Red Dog Mine
Closure and Reclamation Plan

SD C6: Seepage Analysis Report, Red Dog Tailings Main Dam, Future Raises to Closure (URS, 2007)

SEEPAGE ANALYSIS REPORT
RED DOG TAILINGS MAIN DAM
FUTURE RAISES TO CLOSURE
RED DOG MINE, ALASKA

For

TECK COMINCO ALASKA, INC.
URS JOB NO. 33757098
February 26, 2007

February 26, 2007

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Seepage Analysis Report
Red Dog Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska
PO # 1257477, Contract # RD-02-06
URS Job No. 33757098

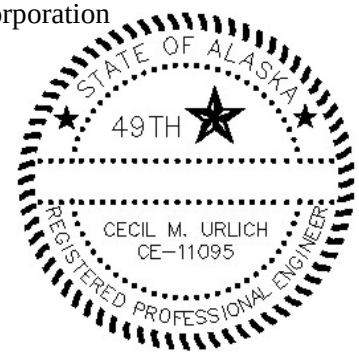
Dear Mr. Coulter:

URS Corporation is pleased to submit one copy of our report to Teck Cominco Alaska, Inc. (TCAK) on the seepage analysis of the Red Dog Tailings Main Dam future raises to closure. The analysis was completed under TCAK Purchase Order No. 1257477 of Contract No. RD-02-06 dated July 25, 2004, and Change Order Nos. 002, 003, 004 and 005.

The seepage analysis was completed by using a three-dimensional model and assuming a permanent water cover over the tailings. The analysis included calibration of the model, estimates of seepage for current and closure dam configurations, and sensitivity evaluations of key material parameter evaluations, a worst case of a failed liner in the dam, and an alternate closure condition of a dry cover.

We thank you for the opportunity to provide engineering support for the tailings main dam future raises to closure. Please call if you have any questions or need additional information.

Sincerely,
URS Corporation



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Executive Summary

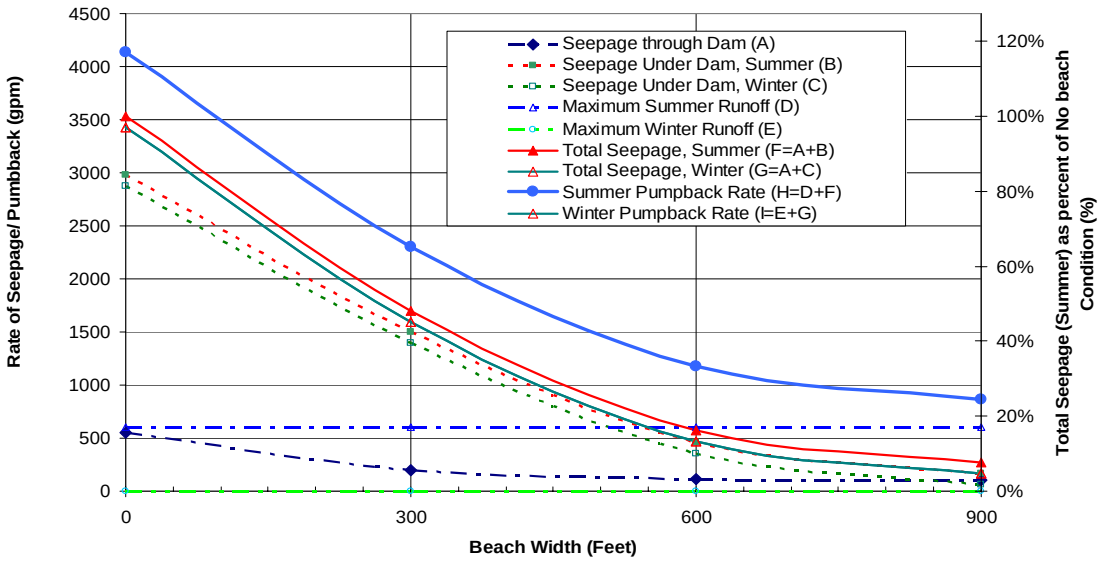
This report describes a three-dimensional (3D) seepage analysis that was completed by URS Corporation for Teck Cominco Alaska Inc. (TCAK) on the tailings main dam at Red Dog Mine. The report describes the calibration of the 3D seepage model to historic seepage pumpback data, estimates of seepage for current and future dam conditions, and sensitivity analyses using a 2D seepage model.

The seepage analysis was completed for the current Stage VII-B condition and the future planned closure condition. The Stage VII-B crest is at elevation 960 feet (El. 960) with maximum permitted tailings and water levels at El 955. For this analysis, the closure crest is assumed to be at El. 986, with maximum projected tailings and water levels at El 975 and El 980.2, respectively.

The 3D seepage analysis models were developed for the previous Stage VI, current Stage VII-B, and future closure configurations of the dam. A 3D explicit finite difference program, FLAC3D (Fast Lagrangian Analysis of Continua in Three Dimensions), was used for the modeling. A sensitivity analysis was completed on select 3D analyses by using the 2D model, GEOFLOW.

Eight major factors influence the seepage, of which five are independent and naturally occurring (precipitation and snow melt, hydraulic conductivities, active layer and permafrost depths, shallow groundwater contributions, and leaks in the geomembrane) and three are operationally controlled and inter-dependent (tailings beach width, impoundment water level and phreatic surface location).

The results of the seepage analysis at closure during summer and winter conditions for different beach widths are graphically shown below in gallons per minute (gpm):



The 3D seepage model was calibrated using historic tailings beach conditions, tailings impoundment water levels, and pumpback rates recorded by TCAK. It is assumed that summer pumpback water is a

combination of seepage from the tailings impoundment and surface runoff to the seepage collection pond, but that winter pumpback water is seepage only because of the frozen ground conditions.

The total seepage shown on the graph is a combination of seepage under the dam plus seepage through the dam, as tabulated below. Seepage under the dam is the major component of total seepage. It occurs through the foundation bedrock under the dam and abutment bedrock around the dam. Seepage through the dam is seepage, or leakage, through the liner system in the dam.

Case	Dam Crest El., feet	Tailings El., feet	Water El., feet	Seepage under Dam (A) gpm	Seepage through Dam (B) gpm	Total Seepage (A+B) gpm
Calibration						
1) No-beach, Summer [1]	950	938	945	1326	308	1634
2) No-beach, Winter [1]	950	938	945	1226	308	1534
3) 600-foot beach, Summer [2]	950	938	940	327	98	425
4) 600-foot beach, Winter [2]	950	938	940	227	98	325
Stage VII-B						
5) 300-foot beach, Summer	960	955	955	595	179	774
6) 300-foot beach, Winter	960	955	955	495	179	674
Closure						
7) No-beach, Summer	986	975	980.2 [3]	2979	553	3532
8) No-beach, Winter	986	975	980.2 [3]	2879	553	3432
9) 300-foot beach, Summer	986	975	980.2 [3]	1496	202	1698
10) 300-foot beach, Winter	986	975	980.2 [3]	1396	202	1598
11) 600-foot beach, Summer	986	975	980.2 [3]	458	115	573
12) 600-foot beach, Winter	986	975	980.2 [3]	358	115	473
13) 900-foot beach, Summer	986	975	980.2 [3]	165	101	266
14) 900-foot beach, Winter	986	975	980.2 [3]	65	101	166

By using the results of the seepage analysis, URS developed the following conclusions on the total seepage rates for future raises of the tailings main dam to closure:

- A tailings beach on the tailings alongside the upstream slope of the dam is an effective means of reducing the total seepage, as shown on the graph above. The benefit of a beach in reducing seepage at closure can be described as follows:
  - No-beach - Estimated seepage is approximately 3,540 gpm in summer and 3,440 gpm in winter.
  - 300-foot-wide beach - Estimated seepage is approximately 1,700 gpm in summer and 1,600 gpm in winter, which is approximately 50 % of the estimated seepage with no beach, or a 50 % reduction in seepage from the no-beach condition.
  - 600-foot-wide beach – Estimated seepage is approximately 600 gpm in summer and 500 gpm in winter, which is 15 to 20 % of the estimated seepage with no beach, or an 80 to 85 % reduction in seepage from the no-beach condition
  - 900-foot-wide beach – Estimated seepage is approximately 300 gpm in summer and 200 gpm in winter, which is 5 to 10 % of the estimated seepage with no beach, or a 90 to 95 % reduction in seepage from the no-beach condition

- The difference between winter and summer seepage rates is primarily from the shallow seepage and ground water flow from the abutments, which was estimated to be about 100 gpm. Pumpback data shows a similar magnitude of difference, when the effects of precipitation and permafrost degradation are not reflected on the pumpback data.
- The pumpback rate is generally higher in summer than winter. The difference depends on changes in tailings operations, climate, freshets, surface runoff, groundwater depth, active layer and permafrost degradation. In years of no significant permafrost degradation, the maximum difference between winter and summer pumpback was about 600 gpm.
- The estimated maximum summer runoff from rain and snow is 600 gpm for a rain-on-snow event. The design criteria is 2.5-inches for a 100-year 24-hour rainfall. The maximum daily snowmelt using a maximum observed snow-water equivalent is 7.44 inches at the end of the 1993-94 winter and the corresponding maximum snowmelt in one day is 2.97 inches.
- The calibration of the 3D model relied heavily on the pumpback records. The pumpback water includes seepage through and under the dam, and seasonal water from other sources such as surface runoff, shallow groundwater, melt water from thawing frozen ground, and water released by "ice dams" that develop on the surface of the active layer as it thaws.
- The phreatic surface in the tailings and in the rockfill buttress upstream of the liner system is mostly influenced by the width of the tailings beach, the horizontal hydraulic conductivity of the tailings, and possibly the hydraulic conductivity of the weathered shale bedrock beneath the tailings impoundment.
- Permafrost under the dam has degraded since the start of tailings operations in the late 1980s by up to 80 and 50 feet vertically on the west and east abutment, respectively. The thaw bulb under the creek has widened by approximately 250 feet, consisting of 200 feet to the east and 50 feet to the west.
- The dam thermistor records indicate that the degradation of permafrost and widening of the thaw bulb along and beneath the dam footprint has slowed considerably over the last few years and may have ceased altogether.Therefore it is reasonable to assume a smaller rate of future permafrost degradation and thaw bulb widening.
- The historical thermistor and seepage pumpback records show that the seepage pumpback rate increased with increasing depth to permafrost beneath the dam. The increase in pumpback rates, and hence the seepage rates, were more pronounced, whenever significant permafrost degradations were noted beneath the dam over a short period of time.
- Future changes in the depth to permafrost will depend on the future construction activities around the dam such as the raises to closure, modifications to the seepage collection system, and changes in tailings management such as the width of the tailings beach and method of cover over the tailings impoundment at closure.

URS developed the following conclusions on the sensitivity to seepage of variations in key material parameters, the extreme worst case of a failed geomembrane in the dam, and an alternate possible closure condition of a dry cover over the tailings:

- Estimated seepage rates are sensitive to changes in hydraulic conductivities of bedrock and tailings. The hydraulic conductivity ranges considered for bedrock ($1x10^{-5}$ to $3x10^{-5}$ cm/sec) and tailings ($1x10^{-6}$ to $5x10^{-5}$ cm/sec) resulted in a 50 to 200 % variation in seepage under dam. However, URS concludes that the parameters selected are appropriate.
- Estimated seepage rates are not very sensitive to changes in current permafrost conditions. Up to 10 feet of aggradation and 20 feet of degradation from the 2005 permafrost surface were considered, and resulted in a 98 to 103 % variation in seepage under the dam. Therefore, URS concludes that the parameters selected are appropriate.

- Leaks in the geomembrane will impact seepage through the dam depending on the number and size. URS estimated leak numbers and sizes from other project histories, industry standards, published liner performance data, tailings dam liner installation quality control, and integrity of geomembrane exposed during Stage VII-B cutoff wall excavations.
- Use of extreme published values of geomembrane leak number and size would result in a 10 to 1000 % variation in seepage through the dam. The leak numbers and sizes selected by URS, in combination with other parameter discussed above, resulted in total seepage rates through the tailings main dam that are consistent with the pumpback records.
- In the extreme worst case of a failed geomembrane that is no longer functional, the phreatic surface in the dam would rise and seepage through the dam for a 900-foot wide beach would be about 2100 gpm in winter and 2200 gpm in summer, which are 7 to 11 times the seepage estimated for a 900-foot beach with intact liner.
- If the geomembrane did fail and was no longer functional, a relatively wide or perimeter tailings beach would be needed to keep the phreatic surface in the dam from rising and reducing the stability of the dam, and to retain the tailings water behind the dam. Dam stability and geomembrane durability are discussed in separate URS reports.
- For an alternate closure case of a dry cover over the tailings, a net inflow of 400 gallons per minute (gpm) into the tailings was estimated by taking the mean annual precipitation of 20.7 inches and assuming a net infiltration of 50% to allow for evaporation and runoff. This inflow would cause the phreatic surface in the tailings to rise above the cover.
- A net inflow of less than 40 gpm to the tailings impoundment would be required to ensure that the phreatic surface in the tailings will remain below the cover. This would be very difficult to achieve because it would require that only 5 % of the direct precipitation could remain in the tailings area and the other 95 % would need to be run off the tailings area.

From the seepage analysis results and conclusions, URS has developed recommendations for seepage control of the tailings main dam future raises to closure as follows:

- Monitoring of the phreatic surface in the rockfill and tailings upstream of the liner will help to provide a year-round indicator of total seepage. The monitoring would provide upstream water level data, in combination with downstream pumpback data, for future verification of the seepage model and updates of the seepage estimate at closure.
- It is recommended that a tailings beach as wide as practically possible be maintained alongside the entire upstream slope of the dam. The efforts to construct and maintain a wider beach will be offset by the need to pump less seepage back to the tailings impoundment. The beach would need to be covered with rock for dust control.
- In order to monitor the upstream phreatic surface in the rockfill and tailings upstream of the liner at closure, it is recommended that piezometers be installed in the tailings under the beach near to and away from the crest of the dam. The piezometers should be installed in lines that are continuations of the current lines of piezometers.
- If a wide tailings beach is not desired for closure, consideration could be given to installing either a barrier wall or grout curtain into competent low permeability bedrock along the toe of the dam upstream of the liner, or a tailings beach further around the impoundment perimeter. The barrier must extend below the cutoff wall, and above the phreatic surface.
- If a dry soil cover option is chosen, the maximum net inflow rate into the tailings impoundment must be kept less than 40 gpm to ensure that the phreatic surface does not rise to the top of tailings and into the dry soil cover. This would be very difficult, if not impossible, to achieve.

1.0 INTRODUCTION

The tailings main dam at Red Dog Mine is a 182-feet high rock fill embankment with a primary seepage control system in the dam and a secondary seepage control system downstream of the dam. The mine operator, Teck Cominco Alaska, Inc. (TCAK), and SRK Consulting (Canada) Inc., are developing a mine closure plan that is based on operations to around year 2030. This will require raising the dam by 26 feet from the Stage VII-B crest elevation of 960 feet (El 960) to a final crest at El 986 and height of 208 feet.

In order to provide technical input to the closure plan for the tailings facility part of the mine, TCAK retained URS Corporation to complete a conceptual design of the tailings main dam at closure. In order to develop the conceptual design, URS completed a geotechnical investigation of the dam foundation for the Stage VII-B and closure configuration, a seepage analysis of the dam from the Stage VII-B configuration through the future raises to closure, and a stability analysis of the dam for the various raises to closure.

This URS report presents the results of the seepage analysis for the conceptual design at closure and for a Stage VII-B dam raise that is under construction. The geotechnical investigation, stability analysis and conceptual design are described in separate reports (URS, 2006, 2007b and 2007c). The seepage analysis was completed under TCAK Purchase Order No. 1257477 of Contract No. RD-02-06, dated July 25, 2004, and Change Orders Nos. 002, 003, 004 and 005.

The seepage analysis work also fulfils a State of Alaska Department of Natural Resources (ADNR) request for "*a detailed engineering evaluation of the expected performance of the next raise and subsequent raises to the final configuration of the system*" in a letter to TCAK titled "Multiple Accounts Analysis for Red Dog Tailings Disposal", dated June 2, 2005.

2.0 PURPOSE AND SCOPE

The primary purpose of the URS seepage analysis of the tailings main dam at was to support the closure plan for the tailings impoundment as follows:

- Recreate the seepage rates reflected by the historic pumpback data
- Estimate the range of total seepage through the dam for future raises to closure.

In order to achieve this purpose, the scope of this seepage analysis was developed to include the following major tasks:

- Review the design, construction and operation records of the seepage control systems
- Compile the impoundment water level, precipitation and pumpback data
- Review historic dam inspection, instrumentation, and thermal analysis reports
- Develop a three-dimensional (3D) seepage model to represent all operation conditions
- Calibrate the 3D model to reproduce the ranges of seepage under the past conditions

- Confirm seepage for the current conditions using the 3D model and calibrated parameters
- Predict seepage for wet cover closure conditions using 3D model and calibrated parameters
- Evaluate the sensitivity of significant model parameters using 2D and 3D seepage models
- Evaluate the sensitivity of the extreme worst case of no liner on seepage using the 3D model
- Evaluate the sensitivity of a dry cover closure option on seepage using the 3D model.

On the basis of discussions with TCAK personnel (Swendseid, 2005 and Thornton, 2005) and a review of the historic pumpback data, it is assumed that summer pumpback water is a combination of total seepage from the tailings impoundment and surface runoff from the catchment area of the seepage collection pond. It is assumed that winter pumpback water is seepage only because of the frozen ground conditions.

The total seepage out of the tailings impoundment is considered to be a combination of seepage under the dam plus seepage through the dam. Seepage under the dam is the major component of the total seepage and is seepage that occurs through the foundation bedrock under the dam and the abutment bedrock around the dam. Seepage through the dam is seepage, or leakage, through the liner system in the dam.

The seepage analysis findings, recommendations and conclusions are described in this report which is organized as follows:

- Section 1.0 provides the introduction to the report
- Section 2.0 outlines the purpose and scope of the seepage analysis and report
- Section 3.0 describes the future raises to closure, seepage controls, and tailings management
- Section 4.0 describes the various factors that affect the total seepage
- Section 5.0 describes the approach used to model the total seepage and the sensitivity analyses completed to check the level of confidence of the results
- Section 6.0 describes the model developed to estimate the seepage under the dam and the results obtained from the seepage and sensitivity analyses
- Section 7.0 describes the approach used to estimate the seepage through the dam and the results obtained from the seepage and sensitivity analyses
- Section 8.0 presents conclusions and recommendations developed from the seepage analysis
- Section 9.0 is a list of references used in completing the seepage analysis

3.0 HISTORIC REVIEW AND FUTURE PLANS

Descriptions of the tailings dam site, design and construction history, construction material types, primary seepage control system, secondary seepage control system, and dam instrumentation history were provided in the geotechnical report (URS 2006) and dam history report (URS 2007a)

Concurrent with the seepage analysis, URS completed stability analyses and conceptual designs of the tailings main dam to closure that are described in separate reports (URS 2007b and 2007c, respectively). The planned future raises to closure, the historic effectiveness of the existing seepage controls, and the planned seepage controls for closure are presented in this section.

3.1 FUTURE RAISES TO CLOSURE

3.1.1 Embankment Raises to Closure

The conceptual design of the tailings main dam future raises to closure (URS 2007c) consists of raising the dam by a total of 26 feet from the current Stage VII-B crest at El. 960 to the closure crest at El. 986, by completing the following construction activities:

- Widen the embankment downstream to provide a footprint for the raises
- Raise the embankment to a final crest elevation of El 986
- Extend the wing wall out from the east abutment to a final crest at El 986
- Construct an open channel spillway along the hillside around the west abutment.

For purposes of the conceptual design, it is assumed that the 26 feet of future raises from the current Stage VII-B crest at El. 960 to the closure crest at El. 986 will be constructed in three stages as follows:

- Stage VIII – Ten feet high from crest El. 960 to 970
- Stage IX – Ten feet high from crest El. 970 to 980
- Stage X – Six feet high from crest El. 980 to 986.

The planned maximum tailings and water levels at the closure stage would be El 975 and El 985.6, respectively, based on the water cover closure option (URS 2007c). For seepage analysis, a pond water level at El 980.2, which includes a permanent water cover of 2.0 feet and autumn and spring inflow of 3.2 feet on top of the maximum tailings of El. 975 was used. These are described in the Conceptual Design Report Red Dog Tailings Main Dam Future Raises to Closure (URS 2007c).

3.1.2 Seepage Cutoff System

As part of the Stage VII-B construction in 2006, the west end of the seepage cutoff wall was extended to the west abutment in the same way as was completed in 2005. However, because of the site constraints on the right abutment area, the cutoff wall transitioned at the west abutment into a curtain wall that will extend along a wing wall for raises beyond Stage VII-B. The cutoff wall was constructed in the same way as for the Stage II to IV raises, as shown on Figure 3-1. Figure 3-2 shows typical details of the liner system, curtain wall and wing wall.

The wing wall will be an extension of the main dam from the right abutment where the dam axis will change from a west-east alignment to a northwest-southeast alignment. The wing wall will contain a vertical curtain wall that was partly constructed as part of Stage VII-B. The curtain wall was connected to the geomembrane part of the liner system. The excavation for the curtain wall has an upper part with inclined slopes and lower part with vertical sides and narrow base. The curtain wall in the lower part was embedded in controlled density fill. The wing wall system was constructed in the following sequence:

- Curtain Wall
- Control Density Fill (vertical wall only)
- Geomembrane
- Soil Type 4 (Trench Backfill, downstream of wing wall)
- Soil Type 8 (Trench Backfill, upstream of wing wall)

A critical part of the wing wall that requires CQA scrutiny, judgment and decision is the depth of excavation for the wall through ice-rich soil and rock, and blasted and fractured rock to competent and ice-free bedrock, and the embedment of the wall into bedrock to reduce seepage under the wing wall.

3.1.3 Closure Cover Options

Several closure options were evaluated for closure and reclamation of the tailings main dam, and the following four options were selected for detailed consideration (SRK, 2006a):

- **Contaminated water cover**: Contaminated inflows would be stored in the tailings impoundment for annual treatment and discharge
- **Clean pond water cover**: Contaminated inflows would be diverted to an alternate storage location and clean water inflows will be stored in the tailings impoundment.
- **Dry soil cover**: A soil/rock layer would cover the tailings and the water level would be below the base of the cover. Contaminated inflows would be diverted to an alternate storage location.
- **Wet soil cover**: A soil/rock layer would cover the tailings and the water level will be maintained within the cover.

For the seepage analysis model presented in this report, a contaminated water cover, which represents conditions similar to the clean pond water cover and wet soil cover options, was considered. A sensitivity analysis of dry cover option was also completed in part to investigate the feasibility of this option.

3.2 HISTORIC SEEPAGE CONTROL EFFECTIVENESS

URS reviewed the pumpback history, impoundment water elevations and tailings deposition history for a 10-year period from March 1996 to December 2006. The pumpback records, shown on Figure 3-3, reflect the total seepage from the tailings impoundment. The following findings were made and conclusions were reached with respect to the effectiveness of historic beaching measures to reduce seepage:

- Tailings beaching started in 1997 with tailings discharge at the west abutment. For the no-beach condition before 1997, the pumpback rates ranged from 850 to 1600 gallons per minute (gpm). The average winter pumpback rate, or lower bound of the seepage range, was 1050 gpm. The average summer pumpback rate, or upper bound of the seepage range, was 1600 gpm. The average impoundment water level during this period was approximately El. 930.
  - A no-beach condition was present during this period.
- From 1997 to 2000, tailings were deposited along the upstream face of the main dam to create a beach. A 600 to 700-foot wide beach was developed by 2000 along the full length of the dam. After 2000, the tailings were not deposited on the beach. From 1997 to 2000, the pumpback rates ranged from 600 to 2600 gpm. The average summer and winter pumpback rates were about 1850 and 850 gpm, respectively. The average impoundment water level was at about El 933. The effectiveness of the tailings beach during this period was as follows:
  - A no-beach condition was present before May 1998
  - A partially effective beach was present from June 1998 to August 1999
  - The 600-foot wide tailings beach became effective in August 1999.
- From December 2000 to May 2003, the pumpback rate dropped to an average of 400 gpm. From April to September 2002, the average pumpback rate was 130 gpm, which indicates that the beach was effective in reducing seepage. The pumpback data shows that the average

summer and winter seepage rates were about 750 and 300 gpm, respectively. The average impoundment water level during this time was approximately El 935. The drop in pumpback rates during this period is believed to be due to less seepage because of the 600 to 700-foot wide beach. The effectiveness of tailings beach during this period is summarized as follows:

  - The 600-foot wide tailings beach condition was present until September 2002
  - A partially effective beach was present from September 2002 to May 2003

- From May 2003 to June 2004, the pumpback rates increased as the beach became partially inundated. From the pumpback data, the representative average summer and winter seepage rates are estimated to be approximately 1400 and 850 gpm, respectively. The average impoundment water level during this period was approximately El 941.
  - The partially inundated beach was not as effective as the full beach in reducing seepage.
- From June 2004 to July 2005, the seepage rate increased due to the beach becoming completely inundated. The representative average summer and winter seepage rates during this period are estimated to be about 1650 and 1050 gpm, respectively. The average impoundment water during this period was approximately El 946.
  - The seepage rate during this period represents a no-beach condition.
- From August 2005 to December 2006, the seepage rate decreased from 1650 gpm to about 600 gpm due to the presence of a 300-foot-wide tailings backfill between the coffer dam and main dam. No pumpback readings were taken from late January to late April 2006 because of a damaged flow meter. After a new meter was installed, the pumpback readings were very low for one week, and then stabilized to the pre-damage values. The low values were not used for the seepage estimates.The average summer and winter seepage rates are estimated to be about 900 and 650 gpm, respectively. The average pond level was about El 948.
  - The seepage rate during this period represents a 300-foot wide beach.

The historical effectiveness of the tailings beach due to varying tailings management practices is pictorially summarized with the pumpback data in Figure 3-4. The reason for high recorded seepage rates prior to 1997 and occasional spikes thereafter is possibly the absence of tailings beach and significant permafrost degradation under the footprint of the dam.

The key factor during the low seepage period of December 2000 to May 2003 is that there was a full tailings beach across the entire upstream face of the dam that kept water away from the dam while the water level in the impoundment continued to rise. The impoundment water level continued to rise during the entire period of record from March 1996 to January 2006, except for some short seasonal fluctuations.

3.3 TAILINGS MANAGEMENT FOR CLOSURE

Recognizing the benefits of a wider tailings beach to reduce the seepage rate and lower the phreatic surface in the tailings main dam, the concept of a coffer dam in the tailings impoundment will continue to provide a tailings beach along the entire crest length of the dam for future raises to closure. A coffer dam will be built over the tailings and the space between the main dam and coffer dam will be filled with tailings to form a tailings beach. The benefits of a wider tailings beach will be balanced with other closure considerations in selecting an optimal beach width and thus locating coffer dam alignments for closure.

4.0 FACTORS INFLUENCING SEEPAGE

The pumpback record of the seepage collection system was used as an estimate of total seepage through and under the tailings main dam from the tailings impoundment. The pumpback record is shown on Figure 3-3. It is assumed that summer pumpback water is seepage from the tailings impoundment plus surface runoff and shallow groundwater to the seepage collection pond, but that the winter pumpback water is seepage only because of the frozen ground conditions.

The total seepage rate through and under the tailings main dam, as reflected in the pumpback record, has fluctuated over time with climate, dam height, tailings management and tailings water level changes. It is difficult to quantify the varied and combined contributions of these factors in the seepage analysis. Also, many of these factors have three dimensions, and in combination, they have 3D impacts. Therefore, in order to most accurately predict future seepage, a 3D seepage model was necessary to model seepage through and under the dam. It is considered that 2D seepage models are not adequate for this purpose.

In order to simplify the 3D modeling efforts, it was necessary to identify the factors that have the most impact on seepage through and under the dam. URS identified the following five major independent model parameters, or naturally occurring variables, for detailed consideration in the seepage model:

- Precipitation and snow melt
- Hydraulic conductivities
- Active layer and permafrost depths
- Shallow groundwater contributions
- Leaks in the geomembrane

In addition, URS identified the following three variables that are operationally controlled and inter-dependent:

- Tailings beach width
- Impoundment water level
- Phreatic surface location.

These five independent and three inter-dependent factors are discussed in the following sections.

4.1 PRECIPITATION AND SNOWMELT

Precipitation and snowmelt in the catchment area of the seepage collection pond impact the seepage pumpback rate. This catchment area is confined by diversion ditches and is estimated to be approximately 52 acres (Dames & Moore, 1987a). This component of pumpback volume is not part of the actual seepage through and under the dam. URS reviewed the precipitation and snowmelt data in conjunction with pumpback record to evaluate their influence on the pumpback rates. It was found that the direct contribution from precipitation and snowmelt to the recorded pumpback rate was not significant except in years when extreme precipitation events were recorded and during the spring freshet each year.

An example of extreme precipitation and freshet occurred from August 7 to 15, 2001. During this time, other factors that influence seepage appeared to remain essentially constant. The total precipitation during this period was 5.86 inches with a peak precipitation of 2.2 inches on August 12, 2001. This event

resulted in a difference of peak pumpback rate of approximately 560 gpm from the baseline seepage rate before the event. The peak pumpback was recorded two days later than the peak precipitation.

Similar patterns were noted on the precipitation and pumpback records for other precipitation events where pumpback or seepage was not likely influenced by other factors. An average difference in pumpback of about 105 gpm per each inch of rainfall precipitation was noted about two days after the rainfall event.

The maximum runoff corresponding to rainfall and snow melt appears to occur during a maximum rain-on-snow event. The rain-on-snow event used to estimate the freeboard for the tailings main dam includes a 100-year 24-hour rainfall with reasonably high snow accumulation (Geomatrix, 2003). Using a maximum observed snow-water equivalent of 7.44 inches (1993 to 1994) at the end of winter, the maximum snowmelt in a single day was estimated to be 2.97 inches. The 100-year 24-hour rainfall was estimated to be 2.5 inches. These values result in 5.47 inches of total moisture being available for runoff. This event would result in a peak pumpback rate approximately 600 gpm higher than the baseline pumpback rate before the event.

Based on the above observations and computations for the design rain-on-snow event and a typical storm event, the runoff contribution to pumpback is estimated to be approximately 600 gpm. The total summer pumpback rate would be obtained by adding the runoff contribution to the total seepage during summer.

4.2 HYDRAULIC CONDUCTIVITIES

The liner system is the only designed impervious component and seepage barrier in the tailings main dam. Assuming a reasonable integrity of the liner system, the hydraulic conductivities of the following materials will be the primary parameters influencing the seepage rate:

- Dam rockfill
- Foundation bedrock
- Impounded tailings.

Ranges of hydraulic conductivity for the dam rockfill, foundation bedrock in various frozen, thawed and weathered conditions, and tailings with different vertical and horizontal properties, were obtained from geotechnical investigations and are summarized in the geotechnical investigation report (URS, 2006). Table 4-1 presents a summary of the hydraulic conductivity parameters for materials in dam, its foundation and tailings.

Of these materials, the most significant impact on the seepage rate was determined to be the hydraulic conductivity of the foundation materials. The foundation bedrock encountered at the dam site is highly faulted and fractured to well beyond the depth of the cutoff wall. Therefore, the hydraulic conductivity of the bedrock could vary widely with depth and location. Also, frozen bedrock would have much lower hydraulic conductivity than the thawed bedrock.

The hydraulic conductivity of tailings will not have a major impact on seepage when there is no tailings beach to prevent water contact with the rockfill upstream of the liner. However, when a beach is present, the head at the upstream rockfill face will be less than the water level in the impoundment due to the low hydraulic conductivity of the tailings. In this case, the phreatic surface will drop along the width of the tailings beach. Conversely, the seepage rate will be higher for a higher hydraulic conductivity of tailings.

Table 4-1: Hydraulic Conductivity of Tailings Main Dam Materials

Material Group	Material Description	Hydraulic Conductivity		Hydraulic Conductivity		References and Notes
		Kh (feet/sec)	Kv (feet/sec)	Kh (cm/sec)	Kv (cm/sec)	
Wethrock	Moderately to highly weathered rock	1.19×10^{-5}	1.19×10^{-5}	3.63×10^{-4}	3.63×10^{-4}	Hydraulic Conductivity estimated from: 1) Packer Test Results (URS, 2006): 1×10^{-4} to 5×10^{-4} cm/sec. 2) Permeability Tests (Table A7, Dames & Moore, 1987a): 4.5×10^{-5} to 9.1×10^{-5} cm/sec
Bedrock	Slightly weathered rock	3.28×10^{-7}	3.28×10^{-7}	1.00×10^{-5}	1.00×10^{-5}	Hydraulic Conductivity estimated from: 1) Packer Test Results (URS, 2005a): 1×10^{-5} to 3×10^{-5} cm/sec.
Frozen	Frozen rock	3.28×10^{-8}	3.28×10^{-8}	1.00×10^{-6}	1.00×10^{-6}	Hydraulic Conductivity estimated from: 1) Packer Test Results (URS, 2005a): 1×10^{-6} to 3×10^{-6} cm/sec.
Rockfill	Rockfill materials for dam and underdrain	6.25×10^{-5}	6.25×10^{-5}	1.9×10^{-3}	1.9×10^{-3}	Hydraulic Conductivity estimated from: 1) Permeability Tests (Table A8, Dames & Moore, 1987a): approximately 5.0×10^{-3} cm/sec 2) Water Management Consultants' seepage model utilized Hydraulic Conductivity value of 0.12 cm/sec.
Tailings	Mine tailings	9.19×10^{-7}	9.19×10^{-8}	2.80×10^{-5}	2.80×10^{-6}	Hydraulic Conductivity estimated from: 1) Bulk Sample (Section 5.0, Golder, June 2003): 2.2×10^{-6} cm/sec 2) Slurried Sample (Section 5.0, Golder, June 2003): 2.8×10^{-5} cm/sec

The upstream face rockfill under the tailings reduces the benefit of a lower phreatic surface that would occur if a tailings beach were present. Because of the high hydraulic conductivity of the rockfill, water that seeps to the upstream face of the dam will flow to the mid-section of the dam which will result in a flatter phreatic surface in the upstream rockfill. Similarly, because of the rockfill, seepage below the liner will mostly flow to the low part of the rockfill before the underdrain. This 3D effect will cause a higher head difference across the liner in the middle of the dam and a lower head difference on the abutments.

The rockfill above and below the liner system, the rockfill blanket, and the high phreatic surface in the upstream rockfill face in the mid- section of the dam, have caused higher seepage rates in the mid-section of the dam and lower seepage rates on the abutments of the dam.

4.3 ACTIVE LAYER AND PERMAFROST DEPTH

Frozen ground has a much lower hydraulic conductivity than thawed ground. The tailings dam site is underlain by continuous permafrost, except at the thaw bulb along the original streambed of the South Fork. The absence of permafrost along a creek bed is known as thaw bulb. In addition to the thaw bulb

along the original creek bed, a small area along the eastern abutment of the tailings dam near T-97-030 also did not have permafrost (Water Management Consultants, 1999).

The presence of, and changes to, the depth of permafrost in the dam footprint impacts the hydraulic conductivity of the foundation. In addition, the seasonal changes in thermal conditions result in freeze and thaw of the active layer which impacts the hydraulic conductivity of the shallow ground in the vicinity of the dam. This seasonal change in hydraulic conductivity of the shallow ground causes changes in contribution from the shallow groundwater to the seepage rate estimated from the pumpback data between summer and winter months.

The depth to permafrost on both abutments and the width of the thaw bulb has changed over time since the construction of the tailings main dam and the development of the impoundment. The permafrost conditions beneath the tailings main dam were estimated from thermal and water level data collected from thermistors and piezometers in the dam, past reports (Dames & Moore, 1987a and 1987b; Water Management Consultants, 1999; and URS, January 2005a), TCAK annual inspection reports (TCAK, 2005), and communications on ongoing permafrost research at the mine (Weaver, 2005).

Permafrost profiles developed from the historical thermistor records along the Stage VI axis of the dam indicate permafrost degradation as shown on Figure 4-1. Historical records of thermistors, TDAM-T-1 and TDAM-T-15, shown on Figures 4-2 and 4-3, also show representative permafrost degradation recorded on west and east abutments of the dam. As shown on Figure 4-1, the permafrost under the dam has degraded since the start of tailings operations. Beneath the dam crest, permafrost has degraded by up to 80 feet on the west abutment and 50 feet on the east abutment. The thaw bulb at the creek bed has widened by about 250 feet of which 200 feet is to the east abutment and 50 feet is to the west abutment.

The thaw bulb behavior follows a trend predicted by a geothermal model developed by Water Management Consultants (1999) for the tailings impoundment. The model predicted a potential thaw bulb widening beneath the impoundment of up to approximately 650 feet during the life of the mine, and 1000 feet 100 years after mining. As shown on Figure 4.1, the thermistor data shows that the permafrost degradation and thaw bulb widening under the dam crest has slowed considerably an dpossibly stopped.

The historical thermistor records summarized on Figure 4-1 show that significant permafrost degradation and widening of the thaw bulb at the creek bed has taken place beneath the tailings main dam. It appears that this degradation has significantly slowed and any changes in the permafrost surface in the recent past appear to be minor compared to the period before 2000. However, future permafrost degradation will depend on several factors including future construction activities and the width of the tailings beach.

In order to complete the 3D seepage analysis, URS used the permafrost profile conditions estimated for 2005, as shown on Figure 4-4, and assumed that the permeability of frozen zones of rock is ten times lower than that of thawed bedrock. This assumption is expected to yield practical seepage results, while keeping the computational time reasonable.

Changes in depth to the frozen zone occur either by degradation and aggradation of the permafrost, or by freeze and thaw of the active layer. These changes affect the amount of water that reaches the seepage pumpback system. The depth to the frozen zone impacts the seepage rates and pumpback data as follows:

- Changes in depth to permafrost result in changes to the thickness of the more pervious foundation material under the dam and to the area of the vertical plane of this material that is available for seepage to pass through:

  - Seepage increases as the permafrost degrades
  - Seepage decreases as the permafrost aggrades.
- Fluctuations of the depth to the frozen surface of the active layer impede and enhance seepage in the following four phases while the active layer changes from being frozen and then goes through the thaw process each year:
  - Seepage is at its lowest rate when the active layer is in its most frozen state and the top surface of the active layer is at its highest elevation.
  - Seepage starts to increase as the active layer starts to thaw and starts to break up into ice blocks that basically act as subsurface ice dams.
  - Seepage starts to go through a surge for a period of time while the ice blocks melt and no longer act as ice dams.
  - Seepage reaches its steady state after the surge is over and the active zone has thawed down to the top surface of the permafrost.

Corresponding to periods of recorded permafrost degradation, significant increase in pumpback rates have been recorded from the pumpback system. Examples of such periods are:

- March 1997 to October 1997
- June 1998 to August 1998
- March 2000 to October 2000.

Water from permafrost degradation will add to the pumpback rate, but does not fully explain the total volumes recorded. Impulsive changes in recorded pumpback rates may be due to the ice dam-like actions caused by seasonal changes in the frozen zone under the main dam. When the frozen zone degrades, a large volume of retained water is released to the seepage collection pond. Seepage under the dam is impeded while the frozen surface aggrades. Hence the recorded pumpback rates fluctuate seasonally.

4.4 SHALLOW GROUND WATER

The shallow groundwater investigations completed by Water Management Consultants (1999) obtained field piezometric data showing that the shallow groundwater flows generally mimic surface topography and flow downhill toward the old streambed of the South Fork of Red Dog Creek.

Any active shallow groundwater flow within the dam area upstream of the liner will discharge into the tailings impoundment. Similarly, any active shallow groundwater flow within the dam area, downstream of the liner and upstream of the seepage collection dam, will likely discharge directly into the underdrain or into the seepage collection pond.

The contribution of shallow groundwater to the seepage pumpback volume is considered seasonal. Shallow groundwater contribution is expected only in summer months, and no contribution is expected in winter months. Because, soil and rock in the abutments above permafrost freeze in winter months, thus limiting or eliminating the window available for shallow groundwater flow towards the underdrain.

4.5 LEAKS IN THE GEOMEMBRANE

The HDPE geomembrane part of the liner system is the seepage barrier in the tailings main dam. Any holes in the HDPE geomembrane will result in leaks through the liner system that will directly contribute

to increased seepage through the dam. Therefore, the number of leaks in the geomembrane and the size of these leaks are an important component of the performance of the dam.

The purpose of a geomembrane is to provide an impervious barrier to water. However, geomembranes are not completely leak proof because they get damaged, punctured and distressed despite the attention to construction quality and workmanship. Liner system studies (Rollin et al, 1999; Collucci and Lavagnolo, 1995; Rollin and Jacquelin, 1998; Wallace, 2006; Koerner, 2006 and Giroud, 2006) found that the sizes and frequency of leaks in geomembranes depend on several factors such as:

- Installer personnel experience, workmanship and equipment
- Degree of construction quality assurance and control (CQA and CQC)
- Quality of underlying subgrade and geotextiles
- Quality and placement of materials overlying the geomembrane
- Type and thickness of geomembrane.

The experience and workmanship of installers is a major factor on the type, number and size of leak holes in the HDPE geomembrane. Most detected leaks are caused by knife cuts and faulty seams. The quality of geomembrane installation in the tailings main dam has benefited from the continuity of the same installation personnel from Stages I through VII-B.

Implementation of rigorous CQA and CQC during geomembrane installations is a key factor in leak control. Rigorous CQA will assure high level of liner installation with low number of leaks. Continuity in work quality has been achieved for Stages I through VII-B with URS (formerly Dames & Moore) and Dowl NANA providing CQA services.

The quality of the subgrade has an impact on leaks in geomembrane liner. The geomembrane liner in the dam is protected by a subgrade consisting of 16 oz geotextile and Type 4 liner bedding and cover consisting of Type 4 liner cover. The liner construction including subgrade preparation at each stage of dam construction was completed in accordance with the technical specifications and was monitored by URS and Dowl NANA to assure a quality liner subgrade and cover to provide high protection of liner.

Different types and thicknesses of geomembrane affect the number and size of leak holes. HDPE is considered to be the most appropriate type of presently available geomembrane for the tailings main dam. The 100-mil thick HDPE geomembrane used in the dam is thicker than the commonly used 60-mil and 80-mil thicknesses that are used in most geomembrane applications.

4.6 TAILINGS BEACH WIDTH

The rockfill in the tailings main dam, both above and below the liner system, is permeable. Therefore, direct contact of the water in the tailings impoundment with the upstream face of the dam will provide the maximum head available from the tailings impoundment, thus increasing the seepage under the dam and through the liner.

The tailings beach acts as a barrier to seepage by keeping water away from the dam, and lowers the effective head difference between the upstream and downstream water levels due to much lower permeability characteristics of the tailings. With increasing width of the tailings beach, the seepage under the dam will be lower due to lower head at the upstream face and longer seepage path through the tailings to the rockfill layer on the upstream face of the dam.

The historical effectiveness of a tailings beach due to varying tailings management practices is shown with the historical pumpback data in Figure 3-4. For the purposes of model calibration, the following "no-beach" and "beach" conditions were selected from the historical records:

- **Beach Conditions:** Historical records show that a tailings beach was fully effective from August 1999 to September 2002. The beach width during the time was 600 to 700 feet. Average summer and winter seepage rates were estimated to be approximately 750 and 300 gpm, respectively, except from April and August 2002 when the pumpback rates were as low as 100 gpm. The average impoundment water level during this time was approximately El 935.
- **No-Beach Conditions:** From June 2004 to May 2005, the tailings beach was completely inundated and a "no-beach" condition was in place. Representative average summer and winter seepage rates during this period are estimated to be approximately 1650 and 1050 gpm, respectively. The average impoundment water level during this time was approximately El 946.

The objective of the conceptual design of the tailings main dam future raises to closure is to develop a technically feasible and economical closure configuration of the dam. Effective seepage controls at closure will provide a direct benefit in reducing the cost and maintenance of seepage pumpback, and an indirect benefit in lowering the phreatic surface in the dam upstream and downstream of the liner.

For the closure configuration, a wider beach is expected to provide improved seepage control. Therefore, for the purpose of predicting seepage from the model, the following future tailings beach conditions were considered in the seepage analysis:

- Stage VII-B - 300-foot wide beach
- Closure – No-Beach
- Closure - 300-foot-wide beach
- Closure - 600-foot-wide beach
- Closure - 900-foot-wide beach.

4.7 IMPOUNDMENT WATER LEVEL

The impoundment water level has significant influence on the total seepage collected at the seepage collection pond. For a given width of tailings beach and tailings characteristics, higher total seepage would be resulted from higher impoundment water levels, due to higher phreatic surface upstream of the liner.

Despite short term fluctuations, the water level within the impoundment has increased over the years as shown on Figure 3-3. The impoundment water level has increased due to continuous deposition of tailings and tailings water from the mine operations. The short term fluctuations of impoundment water level are primarily due to:

- Rate of tailings deposition
- Surface water runoff from the mill site and surrounding areas of mine development
- Surface water runoff collected at the mine water diversion dam and pumped in to the impoundment
- Evaporation in the impoundment
- Changes in seepage rate through and under the dam

- Reuse of impounded water

4.8 PHREATIC SURFACE

The phreatic surface in the tailings main dam, both upstream and downstream of the liner, influences the total seepage collected at the seepage collection pond for the following reasons:

- The difference between the head upstream and downstream of the liner influences the volume of seepage through the foundation of the dam as well as any leakage through the liner.
- The liner area that is in contact with water upstream of the liner through which any potential leakage might occur is dependant on the location of phreatic surface upstream of the liner.

The phreatic surface in the protective rockfill part of the tailings main dam that is upstream of the liner and under the tailings is influenced by the presence and width of the tailings beach and the water level in the tailings impoundment water level. The tailings management practices and water levels have varied over time and are well documented. However, the changes in phreatic surface upstream of the liner have not been directly documented and have to be estimated from the seepage analysis models.

The phreatic surface conditions in and under the tailings main dam downstream of the liner has been documented by the underdrain piezometers, P-8B, P-9A, and P-10A, that are located along the critical section of the dam. The phreatic surface downstream of the liner was estimated from the historical data of underdrain piezometer readings collected and provided by TCAK as well as information presented in past reports (Dames & Moore, 1987a; Dames & Moore, 1987b; URS, January 2005a; and Water Management Consultants, 1999).

The highest piezometric water levels recorded along the critical cross section of the dam, downstream of the liner, are presented on Figure 4-5. The estimated phreatic surface and the 2005 permafrost profile conditions along the axis (Stage VI) of the dam are presented on Figure 4-4.

A review of the historical underdrain piezometer data along with the historical thermistor and pumpback records, revealed the following patterns of seepage:

- The underdrain piezometer water levels correlate well with the pumpback data. The piezometer water level readings were higher when the recorded pumpback readings were higher and vice versa.
- Under similar tailings management and seasonal conditions, the rate of pumpback generally increased with increasing pond water level.
- The normalized head of each piezometer and pumpback rates follow a similar pattern during seasonal fluctuations. The normalized total head (N) is the ratio of total head in feet (Ht) to water surface elevation in feet (P), with pond bottom as datum. N= (Ht-781.2)/(P-781.2).
- When potential permafrost degradation was noted from the thermistor records, the pumpback record showed a corresponding increase in flow rate. Increased pumpback records during noted permafrost degradations have had a corresponding increase of underdrain piezometer readings.
- The total precipitation of rainfall and snow does contribute to the total pumpback volumes, however, no apparent correlation was observed between precipitation and pumpback rates.

Table 4-2: Comparison of Representative Underdrain Piezometers and Pumpback Rate Records

Condition	Width of Beach (feet)	Date	Water Level (feet)	Seepage Rate (gpm)	Piezometer P-8B		Piezometer P-9A		Piezometer P-10A		Comments
					Water El. (ft)	N [1]	Water El. (ft)	N [1]	Water El. (ft)	N [1]	
No Beach	0	7/16/1998	932.2	673.93	816.4	0.23	809.8	0.19	805.4	0.16	
		8/22/1998	935.2	2558.8	821.6	0.26	814.3	0.22	808.5	0.18	possible permafrost degradation
		9/28/1998	935	2129.1	818.6	0.24	811.7	0.2	806.9	0.17	possible permafrost degradation
		11/3/1998	934.4	1711.6[2]	816.4	0.23	810.1	0.19	806	0.16	
		5/31/2004	944.3	1299.5	819.6	0.24	813.1	0.2	809.1	0.17	
		7/23/2004	943.2	1045	819.6	0.24	813.1	0.2	808.3	0.17	
		8/20/2004	944.2	1451.5	821.1	0.25	814.5	0.2	809.2	0.17	possible permafrost degradation
		9/12/2004	944	1258.4	818.9	0.23	812.6	0.19	808	0.17	
		11/5/2004	944.7	1353.5	818.5	0.23	812.2	0.19	807.8	0.16	
		3/21/2005	946.9	832.5	819.5	0.23	812.9	0.19	808.2	0.16	
		4/23/2005	947.1	977.1	819.8	0.23	813.3	0.19	808.4	0.16	
		5/3/2005	947.5	1414.6	821.3	0.24	814.4	0.2	809.1	0.17	possible permafrost degradation
		6/5/2005	948.7	1403.4	821.9	0.24	815.2	0.2	809.6	0.17	possible permafrost degradation
		7/12/2005	947.6	1003.5	815.4	0.21	810.3	0.18	806.4	0.15	
Beach	600	1/25/2001	935.4	455.2	805.9	0.16	800.4	0.12	797.9	0.11	
		7/27/2001	935.2	477.9	806.5	0.16	800.8	0.13	798.5	0.11	
		8/21/2001	935	612.7	808	0.17	802.1	0.14	799.7	0.12	possible permafrost degradation
		8/28/2001	934.6	518.8	806.5	0.17	801.4	0.13	799	0.12	
		12/14/2001	934.4	251.6	804	0.15	798.7	0.11	796.7	0.1	
		2/9/2002	934.9	277.6	803.5	0.15	798.5	0.11	796.4	0.1	
		5/21/2002	937.4	113.5	804.6	0.15	799.7	0.12	797.4	0.1	
		8/28/2002	936.9	439.3	806.8	0.16	801.9	0.13	799.4	0.12	
		9/17/2002	937.7	671	809.6	0.18	804.1	0.15	801.3	0.13	possible permafrost degradation
		10/16/2002	937.8	594.3[2]	809.4	0.18	803.9	0.15	801.2	0.13	
Beach	300	11/5/2005	947.7	700.5	813.1	0.19	806.5	0.15	803.3	0.13	
		12/10/2005	947.9	774.9	811.4	0.18	805.9	0.15	802.7	0.13	

[1] - N - Normalized Total Head (Ht - 781.2) / (P - 781.2), where Ht is total water head and P is impoundment water elevation, where El 781.2 is the elevation of the impoundment bottom at the low point of the pumpback system

[2] – Interpolated value

The change in normalized total head due to changes in depth to the frozen surface is impacted by tailings management practices. Wider beaches appear to reduce the fluctuations in the piezometric levels. For example, the normalized total head varied by up to 0.03 for the no-beach condition, but only up to 0.02 for the 600-foot beach condition.

Table 4-2 summarizes the historical seepage, piezometer, and impoundment water level data for recent beach and no-beach conditions to illustrate the above observations. The phreatic surface downstream of the liner primarily depends on all conditions that affect the total seepage, especially the following:

- Tailings impoundment level
- Tail water conditions controlled by the pumpback system operations
- Permafrost degradation
- Seasonal weather changes.

5.0 SEEPAGE ANALYSIS APPROACH

5.1 COMPONENTS OF SEEPAGE

The total seepage from the tailings impoundment emerges downstream of the tailings main dam and consists of the following two major components:

- **Seepage under the dam (seepage)**: This includes seepage through the dam foundation under the liner system and any shallow groundwater seepage from the east and west abutments.
- **Seepage through the dam (leakage):** This includes leakage through the liner system, which would flow through the rockfill and into the underdrain.

Both these components of seepage flow into the seepage collection pond and are returned to the tailings impoundment by means of the seepage pumpback system as illustrated in Figure 5-1.

5.2 ANALYSIS APPROACH

5.2.1 Decoupled Analysis

The leakage and seepage behavior are coupled and are in generally interdependent. The leakage through the liner impacts the phreatic surfaces upstream and downstream of the liner and thus has a coupled effect on the total seepage behavior. However, a decoupled seepage modeling and leakage analysis approach was chosen for the following reasons:

- A combined seepage model would increase the complexity by requiring additional independent parameters such as leakage coefficient, identification of potential leak locations, and pore-pressures at the boundary.
- A 3D seepage model, even without the leakage component, is relatively complex to use and requires long computer run times for each analysis. Therefore, including a leakage boundary in the model would significantly increase modeling effort and analysis time.
- Calibration of the seepage model requires an iterative process. By keeping the number of independent parameters low, the number of iterations will be reduced and the total computer run time required to calibrate and solve the seepage model will be practical and manageable.
- The historic pumpback data suggests that the component of seepage through the dam, or leakage through the liner, is considerably less than the component under the dam, which is consistent with what would be expected given the full-time CQA provided during the liner installation and the fractured nature of the bedrock under the dam.

If the leakage trough the liner was comparable to the seepage under the dam, a decoupled evaluation approach may not be reasonable. However, for the comparatively smaller leakage values that are evident, the impact from leakage on the phreatic surface would be relatively minor. Therefore, it is considered reasonable to decouple the leakage and seepage behaviors and to analyze the separately.

5.2.2 Analysis Models for Seepage Under Dam

In order to model and capture the 3D effects of seepage due the 3D nature of the dam geometry, ground and bedrock surface topography, subsurface thermal conditions, design assumptions, construction constraints and operations observations, URS used the 3D seepage analysis software FLAC3D (Fast Lagrangian Analysis of Continua in Three Dimensions). FLAC3D is a commercially available 3D explicit finite difference program developed by Itasca Consulting Group, Inc. (2003).

In addition to the 3D seepage model, a 2D seepage model was developed using GEOFLOW software to evaluate the sensitivities of various physical parameters considered in the seepage model. GEOFLOW is a 2D Finite Element Analysis (FEA) program. The 3D effects of the seepage problem were simulated by analyzing 2D models at various cross-sections along the length of the tailings main dam.

The seepage analysis models were developed for the purpose of estimating the seepage under and around the tailings main dam during summer conditions. Difference between summer and winter seepage rates were estimated by adding the shallow groundwater contributions from the abutments and shallow seepage contributions away from the downstream toe of the dam. Using the 3D seepage model, the difference between winter and summer seepage was estimated to be about 100 gpm.

The winter seepage rates under and around the tailings main dam were then estimated by assuming that the difference in seepage between summer and winter is mostly due to changes in the active layer. The most impacted areas are the right and left abutments, where no shallow natural clean groundwater contribution is anticipated around the cutoff system during the winter.

5.2.3 Analysis Model for Seepage Through Dam

For simplicity, the seepage through the tailings main dam was solved as an uncoupled problem. However, the 3D modeling software, FLAC3D, is capable of solving seepage in conjunction with leaky boundaries, but was not used in this project.

In order to solve the leakage problem, assumptions were made on the condition of the liner system and the number and size of leak holes in the geomembrane. On the basis of literature on landfill liner systems (Rollin et al, 1999; Collucci and Lavagnolo, 1995 and Rollin and Jacquelin, 1998), these parameters were established and leakage calculations were completed.

5.3 MODEL CALIBRATION

Several conditions that affect the seepage under and around the tailings main dam have changed during the operational history of the tailings impoundment. These include the winter and summer weather, tailings and water levels in the impoundment, shallow ground water flow from the dam abutments, and width of tailings beach.

Periods reflecting “Beach” and “No-Beach” conditions were selected from the Stage VI pumpback data to minimize the influence and effects on the calibration due to permafrost degradation. Target seepage rates

for summer conditions were established for “Beach” and “No-Beach” conditions as shown on Figure 5-2 and summarized as follows:

- 600-foot wide beach – 500 gpm
- No beach – 1600 gpm

The 3D calibration model was developed to reflect the Stage VI dam geometry, recent permafrost surface, seepage cutoff measures, shallow groundwater conditions in the abutments, and tail water conditions in the seepage collection pond. The permeability values for the materials were varied to represent the variable conditions at the site, especially the fractures and weathered zones in the bedrock, to estimate the seepage under the dam.

The phreatic conditions upstream of the liner system that were obtained from the 3D model results were used in calculations to estimate the seepage through the dam. The seepage results from the seepage model represented the summer seepage rate. The winter seepage rate was estimated by subtracting the seepage within the upper 20-feet of ground to represent the active layer typical of the mine site.

The estimate of total seepage out of the tailings impoundment was obtained by adding the estimates of seepage through the dam to the seepage under and around the dam. The total seepage was compared with the seepage pumpback record, and the seepage model was calibrated using an iterative process to reproduce the range of seepage recorded in the historical pumpback data under the conditions evaluated.

The seepage model was calibrated against the target average seepage values for winter and summer seasons established for the “no beach” and “600-foot beach” conditions. This calibration effort resulted in the following outcome:

- The variability in the different factors impacting seepage under and around the dam is narrowed down and the range of seepage estimate from the historical pumpback data is reasonably reproduced
- The permeability values of different material components were selected for the models predicting future seepage through the dam
- The extent and quantity of leakage through the liner is estimated and quantified for prediction of future leakage

Through this iterative approach, URS developed the most appropriate set of parameters for the hydraulic conductivities of the dam and foundation materials, and used these parameters in the seepage models predicting seepage rates for the Stage VII-B and closure conditions.

5.4 SEEPAGE PREDICTIONS

On the basis of the model parameters established from the calibration of the 3D seepage model for the Stage VI condition of the tailings main dam, URS developed 3D seepage models for the Stage VII-B and closure conditions. Seepage analyses were completed for possible future conditions to closure, and estimates of seepage under and around the dam were made.

5.4.1 Seepage Estimate for Stage VII-B

The 3D model that was developed for predicting seepage under the dam for Stage VII-B included the following additional features to reflect the Stage VII-B design conditions:

- Cutoff wall on the east and west abutments extending from Stage IV to VII-B
- A wing wall on the right abutment connected to the cutoff wall
- Higher tailings and water elevations
- A 300-foot wide tailings beach

Besides the estimate for seepage under the dam, the phreatic conditions upstream of the liner, corresponding to a "300-foot beach" condition, were obtained from the seepage analysis and utilized in the estimation of seepage through the dam. Using these estimates, the total seepage for the closure condition was estimated.

5.4.2 Seepage Estimate for Closure

The 3D model that was developed for predicting seepage under the tailings main dam at closure included the following additional features beyond Stage VII-B to reflect the conceptual design condition at closure:

- A wing wall and wing dam out from Stage VII-B wing wall
- Higher tailings and water elevations
- Various widths of tailings beaches (0, 300, 600 and 900-foot wide beaches)

The various beach widths were evaluated to find the optimal width to reduce the total seepage under closure conditions. Seepage values under and around the dam were estimated using the set of parameters established after calibrating the seepage and leakage models for the Stage VI. Phreatic conditions upstream of the liner were obtained from the seepage analysis and used to estimate seepage through the dam. The total seepage at closure with varying beach conditions was estimated from these analyses.

5.5 PARAMETER SENSITIVITY EVALUATION

The selection of hydraulic conductivities of tailings main dam and foundation materials for the calibration of the 3D seepage model for the Stage VI condition was limited to a typical range of values for each material. The range of hydraulic properties were established based on field and laboratory hydraulic conductivities completed during past and current geotechnical investigations (Dames & Moore 1987b; URS 2006) and from typical range of values published for similar material characterizations.

5.5.1 Seepage Under the Dam

The physical parameters that were used to calibrate the 3D model are considered reasonable, but the solution obtained is not necessarily unique. Several other combinations of hydraulic conductivities and varying components of seepage under the dam could produce an acceptable calibrated response for the conditions analyzed. Therefore, sensitivity analyses were completed of three key calibration parameters for the dam at closure:

- Hydraulic conductivity of bedrock
- Hydraulic conductivity of tailings
- Depth to frozen ground

A 2D seepage model was developed using GEOFLOW software to complete the sensitivity analyses. The reason for using the 2D model instead of the 3D model was the time consuming computation effort to

complete each 3D seepage analysis and verify the results, and the rationale that a 2D analysis would provide the information necessary for purposes of evaluating sensitivity.

The 3D effects of the seepage mechanisms were simulated by analyzing several 2D models at various cross-sections along the dam. For each sensitivity analysis parameter, a range of reasonable maximum to minimum values was identified and the 2D seepage analyses were then completed.

The seepage model primarily reflects the contaminated water cover being considered. No separate seepage modeling effort was undertaken for clean pond water cover and wet soil cover options, since the seepage boundary conditions for these options are sufficiently close to the contaminated water cover. A special-case sensitivity analysis was completed for the dry soil cover option, which has significantly different boundary conditions.

5.5.2 Seepage Through the Dam

The sensitivity of model parameters used for evaluating the seepage through the dam was also completed Several other combinations of geomembrane leaks, as well as geomembrane performance, could produce an acceptable calibrated response for the conditions analyzed. Therefore, sensitivity analyses were completed of two key calibration parameters for the dam at closure:

- Number of holes in HDPE geomembrane
- Size of holes in HDPE geomembrane

A special-case sensitivity analysis was also completed by conservatively assuming the extreme worst case of no liner being present in the tailings main dam. This condition will not exist for at least 750 years (URS 2007c), but is considered in this seepage analysis report for the worst case sensitivity purposes. This special case is documented as follows:

- No liner in the dam

For each of the above model parameters and special case identified for the sensitivity analysis, a range of reasonable maximum to minimum values was selected and leakage analyses were completed. Further details of the sensitivity analyses for seepage through the dam are presented in Section 7.0.

6.0 SEEPAGE UNDER AND AROUND DAM

In order to study the behavior of seepage under the tailings main dam at closure, the following 3D and 2D seepage models were developed:

- 3D seepage models using FLAC3D to calibrate the seepage model and to estimate seepage under the dam for future conditions of the dam
- 2D seepage models using GEOFLOW to study the sensitivity of material parameters on estimates of seepage under the dam

This section presents the details of these models developed, seepage and sensitivity analyses completed, and results obtained to study the seepage under the tailings dam.

6.1 MODEL DESCRIPTION

Analytical methods of seepage analysis are limited and only available for simplified boundary conditions and geometries. Available analytical methods cannot handle material non-homogeneity and difficult boundary conditions except for simple two-layered system in certain configurations. For seepage analyses involving complex geometry and geologic conditions, as is the case for the tailings main dam, simple analytical solutions are not practical and appropriate. Therefore a numerical approach is essential.

The 3D and 2D seepage models for the tailings main dam were developed in FLAC3D and GEOFLOW software, respectively. FLAC3D implements an explicit finite difference approach and GEOFLOW implements a finite element approach of numerical scheme to solve boundary value problems. The models were developed using dam geometry reflecting the stages under consideration and material properties for elements in the dam and its foundation as listed below:

- Embankment, liner and cutoff systems (rockfill, drain, geomembrane)
- Foundation and abutments (weathered and moderately weathered bedrock)
- Tailings impoundment with and without a beach.

6.1.1 Model Geometry

The geometry of the tailings main dam for seepage modeling purposes is relatively complex because of the topographic, geologic and thermal variations at the site. To simplify the model, ground surface elevations were obtained from a contour map at chosen sections along the dam axis and used to represent the site geometry. However, even for this simplified geometry, the data preparation, result interpretation, and input-output verification were time-consuming, particularly for the 3D model.

In a numerical model, the distance to the external boundary conditions from the area of interest (extent) will affect the accuracy and reliability of the solution obtained. The model should extend sufficiently far out so that seepage boundary conditions (pore pressures or flows) at external perimeters can be reasonably estimated and specified. The accuracy of the numerical solution is improved with large model extent, but the computation effort increases with increased nodal points. Considering these constraints, an optimal model extent was chosen with the boundary locations approximately as follows:

- West boundary - 1,500 feet out from the west abutment (Station 0+00), to allow sufficient distance between the dam and the boundary.
- East boundary - 1,920 feet out from the east abutment (Station 25+80), to allow adequate distance between the wing wall and the boundary
- Downstream boundary - 1,500 feet down-gradient of Stage VI dam axis, which is down-gradient of the seepage collection pond.
- Upstream boundary - 1,500 feet up-gradient of the Stage VI dam axis, which is across the tailings impoundment.

The selected extent of the model appeared to work adequately for all of the seepage analyses completed.

6.1.2 Materials and Material Properties

The 3D and 2D seepage models for the tailings main dam were developed using typical ranges of hydraulic conductivities for various materials in the embankment (rockfill), its foundation (frozen, thawed and weathered bedrock), and tailings, as summarized in Section 4.2 Typical hydraulic conductivities were obtained from geotechnical investigations, and the values used in the seepage model for different are summarized in Table 4-1. The different material groups used in the models are as follows:

- **"Rockfill"**: This group represents the rockfill in the embankment, Stage I dam foundation, and underdrain. Due to its higher permeability compared to all other materials and its positions (especially at the under-drain), the rockfill allowed the seepage to flow to the underdrain in the model.
- **"Wethrock"**: This group represents layer of highly weathered shale in the foundation and the abutments above the moderately weathered shale bedrock. For simplicity of the model, this highly weathered shale layer was assumed to have a uniform thickness of 40 feet under the entire site.
- **"Bedrock"**: This group represents the moderately weathered shale bedrock, underlying the "Wethrock" group of material, in the foundation and abutments of the tailings main dam.
- **"Frozen"**: This group represents the rock material ("Wethrock" and "Bedrock") that is frozen in the foundation and abutments of the dam. The extent of the "Frozen" group was represented by the 2005 permafrost profile shown in Figure 4-4. The hydraulic conductivity of this group was taken as 0.1 times the hydraulic conductivity of the thawed bedrock ("Bedrock" group).
- **"Tailings"**: This group represents the tailings in the impoundment upstream of the dam. The hydraulic conductivity of the tailings is anisotropic (vertical permeability lower than horizontal permeability), as shown in Table 4-1. However, the influence of the anisotropic effect on the seepage estimate is considered to be minimal. Therefore, in order to control the computation work, the tailings was modeled as an isotropic material and the anisotropy was neglected.

Initial hydraulic conductivities for these materials were obtained from previous geotechnical reports and confined to the range of values in Table 4-1. From the analysis results of the calibration model, the hydraulic conductivities of the materials were adjusted by trial-and-error to reproduce the target calibration total seepage for the "no-beach" and "600 feet beach" cases.

Final hydraulic conductivity values were obtained from the calibration model and were used for seepage analyses of future conditions to closure. These values are shown on Table 4-1. Figure 6-1 shows an isometric view of the 3D model showing different material groups in the model.

6.2 3D SEEPAGE MODEL

6.2.1 General

The 3D seepage models that were developed for the tailings main dam using FLAC3D were used to calibrate the seepage model and to estimate seepage under and around the dam for future conditions of the dam to closure.

In summary, the finite difference method is a numerical approach of approximation by transforming governing differential equations into corresponding difference equations suitable for computer processing. This numerical approach is capable of handling and solving boundary value flow problems with material non-homogeneity and anisotropy, fully and partially saturated flows, and phreatic surface

determinations. With increased complexity of the boundary value problem being solved, the computation and analysis effort could significantly increase.

6.2.2 Simplifications of Model Geometry

The seepage models for the tailings main dam were developed using the simplified geometry shown in Section 6.1.1. In addition, the dam geometry was further simplified by not modeling the rockfill in the dam downstream of the liner in the 3D model except for the "no liner" sensitivity analysis which could not be conducted otherwise. When this part of the rockfill is included in the 3D model, it was found that the computation effort was larger and convergence of the solution was slower because of the additional model size, varying phreatic surface, and partially saturated flow computations.

The extent of saturated flow in the dam downstream of the liner is essentially confined to a shallow rockfill layer at the contact between the dam and foundation. The hydraulic conductivity of the rockfill is much higher than the hydraulic conductivities of other materials. Therefore, the simplified geometry was not expected to significantly alter the seepage problem being analyzed.

Because of the higher hydraulic conductivity of the rockfill material in the embankment, Stage I foundation, and under-drain, the seepage through the bedrock will abruptly change direction to a horizontal flow at the bedrock-rockfill interface, in accordance with the basic principle of flow between materials with different permeability:

$$k_1/k_2 = \tan\beta/\tan\alpha$$

where: k_1 is the hydraulic conductivity of Soil Type 1

k_2 is the hydraulic conductivity of Soil Type 2

α is the angle between the interface plane and flow line in Soil Type 1

β is the angle between the interface plane and flow line in Soil Type 2

This theoretical observation suggests that the flow under the liner from the upstream side of the liner should stay low above the rockfill to bedrock interface. Therefore, it was considered acceptable to not model the rockfill in the dam downstream of the liner.

6.2.3 Mesh Size, Shape, and Aspect Ratio.

The mesh size, shape, and aspect ratio used in any numerical models can significantly affect the accuracy of the model. Alternately, a finer mesh model will result in a larger number of grid-points that will increase the computation effort, especially in a 3D model. Also, it is difficult to configure the mesh so that the shape and aspect ratio of each zone are theoretically good without introducing many more grid-points that the computer can handle within a reasonable time.

In 3D problems involving such time-stepping schemes, the use of a very fine mesh requires a large amount of computer time. As a result, a balance of obtaining acceptably accurate results in a reasonable time has to be achieved. An alternative to using a very fine mesh through the model is to perform sensitivity analyses on the model by varying the mesh size. Although the sensitivity analyses take time because different mesh configurations must be developed, run and compared, one can not be certain if a mesh configuration is providing acceptable solutions without performing some sensitivity analyses.

On the basis of the mesh sensitivity analyses, the optimal mesh configurations for Stage VI, Stage VII-B and closure configurations of the models were chosen for the final analyses as follows:

- Stage VI: 7,334 zones and 9,090 grid points
- Stage VII-B: 7,746 zones and 9,673 grid points
- Closure: 8,183 zones and 10,190 grid points
- Closure ("No-Liner" case): 9,287 zones and 11,216 grid points

Figure 6-1 shows a model with a fine mesh that was used to compare seepage results obtained with a coarse mesh. Cross sectional views of the final coarse mesh configurations for Stage VI, Stage VII-B and closure configurations are shown on Figures 6-2, 6-3 and 6-4, respectively.

Finer mesh sizes were used in areas of high pore pressure gradient and flow concentration, such as near the underdrain, seepage collection pond, and rock fill upstream of the liner. The fine mesh sizes modeled the flow problem more accurately. Meshes at these locations were refined sufficiently so that the flow behavior in these regions was closely represented. Figures 6-5 and 6-6 show plan and section views of the finer mesh used near underdrain and seepage collection pond, respectively.

The attach logic of FLAC3D was for convenience to model areas of local interest with finer mesh than the surrounding coarse mesh. For the rockfill upstream of the liner, fine and coarse mesh configurations were used and the sensitivity on the seepage results was compared before selecting the grid spacing. A cross-section view of the dam showing the finer mesh in front of the liner is shown on Figure 6-7.

6.2.4 Scaling of Fluid Modulus

Numerical methods generally yield approximate results. The time step used in the solution scheme, along with mesh size, shape and aspect ratio, is an important parameter that influences accuracy and ensures that results converge. The time step depends on bulk fluid modulus, mesh size and hydraulic conductivity. Problems with phreatic surfaces will speed up convergence to steady state if the fluid bulk modulus is lower. Because the fluid bulk modulus value is not as important, an artificially lower value of fluid bulk modulus, but high enough to ensure the stability of time step, will speed up the convergence.

To accomplish this objective, URS modified a FISH routine "F3dmod5.fis" developed by ITASCA, Inc. to perform fluid modulus scaling to customize for this problem (see FLAC3D and FLAC Documents: Fluid-Mechanical Interaction). FISH (in FLAC3D) is a special programming language distributed with FLAC/FLAC3D to customize FISH functionality and to accomplish various modeling tasks.

6.2.5 Boundary Conditions

For a typical seepage problem, one of two types of boundary conditions can be specified on an interior or exterior boundary: flow or pore pressure. The following boundary conditions have been specified in the 3D seepage models for the tailings main dam:

- **Left and right abutment boundaries**: Hydrostatic pore pressures were specified and the groundwater table was set at El. 950 based on field data on the abutment near the crest of the dam. This groundwater elevation was assumed constant and no seasonal fluctuation of water table was included in this study.

- **Upstream boundary of tailings impoundment**: Hydrostatic pore pressures were specified corresponding to the maximum water level in the tailings impoundment at each stage of the dam construction:
  - Stage VI: Dam crest at El. 950; maximum water level at El. 945 for "no beach" case and El 940 for "600-foot beach" case.
  - Stage VII-B: Dam crest at El. 960; maximum water level at El. 955
  - Closure Stage: Dam crest at El. 986; maximum water level at El. 980.2. The maximum water elevation consists of the maximum operating elevation at El. El. 975, plus 2.0 feet of water cover, plus 3.2 feet of spring runoff.
- **Upstream boundary away from tailings impoundment and in abutments**: A no-flow boundary with no flow across the boundary was specified. This specification allows change in pore pressures to take place in the seepage calculations.
- **Downstream boundary directly below creek bed**: Hydrostatic pore pressures were specified corresponding to a water level assumed in the seepage collection pond at El. 787.
- **Downstream boundary away from tailings impoundment and in abutments**: A no-flow boundary was specified.
- **Interface between dam rockfill and foundation**: Where the phreatic surface was expected above an interface such as in the valley above the underdrain, alternate rows of nodes were specified as no flow and flowable boundaries with zero-pore pressure to represent the flowable boundaries with non-zero pore pressures. Where the phreatic surface is expected below or at the interface, flowable boundaries with zero-pore pressure were specified. For the case of the "no liner" analysis in Section 7.3.2, the rockfill dam behind the liner was completely modeled.
- **Pore pressures around seepage collection pond:** These pore pressures reflect the water elevation maintained in the pond (El. 787) and were used to establish interior and exterior pore pressure boundary conditions in that area.
- **Geomembrane part of liner system**: The geomembrane was modeled as a "gap" in the dam foundation. FLAC3D treats each face on the gap as an external impervious boundary, which was modeled to extend to the top of the bedrock to model the cutoff system in the dam. For the case of the "no liner" sensitivity analysis in Section 7.3.2, the geomembrane was not modeled.
- On the abutments where the cutoff system was extended and the wing wall is planned, the liner system was modeled with low permeability materials.
- **Top boundary of tailings impoundment**: Pore pressures were specified to values corresponding to the water height above the tailings as follows:
  - For "no-beach" case a non-zero pore pressure was specified on the entire surface of the tailings pond
  - For "beach" cases "no-flow" boundaries were specified within the areas of beach and non-zero pore pressure was specified on the remaining surface of the tailings pond.

On external boundaries where boundary conditions are not specified explicitly, the boundaries are assumed to be of the "no-flow" type and FLAC3D allows the pore-pressure valued to be changed.

6.2.6 Initial Conditions

There are several factors that influence the run time of the 3D seepage model. The initial pore pressure conditions in the tailings main dam are among the most critical of these factors because the flow in the

dam is three dimensional in nature but the model has a discontinuous phreatic surface because of the liner system. Therefore, estimating the initial pore pressures close to the final conditions is difficult.

6.2.7 Convergence Criteria

FLAC3D uses a non-steady flow solution approach for all conditions including steady flow. The steady flow solution is obtained by using the boundary and initial conditions to solve the unsteady flow problem until the result convergences. The default convergence criterion is a maximum limit on the fluid-flow "ratio", defined as [abs(outflow) – abs(inflow)] / [abs(outflow) + abs(inflow)]; where inflow and outflow are the total volume of fluid entering and leaving the flow domain per unit time, respectively. When the "ratio" falls below a specified limit during the solution process, the flow is considered to have converged.

The primary interest in the seepage analysis for the tailings main dam is the flow at localized areas, such as the seepage collection pond and underdrain. Therefore, the fluid-flow ratio may not necessarily be a good indication of convergence. In order to assure convergence, the flow at several areas and pore pressures at various locations were monitored for convergence.

6.2.8 Seepage Calculation at External Boundary

FLAC3D does not have explicit commands to determine the flow through a particular surface or at a particular nodal point. However, each node retains the value of the unbalanced flow at that node and each zone (element) retains its zonal flow vector.

At steady state conditions, the summation of unbalanced flow at these external boundary nodes should give the value of total seepage outflow through that external boundary. Computations of this nature were accomplished for the tailings main dam by writing a simple FISH (in FLAC3D) function.

6.2.9 Seepage Calculation at Interior Boundary

FLAC3D does not have a direct mechanism to compute flows across an interior boundary. However, each zone retains the flow vector for that zone and its geometry data. Scripts are written to obtain the flow component perpendicular to the boundary, which is denoted by a unit normal vector. This component is the dot product of the unit normal vector and the vector times the area of the zone in the direction of the boundary. The sum of the zonal flow perpendicular the boundary is the total seepage across the boundary.

Computations of this nature were accomplished for the tailings main dam by writing a simple FISH (in FLAC3D) function.

6.2.10 Simple analytical models

Analytical solutions are only available for simple geometry and homogenous problems and are not applicable to a complex seepage problem such as at the tailings main dam. Therefore, URS completed hand calculations using analytical solutions developed for simpler problems by Polubarinova-Kochina and Nelson-Skornyakov (Harr 1967; and Delleur, 1999) to make an order of magnitude comparison with the results obtained from the 3D seepage analysis. In addition, the 2D seepage models previously carried out by URS and Geomatrix (1999) were reviewed and compared.

6.3 2D SEEPAGE MODEL

The 2D seepage models using GEOFLOW were developed to evaluate the sensitivity of the seepage model for various parameters used in the 3D model. For the sensitivity analysis of the tailings main dam, the 2D model was developed only for the closure configurations with a 600-foot wide beach.

6.3.1 Model Geometry

The 2D seepage models were developed for the tailings main dam by using the simplified geometry described in Section 6.1.1. The 3D effects of the seepage were simulated by considering 2D models of sections along the length of the dam at the following stations:

- Station 4+20 (same as Station 21+34)
- Station 8+45 (same as Station 12+50)
- Station 9+00

The seepage under and around the entire tailings main dam was estimated by weight averaging the 2D seepage rates obtained at the above stations,

6.3.2 Material Properties

The materials used in the 2D model and the range of material property values are summarized in Section 7.1.2. For the sensitivity analysis, three parameters were identified as being potentially the most sensitive:

- Hydraulic conductivity of bedrock
- Hydraulic conductivity of tailings
- Depth to frozen ground

The following ranges of values were established for these parameters from the material properties summary on Table 4-1:

- Hydraulic conductivity of bedrock: $1x10^{-5}$ cm/sec (low) to $3x10^{-5}$ cm/sec (high)
- Hydraulic conductivity of tailings: $1x10^{-6}$ cm/sec (low) to $5x10^{-5}$ cm/sec (high)
- Depth to frozen ground: 10-foot aggradation to 20-foot degradation based on 2005 permafrost levels

6.3.3 Boundary conditions

The following boundary conditions were specified in the 2D seepage models for the tailings main dam:

- On the upstream boundary of the tailings impoundment, a “no-flow” boundary was specified
- On the downstream boundary of the tailings impoundment, hydrostatic pore pressures were specified assuming the water table downstream of the dam is at the surface level.
- Pore pressures downstream of the tailings main dam were specified. The water table was assumed to be at the surface level at each of the sections considered in the evaluation.
- A discontinuous phreatic surface was allowed in the model at the liner. The phreatic surface upstream of the liner under the tailings beach footprint and the phreatic surface downstream of the liner within the embankment footprint were allowed to change

- At the tailings impoundment, pore pressures were specified to values corresponding to the water height on top of the tailings;

6.4 MODEL CALIBRATION RESULTS

The 3D seepage model was calibrated for the Stage VI configuration of the tailings main dam. The seepage model was developed to represent the summer seepage conditions. The difference between winter and summer seepage was estimated using the 3D model to be about 100 gpm. Using this difference in summer and winter seepage and the summer seepage estimate from the 3D model, the winter seepage was computed for each analysis case. The following sections describe the calibration and predicted results for seepage under dam. Results are summarized on Table 6-1.

Table 6-1: Summary of Seepage Analysis Results

Case	Dam Crest El., feet	Tailings El., feet	Water El., feet	Flow Under Dam (A), GPM	Leakage through. Liner (B), GPM	Total Seepage (A+B) GPM
Calibration						
1) No-beach, Summer [1]	950	938	945	1326	308	1634
2) No-beach, Winter [1]	950	938	945	1226	308	1534
3) 600-foot beach, Summer [2]	950	938	940	327	98	425
4) 600-foot beach, Winter [2]	950	938	940	227	98	325
Stage VII-B						
5) 300-foot beach, Summer	960	955	955	595	179	774
6) 300-foot beach, Winter	960	955	955	495	179	674
Closure						
7) No-beach, Summer	986	975	980.2 [3]	2979	553	3532
8) No-beach, Winter	986	975	980.2 [3]	2879	553	3432
9) 300-foot beach, Summer	986	975	980.2 [3]	1496	202	1698
10) 300-foot beach, Winter	986	975	980.2 [3]	1396	202	1598
11) 600-foot beach, Summer	986	975	980.2 [3]	458	115	573
12) 600-foot beach, Winter	986	975	980.2 [3]	358	115	473
13) 900-foot beach, Summer	986	975	980.2 [3]	165	101	266
14) 900-foot beach, Winter	986	975	980.2 [3]	65	101	166

Notes:

[1] The "No-beach" calibration for summer and winter was computed for the period of June 2004 to January 2005.

[2] The "600-foot beach" calibration for summer and winter was computed for the period of January 2001 to July 2002.

[3] Closure water level consists of the maximum operating elevation at El. 975, plus 2.0 feet of water cover, plus 3.2 feet of spring runoff.

6.4.1 Stage VI, No Beach

The Stage VI – No Beach case represents actual conditions between June 2004 and July 2005. For modeling purposes, the upstream water level at El. 945 was used although the water elevation fluctuated a few feet during that period. At the start of the calibration process, the hydraulic conductivities of materials

were assumed to be those according to previous reports and studies as summarized in Table 4-1. After a series of time-consuming analyses with various hydraulic conductivities, the Table 4-1 values were used.

Figures 6-8 through 6-14 are plots of the results of a computer seepage analysis for the Stage VI- No-Beach condition during summer months. Pore pressure distribution and equipotential lines are shown in Figures 6-8 and 6-9, respectively, at a cross section at Station 8+00, where the underdrain is located.

The 3D effect on the seepage is shown on Figures 6-10 to 6-14 by flow vectors indicating the major trends in directions of water. Figures 6-11 and 6-12 are zoomed-up views around the interface between the liner and surrounding materials, rockfill drain and bedrock. Since the seepage model assumes that the liner provides a perfect seal, most of the water flows through rockfill material in front of the liner, goes into the bedrock, then quickly moves up and flows over the top of the bedrock.

The permeability of the rockfill material above the bedrock is several orders of magnitude higher than that of bedrock. Therefore, water finds an easier path in the underdrain and most of the flow passes through this material. Figure 6-11 shows the flow patterns in front of the liner face (upstream side). The section is cut perpendicular to the geomembrane surface to show that flow is directed down towards the underdrain and into the bedrock because of the pervious rockfill in front of the liner.

Figure 6-14 shows flow vectors of a section perpendicular to and behind the geomembrane. The flows are all towards the seepage collection pond, but going upwards as water migrates through the bedrock under the liner and tries to flow to the collection pond via the underdrain. The flow components along the dam axis are significant as shown in Figures 6-11 to 6-14, and verify that the 3D effect is a key factor on the seepage in the area. This suggests that flows in the third dimension along the dam axis cannot be ignored.

After completing the calibration, the seepage calculated under and around the dam for the Stage VI - No Beach case is 1,397 gpm. The phreatic surface downstream of the liner in the dam at the critical cross section was calculated from the results of the seepage analysis results and is shown on Figure 6-15.

6.4.2 Stage VI, 600-foot Beach

The Stage VI – 600-foot Beach case represents the actual conditions between January 2001 and July 2002. For modeling purpose, the upstream water elevation at El. 940 was used although the water level fluctuated during that period. Figure 6-16 and 6-17 show the pore pressure distribution and equipotential lines, respectively, at Station 8+00 for the Stage VI configuration of the tailings main dam.

After completing the calibration, the seepage under and around the dam for the Stage VI - No Beach case was estimated to be 277 gpm. The phreatic surfaces downstream of the liner within the dam, and upstream of the liner at the critical cross section, were calculated from the results of the seepage analysis and are shown on Figures 6-18 and 6-19, respectively.

6.5 SEEPAGE PREDICTION RESULTS

This section summarizes the results of the seepage analysis performed for the tailings main dam using the 3D model to predict seepage under the dam for Stage VII-B and closure conditions. Using the 3D model, seepage under the dam was predicted for summer conditions. The difference between winter and summer seepage was estimated using the 3D model to be about 100 gpm. Using the summer seepage results for under the dam, and the estimated difference in seepage between summer and winter, the winter seepage

for each analysis case was estimated. The following subsections describe the predicted results for seepage under the dam. Results are summarized on Table 6-1.

6.5.1 Stage VII-B, 300-foot Beach

For predicting seepage by means of the 3D model for the Stage VII-B configuration of the tailings main dam, an upstream water level at El. 955 was used, which corresponds to the maximum design water in the tailings impoundment. The Stage VII-B configuration includes a 300 feet wide tailings beach along the crest of the dam. Therefore a 300-foot case was the only case analyzed. Figures 6-20 and 6-21 are plots of pore pressure distributions and equipotential contours at Station 8+00, respectively.

The seepage under and around the dam for the Stage VII-B case was estimated to be 670 gpm. The phreatic surfaces downstream of the liner in the dam and upstream of the liner at the critical cross section were calculated from the results of the seepage analysis, and are shown on Figures 6-22 and 6-23, respectively.

6.5.2 Closure Stage

For predicting the seepage at closure by means of the 3D model, an upstream water level at El. 980.2 and crest at El. 986 were used. In order to evaluate the optimum beach width, various widths of beach were considered in the seepage analysis as follows:

- No-beach
- 300-foot beach
- 600-foot beach
- 900-foot beach

Seepage results corresponding to these beach widths at closure are discussed below and summarized on Table 6-1. The phreatic surfaces downstream of the liner in the dam and upstream of the liner at the critical cross section were calculated from the results of the seepage analysis and are shown on Figures 6-24 and 6-25, respectively.

6.5.2.1 Closure, No Beach

Figures 6-26 and 6-27 are plots of pore pressure distributions and equipotential contours at Station 8+00, respectively, for the no-beach condition. The seepage under the dam from the seepage analysis completed for closure case is 3,350 and 3,250 gpm under summer and winter conditions, respectively. The seepage results are summarized on Table 6-1.

6.5.2.2 Closure, 300-foot Beach

Figures 6-28 and 6-29 are plots of pore pressure distributions and equipotential contours at Station 8+00, respectively, for the 300-foot-wide beach condition. The seepage under the dam from the seepage analysis completed for closure case is 1,750 gpm in summer and 1,650 gpm in winter, which is approximately 50 % of the estimated seepage with no beach, or an approximately 50 % reduction in seepage from the no-beach condition. The seepage results are summarized on Table 6-1.

6.5.2.3 Closure, 600-foot Beach

Figures 6-30 and 6-31 are plots of pore pressure distributions and equipotential contours at Station 8+00, respectively, for the 600-foot-wide beach condition. The seepage under the dam from the seepage analysis completed for closure case is 600 gpm in summer and 500 gpm in winter, which is approximately 15 to 20 % of the estimated seepage with no beach, or an approximately 80 to 85 % reduction in seepage from the no-beach condition. The seepage results are summarized on Table 6-1.

6.5.2.4 Closure, 900-foot Beach

Figures 6-32 and 6-33 are plots of pore pressure distributions and equipotential contours at Station 8+00, respectively, for the 900-foot-wide beach condition. The seepage under the dam from the seepage analysis completed for closure case is 300 gpm in summer and 200 gpm in winter. This is approximately 5 to 10 % of the estimated seepage with no beach, or an approximately 90 to 95 % reduction in seepage from the no-beach condition. The seepage results are summarized on Table 6-1.

6.6 SENSITIVITY ANALYSIS RESULTS

In order to evaluate the significance and sensitivity of various model parameters on the seepage under and around the tailings main dam, URS completed sensitivity analyses on the models used for calculating seepage under and through the dam as described in Section 5.5. This section presents the results of the sensitivity analyses completed on seepage models for evaluating seepage under and around the dam.

A 2D seepage model was used to evaluate the sensitivity of the following model parameters:

- Hydraulic conductivity of bedrock and tailings
- Depth to permafrost.

A 3D seepage model was used to evaluate the sensitivity of a dry soil cover on the seepage.

The following subsections describe the results obtained from these sensitivity analyses.

6.6.1 Sensitivity to Hydraulic Conductivity

Variations in the hydraulic conductivities of the tailings and bedrock materials were considered in the sensitivity evaluation by using a 2D seepage model. Low, selected, and high hydraulic conductivity values for each material were selected and used in the analysis. Each analysis is labeled as low/low, high/low, low/high, high/high, and selected to represent the relative position within the range of hydraulic conductivity values of bedrock and tailings, respectively.

The hydraulic conductivity values and seepage results are shown in Table 6-2. The seepage rate results obtained from the sensitivity analysis are graphically shown on Figures 6-34 and 6-35, against the hydraulic conductivities of the bedrock and tailings, respectively.

The sensitivity analysis results were obtained by means of the 2D seepage model, which assumes that the flow in the third dimension is negligible. This is the direction perpendicular to the section and along the dam axis. Although this assumption is not representative of the conditions at the dam, the relative seepage rates show the relative influence of the material parameters on the actual seepage under the dam.

Table 6-2: Sensitivity of Seepage under Dam to Hydraulic Conductivity of Bedrock and Tailings

Case	Description [1]	Bedrock Hydraulic Conductivity		Tailings Hydraulic Conductivity		Seepage Rate	
		(cm/sec)		(cm/sec)		(gpm)	% [2]
1	Low/Low	1.00E-05	Low	1.00E-06	Low	734	51
2	High/Low	3.00E-05	High	1.00E-06	Low	1064	74
3	Selected	1.00E-05	Selected	2.80E-05	Selected	1436	100
4	Low/High	1.00E-05	Low	5.00E-05	High	1594	111
5	High/High	3.00E-05	High	5.00E-05	High	2860	199

Note: [1.] Description x/y, x denotes the range of bedrock permeability, y denotes range of tailings permeability
[2] Based on a seepage rate estimated using selected permeability values and depth of permafrost in 2005

The results are summarized graphically on Figures 6-34 and 6-35. The graphs show that rather than the actual seepage estimates, the relative variations of the seepage are more representative of the sensitivity of the hydraulic conductivity on the seepage. The results show possible variations of permeability may result in a 50 to 200 % variation in the seepage under dam estimate.

6.6.2 Sensitivity to Depth of Permafrost

Variations in the permafrost depth were considered in the sensitivity analysis by means of the 2D seepage model. As discussed in Section 4.3, the historical permafrost degradation and thaw bulb widening along and under the dam crest has slowed considerably over the last few years or possibly stopped. With the increased tailings beach width being considered for closure, it is assumed that the future permafrost degradation and thaw bulb widening will not be as significant as it was during the early stages of the tailings operations (Weaver, 2005). Therefore, the following permafrost degradation and aggradation conditions were considered to evaluate the sensitivity on the seepage at closure:

- 10 feet of aggradation (permafrost is 10 feet higher than 2005 levels).
- No change in permafrost surface (2005 permafrost surface)
- 10 feet of degradation (permafrost is 10 feet lower than 2005 levels)
- 20 feet of degradation (permafrost is 20 feet lower than 2005 levels).

The results of the seepage sensitivity analyses are shown in Table 6-3 and Figure 6-36. All other parameters, including the permeability values were kept the same as were used in the 3D models.

The sensitivity analysis results indicate that under the current permafrost conditions, the seepage rate is not very sensitive to reasonable changes that could occur in the permafrost depth.

Table 6-3: Sensitivity of Seepage under Dam to Changes in Permafrost Depth

Change in Permafrost Depth [2] (feet)	Seepage Rate	
	(gpm)	(%) [1]
10	1407	98%
0	1436	100%
-10	1457	101%
-20	1482	103%

Note: [1.] Based on a seepage rate estimated using depth to permafrost data in 2005.
[2] Depth to permafrost change (decrease) from 2005 data.

6.6.3 Sensitivity to Dry Soil Cover Seepage

One proposed closure option is the Dry Soil Cover (SRK, 2006a), for which the cover material, consisting of Okpikruak or Kivalina shale from the Main pit, would be placed over the tailings. Seepage from the Main Waste stockpile as well as the cover runoff will be diverted to the Aqqaluk pit area.

It is desirable that the cover layer has adequate thickness and moisture content to limit the entry of oxygen to the tailings. However, uncertainties about the material and difficulty of construction over tailings make it unlikely that the cover could function as a low permeability barrier (SRK, 2006a). The most rain falls at the same time that most evaporation occurs, it is unlikely that moisture could be stored and evaporated or taken up by plants during the rainy months (SRK, 2006a). Also, with a net positive water balance, the phreatic surface in the tailings could rise above or into the dry soil cover during wet periods or wet years.

For an alternate closure case of a dry cover over the tailings, SRK (2006b) estimated a net inflow of about 400 gallons per minute (gpm) into the tailings impoundment by taking a total impoundment surface area of approximately 700 acres, a mean annual precipitation of 20.7 inches, and the assumption of a net infiltration of 50% to allow for evaporation and runoff.

In order to investigate the effects of the above-mentioned inflow in the Dry Soil Cover option, URS performed a series of 3D seepage analyses to simulate the seepage condition of the tailings dam. The 3D model was based on that previously described in Section 6.2.

Since the computer model does not cover the entire tailings impoundment, this net water inflow was modeled with two components of boundary inflow as follows:

- Inflow from the top tailings surface boundary of the model
- Inflow from the upstream boundary of the model through highly weathered rock and tailings

The net 400 gpm inflow into the system through the tailings impoundment was assumed to flow towards the tailings main dam. Inflow boundary conditions at the top tailings surface boundary of the model and the upstream boundary of the model through the highly weathered rock and tailings were established to reflect this assumption. The two flow components were estimated to be the ratio of the actual top surface area of the tailings and that in the model, approximately 29.5 and 10.7 million square feet, respectively).

Figures 6-37 and 6-38 show the resulting pore pressure distribution and degree of saturation plots, respectively, based on the above-mentioned inflow components. The results show that the top tailings surface would be completely saturated starting from a plane approximately 1,100 feet upgradient of the liner all the way to the upstream boundary of the model (Figure 6-38). The pore pressure distribution shown in Figure 6-37 indicates that pore pressures are relatively high near the upstream boundary.

In order to evaluate the effect of inflow at the boundaries, an analysis was performed by applying all flow (400 gpm) from the top tailings surface of the model and zero-flow on the upstream boundary. Figures 6-39 and 6-40 are the pore pressure distribution and saturation contours, respectively, from this analysis. The results are similar to those from the previous analysis, except the reduction of pore pressures near the upstream boundary. This suggests that the two sets of prescribed inflow conditions yield similar results near the tailings main dam, and a net inflow of 400 gpm would likely cause the phreatic surface to rise above the dry cover.

For the contaminated water cover, the seepage under the dam was estimated to be 458 and 165 gpm for the 600 and 900-foot beaches, respectively. Therefore, the inflow rate is expected to be lower than 165 gpm to ensure that the phreatic surface would not rise to the dry cover. In other words, the dry cover condition should mimic an infinite beach width.

A series of trial and error analyses were performed to estimate the maximum inflow rate that would ensure an unsaturated soil layer below the dry soil cover. A maximum flow rate was found to be about 40 gpm. Figures 6-41 and 6-42 show the pressure distribution and saturation contours for the analysis with the 25 GPM inflow. Figure 6-43 compares the phreatic surface elevations at the rockfill-tailings interface for different cases analyzed as well as the previously computed 900-foot beach water cover case.

In order to verify the results described above, simplified hand calculations were completed based on an analytical solution. Todd (1980) describes factors that theoretically affect the drawdown curve in an unconfined aquifer as being hydraulic conductivity and flow rate. By estimating an effective hydraulic conductivity of the entire system using the results of the previous beach cases (600 and 900-feet), this simplified approach provided relatively consistent results of a 25 to 40 gpm flow rate as the upper limit.

The seepage analyses described in this section represent steady-state flow condition. However, because the hydraulic conductivities of the materials are low, the time lag would be large, but was not accounted for in these calculations. The calculations assumed a net constant flow into the system over a long time, which is a simplistic because of alternating dry and wet periods, evaporation, and water intake by plants

In summary, it will be very difficult to actually keep the dry cover dry. There will be seasonal fluctuations of rainfall and snow, as opposed to steady assumed conditions of average precipitation and infiltration rates. In reality, the infiltration rates will be higher during snowmelt, resulting in an even more likelihood that the surface of the cover would occasionally be inundated.

7.0 SEEPAGE THROUGH THE DAM

The component of seepage through the tailings main dam would result from any potential breach in the liner system. The most significant part of the seepage through a liner system primarily occurs as leakage through holes in the geomembrane. A relatively minor part of the seepage could occur as fluid permeation

through the intact geomembrane. For the main dam, URS considered only the leakage through holes in the geomembrane, and estimated leakage rates using an approach proposed by Bonaparte et al. (1989).

7.1 LEAKAGE ANALYSIS MODEL

7.1.1 Leakage Equation

The rate of leakage through a HDPE geomembrane consists of two primary components as follows:

- Leakage due to geomembrane permeability
- Leakage due to geomembrane defects (leak hole)

Leakage due to geomembrane permeability is negligible compared to leakage through defects in the geomembrane (Giroud and Bonaparte, 1989; and Giroud et al, 1994). Hence, the rate of leakage through a geomembrane is approximated with the rate of leakage through the leak holes in the geomembrane. The equation for evaluating leakage through the geomembrane defects depends on several factors, including:

- Head difference across the geomembrane liner
- Quality of the contact between the geomembrane and the underlying bedding material

Leakage equations used for landfill liner systems are appropriate for low head differences across the geomembrane in the tailings main dam. The head difference across the liner system in the dam could range from 0 to 150 feet depending on factors such as pond level, beach width, phreatic surface and liner location. Therefore, the leakage equation should be suitable for high head differences across the liner.

The liner bedding helps impede the flow through leaks in the geomembrane. Therefore,, the quality of the geomembrane contact with the liner bedding should be considered in estimating the leakage. Leakage equations applicable for high head conditions and different quality of contact were proposed by Giroud et al. (1994) as follows:

- Poor Contact: $Q = 1.15 i_{avg} a^{0.1} h^{0.9} k^{0.74}$ ..... (7-1)
- Good Contact: $Q = 0.21 i_{avg} a^{0.1} h^{0.9} k^{0.74}$ ..... (7-2)
- Perfect Contact: $Q = 4hk\sqrt{a/\pi}$ ..... (7-3)

where: Q is the seepage rate in cubic meters per second

a is the area of the leak hole in square meters
h is the head difference across the liner in meters
k is the hydraulic conductivity of the liner bedding in meters per second
i_{avg} is a dimensionless factor dependant on thickness of underlying material (D) and h .

The i_{avg} value is equal to 1 when *h* is less than or equal to *D*, and greater than 1 when *h* is greater than D. The thickness of rockfill beneath the geomembrane varies greatly, but for most areas of the liner, an assumption of *h* greater than *D* is appropriate. Therefore, in order to simplify the leakage calculation, the value of i_{avg} was assumed to be 1.0 for the entire geomembrane within the tailings dam.

During construction of the tailings dam, strict construction quality control and assurance (CQC and CQA) procedures were in place for the HDPE geomembrane installation. Therefore, a reasonably good contact condition is expected between the liner bedding and geomembrane. With the liner bedding and cover materials having relatively high permeability, a perfect condition is not realistic for the liner system In the tailings main dam.

In summary, the leakage equation (7-2) corresponding to good contact conditions between liner bedding and geomembrane, with i_{avg} equal to 1.0, was used to calculate the leakage through leak holes in the geomembrane

7.1.2 Average Size and Number of Leak Holes

The number and size of the leak holes can vary depending on the quality of the material under and above the geomembrane, experience level of installer, and CQC and CQA during the installation. Because this information is site specific and cannot be determined after the liner is placed and covered, an estimate based on available literature was necessary. This is the typical method of analysis.

7.1.2.1 Average Number of Leak Holes

For the leakage calculations that were completed for the tailings main dam, representative values of the number and average size of holes were estimated from documented surveys of leaks on landfill liners. The average number of holes reported in early literature (Collucci and Lavagnolo, 1995; Rollin and Jacquelin, 1998; and Rollin et al., 1999), generally varied from 3 to 40 leaks per acre, corresponding to high and low quality subgrade and cover materials, respectively. Average number of leaks as high as 100 leaks per acre are reported in literature for low quality sub-grade materials (Colluci and Lavagnolo, 1995).

More recently (Giroud and Touze-Foltz, 2003), a panel of experts agreed that the number of holes in a geomembrane after installation and before cover materials are placed, ranges from 1 to 2 per acre (1 to 5 per hectare). The number of holes after placement of the cover depends on the cover material quality, overburden stress and equipment loading. Data collected in the last decade shows that 70 to 80 % of the holes are created from the placement of materials on the geomembrane. The panel agreed that the number of holes created from such placement of materials could range from very few to 8 per acre (very few to 20 per hectare). This results in total number of holes ranging from 1 to 10 per acre (1 to 25 per hectare).

It should be noted that the literature data is generally applicable to liner systems in landfills, which are different from the liner system in the tailings main dam. The primary differences are:

- **Geomembrane Thickness**: Geomembranes in typical landfill covers is about 60 mils thick, while the thickness of the geomembrane in the tailings main dam is 100 mils.
- **Maximum Particle Size**: For the landfill liners the maximum particle size for the bedding and cover materials that are in direct contact with the geomembrane is typically limited to ¼-inch. For the tailings main dam liner system, the maximum particle size specified is 1-inch.

A full scale test may be used to evaluate the performance of the 100 mils thick HDPE geomembrane in contact with 1-inch bedding and cover materials. For the purpose of this leakage analysis, the following factors were considered in estimating the average number of holes in the geomembrane:

- From the landfill cover experience, the total number of holes in the geomembrane is expected to range from 1 to 10 per acre
- The thickness of the geomembrane (100 mils) is higher than the thickness of geomembranes used in the landfill covers
- The maximum particle size of materials that are in contact with the geomembrane is typically limited to ¾-inch, in landfill covers
- The maximum particle size of materials that are in contact with the tailings dam geomembrane is limited to 1-inch
- The possible adverse effect of having a higher maximum particle size materials in contact with the geomembrane is mitigated by the use of a thicker geomembrane
- The liner system and the embankment were constructed in eight stages from 1988 to 2006 with varying levels of experience over this time and during some extreme climate conditions.
- Improved liner installation construction techniques and levels of CQC and CQA were accomplished between 1988 and 2006.

In order to complete the leakage calculations in the tailings main dam, an average of 4.9 leaks per acre (12 leaks per hectare) was considered as being a reasonably representative estimate.

7.1.2.2 Average Size of Leak Holes

The size of holes historically recommended by Giroud and Bonaparte (1989) that is still being used by many designers today is as follows:

- 0.15 square inches (1 cm^2) for design
- 0.015 square inches (0.1 cm^2) for performance evaluation

These sizes were proposed for holes in the installed geomembrane before the placement of cover material. With recent CQA practices, 0.015 square inch (0.1 cm^2) holes are more realistic than the 0.15 square inch (1 cm^2) size (Giroud, 2006). However, for holes resulting from the placement of the cover material, a larger size can be used. For this analysis, a distribution of holes with varying size depending on likely causes were considered (Giroud, 2006) and average hole sizes were computed as shown in Table 7-1.

The average values in the literature are based on condition surveys at the time of liner construction. It is expected that after installation, the HDPE geomembrane could deteriorate because of stresses due to dam settlement, loading from increasing dam and tailings heights, and seismic deformations of the dam. However, geomembrane installation and CQA practices have improved with each raise of the dam, which would lead to less and smaller holes in each raise. Therefore, it was assumed that the long-term performance of the liner does not significantly increase the size and number of holes in the geomembrane.

For the tailings main dam leakage calculations, an average leak hole size of 0.126 square inch (0.81 cm^2) was considered as being a reasonably representative estimate.

Table 7-1: Estimate of Average Size of Leak Holes

Cause	Size of Hole (square inch [cm^2])[1]	No. of Holes (holes/acre [holes/hectare])[1]
Geomembrane installation	0.015 [0.1]	1.2 [3]
Placement of material overlying geomembrane	0.015 [0.1] to 0.15 [1.0] Average 0.083 [0.55]	3.4 [8]
Equipment tearing of geomembrane	0.75 [5.0]	0.4 [1]
Average Hole Size	0.126 square inch [0.81 cm^2]	

Notes: [1] Equivalent metric units are within square brackets [].

7.1.3 Hydraulic Head

As shown by the empirical equation in Section 7.1.1, the rate of leakage is proportional to the hydraulic head difference across the liner, which depends on the phreatic surfaces immediately upstream and downstream of the liner. For the different analysis cases considered, these phreatic surfaces could be estimated from the 3D seepage analysis. The upstream phreatic surfaces depend on the width of the tailings beach and were calculated using the results of the seepage analysis. For simplicity and convenience, the downstream surface was assumed to be at the downstream ground surface.

7.1.4 Submerged Liner Area

The number of leak holes is expressed as number of holes in the geomembrane for a given area of liner. Therefore, the leakage estimate also depends on the area of the liner that is in contact with water and is subject to a difference in head. On the basis of the phreatic surfaces obtained for different 3D seepage analysis cases, the submerged liner area in the tailings main dam was estimated for each 3D analysis as summarized on Table 7-2.

Table 7-2: Estimate of Submerged Liner Area

Case	Submerged Liner Area ($feet^2$)
Calibration	
1) No-Beach	681,553
2) 600-foot Beach	455,430
Stage VII-B	
3) 300-foot Beach	570,080
Closure	
4) No Beach	926,083
5) 300-foot Beach	609,084
6) 600-foot Beach	498,330
7) 900-foot Beach	477,361

7.2 LEAKAGE ANALYSIS RESULTS

Using the leakage analysis model and the phreatic surface results obtained from the 3D seepage analysis for the tailings main dam, the component of seepage through the dam were computed. The calculations showing the leakage analyses for different 3D seepage analyses are summarized in the following sections.

7.2.1 Model Calibration Results

Both components of seepage through the tailings main dam and under and around the dam were computed separately. The combined total seepage was compared with the pumpback data to calibrate the 3D seepage model. A summary of the components of seepage through the dam at the end of the model calibration are presented below and in Table 6-1.

- **Stage VI, No Beach**: The rate of seepage through the dam at the end of the iterative model calibration is 308 gpm
- **Stage VI, 600-foot Beach**: The rate of seepage through dam at the end of the iterative model calibration is 98 gpm

7.2.2 Prediction Results

Once the 3D model was calibrated and a set of model parameters were established, the seepage for future conditions of the tailings main dam were predicted for the Stage VII-B and closure conditions using the calibrated model parameters. A summary of the components of seepage through the dam predicted for different future analysis cases considered is presented below and in Table 6-1:

- **Stage VII-B, 300-foot Beach**: The rate of seepage through the dam is 179 gpm
- **Closure, No Beach**: The rate of seepage through the dam is 553 gpm
- **Closure, 300-foot Beach**: The rate of seepage through the dam is 202 gpm
- **Closure, 600-foot Beach**: The rate of seepage through the dam is 115 gpm
- **Closure, 900-foot Beach**: The rate of seepage through the dam is 101 gpm

7.3 SENSITIVITY ANALYSIS RESULTS

In order to evaluate the sensitivity of various parameters on the component of seepage through the tailings main dam, sensitivity analyses were completed on the calculations used for estimating leakage through the HDPE geomembrane as described in Section 5.5 A 2D seepage model was used to evaluate the sensitivity of the following parameters:

- Number of leaks in the liner
- Average hole size

The following subsections describe the results obtained from these sensitivity analyses.

7.3.1 Sensitivity to Number and Size of Leak Holes

Variations in the number of leaks in the HDPE geomembrane were considered in the sensitivity evaluation by using the leakage analysis model described in Section 7.1. Based on the range of values established for the number of holes and average size of holes in the geomembrane, values of low, selected, and high were selected and used in the leakage analysis for 600-foot beach condition at closure.

Each analysis is descriptively called as low/low, high/low, low/high, high/high, and selected to represent the relative position within the range of the number of leaks in the HDPE geomembrane, and the average size of the leaks, respectively.

The leak densities and average size of holes used in the sensitivity analysis and the corresponding seepage results are shown in Table 7-3. The leakage results obtained from the sensitivity analysis are graphically presented against number of leak holes and average hole size on Figures 7-1 and 7-2, respectively.

Table 7-3: Sensitivity of Seepage through Dam to Number and Size of Leak Holes

Case	Description [1]	Number of Leaks		Average Hole Size		Rate of Seepage through Dam	
		(Leaks/Acre)	(Leaks/ 10,000 feet^2)	Area (inch^2)	Radius (inch)	(gpm)	(%)[2]
1	Low/Low	1.6	0.37	0.0016	0.022	25	21
2	High/Low	16.2	3.72	0.0016	0.022	246	215
3	Selected	4.9	1.11	0.1256	0.200	115	100
4	Low/High	1.6	0.37	0.1550	0.222	39	34
5	High/High	16.2	3.72	0.1550	0.222	391	340

Note: [1.] Description x/y, x denotes the range of number of leak holes, y denotes range of average hole size
[2.] Percentage based on a seepage rate estimated using "selected" number and average size of leak holes

The results from the sensitivity analysis show the relative influence of the liner parameters on seepage through the tailings main dam. The results show that possible variations of number of leak holes and average hole size within the range of values established may result in a 10 to 1000 % variation in the seepage estimated through the dam.

7.3.2 Sensitivity to No Liner in Dam

The lifetime of a HDPE geomembrane is generally defined as the time in which the physical and mechanical properties of the geomembrane would degrade by 50% of its initial values. Recent literature (Koerner et al., 2005) and personal communications with geosynthetic experts (Koerner, 2006; Wallace, 2006; and Giroud, 2006) indicates that the lifetime of a HDPE geomembrane ranges from:

- 510 years for buried geomembrane at 6.0^{o}C (43^{o}F) and exposed to semi-aggressive liquids;, to
- 1600 years for buried geomembrane at -2.3^{o}C (28 ^{o}F) and not exposed to liquid.

The subject of geomembrane longevity and durability is discussed in more detail in the conceptual design report of the tailings dam future raises to closure (URS, 2007c).

Colder climates and protected (buried) conditions increase the life of HDPE geomembranes. However, the closure condition will remain in perpetuity, so a potential ultimate failure of the geomembrane was considered. In order to evaluate this extreme condition, a completely degraded liner or "no liner" condition was considered in a sensitivity analysis by using the 3D seepage model outlined in Section 6.2.

The functional purpose of the tailings main dam to retain tailings and water is primarily accomplished by the HDPE geomembrane within the dam and the low permeability tailings in the impoundment. A wide

tailings beach is required to sufficiently lower the phreatic surface upstream of the liner, reduce the seepage through and under the dam, and improve the stability of the embankment

The extreme event of a failure and non-functionality of the HDPE geomembrane can be mitigated by either a wider tailings beach or a perimeter tailings beach around a larger part of the impoundment away from the dam. A 900-foot wide tailings beach parallel to the crest of the dam was considered for the sensitivity evaluation of the "no liner" condition

For this extreme no liner condition, the seepage would occur predominantly through the dam. To be consistent with the definitions of seepage through and under dam, all seepage calculated from the no liner model is referred to in this report as seepage through the dam instead of seepage under the dam.

For this extreme case of no liner and 900-foot wide beach condition at closure, the sensitivity analysis was completed using the 3D model and the following results obtained:

- Seepage under the dam was estimated for the "no liner" case to be 2200 gpm in summer and 2100 gpm in winter. Seepage under the dam for the corresponding case with liner intact and fully functional is 300 gpm in summer and 200 gpm in winter (Section 6.5.2.4). Therefore, if the liner were to fail, the seepage rate would increase approximately 7 to 11 times in summer and winter, respectively.
- Figures 7-3 and 7-4 show the plots of pore pressure distributions and equipotential contours across the dam at Station 8+00, respectively. The down stream phreatic surface is graphically shown in Figure 7-5.

For the "no liner" analysis, the hydraulic conductivity of the rockfill in the dam was increased by 10 times the value in Table 4-1 in order to speed up the computer run. Because the ratio of hydraulic conductivity between rockfill and tailings is several orders of magnitude, this increase does not have a major impact on the results, but saves a considerable amount of analysis time.

The pore pressure drops in the tailings as a result of the beach effect in the same way a sit does in the previous cases with the liner in place. It is expected that the contrast in permeability should cause the pore pressure level to drop deeper at the tailings to rockfill interface for a normal plane strain problem. However, the analysis results suggest that the 3D effect plays a more dominant role in the pore pressure distribution in the rockfill embankment.

In summary, because of the valley-shaped topography of the original ground and bedrock surfaces, and the relatively pervious nature of the rockfill embankment, groundwater from the two abutment areas of the dam tends to flow towards the deepest section through the underdrain. This flow component parallel to the dam axis causes the equalizing effect of pore pressure level higher than expected phreatic surface along this section through the underdrain.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

Based on the results of the seepage analyses presented in this report and summarized in Figure 8-1, URS concludes the following:

- A tailings beach on the tailings alongside the upstream slope of the dam is an effective means of reducing the total seepage, as shown on the graph above. The benefit of a beach in reducing seepage at closure can be described as follows:
  - No-beach - Estimated seepage is approximately 3,540 gpm in summer and 3,440 gpm in winter.
  - 300-foot-wide beach - Estimated seepage is approximately 1,700 gpm in summer and 1,600 gpm in winter, which is approximately 50 % of the estimated seepage with no beach, or a 50 % reduction in seepage from the no-beach condition.
  - 600-foot-wide beach – Estimated seepage is approximately 600 gpm in summer and 500 gpm in winter, which is 15 to 20 % of the estimated seepage with no beach, or an 80 to 85 % reduction in seepage from the no-beach condition
  - 900-foot-wide beach – Estimated seepage is approximately 300 gpm in summer and 200 gpm in winter, which is 5 to 10 % of the estimated seepage with no beach, or a 90 to 95 % reduction in seepage from the no-beach condition
- The difference between winter and summer seepage rates is primarily from the shallow seepage and ground water flow from the abutments, which was estimated to be about 100 gpm. Pumpback data shows a similar magnitude of difference, when the effects of precipitation and permafrost degradation are not reflected on the pumpback data.
- The pumpback rate is generally higher in summer than winter. The difference depends on changes in tailings operations, climate, freshets, surface runoff, groundwater depth, active layer and permafrost degradation. In years of no significant permafrost degradation, the maximum difference between winter and summer pumpback was about 600 gpm.
- The estimated maximum summer runoff from rain and snow is 600 gpm for a rain-on-snow event. The design criteria is 2.5-inches for a 100-year 24-hour rainfall. The maximum daily snowmelt using a maximum observed snow-water equivalent is 7.44 inches at the end of the 1993-94 winter and the corresponding maximum snowmelt in one day is 2.97 inches.
- The calibration of the 3D model relied heavily on the pumpback records. The pumpback water includes seepage through and under the dam, and seasonal water from other sources such as surface runoff, shallow groundwater, melt water from thawing frozen ground, and water released by “ice dams” that develop on the surface of the active layer as it thaws.
- The phreatic surface in the tailings and in the rockfill buttress upstream of the liner system is mostly influenced by the width of the tailings beach, the horizontal hydraulic conductivity of the tailings, and possibly the hydraulic conductivity of the weathered shale bedrock beneath the tailings impoundment.
- Permafrost under the dam has degraded since the start of tailings operations in the late 1980s by up to 80 and 50 feet vertically on the west and east abutment, respectively. The thaw bulb under the creek has widened by approximately 250 feet, consisting of 200 feet to the east and 50 feet to the west.

- The dam thermistor records indicate that the degradation of permafrost and widening of the thaw bulb along and beneath the dam footprint has slowed considerably over the last few years and may have ceased altogether.Therefore it is reasonable to assume a smaller rate of future permafrost degradation and thaw bulb widening.
- The historical thermistor and seepage pumpback records show that the seepage pumpback rate increased with increasing depth to permafrost beneath the dam. The increase in pumpback rates, and hence the seepage rates, were more pronounced, whenever significant permafrost degradations were noted beneath the dam over a short period of time.
- Future changes in the depth to permafrost will depend on the future construction activities around the dam such as the raises to closure, modifications to the seepage collection system, and changes in tailings management such as the width of the tailings beach and method of cover over the tailings impoundment at closure.

URS developed the following conclusions on the sensitivity to seepage of variations in key material parameters, the extreme worst case of a failed geomembrane in the dam, and an alternate possible closure condition of a dry cover over the tailings:

- Estimated seepage rates are sensitive to changes in hydraulic conductivities of bedrock and tailings. The hydraulic conductivity ranges considered for bedrock ($1x10^{-5}$ to $3x10^{-5}$ cm/sec) and tailings ($1x10^{-6}$ to $5x10^{-5}$ cm/sec) resulted in a 50 to 200 % variation in seepage under dam. However, URS concludes that the parameters selected are appropriate.
- Estimated seepage rates are not very sensitive to changes in current permafrost conditions. Up to 10 feet of aggradation and 20 feet of degradation from the 2005 permafrost surface were considered, and resulted in a 98 to 103 % variation in seepage under the dam. Therefore, URS concludes that the parameters selected are appropriate.
- Leaks in the geomembrane will impact seepage through the dam depending on the number and size. URS estimated leak numbers and sizes from other project histories, industry standards, published liner performance data, tailings dam liner installation quality control, and integrity of geomembrane exposed during Stage VII-B cutoff wall excavations.
- Use of extreme published values of geomembrane leak number and size would result in a 10 to 1000 % variation in seepage through the dam. The leak numbers and sizes selected by URS, in combination with other parameter discussed above, resulted in total seepage rates through the tailings main dam that are consistent with the pumpback records.
- In the extreme worst case of a failed geomembrane that is no longer functional, the phreatic surface in the dam would rise and seepage through the dam for a 900-foot wide beach would be about 2100 gpm in winter and 2200 gpm in summer, which are 7 to 11 times the seepage estimated for a 900-foot beach with intact liner.
- If the geomembrane did fail and was no longer functional, a relatively wide or perimeter tailings beach would be needed to keep the phreatic surface in the dam from rising and reducing the stability of the dam, and to retain the tailings water behind the dam. Dam stability and geomembrane durability are discussed in separate URS reports.
- For an alternate closure case of a dry cover over the tailings, a net inflow of 400 gallons per minute (gpm) into the tailings was estimated by taking the mean annual precipitation of 20.7 inches and assuming a net infiltration of 50% to allow for evaporation and runoff. This inflow would cause the phreatic surface in the tailings to rise above the cover.
- A net inflow of less than 40 gpm to the tailings impoundment would be required to ensure that the phreatic surface in the tailings will remain below the cover. This would be very difficult to

achieve because it would require that only 5 % of the direct precipitation could remain in the tailings area and the other 95 % would need to be run off the tailings area.

8.2 RECOMMENDATIONS

From the seepage analysis results and conclusions, URS has developed recommendations for seepage control of the tailings main dam future raises to closure as follows:

- Monitoring of the phreatic surface in the rockfill and tailings upstream of the liner will help to provide a year-round indicator of total seepage. The monitoring would provide upstream water level data, in combination with downstream pumpback data, for future verification of the seepage model and updates of the seepage estimate at closure.
- It is recommended that a tailings beach as wide as practically possible be maintained alongside the entire upstream slope of the dam. The efforts to construct and maintain a wider beach will be offset by the need to pump less seepage back to the tailings impoundment. The beach would need to be covered with rock for dust control.
- In order to monitor the upstream phreatic surface in the rockfill and tailings upstream of the liner at closure, it is recommended that piezometers be installed in the tailings under the beach near to and away from the crest of the dam. The piezometers should be installed in lines that are continuations of the current lines of piezometers.
- If a wide tailings beach is not desired for closure, consideration could be given to installing either a barrier wall or grout curtain into competent low permeability bedrock along the toe of the dam upstream of the liner, or a tailings beach further around the impoundment perimeter. The barrier must extend below the cutoff wall, and above the phreatic surface.
- If a dry soil cover option is chosen, the maximum net inflow rate into the tailings impoundment must be kept less than 40 gpm to ensure that the phreatic surface does not rise to the top of tailings and into the dry soil cover. This would be very difficult, if not impossible, to achieve.

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FIGURES

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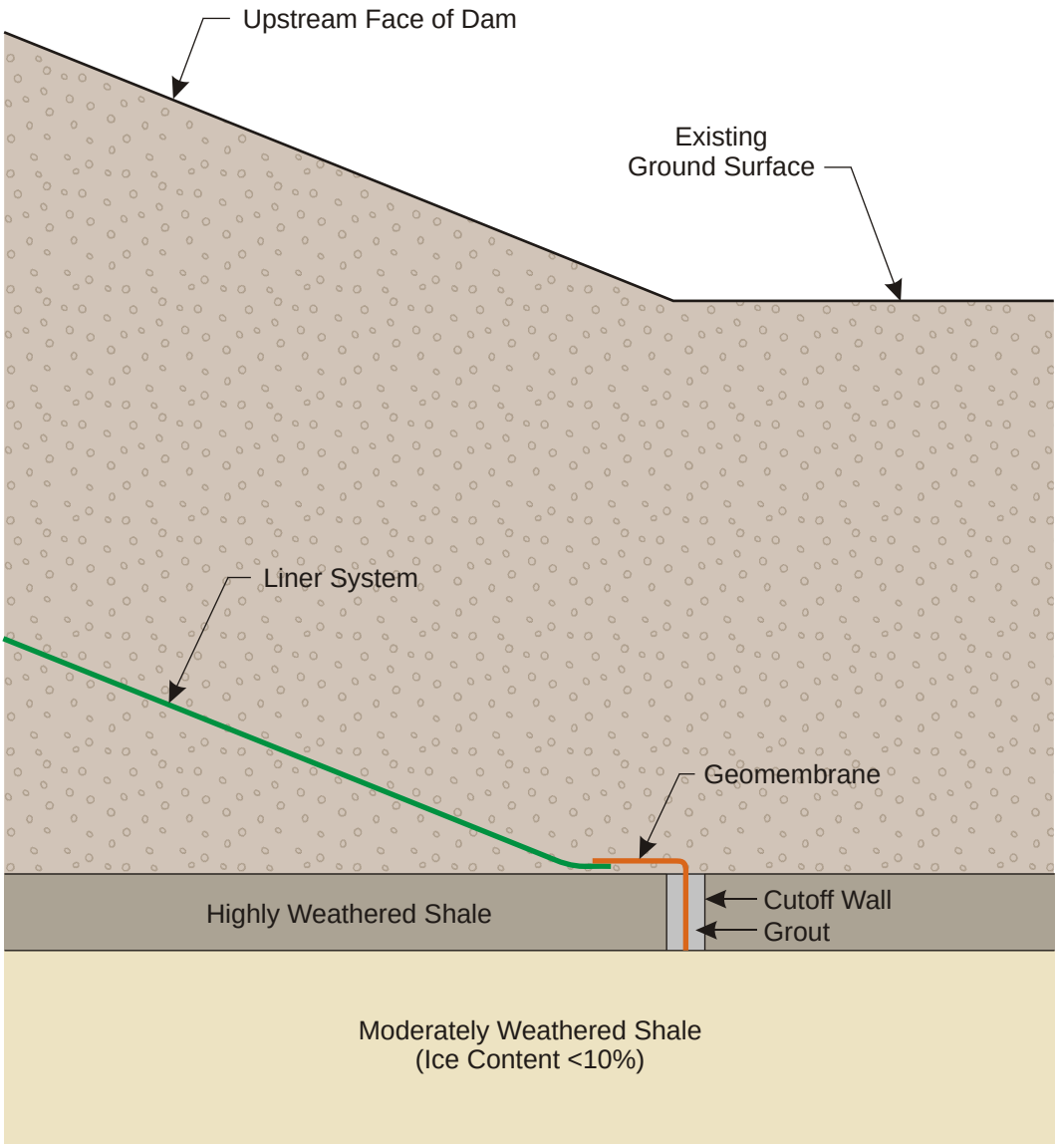

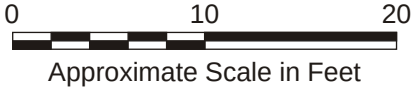

Figure 3-1
Cutoff Wall
(Stages II, III, IV and VII-B)

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

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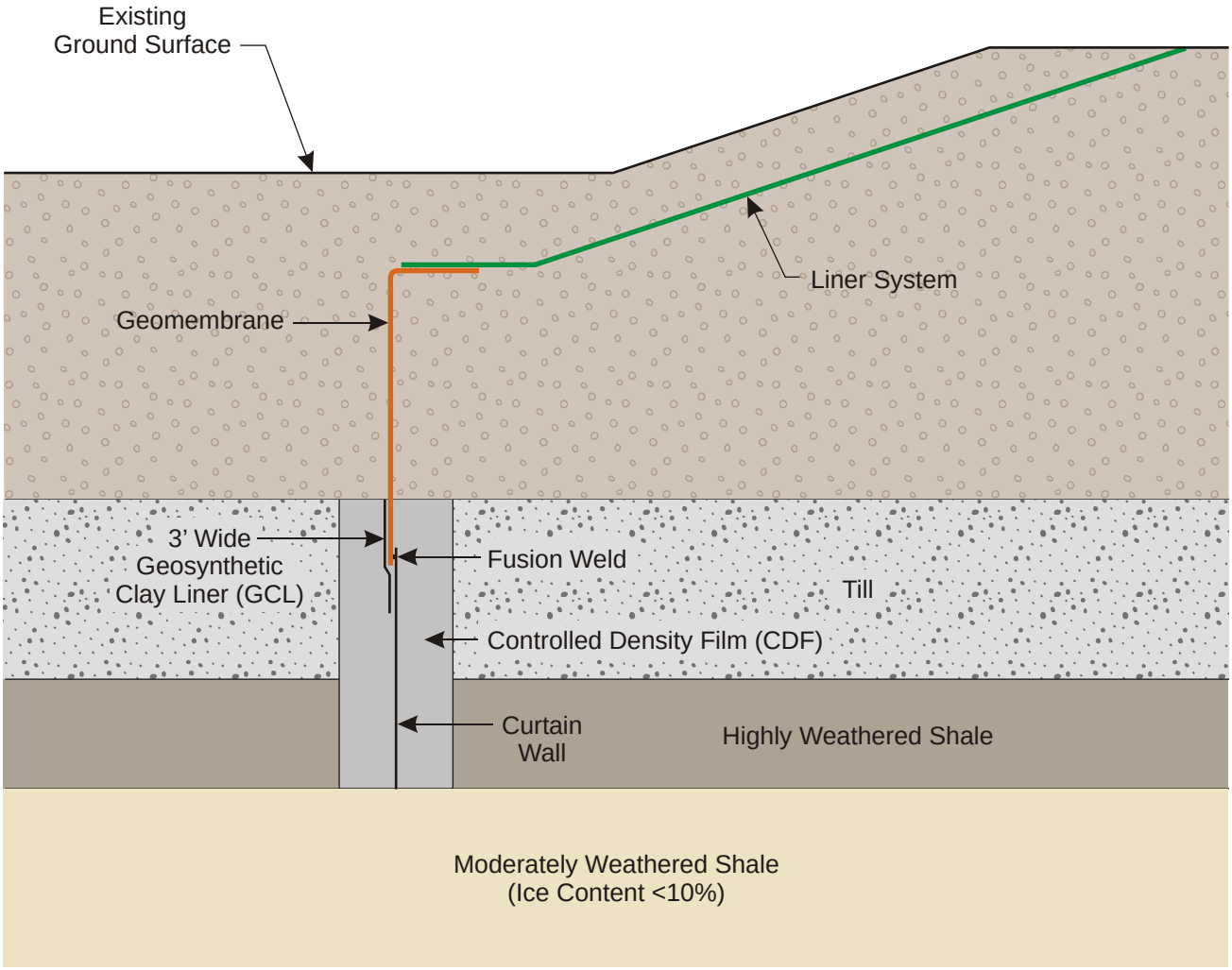

Not to Scale

Figure 3-2
Wing Wall for Stage VII-B and Future Raises to Closure

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

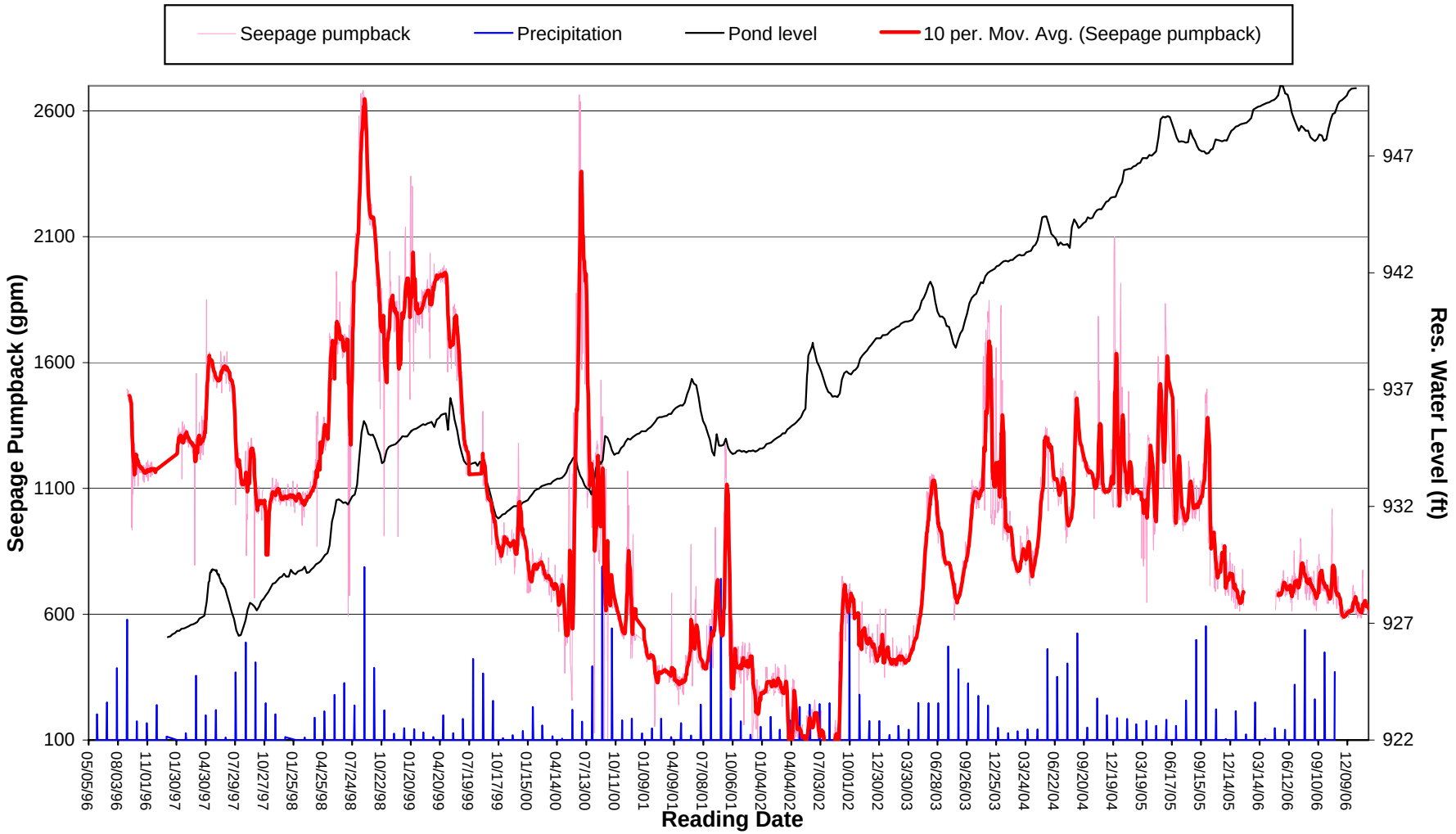

Figure 3-3

Seepage Pumpback Record

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

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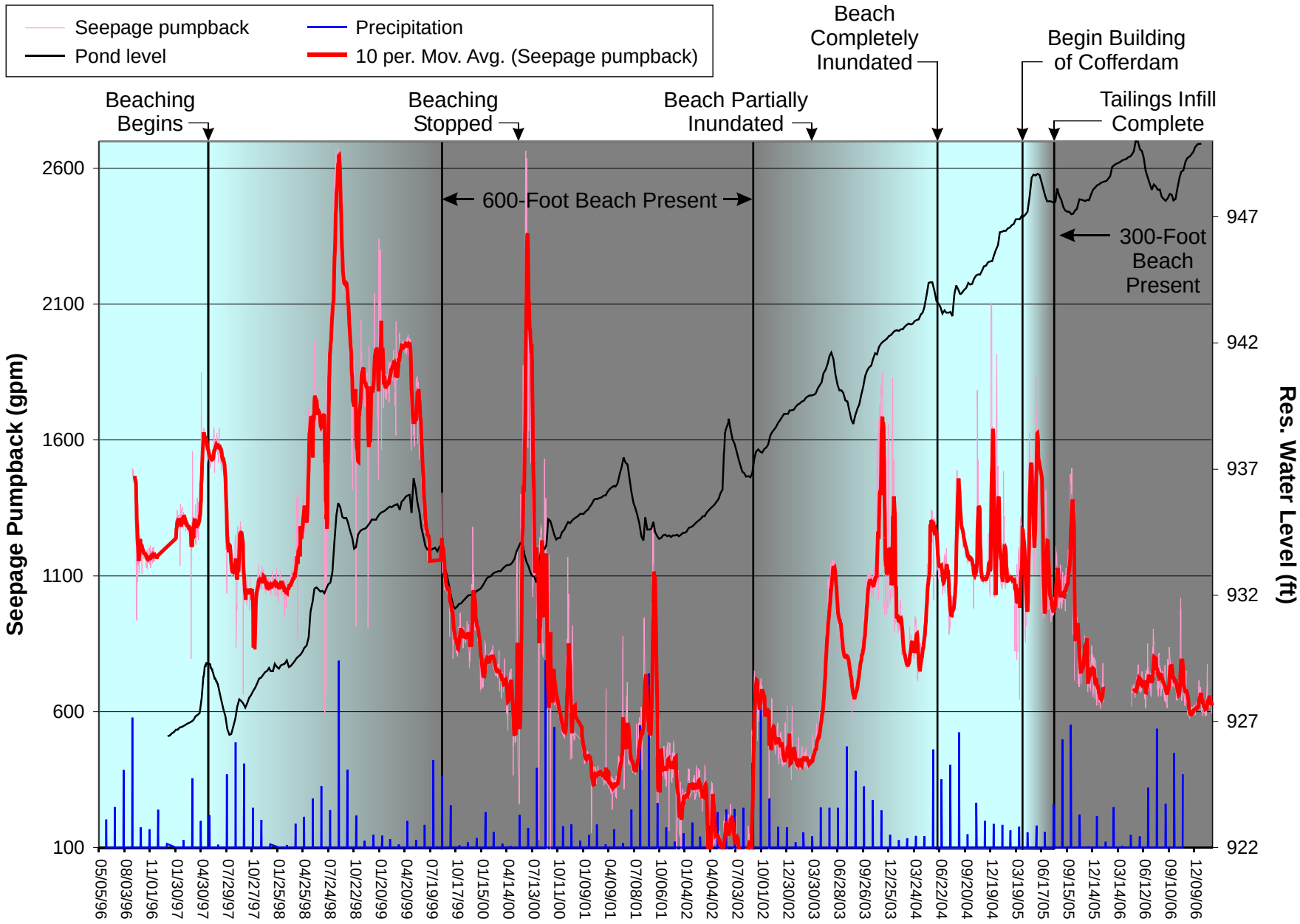
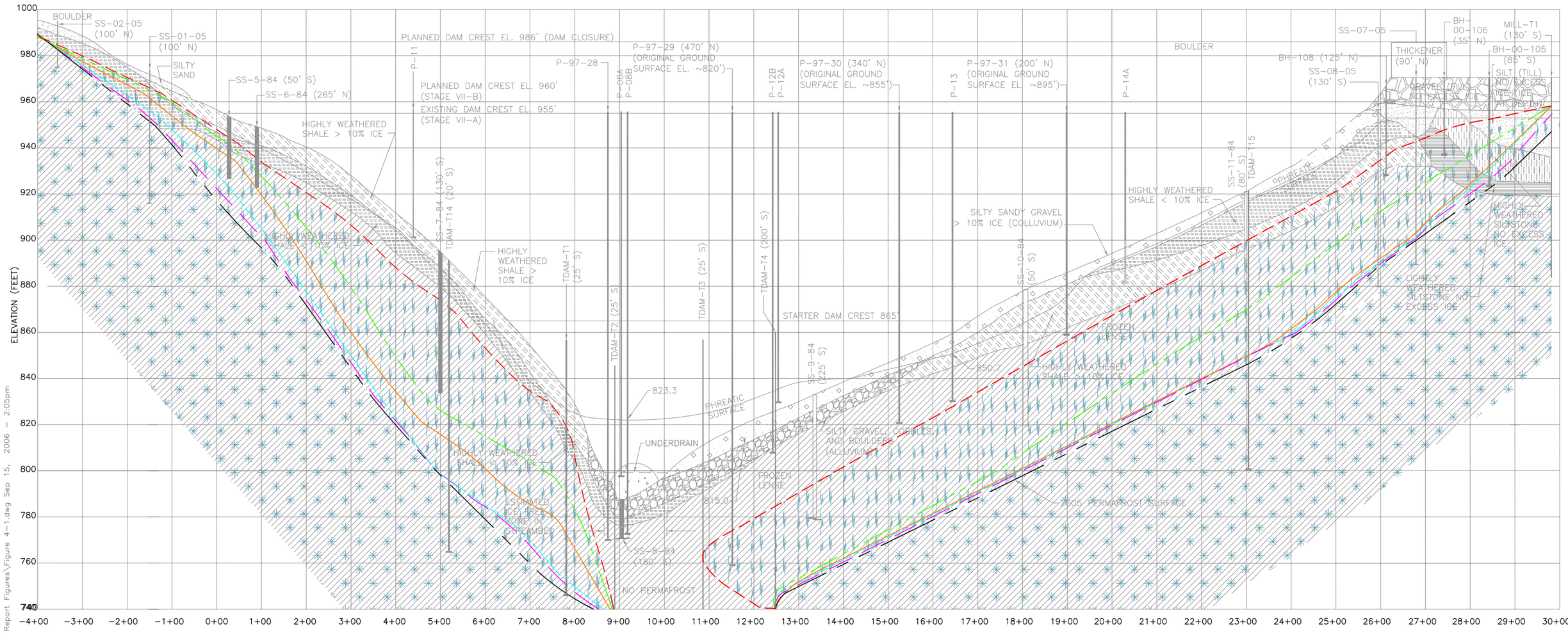


Figure 3-4

Historical Effectiveness of Tailings Beach on Seepage

Job No. 33757098



100 0 100 200
HORIZONTAL SCALE IN FEET

20 0 20 40
VERTICAL SCALE IN FEET

LEGEND

- 1990
- 1995
- 1997
- 2000
- 2002
- 2005

PERMAFROST DEGRADED BETWEEN 1990 AND 2005

PERMAFROST IN 2005

SLIGHTLY TO MODERATELY WEATHERED SHALE

P:\ACAD\PROJECT\TeckCominco Alaska\Seepage Report Figures\Figure 4-1.dwg Sep 15, 2006 - 2:05pm

Job No. 33757098.00001

Figure 4-1

Historical Permafrost Degradation along Dam Crest

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS

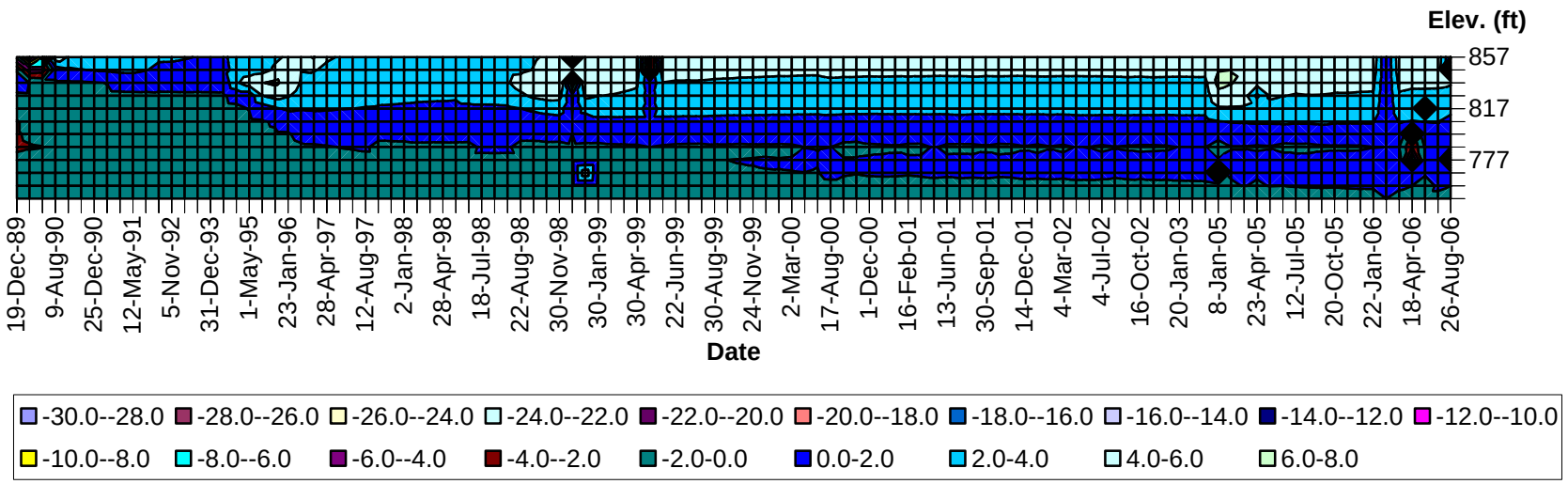


Figure 4-2

Historical Temperature Profile at Thermistor TDAM – T1

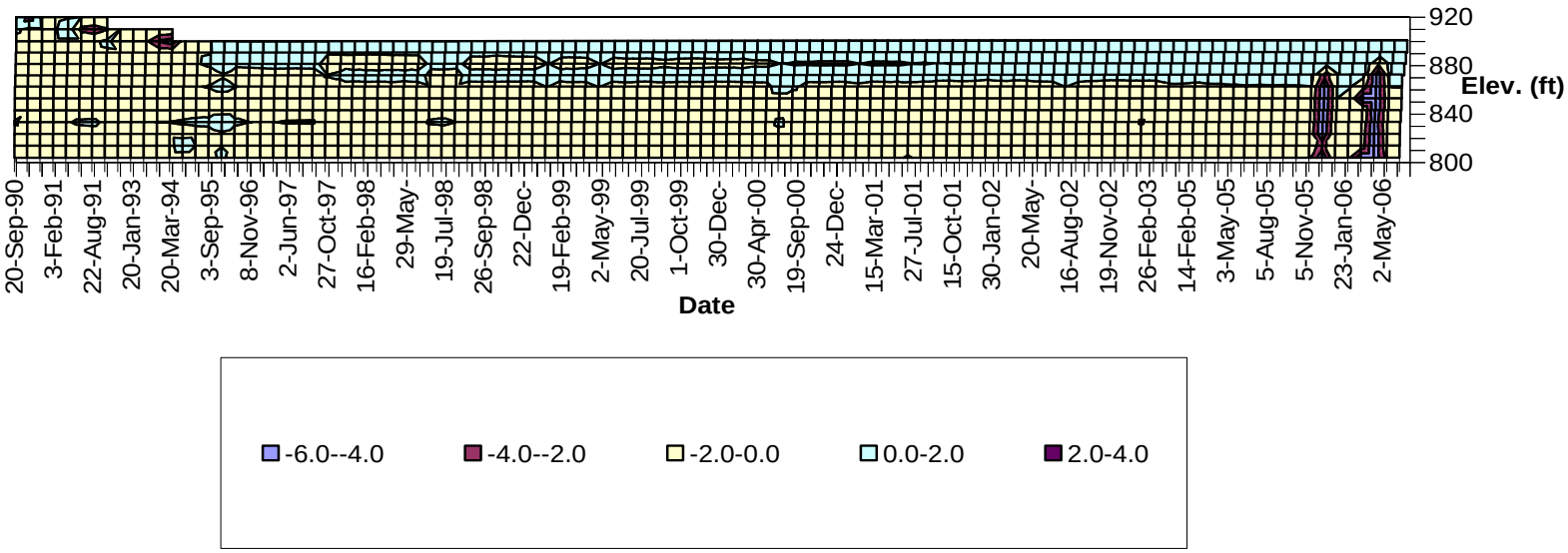
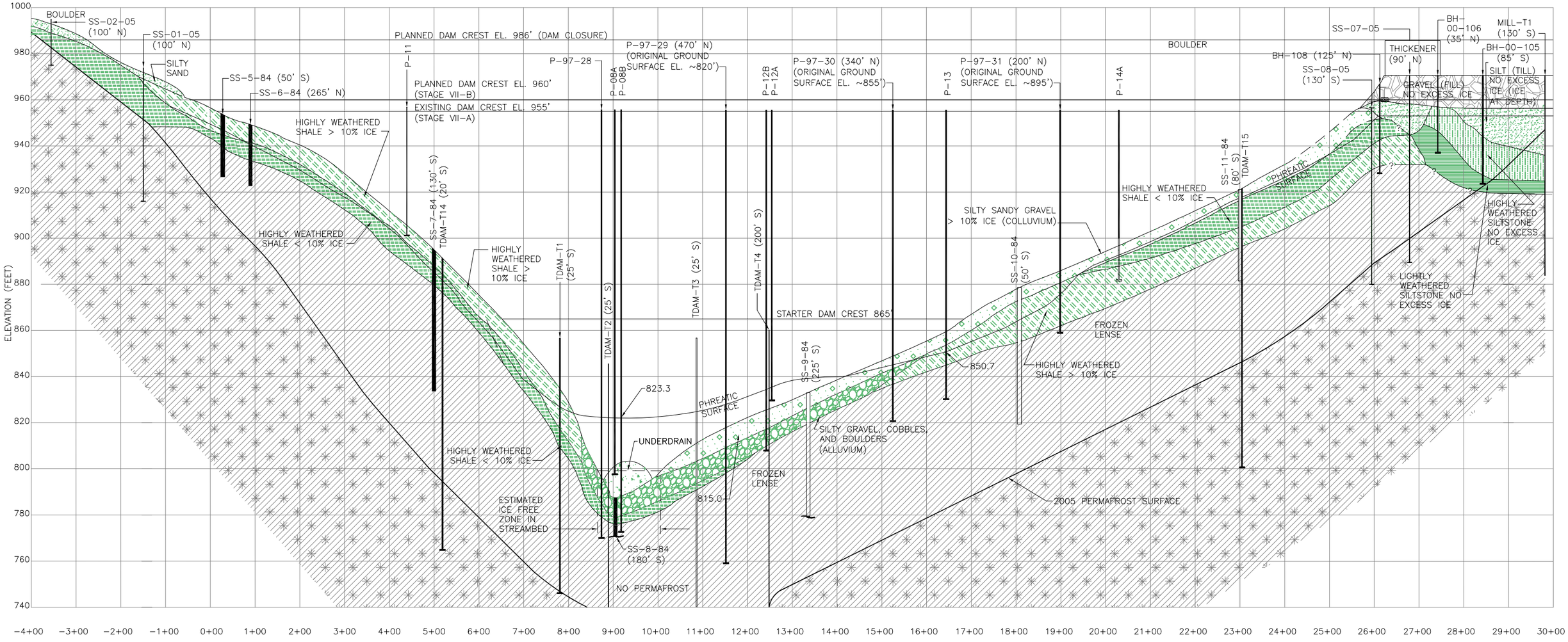


Figure 4-3

Historical Temperature Profile at Thermistor TDAM – T15



100 0 100 200
HORIZONTAL SCALE IN FEET

20 0 20 40
VERTICAL SCALE IN FEET

LEGEND

PERMAFROST IN 2005

SLIGHTLY TO MODERATELY WEATHERED SHALE

P:\ACAD\PROJECT\TeckCominco Alaska\Seepage Report Figures\Figure 4-4.dwg Sep 15, 2006 - 2:06pm

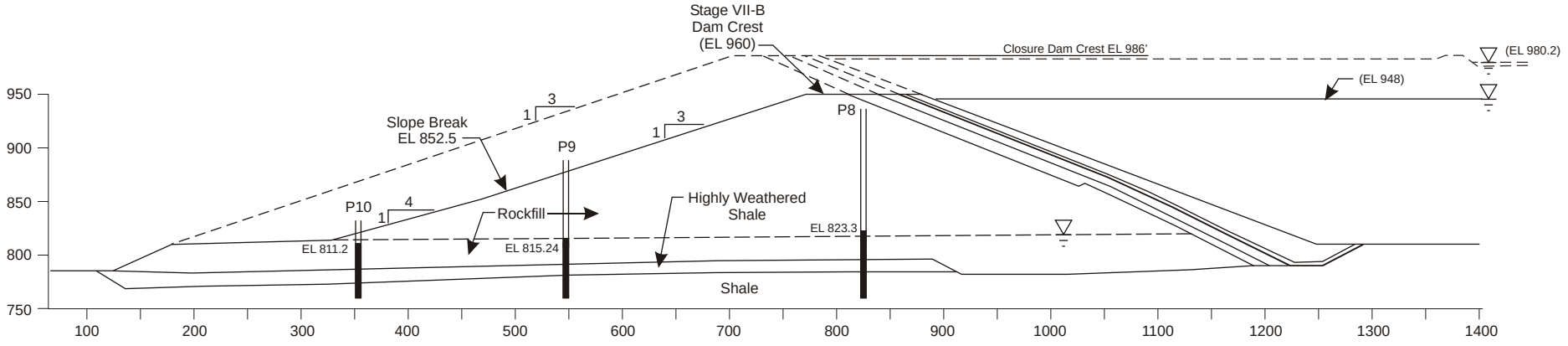
Job No. 33757098.00001

Figure 4-4

Current (2005) Permafrost Profile along Dam Crest

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS



LEGEND

Maximum recorded water level (Stage VII-B)

0 100 200

Horizontal and Vertical Scale in Feet

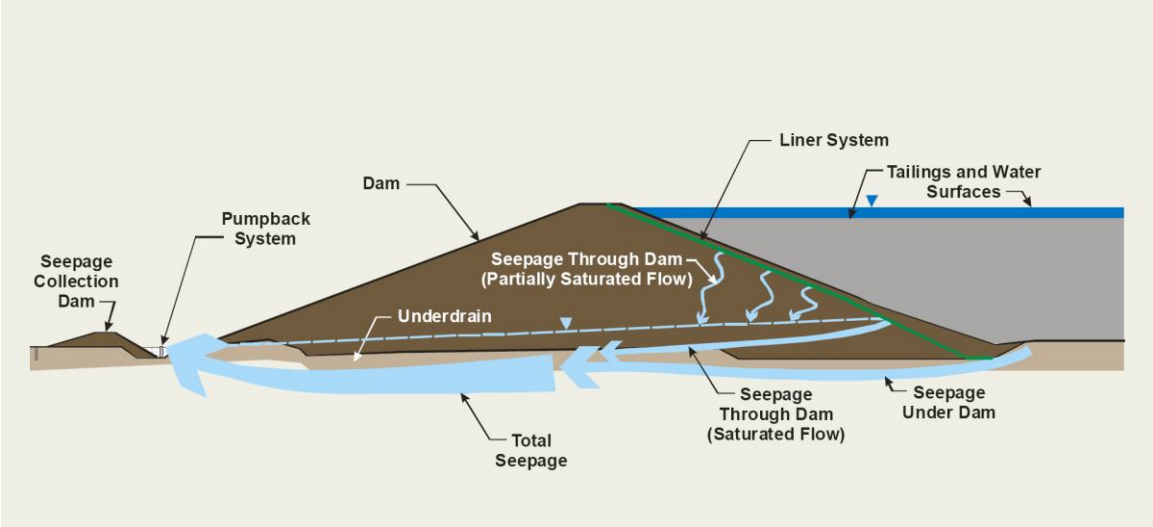
Figure 4-5

Phreatic Surface Downstream of Liner

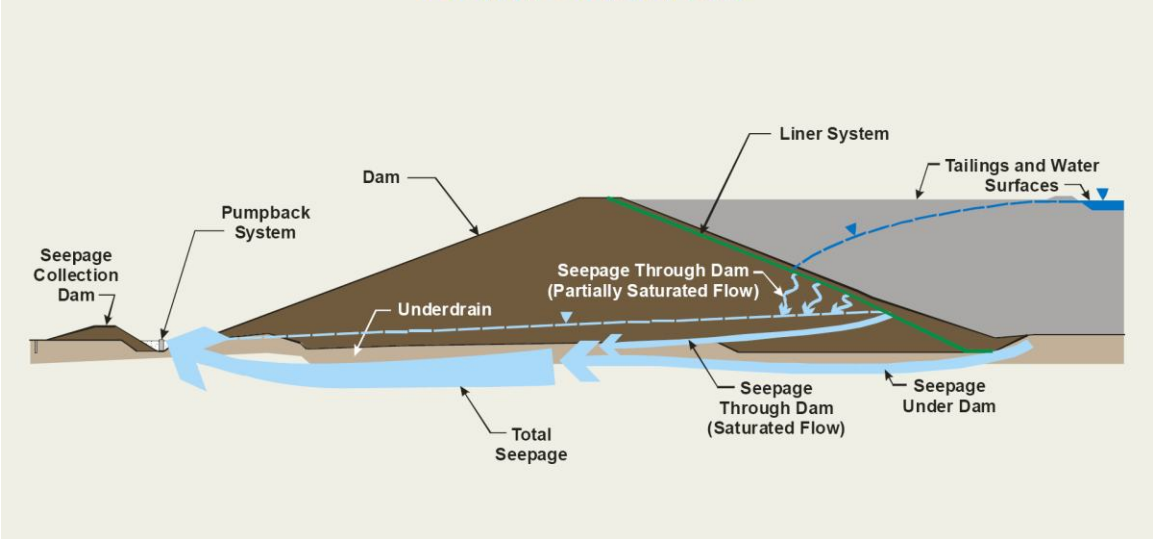
Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska



(a) No-Beach Conditions



(b) Beach Conditions

Figure 5-1
Components of Seepage

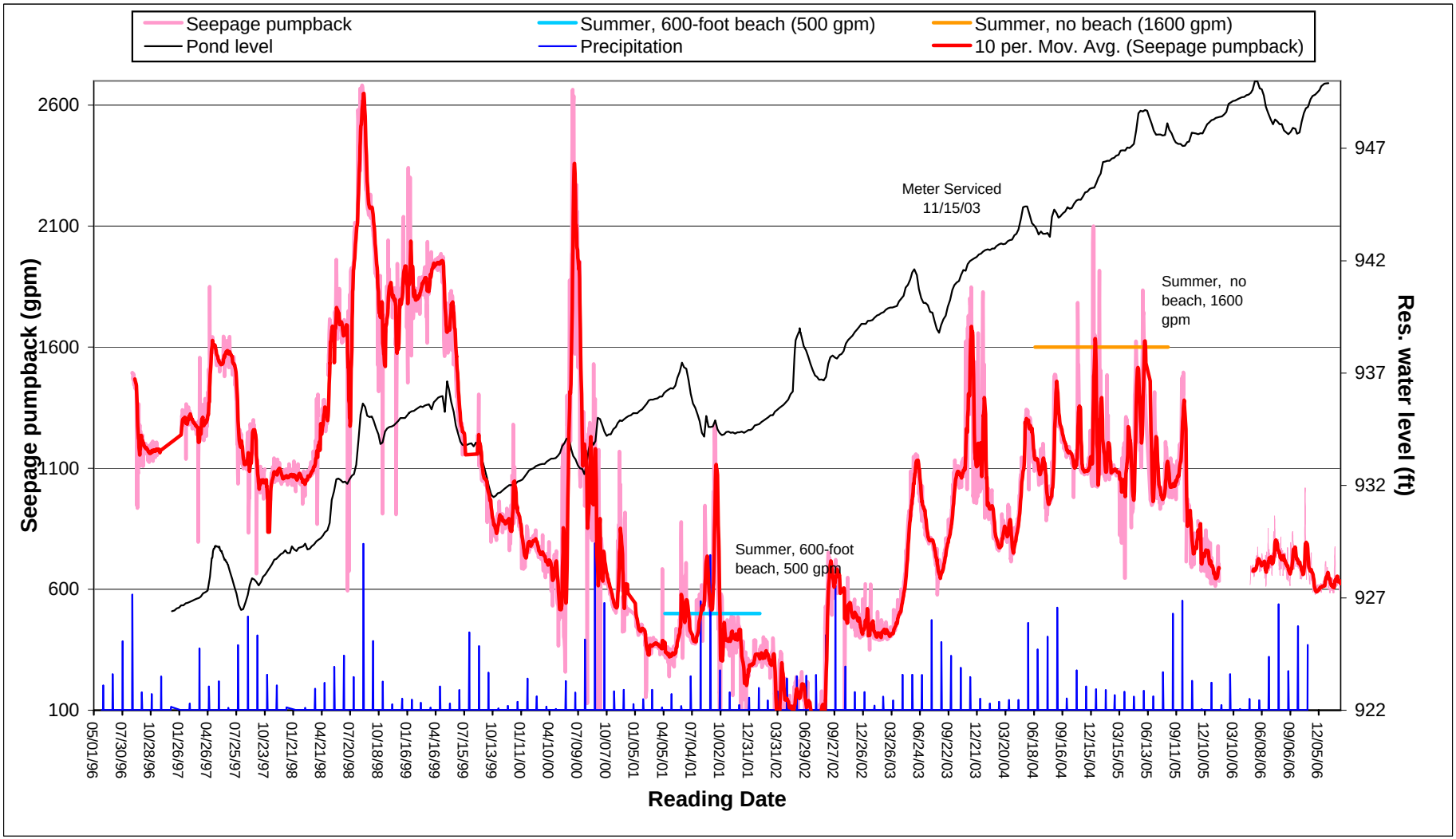


Figure 5-2

Target Seepage Rates

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS Job No. 33757098

URS

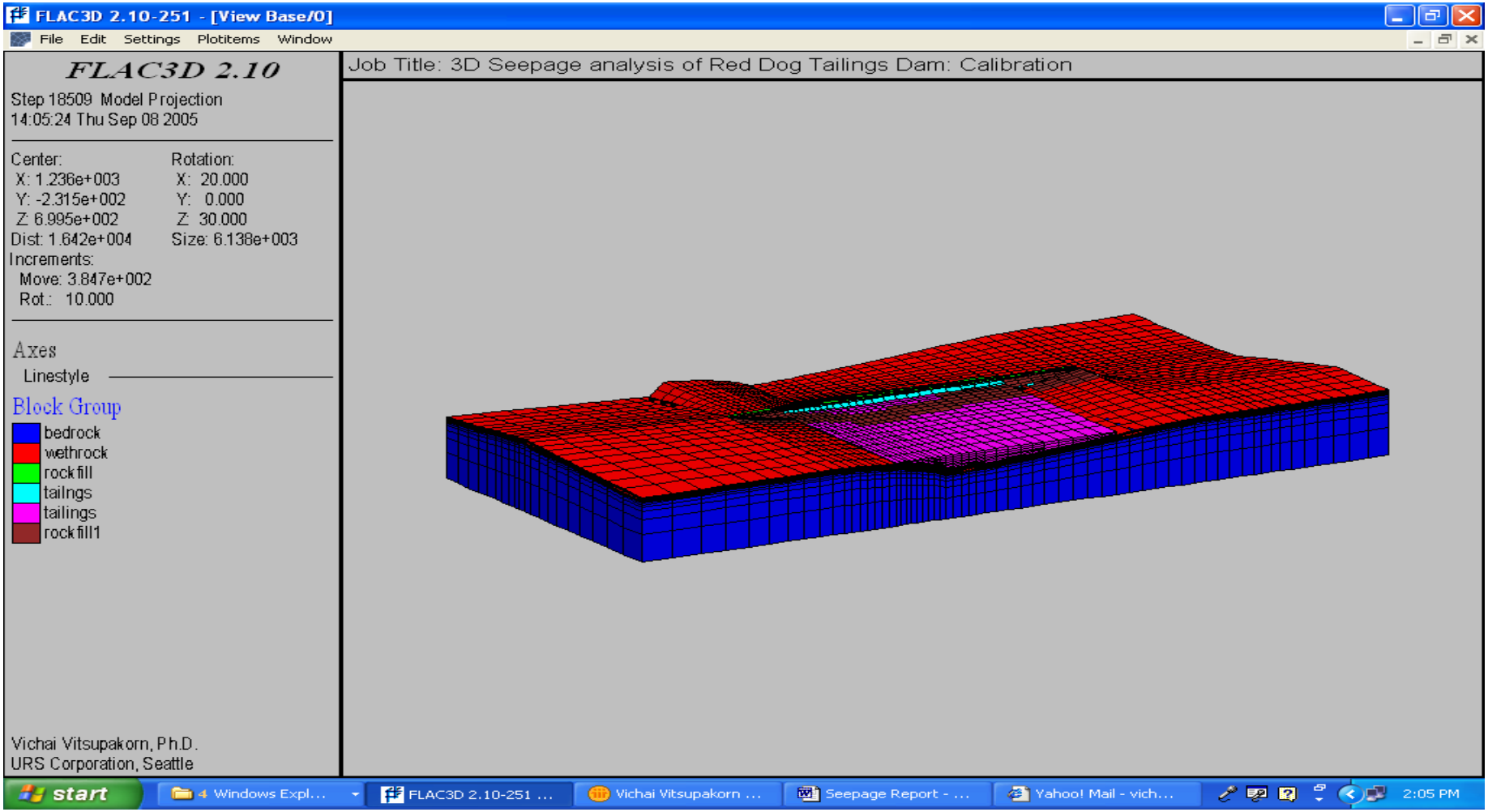


Figure 6-1

Fine Mesh and Different Material Zones for Stage VI

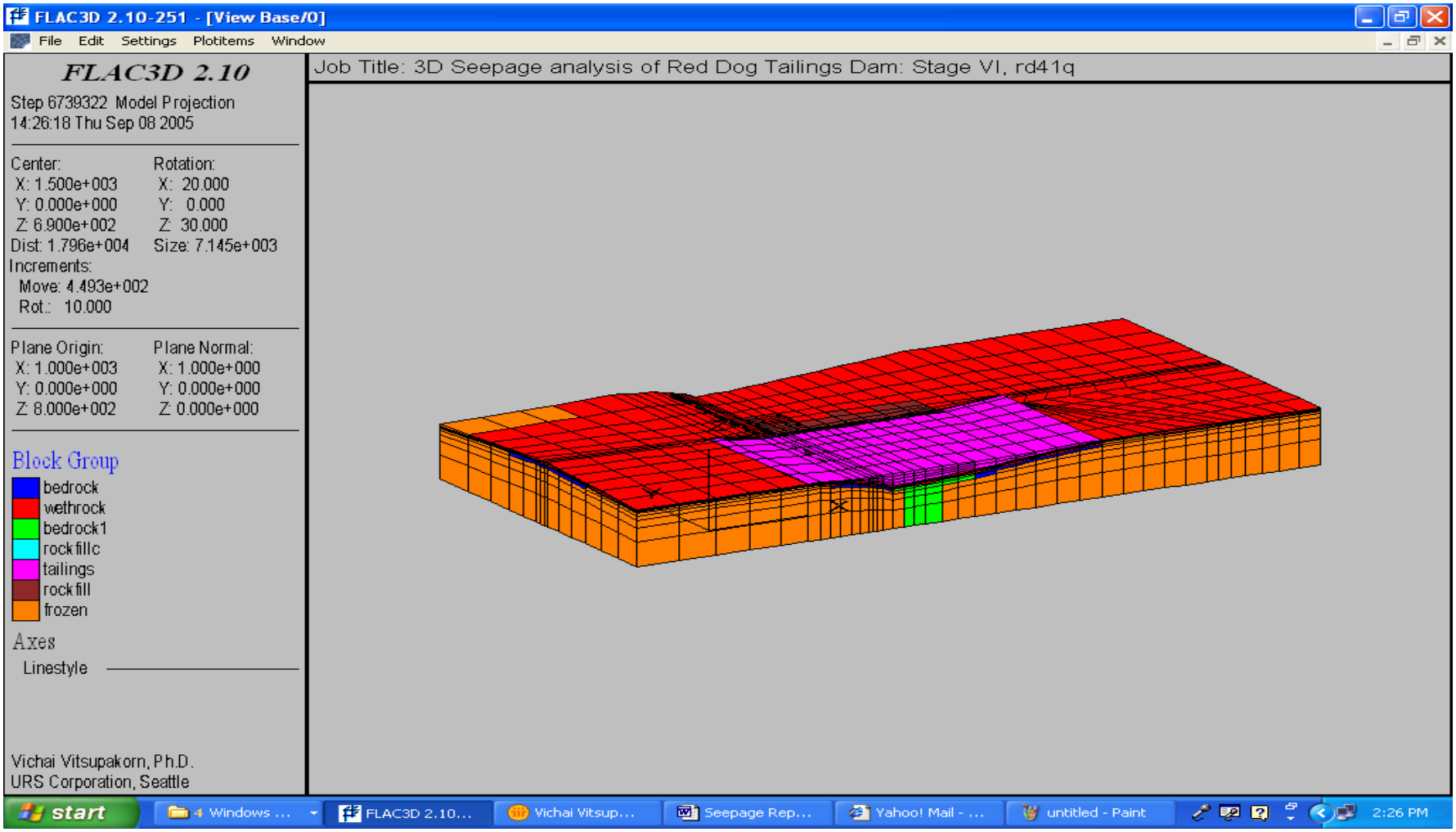


Figure 6-2

Final (Coarse) Mesh for Stage VI

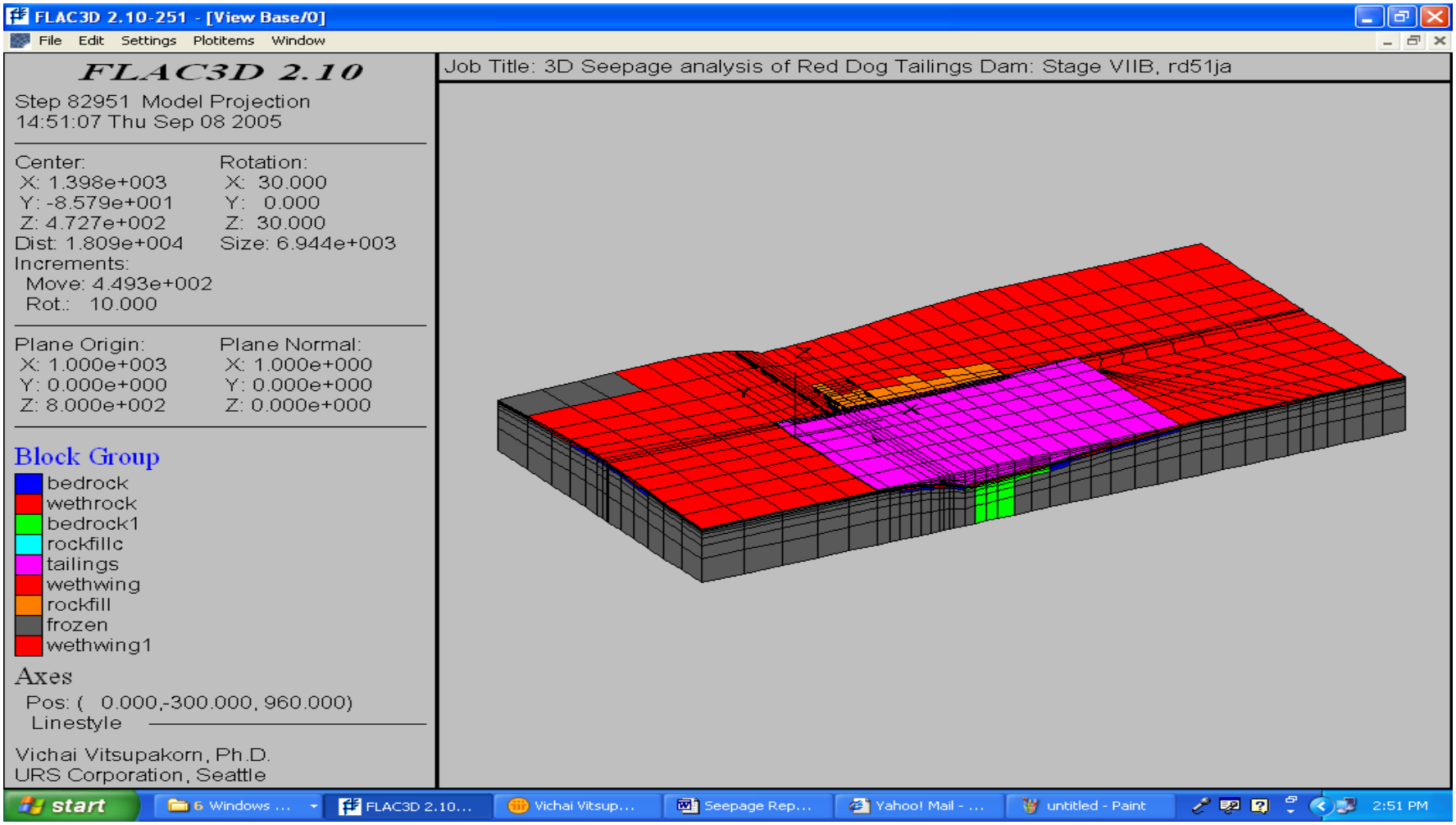


Figure 6-3

Final (Coarse) Mesh for Stage VII-B

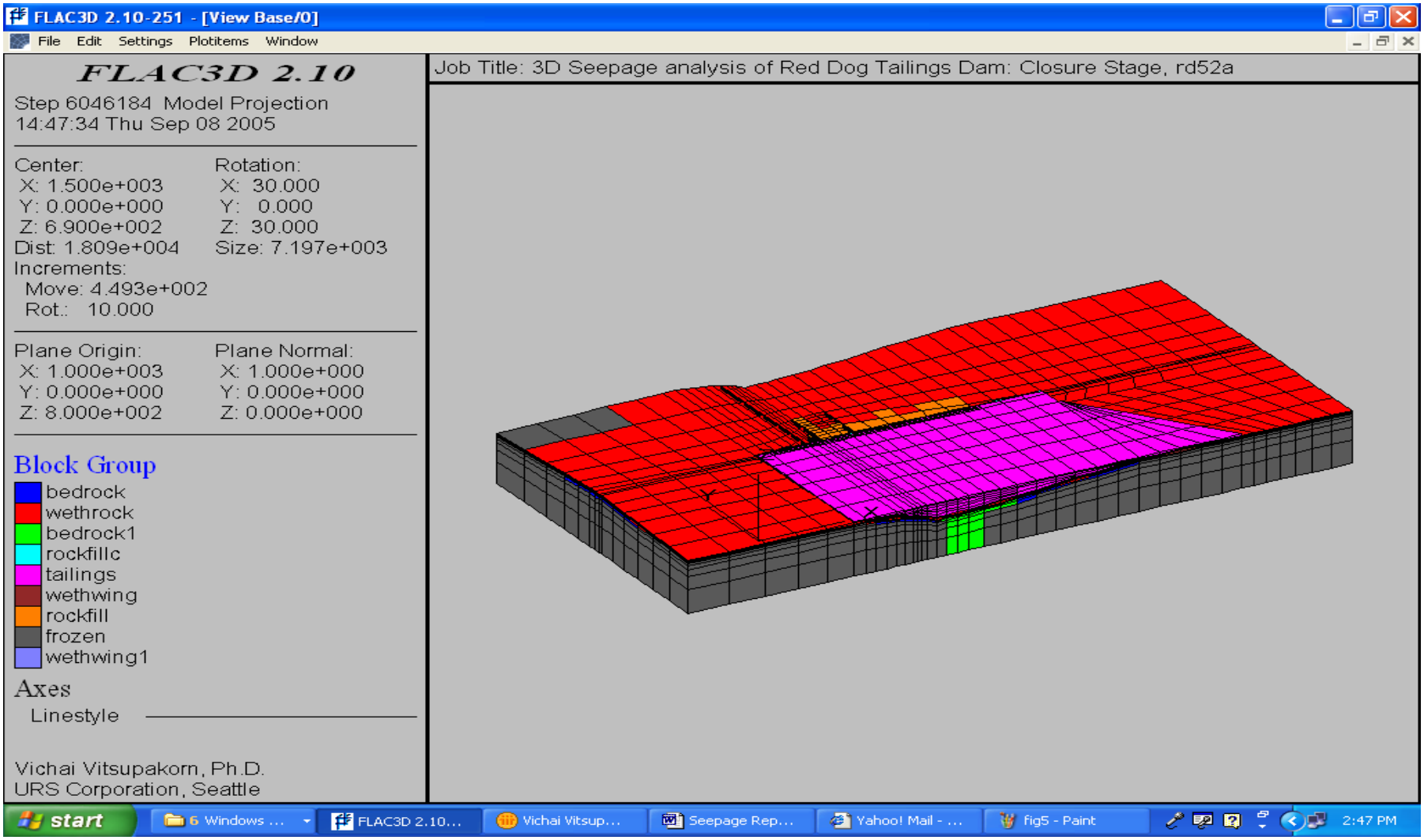


Figure 6-4

Final (Coarse) Mesh for Closure

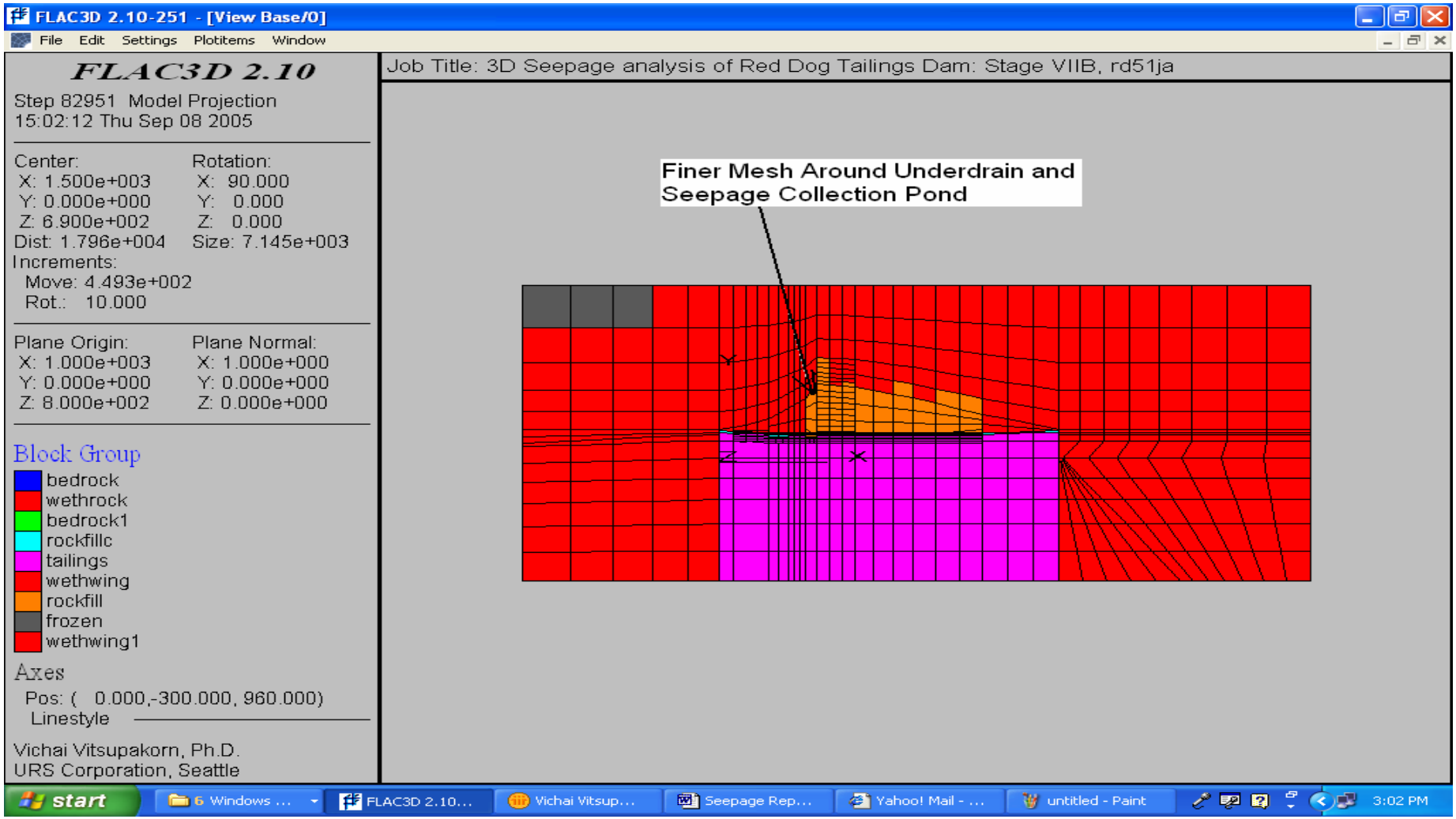


Figure 6-5

Plan View of Fine Mesh near Underdrain and Seepage Collection Pond

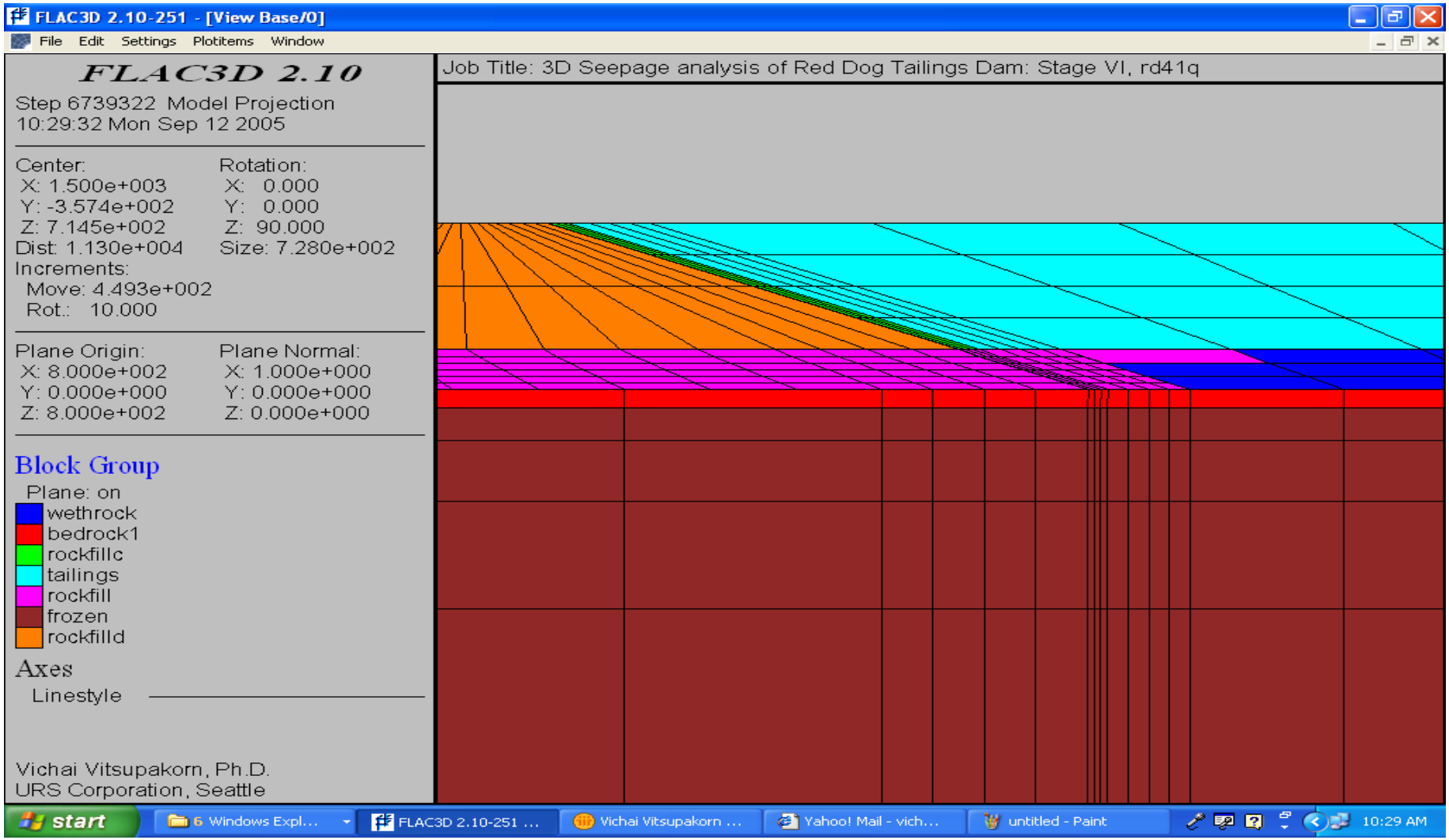


Figure 6-6

Section View of Fine Mesh through Underdrain and Seepage Collection Pond

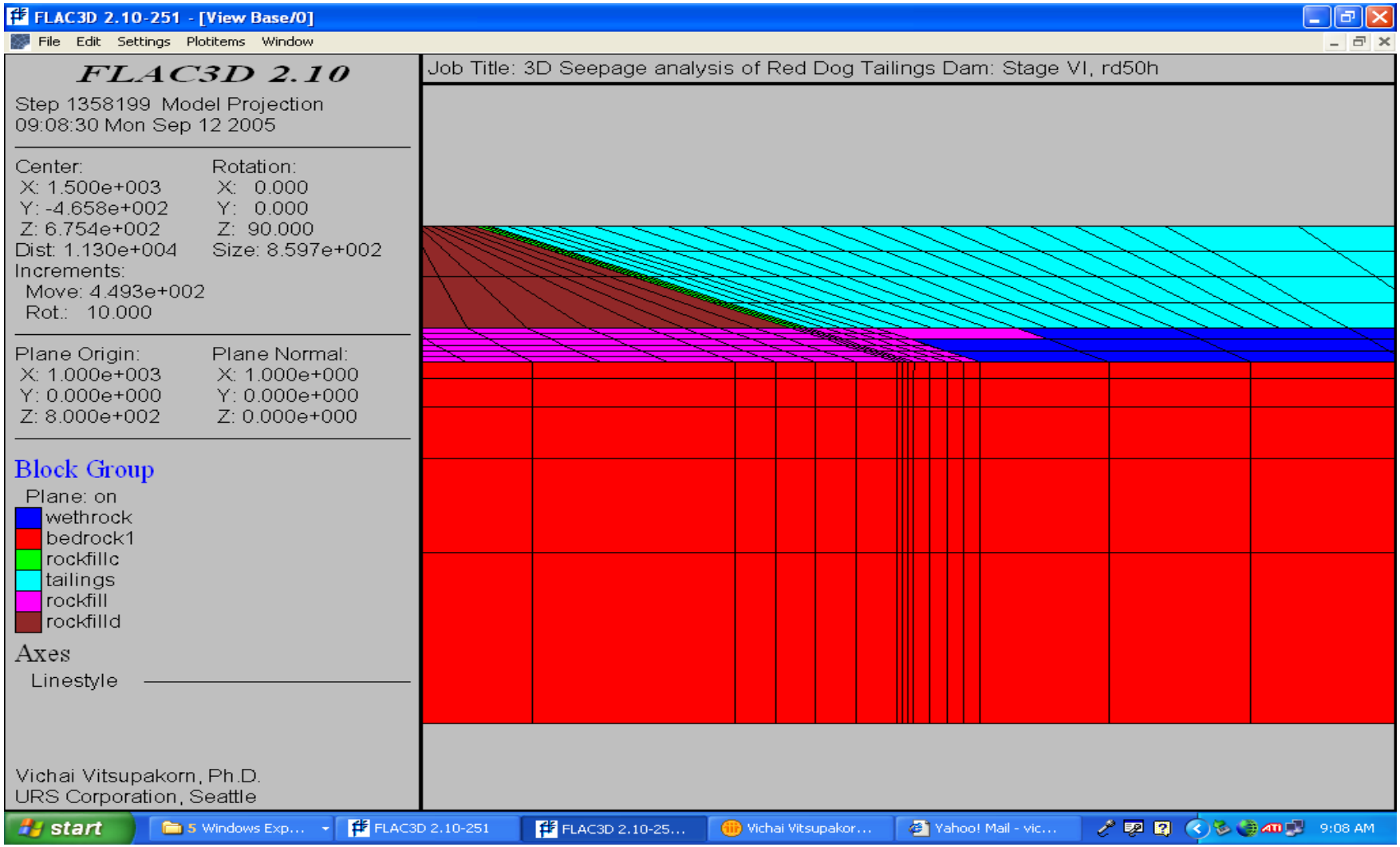


Figure 6-7

Final Mesh in Front of Liner

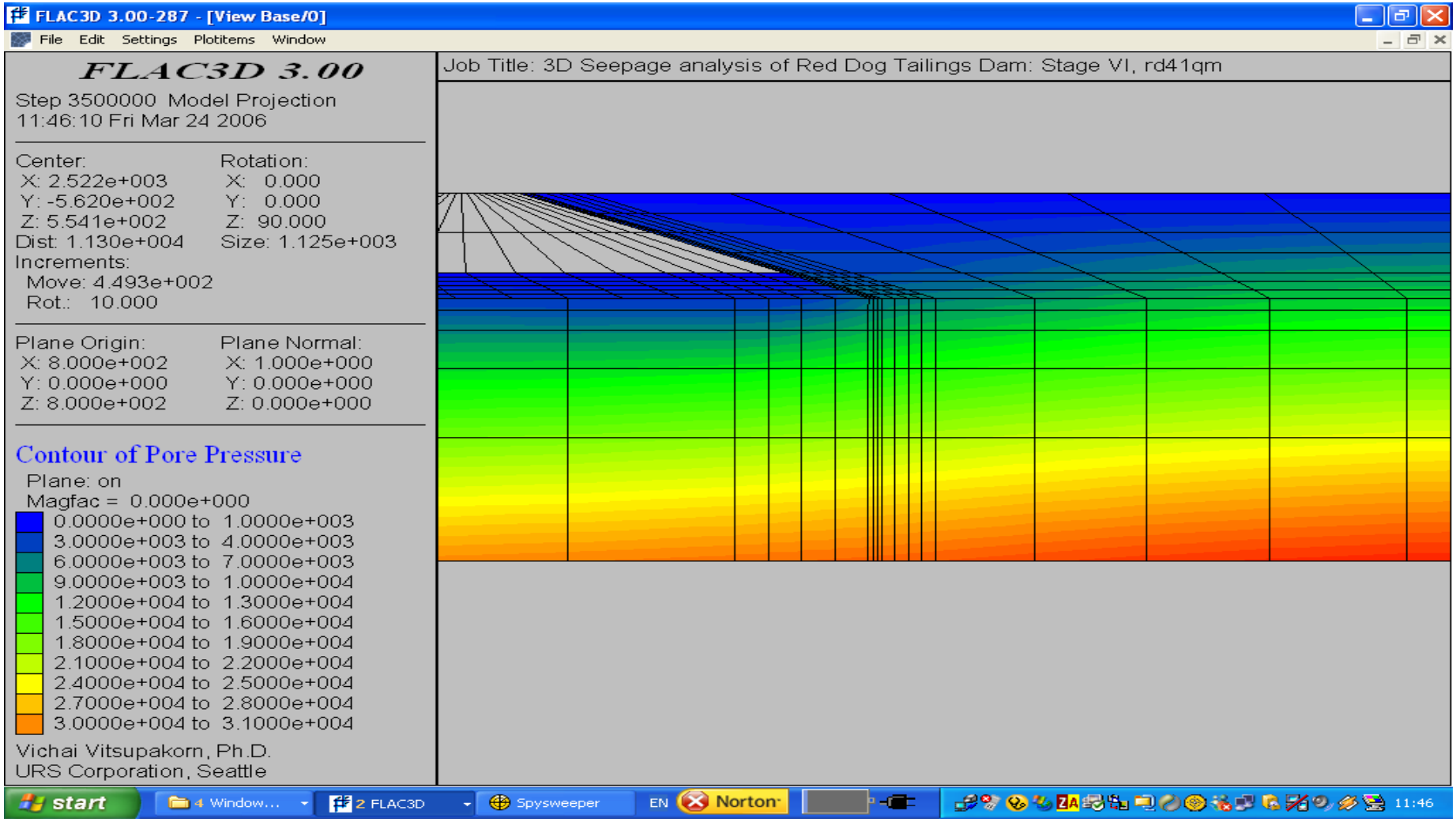


Figure 6-8

Pore Pressure Distribution at Station 8+00: Stage VI, No-Beach

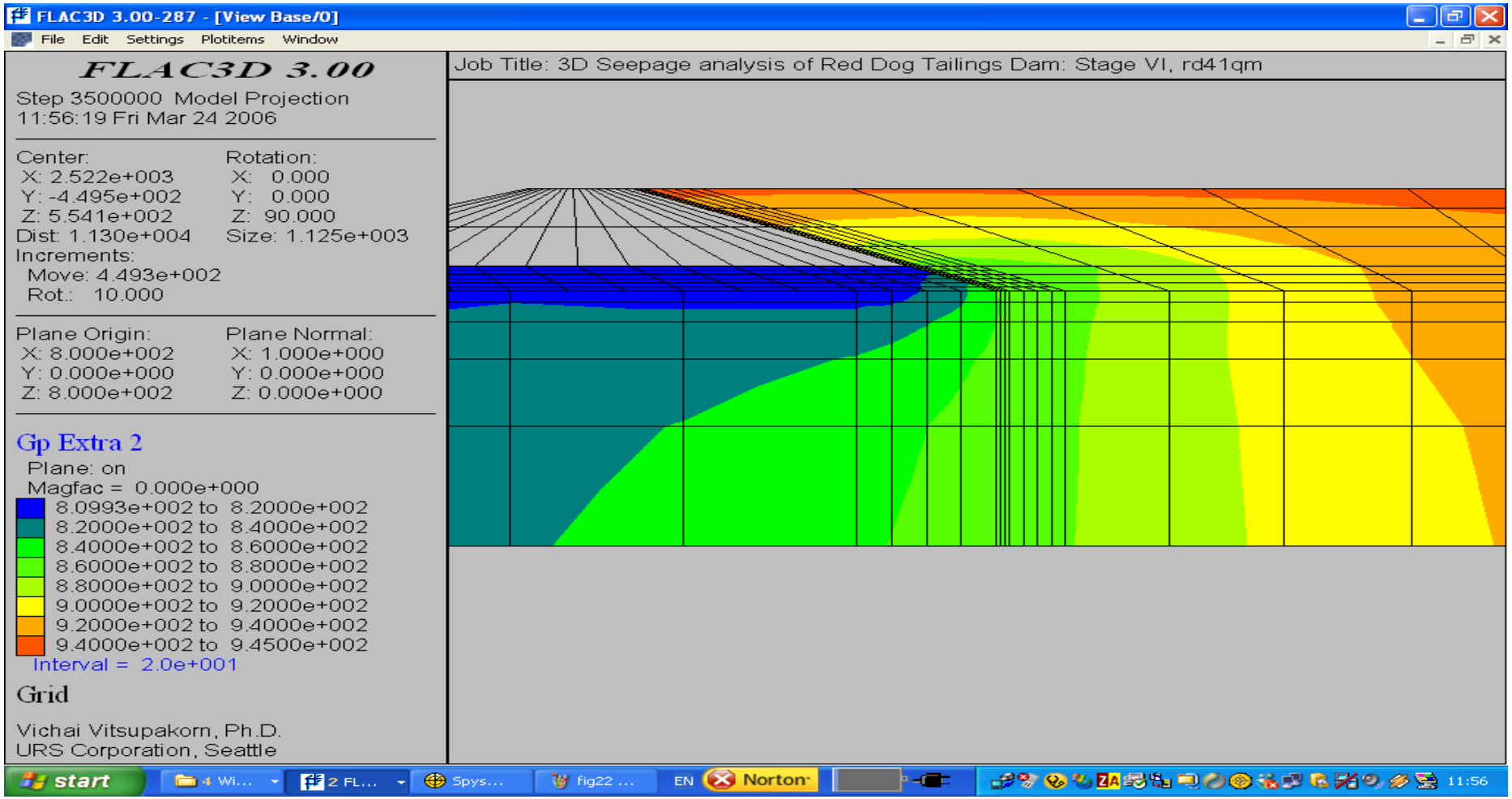


Figure 6-9

Equipotential Lines at Station 8+00: Stage VI, No-Beach

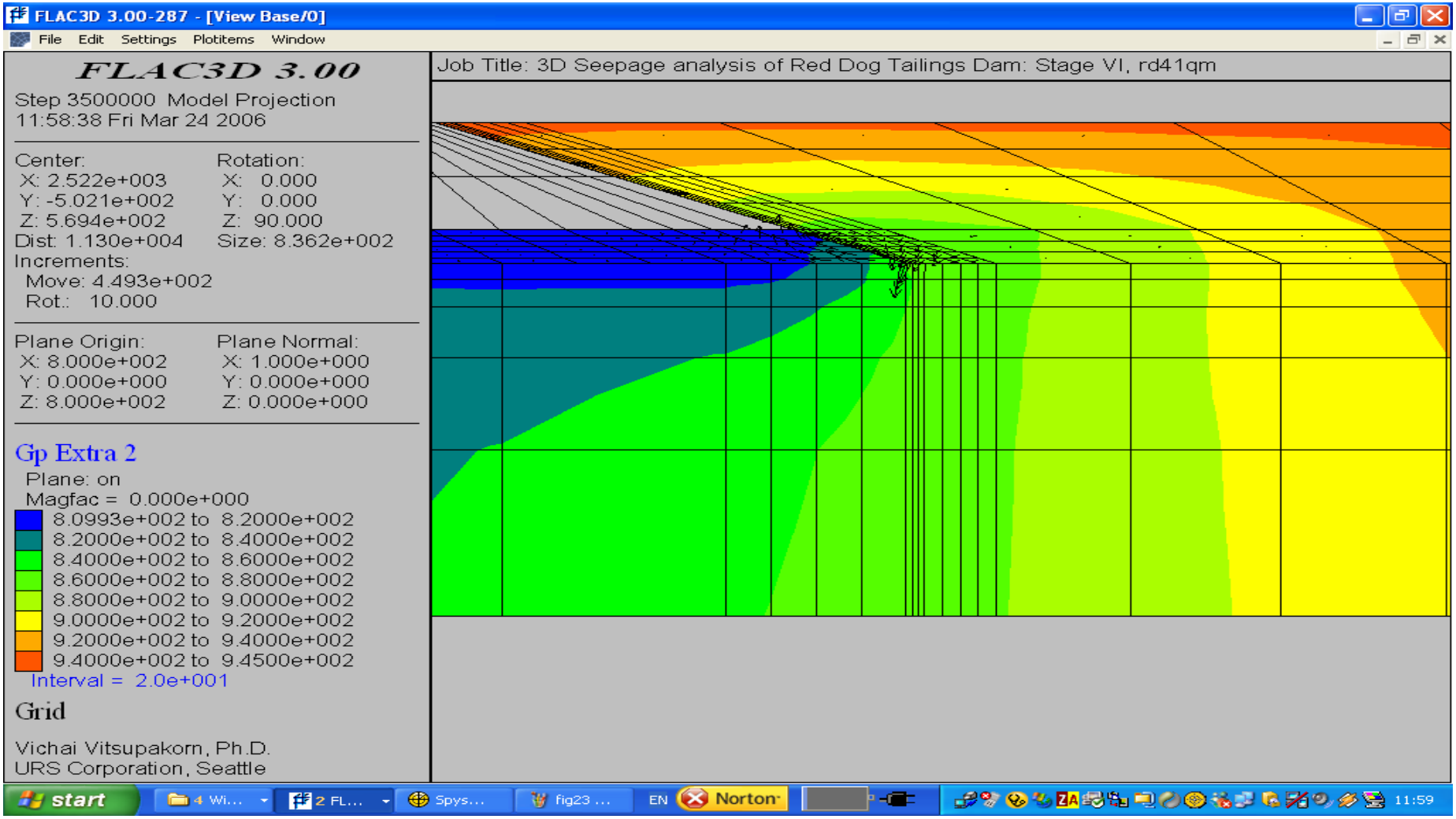


Figure 6-10

Flow Vectors around Liner at Station 8+00: Stage VI, No-Beach

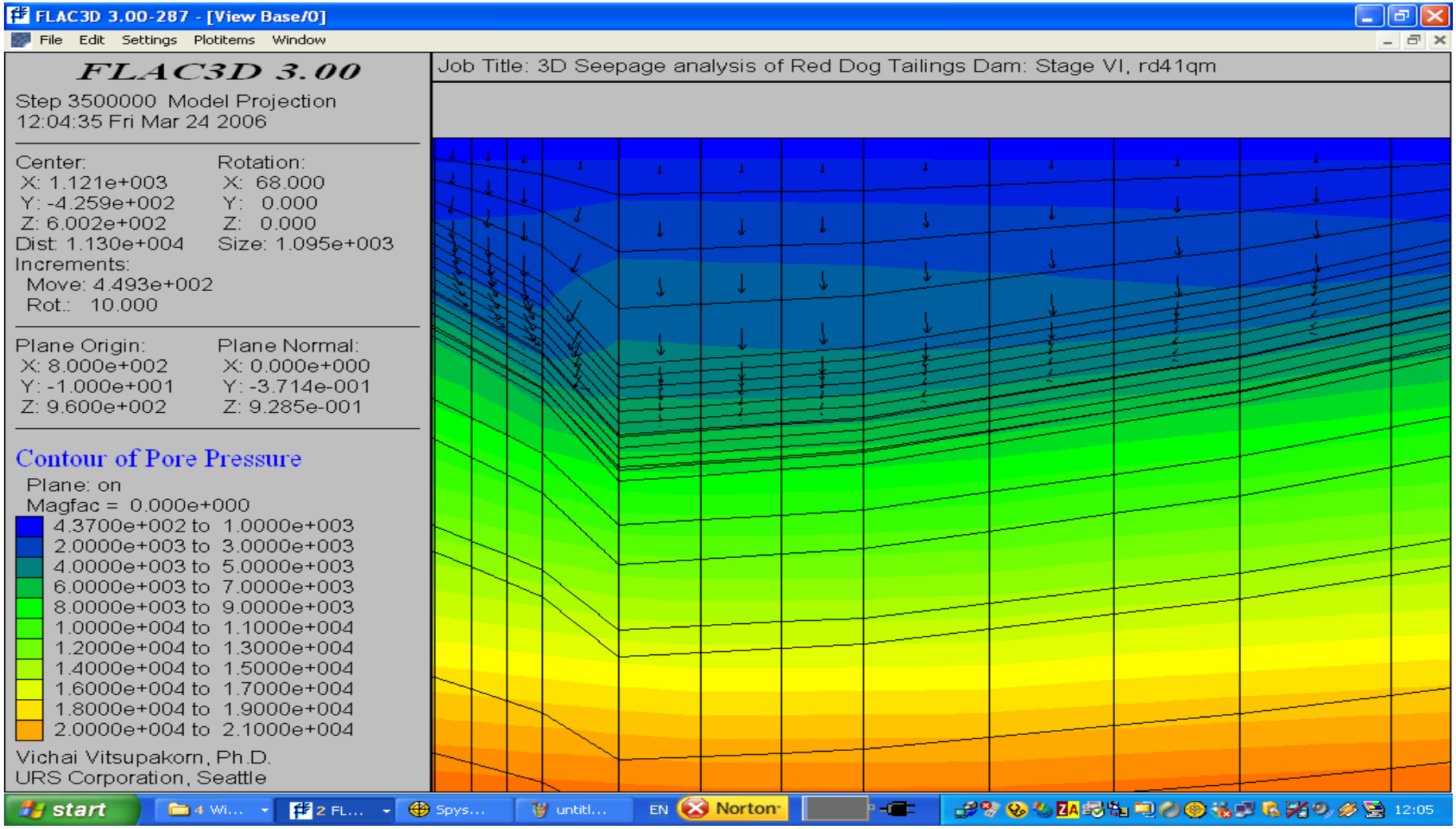


Figure 6-11

Flow Vectors Upstream of Liner, Stage VI, No-Beach

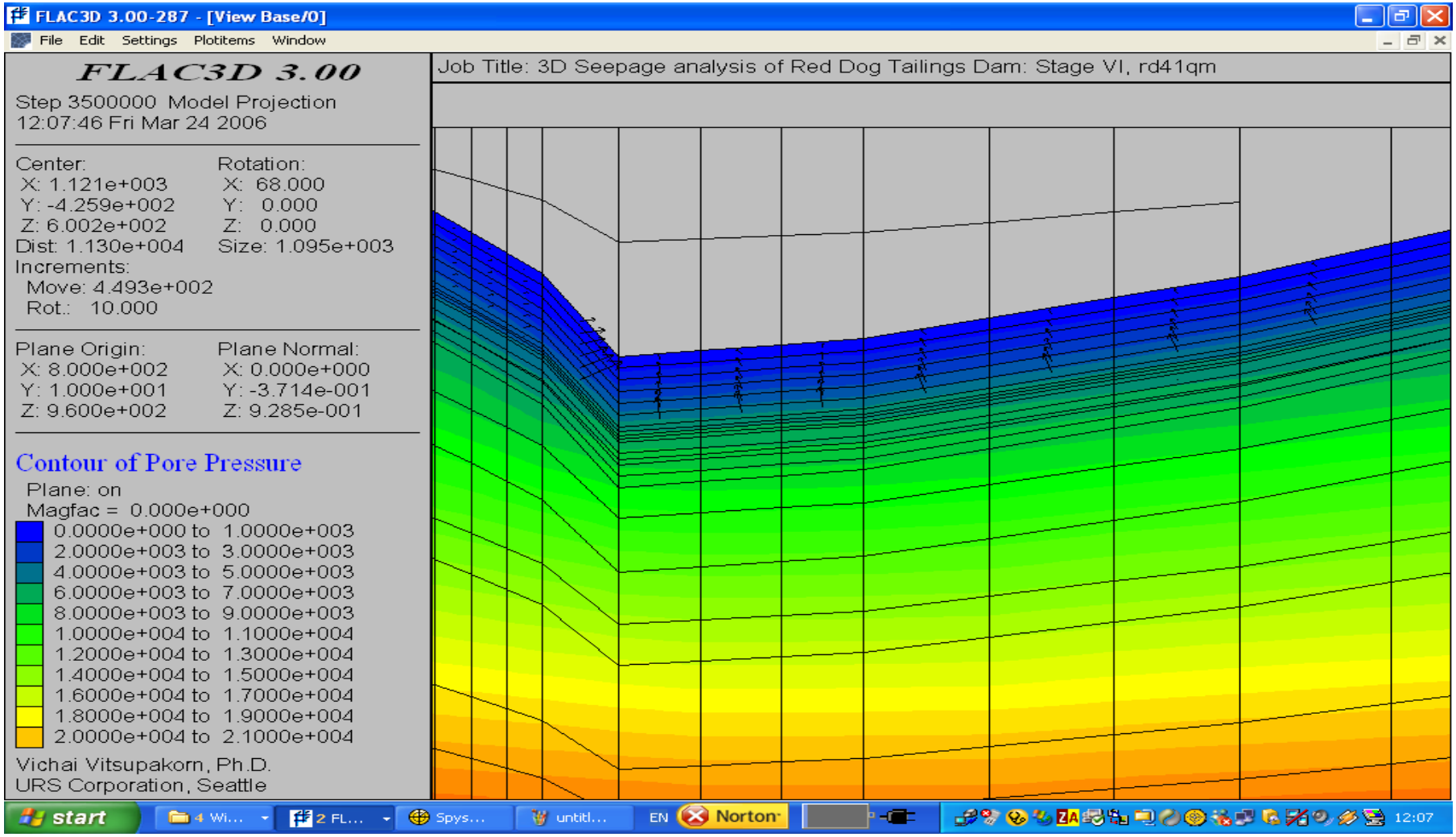


Figure 6-12

Flow Vectors Downstream of Liner, Stage VI, No-Beach

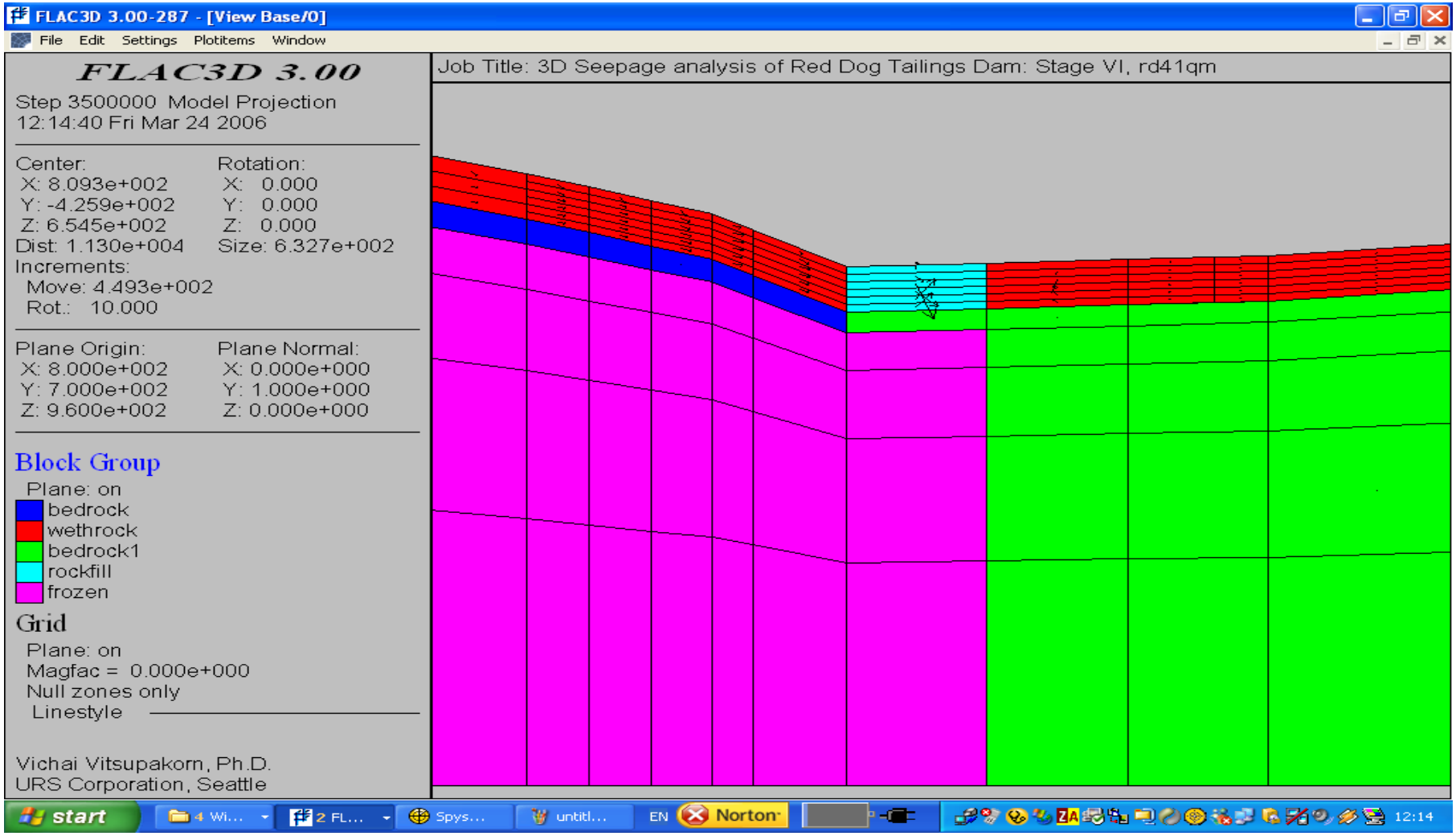


Figure 6-13

Flow Vectors through Seepage Collection Pond, Stage VI, No-Beach

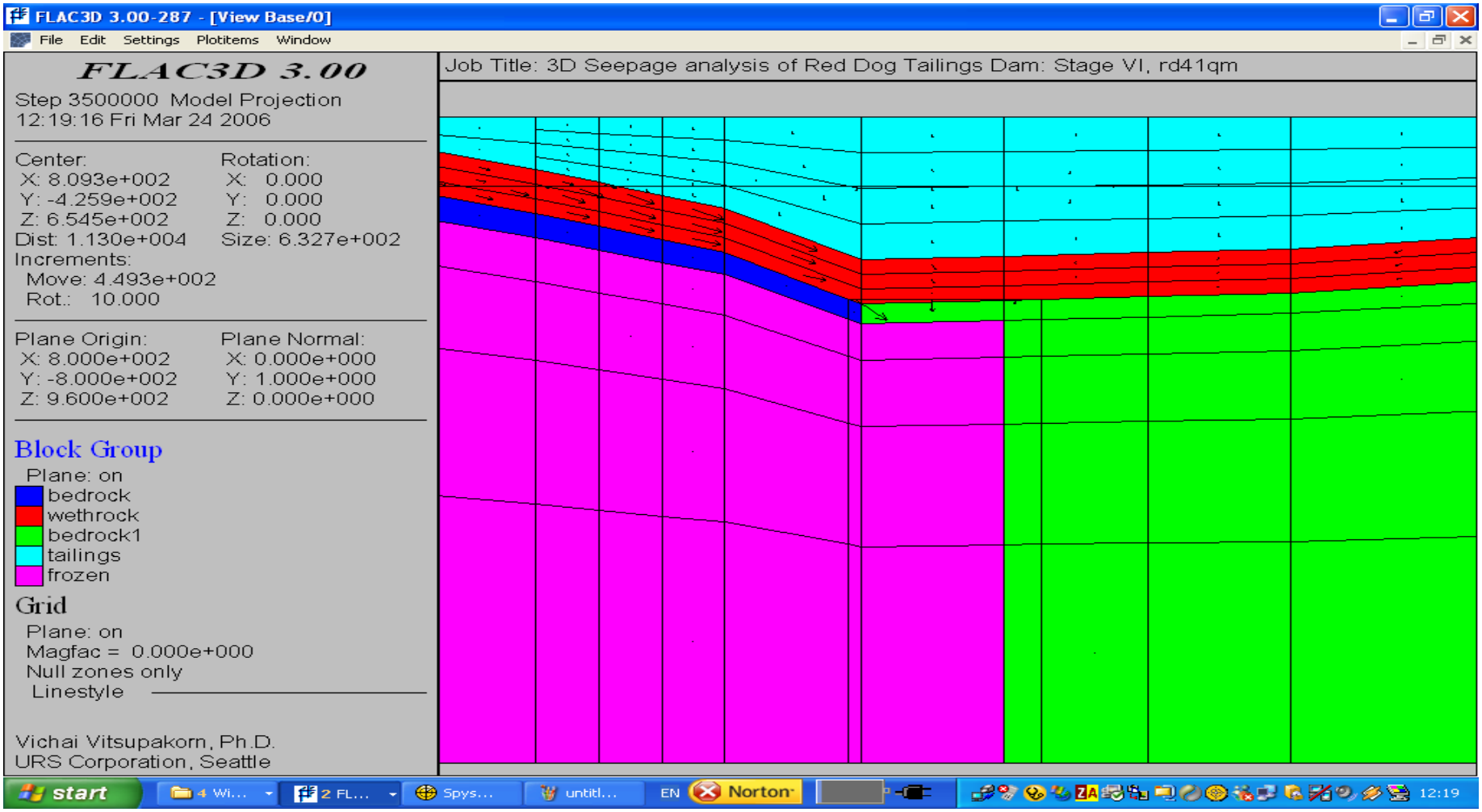
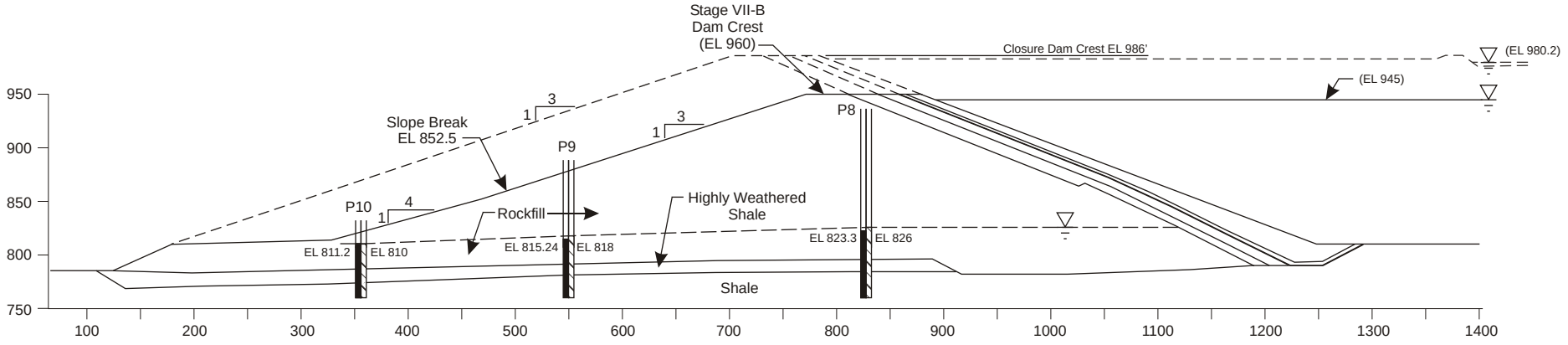


Figure 6-14

Flow Vectors 800 ft Upstream and Parallel to Dam Crest, Stage VI, No-Beach



LEGEND

Maximum recorded water level (Stage VII-B)

Water levels from seepage analysis (Stage VI, No Beach)

0 100 200

Horizontal and Vertical Scale in Feet

Figure 6-15

Phreatic Surface Downstream of Liner for Stage VI, No Beach

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

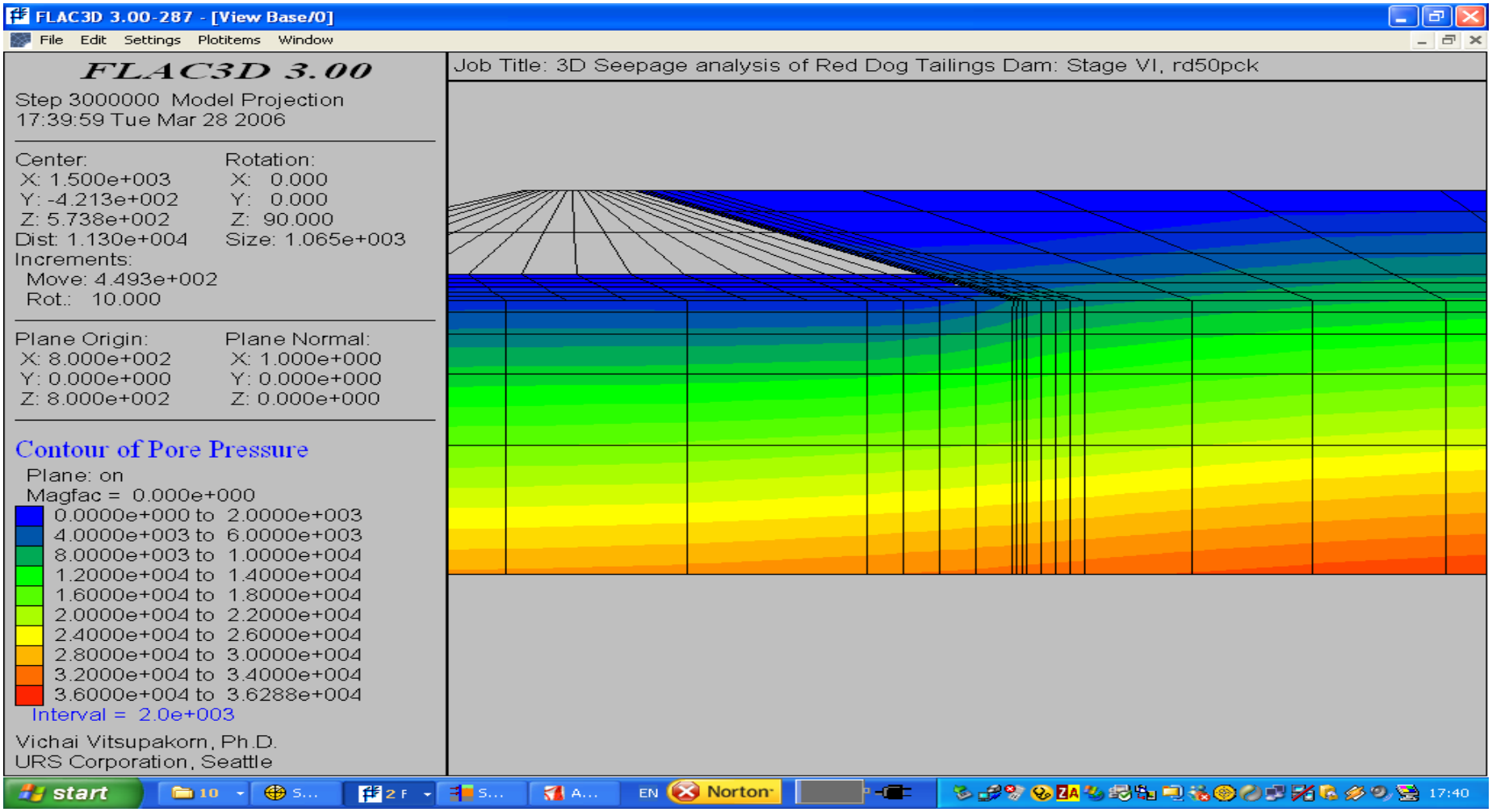


Figure 6-16

Pore Pressure Distribution at Station 8+00: Stage VI, 600-foot Beach

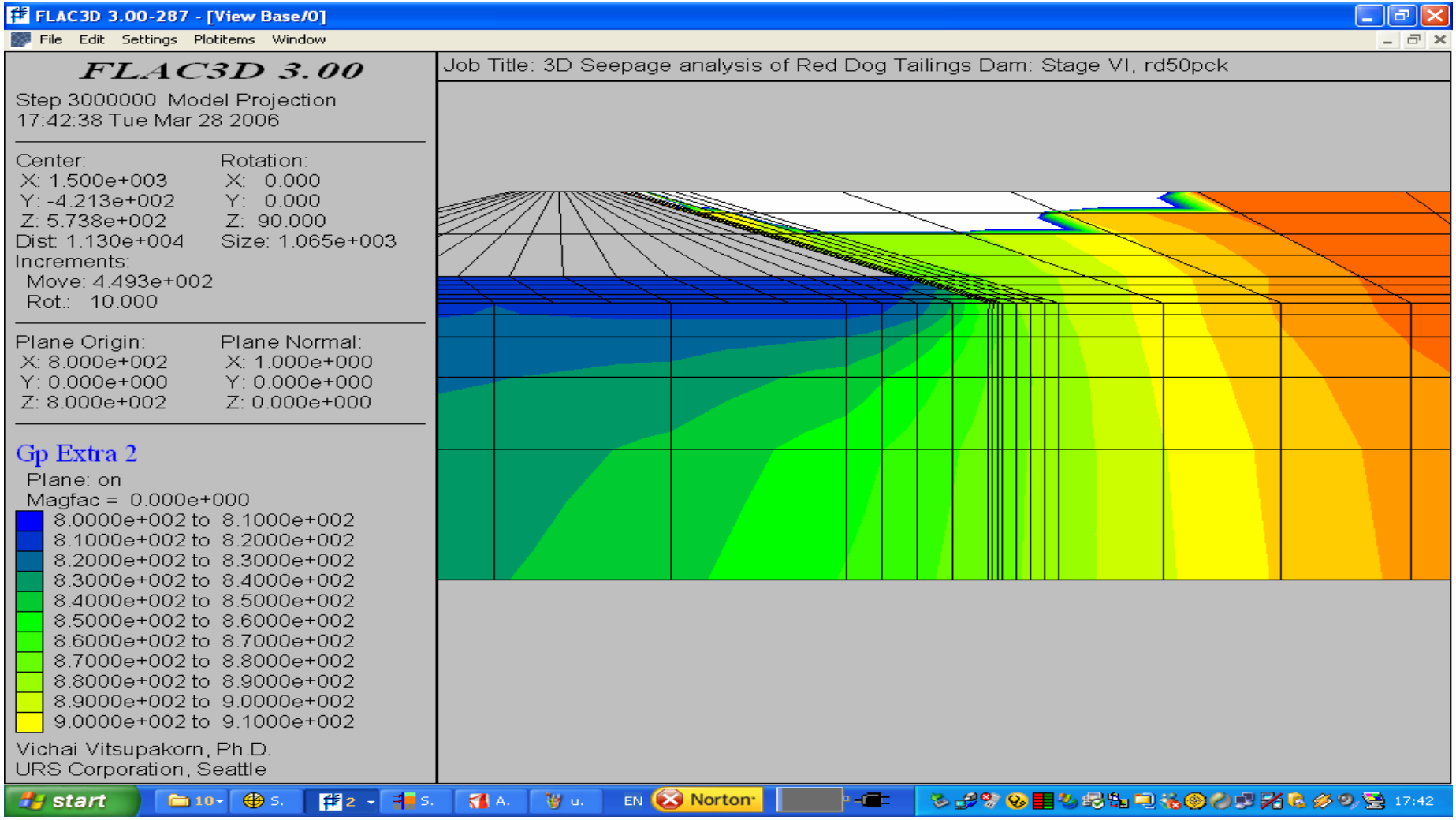
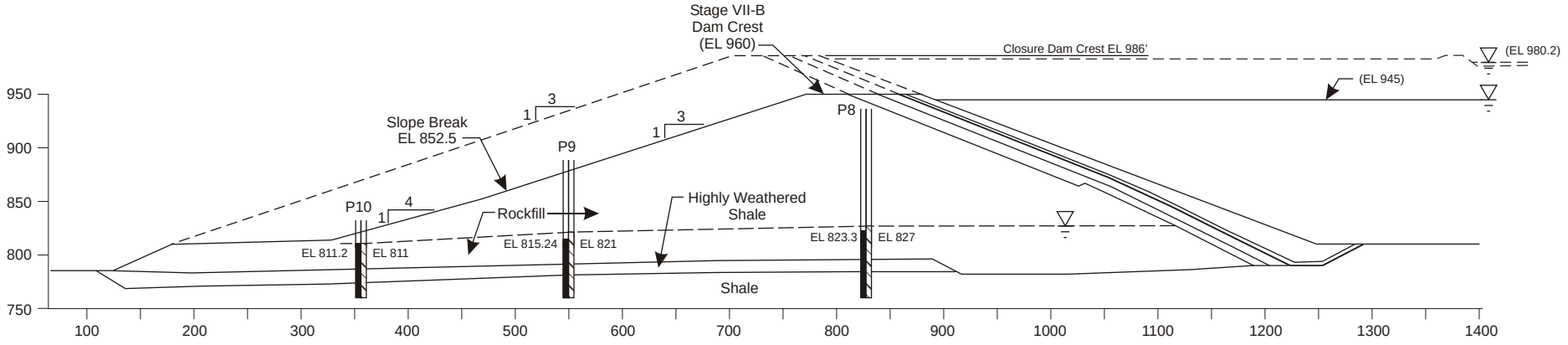


Figure 6-17

Equipotential Lines at Station 8+00: Stage VI, 600-foot Beach



LEGEND

Maximum recorded water level
(Stage VII-B)

Water levels from seepage analysis
(Stage VI, 600-Foot Beach)

0 100 200
Horizontal and Vertical Scale in Feet

Figure 6-18

Phreatic Surface Downstream of Liner for Stage VI, 600-Foot Beach

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

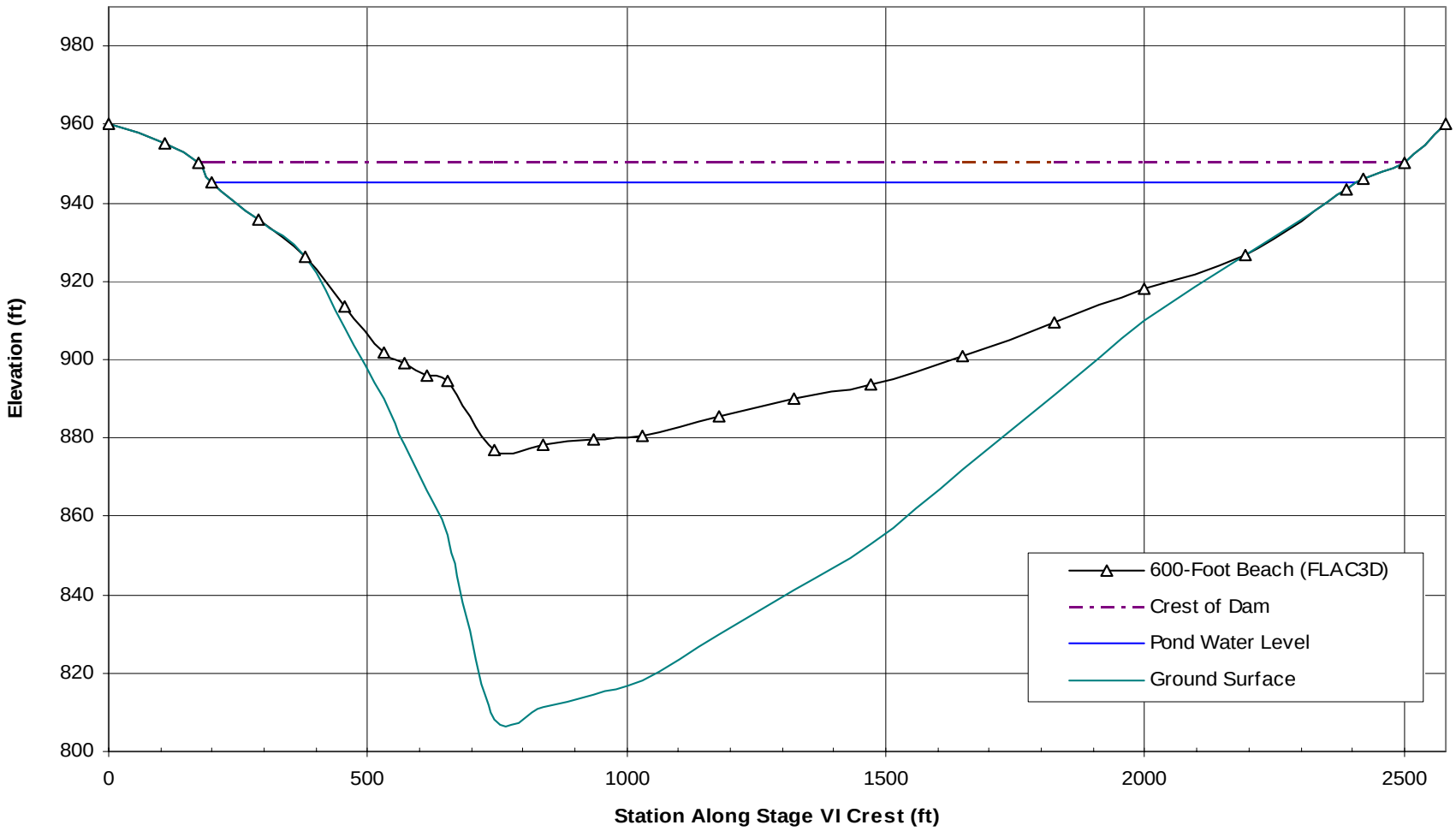


Figure 6-19

Phreatic Surface Upstream of Liner: Stage VI, 600-foot Beach

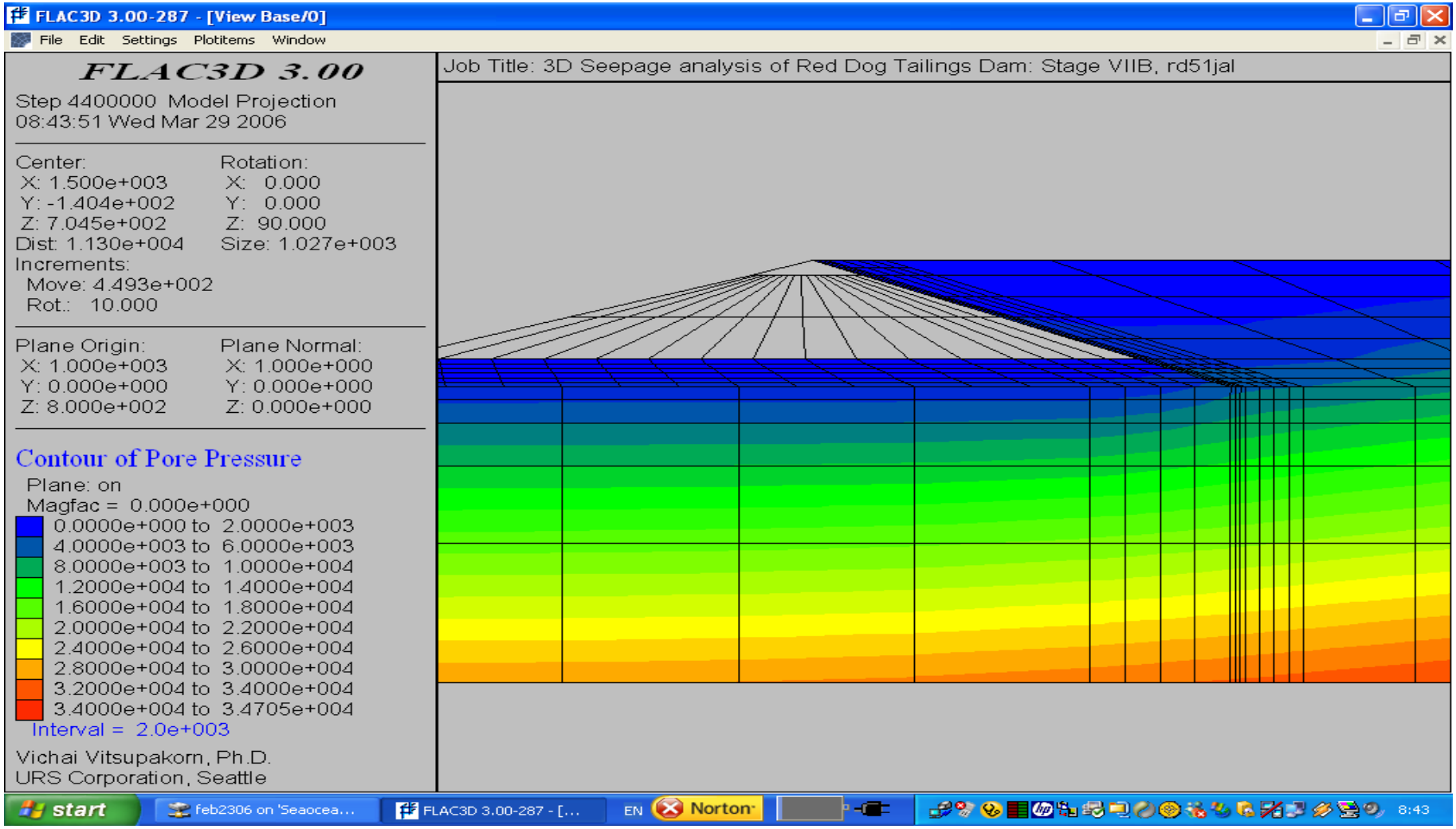


Figure 6-20

Pore Pressure Distribution at Station 8+00: Stage VII-B, 300-foot Beach

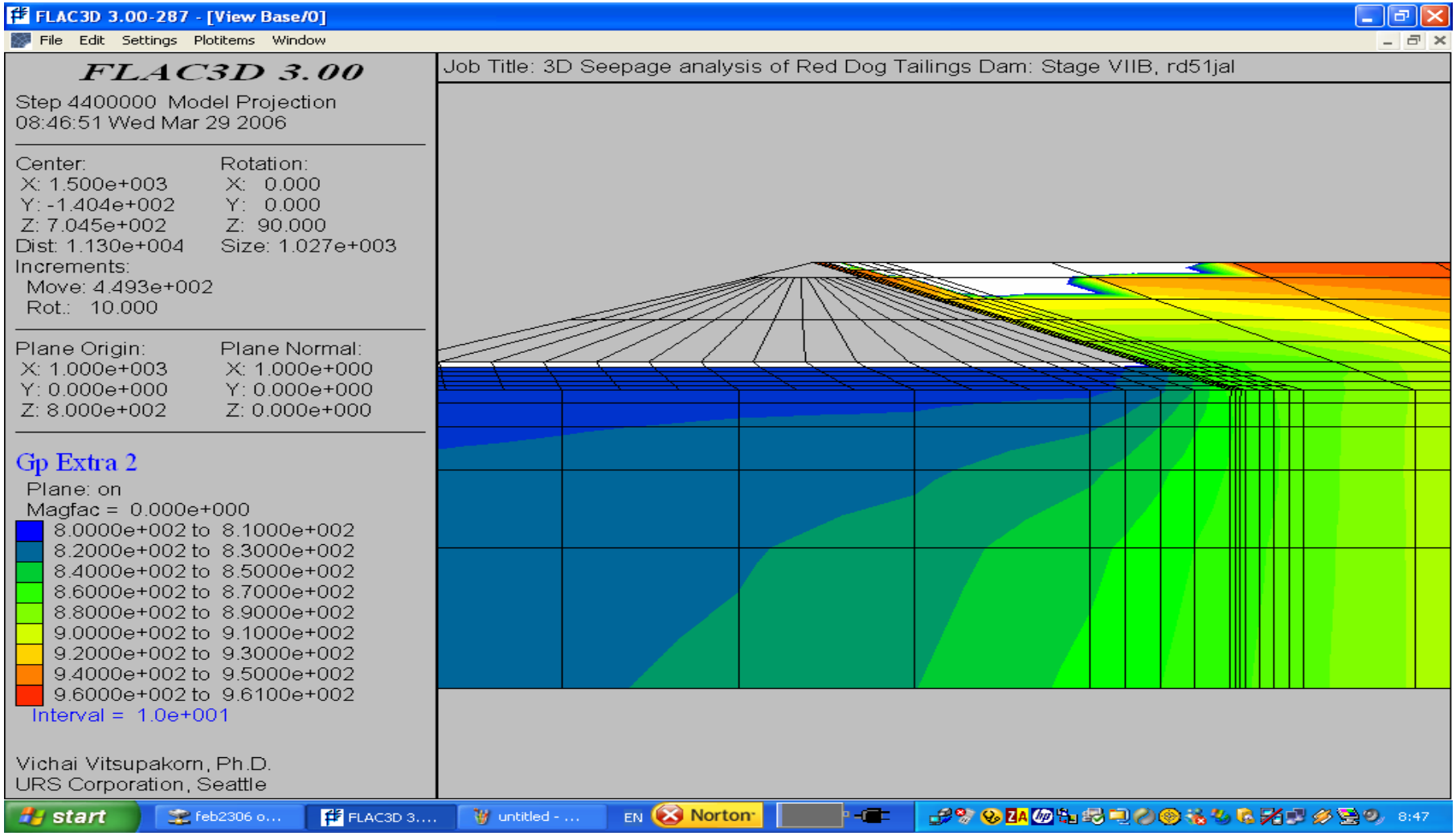
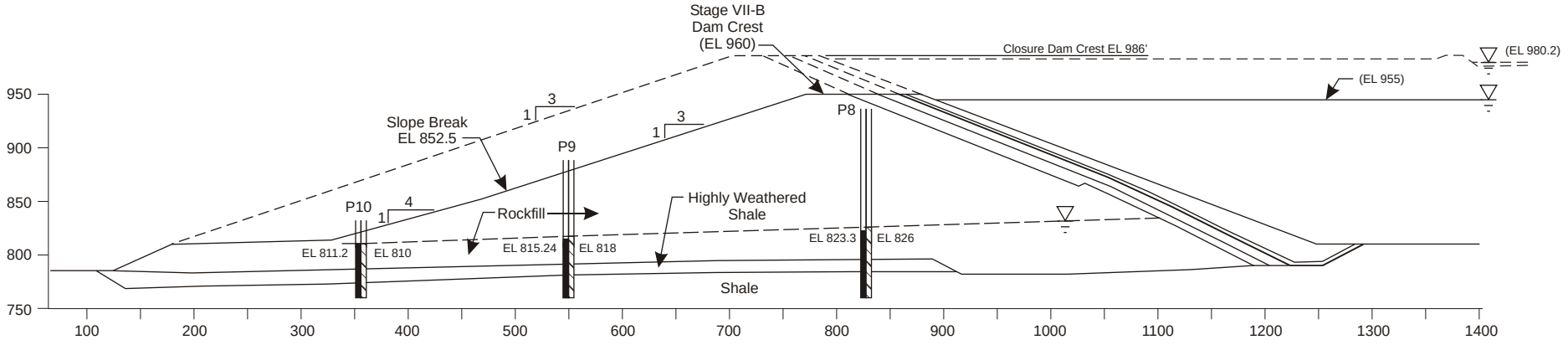


Figure 6-21

Equipotential Lines at Station 8+00: Stage VII-B, 300-foot Beach



LEGEND

Maximum recorded water level
(Stage VII-B)

Water levels from seepage analysis
(Stage VII-B, 300-Foot Beach)

0 100 200
Horizontal and Vertical Scale in Feet

Figure 6-22

Phreatic Surface Downstream of Liner for Stage VII-B, 300-Foot Beach

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

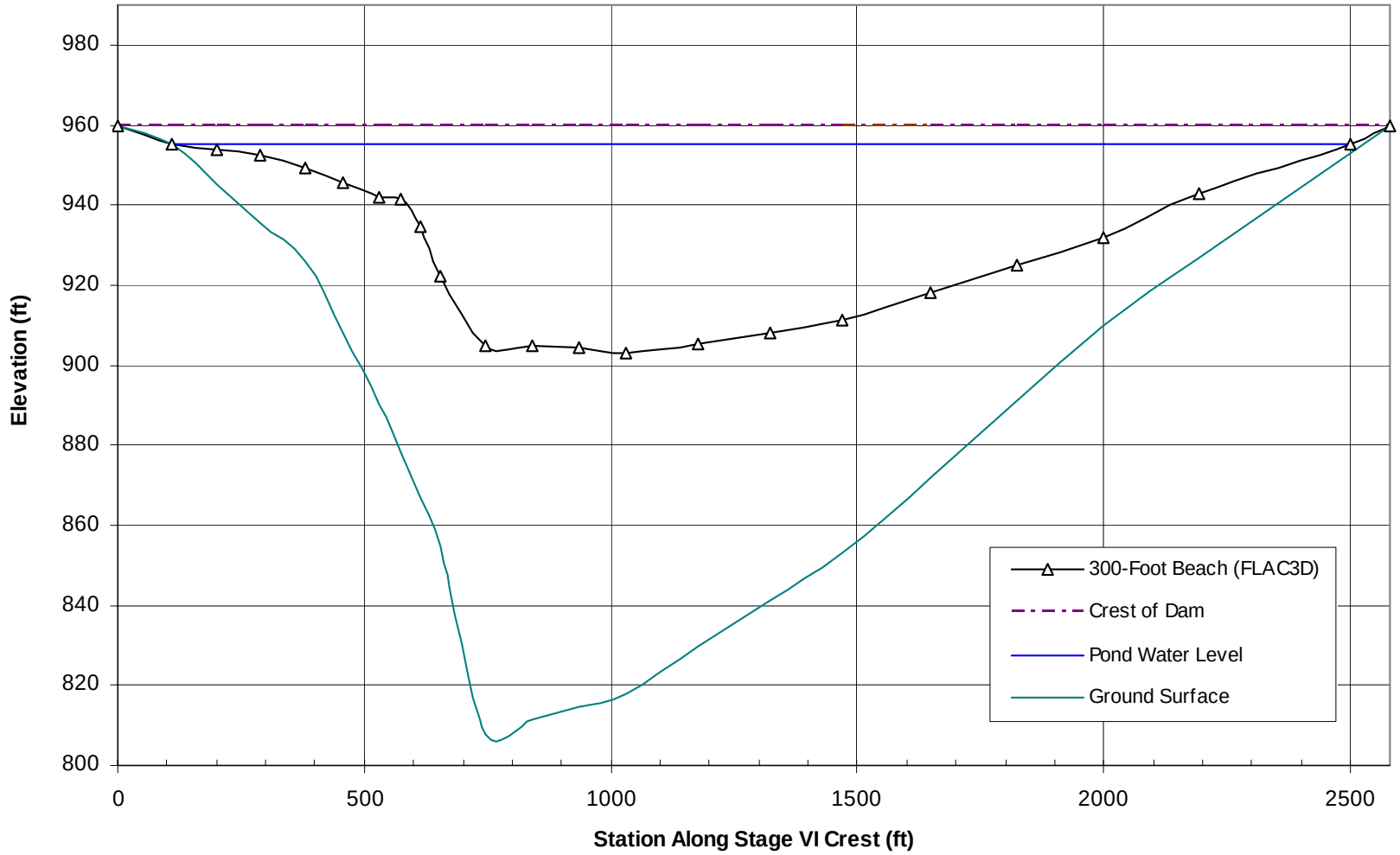
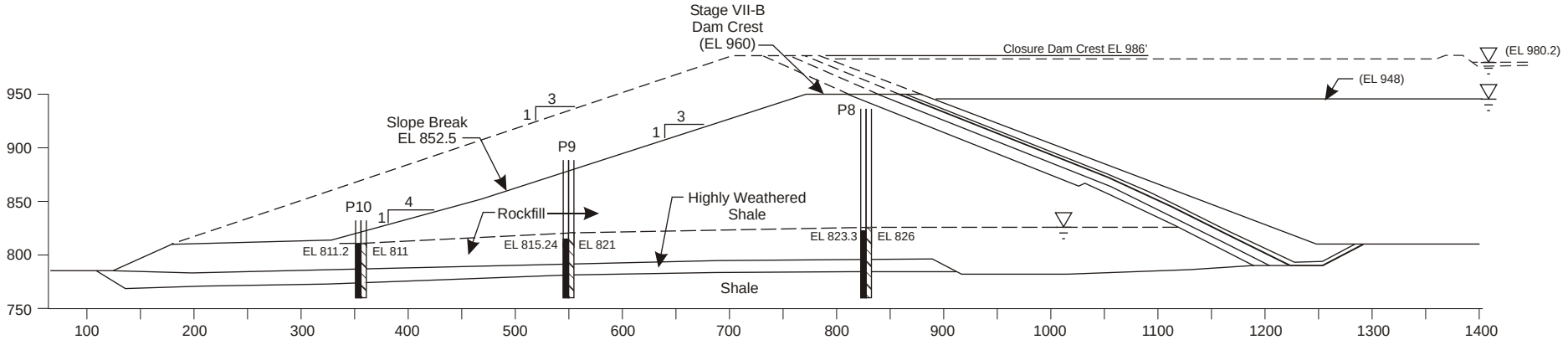


Figure 6-23

Phreatic Surface Upstream of Liner: Stage VII-B, 300-foot Beach



LEGEND

Maximum recorded water level (Stage VII-B)

Water levels from seepage analysis (closure)

0 100 200

Horizontal and Vertical Scale in Feet

Figure 6-24

Phreatic Surface Downstream of Liner Closure, 900-Foot Beach

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

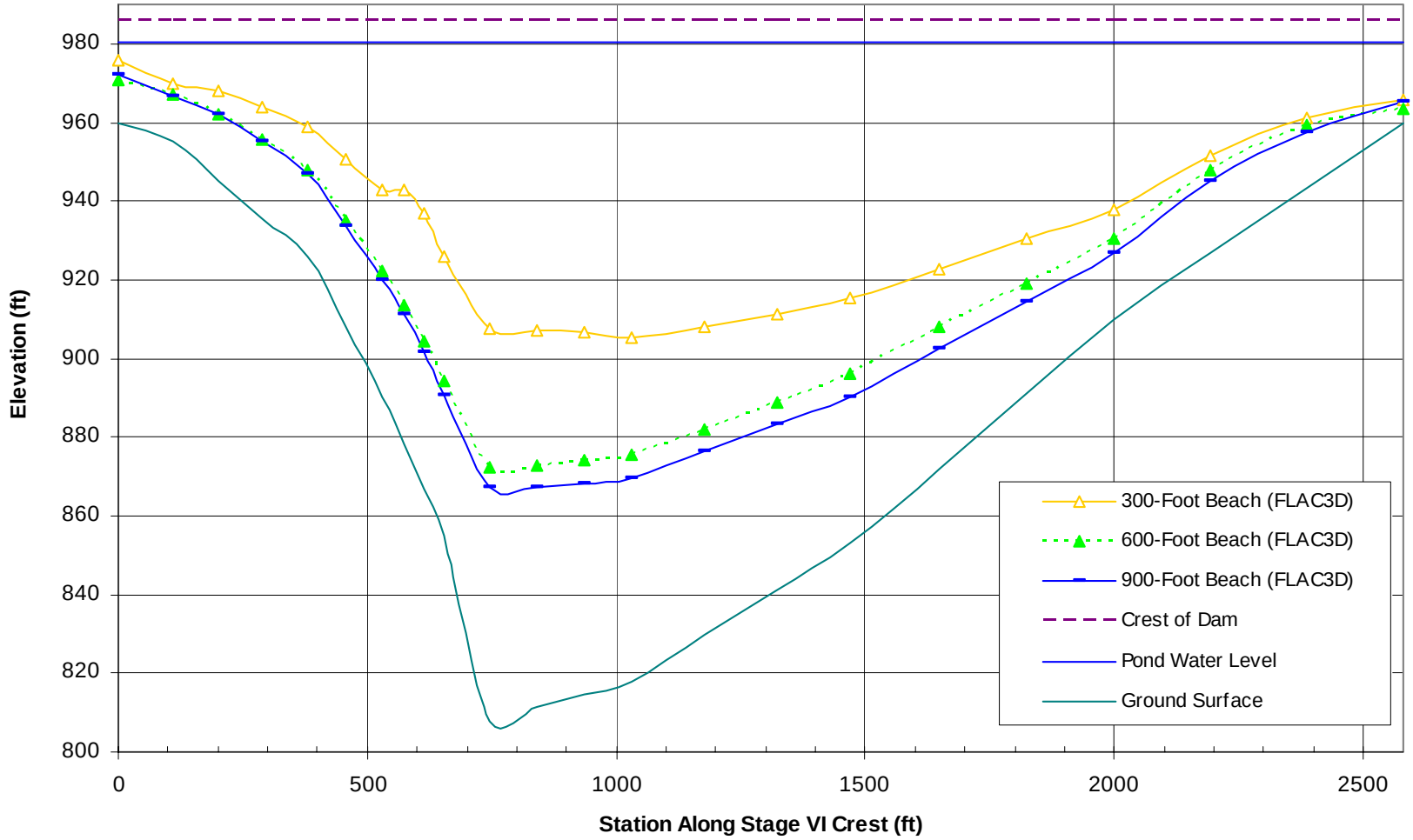


Figure 6-25

Phreatic Surface Upstream of Liner: Closure

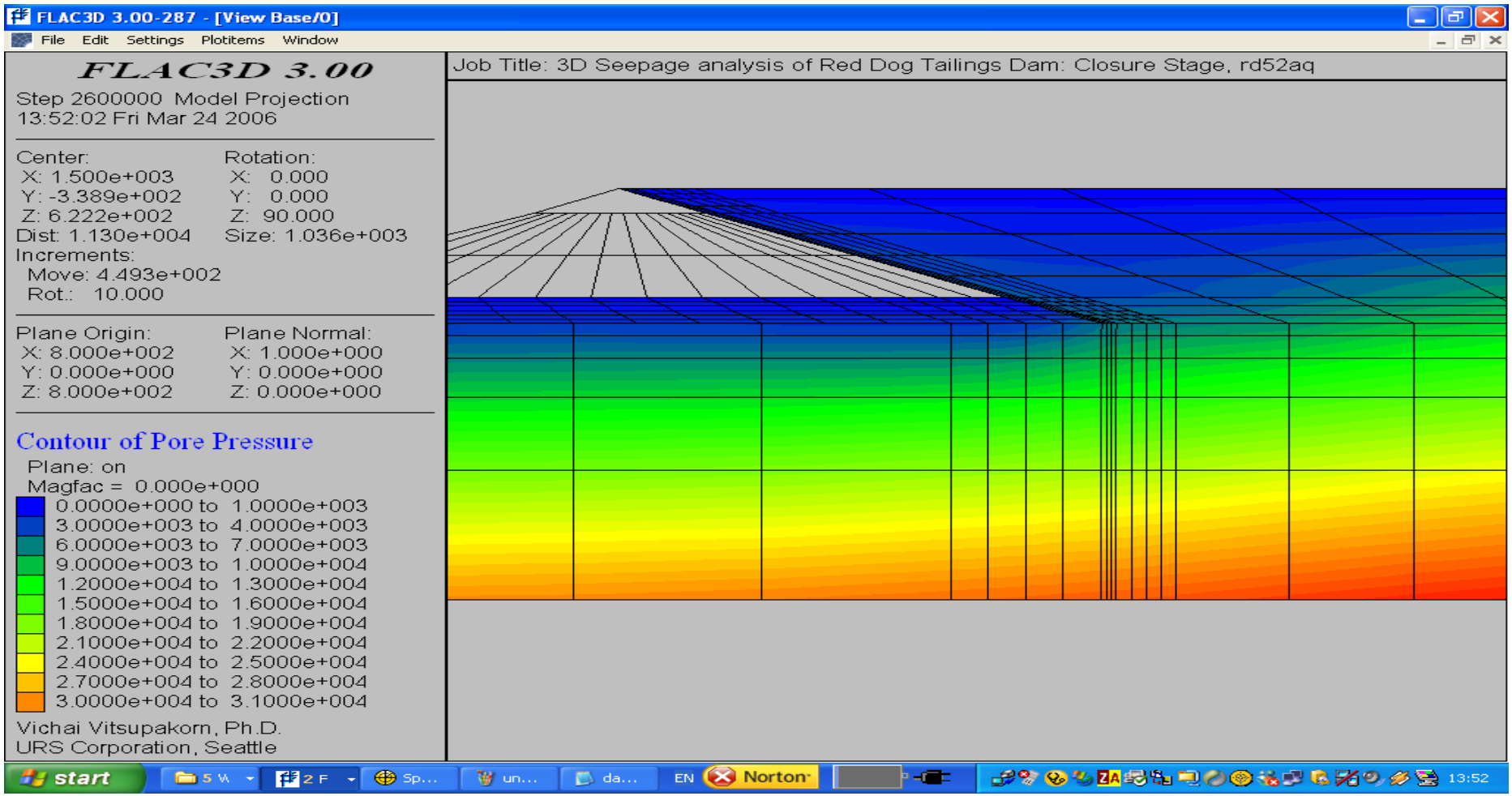


Figure 6-26

Pore Pressure Distribution at Station 8+00: Closure, No Beach

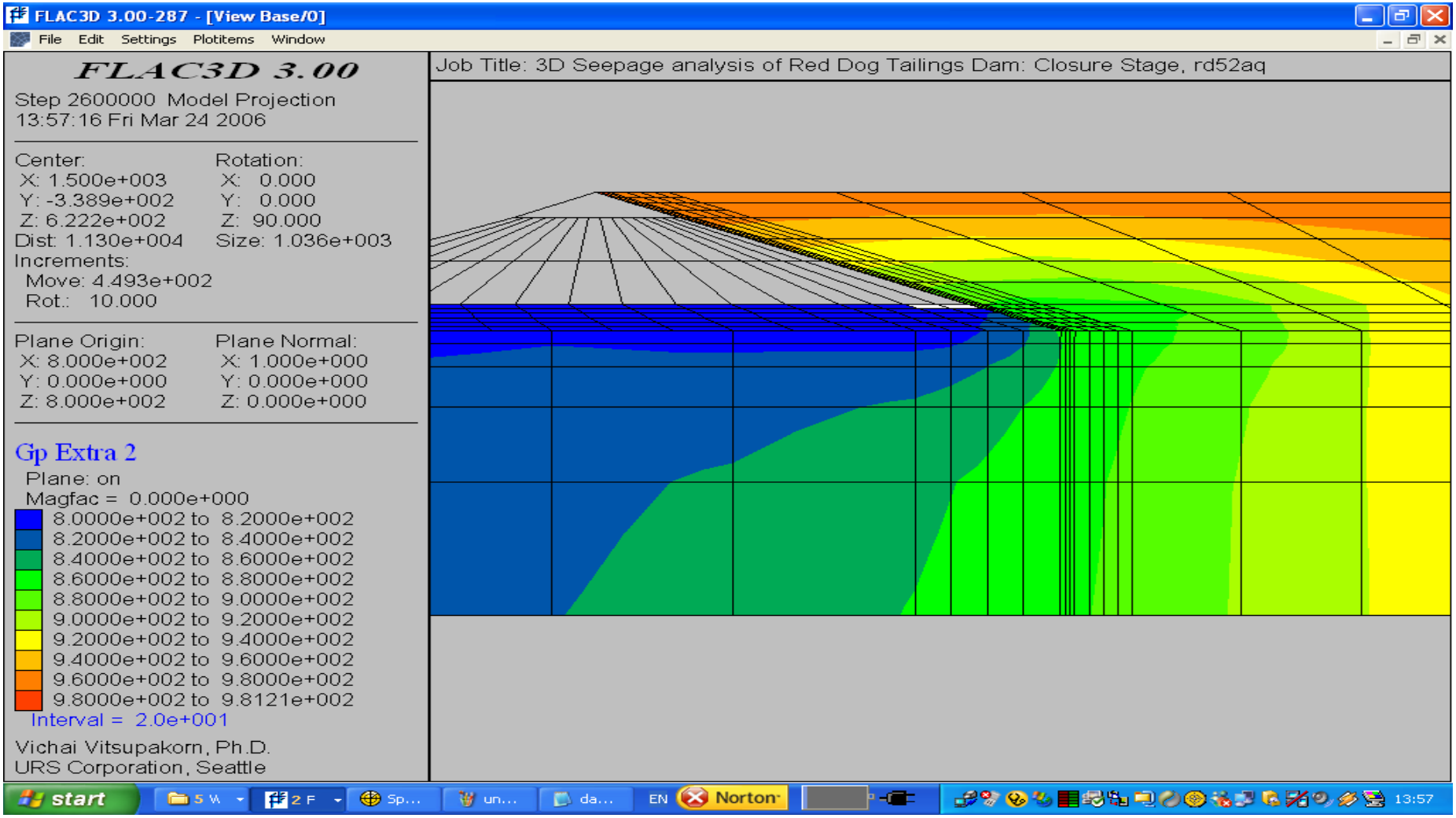


Figure 6-27

Equipotential Lines at Station 8+00: Closure, No Beach

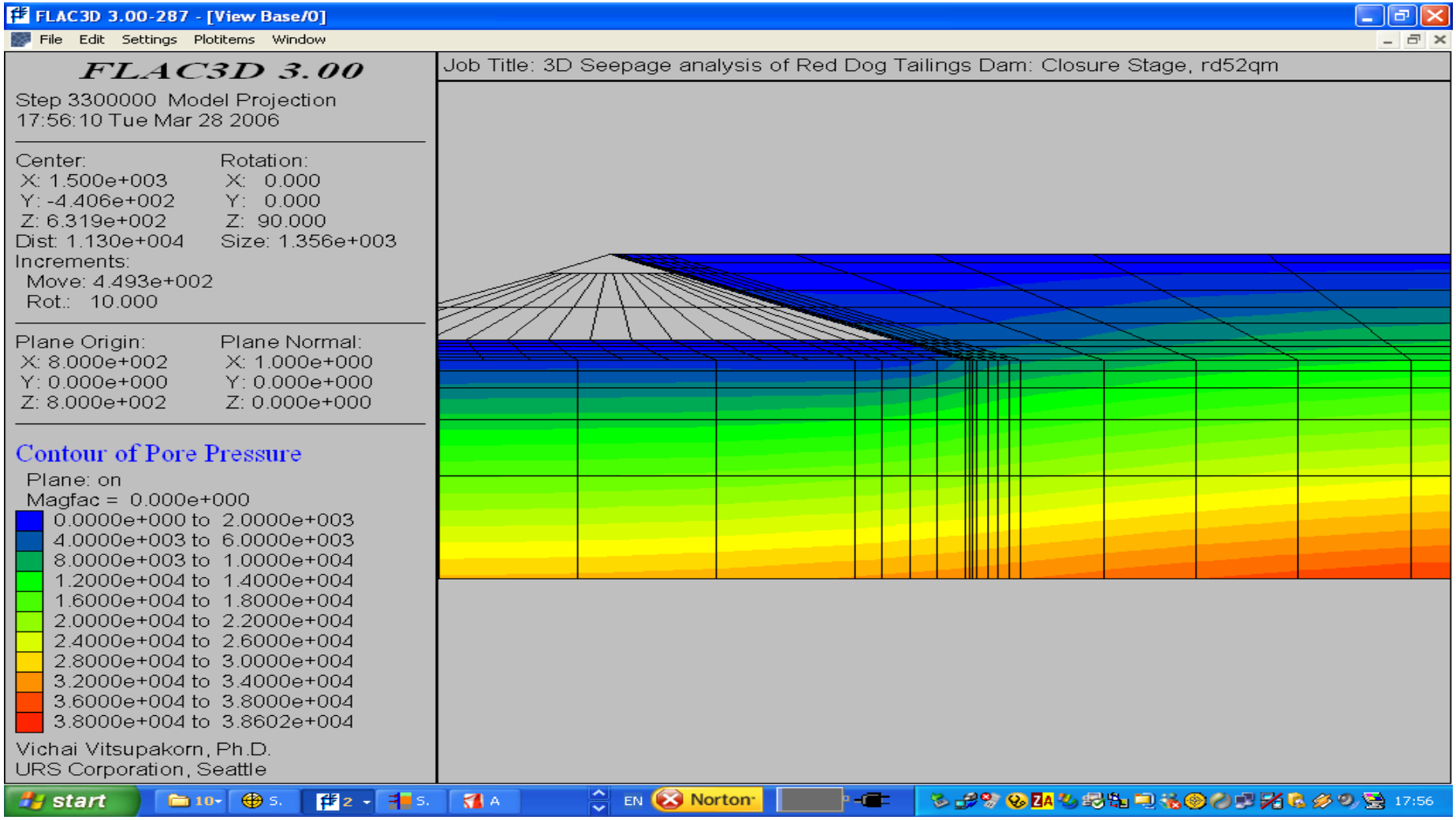


Figure 6-28

Pore Pressure Distribution at Station 8+00: Closure, 300-foot Beach

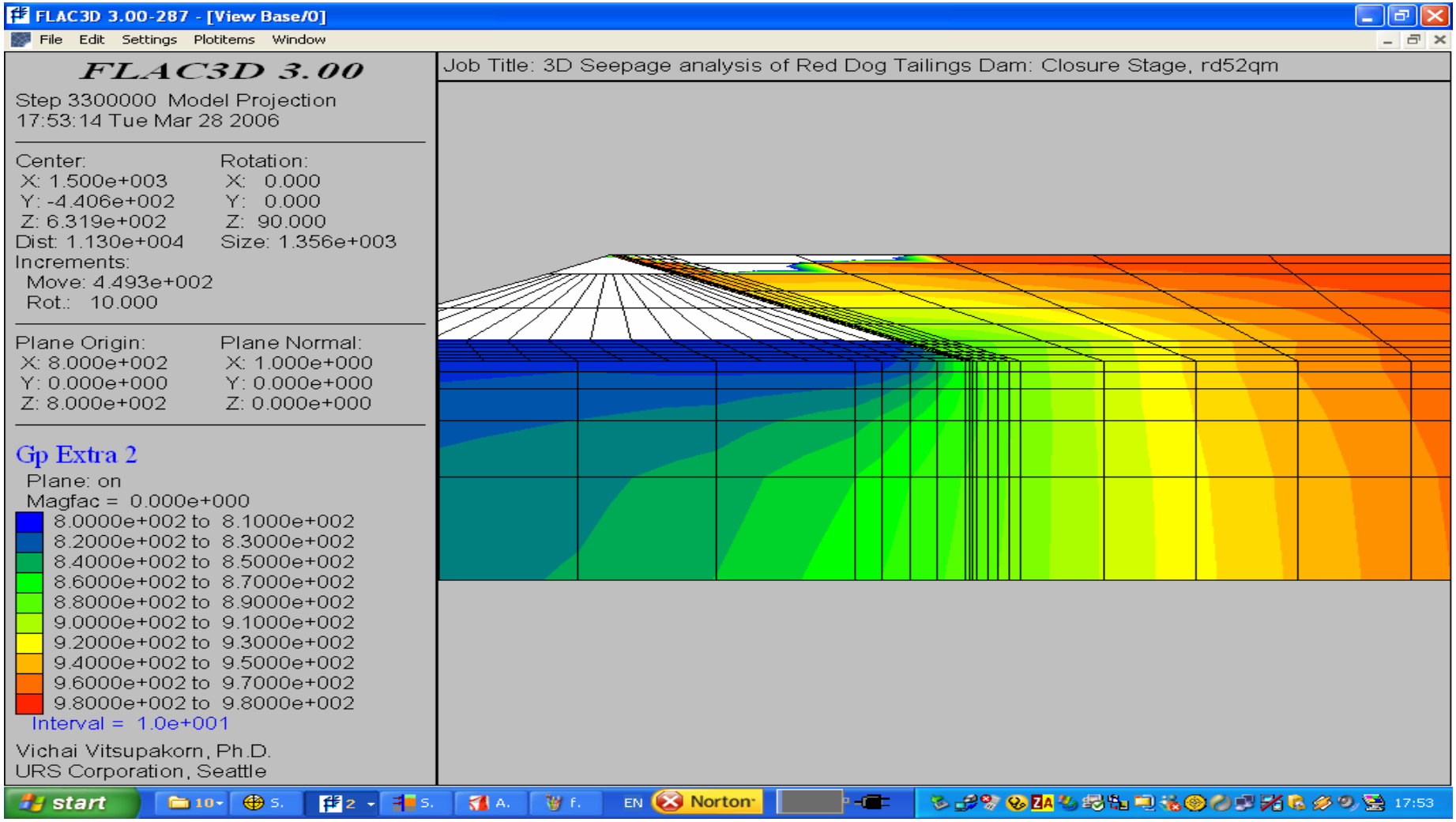


Figure 6-29

Equipotential Lines at Station 8+00: Closure, 300-foot Beach

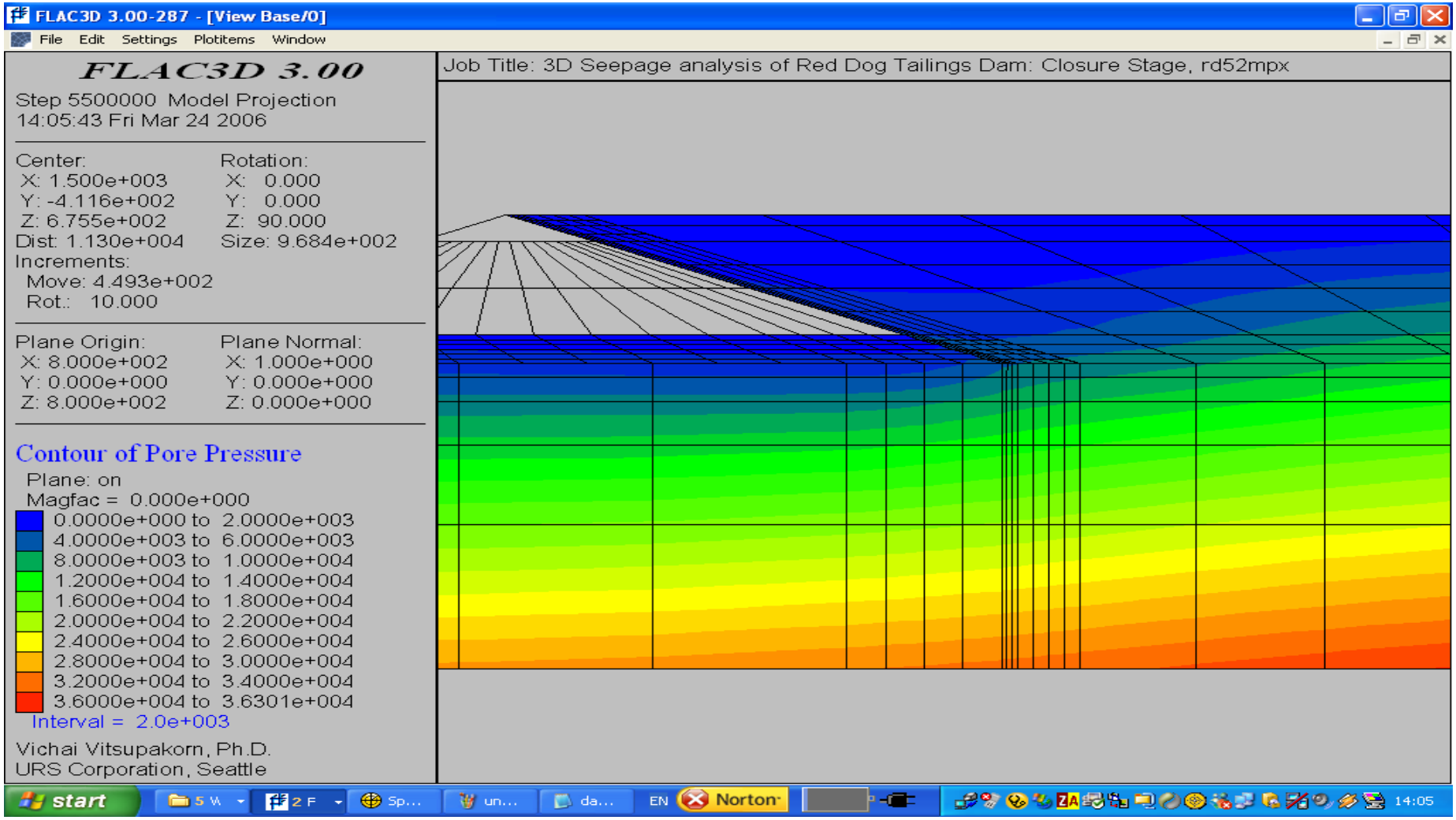


Figure 6-30

Pore Pressure Distribution at Station 8+00: Closure, 600-foot Beach

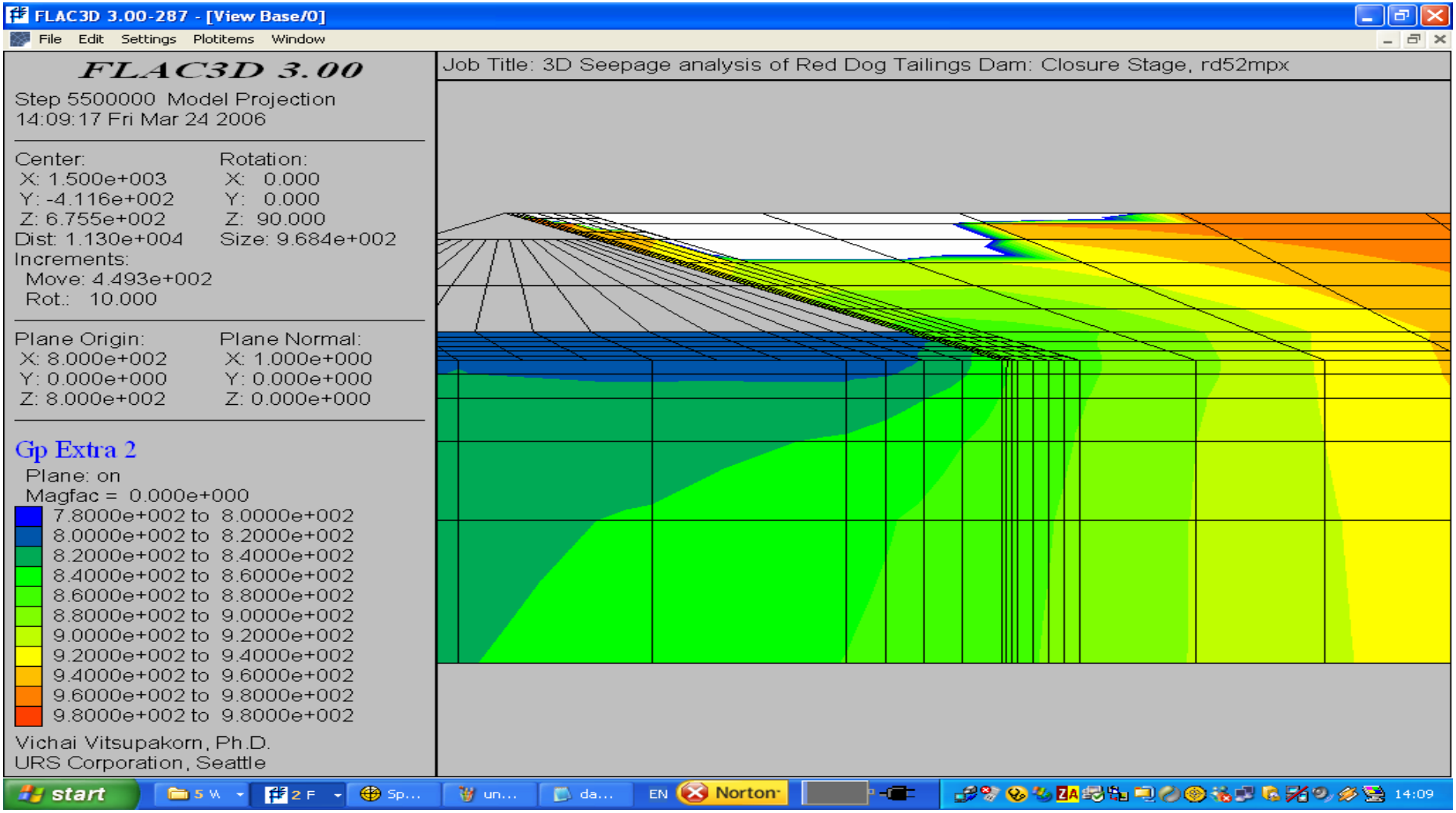


Figure 6-31

Equipotential Lines at Station 8+00: Closure, 600-foot Beach

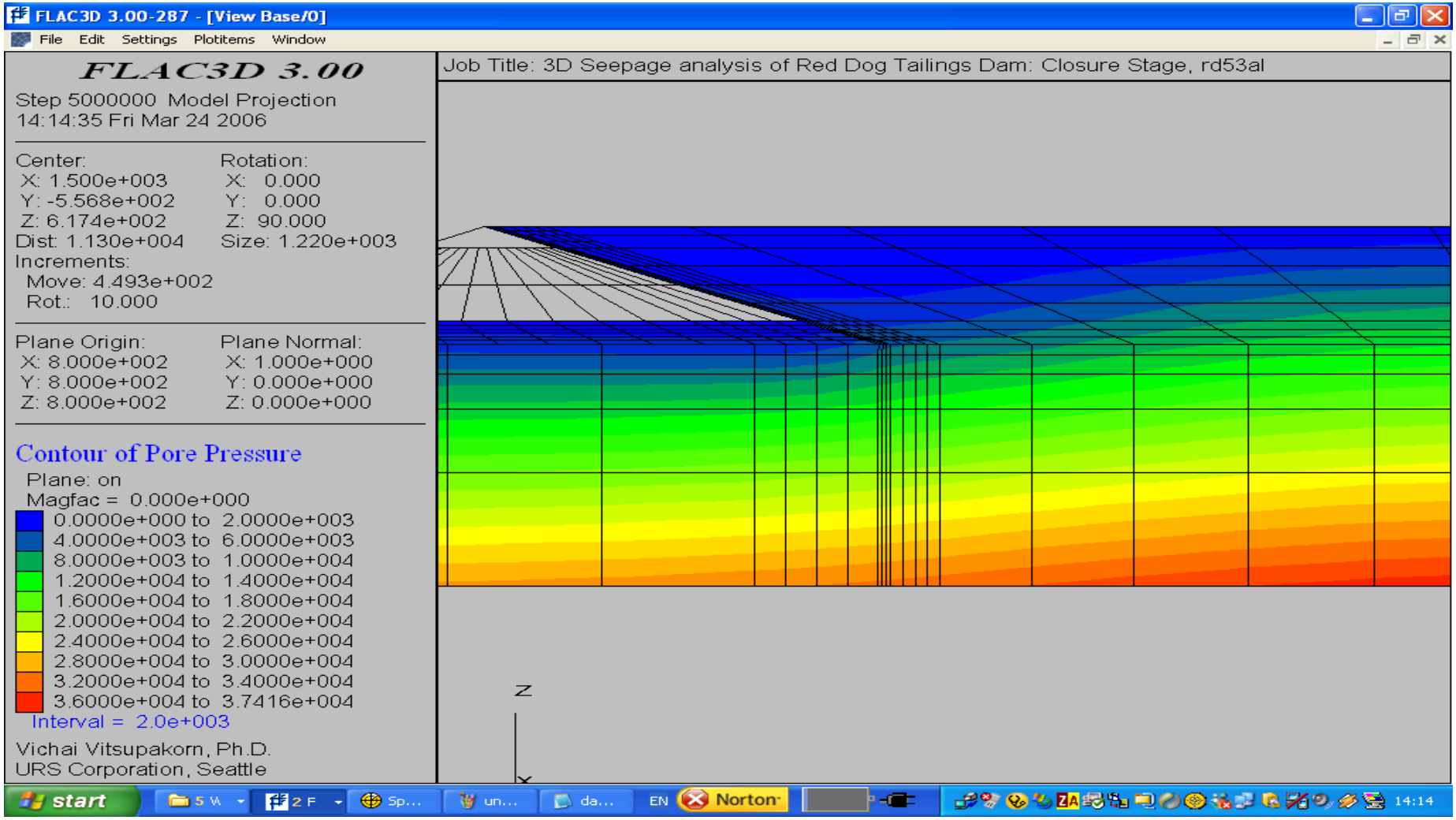


Figure 6-32

Pore Pressure Distribution at Station 8+00: Closure, 900-foot Beach

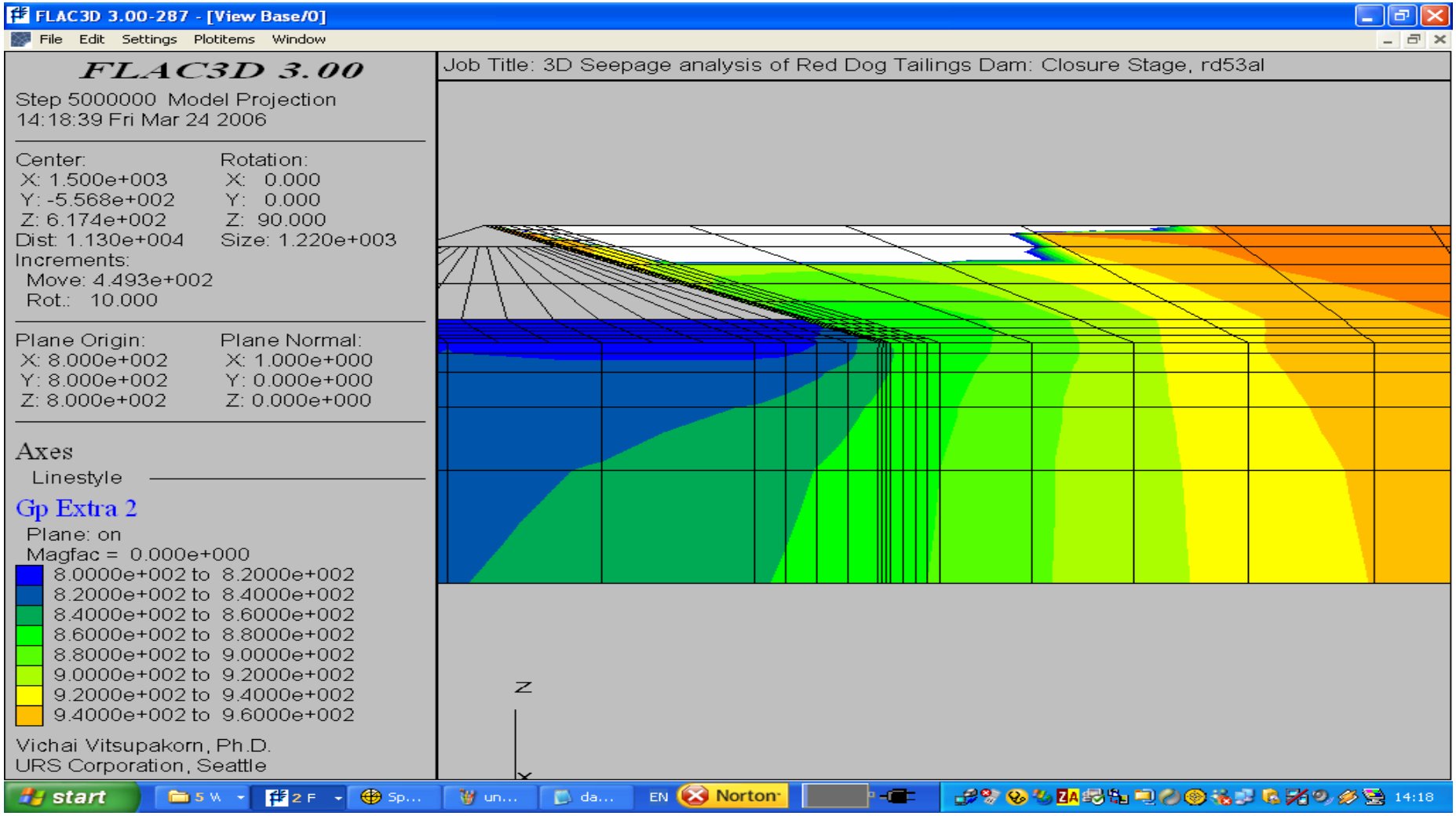
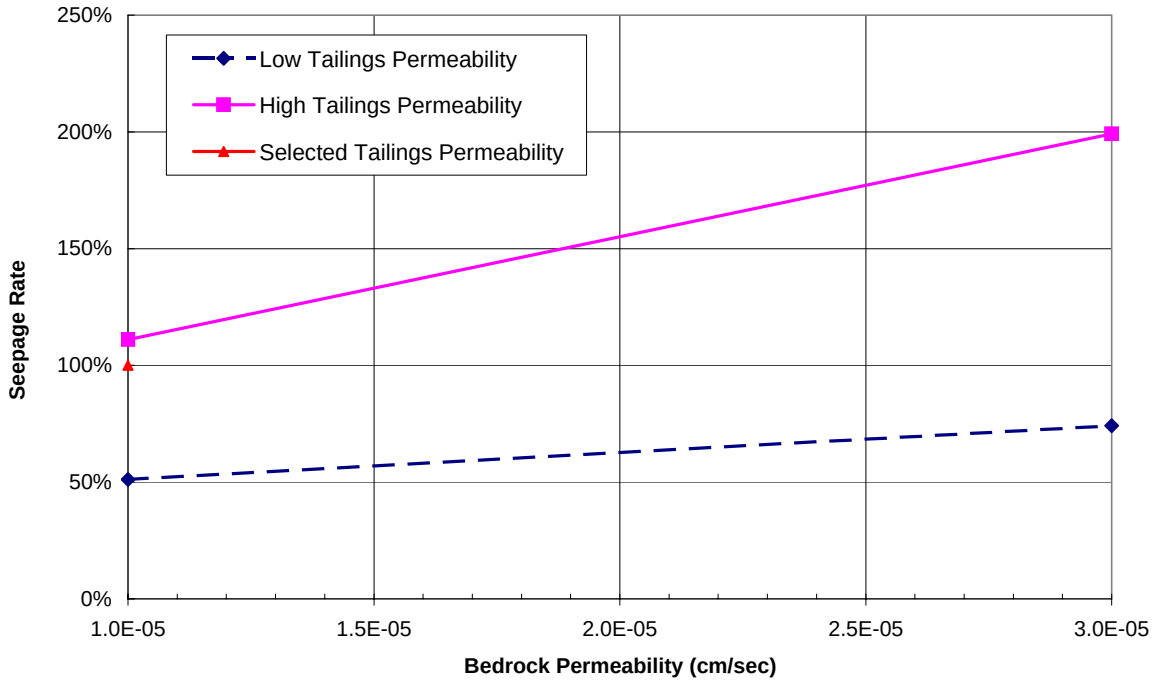


Figure 6-33

Equipotential Lines at Station 8+00: Closure, 900-foot Beach

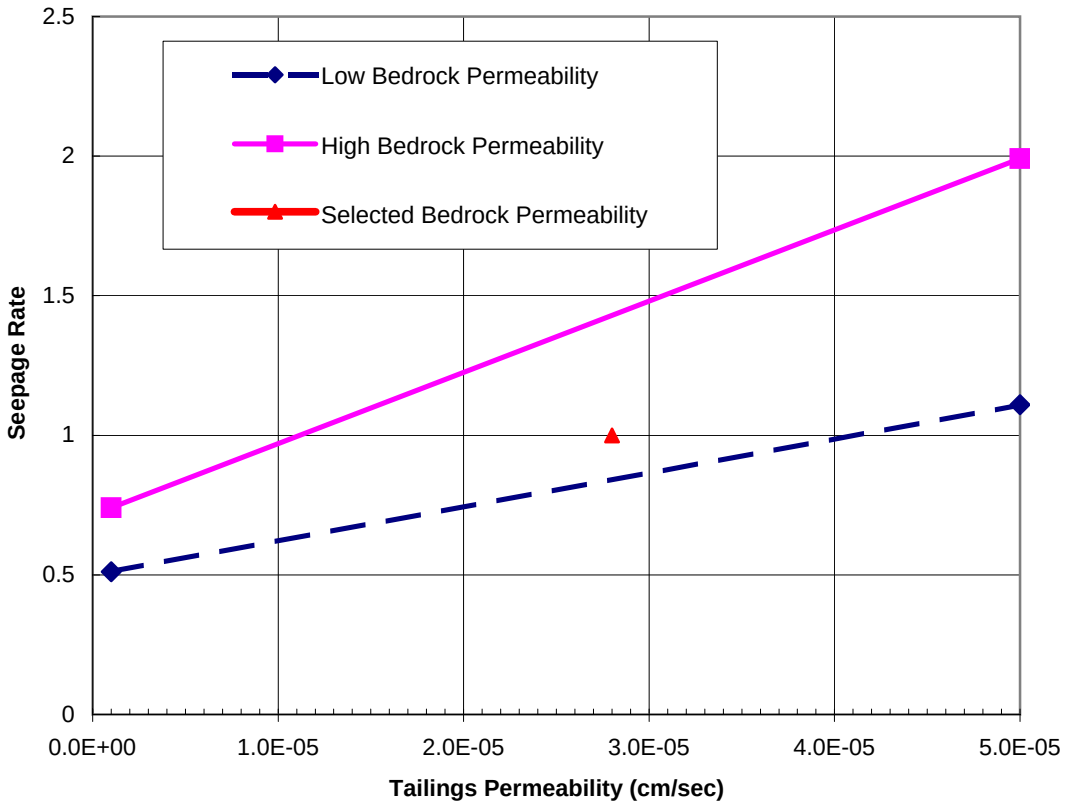


Permeability Values for 2D Sensitivity Analyses

Case	$k_{bedrock}$		$k_{tailings}$		Seepage Rate	
	(cm/sec)		(cm/sec)		(gpm)	(%)[1]
1	1.00E-05	Low	1.00E-06	Low	734	51%
2	3.00E-05	High	1.00E-06	Low	1064	74%
3	1.00E-05	Selected	2.80E-05	Selected	1436	100%
4	1.00E-05	Low	5.00E-05	High	1594	111%
5	3.00E-05	High	5.00E-05	High	2860	199%

Note: [1.] Based on a seepage rate estimated using selected permeabilities, and depth to permafrost data in 2005.

Figure 6-34
Effect of Permeabilty of Bedrock on Seepage Rate
Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska



Permeability Values for 2D Sensitivity Analyses

Case	$k_{bedrock}$		$k_{tailings}$		Seepage Rate	
	(cm/sec)		(cm/sec)		(gpm)	(%)[1]
1	1.00E-05	Low	1.00E-06	Low	734	51%
2	3.00E-05	High	1.00E-06	Low	1064	74%
3	1.00E-05	Selected	2.80E-05	Selected	1436	100%
4	1.00E-05	Low	5.00E-05	High	1594	111%
5	3.00E-05	High	5.00E-05	High	2860	199%

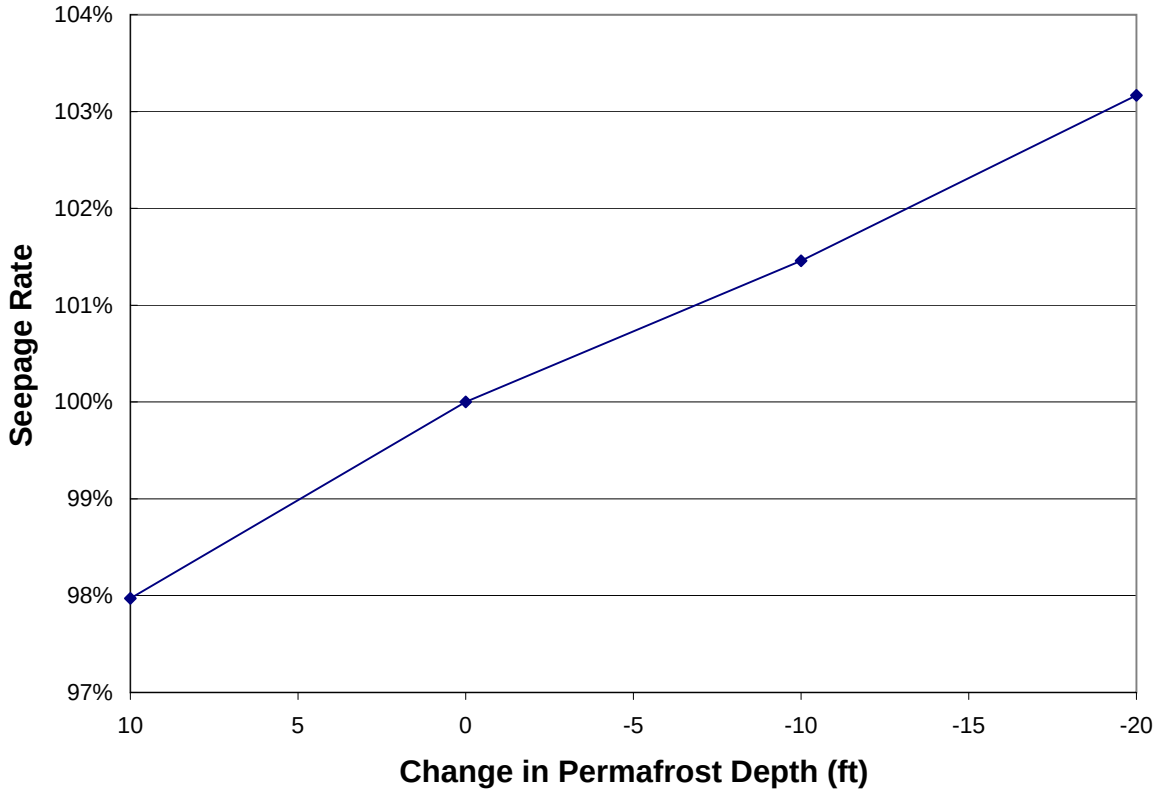
Note: [1.] Based on a seepage rate estimated using selected permeabilities, and depth to permafrost data in 2005.

Figure 6-35

URS Job No. 33757098 **Effect of Permeabilty of Tailings on Seepage Rate**

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska



Summary of Seepage Variation with Different Permafrost Depth

Change in Permafrost Depth[2] (ft)	Seepage Rate	
	(gpm)	(%)[1]
10	1407	98%
0	1436	100%
-10	1457	101%
-20	1482	103%

Note: [1.] Based on a seepage rate estimated using depth to permafrost data in 2005.

[2.] Depth to permafrost change (decrease) from 2005 data.

Figure 6-36

URS Job No. 33757098

Effect of Permafrost Depth on Seepage Rate

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS

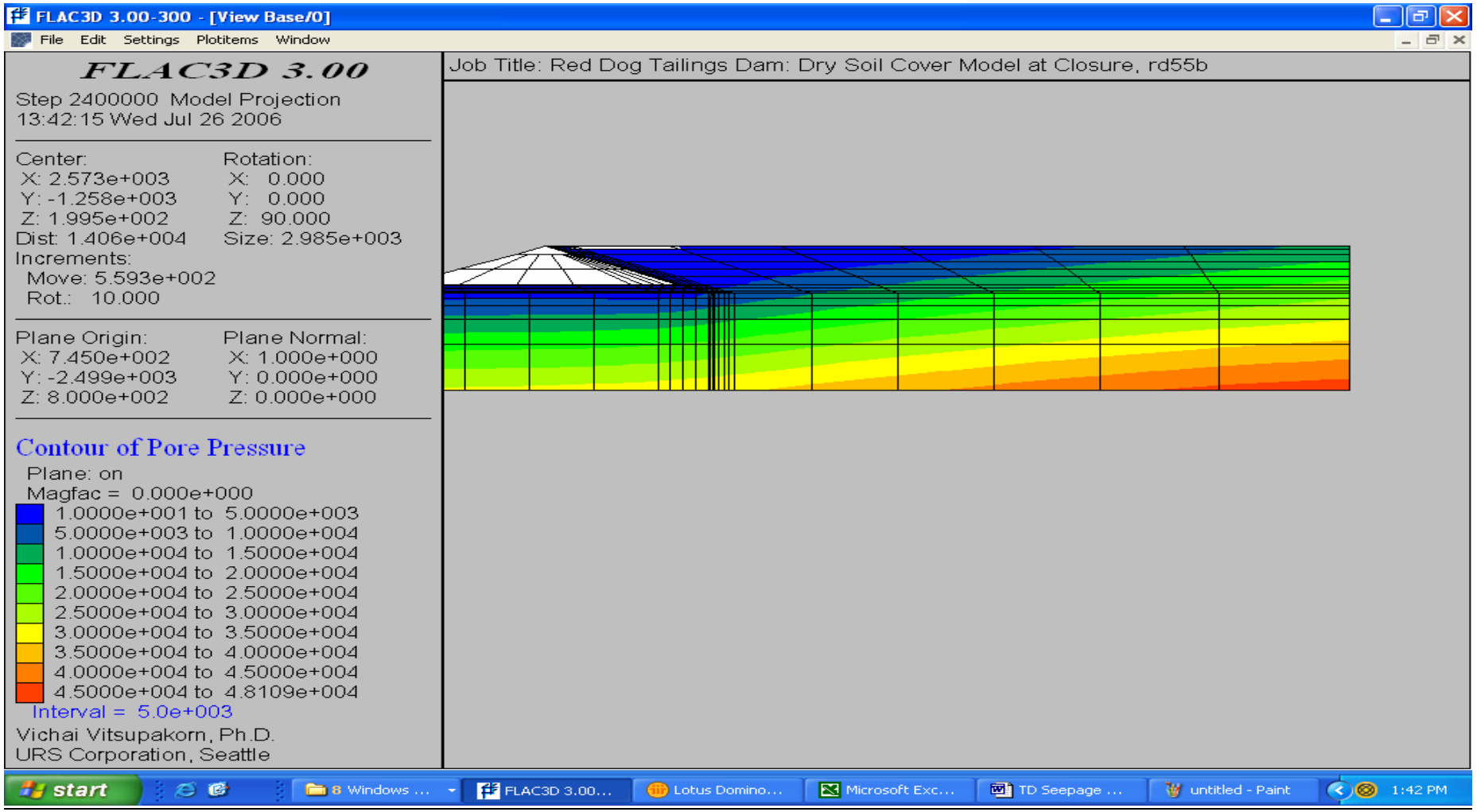


Figure 6-37

URS Job No. 33757098 **Pore Pressure Distribution at Station 7+45: Dry Soil Cover Option-400 gpm (Top and Upstream Inflow)**

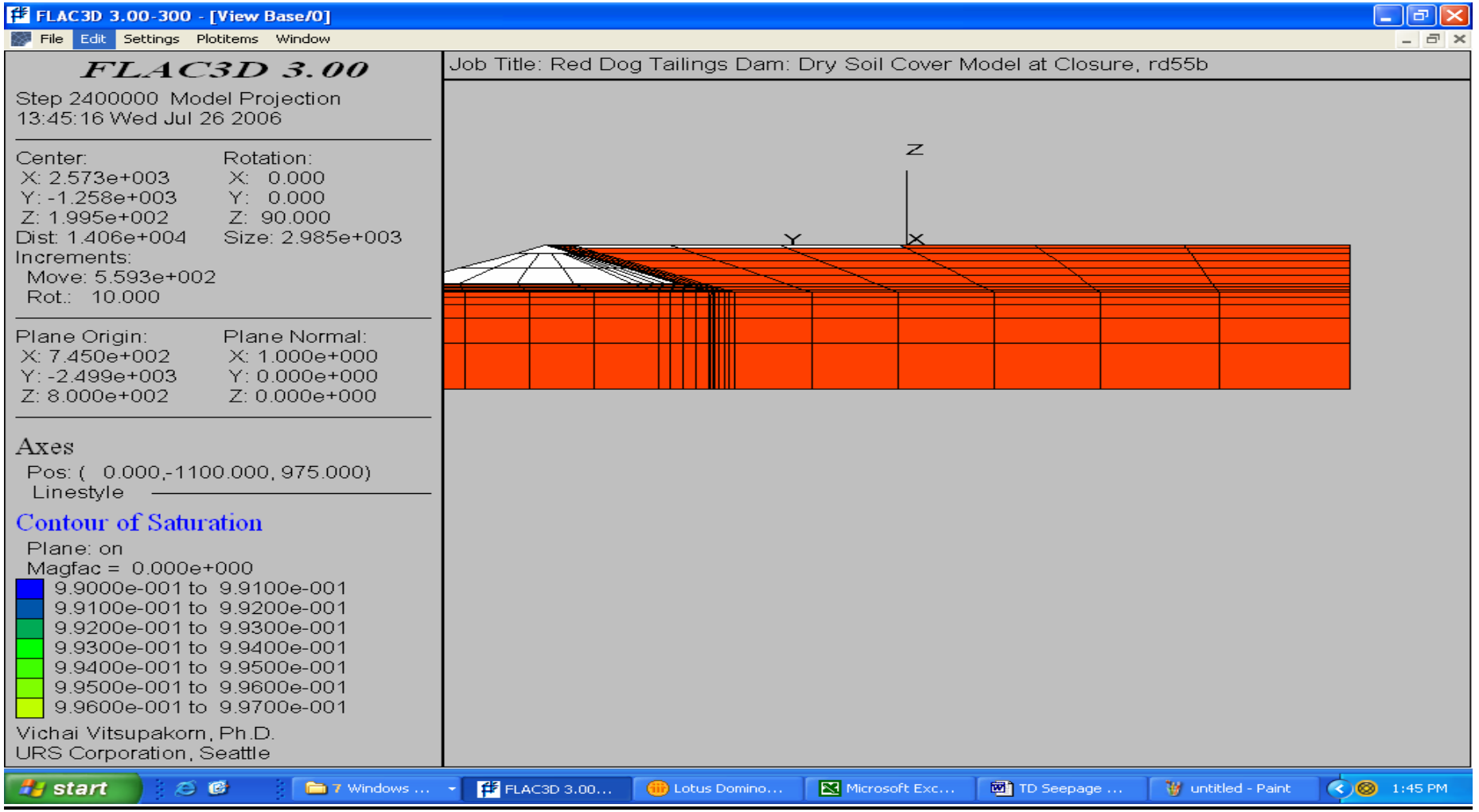


Figure 6-38

URS Job No. 33757098 **Saturation Distribution at Station 7+45: Dry Soil Cover Option-400 gpm (Top and Upstream Inflow)**

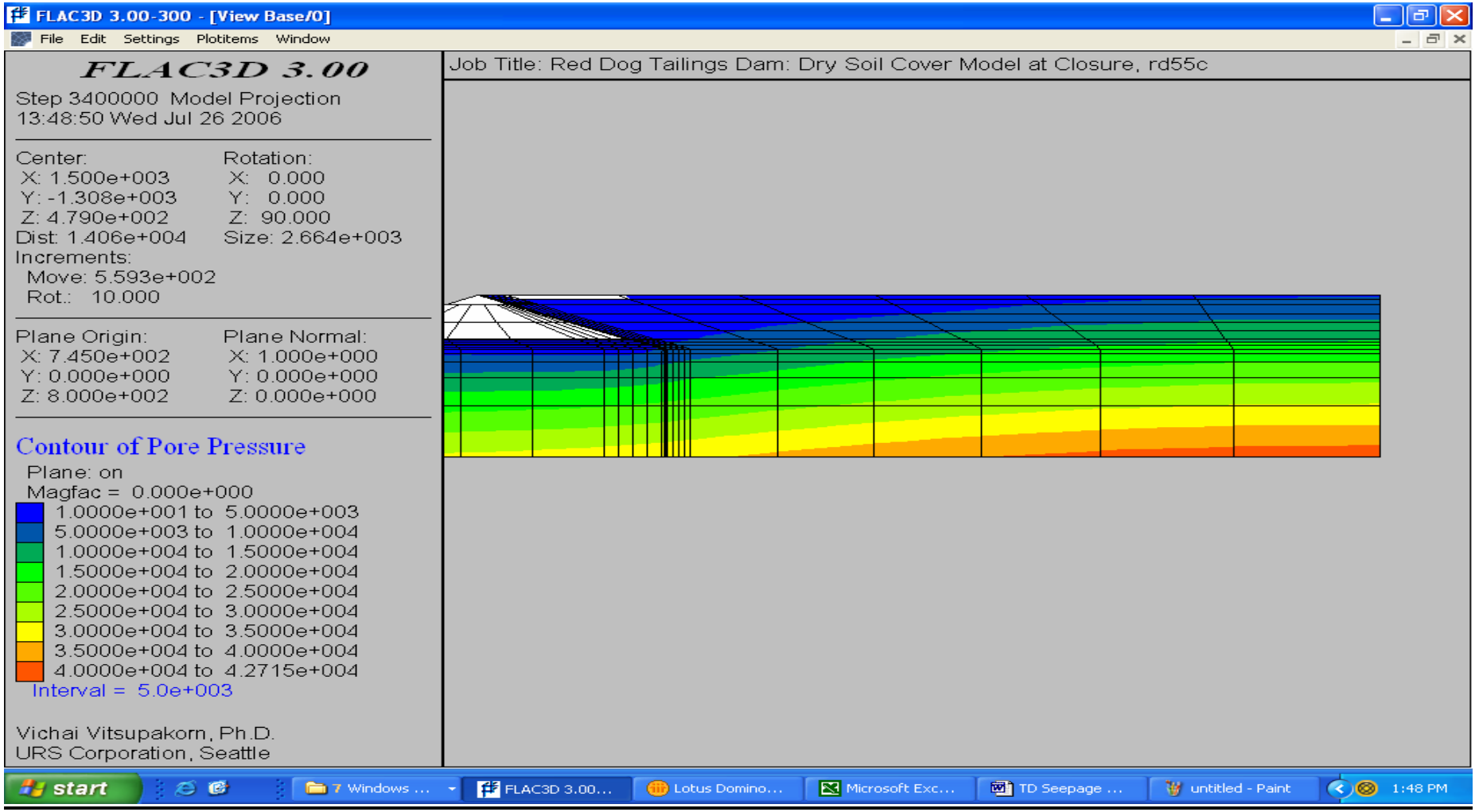


Figure 6-39

Pore Pressure Distribution at Station 7+45: Dry Soil Cover Option-400 gpm (Top Inflow only)

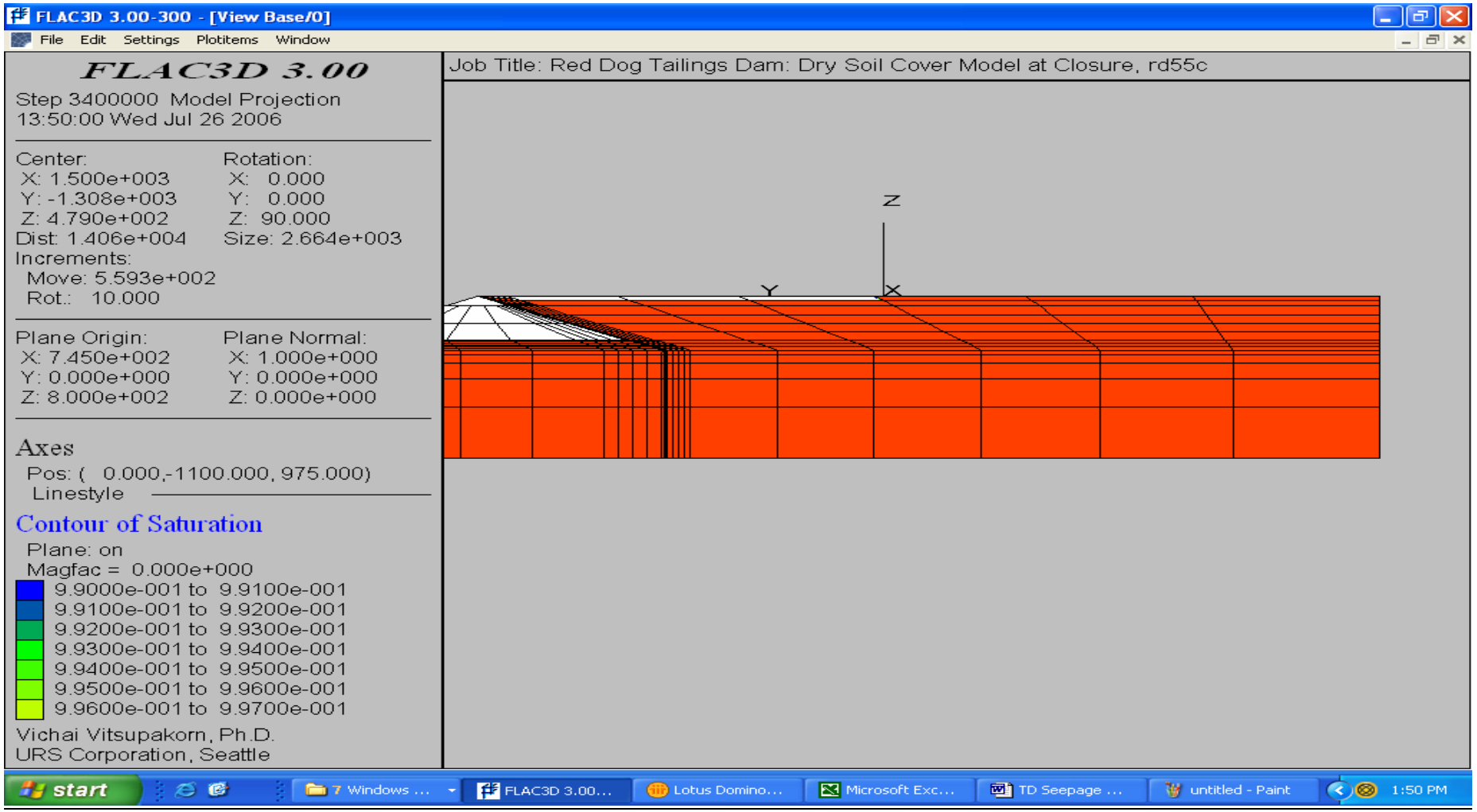


Figure 6-40

Saturation Distribution at Station 7+45: Dry Soil Cover Option-400 gpm (Top Inflow only)

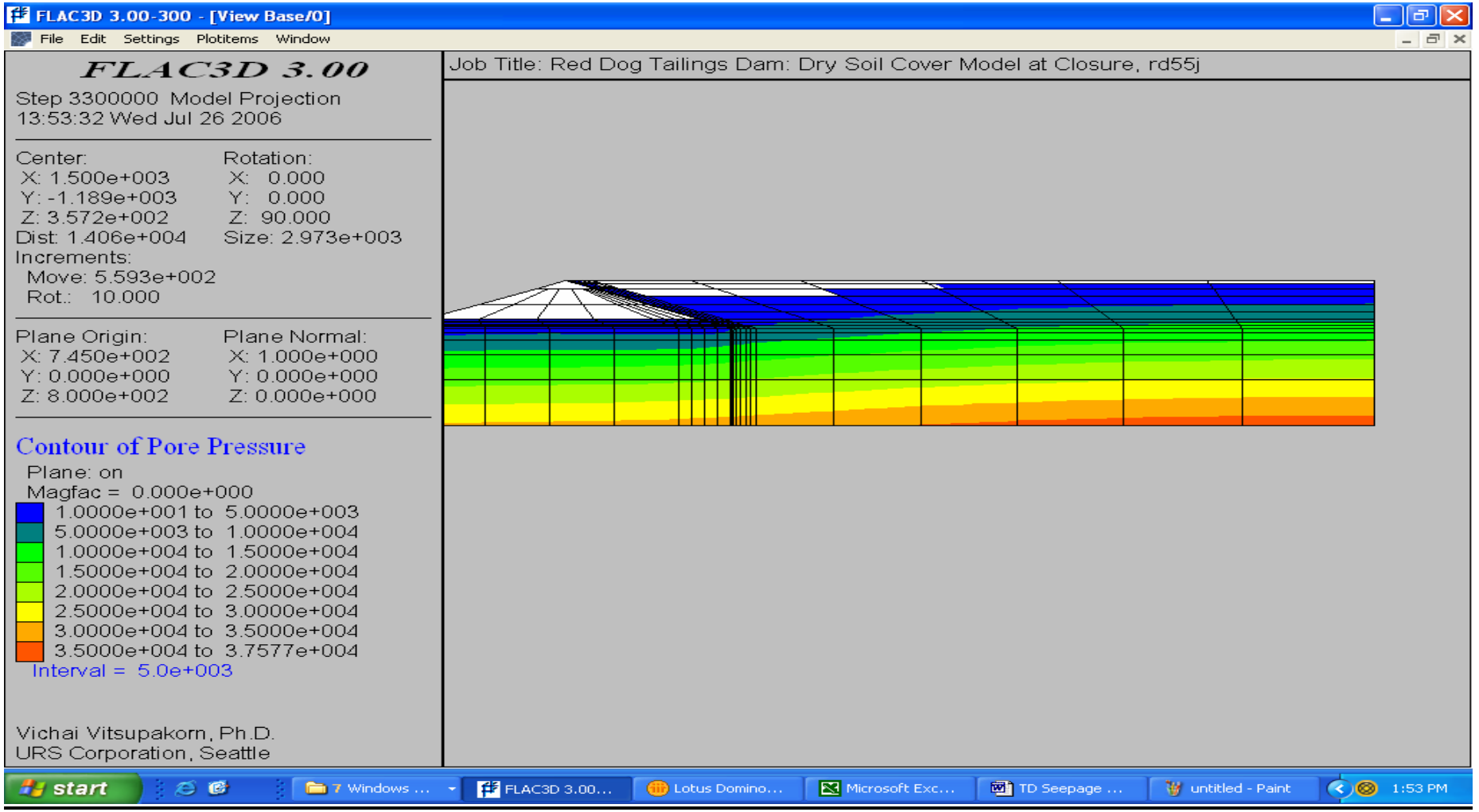


Figure 6-41

Pore Pressure Distribution at Station 7+45: Dry Soil Cover Option-25 gpm

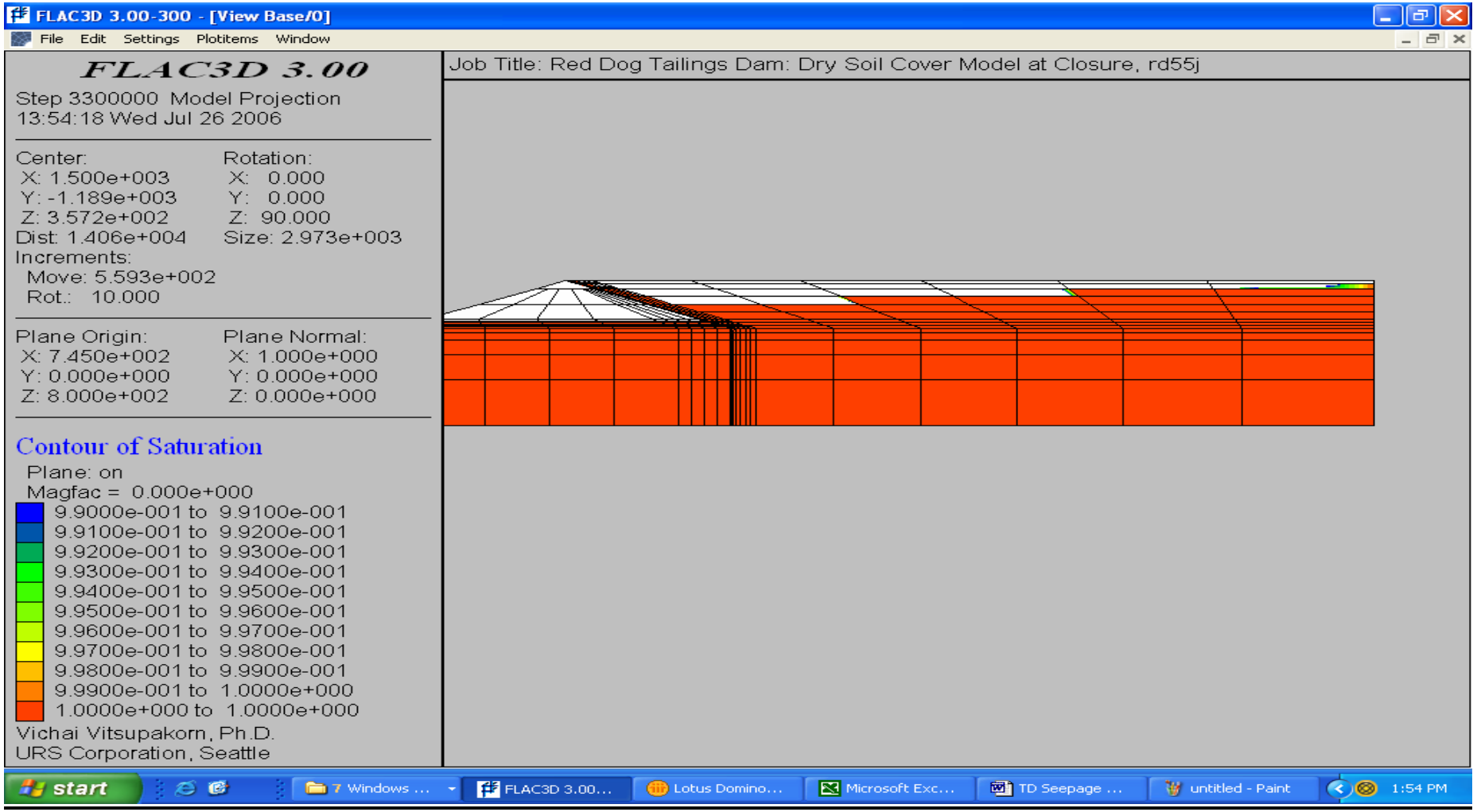


Figure 6-42

Saturation Distribution at Station 7+45: Dry Soil Cover Option-25 gpm

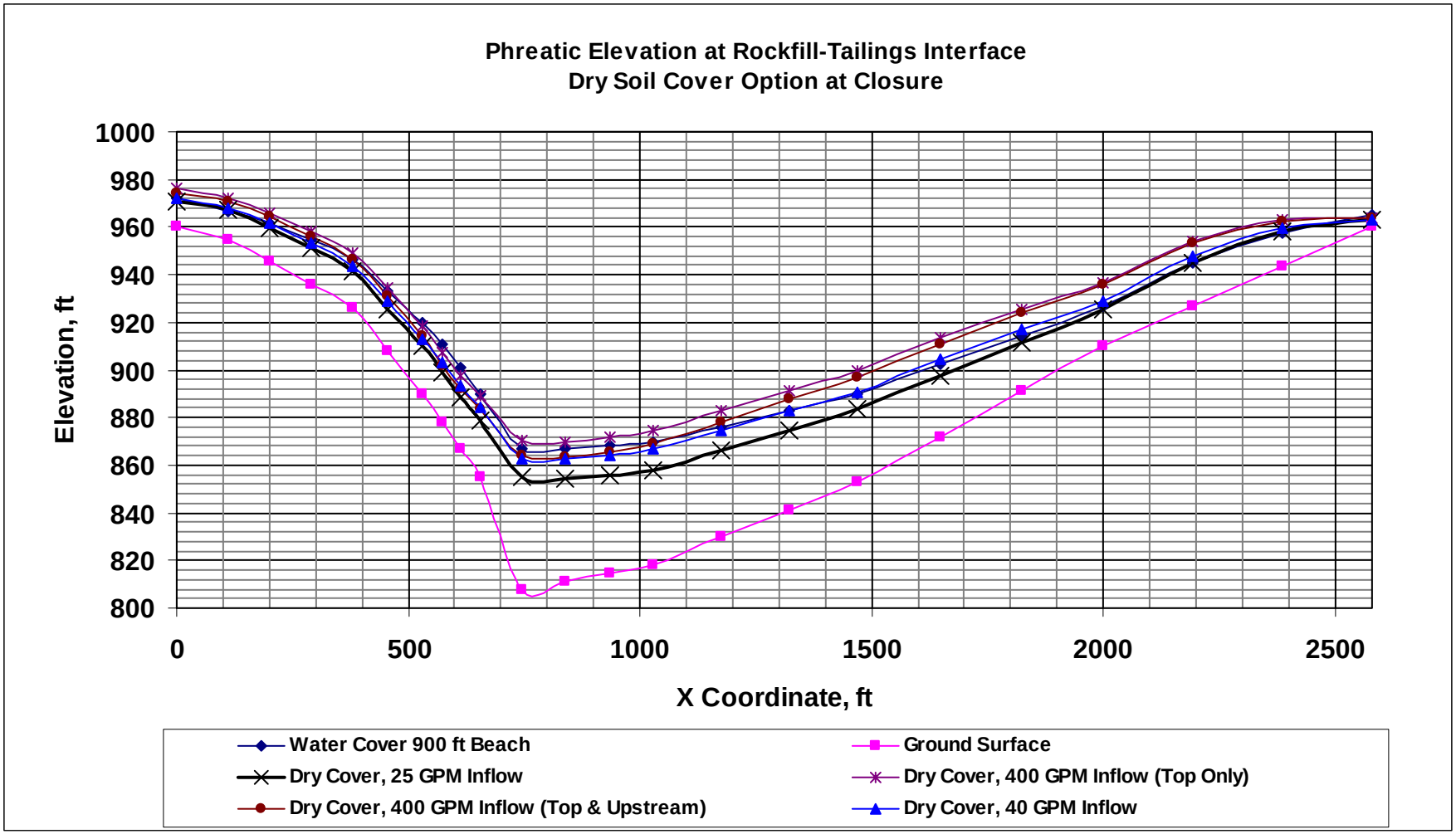


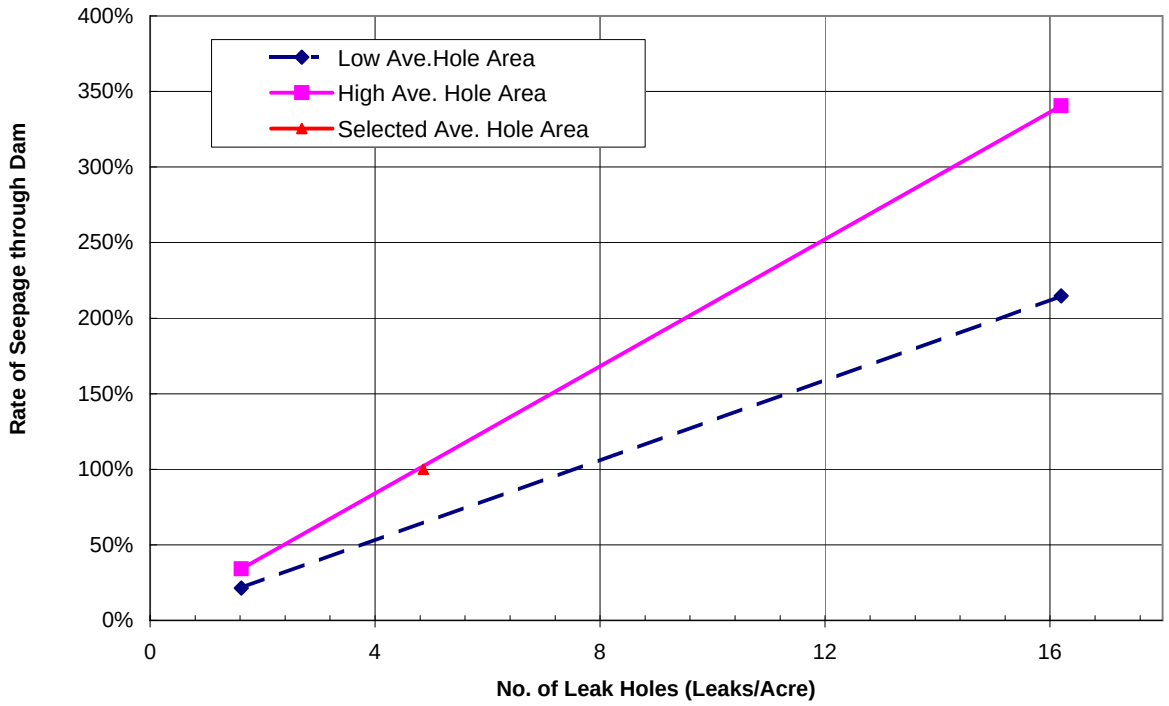
Figure 6-43

URS Job No. 33757098

Phreatic Surface Upstream of Liner: Dry Soil Cover and 900-foot Beach

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska



Case	No. of Leak Holes		Average Area of Leak Holes		Rate of Seepage through Dam	
	(Leaks/acre)		(in^2)		(gpm)	(%)[1]
1	1.6	Low	0.002	Low	24.6	21%
2	16.2	High	0.002	Low	246.4	215%
3	4.9	Selected	0.126	Selected	114.7	100%
4	1.6	Low	0.155	High	39.1	34%
5	16.2	High	0.155	High	390.5	340%

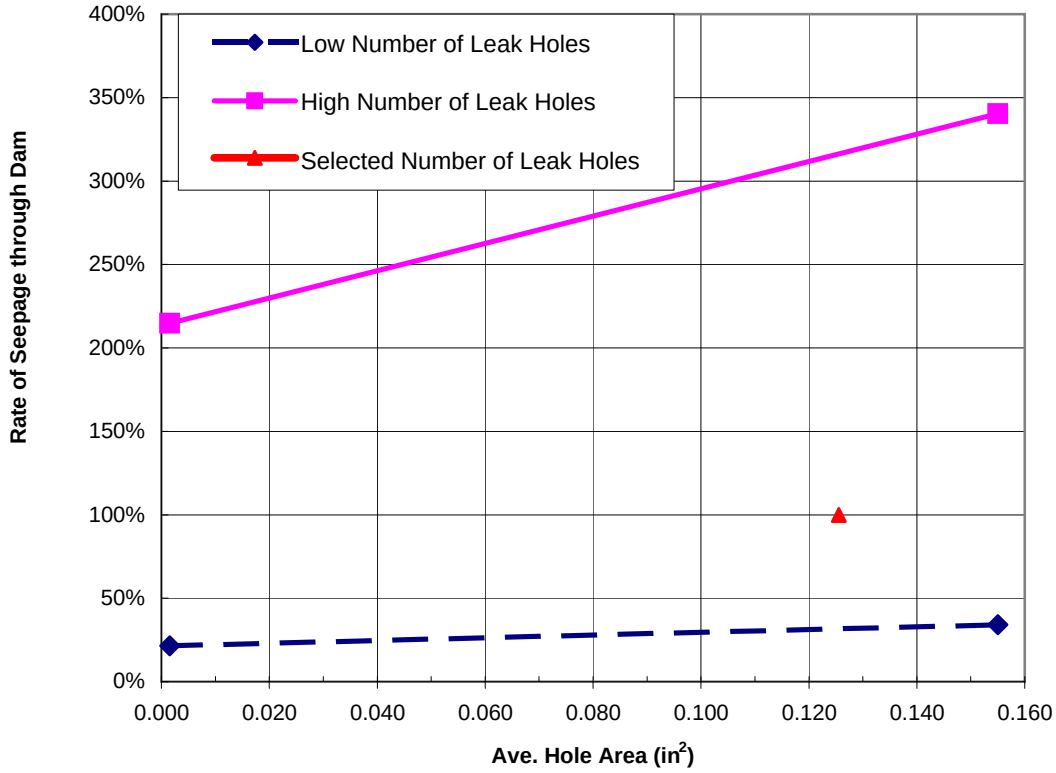
Note: [1.] Based on a seepage rate estimated using selected number and size of leak holes

Figure 7-1

URS Job No. 33757098 **Effect of Number of Leaks on Rate of Seepage through Dam, 600 Feet Beach at Closure**

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska



Case	No. of Leak Holes		Average Area of Leak Holes		Rate of Seepage through Dam	
	(Leaks/Acre)		(in^2)		(gpm)	(%)[1]
1	1.6	Low	0.002	Low	24.6	21%
2	16.2	High	0.002	Low	246.4	215%
3	4.9	Selected	0.126	Selected	114.7	100%
4	1.6	Low	0.155	High	39.1	34%
5	16.2	High	0.155	High	390.5	340%

Note:

[1.] Based on a seepage rate estimated using selected number and size of leak holes

Figure 7-2

URS Job No. 33757098 **Effect of Average Hole Area on Rate of Seepage through Dam, 600 Feet Beach, Closure**

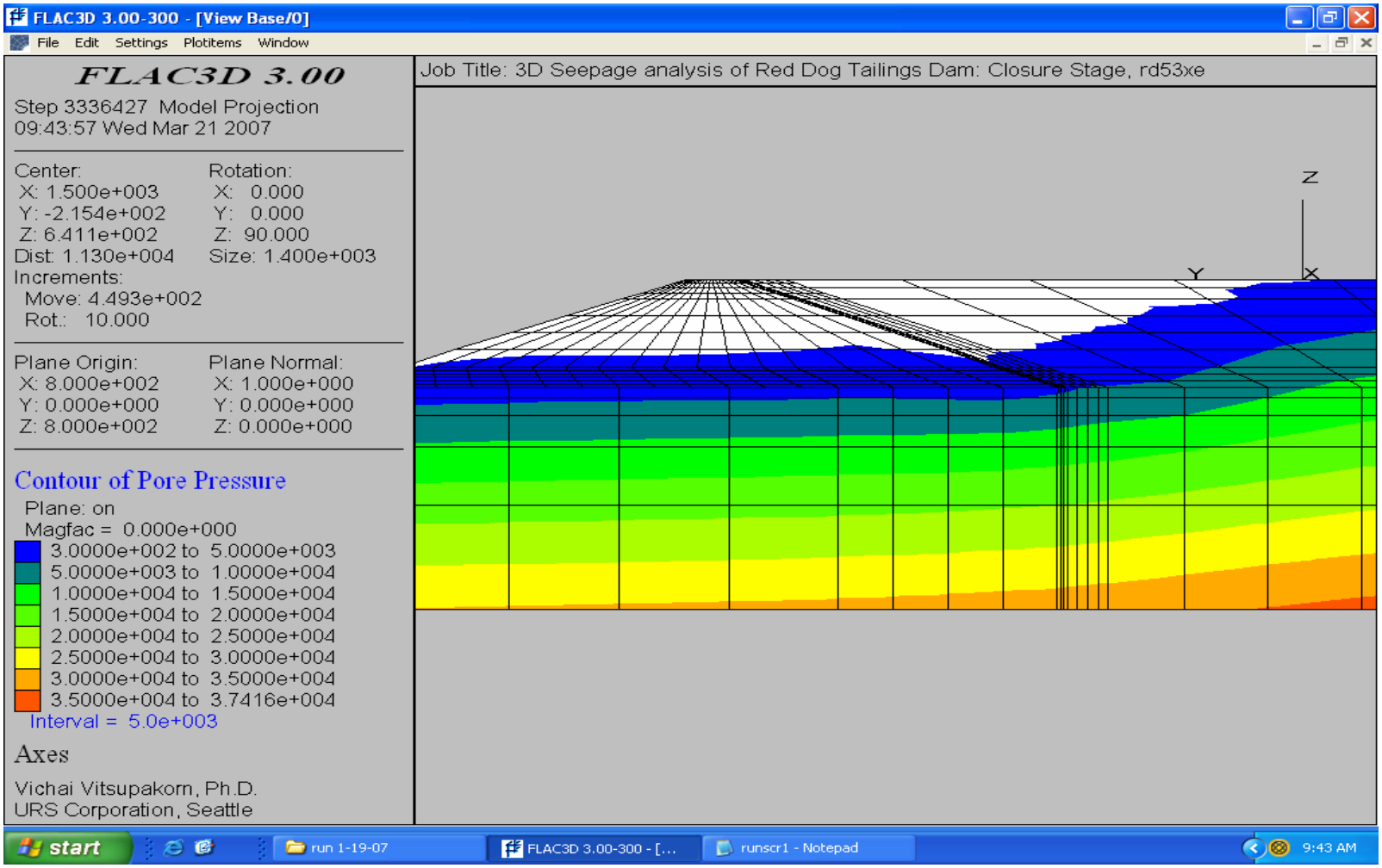


Figure 7-3

URS Job No. 33757098

Pore Pressure Distribution at Station 8+00: Closure, 900-foot Beach, No Liner

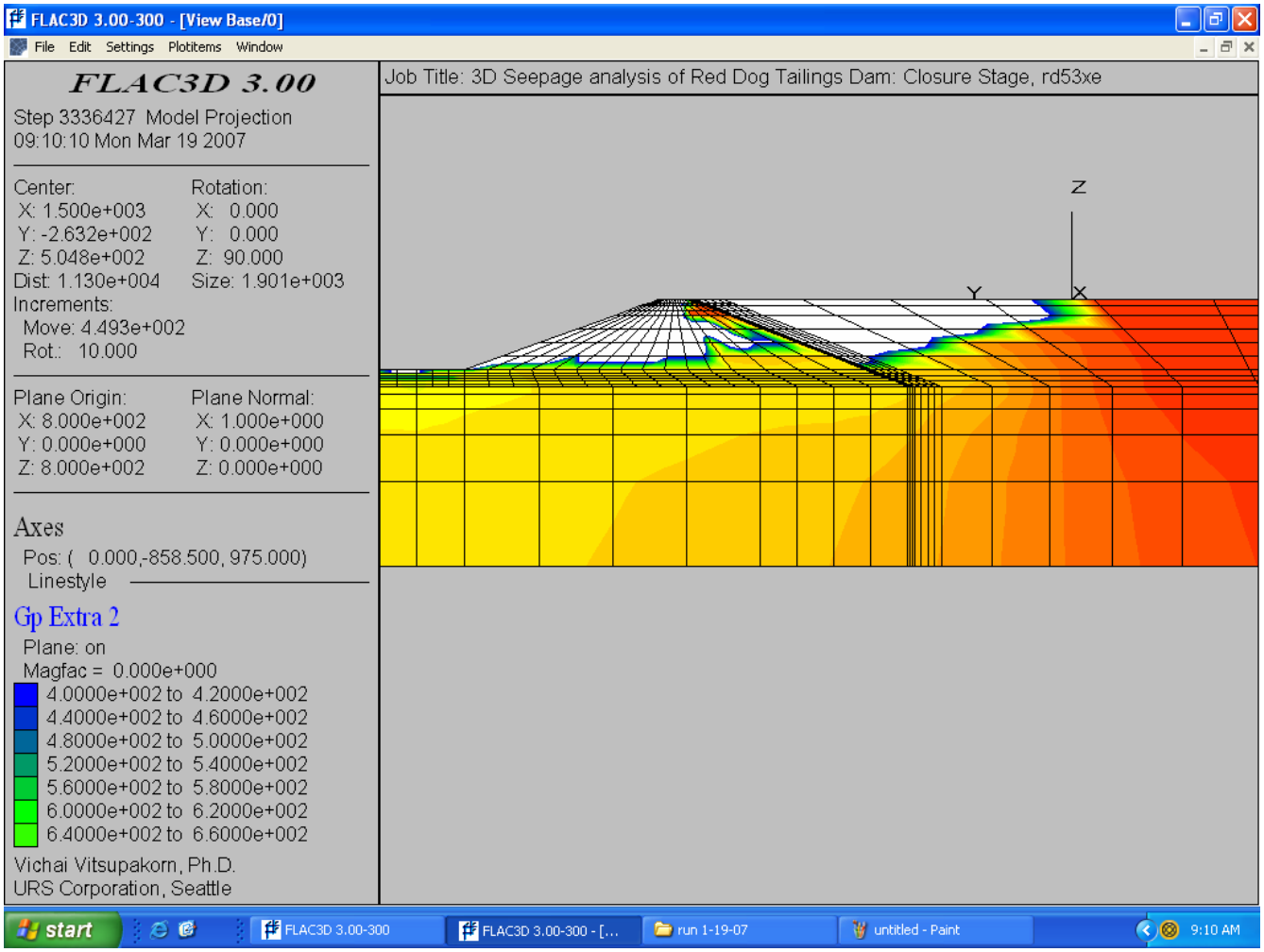
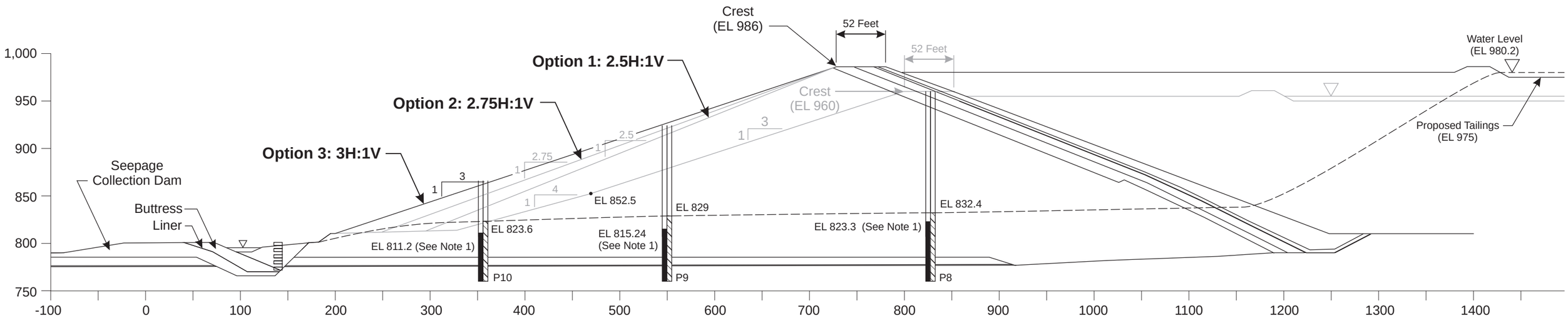


Figure 7-4

Equipotential Lines at Station 8+00: Closure, 900-foot Beach, No Liner

33757098_53.cdr



0 100 200

Horizontal and Vertical Scale in Feet

LEGEND

- Maximum recorded water level (Stage VII-B)
- Water levels from seepage analysis (closure, 900 beach, no liner)

NOTE

1. Maximum recorded phreatic levels shown correspond to operational conditions and dam configuration present till 2005 and do not reflect the potential phreatic surfaces for closure configuration and operations.

Figure 7-5

Downstream Phreatic Surface for Closure
900 Feet Beach, No Liner

Job No. 33757098

URS

Seepage Analysis of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

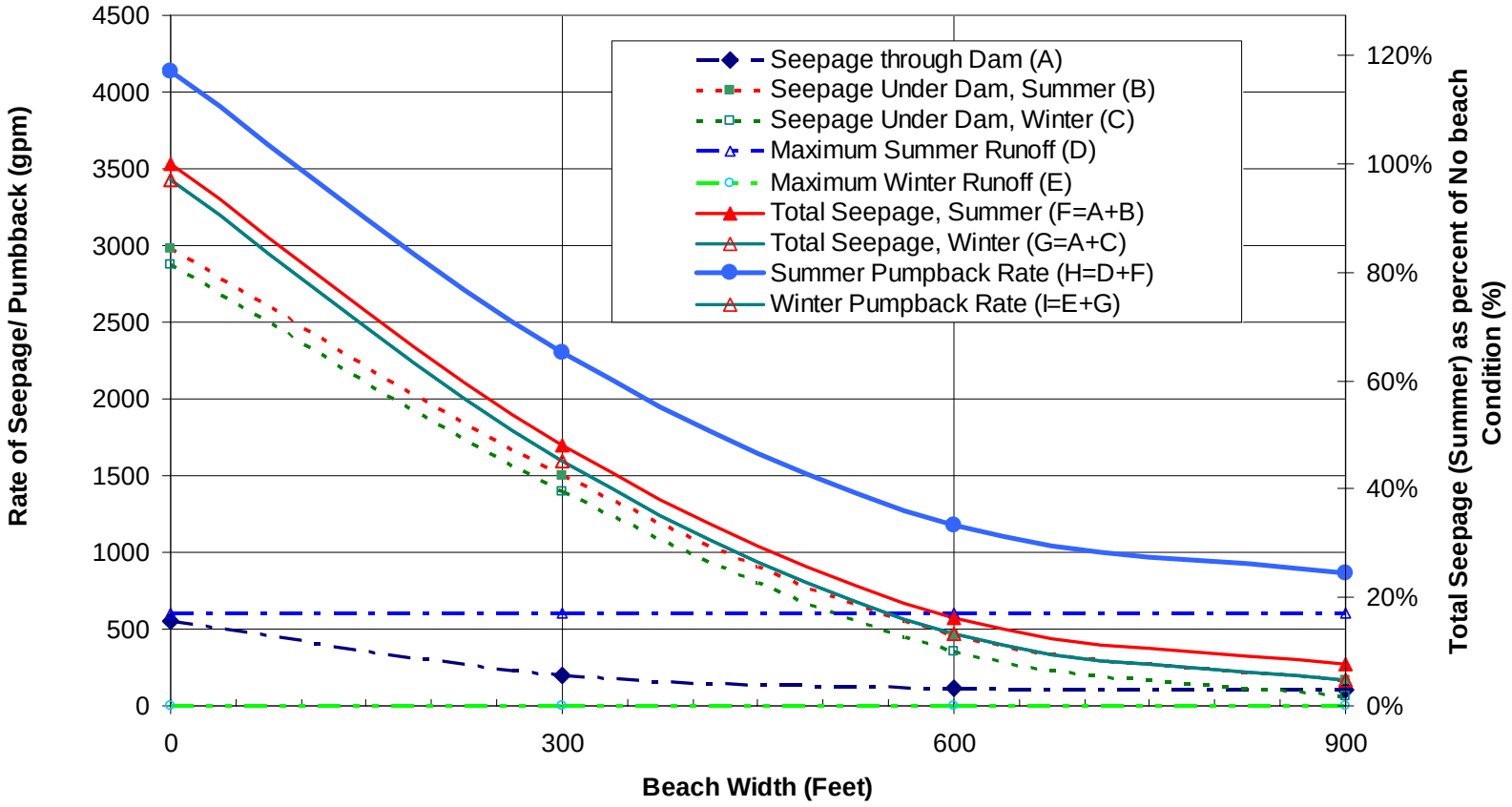


Figure 8-1

Summary of Seepage Results for Closure Conditions