Red Dog Mine
Closure and Reclamation Plan

SD C4: Preliminary Conceptual Design Report, Red Dog Tailings Main Dam, Future Raises to Closure (URS, 2007)

PRELIMINARY
CONCEPTUAL DESIGN REPORT
RED DOG TAILINGS MAIN DAM
FUTURE RAISES TO CLOSURE
RED DOG MINE, ALASKA

For

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

November 26, 2007

Mr. Gary Coulter
Teck Cominco Alaska, Inc.
3105 Lakeshore Drive, Blvd. A, Suite 101
Anchorage, Alaska 99517

Preliminary Conceptual Design 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 this preliminary conceptual design report to Teck Cominco Alaska, Inc. (TCAK) for the Red Dog tailings main dam future raises to closure. The conceptual design was completed under TCAK Purchase Order No. 1257477 of Contract No. RD-02-06 dated July 25, 2004, and Change Order Nos. 002 to 006.

This conceptual design report consolidates the results of work conducted by URS since mid-2004 in support of the tailings impoundment part of an overall mine closure plan that is being developed. The conceptual design assumes that mine operations will continue until the year 2031, and that the tailings main dam will be raised 26 feet above its current height to support the life of the mine.

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



Charles Masala, P. Eng.
Senior Civil Engineer

Cheryl Learned.
Geotechnical Engineer

Cecil M. Urlich, P.E.
Vice President and Project Manager

cc; Mr. George Thornton, TCAK, Red Dog Mine (1)
Mr. Jim Swendseid, TCAK, Red Dog Mine (1)
Mr. Jim Kulas, TCAK, Red Dog Mine (1)
Mr. Daryl Hockley and Ms. Kelly Sexsmith, SRK, Vancouver (1)

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PURPOSE AND SCOPE 1

2.1 PURPOSE 1
2.2 SCOPE 2

3.0 HISTORIC REVIEW AND FUTURE PLANS 4

3.1 DAM DESCRIPTION 4
3.2 HAZARD CLASSIFICATION 5
3.3 TAILINGS IMPOUNDMENT 6
3.4 FUTURE DAM RAISES TO CLOSURE 7
3.5 TAILINGS MANAGEMENT FOR CLOSURE 8

4.0 CONCEPTUAL CLOSURE DESIGN 8

4.1 TAILINGS FACILITY 8
4.1.1 Maximum Impoundment Level 8
4.1.2 Tailings Impoundment Area 9
4.2 TAILINGS MAIN DAM 9
4.2.1 Future Raise Stages and Configurations 9
4.2.2 Future Embankment Configuration 10
4.2.3 Embankment to Wing Wall Connection Bend 10
4.2.4 Future Wing Wall Alignment 11
4.2.5 Curtain Wall 13
4.2.6 Left Abutment Liner Extension 14
4.2.7 Seepage Analysis 14
4.2.8 Stability Analysis 16
4.2.9 Seepage Collection Dam 16
4.2.10 Emergency Spillway 17

5.0 LONG-TERM INTEGRITY OF TAI LINGS MAIN DAM 18

5.1 DAM EMBANKMENT 18
5.1.1 Rockfill Durability 18
5.1.2 Thaw Consolidation 21
5.1.3 Seismic Deformation 22
5.2 LINER SYSTEM 22
5.2.1 Life Expectancy of HDPE Geomembrane 23
5.2.2 HDPE Geomembrane Arctic Performance 25
5.2.3 Life Expectancy of Geotextiles 26
5.2.4 HDPE Geomembrane Response to Embankment Movement 26
5.3 DRAINAGE SYSTEM 29
5.3.1 Tailings Piping Potential and Internal Erosion 29
5.3.2 Chemical Precipitation Potential 32
5.3.3 Riser Pipe Integrity 36
5.4 EMERGENCY SPILLWAY 36
5.5 LONG-TERM TAILINGS MAIN DAM PERFORMANCE 37

6.0 CONCLUSIONS AND RECOMMENDATIONS 37

7.0 REFERENCES 39

TABLES

Table 3-1 Tailings Main Dam Existing Configuration
Table 3-2 Tailings Main Dam Future Configuration
Table 4-1 Tailings Impoundment Water Levels
Table 4-2 Dam Crest and Curtain Wall Lengths
Table 4-3 Seepage Analysis Results
Table 4-4 Stability Analysis Results
Table 4-5 Riprap Conceptual Design
Table 5-1 Lithology of DD2 Quarry Rock
Table 5-2 Unexposed HDPE Geomembrane Lifetime Prediction
Table 5-3 HDPE Geomembrane Lifetime Prediction Extrapolation
Table 5-4 Soil Types Critical Hydraulic Gradients
Table 5-5 Underdrain Plugging Time due to Chemical Precipitation
Table 5-6 Typical Reclaim Water Chemical Properties
Table 5-7 Major Tailings Main Dam Elements Life Expectancy

FIGURES

Figure 3-1 Tailings Impoundment Plan View at Closure
Figure 3-2 Tailings Impoundment Plan View at Closure with Photo Superimposed
Figure 5-1 Dam Crest Settlement due to Thaw Consolidation
Figure 5-2 Geomembrane Conceptual Stages of Chemical Aging
Figure 5-3 Geomembrane Abrupt Deformation
Figure 5-4 Soil Mass and Geomembrane Deformation due to Earthquakes
Figure 5-5 Grain Size Distribution

DRAWINGS

TDC-1 Title Sheet, Index, Map and Notes
TDC-2 References, Legend and Abbreviations
TDC-3 Existing Stage VII-B Dam Plan and Profile
TDC-4 Profile Along Highest Main Dam Section
TDC-5 Profile Across Embankment Crest
TDC-6 Profiles Along Curtain Wall
TDC-7 Closure Stage X Dam Plan and Profile
TDC-8 Profile Across Wing Wall Options
TDC-9 Profile Along Wing Wall Options 4 & 5
TDC-10 Profile Along Wing Wall Option 6
TDC-11 Bend Alternative and Wing Wall Option Plan

TDC-12 Stage VIII Raise Conceptual Plan and Profile
TDC-13 Stage VIII Raise Conceptual Sections
TDC-14 Stage VIII Raise Embankment to Wing Wall Bend
TDC-15 Stage IX Raise Conceptual Plan and Profile
TDC-16 Stage IX Raise Conceptual Sections
TDC-17 Stage IX Raise Embankment to Wing Wall Bend
TDC-18 Stage X Raise Conceptual Plan and Profile
TDC-19 Stage X Raise Conceptual Sections
TDC-20 Stage X Raise Embankment to Wing Wall Bend
TDC-21 Spillway Conceptual Plan and Profile
TDC-22 Spillway Conceptual Crest and Stilling Basin Details
TDC-23 Spillway Conceptual Sections and Details
TDC-24 Spillway Reach 1 Conceptual Profile
TDC-25 Spillway Reach 2 & 3 Conceptual Profiles

APPENDICES

Appendix A: Red Dog Mine Tailings Audit Report Review
Appendix B: Conceptual Design of Spillway at Closure
Appendix C: Geomembrane Durability and Longevity
Appendix D: Underdrain Rockfill Durability and Longevity
Appendix E: Riser Pipe Video Camera Survey Report
Appendix F: Preliminary Technical Specifications
Appendix G: Preliminary Construction Quality Assurance Plan

Executive Summary

This report describes the conceptual design of future raises to closure of the tailings main dam at Red Dog Mine that was completed by URS Corporation for Teck Cominco Alaska Inc. (TCAK). The conceptual design is in support of a mine closure plan that is being developed by TCAK and SRK Consulting Inc. on the basis of mine operations to the year 2031 (SRK, 2007).

The dam is a 182-foot high rock fill embankment that was raised in stages by downstream construction to the current Stage VII-B configuration of crest at elevation 960 feet (El. 960). Maximum permitted tailings and water levels at El. 955 allow for a freeboard of at least 5 feet. For the conceptual design of the future raises, the maximum tailings and water levels are projected to be at El. 975 and El. 980.2, respectively.

The dam contains internal seepage and drainage control systems. The primary seepage control is a liner system on the upstream part of the dam from competent rock to the dam crest. Its key element is a high density polyethylene (HDPE) geomembrane. The drainage control system consists of a toe drain along the upstream toe of the dam under the liner and a rock underdrain along the former creek channel alignment.

The conceptual design of the future raises to closure assumes the conservative scenario of a water cover over the entire tailings impoundment. The water cover will require permanent maintenance of a minimum of two feet of water over the tailings, and active on site care, including a water treatment plant. At El. 986, the tailings impoundment is estimated to cover an area of approximately 725 acres.

The future raises to closure require raising the embankment by 26 feet from the Stage VII-B crest of El. 960 to a final crest at El. 986 and ultimate maximum dam height of 208 feet, extending the dam out as a wing wall either south or southeast of the east abutment where the ground flattens out, and constructing a permanent spillway around the west abutment hillside with a spillway crest at El. 984.5.

The final dam crest at El. 986 and spillway crest at El. 984.5 may change as the closure plan is being developed for the tailings impoundment, as tailings production and storage area projections are updated, and as the disposition of various impacted waters are finalized. However, any crest level changes will have minimal impact to the conceptual designs described in this report.

The conceptual design assumes that the dam will have three raises (Stages VIII, IX and X) of heights 10, 10 and 6 feet, respectively. The raises will increase the crests to El. 970, 980 and 986 feet, and maximum height to 192, 202 and 208 feet, respectively. The raises will be built by downstream construction. The Stage X downstream toe is projected to terminate near the upstream end of the seepage collection pond.

The final dam will include the embankment and a wing wall out from the right abutment. The wall will be built in the same way as the embankment so that they will comprise one dam with continuous seepage and drainage control systems, and a bend at the embankment to wall connection. There are several possible bend and wing wall alignments and configurations that will be resolved in the final designs.

The main embankment raises and most of the wing wall will have upstream slopes of 2.5 horizontal to 1.0 vertical (2.5:1) and downstream slopes of 3:1. The upstream slopes will be the same as the upstream slopes of the previous raises. The downstream slopes will the same as the Stage VII-B downstream slope. The embankment to wall bend slopes may need to be steeper because of space constraints.

The emergency spillway includes a channel around the left abutment. The tailings impoundment at closure will contain all inflows up to the spillway crest at El. 984.5 in order to maintain a zero-discharge operation. In the improbable event that the impounded water exceeds El. 984.5, the spillway is sized to route the probable maximum flood (PMF) through its channel to a maximum water level of El. 985.6.

The long-term integrity and longevity of the dam depends on the durability of the embankment rockfill, liner and drainage systems, and spillway. Some components of these systems were found to have an indefinite or undefined life expectancy. The life expectancies of the major elements are as follows:

Dam Element	**Life Expectancy (years)**	
	Minimum	**Maximum**
Rockfill	Indefinite	
HDPE Geomembrane	500	1600
Geotextile	220	465
Dam Drainage System	Indefinite	
Underdrain	16,500	160,000
Dam Riser Pipe	Indefinite	

The rockfill durability and ability to handle movement caused by thaw consolidation and seismic-induced foundation and seismic deformation is critical to long-term dam stability. The rockfill is inert rock from the DD-2 quarry and the Main Pit, is durable and not reactive, and will remain intact for centuries. Thaw consolidation of the foundation has mostly occurred, and any further thaw will have minimal impact.

A seismic deformation was estimated corresponding to an earthquake of maximum magnitude that has a likelihood of occurring once in 2,475-years. The deformation analysis estimated a maximum horizontal displacement of 21 inches at the crest and maximum shear strain in the dam of up to 0.2 percent. These levels of earthquake-induced displacement and shear strain are tolerable under such an extreme event.

The geomembrane is the key element of the liner system for seepage control. To prevent brittle cracking during cold weather, it is estimated that geomembrane sheets (23 feet by 300 feet) need to have 3.6 and 46.2 inches of slack on the short and long sides, respectively. It was found that the geomembrane could safely accommodate gradual deformations and abrupt deformation induced by the design earthquake.

A drainage system analysis showed that piping will not occur. A liner system analysis showed that the filter soil does not satisfy the “no erosion” criteria, so that if the liner lost its effectiveness, the tailings will erode and not seal the filter soil. However, the low hydraulic gradient in the dam will reduce the potential for internal erosion, and any erosion that could occur will only be in the liner system.

It was estimated that it would take 16,500 to 160,000 years for the underdrain to become plugged as a result of secondary mineral precipitation from tailings water draining through it. Additional analyses of the long-term geochemical changes are required to increase the confidence of the predicted times.

The same soil, rock and geosynthetic materials were used in the dam during all stages of construction, and were assumed for the future raises. However, the final design of each raise should investigate if other materials are more appropriate. This should especially apply to geosynthetics where technology has advanced significantly and new products may better serve the long term needs of the future raises.

The conceptual design shows that the dam can be constructed and operated safely during the remaining mining operations to at least 2031, that long-term dam stability and seepage controls can be maintained in perpetuity after closure, and that post closure operational and maintenance requirements can be achieved.

1.0 INTRODUCTION

The tailings main dam at Red Dog Mine is a 182-foot high rock fill embankment that was raised in stages to the current Stage VII-B level. The mine operator, Teck Cominco Alaska, Inc. (TCAK), and SRK Consulting Inc. are developing a mine closure plan that is based on operations to the year 2031. This will require raising the dam 26 feet from the current crest at elevation 960 feet (El. 960) to a crest at El. 986.

In order to provide technical input to the closure plan for the tailings facility, TCAK retained URS Corporation to complete a conceptual design of the tailings main dam at closure. The conceptual design of the dam at closure assumes three raises (Stages VIII, IX and X) of heights 10, 10 and 6 feet to crests at El. 970, 980 and 986, respectively. At closure the dam will be at the maximum dam height of 208 feet.

In order to develop the conceptual design, URS completed a geotechnical investigation for the Stage VII-B and closure configurations, a historical review of the dam, a seepage analysis for Stage VII-B and the future raises, and a stability analysis for the future raises. These investigations and analyses are summarized in separate reports (URS 2006b, 2007a, 2007b and 2007c).

The conceptual design work also fulfils a State of Alaska Department of Natural Resources (ADNR) request that was provided in a letter to TCAK titled "Multiple Accounts Analysis for Red Dog Tailings Disposal", dated June 2, 2005 (ANDR 2005b). ADNR requested the following:

> *"a conceptual design of the Red Dog Tailings Dam and the Red Dog Back Dam to the ultimate and final configuration, with a detailed engineering evaluation of the expected performance of the dams and tailings storage facility in the next raise and in subsequent raises up to the ultimate height, and in the final, closed configuration."*

This URS report presents the conceptual design of the future raises to closure, along with conceptual design level drawings, and refers to the geotechnical, history, seepage and stability reports. The conceptual design, investigation and analysis work was all performed under TCAK Purchase Order No. 1257477 of Contract No. RD-02-06, dated July 25, 2004, and Change Orders Nos. 002 to 006.

2.0 PURPOSE AND SCOPE

2.1 PURPOSE

The purpose of the conceptual design of the tailings main dam future raises to closure is to demonstrate that the tailings main dam can be constructed and operated safely during mining operations to the year 2031, that long-term dam stability and seepage controls will be maintained after closure, and that post closure design operations and maintenance requirements can be achieved.

As part of a corporate risk assessment of the mine property, TCAK retained BGC Engineering Inc. in 2000 to audit the tailings facility to identify potential risks during current and closure dam configurations and operations. BGC (2001) identified potential risks, described the practices of the time, suggested improvements, and raised the following concerns regarding closure of the tailings impoundment and dam:

- Potential impact of acid rock generation (ARD) from the tailings and waste rock
- Integration of closure plans for the tailings impoundment with the rest of the mine property
- New seismic design criteria for closure if the area is found to be more seismically active
- Feasibility of crowning the tailings and applying other methods of tailings deposition
- HDPE geomembrane long-term performance and compatibility with acidic water.

URS reviewed the audit report and provided comments in a technical memorandum (URS 2004) that is in Appendix A. Meanwhile, the BGC concerns are resolved as part of the mine closure plan being developed by SRK (2006) and the conceptual design of the future raises to closure. The closure plan and conceptual design specifically addresses the following concerns:

- ARD impacts from the tailings and mine waste rock
- Integrated closure of the mine property including the tailings impoundment
- Sub-aerial and paste tailings disposal and wet and dry soil cover options
- Appropriate and updated seismic criteria for closure
- Stable tailings dam configuration with a flooded impoundment
- HDPE geomembrane long-term performance
- HDPE compatibility with dissolved ions, metals and tailings water.

Considering the purpose of the conceptual design of the tailings main dam future raises to closure, and the risks and concerns that were identified, the following conceptual closure design objectives were pursued:

- Demonstrate that the currently projected volume of tailings to closure can be safely stored in the existing tailings impoundment location
- Delineate the aerial coverage of the tailings impoundment at closure and access requirements for any new perimeter dams and relocation of existing facilities that may be required
- Establish the maximum crest elevations of the tailings main dam and construction methods, alignments, slopes, footprints and fill volumes of the future raises
- Analyze the stability of the tailings main dam under static and seismic conditions and determine the optimum downstream slope necessary to achieve long-term post-closure stability
- Estimate the seepage out of the tailings impoundment through and under the tailings main dam at closure by modeling the current seepage using the measured seepage pumpback data
- Assess the ability of the tailings impoundment at closure to safely route the inflow design flood (IDF) which is the probable maximum flood (PMF)
- Demonstrate long-term post-closure performance of the tailings main dam components such as the rockfill embankment, liner system, drainage system and emergency spillway.

2.2 SCOPE

The objective of the initial scope of work was to develop a conceptual design without detailed analyses. The scope of work was developed during discussions between Ms. Kathleen Willman of TCAK, and Messrs. Cecil Urlich and Charles Masala of URS, and included two phases:

- **Phase 1 - Concept Development**. The conceptual development work included reviewing relevant information, delineating the impoundment area, identifying any new perimeter dams and facility relocations that would be needed, developing a preliminary geometry of the tailings main dam and balancing the objective of an emergency spillway with a goal of zero discharge.
- **Phase 2 - Conceptual Design**. The conceptual design work of the tailings main dam future raises to closure included the completion of two-dimensional (2D) seepage analyses, conceptual-level stability analyses, closure planning reviews and coordination with TCAK and SRK, technical memoranda of conceptual design findings, and conceptual design drawings.

As a result of the Multiple Accounts Analysis (2005b), the URS scope was expanded by means of consultations between Messrs. Gary Coulter, Jim Kulas and George Thornton of TCAK, Daryl Hockley and Cameron Scott of SRK, and Cecil Urlich and Sri Rajah of URS. The expanded work included a geotechnical investigation under a separate Stage VII design contract, and new phases of closure work:

- **Phase 3 – Ongoing Analyses and Reporting**. The ongoing analyses and reporting included a 3D seepage modeling and analyses using historic tailings disposal and pumpback data, stability analyses using new geotechnical findings and seismic criteria, conceptual design updates for the main dam and spillway, and draft specifications and construction quality assurance (CQA) plan.
- **Phase 4 – Future Anticipated New Activities**. Future anticipated new activities included a Closure Plan Feedback and Evaluation Workshop held in Anchorage on April 3 to 7, 2006, Failure Modes and Effects Analysis (FMEA) workshop to be held, back dam design peer review, and tailings impoundment closure plan review.

As a result of comments and concerns expressed by State regulators, NANA, and community representatives during the Closure Plan Feedback and Evaluation Workshop, the URS scope was expanded by means of consultations between Messrs. Coulter, Kulas, Thornton, Hockley, Scott and Urlich, to include more detailed evaluations of specific long-term embankment integrity items as follows:

- **Phase 5 – Tasks Added After Workshop.** Tailings main dam long-term integrity items included evaluations of the impacts of a design earthquake on embankment deformation and liner integrity, durability and longevity of the geomembrane part of the liner system, and the potential of plugging of the drainage system as a result of tailings piping and chemical precipitation.

During the Stage VII-B construction in 2006 and 2007, significant experience was obtained as a result of challenges and impediments that were encountered during the excavation and dewatering of the cutoff wall and curtain wall trenches, and in efforts to maintain the stability of the side slopes and work benches. This experience was used to improve the conceptual design of the wing wall as follows:

- **Phase 6 – Use of Stage VII-B Experience.** Considerations were given to alternate types of dam construction and configuration at the embankment to wing wall transition (bend) because of the nearby zinc thickener and freshwater tank, and to several wing wall alignments because of deep zones of fractured rock and a preference to minimize operations impacts to the mine haul road.

This conceptual design of the tailings main dam future raises to closure report has 25 conceptual design drawings appended. Drawing TDC-1 contains a drawing index, vicinity map and general notes. Drawing TDC-2 lists reference drawings and documents, and contains a legend and abbreviations. Drawings TDC-3 to TDC-25 show the conceptual design and are referenced in the text.

3.0 HISTORIC REVIEW AND FUTURE PLANS

3.1 DAM DESCRIPTION

The tailings main dam is a 182-foot-high rock fill embankment on an approximately west-east alignment across the South Fork of Red Dog Creek. It was built and raised in seven stages (Stages I to VII), with Stage VII built in two sequences (Stages VII-A and VII-B). The dam was raised by downstream construction to the current Stage VII-B crest at an elevation of 960 feet (El. 960) as shown in Table 3-1.

Table 3-1 – Tailings Main Dam Existing Configuration

Dam Stage	Year of Construction	Raise Height (ft)	Dam Crest El (ft)	Toe El [1] (ft)	Max. Dam Height [2] (ft)	Crest Width (ft)	Crest Length (ft)
Stage I (Starter Dam)	1988	-	865	790	75	42	1088
Stage II	1989	25	890	788	102	42	1352
Stage III	1990	20	910	786	124	42	1776
Stage IV	1991	15	925	784	141	42	1917
Stage V	1993	15	940	782	158	42	2175
Stage VI	1993	10	950	778	172	52	2427
Stage VII-A	2003/2004	5	955	778	177	52	2550
Stage VII-B	2005 to 2007	5	960	778	182	52	3457

Notes: [1] Lowest elevation of the natural creek bed on the upstream or downstream toe of the dam.
[2] Dam height is the maximum vertical distance from the natural bed of the creek at the downstream toe.
[3]Stage VII-B crest length includes the main dam crest (2640 ft) plus the wing wall crest length (817 ft).

The dam contains internal seepage and drainage controls. The primary seepage control is a liner system on the upstream part of the dam from competent rock to the dam crest. The key element of the liner is a high density polyethylene (HDPE) geomembrane that is cemented into a cutoff wall under the upstream toe of the dam. The drainage control system consists of a toe drain along the upstream toe of the dam under the liner and a rock underdrain along the former creek channel.

Stages I to VII-A of the dam consist of an embankment across the South Fork of Red Dog Creek. Stage VII-B consists of a raise to the embankment and an extension southeast of the right abutment. As a result, the crest length increased significantly from Stage VII-A (2550 feet) to Stage VII-B (3457 feet). The extension is referred to as a wing wall that will be built along with the future raises to closure. The Stage VII-B part of the wing wall is the seepage cutoff component that is referred to as a curtain wall.

The tailings main dam embankment and wing wall consist of the following main components:

- A rockfill embankment across the South Fork of Red Dog Creek
- A rock under-drain from the Stage II downstream toe to the seepage collection pond
- A liner system containing a high-density polyethylene (HDPE) geomembrane
- A HDPE-lined cutoff trench below the upstream toe of the dam
- A HDPE-lined cutoff wall filled with grout and extending below the cutoff trench

- A rockfill buttress and stability berm on the upstream face to protect the HDPE liner
- A pipeline utilidor for tailings discharge, seepage return and mine water discharge
- A pipe bench on the upstream dam face that supports the tailings discharge pipeline
- A curtain wall extending southeast from the right abutment of the embankment
- Proprietary interlocking "GSE CurtainWall" panels along the curtain wall
- Control density fill embedment along the bottom of the curtain wall panels
- Piezometer and thermistor instrumentation in and around the dam and wing wall.

The upstream slope of the embankment is 2.5 horizontal to 1.0 vertical (2.5:1). The downstream slopes were 2:1 for Stage I, 4:1 for Stages II to VI and the lower parts of Stages VII-A and VII-B, and 3:1 for the upper parts of Stages VII-A and VII-B. The Stage VII-A and B downstream slopes are steeper than the previous downstream slopes. The crest width of all raises was 52 feet, which is the present crest width.

Descriptions of the tailings main dam site, design and construction history, material types, primary seepage control system, secondary seepage control system, and instrumentation are provided in the Stage VII-A construction report (URS 2004a), Stage VII-B construction and as-built record drawings (URS 2005b and 2007e, respectively), geotechnical report (URS 2006) and dam history report (URS 2007a).

Concurrent with the conceptual design development, URS completed seepage and stability analyses of the tailings main dam Stage VII-B configuration and the future raises to closure that are described in separate reports (URS 2007b and 2007c, respectively).

3.2 HAZARD CLASSIFICATION

The dam is under the jurisdiction of the Alaska Dam Safety Program (ADSP), is in the National Inventory of Dams as ID# AK 00201, and is classified as a Class II (significant) hazard dam in accordance with Title 11 of Alaska Administrative Code 93.157. There are three classifications for dams based on potential impacts of failure and improper operation of the dam:

- Class I (high)
- Class II (significant)
- Class III (low)

These classifications set the standard for design, construction and operation of a dam, and requirements for inspections and emergency planning. The Class II hazard potential is assigned to a dam if is determined that failure or improper operation of the dam will result in the following:

(a) a significant danger to public health;

(b) the probable loss of or probable significant damage to homes, occupied structures, commercial property, high-value property, major highways, primary roads, railroads, or public utilities, other than property losses or damage limited to the owner of the barrier;

(c) other probable significant losses or damage, other than property losses or damage limited to the owner of the barrier;

(d) probable loss of or significant damage to waters identified under 11 AAC 195.010(a) as important for the spawning, rearing, or migration of anadromous fish.

The dam is classified as Class II because Ikalukrok Creek and the lower part of Red Dog Creek, which are downstream of the dam and below a fish weir, are listed by the Alaska Department of Fish and Game as anadromous streams. It was considered that these creeks would be subject to probable loss of, or significant damage to, waters identified under 11 AAC 195.010 (a) as important for the spawning, rearing, or migration of anadromous fish.

3.3 TAILINGS IMPOUNDMENT

Section 13.2.2 of ADNR (2005a) describes the following perspective on tailings dams and impoundments:

> "*Dams at tailings storage facilities are unique because the service life of the dam is infinite, generally speaking. When the reservoir is full of tailings and the facility is closed, the dam must remain in place and continue to retain the substance for an indefinite period of time while withstanding the effects of surface runoff and groundwater as the system is transformed from an active, operational condition to an inactive, closed condition*".

In order to prudently address this perspective, three closure approaches are commonly considered in order to meet tailings impoundment post-closure design requirements:

- **Walk away** – A walk away solution is one in which no additional monitoring or maintenance activities are required after tailings facility is closed.
- **Passive care** – Passive care is achieved when there is a minimal on-going need for occasional monitoring and infrequent minor maintenance.
- **Active care** – Active care requires an annual on-going need for operation, maintenance and monitoring of the facility, including a water treatment system.

For the Red Dog tailings impoundment, the following four options were considered for closure and reclamation and they all fall under active care (SRK 2006):

- **Water Cover** – A water cover would maintain a year-round pool over the tailings that would restrict oxidation of the sulfide minerals in the tailings, and thereby minimize acid generation and contaminant release. The pond would also minimize the potential for fugitive dust to be released, and provide a storage location for contaminated water from the rest of the site.
- **Clean Pond Water Cover** – A clean pond water cover would also maintain a permanent pool over the tailings to restrict oxidation and acid generation. In this case, however, contaminated water would be diverted to an alternative storage location, most likely the Aqqaluk Pit.
- **Dry Soil Cover** – A dry soil cover would maintain a layer of soil or rock over the tailings, and divert or channel inflows to ensure that the water table does not rise into the cover material.
- **Wet Soil Cover** – A wet soil cover would create a layer of water-saturated soil or rock over the tailings that would be a barrier to oxygen and would prevent oxidation and acid generation in the tailings.

These four options assume that future tailings will be discharged and stored in the impoundment in the current manner, and therefore will require raises of the tailings main dam. The assumed closure design option in this report is "Water Cover" which was selected by TCAK on the basis of feedback from local communities, NANA, State Representatives, and Non-Governmental Agencies.

The water cover option is considered to be conservative because it will result in the most water storage in the tailings impoundment, the highest tailings main dam and back dam structures, the greatest potential for seepage out of the impoundment, and the highest potential for dam instability. The option is an active care closure approach because it will require the continued presence of a water treatment plant.

For the conceptual design of the future raises to closure, the maximum tailings and water levels are projected to be at El. 975 and El. 980.2, respectively. These are described in Section 4.1.1 and in Appendix B. Figures 3-1 and 3-2 show a conceptual level plan view for the closure configuration of the tailings impoundment

3.4 FUTURE DAM RAISES TO CLOSURE

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

- 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 built 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, and
- Stage X: Six feet high from crest El. 980 to 986.

The dimensions these future raises to closure, as assumed for the purposes of this conceptual design report, are shown on Table 3-2. The crest length includes the main embankment and wing wall.

Table 3-2 – Tailings Main Dam Future Configuration

Dam Stage	Raise Height (ft)	Dam Crest El (ft)	Toe El [1] (ft)	Max. Dam Height [2] (ft)	Crest Width (ft)	Crest Length Range (ft) [3,4]
Stage VIII	10	970	778	192	52	3535 to 5085
Stage IX	10	980	778	202	52	3785 to 5450
Stage X	6	986	778	208	52	3890 to 6015

Notes:
[1] Lowest elevation of the natural creek bed on the upstream or downstream toe of the dam.
[2] Dam height is the maximum vertical distance from the natural bed of the creek at the downstream toe.
[3] Stage VIII to X crest length includes the main embankment crest length plus the wing wall crest length.
[4] The range of crest lengths depends on the wing wall alignment option that is selected.

It should be noted that these raise heights were selected for the purpose of the conceptual design and do not reflect a final mine plan for the raises. However, a variation of the number of stages and their heights will not impact the stability and performance of the dam as presented in this conceptual design report.

3.5 TAILINGS MANAGEMENT FOR CLOSURE

Recognizing the benefits of a wider tailings beach to reduce the seepage rate and to 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 the 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 considerations to select optimal beach widths and coffer dam alignments for closure.

It may be necessary to have a wider tailings beach alongside and upstream of the embankment to wing wall connection bend if the upstream and downstream slopes along of the bend are designed to be steeper than the slopes of the main embankment and wing wall.

4.0 CONCEPTUAL CLOSURE DESIGN

4.1 TAILINGS FACILITY

The design and construction history of the tailings main dam are described in the dam history report (URS 2007a), along with the components of the dam. Drawing TDC-3 shows the existing elements of the dam in plan and profile view to Stage VII-B. This section describes the conceptual design of the tailings main dam future raises to closure.

4.1.1 Maximum Impoundment Level

The tailings impoundment is operated as a “zero discharge” facility and it is planned to maintain this condition after closure. In order to maintain “zero discharge”, the maximum tailings surface level will include the components shown in Table 4-1. The estimated values of these components and the methodology of selecting the values are described in Appendix B, and summarized in Table 4-1.

The tailings impoundment will have a permanent water cover of two feet at El. 977. During the autumn to spring inflow, the water level is expected to rise to El. 978.6. With the inclusion of the PMF, wind setup, wave run-up, spillway backwater effects, additional water storage, and spillway depths, the maximum water depth is expected to reach El. 985.6. Therefore, the water depth will range from 2.0 to 10.6 feet.

Table 4-1 – Tailings Impoundment Water Levels

Closure Requirement Component	Depth (feet)	Elevation (feet)
Total tailings storage	Not applicable	975.0
Minimum water cover	2.0	977.0
Autumn to spring inflow	1.6	978.6
Probable maximum flood (PMF)	1.1	979.7
Wind setup	0.3	980.0
Wave run-up	0.9	980.9
Spillway backwater effects	1.1	982.0
Additional water storage	2.5	984.5
Spillway depth	1.1	985.6

The crest of the dam at closure is estimated to be El. 986 as described in Appendix B. However, the final dam and spillway crest heights at El. 986 and 984.5, respectively, may change as the closure plan is being developed for the tailings impoundment, as tailings production and storage area projections are updated, and as the disposition of various impacted waters are finalized. Any crest height changes will have minimal impact to the conceptual designs described in this report.

4.1.2 Tailings Impoundment Area

At the final dam crest height of El. 986, the tailings impoundment area will cover approximately 725 acres as shown in Appendix B, Figure 3-1 and 3-2. The final impoundment will inundate four existing facilities along the east side of the current impoundment. They are the tailings water reclaim plant, the incinerator, the helicopter landing pad, and the fire station.

A tailings back dam is under construction in front of the waste pile along the saddle at the south end of the tailings impoundment. The back dam will prevent the impounded tailings and water from spilling over the saddle to the Bons Creek watershed south of the tailings impoundment. The back dam is not described in this conceptual design report of the tailings main dam future raises to closure.

4.2 TAILINGS MAIN DAM

4.2.1 Future Raise Stages and Configurations

The tailings main dam future raises to closure require raising the embankment by 26 feet from the Stage VII-B crest of El. 960 to a final crest at El. 986 and ultimate maximum dam height of 208 feet, extending the dam out as a wing wall either south or southeast of the east abutment where the ground flattens out to confine the tailings and protect the mill facilities, and constructing a permanent spillway around the west abutment hillside with a spillway crest at El. 984.5.

The final dam will include the main embankment and a wing wall that will extend out from the right abutment. The embankment and wing wall will be of similar construction and will comprise one dam with continuous seepage and drainage control systems and a bend at the embankment to wall connection. There are several possible bend configuration alternatives and wing wall alignment options that must be resolved in the final designs for each future raise, and viewed holistically for the combined future raises.

The conceptual design for raising the tailings main dam to the closure height at El. 986 calls for raising the dam in stages by the same method of downstream construction as was used in the starter dam and Stages II to VII-B. For the conceptual design, it is not necessary to know the actual height of each stage; it was assumed that there will be two 10-foot raises and one 6-foot raises.

The configurations of the tailings main dam for the future raises are provided in Table 3-2. The maximum estimated crest length of the dam will be 2,855 feet of the main embankment plus 1035 to 3160 feet of wing wall depending on which wing wall option is selected. Therefore, the final dam crest would have a length of 3890 to 6015 feet depending on which wing wall option is selected.

As with each of the previous stages, construction of the future raises will require design specifications and a construction quality assurance (CQA) plan. Preliminary design specifications and CQA plan are provided in Appendix F and G, respectively.

4.2.2 Future Embankment Configuration

The main embankment will be raised by 26 feet from the Stage VII-B crest at El. 960 to a final crest at El. 986. The maximum tailings main dam height at closure will be 208 feet, which is calculated as the difference between the maximum crest height (El. 986) and the lowest part of the downstream toe before starter dam construction (El. 778). The crest widths will be 52 feet as for all previous raises.

In order to maintain the required stability of the tailings main dam, the main embankment raises will have upstream slopes of 2.5:1 and downstream slopes of 3:1. The upstream slopes will be the same as at present and the downstream slopes will be the same as the upper part of the present Stage VII-B downstream slope. Details for the future raises are as follows:

- **Stage VIII raise to El. 970** – The plan and profile are shown on Drawing TDC-12. Sections are shown on Drawing TDC-13. Approximately 390,000 cubic yards of rock fill will be required to raise the embankment ten feet from Stage VII to VIII (El. 960 to 970).
- **Stage IX raise to El. 980** – The plan and profile are shown on Drawing TDC-15. Sections are shown on Drawing TDC-16. Approximately 535,000 cubic yards of rock fill will be required to raise the embankment ten feet from Stage VIII to IX (El. 970 to 980).
- **Stage X raise to El. 986** – The plan and profile are shown on Drawing TDC-18. Sections are shown on Drawing TDC-19. Approximately 365,000 cubic yards of rock fill will be required to raise the embankment six feet from Stage IX to X (El. 980 to 986).

A profile of the highest section of the tailings main dam embankment and the subsurface conditions under this section are shown in TDC-4. Subsurface profiles along the tailings main dam crest and the Stage VII-B curtain wall are in TDC-5 and 6. Drawings TDC- 8, 9 and 10 show the subsurface conditions along the wing wall for several potential alignment options that are shown in TDC-7 and 11.

The conceptual design assumes that the Stage VIII, IX and X raises will be built by downstream construction. The downstream toe at the highest section of the dam will move progressively closer to the seepage collection pond for each raise as shown on in TDC-13, 16 and 19. The Stage X downstream toe will extend to near the upstream edge of the seepage collection pond as shown on TDC-18 and TDC-19.

4.2.3 Embankment to Wing Wall Connection Bend

The alignment of the embankment to wing wall transition bend is constrained by existing mill facilities including the zinc thickener and fresh water tank near the right abutment. As a result, conceptual design Alternatives 1, 2 and 3 were developed for constructing the bend to minimize impact to the facilities. Drawings TDC-7, 11, 14, 15, 17, and 20 show plans and cross-sections for the bend alternatives.

Alternatives 1, 2 and 3 are based on downstream, centerline and upstream methods of construction, respectively. Because of space constraints and maximum dam height of 26 feet around the bend, it was decided for comparison purposes to assume downstream slopes of 2.5:1 and upstream slopes of 1.5:1, which are steeper that the main embankment slopes. These slopes will be finalized on the basis of stability analyses that will be required for the Stage VIII design and consideration of the Stage IX and X layouts.

Alternative 1, using downstream construction, is constrained by the presence of the zinc thickener and freshwater tank and their ancillary facilities, and by the alignment of the top of the Stage VII-B liner system at El. 960. These constraints are shown in TDC-14, 17 and 20. The Stage VIII raise is not directly impacted by these constraints, but the Stage VIII design will impact the Stage IX and X designs.

Alternatives 2 and 3 attempt to minimize impacts on the zinc thickener and fresh water tank during the Stage IX and X raises. These alternatives are based on centerline and upstream construction methods, respectively. Cross-sections are shown on TDC-17 and 20. The technical feasibility of Alternatives 2 and 3 can be achieved by a combination of factors including steeper upstream and downstream slopes, a foundation of waste rock on the previous upstream slope, and a horizontal component of the liner system.

URS completed the investigation and design, facilitated permits, and managed construction for an identical project at Kettle River Mine in Republic, Washington. This project involved two downstream raises of a large tailings dam followed by an upstream raise because property ownership downstream of the dam did not allow for future downstream raises.

Steeper upstream and downstream slopes may be possible because the maximum dam height at the bend will be 26 feet compared to 208 feet at the highest dam section, and the head of tailings and water at the bend will be relatively low because of the freeboard requirement and wide tailings beach. The corner of the tailings impoundment at the bend could have a wide beach with minimal impact to the impoundment storage capacity. Replacement of less dense, upper soils may be needed to achieve the steeper slopes.

The use of waste rock as a foundation for centerline and upstream construction is feasible. Waste rock can be placed upstream of the liner system, and some tailings excavation may be required to place the waste rock. Such excavations are feasible as shown by the Stage IV to VII cutoff wall installations at the left and right abutments during Stage VII-B construction. The foundation can be made stable as shown by stability analyses for the pipe rack and dust control structures (Dames & Moore, 1999; URS, 2001).

4.2.4 Future Wing Wall Alignment

Most of the wing wall will have upstream slopes of 2.5:1 and downstream slopes of 3:1. The upstream slopes will be the same as the present upstream slopes. The downstream slopes will be the same as the upper part of the present Stage VII-B downstream slope. However, some upstream and downstream slopes of the wing wall may need to be steeper because of space constraints such as the mine haul road.

During the Stage VII-B design, concepts were developed for the curtain wall part of the future Stage VIII, IX and X wing wall. The wing wall will be an extension of the tailings main dam to the southeast from the right abutment where the dam axis will change from an approximate west-east alignment to an approximate northwest-southeast alignment. The change in the tailings dam alignment was necessitated by the presence of mill site infrastructure directly east of the original dam alignment.

The wing wall will consist of a below-ground curtain wall for seepage cutoff, and a lined embankment that will be a continuation of the main dam embankment described in Section 4.2.2. The liner is a continuation of the curtain wall. The wing wall branches off from the tailings main dam at the bend at the right abutment as described in Section 4.2.3.

The Stage VII-B part of the wing wall is an 817-foot long curtain wall to the Stage VII-B crest at El. 960. The curtain wall installation and the challenges and impediments that were encountered are described in the Stage VII-B construction report (URS 2007d). The curtain wall extends vertically down to competent rock and is tied into the east end of the cutoff wall to provide a continuous seepage control barrier.

The wing wall starts at the eastern-most end of the Stage VII-B cutoff wall as shown in TDC-3. At the southeast end of the Stage VII-B curtain wall, six wing wall alignment options have been considered for the future raises to closure as shown on TDC-11 and summarized as follows:

- **Option 1**: Extend the alignment to the southeast in the same straight line as the Stage VII-B curtain wall and across the mine haul road to the toe of the main waste pile.
- **Option 2**: Turn the alignment north at an anti-clockwise angle from the Option 1 alignment and across the mine haul road to the toe of the main waste pile.
- **Option 3**: Turn the alignment south at a clockwise angle from the Option 1 alignment and across the mine haul road to the toe of the main waste pile.
- **Option 4**: Move the alignment about 40 feet northeast of and parallel to the Option 1 alignment for about 300 feet, turn to the northeast for another 350 feet, and across the mine haul road to the toe of the main waste pile.
- **Option 5**: From the end of the Stage VII-B curtain wall, turn the alignment to the northeast and extend it 600 feet and across the mine haul road to the toe of the main waste pile.
- **Option 6**: From the end of the Stage VII-B curtain wall, turn the alignment south about 2000 feet south between and parallel to the mine haul road and the east side of the tailings impoundment.

Options 1, 2 and 3 are described in a report of a 2005 geotechnical investigation (URS, 2006b). All three options were found to be limited in suitability because of highly weathered shale along the Option 1 and 3 alignments, and alternating slightly to moderately weathered shale along the Option 2 alignment, as shown on TDC-8. Depths of excavation that would be required to install the seepage cutoff along these alignment options would be in excess of 50 feet. The rock along these extended alignments was poorer in quality than the rock that was anticipated and encountered in the Stage VII-B curtain wall installation in 2006 and 2007.

The suitability of the subsurface conditions along the wing wall alignment were evaluated on the basis of the presence of the bedrock and its inclination upwards following the surface elevation until it intersects the wing wall crest at El. 986. In the geotechnical investigation report, URS concluded that Option 2 was preferred, based on the apparent suitability of subsurface conditions and knowledge of the subsurface conditions at the time.

While developing the wing wall conceptual design, URS prepared two possible subsurface profiles along the Option 2 alignment based on the 2005 geotechnical findings. It was assumed that borings SS-18-05 and SS-19-05 were terminated in slightly to moderately weathered shale. However, boring SS-15-05 in the same area showed slightly to moderately weathered shale alternating with highly weathered shale. The interpretation of subsurface conditions in this area was concluded to be uncertain.

In 2007, the Option 2 alignment was re-evaluated in light of the uncertain subsurface conditions and their impact on installing the curtain wall to competent bedrock. A geotechnical investigation was conducted by URS in September 2007 to clarify the subsurface conditions and confirm Option 2 as the preferred alignment. Four borings (SS-01-07 to SS-04-07) were completed on September 24 and 25, 2007 at locations shown on TDC-3.

As a result, Option 2 was found to be not as suitable as first believed, and modifications to this option were developed as Options 4 and 5, shown on TDC-11. The downsides of Options 4 and 5 are the significant and uncertain depths to competent rock, and the fact that curtain wall construction would severely impact the mine haul road operations and ultimately require that the haul road be relocated.

This issue was discussed during a meeting on September 25, 2007 between Gary Coulter of TCAK, Daryl Hockley of SRK, and Cecil Urlich and Cheryl Learned of URS, and with George Thornton and Jim

Swendseid by conference call. Option 6 was suggested to prevent disruption to the haul road and mill operations. The 2005 geotechnical work showed that the Option 6 alignment had shallower competent rock that the other options, but was longer than the other options because of the low topography.

The conceptual design assumes that the wing wall be built along either Option 4, 5 or 6. Options 4 and 5 are shorter with lower embankment costs, but have deeper and uncertain rock conditions will disrupt mine haul road operations. Option 6 will not disrupt mine haul road operations and has shallower depth to competent rock, but has a longer embankment. A hybrid option is a partial Option 6 to the south and turn it east across the mine haul road to the main waste pile during a later phase of mining when impacts to the haul road might be more easily mitigated.

Another factor favoring Option 6 is that as part of the mine closure plan, it is planned to collect seepage from the main waste pile and divert it to the Aqqaluk Pit to prevent it from entering the tailings impoundment (SRK, 2007). The alignment for this diversion system is shown on TDC-7 as a series of cutoff walls and a pipeline. One concept is to construct a cutoff wall. An Option 6 curtain wall could also serve at the main waste pile cutoff wall.

The final decision on the wing wall alignment will be made during the design of the Stage VIII raise, which URS is just starting to develop. The Stage VIII design will need to consider the Stage IX and X requirements. The remainder of the wing wall will be designed along with the Stage VIII, IX and X raises and will be updated in accordance with the engineering practice and technological advances.

4.2.5 Curtain Wall

The curtain wall is a key component of the wing wall and provides seepage cutoff to bedrock. The Stage VII-B design included interlocking, proprietary 100 mil HDPE “GSE CurtainWall” panels embedded in a trench of controlled density fill (CDF) and backfilled to El. 960. The same GSE CurtainWall panels may be used for future raises. However, there are specific concerns regarding constructability that will be addressed during the design of the future raises, starting with Stage VIII.

Installation of the GSE CurtainWall panels requires excavation of material to a stable work bench in order to allow for equipment operation, followed by trenching to competent bedrock. During the Stage VII-B construction of the curtain wall, space limitations and constructability issues were revealed that will be considered during development of designs for future raises. These issues include the following:

- A work bench at least 40 feet wide in firm soils is needed for equipment trafficking and trench excavation.
- The presence of infrastructure may prevent deepening and widening of the excavation for the required work bench width
- Excavator reach limitations may make it difficult to provide the necessary cut slopes for stability and worker safety
- A supply of dewatering pumps and piping is necessary to assure quality and safety during panel installation
- Work may be conducted during frozen and thawed conditions which will effect work bench and trench wall stability and construction quality
- Un-supported GSE CurtainWall panels fold under their self weight, resulting in uneven surfaces that are time-consuming and difficult to weld,

- Welding geomembranes to the GSE CurtainWall panels precluded the use of fusion welding without extensive patching due to the location of the interlocks.

The subsurface profile showing these conditions along the wing wall alignment is presented in Drawings TDC-9 and 10. The estimated curtain wall lengths and ultimate dam crest lengths for each raise are summarized in Table 4-2.

Table 4-2 – Dam Crest and Curtain Wall Lengths

Stage	Dam Crest Length (feet)			Curtain Wall length(feet)		
	Option 4	Option 5	Option 6	Option 4	Option 5	Option 6
VIII	3535	3535	5085	1440	1300	*
IX	3945	3785	5450	1500	1325	*
X	4060	3890	6015	1525	1350	*

* Extension (termination) of the curtain wall will be estimated based on the future geotechnical exploration.

The Stage VII-B curtain wall construction offered an opportunity to evaluate the process and evaluate ways to improve the methodology and design. For future raises, the curtain wall design including GSE CurtainWall panels will be evaluated and if deemed appropriate, re-designed. Other options that could be considered include slurry walls and soil-cement mix walls.

4.2.6 Left Abutment Liner Extension

Competent (moderately to slightly weathered) rock at the left abutment is up to 10 feet deep as shown on Drawing TDC-5. The surface material is silty sand and highly weathered shale which are not suitable for seepage control. In order to control seepage under the dam, a cut off wall composed of a HDPE geomembrane embedded in grout extends to competent underlying bedrock at the upstream toe of the dam. The geomembrane at the top of the cutoff wall continues into liner system in the dam.

During the Stage II to VII-A raises, the west end of the cutoff wall was only extended horizontally to directly below the west end of the dam crest. However, because the silty sand and highly weathered shale are up to 10 feet deep, stopping the cutoff wall to under the west end of the crest leaves the abutment susceptible to seepage around the liner system and along the waste abutment if the water level in the tailings impoundment is within 10 feet of the crest. The dam crest was extended to the cutoff wall for Stage VII-B. A large excavation was required.

The conceptual design of the future raises to closure includes the permanent further reduction of seepage on the left abutment by extending the cutoff wall beyond the dam crest until the wall intersects with the bed rock at elevation 986 feet. This is shown in Drawing TDC-18, and is the way that the Stage VII-B cutoff wall was terminated.

4.2.7 Seepage Analysis

A 3D seepage analysis was completed for the tailings main dam future raises to closure (URS, 2007b). Eight major factors were found to 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 liner) and three are operationally controlled and inter-dependent (tailings beach width, impoundment water level and phreatic surface location.

Seepage under the dam, seepage through the dam, and total seepage are presented in Table 4-3. Seepage under the dam is the largest 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.

Table 4-3 – Seepage Analysis Results

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

[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 water cover and spring runoff.

Sensitivity analyses were conducted of seepage through the dam. Leaks in the HDPE geomembrane will impact seepage depending on the number and size of the leaks. Reasonable numbers and sizes of leaks were estimated from other project histories, industry standards, published liner performance data, the liner installation quality control, and the observed integrity of the geomembrane exposed during the Stage VII-B cutoff wall excavations.

The use of extreme published values of geomembrane leak number and size would result in a 10 to 1000 % variation in seepage through the dam with 600-foot wide beach. In the extreme unlikely 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-wide beach with an intact liner.

4.2.8 Stability Analysis

A stability analysis was completed for the tailings main dam future raises to closure (URS, 2007c). Seven major factors were considered in the stability analyses: dam geometry, rock shear strength, gouge zone shear strength, seismic loading, phreatic surface, construction impact and long-term stability. All of these factors were taken into account to achieve a safe and optimum design of the dam. The stability analysis results are in Table 4-4.

For the end-of-construction condition with a pore pressure coefficient of 0.1 and a gouge internal friction angle of 22 degrees, the downstream slopes of the future raises will be stable at inclines of 2.75:1 or flatter. However, for the long-term condition with a gouge internal friction angle of 22 degrees, the downstream slopes of the future raises will be unstable at inclines of 2.75:1, but will be stable at inclines of 3:1.

Table 4-4 – Stability Analysis Results

Design Condition	Loading Conditions	Minimum Required FS	Computed FS (for gouge friction angle 22 deg. & cohesion 0)						
			2.75H:1V				3H:1V		
			Stage VIII	Stage IX	Closure		Stage VIII	Stage IX	Closure
					with berm	without berm			
End-of-Construction Ru =0.1	Static, Block	1.3	1.46	1.44	1.45	1.35	1.54	1.50	1.53
	Static, Circular	1.3	1.73	1.68	1.65	1.45	1.88	1.70	1.58
Long Term Condition	Static, Block	1.5	1.61	1.58	1.59	1.52	1.70	1.65	1.67
	Static, Circular	1.5	1.81	1.77	1.74	1.59	1.96	1.82	1.71
	Seismic (OBE) Block	1.2	1.23	1.23	1.23	1.18*	1.27	1.27	1.29
	Seismic (OBE) Circular	1.2	1.47	1.46	1.39	1.26	1.55	1.42	1.38
	Seismic (MDE) Block	1.0	0.98*	0.98*	1.00	0.96*	1.01	1.02	1.06
	Seismic (MDE) Circular	1.0	1.22	1.18	1.14	1.04	1.23	1.15	1.14

* Does not meet minimum FS criteria

For the conceptual design, a downstream slope of 3:1 was used for the future raises to closure except around the bend as noted in Section 4.2.3. The upstream slope will be maintained at the current 2.5:1 inclination for the future raises to closure. The crest width of the dam will also be maintained at 52 feet.

4.2.9 Seepage Collection Dam

At the final tailings main dam configuration and crest level of El. 986, the seepage collection dam will be approximately 170 feet downstream of the downstream toe of the tailings main dam. Therefore, the seepage collection dam could remain in place after the closure and continue functioning as a secondary seepage control system unless there were other reasons to relocate it further downstream.

At the final tailings main dam El. 986 the seepage pumpback system will be approximately 95 feet downstream of the toe of the tailings main dam. The seepage pumpback system could remain in place after the closure of the tailings main dam and continue functioning as a secondary seepage control system.

4.2.10 Emergency Spillway

The Alaska dam safety guidelines (ADNR 2005a) states the following with respect to water management at tailings dams:

> "*Managing water at tailings dams represents a unique challenge for designers and operators. During the operating phase, an emergency spillway might not exist and the reservoir must then retain the full volume of the IDF. In this case, a detailed water balance methodology must be developed to carefully monitor and maintain a reserve storage capacity. For closure, the facility must be modified to safely handle an IDF, typically the PMF or some other extreme event".*

The tailings impoundment will contain the inflows shown in Table 4-1, including the probable maximum flood (PMF), and will meet storage requirements at El. 984.5 without requiring an emergency spillway. After closure, the tailings water level will be regulated using an on site pump and treatment system. However, standard dam design practices require that all dams at their completed height have an emergency spillway.

The spillway will be an open channel located above and aligned around the left abutment of the tailings main dam as shown on Figure 3-1 and Drawings TDC-7 and 18. The same type of spillway has functioned well at the freshwater dam since it was constructed in 1989.

The control section at the spillway crest will function as a broad-crested weir at El. 984.5 so that the tailings facility will remain a "zero discharge" operation after closure. During an unexpected and improbable event where the impounded water may exceed El. 984.5, the discharge would occur through the spillway. The spillway is sized to route the PMF through its channel if the level of the impounded water is at El. 984.5 at the start of the probable maximum precipitation (PMP).

Based on the conceptual design, the alignment of the spillway closely borders the west extent of the liner system of the tailings main dam and will impact the alignment of the west perimeter road in the west abutment area. If this is determined to impact the stability of dam or the effectiveness of the liner system, the spillway channel will be moved west. The integration of the spillway and the liner system will be determined at a later stage.

Erosion protection will be required in the spillway because of the high velocity of flow in the outlet channel. Riprap was considered to be the most suitable type of erosion control because of its availability and durability. The extent of riprap required along the outlet channel is shown on Drawings TDC-22 and TCD-23. Riprap is not required in some parts of the channel because the channel bottom is in competent bedrock. Due to the estimated low flow depths in the channel, erosion protection using riprap is not required at channel depths of more than 3 feet from the bottom of the channel.

For riprap design the outlet channel was divided into four reaches as shown on Drawing TDC-21 and in Table 4-5. The reach 1 profile is shown on Drawing TDC-24. The reach 2 and 3 profiles are shown on Drawing TDC-25. The reach 4 profile is not shown because the slope is zero and there is no variation in the bottom profile. Details are provided in Appendix B.

To control erosion at the outlet of the spillway channel, a stilling basin will be constructed at the end of the spillway chute before discharging into the North Fork of Red Dog Creek as shown on Drawing TDC-

22. The stilling basin will dissipate energy from the flow prior to discharge to the creek. The stilling basin is conceptually designed to handle the design discharge of the spillway of 37 cubic feet per second.

Table 4-5 – Riprap Conceptual Design

Reach	Slope	D_{50} (inches)
1	24	12
2	10	6
3	2	3
4	0	Not required

It is presently anticipated that the riprap for erosion control in the stilling basin will be similar to that sized for Reach 3, with a D_{50} of 12 inches.

To prevent re-suspension of tailings at the spillway entrance, tailings should not be placed within 75 feet of the spillway entrance and a gravel blanket should be placed in the vicinity of the spillway entrance. Therefore, the spillway entrance for conceptual design purposes is located more than 200 feet upstream of the crest of the tailings dam, as shown on Drawings TDC-7 and 18.

The conceptual design of the spillway took into account the results of the 2005 geotechnical investigation conducted along the spillway alignment (Appendix B and URS, 2006a). The conceptual design of the spillway sections shown on Drawing TDC-23 has assumed that the organic root mat, clayey silty sand with gravel, and the highly weathered black shale will be over-excavated and replaced with riprap to provide the required protection for the spillway.

5.0 LONG-TERM INTEGRITY OF TAI LINGS MAIN DAM

5.1 DAM EMBANKMENT

The long-term stability of the tailings main dam depends on the durability of rockfill materials and the performance of the dam embankment in accommodating movements that might be caused by thaw consolidation of the dam foundation and seismic deformation triggered by an earthquake.

5.1.1 Rockfill Durability

The rockfill that was used in the Stage I starter dam was obtained from the mill area. The rockfill that was used for the Stage II to VII-B raises was obtained from the DD2 quarry which is located southwest of the tailings impoundment, and from the Main Pit which is located east of the tailings impoundment. The planned sources of rockfill for the future raises to closure are the DD2 quarry and acceptable rock from the continued mining of the Main Pit and the future planned mining of the Aqqaluk Pit.

The DD2 quarry is located west of the southwest corner of the tailings impoundment, and almost due west of the overburden stockpile. The rock mined from DD2 and used in the main embankment consisted mainly of shales and cherts, and various combinations of these two rock types. Construction grade rock fill from the Main Pit was used when the material quality was high enough to met design specifications and large enough to warrant re-routing of the truck fleet from hauling from DD2.

The rockfill from DD2 was identified during quarrying as Siksikpuk Shale, but the materials observed during Stage VII-B construction and during dam inspections were observed to be more representative of Okpikruak Shale (Ks). This is a Cretacious shale that is described in the geology records as:

> *"dark gray to medium gray, soft to moderately hard, clayey shale, thinly bedded, typically (but not always) micaceous. Commonly contains siltstone, and less frequently, sandstone (KSS) . . . can often be distinquished from other shales by scratching with a knife, which produces a characteristic orange-brown streak."*

The embankment consists of seven soil types described in the dam history report (URS 2007a) as follows:

- Soil Type 1 - Random Rockfill (24-inch)
- Soil Type 2 - Rockfill (12-inch)
- Soil Type 3 - Processed Select (3-inch)
- Soil Type 4 - Processed Select (1-inch)
- Soil Type 5 - Rock Drain and Riprap
- Soil Type 6 - Random Rockfill (12-inch)
- Soil Type 7 - Transition Rockfill (12-inch)
- Soil Type 8 - Kivalina Shale (Select finer fraction)
- Soil Type 9 - Random Mine Waste Fill

Soil Types 1 and 2 are the main rockfill material in the dam. Soil Type 2 was used for the Stage 1 Starter dam. Soil Type 1 was used to build the downstream mass of Stages II to VII, and the protective buttress for the liner system on the upstream face of the dam as shown in Drawings TDC 13, 16, and 19.

Soil Types 3, 4, and 7 are part of the liner system with the geotextile and HDPE geomembrane. Soil Type 9 was specified during Stage VII-B construction as mine waste that could be used as fill upstream of the curtain wall. These soils are buried and less susceptible to natural weathering over time. They are expected to outlive the geotextile and HDPE geomembrane, and therefore are not expected to be the controlling factor for the long-term rockfill durability in the dam.

Soil Type 5 was used in the rock drain part of the drainage system and the seepage collection pond. Soil Type 6 was used as rockfill over the rock drain and pumpback chambers in the seepage collection system. These soils do not form the main component of the embankment, and are not expected to be the controlling factor in evaluating the long-term rockfill durability in the tailings main dam.

The rockfill in the Stage I starter dam mass (Soil Type 2) consists of slightly weathered to fresh select chert carbonate and sandstone from the mill site area. This rockfill is well-graded and consists primarily of gravels, cobbles or blasted rock with a maximum diameter of 24 inches, less than 15 percent by weight passing the No. 200 sieve, and less than 5 percent ice by volume. Some flexibility in ice content was allowed depending on the percent passing the No. 200 sieve.

The rockfill in the Stage II to VII-B mass and the Stage I to VII-B buttress (Soil Type 2) consists of moderately weathered to fresher select chert, shale and sandstone, or mine waste. It is well-graded and consists primarily of gravels and cobbles with a maximum diameter of 24 inches, less than 25 percent fines, and less than 5 percent ice by volume. Some flexibility in ice content was allowed depending on the percent passing the No. 200 sieve.

The durability of the rockfill for purposes of designing the future raises will be evaluated by conducting the following procedures:

- Petrographic analysis
- Field observations
- Core measurements
- Laboratory testing.

A petrographic analysis of the rock mass at the macroscopic level consists of identifying the source lithology, conducting field observations to describe the mineral content, and providing a textural description as outlined in ASTM D4992. A key component of the analysis is that it be conducted by an individual that is qualified by education and experience.

At the microscopic level of petrographic analysis, the methods of atomic absorption and x-ray fluorescence can be used to evaluate the chemical structure and relative abundance of elements that make up the rock mass. Table 5-1 provides the lithologic properties of rock core samples taken from the DD2 quarry. The analyses were conducted during the 2005 and 2006 construction of Stage VII-B.

Table 5-1 – Lithology of DD2 Quarry Rock

Sample ID	Facility	Location	Lithology	Mnlz	Salts	pH	EC
RDP1-4	DD2	Gravel Plant	Siksikpuk	None	no	7.15	416
RDP1-5	DD2	North Side of Road	Siksikpuk	None	FeOx	5.28	59.1
RDP1-6	DD2	South Side of Road	Siksikpuk	None	no	5.64	165

The quality of a rock mass and its potential for providing a durable fill were measured by evaluating properties of a rock mass during coring. These properties include density or specific gravity, hardness, weathering characteristics and joint spacing, core recovery ratio (L) and rock quality designation (RQD), described as follows:

- The density of the material from the DD2 quarry was typically 140 pounds per cubic foot (pcf).
- L is a measure of the length of recovered core, and is obtained by dividing the length of recovered core by the length of core run. It is used with the RQD to evaluate the parent rock quality.
- RQD is an index property used to measure the quality of a rock mass. The extracted core is assessed for the frequency of fracturing over the length of a 100 mm core sample.

The rockfill durability was also assessed by making observations and measurements during rockfill placement and dam operations. Monitoring settlement in the dam is a way of evaluating rockfill behavior once the dam is in service. Secondary settlement or consolidation takes place over time and can be measured with settlement monuments and observed visually. Section 5.1.2 describes the observed settlement that appears to have taken place in the main embankment as a result of thaw consolidation.

Typical riprap durability specifications call for tests such as freeze-thaw durability (ASTM C666), specific gravity and absorption (ASTM C 127), sodium sulfate soundness (ASTM C 88) and Los Angeles (LA) Abrasion (ASTM C131/C535).

In June 2004, Alaska Test Lab (ATL) reported the results of an LA Abrasion test (ASTM C131/AASHTO T 96). A degradation of 26 and percent loss of 39.6 percent were recorded. The Alaska State Department of Transportation (ADOT) specifications for riprap require a wear not greater than 50 percent after 500 revolutions in the LA abrasion test. The LA Abrasion test result exceeded the ADOT riprap requirement.

Freeze-thaw durability and sodium or magnesium sulfate soundness tests have not been performed on DD2 material. Typically one or the other, but not both, are performed as a measure of rock durability. These methods test the repeated application of an internal expansive force on the test sample. In the freeze-thaw test, the material is cyclically saturated, frozen and thawed. The sodium or magnesium sulfate soundness test allows salt crystals to enter the material in solution, dry and re-saturate.

There are many ways of measuring rock properties to evaluate its potential durability and longevity when it is used for rockfill in the tailings main dam. The appropriate measures have been made to evaluate the material in DD2 for use as rockfill in the tailings main dam.

The rockfill durability and ability to handle movement caused by thaw consolidation and seismic-induced foundation and seismic deformation is critical to long-term dam stability. The rockfill is inert rock from the DD-2 quarry and the Main Pit, is durable and not reactive, and will remain intact for centuries. Thaw consolidation of the foundation has mostly occurred, and any further thaw will have minimal impact.

5.1.2 Thaw Consolidation

Thaw consolidation of the dam foundation is expected from the degradation of permafrost in the dam foundation. With the various stages of dam construction and operation, the permafrost conditions in the dam foundation have changed significantly. The historical permafrost degradation beneath the main dam crest was monitored by dam thermistors and is summarized in the seepage analysis report (URS, 2007b).

Dam crest settlement has not been monitored. However, the crest settlement between the construction of stages can be estimated from as-built survey records and from observations of the existing anchor trench for each new raise. For example, Stages VI and VII-A were completed in 1993 and 2003, respectively, ten years apart. The excavation and survey of the Stage VI anchor trench in 2003 showed that a maximum crest settlement of about one foot occurred around Sta. 13+75, which is 115 feet (El. 950 minus 835) above the original creek bed. Air photo mapping in 2001 showed the crest at approximately Sta. 12+00 to be at El. 949.3, which represents about 0.7 feet of settlement.

In general, the thaw settlement estimate has two components (Crory, 1973; Watson et al., 1973):

- Settlement caused by the phase change of segregated ice, and
- Subsequent consolidation.

Neglecting the compressibility of the shale bedrock within which most of the permafrost degradation occurred from 1993 to 2003, the settlement of the dam due to thaw consolidation during this period could be approximated by the component of thaw settlement due to the phase change.

Settlement caused by the phase change of segregated ice can be estimated in the following way:

$$S = 1 - \frac{\gamma_{df}}{\gamma_{dth}} Z ,$$

where Z= thickness of originally frozen layer

γ_{df} = dry density of frozen soil

γ_{dth} = dry density of thawed soil.

By using the dry density of frozen and thawed soil and the results of thaw consolidation and laboratory tests (URS 2006b, Appendix B4 and Table 4-3), URS estimated the thaw settlement along the Stage VI crest. The crest elevation based on calculated thaw settlement and observed settlement of the anchored liner is shown on Figure 5-1. The graph shows the predicted thaw settlement along the Stage VI axis.

The predicted thaw settlement varies from 1.2 to 6.3 feet. The maximum thaw settlement was predicted to occur at Sta. 8+00. In the vicinity of the abutments, a settlement of up to 2 feet was predicted. The predicted thaw settlement agrees well with the crest settlement obtained from as-built survey drawings.

The thickness of permafrost degradation is greatest in the middle of the dam and lowest at the abutments. The degradation in the abutments and middle of the dam occurred in highly weathered shale and slightly to moderately weathered shale, respectively. A higher thaw settlement was predicted in the middle part of the dam because slightly to moderately weathered shale has more thaw strain than highly weathered shale.

5.1.3 Seismic Deformation

The conceptual design of the raises to closure considers the seismic stability and permanent deformation during a seismic event that is reasonably expected at the site. The design seismic event considered corresponds to a maximum magnitude of earthquake that likely occurs once in 2,475-years. Permanent deformations of the dam for this earthquake were also estimated by a simplified procedure incorporating dynamic response analyses (URS, 2007c).

The deformation analysis of the tailings main dam provided the following results that may be induced by a 6.5-magnitude earthquake:

- A maximum horizontal displacement in a range of 2.5 to 21 inches.
- A maximum shear strain in the dam of approximately 0.2 percent.

These levels of earthquake-induced displacement and shear strain are considered tolerable under such an extreme event. The computed earthquake-induced displacement is also used in evaluating the potential damage on the geomembrane under the seismic loading. This is discussed in more details in Section 4.2.4.

5.2 LINER SYSTEM

HDPE geomembrane with a thickness of 100 mils was used as the seepage barrier in the liner system of the tailings main dam. Even though HDPE geomembrane has been successfully used for over 25 years now, it is a relatively new materials in civil engineering applications. In order to research the durability and life expectance of HDPE geomembranes, URS conducted a literature search, reviewed an SRK (2004) memorandum, and communicated with the following industry experts:

- Dr. J.P. Giroud, Independent Consultant (Giroud, 2006a)
- Dr. R. Koerner of Geosynthetic Research Institute (Koerner, 2006).

A study by Koerner et al. (2005) has shown that the useful lifetime of HDPE geomembranes is composed of the following three distinct stages of time (Figure 5-2) as follows:

- **Stage A - Antioxidant Depletion Time.** The antioxidants of the geomembrane are gradually depleted due to chemical reactions with oxygen or physical loss.

- **Stage B - Induction Time to Onset of Degradation.** Oxidation occurs in geomembrane without black carbon and antioxidants. This stage starts at an extremely slow oxidation rate, continues at a larger oxidation rate, and eventually returns to a very slow oxidation rate.
- **Stage C - Time to Reach 50 percent Degradation**. The polymer degradation continues until the geomembrane degrades to 50 percent of its initial mechanical and physical properties. At the end of this stage, all tensile properties are severely changed so that the engineering performance of the geomembrane is jeopardized. This is considered to be the end of geomembrane life even though the material still exists and can function with a factor of safety lower than the initial design value.

Rowe & Sangam (2002) used laboratory test results to estimate the lifetime of HDPE geomembrane as a function of in-service temperature (from 10 ^{o}C (50 ^{o}F) to 30 ^{o}C (86 ^{o}F)) and material in contact with the geomembrane. These tests were conducted on non-exposed 80 and 60 mil geomembranes at various temperatures. Later, Koerner & Hsuan (2003) published the lifetime prediction of the material in each of the three stages for temperature ranges of 20 ^{o}C (68 ^{o}F) to 40 ^{o}C (104 ^{o}F). They also found that the predicted lifetime of the geomembrane is largely dependent on temperature and exposure conditions.

Recently, Koerner (2006) (attached in Appendix C) developed a set of revised total geomembrane lifetimes (Table 5-2) using average values of Stage C from three sources (Martin & Gardner, 1983; Viebke et al., 1994; and unpublished data from Geosynthetic Research Institute (Koerner, 2006)) may be used as a practical guide. Koerner (2006) pointed out that unlike the Stage A prediction, more test data on Stages B and C will be needed to gain a higher degree of confidence in the predictions.

Table 5-2 – Unexposed[1] HDPE Geomembrane Lifetime Prediction

In-Service Temperature (C°)	Stage A (years)		Stage B (years)	Stage C (years)			Total Prediction[2] (years)
	Std OIT[3]	HP OIT[4]		Viebke et al., 1994	Martin & Gardner, 1983	GSI Data Koerner, 2006	
20	200	215	30	740	208	35	565
25	135	144	25	441	100	26	353
30	95	98	20	259	49	21	226
35	65	67	15	154	25	17	146
40	45	47	10	93	13	14	96

[1]Exposed geomembrane lifetimes are considerably less than values presented in this table.
[2]Total = Stage A (average) + Stage B + Stage C (average)
[3]Std OIT - standard oxidative induction time
[4]HP OIT - high pressure oxidative induction time
Source: Koerner (2006)

5.2.1 Life Expectancy of HDPE Geomembrane

The lifetime of HDPE geomembrane primarily depends on temperature and material in contact with the geomembrane. Data from Red Dog indicates that the temperature on the geomembrane surface varies from -2.3 (27.9^{o}F) to 6°C (42.8 ^{o}F) at different elevations, which are outside the published temperature ranges. Also, the exposure condition at Red Dog could be considered as intermediary between exposure to air and a combination of air, water and leachate (Giroud, 2006b, attached in Appendix C).

Therefore, an extrapolation of temperature data and interpolation of material in contact was necessary to predict the proper lifetime of the geomembrane in the tailings main dam. Rowe (2005) and Koerner et al. (2005) present computation methods to predict the antioxidant depletion of geomembrane in Stage A which may be used to extrapolate the predicted lifetime from the published literatures.

Table 5-2 lists the results of extrapolations by Giroud (2006c) and URS (Appendix C) based on the two approaches under various conditions. The two data sets in the Stage A period were based on different approaches proposed by Rowe and Sangam (2002). The differences in the results of the approaches ("inferred exposure" or air exposure and "simulated exposure" from laboratory tests) are significant.

Tests to evaluate the Stage B time period are in progress (Giroud 2006b), but the time period is expected to be relatively small. Therefore, an approximate estimate is reasonable without significantly impacting the overall lifetime prediction. Predictions of the Stage C time period may be based on Viebke, et al. (1994), Martin & Gardner (1983), and limited preliminary unpublished tests on polyethylene resins at the Geosynthetic Research Institute (GRI) (Koerner, 2006). Estimated time periods for Stages B and C by for the temperature range of -2.3 (27.9°F) to 6°C (42.8 °F) are presented in Table 5-3.

The total lifetime of the geomembrane specific to Red Dog conditions were obtained by adding the estimated time periods of Stages A, B and C. These are shown in the last two columns of Table 5-3 based on data from two sources. The values based on Koerner et al. (2005) reflect the geomembrane lifetime under the air exposure condition; whereas those based on Rowe (2005) are from test data for a less-aggressive leachate exposure condition.

Table 5-3 – HDPE Geomembrane Lifetime Prediction Extrapolation

Actual temp.	Base temp. range	Stage A (years)				Stage B (years)			Stage C (years)	Total (years)	
		Koerner		Rowe		Koerner	Rowe		Koerner	Koerner	Rowe
		Standard OIT[1]	High pressure OIT[1]	Air exposure	Simulated exposure		Base case	Adjusted case		Standard OIT[1]/Air exposure	Simulated exposure
-2.3°C	20-40°C	1368	1527	1528	822	124	201	195	114	1606	1131
6°C	20-40°C	645	710	735	381	71	73	61	72	790	510

[1]OIT - oxidative induction time

It is noted that tests have been conducted on polyethylene resins used in pipes which resulted in a Stage C time period prediction of 3,500 to 4,000 years at a temperature of 6°C (42.8 °F). However, this cannot be applied to a HDPE geomembrane without more tests. The Table 5-3 values for a HDPE geomembrane lifetime were based on the available information at the time of this report. The geomembrane lifetime is expected to range from:

- 510 years for buried geomembrane exposed to semi-aggressive liquids and at 6.0°C (42.8 °F) to
- 1600 years (for buried geomembrane that is not exposed to liquid and at -2.3°C (27.9°F)).

5.2.2 HDPE Geomembrane Arctic Performance

The HDPE geomembrane lifetime estimates were based on an assumption that the geomembrane will perform in normal conditions without any premature failure. However, it is reported (Peggs 2003, Peggs et al. 1990, and Reddy & Butul 1999) that HDPE geomembranes can occasionally be subjected to a shattering failure due to brittle cracking during cold weather with temperature ranging from 5°C (41°F) to -30°C (-22°F). Failures of this type have been observed on side slopes of uncovered liners. Since the temperature at the geomembrane in the tailings main dam ranges from -2.3°C (27.9°F) to 6°C (42.8 °F), it is prudent to address this potential mode of failure.

Uniaxial and multiaxial tensile tests conducted by Peggs et al. (1990) with HDPE geomembranes in the temperature range of 25°C (77°F) to -30°C (-22°F) showed that the yield stress increases from 3.0 to 5.0 ksi while the yield strain decreases from 12 to 6 percent. Pneumatic burst tests showed that at a temperature of -38°C (-36.4°F), geomembranes displayed strains at yield of less than 2 percent. Peggs et al. (1990) report that brittle cracking in geomembranes can form and propagate rapidly under stress resulting in the premature failure. Reddy & Butul (1999) summarize the factors that induce stress cracking as follows:

- Use of low stress cracking resistant resins
- Stresses induced by restrained thermal contractions at low ambient temperatures
- Overheating seams or spot welding
- Mechanical damage
- Stress induced by lack of support at corners
- Hard particle impingement
- Impact load
- Chemical environment.

Geomembranes are subjected to dimensional changes when they are deployed at a higher temperature than the temperature that they experience during the winter thermal contraction. These changes lead to induced stresses in the geomembrane, which cause stress cracks to start. Therefore, it is necessary to provide the geomembrane with sufficient slack during installation to avoid thermal stresses. The required slack of a geomembrane for a site-specific temperature condition can be computed as follows:

$$\Delta L = L x CTE x \Delta T$$

where: ΔL = required slack of the geomembrane

L = length of the geomembrane sheet in the considered direction

CTE = coefficient of thermal expansion of the geomembrane

ΔT = difference between geomembrane installation temperature and lowest expected service temperature.

Some of the geomembrane in tailings main dam was installed in the fall season with the maximum ambient temperature at 19°C (66.2°F). The measured highest and lowest geomembrane temperatures at the site are 35° (95°F) and -2.3°C (27.9°F), respectively. For these temperatures, the required slackness (slack) of a 23 feet by 300 feet geomembrane sheet (one roll) are estimated with a safety factor of 1.5 to be 3.6 and 46.2 inches on the short and long sides, respectively (Appendix C). Geomembrane failure due to thermal stress can be minimized if these slacks are provided during installation.

5.2.3 Life Expectancy of Geotextiles

Nonwoven polypropylene geotextiles Geotex 1001 and 1601 (SI Geosolutions, 2006) were installed in the tailings main dam. In order to estimate their lifetime in the dam, a Federal Highway Administration (FHWA, 2000) publication was used that describes a deteriorating mechanism of geosynthetic materials having the same polyolefin parent, polyethylene and polypropylene, and recommends an approach to predict the lifetimes. Based on this approach and GRI test results, SI Geosolutions (1997) claims that the mechanical degradation can be less for polypropylene than for polyethylene under some conditions

Geotextiles and HDPE geomembranes manufactured with the same antioxidant composition likely have similar lifetimes. Using the most conservative estimate of current research data (SI Geosolutions, 2006), geotextile lifetime is found to be approximately 100 years at 20°C (68°F). Assuming that 70percent of this lifetime is attributed to the antioxidant depletion time with the conditions described by Koerner et al. (2005), URS computed the geotextile lifetime at 6°C (42.8^{o}F) and -2.3 ^{o}C (27.9^{o}F) to be 222 and 465 years, respectively (Appendix C).

5.2.4 HDPE Geomembrane Response to Embankment Movement

The HDPE geomembrane on the upstream face of the tailings main dam is the only impervious barrier in the dam. The integrity of this barrier during the design seismic event is implicitly assumed in completing the stability and deformation analyses summarized in Section 4.1.3. The ability of the geomembrane to accommodate the permanent seismic deformations of the dam without failure is evaluated in this section.

If the liner fails to function as designed, the phreatic surface in the dam will rise, as shown by a sensitivity evaluation that was completed for the extreme case of liner failure in the seepage analysis (URS, 2007b). This rise of phreatic surface will adversely effect the dam stability (URS, 2007c). Therefore, the long-term performance of the dam would depend on the ability of the geomembrane to accommodate the seismic deformations and settlements of the dam.

5.2.4.1 Seismic Deformations

As discussed in Section 4.1.3, permanent deformation of the dam occurs when the ground acceleration exceeds the yield acceleration of the failure mass corresponding to a critical failure surface. When the ground acceleration decreases, the failure mass will decelerate and stop moving. The total deformations will depend on the ground acceleration history for a particular event and the yield acceleration value.

The permanent deformation of the geomembrane will vary gradually up the height of the dam and may include an abrupt deformation such as when the critical failure surface passes through the liner system as shown on Figure 5-3. The stability analyses completed for the closure configuration of the dam found that the critical failure surfaces, both with block and circular failure surfaces, for the seismic stability of the downstream slope of the tailings dam do not pass through the liner system.

However, the actual deformation of the dam is very dependant on the acceleration time history. Also, when a higher seismic coefficient, up to a value equal to the design peak ground acceleration (PGA) is considered, trial failure surfaces passing through the liner and FS values less than 1.0 are possible. These factors indicate that an abrupt deformation of the liner system in the dam is less likely, but possible.

Considering the likely deformation modes of the geomembrane, URS evaluated the following two conditions to evaluate the geomembrane response to permanent seismic deformations of the dam:

- **Gradual Deformation of Geomembrane**: This is the most likely deformation condition and would vary up the height of the dam.
- **Abrupt Deformation of Geomembrane:** This includes an abrupt deformation along a critical failure surface that passes through the liner system, and is an extreme deformation condition as shown on Figure 5-3.

5.2.4.1.1 Gradual Deformation of Geomembrane

The deformation of the dam around the liner system is assumed to be gradual. The maximum shear strain within the dam, estimated from the simplified deformation analysis, is 0.2 percent. The geomembrane is inclined at 21.8-degrees from horizontal. Therefore, in order for the geomembrane to not slip, the following conditions are necessary:

- The geomembrane would deform along with the surrounding rockfill
- The shear strain in the dam would result in tensile strain in the geomembrane.

Considering an infinitesimal element of soil mass containing an embedded geomembrane element as in Figure 5-4, the tensile strain in the geomembrane was estimated assuming that no slip occurs between the geomembrane and the surrounding rockfill, as follows:

- Horizontal deformation of the soil mass element: $\Delta dx = \gamma_{\max} dy$
- Original length of geomembrane element: $L = \dfrac{dy}{\sin\alpha}$
- Elongation of the geomembrane element due to soil deformation: $\Delta L = \gamma_{\max} dy \cos\alpha$
- The tensile strain in the geomembrane: $\varepsilon = \dfrac{\Delta L}{L} = \gamma_{\max} \cos\alpha \sin\alpha$

Based on the maximum shear strain estimated from the simplified deformation analysis ($\gamma_{\max}$ = 0.2 percent), the tensile strain in the geomembrane was estimated as, 0.07 percent (ε = 0.2 percent by cos 21.8^o by sin 21.8^o).

The yield strain of the geomembrane obtained from tensile tests on a wide specimen is about 10 percent. The actual tensile strain estimate from the maximum shear strain in the dam (0.07 percent) is about 0.7 percent of the yield strain obtained in the laboratory. However, the time to degrade the mechanical properties of the geomembrane to about 50 percent is at least several hundred years, and the maximum tensile strain during an earthquake is only 0.7 percent of the yield strain and 0.1 percent of the failure strain (based on 65 percent of failure strain, Appendix C).

It is concluded that the geomembrane in the tailings main dam can safely accommodate the permanent gradual deformation considered above.

5.2.4.1.2 Abrupt Deformation of Geomembrane

A seismic event can cause displacements of the tailings main dam that may result in abrupt deformations of the geomembrane as shown on Figure 5-3. This section describes the computations that were completed to evaluate the effect of the design that a 2,475-return period seismic event would have on the

geomembrane. Two extreme cases of potential failure surfaces were considered in the evaluation, as follows:

- Failure surface intersects the geomembrane perpendicularly, and
- Failure surface intersects the geomembrane horizontally.

Soil Type 1 rockfill contains larger particles than other soil types within the embankment. The rockfill particle size and arrangements (interlock) will control the thickness of the disturbed zone and shape of the potential sliding plane. Neither side of the liner system is in contact with soil Type 1 rockfill. Rather, the Soil Type 1 fill serves as buttress material on the upstream and downstream faces of the dam.

Based on the construction specification, soil Type 1 is well-graded and consists primarily of gravels and cobbles with maximum diameter of 24 inches, less than 25 percent by weight passing the No. 200 sieve, and less than 5 percent ice by volume. However, based site observation, the average size of soil Type 1 could be as low as 6 inches in diameter.

However, due to over-size particles that are greater than 6 inches in size, and the particle interlock arrangement, the thickness of the disturbed zone would be in the order of one to two feet. Therefore, it was assumed that a one-foot width of disturbed soil pushes and displaces the geomembrane out of its original position.

In order for the liner to continue to function after the seismic event, the geomembrane must be able to accommodate the increase in length, without tensile failure, despite the relative movement of the two zones of soil. Alternatively, a maximum deformation of the geomembrane based on its yield strength may be computed and compared to the maximum displacement expected from the design earthquake. If the former deformation is larger than the earthquake-induced displacement, it can be concluded that the geomembrane will remain intact after the seismic event. URS used the latter approach in this study.

With elongations at peak tensile stress and breakage of 16 to 65 percent, respectively, maximum horizontal deformations were computed based on geometric relationships and limit equilibrium. Calculations were completed at various depths for the two assumed failure surface cases. The estimated maximum horizontal deformations that the geomembrane can tolerate without breakage are as follows:

- Failure surface intersects the geomembrane perpendicularly: 3.6 feet
- Failure surface intersects the geomembrane horizontally: 1.9 feet

If the thickness of the disturbed zone is grater than one foot, the estimated maximum horizontal deformations that the geomembrane can tolerate without breakage would be higher than the above reported values. Since the maximum horizontal displacement from the maximum credible earthquake was estimated to be 21 inches (1.75 feet), it is concluded that the geomembrane in the tailings main dam can safely handle an abrupt deformation induced by the design 2,475-year return period seismic event.

5.2.4.2 Settlement Deformations

Settlement of the tailings main dam has primarily resulted from foundation thaw consolidation and the self weight of the rockfill and tailings. Some additional settlement, expected to be elastic in nature, due to self weight of the rockfill and tailings will occur as the dam is raised and additional tailings are impounded. However, the thaw consolidation is not expected to increase because it appears to have stabilized as described in the seepage report (URS, 2007b).

Thaw consolidation was induced by the permafrost degradation in the rock foundation and generally occurs as a phase change of segregated ice and subsequent consolidation. Phase changes and the resulting settlement for the future raises to closure are expected to be minimal because on-going and future construction activities will be distal to the rock foundation, except around the downstream toe where the liner will not be influenced.

Settlement caused by the consolidation of the compacted rockfill has taken place during and immediately after each raise was constructed. Therefore, most of the settlement has already occurred incrementally as construction progresses so that its effect on the geomembrane has been distributed over the dam height.

Settlement of the wing wall is expected to occur as a result of foundation thaw consolidation and self weight of the rockfill. However, this settlement is expected to be minimal because of the maximum height of the wing wall is approximately 26 feet and the subsurface materials appear to already thawed.

5.3 DRAINAGE SYSTEM

5.3.1 Tailings Piping Potential and Internal Erosion

Seepage force resulting from hydraulic gradients in the tailings main dam can adversely affect the dam in two ways:

- Piping
- Internal erosion.

Piping is generally referred to failure occurring when seepage force is larger than the soil strength. Usually the hydraulic gradient, or "exit gradient", is checked at a point downstream where flow exits the soil mass. At this point, the effective overburden pressure and soil strength are the lowest.

The concept of piping is generally explained in a simplified one-dimensional model in which a soil column is subjected to a head differential (h). Forces on a horizontal section across the soil column are computed as follows:

Downward force: $F_d = \gamma_m LA$,

Upward force: $F_u = (h + L)\gamma_w A$,

where γ_m = saturated soil unit weight,

γ_w = water unit weight,

L = length of soil column,

A = cross-sectional area of soil column,

h = differential head.

Ignoring the side friction along the perimeter of the soil column, soil failure is initiated when the upward (seepage) force becomes larger than the downward force. The hydraulic gradient causing the upward driving force to equal to the downward resisting force, is termed critical hydraulic gradient (i_{cr}). Equating the forces results in a simple formula for evaluating the piping potential of a soil element as follows:

$$\gamma_m LA = (h + L)\gamma_w A.$$

Then $$i_{cr} = \frac{h}{L} = \frac{\gamma_m}{\gamma_w} - 1 = \frac{\gamma'_m}{\gamma_w},$$

where γ'_m = buoyant or effective unit weight of soil

The FS against piping is normally defined as a ratio of critical hydraulic gradient of the soil to induced hydraulic gradient at the point of concern:

$$FOS = \frac{i_{cr}}{i}.$$

Table 5-4 lists the critical hydraulic gradient values of different soil types in the tailings main dam site, which is a function of the soil effective unit weight.

In order to evaluate the piping potential of the soils in the tailings main dam, it was necessary to estimate the induced hydraulic gradient in the dam at controlling points from the 3D seepage model described in the seepage analysis report (URS, 2007b). The conditions in which the piping potential was evaluated are the closure cases with a 600-foot beach and effective liner and a 900-foot beach with no liner. Based on this study, a minimum factor of safety of 2.0 was conservatively obtained, indicating safe conditions.

Table 5-4 – Soil Types Critical Hydraulic Gradient

Soil type	**Soil saturated unit weight, γ_m (pcf)**	**Water unit weight, γ_w (pcf)**	**Soil effective unit weight, γ'_m (pcf)**	**Critical hydraulic gradient i_{cr}**
Tailings (Silt and fine Sand)	120	62.4	57.6	0.92
Liner bedding	138	62.4	75.6	1.21
Filter drain	138	62.4	75.6	1.21
Transition rockfill	138	62.4	75.6	1.21
Random rockfill	140	62.4	77.6	1.24
Highly weathered shale	115	62.4	52.6	0.84
Underdrain	115	62.4	52.6	0.84

Internal erosion is defined by van Zyl and Harr (1981) as the process in which finer particles are moved locally into coarser materials. Internal erosion causes cavities in a dam that may lead to further damage. The potential to this type of seepage erosion can be evaluated by applying filter criteria to estimate blockage stability in areas of transition from fine to coarse materials.

For the tailings main dam, a “filter drain” material was placed next to the geomembrane over the entire upstream face of the dam to provide a drainage path to the underdrain. With the presence of the geomembrane and geotextile liner materials, any movement of the filter drain material into the rockfill dam body is prevented. However, the liner materials could deteriorate with time. Therefore, the internal erosion potential of the filter drain and tailings was evaluated.

The following US Bureau of Reclamation (USBR, 1987 and 1998) filter criteria were used to evaluate the ability of the existing system to prevent internal erosion if the liner is no longer effective:

- Stability criteria: $\frac{(D_{15})_{filter}}{(d_{85})_{base}} \leq 5$ (to prevent the base soil movement into the filter material).
- Permeability criteria: $\frac{(D_{15})_{filter}}{(d_{15})_{base}} \geq 5$ (to provide faster seepage than that of the base soil).

Filter design also must satisfy the following conditions in addition to the stability and permeability criteria:

- Coefficient of uniformity (Cu) should be within the $1.5 \leq C_u \leq 8$ range.
- Maximum particle size of filter material is 75 mm to minimize particle segregation and bridging during placement.
- Maximum percentage of minus 200 size filter material is 5 percent to prevent clogging caused by excessive movement of fines in the filter and to maximize the permeability of the filter.

For the filter criteria evaluation, results of laboratory tests shown in Figure 5-5 were used on Soil Type 3, Soil Type 4, and tailings. The test results were reported by Alaska Test Lab (2005), URS (2005) and Golder Associates (2003). Two scenarios were evaluated as follows:

- Soil types 3 and 4 were considered as filter material; and tailings was considered as base soil. This combination does not meet the stability criteria based on their gradation curves. It was, therefore, concluded that loss of some fines of tailings will result if the liner becomes ineffective.
- Soil types 3 and 4 were considered as filter material and base soil, respectively. This combination of soil types does not meet the stability criteria based on gradation curves. Therefore, it was concluded that loss of some fines of soil type 4 will result if the liner becomes ineffective.

Filter criteria proposed by Foster & Fell (2001) were also considered. This method is based on analysis of laboratory test results as well as characterization of existing dams that have experienced internal erosions. Three internal erosion categories have been defined in this method:

- No erosion: Almost no erosion occurs.
- Some erosion and excessive erosion: The filter seals after some erosion of the base material.
- Continuing erosion: The filter is too coarse to allow the eroded materials to seal it, such as with unrestricted erosion of the base soil.

The method first identifies the base soil group in accordance with the percent fines passing through the 75 μm sieve. Then, the particle size of the filter material for which 15 percent by weight is finer (DF_{15}) is compared with particle size of the base material for which 85 percent by weight is finer. It was found that the Type 3 (filter) soil does not satisfy the “no erosion” criteria, and that continuing erosion of the tailings base material is expected and the eroded tailings will not seal Type 3 soil.

However, the low hydraulic gradient in the dam will reduce the potential for internal erosion and any erosion that might occur would only be from the liner system. The dam stability will not be impacted because of relatively small volume and coarse gradation of the Type 4 material.

5.3.2 Chemical Precipitation Potential

Tailings water that seeps out of the impoundment and through or under the liner system flows through the tailings, and then along the underdrain and into the seepage pond, where it is collected and pumped back up to the tailings impoundment.

The underdrain is composed of drain rock with voids that allow seepage to flow through it and under the dam. An evaluation was made of the underdrain becoming plugged over time by secondary mineral precipitation from the tailings pore water. This water could precipitate secondary minerals (predominantly ferric oxyhydroxides) due to a combination of the type and concentrations of chemical species in the pore water, the pH of the water, geochemical changes that may occur to the water, and other factors.

5.3.2.1 Causes of Precipitation

There are several factors which affect the precipitation rate in the underdrain. The basic, simplified relationships are as follows:

- **Initial iron concentration**: higher initial iron concentration in the seepage water, results in higher iron precipitation based on a linear relationship.
- **pH**: lower pH results in drastically lowered precipitation and higher pH results in increased precipitation.
- **Aeration level**: increased aeration results in proportionally higher precipitation according to a nearly linear relationship.
- **Retention time in the underdrain**: longer retention time results in proportionally more precipitation according to an approximately asymptotic relationship.
- **Temperature:** lower temperature results in less precipitation due to slower kinetics.

It was estimated to take approximately 11 years for water to penetrate from the tailings surface through the tailings and to the underdrain. As aerated water travels from the free water in the pond through the tailings toward the underdrain, oxygen will be consumed in various reactions with the sulphide mineral constituents of the tailings. The tailings will reduce oxidized substances flowing through them, resulting in pore water that will be in a reduced state when it reaches the underdrain.

As the pore water reaches the underdrain, it is in a reduced state, with a low redox potential. However, the underdrain itself is open to the atmosphere. It is assumed that due to air diffusion and convection through the voids in the underdrain, that the underdrain will be aerated to a level of approximately 10 percent of what would occur in the outside air. A value of 10 percent oxygen is representative for a waste rock dump, which is considered similar to the situation for the underdrain (Dames & Moore 1987).

The air will travel by diffusion and convection into the underdrain, from the drain rock in the underdrain, and from the underdrain exit to the atmosphere. Some oxygen will dissolve into the underdrain water. At the underdrain exit, the oxygen concentration in the water would be higher than 10 percent, while inside the dam where the tailings pore water enters the underdrain, the oxygen concentration would be lower than 10 percent due to the difficulty of air transport through the drain rock. Overall, the oxygen concentration throughout the underdrain is assumed to average 10 percent of atmospheric.

The constituents in the water (mainly ferrous iron, but including others) will be affected by this partial aeration. The redox potential of the water will change significantly due to the dissolved oxygen, and will

affect the geochemical stability of the underdrain water and result in precipitation of ferric iron hydroxides. This would occur in two steps:

- Ferrous iron is oxidized to ferric iron, which is the slowest step kinetically and limits the overall rate of the reaction
- The ferric iron ion will hydrolyze some water present, forming a ferric hydroxide precipitate.

At low pH of 2 to 3, the overall reaction is very slow and on the order of thousands of years. At a pH of 4, it would take about 300 years for 50 percent of the ferrous iron to be oxidized to ferric hydroxide. At a moderate pH of 6, 50 percent oxidation of ferrous to ferric hydroxide requires around 275 hours or 11.5 days. In 30 days at a pH of 6, about 81 percent of the ferrous iron would oxidize to ferric hydroxide precipitate. This precipitate has the potential to plug the pore space within the underdrain drainage rock.

5.3.2.2 Mitigating Factors

There are significant factors that reduce the potential of precipitation in the underdrain, such as mitigating the release of precipitated material from the water in the underdrain. For example, the residence time in the underdrain is about one day, assuming a 500-foot long by 50-foot wide by 6-foot high underdrain with 30 percent porosity and flow of 250 gpm. While the aerated water is thermodynamically favored to contain ferric iron, the kinetics of the oxidation of ferrous iron to ferric iron are not instantaneous.

The hydrolysis by ferric ion and formation of iron hydroxide precipitate is not instantaneous, but is faster than the oxidation of ferrous iron to ferric iron. The overall reaction kinetics is affected by the pH of the aerated water. At a low pH, the kinetics of the oxidation of ferrous iron to ferric iron are slow so that no appreciably precipitation will occur in one day. At moderate pH, the oxidation of ferrous iron to ferric iron is faster, as discussed above. Thus, a lower pH will not favor formation of a ferric iron precipitate.

A lower temperature in the underdrain also results in a slower reaction time. A drop from 25°C (77 °F) to 5°C (41°F) results in a ten-fold slowing of the reaction rate. At Red Dog, the average temperature is more likely to be 5°C (41°F) than 25°C (77°F), which is the temperature at which the PHREEQC (USGS 1999) geochemical simulations were run. The temperature in the tailings impoundment has historically varied between -2°C (35.6°F) and 6°C (42.8°F). Therefore, 5°C (41°F) is a reasonable assumption.

Lower levels of oxygen in the underdrain will result in a slower and less complete ferrous iron oxidation reaction. Deep in the dam at the start of the underdrain, the dissolved oxygen levels will be lower, resulting in slower ferrous iron oxidation reaction to begin with. As the water flows toward the exit, the dissolved oxygen level will increase, allowing for a faster ferrous iron oxidation reaction. Also, at times the underdrain may be full of water and not allowing any significant inflow of air from the exit. At these times, oxidation would be minimal, based only on air that flows in from the surrounding drain rock.

The ferric hydroxide precipitate that forms will initially be an amorphous hydrous ferric hydroxide (“yellowboy”) with entrained water, similar in consistency to loose gelatin. It is likely that much of this precipitate will be swept out of the underdrain by water flow. As the precipitate dries out, both inside and outside the underdrain, it will solidify to a more crystalline precipitate. Eventually, other forms of ferric hydroxide may form such as maghemite, hematite, and goethite. These are denser, more crystalline, and take many years to form. They are not a likely problem given the 1-day retention time in the underdrain.

5.3.2.3 Geochemical Modeling

Using the available chemical data on the pumpback water chemistry, the precipitate potential within the underdrain was evaluated using the one-dimensional chemical transport program "PHREEQC", developed by USGS (1999).

The underdrain is 500 feet long, 50 feet wide, and 6 feet deep. The underdrain flowrate of 250 gpm, which reflects the seepage estimate for closure conditions with a 900-feet wide beach, was used in this evaluation. These dimensions and flowrate provide a retention time of approximately three days.

Currently, the average pumpback water pH is 6.27. Once the mine is closed it is assumed that the pumpback water pH will drop to a slightly lower level due to minor oxidation of the tailings and formation of acidic drainage. This effect was not explored in this analysis, but is typically seen in acid generating tailings. Even if the tailings impoundment is kept a flooded, the 250 gpm pumpback of oxidized water would slowly oxidize some of the tailings, dropping the pH to an assumed value of 6.0.

Bacterial oxidation of iron was not considered in this analysis due to high pH. Bacteria responsible for oxidation of iron from ferrous iron to the ferric state are likely to be active at a pH below 3. Since it is unlikely that such a low pH will be present, this aspect was ignored.

The geochemical modeling and analysis were completed using the following representative parameters and assumptions:

- Flushing effect of 50 percent, so that half of all precipitate that forms stays in the underdrain, and the other half is flushed out and pumped back to the impoundment
- Uniform aeration of 10 percent
- Specific gravity of formed precipitate 2.4 g/cm^3 (Lide, 1991-1992)
- Void space within underdrain is 30 percent
- Average temperature in the underdrain is 5°C (41°F)
- pH of tailings water is 6.0
- Retention time in the underdrain is 1 day.

In order to evaluate the sensitivity of pH level and temperature, the analyses were completed for the following sets of parameters and a representative underdrain retention time of one day:

- pH 5.0 and 5°C (41°F): Optimistic assumption of what could happen
- pH 6.0 and 5°C (41°F): Representative long-term condition for what will happen
- pH 6.3 and 5°C (41°F): Present-day condition
- pH 6.0 and 25°C (77°F): Conservative scenario.

The kinetics were also checked for a longer underdrain retention time of 30 days to represent a much slower underdrain flow rate that could occur if the tailings were capped (dry cover) and if the only water to run through was runoff. The analyses were repeated with the same pH level and temperature values. The results from both sets of analyses are presented in Table 5-5 and Appendix D.

Table 5-5 – Underdrain Plugging Time due to Chemical Precipitation

pH	Temperature (^{0}C)	Underdrain Plugging Time (years)	
		Retention time = 1 day	Retention time =30 days
5	5	15,330,000	280,000
6	5	160,000	3,000
6.3	5	40,000	980
6	25	16,500	610

Some problems were encountered with the geochemical modeling program, PHREEQC. It was not determined how to limit the aeration level to 10 percent. The program ignored that only a certain amount of air was available for equilibration, and allowed 100 percent aeration of the underdrain water.

Therefore, the thermodynamic conclusions are likely conservative. However, the thermodynamics were used to estimate which precipitates might form. The analysis was mostly based on kinetic considerations. The reduced aeration of 10 percent was included in the Excel spreadsheet for kinetics, so the results are not significantly affected by this shortcoming.

Data provided by TCAK on July 9, 2007 indicates that typical reclaim water has a pH of 4.9.

Table 5-6 –Typical Reclaim Water Chemical Properties

Sample	ppm Pb	ppm Zn	ppm Fe	ppm Cd	ppm TDS	pH	Comment/Location Description
Pond Side Sump	0.59	1366	167.8	9.62	8390	3.68	During curtain wall construction (P-05-65)
Mill Side of Excavation	<0.078	1567	0.28	4.71	8450	4.76	Taken from mill side wall of excavation near panel C5/C6
Pond & Mill Mix	0.09	1794	18.74	6.46	9500	4.84	Taken from low point near panel C6 in curtain wall excavation
Mill Side	2.75	5035	1.36	23.00	20170	4.90	Taken from wall of excavation near Sta. 107+60 on the mill side
Reclaim	2.5	420	0.6 - 2	4	4600	$4.9^{+/-}$	Typical reclaim concentrations

5.3.2.4 Precipitation Potential in the Underdrain

The geochemical modeling using limited pumpback water chemistry shows that iron hydroxides are likely to precipitate in the underdrain. Based on the physical characteristics and properties of the underdrain, the kinetics of ferrous to ferric iron oxidation, and the ferric iron hydrolysis to ferric iron hydroxide, it can be assumed that the underdrain will eventually plug.

The amount of time for plugging to occur depends on the water pH and retention time in the underdrain. Reasonable assumptions result in a plugging time of 160,000 years. A conservative scenario results in a plugging time of 16,500 years. These conclusions have not taken into account geochemical changes to the tailings, tailings water, pore water or underdrain water quality over time. Additional data on long-term geochemical changes are required to increase the degree of certainty in the predicted times.

Alternately, if the tailings impoundment is capped, it could be expected that the underdrain would plug at a faster rate, due to the longer retention time in the underdrain. A reasonable set of assumptions results in a plugging time of 3,000 years if the tailings pond were capped. A conservative scenario results in a plugging time of 980 years.

5.3.3 Riser Pipe Integrity

A backup seepage collection system was included in the dam to handle any excess freeze-up in the center part of the dam that could block the flow in the underdrain. Two riser pipes were included on the upstream face along the interface between the random rockfill and processed filter material. They are referred to as the west and east riser pipes.

Both riser pipes were extended to the dam crest with the construction of each raise. The extension of the riser pipes for Stage VII-B raise was accomplished using 12-inch outside diameter pipe, which is similar to the existing pipes. The west riser pipe was found to be blocked during an attempt to measure the depth to water as described in the dam history report (URS 2007a).

The riser pipes are set in the toe-drain at the deepest part of the original channel, which is less likely to be affected by thaw settlement from permafrost degradation due to the presence of the thaw bulb along the original stream. However, due to the curved alignment of the stream beneath the dam footprint, settlement due to thaw settlement of the foundation is expected along the entire alignment.

Also, due to the presence of the impoundment on the upstream end, the rate of permafrost degradation on the downstream side of the dam is expected to be different. Therefore, differential settlements of the dam along various cross-sections are to be anticipated and were observed during periodic dam safety inspections (Dames & Moore 1994; Dames & Moore 1998; and URS, 2003).

There was a concern that the blockage in the west riser pipe may have been caused by a failure of the pipe due to differential settlement of the embankment. A downhole camera survey was conducted in 2006 to investigate the blockage and determine the condition of the pipe. The survey report is in Appendix E, and the findings of the survey are described in the dam history report (URS 2007a). There was no evidence of any deformation of the pipe and therefore no evidence of differential settlement.

Future changes to the permafrost will depend on the future construction activities around the dam. Likely changes include the raises to closure, a modified seepage collection system, and tailings management such as the width of the tailings beach and method of cover over the tailings impoundment at closure.

The condition of the riser pipe is not likely to be further impacted by differential settlement due to thaw settlement. Based on the recent instrumentation data for the current configurations, the permafrost degradation beneath the dam has either stopped or significantly decreased. With the presence of a wide tailings beach which insulates the subsurface, further significant permafrost degradation is unlikely.

5.4 EMERGENCY SPILLWAY

The emergency spillway is described in Section 3.2.7. It will be cut in bedrock at the left abutment of the dam. The long term integrity of the spillway depends mainly on the durability of the outlet channel. The geotechnical investigation of the spillway (URS 2006) confirms that the highly weathered rock along the channel alignment will need to be over-excavated and replaced by more competent riprap to provide a long-term durable outlet channel.

5.5 LONG-TERM TAILINGS MAIN DAM PERFORMANCE

Tailings dams are unique because the service life of the dam is infinite, generally speaking (ADWR 2005a). While the preceding sections have demonstrated long-term performance of the main elements of the tailings main dam, the purpose was not to demonstrate infinite design life. The life expectancy of each of the main elements of the tailings main dam are presented in Table 5-7.

Table 5-7 – Major Tailings Main Dam Elements Life Expectancy

Dam Element	Life Expectancy (years)	
	Minimum	Maximum
Rockfill	Indefinite[1]	
HDPE Geomembrane	500	1600
Geotextile	220	465
Drainage System	Indefinite[1]	
Underdrain[2]	16,500	160,000
Riser Pipe	Indefinite[1]	
Emergency Spillway	Indefinite[1]	

[1] Indefinite means undefined life expectancy.
[2] Based on wet cover, conservative and reasonable set of assumptions scenario.

The life expectancy of the tailings main dam is related to the life expectancy of its various elements. The quantifiable life expectancies of the elements range from 220 years for the geotextile to 160,000 years for the underdrain. This long service life will allow for possible modifications to the tailings facility resulting from new technology which might allow for reprocessing tailings that are currently considered not economic to process. The elements with indefinite life expectancy are expected to outlive the elements with quantifiable life expectancy, hence they are not considered to limit the life expectancy of the dam.

Section 2.2 identifies the tailings facility as requiring active care for the water cover option because the water treatment plant will be maintained after closure. Active care can only be achieved by continued monitoring and maintenance activities after closure. A combination of the long-term design performance of tailings main dam components and active care demonstrate that a system is in place under which stability of the tailings main dam can be maintained indefinitely.

The same soil, rock and geosynthetic materials have been used in the tailings main through all its stages of construction, have been assumed for the future raises to closure. However at the time of final design, it will be prudent to investigate if other materials may be more appropriate. The consideration of new materials will especially apply to geosynthetics where technology has advanced significantly in the last decade, and new types of geomembranes and geotexiles may be available that may better serve the long term needs of a the tailings main dam at closure.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The conceptual design of the future raises to closure assumes the scenario of a water cover over the entire tailings impoundment. The water cover will require permanent maintenance of a minimum of two feet of water over the tailings, and active on site care, including a water treatment plant. At El. 986, the tailings impoundment is estimated to cover an area of approximately 725 acres.

The future raises to closure require raising the embankment by 26 feet from the Stage VII-B crest of El. 960 to a final crest at El. 986 and ultimate maximum dam height of 208 feet, extending the dam out as a wing wall either south or southeast of the east abutment where the ground flattens out, and constructing a permanent spillway around the west abutment hillside with a spillway crest at El. 984.5.

The final dam crest at El. 986 and spillway crest at El. 984.5 may change as the closure plan is being developed for the tailings impoundment, as tailings production and storage area projections are updated, and as the disposition of various impacted waters are finalized. However, any crest level changes will have minimal impact to the conceptual designs described in this report.

The conceptual design assumes that the dam will have three raises (Stages VIII, IX and X) of heights 10, 10 and 6 feet, respectively. The raises will increase the crests to El. 970, 980 and 986 feet, and maximum height to 192, 202 and 208 feet, respectively. The raises will be built by downstream construction. The Stage X downstream toe is projected to terminate near the upstream end of the seepage collection pond.

The final dam will include the embankment and a wing wall out from the right abutment. The wall will be built in the same way as the embankment so that they will comprise one dam with continuous seepage and drainage control systems, and a bend at the embankment to wall connection. There are several possible bend and wing wall alignments and configurations that will be resolved in the final designs.

The main embankment raises and most of the wing wall will have upstream slopes of 2.5 horizontal to 1.0 vertical (2.5:1) and downstream slopes of 3:1. The upstream slopes will be the same as the upstream slopes of the previous raises. The downstream slopes will the same as the Stage VII-B downstream slope. The embankment to wall bend slopes may need to be steeper because of space constraints.

The emergency spillway includes a channel around the left abutment. The tailings impoundment at closure will contain all inflows up to the spillway crest at El. 984.5 in order to maintain a zero-discharge operation. In the improbable event that the impounded water exceeds El. 984.5, the spillway is sized to route the probable maximum flood (PMF) through its channel to a maximum water level of El. 985.6.

The long-term integrity and longevity of the dam depends on the durability of the embankment rockfill, liner and drainage systems, and spillway. Some components of these systems were found to have an indefinite or undefined life expectancy. The life expectancies of the major elements are as follows:

Dam Element	**Life Expectancy (years)**	
	Minimum	**Maximum**
Rockfill	Indefinite	
HDPE Geomembrane	500	1600
Geotextile	220	465
Dam Drainage System	Indefinite	
Underdrain	16,500	160,000
Dam Riser Pipe	Indefinite	

The rockfill durability and ability to handle movement caused by thaw consolidation and seismic-induced foundation and seismic deformation is critical to long-term dam stability. The rockfill is inert rock from the DD-2 quarry and the Main Pit, is durable and not reactive, and will remain intact for centuries. Thaw consolidation of the foundation has mostly occurred, and any further thaw will have minimal impact.

A seismic deformation was estimated corresponding to an earthquake of maximum magnitude that has a likelihood of occurring once in 2,475-years. The deformation analysis estimated a maximum horizontal

displacement of 21 inches at the crest and maximum shear strain in the dam of up to 0.2 percent. These levels of earthquake-induced displacement and shear strain are tolerable under such an extreme event.

The geomembrane is the key element of the liner system for seepage control. To prevent brittle cracking during cold weather, it is estimated that geomembrane sheets (23 feet by 300 feet) need to have 3.6 and 46.2 inches of slack on the short and long sides, respectively. It was found that the geomembrane could safely accommodate gradual deformations and abrupt deformation induced by the design earthquake.

A drainage system analysis showed that piping will not occur. A liner system analysis showed that the filter soil does not satisfy the "no erosion" criteria, so that if the liner lost its effectiveness, the tailings will erode and not seal the filter soil. However, the low hydraulic gradient in the dam will reduce the potential for internal erosion, and any erosion that could occur will only be in the liner system.

It was estimated that it would take 16,500 to 160,000 years for the underdrain to become plugged as a result of secondary mineral precipitation from tailings water draining through it. Additional analyses of the long-term geochemical changes are required to increase the confidence of the predicted times.

The same soil, rock and geosynthetic materials were used in the dam during all stages of construction, and were assumed for the future raises. However, the final design of each raise should investigate if other materials are more appropriate. This should especially apply to geosynthetics where technology has advanced significantly and new products may better serve the long term needs of the future raises.

The conceptual design shows that the dam can be constructed and operated safely during the remaining mining operations to at least 2031, that long-term dam stability and seepage controls can be maintained in perpetuity after closure, and that post closure operational and maintenance requirements can be achieved.

7.0 REFERENCES

Alaska Test Lab, 2005, "Grain Size Distribution Curve for Red Dog Airport Project", for Teck Cominco Alaska, September 2005.

Alaska Testlab (ATL), 2004, "Letter: Red Dog Mine Airport Paving", for Teck Cominco Alaska, Inc. June 14, 2004.

Alaska Department of Natural Resources (ADNR), 2005a, "Guidelines for Cooperation with the Alaska Dam Safety Program." Dam Safety and Construction Unit, Water Resources Section Division of Mining, Land and Water, June 30, 2005.

Alaska Department of Transportation (ADOT), 2004, Standard Specifications for Highway Construction. Alaska Department of Transportation and Public Facilities. January 9, 2004.

ADNR, 2005b, "Multiple Accounts Analysis for Red Dog Tailings Disposal." Dam Safety and Construction Unit, Water Resources Section Division of Mining, Land and Water, June 2, 2005.

ASTM, 2007, Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate (C88-05). ASTM International, West Conshohocken, PA. August 16, 2007.

BGC Engineering Inc., 2001, “Red Dog Mine, Cominco Alaska, Tailings Audit.” February 9, 2001.

Bowles, J., 1996, “Foundation Analysis and Design”, Fifth Ed. McGraw-Hill, 1996.

Crory, F. E., 1973, “Settlement Associated with the Thawing of Permafrost.” proc. 2nd Intern. Conf. Permafrost (Yakutsk, U.S.S.R.), North Amer. Contrib., U.S. Nat. Acad. Sci., Washington, D.C., pp. 599-607, 1973.

Dames & Moore, 1986, "Proposed Tailing Dam Design Report”, for Cominco Alaska, Inc., June 6, 1986.

Dames & Moore, 1987, "Waste Storage Areas Geotechnical Design Report." July 16, 1987.

Dames & Moore, 1994, “Dam Safety Report: Alaska Dam Safety Program, Red Dog Mine Tailings Dam, Alaska”, for Cominco Alaska Inc., December 30, 1994.

Dames & Moore, 1998, “Safety Inspection Report: Red Dog Mine Tailings Dam, Alaska”, for Cominco Alaska Inc., September 4, 1998.

Dames & Moore, 1999, “Stability Analysis, Pipe Bench, Red Dog Tailings Dam, Red Dog Mine, Alaska”, for Cominco Alaska, Inc., August 1999.

Farrar, J., 1993, “Bureau of Reclamation Experience in Testing of Riprap for Erosion Control of Embankment Dams,” Rock for Erosion Control, ASTM sSTP 1177, Charles H. McElroy and David A. Lienhart, Eds, American Society for Testing and Materials, Philadelphia, 1993.

FHWA, 2000, “Corrosion/degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.” Publication No. FHWA-NHI-00-044.

Foster, M. and R. Fell, 2001, “Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria.” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 5, ASCE, May 2001.

Giroud J.P., 2006a, “Personal communications regarding HDPE Liner Leakage and Longevity”, 2006.

Giroud, J.P., 2006b, “Draft Memorandum: Red Dog Tailings Dam, Geomembrane Durability, Comments.” May 24, 2006.

Giroud, J.P., 2006c, “Draft Memorandum: Red Dog Tailings Dam, Geomembrane Durability, Additional Comments.” June 2, 2006.

Golder Associates, 2003, "Detailed Design, Tailings Impoundment - Back Dam, Red Dog Mine, Alaska." June 27, 2003.

ICOLD, 2001, “Tailings Dams Risk of Dangerous Occurrences: Bulletin 121” by ICOLD Committee on Tailings Dams and Waste Lagoons (1995-2001)

Koerner, R.M., and Y.G. Hsuan, 2003, “Lifetime Prediction of Polymeric Geomembranes Used in New Dam Construction and Dam Rehabilitation.” Proceedings of the Association of State Dam Safety Officials Conference, Lake Harmony, Pennsylvania, June 4-6, 2003.

Koerner, R.M., Y.G. Hsuan and G.R. Koerner, 2005, "Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions." GRI White Paper # 6, Geosynthetic Institute, June 7, 2005, 19 p.

Koerner, R.M., 2006, "Personal communications regarding HDPE Liner Longevity".

Lide, D. R., 1991/1992, "Hand Book of Chemistry and Physics", 72nd edition, 1991-1992.

Martin, J. R. and Gardner, R. J., 1983 "Use of plastics in Corrosion Resistant Instrumentation", paper presented at 1983 Plastic Seminar, NACE, Dallas, Texas, October 24-27, 1983.

Peggs, I.D., 2003, "Geomembrane Liner Durability: Contributing Factors and the Status Quo." http://www.geosynthetica.net/tech_docs/IDPigsUKpaper.pdf.

Peggs, I.D., J.P. Winfree and J.P. Giroud, 1990, "A Shattered Geomembrane Liner Case History: Investigation and Remediation." http://pgi-tp.cee.uiuc.edu.

Reddy, D.V, and B. Butul, 1999, "A Comprehensive Literature Review of Liner Failure and Longevity Research Report." Florida Center for Solid and Hazardous Waste Management, University of Florida, July 1999, p. 146.

Rowe, R.K. and H.P. Sangam, 2002, "Review Article: Durability of HDPE Geomembranes." Geotextiles and Geomembranes, pp. 77-95.

Rowe, R.K., 2005, "Long-term Performance of Contaminant Barrier Systems." Geotechniques 55, No. 9, pp. 631-678.

SRK Consulting Inc., 2004, "Memo on Red Dog HDPE Geomembrane Liner Longevity Research." p. 02.

SRK Consulting Inc., 2006, "Red Dog Mine Closure Options: Volume 1 – Tailings", for Teck Cominco, March 2006.

SRK 2007, "Plan of Operations for Tailings and Water Management" for Teck Cominco Alaska Inc.

SI Geosolutions, 2006, "Product Data Sheet of Geotex 1001 and Geotex 1601."

SI Geosolutions, 1997, "The Durability of Polypropylene Nonwoven Geotextiles for Waste Containment Applications." Technical note SM-404.

Teck Cominco Alaska (TCAK), 2007, Formations Likely to be Encountered at Red Dog Deposit", from Teck Cominco Alaska correspondence with Tom Krolak, Senior Geologist, April 2007.

USBR, 1987, "Design of Small Earth Dam." United States Department of the Interior - Bureau of Reclamation Third Edition, 1987.

USBR, 1998, "Earth Manual." United States Department of the Interior - Bureau of Reclamation Third Edition, 1998.

URS, 2001, “Report, Slope Stability Evaluation, Tailings Dam and Tailings Beach Dust Control Structure, Red Dog Mine, Alaska”, for Cominco Alaska, Inc., August 7, 2001.

URS, 2003, “Dam Safety Inspection Report, Red Dog Mine, Alaska”, for Teck Cominco Alaska, October 2003.

URS, 2004, “Review of Red Dog Mine Tailings Audit Report by BGC Engineering Inc.”, for Teck Cominco Alaska, October 22, 2004.

URS, 2005, “Design Modification, Numbers 1 through 10: Tailings Main Dam, Stage VII-B Construction, Red Dog Mine, Alaska”, for Teck Cominco Alaska, July through November, 2005.

URS, 2006a, “Technical Memorandum: Conceptual Design of Spillway for Tailings Main Dam at Closure, Red Dog Mine, Alaska”, for Teck Cominco Alaska, March 17, 2006.

URS, 2006b, “Geotechnical Investigation Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska”, for Teck Cominco Alaska, July 28, 2006.

URS, 2006c, “Design Modification, Numbers 11 through 24: Tailings Main Dam, Stage VII-B Construction, Red Dog Mine, Alaska”, for Teck Cominco Alaska, June through October, 2006.

URS Corporation, 2007a, “Dam History Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska”, for Teck Cominco Alaska, February 26, 2007.

URS, 2007b, “Seepage Analysis Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska”, for Teck Cominco Alaska, February 26, 2007.

URS, 2007c, “Stability Analysis Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska”, for Teck Cominco Alaska, April 13, 2007.

URS, 2007d, “Construction Completion Report, Tailings Main Dam, Red Dog Mine, Alaska”, for Teck Cominco Alaska, October 19, 2007.

USGS, 1999, “PHREEQC: A Computer Program for Speciation, Batch-Reaction, One-Dimensional Transport, and Inverse Geochemical Calculations”, freeware, Version 2, http://wwwbrr.cr.usgs.gov/projects/GWC_coupled/phreeqc/index.html

USGS, 1996, “Alaska Resources Data File, DeLong Mountains Quadrangle.” Open File Report No. 00-023, Anchorage, AK, 1996. http://ardf.wr.usgs.gov

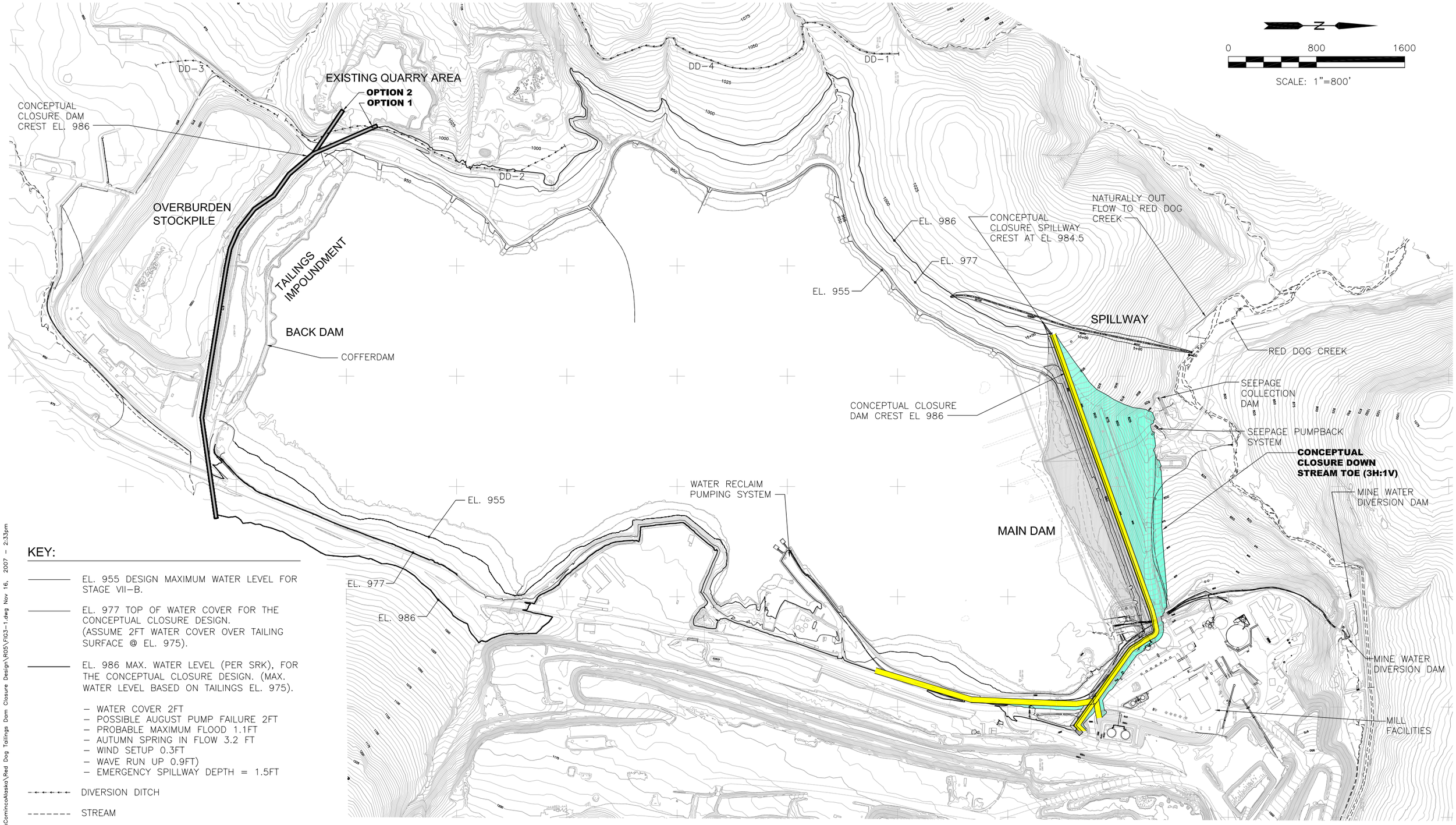
Van Zyl, D. and Harr, M.E., 1981, “Seepage Erosion Analysis of Structures.” Proc., 10th International Conference on Soil Mechnics and Foundation Engineering, Stockholm, Sweden, Vol. 1, pp. 503-509.

Viebke, J., F. Elble, M. Ifwarson and U.W. Gedde, 1994, “Degradation of Unstabilized Medium-density Polyethylene Pipes in Hot-water Applications.” Polymer Engineering and Science, pp. 1354-1361.

Waston, G. H., W. A. Slusarchuk, and R. K. Rowley, 1973, "Determination of Some Frozen and Thawed Properties of Permafrost Soils," Can. Geotech. J. 10:592-606, 1973.

Yanful, E.K., and Mustafa A. Samad, 2004, "Hydraulic Design of Tailings Water Cover at Red Dog Waste Management Area." University of Western Ontario, June 2004.

Figures



KEY:

- EL. 955 DESIGN MAXIMUM WATER LEVEL FOR STAGE VII–B.
- EL. 977 TOP OF WATER COVER FOR THE CONCEPTUAL CLOSURE DESIGN. (ASSUME 2FT WATER COVER OVER TAILING SURFACE @ EL. 975).
- EL. 986 MAX. WATER LEVEL (PER SRK), FOR THE CONCEPTUAL CLOSURE DESIGN. (MAX. WATER LEVEL BASED ON TAILINGS EL. 975).
  - WATER COVER 2FT
  - POSSIBLE AUGUST PUMP FAILURE 2FT
  - PROBABLE MAXIMUM FLOOD 1.1FT
  - AUTUMN SPRING IN FLOW 3.2 FT
  - WIND SETUP 0.3FT
  - WAVE RUN UP 0.9FT)
  - EMERGENCY SPILLWAY DEPTH = 1.5FT
- DIVERSION DITCH
- STREAM

Job No. 33757098

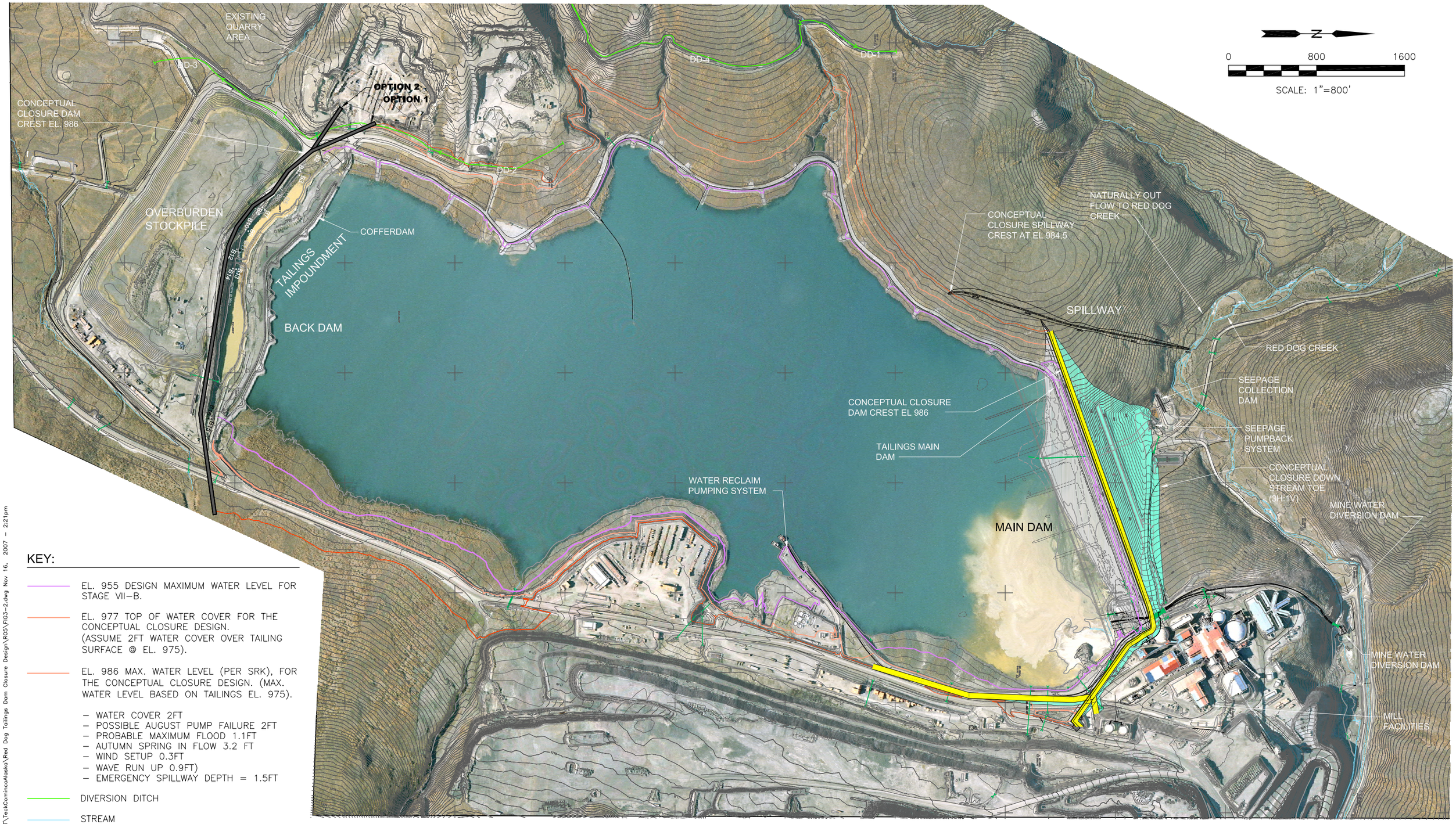
Figure 3-1

Tailings Impoundment Plan View at Closure

Conceptual Closure Design of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS

P:\ACAD\PROJECT\TechCominco Alaska\Red Dog Tailings Dam Closure Design\ROS\FIG3-1.dwg Nov 16, 2007 - 2:33pm



KEY:

- EL. 955 DESIGN MAXIMUM WATER LEVEL FOR STAGE VII–B.
- EL. 977 TOP OF WATER COVER FOR THE CONCEPTUAL CLOSURE DESIGN. (ASSUME 2FT WATER COVER OVER TAILING SURFACE @ EL. 975).
- EL. 986 MAX. WATER LEVEL (PER SRK), FOR THE CONCEPTUAL CLOSURE DESIGN. (MAX. WATER LEVEL BASED ON TAILINGS EL. 975).
  - WATER COVER 2FT
  - POSSIBLE AUGUST PUMP FAILURE 2FT
  - PROBABLE MAXIMUM FLOOD 1.1FT
  - AUTUMN SPRING IN FLOW 3.2 FT
  - WIND SETUP 0.3FT
  - WAVE RUN UP 0.9FT)
  - EMERGENCY SPILLWAY DEPTH = 1.5FT
- DIVERSION DITCH
- STREAM

P:\ACAD\PROJECT\TeckCominco\Alaska\Red Dog Tailings Dam Closure Design\R05\Fig3-2.dwg Nov 16, 2007 - 2:21pm

Job No. 33757098

Figure 3-2

Tailings Impoundment Plan View at Closure with Photo Superimposed

Conceptual Closure Design of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

URS

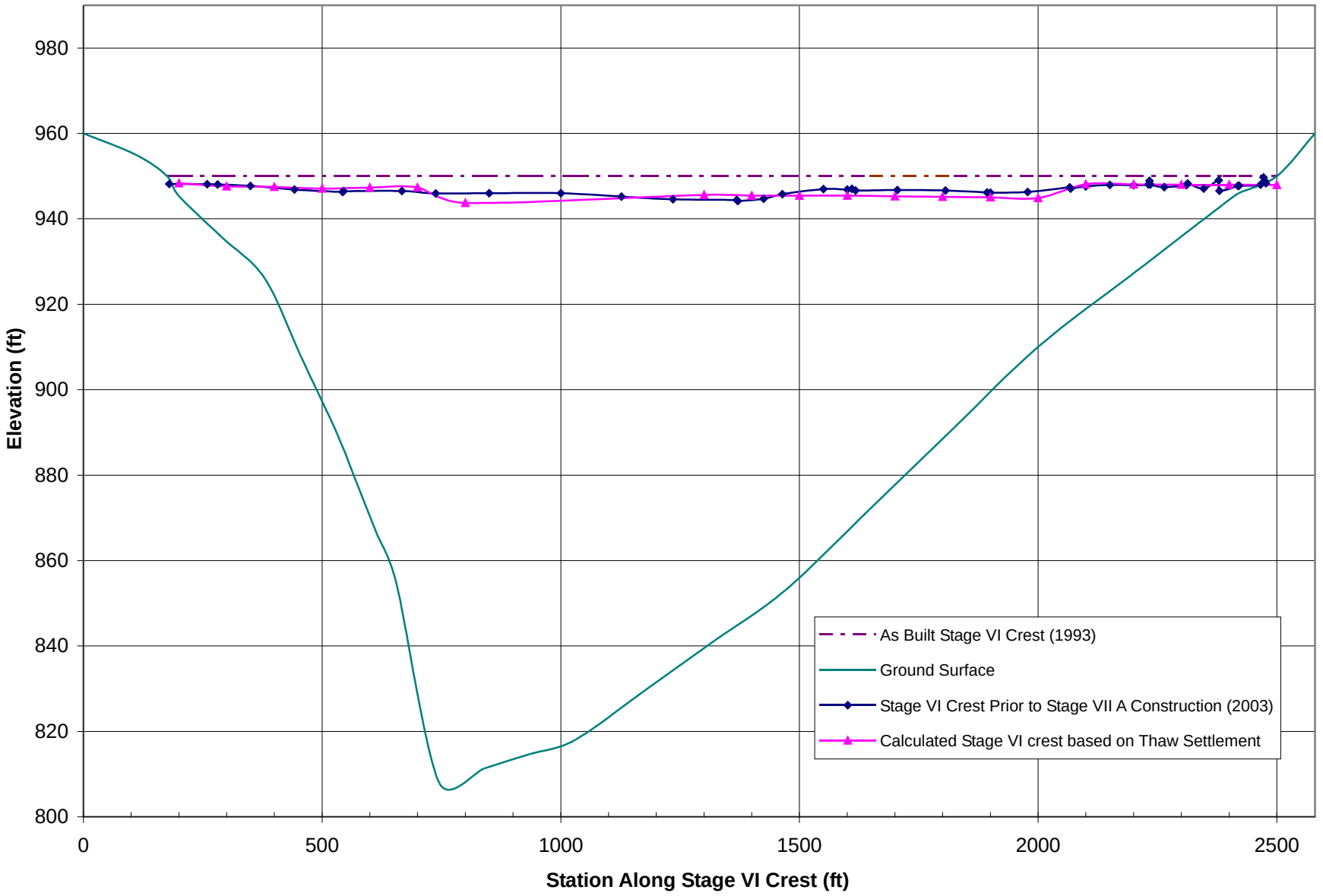
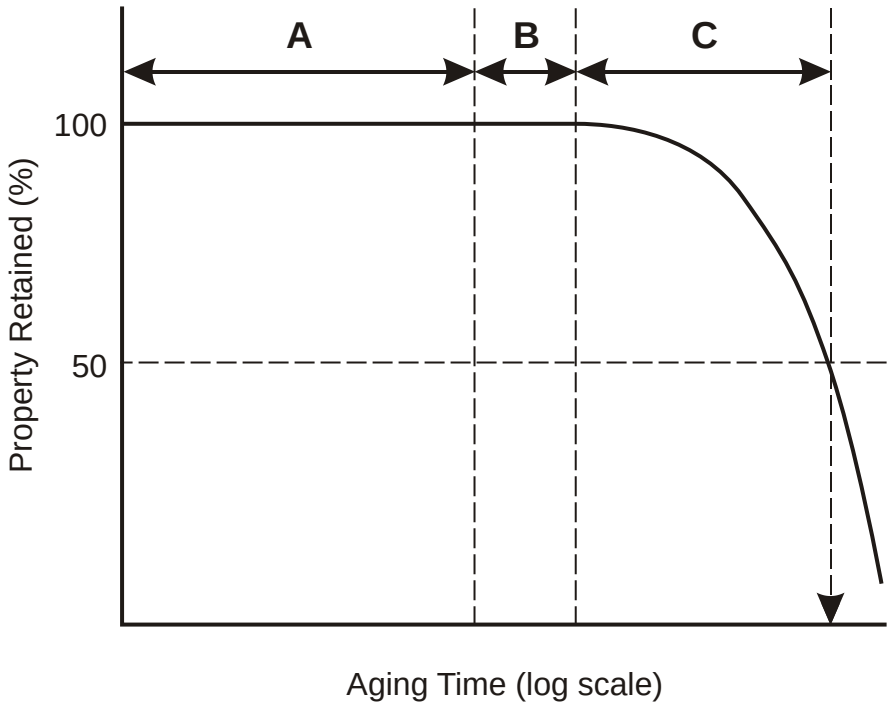


Figure 5-1

Dam Crest Settlement due to Thaw Consolidation

Job No. 33757098

33757098_34.cdr



A = Depletion time of antioxidants

B = Induction time to onset of polymer degradation

C = Time to reach 50% degradation of a particular property

SOURCE:
Modified from Koerner, R.M., Hsuan, Y.G., and Koerner, G.R., "Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions". GRI White Paper #6, Geosynthetic Institute, June 7, 2005.

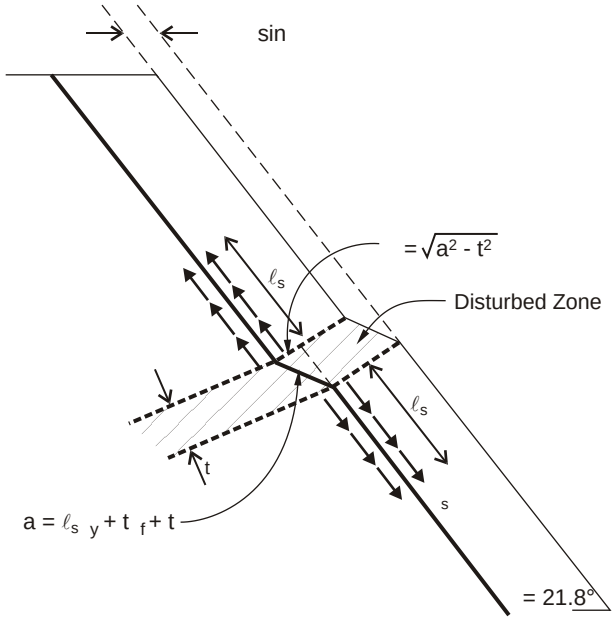
Figure 5-2
Geomembrane Conceptual Stages of Chemical Aging

Job No. 33757098

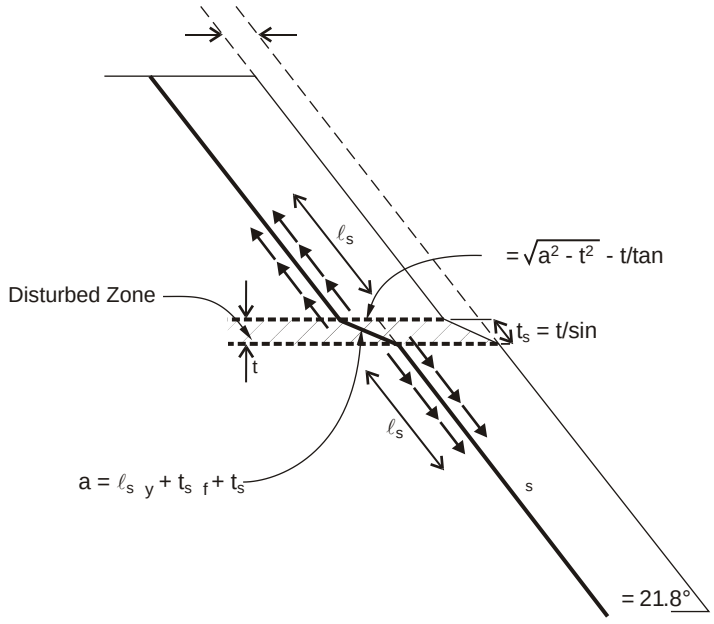
URS

Conceptual Closure Design of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

33757098_29.cdr



(a) Sliding Perpendicular to Liner



(b) Sliding Horizontally

Notes:

1. Failure strain of the geomembrane, $_f = 0.65$
2. Yield strain of the geomembrane, $_y = 0.16$
3. t = thickness of the disturbed zone

Figure 5-3
Geomembrane Abrupt Deformation

Job No. 33757098

URS

Conceptual Closure Design of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

33757098_35.cdr

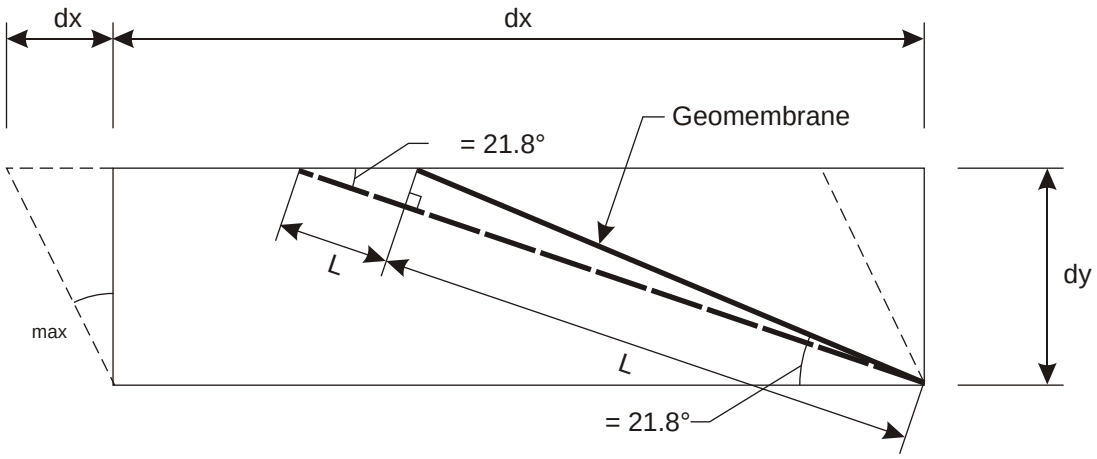


Figure 5-4

Job No. 33757098

Soil Mass and Geomembrane Deformation due to Earthquake

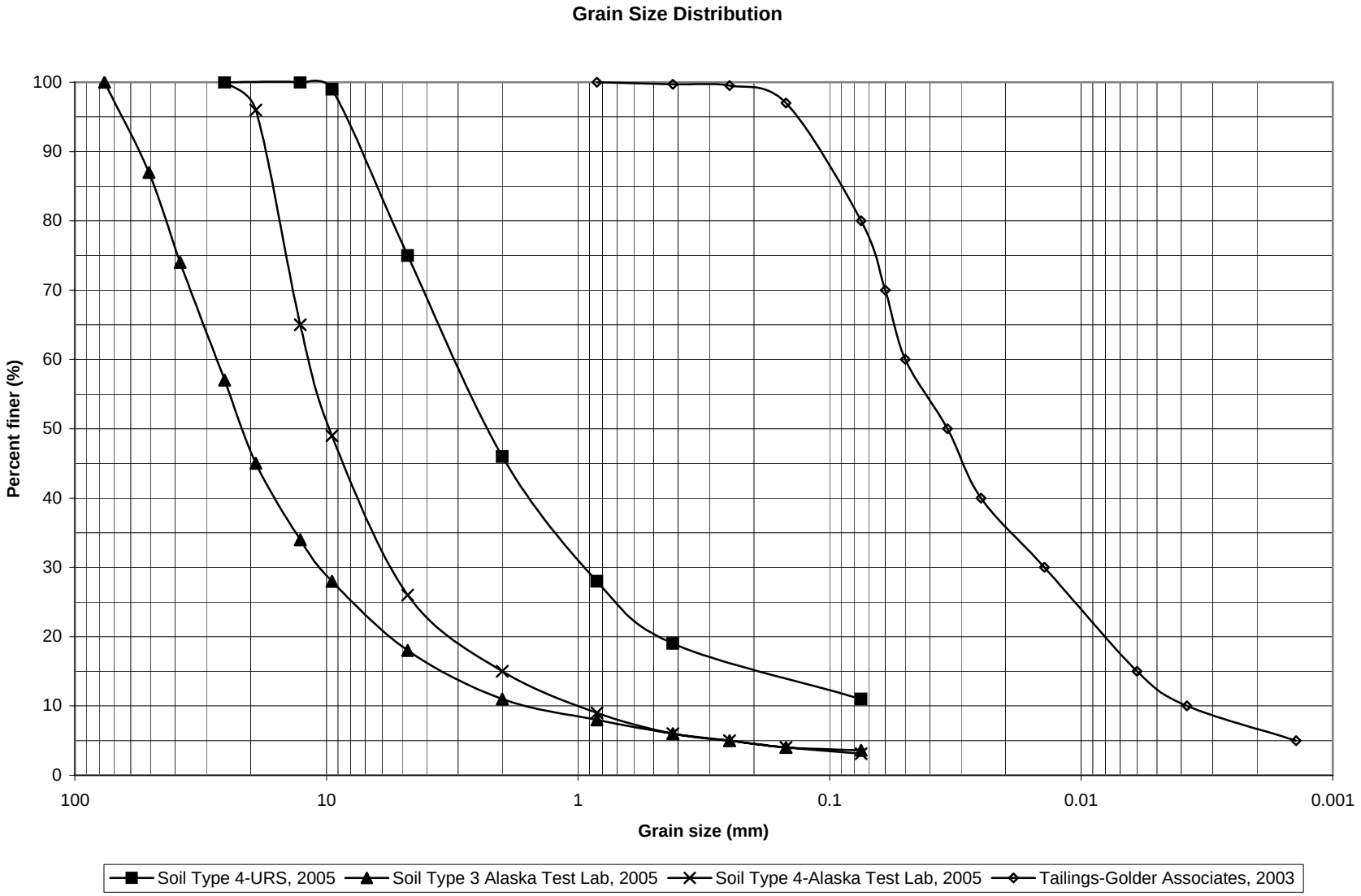


Figure 5-5
Grain Size Distribution

Job No: 33757098

URS

Conceptual Closure Design of Tailings Main Dam
Future Raises to Closure
Red Dog Mine, Alaska

Drawings

PRELIMINARY CONCEPTUAL DESIGN DRAWINGS

FOR

TECK COMINCO ALASKA INC.

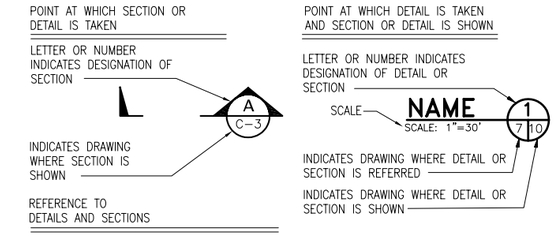
RED DOG MINE TAILINGS MAIN DAM, FUTURE RAISES TO CLOSURE

RED DOG MINE, ALASKA

DRAWING INDEX

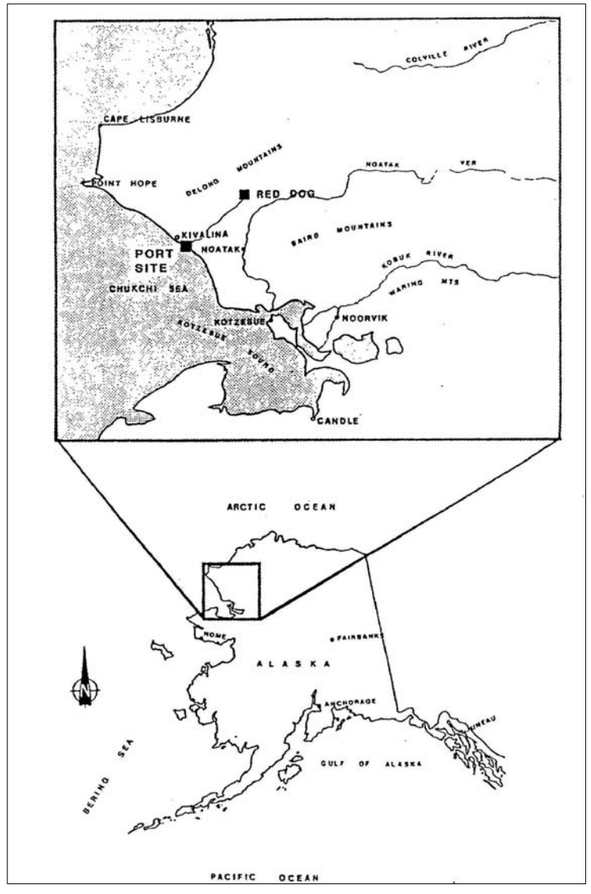
DRAWING NO.	REV. NO.	TITLE
TDC-1	0	TITLE SHEET, INDEX, MAP AND NOTES
TDC-2	0	REFERENCES, LEGEND AND ABBREVIATIONS
TDC-3	0	EXISTING STAGE VII-B DAM PLAN AND PROFILE
TDC-4	0	PROFILE ALONG HIGHEST MAIN DAM SECTION
TDC-5	0	PROFILE ACROSS EMBANKMENT CREST
TDC-6	0	PROFILES ALONG CURTAIN WALL
TDC-7	0	CLOSURE STAGE X DAM PLAN & PROFILE
TDC-8	0	PROFILE ACROSS WING WALL OPTIONS
TDC-9	0	PROFILE ALONG WING WALL OPTIONS 4 & 5
TDC-10	0	PROFILE ALONG WING WALL OPTION 6
TDC-11	0	BEND ALTERNATIVE AND WING WALL OPTION PLANS
TDC-12	0	STAGE VIII RAISE CONCEPTUAL PLAN AND PROFILE
TDC-13	0	STAGE VIII RAISE CONCEPTUAL SECTIONS
TDC-14	0	STAGE VIII RAISE EMBANKMENT TO WING WALL BEND
TDC-15	0	STAGE IX RAISE CONCEPTUAL PLAN AND PROFILE
TDC-16	0	STAGE IX RAISE CONCEPTUAL SECTIONS
TDC-17	0	STAGE IX RAISE EMBANKMENT TO WING WALL BEND
TDC-18	0	STAGE X RAISE CONCEPTUAL PLAN AND PROFILE
TDC-19	0	STAGE X RAISE CONCEPTUAL SECTIONS
TDC-20	0	STAGE X RAISE EMBANKMENT TO WING WALL BEND
TDC-21	0	SPILLWAY CONCEPTUAL PLAN AND PROFILE
TDC-22	0	SPILLWAY CONCEPTUAL CREST AND STILLING BASIN DETAILS
TDC-23	0	SPILLWAY CONCEPTUAL SECTIONS AND DETAILS
TDC-24	0	SPILLWAY REACH 1 CONCEPTUAL PROFILE
TDC-25	0	SPILLWAY REACH 2 & 3 CONCEPTUAL PROFILES

SECTION AND DETAIL DESIGNATION



ARE ALSO DESIGNATED BY X/A-Y, WHERE X IS A NUMBER FOR DETAILS AND A LETTER FOR SECTIONS, AND A-Y IS THE DRAWING ON WHICH THE SECTION OR DETAIL APPEARS

VICINITY MAP



GENERAL NOTES

1. THESE DRAWINGS ARE PRELIMINARY CONCEPTUAL DESIGN DRAWINGS THAT ARE PART OF A PRELIMINARY CONCEPTUAL DESIGN REPORT FOR THE TAILINGS MAIN DAM FUTURE RAISES TO CLOSURE. THESE DRAWINGS ARE NOT FOR CONSTRUCTION AND ARE TO BE USED IN THE CONTEXT OF THE PRELIMINARY CONCEPTUAL DESIGN REPORT. FINAL DESIGN DRAWINGS WILL BE PREPARED FOR THE APPROVAL TO CONSTRUCT EACH FUTURE RAISE.

2. THE TOPOGRAPHIC MAP WAS COMPILED TO MEET HORIZONTAL AND VERTICAL ACCURACY IN ACCORDANCE WITH AMERICAN SOCIETY FOR PHOTOGRAMMETRY AND REMOTE SENSING (ASPRS) CLASS I STANDARDS. AREAS DENOTING VEGETATION COVER ON THE GROUND SHOULD BE CONSIDERED LESS ACCURATE AND NOT USED FOR CONSTRUCTION UNTIL FIELD CHECKED IN ACCORDANCE WITH ASPRS STANDARDS. THIS MAP PROJECTION IS BASED UPON NORTH AMERICAN DATUM OF 1927 (NAD27), ALASKA STATE PLANE ZONE 7 AS EXPRESSED IN U.S. SURVEY FEET.

3. FOR THE CONCEPTUAL DESIGN, THE FUTURE RAISES OF THE TAILINGS MAIN DAM TO CLOSURE WERE ASSUMED TO CONSIST OF THREE STAGES OF CONSTRUCTION AS FOLLOWS:

   a) STAGE VIII-A 10-FOOT HIGH RAISE FROM CREST EL. 960 TO CREST EL. 970.
   b) STAGE IX-A 10-FOOT HIGH RAISE FROM CREST EL. 970 TO CREST 980.
   c) STAGE X-A 6-FOOT HIGH RAISE FROM CREST EL. 980 TO CREST 986.

4. THE NUMBER AND HEIGHTS OF THE RAISES TO EL. 986 COULD BE CHANGED DEPENDING ON MINE OPERATION NEEDS. ANY CHANGES WOULD HAVE NO IMPACT ON THE CONCEPTUAL DESIGN INTENT, AND MINIMAL IMPACT TO THE CONCEPTUAL DESIGN DETAILS.

5. THE FUTURE RAISES AND FINAL DAM CONFIGURATION WILL INCLUDE THE EMBANKMENT AND A WING WALL OUT FROM THE RIGHT ABUTMENT OF THE EMBANKMENT. THE EMBANKMENT AND WALL WILL BE OF SIMILAR CONSTRUCTION AND WILL COMPRISE ONE DAM WITH CONTINUOUS SEEPAGE AND DRAINAGE CONTROL SYSTEMS AND A BEND AT THE EMBANKMENT TO WALL CONNECTION. THERE ARE SEVERAL POSSIBLE BEND AND WALL ALIGNMENTS AND CONFIGURATIONS THAT WILL BE RESOLVED IN THE FINAL DESIGNS OF EACH RAISE.

6. EACH RAISE WILL REQUIRE THE FOLLOWING WORK:

   a) WIDEN THE EXISTING DAM TO PROVIDE A FOOTPRINT FOR THE RAISE, CONSTRUCT THE RAISE TO THE CREST ELEVATION, AND EXTEND THE LINER SYSTEM ON THE UPSTREAM PART OF THE RAISE.
   b) EXTEND THE PRIMARY SEEPAGE CONTROL SYSTEM BY EXTENDING THE EXISTING SEEPAGE CUTOFF SYSTEM ALONG THE WEST ABUTMENT OF THE MAIN DAM EMBANKMENT AND ALONG THE EAST EXTENSION OF THE WING WALL.

7. ALL CONSTRUCTION MATERIAL SOIL TYPES ON THE DOWNSTREAM SIDE OF THE DAM AND ABOVE THE TOP ELEVATION OF THE LINER SHALL BE NON-"MINERALIZED". "MINERALIZED" MATERIAL CAN BE USED FOR CONSTRUCTION ON THE UPSTREAM SIDE OF THE LINER BELOW THE TOP ELEVATION OF THE LINER.

8. THE SAME SOIL, ROCK AND GEOSYNTHETIC MATERIALS WERE USED IN THE DAM DURING ALL PREVIOUS STAGES OF CONSTRUCTION, AND WERE ASSUMED FOR THE FUTURE RAISES IN THE CONCEPTUAL DESIGN. HOWEVER, THE FINAL DESIGN OF EACH RAISE WILL INVESTIGATE IF OTHER MATERIALS ARE MORE APPROPRIATE, ESPECIALLY FOR GEOSYNTHETICS WHERE TECHNOLOGY HAS ADVANCED AND NEW PRODUCTS MAY BE MORE SUITABLE.

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE	REVISIONS NO.	BY	DATE	DESCRIPTION	REVISIONS NO.	BY	DATE	DESCRIPTION
		1				1			
		2				2			
		3				3			
		4				4			
		5				5			

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

TITLE SHEET, INDEX, MAP AND NOTES

JOB NO. 33757098.00001

DRAWING NO. TDC-1 REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-1.dwg Nov 16, 2007 - 3:18pm

REFERENCE DRAWINGS

CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGES I–VI, BY DAMES & MOORE (NOW URS) FOR COMINCO ALASKA INC, RED DOG MINE, ALASKA, 1986, WITH REVISIONS THROUGH 1995.

DRAWING NO.	REV. NO.	TITLE
T–1	5 (FEB 95)	TITLE SHEET, INDEX AND VICINITY MAP
T–2	1 (FEB 95)	GENERAL DATA SHEET
T–3	0 (JUNE 86)	BORING LOGS
T–4	0 (JUNE 86)	BORING LOGS
T–5	3 (FEB 95)	TAILINGS IMPOUNDMENT – GENERAL SITE PLAN
T–6	6 (FEB 95)	TAILINGS DAM – PLAN AND PROFILE
T–7	6 (FEB 95)	TAILINGS DAM – SECTIONS
T–8	1 (JAN 87)	STARTER DAM – PLAN
T–9	1 (JAN 87)	STARTER DAM – SECTION AND DIVERSION PIPE PROFILE
T–10	1 (JUN 86)	TAILINGS DAM – EXCAVATION PLAN AND PROFILE
T–11	0 (AUG 89)	TAILINGS DAM – EXCAVATION SECTIONS
T–12	1 (AUG 89)	TAILINGS DAM – EXCAVATION SECTIONS
T–13	1 (JAN 87)	TOE DRAIN AND DIVERSION PIPE DETAILS
T–14	2 (JAN 88)	PIPE TRENCH DETAILS
T–15	3 (JAN 88)	LINER ANCHOR DETAILS
T–19	3 (AUG 89)	TAILINGS DAM – MISCELLANEOUS DETAILS
T–20	1 (JAN 87)	EMERGENCY SPILLWAY – PLAN, PROFILE AND SECTION
T–20A	2 (DEC 94)	SPILLWAY – SECTION AND DETAILS
T–21	5A (NOV 95)	PROPOSED INSTRUMENT LOCATIONS (AS BUILT)
T–21F	2 (MAY 89)	TAILINGS DAM – PROPOSED INSTRUMENT LOCATIONS (FIELD REVISIONS)
T–22	6 (MAR 95)	INSTRUMENT INSTALLATION DETAILS
T–22F	3 (MAY 89)	TAILINGS DAM – INSTRUMENT INSTALLATION DETAILS (FIELD REVISIONS)
T–23	1 (AUG 89)	CUTOFF WALL SECTION AND DETAILS

CONSTRUCTION COMPLETION REPORT, STAGE VII–A RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, MARCH 30, 2004.

T7–1	4 (MARCH 04)	TITLE SHEET, INDEX AND VICINITY MAP
T7–2	2 (MARCH 04)	EXISTING TAILINGS DAM PLAN AND PROFILE
T7–3	1 (MARCH 04)	ESTIMATED SUBSURFACE PROFILES
T7–4	4 (MARCH 04)	STAGE VII–A PLAN AND PROFILE (AS BUILT)
T7–6	2 (MARCH 04)	STAGE VII–A SECTIONS (AS BUILT)
T7–9	4 (MARCH 04)	EXCAVATION AND LINER ANCHOR SECTIONS (AS BUILT)
T7–10	2 (MARCH 04)	LINER SECTIONS AND DETAILS (AS BUILT)
T7–11	2 (MARCH 04)	CUTOFF WALL SECTIONS AND DETAILS (AS BUILT)
T7–12	3 (MARCH 04)	RISER PIPE AND PIPE PENETRATION DETAILS (AS BUILT)
T7–13	3 (MARCH 04)	STAGE VII–A LEFT ABUTMENT GRADING PLAN (AS BUILT)
T7–15	3 (MARCH 04)	STAGE VII–A RIGHT ABUTMENT GRADING PLAN (AS BUILT)
T7–17	0 (MARCH 04)	STAGE VII–A GEOMEMBRANE PANEL LAYOUT (AS BUILT)
T7–18	0 (MARCH 04)	STAGE VII–A FINAL SURVEY (1/4) (AS BUILT)
T7–19	0 (MARCH 04)	STAGE VII–A FINAL SURVEY (2/4) (AS BUILT)
T7–20	0 (MARCH 04)	STAGE VII–A FINAL SURVEY (3/4) (AS BUILT)
T7–21	0 (MARCH 04)	STAGE VII–A FINAL SURVEY (4/4) (AS BUILT)
T7–22	0 (MARCH 04)	STAGE VII–A CROSS–SECTIONS (AS BUILT)

CONSTRUCTION COMPLETION REPORT FOR STAGE VII–B, RED DOG TAILINGS DAM BY URS CORPORATION FOR TECK COMINCO ALASKA INC., RED DOG MINE OCTOBER 26, 2007.

DRAWING NO.	REV. NO.	TITLE
T7B–1	4	TITLE SHEET, INDEX AND VICINITY MAP
T7B–2	0	EXISTING TAILINGS DAM PLAN AND PROFILE
T7B–3	4	SUBSURFACE PROFILES
T7B–4	3	STAGE VII–B TAILINGS DAM PLAN AND PROFILE
T7B–5	2	RIGHT ABUTMENT CURTAIN WALL 3D
T7B–6	7	RIGHT ABUTMENT CURTAIN WALL DETAILS
T7B–7	4	RIGHT ABUTMENT CUTOFF WALL
T7B–8	1	LEFT ABUTMENT CUTOFF WALL
T7B–9	2	TAILINGS DAM SECTIONS
T7B–10	1	LINER SECTIONS AND DETAILS
T7B–11	1	RISER PIPES AND INSTRUMENTATION
T7B–12	0	MAIN DAM CREST PANEL LAYOUT
T7B–13	0	MAIN DAM CREST PANEL LAYOUT
T7B–14	0	CURTAIN WALL SUBSURFACE PROFILE
T7B–15	0	AND CONNECTION DETAIL AS–BUILT CURTAIN WALL PANEL LAYOUT

LEGEND

PLAN

CONTOURS (EXISTING)
MAJOR
MINOR

CONTOURS (PROPOSED)
MAJOR
MINOR

LINER (EXISTING)

LINER (FUTURE)

MINOR CONTOUR BELOW

LAKE OR WATER LINE

CREST OF DAM

PUMPBACK PIPE LINE

ROADWAY OR DUST CONTROL STRUCTURE

PIPE

POSSIBLE KIVALINA SHALE

GROUT

DAM CREST

DOWNSTREAM EXTEND

HISTORIC DRAINAGE

FLOW PATH

CURTAIN WALL

CUTOFF WALL

SECTION

GEOMEMBRANE

GEOTEXTILE

ESTIMATED PROFILE

GROUND (EXISTING)

PERMAFROST IN 2005

COMPLETELY WEATHERED SHALE

HIGHLY TO COMPLETELY WEATHERED SHALE

HIGHLY WEATHERED SHALE

HIGHTLY WEATHERED SILTSTONE

SILTY SAND & HIGHLY WEATHERED SHALE

MODERATELY TO HIGHLY WEATHERED SHALE

LAMINATED SHALE

SLIGHTLY TO MODERATELY WEATHERED SHALE

SILTSTONE

SANDSTONE

PEAT

TILL

SILTY SANDY GRAVEL

SILTY GRAVEL, COBBLES, AND BOULDERS (ALLUVIUM)

SILTY CLAY WITH SAND AND GRAVEL FILL

SAND & GRAVEL FILL

GRAVEL FILL

ROCK FILL/UNDER DRAIN

ROCK DRAIN

WASTE ROCK

RIP RAP { LARGER / SMALLER

GRAVEL BEDDING

NEW DAM RAISE

BORING AND INSTRUMENTATION LOCATIONS

- DAMES & MOORE FOR TAILINGS DAM (1982)
- DAMES & MOORE FOR TAILINGS DAM (1984)
- DAMES AND MOORE FOR WASTE STORAGE (1984 & 1987)
- EBA ENGINEERING FOR MILL FACILITIES (2000)
- PND FOR RIGHT ABUTMENT WING WALL (2004)
- DAMES & MOORE FOR MILL FACILITIES (1982, 1985)
- URS WITH PIEZOMETERS (2005)
- THERMISTORS (PRE–2005)
- URS WITH PIEZOMETERS & THERMISTERS (2005)
- URS WITH PIEZOMETERS (2005)
- URS WITH NO INSTRUMENTATION (2005–2006)
- URS BORING (2007)

EMBANKMENT CONFIGURATION

SOIL TYPE	MATERIAL
①	RANDOM ROCKFILL (24 INCH)
②	SELECT (24 INCH)
③	PROCESSED SELECT (3 INCH)
④	PROCESSED SELECT (1 INCH)
⑦	SELECT (12 INCH)
⑨	RANDOM MINE WASTE FILL
GEOSYNTHETICS	
GEOMEMBRANE – 100 MIL HDPE GEOTEXTILE – 10 OZ AND 16 OZ NONWOVEN	

STAGE	CREST ELEVATION	COMMENTS
I	865'	EXISTING
II	890'	EXISTING
III	910'	EXISTING
IV	925'	EXISTING
V	940'	EXISTING
VI	950'	EXISTING
VII–A	955'	EXISTING
VII–B	960'	EXISTING
VIII	970'	FUTURE
IX	980'	FUTURE
X	986'	FUTURE

ABBREVIATIONS

DRAWINGS

T	TAILINGS IMPOUNDMENT DAM
T7	TAILINGS DAM STAGE VII–A
T7B	TAILINGS DAM STAGE VII–B
TDC	TAILINGS DAM CONCEPTUAL

DIRECTIONS

N	NORTH
S	SOUTH
E	EAST
W	WEST
NW	NORTHWEST
NE	NORTHEAST
SW	SOUTHWEST
SE	SOUTHEAST

DRAWING LABELS

TDAM	TAILINGS DAM
P	PIEZOMETER
T	THERMISTOR
TP	TEST PIT
BH	BORE HOLE
MILL	MILL
TPT	
D_{50}	GRAIN DIAMETER (IN MM) CORRESPONDING TO 50% PASSING BY WEIGHT (OR MASS)

SHORTENED WORDS

APPROX.	APPROXIMATE(LY)
CDF	CONTROLLED DENSITY FILL
EL	ELEVATION
EST	ESTIMATED
EXIST	EXISTING
FT	FOOT (FEET)
HDPE	HIGH DENSITY POLYETHYLENE
HOR (H)	HORIZON
INC	INCORPORATION
LTD	LIMITED
MAX	MAXIMUM
MIN	MINIMUM
NO	NUMBER
NTS	NOT TO SCALE
OZ	OUNCE
REV	REVISION
STA	STATION
VER (V)	VERTICAL

REVISIONS DESCRIPTION.

INCORP	INCORPORATED
KIV	KIVALINA
RPT	REPORT

TEXT SYMBOLS

#	NUMBER
%	PERCENT

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
TECK COMINCO ALASKA INC., "RED DOG MINE"	
(TOPOGRAPHY MAP, SEPTEMBER 16, 2004, PROJECT 6040524).	

REVISIONS NO.	BY	DATE	DESCRIPTION
△			
△			
△			
△			
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			
△			
△			
△			
△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE–11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

REFERENCES, LEGEND AND ABBREVIATIONS

JOB NO. 33757098.00001

DRAWING NO. TDC–2 | REV. 0

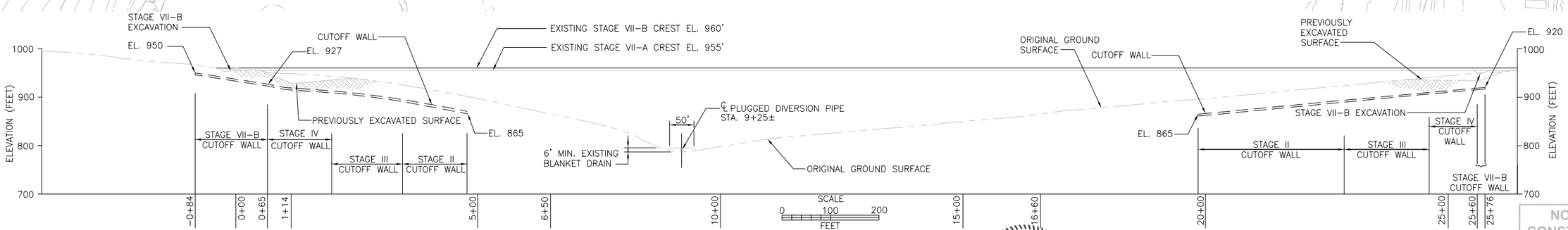
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–2.dwg Nov 16, 2007 – 3:17pm

N
W — E

ROCK DRAIN AND PUMPING CHAMBERS
SEEPAGE COLLECTION DAM
SPILLWAY
LIMITS OF BLANKET DRAIN
EXIST PIPE BEACH
TAILINGS MAIN DAM
LINER
STAGE VI AXIS
STA. 0+00
N5143415.8
E580880.1
PLUGGED DIVERSION PIPE
EAST RISER PIPE
WEST RISER PIPE
CUTOFF WALL
TOE DRAIN JUNCTION
UPSTREAM TOE OF ROCK BUTTRESS
COFFER DAM CREST EL. 960FT
PIPE CROSSING CULVERT
STAGE VI AXIS
STA. 25+60
N 5144278.2
E 583290.5
LIMITS OF 2006 SURVEY
TOE OF GEOMEMBRANE AT CUTOFF TRENCH
APPROX. LIMIT OF CUTOFF TRENCH FLOOR
APPROX. LIMIT OF CUTOFF TRENCH
DUST CONTROL STRUCTURE
COFFER DAM EL. 955
CUTOFF WALL
STAGE II CUTOFF WALL
STAGE III CUTOFF WALL
STAGE IV CUTOFF WALL
STAGE VII–B CUTOFF WALL
CURTAIN WALL (STAGE VII–B)

PLAN

SCALE
0 120 240
FEET



NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE–11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

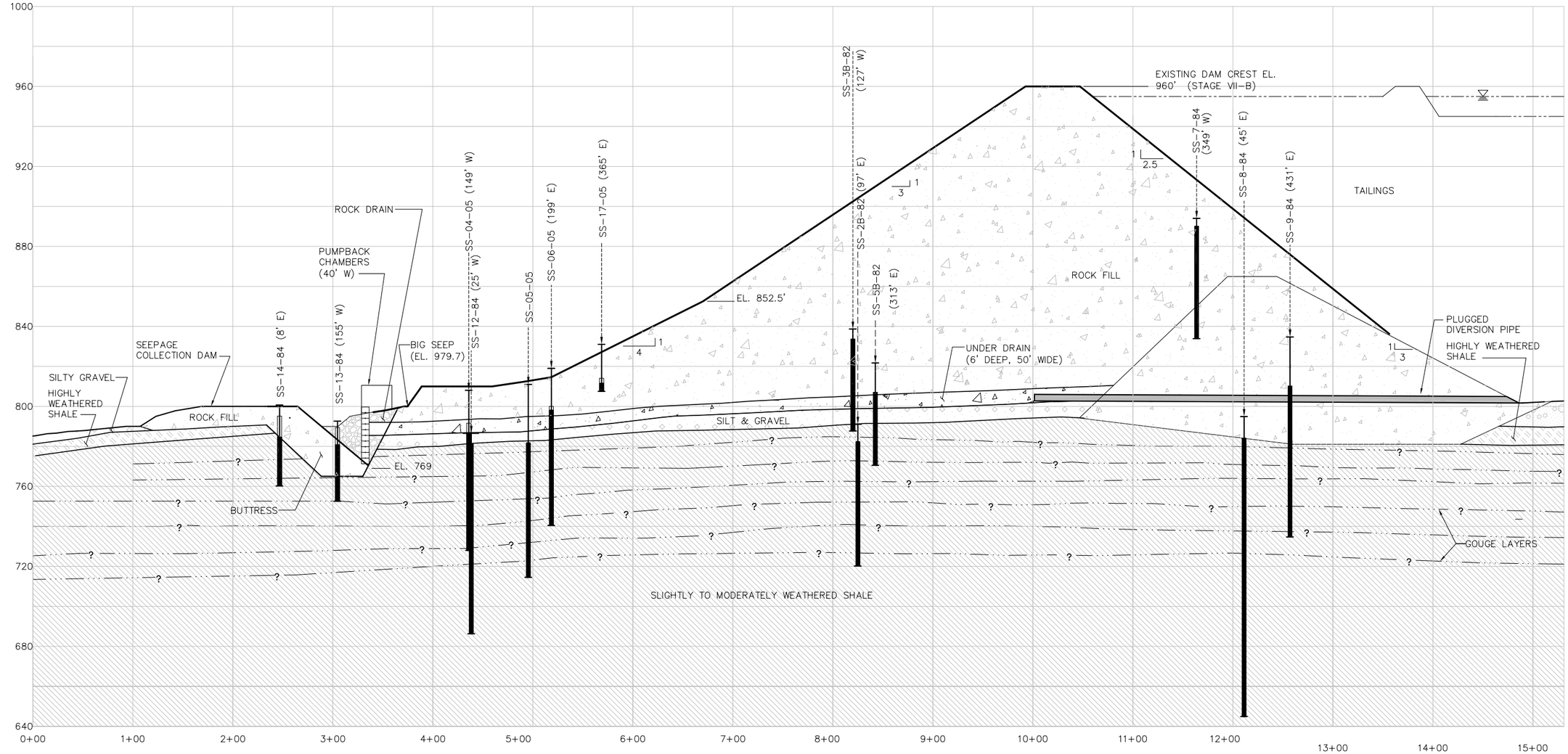
EXISTING STAGE VII–B DAM PLAN & PROFILE

JOB NO. 33757098.00001
DRAWING NO. TDC–3
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–3.dwg Nov 16, 2007 – 3:15pm

NORTHWEST

SOUTHEAST



SUBSURFACE PROFILE
STATION 8+60

(A / 3|4)

HOR. SCALE
0 50 100
FEET

VER. SCALE
0 25 50
FEET

KEY:
- GROUND SURFACE
- SOIL
- HIGHLY WEATHERED BEDROCK
- BEDROCK

NOTE

1. DEPICTS THE UNDERDRAIN, WHICH IS ACTUALLY OFF-SET INTO THE PAGE ALONG ALIGNMENT ≈ 76FT ON AVERAGE. (A / TDC-3)

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA"
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	FOR AGRA SIMONS LTD, APRIL 2000.
	4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

STATE OF ALASKA – 49TH – CECIL M. URLICH CE-11095 – REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

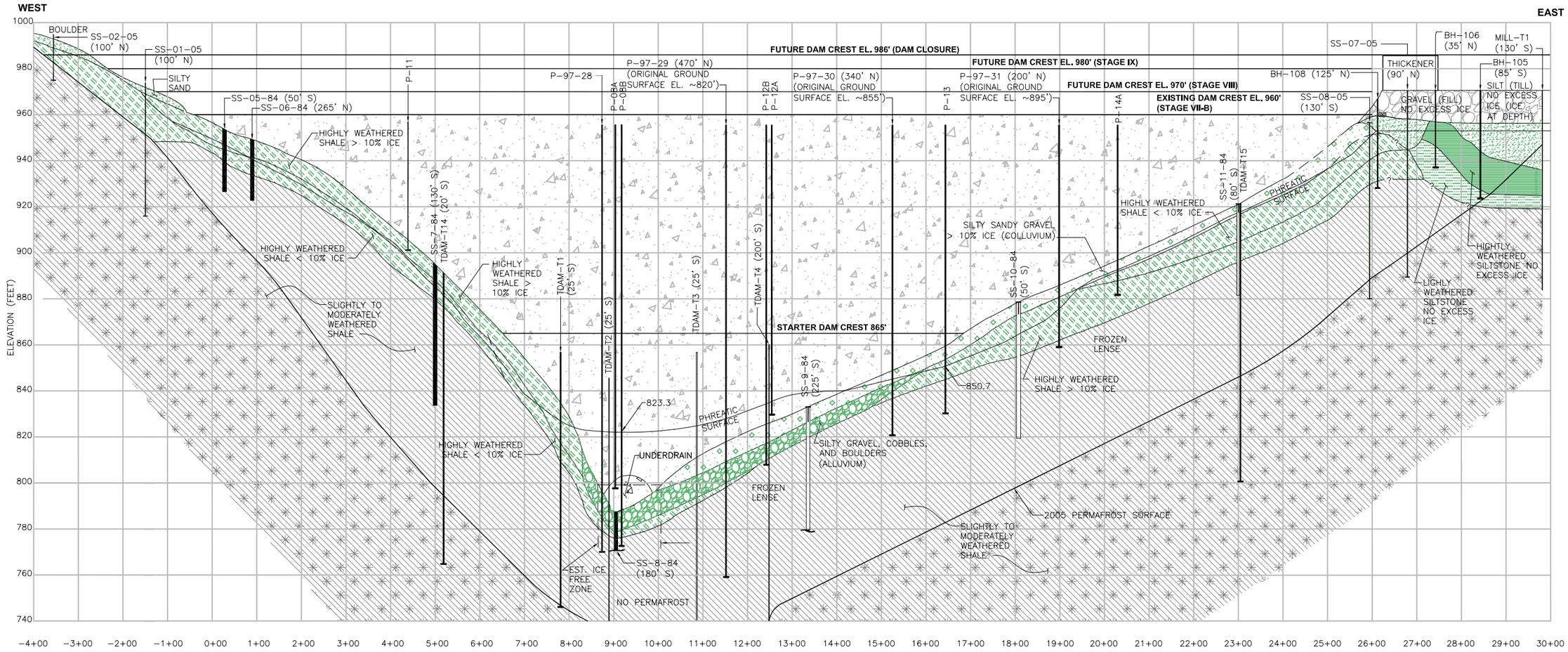
RED DOG MINE TAILINGS DAM, FUTURE RAISES

PROFILE ALONG HIGHEST MAIN DAM SECTION

JOB NO. 33757098.00001

DRAWING NO. TDC-4 REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-4.dwg Nov 16, 2007 - 3:13pm



NOTES:

1. INDICATED SUBSURFACE DESCRIPTIONS REPRESENT GENERAL SOIL AND ROCK CONDITIONS UNDER DAM (BORINGS BY DAMES & MOORE IN 1982 AND 1984), UNDER WASTE STORAGE AREA (BORINGS BY DAMES & MOORE IN 1984 AND 1987), IN MILL AREA NEAR RIGHT ABUTMENT (BORINGS BY EBA IN 2000) ON RIGHT ABUTMENT (TEST PITS BY PND IN 2004), AND UNDER FUTURE DAM RAISES (URS, 2005).
2. ICE PERCENTAGES SHOWN BENEATH THE DAM WERE CALCULATED BY VOLUME PRIOR TO DAM CONSTRUCTION AND TAILINGS DISPOSAL. THESE PERCENTAGES DO NOT REFLECT CURRENT CONDITIONS. THE PERMAFROST AND FROZEN LENS LOCATIONS SHOWN ARE BASED ON CURRENT CONDITIONS AS DESCRIBED IN NOTE 4.
3. SOIL AND ROCK PROFILES SHOWN REPRESENT URS INTERPRETATION OF DATA FROM GEOTECHNICAL INVESTIGATIONS INDICATED IN NOTE 1. SUBSURFACE CONDITIONS ARE ONLY KNOWN AT BORING LOCATIONS AND MAY DIFFER AT OTHER LOCATIONS, ESPECIALLY IN THE RIGHT ABUTMENT AREA WHERE THEY ARE DISCONTINUOUS AND VARIABLE.
4. PERMAFROST PROFILES, ICE LENSES, AND PHREATIC SURFACE SHOWN REPRESENT TECK COMINCO INTERPRETATION OF THERMISTOR AND PIEZOMETER INSTRUMENTATION DATA. CONDITIONS ARE ONLY KNOWN AT THE INSTRUMENTATION LOCATIONS, AND MAY DIFFER AT OTHER LOCATIONS. WATER LEVELS SHOWN WERE COLLECTED THROUGH DECEMBER 2006.
5. MODIFY CONSTRUCTION AND EXCAVATION DEPTHS AS DIRECTED BY ENGINEER TO ACCOUNT FOR ACTUAL SOIL, ROCK, AND ICE CONDITIONS.

PROFILE ACROSS EMBANKMENT CREST

B 3|5

100 0 100 200
HORIZONTAL SCALE IN FEET

20 0 20 40
VERTICAL SCALE IN FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA" FOR AGRA SIMONS LTD, APRIL 2000.
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			
△				△			

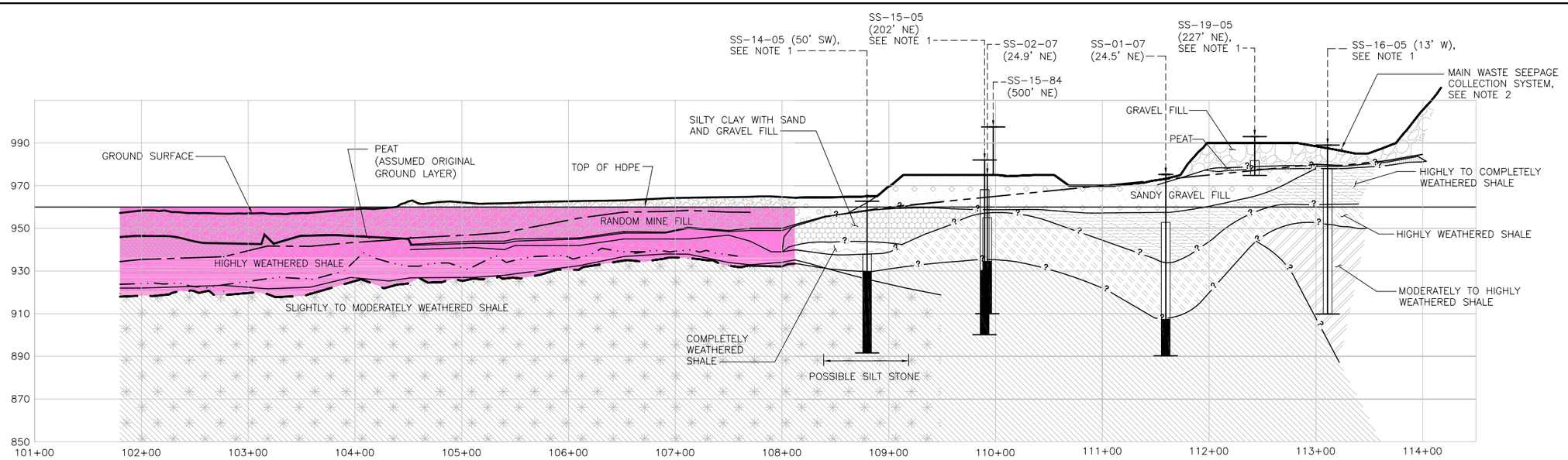
STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED		INIT	DATE
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.
RED DOG MINE TAILINGS DAM, FUTURE RAISES
PROFILE ACROSS EMBANKMENT CREST
JOB NO. 33757098.00001
DRAWING NO. TDC-5
REV. 0

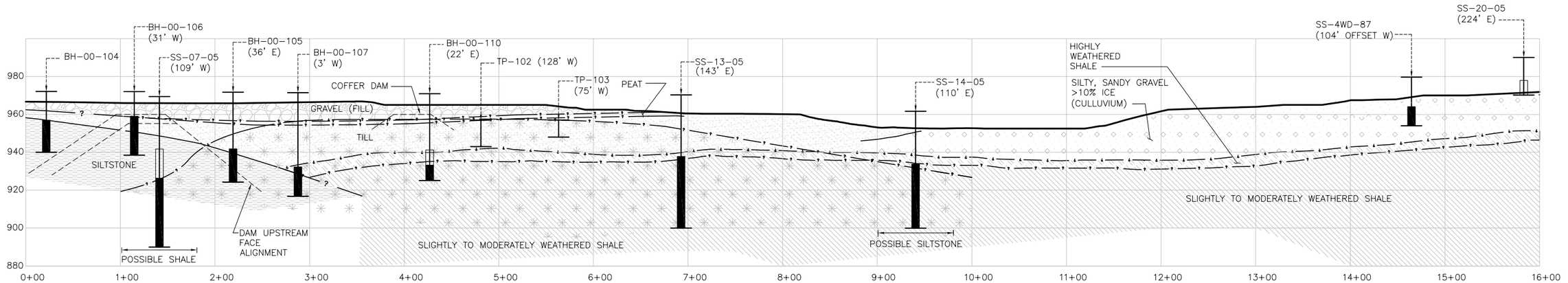
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-5.dwg Nov 16, 2007 - 3:20pm



PROFILE C 3 6

HOR. SCALE 0 50 100 FEET

VER. SCALE 0 25 50 FEET



PROFILE D 3 6

HOR. SCALE 0 50 100 FEET

VER. SCALE 0 25 50 FEET

LEGEND:

- STAGE VII-B CURTAIN WALL
- 2004 TOPOGRAPHY
- 1987 TOPOGRAPHY
- TRENCH BOTTOM
- LAYER BOUNDARIES
- WORKBENCH
- TOP CDF
- TRENCH BOTTOM

NOTES

1. NO FROZEN GROUND NOTED DURING DRILLING IN 2005.
2. 14" DIAMETER, HEAT-TRACED HDPE PIPE.

KEY: GROUND SURFACE; SOIL; HIGHLY WEATHERED BEDROCK; BEDROCK

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA"
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	FOR AGRA SIMONS LTD, APRIL 2000. 4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

NO.	BY	DATE	REVISIONS DESCRIPTION

NO.	BY	DATE	REVISIONS DESCRIPTION

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED	INIT	DATE
DESIGNED BY: S. RAJAH		7-13-05
DRAWN BY: T. PHAM		7-13-05
CHECKED BY: C. MASALA		10-26-07
APPROVED BY: C. URLICH		11-02-07
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

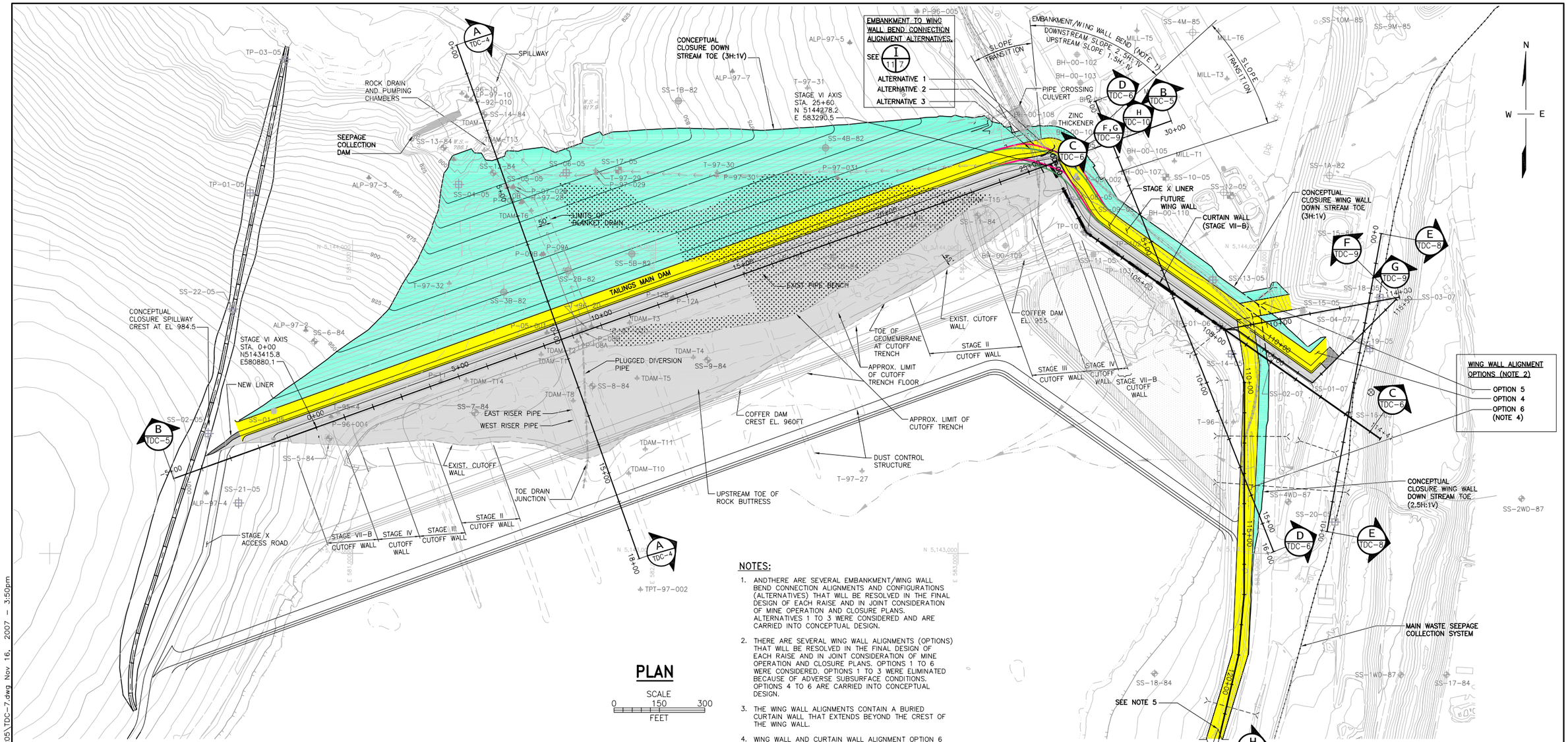
RED DOG MINE TAILINGS DAM, STAGE VII-B

PROFILES ALONG CURTAIN WALL

JOB NO. 33757098.00001

DRAWING NO. TDC-6 REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-6.dwg Nov 16, 2007 - 3:34pm

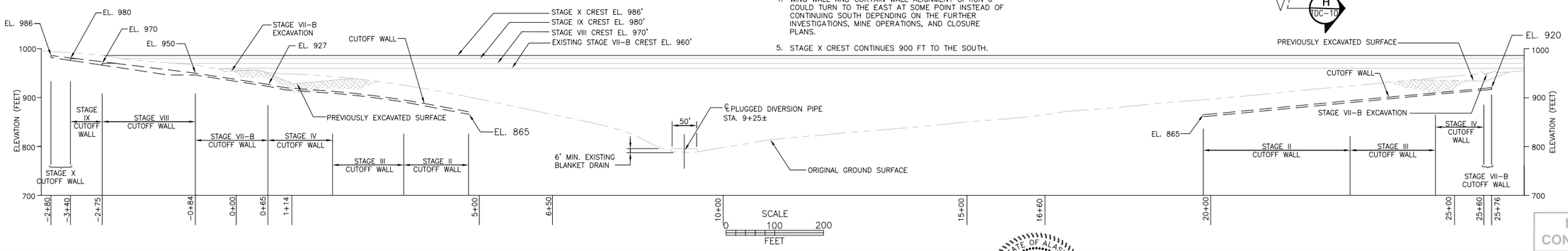


PLAN

SCALE
0 150 300
FEET

NOTES:

1. ANDTHERE ARE SEVERAL EMBANKMENT/WING WALL BEND CONNECTION ALIGNMENTS AND CONFIGURATIONS (ALTERNATIVES) THAT WILL BE RESOLVED IN THE FINAL DESIGN OF EACH RAISE AND IN JOINT CONSIDERATION OF MINE OPERATION AND CLOSURE PLANS. ALTERNATIVES 1 TO 3 WERE CONSIDERED AND ARE CARRIED INTO CONCEPTUAL DESIGN.
2. THERE ARE SEVERAL WING WALL ALIGNMENTS (OPTIONS) THAT WILL BE RESOLVED IN THE FINAL DESIGN OF EACH RAISE AND IN JOINT CONSIDERATION OF MINE OPERATION AND CLOSURE PLANS. OPTIONS 1 TO 6 WERE CONSIDERED. OPTIONS 1 TO 3 WERE ELIMINATED BECAUSE OF ADVERSE SUBSURFACE CONDITIONS. OPTIONS 4 TO 6 ARE CARRIED INTO CONCEPTUAL DESIGN.
3. THE WING WALL ALIGNMENTS CONTAIN A BURIED CURTAIN WALL THAT EXTENDS BEYOND THE CREST OF THE WING WALL.
4. WING WALL AND CURTAIN WALL ALIGNMENT OPTION 6 COULD TURN TO THE EAST AT SOME POINT INSTEAD OF CONTINUING SOUTH DEPENDING ON THE FURTHER INVESTIGATIONS, MINE OPERATIONS, AND CLOSURE PLANS.
5. STAGE X CREST CONTINUES 900 FT TO THE SOUTH.



NOT FOR CONSTRUCTION

REFERENCES	REFERENCES	REVISIONS				REVISIONS			
TITLE	TITLE	NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 2004 & 2007.	△				△			
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.		△				△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

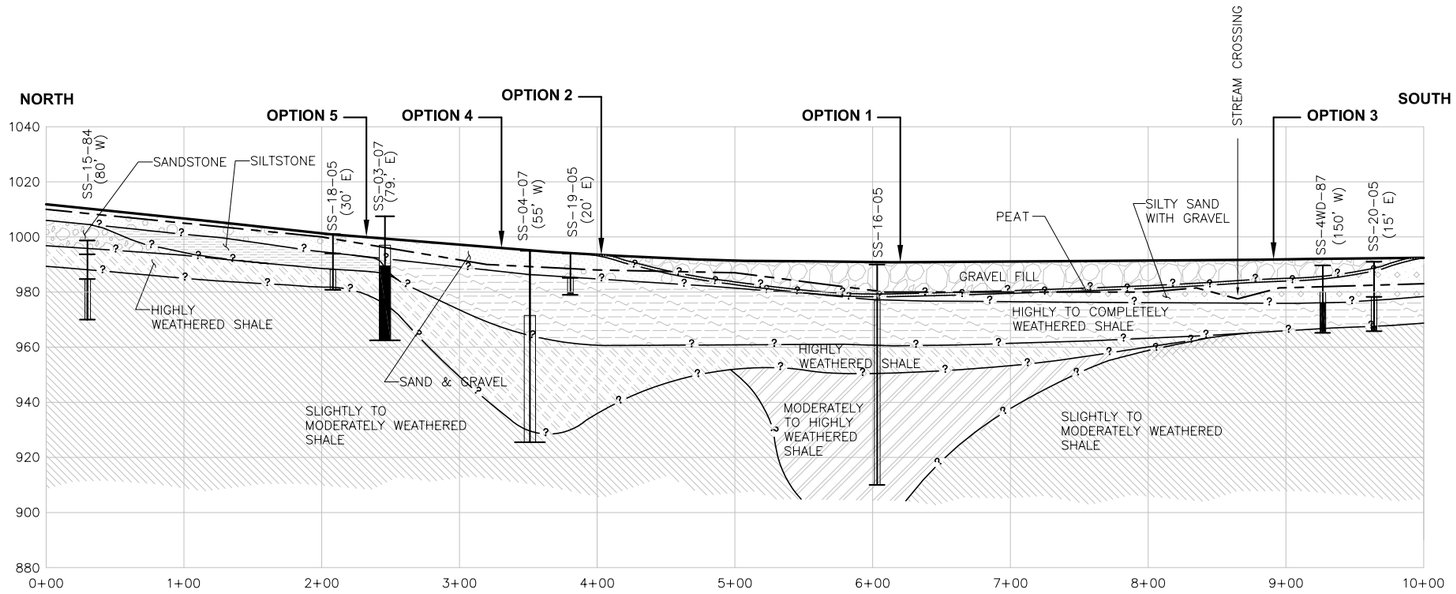
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

CLOSURE STAGE X DAM PLAN & PROFILE

JOB NO. 33757098.00001
DRAWING NO. TDC–7
REV. 0

P:\ACAD\PROJECT\TeckCominccoAlaska\Red Dog Tailings Dam Closure Design\R05\TDC–7.dwg Nov 16, 2007 – 3:50pm



KEY:



LEGEND:

2004 TOPOGRAPHY

1987 TOPOGRAPHY

HOR. SCALE
0 50 100
FEET

VER. SCALE
0 25 50
FEET

PROFILE E (7 | 8)

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA"
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	FOR AGRA SIMONS LTD, APRIL 2000.
	4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			
△				△			

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

RED DOG MINE TAILINGS DAM, STAGE VII-B

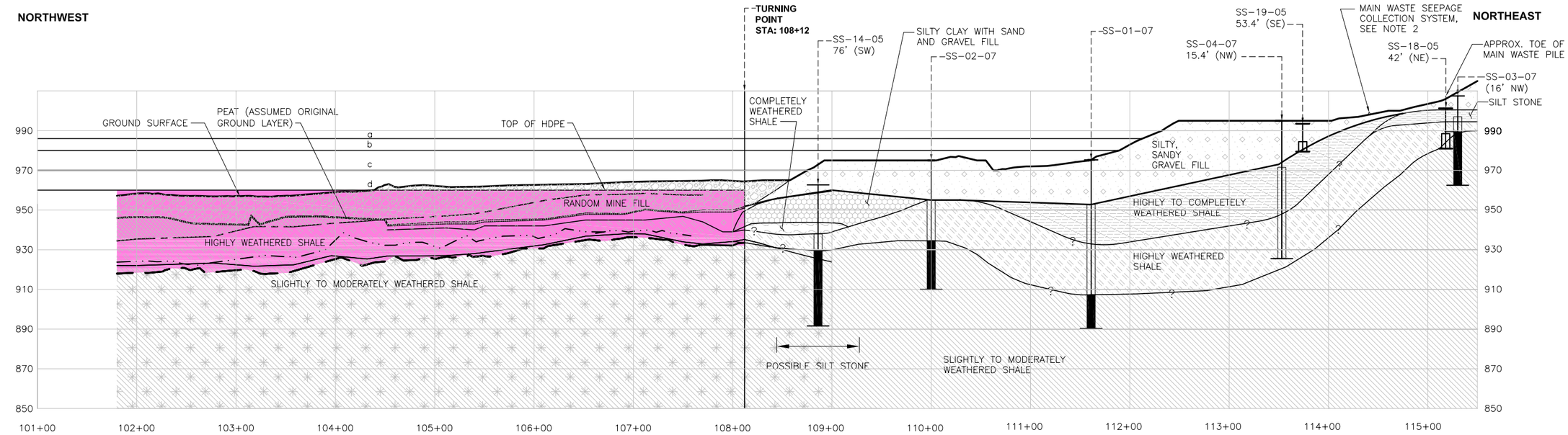
PROFILE ACROSS WING WALL OPTIONS

JOB NO. 33757098.00001

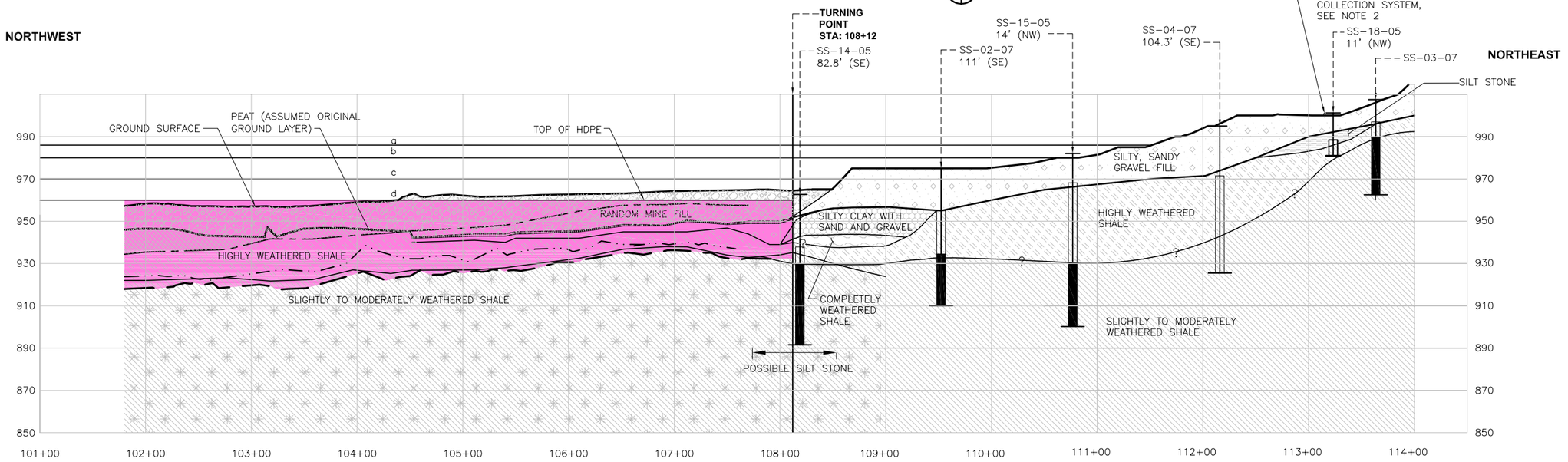
DRAWING NO. TDC-8

REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-8.dwg Nov 16, 2007 - 3:32pm



PROFILE WING WALL OPTION 4 (F 7|9)



NOTES

1. NO FROZEN GROUND NOTED DURING DRILLING IN 2005.
2. 14" DIAMETER HEAT-TRACED HDPE PIPE

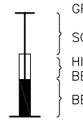
LEGEND:

- STAGE VII-B CURTAIN WALL
- 2004 TOPOGRAPHY
- 1987 TOPOGRAPHY
- TRENCH BOTTOM
- LAYER BOUNDARIES
- WORKBENCH
- TOP CDF

PROFILE WING WALL OPTION 5 (G 7|9)

HOR. SCALE 0 50 100 FEET
VER. SCALE 0 25 50 FEET

KEY:



LEGEND

- a — STAGE X CREST EL. 986
- b — STAGE IX CREST EL. 980
- c — STAGE VIII CREST EL. 970
- d — STAGE VII CREST EL. 960

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA"
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	FOR AGRA SIMONS LTD, APRIL 2000.
	4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

REVISIONS NO.	BY	DATE	DESCRIPTION

REVISIONS NO.	BY	DATE	DESCRIPTION

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED	INIT	DATE
DESIGNED BY: B. SUREN		8-10-07
DRAWN BY: T. PHAM		8-10-07
CHECKED BY: B. KUCK		10-26-07
APPROVED BY: C. URLICH		11-02-07
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

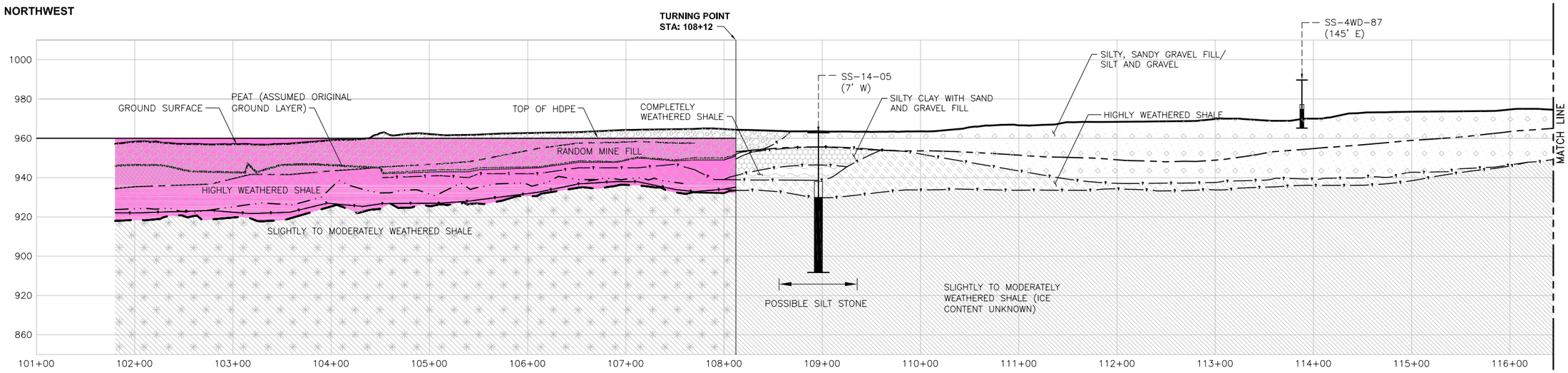
RED DOG MINE TAILINGS DAM, STAGE VII-B

PROFILE ALONG WING WALL OPTIONS 4 & 5

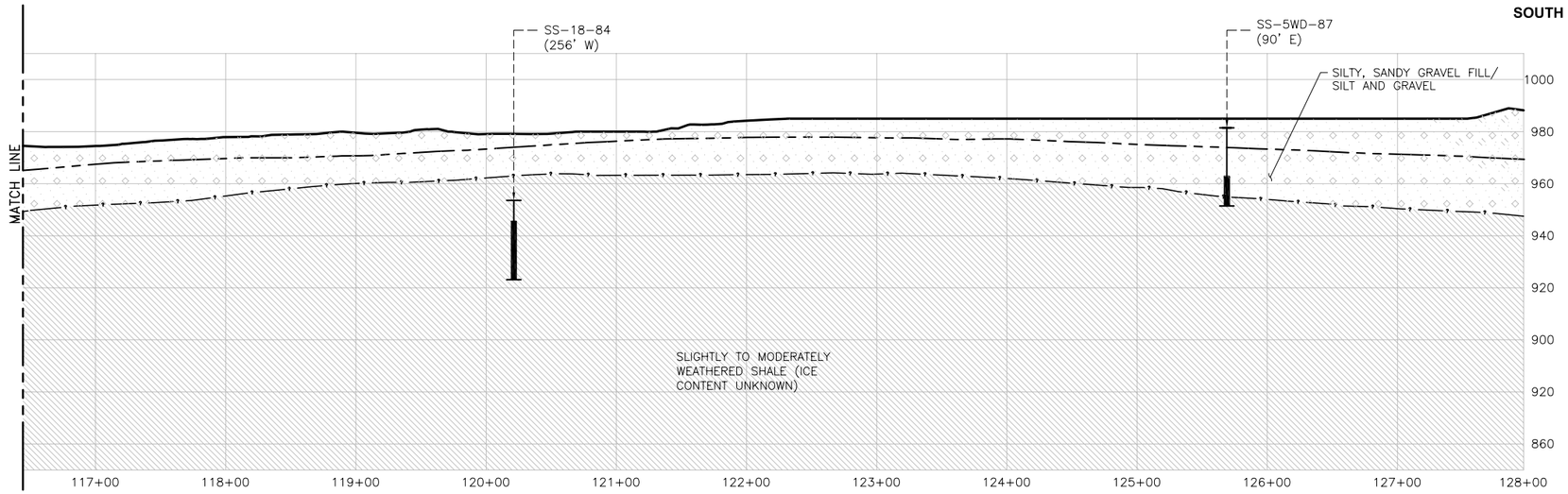
JOB NO. 33757098.00001
DRAWING NO. TDC-9
REV. 0

P:\ACAD\PROJECT\TeckCominco\Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-9.dwg Nov 16, 2007 - 3:23pm

NORTHWEST



SOUTH



LEGEND:

- STAGE VII-B CURTAIN WALL
- 2004 TOPOGRAPHY
- 1987 TOPOGRAPHY
- TRENCH BOTTOM
- LAYER BOUNDARIES
- WORKBENCH
- TOP CDF
- TRENCH BOTTOM

PROFILE WING WALL OPTION 6

(H 7110 / A 1110)

HOR. SCALE
0 50 100
FEET

VER. SCALE
0 25 50
FEET

KEY:
- GROUND SURFACE
- SOIL
- HIGHLY WEATHERED BEDROCK
- BEDROCK

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. DAMES & MOORE, "DESIGN REPORT – PROPOSED TAILINGS DAM, RED DOG MINE DEVELOPMENT" FOR COMINCO ALASKA INC., JUNE 6 1986.	3. EBA ENGINEERING CONSULTANTS, LTD. "GEOTECHNICAL EVALUATION: PROPOSED VIP PROJECT – MILLSITE, RED DOG MINE, ALASKA"
2. DAMES & MOORE, "GEOTECHNICAL DESIGN REPORT FOR COMINCO RED DOG MINE WASTE STORAGE AREAS," FOR COMINCO ALASKA INC JULY 16, 1987.	FOR AGRA SIMONS LTD, APRIL 2000.
	4. PND INC., "GEOTECHNICAL INVESTIGATION FOR PROPOSED COFFER DAM", FOR TECK COMINCO ALASKA, DECEMBER 8, 2004

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
1				1			
2				2			
3				3			
4				4			

STATE OF ALASKA 49TH
CECIL M. URLICH
CE-11095
REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

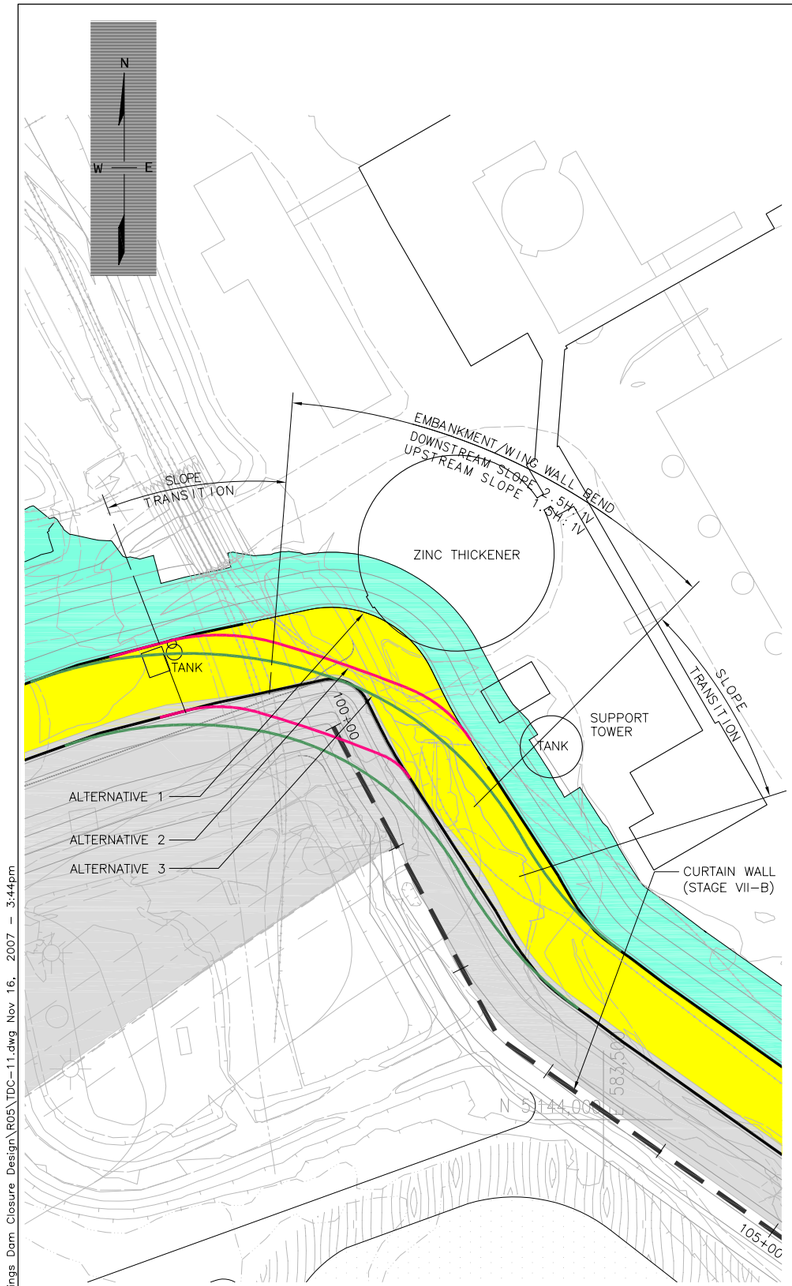
RED DOG MINE TAILINGS DAM, STAGE VII-B

PROFILE ALONG WING WALL OPTION 6

JOB NO. 33757098.00001

DRAWING NO. TDC-10 REV. 0

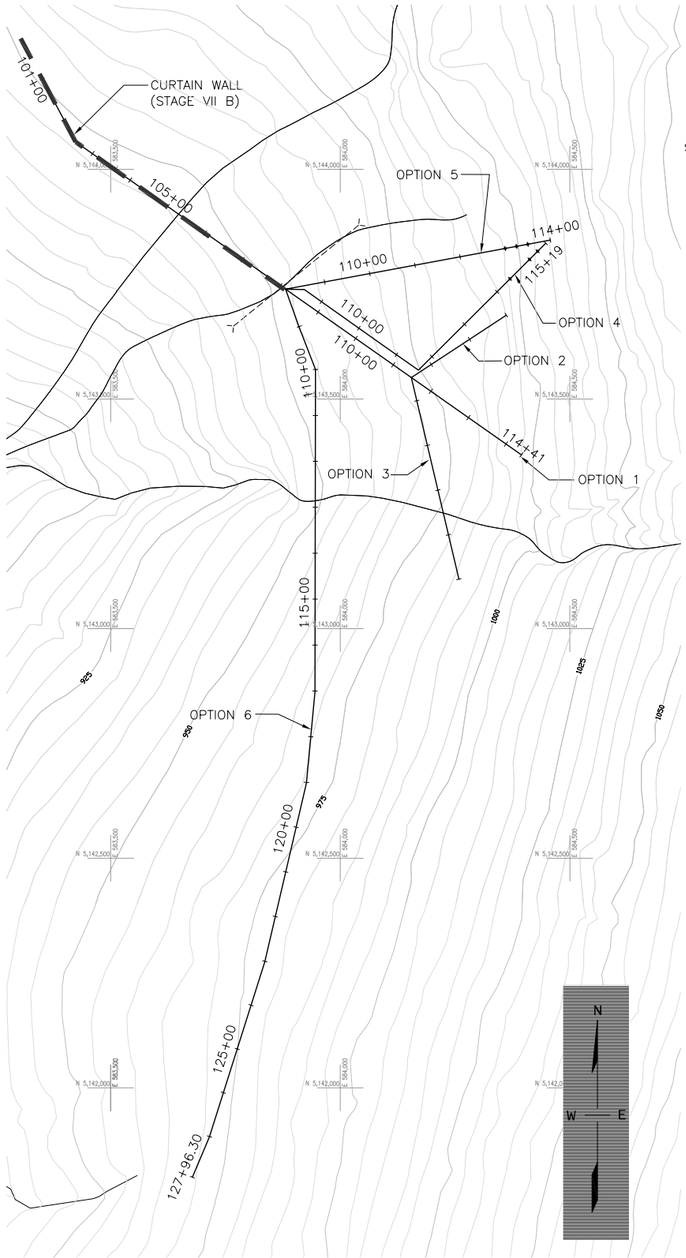
P:\ACAD\PROJECT\TeckCominoAlaska\Red Dog Tailings Dam Closure Design\R05\TDC-10.dwg Nov 16, 2007 - 3:36pm



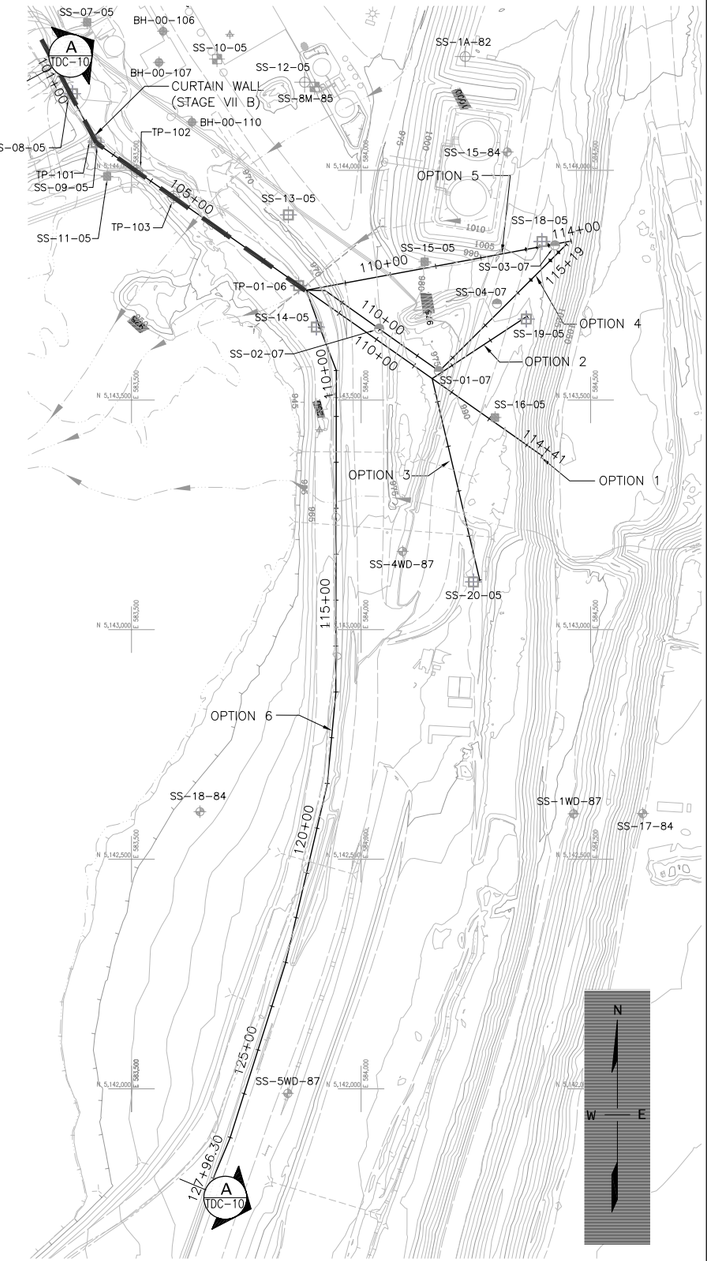
EMBANKMENT/WING WALL BEND ALTERNATIVES

I
7 | 11

SCALE
0 50 100
FEET



PRE-MINING TOPOGRAPHY (1987)



CURRENT TOPOGRAPHY (2004)

WING WALL ALIGNMENT OPTIONS

SCALE
0 150 300
FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM,	3. CONSTRUCTION COMPLETION REPORT, STAGE VII-A & VII-B RAISE, RED
STAGE I-VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC.,	DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG
RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING	4. NORTH PACIFIC AIR SURVEYS "TOPOGRAPHY SHEETS 1 THROUGH
COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	10" JULY 1987.

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			

SCALE: AS NOTED	INIT	DATE
DESIGNED BY: B.SUREN		10-26-07
DRAWN BY: T. PHAM		10-26-07
CHECKED BY: C.MASALA		10-26-07
APPROVED BY: C. URLICH		11-02-07
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

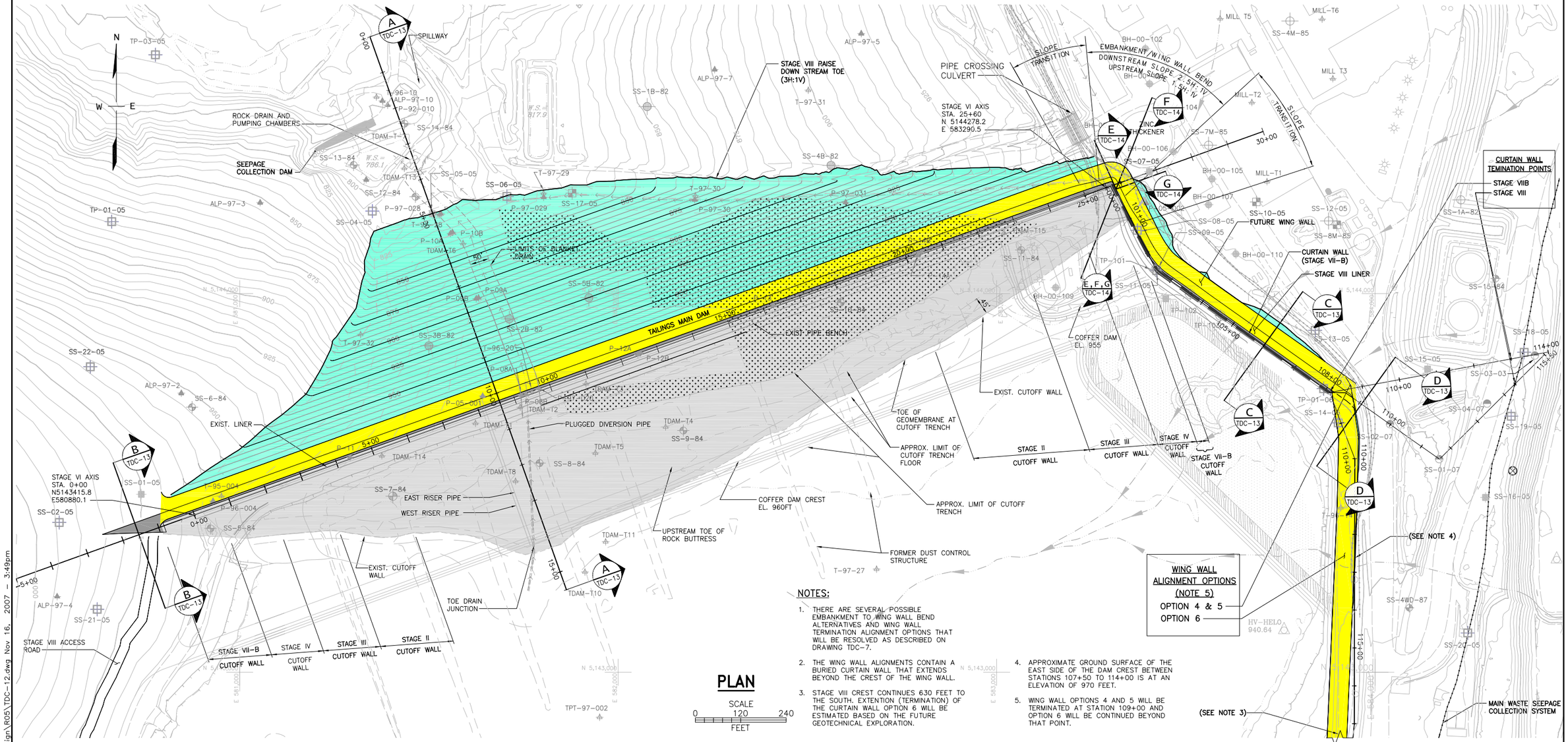
RED DOG TAILINGS MAIN DAM, FUTURE RAISES

BEND ALTERNATIVE AND WING WALL OPTION PLANS

JOB NO. 33757098.00001

DRAWING NO. TDC-11 | REV. 0

P:\ACAD\PROJECT\TeckCominccoAlaska\Red Dog Tailings Dam Closure Design\R05\TDC-11.dwg Nov 16, 2007 - 3:44pm

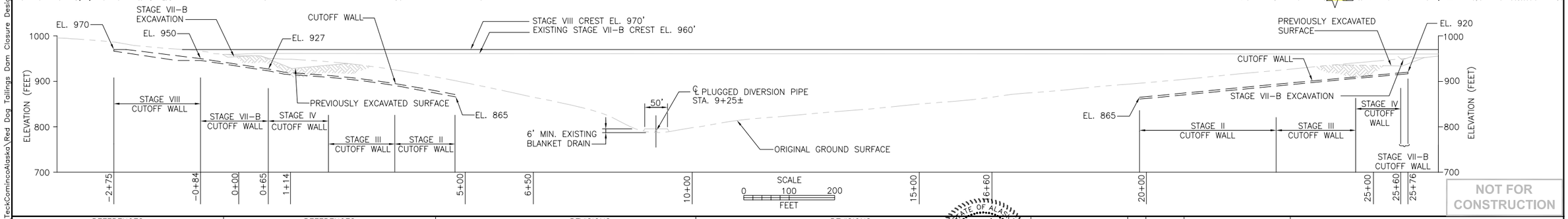


PLAN

SCALE
0 120 240
FEET

NOTES:

1. THERE ARE SEVERAL POSSIBLE EMBANKMENT TO WING WALL BEND ALTERNATIVES AND WING WALL TERMINATION ALIGNMENT OPTIONS THAT WILL BE RESOLVED AS DESCRIBED ON DRAWING TDC–7.
2. THE WING WALL ALIGNMENTS CONTAIN A BURIED CURTAIN WALL THAT EXTENDS BEYOND THE CREST OF THE WING WALL.
3. STAGE VIII CREST CONTINUES 630 FEET TO THE SOUTH. EXTENTION (TERMINATION) OF THE CURTAIN WALL OPTION 6 WILL BE ESTIMATED BASED ON THE FUTURE GEOTECHNICAL EXPLORATION.
4. APPROXIMATE GROUND SURFACE OF THE EAST SIDE OF THE DAM CREST BETWEEN STATIONS 107+50 TO 114+00 IS AT AN ELEVATION OF 970 FEET.
5. WING WALL OPTIONS 4 AND 5 WILL BE TERMINATED AT STATION 109+00 AND OPTION 6 WILL BE CONTINUED BEYOND THAT POINT.



NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM,	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED
STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC.,	DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG
RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING	
COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			
△				△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE–11095
REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

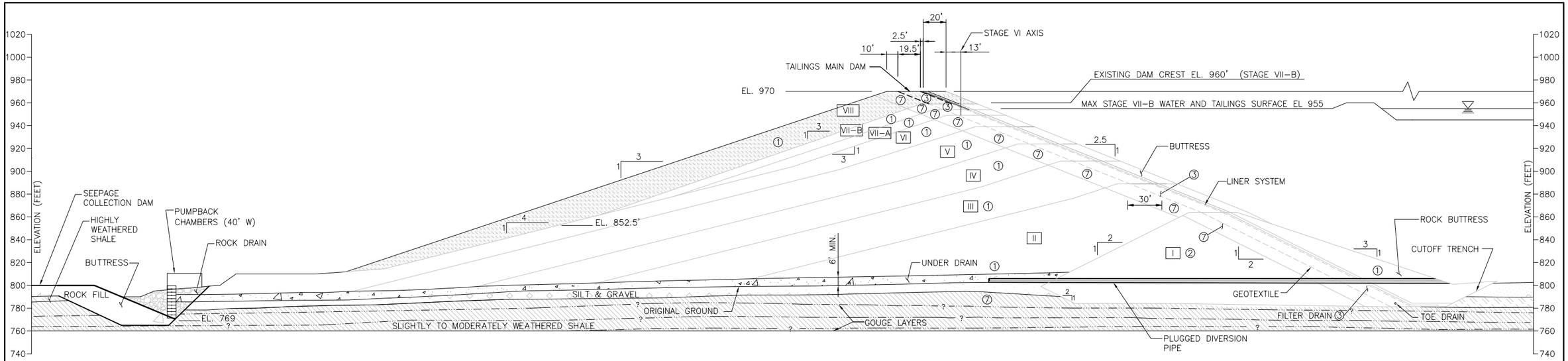
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

STAGE VIII RAISE CONCEPTUAL PLAN AND PROFILE

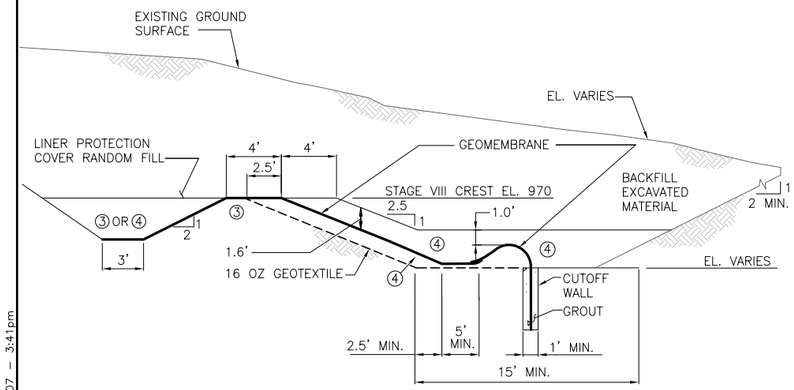
JOB NO. 33757098.00001
DRAWING NO. TDC–12
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–12.dwg Nov 16, 2007 – 3:49pm



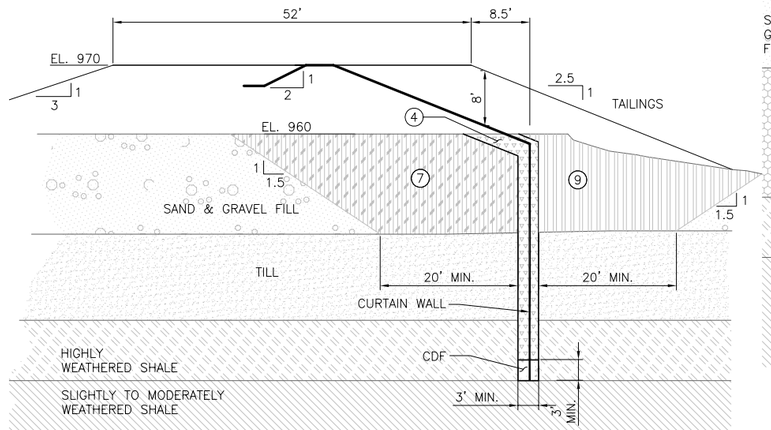
MAIN DAM SECTION
STATION 8+60
A 12|13

SCALE
0 40 80
FEET



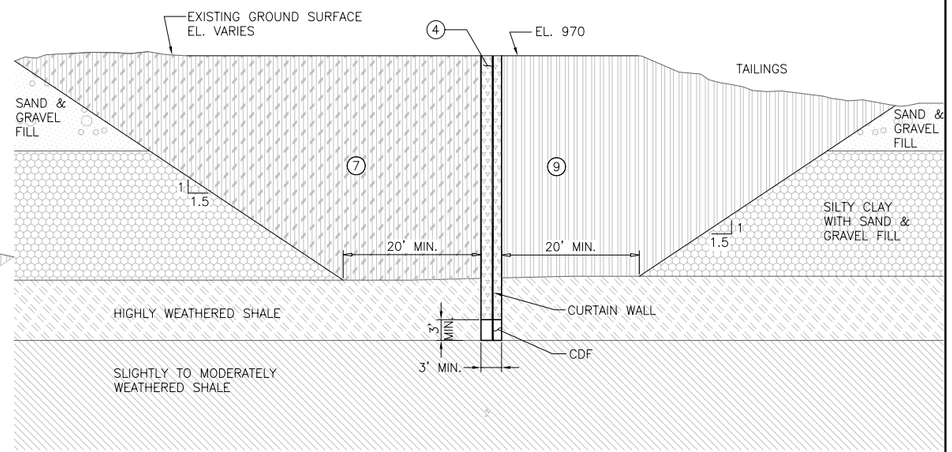
LEFT ABUTMENT SECTION
AT STA. −1+60
B 12|13

SCALE: NTS



WING WALL SECTION
AT STA. 106+00
C 12|13

SCALE
0 10 20
FEET



RIGHT ABUTMENT SECTION
AT STA. 109+08
D 12|13

SCALE
0 10 20
FEET

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION

REVISIONS NO.	BY	DATE	DESCRIPTION

STATE OF ALASKA 49TH
CECIL M. URLICH CE–11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.
RED DOG TAILINGS MAIN DAM, FUTURE RAISES
STAGE VIII RAISE CONCEPTUAL SECTIONS

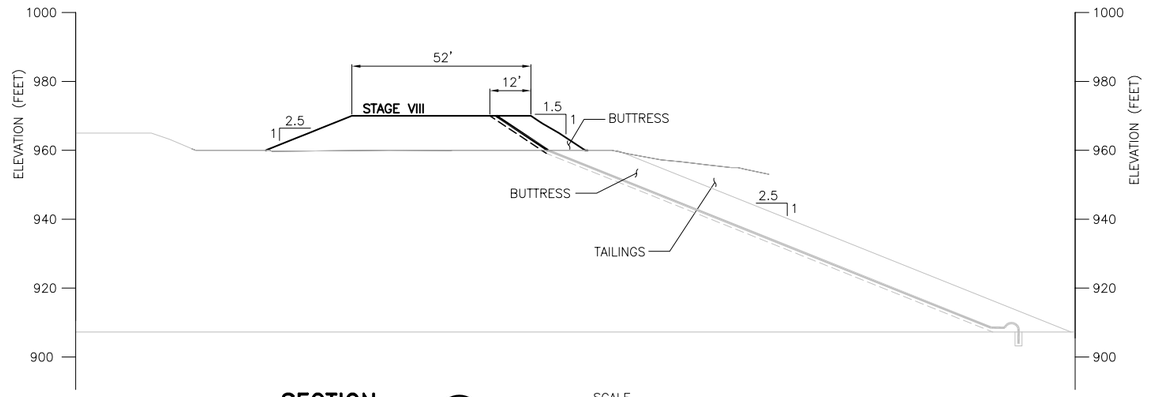
JOB NO. 33757098.00001
DRAWING NO. TDC–13
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–13.dwg Nov 16, 2007 – 3:41pm

LEGEND

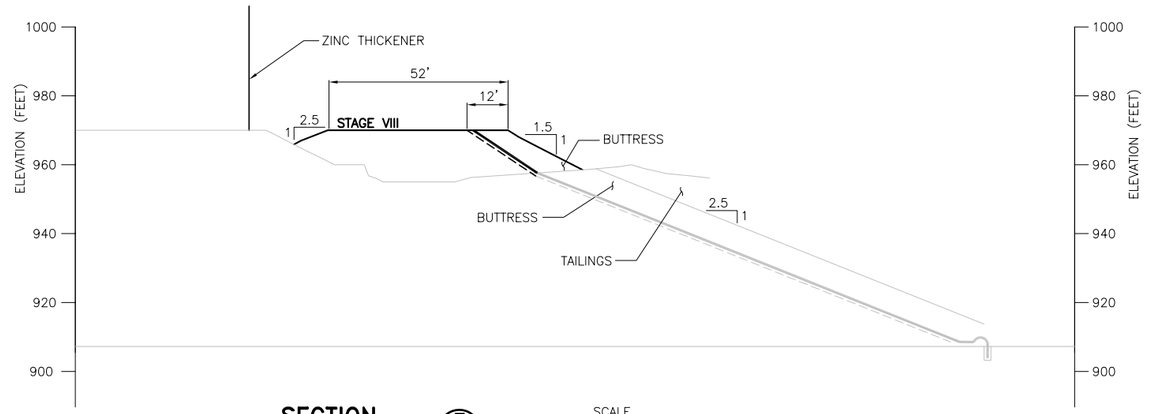
EXISTING LINER SYSTEM

FUTURE LINER SYSTEM



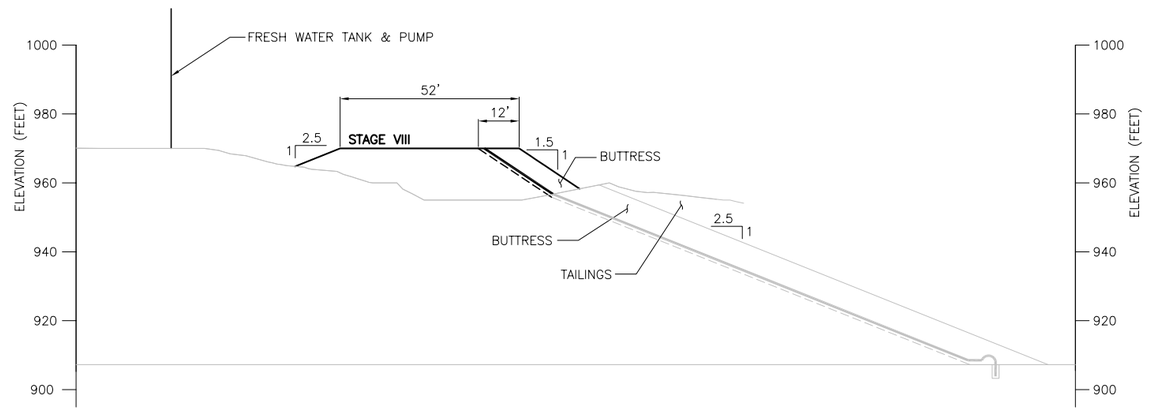
SECTION E 12|14

SCALE 0 20 40 FEET



SECTION F 12|14

SCALE 0 20 40 FEET



SECTION G 12|14

SCALE 0 20 40 FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

NO.	BY	DATE	REVISIONS DESCRIPTION
△			
△			
△			
△			
△			

NO.	BY	DATE	REVISIONS DESCRIPTION
△			
△			
△			
△			
△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE–11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	C. MASALA		8–10–07
DRAWN BY:	T. PHAM		8–10–07
CHECKED BY:	B. SUREN		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

STAGE VIII RAISE EMBANKMENT TO WING WALL BEND

JOB NO. 33757098.00001

DRAWING NO. TDC–14 REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–14.dwg Nov 29, 2007 – 1:15pm

N
W E

SPILLWAY

ROCK DRAIN AND PUMPING CHAMBERS

SEEPAGE COLLECTION DAM

STAGE IX RAISE DOWN STREAM TOE (3H:1V)

STAGE VI AXIS
STA. 25+60
N 5144278.2
E 583290.5

EMBANKMENT TO WING WALL BEND CONNECTION ALIGNMENT ALTERNATIVES
- ALTERNATIVE 1
- ALTERNATIVE 2
- ALTERNATIVE 3

PIPE CROSSING CULVERT

SLOPE TRANSITION

EMBANKMENT / WING WALL BEND
DOWNSTREAM SLOPE 2.5H:1V
UPSTREAM SLOPE 1.5H:1V

ZINC THICKENER

FUTURE WING WALL

STAGE IX LINER

STAGE IX WING WALL DOWN STREAM TOE (3H:1V)

CURTAIN WALL TEMINATION POINTS
- STAGE VIIB
- STAGE VIII
- STAGE IX

LIMITS OF BLANKET DRAIN

TALINGS MAIN DAM

EXIST. PIPE BENCH

COFFER DAM EL. 955

EXIST. CUTOFF WALL

TOE OF GEOMEMBRANE AT CUTOFF TRENCH

APPROX. LIMIT OF CUTOFF TRENCH FLOOR

APPROX. LIMIT OF CUTOFF TRENCH

STAGE II CUTOFF WALL

STAGE III CUTOFF WALL

STAGE IV CUTOFF WALL

STAGE VII-B CUTOFF WALL

CURTAIN WALL (STAGE VII-B)

STAGE VI AXIS
STA. 0+00
N5143415.8
E580880.1

EXIST. LINER

NEW LINER

PLUGGED DIVERSION PIPE

EAST RISER PIPE

WEST RISER PIPE

COFFER DAM CREST EL. 960FT

UPSTREAM TOE OF ROCK BUTTRESS

FORMER DUST CONTROL STRUCTURE

TOE DRAIN JUNCTION

EXIST. CUTOFF WALL

STAGE VIII AND IX ACCESS ROAD

STAGE VII-B CUTOFF WALL

STAGE IV CUTOFF WALL

STAGE III CUTOFF WALL

STAGE II CUTOFF WALL

WING WALL ALIGNMENT OPTIONS
- OPTION 5
- OPTION 4
- OPTION 6

(SEE NOTE 3)

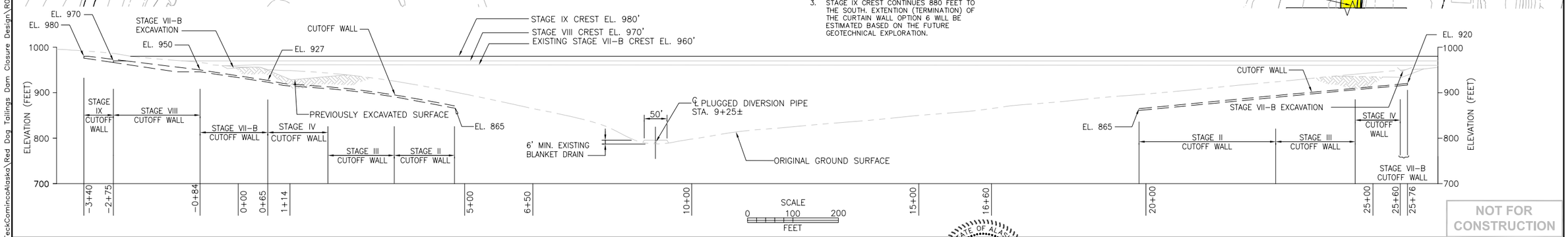
MAIN WASTE SEEPAGE COLLECTION SYSTEM

NOTES:

1. THERE ARE SEVERAL POSSIBLE EMBANKMENT TO WING WALL BEND ALTERNATIVES AND WING WALL TERMINATION ALIGNMENT THAT WILL BE RESOLVED AS DESCRIBED ON DRAWING TDC-7.
2. THE WING WALL ALIGNMENTS CONTAIN A BURIED CURTAIN WALL THAT EXTENDS BEYOND THE CREST OF THE WING WALL.
3. STAGE IX CREST CONTINUES 880 FEET TO THE SOUTH. EXTENTION (TERMINATION) OF THE CURTAIN WALL OPTION 6 WILL BE ESTIMATED BASED ON THE FUTURE GEOTECHNICAL EXPLORATION.

PLAN

SCALE
0 120 240
FEET



NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII-A & VII-B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION

REVISIONS NO.	BY	DATE	DESCRIPTION

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	S. RAJAH		7-13-05
DRAWN BY:	T. PHAM		7-13-05
CHECKED BY:	C. MASALA		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

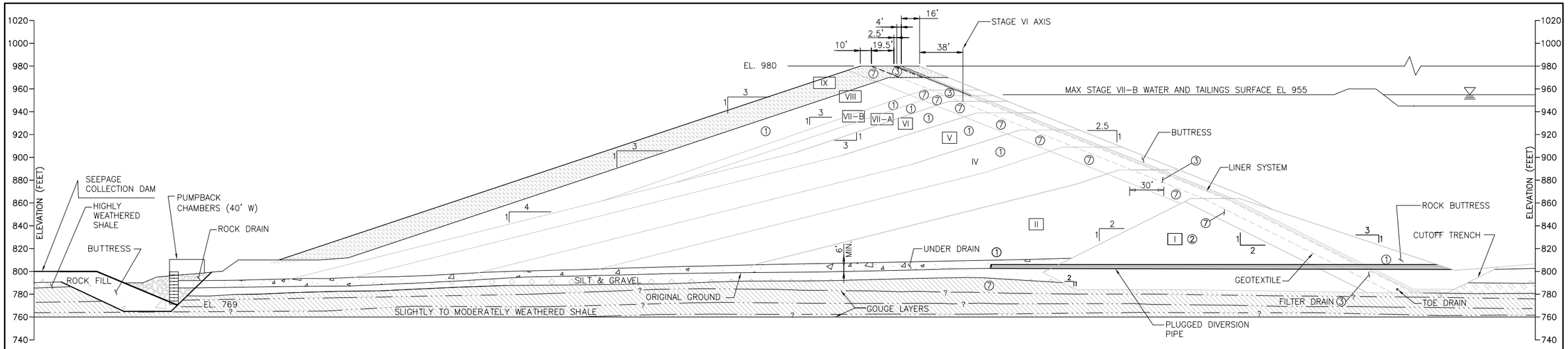
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM, FUTURE RAISES

STAGE IX RAISE CONCEPTUAL PLAN AND PROFILE

JOB NO. 33757098.00001
DRAWING NO. TDC-15
REV. 0

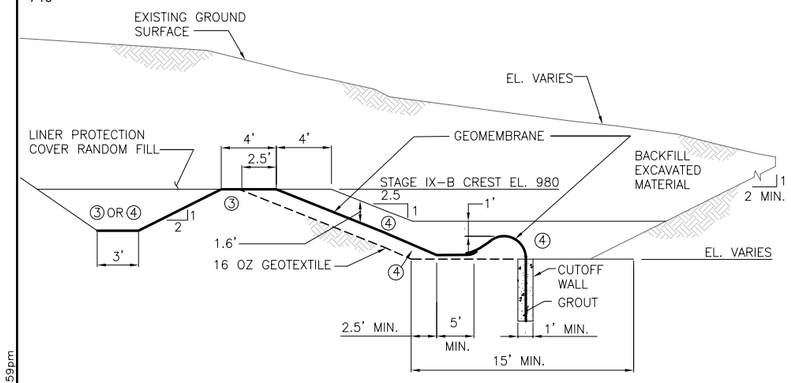
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-15.dwg Nov 16, 2007 - 4:01pm



MAIN DAM SECTION
STATION 8+60

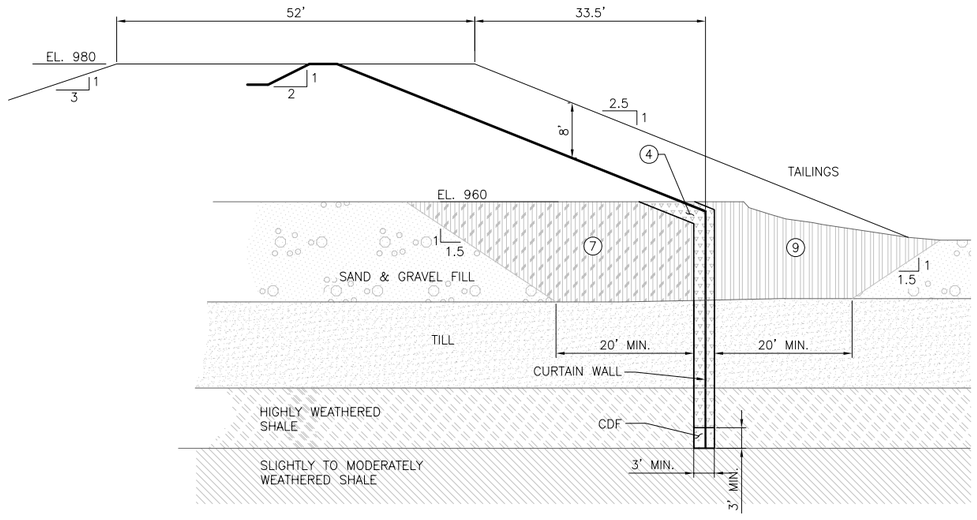
A 15|16

SCALE
0 40 80
FEET



LEFT ABUTMENT SECTION
AT STA. −2+10

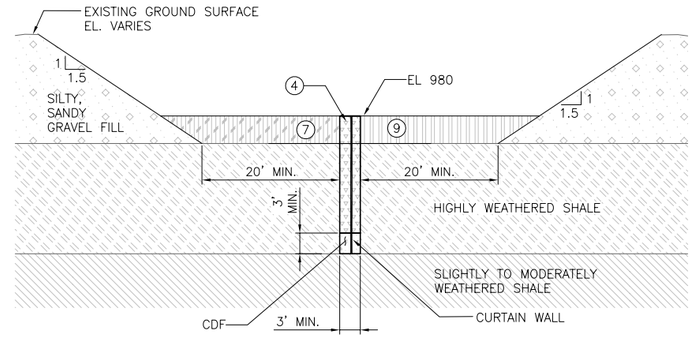
B 15|16 SCALE: NTS



WING WALL SECTION
AT STA. 106+00

D 15|16

SCALE
0 10 20
FEET



WING WALL SECTION
AT STA. 112+80

E 15|16

SCALE
0 10 20
FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			
△				△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

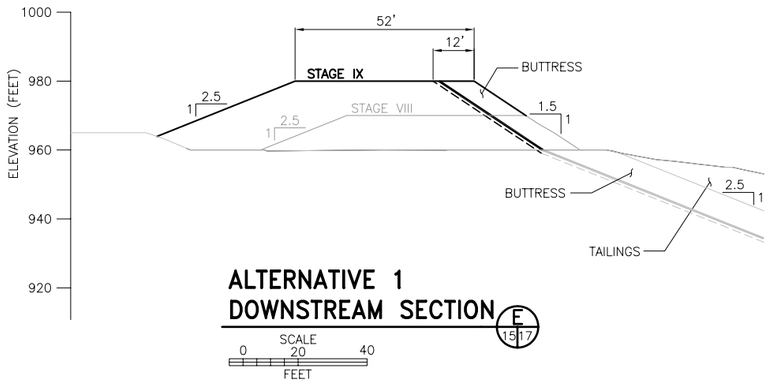
SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.
RED DOG TAILINGS MAIN DAM, FUTURE RAISES
STAGE IX RAISE CONCEPTUAL SECTION

JOB NO. 33757098.00001
DRAWING NO. TDC–16
REV. 0

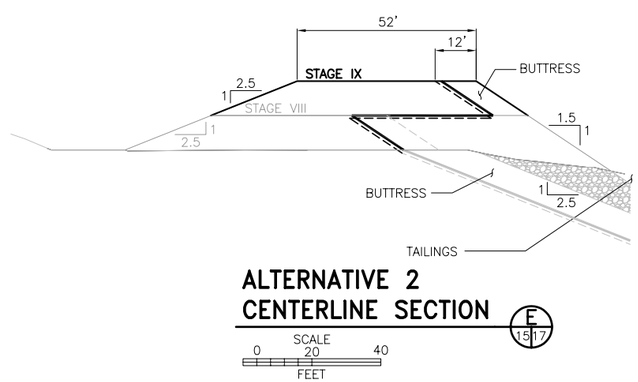
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–16.dwg Nov 16, 2007 – 3:59pm



ALTERNATIVE 1
DOWNSTREAM SECTION

E 15/17

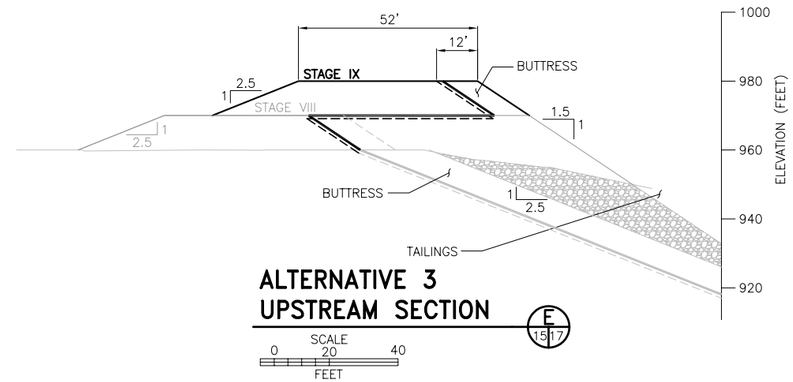
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

E 15/17

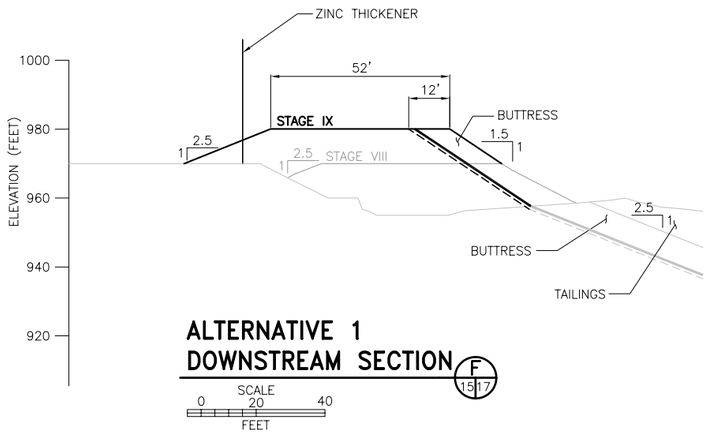
SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

E 15/17

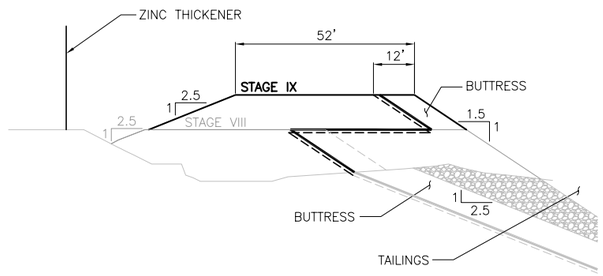
SCALE 0 20 40 FEET



ALTERNATIVE 1
DOWNSTREAM SECTION

F 15/17

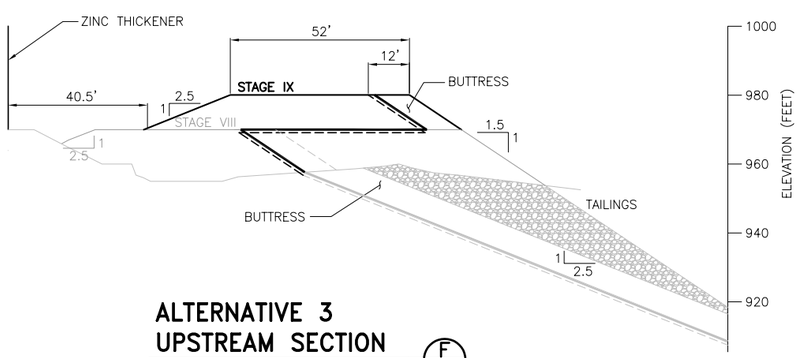
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

F 15/17

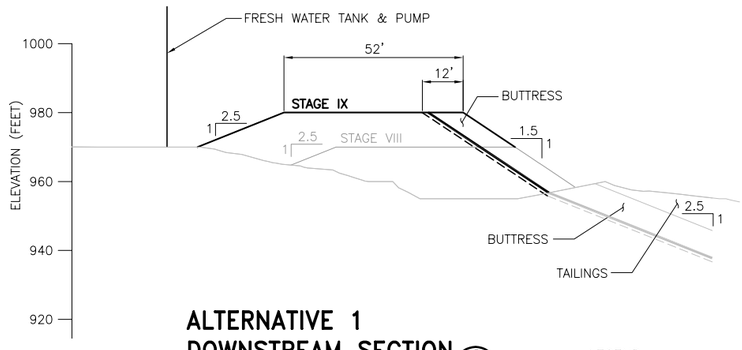
SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

F 15/17

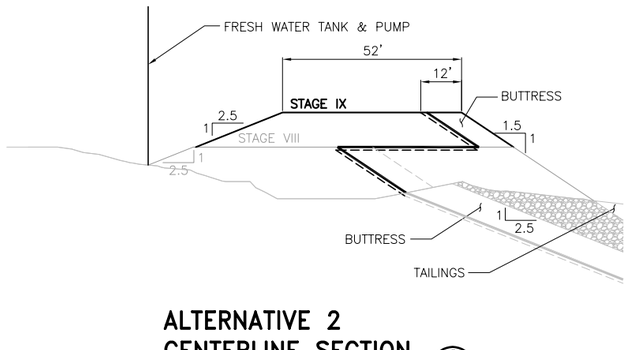
SCALE 0 20 40 FEET



ALTERNATIVE 1
DOWNSTREAM SECTION

G 15/17

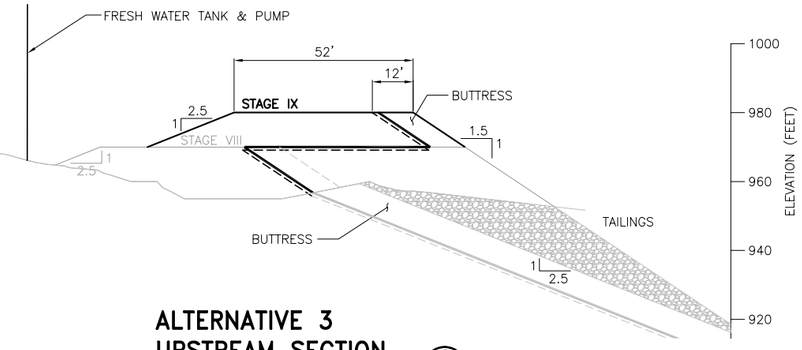
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

G 15/17

SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

G 15/17

SCALE 0 20 40 FEET

LEGEND

EXISTING LINER SYSTEM

FUTURE LINER SYSTEM

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

STATE OF ALASKA 49TH — CECIL M. URLICH CE–11095 — REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	C. MASALA		8–10–07
DRAWN BY:	T. PHAM		8–10–07
CHECKED BY:	B. SUREN		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

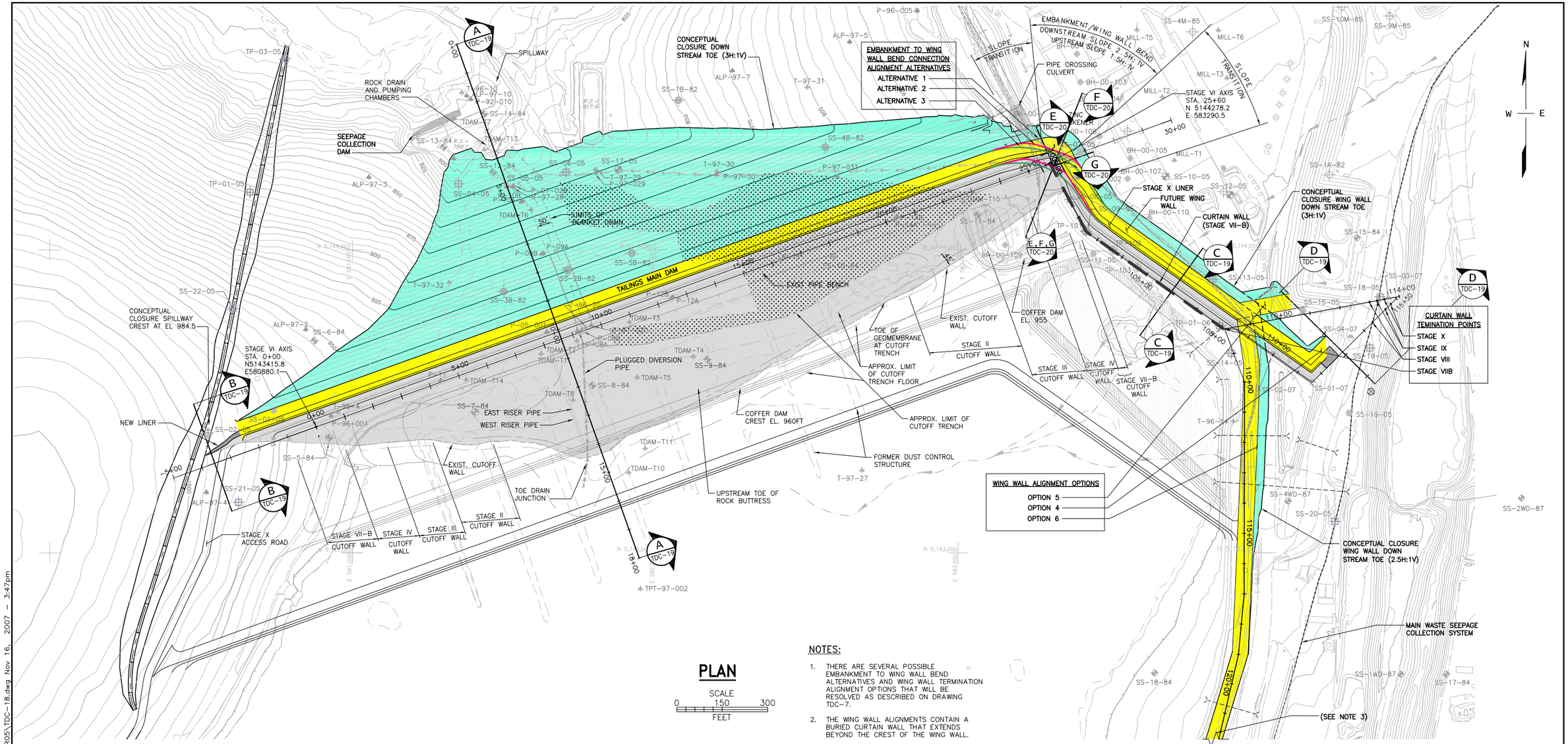
RED DOG TAILINGS MAIN DAM, FUTURE RAISES

STAGE IX RAISE EMBANKMENT TO WING WALL BEND

JOB NO. 33757098.00001

DRAWING NO. TDC–17 REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–17.dwg Nov 29, 2007 – 1:15pm

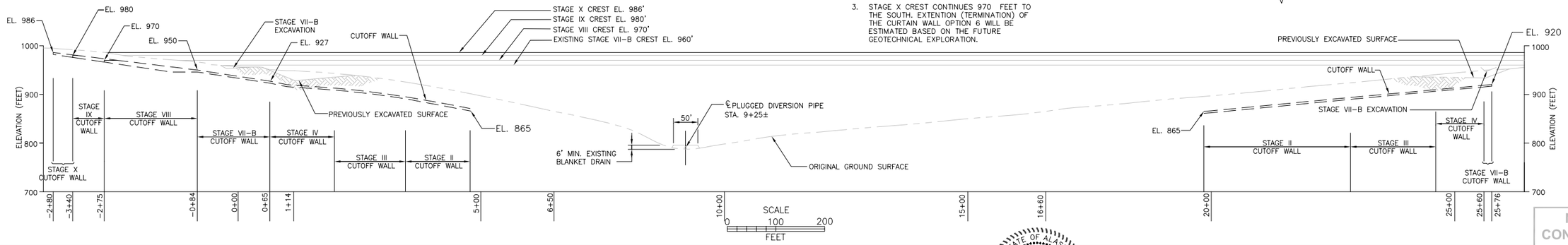


PLAN

SCALE
0 150 300
FEET

NOTES:

1. THERE ARE SEVERAL POSSIBLE EMBANKMENT TO WING WALL BEND ALTERNATIVES AND WING WALL TERMINATION ALIGNMENT OPTIONS THAT WILL BE RESOLVED AS DESCRIBED ON DRAWING TDC–7.
2. THE WING WALL ALIGNMENTS CONTAIN A BURIED CURTAIN WALL THAT EXTENDS BEYOND THE CREST OF THE WING WALL.
3. STAGE X CREST CONTINUES 970 FEET TO THE SOUTH. EXTENTION (TERMINATION) OF THE CURTAIN WALL OPTION 6 WILL BE ESTIMATED BASED ON THE FUTURE GEOTECHNICAL EXPLORATION.



NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION

REVISIONS NO.	BY	DATE	DESCRIPTION

STATE OF ALASKA 49TH — CECIL M. URLICH CE–11095 — REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED	INIT	DATE
DESIGNED BY: S. RAJAH		7–13–05
DRAWN BY: T. PHAM		7–13–05
CHECKED BY: C. MASALA		10–26–07
APPROVED BY: C. URLICH		11–02–07
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

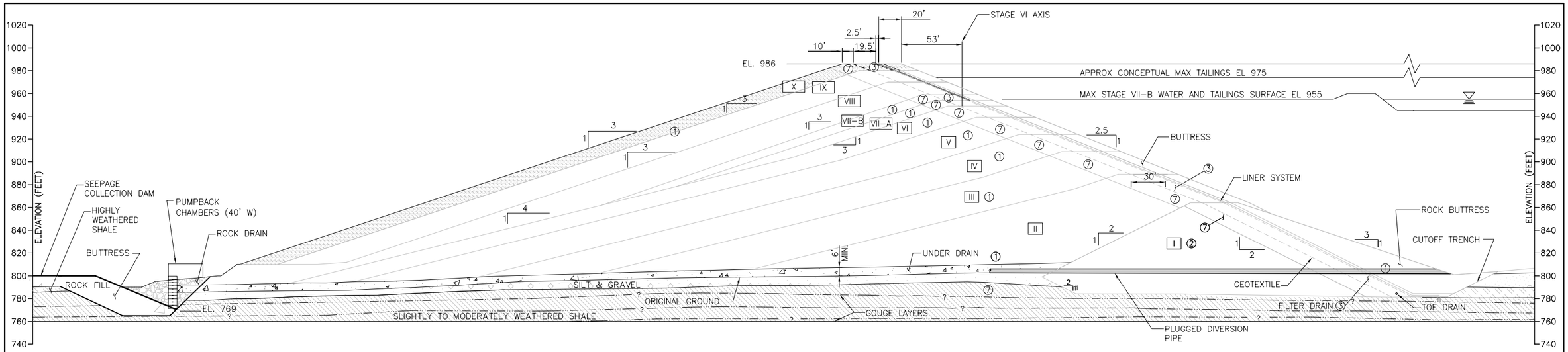
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

STAGE X RAISE CONCEPTUAL PLAN AND PROFILE

JOB NO. 33757098.00001
DRAWING NO. TDC–18
REV. 0

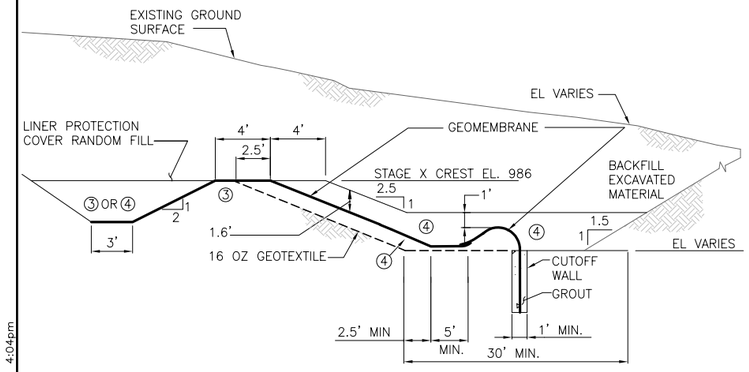
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–18.dwg Nov 16, 2007 – 3:47pm



MAIN DAM SECTION
STATION 8+60

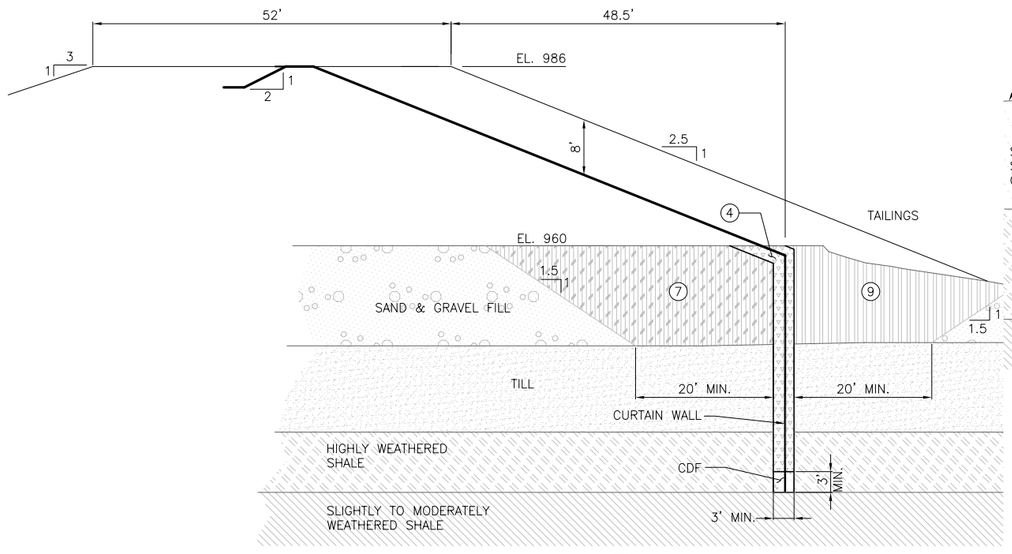
A 17 18

SCALE 0 40 80 FEET



LEFT ABUTMENT SECTION
AT STA. −3+60

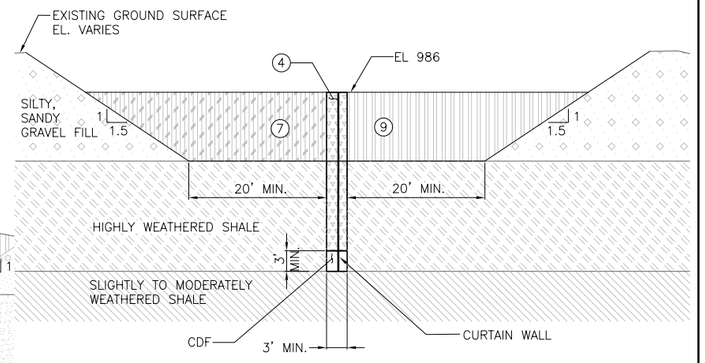
B 17 18 SCALE: NTS



WING WALL SECTION
AT STA. 106+00

C 17 18

SCALE 0 10 20 FEET



WING WALL SECTION
AT STA. 113+00

D 17 18

SCALE 0 10 20 FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES	REVISIONS				REVISIONS			
TITLE	TITLE	NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.	△				△			
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.		△				△			

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

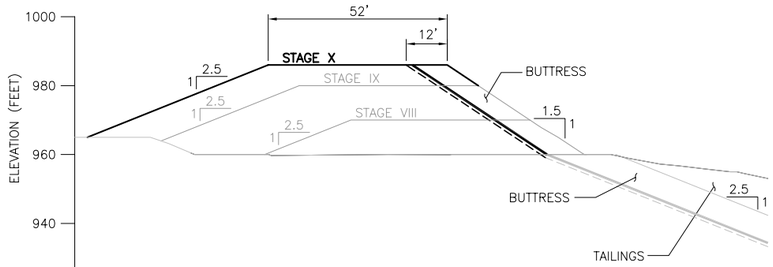
SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	S. RAJAH		7–13–05
DRAWN BY:	T. PHAM		7–13–05
CHECKED BY:	C. MASALA		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.
RED DOG TAILINGS MAIN DAM, FUTURE RAISES
STAGE X RAISE CONCEPTUAL SECTION

JOB NO. 33757098.00001
DRAWING NO. TDC–19
REV. 0

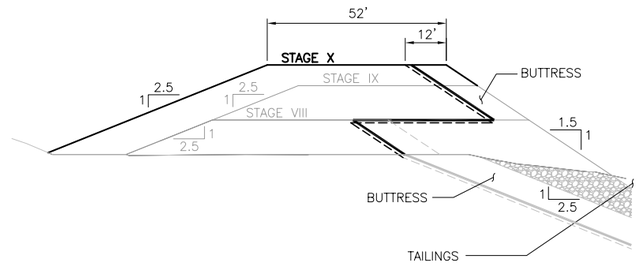
P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–19.dwg Nov 16, 2007 – 4:04pm



ALTERNATIVE 1
DOWNSTREAM SECTION

E 18|20

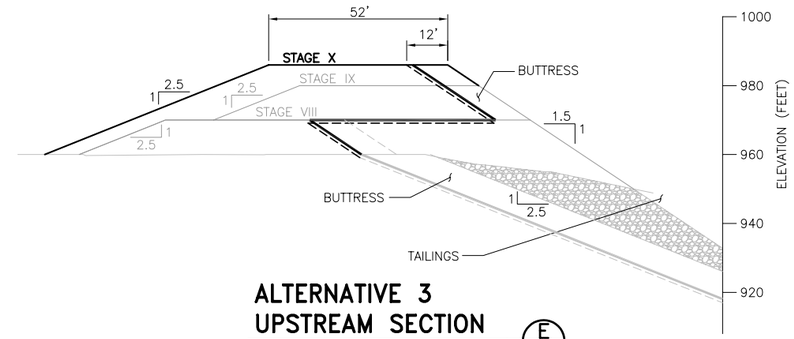
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

E 18|20

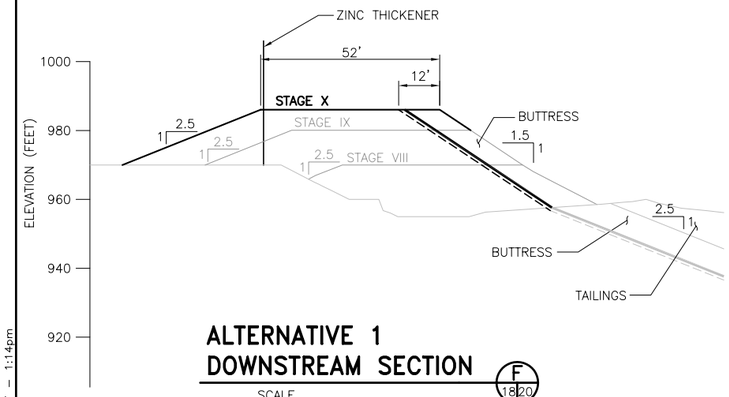
SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

E 18|20

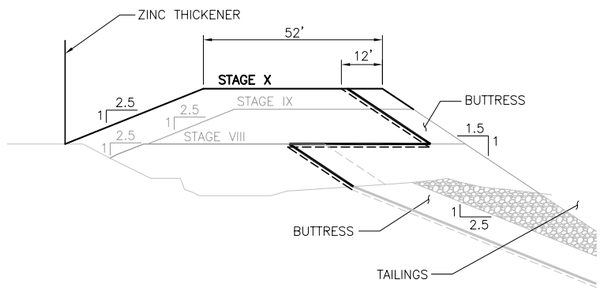
SCALE 0 20 40 FEET



ALTERNATIVE 1
DOWNSTREAM SECTION

F 18|20

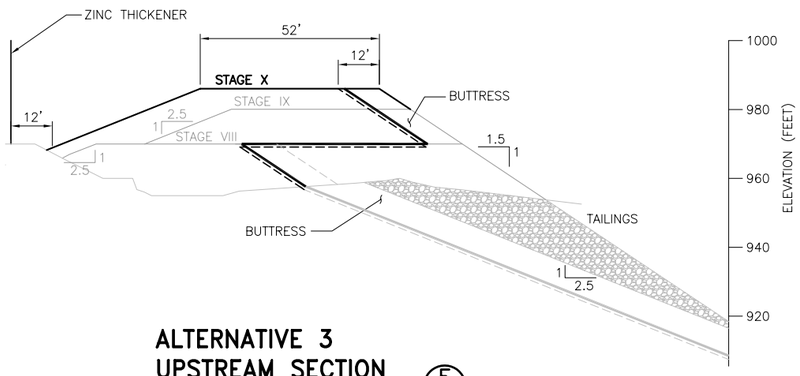
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

F 18|20

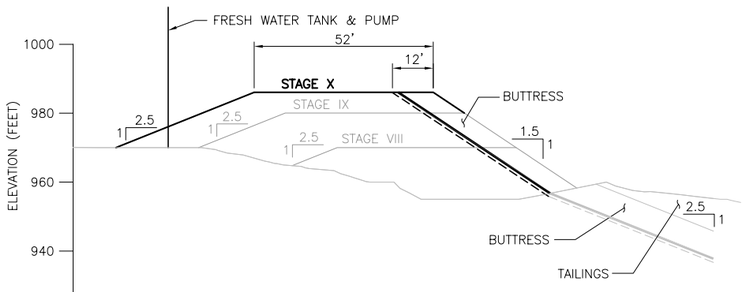
SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

F 18|20

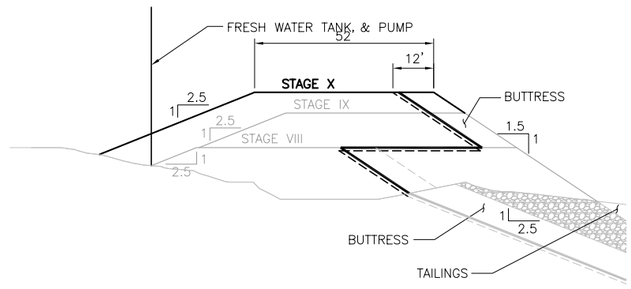
SCALE 0 20 40 FEET



ALTERNATIVE 1
DOWNSTREAM SECTION

G 18|20

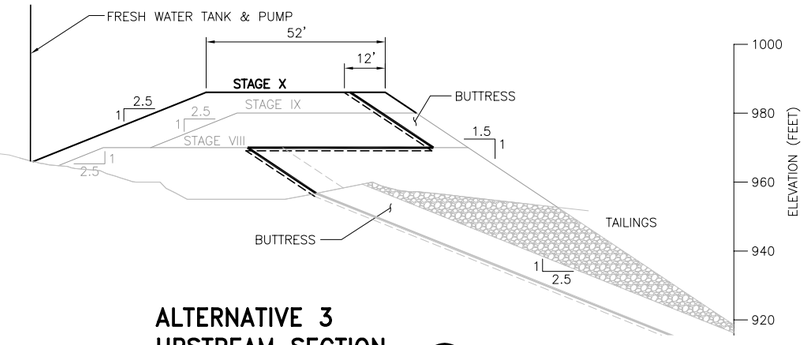
SCALE 0 20 40 FEET



ALTERNATIVE 2
CENTERLINE SECTION

G 18|20

SCALE 0 20 40 FEET



ALTERNATIVE 3
UPSTREAM SECTION

G 18|20

SCALE 0 20 40 FEET

LEGEND

EXISTING LINER SYSTEM

FUTURE LINER SYSTEM

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES	REVISIONS				REVISIONS			
TITLE	TITLE	NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.	△				△			
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.		△				△			

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	C. MASALA		8–10–07
DRAWN BY:	T. PHAM		8–10–07
CHECKED BY:	B. SUREN		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

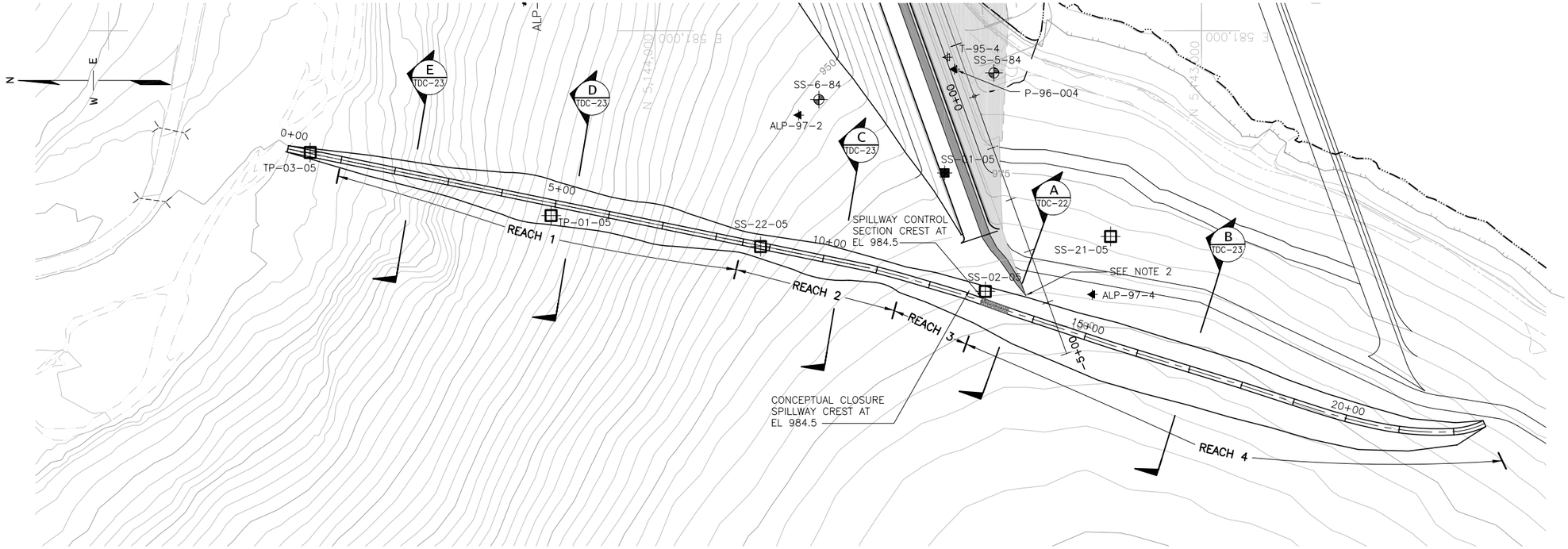
RED DOG TAILINGS MAIN DAM, FUTURE RAISES

STAGE X RAISE EMBANKMENT TO WING WALL BEND

JOB NO. 33757098.00001

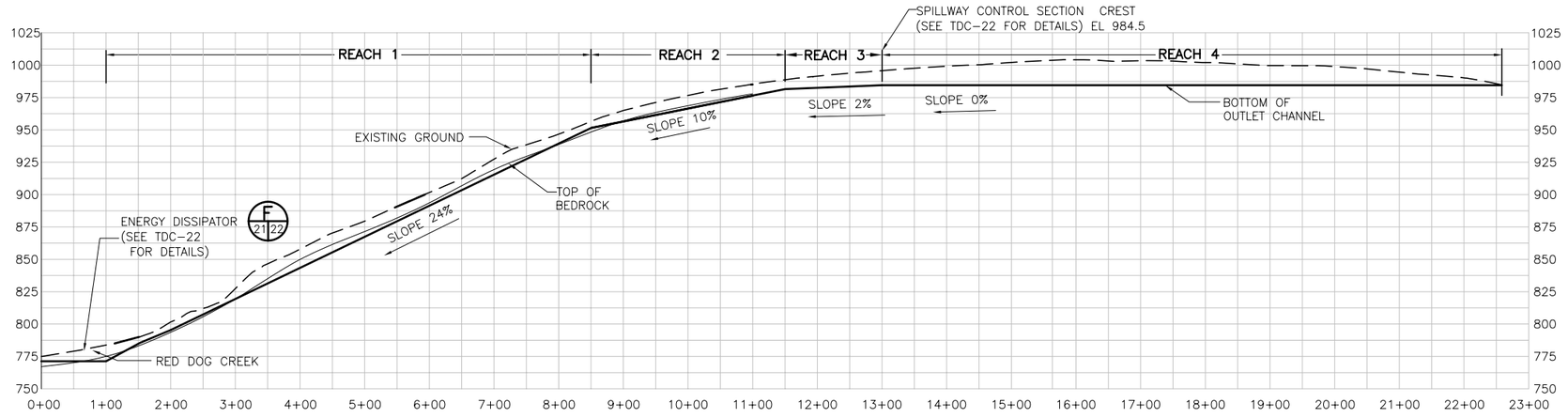
DRAWING NO. TDC–20 | REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–20.dwg Nov. 29, 2007 – 1:14pm



PLAN

SCALE
0 100 200
FEET



PROFILE

HORIZ. SCALE:
0 100 200
FEET

VER. SCALE:
0 50 100
FEET

NOTES:

1. PROFILE OF REACH 4 IS NOT SHOWN BECAUSE THE BOTTOM SLOPE IS 0%.
2. THE EFFECT OF THE PROXIMITY OF THE SPILLWAY CHANNEL AND THE LINER ON THE LEFT ABUTMENT WILL BE DETERMINED LATER.

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES
TITLE	TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS				REVISIONS			
NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
△				△			
△				△			
△				△			
△				△			

STATE OF ALASKA 49TH
CECIL M. URLICH
CE-11095
REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT	DATE
DESIGNED BY:	C. MASALA		01–18–06
DRAWN BY:	T. PHAM		01–18–06
CHECKED BY:	C. TALICH		10–26–07
APPROVED BY:	C. URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

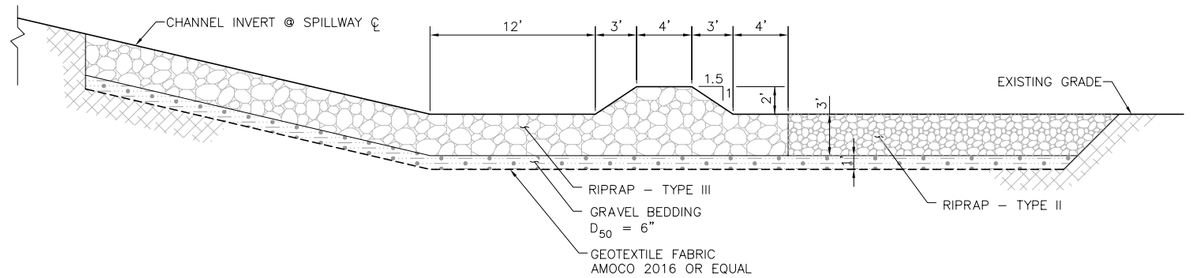
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

SPILLWAY CONCEPTUAL PLAN & PROFILE

JOB NO. 33757098.00001
DRAWING NO. TDC–21
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–21.dwg Nov 16, 2007 – 3:56pm

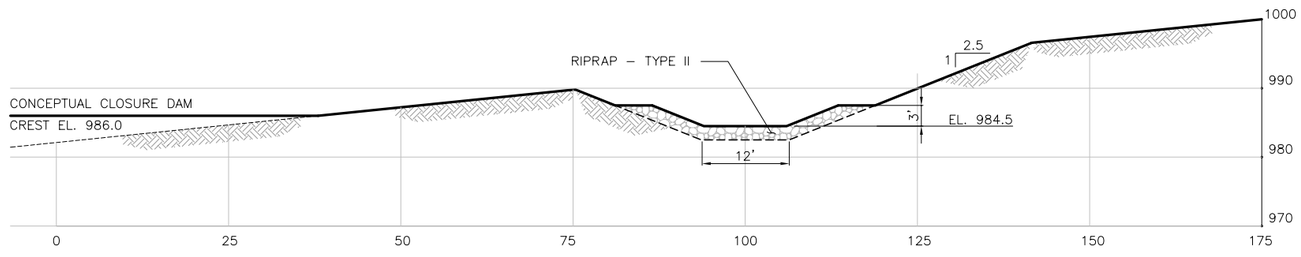


PROFILE

ENERGY DISSIPATOR DETAIL (F 21|22)

NTS

SOIL TYPE	MATERIAL DESCRIPTION
RIPRAP	SELECT MATERIAL CONSISTING OF BLASTED ROCK. THE ROCK SHALL BE DURABLE, BLOCKY, ANGULAR, AND HAVE A MAXIMUM LENGTH TO WIDTH RATIO OF 2. FLAT OR ELONGATED ROCK SHALL NOT BE USED.
	TYPE I – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO THREE (3) INCHES.
	TYPE II – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO SIX (6) INCHES.
	TYPE III – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO TWELVE (12) INCHES.



SPILLWAY CREST CROSS SECTION (A 21|22)

HORIZ. & VER. SCALE:
0 10 20
FEET

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGES I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC. RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. TOPOGRAPHY SEPT. 2004
2. GOLDER REPORT "DETAILED DESIGN TAILINGS IMPOUNDMENT – BACK DAM, 2003"	

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	C. MASALA		4–08–05
DRAWN BY:	T. PHAM		4–08–05
CHECKED BY:	C. TALICH		10–26–07
APPROVED BY:	C URLICH		11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

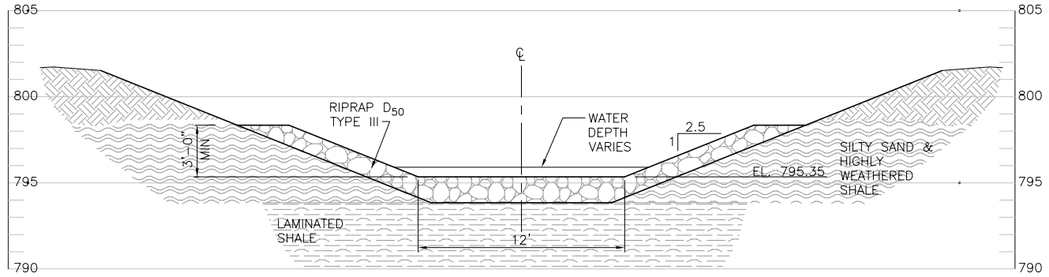
TECK COMINCO ALASKA INC.

RED DOG MINE TAILINGS CONCEPTUAL CLOSURE

SPILLWAY CONCEPTUAL CREST AND STILLING BASIN DETAILS

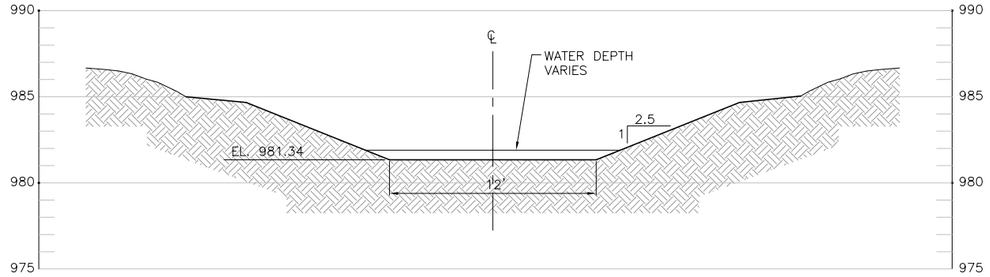
JOB NO. 33757098.00001
DRAWING NO. TDC–22
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–22.dwg Nov 16, 2007 – 3:55pm



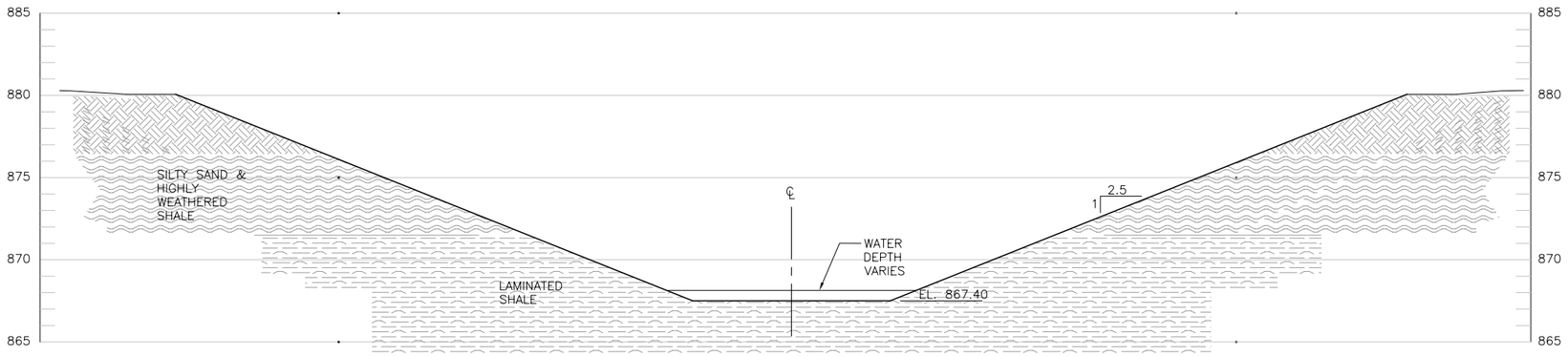
STA 2+00

SPILLWAY OUTLET CHANNEL CROSS SECTION (E 2123)



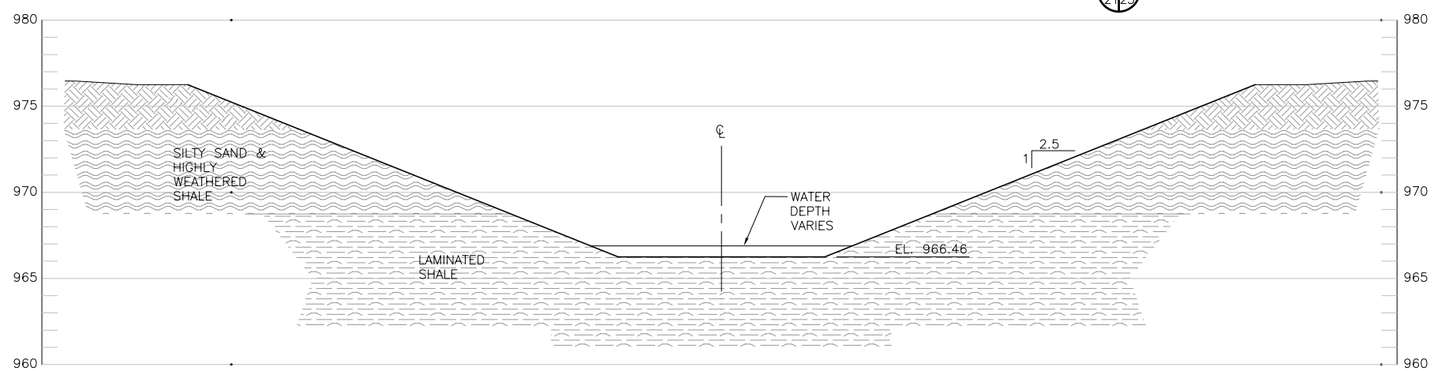
STA 17+00

SPILLWAY UPSTREAM CHANNEL CROSS SECTION (B 2123)



STA 5+00

SPILLWAY OUTLET CHANNEL CROSS SECTION (D 2123)



STA 10+00

SPILLWAY OUTLET CHANNEL CROSS SECTION (C 2123)

STA.	TOP OF OUTLET CHANNEL (NATURAL GROUND) EL. (FT)	BOTTOM OF OUTLET CHANNEL EL. (FT)	DIFFERENCE IN EL. (FT)
0+00	775.00	771.28	3.72
1+00	783.00	771.28	11.72
2+00	801.48	795.35	6.13
3+00	825.66	819.35	6.31
4+00	858.32	843.37	14.95
5+00	880.07	867.40	12.67
6+00	901.68	891.39	10.29
7+00	926.57	915.40	11.17
8+00	946.44	939.41	7.03
9+00	963.46	956.47	6.99
10+00	976.65	966.46	10.19
11+00	985.39	976.47	8.92
12+00	991.53	982.50	9.03
13+00	995.74	984.50	11.24
14+00	999.20	984.50	14.7
15+00	1002.08	984.50	17.58
16+00	1004.15	984.50	19.65
17+00	1003.46	984.50	18.96
18+00	1002.02	984.50	17.52
19+00	999.69	984.50	15.19
20+00	998.94	984.50	14.44
21+00	994.60	984.50	10.10
22+00	990.22	984.50	5.72
22+58.5	984.50	984.50	0.00

NOTE:

1. DUE TO VERY LOW FLOW DEPTH IN THE OUTLET CHANNEL 0.34 TO 1.1 FEET, EROSION PROTECTION IS NOT REQUIRED MORE THAN 3 FEET ABOVE THE CHANNEL BOTTOM.

SOIL TYPE	MATERIAL DESCRIPTION
RIPRAP	SELECT MATERIAL CONSISTING OF BLASTED ROCK. THE ROCK SHALL BE DURABLE, BLOCKY, ANGULAR, AND HAVE A MAXIMUM LENGTH TO WIDTH RATIO OF 2. FLAT OR ELONGATED ROCK SHALL NOT BE USED.
	TYPE I – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO THREE (3) INCHES.
	TYPE II – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO SIX (6) INCHES.
	TYPE III – ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO TWELVE (12) INCHES.

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. TOPOGRAPHY SEPT. 2004
2. GOLDER REPORT "DETAILED DESIGN TAILINGS IMPOUNDMENT – BACK DAM, 2003"	

REVISIONS NO.	BY	DATE	DESCRIPTION
△			
△			
△			
△			
△			

REVISIONS NO.	BY	DATE	DESCRIPTION
△			
△			
△			
△			
△			

STATE OF ALASKA 49TH — CECIL M. URLICH CE-11095 — REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED	INIT	DATE
DESIGNED BY: C. MASALA		01-18-06
DRAWN BY: T. PHAM		01-18-06
CHECKED BY: C. TALICH		10-26-07
APPROVED BY: C. URLICH		11-02-07
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

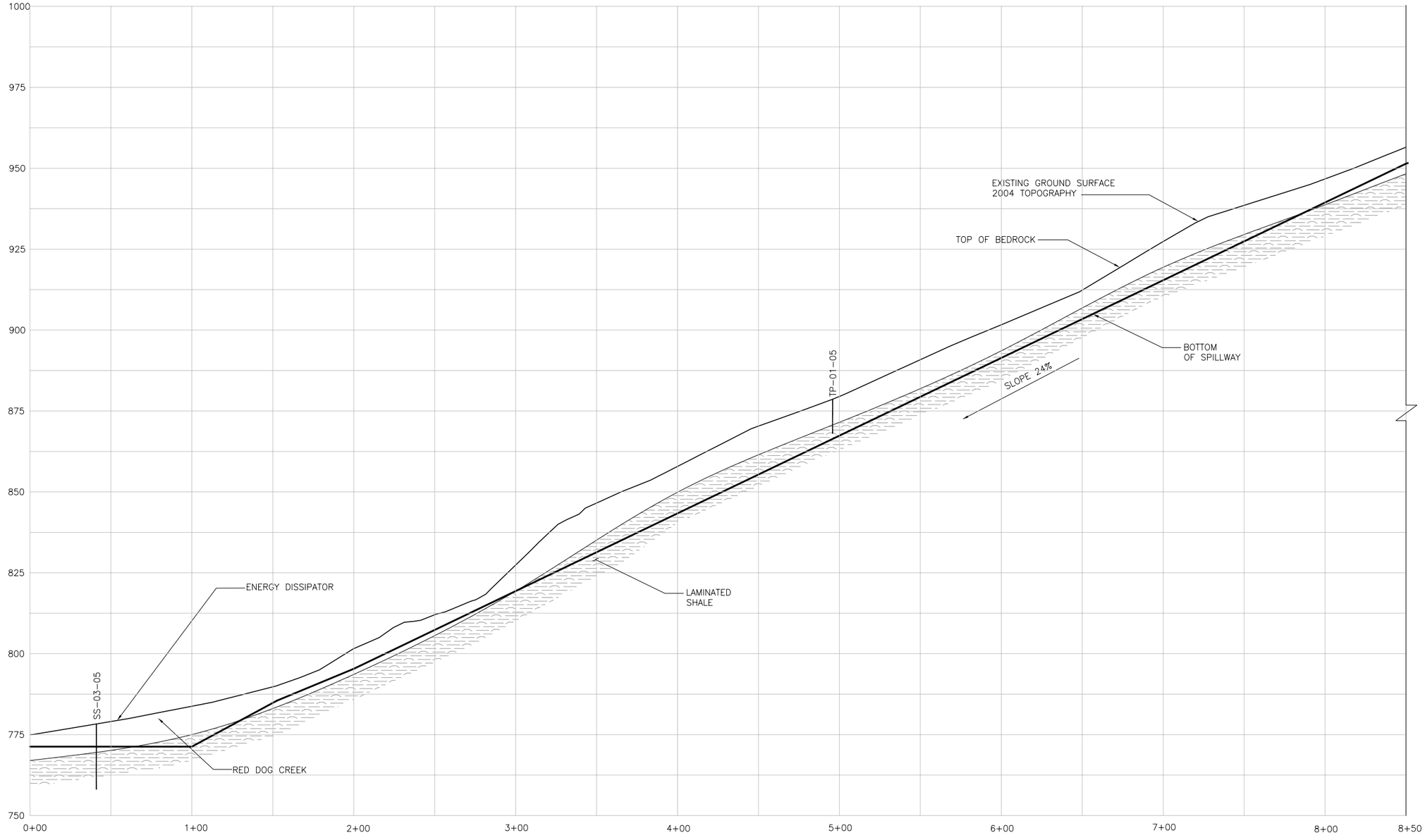
SPILLWAY CONCEPTUAL SECTIONS AND DETAILS

JOB NO. 33757098.00001

DRAWING NO. TDC-23

REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-23.dwg Nov 16, 2007 - 3:54pm



PROFILE – REACH 1: STA 1+00 TO STA 8+50

HORIZ. SCALE
0 30 60
FEET

VER. SCALE
0 15 30
FEET

NOT FOR CONSTRUCTION

REFERENCES TITLE	REFERENCES TITLE
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.	

REVISIONS NO.	BY	DATE	DESCRIPTION
1			
2			
3			
4			
5			

REVISIONS NO.	BY	DATE	DESCRIPTION
6			
7			
8			
9			
10			

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

		INIT	DATE
SCALE:	AS NOTED		
DESIGNED BY:	C. MASALA		01-18-06
DRAWN BY:	T. PHAM		01-18-06
CHECKED BY:	C. TALICH		10-26-07
APPROVED BY:	C. URLICH		11-02-07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101-1616
Telephone (206) 438-2700

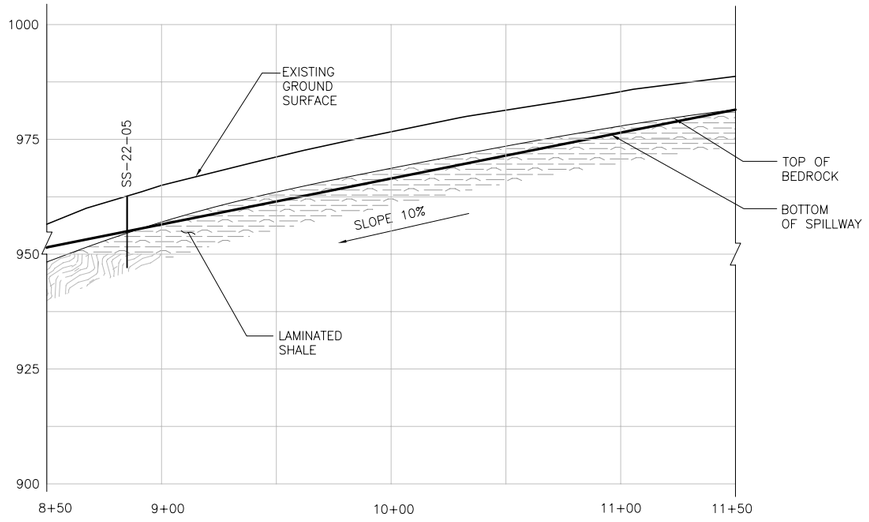
TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

SPILLWAY REACH 1 CONCEPTUAL PROFILE

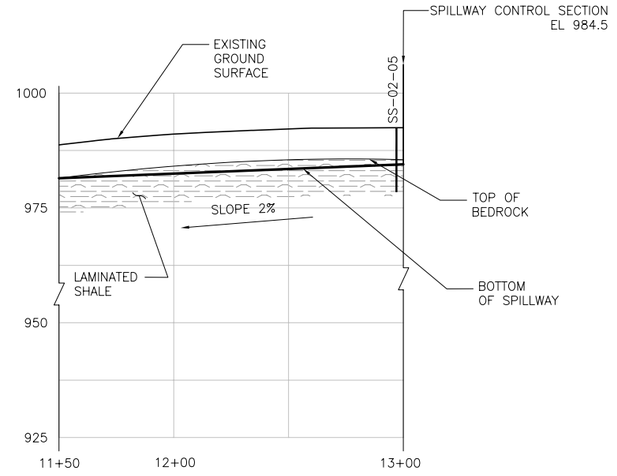
JOB NO. 33757098.00001
DRAWING NO. TDC-24
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC-24.dwg Nov 16, 2007 - 3:53pm



PROFILE – REACH 2: STA 8+50 TO STA 11+50

HORIZ. SCALE
0 30 60
FEET
VER. SCALE
0 15 30
FEET



PROFILE – REACH 3: STA 11+50 TO STA 13+00

HORIZ. SCALE
0 30 60
FEET
VER. SCALE
0 15 30
FEET

NOT FOR CONSTRUCTION

REFERENCES	REFERENCES	REVISIONS				REVISIONS			
TITLE	TITLE	NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
1. CONSTRUCTION DRAWINGS, TAILINGS IMPOUNDMENT DAM, STAGE I–VI, BY DAMES & MOORE, FOR COMINCO ALASKA, INC., RED DOG MINE, ALASKA, 1986 WITH REVISIONS THROUGH 1995.	3. CONSTRUCTION COMPLETION REPORT, STAGE VII–A & VII–B RAISE, RED DOG TAILINGS DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2004 & 2007.	△				△			
2. TECK COMINCO ALASKA, INC. "CONTOUR DRAWINGS SHOWING COMINCO TOPOGRAPHY" SEPTEMBER 16, 2004.		△				△			

STATE OF ALASKA 49TH
CECIL M. URLICH CE-11095
REGISTERED PROFESSIONAL ENGINEER

SCALE: AS NOTED		INIT	DATE
DESIGNED BY: C. MASALA			01–18–06
DRAWN BY: T. PHAM			01–18–06
CHECKED BY: C. TALICH			10–26–07
APPROVED BY: C. URLICH			11–02–07
CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

TECK COMINCO ALASKA INC.

RED DOG TAILINGS MAIN DAM AT CLOSURE

SPILLWAY REACH 2 & 3
CONCEPTUAL PROFILES

JOB NO. 33757098.00001
DRAWING NO. TDC–25
REV. 0

P:\ACAD\PROJECT\TeckCominco Alaska\Red Dog Tailings Dam Closure Design\R05\TDC–25.dwg Nov 16, 2007 – 3:58pm

Appendix A

Memo

Century Square
1501 4th Avenue, Suite 1400
Seattle, Washington 98101
206.438.2700 Telephone
206.438.2699 Fax

To:	Kathleen Willman, Teck Cominco Alaska, Inc.	**Info:**	
From:	Sri Rajah, URS Cecil Urlich, URS	**Date:**	October 22, 2004
RE:	Review of Red Dog Mine Tailings Audit Report by BGC Engineering Inc. URS Project No: 33757098		

This memo summarizes the URS review of the report "Red Dog Mine, Cominco Alaska, Tailings Audit" dated February 9, 2001, by Iain G. Bruce of BGC Engineering Inc. The audit was completed as part of a corporate risk assessment of the mine property, and the report summarizes the results of a mine visit from November 17 to 20, 2000. The URS review also reflects observations made by Cecil Urlich at the mine during a dam Periodic Safety Inspection (PSI) from October 18 to 22, 2004.

INTRODUCTION

The audit report is organized into an Introduction and the following five sections:

- Management Systems
- Tailing Pond Design and Construction Practices
- Water Balance and Tailings Deposition Planning
- Tailing Pond Monitoring Practices and Procedures
- Tailing Pond Closure Planning:

Each section after the Introduction describes TCAK efforts being made at the time of the visit and provides suggestions for improvement. The URS review comments are presented below for each section.

MANAGEMENT SYSTEMS:

Management system improvements suggested by BGC can be categorized as follows:

- Define responsibility and lines of responsibility
- Form a tailings management committee (TMC)
- Complete long-term closure plans
- Modify the Operation and Maintenance (O&M) manual
- Prepare and practice an Emergency Response Plan (ERP).

The responsibility for the tailings system is defined in the O&M manual to be a mine staff member designated as Responsible Party. Mr. Jim Swendseid, P.E., is the Responsible Party and has been since before the audit. The manual describes the responsibilities and required qualifications of the Responsible Party, and provides guidance on support to be provided by TCAK to the Responsible Party.

It is a good idea for a TMC of mill, mining and environmental department staff to meet regularly. Meetings would enhance the interaction of operation and closure plans, and resolve operation practice and closure requirement conflicts. However, closure options are now being evaluated, so operation and closure plans cannot interact because the closure plan is not final. Meanwhile, URS has observed open communications and understandings between the departments and an awareness of objectives. Therefore, URS recommends that a TMC is not needed and that current communication practices should continue.

A long-term closure plan is being developed by TCAK for the mine that includes the tailings facility. The current tailings closure plan is to flood the pond with a minimum water depth of two feet. Other closure options are now being evaluated. They include flooding the pond with a greater depth of water, changing the surface topography by depositing paste tailings, and constructing a wet or dry soil cover.

The current O&M manual was completed in April 2004 as Revision 3 of a manual started in 1989. Revision 4 will be completed early in 2005 after the 2004 PSI. O&M guidelines are provided for the tailings dam, seepage collection dam and pump-back system. The manual includes a Project Data Sheet, sections on Project Description, Responsibility, Routine Observations and Maintenance, Instrumentation Monitoring, and Emergency Action Plan, and Appendices. The manual is not intended to provide a complete management system and deposition plan. However, URS has observed the deposition plan to be well thought out relative to filling low areas and addressing seasonal issues such as dust and freezing.

An Emergency Action Plan (EAP) is Section 6.0 of the O&M manual. It includes sections titled Unusual Occurrences and Procedures, and Emergency Action Planning. An Emergency Situation Notification flow chart identifies TCAK entities (Responsible Party and Incident Command Center), and others (State Dam Safety and Construction Unit, URS, downstream developments, and press and news media).

TAILINGS POND DESIGN AND CONSTRUCTION PRACTICES:

BGC suggests the following improvements in tailings pond design and construction practices:

- Evaluate effect of residual strength of bedrock on dam stability
- Review the seismic criteria in light of recent seismic data
- Establish trigger levels for potential piezometric levels
- Evaluate need for dam deformation monitoring and analysis
- Provide mitigation for potential tailings spill at culvert outlet.

Specific BGC comments on each of the above and the URS response are summarized as follows:

Residual Strength of Shale Bedrock

BGC suggests that the effect of residual strength of shale on dam stability should be re-assessed because of evidence of movement in the form of shears and slickensides. BGC recommended the following:

- Assess a residual value even if it is not an appropriate value to use in the foundation
- Confirm that a weak layer does not underlie the dam in an adverse orientation
- Complete additional stability analyses using residual friction for raises and steeper slopes
- Complete geotechnical logs of boreholes drilled for piezometers or instruments

As a result of the geotechnical investigations for Stages I to VI, Dames & Moore (1986) established the residual strength of the gouge zones in the shale as 24 degrees, and noted large variations in bedding orientation as a result of intense folding and bending of the bedrock. The residual strength was based on

one consolidated direct shear test on the gouge material. URS considers this to be adequate for dam design up to Stage VII, but it should be re-evaluated for the higher and steeper closure raises.

A weak layer in an adverse orientation was assumed under the dam for all stability analyses. The layer is a 6-inch thick gouge zone 15 feet below original ground surface, at the interface between ice-rich and ice-free bedrock. The assumed gouge orientation is the least favorable and provides conservative results. The Dames & Moore (1986) stability analyses estimated an allowable internal friction angle of 6 degrees for the gouge and a seismic safety factor of 1.0 under drained loading. The foundation was found to be acceptable for Stage IV because the measured residual shear strength was higher than 6 degrees.

URS (2002) used a similar stability analysis approach for the Stage VII raise with a steeper slope, and calculated an internal friction angle of 15 degrees and seismic factor of safety of 1.0 for a Maximum Considered Earthquake (MCE). This required internal friction value is much lower than the established residual strength of 24 degrees. In addition, Stage VII is in the foundation footprint of Stage VI. Therefore, URS concluded that the Stage VII dam and foundation were stable, and that no additional work is needed to further investigate the weak pre-sheared zones under the dam for Stage VII stability.

The closure design requires a higher dam and an extension of the dam beyond the current toe. URS (2004) stability analyses show that the final dam can have a steeper slope subject to geotechnical drilling to identify the weakest zones under the new footprint. Meanwhile, any instrumentation drilling should be completed as geotechnical borings and sealed to prevent a surface water to permafrost connection.

Seismicity and Seismic Design Criteria

For Stage I to VI design, the dam was rated as Class III with low hazard potential. For stability analyses, Dames & Moore (1987) established seismic criteria based on National Research Council (1983) seismic hazard map and guidelines, and developed the following horizontal Peak Ground Accelerations (PGA):

- PGA of 0.05g corresponding to an Operating Basis Earthquake (OBE) with a return period of 475 years or a 10% chance of being exceeded in 50 years.
- PGA of 0.10g corresponding to Maximum Credible Earthquake (MCE) that often is assumed to have a return period of 10,000 years.

The Dames & Moore (1987 and 1999) seismic stability analyses used a pseudo-static approach with reduced undrained shear strength and constant seismic lateral force associated with a horizontal seismic coefficient equal to the PGAs described above for OBE and MCE conditions.

By the time of the Stage VII design, the dam was upgraded to Class II with significant hazard potential. For the stability analyses, URS (2002) established new seismic criteria based on USGS hazard maps published for Alaska (Wesson et al, 1999), and developed design criteria and PGAs as follows:

- PGA of 0.10g corresponding to the OBE. A 475-year seismic event with a 10% probability of being exceeded in 50 years was taken as the OBE seismic hazard.
- PGA of 0.20g corresponding to the MCE. A 2,475-year seismic event with a 2% probability of being exceeded in 50 years was taken as the MCE seismic hazard.
- The Maximum Design Earthquake (MDE) was assumed to be the same as the MCE.

The Alaska Dam Safety Program (2003) defines the MDE as "*the greatest earthquake that reasonably could be generated by a specific seismic source, based on seismological and geologic evidence and*

interpretations", and provides a 1,000 to 2,000-year range of return periods for a Class II dam. The MDE described above, provides the most conservative design within standard dam design practice

For Stage VII design, URS (2003) used a pseudo-static approach with reduced undrained shear strength and constant seismic lateral force with seismic coefficient of 50 % of the PGA. This approach was based on studies that showed that earth dams with pseudo-static factors of safety greater than 1.0, obtained by using a seismic lateral force with 50% of the PGA, will not develop large deformations (Hynes-Griffin and Franklin 1984, Kramer 1996, Crouse 2004). URS used the following seismic coefficients for the Stage VII design:

- 0.05g corresponding to the OBE
- 0.10g corresponding to the MCE.

This approach on for Stage VII resulted in the same seismic coefficients as for the Stage I to VI design. The analyses show that Stage VII is stable under seismic loading. However, a review of the seismic criteria and additional stability analyses will be required for the dam at closure because the dam will be higher than Stage VII, the footprint will extend beyond the current toe, and the pond may be flooded.

Trigger Levels for Piezometric Surfaces

The O&M manual Section 5.3.2 identifies trigger levels associated with water levels in piezometers P8, P9 and P10, which are aligned down the slope from the crest to the toe. The trigger levels were specified in previous versions of the manual. For the next update of the manual, URS will list the trigger levels in a separate section, and will require that URS be contacted if the trigger levels are reached.

Dam Deformation Analysis and Monitoring

Section 4.0 of the O&M manual prescribes daily and quarterly inspection procedures. Daily and weekly/ quarterly inspection sheet check lists are in Appendix A. These procedures are being followed by TCAK. Completed check lists include comments on observations of slumps and cracks. The manual also outlines procedures for the Inspector and Responsible Party in the event of any such unexpected observations.

URS considers that deformation analyses and monitoring are not necessary. Bench marks were installed previously on the crest at 200-foot centers, but were buried by the Stage VII-A raise in 2003. Given the good stability history and performance of the dam, and that it is visually inspected daily and traversed several times a day by mine staff, URS recommends against installing bench marks.

In addition, bench marks only indicate movement at the bench mark locations, and not where other movements might occur, such as slumps, cracks and seepage. The daily inspections would notice such movements, as well as any outward movement of the crest. So bench marks could provide a false sense of security. The daily inspections would provide more reliable real time information based on observations.

Potential Tailings Spill at Culvert Outlet

On October 20, 2004, Cecil Urlich and Jim Swendseid inspected the culvert outlet at the east abutment. There is no concern about tailings spills over the dam face because the Stage VII-A construction sloped the ground towards the sand filter coffer dam in the tailings pond. Therefore, any tailings spill will be flow towards the tailings pond. Also, the area is well traveled and a spill would be quickly noticed.

WATER BALANCE AND TAILINGS DEPOSITION PLANNING:

BGC suggests the following water balance and tailings deposition planning improvements:

- Prepare electronic water balance charts to assess monthly water levels
- Consider placing a spillway over the dam to guard against complacency
- Ignore diversion ditches in the water balance for long-term planning
- Include updated evaporation data in new water balance determinations.

Water surface elevations are being generated electronically and used for planning future raises. This data led to the design of Stage VII and construction of Stage VII-A, and is being used for scheduling the design and construction of Stage VII-B, and for closure plans options that are currently being evaluated.

The Stage I to VI design included an emergency spillway as part of the final dam, but included freeboard for the raises so that interim spillways were not needed (Dames & Moore 1987). The spillway was shown to be along the west abutment hillside around the dam in the same way as at the freshwater dam. However, the construction of Stage VII and future raises to closure will push a hillside spillway upslope.

The final spillway could be either along the hillside described above, or across the dam. The best site for a dam spillway appears to be near the west end of the dam for topographic and site impact reasons. The spillway must continue along the hillside and discharge downstream of the seepage collection pond and pump-back system. The discharge chute could be at about the same location as in the original design.

The Stage VII freeboard is 5 feet, which is adequate for the design storm, runoff, waves and wind. URS does not recommend interim spillways to guard against complacency because tailings facility permit is for zero discharge and there is an O&M manual and EAP. It is improper to suggest complacency as a reason for a spillway because TCAK is bound by its “Certificate of Approval to Operate a Dam”.

URS believes that diversion ditches should be used in water balance calculations for long term planning. The ditches were designed and are maintained to reduce runoff into the pond. This is standard practice in tailings operations and closure plans. If the pond is to be flooded in perpetuity, long-term maintenance of the ditches must be in the closure plan, and must be relied on in monitoring the closure system.

The water balance estimates are based on current data that is electronically collected by TCAK, and includes including updated evaporation figures.

TAILINGS POND MONITORING PRACTICES AND PROCEDURES:

BGC suggests the following monitoring practice and procedure improvements:

- Specific trigger levels for underdrain piezometers
- Stability analyses using a range of piezometric surfaces
- Monitor tailing main dam wall deformations.

Section 5.3.2 of the O&M manual presents trigger levels for piezometers P9, P10, and P11. If any of the levels are reached, the O&M manual requires a closer review of the readings, which may include new stability analyses. For the next update of the O&M manual, URS will list the trigger levels in a separate section, and will require that URS be contacted if the trigger levels are reached.

The trigger levels for the piezometer readings in all O&M manuals were determined on the basis of Stage I to VI stability analyses and a range of piezometric surfaces. The trigger levels in the current O&M

manual were determined on the basis of the Stage VII stability analysis and piezometric surfaces, review of historic piezometric readings, and discussions of these readings with the Responsible Party.

Section 4.0 of the O&M manual describes inspection procedures and checklists for monitoring deformations including slumps and cracks. Cracks along the upstream face are documented on the daily and weekly inspection forms, and evaluated. The cracks have been found to be not significant and are typically filled by grading over them. Deformation monitoring is not required as described above.

TAILINGS POND CLOSURE PLANNING:

BGC suggests considering the following elements in the closure plan of the tailings system:

- Potential impact of ARD from tailings and waste rock
- Integrate closure of tailings pond with the rest of the property
- Seismicity for closure if area found to be more seismically active
- Crown tailings and assess other methods of tailings deposition
- HDPE long-term performance and compatibility with acidic water.

ARD impacts from the tailings and mine waste rock have been studied. The closure plan that is being developed is addressing ARD impacts. The flooding option that is being considered for closure of the tailings pond is evaluating a minimum depth of water cover over the tailings ranging from is 2 to 10 feet.

The closure plan is being developed for the mine which includes integration of all components of the property, including the tailings pond, and not looking at the tailings pond in isolation.

URS concludes that the seismic criteria used for the Stage VII design is appropriate for closure. This is discussed above under “Seismicity and Seismic Design Criteria”. URS has completed stability analyses for closure configurations that show the dam is stable even with a flooded pond. Also, we understand that TCAK plans to collect and treat water from the tailings system in perpetuity.

The closure plan that is being developed is considering other methods of tailings deposition and closure methods that would result in changes to the final surface topography of the tailings pond. Methods being considered include subaerial and paste tailings deposition, and wet and dry soil covers for closure.

HDPE has been used for over 25 years for landfills and chemical waste and tailings storage. Extensive testing was completed under USEPA and State scrutiny. In the early 1980s, it was standard to perform USEPA Method 9090 "Compatibility Test for Wastes and Membrane Liners" for hazardous and toxic materials, including tailings in contact with HDPE. Results routinely showed a high chemical resistance of HDPE to alkaline and acidic waters. Exposure in cold climates is less critical than in warm climates. This testing is now seldom performed. URS concludes that HDPE will provide long-term tailings containment with no deterioration from contact with dissolved ions, metals and acid tailings water.

Appendix B

Technical Memorandum

1501 4th Avenue, Suite 1400
Seattle, Washington 98101
206.438.2700 Telephone
206.438.2699 Fax

To:	Gary Coulter, Teck Cominco Alaska, Inc.	**Info:**	George Thornton, TCAK Jim Swendseid, TCAK Daryl Hockley, SRK
From:	Cecil Urlich and Charles Masala, URS	**Date:**	August 6, 2007
RE:	**Conceptual Design of the Spillway for Tailings Main Dam at Final Closure** URS Project No. 33757098; TCAK Contract No. RD-02-06; PO No. 1257477		

1.0 Introduction

This memo summarizes the conceptual design for the spillway that was completed by URS Corporation for Teck Cominco Alaska Inc. (TCAK), for the Red Dog tailings facility for future final closure. The memo also fulfills a State of Alaska Department of Natural Resources (ADNR) requirement for a detailed engineering evaluation as outlined in a letter from ADNR to TCAK titled "Multiple Accounts Analysis for Red Dog Tailings Disposal", dated June 2, 2005.

This memo is an update of two earlier memos dated April 8 and July 22, 2005. The components which make up the total freeboard required for closure presented in Table 1.0 have been updated. The basis for estimating the depth of each component included in the total freeboard is provided in Section 2.0

Based on current TCAK estimation of total tailings storage requirements to closure and the estimated total freeboard requirement, the tailings facility is expected to be closed with the tailings main dam at a maximum crest elevation of 986 feet (El. 986). This elevation was estimated based on the final tailings storage at El. 975, two feet of minimum water cover, and a total freeboard requirement of nine feet as shown in Table 1.0.

The tailings facility is operated as a "zero discharge" impoundment, and the plan is to maintain this condition after closure. The tailings impoundment will contain the water depths shown in Table 1.0, including the probable maximum flood (PMF), and will meet closure design requirements at El. 984.5 without requiring an emergency spillway. After closure, the tailings water level will be regulated using an on site pump and treatment system. However, standard dam design practices require that all dams at their final height should have an emergency spillway.

The spillway will be a side channel above and around the left abutment of the tailings main dam as shown on Drawings TDC-18 and 21. This is similar to the spillway layout at the Red Dog Mine freshwater dam.

Table 1.0 - Spillway Crest Elevation Requirements

Closure Requirements	Components of Closure Requirements	Depth (feet)	Resulting Elevation (feet)
Impounded Tailings at Closure	Total tailings storage	Not Applicable	975.0
Water Cover for Closure	Minimum water cover	2.0	977.0
Freeboard Required for Closure	Autumn to Spring Inflow	1.6	978.6
	PMF	1.1	979.7
	Wind setup	0.3	980.0
	Wave run-up	0.9	980.9
	Spillway backwater Effects (due to 600 ft long upstream beach)	1.1	982.0
	Additonal Water Storage (1 wet year total precipitation)	2.5	984.5
Spillway Channel	Spillway depth	1.5	986.0

During an unexpected and improbable event where the impounded water would exceed El. 984.5, the discharge would occur through the spillway. The spillway is sized to route the PMF through its channel if the level of the impounded water is at El. 984.5 at the start of the probable maximum precipitation (PMP).

2.0 Estimation of Total Freeboard

The total freeboard of the Red Dog tailings facility include: autumn to spring inflow; PMF; wind setup; wave run-up; spillway backwater effects; wet year total precipitation and spillway depths as presented in Table 1.0. The total freeboard is the 9 feet water depth above the two feet minimum water cover that will permanently cover the tailings. The basis for estimating the two feet minimum water cover and each of the components of the total freeboard is presented in the following sections.

2.1 Minimum Water Cover

According to the current mine plan, it is estimated that the total tailings of 88,000,000 tons (79,800,000 tonnes) will be produced by the end of ore processing in 2031. Using the lowest tailings density estimate of 94.3 pounds per cubic feet (pcf), the projected 88,000,000 tons of tailings to be placed in the reservoir will result in the final struck-level tailings surface of approximately El. 975 feet (SRK 2007b). The Red Dog tailings are potentially acid generating. To prevent the generation of acid, a minimum water cover of two feet will be maintained over the tailings surface in perpetuity after closure.

2.2 Autumn to Spring Inflow

From autumn to spring, the water treatment facility is shut down with no discharge of water from the reservoir. However, during this period, there is accumulation of winter precipitation, mine water and freshet in the pond. The estimated maximum dam height for the conceptual closure includes a volume which will be used to store the inflow into the pond from autumn to spring. Initially the autumn to spring total depth of 3.2 feet was estimated by TCAK using historical data from 1995 to 2003 (TCAK 2003). Based on the revised closure plan which results in some of the autumn to spring runoff originally going into the tailings pond, being redirected to the pit, a revised autumn to spring runoff of 1.6 feet was estimated by SRK, 2007b.

2.3 PMF

URS conducted flood routing modeling of the tailings facility as part of the revised spillway design, using the previously estimated probable PMP of 5.9 inches over a catchment area of 2.55 square miles (Geomatrix 2003). The HEC-HMS flood routing model was used in the analyses and the total inflow volume into the reservoir was estimated to be 765 acre feet (URS 2005b). Assuming that the reservoir is full at the beginning of the PMP, at the peak discharge of 37 cubic feet per second (cfs), the maximum depth of flow over the spillway would be 1.1 feet (URS 2005b).

2.4 Wind Setup

Wind setup is inversely proportional to water depth in the pond. Initially a wind setup of 0.06 feet was estimated by Geomatrix (2003) assuming an average water depth in the pond of 24 feet. For the final closure case, TACK (2003) revised the calculation using an average depth of 5 feet resulting in an estimated wind setup of 0.3 feet. The value of wind setup estimated by TACK using an average depth of 5 feet is considered conservative based on the current total freeboard of approximate 9 feet, but it is considered reasonable for inclusion in the estimation of the maximum dam height at closure.

2.5 Wave Run-up

Geomatrix and TCAK estimated the wave run-up height of 3.42 feet onto the main tailings dam (Geomatrix 2003, TCAK 2003). Both Geomatrix and TCAK estimated the wave run-up height with the assumption that there was no beach in front of the main tailings. The calculated wave run-up is inversely proportional to the angle of the upstream face of the dam to the horizontal plane. The placement of a 600 feet beach upstream of the main dam, with elevation equivalent to the maximum elevation of the main tailings dam, covers the upstream face of the main tailing dam and eliminates the effect of the wave run-up. To account for the possibility of the tailings beach and the coffer dam settling overtime, an upstream angle of 5 degrees was used in estimating the wave run-up for the final closure, see Section 5.5. Based on the upstream angle of 5 degrees, the wave run-up was estimated as 0.9 feet (URS 2005a).

2.6 Spillway Backwater Effects

In the unlikely event that the water level in the tailings pond exceed 984.5 feet (spillway control elevation) discharge will be routed from the tailings pond via the spillway channel on the left abutment of the dam, to Red Dog Creek downstream of the tailings dam. The control section of the spillway which will operate as a broad-crested weir will be placed near the center line of the main dam. Due to the 600 feet tailings beach upstream of the main dam, the tailings water pond is expected to be at least 600 feet from the spillway control point. To route the water from the pond to the spillway control point, an upstream channel, at least 600 feet will be installed. This will cause backwater effects in the channel resulting in water depth increase of 1.1 feet in the pond (URS 2007).

2.7 Additional Water Storage Capacity

In the event that the treatment system and discharge of the pond water completely ceased operating, additional storage capacity of 2.5 feet was added to the total freeboard. The 2.5 feet was estimated from the total precipitation of 28.5 inches received in a wet year based on water balance model performed by SRK, 2007. The additional water storage capacity of 2.5 feet will accommodate up to one year of precipitation even in a wet year.

2.8 Spillway Depth

The spillway depth of 1.5 feet above the spillway control point of El. 984.5 was added to the freeboard to allow routing of the PMF in case the reservoir is full when the PMF occurs. Based of the reservoir flood routing, the maximum discharge through the spillway during a PMF would be 37 cfs. For the discharge of 37 cfs the maximum depth of water over the spillway will be 1.1 feet (see Section 5.1).

3.0 Spillway Conceptual Design Approach

The spillway conceptual design that is described in this memo is preceded by two closure spillway designs by Dames & Moore (now URS): the original Stage VI spillway design (Dames & Moore, 1986); and a Stage VII modified spillway design (Dames & Moore, 1995).

As part of final closure spillway design URS has identified and addressed the following items:

- Basis for design of an emergency spillway for the final tailings closure
- Design criteria for a spillway crest at El. 984.5 and final dam crest at El. 986
- Conceptual spillway design based on extrapolation from two previous spillway designs.
- Re-evaluation of reservoir routing model using the Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) model (successor to HEC-1), developed by the U.S. Army Corps of Engineers (USACE). The HEC-HMS model was used to estimate the peak PMF inflow storage into the reservoir; spillway crest maximum water elevation, and the maximum discharge rate through the spillway. Although these values were estimated in both the 1986 and 1995 spillway designs, they were updated by URS to reflect changes due to the increase in the reservoir surface area at El. 986.
- Develop new channel alignment which is further uphill on the left abutment, and roughly parallel to, the 1986 and 1995 alignments.
- Estimate the velocity and depth of flow along the new channel alignment based on changed channel slope and the revised peak discharge from the HEC-HMS model.
- Re-evaluate the design of the crest, channel erosion protection, and stilling basin based on the new model results, revised channel alignment, and updated geotechnical information.
- Evaluate how the wave run-up height will be reduced or dampened by maintaining a wide tailings beach upstream of the tailings main dam.

The conceptual design of the spillway and the analyses that were completed for the design are described in the following sections of this memo.

4.0 Spillway Conceptual Design

4.1 Channel design

From the reservoir routing analysis detailed in Section 5.1, URS estimated the maximum discharge from the spillway to be 37 cfs and the maximum flow water depth over the spillway to be 1.1 feet (URS, 2005b). The maximum reservoir storage was calculated to be 765 acre-feet, which would result in a peak impoundment water level at El. 985.6.

The conceptual design shows that the part of the channel that is downstream of the spillway control point will be relatively steep, ranging in slope from 2 percent at the top to 24 percent at the bottom. This downstream channel slope is dictated by the natural ground slope which increases in steepness from top to

bottom. As part of the requirement for designing erosion protection for steep slopes, the channel alignment should be straight (Section 4.2).

The alignment of the spillway channel for conceptual design purposes was split into three reaches based on channel slope, as presented in Table 2.0, and stationing shown on Drawing TDC-21. The reaches are labeled as Reach 1, 2, 3 and 4 from top to bottom. As in the previous designs, the bottom width of the channel was selected to be 12 feet for ease of construction using a bulldozer.

Table 2.0 - Spillway Outlet Channel Design Details

Reach number	Stationing	Approximate Slope (%)	Length (feet)	Flow Depth (feet)	Flow velocity (feet/sec.)
1	1+00 - 8+50	24	750	0.34	8.60
2	8+50 - 11+50	10	300	0.45	6.54
3	11+50 - 13+00	2	150	0.72	3.92
Crest	13+00	0	50	1.1	<3.92
4	13+00 – 22+60	0	960	1.1 to 2.2	<3.92

The channel side slopes will be constructed to slopes of 2.5 horizontal to 1 vertical (2.5:1) to meet another requirement for the design of erosion protection for steep channel slopes (Section 4.2). The alignment and reaches of the spillway channel are shown on Drawing TDC-21. Typical sections and details at representative stations are shown on Drawing TDC-23. Profiles are shown on Drawing TDC-24 for Reach 1 and Drawing TDC-25 for Reaches 2 and 3.

4.2 Outlet channel erosion protection design

Because of the expected high flow velocity in the outlet channel, erosion protection will be required. Riprap is considered to be the most appropriate material for erosion control because of its availability and durability. The extent of riprap for the conceptual design was based on findings of 2005 geotechnical investigation, which are described in Section 5.6. Riprap was not designed to be used where the bottom of the spillway is below the laminated shale bedrock.

Silty sand and highly weathered shale that is exposed by the spillway excavation will be over-excavated and replaced with riprap as shown on Drawing TDC-23. The conceptual sizing and design of the riprap was developed using the methodology recommended by the U.S Army Corps of Engineers (1994, EM 1110-2-1601) for steep slopes. The conceptual design of riprap on steep slopes included the following considerations:

- The riprap should be angular rock with unit weight of at least 167 pounds per cubic foot
- Channel flow should have low unit discharge, that is, low water depth for a wide channel
- The longitudinal slope of the channel should range from 2 to 20 percent
- Restrict application to straight channel with side slope of 2.5 to 1.0 (horizontal to vertical)
- Riprap should have uniform gradation, or D_{85}/D_{15} ratio equal to or less than 2.

For Reach 3, the slope is 24 percent which exceeds the recommended slope of 2 to 20percent. To account for the steeper slope, the following considerations were made for the conceptual design:

- The calculated D_{50} of the riprap required was increased by 30 percent.

- The spillway is expected to be used in extremely rare events, since according to the design, the reservoir design meets all requirements without requiring the spillway;
- The dam will be operated and inspected after closure, so that if there is discharge through the spillway and if damage has occurred, they would be mitigated and repaired.

The summary of the results for the riprap conceptual design are presented in Table 3.0. Details are presented in Section 5.3 (URS 2007b).

Table 3 - Riprap Conceptual Design

Reach	Slope (percent)	D_{50} (inches)
1	24	12
2	10	6
3	2	3
4	0	No riprap required

4.3 Stilling basin calculation

The spillway channel will discharge into the North Fork of Red Dog Creek, approximately 500 feet downstream of the toe of the seepage collection dam, as shown on Drawing TDC-18. This discharge outlet is at Station 0+00, and is a flat wetland area that is located west of the outer side of a bend in the creek.

To control erosion at the outlet of the spillway channel, a stilling basin would be constructed at the end of the spillway chute. The stilling basin will dissipate the energy from the flow prior to discharging into the creek. The stilling basin is designed to handle the design discharge of the spillway of 37 cfs.

URS completed the conceptual design of the stilling basin by using a method developed by the U.S. Bureau of Reclamation (1994). The length of the required stilling basin for conceptual design purposes was estimated to be 5.4 feet, but was increased to 12 feet to facilitate ease of construction using a bulldozer.

It is presently anticipated that the riprap for erosion control in the stilling basin will be similar to that sized for Reach 3, with a D_{50} of 12 inches.

4.4 Spillway Entrance

The purpose of the spillway entrance is to prevent the re-suspension of the tailings due to bottom scour. Based on the 1995 conceptual spillway design, the velocities and tractive forces responsible for causing re-suspension were found to be below values which would cause re-suspension, within the vicinity of the spillway outlet. To further prevent re-suspension, the following, two considerations were made:

- The tailings should not be placed within 75 feet of the spillway entrance
- A gravel blanket should be placed in the vicinity of the spillway entrance.

Because of these two considerations and the local topography, the spillway entrance is located approximately 600 feet upstream of the crest of the tailings dam, as shown on Drawings TDC-18 and 21.

5.0 Spillway Conceptual Analysis

5.1 Reservoir routing

The purpose of reservoir routing is to estimate the maximum impoundment water elevation and spillway discharge during a PMF. Reservoir routing was conducted using HEC-HMS model. HEC-HMS was used to simulate stormwater runoff into the tailings pond (inflow hydrograph), estimate maximum water elevation in the pond and calculate the resulting maximum discharge through the spillway (outflow hydrograph).

Before running HEC-HMS model, the following parameters were identified and estimated as inputs to the model:

- Catchment area
- Lag time
- Curve number
- Initial abstraction
- Rainfall depth-duration curve
- Reservoir elevation, storage and discharge rating curve.

5.1.1 Catchment Area

For the current spillway design the catchment area of 2.55 square miles was maintained as in the 986 and 1995 spillway designs. The catchment area is estimated with the assumption that the diversion ditches that are currently around the impoundment will be maintained after closure. The 2.55 square miles catchment area includes more than one square mile of the impoundment, so that almost half of the catchment area is the impoundment.

If the current diversion ditches were not maintained after closure, the catchment area would increase to 3.28 square miles.

5.1.2 Lag Time

Lag time is the measure of the time it takes for precipitation to run-off from where it has fallen to the reservoir outlet. Some factors which affect lag time include the ground slope, permeability of the ground and interception from vegetation.

Lag time is usually estimated as 60 percent of the time of concentration. Time of concentration is the travel time from the hydraulically furthermost point in a watershed to the outlet. The time of concentration was estimated using the following Kirpich Formula:

- $T_c = 0.0078 * L^{0.77} S^{-0.385}$

  where Tc = time of concentration in minutes

L = maximum length of flow in feet
S = the watershed gradient in feet per feet or the difference in elevation between the outlet and the most remote point divided by the length L

URS estimated the time of concentration to be 26 minutes, with the corresponding lag time calculated to be 15 minutes (URS 2005b)..

5.1.3 Curve Number

Curve Number (CN) is an empirically derived value which is depended on location, soil-type, land use, and antecedent moisture conditions. The CN provides a measure of how much of the precipitation falling on the catchment will end up as runoff. The higher the CN, the higher the proportion of the rainfall falling on the catchment area will end up as runoff.

Theoretically, the CN varies between 0 and 100. A CN of 100 is applied if it is assumed that all the precipitation falling on the catchment area will end as runoff. A CN of 0 applies when it is assumed that none of the precipitation falling on the catchment area will end as runoff. For the tailings facility, a CN of 100 was assumed for three reasons:

- The design storm occurs when the ground is covered with snow or ice, resulting in most of the precipitation ending as runoff
- Approximately half of the catchment area is covered by the impoundment, which has a CN of 100
- A CN of 100 results in a conservative estimate.

5.1.4 Initial abstraction

Initial abstraction is the measure of the rainfall which falls prior to the beginning of runoff and it includes infiltration and surface storage in the watershed. Initial abstraction is inversely related to the CN. If the CN is assumed to be 100, as is the case for the tailings impoundment area, the initial abstraction is 0. Initial abstraction was estimated using the following formula:

- Ia = 0.2S, where S is maximum retention
- $$S = \frac{1000}{CN} - 10 = \frac{1000}{100} - 10 = 0$$
- Ia = 0 (URS 2007b)

5.1.5 Rainfall depth-duration curve

For a particular frequency or return period of a storm, the rainfall depth-duration curve provides a relationship between rainfall depth (intensity) and duration. The frequency or return period for the PMF is estimated to be once every 10,000 years or more. The rainfall depth is the depth per unit time in inches per hour.

The duration is that time in minutes or hours over which a precipitation of a certain depth is expected. The rainfall depth-duration curve that was developed for the 1995 spillway design for the PMP is shown

on Table 4 and was used was used for the current conceptual design of the spillway at closure (URS 2005b).

Table 4 – Rain Depth-Duration

Duration	5 min	15 min	1 h	2 h	3 h	6 h	12 h	24 h
Precip. Depth (inches)	0.475	0.91	1.59	2.64	3.36	4.8	5.38	5.9

5.1.6 Storage-elevation-discharge curve for the reservoir

The discharge rating curve of a reservoir depends on the type of the spillway designed. The spillway conceptual at the final closure will be a broad-crested weir. Therefore, the following equation was used to estimate discharge through the broad-crested weir (Brater and King, 1976):

- $Q = CLh^{3/2}$

Where Q – Discharge, cubic feet per second
C – Discharge coefficient
L – Width of the weir, feet
H – Height of water above weir, feet

According to Brater and King (1976), the coefficient of the broad-crested weir varies from 2.34 to 3.32 as a function of weir crest breadth and water head. For the conceptual spillway closure design, with the breadth of 12 feet and approximate water head of 1 foot, a 2.6 weir coefficient was selected.

5.1.7 Results of Reservoir Routing

The maximum inflow rate into the reservoir from the surrounding catchment area was estimated as 3,265 cfs, while the maximum discharge through the spillway was estimated as 37 cfs. The maximum depth over the spillway was estimated as 1.1 feet with spillway control point velocity of 3.9 fps (URS 2005b).

5.2 Channel Design

The depth and velocity of flow in the channel were estimate using Manning's formula presented as follows:

- $Q = \frac{1.49}{n} AR^{3/2} S^{1/2}$

Where Q - flow in cubic feet per second (ft^3/s)
A - area of flow
n - Manning coefficient
R - hydraulic radius in feet
S - channel slope

The Manning equation applicable to each reach was solved using Flowmaster a computer program developed by Haestad Methods (URS 2007b).

5.3 Outlet channel erosion protection design

The design of riprap was estimated using the following equation recommended by the U.S. Army Corps of Engineers (1994, EM 1110-2-1601) for steep slopes:

- $$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}}$$

Where: s = slope of bed
q = unit discharge (Q/b) (where Q is the discharge and b is the channel width)

Since D_{50} is more commonly used in specifying riprap design, the D_{30} from the above equation was converted to D_{50} using the following equation:

- $$D_{50} = D_{30} \left(D_{85} / D_{15}\right)^{1/3}$$

From Section 3.2, the D_{85}/D_{15} ratio was assumed to be 2.

The results of the erosion protection for each reach are presented in Section 4.2 (URS 2007b).

5.4 Spillway entrance design

The 1995 spillway design included an analysis of the spillway entrance using permissible tractive force and scouring velocities for 400 sieve particles. The average velocities and tractive force for the PMF flows were estimated at three cross sections located at distances of 20, 45 and 100 feet upstream from the spillway entrance. Both the average velocities and tractive forces were found to be below the values which would cause re-suspension.

However, the calculated velocities and tractive forces were averaged over the entire cross section area. Therefore, there was a possibility that localized scouring of the tailings would occur.

For the current design, the velocities and tractive forces will be less than those calculated in 1995, due to the reduced spillway discharge from 51 to 37 cfs. Therefore, the 1995 spillway entrance design will be conservatively adopted for the current conceptual design purposes.

5.5 Wave run-up

In estimating the final elevation of the conceptual tailings design crest elevation, it was assumed that the wave run-up would be reduced from 3.5 feet (estimated by Geomatrix) to 0.9 feet due to construction of a coffer dam and tailings beach upstream of the main dam. To verify this assumption, a wave run-up calculation was conducted (URS, 2005a).

Wave run-up was estimated using the procedure provided by the U.S. Bureau of Reclamation, 1981. The following equation was used for calculating wave run-up:

- $$R_s = \frac{H_s}{0.4 + (H_s / L)^{0.5} \tan(90 - x)}$$

Where: Rs is the wave run-up
Hs is the significant wave height
L is the wave length
x is the angle of the upstream face of the dam with the horizon

The main parameter which will affect wave run-up by the construction of the tailings beach in the above equation is the angle x. The construction of the tailings beach upstream of the dam results in a significant reduction of x. Since x is directly proportional to the wave run-up, as the angle x reduces, the wave run-up also reduces.

To calculate a wave run-up of 3.5 feet, Geomatrix used the tailings dam upstream slope angle x of 25 degree which corresponds to the upstream slope of 2.5 to 1. To achieve a wave run-up of 0.9 feet used in the conceptual tailings dam elevation design, the beach upstream of the main tailings dam should be place at a slope angle of 5 degrees (9 percent) or less.

5.6 Soil and Rock Conditions

A geotechnical investigation was conducted in 2005 to establish the soil and rock conditions along the alignment of the spillway for purposes of confirming the conceptual design, and specifically to determine the amounts of rock excavation and riprap placement that will be required. This investigation was part of a larger investigation for the conceptual design of the tailings main dam at closure.

The subsurface conditions along the proposed spillway alignment were investigated by drilling four exploratory borings and one test pit at locations determined by URS and TCAK as shown in the attached Drawing TDC-21. These locations were determined by URS in coordination with TCAK, and were approved by ADNR.

5.6.1 Field Exploration

The field exploration program was conducted between November 20 and 22, 2005. During this period, four borings (SS-02-05, SS-03-05, SS-21-05, and SS-22-05) were drilled to depths of 14.1, 20.1, 18.6 and 15.1 feet, respectively, below the ground surface. One test pit (TP-01-05) was excavated to a depth of 10 feet. The boring and the test pit locations within the generalized layout of the spillway alignment are shown in the Drawing TDC-21. The boring locations were surveyed by TCAK.

The borings were drilled by Discovery Drilling Company under the technical direction of a URS geotechnical engineer, Carlos Chaparro, who maintained a continuous log of the borings, visually examined and classified the soils in accordance with the Unified Soil Classification System, and collected samples of the soil and rock that were encountered. Graphical representations of the boring and test pit logs (URS 2006b).

The borings were drilled by a Track Mounted CME 55 drill rig, and advanced by means of a hollow stem auger. The borings were terminated when the auger met refusal in the more competent and less weathered bedrock. Coring of the bedrock below the depth of auger refusal was not attempted because information on the deeper more competent bedrock was not necessary for purposes of conceptual design.

Disturbed samples of soil and highly weathered bedrock were obtained with a continuous sampler barrel or with a 3-inch split spoon sampler. Standard Penetration Tests (SPTs) were conducted on samples collected with split spoon samplers in general accordance with ASTM designation D-1586. The sampler was driven with a 300-lb hammer falling 30 inches. The sampling resistance, measured in blows per foot of sampler penetration ("N" values) or fraction thereof, is shown on the boring logs. The N values in conjunction with visual examinations, were used in evaluating the relative density, consistency, and strength of the soils encountered.

Test Pit TP-01-05 was completed instead of a boring because the ground was considered to be unsafe for a drill rig because it was too steep and very icy. The test pit was excavated by TCAK using a backhoe to a depth of 10 feet. Excavation of the upper 4 feet of the test pit was difficult because the soil was frozen. Less resistance was encountered in soil below the 4-foot depth. Excavation was difficult again at a depth of 8 feet where shale bedrock was encountered.

Detailed description of the field exploration procedures is presented by URS in a separate report Geotechnical Data Report, Tailings Main Dam Future Raises to Closure, for TCAK (URS 2006a).

5.6.2 Subsurface Conditions

The general soil profile from top to bottom that was interpolated from the geotechnical investigation along the alignment of the spillway consists of the following soil and rock types with approximate thicknesses:

- 1 foot of tundra organic root mat
- 2.5 to 6 feet of brown clayey silty sand with gravel
- 1 to 4 feet of highly weathered black shale
- Weak to medium strong laminated shale to the bottom of the borings and test pit.

Drawing TDC-23 shows the cross sections including the spillway structure. Soil profiles along the centerline of the spillway are shown in Drawings TDC-24 and TDC-25.

5.6.3 Geotechnical Design Considerations

The conceptual design of the spillway sections shown on Drawing TDC-23 has assumed that the organic root mat, clayey silty sand with gravel, and the highly weathered black shale will be over-excavated and replaced with riprap to provide the required protection for the spillway.

Drawing TDC-23 shows that the spillway sections at Stations 2+00 will need to be over-excavated by depths of approximately 1 foot. The excavation would terminate at more competent rock so that 1 foot of riprap is needed.

Drawing TDC-23 shows that the spillway sections at Stations 5+00 and 10+00 will terminate at depths of approximately 4 and 2 feet, respectively, in the more competent and less weathered laminated shale. Therefore, that it is most likely that over-excavation and riprap will not be needed at these sections in the middle part of the spillway alignment.

List of Drawings (from Conceptual Design Report)

TDC -18	Stage X Raise Conceptual Plan and Profile
TDC -21	Spillway Conceptual Plan and Profile
TDC -23	Spillway Conceptual Sections and Details
TDC -24	Spillway Reach 1 Conceptual Profile
TDC -25	Spillway Reach 2 and 3 Conceptual Plan and Profile

References

Dames & Moore (1986), “Design Report: Proposed Tailings Dam Red Dog Mine Development”, for Cominco Alaska, June, 1986.

Dames & Moore (1995), “Design Report: Proposed Tailings Dam Modifications Red Dog Mine”, for Cominco Alaska, February, 1995.

Geomatrix, (2003), “Final Calculations Red Dog Tailings Dam Freeboard Analysis”, for Teck Cominco Alaska Inc.

SRK 2007a, “Annual Water and Load Balance” for Teck Cominco Alaska Inc.

SRK 2007b, “Plan of Operations for Tailings and Water Management” for Teck Cominco Alaska Inc.

Teck Cominco Alaska Inc. (2002), Calculations performed by Norman Paley as part of a tailings facility planning study.

Teck Cominco Alaska Inc. (2003), “Design Criteria and Conceptual Tailings Deposition Plan, 2003”

URS, 2005a, “Wave Run-up Calculation for Tailings Dam Closure” Calculation Package SW-02, URS, August 29, 2005 (4 sheets and 6 attachments)

URS, 2005b, “Spillway Conceptual Closure Design” Calculation Package SW-03, URS, August 29, 2005 (8 sheets plus 13 attachments).

URS (2005c) “Conceptual Design of Spillway at Tailings Main Dam for Closure” for Teck Cominco Alaska Inc. April 8, 2005.

URS (2005d) “Conceptual Design of Spillway at Tailings Main Dam at Closure” for Teck Cominco Alaska Inc. July 22, 2005.

URS (2006a), “Geotechnical Data Report, Tailings Main Dam Future Raises to Closure” for Teck Cominco Alaska Inc. February 3, 2006.

URS (2006b) “Geotechnical Investigation Borings (SS-02-05, SS-03-05, SS-21-05, SS-22-05) and Test Pit (TP-01-05) Logs”, URS, March 17, 2006

URS (2007) “Backwater Evaluation for Spillway Closure” Calculation Package SW-04, URS, July, 2007

U.S. Army Corps of Engineers (1994), “Hydraulic Design of Flood Control Channels”, EM 1110-2-1601.

U.S. Army Corps of Engineers (2001), “Hydrologic Modeling System HEC-HMS – User’s Manual” Hydrologic Engineering Center, January 2001.

U.S.Bureau of Reclamation (1981), “Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams.”

U.S. Bureau of Reclamation (1974), “Design of Small Dams”, U.S. Department of the Interior, Washington.

Appendix C

URS

1500 Fourth Avenue
Suite 1400
Seattle, Washington 98101
Ph (206) 438 - 2700
Fx (206) 438 - 2699

Client: Teck Cominco Alaska Inc.

Project Name: Red Dog Mine Tailings Dam

Location: Alaska

Subject: Predict the longevity of geomembranes

Project No.: 33757098

Calculation No.:

Sheet 1 of 17

Appendix A (Pages 3)
Appendix B (Pages 21)
Appendix C (Pages 7)
Appendix D (Pages 48)
Appendix E (pages 13)

Predict the longevity of geomembranes
(Title of Calculations/Document)

Calculations by Tung T. Vu

TV 05/26/06

Assumptions Checked by

6/5/06

Calculations Checked by

B. Surendran 5/30/06

Task Manager review by

6/5/06.

Approved by

6/16/06

Approval notice: If calculations/document are only spot checked, do not require checking, or are assumed to be correct by experience of engineering judgement, it should be so noted here.

Revision No.	Date	By	Checked By (init)	Approval
1				
2				
3				

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

EXECUTIVE SUMMARY

This calculation package summarizes the calculations to predict the longevity of 100-mil HDPE geomembrane in the Red Dog Mine tailings main dam.

URS conducted a literature search to predict the total lifetime of HDPE geomembranes. The lifetime of a geomembrane is considered to be the time at which the geomembrane loses 50% of design properties. The overall lifetime of geomembrane consist of three stages:

- Stage A
- Stage B
- Stage C.

Stage A is the time for the depletion of antioxidants of geomembrane and it is determined based on test results of several authors.

Stage B is the time since the oxygen starts attacking the polymer to the onset of polymer degradation. This period is determined from field data and the published data are widely accepted since it contributes only small part to the overall lifetime of geomembrane.

Stage C is the time of polymer degradation. This is still a current research topic by the fact that the published data range from 35 years to 750 years for a geomembrane at 20°C (68 ^{o}F). Due to a lot of uncertainties associated with this issue, the lowest but latest values provided by GSI are used to predict the overall lifetime of geomembrane.

The lifetime of geomembranes depends on the in-service temperature of the liner. The lower the temperature is, the longer the geomembrane can last. For the temperature conditions present within the tailings dam in the range of -2.3^{o}C (27.9 ^{o}F) to 6°C (42.8 ^{o}F), URS estimated a range of total lifetime of the geomembrane. Depending on the temperature and the level of liquid aggressiveness that the geomembrane is exposed to, the lifetime of the geomembrane is estimated to be from 500 to 1600 years.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

OBJECTIVES:

To predict the longevity of HDPE geomembranes as a liner.

INDEX

1. Summary……..…2
2. Calculations……..…3-17
3. Appendix A – References List……A-1 to 3
4. Appendix B – GRI White Paper #6……..…B1 to 21
5. Appendix C – Personal Communications with Robert Koerner……..…C-1 to 7
6. Appendix D – R.K. Rowe'paper……..…D-1 to 48
7. Appendix E – J.P. Giroud Draft Technical Memo……..…E-1 to 13

DATA:

1. HDPE geomembrane, 100 mil thick.

REFERENCES:

Please see the attached list

ASSUMPTIONS:

Please see in the detail calculations

RESULTS:

The lifetime of a HDPE geomembrane in the Red Dog Mine Tailings Dam is estimated from 500 (at 6 ^{o}C (42.8 ^{o}F)) to 1600 (at -2.3 ^{o}C (27.9 ^{o}F)) years.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

CALCULATIONS:

1. Longevity of HDPE Geomembrane

a. Literature review

Studies by Koerner et al. [7] at the Geosynthetic Institute shown that the lifetime of HDPE geomembranes composes of three distinct stages as illustrated in Fig. 1.

Stage A called "Antioxidant Depletion Time" is the time where the antioxidants of geomembrane are depleted due to chemical reactions with oxygen or physical loss.

Stage B called "Induction Time to Onset of Degradation" is the time where the oxidation occurs in geomembranes without black carbon and antioxidants. This stage starts with extremely slow oxidation rate, then continues with larger oxidation rate, and eventually finishes with very slow oxidation rate again.

Stage C called "Time to Reach 50% Degradation" is the time where the polymer degradation continues to reach 50% of initial mechanical and physical properties of geomembranes. At the end of this stage, all tensile properties are severely changed so that the engineering performance of the geomembrane is jeopardized. This is selected as the end of the lifetime of the geomembrane even though the material still exists and can function with a factor of safety lower than the initial design value.

It is noteworthy that the duration of all three stages strongly depends on the service temperature, especially stage "A" and stage "C".

For Stage "A", Rowe et al. and Koerner et al. [7] conducted separate tests on the antioxidants depletion time of HDPE geomembrane of thickness of 80 and 60 mil respectively, at temperature of 55, 65, 75, and 85°C (131, 149, 167, and 185 ^{o}F) and then interpreted the results for the site temperature range as illustrated in Table 1 and 2. Bonaparte et al. [1] expanded the results for the lower range of temperature as in Table 3. These results show similar antioxidant depletion time of geomembrane in the common temperature range of 20-30°C (68-86 ^{o}F), and the lower temperature the site is the longer time of antioxidants depletion the geomembrane experiences.

For Stage "B", there is very little data available for the induction time of HDPE geomembrane. Bonaparte et al. [1] estimated this value as 20 years at 20°C (68 ^{o}F) based on the aged milk and water containers retrieved from waste landfills. Koerner et al. [7] presented a range of values estimated from field samples as in Table 2. Since this period does not have a significant contribution to the whole lifetime of a geomembrane, the cited values of induction time can be used in computation of service-life time of a geomembrane.

For the stage "C", Bonaparte et al. [1], and Koerner et al. [7] & [8] all refer to the test results by Viebke et al. [2] and [23], and by Martin and Gardner (1983). However, Viebke et al.'s test results were interpreted so differently in the papers by Bonaparte et al. and Koerner et al. as shown in Tables 2 and 4.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

Bonaparte et al. [1] predicted the Stage C of a geomembrane at 20°C (68°F) based on Viebke et al.'s results up to 750 years which yields the total lifetime of geomembrane up to 970 years. There remain two major concerns associated with Bonaparte et al.'s estimation of geomembrane halflife.

First, the basis for using the value of 90 days as the half life of geomembrane at 115°C (239 °F) is not clear. Viebke et al. mentioned that they did the tests with temperature ranging from 70°C (158 °F) to 105°C (221 °F). Therefore, no one knows how Bonaparte et al. interpreted Viebke et al.'s test results to obtain the halflife of geomembrane at 115°C (239 °F) and used that value as the original value for the estimation of geomembrane halflife later on. Normally, the half life of geomembrane at temperature of 20°C (68 °F) is at magnitude of thousands times larger than that at 115°C (239 °F). Thus, the error in estimation of geomembrane half life at 115°C (239 °F) will be multiplied by thousands times for the half life at 20°C (68 °F).

Second, the activation energy used in Bonaparte et al.'s estimation of geomembrane halflife is in question. Actually, the unique technique to estimate the halflife of geomembrane at the temperature different from the test temperature is to use the Arrhenius equation:

$\ln(t_o/t) = (E_{act}/R)(1/T_o - 1/T)$ (1) (Ref. [15])

Where: t_o: time to 50% strength loss at the test conditions
T_o: absolute temperature of the test environment, °K
t: time to 50% strength loss at in-situ conditions
T: absolute temperature of the in-situ environment, °K
R: universal gas constant, 8.314 J/mol
E_{act}: effective activation energy, J/mol

The value of activation energy plays a critical role in this calculation. Different values of activation energy used result in very different values of half life estimation. Viebke et al. stated that the activation energy fitting the Arrhenius temperature dependence for the incubation period was 75±5 kJ/mol and for the degradation phase was 80 kJ/mol but the value of 80 kJ/mol for the degradation phase should only be considered as a rough estimate because of a relatively large scatter in the data. Bonaparte et al. did use the value of activation energy of 80 kJ/mol in their computation of geomembrane halflife. There are two possibilities for their use of this value of activation energy. If they use the activation energy for the degradation period to compute the geomembrane halflife, it is unsuitable assumption since Viebke et al. [22] stated in their reports that "The rapid degradation of the polymer starts when the antioxidant system has been depleted at any location. We believe, however, based on relatively few data that the incubation period of a modern, well-stabilized polyolefin pipe constitutes the major part of the lifetime. The data currently being reported indicate that the polymer degradation period constitutes only 5% to 10% of the lifetime". This raises a question "Is it adequate that Bonaparte et al. and Koerner et al. interpreted the lifetime of the HDPE pipe in the Viebke et al.'s work as the lifetime of only stage C in their model (stage where the antioxidant in the geomembrane is totally depleted)?". It seems that the lifetime of a HDPE pipe from Viebke et al.'s tests is equivalent to the lifetime of Stage A in the model of Bonaparte et al. and Koerner et al, and the Stage C is approximately 10% of sum of Stage A and Stage B. If this assumption is correct, Bonaparte et al. and Koerner et al. doubled the lifetime of the geomembrane in their prediction.

If Bonaparte et al. used the activation energy of 80 kJ/mol for the incubation period in their computation it is a bad selection of input data since it results in an unconservative estimation. Estimation of geomebrane halflife using three different values of activation energy of 70, 75, and 80 kJ/mol and with the same values of other input data comes up with very different values of geomembrane halflife as 280, 463, and 766 years respectively. It is clear that Bonaparte et al. overestimated the halflife of geomembrane by a factor of three by using a value of activation energy on the unconservative side.

In a personal conversation with the writer, Koerner R.M. [6] said that he is not sure about the duration of Stage C that he interpreted from Viebke et al.'s tests. From their own tests with HDPE geomembrane in GSI, he got much smaller values of Stage C lifetime. He suggested that the revised total lifetime of geomembrane using the average value of Stage C from three different sources as shown in Table 5 may be used in practice. However, he is still not sure about the validity of that approach. Only one thing in estimating the lifetime of geomembranes he can be sure is the Stage A, he still leaves Stage B and Stage C as open issues.

In conclusions, the total lifetime of a geomembrane consists of three stages A, B, and C as above indicated. The stage A can be determined from the published papers since it is corroborated by different authors. The stage B can be determined from Bonaparte et al. and Koerner et al.'s papers as well since this stage does not have significant influence in the total lifetime of geomembranes. The stage C is still an open issue and data of this stage presented in Bonaparte et al. and Koerner et al.'s papers are still not reliable due to the above cited uncertainties.

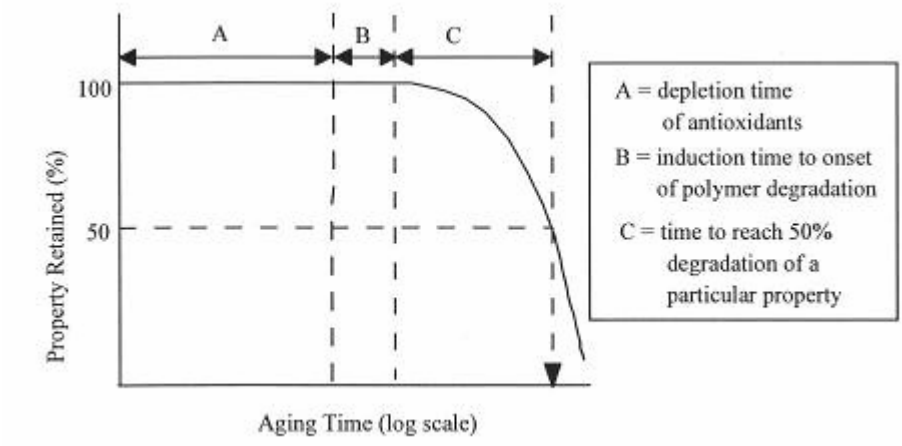


Figure 1: Three conceptual stages in chemical aging of polyolefin geomembranes

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

Table 1. Lifetime prediction of unexposed HDPE geomembranes as a function of in-situ surface temperature.

In-Service Temperature (C°)*	Laboratory tested			Leachate-air	Leachate-water	Air-water	Leachate-unsaturated
	Air	Water	Leachate				
13 [55.4]	390	190	40	210	110	290	160
15 [59]	330	160	36	180	100	250	140
20 [68]	230	120	26	130	70	180	100
25 [77]	160	80	22	90	50	120	70
30 [86]	90	44	12	50	28	70	40

Source: Koerner et al. [7]
* Values in square bracket [ ] are in Fahrenheit

Table 2. Lifetime prediction of unexposed HDPE geomembranes as a function of in-situ surface temperature.

In-Service Temperature (C°)*	Stage A (yrs.)		Stage B (yr)	Stage C (yr)		Total Prediction (yr)
	Std OIT	HP-OIT		Ref. 1	Ref. 2	
20 [68	200	215	30	740	208	712
25 [77]	135	144	25	441	100	435
30 [86]	95	98	20	259	49	270
35 [95]	65	67	15	154	25	170
40 [104]	45	47	10	93	13	109

Ref.1 = Martin & Gardner
Ref.2 = Viebke et al. [23]
Source: Koerner et al. [8]
* Values in square bracket [ ] are in Fahrenheit

Table 3. Extrapolation of Depletion of antioxidant trend to various value of in-situ temperature.

In-Service Temperature (C°)*	Std OIT (Years)	HP-OIT (Years)
30 [86]	90	89
25 [77]	130	131
20 [68]	192	196
15 [59]	286	296
10 [50]	432	455
5 [41]	663	709

Source: Bonaparte et al. [1]
* Values in square bracket [ ] are in Fahrenheit

Table 4. Lifetime prediction of unexposed HDPE geomembranes as a function of in-situ surface temperature.

In-Service Temperature (C°)*	Stage A (yrs.)		Stage B (yr)	Stage C (yr)		Total Prediction (yr)
	Std OIT	HP-OIT		max	min	
20 [68	200	215	30	255	149	449
25 [77]	135	144	25	132	77	270
30 [86]	95	98	20	70	41	173
35 [95]	65	67	15	38	22	111
40 [104]	45	47	10	21	12	73

Stage A measured values from Hsuan and Guan (1997) research via GRI [4]
Stage B estimated values from field sample by GRI
Stage C's values from Gedde et al. [2]
Source: Koerner et al. [7]
* Values in square bracket [ ] are in Fahrenheit

Table 5. Lifetime prediction of unexposed[1] HDPE geomembranes as a function of in-situ surface temperature.

In-Service Temperature (C°)*	Stage A (yrs.)		Stage B (yr)	Stage C (yr)			Total Prediction[2] (yr)
	Std OIT	HP-OIT		Ref. [44]	Ref. [42]	GSI Data	
20 [68	200	215	30	740	208	35	565
25 [77]	135	144	25	441	100	26	353
30 [86]	95	98	20	259	49	21	226
35 [95]	65	67	15	154	25	17	146
40 [104]	45	47	10	93	13	14	96

[1]Exposed geomembrane lifetimes are considerably less than values in this table.
[2]Total = Stage A (average) + Stage B + Stage C (average)
Source: Koerner, R.M., "Personal Communication" [6]
* Values in square bracket [ ] are in Fahrenheit

b. Lifetime prediction.

Studies also show that the lower the temperature on the geomembrane surface is, the longer the longevity of geomembrane will be. For the Red Dog Mine area, statistics data and calculation results show a range of temperature of -2.3 (27.9 ^{o}F) to 6°C (42.8 ^{o}F) on geomembrane surface at different elevations. Therefore, the lifetime of geomembrane is eastimated for this range of temperature.

The techniques used to predict the geomembrane lifetime are presented in R.K. Rowe (2005) and Koerner et al. (2005) papers. The following equations are derived from R.K. Rowe (2005) paper and used in this analysis for convenience.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

Equation 31 from R.K. Rowe (2005):

$$OIT_{(t)} = OIT_o \exp(-st) \quad (1)$$

where s is the antioxidant depletion rate, t is the time for antioxidant depletion from an initial OIT value of OIT_o to a final value of $OIT_{(t)}$.

Equation 32 from R.K. Rowe (2005):

$$s = A \exp(-\frac{E}{RT}) \quad (2)$$

where A is a constant, T is the absolute temperature, E is the activation energy, and R is the universal gas constant.

Equation 1 gives:

$$t = \frac{\ln(OIT_o) - \ln(OIT_{(t)})}{s} \quad (3)$$

hence:

$$\ln(t) = \ln\left[\ln(OIT_o) - \ln(OIT_{(t)})\right] - \ln(s) \quad (4)$$

Equation 2 gives:

$$\ln(s) = \ln(A) - \frac{E}{RT} \quad (5)$$

Combining Equations 4 and 5 gives:

$$\ln(t) = \ln\left[\ln(OIT_o) - \ln(OIT_{(t)})\right] - \ln(A) + \frac{E}{RT} \quad (6)$$

Reference

Equation 6 can be written as follows:

$$\ln(t) = a + \frac{b}{T} \tag{7}$$

where a and b are constants.

Considering two sets of data, (T_1 and t_1) and (T_2 and t_2), Equation 7 gives:

$$\ln(t_1) = a + \frac{b}{T_1} \tag{8}$$

and:

$$\ln(t_2) = a + \frac{b}{T_2} \tag{9}$$

Combining Equations 7 to 9 gives:

$$\frac{\ln(t) - \ln(t_1)}{\ln(t_1) - \ln(t_2)} = \frac{\frac{1}{T} - \frac{1}{T_1}}{\frac{1}{T_1} - \frac{1}{T_2}} \tag{10}$$

hence:

$$\ln(t) = \ln(t_1) + \frac{\frac{1}{T} - \frac{1}{T_1}}{\frac{1}{T_1} - \frac{1}{T_2}}\left[\ln(t_1) - \ln(t_2)\right] \tag{11}$$

hence:

Reference

$$t = \exp\left\{\ln(t_1) + \frac{\frac{1}{T} - \frac{1}{T_1}}{\frac{1}{T_1} - \frac{1}{T_2}}\left[\ln(t_1) - \ln(t_2)\right]\right\} \quad (12)$$

Equation 12 makes it possible to calculate the geomembrane antioxidant depletion time for an absolute temperature, T, using as a basis two sets of temperature and time, (T_1 and t_1) and (T_2 and t_2). It is implicitly assumed that the geomembrane considered at temperature T is the same as the geomembrane used to develop both sets of data, (T_1 and t_1) and (T_2 and t_2). In particular, the OIT values (i.e. OIT_o and $OIT_{(t)}$) are assumed to be the same for all stets of data, i.e. (T_1 and t_1), (T_2 and t_2) and (T and t). Typical values are 135 minutes for OIT_o and 0.5 minute for $OIT_{(t)}$. However, these values are not needed in Equation 12.

Equation 12 is applicable only if the considered phenomenon is consistent with the Arrhenius model. Equation 12 was established for the phenomenon of antioxidant depletion. However, inspection of the sequence of equations presented in Section 2.1 shows that Equation 12 is in fact derived from Equation 7. Therefore, Equation 12 is applicable to all phenomena governed by Equation 7. Equation 7 can be written as follows:

$$\ln(t) = a + \frac{b}{T} \quad (13)$$

hence:

$$t = C \exp\left(\frac{b}{T}\right) \quad (14)$$

where C is a constant.

Equation 14 includes the equation of the Arrhenius model. Therefore, Equation 12 is applicable to all phenomena that are consistent with the Arrhenius model.

Equation 12 is used to extrapolate data from Koerner et al. (2005) and R.K. Rowe (2005) presented in Table 6. It is noted that the most accurate extrapolated values should be the values obtained using the widest temperature range as a basis for the extrapolation, i.e. the 10-60°C (50-140 °F) range. However, the data for the temperature ranges of 10-20°C (50-68 °F) and 40-60°C (104-140 °F) are not available

Reference

from Koerner et al. (2005). Therefore, data for the temperature range of 20-40°C (68-104 °F) from both Koerner et al. (2005) and R.K. Rowe (2005) as in Table 6 are used to extrapolate the geomembrane lifetime.

Stage A: According to Rowe, there is a significant difference in the rate of antioxidant depletion (Stage A) depending on the material in contact with the geomembrane. Rowe indicates that the antioxidant depletion rate in leachate is 2.3 times faster than in water and 4 times faster than in air. In his Rankine Lecture paper, Rowe proposes two approaches for evaluating the duration of Stage A (antioxidant depletion time) in landfills: (i) an inferred exposure, which combines exposures to air, water and leachate using a simple equation; and (ii) a simulated exposure in a laboratory model. These two approaches give depletion times of the same order, e.g. of the order of 100 years at 20°C (68 °F), which are similar to the depletion time in case of exposure to water and about half the depletion time in the case of exposure to air. It is interesting to compare these depletion times with the depletion time proposed by Koerner for Stage A. For 20°C (68 °F), the time proposed by Koerner is of the order of 200 years. From this comparison, it appears that the exposure situation considered by Koerner (which could be referred to as "buried" or "backfilled") is similar (with respect to antioxidant depletion) to exposure to water or a combination of air, water and leachate.

In the case of the tailings main dam, the situation could be considered as intermediary between exposure to air, on one hand, and a combination of air, water and leachate, on the other hand. This approach would be based on the assumption that the liquid in the dam is less aggressive than the leachate considered by Rowe. This would lead to a depletion time between 100 and 200 years at 20°C (68 °F), e.g. a depletion time of the order of 150 years at 20°C (68 °F).

However, the better approach is to estimate the geomembrane lifetime from both sets of data and then the final lifetime can be obtained from these values based on the actual site conditions.

Trial calculations showed that the data from both Koerner et al. (2005) and R.K. Rowe (2005) are approximately consistent with the Arrhenius model for different ranges of base temperature, e.g. 20-30°C (68-86 °F), 30-40°C (86-104 °F), 20-40°C (68-104 °F). The extrapolated lifetime of geomembrane for the temperature range of -2.3°C (27.9 °F) to 6°C (42.8 °F) based on the base temperature range of 20-40°C (68-104 °F) are presented in the columns 2-3, Table 7.

Stage B: Data for an accurate prediction of the Stage B time are insufficient. Relevant data should be available in several years from tests that were started more than ten yeas ago and are still in progress. However, the Stage B time being small, an approximate prediction is possible without a significant error.

Trial calculations show that for Stage B, the data reported by both Koerner et al. (2005) and R.K. Rowe (2005) do not closely follow the Arrhenius model. In fact, the results presented by Koerner vary linearly with temperature. In the case of Rowe's results extrapolated from Rowe's data, the discrepancy may be due to the fact that the times for temperatures of 30°C (86 °F) or higher are very small and extrapolation based on these values may be inaccurate. The extrapolated Stage B lifetime of geomembrane for the temperature range of -2.3°C (27.9 °F) to 6°C (42.8 °F) based on the base temperature range of 20-40°C (68-104 °F) are presented in the columns 7-9, Table 7.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

Stage C: The Stage C lifetime is currently impossible to predict accurately for the reasons presented above. Using data from tests on polyethylene resins used in pipes, times of the order of 3,500 to 4,000 years can be predicted for a temperature of 6°C(42.8°F). However, predictions based on these data are questionable. In this analysis, predictions of Stage C lifetime are based on limited preliminary unpublished tests on polyethylene resins used in geomembrane by Koerner (2006) at the Geosynthetics Research Institute (GRI). Trial calculations show that for Stage C, the data provided by Koerner (2006) closely follow the Arrhenius model. The extrapolated Stage C lifetime of geomembrane for the temperature range of -2.3°C (27.9 °F) to 6°C (42.8 °F) based on the base temperature range of 20-40°C (68-104 °F) are presented in the column10, Table 7.

The total lifetime of geomembrane is the sum of lifetime of threes Stages A, B, and C. The geomembrane lifetime for the air exposure condition is obtained from Koerner et al. (2005) data and for the less-aggressive leachate exposure condition is obtained from R.K. Rowe (2005) data, and presented in the columns 11 and 12, Table 7.

Table 6 Degradation times (years) used for extrapolation

Temperature	Stage 1				Stage 2			Stage 3
	Koerner		Rowe		Koerner	Rowe		Koerner
	Standard OIT	High pressure OIT	Air exposure	Simulated exposure		Base case	Adjusted case	
10°C (50 °F)	-	-	510	280	-	50	30	-
20°C (68 °F)	200	215	235	115	30	15	10	35
30°C (86 °F)	95	98	110	50	20	6	4	21
40°C (104 °F)	45	47	55	25	10	2	1	14
60°C (104 °F)	-		15	6	-	0.4	0.3	-

Table 7: Geomembrane lifetime prediction for the temperature range of -2.3°C (27.9 °F) to 6°C (42.8 °F)

Actual temp.	Base temp. range	Stage 1 (years)				Stage 2 (years)			Stage 3 (years)	Total (years)	
		Koerner		Rowe		Koerner	Rowe		Koerner	Koerner	Rowe
		Standard OIT	High pressure OIT	Air exposure	Simulated exposure		Base case	Adjusted case		Standard OIT/Air exposure	Simulated exposure
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*
-2.3°C(27.9 °F)	20-40°C (68-104°F)	1368	1527	1528	822	124	201	195	114	1606	1131
6°C(42.8 °F)	20-40°C(68-104°F)	645	710	735	381	71	73	61	72	790	510

In conclusion, the lifetime of geomembrane depends strongly on the temperature and site conditions (air, water or leachate) that it is exposed to. The service life of the geomembrane for the Red Dog Mine Tailings Dam are estimated based on the data reported by Koerner et al. (2005) and R.K. Rowe (2005) for the temperature range of -2.3°C (27.9 °F) to 6°C (42.8 °F). Conservative predictions show a range of geomembrane lifetime from 510 years (for buried geomembrane exposed to semi-aggressive liquids and at 6.0°C (42.8 °F)) to 1600 years (for buried geomembrane that is not exposed to liquid and at -2.3°C (27.9 °F)). It should be noted that a service life at 6°C (42.8 °F) greater than 4,000 years could be predicted if some data presented in the literature for Stage C were considered. However, these data based on tests on polyethylene resins used in pipes have not yet been confirmed by tests on polyethylene resins used in geomembranes. These tests are in progress and results should be available in several years. In the meantime it is recommended to use a conservative approach for Stage C time predictions.

Finally, it is important to note that during Stage A, the geomembrane is protected by antioxidants, Therefore, during Stage A the geomembrane should not experience any loss in properties. As a result, it may be predicted that the geomembrane will experience no loss in properties during a period of 400 to 1500 years (depending on the exposure conditions and temperature described above). After this period, the geomembrane properties can be expected to change slowly during a period that is difficult to predict based on current knowledge

c. Geomembrane premature failure in cold weather and remediation

The above lifetime of geomembranes is estimated with the assumption that the geomembrane will be working in normal conditions without any premature failure. However, it is reported by many researchers, Peggs et al. [13] & [14] and Reddy et al. [15], that HDPE geomembranes are normally subjected to a shattering failure due to brittle cracking during cold weather with temperature ranging from 5°C (41°F) to -30°C (-22 °F). Many failures of this type have been observed on side slopes of uncovered liners. Since the temperature in the geomembrane in the Red Dog Mine dam is in the range of -2.3°C (27.9 °F) to 6°C (42.8 °F), it is necessary to account this failure phenomenon for design, specification, and installation of the liner system.

Uniaxial and multiaxial tensile tests conducted by Peggs et al. [14] with HDPE geomembranes in the temperature range of 25°C (77 °F) to -30°C (-22 °F) demonstrated that the yield stress increases from 20.7 MPa to 35.0 MPa while the yield strain decreases from 12% to 6%.

According to Peggs et al. [14], observed brittle stress cracking can be classified into two types: slow crack growth (SCG) and rapid crack propagation (RCP). SCG occurs during application of constant tensile stress well below the yield stress of HDPE. The single unbranched cracks, SGC, grow slowly. When the cracks become of a critical size and reach a critical growth rate, they can propagate very rapidly and branch many times to produce the appearance of shattering. This branching RCP cracks grow very rapidly, at speeds as high as 500 m/s. It is clear that without a stress, there can be no stress cracking. Therefore, if the geomembranes are designed and installed to perform solely as a barrier without stress, the shattering failure will not occur. However, this is not practical in a dam construction site as stress caused by installation, overheating during welding, or temperature during the installation time higher than the life-service time is unavoidable. The only way to minimize the risk of shattering

failure is to minimize the initiation of stress cracking.

Reddy et al. [15] summarized the all factors that influence stress cracking as follows:

a. Use of low stress cracking resistant resins;
b. Stresses induced by restrained thermal contractions at low ambient temperatures;
c. Overheating seams or spot welding;
d. Mechanical damage;
e. Stress induced by lack of support at corners;
f. Hard particle impingement;
g. Impact load; and
h. Chemical environment.

Since many of these factors are well known in design and installation of geomembranes, only emphases on how to minimize stress cracking due to seaming and low ambient temperature are addressed in this paper. For all reported failures, cracks initiated at the seams or spot tack welds, which implies that the seaming technique is the main cause for this problem. Overgrinding and grinding gouges parallel to the seam direction produce stress concentrations normal to the major stresses. Therefore, when preparing for extrusion seaming, grinding should be done perpendicular to the seam rather than along the seam on the assumption that all the grinding marks will not be covered by extrudate. However, if all the grinding marks are covered by the extrusion bead, their direction is immaterial. To prevent overheating the adjacent geomembrane and making it more susceptible to stress cracking, it should not be allowed to add an extruded bead to an already existing seam. Many failures have been initiated at such locations.

According to Peggs [14], the most important parameters to measure in seam testing are shear elongation and peel separation. Shear elongation should exceed 100% of the distance between the edge of the seam and the nearer grip to confirm that the welding procedure has not reduced the ductility of the adjacent geomembrane. It must require that the failure not only occur outside the seam but also be ductile.

When repairs are made, it is inadvisable to repair seams by laying down an additional extruded bead. A wide bead is proposed to use to repair seams containing stress cracks so that the heat affected zone in the parent geoembrane would be well removed from the heat-effected zone of the original seam, thereby preventing an excessive reduction in the stress cracking resistance of the geomembrane.

It is obvious that geomembranes are subjected to dimensional changes when they are deployed at a temperature higher than the design temperature they experience during the winter time due to thermal contraction. The dimensional changes will lead to an induced stress in the geomembrane, which subsequently results in the initiation of stress cracking. Therefore, it is necessary to provide the geomembrane with sufficient slack at installation so that it can accommodate the thermal contraction at the low temperatures expected in service without damage. The coefficients of linear thermal expansion (CTE) of DHPE geomembrane in the practical temperature range of -40°C (-40 ^{o}F) to 90°C (194 ^{o}F) proposed by Peggs et al. [14] are shown in Table 8. This temperature is the temperature in the geomembrane itself, not the ambient temperature. The temperature of the geomembrane can be computed from the ambient temperature and material properties of the geomembrane. Reddy et al. [15] proposed the average temperature in black and white geomembrane exposed to different field conditions at different time of the year as in Table 9.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098 Sheet____ of ____
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

Table 8: Coefficient of linear thermal expansion

Temperature range	Mean CTE
- 40 to 10°C (-40 to 50 °F)	1.4 x 10^{-4}/°C
- 40 to 20°C (-40 to 68 °F)	1.6 x 10^{-4}/°C
- 40 to 30°C (-40 to 86 °F)	1.9 x 10^{-4}/°C
- 40 to 40°C	2.3 x 10^{-4}/°C

Source: Peggs et al. [14]

Table 9: Average temperature in black and white geomembrane

Season of Year	Maximum Ambient Temperature	Black geomembrane		White geomembrane	
		Max. Temp.	Diff. Amb.	Max. Temp.	Diff. Amb.
Winter	5	13	8	2	-3
Spring	22	46	24	38	16
Summer	30	70	40	57	27
Fall	19	35	17	28	10

Source: Reddy et al. [15]

The designed slack of a geomembrane in a given condition can be estimated from the above data using the following equation.

$\Delta L = L \times CTE \times \Delta T$

where: ΔL = estimated slack of geomembrane
L = length of geomembrane sheet in the considered direction
CTE = Coefficient of thermal expansion of geomembrane
ΔT = Difference of temperature at installation and the lowest temperature expected in geomembrane during life-service time.

For the tailings main dam, it is supposed that the black geomembrane is installed in the fall with the maximum ambient temperature of 19°C (66.2 °F). From Table 9, the maximum temperature in the geomembrane is 35°C (95 °F). The average lowest temperature measured on the geomembrane surface is -2.3°C (27.9 °F). This temperature can be considered the lowest temperature in the geomembrane during its lifetime. The slacks of the geomembrane sheet 23 ft x 300 ft (one roll) can be calculated as follows:

On the short side:
$\Delta L = 23 \times 12 \times 2.3 \times 10^{-4} \times (35 + 2.3) = 2.4$ in
On the long side:
$\Delta L = 300 \times 12 \times 2.3 \times 10^{-4} \times (35 + 2.3) = 30.8$ in

Layfield Geosynthetics [11] recommends a factor of safety of 1.0 to 1.5 be used in calculation of geomembrane slack. Therefore, if a factor of safety of 1.5 is used, the geomembrane slacks in short and

long direction in this case are 3.6 in and 46.2 in respectively.

d. Conclusions

It is concluded based on the above analyses that with appropriate prevention of premature failure, The lifetime of a HDPE geomembrane in the Red Dog Mine Tailings Dam is estimated from 500 (at 6 ^{o}C (42.8 ^{o}F)) to 1600 (at -2.3 ^{o}C (27.9 ^{o}F)) years.

2. Longevity of geotextiles

According to SI Geosolutions, the nonwoven geotextiles, Geotex 1001 and Geotex 1601 that are being used in the Red Dog Mine dam are polypropylene products. FHWA describes the same deteriorated mechanism of geosythetic reinforcements that have the same polyolefin parent: polyethylene and polypropylene, and recommends using the same technique as used for HDPE geomembrane to predict their lifetime [23]. SI Geosolutions states that the polypropylene even performs better than polyethylene against the degradation of mechanical properties due to elevated temperatures that is the main factor affecting the lifetime of the geotextiles. SI Geosolutions also use the results of tests for HDPE geomembrane that were conducted by the Gyosynthetic Research Institute and the aforementioned technique to predict the lifetime of a nonwoven polypropylene geotextile [25]. It is clear from the above discussion that the lifetime of a polyolefin product depends greatly on the rate of diffusion of antioxidant to the surrounding environment. Therefore, there might be some factors affecting the lifetime of a polyolefin product associated with the dimensional difference of geotextile fibers and geomembrane sheets. However, there is no document known so far referring to this issue.

Therefore, it can be concluded that if the geotextiles and HDPE geomembranes are manufactured with the same antioxidant composition, the lifetime of geotextiles can be comparable to that of geomembranes. For the most conservative estimation, a lifetime of geotextiles of 100 years at 20°C (68 ^{o}F) is used as recommended by FHWA based on the current research data [24]. Assume that 70% of this lifetime is attributed to the antioxidant depletion time and the testing conditions are the same as those for Koerner et al.'s tests, the lifetime of geotextiles at 6°C (42.8 ^{o}F) is then calculated using above-cited Arrhenius equation:

$\ln(70/t_{A1}) = (56000/8.314)*(1/293 - 1/279)$

$t_{6^{o}C} = 222$ years

Similarly, the lifetime of geotextiles at -2.3^{o}C (27.9 ^{o}F) is predicted as 465 years.

In conclusions, the lifetime of polypropylene nonwoven geotextiles, Geotex 1001 and Geotex 1601, used in the Red Dog Mine main dam is considered to be comparable to that of the HDPE geomembrane. For the most conservative estimation regarding the larger exposure to environment of geotextiles fiber compared to the geomembrane, the estimated lifetime range of geotextiles is 220 to 460 years.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by _____ Date_____

Reference

APPENDIX A

List of References

A1/2

List of References

I. Geomembrane

1. Bonaparte, R., Daniel, D.E., Koerner, R.M. (2002). "Assessment and recommendations for improving the performance of waste containment systems". *Research Report*, United States Environmental Protection Agency

2. Gedde, U.W., Viebke, Leijstrom, H., and Ifwarson, M. (1994). "Long-term properties of hot-water polyolefin pipes – A review". *Polymer Engineering and Science*, pp. 1773-1787.

3. GeoSyntec Consultants (1996). "Assessment of long-term performance of the proposed HDPE geomembrane liner and cap at the Crandon project, TMA facility". *Addendum No 3 to the Feasibility Report / Plan of Operation / Appendix F.*

4. Hsuan, Y.G., "Waste containment technology – Lifetime prediction of the landfill liner". www-gateway.vpr.drexel.edu/files/ NewEH/htmls/Waste_Hsuan_Problem_set.doc

5. Jones, D.R.V, Shercliff, D.A., Dixon, N. (…..). "Difficulties associated with the specification of protection geotextiles using only unit weight". *http://www.geofabrics.com/docs/PAPER_-_Difficulties_with_unit_weight.pdf*

6. Koerner, R.M. (2006). "Personal Communication"

7. Koerner, R.M., Hsuan, Y.G., and Koerner, G.R. (2005). "Geomembrane lifetime prediction: Unexposed and exposed conditions", *GRI White Paper # 6*, Geosynthetic Institute, June 7, 19 p.

8. Koerner, R.M., Hsuan, Y.G., (2003). "Lifetime prediction of polymeric geomembranes used in new dam construction and dam rehabilitation". Proceedings of the *Association of State Dam Safety Officials Conference*, Lake Harmony, Pennsylvania, June 4-6, 2003.

9. Koerner, R.M., (1997). "Designing with geosynthetics". Fourth Edition, Prentice Hall, New Jersey, p. 760.

10. Larson, E., Kelly, R., Wilkes, J., Scuero, A., Dreese, T., (2005). "Geomembrane installation at salt springs CFRD dam". Proceedings of the *Association of State Dam Safety Officials Conference*, Orlando, Florida, September 23-26, 2005.

11. Layfield. (…..). "Geomembrane slack". *http://www.geomembranes.com*

12. Muller, W., and Jakob, I. (2002). "Oxidative resistance of high-density polyethylene geomembranes". *Polymer degradation and stability*, pp. 161-172.

13. Peggs, I.D. (2003). "Geomembrane liner durability: Contributing factors and the status Quo". *http://www.geosynthetica.net/tech_docs/IDPigsUKpaper.pdf*

14. Peggs, I.D., Winfree, J.P., Giroud, J.P. (....). "A shattered geomembrane liner case history: Investigation and remediation". http://pgi-tp.cee.uiuc.edu

15. Reddy, D.V, and Butul, B. (1999). "A comprehensive literature review of liner failure and longevity". *Research report.* Florida Center for Solid and Hazardous Waste Management, University of Florida, July 1999, p. 146.

16. Rowe, R.K., Sangam, H.P. (2002). "Long-term performance of contaminant barrier systems". *Geotechnique 55*, No.9, pp. 631-678.

17. Sangam, H.P., Rowe, R.K., (2002). "Effects of exposure conditions on the depletion of antioxidants from high-density polyethylene (HDPE) geomembranes", pp. 1221-1230.

18. SRK Consulting Engineers and Scientists (2004). "Memo on Red Dog HDPE geomembrane liner longevity research", p. 02.

19. Tao, T., Li, D.F, Yan, J., (1995). "Calculation of coupled stress and seepage for dams with a geotextile/geomembrane composite liner". *Geosynthetics International*, Vol. 2, No. 4, pp. 707 – 721.

20. URS Corporation (2006). "Geomembrane selection for Maria Copa county, Arizona"

21. URS Corporation (2005). "Excerpts from URS report".

22. Viebke, J., Elble, E., Ifwarson, M., and Gedde, U.W. (1994). "Degradation of unstabilized medium-density polyethylene pipes in hot-water applications". Polymer Engineering and Science, pp. 1354-1361.

II. Geotextiles

23. FHWA (2000). "Corrosion/degradation of soil reinforcements for mechanically stabilized earth walls and reinforced soil slopes". Publication No. FHWA-NHI-00-044.

24. SI Geosolutions (2006). "Product data sheet of Geotex 1001 and Geotex 1601".

25. SI Geosolutions (1997). "The durability of polypropylene nonwoven geotextiles for waste containment applications". Technical note SM-404.

III. Geosynthetic behavior

26. Giroud, J.P. (2005). "Quantification of geosynthetic behavior". Geosynthetic International, Vol. 12, No. 1, pp. 1-27

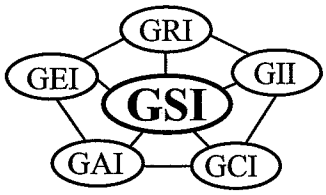
APPENDIX B

GRI White Paper # 6

Geomembrane Lifetime Prediction – Exposed and Unexposed Conditions

B1/21

Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441



GRI White Paper #6

- on -

Geomembrane Lifetime Prediction:
Unexposed and Exposed Conditions

by

Robert M. Koerner, Y. Grace Hsuan and George R. Koerner
Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033 USA

Phone (610) 522-8440
Fax (610) 522-8441

E-mails:
robert.koerner@coe.drexel.edu
grace.hsuan@coe.drexel.edu
gkoerner@dca.net

June 7, 2005

B3/21

Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions

1.0 Introduction

Without any hesitation the most frequently asked question we have had over the past 25 years' is "how long will a particular geomembrane last".* The two-part answer to the question, largely depends on whether the geomembrane is covered in a timely manner or left exposed to the site-specific environment. Before starting, however, recognize that the answer to either covered or exposed geomembrane lifetime prediction is neither easy, nor quick, to obtain. Further complicating the answer is the fact that all geomembranes are formulated materials consisting of (at the minimum), (i) the resin from which the name derives, (ii) carbon black or colorants, (iii) short-term processing stabilizers, and (iv) long-term antioxidants. If the formulation changes (particularly the additives), the predicted lifetime will also change. See Table 1 for the most common types of geomembranes and their approximate formulations.

Table 1 - Types of commonly used geomembranes and their approximate formulations (based on weight percentage)

Type	Resin	Plasticizer	Fillers	Carbon Black	Additives
HDPE	95-98	0	0	2-3	0.25-1
LLDPE	94-96	0	0	2-3	0.25-3
fPP	85-98	0	0-13	2-4	0.25-2
PVC	50-70	25-35	0-10	2-5	2-5
CSPE	40-60	0	40-50	5-10	5-15
EPDM	25-30	0	20-40	20-40	1-5

HDPE = high density polyethylene
LLDPE = linear low density polyethylene
fPP = flexible polypropylene
PVC = polyvinyl chloride (plasticized)
CSPE = chlorsulfonated polyethylene
EPDM = ethylene propylene diene terpolymer

* More recently, the same question has arisen but focused on geotextiles, geogrids, geopipe, fibers of GCLs, etc. This White Paper, however, is focused on geomembranes due to the general lack of information on the other geosynthetics.

The possible variations being obvious, one must also address the degradation mechanisms which might occur. They are as follows accompanied by some generalized commentary.

- Ultraviolet - occurs only when the geosynthetic is exposed; it will be the focus of the second part of this communication.
- Oxidation - this occurs in all polymers and is the major mechanism in polyolefins (polyethylene and polypropylene) under covered conditions.
- Ozone - this occurs in all polymers that are exposed to the environment. The site-specific environment is critical in this regard.
- Hydrolysis - this is the primary mechanism in polyesters and polyamides.
- Chemical - can occur in all polymers and can vary from water (least aggressive) to organic solvents (most aggressive).
- Radioactive - not a factor unless the polymer is exposed to radioactive materials of sufficiently high intensity to cause chain scission, e.g., high level radioactive waste materials.
- Biological - generally not a factor unless biologically sensitive additives (such as low molecular weight plasticizers) are included in the formulation.
- Stress State - a complicating factor which is site-specific and should be appropriately modeled in the incubation process.
- Temperature - clearly, the higher the temperature the more rapid the degradation of all of the above mechanisms; temperature is critical to lifetime and furthermore is the key to time-temperature-superposition which is the basis of the laboratory incubation methods which will be followed.

2.0 Lifetime Prediction: Unexposed Conditions

Lifetime prediction studies at GRI began at Drexel University under U. S. EPA contract from 1991 to 1997 and have continued under GSI consortium funding since that time. Focus to date has been on HDPE geomembranes beneath solid waste landfills due to its common use in this particular challenging application. Incubation of the coupons has been in landfill simulation cells (see Figure 1) maintained at 85, 75, 65 and 55°C. The specific conditions within these cells are oxidation beneath, chemical (water) from above, and the equivalent of 50 m of solid waste mobilizing compressive stress. Results have been forthcoming over the years insofar as three distinct lifetime stages; see Figure 2.

Stage A - Antioxidant Depletion Time

Stage B - Induction Time to Onset of Degradation

Stage C - Time to Reach 50% Degradation (Halflife)

2.1 Stage A - Antioxidant Depletion Time

The purposes of stabilizer antioxidants are to (i) prevent polymer degradation during processing, and (ii) prevent oxidation reactions from taking place during Stage A of service life, respectively. Obviously, there can only be a given amount of antioxidants in any formulation. Once the antioxidants are depleted, additional oxygen will begin to attack the polymer chains, leading to subsequent stages as shown in Figure 2. The duration of the antioxidant depletion stage depends on both the type and amount of antioxidants.

The depletion of antioxidants is the consequence of two processes: (i) chemical reactions with the oxygen diffusing into the geomembrane, and (ii) physical loss of antioxidants from the geomembrane. The chemical process involves two main functions; the scavenging of free radicals converting them into stable molecules, and the reaction with unstable hydroperoxide

(ROOH) forming a more stable substance. Regarding physical loss, the process involves the distribution of antioxidants in the geomembrane and their volatility and extractability to the site-specific environment.

Hence, the rate of depletion of antioxidants is related to the type and amount of antioxidants, the service temperature, and the nature of the site-specific environment. See Hsuan and Koerner (1998) for additional details.

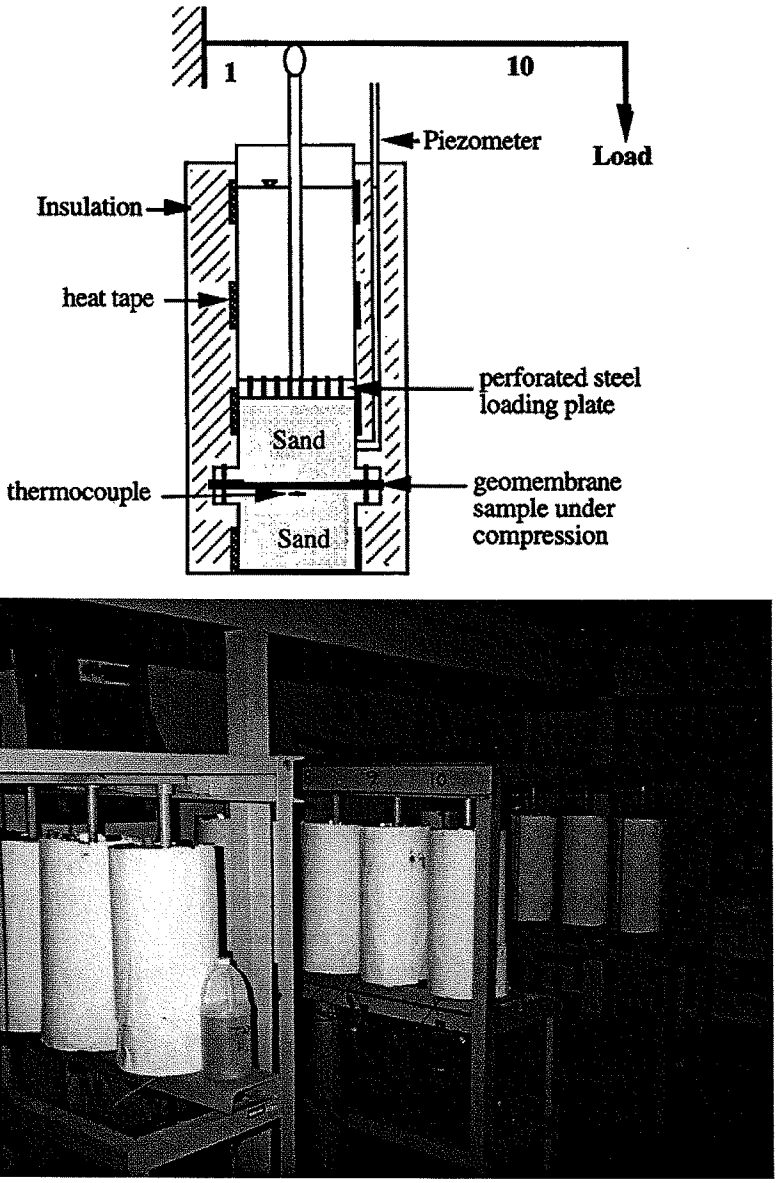


Figure 1. Incubation schematic and photograph of multiple cells maintained at various constant temperatures.

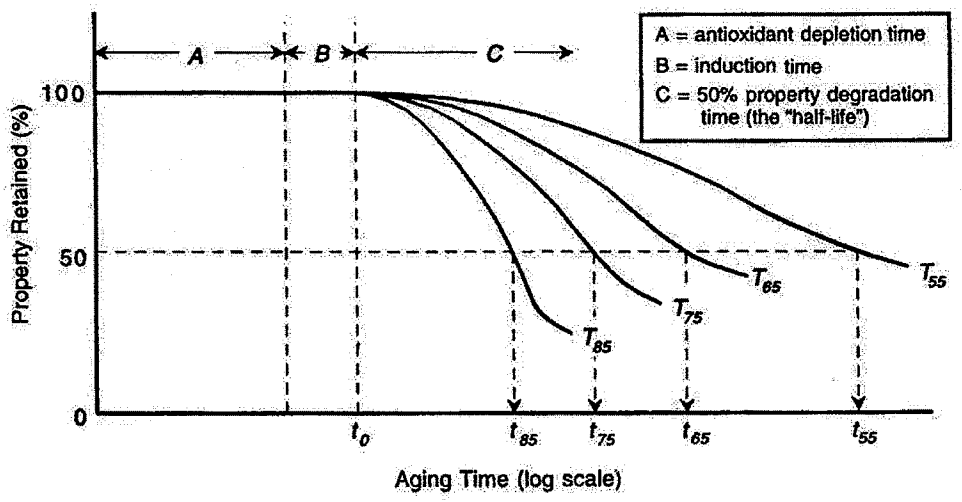


Figure 2. Three conceptual stages in chemical aging of polyolefin geomembranes.

2.2 Stage B - Induction Time to Onset of Degradation

In a pure polyolefin resin, i.e., one without carbon black and antioxidants, oxidation occurs extremely slowly at the beginning, often at an immeasurable rate. Eventually, oxidation occurs more rapidly. The reaction eventually decelerates and once again becomes very slow. This progression is illustrated by the S-shaped curve of Figure 3(a). The initial portion of the curve (before measurable degradation takes place) is called the induction period (or induction time) of the polymer. In the induction period, the polymer reacts with oxygen forming hydroperoxide (ROOH), as indicated in Equations (1)-(3). However, the amount of ROOH in this stage is very small and the hydroperoxide does not further decompose into other free radicals which inhibits the onset of the acceleration stage.

In a stabilized polymer such as one with antioxidants, the accelerated oxidation stage takes an even longer time to be reached. The antioxidants create an additional depletion time stage prior to the onset of the induction time, as shown in Figure 3(b).

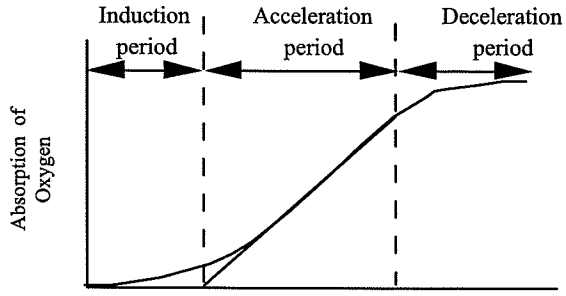
$$RH \rightarrow R\bullet + H\bullet \qquad (1)$$

(aided by energy or catalyst residues in the polymer)

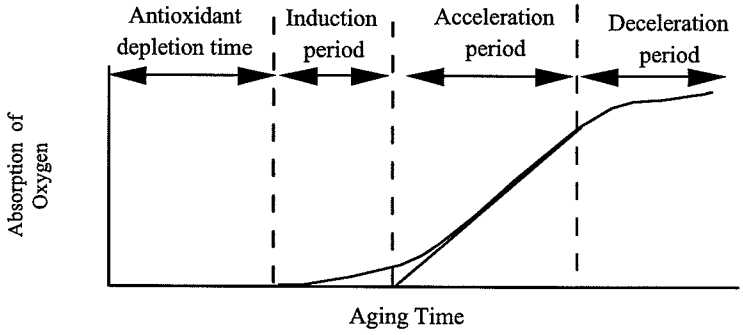
$$R\bullet + O2 \rightarrow ROO\bullet \quad (2)$$

$$ROO\bullet + RH \rightarrow ROOH + R\bullet \quad (3)$$

In the above, RH represents the polyethylene polymer chains; and the symbol "•" represents free radicals, which are highly reactive molecules.



(a) Pure unstabilized polyethylene



(b) stabilized polyethylene

Figure 3. Curves illustrating various stages of oxidation.

2.3 Stage C - Time to Reach 50% Degradation (Halflife)

As oxidation continues, additional ROOH molecules are being formed. Once the concentration of ROOH reaches a critical level, decomposition of ROOH begins, leading to a substantial increase in the amount of free radicals, as indicated in Equations (4) to (6). The additional free radicals rapidly attack other polymer chains, resulting in an accelerated chain reaction, signifying the end of the induction period, Rapopport and Zaikov (1986). This indicates that the concentration of ROOH has a critical control on the duration of the induction period.

$$ROOH \rightarrow RO \bullet OH \bullet \text{ (aided by energy)} \quad (4)$$

$$RO \bullet + RH \rightarrow ROH + R \bullet \quad (5)$$

$$OH \bullet + RH \rightarrow H2O + R \bullet \quad (6)$$

A series of oxidation reactions produces a substantial amount of free radical polymer chains (R•), called alkyl radicals, which can proceed to further reactions leading to either cross-linking or chain scission in the polymer. As the degradation of polymer continues, the physical and mechanical properties of the polymer start to change. The most noticeable change in physical properties is the melt index, since it relates to the molecular weight of the polymer. As for mechanical properties, both tensile break stress (strength) and break strain (elongation) decrease. Ultimately, the degradation becomes so severe that all tensile properties start to change (tear, puncture, burst, etc.) and the engineering performance is jeopardized. This signifies the end of the so-called "service life" of the geomembrane.

Although quite arbitrary, the limit of service life of polymeric materials is often selected as a 50% reduction in a specific design property. This is commonly referred to as the halflife time, or simply the "halflife". It should be noted that even at halflife, the material still exists and

can function, albeit at a decreased performance level with a factor-of-safety lower than the initial design value.

2.4 Summary of Lifetime Research-to-Date

Stage A, that of antioxidant depletion for HDPE geomembranes as required in the GRI-GM13 Specification, has been well established by our own research and corroborated by others, e.g., Sangram and Rowe (2004). The GRI data for Standard and High Pressure Oxidative Induction Time (OIT) is given in Table 2. The values are quite close to one another. Also, as expected, the lifetime is strongly dependent on the service temperature; with the higher the temperature the shorter the lifetime.

Table 2 - Lifetime prediction of HDPE (nonexposed) at various field temperatures

In Service Temperature (°C)	Stage "A" (yrs.)		Stage "B" (yrs.)	Stage "C" (yrs.)		Total Lifetime (ave. values)
	Std OIT	HP-OIT	Field Data	(max.)	(min.)	
20	200	215	30	255	149	449
25	135	144	25	132	77	270
30	95	98	20	70	41	173
35	65	67	15	38	22	111
40	45	47	10	21	12	73

Notes: Stage "A" measured values from Hsuan and Guan (1997) research via GRI
Stage "B" estimated values from field samples by GRI
Stage "C" literature values from Gedde, et al. (1994)

Stage "B", that of induction time, has been obtained by comparing 30-year old polyethylene water and milk containers (containing no long-term antioxidants) with currently produced containers. The data shows that degradation is just beginning to occur as evidenced by slight changes in break strength and elongation, but not in yield strength and elongation. The lifetime for this stage is also given in Table 2.

Stage "C", the time for 50% change of mechanical properties is given in Table 2 as well. The data depends on the activation energy, or slope of the Arrhenius curve, which is very

sensitive to material and experimental techniques. The data is from Gedde, et al. (1994) which is typical of the HDPE resin used for gas pipelines.

Summarizing Stages A, B, and C, it is seen in Table 2 that the halflife of covered HDPE geomembranes (formulated according to the current GRI-GM13 Specification) is estimated to be 449-years at 20°C. This, of course, brings into question the actual temperature for a covered geomembrane such as beneath a solid waste landfill. Figure 4 presents multiple thermocouple monitoring data of a municipal waste landfill liner in Pennsylvania for over 10-years, Koerner and Koerner (2005). Note that for 6-years the temperature was approximately 20°C. At that time and for the subsequent 4-years the temperature increased to approximately 30°C. Thus, the halflife of this geomembrane is predicted to be from 270 to 449 years within this temperature range. The site is still being monitored, see Koerner and Koerner (2005).

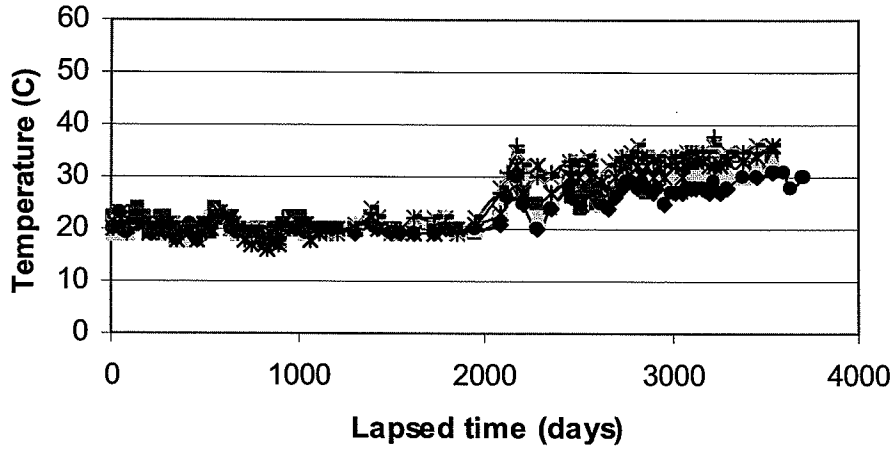


Figure 4. Long-term monitoring of an HDPE liner beneath a municipal solid waste landfill in Pennsylvania.

2.5 Lifetime of Other Covered Geomembranes

By virtue of its widespread use as liners for solid waste landfills, HDPE is by far the widest studied type of geomembrane. Note that in most countries (other than the U.S.), HDPE is the required geomembrane type for solid waste containment. Some commentary on other-than HDPE geomembranes (recall Table 1) follows:

2.5.1 Linear Low Density Polyethylene (LLDPE) geomembranes

The nature of the LLDPE resin and its formulation is very similar to HDPE. The fundamental difference is that LLDPE is a lower density, hence lower crystallinity, than HDPE; e.g., 10% versus 50%. This has the effect of allowing oxygen to diffuse into the polymer structure quicker, and likely decreases Stages A and C. How much is uncertain since no data is available, but it is felt that the lifetime of LLDPE will be somewhat reduced with respect to HDPE.

2.5.2 Plasticizer migration in PVC geomembranes

Since PVC geomembranes necessarily have plasticizers in their formulations so as to provide flexibility, the migration behavior must be addressed for this material. In PVC the plasticizer bonds to the resin and the strength of this bonding versus liquid-to-resin bonding is significant. One of the key parameters of a stable long-lasting plasticizer is its molecular weight. The higher the molecular weight of the plasticizer in a PVC formulation, the more durable will be the material. Conversely, low molecular weight plasticizers have resulted in field failures even under covered conditions. See Miller, et al. (1991), Hammon, et al. (1993), and Giroud and Tisinger (1994) for more detail in this regard.

2.5.3 Crosslinking in EPDM and CSPE geomembrnaes

The EPDM geomembranes mentioned in Table 1 are crosslinked thermoset materials. The oxidation degradation of EPDM takes place in either ethylene or propylene fraction of the co-polymer via free radical reactions, as expressed in Figure 5, which are described similarly by Equations (4) to (6).

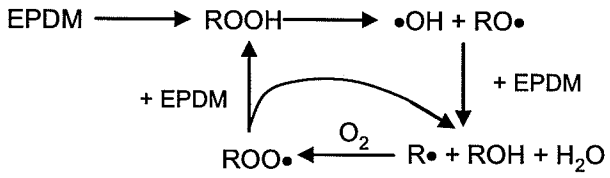


Figure 5. Oxidative degradation of crosslinked EPDM geomembranes, (Wang and Qu, 2003).

For CSPE geomembranes, the degradation mechanism is dehydrochlorination by losing chlorine and generating carbon-carbon double bonds in the main polymer chain, as shown in Figure 6. The carbon-carbon double bonds become the preferred sites for further thermodegradation or cross-linking in the polymer, leading to eventual brittleness of the geomembrane.

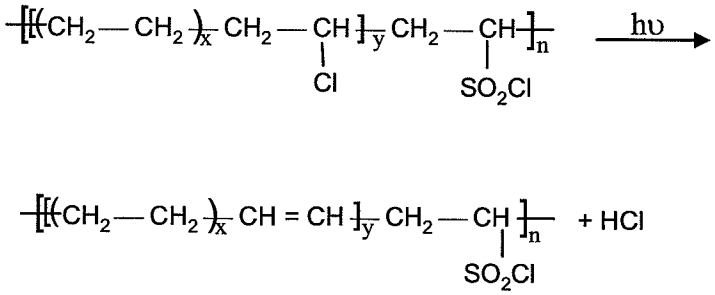


Figure 6. Dechlorination degradation of crosslinked CSPE geomembranes (Chailan, et al., 1995).

Neither EPDM nor CSPE has had a focused laboratory study of the type described for HDPE reported in the open literature. Most of lifetime data for these geomembranes is antidotal by virtue of actual field performance. Under covered conditions, as being considered in this section, there have been no reported failures by either of these thermoset polymers to our knowledge.

3.0 Lifetime Prediction: Exposed Conditions

Lifetime prediction of exposed geomembranes have taken two very different pathways; (i) prediction from anecdotal feedback and field performance, and (ii) from laboratory weathermometer predictions.

3.1 Field Performance

There is a large body of anecdotal information available on field feedback of exposed geomembranes. It comes form two quite different sources, i.e., dams in Europe and flat roofs in the USA.

Regarding exposed geomembranes in dams in Europe, the original trials were using 2.0 mm thick polyisobutylene bonded directly to the face of the dam. There were numerous problems encountered as described by Scuero (1990). Similar experiences followed using PVC geomembranes. In 1980, a geocomposite was first used at Lago Nero which had a 200 g/m^2 nonwoven geotextile bonded to the PVC geomembrane. This proved quite successful and led to the now-accepted strategy of requiring drainage behind the geomembrane. In addition to thick nonwoven geotextiles, geonets, and geonet composites have been successful. Currently over 50 concrete and masonry dams have been rehabilitated in this manner and are proving successful for over 30-years of service life. The particular type of PVC plasticized geomembranes used for these dams is proving to be quite durable. Tests by the dam owners on residual properties show only nominal changes in properties, Cazzuffi (1998). As indicated in Miller, et al. (1991) and Hammond, et al. (1993), however, different PVC materials and formulations result in very different behavior; the choice of plasticizer and the thickness both being of paramount importance.

Regarding exposed geomembranes in flat roofs, past practice in the USA is almost all with EPDM and CSPE and, more recently, with fPP. Manufacturers of these geomembranes regularly warranty their products for 20-years and such warrants appear to be justified. EPDM and CSPE, being thermoset or elastomeric polymers, can be used in dams without the necessity of having seams by using vertical attachments spaced at 2 to 4 m centers, see Scuero and Vaschetti (1996). Conversely, fPP can be seamed by a number of thermal fusion methods. All of these geomembrane types have good conformability to rough substrates as is typical of concrete and masonry dam rehabilitation. It appears as though experiences (both positive and negative) with geomembranes in flat roofs should be transferred to all types of waterproofing in civil engineering applications.

3.2 Laboratory Weatherometer Predictions

For an accelerated simulation of direct sunlight using a laboratory weatherometer one usually considers a worst-case situation which is the solar maximum condition. This condition consists of global, noon sunlight, on the summer solstice, at normal incidence. It should be recognized that the UV-A range is the target spectrum for a laboratory device to simulate the naturally occurring phenomenon, see Hsuan and Koerner (1993), and Suits and Hsuan (2001).

The Xenon Arc Weatherometer (ASTM G155) was introduced in Germany in 1954. There are two important features; the type of filters and the irradiance settings. Using a quartz inner and borosilicate outer filter (quartz/boro) results in excessive low frequency wavelength degradation. The more common borosilicate inner and outer filters (boro/boro) shows a good correlation with solar maximum conditions, although there is an excess of energy below 300 nm wavelength. Irradiance settings are important adjustments in shifting the response although they do not eliminate the portion of the spectrum below 300 nm frequency. Nevertheless, the Xenon

Arc weatherometer is commonly used method for exposed lifetime prediction of all types of geosynthetics.

UV Fluorescent Lamps (ASTM G154) are an alternative type of accelerated laboratory test device which became available in the early 1970's. They reproduce the ultraviolet portion of the sunlight spectrum but not the full spectrum as in Xenon Arc weatherometers. Earlier FS-40 and UVB-313 lamps give reasonable short wavelength output in comparison to solar maximum. The UVA-340 lamp was introduced in 1987 and its response is seen to reproduce ultraviolet light quite well. This device (as well as other types of weatherometers) can handle elevated temperature and programmed moisture on the test specimens.

Research at the Geosynthetic Institute (GSI) is actively pursuing both Xenon and UV Fluorescent devices on a wide range of geomembranes. Table 3 gives the geomembranes being incubated and the current number of hours of exposure.

Table 5 - Details of the GSI laboratory exposed weatherometer study on various types of geomembranes

Geomembrane Type	Thickness (mm)	UV Fluorescent Exposure*	Xenon Exposure*	Comment
1. HDPE (GM13)	1.50	8000 hrs.	6600 hrs.	Basis of GRI-GM13 Spec
2. LLDPE (GM17)	1.00	8000	6600	Basis of GRI-GM-17 Spec
3. PVC (No. Amer.)	0.75	8000	6600	Low Mol. Wt. Plasticizer
4. PVC (Europe)	2.50	7500	6600	High Mol. Wt. Plasticizer
5. fPP (BuRec)	1.00	2745**	4416**	Field Failure at 26 mos.
6. fPP-R (Texas)	0.91	100	100	Field Failure at 8 years
7. fPP (No. Amer.)	1.00	7500	6600	Expected Good Performance

*As of 12 July 2005 exposure is ongoing
**Light time to reach halflife of break and elongation

3.3 Laboratory Weatherometer Acceleration Factors

The key to validation of any laboratory study is to correlate results to actual field performance. For the nonexposed geomembranes of Section 2 such correlations will take hundreds of years for properly formulated products. For the exposed geomembranes of Section

3, however, the lifetimes are significantly shorter and such correlations are becoming possible. In particular, Geomembrane #5 (flexible polypropylene) of Table 3 was an admittedly poor geomembrane formulation which failed in 26 months of exposure at El Paso, Texas, USA. The reporting of this failure is available in the literature, Comer, et al. (1998). Note that for both UV Fluorescent and Xenon Arc laboratory testing of this material, failure (halflife to 50% reduction in strength and elongation) occurred at 2745 and 4416 hours, respectively. The comparative analysis of laboratory and field for this case history allows for the obtaining of acceleration factors for the two incubation devices.

3.3.1 Comparison between field and UV Fluorescent weatherometer

The light source used in the UV fluorescent weatherometer is UVA with wavelengths from 295-400 nm. In addition, the intensity of the radiation is controlled by the Solar Eye irradiance control system. The UV energy output throughout the test is 68.25 W/m^2.

The time of exposure to reach 50% elongation at break

$= 2745$ hr. of light
$= 9{,}882{,}000$ seconds

Total energy in MJ/m^2 $= 68.25\ W/m^2 \times 9{,}882{,}000$
$= 674.4\ MJ/m^2$

The field site was located at El Paso, Texas. The UVA radiation energy (295-400 nm) at this site is estimated based on data collected by the South Florida Testing Lab in Arizona (which is a similar atmospheric location). For 26 months of exposure, the accumulated UV radiation energy is 724 MJ/m^2 which is very close to that generated from the UV fluorescent weatherometer. Therefore, direct comparison of the exposure time between field and UV fluorescent is acceptable.

Field time = 26 Months vs. Fluorescent UV light time: = 3.8 Months Thus, the acceleration factor is 6.8.

3.3.2 Comparison between field and Xenon Arc weatherometer

The light source of the Xenon Arc weatherometer simulates almost the entire sunlight spectrum from 250 to 800 nm. Depending of the age of the light source and filter, the solar energy ranges from 340.2 to 695.4 W/m^2, with the average value being 517.8 W/m^2.

The time of exposure to reach 50% elongation at break

= 4416 hr. of light
= 15,897,600 seconds

Total energy in MJ/m^2 = 517.8 W/m^2 × 15,897,600
= 8232 MJ/m^2

The solar energy in the field is again estimated based on data collected by the South Florida Testing Lab in Arizona. For 26 months of exposure, the accumulated solar energy (295-800 nm) is 15,800 MJ/m^2, which is much higher than that from the Xenon Arc weatherometer. Therefore, direct comparison of halflives obtained from the field and Xenon Arc weatherometer is not anticipated to be very accurate. However, for illustration purposes the acceleration factor based on Xenon Arc weatherometer would be as follows:

Field = 26 Months vs. Xenon Arc = 6.1 Months : Thus, the acceleration factor is 4.3.

4.0 Summary and Recommendations

This White Paper has described research on the geomembrane type which has had the majority of research effort, that being nonexposed HDPE used in landfill applications. While this material promises service lifetime of hundreds of years, the elevated temperatures of exposed or nearly exposed geomembranes in other applications (dams, canals, reservoirs, etc.) is expected to be greatly reduced. It was shown that HDPE decreases its predicted halflife from 449-years at 20°C, to 73-years at 40°C. Other geomembrane types (LLDPE, PVC, EPDM and CSPE) have had essentially no focused effort on lifetime prediction of the type described herein. All are candidates for additional research in this regard.

Exposed geomembrane lifetime was addressed from the perspective of field performance which is very unequivocal. Experience in Europe, mainly with relatively thick PVC containing high molecular weight plasticizers, has given 25-years of service and the geomembranes are still in use. Experience in the USA with exposed geomembranes on flat roofs, mainly with EPDM and CSPE, has given 20^{+}-years of service. The newest geomembrane type in such applications is fPP which currently carries similar warranties. To be noted, however, is that degradation is a very slow process and every time a formulation changes there is uncertainty as to its long-time field performance versus the previous formulation.

Alternatively, exposed geomembrane lifetime can be addressed by using accelerating laboratory weatherometers. GSI is fully involved in such an activity using UV Fluorescent and Xenon Arc weatherometers. Two types of polyethylene, two PVCs, and three fPP geomembranes (seven in total) are being incubated for sufficient time to reach their respective lifetimes. One type of fPP has reached this level and correlation to actual field failure time is reasonable. Analysis of this (poorly formulated) geomembrane results in acceleration factors of 6.8 for UV Fluorescent, and 4.3 for Xenon Arc devices. Based on such acceleration factors, for 20-year lifetime exposed geomembranes typical laboratory weatherometer exposure will be 3-years, or longer. As noted in Table 2 such testing is ongoing and will be continued so as to report our findings at a future date. In this regard we are proceeding as follows so as to develop the required confidence needed for use of geomembranes in long-term, permanent, systems.

(i) Extend HDPE laboratory studies on nonexposed geomembranes to other polymer types such as PVC, LLDPE, fPP, EPDM and CSPE.

(ii) Evaluate, to the extent possible, various additives particularly antioxidants in polyolefins (HDPE, LLDPE and fPP) and plasticizers in PVC.

(iii) Document and analyze geomembrane dam rehabilitation in Europe (and elsewhere) with particular emphasis on durability.

(iv) Document and analyze geomembrane use in flat roofs and other exposed applications, e.g., pond and reservoir liners as well as canal liners.

(v) Initiate a broad research program on lifetime prediction of exposed geomembranes (of all types and formulations) using laboratory weatherometers such as the ongoing study described herein.

Acknowledgements

The financial assistance of the member organizations of the Geosynthetic Institute and its related institutes for research, information, education, accreditation and certification is sincerely appreciated. Their identification and contact member information is available on the Institute's web site at <<geosynthetic-institute.org>>.

References

Cazzuffi, D., "Long-Term Performance of Exposed Geomembranes on Dams in the Italian Alps," Proc. 6th Intl. Conf. on Geosynthetics, IFAI, 1998, pp. 1107-1114.

Chailan, J.-F., Boiteux, C., Chauchard, J., Pinel, B. and Seytre, G., "Effect of Thermal Degradation on the Viscoelastic and Dielectric Properties of Chlorosulfonated Polyethylene (CSPE) Compounds," Journal of Polymer Degradation and Stability, Vol. 48, 1995, pp. 61-65.

Comer, A. I., Hsuan, Y. G. and Konrath, L., "The Performance of Flexible Polypropylene Geomembranes in Covered and Exposed Environments," 6th International Conference on Geosynthetics, Atlanta, Georgia, USA, March, 1998, pp. 359-364.

Gedde, U. W., Viebke, J., Leijstrom, H. and Ifwarson, M., "Long-Term Properties of Hot-Water Polyolefin Pipes - A Review," Polymer Engineering and Science, Vol. 34, No. 24, 1994, pp. 1773-1787.

Giroud, J.-P. and Tisinger, L. G., "The Influence of Plasticizers on the Performance of PVC Geomembranes," PVC GRI-17 Conference, IFAI, Roseville, MN, 1994, pp. 169-196.

Hammon, M., Hsuan, G., Levin, S. B. and Mackey, R. E., "The Re-examination of a Nine-Year-Old PVC Geomembrane Used in a Top Cap Application," 31st Annual SWANA Conference, San Jose, CA, 1993, pp. 93-108.

Hsuan, Y. G. and Guan, Z., "Evaluation of the Oxidation Behavior of Polyethylene Geomembranes Using Oxidative Induction Time Tests," ASTM STP 1326, Riga and Patterson, Eds., ASTM, 1997, pp. 138-149.

Hsuan, Y. G. and Koerner, R. M., "Can Outdoor Degradation be Predicted by Laboratory Acceleration Weathering?," GFR, November, 1993, pp. 12-16.

Hsuan, Y. G. and Koerner, R. M., "Antioxidant Depletion Lifetime in High Density Polyethylene Geomembranes," Jour. Geotech. and Geoenviron. Engr., ASCE, Vol. 124, No. 6, 1998, pp. 532-541.

Koerner, G. R. and Koerner. R. M., "In-Situ Temperature Monitoring of Geomembranes," Proc. GRI-18 Conf. at GeoFrontiers, Austin, TX, 2005, 6 pgs.

Miller, L. V., Koerner, R. M., Dewyea, J. and Mackey, R. E., "Evaluation of a 30 mil PVC Liner and Leachate Collection System," Proc. 29th Annual GRCDA/SWANA Conf., Cincinnati, OH, 1991.

Müeller, W. and Jakob, I., "Oxidative Resistance of High-Density Polyethylene Geomembranes," Jour. Polymer Degradation and Stability," Elsevier Publ. Co., No. 79, 2003, pp. 161-172.

Rapoport, N. Y. and Zaikov, G. E., "Kinetics and Mechanisms of the Oxidation of Stressed Polymer," Developments in Polymer Stabilization—4, G. Scott, Ed., Applied Science Publishers Ltd., London, U.K., 1986, pp. 207-258.

Sangam, H. P. and Rowe, R. K., "Effects of Exposure Conditions on the Depletion of Antioxidants from HDPE Geomembranes", Canadian Geotechnical Journal, Vol. 39, 2002, pp. 1221-1230.

Scuero, A., "The Use of Geocomposites for the Rehabilitation of Concrete Dams," Proc. 4th Intl. Conf. on Geosynthetics, The Hague, Balkema Publ. Co., 1990, pg. 474.

Scuero, A. M. and Vaschetti, G. L., "Geomembranes for Masonry and Concrete Dams: State-of-the-Art Report," Proc. Geosynthetics Applications, Design and Construction, M. B. deGroot, et al., Eds., A. A. Balkema, 1996, pp. 889-898.

Suits, L. D. and Hsuan, Y. G., "Assessing the Photo Degradation of Geosynthetics by Outdoor Exposure and Laboratory Weatherometers," Proc. GRI-15 Conference, Hot Topics in Geosynthetics II, GII Publ., Folsom, PA, 2001, pp. 267-279.

Wang, W. and Qu, B., "Photo and Thermo-Oxidative Degradation of Photocrosslinked Ethylene-Propylene-Diene Terpolymer," Journal of Polymer Degradation and Stability, Vol. 81, 2003, pp. 531-537.

APPENDIX C

Personal Communication with Bob Koerner

cv/7

Tung Vu/Seattle/URSCorp
04/21/2006 04:58 PM

To robert.koerner@coe.drexel.edu
cc Sri Rajah/Seattle/URSCorp@URSCORP
bcc
Subject Geomembrane life time prediction

Dear Mr. Koerner:

I am Tung Vu, a geotechnical engineer in URS Corporation in Seattle, WA. I am estimating the service-life time of a geomembrane used as a dame liner. Thus, I am studying a serie of related articles written by you and some other authors. However, I come up with some issues that I need to discuss with you. I am considering following references:

1. Koerner, R.M., and Hsuan, G.Y. (2003). "Lifetime prediction of polymeric geomembranes used in new dam construction and dam rehabilitation"
2. Koerner, R.M., Hsuan, G.Y., and Koerner, G.R. (2005). "Geomembrane lifetime prediction: Unexposed and exposed conditions" - GRI white paper #6
3. Bonaparte, R., Daniel, D.E., and Koerner, R.M. (2002). "Assessment and recommendations for improving the performance of waste containment systems" - Chapter 2 : Geosynthetic tasks.
4. Hsuan, G.Y., (.....). "Waste containment technology - lifetime prediction of the landfill liner"
5. Gedde, U.W., Viebke, J., Leijstrom, H., and Ifwarson, M. (1994). "Long-term properties of hot-water polyolefin pipes - A review"
6. Viebke, J., Elble, E., Ifwarson, M., and Gedde, U.W. (1994). "Degradation of unstabilized medium-density polyethylene pipes in hot-water applications"

In your all works, the common concept to predict the lifetime of a geomembrane is that the geomembrane consists of 3 stages: Stage A, Stage B, and Stage C. Stage A is the time for the complete depletion of antioxidants to occur and it is taken from Hsuan's research. Stage B is taken from field data. Stage C is taken from Viebke et al. 's research. I have following questions:

Question 1: Is it adequate to consider the lifetime of the pipe in Viebke et al.'s research as the time of only Stage C in your model? Viebke et al. mentioned in their paper that "During regimes A and B, there is a complete depletion of the added antioxidants after which the unprotected polymer rapidly degrades (regim C)" [6], and "The rapid degradation of the polymer starts when the antioxidant system has been depleted at any location. We believe, however, based on relatively few data, that the incubation period of a modern, well-stabilized polyolefin pipe constitutes the major part of the lifetime. The data currently being reported indecate that the polymer degradation period constitutes only 5% to 10% of the lifetime" [5]. So, it is clear that the lifetime of pipe in Viebke et al.'s work is equivalent to the Stage A in your model and Stage C is equivalent to the Regime C in their model (5-10% of lifetime).

Question 2: What does indicate that the lifetime of the pipe in Viebke et al. 's work is the time where the pipe lost its 50% mechanical and physical properties? I can not find anywhere in their paper they mentioned that or I can interpret that factor.

Question 3: How can you interpret Viebke et al.'s results so differently in your articles [1], [2], [3] eventhough the test results in their two articles [5] and [6] are the same? What factors did you use to calculate the maximum and minimum values of Stage C in your article [2]?

Question 4: Which reference mentions the halflife of a geomembrane of 90 days at 115 oC as you used in the article [3]? I have special interest in this value because it has a significant affect on the calculated value of geomembrane lifetime.

Question 5: If I need to extrapolate the average lifetime of a geomembrane at a temperature out of 20 to 40 oC range, what values of activation energy and halflife at 115 oC I should use? The value of activation

energy I back calculated from your data in the article [1] is 80 KJ and in the article [2] is 95 KJ. That is a big difference and that results in a very large difference of lifetime of a geomembrane.

Since I am not an expert in Polymer Engineering, I urge your patience in our communication if my questions seem silly. I am looking forward to talking with you further about these issues.

I appreciate your time and your help.

Best regards,

Tung Vu

This e-mail and any attachments are confidential. If you receive this message in error or are not the intended recipient, you should not retain, distribute, disclose or use any of this information and you should destroy the e-mail and any attachments or copies.

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24

05/7

Marilyn Ashley
<mvashley@verizon.net>
04/24/2006 07:12 AM

To Tung_Vu@urscorp.com
cc
bcc
Subject Geomembrane in Lifetime Prediction

History: This message has been replied to and forwarded.

Dear Mr. Vu,

Attached is the response to your questions from Dr. Koerner. If you have any problems please contact me.

Best regards,

Marilyn Ashley Response to Questions.doc

6/7

To: Tung Vu

From: Bob Koerner

We did not start lifetime prediction until we had Stage A values and felt confident in them. We still do feel that at 20°C a HDPE geomembrane formulated according to GRI-GM13 has 200 years for depletion of the antioxidants. Note that this is not absolute "zero" antioxidant content but an extremely low value of 0.5 minutes in standard OIT and 25 minutes high pressure OIT.

The Stage B is very difficult to evaluate in isolation of Stage A beforehand, and Stage C afterward. It is also the shortest time of the three stages and we have evaluated it by comparing new and old HDPE containers which have no long0term AO's in them during production. We estimated 30 years at 20°C.

Lastly, Stage C is taken from Viebke's work as you mention. There are numerous assumptions that we made including some that you mentioned and (even) some other ones;

- The temperature of 115°C is really too high of an incubation temperature for HDPE with 90°C being the highest recommended value
- Pipe grade resin is slightly higher in density than geomembrane grade resin and the higher crystallinity probably (?) gives higher predicted values

For these reasons and some of those that you cite, we now have revised our HPDE lifetime prediction per the attached table.

TABLE 5.12 LIFETIME PREDICTION OF A BACKFILLED HDPE GEOMEMBRANE AS A FUNCTION OF IN-SITU SERVICE TEMPERATURE[1]

In-Service Temperature (C°)	Stage A (yrs.)		Stage B (yr)	Stage C (yr)			Total Prediction[2] (yr)
	Std OIT	HP-OIT		Ref. [44]	Ref. [42]	GSI Data	
20	200	215	30	740	208	35	565
25	135	144	25	441	100	26	353
30	95	98	20	259	49	21	226
35	65	67	15	154	25	17	146
40	45	47	10	93	13	14	96

[1]Exposed geomembrane lifetimes are considerably less than values in this table.
[2]Total = Stage A (average) + Stage B + Stage C (average)

Reference 44 is Viebke, et al. (1994) and Reference 42 is Martin and Gardner (1983). Both are with pipe resins. We then evaluated some unstabilized HDPE resin making plaques ourselves and obtained the "GSI Data" in the table. The differences is these three

sets of data is huge and leaves us with doubts about the Stage C value. As such, I have averaged the three together, but the validity of this approach is also in doubt.

Lastly, in my lectures I have been focusing only on Stage A (in which I have confidence) and saying that HDPE geomembranes at 20°C have 200 years lifetime without any change in mechanical properties. This leaves Stages B and C open for discussion.

Best wishes in your activities.

APPENDIX D

R.K. Rowe's paper

Long-term performance of contaminant barrier systems

Rowe, R. K. (2005). *Géotechnique* 55, No. 9, 631–678

D2/48

Long-term performance of contaminant barrier systems

R. K. ROWE*

This lecture describes the latest findings with respect to the long-term performance of modern municipal solid waste (MSW) landfill barrier systems. Field data relating to the clogging of leachate collection systems and the latest techniques for predicting their performance are examined. It is indicated that the primary leachate collection systems may have service lives that range from less than a decade to more than a century, depending on the design details, waste characteristics and mode of operation. Recent data indicate that landfill liner temperatures can be expected to reach at least 30–40°C for normal landfill operations. With recirculation of leachate the liner temperature increases faster than under normal operating conditions, and may be expected to exceed 40°C. Temperatures (up to 40–60°C) may occur at the base of landfills where there is a significant leachate mound. Temperature is shown to have a significant impact on both contaminant migration and the service life of the liner system. Field measurements and theoretical calculations show that composite liners are substantially better than single liners in terms of controlling leakage from landfills. Also, the leakage rates with a composite liner are very small, and diffusion will dominate as a transport mechanism for contaminants that can readily diffuse through a geomembrane (GM). Composite liners involving a GM over a geosynthetic clay liner (GCL) gave rise to substantially less leakage than those involving a compacted clay liner (CCL). The observed leakage through composite liners can be explained by the holes in, or adjacent to, wrinkles/waves in the GM, and this leakage can be calculated using simple equations. High-density polyethylene (HDPE) GMs provide an excellent diffusive barrier to ions. However, some organic compounds readily diffuse through HDPE GMs, and a combination of GM and an adequate thickness of liner and attenuation layer are required to control impact to negligible levels. The long-term performance of HDPE GMs is discussed. Based on the currently available data, the service life for HDPE GM in MSW landfill is estimated to be about 160 years for a primary liner at 35°C and greater than 600 years for a secondary GM provided it is at a temperature of less than 20°C. Clay liners are susceptible to both shrinkage and cracking during construction (due to heating by solar radiation or freezing) and after placement of the waste (due to temperature gradients generated by the waste). The former can be controlled by quickly covering the liner with a suitable protection layer. The latter can be controlled by appropriate design. The use of numerical models for predicting the service lives of engineered systems and long-term contaminant transport is demonstrated.

KEYWORDS: case history; environmental engineering; geosynthetics; industrial waste; laboratory tests; landfills; numerical modelling & analysis; pollution migration/control; temperature effects; time dependence

Manuscript received 19 July 2005; revised manuscript accepted 9 September 2005.

Discussion on this paper closes on 2 May 2006, for further details see p. ii.

* GeoEngineering Centre at Queen's-RMC, Department of Civil Engineering, Queen's University, Kingston, Ontario, Canada.

Cet exposé donne les dernières conclusions sur la performance à long terme des systèmes de barrage modernes dans les décharges municipales de déchets solides (MSW). Nous examinons les données de terrain relatives à l'engorgement des systèmes de collecte au leachate ainsi que les dernières techniques pour prévoir leur performance. Nous savons que les systèmes primaires de collection au leachate peuvent avoir des vies utiles allant d'à peine dix ans à plus d'un siècle, selon leur conception, les caractéristiques des déchets et les modes d'exploitation. Des données récentes indiquent que les températures des doublures peuvent atteindre entre 30 et 40°C pour les sites normaux. Avec la recirculation du leachate, la température de la doublure augmente plus vite qu'en conditions d'exploitation normales et elle peut même dépasser les 40°C. Les températures peuvent atteindre jusqu'à 40–60°C à la base des décharges là où se trouve une accumulation significative de leachate. Nous montrons que la température a un impact significatif sur la migration des contaminants et sur la durée de vie utile du système de doublure. Des mesures prises sur le terrain et des calculs théoriques montrent que les doublures composites sont substantiellement meilleures que les doublures simples pour ce qui est du contrôle des fuites. De plus, le nombre de fuites avec une doublure composite est très faible et la diffusion va dominer comme mécanisme de transport des contaminants qui peuvent facilement passer à travers une géomembrane (GM). Les doublures composites constituées d'une GM sur une doublure en argile géosynthétique (GCL) donnent lieu à beaucoup moins de fuites que celles ayant une doublure d'argile compactée (CCL). Les fuites observées à travers la doublure composite peuvent s'expliquer par la présence de trous ou de plissement/vagues dans et à côté de la GM et ces fuites peuvent être calculées en utilisant des équations simples. Des GM en polyéthylène à haute densité (HDPE) sont une excellente barrière diffusive aux ions. Cependant, certains composés organiques se diffusent facilement à travers les GM en HDPE et une combinaison de GM, d'épaisseur adéquate de doublure et de couche d'atténuation est nécessaire pour maintenir l'impact à des niveaux négligeables. Nous étudions la performance à long terme des GM en HDPE. D'après les données disponibles actuellement, la durée de vie utile des GM en HDPE dans les décharges MSW est évaluée à environ 160 ans pour une doublure primaire à 35°C et à plus de 600 ans pour une GM secondaire du moment que sa température reste inférieure à 20°C. Les doublures en argile sont sujettes au rétrécissement et aux cassures pendant la construction (en raison de la chaleur solaire ou du gel) et après mise en place des déchets (en raison des montées de température produites par les déchets). Le premier problème peut être solutionné en couvrant rapidement la doublure d'une couche protectrice appropriée. Le second problème peut être résolu par une conception mieux adaptée. Nous montrons comment utiliser des modèles numériques pour prédire les durées de vie utile des systèmes industriels et du transport des contaminants à long terme.

D3/48

INTRODUCTION

President Carter's declaration of a State of Emergency at Love Canal, NY, in 1978, and the associated evacuation of 236 families from homes around the landfill, highlighted for the public the potential risks associated with waste disposal. Following recognition of the problems of past waste disposal activities, waste disposal has changed very significantly since the 1970s. Today most developed countries have regulations controlling the location, design and operation of landfills. This typically involves limiting the type of waste that can be disposed in the landfill (e.g. not permitting landfill disposal of the liquid hazardous wastes that caused the most serious past problems) and requiring a barrier system to separate the waste and the associated leachate from the groundwater system. In some cases this is hoped (without any quantification) to provide environmental protection. In other cases the barrier system is explicitly (and quantitatively) designed to control contaminant impact to negligible levels. When such systems are well designed and constructed, there is now a wealth of data (mostly in unpublished monitoring reports, but a good publicly available example is given in the data collected by Bonaparte *et al.*, 2002) to suggest that they are performing well. However, most modern landfills with engineered liners have been in operation for less than 20 years. Thus the question arises as to what some of the key factors are that will influence the long-term performance of these systems. This paper addresses this critical question.

Amongst many possible applications, barrier systems can be used to contain possible future chemical spills (e.g. around tanks containing hydrocarbons), contain contaminated fluid in lagoons (e.g. leachate), collect solute from heap leach pads, minimise acid drainage from mine waste, and control contaminant migration from landfills. While there is material in this lecture applicable to this wide range of applications, the focus will be on barrier systems for municipal, industrial, hazardous and incinerator waste landfills (generically referred to as landfills), with particular emphasis on municipal solid waste (MSW) landfills. In the following discussion the term 'barrier system' should be interpreted as a landfill barrier system.

There is a wide variety of possible barrier systems (Rowe *et al.*, 2004), but most involve some combination of traditional soil materials (sand, gravel, clay) and geosynthetic materials. A barrier system typically involves many different components, including filtration/separation layers, drainage layers and protection layers, as well as one or more low-permeability liner. One possible barrier system is shown schematically in Fig. 1. An assessment of the long-term performance of these systems requires consideration of both the soil and geosynthetic components and their interactions, with the performance of one component typically impacting on that of another: this is the primary underlying theme of this lecture.

In many respects, the lecture logically follows from Mitchell's (1991) 31st Rankine Lecture on 'Conductivity phenomena', with the flow of fluids, chemicals and energy playing a key role in what will be discussed in the subsequent sections. Furthermore, these flows are generally coupled. Thus the second theme is the importance of coupled phenomena. Mitchell (1991) and more recently Rowe *et al.* (2004) and Mitchell & Soga (2005) have provided the fundamental framework for much of the discussion of conductivity phenomena to be presented herein.

Given the timescales involved, numerical analysis is a necessity if one wishes to predict the long-term performance of barrier systems. Of course that analysis must be rooted in reality, and it must be conducted by those who understand both the potential of the analysis for improving design and predictive monitoring, as well as its limitations and pitfalls.

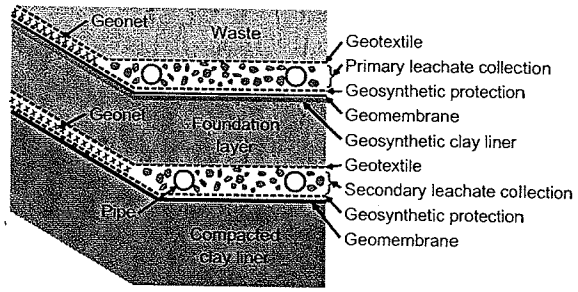


Fig. 1. Schematic showing one possible double composite liner system including, from top down: waste; primary leachate collection system (on the base comprising a geotextile separator/filter between the waste and underlying coarse gravel drainage blanket with perforated leachate collection pipes at a regular spacing and, on the side slope, a geotextile and geonet) used to control the leachate head on the underlying primary liner; a geotextile protection layer (this is often augmented with a sand protection layer not shown here); a primary liner (comprising an HDPE geomembrane, a geosynthetic clay liner, and a foundation layer (omitted in many designs)); a secondary leachate collection/leak detection system (similar configuration to primary system described above) intended to control the head on the secondary liner and to detect and collect leachate leakage through the primary liner; a secondary protection layer; and a secondary liner (comprising an HDPE geomembrane and a compacted clay liner, which in many designs is replaced by a GCL over a suitable attenuation layer)

Laboratory studies are required to identify the key phenomena, to obtain parameters for analysis, and as a relatively controlled benchmark against which the numerical analysis can be tested. There is also a need to test the models against observed field performance where data are available. To the extent that space permits, this lecture seeks to address each of these aspects, and thus the third theme of the lecture is the role of analysis in the assessment of long-term performance of barrier systems. The three themes are each intertwined, and will be touched upon in each of the following sections.

Some basic concepts

In the context of the lecture, a barrier system is intended to control contaminant transport and ensure negligible long-term environmental impact. To do so it is necessary to have a facility that will control contaminant impact for the contaminating lifespan of the landfill. (The reader is referred to Table 1 for a definition of terms not common in geotechnical engineering usage, such as 'contaminating lifespan'.) An evaluation of the potential long-term impact must involve consideration of contaminant transport in the context of the service lives of the various engineered components of the system. It must take account of the fact that components of the system are likely to fail at different times, with the primary leachate collection system (LCS) and geomembrane (GM) liner being the most vulnerable because they are subjected to the severest chemical and biological conditions. Thus one should not expect that each component of the barrier system will function for the entire contaminating lifespan (which, for a large landfill, can be expected to be hundreds of years; Rowe *et al.*, 2004) but, rather, that the system as a whole will provide the long-term protection that is required.

The primary mechanisms for contaminant transport that typically need to be considered for modern facilities are advection and diffusion. Other mechanisms relevant to the clean-up of past problems with unlined disposal sites for

D4/48

hazardous waste or brines, such as density-driven migration, are omitted because the disposal of these contaminants as liquids is generally not permitted in modern waste disposal facilities. Advection is a physical process whereby contaminants introduced into a groundwater flow system migrate in solution (as solutes) or in suspension (e.g. silt and fine sand particles, microbes) along with the movement of leachate or groundwater. It is governed by Darcy's Law, with the Darcy flux, v_a, given by

$$v_a = -ki \quad (1)$$

where k is the hydraulic conductivity (coefficient of permeability) and i is the hydraulic gradient, which is often controlled by the level of leachate mounding on the landfill liner. Diffusion involves the migration of molecules or ions in air, water or a solid (e.g. a GM, as will be discussed later), as a result of their own random movements, from a region of higher concentration to a region of lower concentration. Diffusion can occur in the absence of any bulk air or water movement. Diffusive transport is generally governed by Fick's law, with the diffusive flux f given by

$$f = -Di_c \quad (2)$$

where D is the diffusion coefficient and i_c is the concentration gradient. In addition to being important as a contaminant transport mechanism from a landfill, diffusion is also a critical transport mechanism between leachate and biofilms that develop on drainage media in the landfill, and is therefore an important factor influencing the rate of clogging of LCSs (to be discussed subsequently). Diffusion of water vapour is also an important consideration when modelling the potential desiccation of clay liners.

When dealing with contaminant transport through porous media, the apparent diffusion of a contaminant from a source of high concentration to a receptor of lower concentration is a complex process, which involves molecular diffusion due to the concentration gradient. However, as discussed in more detail by Rowe *et al.* (2004), it is also influenced by factors such as the complex tortuosity of the porous media, osmotic flow, electrical imbalance, and possible anion exclusion. Typically, diffusion parameters are inferred from laboratory tests conducted using the soil of

Table 1. Definition of some terminology used in landfill design (based on Rowe *et al.*, 2004)

Term	Definition
Antioxidant	An additive in polymer formulation intended to halt oxidative reactions during manufacture and over the polymer's service life.
Attenuation	The process whereby the concentrations of chemical species in groundwater or leachate are reduced as contaminants move throughout the barrier system and subsurface.
BOD: biochemical oxygen demand	The amount of oxygen used in the biochemical oxidation of organic matter in water over a specified time under specified conditions. BOD_5 is the BOD measured in a 5-day test. It is a standard test used in assessing wastewater strength.
Biodegradation	The transformation (through metabolic or enzymatic action) of organic substances to smaller molecules via oxidation and reduction mechanisms induced by the metabolic activity of microorganisms.
BTEX	The group of volatile aromatic hydrocarbons: benzene, toluene, ethylbenzene and xylenes.
COD: chemical oxygen demand	A measure of the amount of organic substances in water. Includes non-biodegradable and recalcitrant (slowly degrading) compounds that are not included in BOD_5.
Clogging	A build-up of biofilm, chemical precipitates and small (e.g. silt and sand) particles that are deposited in pipes, granular material (e.g. sand or gravel), and geotextiles that are used in drainage systems. This build-up progressively reduces the hydraulic conductivity of the system and hence its ability to drain fluids (e.g. leachate).
Composite liner	A barrier to contaminant migration composed of two or more liner materials. Typically a geomembrane and compacted clay liner (CCL) or a geomembrane and geosynthetic clay liner (GCL).
Conservative contaminant	An unreactive contaminant that does not degrade and the movement of which is not retarded. A typical example is chloride (Cl^-).
Contaminating lifespan	The period of time during which the landfill will produce contaminants at levels that could have unacceptable impact if they were discharged into the surrounding environment.
CQA	Construction quality assurance.
CQC	Construction quality control.
DCM: dichloromethane	A volatile chlorinated hydrocarbon used in paint removers and chemical processing. Also known as methylene chloride; CH_2Cl_2.
Diffusion	Migration of molecules or ions in air, water or a solid as a result of their own random movements from a region of higher concentration to a region of lower concentration. Diffusion can occur in the absence of any bulk air or water movement.
Geonet (GN)	A planar, polymeric structure consisting of a regular, dense network of integrally connected overlapping ribs.
Geotextile (GT)	A planar, polymeric textile material, which may be woven or nonwoven.
Geomembrane (GM)	A relatively impermeable, polymeric sheet.
Geosynthetic clay liner (GCL)	Factory-manufactured hydraulic barrier consisting of a layer of bentonite clay supported by geotextiles and/or geomembranes, and held together by needle-punching, stitching, or chemical adhesives.
Flux	Rate of movement of mass or heat through a unit cross-sectional area per unit time in response to a concentration, hydraulic, or thermal gradient.
FSS: inorganic (sometimes called fixed) suspended solids	FSS is made up of inert biomass, mineral precipitate, and soil particles. FSS = TSS-VSS.
Geosynthetic	A polymeric material used in geotechnical applications.
Interface transmissivity	The capacity of the space between a geomembrane and an underlying material to convey fluid. Depends on interface contact conditions, and is required to estimate leakage through holes in geomembranes that are part of composite liners.

(*continued*)

25/48

Table 1. ***(continued)***

Term	Definition
Leachate	A liquid produced from a landfill that contains dissolved, suspended and/or microbial contaminants from the waste.
Leachate mound	The surface of gravity-controlled water (leachate) in a landfill. It generally corresponds to the top of the zone of saturation in the waste or leachate collection system. Generally represents the water pressure acting on the primary liner in a landfill.
Leak detection system (LDS)	Refers to geonet or gravel drainage layer used to monitor volume of fluid and chemical concentrations passing through a liner system.
Leakage	The movement of fluid through a hole in a geomembrane under a hydraulic gradient. Typically expressed as litres per hectare per day (lphd).
Methanogenic conditions	Environment under which methane is formed by obligate anaerobes.
OIT: oxidative induction time	A relative measure of a material's resistance to oxidative decomposition as determined by the thermoanalytical measurement of the time interval to onset of exothermic oxidation of a material at a specified temperature in an oxygen atmosphere.
Service life	The period of time for which an engineered component of a barrier system performs in accordance with the design assumptions.
Stress cracking	An external or internal rupture in a plastic caused by a tensile stress less than its short-term mechanical strength.
TSS: total suspended solids	TSS comprises volatile suspended solids (VSS) and inorganic suspended solids (FSS), and is obtained using a gravimetric measurement of the residue retained on a 0·45 μm glass fibre filter dried at 105°C.
Transition metals	Metals that have a partially filled *d* shell. Examples include Cr, Mn, Fe, Co, Ni, Cu, Zn, Pd, Cd.
VOC	Volatile organic compound.
VFA: volatile fatty acid	Fatty acids contain only the elements of carbon, hydrogen and oxygen, and consist of an alkyl radical attached to a carboxyl group. The lower molecular weight, volatile, fatty acids are liquids that are soluble in water, volatile in steam, and have a notable odour. Examples commonly found in leachate include ethanoic (acetic) acid [CH_3COOH], propanoic (propionic) acid [CH_3CH_2COOH] and butanoic (butyric) acid [CH_3CH_2 CH_2COOH]. VFAs represent the primary component of COD, and their concentration is often expressed in terms of COD.
Volatile suspended solids (VSS)	VSS comprises active biomass (typically about 70%), including acetate, butyrate, and propionate degraders, and 'inactive' biomass (about 30%), which includes dead bacteria. VSS is obtained using a gravimetric measurement of the residue retained on a 0·45 μm glass fibre filter dried at 550°C.
Wrinkles (waves)	Unevenness of a geomembrane when placed on a flat surface. Often occurs as a result of thermal expansion (after placement) arising from solar radiation.

interest and a leachate similar to that anticipated in the field application, without explicitly considering the role of all these factors, and hence the corresponding effective diffusion coefficient reflects multiple factors (in addition to molecular diffusion).

For many practical situations (Rowe *et al.*, 2004), contaminant transport modelling through the soil component of barrier systems involves solving the equation for one-dimensional contaminant transport of a single reactive solute through a porous medium:

$$n\frac{\partial c}{\partial t} = nD_e\frac{\partial^2 c}{\partial z^2} - \rho_d K_d\frac{\partial c}{\partial t} \quad (3)$$

subject to appropriate boundary and initial conditions, where c is the concentration at depth z and time t, n is the effective porosity, D_e is the effective diffusion coefficient, ρ_d is the dry density, and K_d is the partitioning coefficient.

Landfills generate heat from the biodegradation of organic wastes and the hydration of certain inorganic wastes (e.g. incinerator ash). The consequent heat flow can influence liner temperatures and consequently contaminant transport (as both k and D are temperature dependent) as well as the service life of both the geomembrane and clay liners (as will be subsequently discussed). Heat flow is governed by Fourier's law, with the heat flux q_T given by

$$q_T = -\lambda' i_T \quad (4)$$

where λ' is the thermal conductivity and i_T is the thermal gradient.

Structure

This lecture will focus on seven key issues relevant to the assessment of the likely long-term performance of barrier systems. It begins with a discussion of the factors associated with the clogging of the LCS and the modelling of the bio-geochemical clogging processes that can ultimately reduce the porosity and hydraulic conductivity to the point where the leachate head on the liner can no longer be controlled to the design level (this is the point where the service life of the LCS has been reached). Based on field data, the temperature of landfill liners will then be reviewed, as this will impact on all subsequent issues. Leakage (advective flow) through composite liners is examined based on both field observations and theoretical calculations. Diffusive transport through natural clayey deposits, compacted clay liners (CCLs), geosynthetic clay liners (GCLs) and geomembranes (GMs) is examined, with emphasis on the potential for contaminant diffusion through a solid GM during the period that it is a very effective barrier to fluid flow (i.e. while there is very low leakage). The service life of primary and secondary high-density polyethylene (HDPE) GMs is then discussed, with particular attention being paid to the effect of liner temperature and the interaction between the liner and leachate on the service life of the GM. The potential for desiccation of clay liners due to thermal gradients induced by the waste is then examined with respect to both experimental evidence and coupled modelling of heat and moisture transfer as well as stress and deformations for the liner. Finally, the lecture considers the long-term performance of barrier systems, in terms both of some field experience and of long-term contaminant transport modelling (taking account of issues such as the service lives of components of the barrier system). One could give a lecture on each of these issues but, given space limitations, only a few key aspects of each issue will be discussed.

This lecture focuses on the geoenvironmental aspects of controlling contaminant migration. While important, this is

but one aspect of landfill design. To ensure a safe landfill, attention must also be paid to environmental geotechnics (especially issues of stability) and landfill operations: these issues are beyond the scope of this lecture.

LONG-TERM PERFORMANCE OF LEACHATE COLLECTION SYSTEMS

Overview

Leachate is the contaminated water predominantly generated from the percolation of precipitation through waste and by the biodegradation of the waste. It contains dissolved, suspended and microbial contaminants. Although modern MSW leachate typically has low concentrations of toxic compounds compared with the levels encountered when there is co-disposal of hazardous waste (as was once common practice), there is still a need to control their migration and prevent groundwater contamination.

The LCS represents a critical component of barrier systems for landfills. The primary function of the LCS is to control the leachate head acting on the underlying liner. The secondary function is to remove leachate for either recirculation or treatment and disposal. Clogging of LCSs involves a reduction in hydraulic conductivity of the drainage material and/or filter to the point where it results in the development of a leachate mound acting on the base of the landfill. As the mound increases in height, the advective transport of contaminants through the liner system increases, increasing the potential for groundwater contamination.

The design of LCSs has evolved considerably over time, and there are many misconceptions about appropriate design and operation procedures. The earliest form of LCS involved a toe drain and/or perimeter drain. These systems minimised surface water contamination by controlling leachate seeps through the landfill cover, but they were ineffective at controlling the height of the leachate mound within the waste and the consequent contaminant escape into the underlying soil and groundwater. The next generation of leachate removal systems involved the inclusion of granular material (French drains) in conjunction with perforated drainage pipes, typically located every 50–200 m along the landfill base. These systems provided greater control of the leachate mound than toe drains or perimeter drains, but the confluence of flow through a limited surface area made the drains particularly prone to clogging (especially when they were wrapped in a geotextile). As discussed below, clogging of either a granular material or a geotextile filter impairs the ability for leachate to enter the leachate collection pipe, causing a build-up of a leachate mound between the drains. This mound induces some flow towards the pipes, but also increases outward flow through the bottom of the landfill.

Modern LCSs typically comprise a continuous blanket of granular material covering the landfill base liner and a regular pattern of leachate collection pipes. Some systems use sand for the drainage blanket; however, it has been shown that sand can readily clog (e.g. Reades *et al.*, 1989). The replacement of sand by gravel has improved the performance of these systems, but clogging will still occur. This raises two questions: (a) how does one design a system to maximise the long-term performance and hence its service life; and (b) what is the service life of the system? This section will:

(*a*) examine the mechanisms governing the clogging of leachate collection systems
(*b*) discuss the insights gained from both field and laboratory studies
(*c*) examine techniques for modelling the performance of these systems.

Keele Valley Landfill (KVL)

The Keele Valley Landfill (KVL), located north of Toronto in Maple, Canada, covers 99 ha and is divided into four quadrants, referred to as Stages 1 to 4. The barrier system consists of a CCL, typically 1·5 m thick, overlain by a 0·3 m thick sand protection layer and an LCS as discussed below. Stage 1 (north-east portion) was constructed between 1983 and 1985. Stage 2 (north-west portion) was constructed between 1986 and 1988. Construction of Stages 3 and 4 was started in 1988 and 1990 respectively and completed in 1994. The LCS in Stages 1 and 2 consists of a series of French drains (50 mm washed, crushed gravel mounds 0·5 m high and spaced 65 m apart) draining into four main 150 mm diameter perforated collection pipes (8 mm diameter perforations) encapsulated in 40 mm washed crushed gravel mounds 0·5 m high and 200 m apart. In Stages 3 (south-east) and 4 (south-west), the LCS consists of a granular drainage blanket (0·3 m thick, 50 mm washed, crushed gravel) draining into the four main leachate collection pipes. The four primary pipes run predominantly north-west to south-east the full length of the landfill, are approximately 900 m, 950 m, 1200 m and 1200 m long, and drain to a common holding tank. In addition, a fifth 300 m long collection pipe drains the southeast corner of Stage 4.

The landfill capacity is approximately 33 million m^3, and the maximum depth of waste at closure was about 65 m. Between the first acceptance of waste in 1984 and landfill closure in December 2002, it received 28 million tonnes of MSW from the Greater Toronto Area. As will be discussed in several sections of this lecture, the KVL has been extensively monitored and studied.

Trends in KVL leachate data

As subsequent sections will discuss both field and laboratory studies related to the KVL leachate, it is useful to look at the changes in the concentration of key components of the leachate collected at the main holding tank at the KVL over the past 21 years. Armstrong & Rowe (1999) found a strong correlation between the variability of the KVL leachate characteristics and both precipitation and the sequence of waste placement. Periods of heavy precipitation tended to result in lower-strength leachate, especially after new sections of the LCS had been constructed. The data also suggested that when fresh waste lifts were placed over older waste, the older waste acted as a bioreactor that treated the leachate generated by the newer waste (much like what was observed in test cells by Ham & Bookter, 1982). These results suggest that planned waste placement can play a role in the treatment of leachate before collection and removal. However, as will be illustrated in the following sections, the waste is not the only place where leachate 'treatment' occurs.

Despite the variability in leachate characteristics, trends can be identified in plots of 2-month rolling average of key characteristics with time (Fig. 2). Chloride represents the 'stable' inorganic constituents, because its concentration is largely independent of biological and chemical interactions. The chloride concentration has increased over the past 21 years, although it may have levelled off over the past couple of years at an average concentration of about 4000 mg/l (Fig. 2). In contrast, the calcium concentration reached a peak annual average value of 1580 mg/l in 1991. The average calcium concentration over the first decade (1984–1993) was about 1200 mg/l. The end-of-pipe concentration of calcium has subsequently decreased to quite low values (average 45 mg/l since closure).

The level of organic contaminants in the leachate can be represented in terms of the chemical oxygen demand

D7/48

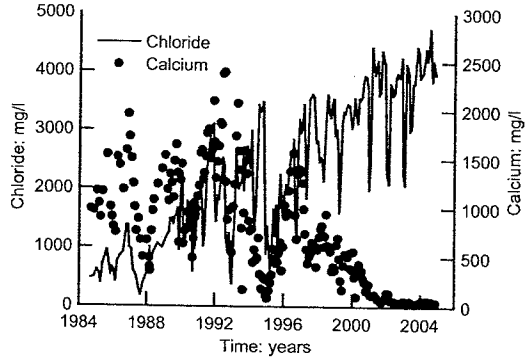


Fig. 2. Variation in end-of-pipe chloride and calcium concentrations with time for the period 1984–2004

(COD), which averaged about 12 200 mg/l over the first decade and has decreased to about 2300 mg/l over the last 2 years since closure. Over the first decade pH was relatively stable, with an average value of 6·3, which increased to an average of 7·4 over the next 9 years and to 7·7 since closure. Thus the leachate data suggest that there is a shift in the end-of-pipe leachate chemistry with the 'stable' inorganic load still increasing while the organic load (represented by COD) and 'unstable' inorganic load (represented by calcium) are decreasing. Of particular note is the strong correlation between the decrease in COD and calcium relative to chloride, as is evident from Fig. 3. This trend suggests that the decrease in calcium is related to the decrease in COD, as has been suggested, based on theoretical considerations, by Rittmann *et al.* (1996). COD has both a readily degradable component—the biochemical oxygen demand (BOD_5)—and a slowly degrading component. Thus the ratio of BOD_5 to COD provides an indication of the amount of COD that can be readily biodegraded. In the KVL leachate this ratio was relatively constant, with an average value of 0·65, typical of 'young' leachate, from 1984 until the end of 2000, after which it rapidly decreased to 0·11. Thus, prior to closure, the decrease in the concentrations of organic constituents (and consequently COD) at the end of the LCS does not appear to have been associated with an apparent 'ageing' of the waste. This is likely a consequence of end-of-pipe leachate reflecting the net effect of degradation of new waste (higher concentrations), treatment within the landfill, and degradation of old waste (lower concentrations).

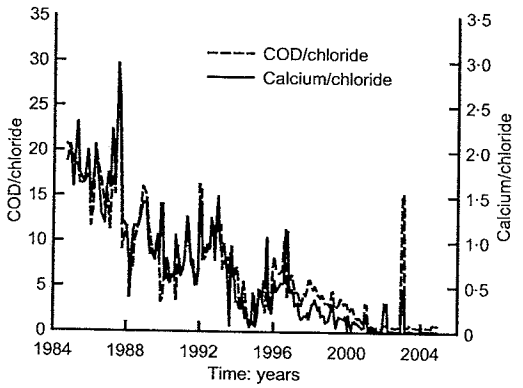


Fig. 3. Variation in end-of-pipe COD/chloride and calcium/chloride with time for the period 1984–2004

Thus 'ageing' or degradation of waste is occurring, but the overall net effect of new and old waste ageing at different rates is being observed. Although not conclusive, the change in the ratio of COD/Cl and in Ca/Cl (Fig. 3), combined with the fact that the average ratio of BOD_5/COD has remained at about 0·65 for almost the first 17 years, suggests that the leachate was being 'treated' as it passed through the LCS. If this is true, then one would expect to find a build-up of organic and inorganic material within the LCS. This will be shown to be the case in the following section.

Field examples of clogging

Clogging of LCSs has been observed in landfills that have involved a wide range of collection systems, ranging from French drains to continuous sand and gravel layers, and in systems both with and without geotextiles (Table 2). For example, field exhumations (Reades *et al.*, 1989; Barone *et al.*, 1993) found that the upper portion of the sand 'protection' layer over the liner at the KVL became clogged within the first four years, and did not contribute to the hydraulic performance of the collection system. In addition to visual evidence of clogging in the upper portion of the sand layer, the lack of flow in the sand layer was clearly evidenced by the development of a clear diffusion profile starting at the top of the sand layer (Fig. 4), which demonstrates that the sand layer was acting as part of the liner in terms of a 'diffusion barrier'. It is particularly notable that both the inorganic contaminant chloride and several organic contaminants (especially toluene) exhibit diffusion profiles through both the sand and the clay. One would not expect to find profiles such as this if there were any significant flow in the sand layer. Koerner & Koerner (1995b) have also reported clogging of a sand protection layer (Case 3, Table 2), where after 10 years the hydraulic conductivity dropped by three orders of magnitude, from 4×10^{-4} m/s to 2×10^{-7} m/s, and leachate was flowing through the waste rather than the sand.

Exhumation of portions of the continuous gravel drainage blanket in later stages of the KVL (Cases 1 and 2 in Table 2; Fleming *et al.*, 1999) indicated a three order of magnitude drop in the hydraulic conductivity of relatively uniform 50 mm gravel near the leachate collection pipe after 4 years (although the hydraulic conductivity was still sufficient to transmit leachate without the development of a leachate mound). Clogging was observed to be substantially less (Case 2 in Table 2) in areas where a geotextile filter was used between the waste and the gravel than where the waste was in direct contact with the gravel. Additional evidence of clogging at the KVL is provided by physical observations of clogging of the main header line leading to the main manhole. In the spring of 2001, the main header was so occluded with clog material that a pipe observation camera could not enter the pipe.

Koerner *et al.* (1994) and Koerner & Koerner (1995b) reported excessive clogging of a geotextile in two cases where the geotextile was wrapped either around the perforated pipe or around the gravel in a drainage trench (Cases 3 and 5 in Table 2) and some clogging in a third case (Case 4).

The field evidence cited above, and in Table 2, shows that there are deposits of both biofilm and inorganic precipitate on the surface of the granular material, which reduces the drainable porosity and hydraulic conductivity of the drainage material. Where clog compositions have been reported (e.g. Brune *et al.*, 1991; Fleming *et al.*, 1999), they comprised over 50% calcite ($CaCO_3$), 16–21% silica where there is no filter, and iron and manganese representing up to 8% and 5% respectively. Calcite was also the dominant mineral in clog scale obtained from a leachate collection pipe in a

Table 2. Summary of observations from exhumation of leachate collection systems (modified from Rowe, 1998a)

Case	Waste type*,† Age; leachate	Collection system design‡	Key observations
1.	MSW and LIW ~4 years COD = 14 800 mg/l BOD_5 = 10 000 mg/l pH = 6·3 Performance: Adequate at time of exhumation	Blanket underdrain: waste over 50 mm relatively uniform gravel; 200 mm, SDR 11 HDPE pipe; 8 mm holes Pipe never cleaned No geotextile between waste and gravel	30–60% loss of void space in upper stone 50–100% loss of void space near pipe Permeability of gravel decreased from ~10^{-1} m/s to ~10^{-4} m/s All lower holes in pipe blocked; majority of upper holes blocked Large clog growth inside pipe
2.	MSW and LIW As for Case 1 above	Blanket underdrain: waste over geotextile over 50 mm gravel (rest as above) (GT: W; M_A = 180 g/m^2, AOS = 0·475 mm, t_{GT} = 0·6 mm, P = 0·04 s^{-1})	Substantially less clogging than observed in Case 1 above where there was no GT 0–20% loss of void space in upper gravel below geotextile
3.	MSW and LIW; LR ~ 10 years COD = 31 000 mg/l BOD_5 = 27 000 mg/l pH ~ 6·9 Performance: No flow in LCS; high leachate mound	Toe drain only Trench with 600 mm of crushed gravel (6–30 mm) around geotextile wrapped 100 mm SDR 41 perforated PVC pipe (GT:HBNW; M_A = 150 g/m^2, AOS = 0·15 mm; t_{GT} = 0·30 mm, P = 1·1 s^{-1})	Flow reduction noted after 1 year Pipe crushed (likely due to construction equipment) Substantial reduction in void space and cementing of gravel. k reduced from 2·5 × 10^{-1} m/s to 1·2 × 10^{-4} m/s Sand (SW; AASHTO #10) layer above GM was clogged and leachate drained on top (not through this layer). k reduced from 4 × 10^{-4} m/s to 2 × 10^{-7} m/s Excessive clogging of GT (k dropped from 4·2 × 10^{-4} m/s to 3·1 × 10^{-8} m/s)
4.	MSW and LIW; LR 6 years COD = 10 000 mg/l BOD_5 = 7500 mg/l pH ~7·5 Performance: Drain functioning adequately	Perimeter drain to control leachate seeps Geotextile wrapped trench with 6–18 mm gravel and 100 mm SDR 30 HDPE perforated pipe (GT: W, M_A = 170 g/m^2; POA = 7%, AOS = 0·25 mm; t_{GT} = 0·41 mm; P = 0·9 s^{-1})	Only small reduction in k of gravel from 5·3 × 10^{-1} m/s to 2·8 × 10^{-1} m/s Marginal clogging of GT (k dropped from 3·7 × 10^{-4} m/s to 1·4 × 10^{-4} m/s)
5.	ISS (included slurried fines 70% finer than 150 μm) 0·5 years COD = 3000 mg/l BOD_5 = 1000 mg/l pH = 9·9 Performance: No flow in LCS	Blanket underdrain: Waste over protection sand (0·075–4 mm) over geotextile (AOS = 0·19 mm) over pea gravel (1–20 mm) drainage layer; 100 mm diameter geotextile wrapped HDPE perforated pipe; 12 mm dia. holes (GT: NPNW, M_A = 220 g/m^2, AOS = 0·19 mm; t_{GT} = 2·7 mm, P = 1·8 s^{-1})	High leachate mound Upper geotextile functioning (k dropped from 4·9 × 10^{-3} m/s to 8·5 × 10^{-5} m/s) Pea gravel relatively clean Geotextile wrapping around perforated pipe excessively clogged (k dropped from 4·9 × 10^{-3} m/s to 4·4 × 10^{-8} m/s) Once geotextile sock removed, leachate flowed freely Heavy geotextile sock clogging at location of perforations in pipe
6.	MSW, LIW, ISS 6 years COD = 24 000 mg/l BOD_5 = 11 000 mg/l pH = 6·1 Performance: No flow of methane into extraction wells	Injection wells – 100 mm perforated PVC pipes schedule 40 (GT: NPNW, M_A = 176 g/m^2, t_{GT} = 2·2 mm, P = 1·1 s^{-1}) LCS not exhumed	Geotextile filter wrapping around extraction wells excessively clogged ~30% extraction wells stopped producing recoverable amounts of methane Geotextiles were black with organic material and caked with fine sediment Permeability reduction by as much as four orders of magnitude (k dropped from 2·3 × 10^{-3} m/s to 7·5 × 10^{-7} m/s).
7.	6 years Performance: Drain functioning adequately	Blanket underdrain: waste over 600 mm clean sand drainage material, HDPE collection pipes wrapped with geotextile, over HDPE geomembrane over 200 mm of clay, with saw tooth trench base configuration	Sections of manholes (consisting of flexible-walled PE segments) were collapsing and allowing drainage sand, cover soil, and waste into LCS. Sand layer permeability was reduced from 1·85 × 10^{-4} to 1·23 × 10^{-4} m/s (33% reduction) GT permittivity reduction was 48% but resulting increased head in the LCS was acceptable

(*continued*)

D8/48

Table 2. (*continued*)

8.	ISS (also accepted salt slag) COD = 51 000 mg/l BOD_5 = 23 300 mg/l pH = 5·9 Ca = 3500 mg/l Performance: Drainage pipes functioning	No description of LCS	Rapid filling (10–20 m/yr) Clogging was particularly intensed despite pipes being flushed at least once a year Clog constituents – calcium (21%), carbonate (34%) Clogging occurred during the early acetogenic phase of decomposition
9.	4 years COD = 1000 mg/l BOD_5 = 40 mg/l pH = 7·0 Ca = 132 mg/l Performance: Drainage pipes functioning	No description of LCS	Slowly filled (~2 m/yr) Very little clogging exhibited between annual flushing events Encrustation consisted of both inorganic precipitate and bacterial slime Leachate phase: stable methanogenic
10.	MSW and ISS Performance: No flow in LCS	Toe drain only	Leachate mounding Leachate drains involving toe drains were clogged Toe drain was replaced
11.	MSW and ISS ~16 years COD = 2300–6500 mg/l BOD_5 = 3900–8300 mg/l pH = 7·0 Performance: Plugging of the leachate header	Blanket underdrain: Waste over LCS (pipe spacing ranging from 50 to 200 m with gravel (5–20 mm) drainage layer) over 100–150 mm thick sand bentonite liner	High leachate mound (20 m) Elevated leachate temperature caused by sludge Clogging occurred during the early acetogenic phase of decomposition Bypass system was installed to divert leachate around a plugged section of the perimeter drain
12.	MSW and incinerator ash 2·5 years Performance: Cell 6 – no flow in LCS	Class 1 – double-lined landfill	Layer of hard mineral substance that filled ~25–100% of the collection pipes Calcite was the only mineral detected
13.	MSW and LIW 12 years COD = 400–6500 mg/l BOD_5 = 15–3400 mg/l pH = 7·2–8·2 Performance: High leachate mound	Blanket underdrain: Waste over a 300 mm thick gravel layer (20–40 mm). Central 150 mm diameter perforated drainage pipe. One vertical leachate pumping well at the lowest point of the cell. LCS not exhumed	Clog deposits (analysed by SEM-EDS) consisted of calcite and ferrous sulphur Residual drainable porosity values ranged from 25% (around the pumping well) to 50%
14.	MSW and LIW 5 years COD = 1400–4000 mg/l pH = 7·6–7·9 Performance: Adequate at time of examination	Blanket underdrain: Waste over a 500 mm thick gravel layer (20–40 mm). Two 225 mm diameter perforated drainage pipes. One central 225 mm diameter HDPE leachate evacuation pipe (evacuation by gravity). High-pressure cleaning of the main pipe and analysis of the collected deposits. LCS not exhumed	Deposits (analysed by SEM-EDS) consisted of calcite, dolomite and ferrous sulphur. Residual drainable porosity values ranged from 10% (in the upper part of the drainage layer) to 50% Main evacuation pipe almost completely clogged (more than 80%)

* MSW: municipal solid waste; LIW: light industrial waste; ISS: industrial solids and sludge; LR: leachate recirculation.
† Cases 1 and 2 (Fleming *et al.*, 1999); Cases 3–6 (Koerner & Koerner, 1995b); Case 7 (Craven *et al.*, 1999); Cases 8–9 (Brune *et al.*, 1991); Case 10 (McBean *et al.*, 1993); Case 11 (Rowe, 1998b); Case 12 (Maliva *et al.*, 2000); Cases 13 and 14 (Bouchez *et al.*, 2003).
‡ GT: geotextile; W: woven; HBNW: heat-bonded non-woven; NPNW: needle-punched non-woven; SDR: standard dimension ratio; M_A, mass per unit area; AOS: apparent opening size; t_{GT}: geotextile thickness; P: permittivity.

D9/48

D10/48

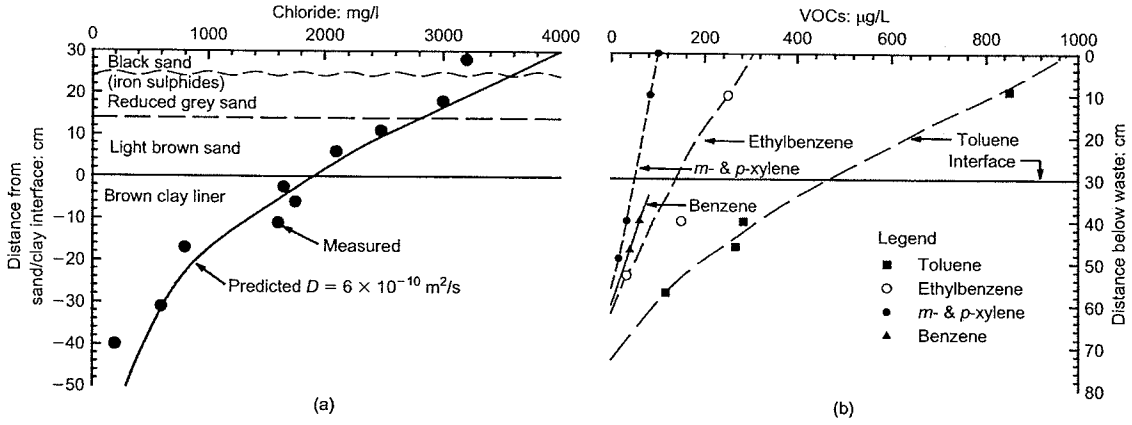


Fig. 4. Diffusion profile through a sand blanket and underlying compacted clay liner at the Keele Valley Landfill after 4·25 years: (a) chloride (modified from Reades *et al.*, 1989); (b) volatile organic compounds (modified from Barone *et al.*, 1993)

Florida landfill that received incinerator ash and MSW (Maliva *et al.*, 2000), and in clog scale obtained from a United Kingdom leachate collection pipe (Manning, 2000).

The leachate data for the KVL examined earlier (Figs 2 and 3) suggest that the clog material (both the organic and inorganic) is being formed by biologically induced processes that involve the removal of some of the organic leachate constituents (as implied by the reduction in COD) and precipitation of some inorganic leachate constituents (as implied by the reduction in calcium concentration). The following sections, which report the findings from laboratory mesocosm and column tests, provide clear evidence that this is indeed the case.

Laboratory mesocosm studies

To assess the performance of different LCS configurations, 18 flow cells (mesocosms) were set up (McIsaac *et al.*, 2000; Fleming & Rowe, 2004; Rowe & McIsaac, 2005) to simulate, at full scale and in real time, the last 0·5 m of a continuous granular blanket adjacent to a leachate collection pipe in a primary LCS. They involved waste material (a mix of waste and cover soil) overlying a 300 mm thick gravel drainage layer graded at 1·5% to a half section of PVC perforated pipe. Selected mesocosms also had a filter/separator layer between the waste material and drainage gravel. The various separator layers examined were a nonwoven needle-punched polypropylene geotextile, a woven slit-film geotextile, a graded granular filter consisting of 4 cm each of well-graded concrete sand and 6 mm nominal size pea gravel, and a 'sacrificial' layer of drainage gravel between the nonwoven geotextile and the waste material. Two sizes of gravel were examined (38 mm and 19 mm). KVL leachate was introduced vertically at a rate corresponding to an infiltration of 200 mm/yr and horizontally corresponding to the same infiltration over a catchment zone corresponding to 50 m spacing of the perforated drainage pipes. The mesocosm tests were conducted under anaerobic conditions at 27°C. Except for the fully saturated mesocosms, the bottom 100 mm of the gravel in each mesocosm remained saturated and the top 200 mm unsaturated during operation. Some of the mesocosms were placed in series, with the effluent from one mesocosm being the influent for another to demonstrate the effect of mass loading on clogging. Most of the mesocosms were terminated after 6 years of operation, although one was terminated after 11 years.

Designs without a separator/filter experienced increased amounts and rates of encrustation in the gravel drainage material and in the leachate collection pipes and perforations compared with those with a filtration/separation medium between the gravel and the waste material. The design with a woven geotextile separator prevented intrusion of the waste (and hence resulted in a void volume occupancy (VVO) of 25% over the full 300 mm of gravel, which was smaller than the VVO of 31% with no filter) but did not prevent migration of fines, because of its large filtration opening size (FOS) of 700 µm, and consequently there was more biological, physical and biochemical clogging within the gravel drainage layer than for mesocosms with either a nonwoven geotextile (VVO = 21%) or granular filter (VVO = 18%) separator/filter design.

The nonwoven geotextile (FOS of 110 µm) separator/filter resulted in increased biological and biochemical clog development in and above the nonwoven filter. The nonwoven geotextile effectively treated and filtered out fines from the vertically flowing leachate and resulted in less clog development within the drainage gravel than in the woven geotextile. Although some clogging of the nonwoven geotextile was observed, leachate still managed to percolate through the geotextile without any evidence of perching of leachate on the geotextile.

For designs with a graded granular filter, the sand layer readily clogged and reduced permeability was observed within the sand layer, which became cemented into clumps of sand and pea gravel. The granular filter was effective at filtering, straining and treating the leachate percolating through the waste layer, and resulted in the least clog mass in the underlying gravel of all the separator/filter designs. However, although no significant perched mounding was noted in the mesocosm, the clogging of the sand does have the potential to cause perched leachate mounding above the filter once the sand layer clogs.

Designs involving the placement of coarse gravel above and below a nonwoven geotextile developed a layer of gritty soft material on the top of the nonwoven geotextile owing to the retention of fines (washed through from the waste layer) and biofilm growth. Because of the large pore space of the gravel above the geotextile, the clog material in this layer was not sufficient to hinder its ability to transmit leachate.

Designs operated with the 300 mm gravel layer fully saturated exhibited an accumulation of soft clog throughout the full layer thickness and greater overall clogging (VVO = 45%) than for mesocosm where the saturated zone was only 100 mm (VVO = 31%). This difference in behav-

D11/48

iour is attributed to longer leachate retention time and an environment more conducive to microbial growth in the fully saturated systems. Note that, for the saturated mesocosm, there was 51% VVO in the lower 100 mm of gravel after only 1·6 years—more than in most other mesocosms after 6 years.

Tests conducted with 19 mm and 38 mm gravel (no filter/separator) demonstrated a more uniform distribution of clog material through the layer thickness for the 19 mm gravel than for the 38 mm gravel (where the clog focused at the bottom of the layer). For 19 mm gravel there was less intrusion of the waste into the upper portion of the gravel than for 38 mm gravel, but there was substantially more accumulation of clog material within the gravel itself.

As might be anticipated, based on the column studies of Rowe *et al.* (2000a), reducing the mass loading reduces clogging. The lowest VVO was for the mesocosm fourth in series, where the leachate reaching this mesocosm by lateral flow had already passed through three other mesocosms and hence had been depleted in nutrients and inorganic elements (e.g. Ca^{2+}). In this mesocosm, most of the clogging was due to the vertically percolating fresh leachate. Examining the four mesocosms in series it was found that there was a clear progression of decreased VVO in the saturated 100 mm (58% > 29–32% > 16% > 12%) with reduced mass loading.

Clogging of geotextiles

The field and mesocosm studies discussed above have demonstrated that, although geotextiles can clog, when used appropriately they can also minimise the clogging of the underlying LCS. Laboratory tests reported by Koerner *et al.* (1994) showed that the hydraulic conductivity of a geotextile can drop by 4–5 orders of magnitude due to clogging (with the lowest values being $3–4 \times 10^{-8}$ m/s). Geotextile clogging follows a similar pattern to that for granular materials. The clogging process begins with bacteria adhering to the geotextile fibres and the development of a fixed biofilm, which grows in small micro communities (Mlynarek & Rollin, 1995). Needle-punched nonwoven geotextiles provide a particularly large surface area for biofilm development. The biomass utilises available nutrients from the surrounding environment for cell growth and secretion of extracellular polysaccharides (EPS). As the biofilms grow larger, they can combine with other neighbouring biofilms and engulf geotextile fibres, thereby creating a natural biological filter. The biofilm occupies voids within the geotextile, and acts to entrap additional bacteria, suspended solids and organic matter from the passing leachate. Kossendey *et al.* (1996) found that microorganisms could not gain significant nourishment from the carbon content of the polymers. However, the provision of nutrients by permeating geotextiles with leachate having a significant organic component resulted in microbial growth and clogging.

Numerous investigators have studied the clogging of geotextiles permeated by MSW leachate. The magnitude of the decrease in hydraulic conductivity k_n (and permittivity P) depends on the geotextile (e.g. openness of the pore structure), the flow rate, and the concentration of the leachate. Koerner & Koerner (1995b) observed a decrease in geotextile hydraulic conductivity of between about two and five orders of magnitude. The lowest hydraulic conductivity observed in the laboratory tests, for a heat-bonded nonwoven geotextile, was similar to the lowest value observed for the clogged geotextiles recovered from the field (4×10^{-8} m/s).

Laboratory column studies

Several investigators have performed laboratory column tests to assess the effect of different variables on the clogging of granular porous media. These studies, combined with the field and mesocosm studies described earlier and with theoretical considerations (Rittmann *et al.*, 1996), have shown that the clogging of granular media involves a combination of biological, chemical and physical processes. Microbes in the leachate attach to solid matter (e.g. granular material, geotextile fibres, and the walls of leachate collection pipes), and a biofilm grows as nutrients are supplied by the leachate. The methanogenesis and acetogenesis of volatile fatty acids in the leachate cause an increase in the pH of the leachate and production of carbonate, which results in precipitation of inert material (predominantly $CaCO_3$), which accumulates on available solid surfaces and, in some cases, also settles to the bottom of the drainage media. Finally there is an accumulation of inorganic particles originally suspended in the leachate.

Brune *et al.* (1991), Paksy *et al.* (1998) and Rowe *et al.* (2000b) demonstrated that particle size and grading have a significant impact on the rate and extent of clogging in a granular medium. Columns with large particle diameters give rise to bioreactors as efficient as smaller particle diameters (i.e. result in a similar reduction in organic and inorganic loading in a given time). However, for larger particles, the clogging is distributed over a large volume of the porous media, and consequently there is much less reduction in hydraulic conductivity and a longer time until the material can be said to be clogged than for smaller particles. This can be attributed to the fact that the larger the grain size and the more uniform the grading, the larger are the pores that must be occluded to cause clogging and the smaller the surface area per unit volume (the specific surface) available for the biofilm growth. Clogging caused a drop in hydraulic conductivity by up to five to eight orders of magnitude.

Leachate contaminant mass loading has a significant impact on the rate and extent of clogging (Rowe *et al.*, 2000a). This is primarily a result of the increased mass of nutrients and inorganic material available for biological activity and precipitation. Higher flow rates give rise to less efficient bioreactors (i.e. a smaller reduction in organic and inorganic loading per unit volume of leachate in a given time); however, this is more than compensated for by the increase in mass loading associated with the higher flow rates.

Temperature also has a significant effect on the rate of clogging because of the increased biological activity associated with increasing temperature (Armstrong, 1998). However, the clogging processes were similar at all temperatures, and the relationship between the hydraulic conductivity and drainable porosity did not appear to be temperature dependent.

The comparative study of the clogging of tyre shreds and gravel (Rowe & McIsaac, 2005) found that 38 mm gravel maintained a hydraulic conductivity greater than 10^{-5} m/s for about three times longer than a similar thickness of large (75 mm × 75 mm) tyre shreds compressed at 150 kPa.

The microbial consortia found within the columns consisted of methanogens, denitrifiers, sulphate-reducing and other facultative anaerobes. Methanogenic bacteria dominated in the most intensely clogged zones within the columns. There was no significant observable difference in the microbial consortia with particle size, flow rate or temperature (Armstrong, 1998; Rowe *et al.*, 2000a, 2000b; Fleming & Rowe, 2004).

Examination of the changes in leachate as it passed through porous media indicated that clogging was predominantly associated with the deposition of inorganic precipitates (mostly $CaCO_3$), but that the precipitation of $CaCO_3$ was linked to the biological processes and the reduction of COD. An empirical relationship between $CaCO_3$ precipita-

tion and COD reduction was found to be essentially independent of particle size and temperature, but did increase somewhat with increased flow rate, which may be due to biofilm shearing effects at higher flow rates. This shearing can increase the effluent COD (i.e. reduce the amount of COD removed, other things being equal) and hence increase the ratio of $CaCO_3$ removed to COD removed.

Summary of clogging mechanisms

Clogging is initiated by microorganisms suspended in leachate attaching to and colonising the porous media surfaces as a biofilm comprising a consortium of microbes. The microbes excrete extracellular polysaccharides, which contribute further to the reduction in pore space and hydraulic conductivity of the material. Leachate contains organic acids and other nutrients conducive to biofilm development. In particular, young leachate (characterised by a high BOD_5/COD ratio) contains a large proportion of short-chain carbon acids (e.g. volatile fatty acids such as acetate, propionate and butyrate), which can be biodegraded by suspended and attached biomass within the filter and drainage material. The development of the biofilm is a function of the growth rate of the microorganisms, the substrate concentration, the attachment of microorganisms from the leachate onto the surface of the porous medium, and the detachment of microorganisms from the biofilm into the passing leachate. Thus clogging involves:

(*a*) retention (from the passing leachate onto the medium) of biomass (VSS) and suspended solids (FSS)
(*b*) fermentation of volatile fatty acids (VFAs), primarily acetate, butyrate and propionate, by biomass attached to the medium and suspended biomass (VSS)
(*c*) precipitation of cations (predominantly as $CaCO_3$) with calcium representing 20–30% of the dry mass of clog material formed using real leachate.

For real leachate (which has high VSS and hence provides a continuous supply of acid-consuming biomass), butyrate degraded before acetate, and acetate degraded before propionate (VanGulck & Rowe, 2004b). For synthetic leachate (which has very low VSS), acetate was the first to degrade, followed by butyrate (VanGulck and Rowe, 2004a). There was little propionate degraded in tests over a 1 year period, because propionate degraders have a much slower growth rate than acetate and butyrate degraders. For leachate with significant concentrations of calcium (i.e. greater than 300 mg/l), the three distinct clogging mechanisms (biological, chemical and physical) resulted in a significant (typically greater than 60%) reduction in drainable porosity and a five to eight order of magnitude decrease in hydraulic conductivity.

The precipitation of Ca^{2+} as $CaCO_3$ is directly correlated with the biodegradation of organic acids (represented in the foregoing discussion in terms of a reduction in COD). When examined in more detail (VanGulck *et al.*, 2003), it is found that the amount of carbonate is related to the conversion of propionate to acetate, butyrate to acetate, and acetate to methane, and can be represented in terms of microbiologically catalysed reactions (Parkin & Owen, 1986), as summarised below.

Propionate is fermented to acetate, carbonic acid, and hydrogen:

$$CH_3CH_2COOH + 3H_2O \rightarrow CH_3COOH + H_2CO_3 + 3H_2 \quad (5)$$

and hydrogen gas is oxidised to reduce carbonic acid to methane:

$$3H_2 + \tfrac{3}{4}H_2CO_3 \rightarrow \tfrac{9}{4}H_2O + \tfrac{3}{4}CH_4 \quad (6)$$

As insignificant H_2 accumulates in a balanced anaerobic system, reactions (5) and (6) can be combined (Parkin & Owen, 1986):

$$CH_3CH_2COOH + \tfrac{3}{4}H_2O \rightarrow CH_3COOH + \tfrac{3}{4}CH_4 + \tfrac{1}{4}H_2CO_3 \quad (7)$$

Thus the fermentation of 1 mole of propionate produces 1 mole of acetate and 0·25 mole of H_2CO_3.

Butyrate is fermented to acetate and hydrogen:

$$CH_3CH_2CH_2COOH + 2H_2O \rightarrow 2CH_3COOH + 2H_2 \quad (8)$$

and hydrogen gas is oxidised to reduce carbonic acid to methane:

$$2H_2 + \tfrac{1}{2}H_2CO_3 \rightarrow \tfrac{3}{2}H_2O + \tfrac{1}{2}CH_4 \quad (9)$$

The combination of reactions (8) and (9) gives:

$$CH_3CH_2CH_2COOH + \tfrac{1}{2}H_2O + \tfrac{1}{2}H_2CO_3 \rightarrow 2CH_3COOH + \tfrac{1}{2}CH_4 \quad (10)$$

and so the fermentation of 1 mole of butyrate consumes 0·5 mole of H_2CO_3 and produces 2 moles of acetate.

Acetate is fermented to methane and carbonic acid:

$$CH_3COOH + H_2O \rightarrow CH_4 + H_2CO_3 \quad (11)$$

Thus the fermentation of 1 mole of acetate produces 1 mole of H_2CO_3. The CO_3^{2-} anion would not be expected to exist at the typical pH of leachate, and the details of the precise reaction mechanism causing precipitation of $CaCO_3$ have not been fully established. It may in part involve a reaction between Ca^{2+} and HCO_3^- in the leachate at typical leachate pH (6–8) and in part a possible reaction between Ca^{2+} and CO_3^{2-} adjacent to the biofilm, which has a higher pH. However, the net result is that $CaCO_3$(s) is precipitated, and although the actual mechanism may be more complicated than indicated above, for the purposes of this discussion it may be represented by

$$CO_3^{2-} + Ca^{2+} \rightarrow CaCO_3(s) \quad (12)$$

These reactions indicate that the fermentation of propionate replaces propionate with acetate, while fermentation of butyrate replaces butyrate with two acetate, thus increasing the acidity of the leachate. The bioconversion of a weak acid (acetic acid with $pK_a \sim 4{\cdot}7$) to a weaker acid (carbonic acid with $pK_a \sim 6{\cdot}3$) causes the pH of the leachate to increase. The increase in carbonic acid shifts the carbonic equilibrium to carbonate, which can combine with metals to precipitate carbonate-bearing minerals. As the fermentation of acetate generates four times more H_2CO_3 than propionate fermentation, and butyrate fermentation actually consumes H_2CO_3, it follows that the fermentation of acetate drives $CaCO_3$(s) precipitation.

Based on the foregoing it is convenient to relate the amount of Ca^{2+} removed from a leachate (by precipitation) to the amount of H_2CO_3 produced by reactions (7), (10), and (11) in terms of a carbonic acid yield coefficient Y_H (VanGulck *et al.*, 2003):

$$Y_H = \frac{Ca^{2+}\text{ removed}}{\text{net } H_2CO_3 \text{ produced}} \quad (13)$$

It follows that the precipitation of $CaCO_3$ will be controlled by the availability of carbonate (and hence the biodegradation of VFAs and consequent reduction in COD) until Ca^{2+} is depleted in the leachate. As the inventory of

D13/48

Ca^{2+} in the landfill is likely very large, the reduction of Ca^{2+} in the leachate may be due to a combination of geochemical changes in the waste that make calcium less soluble (e.g. an increase in pH) and hence reduce the concentration of Ca^{2+} in the leachate entering the LCS, and actual depletion from the leachate as it passes through the collection system and precipitates out of solution. The former can be dealt with in terms of the input function for Ca^{2+} ; the latter is dealt with in the modelling of clogging within the collection system. This then permits the development of numerical techniques that can be used to simulate clogging processes (Cooke *et al.*, 2005a) by modelling the biodegradation of VFAs, and a simple design method (Rowe & Fleming, 1998) based on the limit imposed by the availability of Ca^{2+} in the leachate, as discussed in subsequent subsections.

Practical implications

The various studies discussed above have demonstrated that development of a biofilm occurs relatively quickly, and then, with time, changes from a soft 'slime' to a slime with hard particles (sand-size solid material in a soft matrix), to a solid porous concretion of a coral-like 'biorock' structure (VanGulck & Rowe, 2004b). During the early phases of clog development the biofilm is relatively easy to clean from leachate collection pipes (e.g. by pressure jetting). However, once the biofilm becomes sufficiently established to cause significant precipitation of $CaCO_3$, the inorganic film becomes firmly attached to the adjacent media (e.g. perforation) and to the inside of pipes (Fleming *et al.*, 1999), and becomes very difficult to remove. This implies that leachate collection pipes in landfills should be regularly cleaned to remove the soft biofilm in its early stages of development. It has been found that the rate of clog development accelerates with time, and hence the rate of inspection/cleaning required in pipes may increase with time as biofilm develops and as the concentration of VFAs in the leachate (or COD) increases. Silica (sand and silt particles) can be controlled by the use of an appropriate filter. Although calcium and VFAs are ubiquitous in MSW, management practices such as the sequencing of the placement of waste will influence the concentrations of calcium and VFAs reaching the LCS, as discussed by Armstrong & Rowe (1999). For example, placing new waste over old waste where the biological processes are well established tends to increase the pH of the leachate as it migrates through the old waste, and hence tends to reduce the solubility of Ca^{2+} and thus its concentration in the leachate when it first enters the leachate collection system. Also, practices such as the recirculation of leachate serve to re-inject VFAs and calcium back into the waste, and this will have implications for the performance of the LCS.

Because of the level of 'treatment' that occurs in an LCS before the leachate reaches the collection point, end-of-pipe analyses of the leachate VFA and Ca^{2+} concentrations indicate only what it is like at the collection point, and provide little evidence regarding the nature of the leachate that enters the system.

The following observations led Rowe & Fleming (1998) to propose a simple method for estimating the rate of clogging of different collection system designs:

(*a*) The rate of clogging was related to mass loading.
(*b*) The dry densities of clog material, ρ_c, varied between 1·6 and 2·0 Mg/m^3.
(*c*) Clogging was a problem only for leachates with a significant concentration of Ca^{2+}, Mg^{2+} or Fe^{2+}, with $CaCO_3$ typically representing more than 50% of the clog material.
(*d*) The calcium fraction of total clog material (f_{Ca}) was typically 20–30% of the clog mass.
(*e*) Clogging was greatest near the leachate collection pipe, and decreased with distance from the pipe.

This approach uses calcium as a surrogate for other clog materials that may deposit within the drainage layer (i.e. magnesium, iron, silica). The methodology ignores the reaction kinetics of clogging and conservatively assumes that all calcium entering the drainage layer becomes deposited in the granular layer instantaneously. Thus the maximum possible volumetric yield, $Y(t)$, of clog per unit volume of leachate may be estimated to be

$$Y(t) = \frac{c_L(t)}{\rho_c f_{Ca}} \tag{14}$$

where $c_L(t)$ is the anticipated concentration of calcium in leachate, ρ_c is the dry density of the clog material, and f_{Ca} is the proportion of calcium in the total clog material. It then follows that the volume of pore space filled with clog material over a length of collection system x per unit width at some time t is given by

$$V(t) = \int_0^t \frac{c_L(t) q_0 x}{\rho_c f_{Ca}} \, d\tau \tag{15}$$

where q_0 is the rate of leachate percolation into the porous medium per unit area.

The time required to produce a particular level of clogging may be estimated from this relationship for any $c_L(t)$. For any particular degree of clogging a 'porosity reduction' v_f is defined such that:

$$n_b = n_0 - v_f \tag{16}$$

where n_b is the free porosity remaining at time t, n_0 is the initial porosity, and v_f is the porosity reduction (i.e. the porosity that is occupied by clog material at time t).

The foregoing relationships can be used to deduce clogging for a wide range of practical situations. However, for a blanket drain, the field observations at the KVL suggest that clogging will be greatest towards the collection pipes and least away from the pipes (Fleming *et al.*, 1999). Therefore it is assumed (Fig. 5) that the porosity reduction (v_f) and the portion of the overall thickness of drainage blanket subject to clogging (B') both vary linearly from essentially zero at the up-gradient edge of the drainage path (where lateral flow is zero) to a maximum value $v_f = v_f^*$ and $B' = B$ (where B is the thickness of the drainage layer) within a few metres of the collection pipe. Within a distance a of the pipe, the porosity reduction is assumed to remain constant at a peak value v_f^*, which is equated with a particular degree of performance impairment.

Therefore the volume of mineral clog corresponding to a permeability decrease represented by v_f is given by

$$V_{tot} = \int_0^L v_f(x) B'(x) dx \tag{17}$$

where

$$v_f = \begin{cases} v_f^* x/(L-a) & \text{for } x < L-a \\ v_f^* & \text{for } L-a < x < L \end{cases}$$

$$B'(x) = \begin{cases} Bx/(L-a) & \text{for } x < L-a \\ B & \text{for } L-a < x \end{cases}$$

and hence

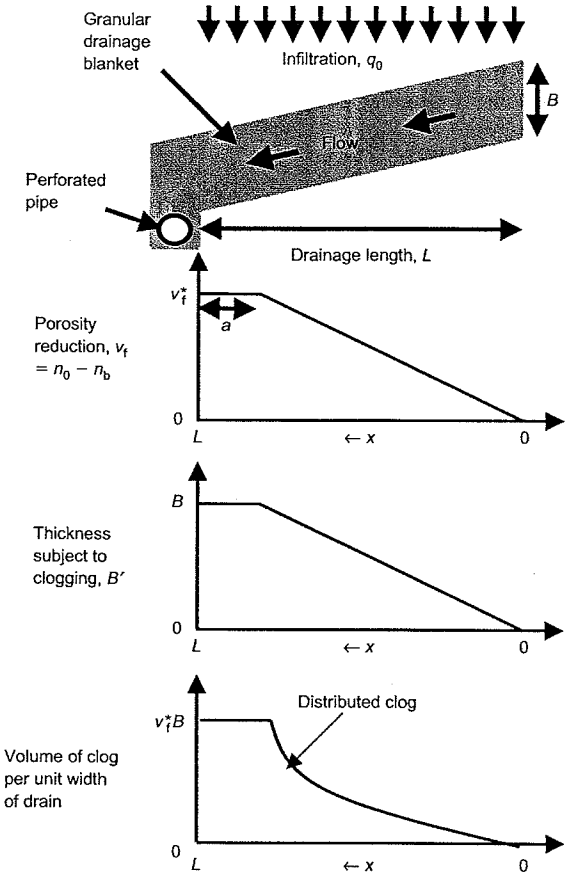


Fig. 5. Schematic of drainage layer configuration and clog development within the underdrain (modified from Rowe & Fleming, 1998)

$$V_{tot} = \frac{v_f^* BL(1 + 2a/L)}{3} \tag{18}$$

The time to reach the degree of clogging represented by V_{tot} is calculated by solving for the rate of mineral volume growth expressed above for constant or time-varying Ca concentration. For a constant Ca concentration c_L, the time to clog, t_c, is given (from equations (14) and (17)) by

$$t_c = \frac{(v_f^* BL/3 + 2v_f^* Ba/3)(\rho_c f_{Ca})}{(c_L q_0 L)} \tag{19}$$

For variable Ca concentration c, where $c = c_{L1}$ for $t < t_1$ and then linearly decreases with time to $c = c_{L2}$ at $t = t_2$, after which it remains constant at c_{L2}, then the total clog volume for $t > t_2$ is given by

$$V(t) = 0{\cdot}5\zeta(c_{L1} - c_{L2})(t_1 + t_2) + \zeta c_{L2} t \tag{20}$$

where $\zeta = q_0 L/(\rho_c f_{Ca})$.

Combining equations (18) and (20) for a time-varying Ca concentration, the time to clog t_c is given (for $t_c > t_2$) by

$$t_c = \frac{(1 + 2a/L)B\rho_c f_{Ca} v_f^*}{3q_0 c_{L2}} - \frac{(c_{L1} - c_{L2})(t_1 + t_2)}{2c_{L2}} \tag{21}$$

It follows from the foregoing that the time to clog decreases with increasing spacing between drains (L), increasing calcium concentrations (c_{L1} and c_{L2}) and increasing infiltration into the drainage blanket (q_0). This technique has been applied to predicting the clogging of old French drain systems, and gives relatively short service life predictions (a decade), consistent with observed field behaviour. When applied to well-designed modern systems with adequately spaced collection pipes and a suitable thickness of uniformly graded coarse gravel (e.g. $L = 25$ m, $B = 0{\cdot}5$ m), it gives predictions of service lives in excess of 100 years, assuming that the pipes are regularly cleaned (Rowe & Fleming, 1998).

The methodology described above has the advantage of simplicity; however, as with any simplified model it also has limitations. It assumes that the entire concentration $c_L(t)$ is used to cause clogging, and that the kinetic processes may be neglected. These assumptions are likely conservative, and would lead to an underestimate of the time required for clogging to occur. On the other hand, while the methodology is dependent on porosity, it does not distinguish the effects of different pore size for similar porosity. For example, one could have 5 mm and 50 mm particles with the same initial porosity, and the time predicted to get the same relative reduction in porosity would be the same. However, as indicated by the laboratory studies discussed earlier, particle size does have an effect because of the difference in the absolute size of the voids and the difference in specific surface (surface area per unit volume) that one has for larger-diameter particles. Thus the calculation method proposed above should be used with caution, and only for relatively uniformly graded granular material with a nominal size greater than 20 mm. A more sophisticated model that can address these issues is described in the next subsection.

Both the column and mesocosm studies discussed above have shown that clogging was greater in saturated systems than in unsaturated systems. Thus, to minimise clogging, LCS should be kept pumped and not allowed to remain in a saturated state for prolonged periods of time.

Studies comparing gravel with tyre shreds (Rowe & McIsaac, 2005) have indicated that an increased thickness of compressed tyre shred may be used to give a service life similar to that of a given thickness of gravel in non-critical zones. However, because of the greater variability in pore size, gravel should continue to be used in critical zones where there is a high leachate mass loading.

Giroud (1996) has discussed the issue of geotextile clogging as part of a broad review of filter design. He tentatively recommends that sand and nonwoven geotextile filters should not be used even if the waste has been stabilised to produce low-strength leachate by pre-treatment. Rather, he recommends the use of monofilament woven geotextiles with a minimum apparent opening size (AOS) of 0·5 mm and a minimum percentage open area (POA) of 15%, with a preference for a POA greater than 30%. The rationale for these recommendations arises from the following observations.

(*a*) The specific surface area for monofilament woven geotextile is much smaller than for nonwoven geotextiles, and this decreases the surface area for biofilm growth.
(*b*) The woven filter allows more effective and rapid movement of fine material (i.e. material not intended to be retained) and leachate through the filter.
(*c*) Because of their compressibility, the filtration characteristics of a nonwoven geotextile vary with applied pressure, and the critical filtration characteristics should be assessed under design pressures, which could be up to 500 kPa.

There is some evidence to suggest that geotextiles selected in accordance with Giroud's (1996) recommendations are likely to experience less clogging and reduction in hydraulic conductivity with time than needle-punched nonwoven or slit

D15/48

film geotextiles normally used. Giroud's (1996) recommendations are based on the premise that one wishes to minimise clogging of the filter. This may indeed be the case for some design situations (e.g. if one insists on wrapping geotextile around pipe in LCS). However, although excessive clogging is undesirable, the processes that cause clogging also provide leachate treatment and, in so doing, (a) decrease the potential for clogging at more critical zones (e.g. near collection pipes) and (b) reduce the level of leachate treatment required after removal of leachate from the landfill. Based on the available evidence, as summarised earlier, it appears desirable to design the LCS to maximise leachate treatment while maintaining its design function. Under these circumstances, using a design such as the one shown in Fig. 6 with a sand and/or nonwoven geotextile filter between the waste and gravel may be desirable provided that perching of leachate on the filter does not have any negative effects (such as side seeps).

Based on published data (Koerner *et al.*, 1994; Rowe, 1998a), it is unlikely that the hydraulic conductivity of a typical nonwoven needle-punched geotextile would be below 4×10^{-8} m/s (1·3 m/yr) for normal conditions and more likely that it would be of the order of 1×10^{-7} m/s or higher. If the geotextile was used in a blanket drain (e.g. Fig. 6), one can quickly establish that there would be negligible perched leachate on the geotextile for typical rates of leachate generation (less than 3×10^{-8} m/s or 1 $m^3/(yr\,m^2)$). Thus, while recognising that geotextiles will clog, theoretical consideration, mesocosm studies and field observations all indicate that an appropriately selected geotextile used to protect gravel in a blanket drain will improve the performance and the service life of the drainage gravel and not cause excessive perched leachate mounding. There are two disadvantages to the use of the geotextile directly between the drainage gravel and the waste (Fig. 6(a)). First, unless considerable care is taken, the placement of the waste could cause tearing of the geotextile (creating holes). Second, if the geotextile is left exposed for any significant period of time before covering by waste, there will be degradation of the geotextile by exposure to ultraviolet rays, which will increase the tendency for it to tear. These problems can be avoided by placing a medium-to-coarse sand protection layer over the geotextile (Fig. 6(b)).

It has also been suggested (e.g. Brune *et al.*, 1991) that carbonate drainage gravel is unsuitable for use in the drainage layer because it could dissolve and contribute to subsequent calcite crystallisation on alkaline biofilms covering drainage gravel. However, there is now a considerable body of evidence (e.g. Rittmann *et al.*, 1996; Owen & Manning, 1997; Jefferis & Bath, 1999; Manning & Robinson, 1999; Bennett *et al.*, 2000) to suggest that calcium carbonate is supersaturated in landfill leachate very early in the life of the landfill, and that it is unlikely that dissolution would occur under these conditions. The evidence points to the Ca^{2+} being leached out of the waste and transported to the point of deposition by the leachate while the CO_3^{2-} arises predominantly from the mineralisation of organic carbon by methanogens (Brune *et al.*, 1991; Owen & Manning, 1997; Bennett *et al.*, 2000).

To address the concern that acetogenic leachate may cause dissolution of dolomitic limestone, Bennett *et al.* (2000) examined drainage gravel exhumed from the KVL (Case 2, Table 2) and looked for incipient dolomite dissolution, which is commonly characterised by the formation of small pits detected at high magnification on crystal surfaces. They were able to show that the surface of the dolomitic limestone exhumed from the KVL LCS was devoid of such pits. However, they did find significant fines (dolomite, quartz, feldspar), washed into the LCS as particulate material suspended in the leachate, which became part of the secondary calcite formed around the gravel. This, combined with the difference in observed clogging at the KVL where a geotextile filter was and was not used (Cases 1 and 2, Table 2), suggests that the clogging of the drainage gravel could be reduced by placing a geotextile filter between the waste and drainage gravel to limit the ingress of fines.

Numerical clogging model

The understanding of clogging derived from the studies summarised above has allowed the development of a numerical model, BioClog, to simulate clogging in both column experiments (Cooke *et al.*, 2005a) and 2D flow systems. A brief summary is presented here. Reactive chemical transport is modelled with consideration of biological growth, mineral precipitation, attachment and detachment of suspended

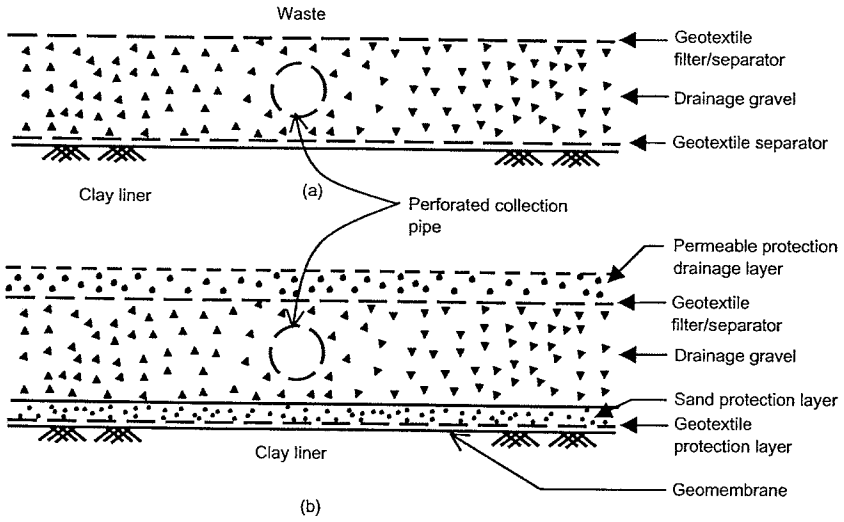


Fig. 6. Typical leachate collection drainage blankets: (a) geotextile alone; (b) geotextile and sand protection layers (note that upper permeable protection drainage layer would be medium to coarse sand or gravel (the latter preferred); the lower sand protection layer would be fine or medium sand)

solids. The porous medium is represented as a fixed-film reactor, and changes in porosity resulting from the development of biofilm and inorganic films can be calculated directly based on a geometric model. The corresponding changes in hydraulic conductivity are then deduced based on empirical relationships derived from laboratory tests. For 2D systems, a flow model is coupled with the transport model.

The transport and fate of three volatile fatty acids (VFAs), dissolved calcium, suspended inorganic solids and suspended biomass are modelled using the finite element method. Fig. 7 gives a schematic representation of species pathways and reactions modelled. Each species has a source/sink term representing the rate of loss or gain resulting from reaction of the species. The term may differ at each node, and is recalculated for each time step. The equations for the reaction terms for each species are given in Cooke *et al.* (2005a). The mechanisms modelled include the following.

(*a*) Acetogenesis of propionate and butyrate, and methanogenesis of acetate by the substrate degraders (both suspended microbes and in the biofilm on the porous media), together with the generation of acetate as a by-product of acetogenesis (equations (5) and (8)). Biofilm substrate utilisation accounts for diffusion of substrate into the biofilm. Once in the biofilm, the substrate consumption is modelled based on Monod kinetics.

(*b*) Detachment and attachment of suspended active biomass from/to the porous media and decay. The rate of detachment of biomass from the biofilm is computed using contributions from two different mechanisms: (i) detachment due to shear stress; and (ii) detachment due to growth rate. Monod kinetics is used to model growth and decay based on the cell decay rate for the species. Attachment is based on a first-order rate coefficient for attachment, which can be calculated using either a particle filtration method or a network method.

(*c*) The loss of suspended inert biomass (i.e. biomass such as dead bacteria, which do not consume substrate) from solution because of attachment to the porous medium, and the gain from detachment of inert biofilm and conversion of suspended active biomass to inert biomass through decay are modelled. The detachment and attachment rates are calculated using the same methodology as for the active suspended biomass. The production of suspended inert biomass is calculated based on the concentrations of suspended active biomass, the decay rate for the species, and the fraction that is refractory.

(*d*) The precipitation of calcium carbonate is based on the carbonic acid yield coefficient (equation (13)), and the net rate of production of carbonic acid calculated from the utilisation of the VFAs (equations (7), (10) and (11)).

(*e*) The increase in suspended inorganic solids due to detachment of inorganic solids is computed using the shear stress model; the attachment is based on the attachment rate, computed in a similar manner to that for the suspended active biomass.

The clog matter is modelled as comprising five separate films: a biofilm for each of the active degraders (acetate, propionate and butyrate degraders), an inert biofilm, and an inorganic solids film. Cooke *et al.* (2005a) provide equations for calculating film thickness based on the same processes as modelled for the reaction terms (Fig. 7):

(*f*) Active biofilms can grow by attachment of suspended active degraders and growth, and may shrink as a result of detachment and decay. Attachment and detachment are modelled as discussed in (*b*) above. Growth and decay are calculated according to the non-steady-state biofilm growth model.

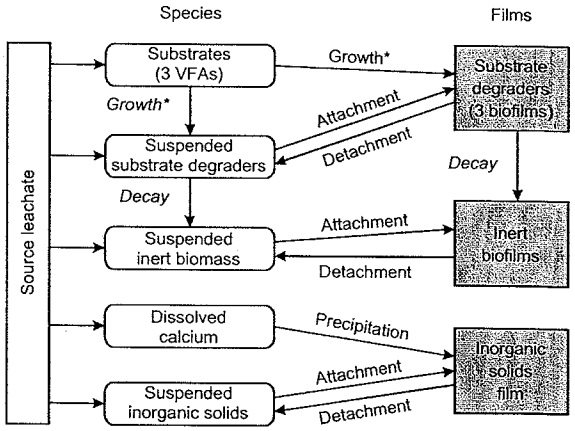


Fig. 7. Schematic of the species pathways and primary reactions incorporated in the BioClog model (after Cooke *et al.*, 2005a). *'Growth' represents growth of substrate degraders due to utilisation of substrate. Note: figure does not indicate production of acetate from degradation of other VFAs

(*g*) Inert biofilm thickness increases because of attachment of inert biomass and conversion of decayed active biomass, and decreases because of detachment.

(*h*) Inorganic solids film thickness increases because of attachment of suspended inorganic solids, precipitation of calcium carbonate, and production of 'other' inorganic solids, and decreases because of detachment. Attachment and detachment are as discussed previously. The mass of precipitate is computed from the mass of calcium removed from the leachate (see (*d*) above). The production of 'other' inorganic solids is calculated by applying a user-defined multiplier to the rate of calcium carbonate production, thereby allowing the model to represent additional precipitates, assuming they accumulate at a rate proportional to calcium carbonate. The contribution of 'other' precipitates is often small (Brune *et al.*, 1991; Fleming *et al.*, 1999; Manning & Robinson, 1999).

The active biofilm, inert biofilm and inorganic solids thicknesses are computed for each element in the finite element mesh. A geometric representation of the porous medium allows calculation of porosity and specific surface in each element based on the total thickness of the active and inactive films, as described by Cooke & Rowe (1999).

Two-dimensional modelling involves a 2D transport model that simulates the processes discussed above together with a 2D flow model. The flow model takes the porosity and corresponding hydraulic conductivity at each point in space, at a given time, from the transport model and calculates the leachate mound subject to these physical parameters and the infiltration from the waste into the collection system. This mound then provides input to the transport model for the next time step.

The model has been calibrated to well-controlled laboratory experiments permeated with synthetic (VanGulck, 2003) and KVL (VanGulck, 2003; Cooke *et al.*, 2005b) leachate. The results indicate that the processes simulated in the model are largely responsible for the biologically induced clogging of landfill LCSs. The application of the model can be briefly illustrated with respect to a laboratory column test involving 6 mm glass beads (VanGulck, 2003). The column was seeded with KVL leachate (to provide a source of

D17/48

relevant bacteria for initial attachment to the beads), but for the bulk of the test the leachate was synthetic leachate with similar chemical characteristics to those of the KVL leachate but without volatile (e.g. microbes) and inorganic (i.e. particulate material) suspended solids. The leachate chemistry was monitored as it moved along the column from the influent to effluent end. Fig. 8 shows the observed variation in the concentration of calcium at the influent and effluent ends of the column. Also shown is the calcium concentration calculated using the BioClog model. Fig. 9 shows the calculated and observed changes in porosity along the column at 247, 295 and 427 days.

The data show three distinct stages that are captured by the model. For the first approximately 250 days (the lag or acclimation phase) the effluent concentration mirrors the influent, and there is no noticeable change in the calcium concentration (Fig. 8). However, Fig. 9 shows that there has been some decrease in porosity, which can be attributed to the development of a biofilm on the glass beads, which is occupying space but which, until the end of the lag phase, has not yet developed sufficiently to have a measurable effect on the calcium concentration. This situation changes in the transition phase, and the biological processes now result in an increasing production of carbonate (due predominantly to increased methanogenesis of acetate) and associated precipitation of calcium carbonate to the point where, in the steady phase, most of the calcium entering the column is precipitated and very little remains in the effluent. As a consequence there is increased clogging, and at test termination (427 days) the porosity at the influent end of the column is less than half the initial porosity, and the hydraulic conductivity has dropped to 10^{-6} m/s (i.e. by five orders of magnitude). Fig. 9(a) shows the glass beads in the first 15 cm of the column from the influent port at test termination; the clogging is visually evident.

It should also be noted that the general trend evident in Fig. 8 (a lag, a transition, and a steady phase) is also evident in the field data from the KVL (Fig. 2) when allowance is

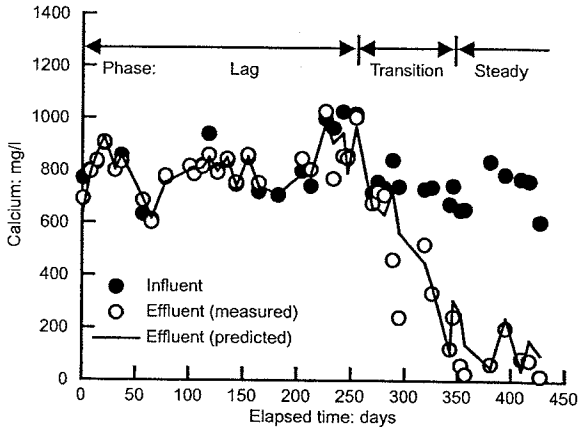


Fig. 8. Observed and calculated variation in effluent calcium concentration with time for a column test using 6 mm glass beads (data from VanGulck, 2003)

made for the operational effects on the leachate concentrations. Given the variability associated with biological processes and landfills in particular, the agreement between the BioClog calculations and the observed behaviour (Figs 8 and 9) is very encouraging (see references previously cited for more detailed comparisons).

Rowe & VanGulck (2004) modelled a 1D flow column filled with 6 mm glass beads and permeated with KVL leachate. The primary difference between synthetic leachate and actual KVL leachate is the continuous supply of volatile and inorganic suspended solids in the influent leachate, which substantially accelerates the clogging process (all other things being equal). The next few paragraphs illustrate the complexities of the interactions being simulated by BioClog.

The change in porosity at 50-day intervals is shown in

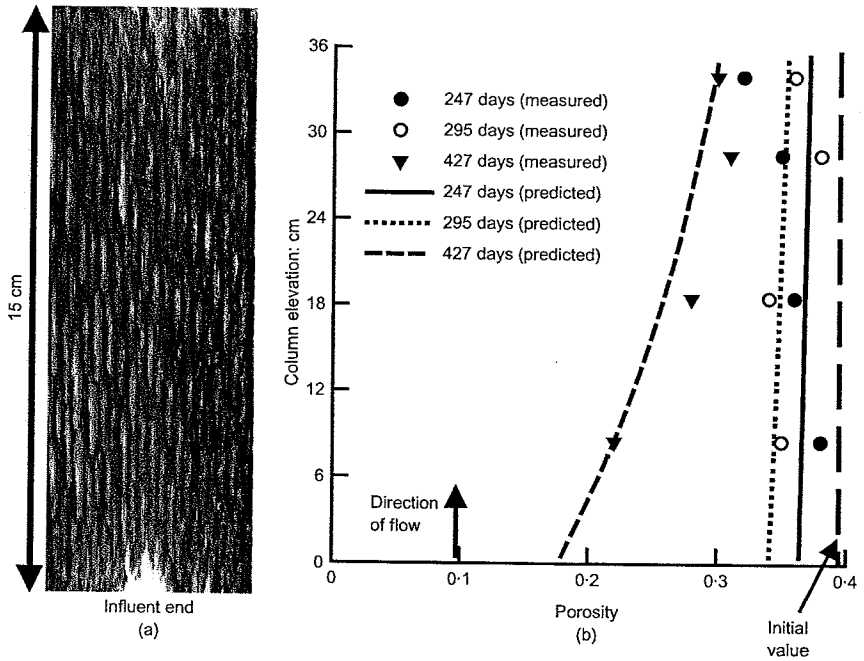


Fig. 9. Results from a column test using 6 mm glass beads: (a) glass beads at the end of a column test, showing greatest clogging at the bottom (influent end) and reduced clogging moving up the column; (b) observed and calculated variation in porosity with position at three times (data from VanGulck, 2003)

D18/48

Fig. 10(a). This change is a consequence of an accumulation of inorganic (mostly $CaCO_3$, Fig. 10(b)) and volatile (biofilm, Fig. 10(c)) films on the glass beads. During the first 50 days, the reduction in porosity was small, with the volatile film being the primary contributor to the clogging that did occur. The volatile film was similar to, or thicker than, the inorganic film for the first 150–200 days. Subsequently, the inorganic film thickness exceeded the volatile film and the column became highly clogged (especially near the influent end). At 200–250 days there was a reduction in volatile film thickness due to biofilm detachment caused by the high seepage velocities (and hence fluid shear stresses) associated with low porosity and a fixed flow rate. In contrast, the inorganic film does not undergo detachment in this case. The column may be regarded as being completely clogged at locations where the porosity drops to about 5%.

The volatile film consists of active VFA-degrading bacteria (e.g. acetate, butyrate and propionate degraders) and non-active or non-substrate-consuming biomass (e.g. dead bacteria). Each degrader has a unique combination of Monod kinetic parameters, and its growth (and decay) is a function of these parameters, of the substrate concentration, and of the attachment and detachment of each degrader to and from the bead surface. As a consequence, the methanogenesis of acetate and fermentation of propionate and butyrate will occur at different rates. In addition, the concentration of acetate results from a combination of a decrease due to methanogenesis of acetate and an increase due to fermentation of propionate and butyrate, and hence is a function of the kinetics of all three primary degraders. This gives rise to results such as shown in Figs 10(d) and 10(e), where there was significant butyrate consumption but very little change in acetate concentration in the leachate within the first 50 days. There was also relatively little change in calcium concentration (Fig. 10(f)), as its precipitation is driven primarily by methanogenesis of acetate, as previously discussed. By 100 days the biofilm had developed sufficiently to cause significant acetate and butyrate removal, with a consequent large removal of calcium. Peak consumption occurred at 150–200 days. At later times, the low porosities resulted in shearing of the biofilm (Fig. 10(c)) and subsequently less consumption of VFAs and precipitation of calcium. Furthermore, the lower porosities imply higher seepage velocities and hence shorter retention times (during which biodegradation can occur) in the column. Propionate removal did not occur within the column owing to the slow net growth rate of propionate degraders compared with acetate and butyrate degraders and the limited (250 day) time frame examined.

The concentration of volatile suspended solids (VSS) in the leachate decreased along the length of the column during the first 150 days (Fig. 10(g)). Because of the high fluid shear stresses arising from clogging, the biofilm detachment in the first 10 cm resulted in the VSS concentration at an elevation of 10 cm being larger than in the influent at 200 days. The detached biomass subsequently reattached to beads further up the column. Detachment becomes even more significant with increased clogging, and hence by 250 days the effluent VSS concentration was slightly larger than the influent concentration.

The attachment of inorganic suspended solids (inert biomass, mineral precipitate, and soil particles, FSS) is a function of the pore throat size and seepage velocity. Clogging results in smaller pore throats and greater potential for attachment (straining); however, smaller pore throats also give rise to higher seepage velocities and lower potential for attachment. The net effect is being observed with time in the column for these particles, with the former dominating at earlier times but the latter dominating once the clogging becomes severe.

After a decade of development, the BioClog model is now reaching the point where it can go beyond column studies and be used to predict clogging in landfill drainage systems. Nevertheless, because of the highly coupled and complex nature of the model, which draws together microbiology, geochemistry, waste water engineering and geotechnical engineering, it is unlikely to be used in routine engineering practice. However, models such as this do provide insight that can both advance understanding of what is observed in laboratory tests and field monitoring, and aid in evaluating the likely difference in the performance of different types of LCS.

TEMPERATURE AT THE BASE OF A LANDFILL

Increases in landfill temperature arise from heat generated by biodegradation of waste or the heat of hydration of incinerated residues (ash). High temperatures (50–70°C) have been reported in many landfills (e.g. Collins, 1993; Yoshida *et al.*, 1996). For example, at the Tokyo Port Landfill in Japan, 35 m of MSW was placed directly on the surface of a natural clayey liner. This landfill has no effective LCS on the base, and a significant (20–25 m above the base of the landfill) leachate mound has developed. The temperatures at the base were up to 50°C 7–10 years after the beginning of landfilling and had reduced to 37–41°C after 20 years (Yoshida & Rowe, 2003). High temperature is not restricted to MSW landfills. At the Ingolstadt landfill in Germany (Klein *et al.*, 2001), MSW incinerator bottom ash was placed to a thickness of about 9 m over a composite (GM over clay) liner system. The temperature at the liner peaked at 46°C 17 months after the start of landfilling, and is subsequently decreasing at 0·6 deg C per month.

Typically the temperature has a maximum value in the main body of the waste and decreases towards the boundaries defined by the surface and the underlying liner (Collins, 1993). The rate of increase in temperature with time, both in the waste and at the liner, may vary depending on the waste management practice that is adopted. For example, the rate of waste placement can affect the rate at which the temperature increases at the liner, as illustrated by Brune *et al.* (1991), who reported temperatures ranging from 24°C to 38°C in a leachate drain beneath 4–6 year old waste at the Altwarmbüchen Landfill in Germany (waste placement rate 10–20 m/yr) but only 14–20°C after a similar period at the Venneberg Landfill (waste placement rate 2 m/yr). Similarly, the rates of percolation of moisture through the landfill cover can also affect temperature, as illustrated by Koerner & Koerner (2006), who monitored the temperature on the GM liner at two landfill cells for the same landfill (50 m of waste) north of Philadelphia, USA (mean annual temperature 12·6°C). Both cells had a similar low-permeability geosynthetic cover, but in one case ('dry cell') there was no additional moisture added whereas in the other case ('wet cell') moisture augmentation began after about 200 days at a rate of approximately 500 m^3 per month. The variation in temperature with time is shown in Fig. 11. The reported temperatures within the waste itself after 8 years and 2 years in the dry and wet cells were 32–43°C and 40–67°C (mean 55°C) respectively (Koerner *et al.*, 2003). For the dry cell, the average liner temperature was consistently about 20°C for 5·5 years. However, after 5·5 years the temperature quickly increased and after 10 years is averaging about 32°C. It may be hypothesised that the 5–6 year delay before the temperature increased reflected the time required for significant heat generation and conduction of that temperature downward to the liner; another similar example will be

D19/48

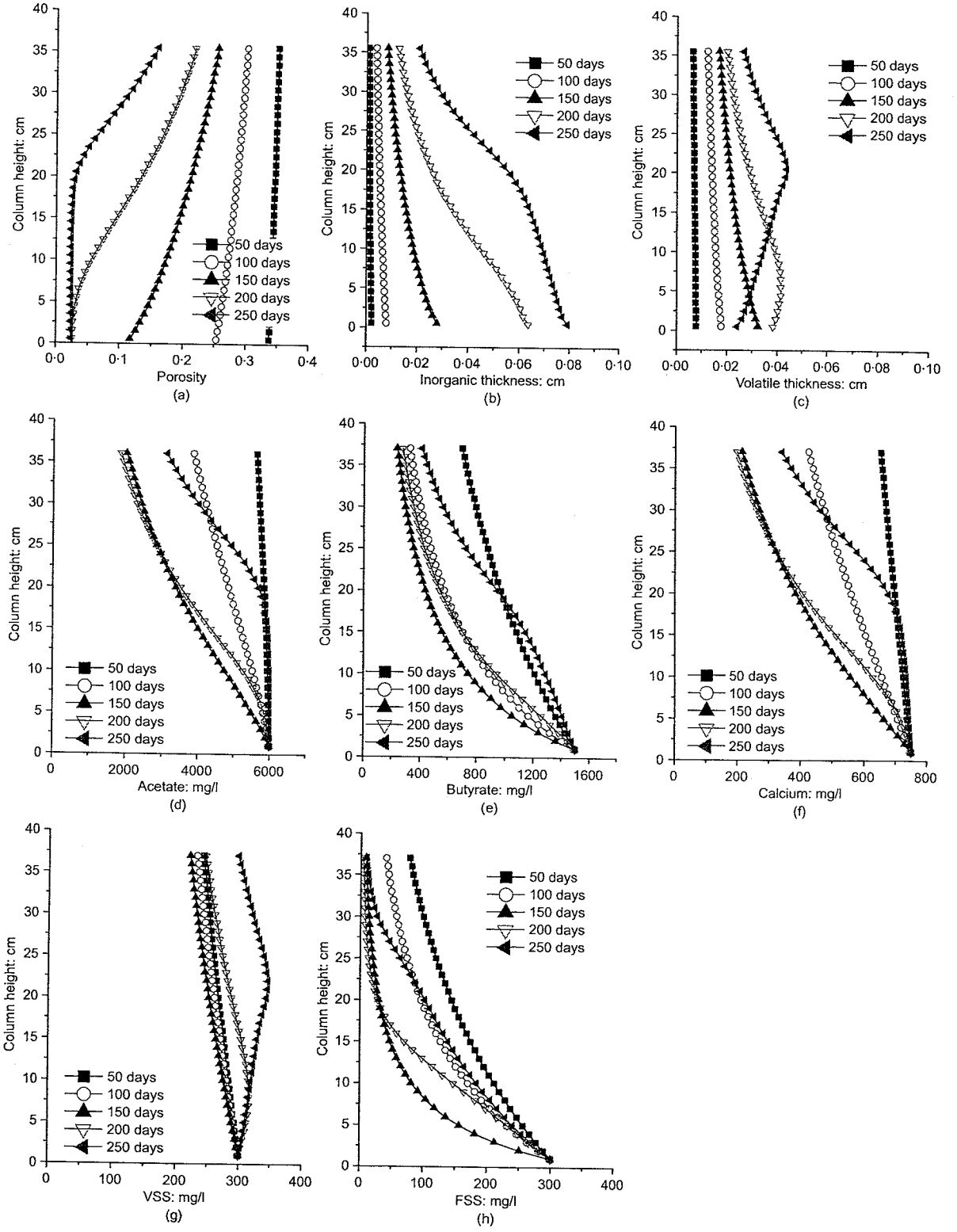


Fig. 10. Various aspects of clogging along a column of glass beads at 50-day intervals based on BioClog model results (after Rowe & VanGulck, 2004): (a) porosity; (b) inorganic film thickness; (c) volatile film thickness; (d) acetate concentration; (e) butyrate concentration; (f) calcium concentration; (g) VSS concentration; (h) FSS concentration

cited below. For the wet cell, the liner temperature was between 25°C and 28°C (i.e. 5–8°C higher than the dry cell) for the first year and then increased over the following 2 years to between 41°C and 46°C.

The KVL provides an example where liner temperature has been monitored above a CCL over a 21-year period. Barone *et al.* (2000) reported data at four locations in the landfill up to 1998. Fig. 12 provides updated data (up to December 2004) at those same four locations. As described in the previous section, Stages 1 and 2 have French drains

D20/48

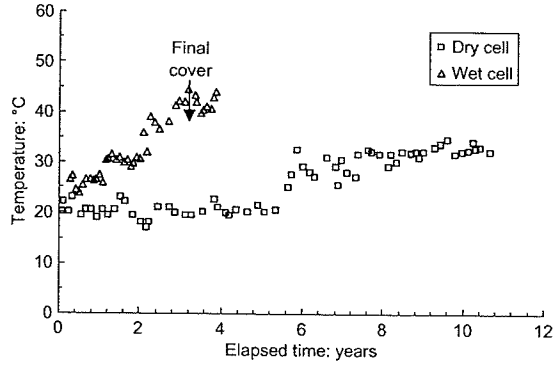


Fig. 11. Average geomembrane liner temperature for dry and wet cells (data replotted from Koerner & Koerner, 2006)

as the LCS. In Stage 1 the temperature increased slowly and remained low (average 12°C) for the first 6 years but then increased rapidly as the leachate mound increased from 1 m to 6·5 m. Subsequently, the temperature stabilised at about 37°C, although the leachate mound continued to increase to about 8·4 m (it has subsequently reduced slightly, probably as a result of the installation of final cover). In Stage 2, the temperature remained relatively constant and low (average 10°C) for the first 5 years but then increased rapidly to 24°C over the next 6 years, as the leachate mound increased from about 1 m to 5 m, and has continued to increase to 35°C (with no sign of it stabilising yet), even though the leachate mound has actually dropped to about 3 m (primarily because of the installation of final cover). In Stages 3 and 4 the blanket drain LCS continues to function well, but since 1998 the temperature has risen from 15°C to 31–32°C (in 2004) at the two monitors shown in Fig. 12, and appears still to be increasing. Fig. 13 shows the change in liner temperature with time at all monitors in the KVL where the leachate head is less than 0·3 m. In these cases it took 8–12 years for the temperature on the liner to reach 20°C but only another 3–4 years to reach 30°C. To the extent that temperatures appear to have levelled off, it is at about 40°C.

The data from the Philadelphia landfill suggest that the injection of fluid significantly accelerated the rate of increase in temperature at the liner. Likewise the build-up of a leachate mound at the KVL increased the rate at which the liner temperature increases. The current evidence suggests that the final maximum liner temperature is higher for landfills where there is a high waste moisture content due either to the addition of fluid (e.g. leachate recirculation) or to mounding of leachate, but it will be necessary to monitor landfill liners with no significant leachate mound for a longer period of time to confirm this, as it may be that, because of the slower rate of increase in temperature with time, these liners have not been monitored for long enough to know the maximum temperature that will be reached. With modest infiltration of leachate and a well-functioning LCS, the liner temperature exceeded 20°C after 5·5–11 years and exceeded 30°C after 7·5–14 years (Figs 11 and 12). These data suggest that estimates of service lives of liners should not be based on a temperature of 20°C but rather on a temperature at least of the order of 30–40°C. The length of time for which the temperature remains this high is unknown at present, but, based on available data, is likely to be at least a few decades for large landfills such as those where data is currently available (i.e. with 30 m of waste above the liner).

Temperature influences both hydraulic conductivity and diffusion coefficient (Collins, 1993). A groundwater temperature of around 10°C is typical of parts of the world where the local mean temperature is 8–10°C. Diffusive and

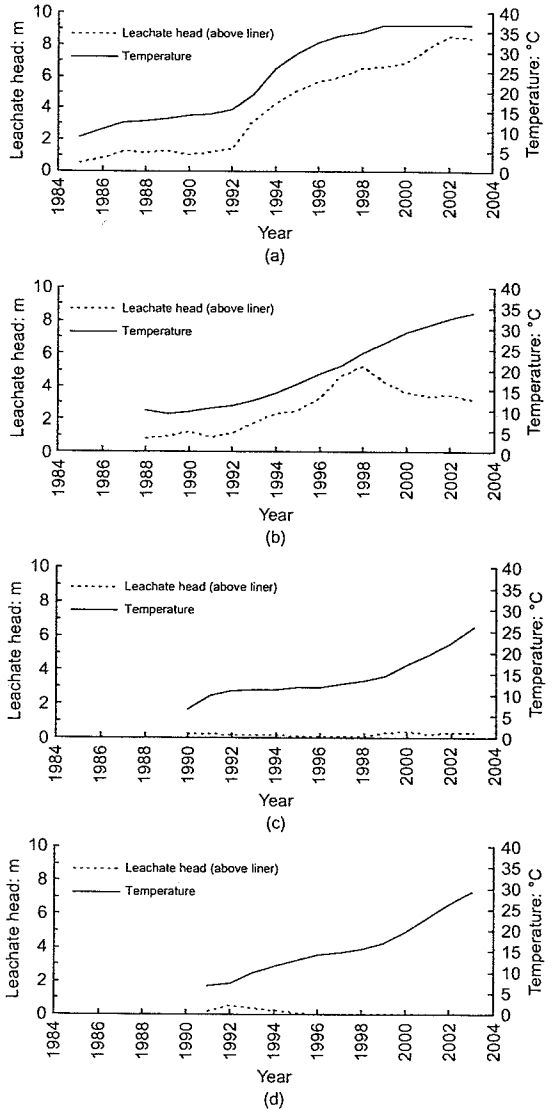


Fig. 12. Variation in leachate head and temperature at four locations in the Keele Valley Landfill (to December 2004): (a) Stage 1, Lysimeter 3; (b) Stage 2, Lysimeter 7; (c) Stage 3, Lysimeter 14B; (d) Stage 4, Lysimeter 16A

advective transport are, respectively, 40% and 30% higher at 20°C than at 10°C, and 100% and 80% higher at 35°C (Rowe *et al.*, 2004). Constant diffusion and hydraulic conductivity parameters may be used if the liner temperature remains relatively constant. However, if the temperature of the liner changes with time then the effect of the change in these parameters with temperature on contaminant impact should be considered. Rowe & Booker (1995, 2005) proposed an approach that readily models the effect of changes in diffusion coefficient and hydraulic conductivity with time. As will be discussed later, temperature also has a significant impact on service lives of geomembranes and clay liners, and on the transmissivity of geonet drainage layers.

LEAKAGE THROUGH LINERS

Change in hydraulic conductivity with time and leakage at the KVL

Leakage through liners is usually interpreted in terms of the actual advective flow through the liner. In the case of

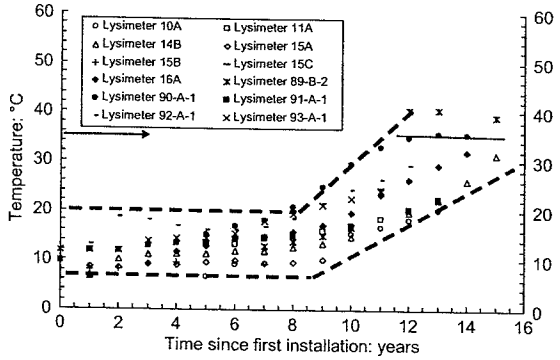


Fig. 13. Variation in liner temperature with time for monitors in parts of the KVL where the leachate head is less than 0·3 m for period to December 2004 (data courtesy of City of Toronto). Arrow and dotted horizontal line indicates 35°C

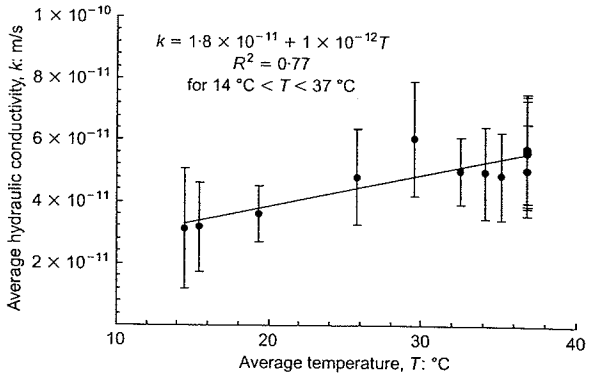


Fig. 15. Variation in annual average hydraulic conductivity with annual average liner temperature for compacted clay liner at Keele Valley Landfill, Lysimeter 3 (data courtesy of City of Toronto)

clay liners this will be given by Darcy's law (equation (1)) and will depend on the hydraulic conductivity k of the clay liner and the hydraulic gradient (i.e. the leachate mound above the liner and the head at the base of the liner). Both the gradient and hydraulic conductivity may vary with time. The latter is discussed below.

Both the inferred and actual hydraulic conductivity of a CCL may vary owing to liner consolidation as the waste thickness increases. For example, at the KVL (Lysimeter 3) the waste thickness increased from 1 m in November 1984 to 33 m in April 1987 and then remained relatively constant. As can be seen from Fig. 14, the annual average k dropped from 4×10^{-10} m/s in 1984 to $3{\cdot}1 \times 10^{-11}$ m/s in 1991. These values are deduced from the measured flow to the lysimeter located in the liner, and the heads above and below the liner, using Darcy's law. Two consolidation-related factors can affect the flow to the lysimeter during this period. First, expulsion of water from voids, due to consolidation, will lead to flow to the lysimeter, which is not explicitly considered in calculating k, and results in the inferred k value being higher than the actual value until consolidation is complete (the effect being greatest in the period 1984–1987 when the waste thickness increased). Second, the decrease in void ratio of the soil due to consolidation will result in an actual decrease in k with time. Both effects occur relatively quickly because, for KVL liner consolidation

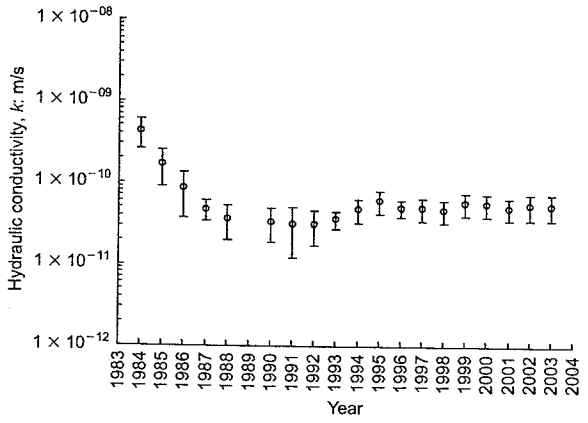


Fig. 14. Variation in annual average hydraulic conductivity with time for compacted clay liner at Keele Valley Landfill, Lysimeter 3 (data courtesy of City of Toronto)

parameters, consolidation is fast (95% consolidation in less than a year for a given load increment). Based on the data shown in Fig. 14 it is evident that the effect of the decrease in void ratio dominates, because there was a significant decrease in inferred k (more than an order of magnitude) between 1984 and 1988.

Hydraulic conductivity can also be influenced by temperature, as discussed earlier. For example, inspection of the data in Fig. 14 shows an increase in annual average k with time between 1991 ($k = 3{\cdot}1 \times 10^{-11}$ m/s, $T_{ave} = 14{\cdot}4$°C) and 1995, after which it remained relatively constant at a value (1995–2003: $k_{ave} = 5{\cdot}3 \times 10^{-11}$ m/s, $T_{ave} = 34{\cdot}8$°C) still well below the specified k of 1×10^{-10} m/s. Inspection of Fig. 12 shows that during the period from 1991 to 1995 the temperature at the top of the liner rose from 14·4°C to 29·8°C and then had an average value of 34·8°C for 1995–2003. Fig. 15 shows the variation in annual average k with annual average temperature between 1991 and 2003. The increase closely corresponds to what one would expect from theoretical consideration of the effect of temperature on k. Since the temperature stabilised, so too has the hydraulic conductivity, suggesting no effect of clay–leachate interaction (Rowe *et al.*, 2004) over the period of time considered. This is consistent with unpublished laboratory studies by Quigley (pers. comm.) that showed no significant change in k due to interaction with MSW leachate after permeation with many pore volumes of leachate.

Based on the hydraulic conductivity of the liner (e.g. Fig. 14) and the leachate head (Fig. 12), the flow through the 1·5 m liner can be calculated. There is some uncertainty regarding the head at the base of the liner (there is unsaturated silty sand of variable thickness between the base of the liner and the water table), and the following leakage estimates are based on a typical 4 m distance between the base of liner and the water table. Thus in Stage 1 of KVL the leakage in 1991 (minimum k, $T \approx 14$°C, before the leachate mound began to rise significantly) was about 120 litres per hectare per day (lphd, the units commonly used for reporting leakage through landfill liners). By 2003 this had increased due to both an increase in k and, most significantly, an 8·4 m leachate mound, to about 430 lphd. In Stages 3 and 4 of the landfill, where the temperature has risen to over 30°C (e.g. Lysimeter 16A, Stage 4, Fig. 12) and the leachate mound is well within the drainage layer (< 0·3 m), a hydraulic conductivity of 5×10^{-11} m/s gives an estimated leakage of 160 lphd. These numbers provide an indication of the magnitude of the expected leakage through a single CCL at KVL with (430 lphd) and without (160 lphd) a significant

leachate mound. They also provide a frame of reference for the discussion of leakage in the following subsection.

Observed leakage through liners involving a geomembrane

Double liner systems have a leak detection system (LDS) (which also serves as a secondary LCS, SLCS) that can be used to provide insight regarding the effectiveness of the primary liner. However, the interpretation of the data from LDS requires careful consideration of sources of fluid other than leakage from the landfill (Gross *et al.*, 1990). These include:

(*a*) water that infiltrated into the LDS layer during construction
(*b*) water arising from consolidation of the CCL
(*c*) groundwater from outside the landfill.

Bonaparte *et al.* (2002) examined data for 72 landfill cells containing one of: a single GM primary liner; a GM/GCL composite primary liner; a GM/CCL composite primary liner; or, in one case, a GM and GCL over a CCL (GM/GCL/CCL) composite primary liner. They identified three periods during which there may be very different flows to the LDS:

(*a*) the initial period of operation
(*b*) the active period of operation
(*c*) the post-closure period (after the final cover has been constructed).

Only the second and third periods are of interest here, because they provide a better indication of liner performance. The landfills examined had LDS composed of sand, gravel, or geonets (GN). All eventually provide useful information, but the gravel and GN systems are likely to provide the best data at early times because the response is faster than for sand systems, and they are unlikely to contain significant construction water or have significant storage for leakage water after the initial period of operation. Table 3 summarises the mean, standard deviations and maximum average monthly flows in the LDS for the three types of liner system discussed above and all types of LDS, and includes cells constructed with and without construction quality control (CQA). All data discussed below are from cells that were constructed with a formal CQA programme unless specifically noted otherwise.

For cells with a primary HDPE GM liner, formal CQA was adopted for 23 of the 25 cells. For these cells, the average monthly flow ranged from 0, to 790 lphd in the active period and from 2 to 100 lphd in the post-closure period (for cells without CQA the corresponding flows were up to 1600 lphd and 330 lphd). The peak flows were generally less than 500 lphd, but exceeded 1000 lphd for two cells. To put these numbers (and those in Table 3) into context, one can compare these leakage rates with those calculated for a 0·6 m CCL. A primary GM liner alone has leakage (Table 3) that often exceeds that for a primary CCL with $k = 5 \times 10^{-11}$ m/s (40–65 lphd) but is normally considerably less than that for a primary CCL with a typically specified $k = 1 \times 10^{-9}$ m/s (860–1300 lphd). However, there are cases where the leakage through a GM alone will exceed that of the CCL with $k = 1 \times 10^{-9}$ m/s.

The combination of a GM and GCL substantially reduces leakage (Table 3) relative to a GM alone. The average flow ranged from 0 to 11 lphd in the active period and from 0 to 2 lphd in the post-closure period. The maximum peak flow was 54 lphd.

The interpretation of leakage rates for GM/CCL systems is more complicated than for the cases discussed above, because part of the fluid collected is consolidation water from the overlying CCL. The leachate chemistry provides some support for the argument that there is consolidation water contributing to the collected fluid. In general, the chloride, COD and, where available, volatile organic compound (VOC) concentrations in the LDS were low compared with values in the leachate (Bonaparte *et al.*, 2002). Notwithstanding uncertainty regarding the proportion of collected fluid that is consolidation water, inspection of Table 3 indicates that the GM/CCL combination is significantly better than the GM

Table 3. Mean and standard deviations of flow in LDS for landfill cells with a primary liner and underlying LDS (in lphd). Numbers have been rounded (Based on data from Bonaparte *et al.*, 2002)

Liner/stage	No. of cells	Average monthly flows: lphd			Peak monthly flows: lphd		
		Mean*	SD†	Max‡	Mean*	SD†	Max‡
Single liner: GM alone							
Active	25	190	330	1600§ 790¶	360	610	3070§ 1830¶
Post-closure	6	130	120	330	330	30	1130
Composite GM/GCL liner							
Active	22	1·5	2·7	11¶	9	16	54¶
Post-closure	5	0·6	0·9	2	4	5	10
Composite GM/CCL or GM/GCL/CCL liner							
Active	11	90	90	370§ 260¶	250	370	1990§ 1240¶
Post-closure	3	50	50	220	60	90	250

*Mean and †standard deviation of reported average and peak average monthly flows: these were obtained for different cells over different periods, and include data obtained for systems with sand, gravel and GN LDS.
‡Maximum value reported.
§Largest value reported, but it is for sand LDS and so may reflect stored water in the LDS shortly after construction.
¶Largest value for liner system with GN LDS.

D23/48

alone. Also, noting that the CCLs in Table 3 all had $k = 1 \times 10^{-9}$ m/s specified, the GM/CCL composite liner was typically significantly better than 860–1300 lphd calculated ($k = 1 \times 10^{-9}$ m/s, $i = 1$–1·5) for a CCL alone.

Given the relatively fast primary consolidation typical of many CCLs, one would expect that the effect of consolidation water would be most significant in the active period and would quickly cease to be significant in the post-closure period, especially for landfills with a GN or gravel LDS that drains quickly. Table 4 summarises results for nine cells where the LDS was a GN or gravel. In several cases there are different cells at the same landfill, and so an assessment can be made of variability that occurs for nominally similar conditions.

Several observations arise from Table 4. First, there is considerable variability in leakage for a similar liner thickness (and maximum waste thickness), and this can be attributed to variability in the liner itself (rather than consolidation). For example, in the post-closure period there is a sixfold difference in average flow and more than a threefold difference in peak flow for two cells at landfill AQ. There is also an order of magnitude difference in the flows for landfills AM and AQ, which have a similar (450 mm) CCL and waste thicknesses.

Second, assuming that the water content and compaction are somewhat similar, if consolidation is an important factor, then one would expect about twice as much consolidation water for 900 mm thick CCLs as for 450 mm thick CCLs. However, during the active period (when consolidation water should be most significant), the flows at landfill AQ (450 mm) are similar to or larger than those for landfills AL and AO (900 mm). In the post-closure period the flows of the two cells at landfill AQ straddle that for landfill AD. More data are needed; however, the available data all lead to the conclusion that variability in the composite liners themselves is significant, and that the difference between the observed flows for the GM/GCL cases and the GM/CCL cases represents differences in leakage, and cannot be explained as the result of consolidation water. This suggests that, on the whole, GM/GCL composite liners can be constructed to give lower leakage than a GM/CCL composite. However, it appears that the flows for landfill AR, which had a GM/GCL/CCL (300 mm) system, are similar to some of those with both 900 mm and 450 mm CCL. While some of this flow is undoubtedly consolidation water, given the relatively small amount of waste (11 m) and thin CCL (300 mm) it appears that there are defects in the GM that are primarily responsible for the higher than otherwise expected leakage. A possible explanation for the differences in the observed flows for those landfills with a GM/GCL compared with those with a GM/CCL liner will be presented in a subsequent section.

It should be emphasised that all of the leakages reported for composite liners are small, and, correspondingly, the contaminant transport by advection will be very small.

Holes in geomembranes

As an intact GM is essentially impermeable to water, leakage through GMs such as those discussed in the previous section must arise from flaws in the GM. These flaws may:

(*a*) be in the GM following manufacturing
(*b*) arise from handling of the GM rolls during delivery to the site, on-site placement and seaming
(*c*) be caused by physical damage during the placement of material such as drainage gravel on top of the liner system
(*d*) arise from subsequent penetration or cracking of the GM during or following placement of the waste.

Holes due to sources (*a*)–(*c*) can be detected by an electrical leak detection survey. Compiling 205 results from four published leak detection surveys, Rowe *et al.* (2004) found that (a) no holes were detected for 30% of the cases, and (b) less than 5 holes/ha were detected for half of the surveys. Nosko & Touze-Foltz (2000) reported 3 holes/ha after installation and 12 holes/ha after placement of drainage layer. Giroud & Bonaparte (2001) suggested that, for GMs installed with strict construction quality assurance, 2·5–5 holes/ha can be used for design calculations of leakage. Colucci & Lavagnolo (1995) reported that 50% of holes had an area of less than 100 mm^2 ($r_0 < 5{\cdot}64$ mm). The number and size of holes will depend on the level of CQC/CQA. As the leak detection surveys are conducted shortly after construction of the liner system, it is uncertain how many holes may develop under combined overburden pressures, elevated temperatures and chemical exposure years after construction. These holes may arise from:

(*a*) indentations at gravel contacts following placement of the waste
(*b*) stress cracking at points of high tensile stress in wrinkles

Table 4. Weighted average flow rates and peak flow in LDS for landfills with composite liners involving GM and CCL and a GN or gravel LDS. All numbers have been rounded to nearest 10. (Based on data from Bonaparte *et al.*, 2002)

	Thickness of			Active operation		Post-closure	
Landfill	Waste: m	GM: mm	CCL: mm	Average:* lphd	Peak:† lphd	Average:* lphd	Peak† lphd
AL1	68	1·5	900	160	370		
AO1	23	1·5	900	120	350		
AO2	23	1·5	900	70	160		
AD7	24	1·5	900	60	390	80	170
AM1	27	2·0	450	10	60		
AM2	27	2·0	450	10	40		
AQ1	21	2·0	450	200	1240	120	250
AQ10	21	2·0	450	40	250	20	80
AR1	11	1·5	6/300‡	170	470		

*Time-weighted, based on the reported values for different time periods.
†Largest peak value reported for a monitoring period.
‡GCL/CCL.

D24/48

(c) substandard seams subjected to tensile stresses.

Large-scale physical testing conducted by Tognon *et al.* (2000), discussed in more detail in a later section, indicated that the protection layer between the GM and the overlying drainage material has a critical effect on the tensile strains induced in the GM. For example, with a protection layer consisting of two layers of nonwoven geotextile having a total mass per unit area of 1200g/m^2, there were over 300 gravel indentations per square metre in the GM and a peak strain (13%) close to the yield strain. If only one out of every 10 000 indentations (i.e. 0·01%) eventually resulted in a defect, this would correspond to over 300 holes/ha. Consequently, if one is to design with 2·5 or even 5 holes/ha it is essential to have both a rigorous CQC/CQA programme and a protection layer that will limit the tensions that develop in the GM to a small value.

Wrinkles in geomembranes

Wrinkles in a GM arise both from construction practice and, in particular, from thermal expansion when the GM is heated by the sun after placement. These wrinkles do not disappear when the GM cools or when it is loaded (Stone, 1984; Soong & Koerner, 1998; Brachman & Gudina, 2002). Pelte *et al.* (1994) reported wrinkles 0·2–0·3 m wide, 0·05–0·1 m high at spacing of 4–5 m. Touze-Foltz *et al.* (2001a) reported wrinkles 0·1–0·8 m wide, 0·05–0·13 m high at spacing of 0·3–1·6 m. Rowe *et al.* (2004) reported a case where there were 1200 wrinkles/ha. Wrinkles are important because of the increased potential for contaminant migration through a hole in the GM at or near the wrinkle. There is also increased potential for development of future holes due to stress cracking at points of high tensile stress in the wrinkle. As wrinkles are often interconnected, the length of a wrinkle should be regarded as the total linear distance that fluid can migrate along a wrinkle and its interconnections.

Geomembrane–clay liner interface properties

Leakage through a defect in a composite liner (GM over clay) depends on the contact between the GM and the underlying soil (Brown *et al.*, 1987). Imperfect contact allows for a transmissive zone between the two liners, and hence, once fluid has migrated through the defect, it can then move laterally in the interface between the two liners before fully percolating through the clay liner. In addition to wrinkles, discussed above, there are three primary sources of this imperfect contact (Rowe, 1998a). First, even for a well-compacted soil, there may be protrusions related to particle size distribution in the liner material, and these protrusions will create some gap in which water may flow. Second, when compacting clay liners to obtain low hydraulic conductivity, it is usually desirable to compact at water contents 2–4% above the standard Proctor optimum value; however, this is often close to the plastic limit of the soil, and it is difficult to obtain a smooth surface because of rutting caused by construction equipment, either during construction of the liner or when the overlying GM and leachate collection layer is being placed. These undulations/ruts tend to provide linear zones where there will not be intimate contact between a GM and CCL (e.g. Cartaud *et al.*, 2005a). Third, the placement of a geotextile between the GM and clay liner either as a separate layer or as part of GCL may provide a continuous, potentially transmissive, layer. This is particularly true when there is a separate layer between the GM and a CCL, as is common in France (Cartaud & Touze-Foltz, 2004). To a lesser extent the presence of a geotextile component of a GCL in contact with a GM also provides for some transmissivity, although much less than for a geotextile over a CCL, because bentonite swelling into the geotextile substantially reduces the geotextile transmissivity (Touze-Foltz, pers. comm.).

Giroud & Bonaparte (1989) characterised two typical GM–CCL contacts: 'good' and 'poor'. Based on calculations for typical liner properties ($k = 10^{-9}$ m/s), these descriptors can be related to average transmissivities of the GM/CCL interface of $1{\cdot}6 \times 10^{-8}$ m^2/s and 1×10^{-7} m^2/s respectively (Rowe, 1998a). These values are useful for performing calculations; however, the actual interface conditions are far more complex. For a CCL with a hydraulic conductivity k of 1×10^{-9} m/s, 'good' and 'poor' contact correspond to an average 'gap' between the GM and the CCL of 27 μm and 50 μm respectively. Cartaud *et al.* (2005a, 2005b) used laser rugosimetry to study the topography of the interface between 2 mm thick HDPE GM and CCL, and found that, within a 1 m^2 area, the apertures could vary from direct contact up to 10 mm. Using numerical simulations for the aperture topography obtained at different pressures, they showed a three order of magnitude reduction in fluid velocities at the interface, and hence a substantial reduction in the transmissivity of the interface, as the applied pressure increased from 6 to 64 kPa.

When a GM is placed over a GCL, there is greater potential for obtaining good contact with a low-permeability layer than when placed over a CCL. This is because the GCL can be placed flat on a well-compacted, smooth and firm foundation (e.g. Giroud, 1997). Away from wrinkles, the factor controlling the leakage for GM over a GCL is the transmissivity of the interface between the GM and GCL. Harpur *et al.* (1993) reported tests for the transmissivities of these interfaces and reported values between 2×10^{-12} m^2/s (at a normal stress of 7 and 70 kPa) for bentonite in direct contact with the GM, and 2×10^{-10} m^2/s (7 kPa) and 1×10^{-12} m^1/s (70 kPa) for a GCL with a woven geotextile in contact with the GM. The highest of these transmissivities was two orders of magnitude below that for 'good' contact for a typical CCL discussed above ($1{\cdot}6 \times 10^{-8}$ m^2/s). This difference in interface transmissivity provides at least a partial explanation for why the leakage through composite liners with a GCL may be lower than that with a CCL.

Where there are wrinkles in a GM over a GCL, there is potential for the bentonite to 'extrude' laterally from the GCL in direct contact with the GM to the zone below the wrinkle. Provided that there is sufficient applied compressive stress (Giroud, 1997), and the wrinkle is small, this would reduce or eliminate the gap between the GM and GCL at that location and hence reduce the interface transmissivity by establishing closer contact between the GM and GCL. However, this also thins the GCL (Stark, 1998; Dickinson & Brachman, 2003). For large wrinkles there may be: (a) significant thinning of the GCL adjacent to the wrinkle, which could impact negatively on the GCL performance; and (b) failure to fill the gap between the GCL and GM.

Solutions for assessing leakage through composite liners—no significant wrinkles

Leakage through composite liners can be calculated using empirical equations or analytical equations, or from numerical analysis. Empirical equations are established by curve-fitting families of solutions from analytical equations (Giroud & Bonaparte, 1989; Giroud, 1997), and although they may be convenient to those without access to analytical solutions, they really do not provide any original information. Analytical equations involve either the assumption of perfect contact (Rowe & Booker, 2000) or lateral migration in a transmissive zone below the GM combined with 1D

D25/48

flow into the underlying soil (Jayawickrama *et al.*, 1988; Rowe, 1998a; Touze-Foltz *et al.*, 1999, 2001b). Numerical methods allow modelling of the actual three-dimensional conditions near the hole (Foose *et al.*, 2001) or the complete variability of the interface topography if it is known (Cartaud *et al.*, 2005b). This raises three questions.

(*a*) Does the assumption of 1D flow in the subsoil have any practical effect on the accuracy of the analytical solutions?
(*b*) Does the additional flexibility afforded by numerical analysis imply that it should be used for calculating leakage in practical problems?
(*c*) Do these equations, which neglect the presence of significant wrinkles, give reasonable predictions of observed leakage through primary liners?

These three questions are addressed below.

Except for the case where a low-permeability soil is deposited as a slurry in contact with the GM and it consolidates with time under a large compressive stress, perfect contact between the GM and the subsoil (as assumed by Rowe & Booker, 2000) is likely to provide a lower bound to the leakage through holes in a GM that is placed above a CCL or GCL. Assuming that there is a uniform transmissive zone between the GM and clay liner, one can use Rowe's (1998a) direct contact analytical equation or Giroud's (1997) empirical charts to calculate leakage. However, the question regarding the significance of the assumption of 1D flow in the liner needs to be addressed. Foose *et al.* (2001) partly addressed this question and showed that, for a GM over a GCL, Rowe's (1998a) direct contact analytical solution provided very good agreement with the results of numerical analysis for the typical reported transmissivity values discussed above. Similarly the author has found excellent agreement (error of less than 4%) between the direct contact analytical solution and the axisymmetric numerical analysis for interface transmissivities within the practical range of interest for both the GM/GCL composite liner and the GM/CCL composite liner.

Both the numerical and analytical solutions discussed above assume a uniform transmissivity of the interface. Analytical solutions have also been developed for regular variations in the transmissivity of the interface (Touze-Foltz *et al.*, 2001b), and numerical methods are available to model more complex situations (Cartaud *et al.*, 2005b). These may be useful in interpreting laboratory tests where the actual interface topography is well defined, but in practical situations the interface topography will vary significantly (e.g. as is evident from the work of Cartaud *et al.*, 2005a, 2005b), but is unknown at the location of the (assumed) holes used in design calculations. Thus for practical purposes, the more simplified approaches used in conjunction with a range of likely transmissivities will provide the information needed for design purposes, and the really critical question is whether these equations, which neglect the presence of significant wrinkles, give reasonable predictions of observed leakage through primary liners. To address this question, it is useful to calculate leakage for typical interface conditions and a range of numbers of holes and compare these results with leakage reported by Bonaparte *et al.* (2002).

The upper two curves in Fig. 16 show the calculated leakage through a composite liner (GM + CCL) for poor contact and good contact, together with the shaded zone defined by the mean value of the average monthly flows in the active (90 lphd) and post-closure (50 lphd) periods for the different landfill cells summarised in Table 3. With poor contact conditions 20–30 holes/ha would be required to explain the typical leakage. For good contact conditions more than 90 holes/ha would be required to explain the lower value for typical leakage, and in excess of 100 holes/ha would be required to explain the upper end of the range. A very large number of holes (≫ 100 holes/ha) would be required to explain the maximum flow shown by an arrow at the right of the figure.

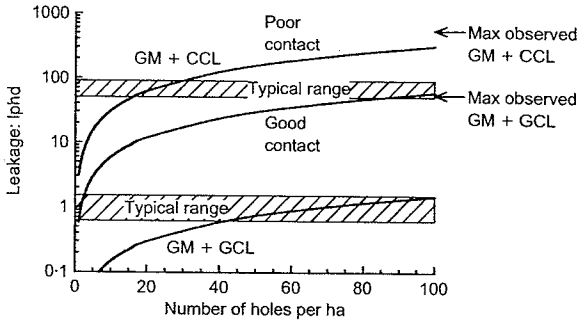


Fig. 16. Comparison with observed leakage and calculated leakage through composite liners, neglecting significant wrinkles. Upper solid curve for GM + 0·6 m CCL ($k = 1 \times 10^{-9}$ m/s) and poor contact ($\theta = 1 \times 10^{-7}$ m^2/s); centre solid curve for GM + 0·6 m CCL ($k = 1 \times 10^{-9}$ m/s) and good contact ($\theta = 1{\cdot}6 \times 10^{-8}$ m^2/s); and lower solid curve for GM + 10 mm GCL ($k = 2 \times 10^{-10}$ m/s; $\theta = 2 \times 10^{-10}$ m^2/s). In each case hole radius $r_0 = 10$ mm, depth of fluid above GM is 0·3 m. The upper shaded region shows typical field leakage rates for GM/CCL, with the upper and lower limits defined by the mean value of the average monthly flows in the active (90 lphd) and post-closure (50 lphd) periods for the different landfill cells summarised in Table 3. The lower shaded region shows typical field leakage rates (0·6–1·5 lphd) for GM/GCL leakage (again from Table 3 and based on data reported by Bonaparte *et al.*, 2002). Upper and lower arrows on right-hand side show maximum observed leakage for GM/CCL and GM/GCL based on data reported by Bonaparte *et al.* (2002)

The calculated leakage for a GM + GCL composite is shown by the lowest curve in Fig. 16, and the lower shaded zone shows the observed typical (0·6–1·5 lphd) leakage (again from Table 3). About 40 holes/ha would be required to explain the bottom end of the typical range and more than 100 holes/ha to explain the upper end of the typical range. The maximum observed leakage shown by the lower arrow on the right is more than an order of magnitude greater than the predictions with 100 holes/ha. Thus it would appear that the predications above, which assume no major wrinkles, are not consistent with the observed leakage and the likely number of holes/ha for either the GM/CCL or GM/GCL systems.

Solutions for assessing leakage through composite liners with wrinkles

Rowe (1998a) presented an analytical solution for the case where a hole coincides with a wrinkle in the GM of length L and width $2b$ (Fig. 17). The transmissivity beneath the wrinkle is much greater than the interface transmissivity, θ, where the GM is in contact with the underlying soil. It is also assumed that $L \gg b$ such that the effects of leakage at the ends of the wrinkle can be neglected. This solution assumes unobstructed lateral flow along the length L and across the width $2b$ of the wrinkle, and then lateral flow between the GM and the soil outside the wrinkle. One-dimensional, vertical flow is assumed from the transmissive layer through the underlying soil beneath the wetted distance from the wrinkle (this is an approximation). Rowe's solution allows consideration of interactions between adjacent similar wrinkles assumed to be spaced at a distance $2x$ apart, and the leakage Q is given by

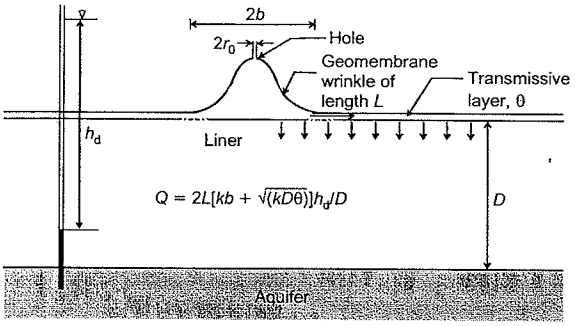


Fig. 17. Schematic defining leakage through a composite liner with a wrinkle. Assumes lateral migration at interface and vertical flow in clay liner

$$Q = 2Lk\left(b + \left\{1 - \exp\left[-\alpha(x-b)\right]\right\}/\alpha\right)\frac{h_d}{D} \qquad (22)$$

where L is the length of the wrinkle; $2b$ is the width of the wrinkle; k is the hydraulic conductivity of the clay liner; θ is the transmissivity of the GM–clay liner interface; $\alpha = [k/(D\theta)]^{0.5}$; h_d is the head loss across the composite liner; and D is the thickness of the clay liner. Assuming no interaction with an adjacent wrinkle, the leakage Q is given by

$$Q = 2L\left[kb + (kD\theta)^{0.5}\right]\frac{h_d}{D} \qquad (23)$$

The leakage calculated using this wrinkle analytical solution was compared with that from a 2D finite element analysis and found to be in excellent agreement (an error of 5% or less) for both the GM/GCL composite liner and the GM/CCL composite liner for the range of cases considered.

Figure 18 shows the calculated leakage (using equation (23)) through GM over a 0·6 m CCL for wrinkles of total length 10 m and 100 m together with the typical leakage range previously shown in Fig. 16 and summarised in Table 3. With 10 m long wrinkles, between 12–22 holed wrinkles/ha are required to explain the typical leakage, and for 100 m long wrinkles only 1–3 holed wrinkles/ha would be required to explain the typical leakage. A similar comparison for a GM over a GCL indicated that the typical range of leakage could be explained by 2–3·5 holed 10 m long winkles/ha, and the maximum leakage could be explained by about 5 holed 100 m long winkles/ha. Thus the typical observed leakage for composite liners with both CCLs and GCLs can be readily explained by holes in wrinkles for the typical number of holes/ha and reasonable combinations of other parameters.

Some factors that can influence leak detection

As noted in previous sections, leakage through composite liners (and hence the fluid collected in LDS) is controlled by the leachate head, the number and size of holes in the GM, wrinkles, the transmissivity of the GM/clay liner interface, and the hydraulic conductivity and thickness of the liner. However, there are a number of additional factors that can influence the amount of fluid that is collected. These include:

(*a*) the hydraulic characteristics of the material above the GM
(*b*) consolidation of the liner
(*c*) the initial degree of saturation of the soil and geosynthetics below the GM

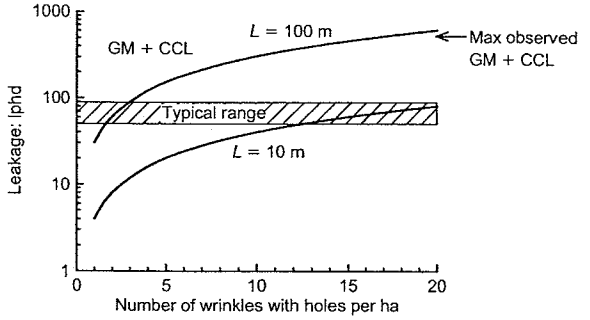


Fig. 18. Comparison with observed leakage and calculated leakage through composite liner accounting for significant wrinkles. Calculated curves for GM + 0·6 m CCL ($k = 1 \times 10^{-9}$ m/s; good contact $\theta = 1{\cdot}6 \times 10^{-8}$ m²/s), wrinkle width, $2b = 0{\cdot}2$ m, and depth of fluid above GM, $h_w = 0{\cdot}3$ m; based on Rowe *et al.* (2004). Typical field leakage rates and maximum observed values from Table 3 and based on data reported by Bonaparte *et al.* (2002)

(*d*) the potential for the GCL to significantly reduce the transmissivity of an underlying geonet drainage layer.

These will each be briefly discussed below.

The leakage through a hole in a GM cannot exceed (a) the flow from Bernoulli's equation if the permeability of the overlying layer is high enough, or (b) the capacity of the fluid to drain to the hole. Giroud *et al.* (1997b) considered both of these cases, and one can (iteratively) calculate the limiting flow Q through a hole from

$$h_w = \left\{\frac{r_0^2 q_0}{2k_{om}} + \frac{Q}{2k_{om}\pi}\left[\ln\left(\frac{Q}{\pi r_0^2 q_0}\right) - 1\right] + \frac{1}{4g^2}\left(\frac{Q}{1{\cdot}88 r_0^2}\right)^4\right\}^{0.5} \qquad (24)$$

where q_0 is the liquid supply (e.g. permeation through the waste per unit area reaching the LCS), k_{om} is the hydraulic conductivity of the permeable leachate collection layer over the GM, r_0 is the radius of the hole in the GM, and g is the acceleration due to gravity. This expression reduces to the Bernoulli equation as k_{om} tends to infinity. For typical sizes of hole, equation (24) may limit leakage only for very long wrinkles.

Expulsion of water from voids due to consolidation will lead to flow to the LDS. It will also result in a reduction in hydraulic conductivity with time, as discussed earlier for the KVL. The significance of this mechanism will depend on the properties of the liner; however, for CCLs such as that at Keele Valley (discussed earlier) and Halton (Rowe *et al.*, 2000c) consolidation occurs quickly (95% consolidation in less than a year), and the rate of expulsion of fluid is controlled largely by the rate of loading due to increasing waste thickness above the liner. This may increase apparent leakage; however, as discussed earlier in the context of the Bonaparte *et al.* (2002) data, there is no evidence that this is significantly affecting the reported leakage rates.

Rowe & Iryo (2005) have examined the hydraulic performance of an LDS below a composite liner consisting of a GM over a GCL. The time for leakage to be detected was shown to be highly dependent on the initial degree of saturation of the material below the GM and the distance between the hole/wrinkle and the drainage point in the system. Under some circumstances this could result in leakage not being detected for a considerable period of time

(several years). The predicted leakage was shown to be of a similar magnitude to that reported in field monitoring and discussed earlier. The effect was at a minimum (although not necessarily negligible) for a GCL alone between the GM and the underlying LDS, and greatest when the GCL was on an unsaturated foundation layer located above the LDS. This raises the caution that it may take considerable time before the full steady-state leakage is established for situations where the soil and geosynthetics below the GM are not initially saturated.

Although geonets have a number of advantages in terms of the potential speed of leak detection, there is potential for a significant reduction in the flow capacity of the geonet due to intrusion of adjacent materials into the structure of the geonet. For example, Hwu *et al.* (1990) showed that the flow rate could be reduced by an order of magnitude as a result of geotextile intrusion into the apertures of geonets. The intrusion could be reduced by stiffening the geotextile by resin treating, burnishing, and scrim reinforcing, and there was a substantial benefit from using a composite fabric (a needle-punched nonwoven over a woven slit-film geotextile). Drainage layers can be designed with a three-layer system, and limited opening size adjacent to the geotextile, to minimise the effect of geotextile/soil intrusion.

Special care is needed when the geonet is overlain by a GCL (e.g. in an LDS below a composite liner) owing to the potential swelling of the bentonite into the geonet. Here the choice of the geosynthetic between the bentonite core and the geonet may be critical. Shaner & Menoff (1992) examined the effect of using a number of geotextiles placed between the GCL and the underlying geonet to limit the intrusion of the bentonite into the geonet. Legge & Davies (2002) also examined this issue. Fig. 19 shows the flow rate for a bi-planar geonet in a standard test between two steel plates. Also shown is the flow rate for a GCL in direct contact with the geonet, and finally for the case of a geogrid placed between the GCL and geonet. Legge & Davies (2002) reported that when the applied pressure of 400 kPa was maintained for 14 h for the GCL/geonet system the flow rate dropped to zero under a unit gradient (165 mm head difference). As this gradient greatly exceeds that typically encountered in landfills (< 0·05 on the base and < 0·33 on side slopes), it is apparent that a geonet should not be used as an LDS below a GCL unless a reinforcing layer sufficient to prevent intrusion of the bentonite into the geonet is placed between the GCL and geonet. There should also be a geosynthetic, either as part of the GCL or between the GCL and geonet, which will prevent potential internal erosion of the bentonite into the geonet (Rowe & Orsini, 2003). The details regarding what measures were taken to avoid bentonite intrusion in the landfills examined by Bonaparte *et al.* (2002) were not reported, and hence the effects of bentonite intrusion into geonets on the leakage detected are not known. However, this mechanism could serve to slow (and hence delay the reporting) of leachate to the sump and, if there is no reinforcing layer and sufficient pressure, could reduce or even cut off flow to the sump. Thus care is needed in the design and construction of these systems to ensure that the swelling of the GCL under vertical stress does not compromise the drainage function of any underlying geonet. It should also be noted that Davies & Legge (2003) have shown that this geonet transmissivity can also be substantially reduced by an increase in temperature.

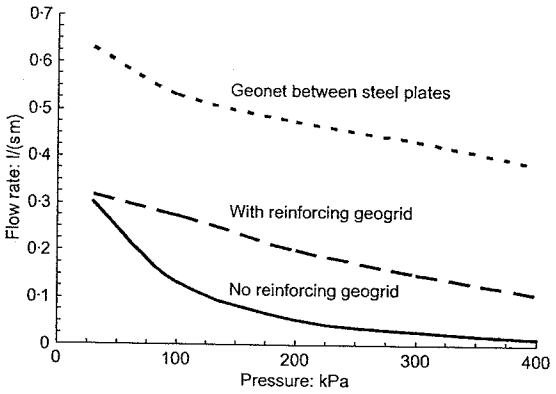


Fig. 19. Variation in flow rate with applied pressure for a transmissivity test at 165 mm head for: geonet between steel plates (standard test); GCL, geogrid and geonet; and GCL in direct contact with geonet. Note: GCL has polypropylene woven ($M_A = 270$ g/m^2) cover and composite nonwoven ($M_A = 270$ g/m^2)-woven slit film ($M_A = 110$ g/m^2) carrier geotextiles, and bentonite ($M_A = 3700$ g/m^2). Geonet was 3·3–4·4 mm thick at 2 kPa ($M_A = 700$ g/m^2). Woven biaxial geogrid had high-strength polyester yarns with polymeric coating and grid aperture 1 mm × 1·3 mm, tensile strength of 32·6 kN/m, and tensile stiffness of 200 kN/m at 2% strain. Partially adapted from Legge & Davies (2002), and with additional data from Legge & Davies (pers. comm.)

DIFFUSION THROUGH LINERS

Any assessment of long-term impact for a landfill requires contaminant transport modelling. Although this is a common practice in some parts of the world, there are still jurisdictions where advective transport (leakage as discussed in the last section) is the only consideration in assessing potential impact. However, as will be demonstrated below, diffusive transport may result in contaminant transport over even relatively small times.

Othman *et al.* (1996) reported significant levels of 1,1 DCA and DCM in the leak detection system for a landfill with a 1·5 mm thick HDPE GM and 6 mm thick GCL composite liner and a 5 mm thick geonet LDS. This same system had exhibited low leakage rates (average value of 3·6 lphd from 14 to 21 months, reducing to 0·7 lphd between 22 and 31 months), and the inorganic chemistry was not characteristic of leachate. This implies that any leakage through holes in the GM was very small. However, the concentrations of DCM and 1,1 DCA were significant over a period of between 6 and 28 months after commencement of waste placement. This suggests possible diffusion through the GM/GCL system. Workman (1993) also examined the chemistry of the fluid in the LDS of a number of cells and detected several VOCs including chloroethane, ethylbenzene and trichloroethene at low concentrations. Workman hypothesised that they migrated as gases from the primary to secondary system at the side slopes. Although this is possible, the alternative hypothesis of migration through the primary liner by diffusion warrants further consideration. In this section the diffusion through naturally deposited and compacted clays, GCL and GMs will be examined.

Diffusion through naturally deposited clay

Natural diffusion profiles through clays deposited at the end of the last glacial period in southern Ontario, Canada, have provided an ideal means of assessing long-term diffusion through thick (30–40 m) low-permeability deposits. These cases (Desaulniers *et al.*, 1981; Quigley *et al.*, 1983; Rowe & Sawicki, 1992) have demonstrated the migration by diffusion over a period of 10 000–12 000 years (against small downward advective flow in two of the cases). The corresponding diffusion coefficients (2×10^{-10}, 3×10^{-10}

D28/48

and $4–6 \times 10^{-10}$ m^2/s) varied over only a relatively small range, which can be attributed, in part, to the different tortuosity of the three clayey soils and, in part, to the effect of the need to maintain an ion balance and hence the dependence of the rate of chloride migration on the rate of migration of associated cations.

Near the Sarnia site, dissolved gases originating from the underlying sedimentary bedrock have also been diffusing upwards in the groundwater. These gases come out of solution upon stress release, and have been observed venting in boreholes drilled into the till. These gases have contributed to two geotechnical problems (Rowe *et al.*, 2002) in the area. In one of these cases, excavation to a depth of 25 m for a new cell for a hazardous waste landfill resulted in venting of gas and water through the remaining 14–15 m of low-permeability till at three separate locations, and resulted in the abandonment of the cell. This case highlights the implication of the diffusion of gas and the importance of identifying the presence of dissolved gas and assessing its implications with respect to stability when considering excavations in areas underlain by gas.

Diffusion profiles have also been observed migrating downwards from landfills in the same till deposit. For example, an extensive study of the Confederation Road Landfill showed that, over the first 15 years, chloride migrated about 1 m. An effective diffusion coefficient of $6·3 \times 10^{-10}$ m^2/s based on short-term (1 week) laboratory tests provided a good prediction of the observed profile (Quigley & Rowe, 1986). It should be noted that the temperature below waste is higher than the long-term groundwater temperature, and that this will increase the diffusion coefficient, other things being equal, as discussed earlier. Most cations moved much less than chloride, with potassium experiencing significant ion exchange and, as a result, migrating less than half the distance of chloride. Heavy metals (lead, copper, zinc, iron and manganese) migrated less than 0·2 m (likely only 0·1 m) in the same time period and were largely removed from solution by precipitation (Yanful *et al.*, 1988).

Diffusion through compacted clay at the Keele Valley Landfill

As discussed earlier, the 0·3 m thick sand blanket at the KVL clogged significantly in the upper few centimetres, and, as a consequence of negligible flow through the sand, a diffusion profile developed within both the sand blanket and the underlying CCL (Fig. 4). Over a period of 4·25 years, chloride diffused about 0·75 m, and the profile was well predicted using a diffusion coefficient of 6×10^{-10} m^2/s obtained from short-term diffusion tests (Fig. 4(a)). This is consistent with the diffusion coefficient obtained for the Confederation Landfill noted above. These two cases illustrate how diffusion gives rise to rapid contaminant migration over the first few years, but the rate of advance of the contaminant front decreases with time (Rowe *et al.*, 2004). At the Keele Valley Landfill, chloride migrated 0·7–0·75 m in 4·25 years, whereas at the Confederation Road landfill it had migrated only a little over 1 m in 15 years. The organic contaminants appear to have diffused almost 0·6 m in 4·25 years (especially toluene, Fig. 4(b)).

Diffusion through GCLs

Lake & Rowe (2000a) examined the diffusion of sodium chloride through GCLs and showed that the diffusion coefficient decreased linearly with decreasing final bulk GCL void ratio. The method of GCL manufacture did not significantly affect the diffusion coefficients at a given void ratio; however, the method of manufacture can influence swelling (Lake & Rowe, 2000b) and hence the diffusion coefficient, especially for samples hydrated at low stress. At low concentrations (3–5 g/l, 0·05–0·08M) the diffusion coefficient deduced at a given void ratio was independent of test method adopted. However, as the NaCl concentration increased to 0·6M or 2·0M there was an increase in the diffusion coefficients in tests conducted at constant void ratio. This increase was largely mitigated when a constant stress was applied to the sample throughout the testing. For the range of conditions examined, the chloride diffusion coefficient was between 1×10^{-10} m^2/s and 4×10^{-10} m^2/s. This is slightly smaller than, but of a similar order of magnitude to, that for diffusion from waste disposal sites through natural and compacted clay (6×10^{-10} m^2/s), as noted earlier.

Lake & Rowe (2004) examined diffusion of DCM, DCA, TCE, benzene and toluene through a GCL at room temperature. They concluded that the order of the rate of mass transport through the GCL was dichloromethane (DCM) ≈ 1,2 dichloroethane (DCA) > benzene > trichloroethylene (TCE) > toluene. The diffusion coefficients at room temperature (at confining pressures less than 10 kPa) ranged from approximately 2×10^{-10} m^2/s to 3×10^{-10} m^2/s. The difference in mass transport was attributed largely to varying degrees of sorption of the different compounds to the geotextile component of the GCL. Rowe *et al.* (2005) extended this work by examining the effect of temperature on the diffusion of benzene, toluene, ethylbenzene, *m*- and *p*-xylene and *o*-xylene (BTEX). They confirmed that the geotextile component of a GCL was the primary contributor to sorption of hydrocarbons by the GCL, with partitioning coefficients (K_d at 22°C and 7°C in ml/g) for the entire GCL following the order: *m*- and *p*-xylene (42, 25) > ethylbenzene (36, 22) > *o*-xylene (27, 14) > toluene (15, 8·7) > benzene (4·4, 2·6). The diffusion coefficients (at 22°C and 7°C in m^2/s) followed the order benzene ($3·7 \times 10^{-10}$, $2·2 \times 10^{-10}$) > toluene ($3·1 \times 10^{-10}$, $1·8 \times 10^{-10}$) > ethylbenzene ($2·9 \times 10^{-10}$, $1·7 \times 10^{-10}$) > *m*- and *p*-xylene ($2·5 \times 10^{-10}$, $1·5 \times 10^{-10}$) ≈ *o*-xylene ($2·6 \times 10^{-10}$, $1·5 \times 10^{-10}$). The reduction in both the diffusion and sorption coefficients with decreasing temperature had opposite effects on mass transport through the GCL. However, the decrease in transport due to a reduced diffusion coefficient is more significant than the increased transport due to smaller sorption, and the net effect was reduced mass transport at lower temperature.

As the typical sodium bentonite used in GCL provided relatively little sorption for volatile organic compounds (VOCs), Lake & Rowe (2005) examined the potential improvement in the sorption that could be achieved for several organoclays and bentonite-activated carbon mixtures. Batch tests performed for four different organoclays showed two to three orders of magnitude higher sorption to DCM, DCA, TCE, benzene, and toluene relative to typical bentonite currently used in GCLs. A mixture of 2% of powdered activated carbon with powdered bentonite (2% PAC/PB) gave much higher sorption than for the organoclays examined for TCE, benzene and toluene. DCM sorption to the 2% PAC/PB mixture was less than for the organoclays, whereas DCA sorption was similar between the 2% PAC/PB mixture and the organoclays. Contaminant transport modelling was used to assess the engineering relevance of the potential improvement in GCL sorption due to the use of these alternative materials. Because of the relatively thin nature of the GCL, it was found that, despite the large potential increases in GCL sorption, the replacement of standard bentonite with the alternative materials had only a slight effect on contaminant migration. Thus the increased costs associated with modifying GCLs likely outweigh the

D29/48

benefit of such additives when considering the use of GCLs as part of the liner system for MSW landfills.

Given the fact that the diffusion coefficients and sorption capacity of the GCL are not substantially better than for a CCL, and yet the GCL is 50 to 100 times thinner than a typical CCL, it follows that in order to have equivalent performance as a diffusion barrier a GCL will need to be combined with a foundation soil such that the total thickness of the GCL and foundation soil (attenuation layer) is similar to the thickness of the CCL. This will be demonstrated in a subsequent section.

Diffusion through geomembranes and composite liners

Geomembranes are solids. Thus, putting aside leakage through holes as discussed earlier, there is no 'flow' through an intact GM. However, water and contaminants can potentially migrate through the GM by molecular diffusion. This migration is a molecule-activated process that can be envisaged as occurring by steps or jumps over a series of potential barriers, following the path of least resistance. For dilute aqueous solutions the process involves three key steps (Haxo & Lahey, 1988):

(*a*) partition of the contaminant between the medium containing the contaminant and the inner (i.e. contacting) surface of the GM (adsorption)
(*b*) diffusion of the permeant through the GM
(*c*) partition between the outer surface of the GM and the outer medium (desorption).

The diffusive movement of a contaminant (or water) through a GM involves a cooperative rearrangement of the penetrant molecule and the surrounding polymer chain segments. The process requires a localisation of energy to be available to allow a diffusive jump of the penetrant molecule in the polymer structure. The penetrant molecule and part of the polymer's molecular chain may share some common volume both before and after the jump. However, this jump will involve the breaking of some van der Waals forces or other interaction between the component molecules and polymer segments (Rogers, 1985). Thus, the diffusive motion depends on the energy availability and the relative mobilities of the penetrant molecules and polymer chains. This will depend on temperature, the size and shape of the penetrant, the nature of the polymer and, potentially, concentration.

The extent to which permeant molecules are sorbed in a polymer depends upon the activity of the permeant within the polymer at equilibrium (Müller *et al.*, 1998). When a GM is in contact with a fluid, there will be a relationship between the final equilibrium concentration in the GM, c_g, and the equilibrium concentration in the fluid, c_f. For the simplest case where the permeant does not interact with the polymer (e.g. as is the case for dilute solutions such as typical landfill leachates and HDPE), the relationship between the concentration in the fluid and the GM is given by (Henry's law)

$$c_g = S_{gf} c_f \tag{25}$$

where S_{gf} is called a partitioning coefficient and in principle is a constant for the given molecule, fluid, GM and temperature of interest.

At the second stage of the migration, diffusion of the sorbed penetrant within the material can be described by Fick's first law:

$$f = -D_g \frac{dc_g}{dz} \tag{26}$$

where f is the mass flux, D_g is the diffusion coefficient in the GM, c_g is the concentration of diffusing substance in the GM, and z is the direction parallel to the direction of diffusion. In transient state, the governing differential equation is (Fick's second law)

$$\frac{\partial c_g}{\partial t} = D_g \frac{\partial^2 c_g}{\partial z^2} \tag{27}$$

which must be solved for the appropriate boundary and initial conditions.

The last stage in the migration process is permeant desorption from the GM to the outer solution. This stage is similar to the first with an inverted process, and the contaminant concentration in the adjacent fluid can be expressed as

$$c'_g = S'_{gf} c_f \tag{28}$$

where S'_{gf} is the contaminant partitioning coefficient between the outside fluid and the GM. In the simplest case where the solutions on either side of the GM are aqueous, these two partitioning coefficients may be assumed to be the same ($S_{gf} = S'_{gf}$).

As the primary interest is in the concentrations of contaminant in water (not the GM), it is convenient to express the diffusion equations in terms of the concentration in adjacent solutions. Substituting equation (25) into equation (26), the flux from an aqueous solution on one side of the GM to an aqueous solution on the other side is given by

$$f = -D_g \frac{dc_g}{dz} = -S_{gf} D_g \frac{dc_f}{dz} = -P_g \frac{dc_f}{dz} \tag{29}$$

where the permeation coefficient (called the permeability in the polymer literature), P_g, is given by

$$P_g = S_{gf} D_g \tag{30}$$

and where P_g is a mass transfer coefficient that takes into account the partitioning and diffusion processes.

There are various methodologies that can be used (Rowe, 1998a) to deduce the partitioning, diffusion and permeation coefficients. The technique that best matches actual situations uses a two-compartment diffusion cell (insert to Fig. 20), where contaminant diffusion is monitored from the source to the receptor. When equilibrium is reached, the value of S_{gf} can be deduced from mass balance considerations. The diffusion coefficient can then be calculated knowing the value of S_{gf} by fitting the variation of the source and receptor concentration with time using computer software that models the boundary conditions, phase change and transport through the GM (e.g. Rowe & Booker, 2005).

The permeation coefficient P_g is highly dependent on the similarity of the penetrant and polymer. For example, Eloy-Giorni *et al.* (1996) indicated values of $S_{gf} = 8 \times 10^{-4}$ and $D_g = 2{\cdot}9 \times 10^{-13}$ m^2/s, giving a very low value of $P_g = 2{\cdot}3 \times 10^{-16}$ m^2/s for water and HDPE. Similarly, August & Tatzky (1984) found that strongly polar penetrant molecules have very low permeation coefficients through polyethylene (with the permeation coefficients being in the following order: alcohols < acids < nitroderivatives < aldehydes < ketones < esters < ethers < hydrocarbons). August *et al.* (1992) found that there was negligible diffusion of heavy metal salts (Zn^{2+}, Ni^{2+}, Mn^{2+}, Cu^{2+}, Cd^{2+}, Pb^{2+}) from a concentrated (0·5M) acid solution (pH = 1–2) through HDPE over a 4 year test period.

Chloride diffusion tests have now been running for about 12 years, and the results of this monitoring for one test are shown in Fig. 20. The receptor concentration remains below about 0·02% of the source concentration and lies within the range of analytical uncertainty for the chemical analysis. However, these data do allow an upper bound of 3×10^{-17} m^2/s to be placed on the permeation coefficient. In practical terms, this means negligible migration of chlor-

P 39/48

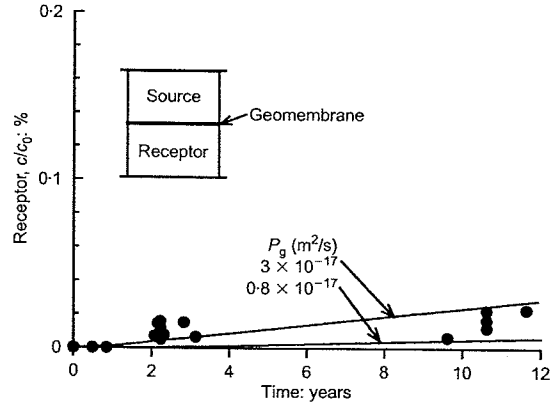


Fig. 20. Increase in chloride concentration in receptor as percentage of source concentration for a two-compartment diffusion test on 2 mm thick HDPE with an aqueous sodium chloride source solution ($c_0 = 2200$ mg/l)

ide (and likely other similar ions) through an HDPE GM for so long as it remains intact (i.e. for its service life, discussed later).

Figure 21 (▲) shows the increase in receptor concentration for diffusion of dichloromethane (DCM) through a 2 mm thick HDPE GM. In contrast to the case of chloride (Fig. 20), where after almost 12 years the receptor is less than 0·02% of the source concentration, for DCM the receptor reached 20% of the initial source concentration in only 70 days. This translates into a permeation coefficient $P_g = 4 \times 10^{-12}$ m²/s and demonstrates relatively rapid diffusion through HDPE. To put these results in context, Fig. 21 also shows the increase in receptor concentration for tests involving 30 mm of compacted clay (■) and a composite liner comprising the 2 mm GM and 30 mm of compacted clay (●). The mass transport through 30 mm of compacted clay was much faster (corresponding to a diffusion coefficient $D_g = 3{\cdot}5 \times 10^{-12}$ m²/s) than for the 2 mm thick GM, indicating that although the GM does not prevent diffusion, it does slow it compared with even a 15-fold thicker clay plug. The composite liner, not surprisingly, proves to be a better barrier than either the GM or clay alone. It will be

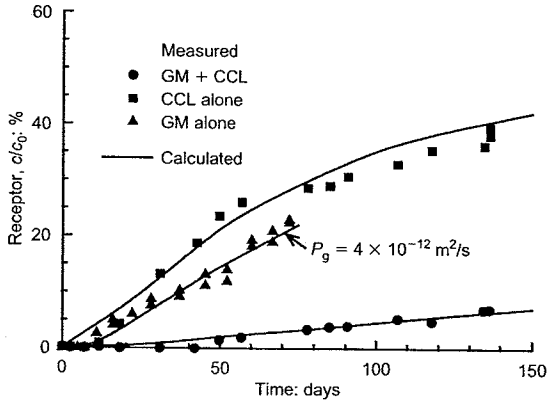


Fig. 21. Increase in DCM concentration in receptor chamber as percentage of source concentration for a two-compartment diffusion test examining: 2 mm thick HDPE (GM alone); 30 mm of compacted clay; and 2 mm thick HDPE GM over 30 mm of compacted clay. All cases involve an aqueous DCM source solution with $c_0 = 5$ mg/l. Data based on Sangam & Rowe (2001a, 2001b)

shown later that the impact on an underlying aquifer can be controlled by the use of a GM in association with suitable thickness of compacted clay and attenuation layer. Given the rapid diffusion through the GM it may also be anticipated (as will be later demonstrated) that the impact of contaminant migration of VOCs such as DCM will occur during the service life of the GM (in contrast to chloride, discussed above, where the GM is likely to control impact for its service life provided that it does not have too many holes initially).

Published values for the partitioning coefficient S_{gf} and diffusion coefficient D_g have been summarised by Rowe *et al.* (2004) together with effective diffusion coefficient (D_e) and sorption (K_d) values for CCLs and GCLs. These values should be used with caution, because they are dependent on temperature and, in some cases, the chemical composition and concentrations in the contaminant source. The values may also vary because of polymer crystallinity, additives etc., and hence published values should be used only as an initial guide; they do not replace experimentally determined values for the GM of interest for projects where uncertainty regarding the diffusion coefficient or sorption could have a significant impact.

SERVICE LIFE OF GEOMEMBRANE LINERS

Primary and secondary GMs have different exposure conditions and hence may be expected to have different service lives. The primary GM is subject to direct contact with leachate and temperatures higher than normal groundwater temperatures. For double composite lined landfills, leachate that passes through a primary clay liner can be expected to have lower concentrations of important constituents such as transition metals and organic compounds (due to cation exchange, precipitation and biodegradation as it migrates through the clay) than the leachate in contact with the primary GM. The secondary GM is also further from the waste and hence is likely to be at a lower temperature than the primary liner if there is adequate thermal insulation between the primary and secondary GM (e.g. as may be provided by a CCL and granular LDS); however, it does have greater exposure to oxygen. In most other respects they are similar, and so the factors affecting the service life of both primary and secondary GMs will be discussed in this section.

Potential failure mechanisms

Given appropriate engineering design and CQC/CQA during the manufacture and installation, experience (e.g. Bonaparte *et al.*, 2002) has shown that GM liners may be expected to perform satisfactorily in the short term. However, the service life (i.e. long-term performance) of the GM is still uncertain, because it is influenced primarily by the synergistic effects of chemical and physical stresses over an extended period of time. Of particular relevance to this service life are the tensile strains (which can be induced by the overlying drainage material and wrinkles in the GM), the GM stress crack resistance, crystallinity, oxidative induction time (as a measure of the antioxidants in the GM), the exposure to chemicals in the leachate, and temperature.

Geomembrane protection

Typically GMs are designed from consideration of short-term puncture resistance. For example, Badu-Tweneboah *et al.* (1998) proposed a technique for assessing the effectiveness of protection layers for HDPE GM liners. Reddy *et al.* (1996) reported on a field trial of construction damage

D31/48

which concluded that 'a geotextile as light as 270 g/m^2... completely protects the GM from construction loading.' Narejo *et al.* (1996) showed that there is a linear increase in protection resistance with increasing thickness (mass per unit area) of the protection layer, and proposed a methodology for selection of geotextile protection layers that will provide short-term protection against puncture under the loads applied by the overlying waste. However, there are some who believe that protection against short-term puncture, although necessary, is not sufficient to ensure adequate long-term performance, and so the German approach (Bishop, 1996; Seeger & Müller, 1996) focuses on minimising the contribution of indentation (and subsequent potential stress cracking), due to stresses induced by the collection gravel under long-term loading conditions, to a very low strain level (~0·25%).

Tognon *et al.* (2000) developed a method for estimating the local strain in a GM due to the indentation of gravel particles. This approach, which was calibrated against direct strain measurements, considers the combined membrane and bending strains. It provides a better representation of the distribution of strain than techniques previously used (e.g. arch elongation), and enhances the evaluation of the peak strains in the GM caused by local indentations. Tognon *et al.* (2000) reported results from a series of large-scale tests conducted using different protection layers. It was found that the best protection for the underlying GM was provided by a sand filled geocushion ($M_A = 2130$ g/m^2) or a special rubber geomat with a polyester scrim ($M_A = 6000$ g/m^2), which limited strains induced by coarse (40–50 mm) angular gravel to 0·9% at 900 kPa and 1·2% at 600 kPa respectively and the number of indentations/m^2 to 78 and 38 respectively. The largest GM strains were observed when the nonwoven geotextiles were used with a maximum strain of 8% and 350 indentations/m^2 being obtained with a 435 g/m^2 geotextile at 250 kPa and a maximum strain of 13% and 338 indentations/m^2 with 1200 g/m^2 of geotextile at 900 kPa. In each case there was adequate protection to avoid holes in the GM in the short term; however, the short-term strain of 13% is very close to yield, and it is not clear what the long-term performance would be with these large, locally induced strains.

The large difference in maximum strains (7·5% and 1·2% respectively at a pressure of 600 kPa) observed for the two rubber geomats suggests that the tensile stiffness provided by the polyester grid in the second geomat played a significant role in reducing lateral deformation of the rubber and hence reducing indentation and strains in the GM. This suggests that the tensile stiffness of the protection layers may be a critical factor in minimising strains in GMs. Although consistent with previous findings, in that a combination of increased cushioning and tensile strength (both of which increase with increasing mass per unit area for needle-punched geotextiles) can be expected to be beneficial in providing GM protection, this does imply that additional benefits may be gained from the use of composite materials (such as the scrim-reinforced rubber mat) as an alternative to simply increasing the mass of nonwoven geotextiles in an attempt to improve GM protection.

The geomembrane strains reported by Tognon *et al.* (2000) were obtained from relatively short-term tests (200–720 min) and at room temperature (24 ± 1°C). Time dependence of the strains resulting from the local incursion of the gravel particles will be based on the deformation/relaxation characteristics of the polyethylene, protection layer, and the time-dependent behaviour of the underlying clay. Thus the peak short-term strain may not represent the maximum localised strain that could develop in longer-term tests, and further research is needed to clarify the time-dependent effects on the local strains caused by the gravel particles. Nevertheless, it is clear that the sand cushion and the scrim-reinforced rubber mat provided far superior protection and are likely to give smaller short- and long-term strains than the nonwoven geotextile layers examined. From a practical perspective, this suggests the desirability of a protection layer involving a sand protection layer (potentially in conjunction with a geotextile).

Wrinkles (waves)

Wrinkles were previously discussed in the context of their impact on increased leakage due to the easy distribution of fluid passing though any hole in the wrinkle. They may also contribute to the formation of holes because of the increased potential for stress cracking related to the tensile strains induced by the wrinkle and its interaction with any adjacent gravel. Brachman & Gudina (2002) have examined the interaction between the granular material and the wrinkle using specially designed apparatus. They found, when the wrinkle was in gravel, there was both pinching and flattening at the top of the GM, which gave rise to increased tensions in the GM. This reinforces the desirability of having a sand protection layer, which is of sufficient thickness to cover the wrinkles, between the gravel drainage layer and the underlying GM (e.g. a design as shown in Fig. 6(b)).

Service life of geomembranes

Given the relatively short history of GM use in waste containment applications, there are relatively few field data relating to the long-term performance of GMs. Brady *et al.* (1994) observed relatively little change in samples over a 30-year period, although the impact resistance changed after 15·5 years and had reduced by about 50% after 30 years. Rollin *et al.* (1994) found that HDPE GM used in a contaminated soil containment facility for 7 years exhibited lower tensile force and elongation at rupture than the original material. Hsuan *et al.* (1991) examined HDPE GM samples from a 7-year-old leachate lagoon and concluded that over the 7-year period there was no substantial change in the internal structure of the GM or its engineering/hydraulic containment properties. In contrast, Rowe *et al.* (1998, 2003) found that an exposed GM exhumed from a leachate lagoon liner after 14 years of operation had very low standard oxidative induction (OIT) values as well as low tensile break properties. The results of the Melt Index test suggest that the degradation was induced by a chain scission reaction in the polymers. The GM was severely cracked (a large portion of this cracking was due to improper maintenance activities on the lagoon), and was found to be highly susceptible to stress cracking (this case will be examined further in a later section). Geomembrane from the bottom of the same lagoon (i.e. that had been covered by leachate rather than being exposed to the elements) experienced slower depletion of antioxidant and appeared to have been sufficiently protected by the antioxidant package to prevent significant oxidation degradation over the 14-year period.

These observations are useful in that they indicate that reasonable GM performance might be expected for buried GMs for at least 15 years, if adequately protected from damage, but they are not sufficient to provide an estimate of long-term GM performance or the service life of HDPE GMs. Consequently, accelerated ageing tests have been developed in an effort to simulate long-term exposure of HDPE. Rowe & Sangam (2002) have provided a relatively recent review and discussion of basic concepts and mechanisms influencing the durability of HDPE and the service life of GM liners. This will not be repeated here other than to

D 32/48

highlight the basic concepts needed to present the most recent results relating to service life of HDPE GMs.

HDPE GMs typically comprise 96–97·5% of polyethylene resin, 2–3% of carbon black and 0·5–1·0% of other additives such as antioxidants and stabilisers (Hsuan & Koerner, 1998). Two key properties are the stress crack resistance (a function of the resin used) and the oxidative induction time (OIT: a measure of the amount and characteristics of the antioxidant package used in the GM). Common GM specifications (e.g. GRI-GM13) require both a minimum stress crack resistance and antioxidant package (OIT). Based on GRI-GM13, GMs used as liners should not fail in 300 h in a single-point notched constant-load test (SP-NCLT; ASTM D5397 (ASTM, 2005)) at 30% of the yield stress. They should also have a minimum OIT value of 100 min for the standard test (ASTM D3895 (ASTM, 2004a)) or 400 min in the high-pressure test (ASTM D5885 (ASTM, 2004b)). After oven-ageing at 85°C for 90 days (ASTM D5721 (ASTM, 2002)), the GM should retain 55% of the standard Std-OIT or 80% of the HP-OIT value. The assessment of service lives presented in this section assumes that the GM meets these criteria, and in fact the calculated service lives are based on test data for a GM with an initial OIT of about 135 min (ASTM D3895).

In a GM, the polyethylene chains may be neatly folded and tightly packed as crystal lamellae or in looser amorphous layers where the chains or chain segments are disordered (Apse, 1989). The lamellae are linked via tie molecules that start and end in adjacent lamellae and influence some of the polyethylene properties. Thus changes in the packing structure or molecules may affect the durability and long-term performance of the GM.

As discussed by Hsuan & Koerner (1998), physical and chemical ageing may take place simultaneously. With physical ageing, the polymer attempts to establish equilibrium from its as-manufactured non-equilibrium state. There is an increase in the crystallinity of the material but, by definition, there is no breaking of covalent bonds (Petermann *et al.*, 1976). Physical ageing may change crystallinity-dependent properties such as the diffusion coefficient, which gets lower (better) with higher crystallinity (Rowe *et al.*, 2003). With chemical ageing there will be bond scission in the backbone of the macromolecules, intermolecular cross-linking and/or chemical reactions in the side-chains (Schnabel, 1981) that will eventually lead to a decrease in mechanical properties and eventually to failure. Hence chemical ageing is of greatest interest with respect to long-term performance.

Chemical ageing is commonly envisaged as having three distinct stages (Viebke *et al.*, 1994; Hsuan & Koerner, 1998):

(*a*) depletion time of antioxidants
(*b*) induction time to the onset of polymer degradation
(*c*) degradation of the polymer to decrease some property (or properties) to an arbitrary level (e.g. to 50% of the original value).

Hsuan & Koerner (1998) note that the oxidation reaction of polyethylene can be increased in the presence of transition metals (e.g. Co, Mn, Cu, Pd and Fe). As these are all potentially present in leachate, it may be expected that GMs will age faster in leachate than in water (even in an anaerobic environment commonly associated with leachate).

It is not feasible to measure the length of these stages for actual field conditions because of the long time required to obtain useful results. Consequently, tests are conducted at elevated temperatures to accelerate ageing, and the results are extrapolated to temperatures expected at the base of a landfill (discussed earlier). This is believed to be reasonable provided that the fundamental mechanisms being evaluated (e.g. oxidation) are not altered by the elevated exposure.

Antioxidant depletion rates (Stage 1) have been examined by Hsuan & Koerner (1998) for a GM with water-saturated sand above, dry sand below, and subject to a stress of 260 kPa at temperatures of 55–85°C for up to 2 years. Mueller & Jacob (2003) reported results for air ageing and water bath ageing of HDPE GMs, and concluded that the service life 'is essentially determined by the slow loss of stabilisers due to migration.' Their paper reports tests involving nine (2·5 mm thick) HDPE geomembranes over a period of 13 years. They reported two-stage depletion of antioxidants, with the second stage being considerably slower and being noted only after two or three years. As they did tests at only one temperature (80°C), they could not produce an Arrhenius plot to establish the activation energy. However, they claim that the activation energy values for antioxidant depletion estimated by Hsuan & Koerner (1998) are too low and should be much higher.

Sangam & Rowe (2002) investigated the antioxidant depletion rates for unstressed specimens immersed in air, water and synthetic leachate (at 22–85°C). After different periods of immersion, samples were removed, tested to obtain the OIT, and then the ln(OIT) was plotted against the period of incubation (Fig. 22). The linear plot implies a first-order relationship between OIT and time, and hence the OIT (an indicator of the total amount of antioxidants) remaining at time t can be given by

$$\mathrm{OIT}(t) = \mathrm{OIT}_0 e^{-st} \tag{31}$$

where OIT_0 is the initial OIT value (typically in minutes) and s is the rate of antioxidant depletion (typically in month^{-1}). The antioxidant depletion rate s in leachate was 2·3 times faster than in water and 4 times faster than in air. The synthetic leachate (pH = 6) was highly reduced ($E_h \approx -328$ mV). The loss of antioxidants can in part be attributed to diffusion out of the GM into the fluid (extraction). However, as the same mechanism was also possible for water, this does not represent the complete explanation. The higher rate observed for leachate-immersed samples (relative to water-immersed samples) may be attributed to the transition metals and other inorganic compounds present in the leachate. Osawa & Saito (1978) and Wisse *et al.* (1990) have indicated that reaction with transition metals increases the consumption of antioxidant even in a relatively anoxic

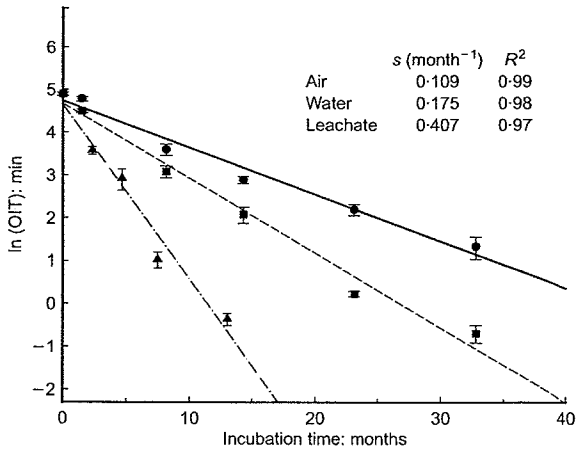


Fig. 22. Plot of ln(OIT) with incubation time at 85°C for different exposure conditions. The slope of the regression curves gives depletion rate s for a given exposure condition and temperature (modified from Sangam & Rowe, 2002)

D33/48

environment. Because of the low diffusion coefficient for metals in HDPE (August *et al.*, 1992), the reaction with the antioxidant may be expected to occur predominantly at the surface of the GM.

Assuming that diffusion, ageing and degradation are activated processes in which the activation energy E_a remains relatively constant, the time required to deplete the antioxidants at any given temperature (for the same exposure conditions) can be inferred using a time–temperature shift (Arrhenius modelling) based on the relationship (Hsuan & Koerner, 1998)

$$s = A \exp\left(\frac{-E_a}{RT}\right) \tag{32}$$

where s represents antioxidant depletion rate; T is the absolute temperature (K); A is a constant (often called the collisional factor), and is material-exposure system dependent; E_a is the activation energy (J/mol); and R is the universal gas constant (8·314 J/(mol K)).

The activation energy can be deduced from the antioxidant depletion rates obtained at different temperatures by plotting ln(s) against $1/T$ (Fig. 23) and then inferring parameters from a regression curve through the data (Fig. 23). Based on this information the antioxidant depletion time at any temperature of interest can be deduced by first establishing the depletion rate s from equation (32) and then calculating the time to deplete the antioxidant from the initial OIT value ($OIT_0 = 133$ min for this case) to the final (residual) value (taken to be 0·5 min when all antioxidant is depleted) from equation (31). The depletion times at temperatures between 10°C (potentially applicable to a secondary GM) and 60°C (upper end of likely range for a primary liner) so deduced are given in Table 5. As discussed earlier, primary liners are most likely to reach temperatures of 30–40°C, and 35°C is taken as a typical median temperature.

The results shown in Table 5 indicate a very large range of times required to deplete the antioxidants depending on the exposure condition, ranging between 50 and 510 years at 10°C and 10 to 80 years at 35°C. At 60°C, all depletion times are short (3–15 years). These results clearly demonstrate the critical importance of liner temperature on the service life of HDPE GMs. In addition they show that leachate exposure has a significant impact on antioxidant depletion times compared with both water and air immersion, and hence that it is essential to consider chemical exposure. However, they also show a need to realistically simulate the chemical exposure conditions (i.e. leachate on only one side of liner), because simple immersion of the GM in a chemical solution produces conditions too severe to characterise its lifetime in a landfill.

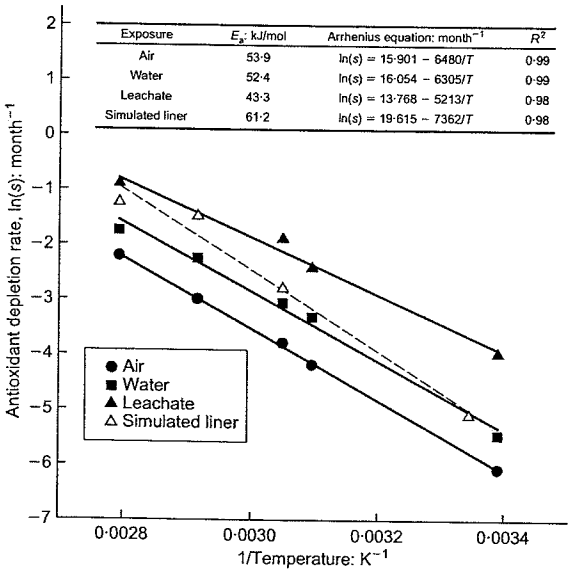


Fig. 23. Arrhenius plot of antioxidant depletion rate for different exposure conditions. Simulated liner with geotextile above and GCL and unsaturated sand below the geomembrane. E_a = activation energy (data for air, water and leachate from Sangam & Rowe, 2002)

Sangam & Rowe (2002) noted that a GM used in a primary composite liner for a landfill will perpetually have leachate on its top surface and soil (e.g. CCL or GCL) in contact with the bottom surface. They suggested that, in the absence of other data, the depletion time could be estimated based on leachate exposure on one side of the GM but considering the response to the contact with the underlining clay as being represented as an average of the exposure to air and water conditions. Based on this assumption the inferred liner depletion time t_{il} was calculated from the depletion times in air, t_{air}, water, t_{water}, and leachate, $t_{leachate}$, as follows:

$$t_{il} = 0{\cdot}25(t_{air} + t_{water}) + 0{\cdot}5t_{leachate} \tag{33}$$

and the resulting times (Table 5) are 210 years at 10°C and 35 years at 35°C. These are considered a better estimate of the likely field depletion time than those for air, water or leachate alone. However, they are only an approximation.

Recognising the limitations of immersion tests, a series of tests was initiated to better simulate the ageing of GM liners. These tests involve cells where the configuration consists (from bottom up) of an unsaturated sand subgrade, GCL, 1·5 mm thick HDPE GM, geotextile protection layer, and gravel. The gravel is saturated with leachate (which is changed regularly). These 'simulated liner' cells were placed in baths and maintained at temperatures of 26, 55, 70 and 85°C. The test cells were periodically terminated and the GM tested. Based on the OIT results the activation energy and relationship between depletion time and temperature were deduced (Fig. 23). The depletion times for the temperatures of interest for this simulated liner were then calculated, and are given in Table 5.

Compared with the simulated liner results (Table 5), the inferred liner depletion times (deduced from equation (33)) are conservative at 10°C (280 compared with 210 years) but similar at 35°C (both 35 years) and 60°C (6 and 7 years). However, this agreement should be viewed with some caution. Although the GMs used in the simulated liner tests and the immersion tests (used to get the inferred liner results) had very similar initial OIT values (135 min and 133 min), they did have different crystallinity (49% and 44%) and thickness (1·5 mm and 2 mm). Of these, the most significant difference is the thickness.

One would expect the depletion time for the 2 mm thick GM to be greater than that for 1·5 mm (assuming that depletion is predominantly diffusion controlled): thus the fact that the inferred results obtained for the 2 mm GM are less than those for the 1·5 mm simulated liner suggests that this approach is likely more conservative than appears from a direct comparison of the numbers in Table 5. This is consistent with the suggestion by Sangam & Rowe (2002) that the approach given by equation (33) is conservative. There are several reasons for conservatism. First, for a predominantly diffusion-controlled process, averaging of times for immersed condition to estimate times for different exposure conditions on each side is likely to underestimate the depletion time for the mixed condition. Second, in the

D34/48

Table 5. Estimated antioxidant depletion time for an HDPE geomembrane

Temperature: °C	Air,* t_{air}: years	Water,* t_{water}: years	Leachate,* $t_{leachate}$: years	Inferred liner,† t_{il}: years	Simulated liner,‡ t_{sl}: years
10	510	235	50	210	280
20	235	110	25	100	115
30	110	55	15	50	50
35	80	40	10	35	35
40	55	30	8	25	25
50	30	15	5	15	10
60	15	8	3	7	6

All times greater than 10 have been rounded to nearest 5 years.
*2 mm HDPE, $OIT_0 = 133$ min (ASTM D3895 (ASTM, 2004a)), crystallinity = 44%; based on data from Sangam & Rowe (2002; Fig. 23).
†Inferred from immersion data, $t = 0{\cdot}25(t_{air} + t_{water}) + 0{\cdot}5t_{leachate}$.
‡1·5 mm HDPE, $OIT_0 = 135$ min (ASTM D3895 (ASTM, 2004a)), crystallinity = 49%.

immersion tests, the leachate strength was essentially constant because the leachate was regularly replaced, and so the exposure was essentially constant over the testing period. In the simulated liner tests (or field) the leachate above the GM is regularly changed, but the soil below is not changed, and hence one can expect a build-up of antioxidant in the soil at the interface, which would slow diffusion of antioxidant to this side of the GM.

The simulated liner results presented in Table 5 represent the best information on depletion time of antioxidants currently available for GM liners in landfills. However, as discussed earlier, this represents only the first stage of the service life, and although these numbers give a likely lower bound to the expected service life, a more realistic estimate should also consider Stages 2 and 3. At present there are very limited data for the other stages of ageing. Karlsson *et al.* (1992) and Gedde *et al.* (1994) have examined medium-density (934 kg/m^3) polyethylene gas pipe with a wall thickness comparable to that of a GM (2·2 mm). Their tests involved having stagnant water inside the pipe and circulating air around the outside for samples aged at a range of temperatures between 70°C and 105°C. Karlsson *et al.* (1992) found that there was a much greater loss of antioxidants near the wall exposed to water than near the wall exposed to air, and as a consequence identified an asymmetric antioxidant diffusion profile through the wall of the pipe. Failure of the pipes was generally initiated from oxidised zones on the inside (water side) of the pipe, and they suggested (based on limited data) that the chain scissions occur randomly in the amorphous phase. They found that oxidation was accompanied by an increase in mass crystallinity and a significant decrease (30%) in molar mass.

Viebke *et al.* (1994) published data relating to both the induction (Stage 2) and polymer degradation (Stage 3) stages of degradation for polyethylene gas pipe with minimal antioxidant and a wall thickness comparable to a GM thickness (2·1 mm). Their tests involved having stagnant deionised water inside the pipe and circulating air around the outside for samples aged at temperatures of 70, 80, 95 and 105°C. For the induction stage (Stage 2), an activation energy of 75 kJ/mol was deduced, and for the degradation stage (Stage 3) an activation energy of 80 kJ/mol was inferred. Based on these values, estimated times t_V for Stage 2 can be calculated for this air/water system (Table 6, column 3). Recognising that interaction with leachate may also influence degradation, a second estimate may be obtained by adjusting based on the difference in the effect observed in the test conducted by Sangam & Rowe (2002). It is considered that a lower-bound estimate could be obtained by:

(*a*) calculating an air/water time, $t_{a/w}$:

$$t_{a/w} = 0{\cdot}5(t_{air} + t_{water}) \quad (34)$$

from results given in Table 5

(*b*) calculating t_{il} for the inferred liner from equation (33)

(*c*) calculating the adjusted Stage 2 time t_{Va} from

$$t_{Va} = t_V \times t_{il}/t_{a/w} \quad (35)$$

and the corresponding value of t_{Va} is given in Table 6 (column 4). It should be noted that these times are relatively short, and less than the 25 years inferred by Bonaparte *et al.* (2002) based on a comparison of the properties of HDPE milk and water containers recovered from landfills after 25 years relative to new containers (assuming that they had the same polymer resin and manufacturing process). Although these containers were probably not immersed in leachate (as adjacent newspapers could still be read) they likely were at typical landfill temperature, and so suggest that the estimates in Table 6 (columns 3 and 4) may be conservative.

As noted earlier, Viebke *et al.* (1994) reported activation energy for Stage 3 of 80 kJ/mol (although they caution that this should be considered a 'rough estimate'). Using this and a half-life at 115°C of 90 days, as reported by Hsuan & Guan (1998) and Bonaparte *et al.* (2002), one can calculate Stage 3 half-lives at different temperatures t_B, as given in column 5 of Table 6. These may then be adjusted for the possible effect of leachate exactly as described above for Stage 2, and hence the adjusted values, t_{Ba}, are calculated from

$$t_{Ba} = t_B \times t_{il}/t_{a/w} \quad (36)$$

and are given in column 6 of Table 6. The service life estimates can then be obtained by summing the Stage 1, 2 and 3 contributions, and are given in column 7 (unadjusted) and 8 (adjusted) of Table 6.

A number of observations can be made based on the numbers presented in Table 6. First, the estimated service lives of secondary GMs at temperatures in the range 10–20°C are between 565 and 2775 years depending on the assumptions. This is a large range, but they are also large times, and suggest that secondary GM liners are likely to have a service life of 600 years or more if maintained at a temperature less than 20°C. Second, at a 'typical' primary liner temperature of 35°C the service life is of the order of 130–190 years (median 160 years). Finally, at temperatures of 50–60°C the service lives are very short (15–50 years). The foregoing assumes leachate on one side of the liner. In areas where there is a hole in the GM, and there is leachate both above and below the GM, the service life may be less than indicated in Table 6. Also, for secondary liners the

D35/46

Table 6. Estimated times for three stages of degradation and resulting service lives for different methods of calculation for an HDPE geomembrane (All times greater than 10 have been rounded to nearest 5 years. Because of rounding, numbers may not add up exactly)

(1) Temp: °C	(2) Stage 1: years Simulated,* t_{siml}	(3) Stage 2: years Base,† t_V	(4) Stage 2: years Adjusted,‡ t_{Va}	(5) Stage 3: years Base,§ t_B	(6) Stage 3: years Adjusted,¶ t_{Ba}	(7) Service life: years Unadjusted,** t_{SL}	(8) Service life: years Adjusted,†† t_{SLa}
10	280	50	30	2445	1380	2775	1690
20	115	15	10	765	440	900	565
30	50	6	4	260	150	315	205
35	35	4	2	155	90	190	130
40	25	2	1	95	55	120	80
50	10	1	0·6	35	20	50	35
60	6	0·4	0·3	15	9	20	15

*Based on simulated liner antioxidant depletion tests (Table 5).
†Calculated using data from Viebke *et al.* (1994) for 2·1 mm wall thickness pipe with water inside and air outside.
‡As per previous note, but adjusted for possible effect of leachate using equation (35) and data from Table 5.
§Calculated using activation energy from Viebke *et al.* (1994) for 2·1 mm wall thickness pipe with water inside and air outside and half-life of 90 days at 115°C from Bonaparte *et al.* (2002).
¶As per previous note, but adjusted for possible effect of leachate using equation (36) and data from Table 5.
**$t_{SL} = t_{siml} + t_V + t_B$.
††$t_{SLa} = t_{siml} + t_{Va} + t_{Ba}$.

temperature will depend on the thermal insulation provided by the material between the primary and secondary GM liner. In the case of double composite liner systems involving just a GM and GCL as the primary liner (i.e. no foundation layer between the GCL and LDS), unpublished data indicate that the temperature of the secondary GM may be only about 3°C less than that of the primary GM (Legge, pers. comm.), and under these circumstances the service life of the secondary GM will be substantially less than the values discussed above and much closer to that of the primary GM.

The service lives presented in Table 6 are useful for obtaining a general idea of the order of magnitude of the GM service life, and for highlighting the importance of liner temperature. However, they should be used with considerable caution. Only the results for Stage 1 are based on actual tests on GM typically used in landfill application in a simulated liner configuration. The uncertainty regarding the Stage 2 and 3 components highlights the need for additional testing to examine their contributions to service life. This testing is under way, but is very time consuming.

The calculated antioxidant depletion times (Table 5) and service lives (Table 6) discussed above all assume a constant temperature. However, the temperature of the liner is likely to vary with time (e.g. Figs 11–13), and this will influence both the antioxidant depletion time and the service life. As the parameters for the antioxidant depletion times are the most reliable for a simulated liner (Table 5), the significance of variable temperature will be illustrated only with respect to the data obtained for this condition in Stage 1. Consider an idealised variation in temperature with time, with a lag period ($0 < t < t_1$) prior to temperature increase (e.g. see Fig. 11, 'Dry cell'), a period where the temperature increases linearly from T_0 to a maximum value T_p ($t_1 < t < t_2$: e.g. see Fig. 11, 'Dry cell'), a period where temperature remains constant at the maximum, T_p ($t_2 < t < t_3$: e.g. see Fig. 12(a)), a period when the temperature decreases linearly with time from T_p to T_0 ($t_3 < t < t_4$), and finally a period ($t_4 < t$) when the temperature is back to its original value, T_0. The variation in OIT with time can be modelled using equations (31) and (32), where temperature T is a function of time for different assumed values of the times t_1–t_4 and temperatures T_0 and T_p. The times t_1 and t_2 can be estimated from data such as those given in Figs 11–13. There is much more uncertainty regarding times t_3 and t_4. More data are needed to define the temperature history of landfills, and unfortunately this can only be realised in real time; however, numerical examples can provide some insight regarding the potential significance of the temperature history for the time at which the antioxidants are depleted. The results for several cases are summarised in Table 7.

For modelling the depletion that occurs during the period when the temperature is increasing, the average depletion rate s_{av} can be used, where it is given by

$$s_{av} = \frac{\int_{t_1}^{t_2} A \exp\left\{\frac{-E_a}{R[T_0 + \xi(t - t_1)]}\right\} dt}{t_2 - t_1} \quad (37)$$

where ξ is the rate of temperature increase with time ($\xi = (T_p - T_0)/(t_2 - t_1)$), T_0 and T_p are in K, and all other terms are as previously defined. Likewise, when temperature is decreasing it is given by

$$s_{av} = \frac{\int_{3}^{t_4} A \exp\left\{\frac{-E_a}{R[T_p + \xi(t - t_3)]}\right\} dt}{t_4 - t_3} \quad (38)$$

where ξ is the rate of temperature decrease with time ($\xi = (T_p - T_0)/(t_4 - t_3)$), and all other terms are as previously defined.

Referring to Table 7, Cases 1–4 all correspond to an initial increase in temperature from $T_0 = 10$°C to $T_p = 35$°C after 8 years but different times at the peak temperature and times for the decrease to occur. Assuming only a short, 12-year period at peak temperature and a decrease over a decade (Case 1), the antioxidant depletion time is 150 years. For otherwise similar conditions but a 30-year period over which the temperature decreases, the depletion time drops to 100 years (Case 2). However, longer periods of time at the peak concentration have a profound effect, with the depletion time dropping to about 40 years for Cases 3 and 4 where the liner is at 35°C for 32 years. Cases 5–10 similarly illustrate the significance of the time period at the peak concentration for the time to antioxidant depletion. Cases 11–13 illustrate the potential effects of leachate recirculation on the time to antioxidant depletion, and show a value of 25 years (Case 11) assuming the peak temperature is maintained for only 10 years and reduces to the baseline value over the following decade, and 20 years (Cases 12 and 13)

D36/48

Table 7. Examination of the effect of liner temperature history on the time to antioxidant depletion. Geomembrane is assumed to have $OIT_0 = 135$ min and a depletion rate defined by parameters given for the simulated liner in Fig. 23. All times have been rounded to nearest 5 years

Case	t_1: years	t_2: years	t_3: years	t_4: years	T_0: °C	T_p: °C	Time to OIT depletion: years
1	0	8	20	30	10*	35*	150
2	0	8	20	50	10*	35*	100
3	0	8	40	50	10*	35*	40
4	0	8	40	70	10*	35*	40
5	8†	14†	20	50	10†	35†	145
6	8†	14†	40	70	10†	35†	45
7	6‡	10‡	20	50	20‡	35‡	60
8	6‡	10‡	40	70	20‡	35‡	40
9	3§	7§	20	50	15§	37§	50
10	3§	7§	40	70	15§	37§	35
11	0¶	4¶	14	24	20¶	45¶	25
12	0¶	4¶	20	30	20¶	45¶	20
13	0¶	4¶	30	40	20¶	45¶	20

* Idealised based on Lysimeter 7, Fig. 12.
† Idealised based on Fig. 13.
‡ Idealised based on dry cell, Fig. 11.
§ Idealised based on Lysimeter 3, Fig. 12.
¶ Idealised based on wet cell, Fig. 11.

when the temperature remains at peak for 16 years. The difference of only 6 years at the peak temperature reduces the time to antioxidant depletion from 25 years to less than 20 years (18 years), which is within the operating life of many landfills. These results highlight the importance of considering the mode of operation when developing a liner design. Thus if leachate recirculation is envisaged, then consideration should be given both to enhancing the LCSs design (as discussed earlier) and to having a double liner system where the secondary GM is thermally separated from the primary liner. The thermal separation is required to ensure that the secondary GM is at a much lower temperature than the primary GM so that it will have an adequate service life. This is particularly important for containing conservative contaminants that do not degrade as a result of the enhanced biological activity induced by recirculation of leachate, and for operating the landfill as a bioreactor.

SERVICE LIFE OF CLAY LINERS

The clay liner beneath a GM is intended to substantially decrease leakage through any holes in the GM, to act as a partial diffusion barrier, and to provide attenuation of certain (e.g. organic and metal) contaminants. To do so, the bulk hydraulic conductivity must remain less than or equal to the design value for the contaminating lifespan of the landfill.

Provided that a CCL has been properly designed and constructed, and provided that appropriate account has been taken of the potential for an increase in hydraulic conductivity due to clay–leachate interaction (Rowe *et al.*, 2004), the CCL can be expected to have a hydraulic conductivity below the specified design value for thousands of years provided that it is not allowed to desiccate after placement. Desiccation will be discussed in the following subsections.

A GCL used as part of a composite base liner system may also be expected to have a service life of thousands of years provided that (Rowe *et al.*, 2004):

(*a*) there is no significant loss or movement of bentonite from the GCL during placement. Loss into underlying drainage layers is of particular concern, and a filter may be required to avoid bentonite loss for some GCLs (e.g. Estornell & Daniel, 1992). Construction procedures should be selected to ensure a uniform distribution of the bentonite in the GCL after placement. The potential for bentonite movement may vary from one GCL product to another. GCLs are not all the same, and the construction specifications should recognise this fact.

(*b*) there is no significant lateral movement (thinning) of bentonite during and following hydration that would cause an uneven distribution of the bentonite in the GCL during the contaminating life of the landfill (e.g. Stark, 1998). Particular care is needed on side slopes (especially steep side slopes) to avoid bentonite migration downslope, in both the 'dry' and hydrated states.

(*c*) the choice of GCL and the design are such that there is no significant long-term loss of bentonite due to migration (internal erosion) through the GCL under the hydraulic gradients that may occur either during or after termination of the operation of the LCS (Rowe & Orsini, 2003).

(*d*) the geosynthetic component of the GCL is not critical to the long-term performance of the bentonite component of the GCL (otherwise the service life of the GCL is controlled by that of the geosynthetic component).

(*e*) the seams between GCL sheets are installed to ensure intimate contact and adequate overlap. The design and the construction procedures should be such that the seams do not open up prior to, during, or following waste placement (e.g. due to drying, shear induced by construction equipment, or differential settlement). Some GCL products with an initial moisture content greater than 20% have been found to experience opening up (by as much as 300 mm) of the seams between sheets of GCL on side slopes and hence a loss of composite action (Thiel & Richardson, 2005). This problem has been attributed, at least in part, to shrinkage of these GCLs with high initial moisture content after placement. There does not appear to have been any problem with products having initial moisture less than about 15%. However, even for these GCLs care is needed to ensure adequate overlap during construction, to ensure that there is appropriate moisture in the underlying subgrade, and to have the GM and GCL covered and protected from thermal gradients due to exposure to solar radiation.

(*f*) the design hydraulic conductivity is based on considerations of clay–leachate compatibility, considering

D37/48

hydrating conditions, groundwater geochemistry, applied stress, and GCL and leachate characteristics (Rowe, 1998a; Rowe *et al.*, 2004).

(*g*) the GCL does not desiccate. This is the focus of the following discussion.

Desiccation can be related to a change in water content in the clay that could occur:

(*a*) after construction of the CCL and before placing the GM
(*b*) after placing the GM and before covering with waste
(*c*) after placement of waste.

These will be discussed below.

Desiccation before placement of waste

Unless a CCL is kept moist, desiccation can occur within hours of placement. Factors influencing this desiccation include the grain size distribution of the liner material, mineralogy, water content, the climatic conditions (especially temperature and wind), and the length of exposure to drying conditions. Special care is required when the CCL is covered with a GM, because the temperature of a GM exposed to the sun may reach 80°C (Felon *et al.*, 1992), and temperatures of 60–70°C are common. White-coated GMs have been used to reduce temperature, but even in this case temperatures close to 60°C have been reported on a hot (30°C ambient) day (Koerner & Koerner, 1995a).

When a GM is heated there is potential for:

(*a*) evaporation of water from the underlying CCL into any air space between the clay and the GM
(*b*) movement of evaporated moisture downslope upon cooling of the GM
(*c*) movement of moisture from the region of higher temperature to the region of lower temperature.

Mechanisms (*a*) and (*b*) are particularly problematic on side slopes, and have been found to result in desiccation cracking of the clay liner (e.g. Basnett & Bruner, 1993). Mechanism (*c*) is critical when there is a sustained temperature gradient (Döll, 1997), and will be discussed in more detail in the following subsection. Both mechanisms decrease the water content in the clay below the GM, which, in turn, can cause shrinkage and consequent desiccation. Corser *et al.* (1992) examined a test liner/cover left exposed for six months (April–October; temperature of GM reaching 43°C). Where the GM was in intimate contact with the clay there was no significant desiccation. However, in areas where there was an air space between the GM and clay (i.e. at wrinkles/waves) the clay had dried and cracked, with cracks up to 6 mm wide extending to a depth of about 75 mm.

The desiccation of CCLs prior to waste placement can be prevented by adopting appropriate construction procedures. In particular it is essential to cover the GM with a suitable thermal insulation layer (e.g. the protection layer, leachate collection layer and a layer of waste) as soon as possible after placement of the GM.

Desiccation after placement of the waste

The risk of desiccation of clay liners that have been covered, and are no longer subject to solar heating or freezing, depends on five key factors (Holzlöhner, 1995): the properties of the clay liner, the properties of the underlying subgrade, the overburden pressure, the temperature gradient across the liner, and the depth to the water table. There are limited data regarding the desiccation potential of CCLs and GCLs under thermal gradients. However, Southen and Rowe (2004, 2005b) have experimentally investigated the potential for moisture redistribution for geosynthetic clay liners forming part of a composite liner system when subjected to thermal gradients, based on both small (Southen & Rowe, 2004) and large-scale (Southen & Rowe, 2005b) experiments. In particular, the large-scale laboratory testing examined two different subsoils and two GCL materials. Both the spatial and temporal variation in temperature and water content within and beneath the GCLs were monitored. The effects of applied temperature gradient, initial GCL and subsoil water content, and the type of GCL were studied. The following sections describe the modelling of the thermo-hydro-mechanical behaviour of soils and some of the findings from both the experimental work and this modelling.

Numerical modelling of liner desiccation: theory

Of the numerical models that have been developed to describe the thermal behaviour and moisture movement in soils, two were directed at applications involving landfill liners, and will be discussed here. Döll (1997) developed the SUMMIT program using temperature and capillary pressure (suction) as the basic variables for the analysis of thermally driven moisture movement, subject to the assumptions of a rigid, heterogeneous, unsaturated medium. Southen & Rowe (2005a) used Döll's SUMMIT model to analyse their large-scale laboratory experiments, and found that the predicted variations in water content throughout the subsoil were in reasonable agreement with experimental results. The predictions of temperature in the subsoil were less accurate, but still in reasonable agreement with experimental results. However, the SUMMIT model underestimated the volumetric water content in the GCL by as much as a factor of two, and the temperature gradient through the GCL was consistently underpredicted. The poor predictions for the GCL were attributed primarily to the assumption of a rigid medium.

To address the shortcomings of the SUMMIT model, Zhou & Rowe (2003) developed a model that incorporates consideration of fully coupled heat–moisture–air flow, a non-linear constitutive relationship, the dependence of void ratio and volumetric water content on stress, capillary pressure and temperature, and the effect of mechanical deformation on all governing equations. They also proposed a mass conservative numerical scheme to improve the accuracy of the finite element solution to the governing equations.

Numerical modelling of liner desiccation: findings

Heibrock (1997) used Döll's (1997) SUMMIT model to numerically examine composite liners and from that to infer the potential for cracking of CCLs. In one example, cracking was predicted to a depth of 1 m in 20 years for a CCL on a silt loam where the temperature at the top of the liner was 40°C. However, in a second example there was no cracking predicted for a liner on loess with a much higher unsaturated hydraulic conductivity than the silt loam. In this case the calculated downward vapour flux of water was much smaller than the upward liquid flux from the water table induced by suction, and there was no drying.

Using the theoretical framework described in the previous section, Southen (2005) modelled the behaviour of the large-scale laboratory tests. To illustrate the results, consideration will be given here to a few aspects of one test involving a GM over a needle-punched and thermally treated GCL (4240 g/m^2 granular sodium bentonite between a 105 g/m^2 slit-film woven and a 200 g/m^2 needle-punched, nonwoven geotextile). The GCL rested on 1 m of silty sand (19% silt content, optimum water content 12%, and maximum dry

density 1·95 Mg/m^3). The sand and GCL were placed with volumetric water contents of 7·3% and 71% respectively, and a pressure of 50 kPa was applied. The test was left at room temperature (21°C) for 50 days to equilibrate. The temperature on the liner was then raised to, and maintained at, 56°C for 232 days, after which the test was terminated (at day 282).

Figure 24 shows the variation in volumetric water content with depth initially, after the 50-day pre-heat period, and after 232 days of heating (i.e. at test termination). During the isothermal pre-heating stage the GCL took up water from the underlying subsoil and approached saturation at the start of the heating phase. There was moisture redistribution within the subsoil under gravitational forces, such that the volumetric water content decreased by 0·7–2% in the upper portion of the subsoil and increased by 0·5–1·5% at the base. At the end of the test (day 282) the volumetric water content of the GCL had reduced to 9·5% (from 71%), while in the subsoil adjacent to the GCL the volumetric water content had reduced to the residual value of 0·5%. The volumetric water contents predicted by the numerical model fit the general trends observed in the experimental data, and agreement is especially good within the GCL and in the upper 5 cm of the subsoil.

The variation of capillary pressure is tied to the water content distribution, and hence large changes in capillary pressure were calculated, ranging from approximately 60 000 kPa at the GCL interface to approximately 50 kPa at a depth of 0·1 m. The downward migration of water vapour leads to a decrease in unsaturated hydraulic conductivity by several orders of magnitude, and this inhibits the upward transport of liquid water needed to offset the downward vapour transport. The temperature distribution was reasonably modelled. One of the key advantages of the Zhou & Rowe (2003) model over Döll's (1997) SUMMIT model is its ability to model deformable media and calculate stress changes arising from the thermo-hydro-mechanical response. The observed GCL void ratio decreased from 2·62 to 1·76, and the calculated final void ratio (1·78) was in very good agreement with that observed. The calculated variation of net total horizontal stress with depth is shown in Fig. 25. The capillary pressure had very little effect on predicted horizontal stress within the subsoil, because the compressibility of the silty sand with respect to changes in capillary pressure was very low. However, for the highly compressible GCL, the high capillary pressures caused the development of predicted tensile horizontal stresses within the GCL. Air pressure p_a was atmospheric at the beginning and end of the tests, such that the net total stresses were equal to the total stresses at these times. There was some transient variation in air pressure, but the effects were minor. The predicted desiccation cracking of the GCL was confirmed by physical observations.

Southen (2005) conducted a parametric study to evaluate the effect of applied temperature, overburden stress, depth to the aquifer, foundation layer and, for a CCL, liner thickness on the potential for desiccation cracking. This, combined with the experimental data (Southen & Rowe, 2004, 2005b), led to a number of tentative conclusions, although it should be emphasised that more work needs to be done on this challenging topic, especially on the determination of relevant soil parameters.

For both types of composite liner system (GM/GCL and GM/CCL), liner temperature was a key factor affecting the potential desiccation. For single composite liners involving a GCL, it was found that:

(*a*) the properties of the foundation layer underlying the GCL had a critical influence on the potential for desiccation. The unsaturated soil characteristics were important as well as the initial water content. Other things being equal, the higher the initial water content of the foundation soils (up to optimum water content) the better.

(*b*) the higher the overburden stress at the time of GCL hydration, the less the risk of desiccation. This implies that the waste should be placed over the composite liner as quickly as possible after the liner is placed to minimise the potential for both short-term (e.g. solar induced) and long-term (waste temperature induced) desiccation cracking.

(*c*) the risk of desiccation increases with increasing distance to the underlying water table. However, for the cases examined, the offsetting effects of reduced water content and temperature gradient limited the effect of aquifer depth, and there was no significant increase in risk of desiccation for aquifer depths greater than about 5 m below the GCL.

For single composite liners involving a CCL, it was found that:

(*a*) the properties of the liner had a significant effect on the distribution of moisture and stress. The properties of the underlying materials were of relatively less importance, although they still had some impact.

(*b*) although an increased overburden stress reduced the

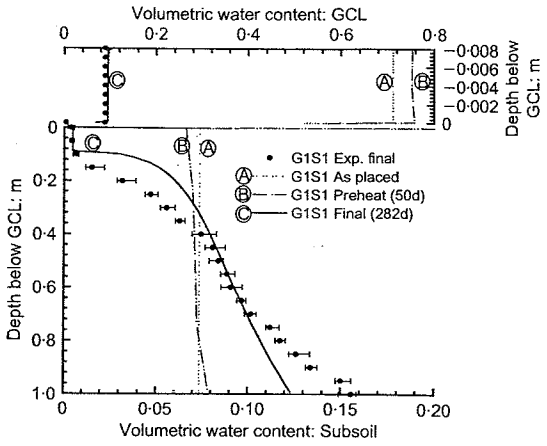


Fig. 24. Calculated and observed volumetric water content (–) profiles

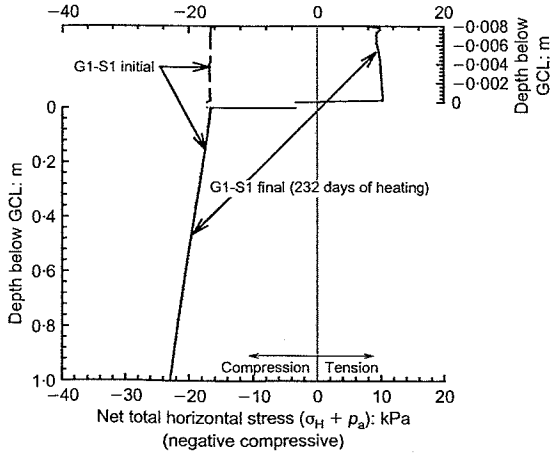


Fig. 25. Net total horizontal stress profiles. σ_H is total stress (tension positive), p_a is air pressure

D39/48

risk of desiccation, it was not as significant as for a GCL.

(*c*) the depth to the underlying aquifer did not significantly affect the final (equilibrium) distribution of moisture within the CCL, but did increase the potential to develop transient peak net horizontal tensile stresses.

(*d*) the thickness of the CCL had a minor effect on predicted water content and stress distributions at equilibrium, but thicker liners were found to have significantly higher peak net horizontal tensile stresses.

To date, the potential for long-term desiccation of clay liners making up part of a composite liner has received relatively little attention in the design of landfills. The work discussed above shows that there is real potential for desiccation, but also suggests that this can be mitigated by appropriate design and construction. This is an area where more research is needed.

LONG-TERM CONTAMINANT IMPACT

Field examples for composite liners

Rowe *et al.* (2003) examined the performance of a composite liner involving a 1·5 mm HDPE GM over a 3 m thick CCL after 14 years of use as a leachate lagoon liner. The GM had no overlying protection layer. The OIT values obtained for the GM were all very low (typically less than 7 min). For the part of the GM exposed to the sun over the 14-year period, the OIT was only about 1 min, implying that the antioxidants originally in the GM (the initial OIT value is unknown) were almost completely depleted. The low measured tensile break properties and stress cracking resistance suggest that some oxidation had already occurred. When inspected at decommissioning, the GM had 82 cracks, holes and patches over an area of 1552 m^2, corresponding to 528 defects per hectare that had developed within 14 years of operation. Of these, 70% (348 defects/ha) were above the leachate level and 30% (180 defects/ha) were below the leachate level. Despite the fact that only 7% of the unrepaired holes were below the leachate level on the side slopes, this was sufficient to allow liquid to get between the GM and clay liner. No cracks or holes were found at the bottom of the lagoon at the time of decommissioning, although 6 of the 54 patches were on the bottom. Thus although there was external evidence that leachate had come into contact with the underlying clay liner, there remained the questions as to how long the GM had served as an effective barrier and to what extent contaminant had migrated through the CCL.

Soil samples taken from the liner at five different locations were analysed and contaminant concentration profiles were established, as shown for chloride in Fig. 26. The data from the five locations were quite consistent, suggesting that once leachate reached the clay liner it quickly spread over the entire base. The back-diffusion profile at the top was a result of water in the lagoon following removal of leachate but before decommissioning. Data show that chloride had migrated to a depth of 1·7 m (but did not have any adverse impact on the groundwater, as the CCL was 3 m thick). The clay hydraulic conductivity was about $2{\cdot}2 \times 10^{-10}$ m/s. Based on this and the possible gradient, the maximum advective transport was about 0·4 m assuming that the GM failed immediately after construction. Thus the fact that chloride had migrated 1·7 m implies that diffusion was the dominant transport mechanism. The chloride diffusion coefficient for the clay was about 7×10^{-10} m^2/s.

Figure 26 shows calculated chloride profiles assuming the leachate was in direct contact with the compacted clay (i.e. the GM was ineffective) at the end of construction (time = 0 years) and at various times after construction (4, 6, 8 and 10 years). Also shown is the profile expected with 2·5 holes/ha (GM in direct contact with the CCL). A comparison of the calculated and observed profiles shows a poor fit for 6, 8 and 10 years. However, assuming the GM functioned effectively for elapsed times of 0 and 4 years provides a reasonable fit to the majority of the data. Thus it appears that the GM ceased functioning effectively somewhere between 0 and 4 years after construction. This case highlights the need to have an adequate attenuation layer in the event of a failure of the GM, and the need to provide adequate protection to the GM. In this case the damage was attributed to a combination of inadequate design (no protection layer) and operating procedures that caused damage to the GM.

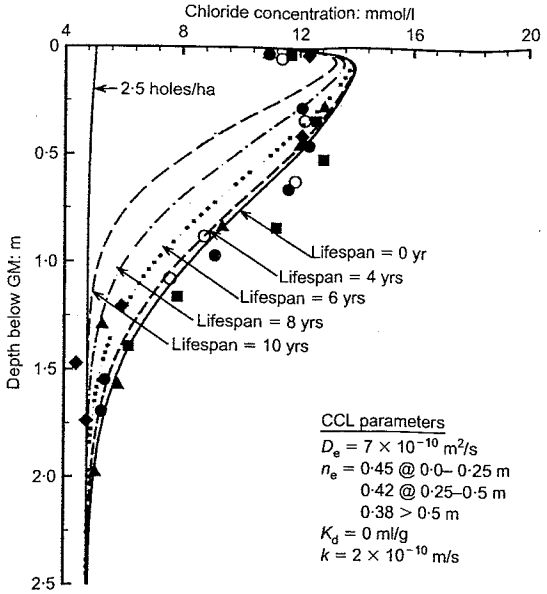


Fig. 26. Chloride concentration profile through compacted clay liner, based on samples from five boreholes, together with prediction of pore fluid concentration for different assumed geomembrane service lives (modified from Rowe *et al.*, 2003)

The foregoing example illustrates that the GM is only as good as the design and operations allow it to be. Although it also highlights the value of the clay component of a composite liner, it should not be concluded from the foregoing example that the GM plays a small to negligible role, if properly designed and protected. For example, a much more positive example of composite liner performance was first reported by Rowe *et al.* (1997) for two test sections at the KVL. For most of the KVL the liner is a CCL (minimum 1·2 m thick), and its performance was discussed earlier. At the locations discussed here the CCL is 2·4 m thick but otherwise as previously described. However, in addition to the normal CCL, a test section was prepared with a composite liner comprising a 2 mm GM over the 2·4 m CCL. In both the normal section (CCL only) and composite section (GM/CCL) the liner was covered with 0·3 m of fine to medium sand and 0·3 m of 50 mm gravel. The migration of ionic leachate constituents was monitored using 12 electrical conductivity sensors installed in the clay liner to a depth of about 0·8 m at each of the two sections. The sensors measure the electrical resistivity of the clay liner, which is a function of soil mineralogy, density, temperature and the concentration of ions in the clay pore water. Details of the method of measurement and the relationship between conductivity and concentration are given by Rowe *et al.* (1997). The increment in pore water conductivity required to give a

clear response from the sensors is 200 μS/cm, which corresponds to an increment in chloride concentration of about 25 mg/l. Rowe *et al.* (1997) reported that after about 5 years salt had migrated to detectable levels through the 0·3 m sand blanket and about 0·7 m into the clay liner where there was no GM. At the GM section there was no detectable migration below the GM.

At the time of writing, there are now 12 years of data (Fig. 27). A clear diffusion profile is evident for the section with compacted clay only, and this extends below the maximum depth of monitoring (0·8 m). Thus over the 12-year period ionic species have migrated more than 1·1 m (through 0·3 m of sand and more than 0·8 m of clay). Advective transport during this 12-year period is calculated to be less than 0·15 m. In contrast, at the composite lined section, even after 12 years, there is no evidence of a concentration profile for ionic species, and the measured conductivity is still representative of background values. This suggests that (a) there is negligible advective flow (leakage) through the GM near the conductivity sensors, and (b) there has been negligible diffusion of ionic species through the GM in 12 years. The lack of holes can be partly attributed to the 0·3 m sand protection layer between the GM and the coarse gravel (note that the same protection layer is used above the clay alone to minimise potential desiccation prior to waste placement). The lack of a diffusion profile after 12 years is entirely consistent with the findings from the laboratory tests (initiated at about the same time) discussed previously (Fig. 20).

Thus the direct comparison of the performance of a single CCL and composite liner at KVL for otherwise identical conditions clearly demonstrates the substantial better performance of the composite lined section in terms of minimising the migration of ionic contaminants and hence minimising long-term contaminant impact.

Factors influencing prediction of long-term contaminant impact

The evaluation of the suitability of a barrier system for a landfill should, in principle, be case dependent and should consider the potential impact on water quality (especially groundwater) due to the proposed landfill. To provide adequate environmental protection the combined engineered system and natural attenuation capacity of the site should be such that there will be negligible impact on groundwater quality for the entire contaminating lifespan of the landfill. As the contaminating lifespan depends on the nature the of waste, the mass of waste per unit area, the waste moisture content, and infiltration through the waste (Rowe *et al.*, 2004) a barrier system that may be perfectly suitable for one landfill will not necessarily be suitable for another (e.g. larger) landfill. In order to make a reasoned, quantitative assessment of potential impact (and hence the suitability of a proposed barrier system) it is usually necessary to solve the advection–dispersion equation subject to appropriate boundary conditions (see Rowe *et al.*, 2004 for a detailed discussion). Factors that influence the potential impact include:

(*a*) the landfill source concentration and its decay characteristics
(*b*) the advective flux (leakage) across the barrier system
(*c*) the thickness of any 'attenuation' layer between the base of the low-permeability liner(s) and the receptor aquifer (or distance to the water table if the liner is constructed in the unsaturated portion of the aquifer)
(*d*) diffusion (especially for VOCs) across the barrier system
(*e*) sorption/retardation in the liner and underlying soil
(*f*) biodegradation
(*g*) dilution in the aquifer
(*h*) the service life of the engineered components of the landfill (e.g. covers/caps, LCSs, liners)
(*i*) the mode of operation of the landfill
(*j*) consolidation of the liner and any underlying compressible soils.

Items (*a*)–(*i*) can be modelled using existing contaminant transport theory (Rowe & Booker, 1998; Rowe *et al.*, 2004) and codes (e.g. Rowe & Booker, 2005). However, item (*j*), the effect of liner consolidation, has received relatively little attention. Questions have been raised regarding the significance of the contribution of consolidation water to flows in leak detection systems (Bonaparte & Gross, 1993; Moo-Young *et al.*, 2004) and the effect of consolidation-induced advection on the rapid migration of VOC into leak detections systems, as reported by Workman (1993) and Othman *et al.* (1996). The issue of consolidation contributing to flows in LDS was discussed earlier. With respect to contaminant transport, Peters & Smith (2002) developed equations that account for the effects of consolidation on the transport of chemicals through clay liners. Results presented for a quasi-steady-state problem (with time-dependent porosity due to consolidation) suggested that consideration of consolidation has only a small effect on the concentration distribution and mass flux. These findings are consistent with those of Rowe & Nadarajah (1995), who found that, for typical liner properties, the change in flow due to consolidation was within the typical range of uncertainty concerning the hydraulic conductivity, and that the effect on contaminant transport and hence impact on a receptor aquifer was not significant for the cases examined. Rowe & Nadarajah (1995) also demonstrated that, to sufficient accuracy, the effect of the build-up of a leachate mound can be modelled as a sequence of steady-state flows without the need for a full consolidation analysis. Thus the effects of consolidation appear to be small for typical low-compressibility CCLs (e.g. Rowe *et al.*, 2000c), and hence conventional advective-diffusive contaminant transport modelling appears suitable for a wide range of practical cases. However, there may be situations involving highly compressible liners where consolidation-induced transport may be potentially significant, and a sophisticated one-dimensional, large-deformation model of coupled mechanical consolidation and solute transport has been developed to model these situations (Lewis, pers. comm).

Although there may be a clearly defined long-term trend

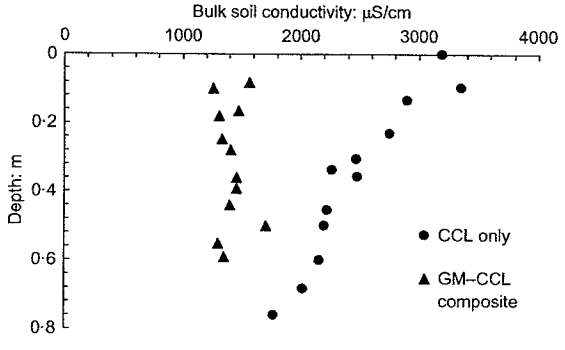


Fig. 27. Bulk soil conductivity profile through compacted clay liner at the Keele Valley Landfill after 12 years. Based on conductivity sensors in the compacted clay: ●, in a section of the landfill with a 2·4 m thick CCL; ▲, in an adjacent section with a trial composite liner comprising a 2 mm HDPE geomembrane over a 2·4 m thick CCL

in leachate characteristics, there is considerable variability in leachate characteristics around this long-term trend (Fig. 2). This raises the question of how important it may be to explicitly model the variability as opposed to the general trend. This issue was addressed by Rowe & Nadarajah (1996), who examined the effects of both cyclical (e.g. seasonal) and random variability about a mean concentration on contaminant transport through a clay liner. For example, Fig. 28 shows the calculated variation in chloride concentration with time at various depths in a clay liner for a leachate source that varies randomly about a mean of 2330 mg/l (standard deviation 620 mg/l; values based on KVL data for 1993). The solid lines represent the variation in concentration calculated using the mean concentration, and the dashed lines show those in response to the randomly varying source (Fig. 28(a)). The significant variability at the source is rapidly damped out by diffusion as the contaminant plume migrates through the clay (Figs 28(b) to 28(e)). The effect is already small by 0·25 m, minimal by 1 m, and negligible at 2 m.

The factors discussed in previous sections of this paper come together when one is faced with predicting the long-term impact of a landfill for a given barrier system. The service life of an LCS can be estimated and used as part of the input to a contaminant transport model. The temperature

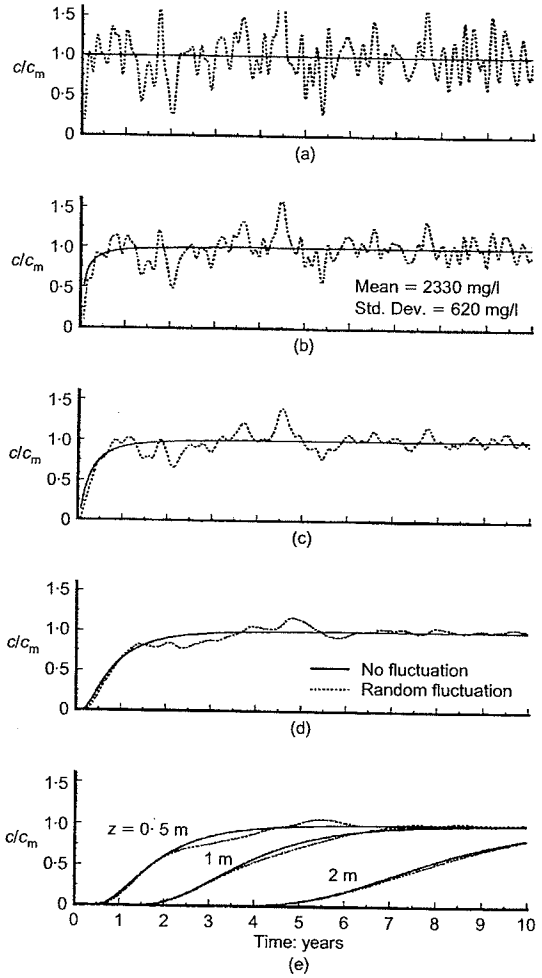


Fig. 28. Effect of random temporal fluctuations of the contaminant source on chloride concentrations c, relative to the mean source concentration c_m, at various depths in a clay liner (modified from Rowe & Nadarajah, 1996): (a) $z = 0$ m; (b) $z = 0{\cdot}05$ m; (c) $z = 0{\cdot}1$ m; (d) $z = 0{\cdot}25$ m; (e) $z = 0{\cdot}5$, 1 and 2 m

of the liner also needs to be considered in assessing suitable transport parameters, and in assessing the service life of the GM and any underlying clay liner. The leakage through the liner needs to be calculated, and the diffusion/sorption/permeation coefficients need to be appropriately selected, as discussed earlier. Finally one needs to evaluate the relative performance of different design options. In the following subsection the long-term impact is examined for both a single and a double composite-lined landfill system. Particular attention is focused on the question of equivalence of systems involving both a CCL and a GCL. The results to be presented were obtained using the program POLLUTE (Rowe & Booker, 2005). The theory upon which the modelling is based (Rowe *et al.*, 2004) allows consideration of the GM, clay liners and any natural aquitards and aquifers, changes in advective velocity, and diffusion properties at different times (e.g. the effects of leachate mounding, failure of a GM, or an increase in temperature can be modelled).

Equivalence of liner systems

There is growing interest in the use of GCLs as a replacement for conventional CCLs. Because, in many jurisdictions, regulations prescribe acceptable barrier system configurations in term of CCLs, this raises the question as to whether a liner involving a GCL is equivalent to one involving a CCL. Several people have attempted to address this question. For example, both Giroud *et al.* (1997a) and Richardson (1997) discussed this in the context of leakage through GCLs and CCLs and the steady-state advective travel time (i.e. neglecting the effect of diffusion on the first arrival time). Richardson also discussed chemical absorption capacity and dilution potential. However, to really assess equivalence from an environmental perspective it is necessary to assess the equivalence in terms of contaminant impact on a receptor aquifer beneath a landfill by solving the advection–dispersion equation. Rowe (1998a) provided a framework to model the contaminant transport through GM/CCL and GM/GCL composite liners considering items (*a*)–(*i*) of the previous subsection, and showed that the peak chloride and dichloromethane concentrations in an underlying aquifer were smaller for a GM/GCL composite liner than for a GM/CCL composite liner provided that the total distance between the GM and the aquifer was the same (i.e. the GCL was used with a more permeable soil that acted as an attenuation layer but not a low-permeability liner).

In selecting parameters for use in conjunction with a contaminant transport analysis, consideration should be given to:

(*a*) the potential for clay–leachate interaction and its effect on hydraulic conductivity (Rowe, 1998a)
(*b*) the interaction with the adjacent GM and the effect on leakage (as discussed earlier)
(*c*) diffusion and sorption (as discussed earlier)
(*d*) the leachate head and corresponding gradient (which will be affected by clogging of the LCS, as discussed earlier)
(*e*) the provision of appropriate protection to the GM and GCL to minimise potential squeezing and local thinning of the GCL (Stark, 1998; Dickinson & Brachman, 2003)
(*f*) the potential for desiccation and shrinkage of the GCL (as discussed earlier).

To illustrate the effect of both diffusion and the finite service lives of different components of the barrier system, consideration will now be given to contaminant transport through composite liner systems consisting of a GM on top of a CCL or GCL. The properties of the HDPE GM, soil

D 42/48

and landfill are as described by Rowe *et al.* (2004). The modelling was based on the following assumptions. The landfill operated for 20 years prior to closure, and all time is expressed relative to the middle of this period. Between times 0 and t_1, a low-permeability cover limited infiltration to 3×10^{-4} m^3/(m^2 yr), and the leachate head on the liner was 0·3 m. Operation of the LCS was terminated at time $t_1 = 40$ years, and the leachate head increased to a maximum of 12 m at time $t_2 = 60$ years. Between times 40 and 60 years an average 6 m head acts on the liner. After 40 years (t_1) the cover was no longer maintained, and the infiltration increased to 0·15 m /yr. The primary GM failed at time $t_3 = 160$ years (the median value at 35°C discussed earlier). For the double-lined system, the secondary GM failed at time t_4 and two values were considered: 300 and 400 years.

Single composite liners

The base case for the single composite liner system, shown schematically in Fig. 29, involved a GM, a 0·6 m CCL and a 0·5 m attenuation layer (GM/CCL/AL) above an aquifer. The second system involved a GM, GCL and 1·1 m attenuation layer (so that the distance from the GM to the receptor aquifer was identical for the two cases, as was discussed in the section on diffusion) (GM/GCL/AL). Fig. 29 shows the calculated dichloromethane (DCM) impacts on the underlying aquifer for these two liner systems. Also shown are the calculated impacts for two other similar systems, where the thickness of the clay liner and attenuation layers has been increased so that the distance from the GM to the aquifer is 3·75 m. In this diffusion-controlled system the peak impact decreases as the total thickness of the soil barrier increases. The impact for the 1·1 m thick system is less than the 50 μg/l limit imposed by some jurisdictions but well above the maximum acceptable concentration of 5 μg/l imposed in other jurisdictions. For the 3·75 m thick system the impact is below 5 μg/l. In both cases the GM/CCL/AL liner system gave a greater impact than that for GM/GCL/AL and similar total thickness, although the difference was of no practical significance, and the two systems may be regarded as equivalent from a contaminant impact perspective.

In all cases the peak impact occurred well before the GM failed ($t_3 = 160$ years), and is controlled by diffusion through the GM and underlying soil. In this particular case, the service life of the primary GM does not have any effect on DCM impact provided it exceeds 100 years. As the concentration of DCM in the landfill is decreasing faster than in the soil (because of the greater biodegradation in the landfill), when the GM does fail the concentration reaching the aquifer decreases (because of dilution of the DCM in the pore fluid by leachate now advecting from the landfill).

For chloride there is a very different response. While the GM is intact there is negligible chloride reaching the aquifer. However, as the decrease in chloride concentration in the landfill is only by dilution (there is no biodegradation or sorption), for this size of landfill there is still significant chloride in the leachate at the time the GM fails, and the peak impact in the aquifer (at about 180 years) is about 700 mg/l. This greatly exceeds a common drinking water objective of 250 mg/l. Thus, for this landfill, none of the single composite liner designs considered could be judged to provide adequate environmental protection unless the service life of the primary GM were very substantially greater than 160 years. At present there is a paucity of evidence to support such a long life for a primary GM in a MSW landfill, and a double-lined system would be required to provide adequate environmental protection.

Double composite liners

The impact of chloride and other conservative contaminants whose concentration decreases only by dilution can be controlled by a suitable double-lined system. Fig. 30 shows a schematic of a double-lined system with a GM/GCL/AL forming both the primary and secondary liner system above an aquifer. A system involving a GM and 0·75 m thick CCL primary liner, and a GM/0·75 m thick CCL and 1 m thick attenuation layer for a secondary liner system, was also modelled. In both cases the head acting above the secondary GM was taken to be 0·03 m for the service life of the secondary LCS (1000 years). The results for both cases are shown in Fig. 30 for two different times of secondary GM failure. Failure of the primary GM at 160 years had negligible impact on the aquifer until failure of the secondary GM at time t_4. If the service life of the secondary GM were 300 years the impact on the aquifer would exceed 420 mg/l (i.e. well in excess of 250 mg/l). However, for a service life of 400 years the impact is less than 250 mg/l and hence below the typical drinking water objective. Both these service lives are below the estimated service life of a secondary GM of 600 years, as discussed earlier (for $T < 20$°C). As in the case of a single liner, there was only a small difference between the results for the liner system with a CCL and that with a GCL, suggesting that again the two systems are practically equivalent for the case examined.

CONCLUSIONS

This lecture has focused on the long-term performance of barrier systems, with particular emphasis on data collected

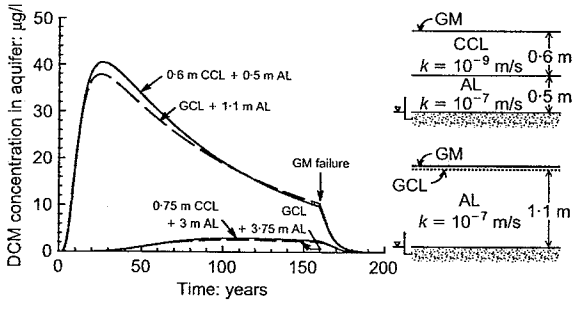


Fig. 29. Calculated variation in dichloromethane (DCM) concentration in an underlying aquifer with time for different single liner configurations for $t_1 = 40$ years, $t_2 = 60$ years and $t_3 = 160$ years (modified from Rowe *et al.*, 2004)

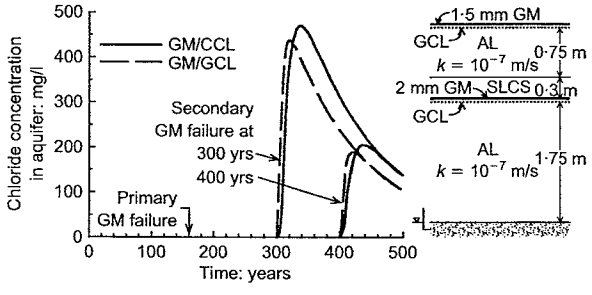


Fig. 30. Calculated variation in chloride concentration in an underlying aquifer with time for different double composite liner configurations for $t_1 = 40$ years, $t_2 = 60$ years and $t_3 = 160$ years (modified from Rowe *et al.*, 2004)

D43/48

over the past 25 years. Many of the studies cited in this lecture are ongoing, and will provide considerably more information in coming years, which may well cause us to revise the present findings (particularly on issues such as the service life of HDPE GMs). As a considerable amount of research is required to make meaningful comments about long-term performance, in this lecture attention is restricted to those materials and systems for which there has been adequate study. There are many other materials and systems that have been proposed, but their use in landfill base liners should be questioned until data have been gathered that will allow a meaningful assessment of how they are likely to behave in the long term relative to the materials currently in use.

Based on the results and discussion presented herein, a number of practical conclusions can be drawn regarding the seven issues that have been discussed. An overarching conclusion is that, even with the best designs and products, the components of a barrier system will fail to provide adequate long-term (or even short-term) performance if there is poor installation, and/or subsequent operations do not comply with good practice. Thus all of the following conclusions assume that there has been appropriate design and construction (taking account of the current state of knowledge and with appropriate CQC/CQA programmes), and that the operating procedures are such as to maintain the integrity of the design. The available laboratory and field evidence, combined with modelling, indicates that primary LCSs in MSW landfills have a finite service life, which could range from less than a decade to more than a century depending on the design details, waste characteristics and mode of operation. The service life of LCSs can be extended by:

(*a*) the use of uniform coarse gravel in the drainage blanket
(*b*) minimising the flow in critical regions (e.g. by selection of pipe spacing and prudent location of sumps)
(*c*) increasing the thickness of gravel in critical areas and having alternative drainage paths
(*d*) using a suitable filter between the waste and the coarse gravel
(*e*) appropriate sequencing of waste placement
(*f*) regularly cleaning the leachate collection pipes.

There are also activities that may reduce the service life, including:

(*a*) disposal of sewage sludge near the LCS
(*b*) disposal of fines near the collection system.

When a collection system does clog it will result in an increased head on the primary liner, causing greater advection through the liner system. It may also increase the liner temperature.

It has been well known that high temperatures (up to 40–60°C) may be anticipated at the base of landfills where there is a significant leachate mound. Recent data now indicate that even with an effective LCS and negligible leachate mound the liner temperature can be expected to reach 30–40°C for normal landfill operations. With recirculation of leachate the liner temperature increases faster than under normal operating conditions, and may be expected to be greater than 40°C to 45°C.

Both monitoring of leakage through different primary liner systems and theoretical calculations have demonstrated that composite liners are substantially better than single liners in terms of controlling leakage from landfills, and that although there will typically be some leakage, the leakage rates with a composite liner are very small. Composite liners involving a GCL have resulted in substantially less reported leakage than those involving a CCL. This has been attributed, at least in part, to better contact between the GM and GCL and a consequent small interface transmissivity. It has also been shown that considerable care is needed in the design of double-lined systems where the primary and secondary GM are separated only by a GCL and geonet. Problems may include swelling of clay into the geonet, which substantially reduces flow in the geonet, and high temperatures on the geonet and secondary liner (reducing flow in the geonet and potentially causing a very substantial reduction in the service life of the secondary GM).

The observed leakage through composite liners (both with a CCL and with a GCL) is considerably greater than would be expected for the typical number of holes in GMs if the GM were in intimate contact with the underlying clay liner. However, the observed leakage can be explained by the holes being in, or closely adjacent to, wrinkles/waves in the GM, and a simple equation (Rowe, 1998a) can be used to estimate leakage through composite liners for this situation. Even with holes in wrinkles, the leakage rates are small and diffusion will dominate as a transport mechanism for contaminants that can readily diffuse through a GM.

HDPE GMs provide an excellent diffusive barrier to ions, and both laboratory and field tests conducted over a 12-year period (and still ongoing) show negligible (i.e. at the limit of the quantification techniques being used) diffusion of salts through HDPE GMs. On the other hand, both laboratory and field observations suggest relatively rapid diffusion of volatile organic compounds through HDPE GMs. They also diffuse through clay. However, migration of these volatile organic compounds is slowed by the GM, and can be controlled to acceptable levels by a combination of GM and an adequate thickness of liner and attenuation layer below the GM.

The long-term performance of HDPE GMs depends on:

(*a*) the selection of suitable material (e.g. meeting the requirements of GRI GM13)
(*b*) appropriate protection of the GM from both damage and stress concentrations
(*c*) the chemistry of the contacting fluid (e.g. contact with leachate decreases service life far more than contact with water)
(*d*) temperature.

There are three stages to the ageing of an HDPE GM. At present there are some good data for antioxidant depletion (Stage 1) of GMs in simulated liner situations. However, projections for Stages 2 (induction) and 3 (degradation) are based on very limited data relating to pressure pipes. More data are required for conditions more relevant to landfill situations to provide truly confident estimates of GM service life (and these tests are presently in progress but results will not be available for several more years). Given this caveat, based on the currently available data it would appear that the service life for HDPE GM in MSW landfill is likely about 160 years for a primary liner at 35°C and greater than 600 years for a secondary GM provided it is at a temperature of less than 20°C (this will be achieved only if there is an adequate thickness of soil between the primary and secondary systems to provide an adequate thermal barrier).

Clay liners are susceptible to both shrinkage and cracking during construction (because of heating by solar radiation or freezing) and after placement of the waste (because of temperature gradients generated by the waste). The former is critical for CCLs and some GCLs (those manufactured at greater than 15% initial water content) but can be controlled by appropriate placement of a suitable thermal protection layer (e.g. sand that is kept watered, or by quick placement of waste). The latter requires more sophisticated consideration. Based on laboratory tests and modelling it appears that the potential for thermally induced desiccation can be miti-

D 44/48

gated by controlling the applied temperature gradient (the lower the better), the initial water content of the subsoil (higher is better, within reason), and the initial water content of the GCL prior to the application of stress. Based on the studies reported herein, it appears that although there are certainly situations where desiccation can occur, it can be avoided by appropriate design and construction.

Numerical models can be effective in predicting the service lives of engineered systems and are essential to predicting long-term contaminant transport and assessing the equivalence and suitability of different barrier systems.

This lecture has sought to demonstrate that the assessment of the long-term performance of landfill barrier systems requires an understanding of both soil mechanics and geosynthetics. It has also sought to demonstrate that components of the barrier system (LCS, GM and CCL or GCL) should not be considered in isolation, but rather as part of a system where the characteristics and performance of one component impact on that of another. Finally, it has highlighted the importance of modelling coupled phenomena for assessing the long-term performance of LCSs, the desiccation of CCLs and GCLs, and contaminant transport through barrier systems.

ACKNOWLEDGEMENTS

The author is extremely grateful to the City of Toronto for providing extensive monitoring data, access for research purposes, and their willingness to allow data to be freely published. In doing so they have allowed identification of issues, advancement in design and greater environmental protection both for the KVL and for future landfills. In this regard special thanks are due to L. Ciardullo. F. Barone of Golder Associates and L. Josic and D. Bleiker of AMEC Earth and Environmental gave considerable assistance in providing both data and many useful insights with respect to the KVL. Many people offered information and assistance in the preparation of this lecture. The following people are thanked for their assistance in the preparation of figures: R. Brachman (Figs 1, 6, 16, 18, 20, 21, 29, 30), J. VanGulck (Figs 2, 3, 8, 9), S. Rimal (Figs 4, 23), Z. Islam (Figs 11–15, 28), K. AbdelAtty (Fig. 17), J. Southen (Figs 24, 25), C. Lake (Fig. 26) and F. Barone (Fig. 27). The careful review of sections of the written text by F. Barone, A. Bouazza, R. Brachman, P. Davies, Z. Islam, R. Krushelnitzky, C. Lake, K. Legge, R. McWatters, T. Mukunoki, S. Sheridan, N. Touze-Foltz, G. Hsuan and J. VanGulck is very much appreciated. The author is indebted to the many students and collaborators who have contributed to this research over the past 25 years, and in particular to the late Professors J. R. Booker and R. M. Quigley. The Natural Science and Engineering Research Council of Canada funded the majority of the research reported herein. Additional funding was provided by the Province of Ontario through CRESTech and by Terrafix Geosynthetic Inc. and NAUE GmbH & Co.

NOTATION

- A constant (often called collision factor)
- a distance in drainage layer where porosity reduction is maximum (m)
- B thickness of a drainage layer (m)
- B' portion of the thickness of drainage layer (m)
- b half-width of a wrinkle (m)
- c contaminant concentration (mg/l, μg/l)
- $c(t)$ contaminant concentration at time t (mg/l; μg/l)
- c_f contaminant concentration in solution (mg/l; μg/l)
- c_g contaminant concentration in geomembrane (mg/l; μg/l)
- c_0 initial concentration (mg/l; μg/l)
- c_L contaminant concentration in leachate (mg/l; μg/l)
- D diffusion coefficient in soil (m^2/s; m^2/yr)
- D thickness of clay liner (m)
- D_e effective diffusion coefficient in soil (m^2/s; m^2/yr)
- D_g diffusion coefficient in geomembrane (m^2/s; m^2/yr)
- E_a activation energy (J/mol)
- f mass flux of contaminant ($g/(m^2\,yr)$)
- f_{Ca} proportion of calcium in total clog material (–)
- g gravitational acceleration (m/s^2)
- h hydraulic head (m)
- h_d head loss across composite liner (m)
- h_w leachate head acting on top of geomembrane (m)
- i hydraulic gradient
- i_c concentration gradient
- K_d partitioning or distribution coefficient (ml/g)
- k hydraulic conductivity of liner (m/s)
- k_{ave} average hydraulic conductivity of liner (m/s)
- k_f hydraulic conductivity of foundation/attenuation layer (m/s)
- k_n geotextile normal hydraulic conductivity (m/s)
- k_{om} hydraulic conductivity of leachate collection layer (m/s)
- L drainage length (m) in equations (18–21) and length of a wrinkle (m) in equation (22)
- M_A mass per unit area (g/m^2)
- n porosity (–)
- n_e effective porosity (–)
- n_0 initial porosity (–)
- P geotextile permittivity (s^{-1})
- P_g 'permeability' of polymer to given contaminant ($P_g = S_{gf}D_g$) (m^2/s; m^2/yr)
- p_a pore air pressure (kPa)
- Q flow through a hole in a geomembrane (m^3/s)
- q_0 permeation through landfill cover reaching leachate collection system per unit area (m/s; m/yr)
- q_T heat flux (W/m^2)
- R universal gas constant (8·314 J/(mol K))
- r_0 radius of a hole in a geomembrane (m)
- S_{gf} partitioning (solubility, Henry) coefficient (–)
- S antioxidant depletion rate ($month^{-1}$)
- s_{av} average antioxidant depletion rate ($month^{-1}$)
- T temperature (K)
- T_{ave} average temperature (°C; °K)
- t time (s; yr)
- t_c time to clog (day, yr)
- t_{GM} thickness of geomembrane (mm)
- t_{GT} thickness of geotextile (mm)
- t_{GCL} thickness of GCL (mm)
- $V(t)$ volume of pore space filled with clog material at time t (m^3)
- V_{tot} volume of mineral clog corresponding to permeability decrease (m^3)
- v_a Darcy velocity (Darcy flux) through a layer (m/s; m/yr)
- v_b Darcy velocity (Darcy flux) in an aquifer (m/yr)
- v_f porosity reduction (–)
- v_f^* maximum porosity reduction (–)
- $Y(t)$ maximum possible volumetric yield
- Δ density of a fluid (t/m^3; kg/m^3; g/cm^3)
- Δ_c density of clog material (t/m^3; kg/m^3; g/cm^3)
- Δ_d dry density of soil (t/m^3; kg/m^3; g/cm^3)
- Δ_{GM} density of geomembrane (t/m^3; kg/m^3; g/cm^3)
- θ transmissivity between geomembrane and clay liner (m^2/s)
- λ' Fourier thermal conductivity
- τ tortuosity (–)
- σ_H applied horizontal stress (kPa)

ABBREVIATIONS

- AOS apparent opening size (mm, μm)
- AL attenuation layer
- BOD_5 biochemical oxygen demand measured in a 5-day test.
- BTEX benzene, toluene, ethyl benzene and xylenes
- CCL compacted clay liner
- COD chemical oxygen demand
- CQA construction quality assurance
- CQC construction quality control
- DCA dichloroethane
- DCM dichloromethane
- EPS extracellular polysaccharides

D44/48

FOS	filter opening size (mm, μm)
FSS	fixed (inorganic) suspended solid (mg/l)
GCL	geosynthetic clay liner
GM	geomembrane
GN	geonet
GT	geotextile
HBNW	heat-bonded nonwoven
HDPE	high-density polyethylene
HP-OIT	high-pressure oxidative induction time
ISS	industrial solids and sludge
KVL	Keele Valley Landfill
LCS	leachate collection system
LDS	leak detection system
LIW	light industrial waste
LR	leachate recirculation
lphd	litres per hectare per day
MSW	municipal solid waste
NPNW	needle-punched nonwoven
OIT	oxidative induction time (min)
PAC	powdered activated carbon
PB	powdered bentonite
PLCS	primary leachate collection system
POA	percentage open area (%)
PVC	polyvinyl chloride
SDR	standard dimension ratio for pipes (–)
SLCS	secondary leachate collection system (also called a leak detection system)
SP-NCLT	single-point notched constant-load test
STd-OIT	standard oxidative induction time
TCE	trichloroethylene
TSS	total suspended solids (mg/l)
VFA	volatile fatty acids
VOC	volatile organic compounds
VSS	volatile suspended solids (mg/l)
VVO	void volume occupancy (%)
W	woven

REFERENCES

Apse, J. L. (1989). Polyethylene resins for geomembrane applications. In *Durability and aging of geosynthetics* (ed. R. M. Koerner), pp. 159–176. Amsterdam: Elsevier Applied Science Publishers.

Armstrong, M. D. (1998). *Laboratory program to study clogging in a leachate collection system.* MESc thesis, University of Western Ontario, London, Ontario, Canada.

Armstrong, M. D. & Rowe, R. K. (1999) Effect of landfill operations on the quality of municipal solid waste leachate. *Proc. 3rd Int. Landfill Symp., Cagliari*, 81–88.

ASTM (2002). *Standard practice for air-oven aging of polyolefin geomembranes*, D5721-95. West Conshohocken, PA: American Society for Testing and Materials.

ASTM (2004a). *Standard test method for oxidative-induction time of polyolefins by differential scanning calorimetry*, D3895-04. West Conshohocken, PA: American Society for Testing and Materials.

ASTM (2004b). *Standard test method for oxidative-induction time of polyolefin geosynthetics by high-pressure differential scanning calorimetry*, D5885-04. West Conshohocken, PA: American Society for Testing and Materials.

ASTM (2005). *Standard test method for evaluation of stress crack resistance of polyolefin geomembranes using notched constant tensile load*, D5397-99. West Conshohocken, PA: American Society for Testing and Materials.

August, H. & Tatzky, R. (1984). Permeability of commercially available polymeric liners for hazardous landfill leachate organic constituents. *Proceedings of the international conference on geomembranes*, Denver, Vol. 1, pp. 163–168.

August, H., Tatzky-Gerth, R., Preuschmann, R. & Jakob, I. (1992). Permeations verhalten von Kombinationsdichtungen bei Deponien und Altlasten Gegenueber Wassergefaehrdenden Stoffen, Project 10203412 of the BMBF. Berlin: BAM.

Badu-Tweneboah, K., Giroud, J. P., Carlson, D. S. & Schmertmann, G. R. (1998). Evaluation of the effectiveness of HDPE geomembrane liner protection. *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, 279–284.

Barone, F. S., Costa, J. M. A. & Ciardullo, L. (2000). Temperatures at the base of a municipal solid waste landfill. *Proc. 6th Can Environ. Engng Conf., London, Ontario*, 41–48.

Barone, F. S., Costa, J. M. A., King, K. S., Edelenbos, M. & Quigley, R. M. (1993). Chemical and mineralogical assessment of in situ clay liner Keele Valley Landfill, Maple, Ontario. *Proceedings of the Joint CSCE–ASCE National Conference on Environmental Engineering, Montreal*, 1563–1572.

Basnett, C. R. & Bruner, R. J. (1993). Clay desiccation of a single-composite liner system. *Proc. Geosynthetics 93, Vancouver*, 1329–1340.

Bennett, P. J., Longstaffe, F. J. & Rowe, R. K. (2000). The stability of dolomite in landfill leachate collection systems. *Can. Geotech. J.* **37**, No. 2, 371–378.

Bishop, D. J. (1996). Discussion of 'A comparison of puncture behavior of smooth and textured HDPE geomembranes' and 'Three levels of geomembrane puncture protection' by Narejo, D.B. *Geosynthetics Int.* **3**, No. 3, 441–443.

Bonaparte, R. & Gross, B. A. (1993). *LDCRS flow from double lined landfills and surface impoundments*, EPA/600/R-93/070. Springfield, VA: NTIS Publication PB93-179885.

Bonaparte, R., Daniel, D. & Koerner, R. M. (2002). *Assessment and recommendations for improving the performance of waste containment systems*, EPA Report CR-821448-01-0. Washington, DC: US Environmental Protection Agency.

Bouchez, T., Munoz, M.-L., Vessigaud, S., Bordier, C., Aran, C., & Duquennoi, C. (2003). Clogging of MSW landfill leachate collection systems: prediction methods and in situ diagnosis. *Proc. 9th Int. Waste Management and Landfill Symp., Cagliari* (CD-ROM).

Brachman, R. W. I. & Gudina, S. (2002). A new laboratory apparatus for testing geomembranes under large earth pressures. *Proc. 55th Can. Geotech. Conf.*, Niagara Falls, ON, 993–1000.

Brady, K. C., McMahon, W. & Lamming, G. (1994). *Thirty year ageing of plastics*, Project Report 11, E472A/BG. Crowthorne: Transport Research Laboratory.

Brown, K. W., Thomas, J. C., Lytton, R. L., Jayawickrama, P. & Bahrt, S. (1987). *Quantification of leakage rates through holes in landfill liners*, USEPA Report CR810940. Cincinnati, OH: US Environmental Protection Agency.

Brune, M., Ramke, H. G., Collins, H. & Hanert, H. H. (1991). Incrustation process in drainage systems of sanitary landfills. *Proc. 3rd Int. Landfill Symp., Cagliari*, 999–1035.

Cartaud, F. & Touze-Foltz, N. (2004). Influence of geotextiles at the interface of landfill bottom liners. *Proc. 3rd Eur. Conf. on Geosynthetics, Munich*, 495–500.

Cartaud, F., Touze-Foltz, N. & Duval, Y. (2005a). Experimental investigation of the influence of a geotextile beneath the geomembrane in a composite liner on leakage through a hole in the geomembrane. *Geotextiles and Geomembranes* **23**, No. 2, 117–143.

Cartaud, F., Goblet, P. & Touze-Foltz, N. (2005b). Numerical simulation of the flow in the interface of a composite bottom liner. *Geotextiles and Geomembranes* **23**, No. 6, 513–533.

Collins, H. J. (1993). Impact of the temperature inside the landfill on the behaviour of barrier systems. *Proc. 4th Int. Landfill Symp., Cagliari* **1**, 417–432.

Colucci, P., & Lavagnolo, M.C. (1995). Three years field experience in electrical control of synthetic landfill liners. *Proc. 5th Int. Landfill Symp., Cagliari*, 437–452.

Cooke, A. J. & Rowe, R. K. (1999). Extension of porosity and surface area models for uniform porous media. *ASCE J. Environ. Engng* **125**, No. 2, 126–136.

Cooke, A. J., Rowe, R. K. & Rittmann, B. E. (2005a). Modelling species fate and porous media effects for landfill leachate flow. *Can. Geotech. J.* **42**, No. 4, 116–1132.

Cooke, A. J., Rowe, R. K., VanGulck, J. F. & Rittmann, B. E. (2005b). Application of the Bioclog model for landfill leachate clogging of gravel-packed columns. *Can. Geotech. J.* (in press).

Corser, P., Pellicer, J. & Cranston, M. (1992). Observations on long-term performance of composite clay liners and covers. *Geotech. Fabrics Report*, **10**, No. 8, 6–16.

Craven, W., Townsend, T. G., Vogel, K. & Laux, S. (1999). Field investigation of landfill leachate collection system clogging.

P45/48

Practice Periodical of Hazardous, Toxic, and Radioactive Waste Management **3**, No. 1, 2–9.

Davies, P. L. & Legge, K. R. (2003). Geosynthetic clay liners: use and abuse in South Africa. *Proc. 9th Int. Waste Management and Landfill Symp., Cagliari* (CD-ROM).

Desaulniers, D. D., Cherry, J. A. & Fritz, P. (1981). Origin, age and movement of pore water in argillaceous quaternary deposits at four sites in Southwestern Ontario. *J. Hydrol.* **50**, 231–257.

Dickinson, S. & Brachman, R. W. I. (2003). Thickness of a GCL beneath a geomembrane wrinkle and coarse gravel backfill under large pressure. *Proc. 56th Can. Geotech. Conf., Winnipeg*, 465–472.

Döll, P. (1997). Desiccation of mineral liners below landfills with heat generation. *ASCE J. Geotech. Geoenviron. Engng* **123**, No. 11, 1001–1009.

Eloy-Giorni, C., Pelte, T., Pierson, P. & Margarita, R. (1996). Water diffusion through geomembranes under hydraulic pressure. *Geosynthetics Int.*, **3**, No. 6, 741–769.

Estornell, P. M. & Daniel, D. E. (1992). Hydraulic conductivity of three geosynthetic clay liners. *ASCE J. Geotech. Engng* **118**, No. 10, 1592–1606.

Felon, R., Wilson, P. E. & Janssens, J. (1992). Résistance mécanique des membranes d'étanchéité: membranes armées en théorie et en pratique. In *Vade mecum pour la realisation des systemes d'étancheite drainage artificiels pour les sites d'enfouissement technique en Wallonie*. Journées d'Études Ulg 92.

Fleming, I. R. & Rowe, R. K. (2004). Laboratory studies of clogging of landfill leachate collection and drainage systems. *Can. Geotech. J.* **41**, No. 1, 134–153.

Fleming, I. R., Rowe, R. K. & Cullimore, D. R. (1999). Field observations of clogging in a landfill leachate collection system. *Can. Geotech. J.* **36**, No. 4, 685–707.

Foose, G. J., Benson, C. H. & Edil, T. B. (2001). Prediction leakage through composite landfill liners. *ASCE J. Geotech. Geoenviron. Engng* **127**, No. 6, 510 – 520.

Gedde, U. W., Viebke, J., Leijstrom, H. & Ifwarson, M. (1994). Long-term properties of hot-water polyolefin pipes: a review. *Polymer Engng Sci.* **34**, No. 24, 1773–1787.

Giroud, J. P. (1996). Granular filters and geotextile filters. *Proc. Geofilters '96, Montreal*, 565–680.

Giroud, J. P. (1997). Equations for calculating the rate of liquid migration through composite liners due to geomembrane defects. *Geosynthetics Int.* **4**, No. 3–4, 335–348.

Giroud, J. P. & Bonaparte, R. (1989). Leakage through liners constructed with geomembranes. Part II: Composite liners. *Geotextiles and Geomembranes* **8**, No. 2, 71–111.

Giroud, J. P. & Bonaparte, R. (2001). Geosynthetics in liquid-containing structures. In *Geotechnical and Geoenvironmental Engineering Handbook* (ed. R. K. Rowe), pp. 789–824. Norwell: Kluwer Academic Publishing.

Giroud, J. P., Badu-Tweneboah, K. & Soderman, K. L. (1997a). Comparison of leachate flow through compacted clay and geosynthetic clay in landfill liner systems. *Geosynthetics Int.* **4**, No. 3–4, 391–431.

Giroud, J. P., Khire, M. V. & Soderman, K. L. (1997b). Liquid migration through defects in a geomembrane overlain and underlain by permeable media. *Geosynthetics Int.* **4**, Nos 3–4, 293–321.

GRI-GM13. (2003). *Specification high density polyethylene geomembranes*. Philadelphia, PA: Geosynthetics Research Institute.

Gross, B. A., Bonaparte, R. & Giroud, J. P. (1990). Evaluation of flow from landfill leakage detection layers. *Proc. 4th Int. Conf. on Geosynthetics, The Hague* **2**, 481–486.

Ham, R. K. & Bookter, T. J. (1982). Decomposition of solid waste in test lysimeters. *ASCE J. Environ. Engng* **108**, No. 6, 1147–1170.

Harpur, W. A., Wilson-Fahmy, R. F. & Koerner, R. M. (1993). Evaluation of the contact between geosynthetic clay liners and geomembranes in terms of transmissivity. *Proceedings of a GRI Seminar on Geosynthetic Liner Systems, Philadelphia, PA*, pp. 143–154.

Haxo, H. E. Jr & Lahey, T. (1988). Transport of dissolved organics from dilute aqueous solutions through flexible membrane liner. *Haz. Waste & Haz. Mater.* **5**, 275–294.

Heibrock, G. (1997). Desiccation cracking of mineral sealing liners. *Proc. 6th Int. Landfill Symp., Cagliari* **3**, 101–113.

Holzlöhner, U. (1995). Moisture balance, risk of desiccation in earthen liners. In *State of the art report: Landfill liner systems* (eds U. Holzlöhner, H. August, T. Meggyes and M. Brune), pp. H1–H22. Sunderland: Penshaw Press.

Hsuan, Y. G. & Guan, Z. (1998). Antioxidant depletion during thermal oxidation of high density polyethylene geomembranes *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, 375–380.

Hsuan, Y. G. & Koerner, R. M. (1998). Antioxidant depletion lifetime in high density polyethylene geomembranes. *ASCE J. Geotech. Geoenviron. Engng* **124**, No. 6, 532–541.

Hsuan, Y. G., Lord, A. E. Jr & Koerner, R. M. (1991). Effects of outdoor exposure on a high density polyethylene geomembrane. *Proc. Geosynthetics '91, Atlanta* **1**, 287–302.

Hwu, B., Koerner, R. M. & Sprague, C.J. (1990). Geotextile intrusion into geonets. *Proc. 4th Int. Conf. on Geotextiles, The Hague*, 351–356.

Jayawickrama, P., Brown, K. W., Thomas, J. C. & Lytton, R. L. (1988). Leakage rates through flaws in membrane liners. *ASCE J. Environ. Engng* **114**, No. 6, 1401–1419.

Jefferis, S. A. & Bath, A. (1999). Rationalising the debate on calcium carbonate clogging and dissolution in landfill drainage materials. *Proc. 7th Int. Landfill Symp., Cagliari*, 277–284.

Karlsson, K., Smith, G. D. & Gedde, U. W. (1992). Molecular structure, morphology and antioxidant consumption in medium density polyethylene pipes in hot-water applications. *Polymer Engng Sci.* **32**, No. 10, 649–657.

Klein, R., Baumann, T., Kahapka, E. & Niessner, R. (2001). Temperature development in a modern municipal solid waste incineration (MSWI) bottom ash landfill with regard to sustainable waste management. *J. Haz. Mater.* **B83**, 265–280.

Koerner, G. R. & Koerner, R. M. (1995a). Temperature behaviour of field deployed HDPE geomembranes. *Proc. Geosynthetics '95, Nashville*, 921–937.

Koerner, R. M. & Koerner, G. R. (1995b). *Leachate clogging assessment of geotextile (and soil) landfill filters*, Report CR-819371. Washington, DC: US Environmental Protection Agency.

Koerner, G. R. & Koerner, R. M. (2006). Long term temperature monitoring of geomembranes at dry and wet landfills. *Geotextiles and Geomembranes* **24** No. 1 (in press).

Koerner, G. R., Koerner, R. M. & Martin, J. P. (1994). Design of landfill leachate-collection filters. *ASCE J. Geotech. Engng* **120**, No. 10, 1792–1803.

Koerner, R. M., Knight, A. & Eith, A. (2003). Benefits of leachate recirculation: a case study. *Proceedings of the waste technology conference, New Orleans*, (CD-rom).

Kossendey, T. H., Gartung, E. & Schmidt, S. (1996). Microbiological influences on the long-term performance of geotextile filters. *Proc. Geofilters '96, Montreal*, 115–124.

Lake, C. B. & Rowe, R. K. (2000a). Diffusion of sodium and chloride through geosynthetic clay liners. *Geotextiles and Geomembranes* **18**, No. 2, 102–132.

Lake, C. B. & Rowe, R. K. (2000b). Swelling characteristics of needlepunched, thermally treated GCLs. *Geotextiles and Geomembranes* **18**, No. 2, 77–102.

Lake, C. B. & Rowe, R. K. (2004). Volatile organic compound (VOC) diffusion and sorption coefficients for a needlepunched GCL. *Geosynthetics Int.* **11**, No. 4, 257–272.

Lake, C. B. & Rowe, R. K. (2005). A comparative assessment of volatile organic compound (VOC) sorption to various types of potential GCL bentonites. *Geotextiles and Geomembranes* **23**, No. 4, 323–347.

Legge, K. R. & Davies, P. L. (2002). An appraisal of the performance of geosynthetic material used in waste disposal facilities in South Africa. *Proc. WasteCon 2002, Durban* (CD-ROM).

Maliva, R. G., Missimer, T. M., Leo, K. C., Statom, R. A., Dupraz, C., Lynn, M. & Dickson, J. A. D. (2000). Unusual calcite stromatolites and pisoids from a landfill leachate collection system. *Geology* **28**, No. 10, 931–934.

Manning, D. A. C. (2000). Carbonates and oxalates in sediments and landfill: monitors of death and decay in natural and artificial systems. *J. Geolog. Soc. London* **157**, 229–238.

Manning, D. A. C. & Robinson, N. (1999). Leachate-mineral reactions: implications for drainage system stability and clogging. *Proc. 7th Int. Landfill Symp., Cagliari*, 269–276.

McBean, E. A., Mosher, F. R. & Rovers, F. A. (1993). Reliability-based design for leachate collection systems. *Proc. 3rd Int. Landfill Symp., Cagliari*, 433–441.

D46/48

McIsaac, R. S., Rowe, R. K., Fleming, I. R. & Armstrong, M. D. (2000). Leachate collection system design and clog development. *Proc. 6th Can. Environ. Engng Conf., London, ON*, 66–73.

Mitchell, J. K. (1991). Conduction phenomena: from theory to geotechnical practice. *Géotechnique* **41**, No. 3, 299–340.

Mitchell, J. K. & Soga, K. (2005). *Fundamentals of soil behavior*, 3rd edn. New York: Wiley.

Mlynarek, J. & Rollin, A. L. (1995). Bacterial clogging of geotextiles: overcoming engineering concerns. *Proc. Geosynthetics '95*, Nashville, TN, 177–188.

Moo-Young, H., Johnson, B., Johnson, A., Carson, D., Lew, C., Liu, S. & Hancock, K. (2004). Characterisation of infiltration rates from landfills: supporting groundwater modeling efforts. *Environmental Monitoring and Assessment* **96**, 283–311.

Mueller, W. & Jacob, I. (2003). Oxidative resistance of HDPE. *Polymer Degradation and Stability* **79**, 161–172.

Müller, W., Jakob, R., Tatzky-Gerth, R. & August, H. (1998). Solubilities, diffusion and partitioning coefficients of organic pollutants in HDPE geomembranes: experimental results and calculations. *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, 239–248.

Narejo, D., Koerner, R. M. & Wilson_Fahmy, R. F. (1996). Puncture protection of geomembranes. Part II: Experimental. *Geosynthetics Int.* **3**, No. 5, 629–653.

Nosko, V. & Touze-Foltz, N. (2000). Geomembrane liner failure: modeling of its influence on contaminant transfer. *Proc. 2nd Eur. Geosynthetics Conf., Bologna*, 557–560.

Osawa, Z. & Saito, T. (1978). The effects of transition metal compounds on the thermal oxidative degradation of polypropylene in solution. In *Stabilisation and degradation of polymers*, Advances in Chemistry, Series 169, pp. 2897–2907. American Chemical Society.

Othman, M. A., Bonaparte, R. & Cross, B. A. (1996). Preliminary results of study of composite liner field performance. *Proc. 10th GRI Conf.: Field performance of geosynthetics and geosynthetic related systems, Drexel University, Philidelphia*, pp. 110–137.

Owen, J. A. & Manning, D. A. C. (1997). Silica in landfill leachates: implications for clay mineral stabilities. *Appl. Geochem.* **12**, 267–280.

Paksy, A., Powrie, W., Robinson, J. P. & Peeling, L. (1998). A laboratory investigation of microbial clogging in granular landfill drainage media. *Géotechnique* **48**, No. 3, 389–401.

Parkin, G. F. & Owen, W. F. (1986). Fundamentals of anaerobic digestion of wastewater sludges. *ASCE J. Environ. Engng* **112**, No. 5, 867–920.

Pelte, T., Pierson, P. & Gourc, J. P. (1994). Thermal analysis of geomembranes exposed to solar radiation. *Geosynthetics Int.* **1**, No. 1, 21–44.

Petermann, J., Miles, M. & Gleiter, H. (1976). Growth of polymer crystals during annealing. *J. Macromol. Sci. Phys. B* **12**, No. 3, 393–404.

Peters, G. P. & Smith, D. W. (2002). Solute transport through a deforming porous medium. *Int. J. Numer. Anal. Methods Geomech.* **26**, 683–717.

Quigley, R. M. & Rowe, R. K. (1986). Leachate migration through clay below a domestic waste landfill, Sarnia, Ontario, Canada: Chemical interpretation and modelling philosophies. In *Hazardous and industrial solid waste testing and disposal*, ASTM STP 933, pp. 93–103. American Society for Testing and Materials.

Quigley, R. M., Gwyn, Q. H. J., White, O. L., Rowe, R. K., Haynes, J. E. & Bohdanowicz, A. (1983). Leda clay from deep boreholes at Hawkesbury, Ontario. Part I: Geology and geotechnique. *Can. Geotech. J*, **20**, No. 2, 288–298.

Reades, D. W., King, K. S., Benda, E., Quigley, R. M., LeSarge, K. & Heathwood, C. (1989). The results of on-going monitoring of the performance of a low permeability clay liner, Keele Valley landfill, Maple, Ontario. *Proceedings of a focus conference on eastern regional ground water issues*, National Water Well Assoc., Kitchener, ON, pp. 79–91.

Reddy, K. R., Bandi, S. R., Rohr, J. J., Finy, M. & Siebken, J. (1996). Field evaluation of protective covers for landfill geomembrane liners under construction loading. *Geosynthetics Int.* **3**, No. 6, 679–700.

Richardson, G. N. (1997). GCLs: alternative subtitle D liner systems. *Geotech. Fabrics Report*, **15**, No. 5, 36–42.

Rittmann, B. E., Fleming, I. R. & Rowe, R. K. (1996). Leachate chemistry: its implications for clogging. *Proc. ASCE North American Water and Environmentt Congress '96, Anaheim* (CD-ROM).

Rogers, C. E. (1985). Permeation of gases and vapors in polymers. In *Polymer permeability* (ed. J. Comyn), pp. 11–73. London: Elsevier Applied Science.

Rollin, A. L., Mlynarek, J., Lafleur, J. & Zanescu, A. (1994). Performance changes in aged in-situ HDPE geomembrane. In *Landfilling of wastes: Barriers* (eds T. Christensen, R. Cossu and R. Stegmann), pp. 431–443. London: E & FN Spon.

Rowe, R. K. (1998a). Geosynthetics and the minimization of contaminant migration through barrier systems beneath solid waste. *Proc. 6th Int. Conf. on Geosynthetics*, Atlanta **1**, 27–103.

Rowe, R. K. (1998b). From the past to the future of landfill engineering through case histories. *Proc. 4th Int. Conf. on Case Histories in Geotech. Engng, St Louis*, 145–166.

Rowe, R. K. & Booker, J. R. (1995). A finite layer technique for modelling complex landfill history. *Can. Geotech. J.* **32**, No. 4, 660–676.

Rowe, R. K. & Booker, J. R. (1998). Modelling impacts due to multiple landfill cells and clogging of leachate collection systems. *Can. Geotech. J.* **35**, No. 1, 1–14.

Rowe, R. K. & Booker, J. R. (2000). Theoretical solutions for calculating leakage through composite liner systems. *Developments in theoretical geomechanics: Proceedings of The John Booker Memorial Symposium*, Sydney, pp. 580–602.

Rowe, R. K. & Booker, J. R. (2005). *POLLUTEv7: Pollutant migration through a nonhomogeneous soil*. GAEA Environmental Engineering Ltd, Whitby, Canada.

Rowe, R. K. & Fleming, I. R. (1998). Estimating the time for clogging of leachate collection systems. *Proc. 3rd Int. Congress on Environ. Geotech., Lisbon* **1**, 23–28.

Rowe, R. K. & Iryo, T. (2005). Effect of geosynthetics on the hydraulic performance of leak detection systems. *Can. Geotech. J.* **42**, No. 5, 1359–1376.

Rowe, R. K. & McIsaac, R. (2005). Clogging of tire shreds and gravel permeated with landfill leachate. *ASCE J. Geotech. Geoenviron. Engng* **131**, No. 6, 682–693.

Rowe, R. & Nadarajah, P. (1995). Transport modelling under transient flow conditions. *Proc. 5th Int. Symp. on Numerical Models in Geomechanics, Davos*, 337–342.

Rowe, R. K. & Nadarajah, P. (1996). Effect of temporal fluctuation of leachate concentration in landfills. *Proc. 2nd Int. Cong. on Environ. Geotech., Osaka*, 305–310.

Rowe, R. K. & Orsini, C. (2003). Effect of GCL and subgrade type on internal erosion in GCLs. *Geotextiles and Geomembrane* **21**, No. 1, 1–24.

Rowe, R. K. & Sangam, H. P. (2002). Durability of HDPE geomembranes. *Geotextiles and Geomembranes* **20**, No. 2, 77–95.

Rowe, R. K. & Sawicki, D. W. (1992). The modelling of a natural diffusion profile and the implications for landfill design. *Proc. 4th Int. Symp. Numer. Methods in Geomech., Swansea*, 481–489.

Rowe, R. K. & VanGulck, J. F. (2004). Filtering and drainage of contaminated water. *Proc. 4th Int. Conf. on GeoFilters, Stellenbosch*, 1–63.

Rowe, R. K., Barone, F. S. & Hrapovic, L. (1997). Laboratory and field studies of salt diffusion through a composite liner. *Proc. 6th Int. Landfill Symp., Cagliari* **3**, 241–250.

Rowe, R. K., Hsuan, Y. G., Lake, C. B., Sangam, P. & Usher, S. (1998). Evaluation of a composite (geomembrane/clay) liner for a lagoon after 14 years of use. *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, 191–196.

Rowe, R. K., Armstrong, M. D. & Cullimore, D. R. (2000a). Mass loading and the rate of clogging due to municipal solid waste leachate. *Can. Geotech. J.* **37**, No. 2, 355–370.

Rowe, R. K., Armstrong, M. D. & Cullimore, D. R. (2000b). Particle size and clogging of granular media permeated with leachate. *ASCE J. Geotech. Geoenviron. Engng* **126**, No. 9, 775–786.

Rowe, R. K., Caers, C. J., Reynolds, G. & Chan, C. (2000c). Design and construction of barrier system for the Halton landfill. *Can. Geotech. J.* **37**, No. 3, 662–675.

Rowe, R. K., Goveas, L. & Dittrich, J. P. (2002). Excavations in gassy soils. *Geotech. Engng* **155**, No. 3, 159–161.

Rowe, R. K., Sangam, H. P. & Lake, C. B. (2003). Evaluation of an HDPE geomembrane after 14 years as a leachate lagoon liner. *Can. Geotech. J.* **40**, No. 3, 536–550.

Rowe, R. K., Quigley, R. M., Brachman, R. W. I. & Booker, J. R. (2004). *Barrier systems for waste disposal facilities*. London: Taylor & Francis (E & FN Spon).

Rowe, R. K., Mukunoki, T. & Sangam, P. H. (2005). BTEX diffusion and sorption for a geosynthetic clay liner at two temperatures. *ASCE J. Geotech. Geoenviron. Engng* **131**, No. 1, 1211–1221.

Sangam, H. P. & Rowe, R. K. (2001a). Migration of dilute aqueous organic pollutants through HDPE geomembranes. *Geotextiles and Geomembranes* **19**, No. 6, 329–357.

Sangam, H. P. and. Rowe, R. K (2001b). The role of HDPE geomembranes in retarding the diffusive migration of organic contaminants through composite liner systems. *Proc. 8th Int. Landfill Symp., Cagliari* **3**, 245–254.

Sangam, H. P. & Rowe, R. K. (2002). Effects of exposure conditions on the depletion of antioxidants from high-density polyethylene (HDPE) geomembranes. *Can. Geotech. J.* **39**, No. 6, 1221–1230.

Schnabel W. (1981). *Polymer degradation: principles and practical applications*. Toronto: Hanser.

Seeger, S. & Müller, W. (1996). Requirements and testing of protective layer systems for geomembranes. *Geotextiles and Geomembranes*, **14**, 365–376.

Shaner, K. R. & Menoff, S. D. (1992). Impacts of bentonite geocomposites on geonet drainage. *Geotextiles and Geomembranes* **11**, No. 4–6, 503–512.

Soong, T. Y. & Koerner, R. M. (1998). Laboratory study of high density polyethylene geomembrane waves. *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, 301–306.

Southen, J. M. (2005). *Thermally driven moisture movement within and beneath geosynthetic clay liner*. PhD thesis, University of Western Ontario.

Southen, J. M. & Rowe, R. K. (2004). Investigation of the behavior of geosynthetic clay liners subjected to thermal gradients in basal liner applications. *J. ASTM International* **1**, No. 2 ID JAI11470, www.astm.org. Also in *Advances in geosynthetic clay liner technology* (eds R. E. Mackey and K. von Maubeuge), pp. 121–133. West Conshohocken, PA: ASTM International.

Southen, J. M. & Rowe, R. K. (2005a). Modeling of thermally induced desiccation of geosynthetic clay liners. *Geotextiles and Geomembranes* **23**, No. 5, 425–442.

Southen, J. M. & Rowe, R. K. (2005b). Laboratory investigation of GCL desiccation in a composite liner subjected to thermal gradients. *ASCE J. Geotech. Geoenviron. Engng* **131**, No. 7, 925–935.

Stark, T. D. (1998). Bentonite migration in geosynthetic clay liners. *Proc. 6th Int. Conf. on Geosynthetics, Atlanta* **1**, 315–320.

Stone, J. L. (1984). Leakage monitoring of the geomembrane for proton decay experiment. *Proceedings of the international conference on geomembranes*, Denver, Vol. 2, pp. 475–480.

Thiel, R. & Richardson, G. N. (2005). Concern for GCL shrinkage when installed on slopes, *Proc. ASCE GeoFrontiers Conf.*, Austin, Paper GRI-18 (CD-ROM).

Tognon, A. R., Rowe, R. K. & Moore, I. D. (2000). Geomembrane strain observed in large-scale testing of protection layers. *ASCE J. Geotech. Geoenviron. Engng* **126**, No. 12, 1194–1208.

Touze-Foltz, N., Rowe, R. K. & Duquennoi, C. (1999). Liquid flow through composite liners due to geomembrane defects: analytical solutions for axi-symmetric and two-dimensional problems. *Geosynthetics Int.* **6**, No. 6, 455–479. Erratum: (2000) 7, No. 1, 77.

Touze-Foltz, N., Schmittbuhl, J. & Memier, M. (2001a). Geometric and spatial parameters of geomembrane wrinkles on large scale model tests. *Proc. Geosynthetics 2001, Portland*, 715–728.

Touze-Foltz, N., Rowe, R. K. & Navarro, N. (2001b). Liquid flow through composite liners due to geomembrane defects: nonuniform hydraulic transmissivity at the liner interface. *Geosynthetics Int.* **8**, No. 1, 1–26.

VanGulck, J. F. (2003). *Biodegradation and clogging in gravel size material*. PhD thesis, Queen's University, Kingston, Ontario.

VanGulck, J. F. & Rowe, R. K. (2004a). Evolution of clog formation with time in columns permeated with synthetic landfill leachate. *J. Cont. Hydrol.* **75**, 115–135.

VanGulck, J. F. & Rowe, R. K. (2004b). Influence of landfill leachate suspended solids on clog (biorock) formation. *Int. J. Integrated Waste Management Sci. Tech* **24**, 723–738.

VanGulck, J. F., Rowe, R. K., Rittmann, B. E. & Cooke, A. J. (2003). Predicting biogeochemical calcium precipitation in landfill leachate collection systems. *Biodegradation* **14**, 331–346.

Viebke, J., Elble, E., Ifwarson, M. & Gedde, U. W. (1994). Degradation of unstabilized medium-density polyethylene pipes in hot-water applications. *Polymer Engng Sci.* **34**, No. 17, 1354–1361.

Wisse, J. D. M., Broos, C. J. M. & Boels, W.H. (1990). Evaluation of the life expectancy of polypropylene geotextiles used in bottom protection structures around the Ooster Schelde storm surge barrier: a case study. *Proc. 2nd Int. Conf. on Geotextiles, Las Vegas* **1**, 283–288.

Workman, J. P. (1993). Interpretation of leakage rates in double lined systems. *Proc. 7th GRI Conf.: Geosynthetic Liner Systems, Philadelphia*, 91–108.

Yanful, E. K., Quigley, R. M. & Nesbitt, W. (1988). Heavy metal migration at a landfill site. Part II: Metal partitioning and geotechnical implications. *Appl. Geochem.* **3**, 623–629.

Yoshida, H. & Rowe, R. K. (2003). Consideration of landfill liner temperature. *Proc. 8th Int. Waste Management and Landfill Symp., Cagliari* (CD-ROM).

Yoshida, H., Hozumi, H. & Tanaka, N. (1996). Theoretical study on temperature distribution in a sanitary landfill. *Proc. 2nd Int. Congress on Environ. Geotech., Osaka* **1**, 323–328.

Zhou, Y. & Rowe, R. K. (2003). Development of a technique for modelling clay liner desiccation. *Int. J. Numer. Anal. Methods Geomech.* **27**, 473–493.

VOTE OF THANKS

PROFESSOR W. POWRIE, Department of Civil and Environmental Engineering, University of Southampton

When the 45th Rankine lecturer was announced 12 months ago, the many people who know Professor Rowe expected this year's lecture to be educational, interesting and entertaining. This sense of expectation is evidenced by the many people in the hall who have travelled long distances to be here tonight. They will not have been disappointed: Professor Rowe has spoken eloquently and convincingly on the long-term performance of contaminant barriers—a subject he has in recent years made his own.

Professor Rowe's lecture has been an elegant fusion of chemistry, microbiology, materials science, conduction phenomena and soil mechanics. These are the key elements of modern geoenvironmental engineering. At a recent workshop on waste and landfill mechanics, the acceptance of pH as a geotechnical parameter was remarked upon. This evening's lecture has reminded us that there are several other parameters that must be added to this list.

In 1828, in the original charter for the Institution of Civil Engineers, Thomas Tredgold defined civil engineering in terms that included the phrase 'directing the great sources of Power in Nature for the use and convenience of man'.[1] While this is still relevant today—especially when we think of the potential for exploiting solar, wind and marine current energy—it is not the whole picture. We might define civil engineering as the application of science and other disciplines and skills to the solution of society's problems. Time moves on, and society's problems—and expectations—change. Among the problems that society now faces, the protection of the environment is paramount. Professor Rowe's lecture, which focused on exactly this, was therefore very timely. It was also highly relevant to Europe, particu-

[1] The full definition is reproduced in a paper by P. J. Jowitt in *Proceedings of the Institution of Civil Engineers, Engineering Sustainability*, **157**, No. 2, 79–88.

D48/48

larly in view of impending legislation that will change fundamentally the nature of landfilled wastes: although the amount should reduce, its pollution potential will probably increase, and the design of effective contaminant barriers will be even more essential than it is now.

Professor Rowe's lecture has reminded us that Tredgold's 'great sources of Power in Nature' might include a mass of microorganisms. It contained a wealth of fundamental science presented with great clarity—the hallmark of an expert—and extensive reference to real data. The elegant and effective models for quantifying the complex processes and interactions involved will be of immense relevance and value to practice. The lecture represents a landmark in the development and maturity of environmental geotechnics as a subject, and will be remembered and referred to for a long time to come. It is with very great pleasure that I invite you all to join me in thanking Professor Rowe for what has been an excellent and memorable 45th Rankine Lecture.

Job: Future Raises to closure, Tailings Main dam Red Dog Mine, Alaska Project No.33757098
Description: Predict the longevity of HDPE geomembranes as a liner
Computed by: TV Date 5/26/06
Checked by: SB Date 5/30/06

Reference

APPENDIX E

Appendix E – J.P. Giroud

Draft Technical Memo

B2/13

Draft Memorandum by J.P. Giroud

RED DOG TAILINGS DAM, GEOMEMBRANE DURABILITY, COMMENTS

2006.05.24

---

1 COMMENTS

1.1 Comment on uncertainty

Geomembrane degradation includes three stages: Stage 1 (antioxidant depletion), Stage 2 (induction to polymer degradation) and Stage 3 (polymer degradation). I agree with comments made by Tung Vu in an e-mail dated May 22. In particular, I agree that there is great uncertainty regarding the geomembrane degradation times given by Koerner and by Rowe for Stage 3 (the polymer degradation stage). Both Koerner and Rowe acknowledge this uncertainty in their publications.

In the memorandum from Bob Koerner to Tung Vu, Bob confirms that he has doubts regarding the geomembrane degradation times mentioned in the literature (essentially by Koerner and by Rowe) for Stages 2 and 3. These times are derived from interpretation of data from a single source, Viebke et al (1994). These data were obtained from tests on pipes, not on geomembranes. Koerner says in his memorandum, that, due to these doubts and based on some (unpublished) tests he has made, he has revised his HDPE lifetime prediction. In fact, he proposes a drastic revision because his recent tests give very small degradation times for Stage 3, e.g. 35 years at 20°C compared to more than 700 years based on interpretation (by both Koerner and Rowe) of the tests by Viebke et al. I called Kerry Rowe and mentioned to him that, privately, Koerner was questioning the interpretation of Viebke et al.'s tests as published by himself or by Rowe and was now mentioning degradation times 20 times smaller. Kerry Rowe said he was not aware of this new Koerner's position. Rowe agreed that the only published source of information on Stage 3 at this time was the tests by Viebke et al. and he acknowledged that "there was a lot of room for interpretation" of these tests. However, he said that he felt that a degradation time of 35 years for Stage 3 was probably too small, but he added that he would need several more years before the testing program he started more than ten years ago would give results that are interpretable. Essentially, this discussion with Kerry Rowe confirms that, based on information currently available, there is great uncertainty beyond Stage 1.

1.2 Comments on material in contact

According to Rowe, there is a significant difference in the rate of antioxidant depletion (Stage 1) depending on the material in contact with the geomembrane. Rowe indicates that the antioxidant depletion rate in leachate is 2.3 times faster than in water and 4 times faster than in air. In his Rankine Lecture paper, Rowe proposes two approaches for evaluating the duration of Stage 1 (antioxidant depletion time) in landfills: (i) an inferred exposure, which combines exposures to air, water and leachate using a simple equation; and (ii) a simulated exposure in a laboratory model. These two approaches give depletion times of the same order, e.g. of the order of 100 years at 20°C, which are similar to the depletion time in case of exposure to water and about half the depletion time in the case of exposure to air. It is interesting to compare these depletion times with the depletion time proposed by Koerner for Stage 1. For 20°C, the time proposed by Koerner is of the order of 200 years. From this comparison, it appears that the exposure situation considered by Koerner (which could be referred to as "buried" or "backfilled") is similar (with respect to antioxidant depletion) to exposure to water or a combination of air, water and leachate.

In the case of the Red Dog Tailings Dam, the situation could be considered as intermediary between exposure to air, on one hand, and a combination of air, water and leachate, on the other hand. This approach would be based on the assumption that the liquid in the Red Dog Tailings Dam is less aggressive than the leachate considered by Rowe. This would lead to a depletion time between 100 and 200 years at 20°C, e.g. a depletion time of the order of 150 years at 20°C.

2 EXTRAPOLATION

2.1 Equations used for extrapolation

Equation 31 from Rowe:

$$OIT_{(t)} = OIT_o \exp(-st) \tag{1}$$

where s is the antioxidant depletion rate, t is the time for antioxidant depletion from an initial OIT value of OIT_o to a final value of $OIT_{(t)}$.

Equation 32 from Rowe:

$$s = A \exp(-\frac{E}{RT}) \tag{2}$$

where A is a constant, T is the absolute temperature, E is the activation energy, and R is the universal gas constant.

Equation 1 gives:

$$t = \frac{\ln(OIT_o) - \ln(OIT_{(t)})}{s} \tag{3}$$

hence:

$$\ln(t) = \ln\left[\ln(OIT_o) - \ln(OIT_{(t)})\right] - \ln(s) \tag{4}$$

Equation 2 gives:

$$\ln(s) = \ln(A) - \frac{E}{RT} \tag{5}$$

Combining Equations 4 and 5 gives:

$$\ln(t) = \ln\left[\ln(OIT_o) - \ln(OIT_{(t)})\right] - \ln(A) + \frac{E}{RT} \tag{6}$$

Equation 6 can be written as follows:

$$\ln(t) = a + \frac{b}{T} \tag{7}$$

where a and b are constants.

Considering two sets of data, (T_1 and t_1) and (T_2 and t_2), Equation 7 gives:

$$\ln(t_1) = a + \frac{b}{T_1} \tag{8}$$

and:

$$\ln(t_2) = a + \frac{b}{T_2} \tag{9}$$

Combining Equations 7 to 9 gives:

$$\frac{\ln(t)-\ln(t_1)}{\ln(t_1)-\ln(t_2)}=\frac{\frac{1}{T}-\frac{1}{T_1}}{\frac{1}{T_1}-\frac{1}{T_2}} \tag{10}$$

hence:

$$\ln(t)=\ln(t_1)+\frac{\frac{1}{T}-\frac{1}{T_1}}{\frac{1}{T_1}-\frac{1}{T_2}}\left[\ln(t_1)-\ln(t_2)\right] \tag{11}$$

hence:

$$t=\exp\left\{\ln(t_1)+\frac{\frac{1}{T}-\frac{1}{T_1}}{\frac{1}{T_1}-\frac{1}{T_2}}\left[\ln(t_1)-\ln(t_2)\right]\right\} \tag{12}$$

Equation 12 makes it possible to calculate the geomembrane antioxidant depletion time for an absolute temperature, T, using as a basis two sets of temperature and time, (T_1 and t_1) and (T_2 and t_2). It is implicitly assumed that the geomembrane considered at temperature T is the same as the geomembrane used to develop both sets of data, (T_1 and t_1) and (T_2 and t_2). In particular, the *OIT* values (i.e. OIT_o and $OIT_{(t)}$) are assumed to be the same for all stets of data, i.e. (T_1 and t_1), (T_2 and t_2) and (T and t). Typical values are 135 minutes for OIT_o and 0.5 minute for $OIT_{(t)}$. However, these values are not needed in Equation 12.

Equation 12 is applicable only if the considered phenomenon is consistent with the Arrhenius model. Equation 12 was established for the phenomenon of antioxidant depletion. However, inspection of the sequence of equations presented in Section 2.1 shows that Equation 12 is in fact derived from Equation 7. Therefore, Equation 12 is applicable to all phenomena governed by Equation 7. Equation 7 can be written as follows:

$$\ln(t)=a+\frac{b}{T} \tag{13}$$

hence:

$$t = C \exp\left(\frac{b}{T}\right) \tag{14}$$

where C is a constant.

Equation 14 includes the equation of the Arrhenius model. Therefore, Equation 12 is applicable to all phenomena that are consistent with the Arrhenius model.

2.2 Application of the equation

I have used Equation 12 to extrapolate data from Koerner and Rowe. The data from Koerner and Rowe used in the extrapolation are presented in Table 1 and the extrapolated times are presented in Table 2.The following comments can be made on the degradation times obtained:

- If the data presented in Table 1 were perfectly consistent with the Arrhenius model, the times presented in Table 2 would be the same in any given column, i.e. the same time would be obtained regardless of the temperature range used as a basis for the extrapolation.
- If values in a given column of Table 2 are different (which is always the case), the most accurate value should be the value obtained using the widest temperature range as a basis for the extrapolation, i.e. the 10-60°C range (when available) or the 20-40°C range.
- It appears from Table 2 that, for Stage 1, the results are approximately consistent with the Arrhenius model.
- It appears from Table 2 that, for Stage 2, the results do not closely follow the Arrhenius model. In fact, the results presented by Koerner vary linearly with temperature. In the case of Rowe's results extrapolated from Rowe's data, the discrepancy may be due to the fact that the times for temperatures of 30°C or higher are very small and extrapolation based on these values may be inaccurate.
- It appears from Table 2 that the extrapolated degradation times based on Koerner's data for Stage 3 approximately follow the Arrhenius model.

Based on the degradation times presented in Table 2, the following degradation times can be proposed for a temperature of 6°C:

- 650 to 700 years for Stage 1 in case of a buried geomembrane that is not exposed to liquid.
- 400 years for Stage 1 in case of a buried geomembrane exposed to semi-aggressive liquids.
- 70 years for Stage 2 in case of a buried geomembrane that is not exposed to liquid.
- 40 years for Stage 2 in case of a buried geomembrane exposed to semi-aggressive liquids.
- 70 years for Stage 3, which should be conservative.

For the sake of comparison, extrapolation based on the traditional interpretation of Viebke et al.'s tests by Koerner and Rowe would give degradation times of the order of 3,500 to 4,000 years at 6°C.

3 TENTATIVE CONCLUSION

Geomembrane degradation includes three stages: Stage 1 (antioxidant depletion), Stage 2 (induction to polymer degradation) and Stage 3 (polymer degradation). Data are available for a relatively accurate prediction of the Stage 1 time. However, this time depends significantly of the exposure conditions (from geomembrane buried in dry soil to geomembrane in contact with aggressive liquids). Based on available data and the likely exposure conditions, the Stage 1 time can be predicted to be between 400 years (buried geomembrane exposed to semi-aggressive liquids) and 700 years (buried geomembrane that is not exposed to liquid). For the Red Dog Tailings Dam, it may be appropriate to interpolate between these two values.

Data for an accurate prediction of the Stage 2 time and the Stage 3 time are insufficient. Relevant data should be available in several years from tests that were started more than ten yeas ago and are still in progress. However, the Stage 2 time being small, an approximate prediction is possible without a significant error. Based on available data and the likely exposure conditions, the Stage 2 time can be predicted to be between 40 and 70 years.

The Stage 3 time is currently impossible to predict accurately for the reasons presented above. Using data from tests on polyethylene resins used in pipes, times of the order of 3,500 to 4,000 years can be predicted for a temperature of 6°C. However, predictions based on these data are questionable. Predictions based on limited preliminary unpublished tests on polyethylene resins used in geomembrane give Phase 3 times of the order of 70 years.

In conclusion, a conservative prediction of the service life of the geomembrane at 6° C is between 510 years for (for buried geomembrane exposed to semi-aggressive liquids) to 790 years (for buried geomembrane that is not exposed to liquid). It should be noted that a service life at 6°C greater than 4,000 years could be predicted if some data presented in the literature for Stage 3 were considered. However, these data based on tests on polyethylene resins used in pipes have not yet been confirmed by tests on polyethylene resins used in geomembranes. These tests are in progress and results should be available in several years. In the meantime it is recommended to use a conservative approach for Stage 3 time predictions.

Finally, it is important to note that during Stage 1, the geomembrane is protected by antioxidants, Therefore, during Stage 1 the geomembrane should not experience any loss in properties. As a result, it may be predicted that the geomembrane will experience no loss in properties during a period of 400 to 700 years (depending on the exposure conditions described above). After this period, the geomembrane properties can be expected to change slowly during a period that is difficult to predict based on current knowledge.

* * * * *

Table 1 Degradation times (years) used for extrapolation

Temperature	Stage 1				Stage 2			Stage 3
	Koerner		Rowe		Koerner	Rowe		Koerner
	Standard OIT	High pressure OIT	Air exposure	Simulated exposure		Base case	Adjusted case	
10°C	-	-	510	280	-	50	30	-
20°C	200	215	235	115	30	15	10	35
30°C	95	98	110	50	20	6	4	21
40°C	45	47	55	25	10	2	1	14
60°C	-		15	6	-	0.4	0.3	-

Table 2 Extrapolated degradation times (years) for a temperature of 6°C

Base temperature range	Stage 1				Stage 2			Stage 3
	Koerner		Rowe		Koerner	Rowe		Koerner
	Standard OIT	High pressure OIT	Air exposure	Simulated exposure		Base case	Adjusted case	
20-40°C	645	710	735	381	71	73	61	72
20-30°C	620	710	745	408	56	60	40	76
30-40°C	710	709	711	323	129	116	167	63
10-60°C	-	-	714	404	-	79	47	-

Draft Memorandum by J.P. Giroud

RED DOG TAILINGS DAM, GEOMEMBRANE DURABILITY, ADDITIONAL COMMENTS

2006.06.02

---

1 INTRODUCTION

Kerry Rowe has given me the equations he used for generating the key columns for Stages 2 and 3 of Table 6 in his Rankine Lecture paper. Also, from his Rankine Lecture paper, I reconstructed the equation he used for Stage 1.

The equations are given below.

2 EQUATIONS FROM ROWE

2.1 Equations for Stage 1

From Equation 32 in the Rankine Lecture paper:

$$\ln(s) = \ln(A) - \frac{E}{RT} \tag{A-1}$$

where R is the universal gas constant (8.314 J/(mol °K)).

From Figure 23 in the Rankine Lecture paper, for exposure to air:

$$\ln(s) = 15.901 - \frac{6480}{T} \tag{A-2}$$

From Figure 23 in the Rankine Lecture paper, for the case of simulated liner:

$$\ln(s) = 19.615 - \frac{7362}{T} \tag{A-3}$$

Incidentally, comparing Equations A-1 and A-2 gives the following value for the energy of activation in the case of exposure to air:

$$E = 53.874\,\text{kJ/(mol °K)}$$

Incidentally, comparing Equations A-1 and A-3 gives the following value for the energy of activation in the case of the simulated liner:

$$E = 61.208\,\text{kJ/(mol °K)}$$

The time for geomembrane degradation is then calculated using the following equation from Equation 31 in the Rankine Lecture paper:

$$t = \frac{\ln\left(\frac{OIT_o}{OIT}\right)}{s} \qquad \text{(A-4)}$$

The initial OIT of the geomembrane was: 135 minutes in the case of exposure to air; and 133 minutes in the case of the simulated liner.

Combining Equations A-2 and A-4 gives the following time for antioxidant depletion for the case of exposure to air:

$$t = \frac{\ln\left(\frac{133}{0.5}\right)}{\exp\left(15.901 - \frac{6480}{T}\right)} \quad \text{with } t \text{ in months} \qquad \text{(A-5)}$$

Combining Equations A-3 and A-4 gives the following time for antioxidant depletion for the case of the simulated liner:

$$t = \frac{\ln\left(\frac{135}{0.5}\right)}{\exp\left(19.615 - \frac{7362}{T}\right)} \quad \text{with } t \text{ in months} \qquad \text{(A-6)}$$

Values of the time for antioxidant depletion are presented in the third column of Table A-1 for the case of exposure to air (calculated using Equation A-5) and in the fourth column of Table A-1 for the case of the simulated liner (calculated using Equation A-6).

It is important to note that extrapolation using Equation 12 of the memorandum titled "Red Dog Tailings Dam, Geomembrane Durability, Comments" based on the values in Table A-1 for Stage 1 give the correct values for 6° C. This indicates that the discrepancies observed in the memorandum titled "Red Dog Tailings Dam, Geomembrane Durability, Comments" were due only to the rounding of numbers in Table 6 of the Rankine Lecture paper.

2.2 Equations for Stage 2

The following equation was given to me by Kerry Rowe:

$$\log(t) = \frac{3918}{T} - 8.2 \quad \text{with } t \text{ in hours} \tag{A-7}$$

This equation is based on an interpretation of tests by Viebke et al. (1994).

Equation A-7 cann be written as follows:

$$t = 10^{\left(\frac{3918}{T} - 8.2\right)} \quad \text{with } t \text{ in hours} \tag{A-8}$$

Values of degradation times calculated using Equation A-8 are presented in the last column of Table A-1.

Again, it is important to note that extrapolation using Equation 12 of the memorandum titled "Red Dog Tailings Dam, Geomembrane Durability, Comments" based on the values in the last column of Table A-1 give the correct value for 6° C in Stage 2. This indicates that the discrepancies observed in the memorandum titled "Red Dog Tailings Dam, Geomembrane Durability, Comments" were due only to the rounding of numbers in Table 6 of the Rankine Lecture paper.

2.3 Equation for Stage 3

The following equation was given to me by Kerry Rowe:

$$t = t_o \exp\left[\frac{E}{R}\left(\frac{1}{T} - \frac{1}{T_o}\right)\right] \quad \text{with } t \text{ in days} \tag{A-9}$$

Rowe used t_o = 90 days for T_o = 115°C = 388°K, based on data from Koerner and Bonaparte et al. (2000). He used an activation energy of 80 kJ/(mol °K).

A degradation time of 2444 years is obtained using Equation A-9 for 10° C. In Table 6 of the Rankine Lecture paper, this value is rounded as 2445 years.

3 CONCLUSION

The Arrhenius model is used for all three stages of geomembrane degradation.

Equation 12 of the memorandum titled "Red Dog Tailings Dam, Geomembrane Durability, Comments" provides appropriate extrapolation of degradation times in all three stages. Extrapolation using Equation 12 requires the knowledge of only two sets of data (time, temperature). The accuracy of the extrapolation depends on the accuracy of the data.

* * * * *

Table A-1. Degradation times for Stages 1 and 2 calculated with equations used by Kerry Rowe in Table 6 of his Rankine Lecture paper.

Temperature		Degradation time		
		Stage 1		Stage 2
°C	°K	Air	Simulated	
6	279	706.07	407.35	79.53
10	283	508.48	280.54	50.35
20	293	232.75	115.45	16.96
30	303	112.17	50.38	6.14
40	313	56.64	23.18	2.37
60	333	16.34	5.64	0.42

Note the decimal figures do not imply that the results are physically accurate. They are given to facilitate extrapolations and interpolations.

Appendix D

URS

Technical Memorandum

Date: July 7, 2006

To: Cecil Urlich and Charles Masala

From: Kris Turschmid, Mark Mierzejewski, and Keith Mountjoy

Subject: **Red Dog Tailings Dam Underdrain Analysis**

Introduction

The Red Dog Mine Tailings Main Dam includes an underdrain to facilitate the discharge of seepage from the tailings impoundment that passes through and around the primary seepage control system of the dam. The underdrain is located along the alignment of the original channel of the South Fork of Red Dog Creek, and has dimensions of approximately 750 feet long, 50 feet wide and 6 feet deep.

As part of the current mine closure planning studies, the question has been raised of how long it would take the underdrain to become plugged over time by secondary mineral precipitation from the seepage water as it discharges through the underdrain. The flow in the underdrain could precipitate secondary minerals (predominantly ferric oxyhydroxides) due to a combination of type and concentrations of chemical species present in the water as it enters the underdrain, the pH of the water in the underdrain, geochemical changes that may occur to the water primarily due to exposure to oxygen, and other factors. The flow of water on a basic level is from the pond at the surface of the tailings impoundment, down through the tailings, along the underdrain, into the seepage collection pond, and back up to the tailings pond.

The objective of this technical memorandum is to provide an estimation of how long it would take for the underdrain to plug after mine closure, for several different sets of assumptions.

Discussion of Causes of Precipitation

There are several factors which affect the precipitation rate in the underdrain. The basic, simplified relationships are as follows, and are discussed in more detail below:

- Initial iron concentration – more iron results in proportionally more precipitation according to a linear relationship.
- pH has a very strong effect – lower pH results in drastically less precipitation.
- Aeration level has a nearly linear effect – more aeration results in proportionally more precipitation according to a nearly linear relationship.
- Retention time in the underdrain – longer retention time results in proportionally more precipitation according to an approximately logarithmic relationship.
- Temperature – lower temperature results in less precipitation due to slower kinetics.

After closure, it is estimated that there will be a minimum of 2 feet and maximum of 8.3 feet of free water in the tailings impoundment. (Appendix B of this report). It is assumed that this entire depth will be oxygenated due to contact with the air. Below this free water is the strongly acid-generating tailing deposit, which is assumed to be denser than any organic material which may fall in and settle on the bottom of the free water pond. Organic

material will not have a significant effect on the oxygen in the pond, due to the shallow depth. Decomposition of organics is not a significant factor in the analysis.

It is assumed that reoxygenation from the atmosphere will not penetrate far into the tailings, which is composed of various sulphides and common rock-forming mineral constituents, saturated with water. The tailings are composed of barite, galena, anglesite, sphalerite, and pyrite. The tailings are overwhelmingly acid-generating, due to the high sulphide content of 10 to 21% by weight (SRK, 2006 and Yanful & Samad, 2004). URS estimates that the flow rate through the tailings is such that it will take approximately 11 years for water to penetrate from the free water through the tailings to the underdrain. As aerated water percolates from the free water pond through the tailings to the underdrain, oxygen will be consumed in various reactions with the sulphide mineral constituents of the tailings. The tailings will reduce oxidized substances flowing through them, resulting in tailings pore water that will be in a reduced state when it reaches the underdrain.

As the tailings pore water reaches the underdrain, it is in a reduced state, with a low redox potential. However, the underdrain itself is open to the atmosphere. It is assumed that due to air diffusion and convection through the porous underdrain drain rock, that the underdrain will be aerated to a level of approximately 10% of what would occur in the outside air. A value of 10% oxygen is a representative value for a waste rock dump, which is considered similar to the underdrain. The air will travel by diffusion and convection into the underdrain, both from the drainage rock comprising the underdrain and from the exit of the underdrain itself, which is open to the atmosphere. Some of the oxygen will dissolve into the underdrain water. At the exit of the underdrain, the oxygen concentration in the water would be higher than the assumed 10%, while deep in the upstream part of the dam where tailings pore water enters the underdrain, the oxygen concentration would be much lower than the assumed 10% due to the difficulty of air transport through the drainage rock. Overall, the oxygen concentration throughout the underdrain is assumed to average approximately 10% of atmospheric.

The constituents in the water (mainly ferrous iron, but including others) will be affected by this partial aeration. The redox potential of the water will change significantly due to the dissolved oxygen. This will greatly affect the geochemical stability of the underdrain water and result in precipitation of ferric iron hydroxides. This would occur in two steps: (1) the ferrous iron must be oxidized to ferric iron, which is the slowest step kinetically, and therefore determines the overall rate of the reaction: and (2) the ferric iron ion will hydrolyze some water present, forming a ferric hydroxide precipitate. At low pH (pH=2 to 3), the overall reaction is very slow, on the order of many thousands of years. At a pH of 4, it would take approximately 300 years for 50% of the ferrous iron to be oxidized to ferric hydroxide. At a moderate pH of 6, 50% oxidation of ferrous to ferric hydroxide requires approximately 275 hours or 11.5 days. In 30 days at a pH of 6, approximately 81% of the ferrous iron would oxidize to ferric hydroxide precipitate. This precipitate has the potential to plug the pore space within the underdrain drainage rock, eventually potentially causing the saturation of the dam.

After exiting the underdrain, the oxygenated water will be collected and pumped back up to the tailings impoundment. There, it will eventually flow downward through the mine tailings again, resulting in reduction of any precipitates that may have formed. The flow through the tailings is estimated to take approximately 11 years, which is sufficiently long to reduce any iron present in the pore water back to soluble ferrous iron.

There are significant mitigating factors which will partially alleviate the problem of precipitation within the underdrain. These mitigating factors are discussed below.

Mitigating Factors

There are several factors which mitigate the release of precipitated material from the water in the underdrain. Firstly, the residence time in the underdrain is approximately one day, assuming a 500-foot long by 50-foot wide by 6-foot high underdrain with 30% porosity, and underdrain flow of 250 gpm. While the aerated water is thermodynamically favored to contain ferric iron, the kinetics of the oxidation of ferrous iron to ferric iron are not instantaneous. The hydrolysis by ferric ion and formation of iron hydroxide precipitate is also not instantaneous, but is much faster than the oxidation of ferrous iron to ferric iron. The overall reaction kinetics are significantly affected by the pH of the aerated water. At a low pH, the kinetics of the oxidation of ferrous iron to ferric iron are extremely slow, slow enough that no appreciably precipitation will occur in the 1-day window. At moderate pH the oxidation of ferrous iron to ferric iron is much faster, as discussed above. Thus, a lower pH will not favor the formation of a ferric iron precipitate.

A lower temperature in the underdrain also results in a slower reaction time. A drop from 25°C (77°F) to 5°C (41°F) results in approximately a slowing of the reaction rate by 10 times. As the tailings impoundment is in a cold part of the world, it could be assumed that 5°C is more likely to be the average temperature than 25°C (77°F), which is the temperature at which the Phreeqc (see references) geochemical simulations were run. In fact, the tailings impoundment is likely to vary between approximately -2°C (28.4°F) and 6°C (42.8°F), so assuming 5°C (41°F) is reasonable.

Lower levels of oxygen present in the underdrain will result in a slower and less complete ferrous iron oxidation reaction as well. Deep in the dam, at the beginning of the underdrain, the dissolved oxygen levels will be much lower, resulting in a much slower ferrous iron oxidation reaction to begin with. As the water flows toward the exit of the drain, the dissolved oxygen level will increase, allowing for a faster ferrous iron oxidation reaction. In addition, there may be times when the underdrain is full of water, not allowing any significant inflow of air from the exit of the underdrain. At these times, oxidation would be minimal, based only on air that flows in from the surrounding drain rock.

The ferric hydroxide precipitate that does form will initially form as an amorphous hydrous ferric hydroxide substance (“yellowboy”) with significant quantities of entrained water, similar in consistency to loose gelatin. It is believed that much of this precipitate will be “swept” out of the underdrain by the water movement. As the precipitate dries out somewhat both inside and outside the underdrain, it will solidify more and form a more crystalline precipitate. As time goes on, other forms of ferric hydroxide may form including maghemite, hematite, and goethite. These substances are denser and more crystalline, but will take many years to form, thus are not a likely problem given the 1-day retention time in the underdrain.

Limitations of this analysis

There is currently not enough chemistry data with respect to temporal changes in the tailings and tailings pore water chemistry. The results of kinetic tests on the tailings would be useful to determine sulfide oxidation rates, neutralization rates, construct a water balance, and examine the possibility of the eventual exhaustion of the reductive capacity of the mine tailings.

Bacterial oxidation of iron was not considered in this analysis. Bacteria are likely to be active at low pHs, below 3. Since it is unlikely that such a low pH will be present, this aspect was ignored.

There were some problems encountered with Phreeqc, the geochemical modeling program used to analyze the system. Specifically, it was not determined how to limit the aeration level to the 10% level. Despite input into Phreeqc that only a certain amount of air was available for equilibration, the program ignored this and allowed full (100%) aeration of the underdrain water. Thus, the thermodynamic conclusions are likely to be overly conservative. However, the thermodynamics were only used to determine which precipitates might form. The bulk of the analysis was based on kinetic considerations. The reduced aeration of 10% was included in the Excel spreadsheet used for kinetics, so the results below are not significantly affected by this shortcoming.

Results:

The following assumptions were made in the underdrain analysis:

- 50% "flushing effect" (i.e. half of all precipitate that forms stays in the underdrain, while the other 50% is flushed out to be pumped back above the dam)
- 10% uniform aeration
- no bacterial influence
- formed precipitate specific gravity of 2.4 g/cm3 (see CRC Handbook)
- 500-foot log by 50-foot wide by 6-foot high underdrain with void space of 30%,
- 250 gpm underdrain flow rate.

It is estimated that these underdrain dimensions and flow rate will result in a retention time of water in the underdrain of approximately 1 day. The following times were obtained for the plugging of the underdrain:

- pH 5 and temperature 5°C (41°F) - the underdrain will be fully plugged in 15.33 million years. This is considered an optimistic assumption of what could happen.
- pH 6 and temperature 25°C (77°F) - the underdrain will be fully plugged in 16,506 years. This is considered a conservative "worst case" scenario.
- pH 6.3 (the current average pumpback water pH) and temperature 5°C (41°F) - the underdrain will be fully plugged in 40,372 years. This is considered to be what is currently occurring.
- pH 6 and temperature 5°C (41°F) - the underdrain will be fully plugged in 160,282 years. This is considered a reasonable scenario for what will actually happen in the future.

Currently, the average pumpback water pH is 6.27, however, once the mine is closed it is reasonable to assume that the pumpback water pH will drop to a slightly lower level due to a minor amount of oxidation of the mine tailings and formation of sulfuric acid. This effect was not explored in this analysis, but is typically seen in acid generating mine tailings. Even if the tailings impoundment is maintained in a flooded state, the 250 gpm pumpback of oxidized water would slowly oxidize some of the mine tailings, dropping the pH to an assumed value of 6.0.

The kinetics were also checked for a longer underdrain retention time of 30 days (i.e. a much slower underdrain flowrate which could occur if the tailings were capped and if the only water to run through was runoff water), otherwise using all the same assumptions. These results are as follows:

- pH 5 and temperature 5°C (41°F) - the underdrain will be fully plugged in 280,266 years if the tailings pond were capped.
- pH 6 and temperature 25°C (77°F) - the underdrain will be fully plugged in 614 years if the tailings pond were capped.
- pH 6.3 (the current average pumpback water pH) and temperature 5°C (41°F) - the underdrain will be fully plugged in 982 years if the tailings pond were capped.

- pH 6 and temperature 5°C (41°F) - the underdrain will be fully plugged in 3,035 years if the tailings pond were capped.

CONCLUSION:

Based on the results of geochemical modeling using limited pumpback water chemistry data, iron hydroxides are likely to precipitate within the underdrain. Based on the physical characteristics and properties of the underdrain, the kinetics of ferrous to ferric iron oxidation and subsequent ferric iron hydrolysis to ferric iron hydroxide, it can be assumed that the underdrain will eventually plug.

The calculated amount of time for this plugging to occur is dependent on the assumptions used and is primarily dependent on the pH of the underdrain water and retention time in the underdrain:

- A reasonable set of assumptions results in a plugging time of 160,282 years.
- A conservative “worst case” scenario results in a plugging time of 16,506 years.

These conclusions have not taken into account geochemical changes to the tailings, tailings water, tailings pore water or underdrain water quality over time. Additional information addressing these long-term geochemical changes is required to increase the degree of certainty in these predicted times.

Alternately, if the tailings impoundment is capped, it could be expected that the underdrain would plug at a faster rate, due to the longer retention time in the underdrain:

- A reasonable set of assumptions results in a plugging time of 3,035 years.
- A conservative “worst case” scenario results in a plugging time of 982 years.

Supporting Material and References:

"PHREEQC (Version 2)--A Computer Program for Speciation, Batch-Reaction, One-Dimensional Transport, and Inverse Geochemical Calculations" put out by USGS, freeware. http://wwwbrr.cr.usgs.gov/projects/GWC_coupled/phreeqc/index.html

SRK, 2006, "Red Dog Mine Closure Options, Volume 1 – Tailings," for Teck Cominco Alaska (TCAK), March 2006.

Yanful, E. K. and Samad, M. A., 2004, "Draft Final Report of Hydraulic Design of Tailings Water Cover at Red Dog Waste Management Area" for Teck Cominco Alaska (TCAK), Geotechnical Research center, Department of Civil and Environmental Engineering, University of Western Ontario, London ON, Canada, N6A 5B9, June 2004.

Inputs for Phreeqc model:

Took instrument readings from excel spreadsheets for pumpback water, took average, and plugged that into Phreeqc. Also checked based on 90% worst-case, but the same species precipitated. Pumpback water measurements were collected from January 2000 through April 2006.

Used wateq4f thermodynamic database.

Constituents not present in the database were ignored (but were only present in ug/L concentrations)

Equilibrated this solution with differing amounts of atmospheric oxygen and CO2 to simulate different levels of aeration.

Also equilibrated aerated solution with pyrite, barite, quartz, galena, and sphalerite. This simulates flow of water through the mine tailings.

Also aerated the water which was simulated to run through the mine tailings.

Results:

Pumpback water which is aerated to a 10% level resulted in precipitation of $Fe(OH)_3$ and $Fe(OH)_{2.7}Cl_{0.3}$, goethite, hematite, jarosites, and pyrolusite.

Water which has flowed through the tailings, then been aerated to 10% level resulted in similar precipitates.

Aeration to higher levels resulted in similar precipitates.

Reduced water which was not aerated had a different set of possible precipitates, such as several sulfide species. It would be unlikely for those precipitates to actually form in the underdrain, as they would have already precipitated within the tailings, and these precipitates would not form when any oxygen is present.

Note that these Phreeqc results are thermodynamic ONLY, and do not take into account the kinetics (which in this case are important). To predict the kinetics, an Excel spreadsheet was produced which predicted the oxidation of ferrous iron to ferric hydroxide at various pH. The operative equation in the Excel spreadsheet is:

- $d[Fe^{2+}] / dt = -k [Fe^{2+}] [O_{2\ (aq)}] [OH-]^2$ where k=1.5e16 L^3/mol^3 min.
  (source: http://web.njit.edu/~hsieh/ene670/oxidFE.html)

Appendix E

KLEINFELDER

February 20, 2007
Kleinfelder Project No: 75660

URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101-1616

Attention: Mr. Cecil Urlich

RE: RISER PIPE VIDEO INSPECTION
RED DOG MINE, ALASKA

Dear Cecil:

This letter summarizes the results of Kleinfelder's riser pipe video inspection work at the Red Dog Mine in Alaska. Kleinfelder arrived at the site on Wednesday, September 27, 2006 and completed the work on October 5, 2006.

BACKGROUND



Figure 1

The Red Dog Mine is located in northwestern Alaska approximately 90 miles north of Kotzebue (Figure 1). Two riser pipes were installed during construction of the main tailings pond dam to monitor the water level in the dam soils and to serve as pressure release wells in the event of increased water levels within the dam. Based on the existing plans and conversations with URS, the riser pipes were constructed of HDPE pipe with diameters varying from approximately 5.75 inches to 10 inches. The HDPE pipes daylight on the downstream side, near the crest, of the dam and slope downward in the upstream direction at slopes between approximately 2:1 (H:V) and 2.5:1 (H:V).

Copyright 2007 Kleinfelder, Inc.

VIDEO INSPECTION

Kleinfelder completed an initial video inspection on September 28, 2006 to observe the condition of the east and west riser pipes (Figure 2). The camera inspection system consisted of a motorized "tractor" with a lens that was able to pan side to side and rotate 360. The entire inspection process was recorded on a digital video tape. Compact discs of the video inspection were provided to URS and Teck Cominco following the completion of the work. Due to a constriction in the riser pipes the initial camera inspection was unable to advance passed approximately 20.8 feet in the west riser pipe and 23 feet in the east riser pipe.



Figure 2

URS and Teck Cominco approved additional video inspection work of both of the riser pipes to allow inspection further into the pipes. The additional video inspection work was completed using a manual "push" camera system capable of inspecting pipes for a length of approximately 300 feet.

Video inspection, using the manual push system, of the west and east riser pipes was completed on October 5, 2006. Inspection of the west and east riser pipes consisted of depth measurements of pipe joints, changes in pipe diameter and maximum depth of inspection. Debris consisting of shavings of HDPE pipe, dirt and rocks were observed throughout the inspected pipe lengths. The inspection of the west riser pipe was completed to a depth of 297.5 feet which was the maximum reach of the camera system. Inspection of the east riser pipe was completed to a depth of 133 feet and was terminated due to dirt, rock debris and what appeared to be a geotextile fabric blocking the entire pipe (Figure 3). No water was observed in either riser pipe at the time of the inspections.



Figure 3

The video inspection and a spreadsheet noting recording times, depths and comments were documented on compact disc. Two copies of the compact disc were delivered to URS for distribution.

Copyright 2007 Kleinfelder, Inc.

Kleinfelder appreciates the opportunity to assist URS with the main tailings dam riser pipe video inspection and looks forward to working with you again in the future.

Sincerely,

KLEINFELDER INC.

Dana Divine for

Chris W. Allen, L.G.
Water Resources Specialist

Jim Bailey

Jim Bailey, L.HG.
Well Services Director

Kleinfelder Inc.
2405 140th Avenue NE, Suite A101
Bellevue, WA 98005
Telephone: (425) 562-4200
Fax: (425) 562-4201

Copyright 2007 Kleinfelder, Inc.

Appendix F

FINAL TECHNICAL SPECIFICATIONS
FUTURE RAISES TO CLOSURE
RED DOG TAILINGS DAM
RED DOG MINE, ALASKA

For

TECK COMINCO ALASKA, INC.
URS JOB NO.: 33755331
November 26, 2007

FINAL SPECIFICATIONS

RED DOG TAILINGS IMPOUNDMENT
FUTURE RAISES TO CLOSURE

Prepared by
URS Corporation
Century Square
1501 4th Avenue, Suite 1400
Seattle, Washington 98101-1616
Telephone: 206/438-2700

Prepared for
Teck Cominco Alaska Inc.
Red Dog Mine, 3105 Lakeshore Drive,
Building A, Suite 101, Anchorage, AK 99517

November 16, 2007

TABLE OF CONTENTS

DIVISION 1 GENERAL REQUIREMENTS

01110 SUMMARY WORK

01330 SUBMITTAL PROCEDURES

01450 QUALITY CONTROL

DIVISION 2 SITEWORK

02020 SITE CONDITIONS

02050 BLASTING

02102 CLEARING, STRIPPING, AND GRUBBING

02140 DEWATERING

02151 SHORING

02200 EARTHWORK

02220 TRENCH EXCAVATION, BACKFILLING, AND COMPACTION

02225 EXCAVATION

02274 LINER BEDDING, LINER COVER, AND LINER ANCHORAGE

02276 GEOTEXTILES

02277 GEOSYNTHETIC CLAY LINER

02770 GEOMEMBRANE

02771 GEOMEMBRANE CURTAIN WALL

DIVISION 3 CONCRETE

03310 CONTROLLED DENSITY FILL (CDF)

03600 GROUT

DIVISION 13 SPECIAL CONSTRUCTION

13340 INSTRUMENTATION

SECTION 01110
SUMMARY OF WORK

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A. All Sections

1.02 RELATED DOCUMENTS

A. Contract Documents including all General and Supplementary Conditions prepared by Teck Cominco Alaska, Inc.

B. URS Corporation, "Conceptual Design Drawings for Red Dog Mine Tailings Dam, Future Raises to Closure", for Teck Cominco Alaska Inc., Sheets TDC-1 through TDC-24, November 26, 2007.

C. URS Corporation, "Preliminary Technical Specifications for Future Raises to Closure, Red Dog Mine Tailings Dam", for Teck Cominco Alaska, Inc., issued November 26, 2007 and subsequent revisions.

D. Guidelines for Cooperation with the Alaska Dam Safety Program, prepared by Dam Safety and Construction Unit, Water Resources Section, Division of Mining, Land and Water, Alaska Department of Natural Resources (ADNR) dated June 30, 2005.

1.03 DESCRIPTION

A. This section summarizes the work required for construction of the Red Dog Mine tailings dam Future Raises to Closure and the Parties of the Work, as defined in Paragraph 1.04 of this Section.

1.04 PARTIES OF THE WORK

A. The successful completion of the construction of the different elements of the Future Raises to Closure will depend on the interaction and cooperation of many parties. The following parties are represented in the project:

A. **Owner:** The Owner is Teck Cominco Alaska, Inc. (TCAK) of Anchorage. The Owner will complete site preparations before construction; purchase the geosynthetic, concrete and instrumentation materials; select the Contractor; provide earthwork, geosynthetic and concrete materials to the Contractor; provide a geosynthetic material storage area, concrete batch plant, and soil testing laboratory for Contractors use; and install instrumentation after construction.

B. **Engineer**: The Engineer is URS Corporation (URS) of Seattle. The Engineer designed the raise and is also referred to as the Geotechnical Engineer. The Engineer is responsible for reviewing Contractor submittals pertaining to the design, implementing the CQA program, confirming the cutoff wall and wing wall excavation depths, completing the construction completion report, and certifying that construction is completed in compliance with the Plans and Specifications.

C. **Construction Manager**: The Construction Manager (CM) is TCAK of Anchorage. The CM is responsible for coordinating and monitoring Contractor activities, reviewing Contractor submittals and identifying those that require review by the Engineer and CQA Field Manager, processing the submittals and providing responses to the Contractor, coordinating the CQA field monitoring program, and directing the site preparation and instrumentation activities.

D. **Contractor**: The Contractor will be selected by TCAK. The Contractor is responsible for constructing the earthwork, liner system, cutoff system and wing wall for the raise, installing the geosynthetic and concrete materials, supplying a portable tensiometer for geosynthetics testing, and repairing any damage that may be caused to the existing embankment, liner and cutoff systems as a result of the Stage VII-B construction.

E. **Manufacturer**: The geosynthetic material Manufacturer will manufacture and supply the geosynthetic materials, and provide QC documentation and installation guidelines. The materials will be purchased by the Owner and installed by the Contractor. Approved manufacturers/suppliers are GSE Lining Technology, Inc., Houston, TX, 800.435.2008; and Northwest Linings & Geotextile Products Inc., Kent, WA, 253.872.0244.

F. **CQA Personnel**: CQA Personnel for the raise will consist of a CQA Project Director, a CQA Field Manager, CQA Field Monitors, and a CQA Document Controller. The CQA Project Director and CQA Field Manager, CQA Field Monitors and CQA Document Controller are employees of URS. URS will be contracted by the Owner to provide the CQA personnel.

G. **Laboratory**: The Laboratory consists of the Alaska Testlab (ATL) laboratory in Anchorage, the Owner's on-site testing facility, URS's on-site density/moisture gage, the soil and concrete testing lab provided by the Owner and the Contractor's on-site tensiometer. The Laboratory is responsible for testing to confirm that the earthwork, geosynthetics and concrete materials used in the construction comply with the Specifications.

H. **Surveyor**: The Surveyor will be selected by TCAK. The Surveyor will be responsible for the construction surveying. The Surveyor will confirm the raise stationing, set up the wing wall stationing, complete panel layouts of all liner, cutoff wall and wing wall components, prepare plans and profiles of the cutoff and curtain wall alignments and terminations, and prepare an as-built topographic survey of the tailings dam.

1.05 PROJECT DESCRIPTION

A. The Future Raises to Closure will be conducted in three stages for the purpose of its conceptual design. These three stages are:

1. From Stage VII-B (El.960) to Stage VIII (El. 970)

2. From Stage VIII (El.970) to Stage IX (El.980)

3. From Stage IX (El.980) to Stage X at Closure (El.986).

B. Before the start of Future Raises to Closure, the dam was completed through Stage VII-B to Elevation 960 feet (El. 960). The Future Raises to Closure include extending the cutoff wall, raising the main embankment and wing wall, and inserting additional instrumentation as deemed appropriate.

C. Work to be completed as part of the Future Raises to Closure of the Red Dog Mine tailings dam construction includes the following:

1. Site Preparation;

2. Cutoff System,

3. Wing Wall System,

4. Liner System,

5. Embankment, and

6. Instrumentation.

D. Site preparation will be completed by the Owner and the Contractor with the support of CQA personnel. Currently, it is anticipated that select components of the mill infrastructure will need to be relocated. Future raises may require construction of additional coffer dams in order to extend the existing cutoff wall toward the west abutment and to extend the curtain wall to the south. The Owner will ensure that this takes place prior to the construction of future raises when it is appropriate.

E. The Future Raises to Closure embankment will be constructed using rock fill materials that will be produced at the mine and placed by conventional earthwork delivery, spreading and compaction procedures. These rock fill materials are designated as soil types as follows:

1. Soil Type 1 – Random Rockfill (24-inch)

2. Soil Type 2 – Select Fill (24-inch)

3. Soil Type 3 – Processed Select (3-inch)

4. Soil Type 4 – Processed Select (1-inch)

5. Soil Type 7 – Select (12-inch)

6. Soil Type 8 – Kivalina Shale (Select fill material)

7. Soil Type 9 – Random Mine Waste Fill

C. Liner System (material purchase by Owner, installation by Contractor): The liner system will be an extension of the existing liner system. The existing system terminates at the present west abutment, and at the west side of the pipeline utilidor near the east abutment. The liner system will be constructed from bottom to top as follows, with the purpose of each soil type shown in parentheses:

1. Soil Type 1 (Random Rockfill)

2. Soil Type 7 (Transition Rockfill)

3. 10 oz/yd2 Geotextile (Type 1 Geotextile)

4. Soil Type 3 (Filter Drain)

5. 16 oz/yd2 Geotextile (Type 2 Geotextile)

6. Soil Type 4 (Liner Bedding)

7. 100 mil High Density Polyethylene (HDPE) Geomembrane

8. Soil Type 4 (Liner Cover)

9. Soil Type 1 (Protective Buttress).

F. The bottom of the new geomembrane will be welded to the top of the geomembrane in the existing liner system, and to the top of the geomembrane part of the cutoff system outside the existing liner system.

G. The Future Raises to Closure cutoff system will be a cutoff trench and cutoff wall that will extend from the existing cutoff trench and cutoff wall at the west abutment. The existing cutoff trench and cutoff wall terminates at the west extent of the existing dam. The cutoff system will be extended in the following sequence:

1. Excavation to reveal the existing cutoff wall terminus;

2. Excavation of a trench to bedrock,

3. Installation of 100 mil HDPE Geomembrane Liner with vertical support system,

4. Grout to predetermined depth,

5. Backfill with Soil Type 4 (Liner Cover),

6. Backfill with Soil Type 1 (Random Rockfill)

H. The wing wall system consists of below ground and above ground components, the curtain wall and wing wall respectively. The wing wall system is an extension of the main tailings dam where the dam axis changes from a west-east alignment to a northwest-southeast alignment. The curtain wall is a vertical cutoff system constructed of 100 mil HDPE liner. Construction of the curtain wall is described in Section 1.2.5.1; the wing wall will be constructed in the following sequence:

1. Soil Type 1 (Random Rockfill)

2. Soil Type 7 (Transition Rockfill)

3. 10 oz/yd2 Geotextile (Type 1 Geotextile)

4. 16 oz/yd2 Geotextile (Type 2 Geotextile)

5. Soil Type 4 (Liner Bedding)

6. 100 mil High Density Polyethylene (HDPE) Geomembrane

7. Soil Type 4 (Liner Cover)

I. The seepage cutoff for the Future Raises to Closure will be an extension of the Stage VII-B curtain wall. The curtain wall the vertical part of the wing wall system that extends below El. 960 along the wing wall alignment. It was constructed of GSE CurtainWall panels and will be extended for Stages VIII, IX and X. It may be extended using the same GSE CurtainWall panels; if so, the construction sequence will be as follows:

1. Site preparation including snow clearing and work bench excavation

2. Trench excavation to bedrock

3. Curtain panel deployment and attachment to framing

4. Vertical suspension of curtain panel(s) in the trench

5. CDF pour and set

6. Soil Type 4 backfill (Liner Cover)

7. Soil Type 7 backfill (Downstream Fill)

8. Soil Type 9 backfill (Upstream Fill)

9. Soil Type 1 (Protective Buttress).

J. Instrumentation: After the Contractor completes the construction of the Future Raises to Closure raise, the Owner will install a piezometers and thermistors as agreed to monitor the dam's performance.

1.06 CODES AND REGULATIONS

A. Work shall meet the requirements of applicable laws, statutes, regulations, ordinances, safety regulations of federal, state, and county jurisdictions and as may be further referenced in the Contract Documents.

B. Comply with provisions of federal, state, and local statutes, ordinances, and regulations dealing with the prevention of environmental pollution of natural resources that affect the project.

1.11 QUALITY CONTROL

A. Section 01450 – Quality Control.

PART 2 – PRODUCTS (NOT USED)

PART 3 – EXECUTION (NOT USED)

END OF SECTION 01110

SECTION 01330
SUBMITTAL PROCEDURES

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Blasting:	Section 02050
B.	Dewatering:	Section 02140
C.	Shoring:	Section 02151
D.	Earthwork:	Section 02200
E.	Excavation:	Section 02225
F.	Geomembrane:	Section 02770
G.	Curtain Wall:	Section 02771
H.	Controlled Density Fill (CDF):	Section 03310
I.	Grout:	Section 03600

1.02 DESCRIPTION

A. This section includes the administrative and procedural requirements for submittals, specifically:

1. Submittal procedures, requirements and variations from Contract Documents.
2. Progress schedules.
3. Proposed products list.
4. Shop drawings.
5. Product data and samples.
6. Test reports.

1.03 SUBMITTAL PROCEDURES

A. Contractor shall make all submittals in accordance with this procedure.

B. Contractor shall certify all submittals for accuracy, completeness, and compliance with contract requirements. Contractor shall indicate approval on each submittal as evidence of coordination and review.

C. Shop drawings submitted without evidence of the Contractor's approval will not be reviewed.

D. Where construction or erection procedures require design and calculations, a Professional Engineer licensed in the state of Alaska shall perform these services, affix seal, and sign.

E. The review of submittals by the Construction Manager will be limited to general design requirements only and shall in no way relieve the Contractor from responsibility for errors or omissions contained therein.

F. Acceptance of submittals by the Construction Manager shall not relieve the Contractor from responsibility for the safety of their methods or equipments or from responsibility for complying with the requirements of all applicable codes and of this contract, except with respect to specifically approved variations.

G. Work done prior to the Construction Manager's approval shall be at the Contractor's risk.

H. Submittals shall be acted upon by the Construction Manager as promptly as possible, and returned to the Contractor. Delays, re-sequencing or other impacts to work resulting from Contractor's submission of unchecked or not reviewed, incomplete, inaccurate or erroneous, or nonconforming submittals, which will require Contractor's resubmission of a submittal for Construction Manager's review, shall not constitute a basis of claim for adjustment in Contract Price or Contract Time.

I. Construction Manager reserves the right to withhold action on a submittal requiring coordination with other submittals until related submittals are received.

J. Provide a "Priority List" when submitting several submittals to the Construction Manager at one time.

K. Submittal sequencing should coincide with the submittal schedule.

L. Contractor will allow for permit coordination with regulatory agencies. Contractor shall obtain information regarding the permit time frame required by the regulatory agencies.

1.04 SUBMITTAL REQUIREMENTS

A. Identify each submittal and re-submittal by showing the following information:

1. Name and address of submitter, name and telephone number of the individual who may be contacted for additional information;
2. Project name as it appears in the Contract Documents;
3. Contractor, subcontractor, or supplier name and address;
4. Drawing sheet and detail number(s) and/or specification(s) section number to which the submittal applies; and
5. Submit only pertinent catalog pages, and mark each copy of standard printed data to identify pertinent products.

B. Submittal Format:

1. Make all shop drawing prints in blue or black line on white background;
2. Size shall be 22 x 36 inches, 8 1/2 x 11 inches, or 11 x 17 inches;
3. Make all shop drawings accurate to a scale sufficiently large to show the pertinent features of the item and its method of connection;
4. Transmit each submittal with a sequentially numbered form approved by the Construction Manager. Resubmittals are to have the original number with an alphabetic suffix;
5. Provide a 4-inch by 5-inch space on each drawing and on each submittal transmittal form for the Construction Manager's review comments.

C. Three of each submittal shall be submitted to the Construction Manager.

1.05 SUBMITTAL REVIEW

A. Make all submittals far enough in advance of scheduled installation to provide time for review and approval, for possible revision and resubmittal, and for orders and delivery.

B. Allow ten working days for the Construction Manager's review and return of submittals or resubmittals.

C. Allow an additional one week for submittals which deviate from the Contract Documents.

D. The Construction Manager will review only those items required by the Contract Documents. Information submitted by the Contractor that is not required will be returned marked "Information Only."

E. In the event a single submittal contains both required and non-required information, only the required information is subject to the Construction Manager's review.

F. Submittals will be checked for conformance with the design concept of the project and compliance with the information given in the Contract Documents.

G. The Construction Manager's review of drawings or data prepared by a Professional Engineer licensed in Alaska will be limited to the submittal's effect on the integrity of the completed project.

H. Submittals will be marked to indicate the result of the Construction Manager 's review, as follows:

1. “NO EXCEPTION TAKEN" - Revision of drawing or data will not be required;
2. "MAKE CORRECTION NOTED" - Contractor shall revise the drawing or data as indicated. Resubmittal is not required;

3. "REVISE AND RESUBMIT" - Contractor shall revise the drawing or data and shall resubmit the revised drawing or data to the Construction Manager for review and approval; or
4. "REJECTED" - Submittal does not conform to Contract Documents. Contractor shall resubmit in a form that conforms to Contract Documents to the Construction Manager for review and approval.

I. Copies marked "NO EXCEPTION TAKEN" or "MAKE CORRECTION NOTED" authorize the Contractor to proceed with construction or fabrication covered by those drawings or data sheets with corrections, if any, incorporated.

J. No revision in any way shall be made after a drawing has been marked "NO EXCEPTION TAKEN" without resubmitting the drawing.

K. When prints of drawings have been marked "REVISE AND RESUBMIT," the Contractor shall make the necessary corrections. Every revision shall be shown by number, date and subject in a revision block, and in addition, each revised drawing shall have its latest revision clearly indicated by clouding around the revised areas on the drawing. Drawings submitted without these indications will be considered nonconforming.

L. The applicable parts of the requirements of the above paragraphs with reference to the drawings shall apply equally to design data, catalog cuts, illustrations, printed specifications, draft reports or any other submittal furnished for review.

1.06 VARIATIONS FROM CONTRACT REQUIREMENTS

A. Variations from the Contract Documents shall be specifically and separately approved by the Construction Manager.

B. If working drawings show variations from the contract requirements, describe such variations in writing and submit for approval separate from the drawings.

1.07 GENERAL SUBMITTAL

A. Submit erosion and sediment control plan prior to starting work.

1.08 OTHER SUBMITTALS

A. All other submittals are presented in the related sections presented in Section 1.01.

1.09 SHOP DRAWINGS

A. Shop Drawings shall be submitted to the Construction Manager to be reviewed.

1.10 TEST REPORTS

A. Submit test reports for information for the limited purpose of assessing conformance with information given and the design concept expressed in the Contract Documents.

1.11 SUBMITTAL REGISTER

A. The Contractor shall prepare a submittal register of required submittals and proposed schedule of submittals with initial project schedule.

B. In the event of a discrepancy between the Contract Documents and the Submittal Register, the Contract Documents shall take precedence.

PART 2 PRODUCTS (NOT USED)

PART 3 EXECUTION (NOT USED)

END OF SECTION 01330

SECTION 01450
QUALITY CONTROL (QC)

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A. All Sections

1.02 GENERAL

A. Quality control services include inspections, tests and related actions including reports performed by independent agencies, government authorities, and the Contractor. They do not include Contract enforcement activities performed by the Construction Manager.

B. Inspection and testing services are required to verify compliance with requirements specified or indicated. These services do not relieve the Contractor of responsibility for compliance with Contract Document requirements.

C. Quality control requirements for individual construction activities are shown in the Sections that specify those activities.

D. Inspections, tests and related actions specified are not intended to limit the Contractor's quality control procedures that facilitate compliance with Contract Document requirements.

E. Requirements for the Contractor to provide quality control services required by the Construction Manager, County, or authorities having jurisdiction are not limited by provisions of this Section.

1.03 DESCRIPTION

A. Provide QC for all work.

B. The planned work and construction activities for the Future Raises to Closure will include the following elements:
   1. Embankment;
   2. Liner System;
   3. Cutoff System; and
   4. Emergency Spillway (only for the final raise).

C. The Future Raises to Closure construction of these elements includes the following activities:

1. Geomembrane manufacture, fabrication, and installation for the liner and cutoff systems.

2. Geosynthetic clay liner (GCL) manufacture, fabrication, and installation for the right abutment cutoff wall.

3. Curtain wall manufacture, fabrication, and installation for the southeast extent of the wing wall system.

4. Geotextile manufacture, fabrication, and installation for the liner system.

5. Earthwork supply, placement, and compaction for the embankment, liner and cutoff systems

6. Grout and CDF manufacture and installation for the cutoff system.

1.04 DEFINITIONS

A. There is often confusion between the meanings of quality control (QC) and quality assurance (QA). This QC Section refers to the provision of QC, in this case specifically referring to the QC of the materials comprising the dam raise. In the context of this document:

1. **Quality Control (QC)** refers to actions taken by the Contractor (including those parties charged with the manufacture, fabrication, delivery, and installation of the geosynthetics and grout, and the placement and compaction of the soils materials), that provide a means to determine and sometimes quantify the characteristics of the product. The results of a QC program are compared to the Specifications or other contractual or regulatory requirements. During each aspect of the handling of these materials, QC is provided by the manufacturer, fabricator, or installer of geosynthetics and grout, or the supplier and earthworks contractor for the soils, to ensure that the materials and workmanship conform to the Drawings and Specifications.

2. **Quality Assurance (QA)** is a planned and systematic pattern of means and actions intended to provide confidence that materials and procedures conform to the Drawings, Specifications, and any applicable regulatory requirements. QA is provided by an independent party on behalf of the Owner. Although QA is as important during all phases of the soils or geosynthetics materials handling, QA refers specifically to those actions taken in relation to the installation of the geosynthetics and grout, and the placement and compaction of the soils materials. CQA is the most critical component of an overall QA program, because field conditions are the most variable and most difficult to control.

1.05 RESPONSBILITIES

A. Construction Manager shall be responsible for construction quality assurance (CQA). The Construction Manager shall be responsible for observing and

documenting periodic verification, checking, or testing for confirming that the quality of the curtain wall is in accordance with the Contract Documents.

B. Contractor's Responsibilities:

1. Contractor shall be responsible for QC. Contractor shall engage and pay for the services of qualified staff or a qualified subcontractor to perform QC for monitoring and documenting the quality of the curtain wall in accordance with the Contract Documents.

2. The Contractor and each agency engaged by the Contractor to perform inspections, tests and similar services shall coordinate the sequence of activities to accommodate required services with a minimal delay.

3. Installer shall be responsible for QC. Installer shall engage and pay for the services of qualified staff or a qualified subcontractor to perform QC for monitoring and documenting the quality of the curtain wall in accordance with the Specifications.

4. The subcontractor and/or staff performing QC on behalf of the Installer shall have appropriate education and prior experience in conducting the specific quality control activities. All QC staff shall be approved prior to starting work at the site by the Construction Manager.

5. The Installer shall complete QC inspection, testing or any other action, as considered necessary by the Installer to ensure that the Work has been completed in accordance with the Drawings and Specifications. Notwithstanding the results of the Contractor's QC program, compliance of the Work with the Drawings and Specifications shall be defined by the results of the Construction Manager's CQA program.

C. Defective Work:

1. Remove and replace any work found defective or not complying with requirements of Contract Documents, at no additional cost.

2. Work will be observed as it progresses, but failure to detect any defective work or materials shall not in any way prevent later rejection when such defect is discovered, nor shall it obligate the Owner for final acceptance.

3. All observation and testing required because of defective work or ill-timed notices shall be performed at the Contractor's expense.

PART 2 – PRODUCTS (NOT USED)

PART 3 – EXECUTION

3.01 REPAIR AND PROTECTION:

A. General: Upon completion of observation, testing, sample taking, and similar services, repair damaged construction and restore substrates and finishes to eliminate deficiencies, including deficiencies in visual qualities of exposed finishes.

B. Protect construction exposed by or for quality control and quality assurance activities, and protect repaired construction.

C. Repair and protection is the Contractor's responsibility, regardless of the assignment of responsibility for observation, testing or similar services.

D. Any Work that does not satisfy the requirements of the Drawings and Specifications shall be made good in accordance with the requirements of the Specification or as directed by the Construction Manager at the sole expense of the Contractor.

END OF SECTION 01450

SECTION 02020
SITE CONDITIONS

PART 1 – GENERAL

1.01 SUBSURFACE CONDITIONS

A. Subsurface profiles and locations of borings are shown on the Drawings.

B. The information is based on data obtained from a limited number of borings and as such is not meant to warranty the complete sub-surface conditions.

C. Prior to bidding, bidders may conduct their own subsurface investigation with the approval of the Owner.

1.02 SURFACE CONDITIONS

A. Surface contours shown on the drawings were obtained from 5-foot contour interval maps dated 2007.

B. The location of such design features as the contact between the top of excavated slopes and the ground surface and the contact between the toe of constructed slopes and the ground surface may vary due to surface irregularities not shown on the Drawings.

1.03 ERRORS OR DISCREPANCIES

A. It is the duty of the Contractor to promptly notify the Construction Manager in writing of any site condition that differs from the condition represented on the Drawings.

END OF SECTION 02020

SECTION 02050
BLASTING

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Site Conditions:	Section 02020
B.	Earthwork:	Section 02200
C.	Trench Excavation, Backfilling, and Compaction:	Section 02220
D.	Excavation:	Section 02225
E.	Quality Control:	Section 01450

1.02 GENERAL CONDITIONS

A. Perform no drilling or blasting work until Dam Contractor's general plan for such operations has been approved by the Construction Manager and Engineer.

B. Conform to the requirements specified by the State of Alaska and Federal Government for transportation, handling, storage and use of explosives.

C. Use only skilled and licensed operators to perform blasting.

1.03 SUBMITTALS

A. Prior to commencing excavation work submit to the Construction Manager and Engineer a general plan of operations for blasting, including controlled blasting techniques and controls for flyrock and ground vibration.

B. Submit to the Construction Manager copies of the original invoices for all blasting materials delivered to the site.

C. Submit to the Construction Manager and Engineer a log of completed blasting activities for each day showing location, size, and depth of blast holes, and quantities of materials loaded into each hole.

PART 2 – PRODUCTS

2.01 EXPLOSIVES

A. Selection of explosives and explosive equipment to be used is at the option of the Dam Contractor.

PART 3 – EXECUTION

3.01 GENERAL

A. Select the drilling and blasting pattern, types and quantities of explosives and delay systems.

B. The Engineer will observe the loading of drill holes to verify conformance with the blasting plans prior to loading each drill hole.

C. Control blasting to produce sound rock faces at the final excavation lines. The type, distribution, and quantity of explosives detonated shall be such that existing fractures shall not be opened and new fractures shall not be created.

3.02 OVERSHOOTING

A. Perform blasting and excavation to the lines and grades shown on the Drawings or as directed by the Engineer.

B. Excessive blasting, overshooting and the corrective measures required by the Engineer will be at the Dam Contractor's expense.

C. Backfill overexcavated areas in accordance with Section 02200.

END OF SECTION 02050

SECTION 02102
CLEARING, STRIPPING, AND GRUBBING

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Quality Control	Section 01450
B.	Earthwork	Section 02200
C.	Trench Excavation and Backfill	Section 02220
D.	Excavation	Section 02225

1.02 DESCRIPTION

A. This section includes the clearing, ice, snow and woody debris, stripping, and grubbing of trees, brush, vegetation, in the vicinity of the dam sites. The focus will be on clearing snow and ice from the site prior to and during construction activities taking place for completing the wing wall construction.

B. **Clearing** includes the removal and disposal of all trees, snags, stumps, shrubs, and brush greater than 12 inches in height and rocks greater than 24 inches in diameter.

C. **Stripping** includes removal of all ice, snow and woody debris from areas designated for the wing wall excavation.

D. **Grubbing** includes removal and disposal of wood and root matter to a depth of 18 inches below the ground surface and no grubbing is currently anticipated.

1.03 PROTECTION

A. Perform all clearing, stripping and grubbing work in conformance with applicable codes and regulations of the State of Alaska and federal agencies.

B. Comply with all required permits and regulations.

PART 2 – PRODUCTS

2.01 EQUIPMENT

A. Selection of equipment to be used is at the option of the Contractor.

PART 3 – EXECUTION (Not used)

END OF SECTION 02102

SECTION 02140
DEWATERING

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Quality Control	Section 01450
B.	Site Conditions	Section 02020
C.	Trench Excavation and Backfill	Section 02220
D.	Excavation	Section 02225

1.02 DESCRIPTION

A. Provide personnel, equipment, and structures necessary to remove and treat water from work areas and to maintain a dry work area.

1.03 SUBMITTALS

A. Prior to commencing earthwork, submit to the Construction Manager a general dewatering plan.

PART 2 – PRODUCTS

1.04 EQUIPMENT

A. Selection of equipment to be used is at the discretion of the Contractor. A sump system and or drainage gallery may be necessary to ensure proper drainage is achieved.

PART 3 – EXECUTION

1.05 GENERAL

A. Maintain dry conditions in the dam foundation and trench excavations.

B. Prior to placement of fill materials, liner materials, geotextiles, remove all ponded water, mudding conditions and stop water inflows from work area.

C. Perform dewatering operations to the satisfaction of the Construction Manager and the CQA field manager.

1.06 WATER TREATMENT

A. Treat water as required and in accordance with applicable federal and State of Alaska regulations before discharging. Water will be discharged directly to the tailings pond and subsequently treated.

END OF SECTION 02140

SECTION 02151
SHORING

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Quality Control	Section 01450
B.	Site Conditions	Section 02020
C.	Dewatering	Section 02140
D.	Earthwork	Section 02200
E.	Trench Excavation and Backfill	Section 02220
F.	Excavation	Section 02225

1.02 SAFETY

A. Contractor is responsible for ensuring that all excavations are adequately shored, braced or sloped to provide working conditions that are safe and free from the hazard of caving or sloughing.

B. Contractor is responsible for ensuring that all excavations and shoring meet the applicable regulations of MSHA, OSHA, State of Alaska or other governing regulatory agency.

1.03 SUBMITTALS

A. Develop an excavation and shoring plan showing the proposed shoring or bracing, designed and approved by a civil/structural engineer registered in the State of Alaska and bearing the engineer's stamp.

B. Submit the excavation and shoring plan to the Construction Manager according to Section 01330, at least 14 days prior to starting any excavation work.

PART 2 – PRODUCTS

2.01 MATERIALS

A. Use structural elements in shoring and bracing operations that are free of defects and of adequate strength to provide complete support of the excavations as shown in the excavation and shoring plan.

B. Materials used in the shoring operations shall be at the discretion of the Contractor.

PART 3 – EXECUTION

3.01 GENERAL

A. Install shoring in accordance with the excavation and shoring plan.

END OF SECTION 02151

SECTION 02200
EARTHWORK

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Site Conditions:	Section 02020
B.	Clearing, Stripping, and Grubbing:	Section 02102
C.	Dewatering:	Section 02140
D.	Trench Excavation, Backfilling, and Compaction:	Section 02220
E.	Excavation:	Section 02225
F.	Filter Drain, Liner Bedding, Liner Cover, Liner Anchorage:	Section 02274
G.	Tailings Dam:	Section 02290
H.	Quality Control:	Section 01450

1.02 DESCRIPTION

A. This section describes all soil material types and placement methods to be used in the construction of the tailings dam, and associated features.

B. This section describes the quality control procedures to be used by the Contractor and approved by the Engineer during construction of the project.

1.03 QUALITY CONTROL METHODS

A. Soils and Backfill: Moisture density standard ASTM D1557 or AASHTO T-180 method unless otherwise specifically stated.

B. In-place Density Determination: Sandcone method ASTM D1556 or Nuclear Method ASTM D2922.

C. Classification of Soils: ASTM D2487.

D. Rockfill Durability: Los Angeles Abrasion AASHTO T 96 or ASTM C535 and Freeze-Thaw Durability ASTM C666.

E. Liquid Limit, Plastic Limit, and Plasticity Index of Soils: ASTM D4318.

1.04 SUBMITTALS

A. Prior to the start of earthwork, the Contractor shall submit a pre-construction plan and schedule to the Construction Manager for approval. The plan shall include a description of methods to be used for all excavation, backfill, placement, compaction, and grading operations for each soil type.

B. The CQA Field Manager will review this plan for conformance with the Specifications.

PART 2 – PRODUCTS

2.01 GENERAL

A. The Engineer will classify excavated materials to be used for the construction of the tailings dam into one of seven types.

B. Test each material source in accordance with EPA Toxicity standards. All materials downstream or at an elevation above the liner shall be non-toxic.

C. Crushed material will consist of evenly graded stones that are hard, angular, and have no more than 50% wear at 500 revolutions as determined by AASHTO T 96 or ASTM C535. The material will have no more than 25% loss after 25 freeze-thaw cycles using ASTM C666.

D. Do not use Kivalina shales in construction, unless clearly specified.

2.02 SOIL TYPE 1, RANDOM ROCKFILL (24 INCH)

A. Obtain soil type 1, moderately weathered to fresher cherts, shales, and sandstones, from the Millsite area or from mine waste material.

B. Use materials conforming to the following specifications:

1. Maximum 24-inch particle size;

2. Well-graded material consisting primarily of gravels and cobbles;

3. Less than 25% passing the No. 200 sieve; and

4. Less than 5% ice by volume.

C. Contingent upon approval of the Engineer, soil type 1 having less than 5% passing the No. 200 sieve may have an average of up to 1% ice by volume.

D. Soil type 1 having less than 20% passing the No. 200 sieve and up to 15% ice by volume may be used in construction of the upstream rock buttress.

2.03 SOIL TYPE 2, SELECT (24 INCH)

A. Obtain soil type 2, a slightly weathered to fresh select chert-carbonate material, and sandstones from the millsite area.

B. Do not use mine waste material as soil type 2.

C. Use material conforming to the following specifications:

1. Maximum 24-inch particle size;

2. Well-graded material consisting primarily of gravels and cobbles;

3. Less than 15% passing the No. 200 sieve; and

4. Less than 5% ice by volume.

D. Contingent upon approval of the Engineer, soil type 2 having less than 5% passing the No. 200 sieve may have an average of up to 10% ice by volume.

2.04 SOIL TYPE 3, PROCESSED SELECT (3 INCH)

A. Obtain soil type 3, a slightly weathered to fresh chert-carbonate material, and sandstones from the millsite area or from the alluvial/colluvial materials found in the stream bed.

B. Crush and process millsite materials to:

1. A well-graded aggregate with a maximum particle size of 3-inch;

2. Less than 5% finer than the No. 200 sieve; and

3. Less than 5% ice by volume.

C. Crush and/or process alluvial/colluvial materials to:

1. A well-graded aggregate with a maximum particle size of 3-inch;

2. Less than 5% finer than the No. 200 sieve; and

3. Less than 5% ice by volume.

D. Contingent upon approval of the Engineer, soil type 3 ' having less than 5% passing the No. 200 sieve may have an average of up to 10% ice by volume.

2.05 SOIL TYPE 4, PROCESSED SELECT (1 INCH)

A. Obtain soil type 4, a slightly weathered to fresher chert-carbonate material, and sandstones from the millsite area, or from the alluvial/colluvial materials found in the stream bed.

B. Crush and process millsite materials to:

1. A well-graded aggregate with a maximum particle size of 1-inch;

2. Less than 10% finer than the No. 200 sieve; and

3. Less than 5% ice by volume.

C. Crush and/or process alluvial/colluvial materials to:

1. A well-graded aggregate with a maximum particle size of 1-inch;

2. Less than 10% finer than the No. 200 sieve; and

3. Less than 5% ice by volume.

D. Contingent upon approval of the Engineer, soil type 4 having less than 5% passing the No. 200 sieve may have an average of up to 10% ice by volume.

2.06 SOIL TYPE 5, ROCK DRAIN AND RIPRAP

A. Obtain soil type 5 from mill site area.

B. Use slightly weathered to fresh processed chert-carbonate con-forming to the following specifications:

1. Greater than 85% of material between 6 and 18 inches;

2. Less than 15% total accumulated materials smaller than 6 inches; and

3. Less than 10% ice by volume.

C. Contingent upon approval of the Engineer, soil type 5 may have an average of up to 10% ice by volume.

2.07 SOIL TYPE 6, RANDOM ROCKFILL (12 INCH)

A. Obtain soil type 6, moderately weathered to fresher cherts, shales, and sandstones, from the millsite area, from the mine waste material.

B. Use materials conforming to the following specifications:

1. Maximum 12-inch particle size;

2. Well-graded material consisting primarily of gravels and cobbles;

3. Less than 25% passing No. 200 sieve; and

4. Less than 5% ice by volume.

C. Contingent upon approval of the Engineer, soil type 6 having less than 5% passing the No. 200 sieve may have an average of up to 10% ice by volume.

2.08 SOIL TYPE 7, SELECT (12 INCH)

A. Obtain soil type 7, a slightly weathered to fresh select chert-carbonate material, and sandstones from the millsite area.

B. Do not use mine waste materials as soil type 7.

C. Use material conforming to following specifications:

1. Maximum 12-inch particle size;

2. Well-graded material consisting primarily of gravels and cobbles;

3. Less than 15% passing the No. 200 sieve; and

4. Less than 5% ice by volume.

D. Contingent upon approval of the Engineer, soil type 7 having less than 5% passing the No. 200 sieve may have an average of up to 12% ice by volume.

2.09 SOIL TYPE 9, RANDOM MINE WASTE (1-INCH)

A. Obtain soil type 9, from mine waste or the overburden stockpile area south of the tailings impoundment.

B. Use materials conforming to the following specifications:

1. Maximum 1-inch particle size;

2. Well-graded material consisting primarily of gravels and cobbles;

3. Less than 15% water content by weight; and

4. No visually detectable ice.

C. Contingent upon approval of the CQA Field Manager in consultation with the Engineer, soil type 9 having more less than 5% passing the No. 200 sieve have an average of up to 10% ice by volume .

PART 3 – EXECUTION

3.01 FINISH ELEVATIONS

A. Contours used to illustrate the final top of the backfilled surface of the cutoff trench are intended to show approximate final elevation for drainage control and may be adjusted with the written approval of the Engineer.

B. Establish a general survey control grid at a spacing of 25 feet in each direction in order to establish surface contours before and after construction.

C. Finished embankment surfaces must be within 0.5 feet of the elevations and grades shown on the Drawings.

3.02 SOIL MATERIAL CONTROL

D. The Engineer will classify all materials removed from excavations or delivered to stockpiles as appropriate or inappropriate for use in construction.

E. Stockpile materials that are acceptable as soil material types 1 through 7 in areas designated by the Construction Manager.

F. The Contractor will perform sufficient testing as necessary for the Engineer to classify the materials removed from excavations or delivered to stockpiles.

3.03 PLACEMENT AND COMPACTING

G. Perform soil compaction by the method, specified and as required to provide the minimum percentage of density specified for each area in accordance with ASTM D1557.

H. Soil Types 1, 2, 6, 7 and 9 Random Rockfill (24 and 12 Inch), Select (24 and 12 Inch) and Mineralized Rockfill (1-Inch).

1. Place soil types 1, 2, 6 and 7 to the lines and grades shown on the Drawings.

2. Place material in horizontal lifts having a maximum loose thickness of 3 feet that will provide a well-graded distribution.

3. Compact each lift with a minimum of four complete passes of a 10-ton vibratory compactor or as approved by the Engineer. Additional passes of

the vibratory compactor may be necessary if in the opinion of the Engineer sufficient compaction was not obtained.

4. Conform to Section 02290 for Rock Buttress placement.

I. Soil Type 3, Processed Select (3 Inch)

1. Place soil type 3 to the lines and grades shown on the Drawings.

2. Place soil type 3 in horizontal lifts having a maximum thickness of 12 inches when loose that will provide a well-graded distribution except as noted in Section 02220.

3. Place material used as filter drain in such a way as to maintain the gradation and to eliminate contamination by foreign materials.

4. Compact soil type 3 to at least 90% of maximum dry density.

J. Soil Type 4, PROCESSED SELECT (1 INCH)

1. Place soil type 4 to the lines and grades shown on the Drawings.

2. Place soil type 4 in horizontal lifts having a maximum thickness of 12 inches when loose that will provide a well-graded distribution except as noted in Section 02220.

3. Place soil type 4 in such a way as to maintain the gradation and to eliminate contamination by foreign materials.

4. Compact soil type 4 to at least 90% of maximum dry density except as noted below.

5. Compact soil type 4 to at least 95% of maximum density when placed as pipe bedding, haunching, or pipe backfill.

6. No compaction is required for soil type 4 when placed in areas designated as liner cover.

K. Soil Type 5, Rock Drain and Riprap

1. Place soil type 5 to the lines and grades shown on the Drawings.

2. Place in a manner that will provide a well-graded distribution of rock.

3. No compaction control is required for the placement of rock riprap materials.

4. Place rock drain in horizontal lifts having a maximum loose thickness of 3 feet that will provide a well-graded distribution of rock.

5. No compaction control is required for placement of rock drain material.

L. Moisture Control

1. Where subgrade or a placed layer of soil material must be moisture-conditioned before compacting, uniformly apply water to surface of subgrade or soil material to prevent free water appearing on surface and ensure that moisture is uniformly distributed in the soil prior to compacting operations.

2. Remove and replace, or scarify and air dry, soil material that is too wet to permit compacting to the specified density.

3. Soil material that has been removed because it is too wet to permit compacting may be stockpiled or spread and allowed to dry. Assist drying by discing, harrowing, or pulverizing until moisture content is reduced to a satisfactory value as determined by moisture-density relation tests approved by the Engineer.

3.02 COMPACTION QUALITY CONTROL

A. The Contractor will perform field density testing as necessary for approval by the Engineer of the in-place fill and backfill material.

B. Based upon field density test results, the Contractor shall obtain approval from the Engineer for subgrades and fills prior to placement of subsequent layers of materials.

C. Densify or excavate and recompact all fill and backfill material which fails to meet the specified moisture and density requirements as necessary to meet the compaction criteria.

D. All work resulting from the removal of previously placed soil materials which did not conform to specifications shall be at the Dam Contractor's expense, except as specified in Paragraph 3.05.

3.03 MAINTENANCE DURING CONSTRUCTION

A. Protection of newly graded areas:

1. Protect newly graded areas from traffic and erosion, and keep free from trash, weeds, and ponding water; and

2. Repair and reestablish grades to the specified tolerances as necessary.

B. Where completed compacted areas are disturbed by subsequent construction operations or adverse weather, scarify the surface, reshape, and compact to the required density prior to further construction.

3.04 SCHEDULING

A. Perform construction during the summer and fall construction season or at the owner's option prior to spring thaw.

B. The Construction Manager, Contractor and CQA Field Manager will work together to develop a detailed construction plan showing the proposed construction schedule. A plan will be submitted by the CQA Field Manager to the Construction Manager 90 days prior to any construction operations.

END OF SECTION 02200

SECTION 02220
TRENCH EXCAVATION, BACKFILLING, AND COMPACTION

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Site Conditions:	Section 02020
B.	Clearing, Stripping, and Grubbing:	Section 02102
C.	Dewatering:	Section 02140
D.	Shoring:	Section 02151
E.	Earthwork:	Section 02200
F.	Excavating:	Section 02225
G.	Tailings Dam:	Section 02290
H.	Controlled Density Fill (CDF):	Section 03310
I.	Mortar and Grout:	Section 03600
L.	Quality Control:	Section 01450

1.02 DESCRIPTION

A. This section describes the trench excavation and backfilling for the curtain wall component of the wing wall to be constructed in Future Raises to Closure.

The curtain wall and cutoff wall will require excavation through gravel fill with some snow and ice, original ground surface soils and organics, and glacial till and weathered rock to below the groundwater table to depths indicated on the drawings.

1.03 EXCAVATION SAFETY

A. Conform to Section 02151.

PART 2 – PRODUCTS

2.01 MATERIALS

A. The CQA Field Manager will classify excavated materials as common or rock in accordance with the criteria presented in Section 02225.

Use native and imported materials for backfill as described in this section and as described and classified in Section 02200.

PART 3 – EXECUTION

3.01 SCHEDULE

A. Provide time to the CQA Field Manager and CQA Field Monitors to inspect and classify the materials as they are being excavated, and to complete geotechnical logging and take photos of the excavated and exposed materials.

Provide time and assistance to the CQA Field Manager in consultation with the Engineer to confirm the termination depths of the excavations.

3.02 EXCAVATIONS

A. Excavate all trenches to true and relatively smooth bottom grades in accordance with the lines and grades shown on the Drawings. Dimensions shown on the Drawings are based on borings and interpreted subsurface conditions between borings, and may vary if different subsurface conditions are encountered.

B. Provide assistance to the CQA Field Manager and CQA Field Monitors to inspect and classify the materials as they are being excavated, and complete geotechnical logging and take photos of the excavated and exposed materials.

C. Obtain approval of all subgrades from the CQA Field Manager prior to terminating the excavation. The CQA Field Manager in consultation with the Engineer will determine the final acceptable depth of excavation.

D. Do not place earth, geosynthetic or concrete materials in the trench until the finished excavated dimensions and surfaces are approved by the CQA Field Manager in consultation with the CQA Project Director and the Engineer.

E. Dispose excavated materials that are classified by the CQA Field Manager as being unsuitable for construction fill, in areas as designated by the Construction Manager.

F. Stockpile materials that are classified by the CQA Field Manager as being suitable as a soil type for construction fill, in separate locations according to soil type, at least 100 feet from the edge of the final excavated slopes.

3.03 CURTAIN WALL

A. Excavate the curtain wall trench to the top of the till as shown on the Drawings to create a work bench at least 45 feet wide with 1.5H:1V side slopes.

B. From the work bench, excavate the vertical curtain wall trench to the bottom of highly weathered shale, fully penetrating all visibly detectable ice, fractured rock and blast damaged rock capable of being excavated as shown on the Drawings.

C. Elevation of the work bench may be lowered below the top of the till to complete the trench excavation to the required depth with the equipment on site.

D. Provide the CQA Field Manager and CQA Field Monitors with excavated material on a bucket-by-bucket basis as it being excavated and so that they can inspect and classify the materials, and complete geotechnical logging and take photos of the excavated materials and the excavation bottom and side slopes.

E. Final depth of the curtain wall shall be based on observations of the CQA Field Manager as determined by CQA Field Manager in consultation with the CQA Project Director and the Engineer.

F. Insert the geomembrane curtain wall vertically in the center of the vertical trench.

G. Install a sandbag bulkhead for a 3 to 5-foot depth in the trench. Place sandbags forward of the leading edge of the curtain panel interlock.

H. Fill the curtain wall trench with CDF (Section 03310) to the top of the sandbags making sure that CDF stays contained forward of the sandbags.

I. Fill above the CDF with soil type 4 on the downstream side after the CDF has set for at least 24 hours or as directed by the CQA Field Manager.

3.04 TRENCH BACKFILLING

A. Place CDF or type 4 backfill material as shown on drawings to the dimensions indicated.

B. Type 4 material placed around the curtain panels will not be compacted until material reaches the trench top. At which point, material will be compacted.

C. Comply with material placement requirements as specified.

3.05 SCHEDULING

A. Perform trenching prior to spring thaw.

END OF SECTION 02220

SECTION 02225
EXCAVATION

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Site Conditions:	Section 02020
B.	Blasting:	Section 02050
C.	Clearing, Stripping, and Grubbing:	Section 02102
D.	Dewatering:	Section 02140
E.	Shoring:	Section 02151
F.	Earthwork:	Section 02200
G.	Tailings Dam:	Section 02290
H.	Quality Control:	Section 01450

1.02 DESCRIPTION

A. This section describes the excavation, removal, and disposal of overlying material to expose bedrock containing less than 10% ice by volume for cutoff trench and cutoff wall excavation.

1.03 EXCAVATION SAFETY

A. Conform to Section 02151.

1.04 CLASSIFICATION

A. The CQA Field Manager will classify all excavated materials according to material type, degree of weathering, and percent ice.

B. The CQA Field Manager will classify all excavated materials to be used in dam construction according to criteria presented in Section 02200.

C. Common excavation includes all materials not classified as rock.

D. Excavated materials conforming to the following conditions will be considered rock excavations:

1. Individual excavated pieces of natural material having a volume exceeding 3/4 cubic yard; and

2. In situ material that cannot be ripped by a bulldozer having size and power equivalent to a Caterpillar D-9 operating at 80% to 90% power using a single shank ripper.

E. The CQA Field Manager will collect samples for testing from the materials removed from excavations or delivered to stockpiles. The Contractor shall be responsible for submitting the samples to the Laboratory for testing.

1.05 SUBMITTALS

A. Submit to the Construction Manager and Engineer a detailed plan and schedule for excavation 90 days before beginning any construction operations.

PART 2 – PRODUCTS

2.01 EXCAVATED MATERIALS

A. Materials to be excavated include:

1. Surface Organics.

2. Ice and snow.

3. Tailings.

4. Alluvium and colluvium.

5. Highly weathered rock.

6. Moderately weathered bedrock, including shale, siltstone and sandstone with more than 5% ice by volume.

2.02 EQUIPMENT

A. Excavation equipment to be used is at the option of the Dam Contractor.

B. Perform blasting in accordance with Section 02050.

PART 3 – EXECUTION

3.01 SCHEDULE

A. Perform all work described in this section during the construction period.

B. Provide time and assistance to the CQA Field Manager and CQA Field Monitors to inspect materials as they are being excavated, conduct geotechnical logging and take photos of exposed slopes.

C. Provide time and assistance to the CQA Field Manager and Engineer to determine the termination depths of the excavations.

3.02 GENERAL

A. Excavate all material encountered within lines and grades shown on the Drawings and as described in 2.01. Dimensions shown on the Drawings are based on borings and interpreted subsurface conditions between borings, and may vary if different subsurface conditions are encountered.

B. Obtain approval of all subgrades from the CQA Field Manager prior to terminating the excavation. The CQA Field Manager in consultation with the Engineer will determine final acceptable depth of excavation.

C. Do not place earth, geosynthetic and concrete materials in the trench until the finished excavated dimensions and surfaces are approved by the CQA Field manager in consultation with the Engineer.

D. Dispose of unsuitable excavated materials in areas as designated by the Construction Manager.

E. Stockpile materials that are suitable as a soil type for construction fill at least 100 feet from the edge of the final excavated slopes.

END OF SECTION 02225

SECTION 02274
LINER BEDDING, LINER COVER, AND LINER ANCHORAGE

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Earthwork:	Section 02200
B.	Trench Excavation, Backfilling and Compaction:	Section 02220
C.	Geotextiles:	Section 02276
D.	Tailings Dam:	Section 02290
E.	Geomembrane:	Section 02770
F.	Quality Control:	Section 01450

1.02 DESCRIPTION

A. This section describes the liner bedding, liner cover, and liner anchorage material on the upstream face of the tailings dams.

PART 2 – PRODUCTS

2.01 MATERIALS

A. Use soil type 4, processed select (1 inch), for liner bedding, liner cover, and liner anchorage, as described in Section 02200.

PART 3 – EXECUTION

3.01 GENERAL

A. Place liner bedding, liner cover, and anchorage material to the lines and grades shown on the Drawings, or as directed by the CQA Field Manager.

3.02 PLACEMENT

A. Place, liner bedding, liner cover, and anchorage material after the installed geotextile and geomembrane is approved by the CQA Field Manager.

B. Place liner bedding, liner cover, and liner anchorage material in accordance with Section 02200.

C. Place materials over and around the riser pipe in a manner to avoid any displacement of the pipe in line or grade.

D. Compact material immediately adjacent to riser pipe with hand-operated equipment in accordance with Section 02220.

END OF SECTION 02274

SECTION 02276
GEOTEXTILE

PART 1 - GENERAL

1.01 RELATED WORK

A.	Quality Control	Section 01450
B.	Earthwork	Section 02200
C.	Geomembrane	Section 02770
D.	Liner Bedding, Cover and Anchorage	Section 02274

1.02 DESCRIPTION

A. This section summarizes the installation of geotextiles for liner protection, separation, filtration, and stabilization.

B. Owner will purchase all geotextile for the project in sufficient quantity and store it on the project site.

C. Prior to beginning work, the Contractor shall inspect and inventory the geotextile and thereafter, accept responsibility for storage and handling of material on site.

1.03 REFERENCES

A. The listed publications are part of the Specification to the extent referenced. The publications are referred to by basic designation only. The most recent version of the publication and test method shall be applicable in all cases.

B. American Society for Testing and Materials (ASTM) D1117-01: Standard Guide for Evaluating Nonwoven Fabrics.

C. ASTM D4355-02 Standard Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus).

D. ASTM D4439-04: Standard Terminology for Geosynthetics.

E. ASTM D4533-04: Standard Test Method for Trapezoid Tearing Strength of Geotextiles.

F. ASTM D4632-91(2003) Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.

G. ASTM D4751-04: Standard Test Method for Determining Apparent Opening Size of a Geotextile.

H. ASTM D4833-00e1: Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.

I. ASTM D4873-02: Standard Guide for Identification, Storage, and Handling of Geosynthetic Rolls and Samples.

J. ASTM D5261-92(2003): Standard Test Method for Measuring Mass per Unit Area of Geotextiles.

1.04 DEFINITIONS

A. Definitions are in accordance with ASTM D4439-04 unless otherwise indicated.

B. Geotextile: A permeable geosynthetic comprised of a stable network of fibers used as a separator or cushion.

C. Installer: The Contractor (or subcontractor to the Contractor) shall be the installer and be responsible for field handling, transporting, storing, deploying, seaming, temporary restraining (against wind), and installing the geotextiles. The installer may also be referred to as the geotextile subcontractor.

D. Manufacturer: The party, also referred to as the geotextile manufacturer or fabricator, responsible for the production of the geotextile.

E. Minimum Average Roll Value (MARV): Minimum of a series of average roll values representative of geotextile furnished.

F. Overlap: Distance measured perpendicular from overlying edge of one sheet to underlying edge of adjacent sheet.

1.05 STORAGE, AND HANDLING

A. Storage and handling of geotextile shall conform to ASTM D4873.

B. Handle products in a manner that maintains undamaged condition.

C. Immediately restore any damaged protective covering.

D. Do not store products directly on ground.

E. Protect geotextile from ultraviolet light exposure, precipitation, inundation, mud, dirt, dust, puncture, cutting, and other damaging or deleterious condition.

F. If stored outdoors, elevate and protect geotextile with a waterproof cover.

PART 2 - PRODUCTS

2.01 GENERAL

A. Geotextiles shall be nonwoven pervious sheets of polyester, polypropylene, polyethylene, or polyamide fibers oriented into a stable network so that the fibers retain their relative positions during handling, placement and long-term service.

B. Geotextiles shall be composed of continuous or staple fibers held together through needle-punching.

C. Geotextile products shall not be heat burnished.

D.

E. Geotextile edges shall be finished to prevent material from pulling from the fabric.

2.02 GEOTEXTILE – TYPE 1

A. Geotextile – Type 1 shall be used as shown on the Drawings and shall conform to the physical requirements in Table 1.

Table 1 - Physical Requirements for Geotextile Type 1

Physical Characteristic	Unit	Test Method	Requirement (MARV)
Mass (Weight) per Area	oz./sq. yd	ASTM D5261	10.0
Grab Tensile Strength	lb	ASTM D4632	230
Puncture Strength	lb	ASTM D4833	120
Trapezoid Tearing Strength	lb	ASTM D4533	95

2.03 GEOTEXTILE – TYPE 2

A. Geotextile–Type 2 shall be used as indicated on the Drawings and conform to the requirements of Table 2.

Table 2 - Physical Requirements for Geotextile Type 2

Physical Characteristic	Unit	Test Method	Requirement (MARV)
Mass (Weight) per Area	oz./sq. yd	ASTM D5261	16.0
Grab Tensile Strength	lb	ASTM D4632	370
Puncture Strength	lb	ASTM D4833	170
Trapezoid Tearing Strength	lb	ASTM D4533	145

2.04 LABELING

B. Contractor shall ensure that all geotextile has been marked and tagged with the following information:

1. Manufacturer’s name.
2. Product identification.
3. Lot number.
4. Roll number.
5. Roll dimensions.

PART 3 - EXECUTION

3.01 PREPARATION

A. Prepare areas in which geotextile is to be placed to lines and levels shown on the Drawings or as directed by the CQA Field Manager. The surface shall be prepared in accordance with Section 02200 – Earthwork.

B. Prior to installation of the geotextile, examine and ensure that the underlying surface is free of any sharp objects that may damage the geotextile and is in conformance with Contract Documents.

3.02 INSTALLATION

A. Geotextile shall be placed at the locations shown on the Drawings or as directed by the CQA Field Manager.

B. Geotextile shall be placed in accordance with the manufacturer's recommendations, standards, and guidelines and the requirements of this Specification.

C. During installation, geotextile shall be rejected if it has defects, ribs, holes, flaws, needles or needle fragments, deterioration or damage incurred by manufacturing, transportation, storage or placement. Visual review of the geotextile shall be performed once it has been placed and prior to placing overlying materials.

D. Geotextile shall be placed as directed by the CQA Field Manager, and shall be laid smooth and free of tension, stress, folds, wrinkles, or creases.

E. Adjacent strips shall be laid smooth and provide at least 12 inches of overlap for each joint. Overlap joints and seams shall be measured as a single layer.

F. Cut geotextile using approved cutter only. Take care to protect other in-place geosynthetic materials when cutting geotextile.

G. Securing pins or other methods that may damage the geotextile are not allowed. Sandbags or other methods approved by the CQA Field Manager shall be used to secure the geotextile during installation, and until replaced with cover material.

H. The geotextile shall be protected at all times from contamination by surface water runoff or any other means.

I. Do not entrap excessive dust, stones, or moisture in the geotextile that could damage or clog drains or filters or hamper subsequent seaming. Any geotextile so contaminated shall be removed and replaced with uncontaminated material at the Contractor's expense.

J. When placing soil or other materials over geotextile, Contractor shall ensure:

1. Geotextile is not damaged through puncture, tear, or other mechanism;
2. There is no slippage of the geotextile on underlying layers; and
3. No excessive tensile stresses are generated in the geotextile.

K. Do not operate machinery directly on the geotextile. If the geotextile is covered with less than a 2 feet of soil, no equipment with a ground pressure greater than 5 psi shall operate within 2 feet of the area underlain by the geotextile.

3.03 REPAIRS

A. Replace torn or damaged areas and holes by placing an overlay of the same geotextile having dimensions at least 12 inches greater than the tear or hole in all directions.

B. When the maximum dimension of a tear or puncture exceeds 10 percent of the width of the geotextile roll, the entire sheet shall be removed and replaced with a new sheet.

C. Remove any soil or other material which may have penetrated the torn geotextile and repair any damage to other materials or layers.

3.05 ACCEPTANCE

A. The Contractor shall retain all ownership and responsibility for the geotextile from the time of completion of the geotextile inventory.

B. The Construction Manager will accept the geotextile installation at the completion of:

1. Installation
2. Documentation
3. Verification of the adequacy of all seams, repairs, including associated testing
4. Construction Manager receipt of required written certification documents.

END OF SECTION 02276

SECTION 02277
GEOSYNTHETIC CLAY LINER (GCL)

PART 1 - GENERAL

1.01 RELATED WORK

A.	Earthwork:	Section 02200
B.	Geomembrane:	Section 02770
C.	Geotextile:	Section 02276
D.	Quality Control:	Section 01450

1.02 DESCRIPTION

A. Manufacturing, delivery, storage, and installing of Geosynthetic Clay Liner (GCL). The use of GCL is not anticipated for the construction of Future Raises to Closure. It has been considered in previous design discussions and drafts. In the event that it is reincorporated, the specification has been included.

1.03 REFERENCES

A. The publications listed herein form a part of this Specification to the extent referenced. The publications are referred to in the text by basic designation only. The most recent version of the publication and test method shall be applicable in all cases.

B. American Society for Testing and Materials (ASTM) D4439-04: Standard Terminology for Geosynthetics.

C. ASTM D4873-02: Standard Guide for Identification, Storage, and Handling of Geosynthetic Rolls and Samples.

D. ASTM D5887-04: Standard Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeameter.

E. ASTM D5890-02: Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners.

F. ASTM D5993-99(2004): Standard Test Method for Measuring Mass Per Unit of Geosynthetic Clay Liners

1.04 DEFINITIONS

A. Definitions shall be in accordance with ASTM D4439-04, unless otherwise indicated.

B. Geosynthetic Clay Liner (GCL): A composite of a uniform layer of low permeability granular sodium bentonite encapsulated between two geotextiles used as a hydraulic barrier. The composite is stabilized by needle punching through the top and bottom layers of geotextile.

C. Installer: The Contractor (or a Subcontractor to the Dam Contractor) shall act as the Installer, i.e., the party responsible for field handling, transporting, storing, deploying, seaming, temporary restraining (against wind), and installation of the GCL. The Installer may also be referred to as the GCL Subcontractor.

D. Manufacturer: The party also referred to as the GCL manufacturer, responsible for the production of the GCL in accordance with this Specification.

E. Minimum Average Roll Value (MARV): Minimum of a series of average roll values representative of geosynthetic material furnished.

F. Overlap: Distance measured perpendicular from overlapping edge of one sheet to underlying edge of adjacent sheet.

1.05 SUBMITTALS

A. At least 10 working days prior to shipping GCL to the site, the Dam Contractor shall notify the Construction Manager of the date of shipping.

B. Prior to shipping the GCL to the site, submit the following:
   1. Manufacturer's material specifications, product literature, and sample measuring not less than 12 inches square for the proposed GCL. Label the sample with brand name and furnish documentation of the roll number from which the sample was obtained.
   2. Manufacturer's written certification that:
      a. The proposed GCL has the material property values required by this Specification;
      b. The GCL manufacturer has continuously inspected the GCL for the presence of needles and found GCL to be needle-free;
      c. The loose bentonite for seaming consists of the same natural sodium bentonite as the GCL; and
      d. The bentonite will not shift during transportation or installation thereby causing thin spots in the body of the GCL.
   3. Manufacturer source quality control testing results.
   4. Written procedures for storing, handling, installing, repairing and seaming the GCL.
   5. A list of exceptions (if any) to the requirements specified in this section.

Exceptions and requests for substitutions shall be submitted to the Construction Manager.

C. Prior to placement of the geotextile, the Dam Contractor shall submit resume(s) of the individual(s) or subcontractor(s) who will perform the construction quality control (CQC) activities. The resume(s) shall demonstrate that the individual(s) or subcontractor(s) are suitably qualified to perform CQC activities.

1.06 DELIVERY, STORAGE, AND HANDLING

A. Delivery, storage, and handling of the GCL shall conform to ASTM D4873.

B. GCL shall be supplied in rolls wrapped individually in impermeable and relatively opaque protective covers.

C. Each roll shall be labeled with the lot number, roll number, and other information necessary to identify it for inventory and CQC and CQA testing.

D. Upon delivery at the site, the Dam Contractor and the Construction Manager shall inspect the surfaces of all rolls for defects and for damage. This inspection shall be conducted without unrolling rolls unless defects or damages are found or suspected. The Construction Manager will determine:

1. Rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws.
2. Rolls that are not properly labeled. No unlabelled rolls shall be used for any application. Unlabelled rolls shall be removed from the site and replaced at the Dam Contractor's expense.

E. Immediately repair any damaged protective covering. Preserve integrity and legibility of GCL roll labels.

F. Store and protect GCL from dirt, water, ultraviolet light exposure, vandalism, and other sources of damage.

PART 2 - PRODUCTS

2.01 GEOSYNTHETIC CLAY LINER

A. The GCL shall be Bentofix Thermal Lock NWL, or approved alternate.

B. The bentonite in the GCL shall be sodium montmorillonite clay with a minimum free swell value of 20.

C. The finished GCL shall have a minimum bentonite weight of 0.75 lbs/ft^2.

D. The finished GCL shall have a maximum hydraulic conductivity of 5×10^{-9} cm/sec under a gradient of 1.

E. The GCL shall be manufactured so that the bentonite is stabilized by needle punching through the top and bottom layers of geotextile to enhance the internal shear strength of the GCL and to maintain the integrity of the GCL under hydration.

F. The GCL shall be manufactured so that the bentonite is continuously contained throughout the GCL and so that no displacement of the bentonite occurs when the material is unrolled, moved, cut, torn, or punctured.

G. The encapsulating geotextiles used in the GCL shall be non-woven, needle-punched polypropylene with a minimum nominal weight of 6 oz/yd^2.

2.02 MANUFACTURER SOURCE QUALITY CONTROL

A. The GCL manufacturer shall perform source quality control testing to confirm the manufacturer's published material characteristics and demonstrate compliance with this Section. Testing shall be performed at a minimum frequency of once per lot or once every 100,000 square feet, whichever results in the greater number of tests.

B. The Contractor shall provide the GCL manufacturer's source quality certification for all rolls of GCL shipped to the site. Each quality control certificate shall include roll identification numbers and results of quality control tests. The quality control certificate shall be signed by a responsible party employed by the manufacturer, such as the production manager.

C. The GCL manufacturer shall examine entire GCL surface using a metal detector or other suitable method to verify that no needles or other sharp objects are present. The manufacturer shall certify in writing that the GCL surfaces are needle-free.

2.03 LABELING

A. Each GCL roll shall be marked or tagged with the following information:
   1. Manufacturer's name;
   2. Product identification;
   3. Lot number;
   4. Roll dimensions; and
   5. Roll weight.

B. Mark special handling requirements on rolls.

2.04 ACCESSORY BENTONITE

A. Bentonite for seaming shall be in powder or granular form, and shall be equivalent to the bentonite used in the manufacture of the GCL. The GCL manufacturer shall recommend the accessory bentonite.

PART 3 - EXECUTION

3.01 GENERAL

A. Install GCLs at the locations, lines, and grades shown on the Drawings. All GCLs shall be installed in accordance with the Specification.

B. Materials and Work which fail to meet the requirements of the Specification shall be removed and disposed of at the Dam Contractor's expense.

3.02 HANDLING AND PLACEMENT

A. The Dam Contractor shall handle and deploy all GCLs in such a manner as to ensure that they are not damaged.

3.03 SUBGRADE PREPARATION

A. Subgrades shall be a firm, unyielding surface.

B. The subgrade surface shall be free of debris, organic material, frozen soil, ice, rocks, and other deleterious material.

C. The subgrade surface shall be free of standing water or excessive moisture.

D. The subgrade surface shall be smooth and free of protrusions, ruts, and other irregularities greater than ½ inch high.

E. No GCL shall be deployed until the subgrade surface has been reviewed by the Construction Manager.

3.04 DEPLOYMENT

A. GCL shall be deployed so that seams run up and down (not across) the slope.

B. Adjacent panels of GCL shall be overlapped 6 to 12 inches in accordance with the manufacturer's recommendations.

C. Any wrinkles shall be removed by adjusting and smoothing the GCL after placement.

D. GCL shall not be deployed during precipitation or in the presence of moisture, ponded water, snow, or in other situations that could cause premature hydration of the bentonite. Any GCL that hydrates prematurely shall be removed and replaced at the Contractor's expense.

E. Deploy only that area of GCL which can be covered during the same day, unless otherwise approved by the Construction Manager.

F. Weight the GCL with sandbags or equivalent as necessary to prevent wind uplift.

G. Cut GCL with a geotextile cutter (hook blade), scissors, or other approved device. Protect adjacent materials from damage due to GCL cutting.

H. Do not entrap stones, debris, or other deleterious material within seams or beneath the GCL.

3.05 SEAMING

A. Contacting seam surfaces shall be clean and free of dirt or native soil with all edges pulled tight to maximize contact and to smooth out any wrinkles or creases.

B. Loose bentonite shall be applied at the minimum rate recommended by the GCL manufacturer. In no case shall the minimum rate be less than ¼ pound per liner foot along all seams or overlaps.

3.06 PROTECTION

A. Care shall be taken during placement of material above the GCL to prevent slippage of the GCL, the development of tensile stresses in the GCL, or any other damage. Any damage to the GCL shall be repaired at the Dam Contractor's sole expense.

B. Do not operate machinery directly on the GCL. If the GCL is covered with less than 2 feet of soil, no equipment with a ground pressure of greater than 5 pounds per square inch shall operate within 2 feet of the area underlain by the GCL.

3.07 REPAIRS

A. Remove damaged or hydrated areas of GCL using an approved cutter.

B. Place a patch of the same material extending at least 1 foot in all directions beyond the flaw or damaged area.

C. Seam around the perimeter of the patch per the requirements for initial seaming as described in this Section.

3.08 ACCEPTANCE

A. The Dam Contractor shall retain all ownership and responsibility for the GCL until acceptance by the Construction Manager.

B. The Construction Manager will accept the GCL when:
   1. The installation is complete;
   2. Documentation is complete;
   3. Verification of the adequacy of all seams and repairs is complete; and
   4. The required written certification documents have been received by the Construction Manager.

END OF SECTION 02277

SECTION 02770
GEOMEMBRANE

PART 1 - GENERAL

1.04 RELATED WORK

A.	Earthwork:	Section 02200
B.	Tailings Dam:	Section 02290
C.	Seepage Dam:	Section 02294
D.	Geotextile:	Section 02276
E.	Filter Drain, Liner Bedding, Liner Cover, and Liner Anchor:	Section 02274
F.	Geosynthetic Clay Liner:	Section 02277
G.	Quality Control:	Section 01450

1.05 DESCRIPTION

A. Manufacture, storage, delivery, and installation of high density polyethylene (HDPE) geomembrane materials for the liner and other miscellaneous applications.

1.06 REFERENCES

A. The publications listed below form a part of this Specification to the extent referenced. The publications are referred to in the text by basic designation only. The most recent version of the publication and test method shall be applicable in all cases.

B. American Society for Testing and Materials (ASTM) D1004-03: Standard Test Method for Initial Tear Resistance of Plastic Film and Sheeting.

C. ASTM D1238-04c: Standard Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer.

D. ASTM D1505-03: Standard Test Method for Density of Plastics by the Density-Gradient Technique.

E. ASTM D1603-01: Standard Test Method for Carbon Black In Olefin Plastics.

F. ASTM D3895-04: Standard Test Method for Oxidative-Induction Time of Polyolefins by Differential Scanning Calorimetry.

G. ASTM D4218-96(2001): Standard Test Method for Determination of Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique.

H. ASTM D4439-04: Standard Terminology for Geosynthetics.

I. ASTM D4833-00e1: Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.

J. ASTM D4873-02: Standard Guide for Identification, Storage, and Handling of Geosynthetic Rolls and Samples.

K. ASTM D5199-01: Standard Test Method for Measuring Nominal Thickness of Geosynthetics.

L. ASTM D5596-03: Standard Test Method for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics.

M. ASTM D5641-94(2001)e1: Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber.

N. ASTM D5885-04: Standard Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High-Pressure Differential Scanning Calorimetry.

O. ASTM D6392-99: Standard Test Method for Determining the Integrity of Nonreinforced Geomembrane Seams Produced Using Thermo-Fusion Methods.

P. Geosynthetic Research Institute (GRI):
   1. GM6: Pressurized Air Channel Test for Dual Seamed Geomembranes;
   2. GM11: Accelerated Weathering of Geomembranes using a Fluorescent UVA-Condensation Exposure Device;
   3. GM13: Test Properties, Testing Frequency and Recommended Warranty for High Density Polyethylene Smooth and Textured Geomembranes; and
   4. GM14: Selecting Variable Intervals for Taking Geomembrane Destructive Seam Samples Using the Method of Attributes.

1.07 DEFINITIONS

A. Definitions shall be in accordance with ASTM D4439-02, unless otherwise indicated.

B. Batch: refers to the quantity of resin, usually the capacity of one railcar, used in the manufacture of the HDPE geomembrane sheet. The finished sheet shall be identified by a roll number corresponding to the particular batch of resin used.

C. Bridging: When the geomembrane becomes suspended over the subgrade due to expansion or contraction of the material or poor installation.

D. Extrudate: The molten polymer that is produced from an extruder during extrusion seaming. The polymer is initially in the form of a ribbon rod, bead, or pellets.

E. Geomembrane: An essentially impermeable geosynthetic sheet used for the purpose of liquid, gas, and solids containment. HDPE geomembrane shall be manufactured and installed in accordance with the requirements of this Specification.

F. Geomembrane Subgrade: The surface upon which the geomembrane lies.

G. Installer: The Contractor (or a subcontractor to the Contractor) shall act as the installer, i.e. the party responsible for field handling, transporting, storing, deploying, seaming, temporary restraining (against wind), and installation of the geomembrane. The installer is also referred to as the geomembrane subcontractor.

H. Manufacturer: The party, also referred to as the geomembrane manufacturer or fabricator, is responsible for the production of the geomembrane rolls from resin and for the quality of the resin.

I. Master Welder: The individual to whom the installer delegates responsibility for oversight of geomembrane seaming operations. The Master Welder shall be qualified in accordance with the requirements of this Specification.

J. Overlap: Distance measured perpendicular from overlapping edge of one sheet to underlying edge of adjacent sheet.

K. Panel: The unit area of geomembrane that will be seamed in the field. A panel is identified as a roll or portion of a roll without any internal seams.

L. Panel Layout Drawings: Drawings submitted by the Installer indicating panel numbers, field seams, and details.

1.08 SUBMITTALS

A. At least 10 working days prior to shipping geomembrane to the site, the Contractor shall notify the Construction Manager of the date of shipping.

B. Prior to shipping the geomembrane to the site, the Contractor shall submit the following Product Data.

1. Resin Data:
    a. Statement of production date or dates;
    b. Certification stating that the resin meets the product requirements (see below);
    c. Certification stating that all resin is from the same manufacturer;
    d. Copy of quality control certificates issued by the manufacturer;
    e. Test reports from the manufacturer; and
    f. Statement that no reclaimed polymer is added to resin during manufacture of actual geomembrane to be used in this project.

2. Geomembrane Roll Data:
    a. Statement of production date or dates;
    b. Laboratory test results and certification stating that the geomembrane meets the product requirements;
    c. Certification stating that all geomembrane rolls are furnished by one supplier, and that all rolls are manufactured from one resin type obtained from one resin supplier;
    d. Copy of quality control certificates indicating compliance with requirements of this Specification;
    e. Test reports from the manufacturer;
    f. Statement certifying that no reclaimed polymer is added to the resin;
    g. Statement listing percentages/total of processing aids, antioxidants, and other additives other than carbon black added to or in the resin;
    h. Manufacturer's recommended geomembrane delivery, storage, and handling instructions;
    i. Manufacturer's recommended geomembrane installation instructions; and
    j. Sample warranties for review by the Construction Manager.

3. Extrudate Beads and/or Rod Data:
    a. Statement of production date or dates;
    b. Laboratory certification stating that the rod meets the product requirements;
    c. Certification stating that all rod is manufactured by one manufacturer and resin is supplied from one supplier;
    d. Copy of quality control certificates issued by the manufacturer;
    e. Test reports from the manufacturer; and
    f. Certification stating that the extrudate bead or rod resin is the same type, from the same manufacturer and compatible with the resin used to manufacture the geomembrane supplied for this project.

C. Prior to shipping the geomembrane to the site, the Manufacturer or Supplier shall submit three samples of the HDPE geomembrane to be supplied for the work. Each sample shall be one piece at least 12 inches wide and 48 inches long. Label each sample with brand name and furnish documentation of the lot and roll number from which the sample was obtained.

D. Prior to shipping the geomembrane to the site, Contractor shall submit a listing of exceptions (if any) to the requirements specified herein. Exceptions and requests for substitutions shall be submitted to the Construction Manager.

E. Prior to the placement of any geomembrane, the CQA Field Manager shall work with the Contractor and Construction Manager to submit the following Schedules and Drawings.
    1. An estimated installation schedule. Indicate all weather delays and shift changes built into schedule.

2. Installation layout drawings showing the panel layout indicating both fabricated (if applicable) and field seams, and details not conforming to the Contract Documents.
3. Layout drawings shall be drawn to scale and shall be suitable for use as construction drawings and shall include information such as dimensions, panel numbering, and installation details.

F. Prior to placement of any geomembrane, the Contractor shall submit qualifications for the following individuals or subcontractors that demonstrate they are suitably qualified to complete the designated work in accordance with this Specification.
1. Quality Control (QC) Officer.
2. The geomembrane installation subcontractor, and the supervisor who shall be responsible for the Work.
3. The master welder.
4. The installation seamers performing seaming operations.

G. Prior to the commencement of geomembrane installation on any given day, the CQA Field Manager shall:
1. Ensure that all quality control documentation has been completed.
2. Ensure that the subgrade has been cleared of any sharp material that may puncture the geomembrane.

H. The following submittals are required as a condition of acceptance of the project by the Construction Manager.
1. Certificate stating the geomembrane has been installed in accordance with the Contract Documents.
2. The Warranty obtained from the manufacturer.
3. The installation Warranty.
4. Record Drawings showing all changes from the approved installation drawings. The record drawings shall be reproducible and include the identification and dimensioned location of each seam, repair, cap strip, penetration, boot, and sample taken from the installed geomembrane.
5. Copies of all material and seam QC test results. Each test result shall be identified by sample date, test date, sample location, name of individual who performed the test, and the standard test method used.

1.09 DELIVERY, STORAGE, AND HANDLING

A. Delivery, storage and handling of the geomembrane shall conform to ASTM D4873-02 and the manufacturer’s recommendations.

B. Contractor shall be responsible for coordination and payment of shipping, delivery, unloading, storing, handling and installing the geomembrane.

C. Each roll shall be labeled with the lot number, roll number, and other information necessary to identify it for inventory and CQC and CQA testing.

D. Delivery:
   1. Deliver materials to the site only after the Construction Manager accepts required submittals;
   2. Damaged rolls, as determined by the Construction Manager, shall be removed from the site; and
   3. Deliver in rolls, do not fold.

E. Upon delivery at the site, the Contractor and the Construction Manager shall inspect the surfaces of all rolls for defects and for damage. This inspection shall be conducted without unrolling rolls unless defects or damages are found or suspected. The Construction Manager will determine:
   1. Rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws;
   2. Rolls or factory panels which include repairable flaws; and
   3. Rolls that are not properly labeled. No unlabelled rolls shall be used for any application. Unlabelled rolls shall be removed from the site and replaced at the Contractor's expense.

F. Immediately repair any damaged protective covering. Preserve integrity and legibility of geomembrane roll labels.

G. Storage on Site:
   1. Store geomembrane rolls to protect from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, vandalism, excessive heat or any other deleterious condition;
   2. Store geomembrane rolls on prepared surface;
   3. Do not stack geomembrane more than 3 rolls high; and
   4. Contractor is responsible for off-loading, storage, and transporting material from storage area to area of installation.

H. Handling on Site:
   1. Use appropriate handling equipment to load, move, and deploy geomembrane rolls. Appropriate handling equipment includes cloth chokers, spreader bars and roll bars;
   2. Dragging panels on ground surface shall not be permitted; and
   3. Do not fold geomembrane material. Folded geomembrane shall be rejected.

1.10 QUALIFICATIONS

A. Manufacturer Qualifications:
   1. Manufacturer shall be a commercial entity normally engaged in manufacture of geomembranes for waste containment applications;
   2. Manufacturer shall have at least 5 years continuous experience in the manufacturing of geomembrane rolls of the type specified;
   3. Manufacturer shall have experience totaling 10 million square feet of manufactured geomembrane, of the type specified, for at least 10 completed facilities; and

4. Manufacturer shall satisfy all appropriate trade certifications.

B. Installer Qualifications:
   1. Installer shall have at least 5 years continuous experience in the installation of the specified geomembrane, or meet the satisfaction of the Construction Manager; and
   2. Installer shall have experience totaling 5 million square feet of installed geomembrane, of the type specified, for at least 10 completed facilities, or meet the satisfaction of the Construction Manager.

C. Seamers shall have seamed a minimum of 500,000 square feet of the specified geomembrane or have successfully passed seaming tests to the satisfaction of the Construction Manager.

D. The Contractor and Installer shall agree to participate in and conform with all items and requirements of CQA and QC programs as outlined in this Specification.

1.11 WARRANTY

A. Provide a manufacturer's warranty for the geomembrane material in compliance with provisions of the General Terms and Conditions of the Contract. Provide a minimum 5-year warranty for the materials against deterioration.

B. Provide an installation warranty for the geomembrane material in compliance with the General Terms and Conditions of the Contract. Provide a minimum 2-year non-pro rata warranty for the installation against defects.

C. The warranties shall be provided to the Owner as purchaser and shall be signed by authorized representatives of the geomembrane manufacturer and Installer. The terms of the warranties shall, at a minimum, include, in addition to the requirements of this section, the applicable provisions of GRI GM13.

PART 2 - PRODUCTS

2.01 HDPE GEOMEMBRANE RESIN

A. Resin shall be new HDPE of first quality, compounded, and manufactured specifically for producing HDPE geomembrane.

B. The manufacturer will not mix resin types or use recycled materials or seconds in manufacturing.

C. Resin shall meet the requirements in Table 1.

Table 1
Geomembrane Resin Requirements

Physical Characteristic	Units	Test Method	Requirement
Density (min.)	g/cm^3	ASTM D1505	0.932

Table 1 Geomembrane Resin Requirements			
Physical Characteristic	**Units**	**Test Method**	**Requirement**
Melt Index	g/10 min	ASTM D1238 Condition 190/2.16	≤ 1.0

2.02 HIGH DENSITY POLYETHYLENE (HDPE) GEOMEMBRANE

A. Manufacturing:

1. Do not exceed a combined maximum total of 1 percent by weight of additives other than carbon black or pigment. Identify percentage of processing aids, antioxidants, and other additives other than carbon black.
2. Do not exceed 3.5 percent by weight of finished geomembrane for total combined processing aids, antioxidants, carbon and other additives. Do not exceed 3% carbon black by weight.
3. All additives for UV protection, thermal stability, color, or processing agents must not "bloom" to the surface over time or inhibit welding.
4. Use materials produced in North America or as accepted by the Construction Manager.
5. Provide finished product free from blemishes, holes, pin holes, bubbles, blisters, excessive gels, undispersed resins, and/or carbon black, contamination by foreign matter and nicks or cuts on edges.
6. Roll manufactured sheets or panels for shipment.
7. The HDPE geomembrane delivered to the site shall come from a maximum of two lots.

B. HDPE geomembrane shall conform to the physical requirements in Table 2.

Table 2 **Properties for HDPE Geomembrane**				
Physical Characteristic	**Qualifier**	**Units**	**Test Method**	**Requirement**
Sheet Thickness (nominal)	min. avg. value	mil	ASTM D5199	100
	lowest individual reading	-	-	90
Top and Bottom Surfaces	min. avg. value	mil	GRI GM13 and GRI GM17	Top and Bottom Surfaces
Specific Gravity (Sheet)	lowest individual reading	-	ASTM D1505	0.94
Tensile Properties (each direction)			ASTM D6693 (Type IV)	
Strength at Yield	min. avg. value	lb/in		210
Elongation at Yield	min. avg. value	%		12
Strength at Break	min. avg. value	lb/in		380
Elongation at Break	min. avg. value	%		700
Tear Resistance	min. avg. value	lb	ASTM D1004	70
Puncture Resistance	min. avg. value	lb	ASTM D4833	180

Table 2 Properties for HDPE Geomembrane				
Physical Characteristic	**Qualifier**	**Units**	**Test Method**	**Requirement**
Carbon Black Content	Range	%	ASTM D1603 or D4218	2-3
Carbon Black Dispersion	minimum 8 of 10	category	ASTM D5596	Category 1 or 2
Oxidation Induction Time	min. avg. value	minutes	ASTM D3895 or D5885	100
UV Resistance	min. avg. value	% retained after 1,600 hours	GM11 and ASTM D5885	50

2.03 EXTRUDATE ROD OR BEAD

A. Extrudate rod or bead shall:
   1. Meet the manufacturer's requirements;
   2. Be made from same resin as the geomembrane;
   3. Have thoroughly dispersed additives throughout rod or bead;
   4. Contain 2 to 3 percent carbon black; and
   5. Be free of contamination by moisture or any other foreign matter.

2.04 WELDING FIELD SEAMS

A. The only approved processes for welding seams are:
   1. Extrusion Welding; and
   2. Fusion Welding.

B. Proposed alternate processes shall be documented and submitted for approval by the Construction Manager.

C. The physical properties of the type of resin used in extrusion welding shall be the same as those of the resin used in the geomembrane.

D. Geomembrane seams shall conform to the requirements in Table 3.

Table 3 Minimum Geomembrane Seam Properties			
Physical Characteristic	**Unit**	**Test Method**	**Requirement**
Shear Strength (at yield point)	lb/in. width	ASTM D6392	200 and FTB[1]
Peel Adhesion	lb/in. width	ASTM D6392	130 and FTB[1]

Notes:
1. Film Tear Bond (FTB) is defined as failure of one of the sheets by tearing, instead of separating from the other sheet at the weld interface area (i.e., the sheet fails before the weld fails).

2.05 EQUIPMENT

A. The Contractor shall provide extrusion welding equipment that is equipped with gauges showing temperatures in the extruder apparatus and at the nozzle (temperature at the nozzle may be measured by external temperature gauge).

B. The Contractor shall provide fusion welding equipment that is an automated variable speed vehicular mounted apparatus equipped with devices adjusting and giving temperatures at wedge. The pressure shall be controlled by a spring, pneumatic control, or other system that allows for variation in sheet thickness. Rigid frame fixed position equipment is not acceptable.

C. Seam welding accessories shall meet the following requirements:
   1. The Contractor shall maintain sufficient operational seaming equipment to continue work without delay;
   2. Only apparatus that the Construction Manager has specifically approved by make and model shall be used;
   3. Equipment shall be powered by an electric generator(s) capable of providing constant voltage under the maximum anticipated combined load; and
   4. Electric generators shall not be placed on the geomembrane, unless a protective lining and splash pad large enough to catch spilled fuel is provided by the Contractor.

D. Equipment used for seaming shall not damage the geomembrane.

E. The Contractor shall provide a tensiometer for onsite shear and peel testing of geomembrane seams. The tensiometer shall be:
   1. Motor driven and have jaws capable of traveling at a measured rate of 2 inches per minute;
   2. Be in good working order, be built to ASTM specifications, and be accompanied by evidence of calibration within the last 12 months; and
   3. Be equipped with a gauge that measures the force exerted between the jaws to an accuracy of less than 1 pound and has a digital readout.

F. The Contractor shall provide a punch press for the onsite preparation of specimens for testing. The press shall be capable of cutting specimens in accordance with ASTM D6392.

G. The Contractor shall provide a vacuum box for onsite testing of geomembrane seams as per the following requirements. In addition to the vacuum box, the Contractor shall provide a soapy solution and applicator.
   1. The vacuum box shall have a transparent viewing window on top and a soft, closed-cell neoprene gasket attached to the bottom.
   2. The housing shall be rigid and equipped with a bleed valve and vacuum gauge.
   3. A separate vacuum source shall be connected to the vacuum box.
   4. The equipment shall be capable of inducing and holding a vacuum of 5 pounds per square inch - gauge pressure.

H. The Contractor shall provide the following equipment for pressure testing of geomembrane seams.

1. An air pump (manual or motor driven) equipped with a pressure gauge capable of generating and sustaining pressures over 30 pounds per square inch and mounted on a cushion to protect the geomembrane;
2. A sharp hollow needle or other approved pressure feed device;
3. A rubber hose with fittings and connections between the pump and needle or other approved pressure feed device; and
4. A gauge with an accuracy of 1 pound per square inch.

2.06 MANUFACTURER SOURCE QUALITY CONTROL

A. The manufacturer shall perform source quality control testing on the geomembrane at the manufacturing plant as indicated in Table 4.

Table 4
Manufacturing HDPE Geomembrane Quality Control Tests

Test	Test Method	Minimum Frequency
Sheet Thickness	ASTM D5199	5 per roll
Density	ASTM D1505	See Note 1
Tensile Strength at Yield	ASTM D6693	See Note 2
Elongation at Yield	ASTM D6693	See Note 2
Tear Resistance	ASTM D1004	See Note 2
Puncture Resistance	ASTM D4833	See Note 2
Carbon Black Content	ASTM D1603	See Note 2
Carbon Black Dispersion	ASTM D5596	See Note 2

Notes:
1. One per 100,000 square feet of sheet produced, or one per resin batch, whichever results in the greatest number of tests.
2. One test per resin batch on typical sheet and seam.

B. The objective of the manufacturer's source quality control testing shall be to confirm the manufacturer's published material characteristics and demonstrate the materials compliance with this Specification.

C. The manufacturer shall reject rolls for which quality control requirements are not met.

D. The manufacturer shall certify the quality of all rolls of geomembrane shipped to the site.

E. The Contractor shall provide the results of the manufacturer's source quality control tests to the Construction Manager for all rolls of geomembrane shipped to site.

2.07 LABELING

A. Mark or tag all geomembrane rolls with the following information:
1. Manufacturer's name;
2. Product identification;
3. Lot number;

4. Roll number; and
5. Roll dimensions.

B. Mark special handling requirements on rolls.

PART 3 - EXECUTION

3.01 GEOMEMBRANE SUBGRADE

A. Prepare areas in which geomembrane is to be placed to the lines and levels shown on the Drawings or as directed by the Construction Manager. The surface shall be prepared in accordance with Section 02200 – Earthwork and the following requirements.

B. The subgrade on which the geomembrane is to be installed shall be smooth and free of any gravel, rocks, sharp objects, grade stakes or hubs or any other protrusions or deleterious material that may damage the geomembrane.

C. The Contractor shall be solely responsible for protection of the geomembrane subgrade surface, including the intrusion of surface water beneath the geomembrane. Any damage to the geomembrane subgrade, including damage caused by geomembrane installation, shall be repaired at the Contractor's sole expense. Repair of damaged subgrade shall be completed prior to deployment of geomembrane in that area.

D. Edges of anchor trenches, other excavations and grade changes shall be rounded to a minimum 6-inch radius, chamfered to an angle of no less than 135°, or, with the approval of the Construction Manager, cushioned using a geotextile.

E. Geomembrane shall not be placed in an area, which has been softened as a result of precipitation.

3.02 EXAMINATION OF GEOMEMBRANE SUBGRADE

A. The Installer shall provide to the Contractor written acceptance of the geomembrane subgrade for each day of geomembrane installation.

B. The Contractor shall observe the geomembrane subgrade daily to evaluate desiccation cracking.

C. Once the geomembrane subgrade is accepted by the Installer, it shall be the Installer's responsibility to indicate to the Construction Manager any change in the condition of the subgrade that may require repair work.

3.03 PREPARATION

A. Geomembrane material shall only be installed, including deployment and welding, when the ambient air temperature is between 0°F and 100°F, unless otherwise approved in writing by the Construction Manager.

B. If the Installer wishes to use methods that may allow deployment and welding at ambient temperatures below 0°F or above 100°F the Installer shall demonstrate and certify that such methods produce welds that are entirely equivalent to those produced at ambient temperatures above 0°F and below 100°F and that the overall quality of the geomembrane is not adversely affected.

C. Geomembrane shall not be placed during precipitation, in presence of excessive moisture (e.g., fog, dew), in areas of ponded water, or during excessive winds.

D. The Installer shall perform and test trial welds on samples of geomembrane to verify the performance of welding equipment, methods, and conditions.

E. No welding equipment or welder shall be allowed to perform production welds until equipment and welders have successfully completed trial welds.

F. Trial welds shall be performed at the following minimum frequency:

1. At least 2 per day, with one prior to the start of work and one at mid shift;
2. As directed by the Construction Manager;
3. Every 2 hours when using a fusion welder for cross welds;
4. At least one per welder per shift; and
5. When the ambient temperature changes more than 20°F since the previous trial weld.

G. Trial welds shall be performed in the same surroundings and environmental conditions as the production welds (e.g., in contact with the geomembrane subgrade and similar ambient conditions).

H. All trial welds shall be tested as follows.

1. Samples at least 4 feet long and 2 feet wide with the weld centered lengthwise.
2. 8, 1-inch wide test strips shall be cut from the trial weld.
3. Each specimen shall be quantitatively tested in the field using a tensiometer, 3 for peel, and then 3 for shear in accordance with ASTM D6392.
4. The remaining 2 samples shall be archived by the Contractor, and shall be marked with the welder initials, date, time and machine identification.
5. A trial weld specimen shall pass when the results for both peel and shear tests exceed the required strength (psi) for 2 out of 3 specimens for both peel and shear tests.
6. For double-wedge fusion welding, both welds shall be individually tested and both welds shall be required to pass in peel.

7. If a trial weld fails, neither the welding apparatus nor welder shall be accepted and shall not be used for welding until deficiencies are corrected and two passing full trial weld samples and tests for both peal and shear are achieved consecutively.

3.04 INSTALLATION

A. Installation each day shall not begin until the Contractor has submitted to the CQA Field Manager the previous working day's QC documentation.

B. Deployment:

1. The Contractor shall give careful consideration to the timing and temperature during deployment. The Contractor shall focus on verifying that there is no:
   a. Bridging or excessive stress in the geomembrane; and
   b. Wrinkles in the geomembrane that will fold when covered with soil.
2. Panel Identification: Assign an identifying code number to each panel that is consistent with the Contractor panel layout drawing. The coding shall be coordinated with the CQA Field Manager.
3. Daily Panel Deployment: Deploy no more panels in one day than can be welded during that same day.
4. Panels shall be oriented perpendicular to the line of the slope crest (i.e., down and not across slope).
5. Do not damage geomembrane by handling, by equipment trafficking, leakage of hydrocarbons, or any other means.
6. Unroll geomembrane panels using methods that will not damage, stretch or crimp the geomembrane.
7. Use deployment methods that minimize wrinkles and differential wrinkles between adjacent panels.
8. Place sandbags, or some equivalent form of ballast approved by the CQA Field Manager, on the geomembrane to prevent uplift from wind.
9. Visually inspect the geomembrane for imperfections, and mark faulty or suspect areas for repair.
10. Sufficient geomembrane shall be installed to account for shrinkage and contraction while avoiding wrinkles, and shall be stress-free with no bridging before it is covered.
11. Before wrinkles fold over, attempt to push them out, and wrinkles that cannot be pushed out shall be cut out and the cuts repaired prior to burial.
12. Remove wrinkled or folded material.
13. Personnel walking on the geomembrane shall not engage in activities or wear types of shoes that could damage the geomembrane.
14. Smoking shall not be permitted while working on the geomembrane.
15. The geomembrane surface shall not be used as a work area, for preparing patches, storing tools and supplies, or other uses.
16. Protect the geomembrane in areas of heavy traffic by placing a protective cover that is compatible with and shall not damage the geomembrane.

17. Do not allow traffic directly on the geomembrane. If the geomembrane is covered with less than 2 feet of soil, no equipment with a ground pressure greater than 5 psi shall operate within 2 feet (vertical) of the area underlain by the geomembrane.

C. Weld Layout.

1. Welds shall be oriented perpendicular to the line of the slope crest (i.e., down and not across slope).
2. Minimize the number of field welds in corners, odd-shaped geometric locations and exterior corners.
3. Keep horizontal welds (welds running approximately parallel to slope contours) at least 6 feet away from the toe or crest of a slope.
4. Use a weld numbering system that is compatible with the panel numbering system. Coordinate the weld numbering system with the CQA Field Manager.
5. Panels shall be shingled on all slopes such that the upper panel is overlapped above the lower panel.

D. General Welding Procedures.

1. Do not commence welding with welding apparatus until a trial weld test sample, made by that apparatus, passes the trial weld test.
2. During all welding operations, at least one Master Welder shall be present and shall provide supervision over other welders.
3. Clean the geomembrane surface of grease, moisture, dust, dirt, debris, or other foreign material.
4. Overlap panels a minimum 6 inches for extrusion welding and fusion welding.
5. Solvents or adhesives shall not be used unless the product is approved, in writing, by the CQA Field Manager.
6. Place sandbags, or some equivalent form of ballast approved by the CQA Field Manager, on the geomembrane to prevent uplift from wind.
7. Welds shall be continuous to the outside edge of panels placed in anchor trenches.
8. If required, provide a firm substrate by using a flat board, a conveyor belt, or similar hard surface directly under the weld overlap to achieve firm support.
9. Electric generators shall be placed on a smooth base such that no damage occurs to the geomembrane.
10. The geomembrane shall be protected from damage in heavily trafficked areas.
11. Provide adequate illumination, if welding operations are carried out at night.
12. Personnel walking on the geomembrane shall not engage in activities or wear types of shoes that could damage the geomembrane.
13. Smoking shall not be permitted while working on the geomembrane.
14. Fishmouths or wrinkles at weld overlaps shall be cut along the ridge of the wrinkle to achieve a flat overlap. The cut fishmouths or wrinkles shall be

extrusion welded or patched where the overlap is more than 3 inches. When there is less than 3 inches overlap, an oval or round patch extending a minimum of 6 inches beyond the cut in each direction shall be used.

15. Log every two hours:
    a. Ambient temperature on the geomembrane surface being welded.
    b. Extrudate temperatures in the barrel and nozzle for an extrusion welder.
    c. Preheat temperature.

E. Extrusion Welding.

1. Extrusion welding shall be used only for repairs (e.g., patches, caps) and areas not accessible to fusion welding equipment.
2. Adjacent panels shall be tack bonded together using procedures that do not damage the geomembrane, allow the required tests to be performed, and are not detrimental to final welding. Tack-welded panels shall not be left overnight.
3. Welding apparatus shall be free of heat-degraded extrudate before welding. Purge extruder prior to beginning seam until all heat-degraded extrudate has been removed from barrel.
4. The edge of the top sheet of geomembrane shall be beveled to a minimum of 45 degrees and to the full thickness of geomembrane before extrusion welding.
5. The geomembrane surface shall be abraded a maximum of one quarter of an inch beyond the weld bead area, using a disc grinder, or equivalent, not more than 1 hour before welding.
6. The ends of all welds, which are more than 5 minutes old, shall be ground when restarting the weld.
7. Grinding depth shall not exceed 4 mils.
8. Grind across, not parallel to, welds.
9. Change grinding discs frequently. Do not use clogged discs.
10. Maintain one spare operable extrusion welding apparatus on-site at all times.

F. Fusion Welding

1. Fusion Welding shall be double wedge welding.
2. Fusion Welding shall be used for all welds except those that are not accessible with Fusion Welding Equipment.
3. Edges of cross welds shall be beveled (top and bottom) prior to welding.
4. If welding cross seams, conduct field test welds at least every 2 hours, otherwise, once prior to start of work and once at mid-day.
5. Protect against moisture build-up between panels.
6. Place smooth insulating plate or fabric beneath hot welding apparatus after usage.
7. Extrusion weld a repair patch over all weld intersections.
8. Maintain one spare operable welding apparatus on-site.

G. Inspection.

1. Examine all welds and non-weld areas of the geomembrane for defects, holes, blisters, undispersed raw materials, and any sign of contamination by foreign matter. The surface of the geomembrane shall be clean at the time of the examination.
2. Repair and non-destructively test each suspect location both in weld and non-weld areas. Do not cover geomembrane at locations that have been repaired until test results with passing values are available.

3.05 FIELD QUALITY CONTROL

A. General.

1. The Contractor and all subcontractors, including the Installer, shall participate in and conform with all terms and requirements of the QC program specified.

B. Non-Destructive Testing.

1. Non-destructive testing, as specified herein, shall be completed by the Contractor as part of their QC. The Contractor may undertake non-destructive testing in addition to that specified.
2. The Contractor shall pressure test all fusion welds. Fusion welds that, in the opinion of the CQA Field Manager, cannot be pressure tested shall be vacuum tested.
3. The Contractor shall vacuum test all extrusion welds and those fusion welds that, in the opinion of the CQA Field Manager, cannot be pressure tested.
4. Both pressure testing and vacuum testing shall be completed as specified over the full length of the weld.
5. All testing shall be performed as the welding progresses and not at the completion of all field welding. Complete all required repairs in accordance with this Specification.
6. Pressure testing shall be completed using the equipment specified in Paragraph 2.02 and according to the following procedure.
    a. Seal both ends of the weld to be tested.
    b. Insert needle or other approved pressure-feed device into the channel created by the double-wedge weld.
    c. Energize the air pump to a minimum pressure of 30 pounds per square inch.
    d. Close the valve, and sustain the pressure for at least 10 minutes.
    e. If pressure loss exceeds 3 pounds per square inch, or does not stabilize, locate faulty area and repair in accordance with this Specification. Repeat the pressure testing procedure either side of the repair.
    f. Puncture opposite end of the weld to release air. If the pressure was sustained as above and the pressure releases from the open end, the weld can be accepted.

g. If the pressure does not release, there is a blockage present. Locate the blockage, and repeat the testing procedure on both sides of blockage.

h. Remove needle or other approved pressure-feed device and seal penetration holes by extrusion welding.

7. Vacuum Testing shall be completed using the equipment specified in Paragraph 2.02 and according to the following procedure.

a. Apply soapy solution to the weld to be tested.

b. Place the box over the wetted weld area ensuring a leak-tight seal is created.

c. Energize the vacuum pump and reduce the vacuum box pressure to approximately 5 pounds per square inch - gauge pressure.

d. Examine the geomembrane through the viewing window for the presence of soap bubbles for not less than 15 seconds.

e. All areas where soap bubbles appear shall be marked and repaired in accordance with this Specification.

f. If no bubble(s) appear after 15 seconds, close vacuum valve and open bleed valve, move box over next adjoining area with minimum 3 inches overlap, and repeat the testing process.

C. Welds that the CQA Field Manager agrees can not be nondestructively tested, shall be treated by the Contractor as follows:

1. If the weld is accessible to testing equipment prior to final installation, non-destructively test the weld prior to final installation.

2. If the weld cannot be tested prior to final installation, a cap strip shall be provided over the weld. The welding and cap-stripping operations shall be observed by both the CQA Field Manager and the Contractor QC representative to ensure uniformity and for completeness.

D. Destructive Testing.

1. Destructive testing, as specified herein, shall be completed by the Construction Manager as part of CQA. The Contractor may undertake destructive testing in addition to that completed by the CQA Field Manager as part of its QC.

2. Samples for destructive testing shall be collected by the Contractor, at locations specified by the CQA Field Manager, at a frequency of at least one per 500 feet of seam length.

3. The CQA Field Manager shall specify test locations based on the minimum frequency or suspicion of excess crystallinity, contamination, offset welds, or suspected defect, and may increase the test frequency based on marginal results in accordance with GRI GM 14.

4. Sampling Procedures:

a. The Contractor shall cut samples at locations designated by the CQA Field Manager as welding progresses.

b. The sample shall be a minimum of 12 inches wide by 36 inches long with the weld centered lengthwise.
c. The CQA Field Manager shall provide a unique number for each sample, and the Contractor shall mark the sample number and location, including dimensions, on their panel layout drawing.

5. Contractor shall repair holes in the geomembrane resulting from destructive test sampling in accordance with specified repair procedures, and test the continuity of the repair in accordance with the non-destructive testing Specification.
6. The Contractor shall divide the sample into 3 parts as follows:
   a. A one-inch wide strip from each end of each part;
   b. One, 12 inch by 12 inch portion to be retained by the Contractor; and
   c. One, 12 inch by 12 inch portion for the Construction Manager.
7. The Contractor shall perform, at a minimum, the following field tests to determine the geomembrane weld compliance with the Specification:
   a. Weld strength: ASTM D6392;
   b. Peel Adhesion: ASTM D6392;
   c. At least 5 specimens shall be tested for each test method. Four of 5 specimens shall meet minimum requirements for both shear and peal tests;
   d. None of the peel specimens may peel 100 percent, or the entire sample shall be considered as failing;
   e. All tests shall exhibit a Film Tearing Bond type of separation in which the geomembrane material tears before the weld;
   f. If any test samples fail to achieve the minimum requirements of this Specification, follow the failed test procedures as per this Specification.
8. The Contractor shall allow two days from delivery of the destructive test sample to the CQA Field Manager before placing material over the geomembrane.
9. If the Contractor chooses to place material over the geomembrane before two days from delivery of the sample to the CQA Field Manager, and any tests fail to achieve the Specification, the Contractor shall, at its expense, remove the overlying material and follow the failed test procedures as per this Specification.

E. Failed Weld Procedures.

1. Contractor shall follow these procedures when there is a destructive test failure. The procedures shall apply to test failures determined by the Contractor, based on field tensiometer results.
2. Whenever a sample fails, the Contractor shall provide additional testing for welds that were welded by the same welder and welding apparatus or welded during the same time shift.
3. With respect to the weld from which the failed sample was taken, the Contractor shall follow one of the following 2 options.

4. Option 1:
    a. Reconstruct the weld between any 2 passing test locations. The weld flap shall not be extrusion welded;
    b. Reconstruction methods shall include cap stripping of weld, or replacing weld with a new 1 foot wide panel and welding in place.
5. Option 2:
    a. Trace the weld at least 10 feet minimum in both directions from the location of the failed test, or to the end of the weld;
    b. Obtain a small sample at both locations for an additional field test;
    c. If the field test samples pass, then reconstruct the weld or cap between the 2 test sample locations that bracket the failed test location;
    d. If any sample fails, then repeat the process to establish the zone in which the weld must be reconstructed.

F. The Construction Manager shall accept welds only as follows.
1. Welds that have passed non-destructive testing and that are bracketed by 2 locations from which samples have passed destructive tests.
2. For reconstructed welds exceeding 50 feet, a sample taken from within the reconstructed weld shall pass destructive testing.

3.06 REPAIR PROCEDURES

A. Contractor shall remove damaged geomembrane and replace with acceptable geomembrane materials if damage cannot be repaired to the satisfaction of the CQA Field Manager.

B. Repair, removal, and replacement shall be at the Contractor's sole expense if the damage results from Contractor, or any of the Contractor's subcontractor activities.

C. Contractor shall repair any portion of the geomembrane exhibiting a flaw, or failing a destructive or non-destructive test.

D. Agreement upon the appropriate repair method shall be reached between the CQA Field Manager and Contractor. Acceptable repair procedures include, but are not limited to:

1. Patching: Patching is used to repair large holes (over 3/8-inch diameter), tears, pinholes or other minor localized flaws.
2. Abrading and re-welding: Used to repair small sections of welds.
3. Spot welding or seaming: Used to repair small tears (less than 2 inches long), pin holes or other minor, localized flaws.
4. Capping: Used to repair large lengths of failed welds.
5. Removing the weld and replacing with a strip of new material.

E. Repairs shall satisfy the following requirements as indicated.

1. For repairs incorporating extrusion welding, abrade the geomembrane surfaces to be repaired no more than one hour prior to the repair.
2. For all repairs, clean and dry all surfaces at the time of repair.
3. All repair procedures, materials, and techniques shall be accepted in advance of the specific repair by the Construction Manager.
4. Repairs incorporating patches or caps shall extend at least 6 inches beyond the edge of the defect, and round all corners of material to be patched and the patches to a radius of at least 4 inches.
5. Unless otherwise instructed by the CQA Field Manager, cut the geomembrane below large caps to avoid water or gas collection between the sheets.

F. Verification of repair:

1. Contractor shall number and log each patch repair, and coordinate repair numbering with the CQA Field Manager.
2. Contractor shall non-destructively test each repair using methods specified in the Specification.
3. Contractor shall complete destructive tests at the discretion of the CQA Field Manager.
4. Repairs shall be reconstructed until non-destructive and destructive (where required) test results achieve the requirements of this Specification.

3.07 GEOMEMBRANE ACCEPTANCE

A. The Contractor shall retain all ownership and responsibility for the geomembrane from the time of completion of geomembrane inventory at the beginning for the project.

B. The Construction Manager shall accept geomembrane installation when:
1. All required documentation from the installer has been received and accepted;
2. The installation is complete;
3. Test reports verifying completion of all field welds and repairs, including associated non-destructive and destructive testing, have been received in accordance with this Specification; and
4. The Contractor has submitted to the Construction Manager, all written certification documents and drawings required by this Specification.

3.08 MATERIALS IN CONTACT WITH GEOMEMBRANE

A. Material placed above the geomembrane shall be in accordance with the requirements of Section 02200 – Earthwork.

B. Requirements of this article apply to placing soil directly on a geomembrane or on a geomembrane that is covered with a layer of geotextile.

C. Temperature: Do not place soil materials on the geomembrane at ambient temperatures below 0 degrees F or above 100 degrees F. The CQA Field Manager will carefully observe and ensure that the geomembrane does not get brittle, crack or is otherwise damaged, especially during temperature extremes.

D. Spreading Equipment: Equipment used for spreading soil shall be a light low ground pressure dozer with a ground pressure not exceeding 5 psi, or approved equal.

E. Do not allow any vehicular traffic directly on geomembrane. A minimum soil thickness of 1 foot shall be maintained between spreading equipment and the geomembrane. Rubber-tired hauling vehicles shall operate on a minimum soil thickness of 3 feet.

F. Spreading Operations: Spreading equipment shall not spin their tracks, make sharp turns, or make sharp, rapid starts or stops. Soil materials shall be pushed carefully from previously placed material and not dumped directly onto geosynthetics.

G. When placing soil or other cover materials above the geomembrane, Contractor shall ensure that:

1. The geomembrane is not damaged through puncture, tear, or other mechanism.
2. There is no slippage of the geomembrane on underlying layers.
3. There is no excessive tensile stress developed in the geomembrane.

H. The Contractor shall carefully install materials in contact with geomembrane surfaces to minimize the potential for future damage to the geomembrane. Loosely placed geotextile may be used as protection, if approved by the CQA Field Manager.

I. Clamps, clips, bolts, nuts, or other fasteners used to secure the geomembrane to each appurtenance shall have lifespan equal to or exceeding that of the geomembrane.

J. Pipes and Other Appurtenances:

1. Install geomembrane around appurtenances, such as pipes, protruding through geomembrane as shown in Drawings. Unless otherwise specified, install a geomembrane sleeve or apron around each appurtenance prior to installation of the surrounding geomembrane.
2. After material is placed and welded, complete the final field weld connection between appurtenance sleeve or apron and geomembrane. Maintain sufficient initial overlap of appurtenance sleeve so shifts in location of geomembrane can be accommodated.
3. Extreme care shall be taken while welding around appurtenances because neither non-destructive nor destructive seam testing may be feasible. Do not damage the geomembrane while making connections to appurtenances.

END OF SECTION 02770

SECTION 02771
CURTAIN WALL

PART 1 - GENERAL

1.01 RELATED WORK

A.	Quality Control	Section 01450
B.	Earthwork	Section 02200
C.	Geotextile	Section 02276
D.	Liner Bedding, Cover and Anchor	Section 02274
E.	Geomembrane	Section 02770

1.02 DESCRIPTION

A. This section summarizes the installation of curtain wall part of the wing wall.

B. Owner will purchase all the curtain wall required for the project in sufficient quantity and store it on the project site.

C. Prior to beginning work, the Contractor shall inspect and inventory the curtain wall and thereafter, accept responsibility for storage and handling of material on site.

1.03 REFERENCES

A. The publications listed below form a part of this Specification to the extent referenced. The publications are referred to in the text by basic designation only. The most recent version of the publication and test method shall be applicable in all cases.

B. American Society for Testing and Materials (ASTM) D1004-03: Standard Test Method for Initial Tear Resistance of Plastic Film and Sheeting.

C. ASTM D1238-04c: Standard Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer.

D. ASTM D1505-03: Standard Test Method for Density of Plastics by the Density-Gradient Technique.

E. ASTM D1603-01: Standard Test Method for Carbon Black In Olefin Plastics.

F. ASTM D3895-04: Standard Test Method for Oxidative-Induction Time of Polyolefins by Differential Scanning Calorimetry.

G. ASTM D4218-96(2001): Standard Test Method for Determination of Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique.

H. ASTM D4439-04: Standard Terminology for Geosynthetics.

I. ASTM D4833-00e1: Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.

J. ASTM D4873-02: Standard Guide for Identification, Storage, and Handling of Geosynthetic Rolls and Samples.

K. ASTM D5199-01: Standard Test Method for Measuring Nominal Thickness of Geosynthetics.

L. ASTM D5596-03: Standard Test Method for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics.

M. ASTM D5885-04: Standard Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High-Pressure Differential Scanning Calorimetry.

N. Geosynthetic Research Institute (GRI):
1. GM11: Accelerated Weathering of Geomembranes using a Fluorescent UVA-Condensation Exposure Device; and
2. GM13: Test Properties, Testing Frequency and Recommended Warranty for High Density Polyethylene Smooth and Textured Geomembranes;

1.04 DEFINITIONS

A. Definitions shall be in accordance with ASTM D4439-02, unless otherwise indicated.

B. Batch: refers to the quantity of resin, usually the capacity of one railcar, used in the manufacture of the HDPE geomembrane sheet used in the manufacture of curtain wall. The finished sheet shall be identified by a roll number corresponding to the particular batch of resin used.

C. Bridging: When the curtain wall becomes suspended over the subgrade due to expansion or contraction of the material or poor installation.

D. Curtain Wall: A series of connected, flexible, HDPE, vertical curtain wall panels used as a hydraulic barrier. The curtain wall is comprised of geomembrane panels interconnected at the edges with mechanical interlocks. The interlocks utilize a hydrophilic seal to complete the hydraulic barrier system.

E. Geomembrane: An essentially impermeable geosynthetic sheet used for the purpose of liquid, gas, and solids containment.

F. Installer: The Contractor (or subcontractor to the Contractor) shall be the installer and be responsible for field handling, transporting, storing, deploying, seaming, temporary restraining (against wind), and installation of the curtain wall. The installer is also referred to as the curtain wall subcontractor.

G. Manufacturer: The party, also referred to as the curtain wall manufacturer or fabricator, is responsible for the production of the curtain wall rolls from resin and for the quality of the resin.

H. Panel: Each curtain wall unit that will be installed in the field.

I. Panel Layout Drawings: Drawings submitted by the Installer indicating panel numbers, seams, and details.

1.05 SUBMITTALS

A. Prior to the placement of any curtain wall, the Construction Manager, Contractor and CQA Field Manager will develop Schedules and Drawings.

1. Installation schedule, including the hours worked per week, per day and per shift. Indicate potential weather delays and shift rotations built into schedule.
2. Installation layout drawings showing the panel layout indicating both fabricated (if applicable) and field seams, and details not conforming to the Contract Documents.
3. Layout drawings shall be drawn to scale and shall be suitable for use as construction drawings and shall include information such as dimensions, panel numbering, and installation details.

B. Prior to placement of any curtain wall, the Contractor shall submit qualifications for the following individuals or subcontractors that demonstrate they are suitably qualified to complete the designated work in accordance with this Specification.

1. Quality Control (QC) Officer.
2. Installation subcontractor and supervisor to be responsible for the Work.
3. Installation Welders performing interlock welding operations.

C. Prior to placement of any curtain wall, the Contractor shall submit an Installation Plan describing the proposed methods and equipment for curtain wall deployment, panel layout, welding, repair, and protection. The Plan shall also include quantity and type of all equipment to be used, but not limited to the welding method to be used.

D. The Contractor shall submit documentation stating that the Installer has contacted GSE Lining Technologies, Inc. to review the Installation Plan, including but not limited to, the installation method and QC procedures. A copy of the GSE Lining Technologies, Inc. review comments shall be included with the documentation.

E. The following submittals are required as a condition of acceptance of the project by the Construction Manager.

1. Certificate that curtain wall has been installed in accordance with the Contract Documents.
2. The installation Warranty.
3. Record Drawings in reproducible form showing all changes from the approved installation drawings, and identification and dimensioned location of each weld, repair, cap strip, penetration, boot, and sample taken from the installation.
4. Copies of all material and weld QC test results, with each test result identified by test date, test location, name of individual who performed the test, and the standard test method used.
5. Seam Test Summary Report showing the normal distribution of all test results.

1.06 STORAGE, AND HANDLING

A. Storage and handling of the curtain wall material shall conform to ASTM D4873-02 and the manufacturer's recommendations.

B. Storage on Site:

1. Store curtain wall rolls to protect from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, vandalism, excessive heat or other deleterious conditions.
2. Store curtain wall rolls on prepared surface.
3. Store hydrophilic seal material in their original boxes in a dry location.

C. Handling on Site:

1. Use appropriate handling equipment to load, move, and deploy curtain wall panels or rolls. Appropriate handling equipment includes cloth chokers, spreader bars and roll bars;
2. Dragging panels on ground surface shall not be permitted; and
3. Do not fold curtain wall panels. Folded curtain wall panels shall be rejected.

1.07 QUALIFICATIONS

A. Installer Qualifications:

1. Installer shall have at least 5 years continuous experience in the installation of the specified curtain wall, or meet the satisfaction of the CQA Field Manager.
2. In addition the installer shall also meet the installer qualifications in Section 02770, Paragraph 1.07.

B. The Contractor and Installer shall agree to participate in and conform with all items and requirements of QC programs as outlined in this Specification.

1.08 WARRANTY

A. Provide an installation warranty for the curtain wall material in compliance with the General Terms and Conditions of the Contract, and a minimum 2-year non-pro rata warranty for the installation against defects.

B. The warranties shall be provided to the Owner as purchaser and shall be signed by authorized representatives of the Installer. The terms of the warranties shall, at a minimum, include, in addition to the requirements of this section, the applicable provisions of GRI GM13.

PART 2 – PRODUCTS

2.01 FIELD WELDING

A. Field welds shall be performed per Section 02770 – Geomembrane.

2.02 EQUIPMENT

A. Curtain wall installation equipment shall be per the Installation Plan.

B. Field weld equipment shall be per Section 02770 – Geomembrane.

2.03 LABELING

A. Contractor shall ensure that all curtain wall has been marked and tagged with the following information:

1. Manufacturer's name
2. Product identification.
3. Lot number.
4. Roll number.
5. Roll dimensions.

B. Mark special handling requirements on rolls.

PART 3 - EXECUTION

3.01 SITE CONDITIONS

A. The work site shall be clear of all debris and obstacles that may interfere with the installation of the curtain wall system, including the subsurface

B. All overhead obstacles up to a height sufficient to clear installation equipment must be removed, deactivated or relocated so as not to delay the progress of the installation

C. All underground utilities and other obstacles shall be located and their positions clearly marked along the path of the installation. In addition, the Installer shall

obtain a set of detailed drawings showing the location, type, and depth of each obstruction.

D. If the location of utilities is unknown or unclear, the Contractor shall use a backhoe to excavate and explore along the path of installation.

E. When installing curtain wall panels in an open trench, the trench walls shall be smooth and free of any gravel, rocks, sharp objects, grade stakes or hubs or any other protrusions or deleterious material that may damage the curtain wall panels or inhibit installation.

F. Prepare areas in which curtain wall is to be placed to the lines and levels shown on the Drawings or as directed by the CQA Field Manager. The surface shall be prepared in accordance with Section 02200 – Earthwork and the following requirements.

G. The Contractor shall be responsible for protection of the curtain wall trench walls and subgrade surfaces, including the intrusion of surface water. Any damage to the curtain wall trench walls or subgrade, including damage caused by curtain wall installation, shall be repaired at the Contractor expense. Repair of damaged trench walls or subgrade shall be completed prior to deployment of curtain wall in that area.

H. Edges of the trench, other excavations and grade changes shall be rounded to a minimum 6-inch radius, chamfered to an angle of no less than 135°, or with the approval of the CQA Field Manager, cushioned using a geotextile.

I. Curtain wall shall not be placed in an area softened as a result of precipitation.

3.02 EXAMINATION OF WING WALL AND SUBGRADE

A. The Installer shall provide to the Contractor written acceptance of the curtain wall trench walls and subgrade for each day of curtain wall installation.

B. The Contractor shall observe the curtain wall trench walls and subgrade daily to evaluate desiccation cracking.

C. Once the curtain wall trench walls and subgrade is accepted by the Installer, it shall be the Installer's responsibility to indicate to the CQA Field Manager any change in the condition of the subgrade that may require repair work.

3.03 PREPARATION

A. Curtain wall panels shall only be installed, including deployment and seaming, when the ambient air temperature is between 0°F and 100°F, unless otherwise approved in writing by the Construction Manager.

B. If the Installer wishes to use methods that may allow deployment and seaming at ambient temperatures below 0°F or above 100°F the Installer shall demonstrate and certify that such methods produce seams that are equivalent to those produced at ambient temperatures above 0°F and below 100°F and that the overall quality of the curtain wall is not adversely affected.

3.04 INSTALLATION

A. The Contractor shall produce an Installation Plan describing the proposed methods and equipment for curtain wall deployment, panel layout, seaming, repair, QC and protection. The Installation Plan shall also include quantity and type of all equipment to be used. The Installation Plan shall be reviewed by GSE Lining Technology, Inc. and approved by the Construction Manager prior to the start of installation.

B. Installation each day shall not begin until the Contractor has submitted to the Construction Manager the previous working day's QC documentation.

C. Wherever possible, Contractor shall complete deployment, seaming and backfilling of the curtain wall at the same temperature or within as narrow a temperature range as practical. Contractor shall avoid completing these activities during extreme hot or cold conditions.

D. Deployment:

1. Contractor shall give careful consideration to the timing and temperature during deployment. The Contractor shall focus on verifying that:
   a. There is no bridging or excessive stress in the curtain wall; and
   b. There are no wrinkles in the curtain wall that will fold over when the trench is backfilled with soil material.
2. Panel Identification: Assign an identifying code number to each panel that is consistent with the Contractor panel layout drawing, and coordinate the coding with the CQA Field Manager.
3. Panels shall be oriented vertically in the trench.
4. Do not damage curtain wall by handling, by equipment trafficking, leakage of hydrocarbons, or any other means.
5. Unroll curtain wall panels using methods that will not damage, stretch or crimp the curtain wall.
6. Use deployment methods that minimize wrinkles and differential wrinkles between adjacent panels.
7. Protect the curtain wall panels to prevent uplift of panels from wind.
8. Visually inspect the curtain wall for imperfections, and mark faulty or suspect areas for repair.
9. Sufficient curtain wall shall be installed to account for shrinkage and contraction while avoiding wrinkles. Installed curtain wall shall be stress-free with no bridging before it is covered.
10. Remove wrinkled or folded material.

11. Smoking shall not be permitted while working on the curtain wall.

E. Interlock Seam Layout.

1. Use a seam numbering system that is compatible with the panel numbering system, and coordinate the numbering system with the CQA Field Manager.

F. Interlock Seaming Procedures.

1. During all interlock seaming operations, at least one Seamer shall be present and shall provide supervision over the work.
2. The hydrophilic seal shall be monitored during installation to assure that the rate of hydrophilic seal insertion is the same as the curtain wall.
3. Provide adequate illumination, if interlock seaming operations are carried out at night

G. General Welding Procedures.

1. Conform with Section 02770 – Geomembrane.

H. Extrusion Welding.

1. Conform with Section 02770 – Geomembrane.

I. Fusion Welding

1. Conform with Section 02770 – Geomembrane.

J. Inspection.

1. Examine all seams and non-seam areas of the curtain wall for defects, holes, blisters, undispersed raw materials, and sign of contamination by foreign matter, and ensure that the surface of the curtain wall is clean during the examination.
2. Repair and non-destructively test each suspect location both in seams and non-weld areas. Do not install curtain wall in the trench until inspection has shown the curtain wall and seams to be satisfactory.

3.05 FIELD QUALITY CONTROL

A. General.

1. Contractor and subcontractors, including the Installer, shall participate in and conform with all terms and requirements of the QC program specified herein.

B. Non-Destructive Testing.

1. Non-destructive testing, as specified herein, shall be completed by the Contractor as part of their QC. The Contractor may undertake non-destructive testing in addition to that specified.

a. Elevation of ground surface at the location of each curtain wall panel seam
b. Elevation of trench bottom at the location of each curtain wall panel seam

2. The CQA Field Manager will visually confirm that the panel is resting on the bottom of the trench.
3. Welded seams non-destructive testing shall be as specified per Section 02770 – Geomembrane.
4. All testing shall be performed as the seaming progresses and not at the completion of all the field seaming. Complete all required repairs in accordance with this Specification.

C. Destructive Testing.

1. Conform with Section 02770 – Geomembrane.

D. Failed Weld Procedures.

1. Conform with Section 02770 – Geomembrane.

E. Failed Interlock Procedures.

1. Failed interlock interconnection shall result in removal of the end panel and reinstallation of the panel until the seam passes testing.

F. The CQA Field Manager will accept welded seams only as follows.

1. Seams that have passed non-destructive testing and, if applicable, that are bracketed by 2 locations from which samples have passed destructive tests.
2. For reconstructed weld seams exceeding 50 feet, a sample taken from within the reconstructed weld shall pass destructive testing.

3.06 REPAIR PROCEDURES

A. Repair procedures shall be as specified per Section 02770 – Geomembrane.

3.07 CURTAIN WALL ACCEPTANCE

A. The Contractor shall retain all ownership and responsibility for the curtain wall from the time of completion curtain wall inventory at the beginning for the project.

B. The Construction Manager shall accept curtain wall installation when:

1. All required documentation from Installer has been received and accepted;
2. Installation is complete;
3. Test reports verifying completion of all field seams and repairs, including associated non-destructive and destructive testing, have been received in accordance with this Specification; and
4. Contractor has submitted to the Construction Manager, all written certification documents and drawings required by this Specification.

3.08 MATERIALS IN CONTACT WITH CURTAIN WALL

A. Material placed in contact with the curtain wall shall be in accordance with the requirements of Section 02200 – Earthwork and the Drawings.

B. Requirements of this article apply to placing soil directly in contact with the curtain wall or on a curtain wall that is covered with a layer of geotextile or GCL.

C. Temperature: Do not place soil materials in contact with the curtain wall at ambient temperatures below 0 degrees F or above 100 degrees F.

D. Spreading Equipment: Equipment used for spreading soil shall be a light low ground pressure dozer with a ground pressure not exceeding 5 pounds per square inch, or approved equal.

E. When placing soil or other cover materials in contact with or above the curtain wall, the Contractor shall ensure that:

1. Curtain wall is not damaged through puncture, tear, or any other mechanism;
2. There is no slippage of the curtain wall; and
3. There is no excessive tensile stress developed in the curtain wall.

F. Contractor shall carefully install materials in contact with curtain wall surfaces such that the potential for future damage to the curtain wall is minimized. Loosely placed geotextile may be used as protection if approved by the CQA Field Manager.

G. Clamps, clips, bolts, nuts, or other fasteners used to secure the curtain wall to each appurtenance shall have lifespan equal to or exceeding that of the curtain wall.

END OF SECTION 02771

SECTION 03310

CONTROLLED DENSITY FILL (CDF)

PART 1 GENERAL

1.01 RELATED SECTIONS:

A.	Quality Control	Section 01450
B.	Earthwork	Section 02200
C.	Trench Excavation and Backfill	Section 02220
D.	Curtain Wall	Section 02771
E.	Geomembrane	Section 02770
F.	Dewatering	Section 02140

1.02 DESCRIPTION

A. This section specifies the manufacture and placement of Controlled Density Fill (CDF) which may be used for embedment of the GSE CurtainWall, if the GSE CurtainWall is used for extension of the seepage cutoff along the wing wall alignment during future raises to closure of the tailings main dam.

B. The purpose of the CDF is to secure the placement of the GSE CurtainWall to the bottom of the narrow trench and provide a seal between the GSE CurtainWall and competent bedrock for seepage control.

1.03 QUALITY CONTROL

A. Referenced Standards: This Section incorporates by reference the latest revision of the following documents. These are part of this Section as specified and modified. In case of conflict between the requirements of this Section and that of the listed documents, the requirements of this Section shall prevail.

1.	ASTM C150	Specification for Portland Cement
2.	ASTM C260	Specification for Air-Entraining Admixtures for Concrete
3.	ACI 229R	Controlled Low Strength Materials.

1.04 SUBMITTALS

A. Provide information on the following CDF components to the Construction Manager:

1. Cement,

2. Fine aggregate gradation, and

3. Admixtures.

PART 2 PRODUCTS

1.05 CDF

A. CDF shall be composed of Portland cement, aggregate, admixtures, and water.

1. Minimum 7-day compressive strength of the CDF shall be 50 psi (ASTM C39).

2. Minimum 28-day compressive strength of the CDF shall be 100 psi (ASTM C39).

3. Permeability of 10^{-6} cm/sec (ASTM D5084).

4. Water content shall be sufficient to produce the fluid workable mix but not exceed that required to maintain the fine aggregate in suspension and prevent segregation.

B. Mix proportioning shall be based on the mix design below:

1.	Cement: Portland (Type I or III) or API Class G):	100 lb
2.	Fine Aggregate (Sand):	2850 lb
3.	Water (Clean):	480 lb
4.	Air Entrainment:	5 %
5.	Air Entrainer Admixture – MBAE 90 (5 fl oz/cwt):	5 fl oz
6.	Water Reducer Admixture – Polyheed (8 fl oz/cwt):	8 fl oz
7.	Superplasticizer– Glenium 1000 (5 fl oz/cwt):	5 fl oz
8.	Freeze Point Depressant– Pozzutec 20 (80 fl oz/cwt):	80 fl oz

C. Fine aggregate grading shall be as tabulated below, or as approved by the CQA Field Manager in consultation with the Engineer following the Contractor submittal to be based on available materials and quarry processing capabilities:

Sieve Size Square Opening	Percent Passing By Weight
No. 4	95 - 100
No. 8	80 - 90
No. 16	55 - 77
No. 30	30 - 60
No. 50	12 - 30
No. 100	0 – 10

PART 3 EXECUTION

1.06 MIXING

A. The CDF shall be thoroughly mixed using the following batch plant provided by the Owner at the site:

1. Haganator Model HSE-10300A Mobile Concrete Batching Plant that includes:

a) 10-cubic yard two-compartment decumulative aggregate weigh batcher

b) 24-inch deep troughing batch transfer belt conveyor

c) 10 cubic yard/95 cubic foot cement weigh batcher

d) 300 barrel (4 cu.ft/bbl., 59 tons) in-truss design cement silo.

e) Single 7-inch multi-flight cement feeder (21 cfm capacity)

f) 5 hp air compressor (19.8 cfm) and 80 gallon air receiver

g) 2-inch Badger electronic water meter (20 to 160 gpm)

h) Batch controls

B. Start each batch of mixing only when approved by the CQA Field Manager.

C. Complete all mixing in the presence of a CQA Field Monitor.

D. Continue mixing until the cement and water are thoroughly dispersed throughout the material.

E. Place CDF within one hour after mixing.

1.07 INSTALLATION

A. Dewater trench and clean trench of water, loose soil and rock, ice and snow and other debris before placing CDF.

B. Start placement of CDF only when approved by the CQA Field Manager.

C. Place CDF concurrently on both sides of the GSE CurtainWall while the GSE CurtainWall is held vertically to the bottom of the excavated trench centerline

D. Minimum CDF temperature at time of placement is -7°C (20°F)

E. Keep excavated trench relatively dry and free of snow.

F. Comply with ACI 304R for measuring, mixing, transporting and placing CDF.

1.08 RESPONSIBILITY

A. Contractor shall take full responsibility for ensuring that the integrity of the completed CDF and GSE CurtainWall conforms to the Drawings and Specifications.

END OF SECTION 03310

SECTION 03600
GROUT

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A. Quality Control: Section 01450

B. Geomembrane: Section 02770

1.02 DESCRIPTION

A. This section specifies the manufacture and placement of grout for the embedment of the geomembrane in the cutoff wall below the cutoff trench along the left and right abutments, and the connection to the existing cutoff walls.

B. The purpose of the grout is to secure the placement of the geomembrane to the bottom of the narrow trench and provide a seal between the geomembrane and competent rock for seepage control.

1.03 SUBMITTALS

A. Provide following grout components information to the CQA Field Manager and Construction Manager:

1. Cement.
2. Fine aggregate (sand) gradation.
3. Admixtures.

PART 2 – PRODUCTS

2.01 GROUT

A. Grout shall be composed of:

1. Portland Cement: ASTM C150, Type I or III or ACI Class G.

2. Fine aggregate: ASTM C33 or as approved by CQA Field Manager.

3. Air-Entraining Admixture: ASTM C 260.

4. Water-Reducing Admixture: ASTM C 494, Type A/E/F/D.

B. Mix Proportioning:

1. Cement, water, fine aggregate and admixtures are proportioned by weight.

2. Adjust Grout mix proportions in the field, based on setting performance of grout placed in a trial slot just prior to grouting of cutoff wall.

C. Fine aggregate grading shall be as tabulated below, or as approved by the CQA Field Manager in consultation with the Engineer following the Contractor submittal to be based on available materials and quarry processing capabilities:

Sieve Size	Percent Passing By Weight
No. 4	95 - 100
No. 8	80 - 90
No. 16	55 - 77
No. 30	30 - 60
No. 50	12 - 30
No. 100	0 – 10

D. Batch Proportions:

Material	Weight (lb/cu. yd.)
1. Cement (Type I or III or API Class G):	500
2. Water:	301
3. Admixtures:	37
4. Coarse Aggregate:	1358
5. Fine Aggregate:	1623
6. Air:	5%
Total	3824

Batch weights based on saturated surface dry (SSD) aggregate
Pozzutec20 contains approximately 40% free water
Add Pozzutec20 on site, place grout within approximately 30 minutes (i.e. prior to loss of slump)

E. Admixtures:

Admixture	fl. oz./cwt
Air Entrainment, MB AE90:	2
Water Reducer, Polyheed:	8
Superplasticizer, Glenium1000:	5
Freeze Point Depressant, Pozzutec20:	80

Batch micro-air, Pozzolith 322N and Rheobuild 1000 to obtain 1 to 2-inch slump
Adjust Reobuild 100 dosage rate to obtain slump, do not increase batch water
Dosage rates for micro-air and Rheobuild 1000 to be adjusted in field.
Quantities shown are maximum values.
Micro-air dosage rate to based on air content after addition of Pozzutec20

PART 3 – EXECUTION

3.01 PRODUCT HANDLING AND STORAGE

A. The shelf life recommended by the Manufacturer shall not be exceeded by any components used in the grout.

3.02 MIXING

A. The grout shall be thoroughly mixed using the Owner's existing batch plant at the site described below, or other approved type mixer.

1. Haganator Model HSE-10300A Mobile Concrete Batching Plant or equivalent model that includes:

a) 10-cubic yard two-compartment decumulative aggregate weigh batcher

b) 24-inch deep troughing batch transfer belt conveyor

c) 10 cubic yard/95 cubic foot cement weigh batcher

d) 300 barrel (4 cu.ft/bbl., 59 tons) in-truss design cement silo.

e) Single 7-inch multi-flight cement feeder (21 cfm capacity)

f) 5 hp air compressor (19.8 cfm) and 80 gallon air receiver

g) 2-inch Badger electronic water meter (20 to 160 gpm)

h) Batch controls

B. Start each batch of mixing only when approved by the CQA Field Manager.

C. Complete all mixing in the presence of a CQA Field Monitor.

D. Continue mixing until the cement and water are thoroughly dispersed throughout the material.

E. Place grout within one hour after mixing.

3.03 INSTALLATION

A. Dewater trench and clear trench of water, loose soil and rock, ice and snow and other debris before placing grout.

B. Start placement of grout only when approved by the CQA Field Manager.

C. Place grout concurrently on both sides of geomembrane while geomembrane is held vertically to the bottom of the excavated trench centerline.

D. Minimum placement width of fresh grout is 6 inches.

E. Minimum rock temperature at time of placement is -8ºc (17ºF).

F. Minimum grout temperature at time of placement is -7ºC (20ºF).

G. Keep excavated free of visible ice and snow.

H. Comply with ACI 304R for measuring, mixing, transporting and placing grout.

I. Do not add water or any other materials to the grout during placement.

3.04 RESPONSIBILITY

A. Contractor shall take full responsibility for ensuring that the integrity of the completed curtain wall conforms to the Drawings and Specifications.

END OF SECTION 03600

SECTION 13340
INSTRUMENTATION

PART 1 – GENERAL

1.01 RELATED WORK SPECIFIED ELSEWHERE

A.	Site Conditions:	Section 02020
B.	Clearing, Stripping, and Grubbing:	Section 02102
C.	Earthwork:	Section 02200
D.	Trench Excavation, Backfilling, and Compaction:	Section 02220
E.	Grout:	Section 03600
F.	Quality Control:	Section 01450

1.02 QUALITY CONTROL AND ASSURANCE

A. Provide for the equipment manufacturer's representative to be on site during installation of all instrumentation for verification that equipment is installed and operating in accordance with manufacturer's standards.

1.03 SUBMITTALS

A. Submit to the Construction Manager and Engineer data on all, items that are necessary to provide a complete and functioning system. Include at a minimum the following information:

1. Manufacturer's name;

2. Model numbers;

3. Wiring diagrams;

4. Wiring and connectors;

5. Performance data on all equipment;

6. Operation and maintenance; and

7. Detail of installations and installation schedule.

1.04 SCOPE OF WORK

A. Furnish materials and equipment and perform all necessary work to complete instrumentation installations.

B. Furnish and install all instrumentation to the lines and elevations shown on the Drawings or as directed by the Engineer.

C. No additional compensation above the prices bid in the schedule will be made for construction delays caused by the installation of instruments in the dam.

D. Place all units of instrumentation in operation at the earliest period during construction, as approved by the Engineer.

E. Provide the following instrument systems:

1. Vibrating wire piezometers - for pore water pressure monitoring;

2. Thermistors - for ground temperature monitoring, and

3. Vibrating wire readout box – for downloading piezometer and thermistor data.

F. Maintain the existing instrumentation in operational condition during construction of the dam as directed in Section 3.0.

PART 2 – PRODUCTS

2.01 GENERAL

A. Obtain all instruments of one type and related operating equipment from Geokon. Obtain the supplier's certification that each instrument is correctly operating and submit certification for acceptance by the Engineer.

2.02 VIBRATING WIRE PIEZOMETERS

A. Provide vibrating wire piezometers capable with the following minimum specifications:

Table 12-2 Vibrating Wire Piezometer Specifications	
Standard Ranges	350, 700 kPa; 1, 2 and 3 Mpa
Over Range	2 × rated pressure
Resolution	0.025% F.S. (minimum)
Accuracy	±0.1% F.S.
Linearity[1]	<0.5% F.S. (±0.1% F.S. optional)
Temperature Range[2]	−20°C to +80°C

B. An acceptable vibrating wire piezometer should be approved by the Contract Manager and Engineer.

C. Provide water- and weather-tight, terminal box for the housing of vibrating wire leads at each piezometers installation location. Provide terminal box acceptable by the Engineer.

D. Provide readout boxes capable of data readout from the installed vibrating wire piezometers. Provide portable readout boxes acceptable by the Engineer.

Instrumentation Readout Box Specifications	
Excitation Range	400 Hz to 6000 Hz, 5 Volt Square Wave
Resolution	0.1 µs
Timebase Accuracy	±50 ppm
Temperature Range	−10°C to +50°C

E. Backfill material shall include the following:

1. Provide cement grout; and

2. Provide 50/50 % bentonite/cement grout slurry mixture.

F. Where materials or equipment required to complete the piezometer installation are not specifically described or covered by these Specifications, provide high-quality commercial grades of materials as directed or approved by the Engineer.

2.03 THERMISTOR STRINGS

A. Use Teflon-insulated, Teflon-jacketed, 25 pair/24 AWG solid conductor cable configured for application in the telephone industry. Thermistor strings will have adhere to the following minimum specifications:

Thermistor String Minimum Specifications	
Standard Range	-50°C to 150°C
Resolution	0.1 °C
Accuracy	+/- 0.5°C
Temperature Range	−20°C to +80°C

B. Provide an acceptable thermistor acceptable by the Engineer.

C. Provide one steel terminal pipe and panel for housing instrumentation leads from thermistors.

D. Provide two terminal panels for thermistors terminated in terminal boxes. Terminal panels shall be located within and compatible with piezometer terminal boxes indicated in subsection 2.02.

E. Provide connectors at the readout end of the cables. Connectors shall be identified by engraved identification numbers and embossed adhesive plastic tape.

F. Provide two compatible portable switch boxes to allow thermistors to be read in sequence.

G. Provide portable resistance measuring devices.

H. Backfill material shall include the following:

1. Provide cement grout; and

2. Provide 50/50 % bentonite/cement grout slurry mixture.

PART 3 – EXECUTION

3.01 INSTALLATION

A. Install all instrumentation to the lines and elevations shown on the Drawings or as directed by the Engineer.

B. Do not bury instrumentation until it is tested and approved by the Engineer. The Engineer will make functional checks and will perform calibration tests on the instruments and if the instruments prove to be unsatisfactory, they shall be replaced in kind by the Contractor at his expense. A calibrated reading should be obtained per the manufactures recommendations prior to instrumentation placement.

C. Protect all equipment until the conclusion of this Contract. Provide markers and barricades as necessary. Replace in kind any equipment damaged or rendered inoperable as a result of the Contractor's operations, at the expense of the Contractor.

D. Do not penetrate, puncture, or otherwise damage the liner material with the instruments, wiring, piping, or other related equipment.

E. Complete installation of piezometers at the direction of the Construction Manager.

3.02 VIBRATING WIRE PIEZOMETER AND THERMISTOR STRING INSTALLATION

A. Hole Drilling

1. Drill minimum 4-inch-diameter holes for vibrating wire piezometer and thermistor string installation. Drill Hole P-05-002 and P-05-003 includes thermistors at 10 feet interval through the depth of the piezometer. Locate and install vibrating wire piezometers/thermistors to the elevations shown on the Drawings or as directed by the Engineer.

2. Use only drilling fluid suitable for drilling in permafrost as approved by the Engineer. The use of drilling mud, mud emulsions, or any other material that might affect the permeability or integrity of the foundation and embankment material, or piezometer tips will not be permitted.

3. Clean each hole of cuttings and other debris. Perform the cleaning so the foundation and embankment materials surrounding the hole are not unnecessarily disturbed.

4. Log each hole during the drilling operations. Remove temporary casing during installation of the piezometer and during backfilling operations. All temporary casing shall remain the property of the Contractor.

B. Instrument Installation, Vibrating Wire Piezometer and Thermistor Strings

1. Install the vibrating wire piezometers and thermistor strings at the elevations shown on the Drawings.

2. Clean all holes drilled for the purpose of piezometer installation as approved by the Engineer.

3. Pull temporary casing, if any, no more than 10 feet ahead of piezometer placement. Lower vibrating wire piezometer and/or thermistors until the piezometer tip is at, the elevation shown on the Drawings. Grout the vibrating wire piezometer and thermistors in place at the desire tip elevation.

4. Test piezometer and thermistors before installation. If the instrumentation is functioning properly, the installation may continue; if not, improperly functioning instrumentation shall be replaced at Contractor's expense.

5. When the installation continues, backfill as necessary, with cement grout.

6. After the vibrating wire piezometer and thermistor strings have been placed at the required elevation, backfill with grout or 50/50% bentonite/cement grout mixture to the top of the drill hole. Obtain one installation reading for each vibrating wire piezometer and thermistor string once the installation and placement of instrumentation has finished. Be sure to allow time for the instrument to equilibrate in accordance with the manufacturer’s (Geokon) instructions.

7. A 1/2-inch carrier tube may be used to place the piezometer and thermistor leads during installation.

8. For piezometers and thermistors connected to instrumentation terminal boxes, thread piezometer wires through to terminal box locations. Connect

wiring to terminal panels and label wires, and terminal connections. Test vibrating wire piezometers and thermistors.

9. Once all installation has finished, obtain the initial reading for both the piezometer and thermistor strings.

3.03 ACCEPTANCE AND TESTING

A. At the time of delivery from the instrument manufacturer, submit all instruments to the Engineer for inspection and acceptance. Provide a clean, lighted, temperature-controlled, dry area for storage of instrumentation materials.

B. Submit to the Construction Manager and Engineer, Contractor's and manufacturer's test results from tests performed during installation. All instruments that fail required tests will be rejected by the Engineer and a new replacement instrument shall be provided at the Contractor's expense. Test replacements before use in installation.

C. The Contractor shall be solely responsible for supplying, on schedule, materials and equipment for instrument installation. No additional payment will be made for construction delay due to testing or to the Contractor's failure to provide instrument installation materials or replacement materials for rejected assemblies.

3.04 WARRANTY AND GUARANTEE

A. Obtain from the manufacturer/installer a written guarantee for a time period of at least 5 years during which the instrumentation materials and workmanship specifically provided or performed under this project shall be free from any significant defects. Such written guarantee shall provide for the total and complete repair or replacement of the defect or defective materials upon written notification and demonstration by the Owner of the specific non-conformance of the material or installation with the project Specifications. Such defects or nonconformance shall be repaired or replaced at no cost to the Owner within 30 days of notification, as approved by the Owner or Construction Manager.

END OF SECTION 13340

Appendix G

PRELIMINARY
CONSTRUCTION QUALITY
ASSURANCE PLAN
FUTURE RAISES TO CLOSURE
RED DOG TAILINGS DAM
RED DOG MINE, ALASKA

For

TECK COMINCO ALASKA, INC.
URS JOB NO.: 33755331
November 26, 2007

PRELIMINARY
CONSTRUCTION QUALITY ASSURANCE PLAN

FUTURE RAISES TO CLOSURE
RED DOG TAILINGS DAM
RED DOG MINE, ALASKA

Prepared by

URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101-1616

For

Teck Cominco Alaska Inc.
3105 Lakeshore Drive, Building A, Suite 101
Anchorage, Alaska 99517



November 26, 2007

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1
1.1 DEFINITION: QUALITY CONTROL VS. QUALITY ASSURANCE 2
1.2 PLANNED CONSTRUCTION ACTIVITIES 2
1.3 REFERENCE DOCUMENTS 6

2.0 PARTIES TO THE WORK 7

3.0 CQA PERSONNEL AND RESPONSIBILITIES 9
3.1 CQA PROJECT DIRECTOR 10
3.2 CQA FIELD MANAGER 10
3.3 CQA FIELD MONITORS 12

4.0 MEETINGS AND SITE VISITS 13
4.1 PRE-CONSTRUCTION MEETING 13
4.2 PROGRESS MEETINGS AND REPORTS 14
4.3 SITE VISITS 14

5.0 EARTHWORK CQA 15
5.1 PROJECT PLANNING & SCHEDULE DEVELOPMENT 15
5.2 SOURCE QUALITY CONTROL 15
5.3 EXCAVATION 16
5.4 SOILS PURCHASE, DELIVERY, STORAGE AND HANDLING 18
5.5 PLACEMENT AND COMPACTION 18
5.6 SOIL TESTING 19

6.0 GEOMEMBRANE CQA 21
6.1 MANUFACTURE 21
6.2 SOURCE QUALITY CONTROL 23
6.3 PURCHASE, DELIVERY, STORAGE AND HANDLING 24
6.4 INSTALLATION 27

7.0 GEOTEXTILE CQA 30
7.1 MANUFACTURE 30
7.2 PURCHASE, DELIVERY, STORAGE AND HANDLING 31
7.3 GEOTEXTILE INSTALLATION 33

8.0 GEOSYNTHETIC CLAY LINER (GCL) CQA 34
8.1 MANUFACTURE 34
8.2 PURCHASE, DELIVERY, STORAGE AND HANDLING 36
8.3 GCL INSTALLATION 37

9.0 CURTAIN WALL CQA 37
9.1 MANUFACTURE 37
9.2 SOURCE QUALITY CONTROL 37
9.3 PURCHASE, DELIVERY, STORAGE AND HANDLING 37

9.4 GSE CURTAINWALL PANEL INSTALLATION 37

10.0 GROUT CQA 37
10.1 GROUT MIX 37
10.2 GROUT INSTALLATION 37
10.3 GROUT TESTING 37

11.0 CONTROLLED DENSITY FILL (CDF) CQA 37
11.1 CDF MIX 37
11.2 CDF INSTALLATION 37
11.3 CDF TESTING 37

12.0 INSTRUMENTATION CQA 37
12.1 MANUFACTURE 37
12.2 SOURCE QUALITY CONTROL 37
12.3 INSTRUMENTATION 37
12.4 EQUIPMENT INSTALLATION 37

1.0 INTRODUCTION

This Construction Quality Assurance (CQA) Plan outlines the CQA procedures to be applied to the construction of the earthwork, geosynthetic, concrete and instrumentation components of the Future Raises to Closure of the Red Dog Mine tailings main dam. The Future Raises to Closure will be conducted in three stages for the purpose of its conceptual design. These three stages are:

- From Stage VII-B (El. 960) to Stage VIII (El. 970);
- From Stage VIII (El. 970) to Stage IX (El.980); and
- From Stage IX (El.980) to Stage X at Closure (El.986).

Before the start of future raises to closure, the dam was completed through Stage VII-B to Elevation 960 feet (El. 960). The Future Raises to Closure include extending the cutoff wall in the left abutment, raising the main embankment and wing wall system, and inserting additional instrumentation as deemed appropriate.

This CQA Plan is to be used in conjunction with the following documents:

- URS Corporation, 2006, "Geotechnical Investigation Report, Red Dog Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska Inc., July 28, 2006.

- URS Corporation, "Seepage Analysis Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska (TCAK), February 26, 2007.

- URS Corporation, "Stability Analysis Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska (TCAK), April 13, 2007.

- URS Corporation, "Dam History Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska (TCAK), September 14, 2007.

- URS Corporation, "Conceptual Design Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska (TCAK), November 26, 2007.

- URS Corporation, "Construction Drawings for Red Dog Mine Tailings Dam, Future Raises to Closure", for Teck Cominco Alaska Inc., Sheets TDC-1 through TDC-24, November 26, 2007.

- URS Corporation, "Preliminary Technical Specifications for Future Raises to Closure, Red Dog Mine Tailings Dam", for Teck Cominco Alaska, Inc., November 26, 2007.

- URS Corporation, "Design Modification Nos. 1 through 31" to Teck Cominco Alaska Inc., various dates between June 20, 2005 and August 10, 2007 and future design modifications.

Any conflict between the requirements of the CQA Plan and the other project documents shall be reported to the Engineer for clarification or adjudication as required. Significant care must be applied to monitoring and documenting the activities described in the CQA Plan.

1.1 DEFINITION: QUALITY CONTROL VS. QUALITY ASSURANCE

There is often confusion between the meanings of quality control (QC) and quality assurance (QA). This CQA Plan refers to the provision of QA, in this case specifically referring to the CQA of the materials comprising the dam raise. In the context of this CQA Plan, QC and QA are defined as follows.

Quality Control (QC) refers to actions taken by the Contractor (including those parties charged with the manufacture, fabrication, delivery, and installation of the geosynthetic, concrete and instrumentation materials, and the placement and compaction of the soil materials), that provide a means to determine and sometimes quantify the characteristics of the product. The results of a QC program are compared to the Specifications or other contractual or regulatory requirements. During each aspect of the handling of these materials, QC is provided by the Manufacturer, fabricator, or installer of geosynthetic, concrete and instrumentation materials, or the supplier and contractor for the earthworks, to ensure that the materials and workmanship conform to the Drawings and Specifications.

Quality Assurance (QA) is a planned and systematic pattern of means and actions intended to provide confidence that materials and procedures conform to the Drawings, Specifications, and any applicable regulatory requirements. QA is provided by an independent party on behalf of the Owner. Although QA is as important during all phases of handling the soil, geosynthetic, concrete and instrumentation materials, CQA refers specifically to those actions taken in relation to the installation of the geosynthetic, concrete and instrumentation materials, and the placement and compaction of the soil materials. CQA is the most critical component of an overall QA program, because field conditions are the most variable and most difficult to control, and therefore the design is only fully finalized once the construction is completed.

1.2 PLANNED CONSTRUCTION ACTIVITIES

The planned construction activities for the Future Raises to Closure include the following elements:

- Site Preparation (by Owner and Contractor with CQA Personnel support);
- Cutoff System (material purchase by Owner, installation by Contractor, material inspection by CQA Personnel),
- Wing Wall System (material purchase by Owner, installation by Contractor, material inspection by CQA Personnel),
- Liner System (material purchase by Owner, installation by Contractor, material inspection by CQA Personnel),
- Embankment (material sources provided by owner, construction by Contractor, material inspection by CQA Personnel),
- Instrumentation (by Owner).

The Future Raises to Closure construction of these elements includes the following activities:

- Earthwork material supply, placement and compaction for the embankment and liner, cutoff and

wing wall systems. There are seven gradations of soil and rock fill: Soil Types 1, 2, 3, 4, 7, 8 and 9.

- Geosynthetic materials manufacture, fabrication and installation for the liner, cutoff and wing wall systems. There are four types of geosynthetics: geomembrane liner, geomembrane curtain, geotextile and geosynthetic clay liner (GCL).
- Concrete materials manufacture and installation for the cutoff and wing wall systems. There are two types of concrete materials: grout and controlled density fill (CDF).
- Instrumentation installation of piezometers and thermistors.

The Contractor is responsible for maintaining the integrity of the existing embankment, liner and cutoff systems and instrumentation while exposing them for the new construction.

1.2.1 Site Preparation

Site preparation will be completed by the Owner and the Contractor with the support of CQA personnel. Currently, it is anticipated that select components of the mill infrastructure will need to be relocated. Future raises may require construction of additional coffer dams in order to extend the existing cutoff wall toward the left abutment and to extend the cutoff wall to the south/southeast. The Owner will ensure that this takes place prior to the construction of future raises when it is appropriate.

1.2.2 Embankment

The future raises to closure embankment will be constructed using rock fill materials that will be produced at the mine and placed by conventional earthwork delivery, spreading and compaction procedures. These rock fill materials are designated as soil types as follows:

- Soil Type 1 – Random Rockfill (24-inch)
- Soil Type 2 – Select Fill (24-inch)
- Soil Type 3 – Processed Select (3-inch)
- Soil Type 4 – Processed Select (1-inch)
- Soil Type 7 – Select (12-inch)
- Soil Type 8 – Kivalina Shale (Select fill material)
- Soil Type 9 – Random Mine Waste Fill

Some soil types have different uses as shown in Sections 1.2.3, 1.2.4 and 1.2.5. Soil Types 5 and 6 were developed for Stages I to VI, but are not required in Future Raises to Closure, and are not included in this CQA Plan. Soil Type 9 was developed for backfill upstream of the seepage cutoff wall in the right abutment.

1.2.3 Liner System

The future raises to closure liner system will be an extension of the existing liner system. The existing liner system is anchored at El. 960 along the dam crest and along the wing wall alignment to its southeast extent. The future raises to closure liner system will be constructed from bottom to top as follows, with the purpose of each soil type shown in parentheses:

- Soil Type 1 (Random Rockfill);
- Soil Type 7 (Transition Rockfill),
- 10 oz/yd^2 Geotextile (Type 1 Geotextile),
- Soil Type 3 (Filter Drain),
- 16 oz/yd^2 Geotextile (Type 2 Geotextile),
- Soil Type 4 (Liner Bedding),
- 100 mil High Density Polyethylene (HDPE) Geomembrane,
- Soil Type 4 (Liner Cover), and
- Soil Type 1 (Protective Buttress).

The bottom of the new geomembrane will be welded to the top of the geomembrane in the existing liner system, and to the top of the geomembrane part of the cutoff system outside the existing liner system.

1.2.4 Cutoff System

The future raises to closure seepage cutoff system consists of a cutoff wall that will extend from the existing cutoff wall at the left abutment. The existing cutoff trench and cutoff wall terminates at the west extent of the existing dam. The cutoff system will be extended in the following sequence:

- Excavation to reveal the existing cutoff wall terminus;
- Excavation of a trench to bedrock,
- Installation of 100 mil HDPE Geomembrane Liner with vertical support system,
- Grout to predetermined depth,
- Backfill with Soil Type 4 (Liner Cover),
- Backfill with Soil Type 1 (Random Rockfill)

A critical part of the cutoff system that requires CQA scrutiny, judgement and decision is the depth of excavation for the cutoff 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 this bedrock to reduce seepage under the dam.

1.2.5 Wing Wall System

The wing wall system consists of below ground and above ground components, the curtain wall and wing wall respectively. The wing wall system is an extension of the main tailings dam where the dam embankment axis changes from a west-east alignment to a northwest-southeast alignment. The curtain wall is a vertical seepage cutoff system constructed of 100 mil HDPE liner. Construction of the curtain wall is described in Section 1.2.5.1; the wing wall embankment will be constructed in the following sequence:

- Soil Type 1 (Random Rockfill)
- Soil Type 7 (Transition Rockfill)
- 10 oz/yd^2 Geotextile (Type 1 Geotextile)
- 16 oz/yd^2 Geotextile (Type 2 Geotextile)
- Soil Type 4 (Liner Bedding)
- 100 mil High Density Polyethylene (HDPE) Geomembrane
- Soil Type 4 (Liner Cover)
- Soil Type 1 (Protective Buttress).

1.2.5.1 Curtain Wall

The Stage VII-B curtain wall is part of a wing wall system that extends below ground along with wing wall alignment. The curtain wall is a continuation of the seepage cutoff wall where the dam changes from a west-east alignment to a northwest-southeast alignment. The east end of the cutoff wall at the right abutment was tied to the curtain wall by a field-fabricated, large-radius geomembrane bend.

During the Stage VII-B construction, the curtain wall was installed along the wing wall alignment from Sta. 101+00 to 108+12. The curtain wall was constructed using GSE CurtainWall panels; however, this material may be changed during future raises to closure.

If GSE CurtainWall panels are used for future raises to closure, the curtain wall will be constructed in the following sequence:

- Site preparation including snow clearing and work bench excavation;
- Trench excavation to bedrock,
- Curtain panel deployment and attachment to framing,
- Vertical suspension of curtain panel(s) in the trench,
- CDF pour and set,
- Soil Type 4 backfill (Liner Cover),
- Soil Type 7 backfill (Downstream Fill), and
- Soil Type 9 backfill (Upstream Fill).

A critical part of the curtain 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.

1.2.6 Instrumentation

After the Contractor completes the construction of the future raises to closure, the Owner will install piezometers and thermistors as agreed to monitor the dam's performance.

1.3 REFERENCE DOCUMENTS

- URS Corporation "Preliminary Conceptual Design Report, Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska." for Teck Cominco Alaska, November 26, 2007.

- URS Corporation, "Construction Drawings for Red Dog Mine Tailings Dam, Future Raises to Closure, TDC-1 through TDC-24", for Teck Cominco Alaska Inc., November 26, 2007.

- URS Corporation, "Preliminary Technical Specifications for Future Raises to Closure, Red Dog Mine Tailings Dam", for Teck Cominco Alaska, Inc., November 26, 2007.

- URS Corporation "Seepage Analysis Report: Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska", for Teck Cominco Alaska, February 26, 2007.

- URS Corporation "Stability Analysis Report: Tailings Main Dam, Future Raises to Closure, Red Dog Mine, Alaska." for Teck Cominco Alaska, April 13, 2007.

- URS Corporation, "Construction As-Built Record Drawings, Red Dog Mine Tailings Main Dam, Stage VII-B Raise, Sheets T7B-1 through T7B-17", for Teck Cominco Alaska Inc., October 26, 2007.

- URS Corporation, "Design Modification Nos. 1 through 31", to Teck Cominco Alaska Inc., various dates between June 20, 2005 and August 10, 2007 and future design modifications.

- URS Corporation, "Geotechnical Investigation Report for Red Dog Tailings Main Dam Future Raises to Closure" for Teck Cominco Alaska, Inc., July 28, 2006.

- Alaska Department of Natural Resources (ADNR), "Guidelines for Cooperation with the Alaska Dam Safety Program", prepared by Dam Safety and Construction Unit, Water Resources Section, Division of Mining, Land and Water, ADNR, June 30, 2005.

2.0 PARTIES TO THE WORK

The successful completion of the construction of each element of the future raises to closure will depend on the interaction and cooperation of many parties. The following parties are represented in the project:

- ***Owner***: The Owner is Teck Cominco Alaska, Inc. (TCAK) of Anchorage. The Owner will complete site preparations before construction; purchase the geosynthetic, concrete and instrumentation materials; select the Contractor; provide earthwork, geosynthetic and concrete materials; provide a geosynthetic material storage area, concrete batch plant, concrete and soil testing laboratory for CQA Personnel's use; and install instrumentation.
- ***Engineer:*** The Engineer is URS Corporation (URS) of Seattle. The Engineer designed the raise and is also referred to as the Geotechnical Engineer. The Engineer is responsible for reviewing Contractor submittals pertaining to the design, implementing the CQA program, confirming the cutoff wall excavation depths, completing the construction report, and certifying that construction is in compliance with Plans and Specifications.
- ***Construction Manager:*** The Construction Manager (CM) is TCAK. The CM is responsible for coordinating and monitoring Contractor activities, reviewing Contractor submittals, identifying those that require review by the Engineer and CQA Field Manager, processing the submittals and providing responses to the Contractor, coordinating the CQA field monitoring, and directing site preparation and instrumentation activities.
- ***Contractor:*** The Contractor will be selected by the Owner. The Contractor is responsible for constructing the earthwork, liner system, cutoff system and curtain wall, installing geosynthetic and concrete materials, supplying a portable tensiometer for geosynthetics testing, and repairing any damage that may be caused to the existing embankment, liner and cutoff systems as a result of the Stage VII-B construction.
- ***Geosynthetic Manufacturer:*** The geosynthetic material Manufacturer will manufacture and supply the geosynthetic materials, and provide QC documentation and installation guidelines. The materials will be purchased by the Owner and installed by the Contractor. Approved manufacturers/suppliers are GSE Lining Technology, Inc., Houston, TX, 800.435.2008; and Northwest Linings & Geotextile Products Inc., Kent, WA, 253.872.0244.
- ***Instrumentation Manufacturer:*** The instrumentation equipment Manufacturer is responsible for manufacturing and supplying the instrumentation equipment, and providing QC documentation, installation guidelines, and material warranty, in accordance with Section 12.0. The Owner will purchase and install the instrumentation after the raise construction is completed. The approved Manufacturer/Supplier is Geokon, Inc., Lebanon, NH, 603.448.1562.
- ***CQA Personnel:*** CQA Personnel for the raise will consist of a CQA Project Director, a CQA Field Manager, CQA Field Monitors, and a CQA Document Controller. The CQA Project Director and CQA Field Manager, CQA Field Monitors and CQA Document Controller are employees of URS. URS will be contracted by the Owner to provide the CQA personnel. CQA personnel qualifications and responsibilities are described in Section 3.0.
- ***Laboratory:*** The Laboratory consists of the NANA/DOWL/Alaska Testlab (ATL) laboratory in Anchorage, the Owner's on-site testing facility, URS's on-site density/moisture gage, the soil and concrete testing lab provided by the Owner and the Contractor's on-site tensiometer. The Laboratory is responsible for testing to confirm that the earthwork, geosynthetics and concrete materials used in the construction comply with the Specifications. Laboratory staff

qualifications and responsibilities are described in Section 3.0.

- ***Surveyor:*** The Surveyor will be selected by TCAK. The Surveyor will be responsible for construction surveying. The Surveyor will perform a pre-construction survey, confirm the raise stationing, set up the wing wall stationing, complete panel layouts of all liner, cutoff wall and wing wall components, prepare plans and profiles of the cutoff and curtain wall alignments and terminations, and prepare an as-built topographic survey of the tailings dam.

3.0 CQA PERSONNEL AND RESPONSIBILITIES

The CQA Project Director is responsible for the provision of CQA services for construction of the future raises to closure. General responsibilities are as follows.

- ***CQA Project Director*** - The CQA Project Director is a registered professional engineer (PE) in Alaska who is familiar with the dam design, tailings operations and dam site geotechnical conditions. The CQA Project Director is responsible for CQA program activities on behalf of the CM. The CQA Project Director will review site activities including daily field reports and laboratory test results, approve the cutoff wall and wing wall termination depths in collaboration with the Engineer, address any deficiencies that are encountered and design modifications that are requested, and visit the site if needed during construction.
- ***CQA Field Manager*** - The CQA Field Manager is the on-site representative of the CQA Project Director. The CQA Field Manager will have experience on large earthwork construction and liner installation projects, with detailed geotechnical logging of soil, rock and ice lenses exposed in excavations, arctic conditions of permafrost and ice-rich soils and rock, and dam site geotechnical conditions. The CQA Field Manager will liaison with the CM and Contractor, and supervise the CQA Field Monitors. The CQA Field Manager will report administratively to the CM and technically to the CQA Project Director.
- ***CQA Field Monitors*** - The CQA Field Monitors will assist the CQA Field Manager by monitoring and documenting the Contractor activities and completing the required QA testing. CQA personnel will be contracted by the CM and must have appropriate experience on large earthwork construction and liner installation projects in arctic conditions of permafrost and ice-rich soils and rock.
- ***CQA Document Controller*** – The CQA Document Controller will assist the CQA Project Director and CQA Field Manager in producing, tracking and filing all documents associated with pre-construction, construction and post-construction activities. The CQA Document Controller will file all documents electronically and in hard-copy. This includes but is not limited to tracking and filing the following:
- Pre-construction: CQA Plan, construction drawings and specifications, project schedules, internal and external communications.
    - **Pre-construction**: CQA Plan, construction drawings and specifications, project schedules, internal and external communications.
    - **Construction**: Construction documentation including daily and weekly reports, photos, field and laboratory test results.
    - **Post-construction**: Construction Completion Report, Contractor field documentation and construction report, as-built drawings, and internal and external communications.
- ***Laboratory Personnel*** - Soil test staff will be experienced in soil testing and familiar with American Society for Testing and Materials (ASTM), U.S. Army Corps of Engineers (COE) and other applicable test standards. Soil test staff will also be experienced in the transport, use, and maintenance of the density/moisture gage. Tensiometer operators will be experienced in geosynthetic testing and familiar with ASTM, Federal Test Method Standards (FTMS), National Sanitation Foundation (NSF) and other applicable test standards.

Specific functions and responsibilities of these personnel are presented in the following sections.

3.1 CQA PROJECT DIRECTOR

The CQA Project Director is a registered professional engineer in Alaska who is in direct charge of the CQA program and is responsible for certifying the work. In particular, the CQA Project Director will:

- Review the Geotechnical Report, Drawings, Specifications, CQA Plan and Design Modifications, check for consistency between them, and review materials referenced therein.
- Review comments received from the State of Alaska Department of Natural Resources (ADNR) Dam Safety and Construction Unit, and the TCAK and URS responses to these comments.
- Ensure that project-specific comments and revisions are incorporated into the Drawings, Specifications and CQA Plan, in collaboration with the CM and the Engineer.
- Administer the CQA program, including briefing the CQA Field Manager, supervising on-site CQA personnel, communicating daily regarding construction progress with the CQA Field Manager and communicating with the CQA Document Controller to assure a complete construction file is maintained.
- Review daily and weekly reports immediately upon receipt, review and interpret laboratory test data, and review excavation and construction photographs.
- Review geotechnical logs of cutoff and wing wall excavations with the Engineer, and direct the CQA Field Manager in determining the final excavation depth to competent bedrock.
- Review Contractor submittals forwarded by the CM and CQA Field Manager, collaborate with the Engineer as needed, and respond to the submittals in a timely manner.
- Be on-call during all construction activities and attend site meetings and conference calls as requested by the Owner and CM.
- Review changes to the Drawings and Specifications necessitated by differing field conditions, material availability, constructability constraints, remote site constraints, and adverse weather.
- After collaboration with the Engineer, provide “Design Modifications” to the Owner and CM on any changes to the Drawings and Specifications that are required.
- Prepare, with the CQA Field Manager, Engineer, and CQA Document Controller the final CQA Report and the Construction Completion Report (Certificate) of the Engineer.

A critical part of the CQA Project Director’s responsibilities is to ensure that the cutoff and wing wall excavations and installations are completed to the depths and quality necessary to ensure that a barrier is installed in accordance with the design intent of minimizing seepage out of the tailings impoundment.

To achieve this critical responsibility, the CQA Project Director will regularly communicate with the CQA Field Manager and Engineer, and review the geotechnical logs and digital photographs that will be taken by the CQA Field Manager and transmitted electronically as described in Section 3.2.

If additional direction is needed to ensure that the cutoff and wing wall design intent is met, the CQA Project Director or a qualified alternate will visit the site during the critical parts of the excavation.

3.2 CQA FIELD MANAGER

The CQA Field Manager is the on-site CQA representative. The CQA Field Manager will:

- Serve as the on-site CQA representative reporting to the CQA Project Director, and supervise other on-site CQA personnel.
- Review the quality of the materials, status of the Laboratory and batch plant, and qualifications of CQA personnel for conformance with the Specifications.
- Review the Drawings, Specifications, CQA Plan and geotechnical data for the site, and ensure that CQA personnel are informed of the requirements of the work.
- Attend any relevant Pre-construction and site Progress Meetings and any CQA-specific meetings needed to review the installation and CQA activities, and prepare meeting notes.
- Assign the daily responsibilities of CQA personnel, to ensure that relevant activities of the Contractor are monitored and documented.
- Ensure that materials are classified as they are excavated either into soil types to be stockpiled for use as construction fill, or into unsuitable materials that require disposal.
- Inspect and photograph excavated materials on a representative basis as they are removed from the cutoff wall and curtain wall excavations.
- Ensure that geotechnical logs and photographs are taken of cutoff and wing wall excavations, with particular attention to ice content, blasted rock and fractured rock.
- Ensure that cutoff and wing wall excavations continue through ice-rich soils, blasted rock and fractured rock, and are terminated in competent bedrock in accordance with the Specifications.
- Inspect the sides and bottom of the curtain wall excavation from the top of the excavation using binoculars and a flashlight.
- Review daily reports prepared by CQA personnel, prepare daily field reports at the end of each work day, and prepare weekly summary reports at the end of each week.
- Communicates with the CQA Document Controller to ensure complete filing of all construction documentation.
- Submit all daily and weekly reports and digital photographs electronically to the CM and CQA Project Director as soon as the reports are completed and the photos are cataloged and labeled.
- Collect and review all documentation provided by the Manufacturers, Contractor, Surveyor and CQA Field Monitors.
- Document and forward conformance samples to the Laboratory as appropriate, and review results for conformance and acceptability.
- Record on-site activities that could result in damage to existing embankment, the existing liner and cutoff systems, and Owner property and facilities.
- Designate an on-site CQA person to act on behalf of the CQA Field Manager whenever the CQA Field Manager is off the site upon approval of the Owner, to ensure continuity during operations.
- Prepare, with the CQA Project Director, Engineer and CQA Document Controller the final CQA Report and Construction Completion Report (Certificate) of the Engineer.

The CQA Field Manager will report verbally and with daily and weekly reports, to the CM and CQA Project Director to ensure that any problems are addressed as soon as they occur, and that any potential problems are identified on a timely basis and acted on.

To approve the cutoff and wing wall excavations, and inspect the geosynthetic and concrete material installations in these excavations, the CQA Field Manager will walk the excavations along the bottom of the upper sloped sections safely back from the vertical sections, but will not enter the vertical sections.

Geotechnical logging of vertical sections will be completed by inspecting representative bucket of material as it is excavated, and inspecting vertical sides and bottoms with the aid of binoculars, flashlights and digital photography to the extent possible with respect to safety.

The CQA Field Manager will transmit the geotechnical logs and digital photos electronically to the CQA Project Director before the final depth of each excavation is approved.

3.3 CQA FIELD MONITORS

A sufficient number of CQA Field Monitors are required on site to ensure that the Contractor activities are adequately monitored and documented. The CQA Field Manager may serve as a CQA Field Monitor while supervising other CQA personnel. Activities to be monitored and duties to be carried out include:

- Monitor geosynthetic material logistics, including the handling, storage and protection of the materials prior to deployment;
- Examine all soils for use as components of the dam raise, and collection of samples for laboratory testing to verify conformance with the Specifications;
- Monitor deployment of all geosynthetic materials for damage or flaws, marking any such areas for repair, and documenting the location, size, time, and date of these activities;
- Monitor and document the seaming and joining of all geosynthetic materials, and take captioned photographs of all installed liner panels, which shall be included in the final CQA Report;
- Walk over all completed geosynthetic areas before they are covered to ensure that all welds have been tested, and flaws and damage have been identified, repaired, tested and passed.
- Monitor and conduct *in situ* and laboratory testing of all soils used to ensure that specified materials are used, placed and compacted in accordance with the specifications.
- Monitor and document batch plant operations to ensure that grout and CDF materials are produced in aggregate, cement and water proportions in accordance with the Specifications.
- Note and document any activities that could result in damage to the liner, cutoff and wing wall systems, and report to the CQA Field Manager so that corrective action can be taken.
- Monitor and document all excavation, geosynthetic material placement, concrete material placement, and trench backfill to ensure conformance with the Specifications.
- Communicate with the CQA Document Controller to ensure complete filing of all construction documentation.

The CQA Field Monitors shall assist the CQA Field Manager as needed during the cutoff and wing wall excavations to facilitate the geotechnical logging described in Section 3.2.

4.0 MEETINGS AND SITE VISITS

Meetings of all parties involved with construction of the Future Raises to Closure are required at various times during the project, as required by the CM, to establish work schedules, resolve problems, mitigate potential problems, and maintain open communications. These are outlined in the following sections.

4.1 PRE-CONSTRUCTION MEETING

A Pre-construction Meeting will be held to introduce parties, confirm responsibilities, resolve any issues prior to the start of work, and clarify the CQA requirements. The CQA Plan shall be made available to all parties. In addition, the following points shall be discussed:

- Contractor schedule, milestones, critical path items, work hours and shift logistics.
- Assignment of responsibilities to each party.
- Confirmation that sufficient personnel are provided to meet project requirements.
- Timing and distribution of reports for both work schedules and CQA documentation.
- Lines of authority, reporting and communication.
- Confirmation of scope and items to be included in the pre-construction survey.
- QC procedures and standards for earthwork, geosynthetic and concrete materials.
- Testing logistics and frequency for earthwork, geosynthetics and concrete materials.
- Methodology for review and acceptance of work between the CM and the Contractor.
- Office space and computer network access for Contractor and CQA personnel
- Coordination with ongoing tailings, dam, mill and mining operations, and other site activities.
- Owner site safety procedures, behavior requirements, and site accessibility.
- Time and place of the progress meetings.

Either as part of the Pre-construction Meeting or separately, additional topics pertinent to the CQA program will be discussed and agreed upon by the CM and Contractor, including the following:

- Site walk to confirm the status of the crest of the previous stage, right abutment area, quarry and borrow areas, batch plant, crusher and geosynthetic material storage locations.
- Specific methods of deployment to be used for the geosynthetic materials and operations of the batch plant.
- Review of repair procedures required for different types of flaws or damage, and requirements for welding, testing, monitoring, and documenting repairs.

- Review of precautions needed to protect the subgrade from deterioration due to placement of geosynthetics, and exposure to weather conditions prior to placement of the cover materials.

- Identification of equipment laydown areas, maintenance and repair facilities, roads available for Contractor use, and any areas that are out of bounds to construction personnel.

This meeting shall be documented by the CQA personnel and maintained in the project documents.

4.2 PROGRESS MEETINGS AND REPORTS

The CM shall conduct regularly scheduled Progress Meetings, at a frequency to be determined by the CM, to review the following:

- Work progress with respect to schedule;

- Actual and potential problems, and

- Measures to resolve problems related to the CQA program.

These meetings shall be documented and the decisions reached passed on to all affected parties. The frequency of periodic meetings may be extended by the Owner, if considered warranted by the CM, based on the performance of the Contractor.

The time and place of the first Progress Meeting shall be determined during the Pre-construction Meeting. During each Progress Meeting, the time and place of the next Progress Meeting shall be confirmed. These meetings shall be designated as Progress Meeting No. 1, Progress Meeting No. 2, etc.

The CM shall ensure that Daily Progress Reports are prepared that document the Contractor activities for each day worked, and that Weekly Summary Reports are prepared that outline progress, problems, and resolutions. Areas of concern and potential problems will be identified and addressed at the next Progress Meeting, unless it is of sufficient importance or urgency as to warrant an *ad hoc* meeting.

4.3 SITE VISITS

The CQA Project Director may conduct periodic site visits to ensure that outstanding issues are resolved on a timely basis, and review the progress and methods of construction. Site visits will be made in coordination with the CM if problems arise that either cannot be easily resolved or may impact the design, and to review aspects of the project that are critical to the performance of the design.

The State Dam Safety Engineer and other State representatives and consultants may conduct site visits and inspect the construction activities. These site visits will be coordinated through the CM.

5.0 EARTHWORK CQA

Prior to the start of Contractor's future raises to closure construction activities, the Owner will complete site preparation activities including any associated grading and earthwork.

5.1 PROJECT PLANNING & SCHEDULE DEVELOPMENT

Prior to commencing construction of any future raise, the Contractor, CM and CQA Field Manager will develop a pre-construction plan and schedule. This plan should include an estimated timeline for all excavation, backfill, placement, compaction and grading for each soil type. It should also include an estimated timeline for liner deployment and welding.

Items to be considered while developing this plan also include complicating factors such as:

- Sub-freezing temperatures and how they would affect soil conditions for excavation and backfill and concrete pouring conditions.
- Availability of personnel and equipment on site including excavators, haul trucks, man-lifts, loaders, dozers, pumps (dewatering), etc.
- The need for specialty equipment (ice and rock excavation) that would require additional planning for acquisition or barge transport to the site.
- Material testing equipment for getting time-sensitive test results for soil and concrete.

5.2 SOURCE QUALITY CONTROL

5.2.1 Soils Source Selection

All soil types used in the future raises to closure raise shall be selected in accordance with the Specifications and require the classification and approval of the CQA Field Manager. The CM with support from the CQA Project Director will be responsible for identifying material sources, ensuring adequate supply, conducting source quality control tests, and making the necessary materials available to the Contractor.

Prior to beginning any earthwork, the CM and CQA Field Manager work together to ensure that the following source quality control tests have been completed and results documented in the project files:

- EPA Toxicity standard test to facilitate classification of mineralized and non-mineralized soils.
- Moisture-density characteristics in accordance with ASTM D1557.
- Gradation, index and classification tests.
- Permeability tests on CDF.

The CQA Field Manager shall review the test results, identify the "mineralized" and non-mineralized" soil sources prior to any earthwork construction, and ensure that the identified materials conform to the Specifications.

5.2.2 Reuse of Excavated Material

Soil and rock materials excavated during construction can either be used in the construction or disposed. The CQA Field Manager shall determine the suitability of excavated material for reuse in the construction. All material identified as non-suitable for reuse shall be classified as "waste".

The CQA Field Manager or his or her designator shall identify and plan any laboratory soil testing on the excavated material that may be considered necessary to assist in the classification process. The laboratory testing could include:

- Toxicity based on EPA standards for classifying mineralized and non-mineralized soils.
- Moisture density characteristics in accordance with ASTM D1557.
- Gradation, index and classification tests.

Based on the laboratory test results and field observations on the materials identified as suitable for reuse, CQA Field Manager shall:

- Classify in accordance with the Unified Soil Classification System (USCS).
- Record the degree of weathering.
- Record the percent ice by volume in accordance with the Description and Classification of Frozen Soils.

Based on the above, CQA Field Manager or his or her designator shall further classify the suitable excavated materials for reuse in construction into one of the soil Types defined in the Specifications.

5.3 EXCAVATION

The Contractor is required to complete the earthwork activities to the lines and grades specified in the Plans and Specifications. However, the bottom elevations of the cutoff and wing wall excavations shown on the Drawings may change because they are based on interpolations between geotechnical borings and engineering interpretations.

The CQA Field Manager in consultation with the Engineer will determine the bottom elevations of the cutoff trench, cutoff wall, and wing wall trench. The top of the wing wall trench and the bench elevation may be adjusted by the Contractor with the CQA Field Manager's approval to suit the excavation equipment and the geology encountered.

The Contractor is required to complete the excavation in accordance with the Drawings and Specifications. The CQA Field Manager shall observe, review, and approve the final depth of the excavations for the cutoff trench, cutoff wall, and wing wall. During the excavation work, the Contractor

is required to plan his or her activities to facilitate the CQA Field Manager's observation, classification, and logging activities by:

- Showing representative buckets of excavated material to the CQA Field Manager or his or her designator and spreading material in temporary piles for the CQA Field Manager's inspection and characterization.

- Maintaining the size of the temporary piles to facilitate access of the CQA Field Manager or designator to the entire material from representative buckets of excavated material.

- Moving the inspected material to temporary holding stockpiles for further classification or to a material stockpile according to the classification assigned by the CQA Field Manager or designator.

- Obtaining approval of all subgrades from the CQA Field Manager prior to terminating the excavation.

During excavation, the CQA Field Manager or his or her designator shall observe the excavated material and subgrade conditions and perform the following activities:

- Observe the excavation to document that exposed bedrock contains less than the specified ice by volume in areas shown on the Drawings as foundation, cutoff trench and wing wall excavations.

- Observe the soil types during excavation, and record the depth and location of changes in soil type, as well as any other pertinent geologic information on a geologic map.

- Notify the Owner and CM immediately if changed or unexpected geologic conditions are encountered.

- Classify all excavated materials according to material type as described in the Specifications and in accordance with the Unified Soil Classification System (USCS).

- Classify all materials with respect to the degree of weathering and percent ice by volume, in accordance with the Description and Classification of Frozen Soils.

- Identify excavated materials not suitable for reuse and classify them as "waste". Ensure that the Contractor stockpiles "waste" materials in a separate stockpile for removal from site.

- Inspect and photograph excavated materials as they are removed from the cutoff wall and curtain wall excavations.

- Complete geotechnical logs and take photographs of cutoff and wing wall excavations, with particular attention to ice content, blasted rock, and fractured rock.

- Ensure that cutoff and wing wall excavations continue through ice-rich soils, blasted rock and fractured rock, and are terminated in competent bedrock in accordance with the Specifications.

- Inspect the sides and bottom of the curtain wall excavation from the top of the excavation using binoculars and a flashlight when necessary.

- Direct the Contractor to suitable areas for stockpiling excavated materials acceptable for use in

construction and classifying them into soil types and mineralized and non-mineralized types.

- Ensure that the Surveyor completes the survey of the bottom of the excavated surface prior to backfilling.
- Review the Contractor's work and confirm that depths and slopes of excavations, ramps, trenches, ditches, roads, and other pertinent features meet design requirements.
- Determine final acceptable depth of excavation of the cutoff trench, cutoff wall trench and wing wall trench

The CQA Field Manager and CQA Field Monitors shall record all observations on daily field monitoring report forms, including drawings and geologic maps as appropriate.

5.4 SOILS PURCHASE, DELIVERY, STORAGE AND HANDLING

5.4.1 Purchase Responsibility

The CM with the aid of the CQA Field Manager shall identify material sources, ensure adequate supply, conduct source quality control tests, and make the necessary materials available to the Contractor. The CM shall also ensure that the CQA Field Manager has approved the material sources and types, prior to beginning any earthwork.

5.4.2 Product Delivery, Storage and Handling

The Contractor is required to obtain materials from stockpiles at the material sources within the mine area, and to transport these materials to the construction site. The CQA Field Manager shall:

- Inspect, characterize, and classify the materials into "mineralized" and "non-mineralized" soils, and into one of the soil types defined in the Specifications.
- Direct the Contractor to suitable areas for stockpiling.
- Ensure that the Contractor stores the materials in separate stockpiles as follows:
  - mineralized
  - non-mineralized
  - each soil type
  - waste

The Contractor is required to remove all waste material and dispose of it at locations identified by the CM.

5.5 PLACEMENT AND COMPACTION

The Contractor will complete the placement and compaction for the earthwork activities in accordance with the Drawings and Specifications. In order to ensure conformance to the Specifications, the CQA

Field Manager or his or her designator shall perform the following activities during placement of the different soil Types:

- Prior to the placement of each soil Type, verify that the subgrade has been prepared (scarified, moisture-conditioned, and compacted) in accordance with the Specifications.
- Review the soil testing data to verify that materials conform to the Specifications.
- Visually observe soils for contamination with debris or deleterious material, and inclusion of ice and snow.
- Conduct visual observations and tests to confirm that placed material meets the Specifications, including lift thickness, gradation, percent compaction, and in-place density and moisture.
- Verify that final lines and grades conform to design requirements and that minimum thicknesses have been achieved prior to placing overlying layers
- Where soils are placed over geosynthetics, observe the soils placement to confirm minimum thickness under equipment to prevent damage to the underlying geosynthetic materials.
- Visually observe to detect any damage to underlying geosynthetic materials, if present.
- Monitor the confirmation surveying performed by the Surveyor. Evaluate, determine, and modify the extent of the confirmation surveying.
- Monitor and inspect all soil placement and compaction to verify compliance with the Specifications, especially above the geosynthetic components to avoid damage to them.

If *in situ* testing indicates that moisture contents or densities are outside the Specification limits, the failing area shall be reworked or removed and replaced. These areas shall be retested and the repair process repeated as necessary until passing results are achieved.

Observations shall be recorded on daily field monitoring report forms, drawings, and test data forms.

5.6 SOIL TESTING

5.6.1 Laboratory Soil Testing

Laboratory testing of soils to be used in the future raises to closure raise shall be carried out for purposes of soils classification, acceptance, and QA during construction. All testing shall be in accordance with the associated ASTM standard or other procedure listed in the Specifications.

All laboratory equipment shall be in current calibration, traceable to nationally recognized standards. The CQA Field Manager shall be given copies of calibration certificates for the project records.

The CQA Field Manager or his or her designator shall be responsible for identifying and collecting samples for testing from materials removed from excavations or delivered to stockpiles. The CQA Field Monitors shall conduct the laboratory tests. The laboratory testing shall include:

- Compaction tests:

- ♦ For Soil Types 4 and 8 at least three compaction tests in accordance with ASTM D1557, regardless of the quantity of materials placed and compacted. Appropriate soil correction factors shall be used.

- ♦ For Soil Types 1, 3, and 7, the compaction criteria are established from a test fill (in lieu of laboratory tests) to specify the equipment, lift thickness and number of passes.

- Gradation, index and classification tests.

Prior to use in construction, the CM and CQA Field Manager shall evaluate test results, and the CQA Project Director shall have final approval authority of soil use in construction.

The CQA Field Manager shall identify and require any additional laboratory testing during construction that may be necessary to ensure that the materials do not vary significantly within each soil type, or adversely during the course of the work, and that the materials consistently meet the Specifications.

5.6.2 Field Soil Testing

The CQA Field Monitors shall provide *in situ* testing of the soils after placement and compaction of each layer, to determine their compacted properties and verify conformance with the Specifications. The Contractor is required to seek and obtain the CQA Field Manager's approval for each layer of subgrade and fill prior to placing subsequent layers of materials.

For Soil Types 4 and 8, the *in situ* soil testing shall be carried out using a nuclear moisture-density gauge and the dry density and moisture content determined in the field. The CQA Field Manager or his or her designator shall verify conformance with the Specifications using the results of the *in situ* testing and the results of the laboratory compaction tests. The *in situ* test frequency shall be as necessary for approval by the CQA Project Director.

For Soil Types 1, 2, 3, 7 and 9 the Contractor is required to place the first quantity of these materials as test fills and demonstrate the compaction equipment, lift thickness, and approach to the CQA Field Manager prior to further placement of these materials. The CQA Field Manager may require that the Contractor build a test fill to demonstrate the compaction approach using Soil Types 4 and 8 as well.

Based on the test fill demonstration by the Contractor, the CQA Field Manager or his or her designator will establish the lift thickness, equipment and minimum number of passes for each soil type. The Contractor is required to adhere to these compaction criteria.

If the CQA Field Manager or CQA Field Monitors observe a change in either the characteristics of a soil type or the method of compaction, the CQA Field Manager shall require a new test fill and revise the compaction criteria. The CQA Field Manager shall not approve any change in the established compaction criteria without a new test fill evaluation.

6.0 GEOMEMBRANE CQA

This Section presents information regarding CQA for the High Density Polyethylene (HDPE) geomembrane. The geomembrane will be a HDPE resin manufactured in 100-mil thick sheets and delivered to the site in rolls. This geomembrane will function as the barrier part of the liner system.

6.1 MANUFACTURE

6.1.1 Manufacturer Qualifications

The CM shall select a geomembrane Manufacturer, that is a commercial entity normally engaged in the manufacture of geomembranes for waste containment. The Manufacturer shall have the following:

- At least 5 years continuous experience in manufacturing the specified type of geomembrane
- Manufactured at least 10 million square feet of geomembrane for at least 10 complete facilities
- Satisfaction of all appropriate trade certifications.

6.1.2 HDPE Geomembrane Resin

The geomembrane resin shall be new HDPE of first quality, compounded and manufactured specifically for producing HDPE geomembrane. Recycled materials or seconds shall not be used. Resin types shall not be mixed during manufacturing. The resin shall meet the requirements in Table 6-1.

Table 6-1 Geomembrane Resin Requirements

Physical Characteristic	Units	Test Method	Requirement
Density (min.)	g/cm^3	ASTM D1505	0.932
Melt Index	g/10 min	ASTM D1238 Condition 190/2.16	≤ 1.0

6.1.3 HDPE Geomembrane

The geomembrane shall be manufactured from HDPE resin, in 100-mil thick sheets and conform to the physical requirements in Table 6-2. The Manufacturer shall use materials produced in North America or as accepted by the CM. The Manufacturer shall identify percentage of processing aids, antioxidants, carbon black, and other additives. The geomembrane shall also meet the following requirements:

- Additives other than carbon black or pigment shall not exceed a combined maximum total of 1% by weight;
- Carbon black content shall not exceed 3% by weight,

- The total combined processing aids, antioxidants, carbon and other additives shall not exceed 3.5% by weight of finished geomembrane,
- All additives for UV protection, thermal stability, color, or processing agents shall not "bloom" to the surface over time or inhibit welding,
- Finished product shall be free of blemishes, holes, pin holes, bubbles, blisters, excessive gels, undispersed resins, and carbon black, contamination by foreign matter, and nicks or cuts on edges,
- Manufactured geomembrane sheets or panels shall be rolled for shipment, and
- Geomembrane delivered to the site shall come from a maximum of two lots.

Table 6-2 Properties and Quality Control Tests for HDPE Geomembrane

Physical Characteristic	Qualifier	Units	Test Method	Min Frequency	Requirement
Sheet Thickness (nominal)	min. avg. value	mil	ASTM D5199	5 per roll	100
	lowest individual reading	-	-		90
Top and Bottom Surfaces	min. avg. value	mil	Textured	10	Top and Bottom Surfaces
Specific Gravity (Sheet)	lowest individual reading	-	ASTM D1505	See Note 1	0.94
Tensile Properties (each direction)			ASTM D6693 (Type IV)	See Note 2	
Strength at Yield	min. avg. value	lb/in	ASTM D6693	See Note 2	210
Elongation at Yield	min. avg. value	%	ASTM D6693	See Note 2	12
Strength at Break	min. avg. value	lb/in	ASTM D6693	See Note 2	380
Elongation at Break	min. avg. value	%	ASTM D6693	See Note 2	700
Tear Resistance	min. avg. value	lb	ASTM D1004	See Note 2	70

Table 6-2 Properties and Quality Control Tests for HDPE Geomembrane					
Physical Characteristic	**Qualifier**	**Units**	**Test Method**	**Min Frequency**	**Requirement**
Puncture Resistance	min. avg. value	lb	ASTM D4833	See Note 2	180
Carbon Black Content	Range	%	ASTM D1603 or D4218	See Note 2	2-3
Carbon Black Dispersion	min. 8 of 10	category	ASTM D5596	See Note 2	Category 1 or 2
Oxidation Induction Time	min. avg. value	minutes	ASTM D3895 or D5885	See Note 2	100
UV Resistance	min. avg. value	% retained after 1,600 hours	GM11 and ASTM D5885	See Note 2	50
Notes: 1. Either 1 per 100,000 sf of sheet, or 1 per resin batch, whichever results in the greatest number of tests 2. One test per resin batch on typical sheet and seam.					

6.1.4 Extrudate Rod or Bead

Extrudate rod or bead shall meet the following requirements:

- Manufactured from the same resin as the geomembrane;
- Contain thoroughly dispersed additives throughout the rod or bead;
- Contain 2 to 3 percent carbon black; and
- Free of contamination by moisture or any other foreign matter.

6.2 SOURCE QUALITY CONTROL

6.2.1 Quality Control Tests and Certification

The Manufacturer shall perform source QC testing on the geomembrane at the manufacturing plant as indicated in Table 6-2. The objective of this testing shall be to confirm the Manufacturer published material characteristics and demonstrate the materials compliance with the specifications.

The Manufacturer shall reject rolls for which QC requirements are not met, and shall certify the quality of all rolls of geomembrane shipped to the site. The physical property test results of the geomembrane shall be submitted to the CM prior to shipping in accordance with the frequency outlined in Table 6-2.

The CQA Project Director, in collaboration with the Engineer, shall review the documentation to check if the geomembrane conforms to the specifications.

6.2.2 Labeling

The Manufacturer shall mark or tag all geomembrane rolls with the following information:

- Manufacturer's name;
- Product identification,
- Lot number,
- Roll number,
- Roll dimensions, and
- Special handling requirements.

6.2.3 Warranty

The Manufacturer shall provide a Manufacturer's warranty for the geomembrane material. The warranty shall include a minimum 5-year warranty for the materials against deterioration.

6.3 PURCHASE, DELIVERY, STORAGE AND HANDLING

6.3.1 Purchase Responsibility

The Owner will purchase the geomembrane from the Manufacturer (or Supplier) in sufficient quantity and store it at the mine site. The Manufacturer shall submit product data, QA/QC documents, and samples to the CM for the CQA Project Director's approval prior to shipping. The CM shall ensure that the CQA Project Director has approved the products, prior to authorizing the shipment.

6.3.2 Manufacturer QA/QC Documents

The Manufacturer (or Supplier) shall submit the following product data to the CM and product samples directly to the Engineer, and wait for the CQA Project Director's approval prior to shipping the geomembrane to the site:

- Resin Data:
  - Production date or dates;
  - Certification that the resin meets the product requirements (see below),
  - Certification that all resin is from the same Manufacturer,
  - Copy of quality control certificates issued by the Manufacturer,
  - Test reports from the Manufacturer, and

- Statement that no reclaimed polymer was added to the resin during the manufacture.

- Geomembrane Roll Data:
  - Production date or dates;
  - Laboratory test results and certification stating that the geomembrane meets requirements,
  - Certification stating that all geomembrane rolls are furnished by one supplier,
  - Certification that all rolls are manufactured from one resin type obtained from one supplier,
  - Copy of quality control certificates indicating compliance with the Specifications,
  - Test reports from the Manufacturer,
  - Statement that no reclaimed polymer is added to the resin,
  - List of percentages/total of processing aids, antioxidants, and other additives other than carbon black added to or in the resin,
  - Manufacturer recommended geomembrane delivery, storage, and handling instructions,
  - Manufacturer recommended geomembrane installation instructions, and
  - Sample warranties for review by the CM.

- Extrudate Beads and/or Rod Data:
  - Statement of production date or dates;
  - Laboratory certification stating that the extrudate meets the product requirements,
  - Certification stating that all extrudate is manufactured by one Manufacturer,
  - Certification that resin is supplied from one supplier,
  - Copy of quality control certificates issued by the Manufacturer,
  - Test reports from the Manufacturer, and
  - Certification that the extrudate bead or rod resin is the same type, from the same Manufacturer, and compatible with the resin used to manufacture the geomembrane.

The CQA Project Director and CQA Field Manager shall review the QC certificates and test results for conformance with the Specifications and that the materials on-site correspond to materials for which the documentation was prepared. Any discrepancies and non-conformance shall be reported to the CM, with details of the discrepancies and nature of the materials on-site. Any geomembrane that fails to meet the physical property requirements shall not be deployed.

6.3.3 Product Delivery, Storage and Handling

Delivery, storage and handling of the geomembrane shall conform to ASTM D4873-02 and the Manufacturer's recommendations. The Manufacturer shall label each roll with the lot number, roll number, and other information necessary to identify it for inventory and QC and CQA purposes.

6.3.3.1 Delivery

The Manufacturer shall deliver materials to a location to be determined by the CM only after the CM has accepted the required submittals from the Supplier. The CM acceptance shall be based on the CQA Project Director's review findings.

All geomembrane shall be delivered in rolls. Folded geomembrane shall be rejected. Upon receipt at the site, the CM shall inspect the surfaces of all rolls for defects and for damage. This inspection shall be made without unrolling rolls unless defects or damages are seen or suspected. The CM will determine:

- Rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws;
- Rolls or factory panels which include repairable flaws, and
- Rolls that are not properly labeled.

No unlabelled rolls shall be used on the Future Raises to Closure raise, and shall be returned to the Manufacturer and replaced by labeled rolls.

The CM will ensure that any damaged protective covering is repaired immediately, and that the integrity and legibility of geomembrane roll labels is preserved.

6.3.3.2 Storage and Handling on Site

Upon receipt of delivery, the CM will store the geomembrane on site. Upon the Contractor's arrival on site, the Contractor shall inspect the geomembrane, inventory the materials, and accept responsibility for the future storage and handling of the materials.

Storage of geomembrane rolls shall be subjected at all times to the following conditions:

- Protect from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, vandalism, excessive heat and cold, or any other deleterious condition;
- Store on a prepared surface, and
- Not stack more than three rolls high.

Handling of the geomembrane rolls shall at all times be subject to the following conditions:

- Use cloth chokers, spreader bars and roll bars to load, move and deploy rolls;
- No dragging of rolls on the ground, and

- No folding of rolls.

The CM and CQA Field Manager shall monitor all geomembrane handling operations and storage procedures to prevent damage to the materials.

6.4 INSTALLATION

6.4.1 Deployment

The CQA Field Manager will continuously monitor and document the deployment of the geomembrane, document the time and location of material placement, note any damage to the materials, and record weather and subgrade conditions, to ensure compliance with the Specifications and the seam and geomembrane layout plan.

For any damage to the materials, the Specifications define the repair method to be used. The Contractor will complete all repairs in accordance with the Specifications.

6.4.2 Seaming

All field seaming shall conform to the approved seam and geomembrane layout plan. Field welding operations must be completely monitored and documented by the CQA Field Manager to ensure conformance with the Specifications. Documentation shall include (at a minimum):

- Record of trial welds including date, time, welder identification (ID), machine ID and settings, and test results;
- Seam number, based on a seam numbering system agreed to by the CQA Field Manager and Contractor,
- Date, time, welder ID, machine ID and settings for each weld, and
- Ambient temperature.

Before each day of liner welding, the Contractor will provide a three-foot long test weld by each welding machine. The test welds will undergo destructive weld tests in accordance with the Specifications.

6.4.3 Nondestructive Seam Testing

The purpose of nondestructive testing is to detect any discontinuities or holes that may exist in the welds, and indicate whether a seam has any unwelded sections. Nondestructive tests include:

- Acoustic method – ultrasonic pulse echo;
- Acoustic method – continuous wave resonant frequency,
- Vacuum box,
- Double seam pressurization, and
- Spark testing.

The Contractor is required to perform nondestructive seam tests. Nondestructive testing must be performed over the entire length of all seams in accordance with the Specifications.

The CQA Field Manager shall observe and document the testing to verify compliance with the Specifications, and shall record weld defects and repairs. Documentation will include the date, time, location, and pass or fail for each test.

The CQA Field Manager will outline the failed areas with a waterproof marker compatible with the geomembrane, inform the Contractor of any required repairs, and record the repairs needed.

6.4.4 Destructive Seam Testing

Destructive seam tests will be performed at intervals of at least one test per 500 linear feet for geomembrane welds. The CQA Field Manager may reduce the testing frequency in accordance with GRI GM14. Reasons for performing more tests may include, but are not limited to:

- Wrinkling in seam area;
- Excess crystallinity,
- Suspect welding equipment or techniques,
- Weld contamination,
- Insufficient overlap,
- Adverse weather conditions,
- Possibility of moisture, dust, dirt, debris, and other foreign material in the weld, and
- Failing tests.

The CQA Field Manager will select locations where weld samples will be cut for destructive testing. The locations will be selected so that at least one test per 500 linear feet of weld length is conducted. This is an average frequency for the entire installation; individual samples may be taken at different intervals. Vertical welding such as that conducted on the curtain wall will not be destructively tested; vertical welds will only undergo non-destructive testing.

If the number of failed samples exceeds 2 percent of the tested samples, the testing frequency may be increased at the discretion of the Engineer. Samples taken as a result of failed tests do not count toward the total number of required tests.

The CQA Field Manager shall not inform the Contractor in advance of destructive sample locations. The Contractor shall collect samples in accordance with the Specifications. The CQA Field Manager shall:

- Observe the sample cutting operations;
- Mark each sample with an identifying number that contains the seam number, destructive test number, welder, date, and time, and

- Record the sample location and reason for taking the sample.

The CQA Field Manager shall observe the Contractor field tests. A geomembrane weld sample passes when the break is a ductile, film-tearing bond (FTB). The test strip must break at the edge of the weld, but not within the weld. The weld strength must meet the values listed in the Specifications. If a field test sample fails, the Contractor shall perform repairs in accordance with the Specifications.

Two types of destructive tests shall be performed:

- Peel adhesion (peel) that addresses weld quality, and
- Bonded weld strength (shear) that measures the continuity of tensile strength through the weld and into the parent material.

The destructive tests shall be conducted concurrent with the welding. The CQA Field Manager shall record the results on a destructive seam test form, panel/seam log, and liner layout plan.

If the test fails in either peel or shear, the Contractor shall perform repairs as outlined in the Specifications. This process shall be repeated until passing tests bracket the failed weld section. All welds must be bounded by locations from which passing laboratory tests have been taken.

6.4.5 Liner Anchor

The CQA Field Manager shall monitor anchor trench backfilling to ensure that anchors are constructed in accordance with the Drawings and Specifications and the geomembrane is not damaged.

6.4.6 Repairs

Any geomembrane with a flaw, that fails a nondestructive or destructive test, where destructive tests were cut, or where nondestructive tests left cuts or holes, must be repaired in accordance with the Specifications. The CQA Field Manager shall locate, record, and monitor and document all repairs to ensure that they are completed in accordance with the Specifications.

7.0 GEOTEXTILE CQA

This section presents information regarding CQA for two types of non-woven geotextile installations:

- Type 1 - 10-oz/yd^2 geotextile that will act as a filter to prevent filter drain material from migrating into the transition rock.
- Type 2 - 16-oz/yd^2 geotextile that will add shear strength to the slope and will act as a filter to prevent liner bedding material from migrating into the filter drain.

7.1 MANUFACTURE

7.1.1 Type 1 and Type 2 Geotextiles

Type 1 and Type 2 geotextiles shall comprise a non-woven, pervious sheet of polyester, polypropylene, polyethylene, or polyamide fibers oriented into a stable network so that the fibers retain their relative position with respect to each other during handling, placement and service. They shall be composed of continuous or staple fibers held together through needle-punching, and made primarily from virgin materials with no more than 5 percent by weight of clean recycled polymer.

The Type 1 and Type 2 geotextiles shall conform to the physical requirements in Table 7-1.

Table 7-1 Physical Requirements (MARV) for Type 1 and Type 2 Geotextiles

Physical Characteristic	Unit	Test Method	Type 1 Geotextile	Type 2 Geotextile
Mass (Weight) per Area	oz./ yd^2	ASTM D5261	10.0	16.0
Grab Tensile Strength	lb	ASTM D4632	230	370
Puncture Strength	lb	ASTM D4833	120	170
Trapezoid Tearing Strength	lb	ASTM D4533	95	145

7.1.2 Source Quality Control

The Manufacturer shall perform source QC testing on the geotextile at a minimum frequency of once every 100,000 square feet. The objective of this testing shall be to confirm the Manufacturer's published material characteristics and demonstrate the material's compliance with the Specifications.

The Manufacturer shall reject rolls for which the QC requirements are not met, and certify the quality of all rolls shipped to the site. The Manufacturer shall submit the results of the QC tests to the CM of all rolls of geotextile shipped to the site. This submittal shall be made prior to shipping the geotextile.

The CQA Project Director, in collaboration with the Engineer, shall review the documentation to check if the geotextile conforms to the Specifications.

7.1.3 Labeling

The Manufacturer shall mark or tag all geotextile rolls with the following information:

- Manufacturer's name;
- Product identification,
- Lot number,
- Roll number,
- Roll dimensions, and
- Special handling requirements.

7.2 PURCHASE, DELIVERY, STORAGE AND HANDLING

7.2.1 Purchase Responsibility

The Owner will purchase all geotextiles from Manufacturer (or Supplier) in sufficient quantity and store them at the mine site. The Manufacturer shall submit product data, QA/QC documents, and samples to the CM for the CQA Project Director's approval prior to shipping. The CM shall ensure that the CQA Project Director has approved the products, prior to authorizing the shipment.

7.2.2 Manufacturer QA/QC Documents

The Manufacturer (or Supplier) shall submit the following product data to the CM and product samples directly to the Engineer, and wait for the CQA Project Director's approval prior to shipping the geotextile to the site:

- Geotextile Roll Data:
  - ♦ Written procedures for storing, handling, installing, repairing and welding geotextiles;
  - ♦ Manufacturer material specifications, product literature, and samples of all materials,
  - ♦ Manufacturer certifications for all furnished geotextiles, verifying that they have specified property values of Minimum Average Roll Values (MARV) for geotextiles furnished, and
  - ♦ Manufacturer's Source Quality Control testing results as required by Section 7.1.2.
- Product Samples (directly to the CQA Field Manager):
  - ♦ Two samples of each type of geotextile supplied for the work, with each sample being one piece and at least 12 inches wide and 48 inches long, and
  - ♦ Label each sample with brand name and furnish documentation of the lot and roll number from which the sample was obtained.

The CQA Project Director and CQA Field Manager shall review all QC certificates and test results for conformance with the Specifications and to confirm that the materials on-site correspond to the materials for which the documentation was prepared. Any discrepancies and non-conformance shall be reported to the CM with details of the discrepancies and nature of the materials actually on-site. Any geotextile that fails to meet the physical property requirements shall be not deployed.

7.2.3 Product Delivery, Storage and Handling

Delivery, storage and handling of the geotextile shall conform to ASTM D4873-02 and the Manufacturer's recommendations. The Manufacturer shall label each roll with the lot number, roll number, and other information necessary to identify it for inventory and QC and CQA purposes.

7.2.3.1 Delivery

The Manufacturer shall deliver materials to a location to be determined by the CM only after the CM has accepted the required submittals from the Supplier. The CM acceptance shall be based on the CQA Project Director's review findings.

All geotextile rolls shall be shipped in a closed trailer. Shipment shall be protected with suitable wrapping for protection against moisture and ultraviolet exposure. Upon delivery, the CM shall inspect the surfaces of all rolls for defects and for damage. This inspection shall be conducted without unrolling rolls unless defects or damages are found or suspected. The CM will determine:

- Any rolls or parts of rolls with severe flaws that should be rejected and removed from the site;
- Any rolls or parts of rolls that contain flaws that can be repaired, and
- Rolls that are not properly labeled.

No unlabelled rolls shall be used on the future raises to closure, and shall be returned to the Manufacturer and replaced by labeled rolls.

The CM will ensure that any damaged protective covering be repaired immediately, and that the integrity and legibility of geotextile roll labels are preserved.

7.2.3.2 Storage and Handling on Site

Upon receipt of delivery, the CM will store the geotextile on site. Upon the Contractor's arrival on site, the Contractor shall inspect the geotextile in accordance with the Specifications, inventory the materials, and accept responsibility for the future storage and handling of the materials. At all times, the storage of geotextiles shall be subjected to the following conditions:

- Protected from ultraviolet light exposure, precipitation, inundation, mud, dirt, dust, puncture, cutting, and other damaging or deleterious condition.
- If stored outdoors:
    - Elevated off of the ground, and

- Protected and with waterproof covers.

- Handled in a manner that prevents damage.

The CM and CQA Field Manager shall monitor all geotextile handling operations and storage procedures to ensure damage prevention to the geotextile.

7.3 GEOTEXTILE INSTALLATION

7.3.1 Deployment

The CQA Field Manager shall continuously monitor and document the deployment of the geotextile, document the time and location of material placement, note any damage to materials, and record weather and subgrade conditions to ensure conformance with the Specifications.

For any damage to the materials, the Engineer shall prescribe the method of repair to be used, based on the nature and size of the problem. All repairs shall be performed in accordance with the Specifications.

7.3.2 Installation

The CQA Field Manager shall periodically monitor the geotextile installation operations for proper overlap and technique. Upon completion, the CQA Field Manager shall confirm that all geotextiles have been properly overlapped in accordance with the Specifications.

7.3.3 Repairs

Any damage or flaws to the geotextile shall be repaired in accordance with the specifications. The CQA Field Manager shall locate, record, and monitor and document all repairs to ensure that they are completed in accordance with the Specifications.

8.0 GEOSYNTHETIC CLAY LINER (GCL) CQA

This Section presents information regarding CQA for the Geosynthetic Clay Liner (GCL). This GCL was originally designed to function as a hydraulic barrier at the seams between GSE CurtainWall panels and Geomembrane along the top of the panel, but was later removed from the design. However, it is included in this CQA plan in the event that it may be needed in another situation.

8.1 MANUFACTURE

The GCL is manufactured as a composite of a uniform layer of low permeability granular sodium bentonite encapsulated between two geotextiles. The composite shall be stabilized by needle punching through the top and bottom layers of geotextile. The GCL specified shall conform to the following requirements:

- Bentonite in the GCL shall be sodium montmorillonite clay with a minimum free swell value of 20.
- The GCL shall have a minimum bentonite weight of 0.75 lbs/feet^2.
- The GCL shall have a maximum hydraulic conductivity of 5 x 10^{-9} cm/sec under a gradient of 1.0.
- The bentonite shall be stabilized by needle punching through the top and bottom geotextiles to enhance the internal shear strength of the GCL and to maintain its integrity under hydration.
- The bentonite shall be continuously contained throughout the GCL and so that no displacement of the bentonite occurs when the material is unrolled, moved, cut, torn, or punctured.
- The encapsulating geotextiles used in the GCL shall be non-woven, needle-punched polypropylene with a minimum nominal weight of 6 oz/yd^2.
- The minimum interface shear strength between the GCL and HDPE geomembrane in the final cover system shall have the following values:
  - Peak Strength:

    Friction Angle = 26 degrees

    Cohesion = 270 pounds per square foot (psf)
  - Residual Strength:

    Friction Angle = 18.4 degrees

    Cohesion = 0 psf

- Bentonite for installing the GCL shall be in powder or granular form, and shall be equivalent to the bentonite used in the manufacture of the GCL.

- The GCL manufacturer shall include adequate accessory bentonite with the GCL shipment.

The following product is considered to have acceptable interface shear strengths and other properties specified above and do not need not to be retested for the interface shear strength:

- Bentofix Thermal Lock NWL.

The Engineer may approve a substitute GCL after evaluating its properties with or without additional interface shear strength tests. Any additionally required tests shall comply with the following:

- Comply with ASTM D5321-02 and the requirements of this paragraph;

- Complete at least two shear tests for each interface,

- Measure strengths for each test at three normal loads of 100, 300 and 500 psf, respectively,

- Perform tests on 12-inch square samples and final shear displacement shall be 2 inches,

- Perform tests under saturated conditions,

- Allow GCLs to hydrate for at least 24 hours before testing under normal loads of at least 100 psf, and

- Compact soil materials (if any) to the same condition as required for construction.

The Engineer may accept other combinations of friction angle and adhesion which can be shown to provide comparable strength.

8.1.1 Source Quality Control

The Manufacturer shall perform source QC testing on the GCL to confirm the published material characteristics and demonstrate compliance with the Specifications. Testing shall be performed at least once per lot or once every 100,000 square feet, whichever results in the greater number of tests.

The Manufacturer shall reject rolls for which the QC requirements are not met, and shall certify the quality and submit the results of the QC tests of all rolls shipped to the site. Each QC certificate shall include roll identification numbers and results of the QC tests.

The GCL manufacturer shall examine the entire GCL surface using a metal detector or other suitable method to verify that no needles or other sharp objects are present. The manufacturer shall certify in writing that the GCL surfaces are needle-free.

These submittals shall be made prior to shipping the GCL to the site to facilitate review and approval by the CQA Project Director and Engineer of the material to be shipped. The CM shall review the documentation to check if the GCL conforms to the requirements of the Specifications.

8.1.2 Labeling

Manufacturer shall mark or tag each GCL roll with the following information:

- Manufacturer name;
- Product identification,
- Lot number,
- Roll number,
- Roll dimensions, and
- Special handling requirements.

8.2 PURCHASE, DELIVERY, STORAGE AND HANDLING

8.2.1 Purchase Responsibility

The Owner will purchase all GCL from Manufacturer (or Supplier) in sufficient quantity and store it on the site. The Manufacturer shall submit product data, QA/QC documents, and samples for the CQA Project Director's approval prior to shipping. The CM shall ensure that the CQA Project Director has approved the products, prior to authorizing the shipment.

8.2.2 Manufacturer QA/QC Documents

The Manufacturer (or Supplier) shall submit the following product data to the CM and product samples directly to the Engineer, and get the CQA Project Director's approval prior to shipping the GCL to the site:

- GCL Data:
  - ♦ Manufacturer's written certification of the following:
    - ▪ The proposed GCL has material property values required by the Specifications.
    - ▪ The Manufacturer has continuously inspected the GCL for the presence of needles and found it to be needle-free.
    - ▪ Loose bentonite for the seaming consists of the same natural sodium bentonite as used in the GCL.
    - ▪ Bentonite will not shift during transportation or installation thereby causing thin spots in the body of the GCL.
  - ♦ Manufacturer source quality control testing results.
  - ♦ Written procedures for storing, handling, installing, repairing and seaming the GCL.
- Product Samples (directly to the CQA Field Manager):

- ♦ Two samples of GCL that was manufactured for the project, with each sample being one piece and at least 12 inches square.

- ♦ Label each sample with brand name and furnish documentation of the roll number from which the sample was obtained.

The CQA Project Director and CQA Field Manager shall review all QC certificates and test results for conformance with the Specifications and that the materials on-site correspond to the materials for which the documentation was prepared. Any discrepancies and non-conformance shall be reported to the CM with details of the discrepancies and nature of the materials actually on-site. Any GCL that fails to meet the physical property requirements shall not be deployed.

8.2.3 Product Delivery, Storage and Handling

Delivery, storage and handling of the GCL shall conform to ASTM D4873-02 and the Manufacturer's recommendations. The Manufacturer shall label each roll with the lot number, roll number, and other information necessary to identify it for inventory and QC and CQA purposes.

8.2.3.1 Delivery

Manufacturer shall deliver materials to a location to be determined by the CM only after the CM has accepted the required submittals from the Supplier. The CM acceptance shall be based on the CQA Project Director's review findings.

All GCL shall be supplied in rolls wrapped individually in impermeable and relatively opaque protective covers. Upon receipt at the site, the CM shall inspect the surfaces of all rolls for defects and for damage. This inspection shall be made without unrolling rolls unless defects or damages are seen or suspected. The CM will determine:

- Any rolls or parts of rolls with severe flaws that should be rejected and removed from the site;
- Any rolls or parts of rolls that contain flaws that can be repaired, and
- Rolls that are not properly labeled.

No unlabelled rolls shall be used on the Future Raises to Closure, and shall be returned to the Manufacturer and replaced by labeled rolls.

The CM will ensure that any damaged protective covering be repaired immediately, and that the integrity and legibility of the GCL roll labels are preserved.

8.2.3.2 Storage and Handling on Site

Upon receipt of delivery, the CM will store the GCL on site. Upon the Contractor's arrival at the site, the Contractor shall inspect the GCL, inventory the materials, and accept responsibility for storage and handling of material on site. At all times, the storage of GCL shall be subjected to following conditions:

- Protection from dirt, water, ultraviolet light exposure, vandalism, and other sources of damage.

The CM and CQA Field Manager shall monitor all GCL handling operations and storage procedures to ensure damage prevention to the GCL.

8.3 GCL INSTALLATION

8.3.1 Deployment

The CQA Field Manager shall continuously monitor and document the deployment of the GCL, time and location of material placement, note any damage to the materials, and record weather and subgrade conditions, to ensure conformance with the Specifications and Manufacturer guidelines.

For any damage to the materials, repairs shall be performed in accordance with the Specifications.

8.3.2 Seaming

The CQA Field Manager shall monitor the GCL seaming operations for proper overlap and technique. Upon completion, the CQA Field Manager shall confirm that all GCLs have been properly seamed in accordance with the Specifications.

8.3.3 Repairs

Any damage or flaws to the geotextile shall be repaired in accordance with the specifications. The CQA Field Manager shall locate, record, and monitor and document all repairs to ensure that they are completed in accordance with the Specifications.

9.0 CURTAIN WALL CQA

This section presents information regarding CQA for the curtain wall constructed of GSE CurtainWall panels. GSE CurtainWall panels were used to construct the seepage cutoff wall extending from the right abutment to the south/southeast. It has not yet been determined whether the GSE CurtainWall panels will be used for future raises to closure extensions of the curtain wall; it has been included in this CQA plan in the event that it is used for seepage cutoff in future raises.

The curtain wall will consist of vertical 100 mil thick HDPE geomembrane panels interlocked at each end. The curtain wall will function as the seepage barrier part of the wing wall extending to the southeast from the right abutment.

9.1 MANUFACTURE

9.1.1 Manufacturer Qualifications

The CM will select a curtain wall Manufacturer based on the following guidelines:

- At least 5 years of continuous experience in manufacturing geomembranes.
- Experience with geomembrane vertical barrier system with seepage-proof end interlocks.
- Manufactured at least 10 million square feet of geomembrane for at least 10 complete facilities.
- Satisfaction of all appropriate trade certifications.

9.1.2 HDPE Geomembrane Resin

The geomembrane resin shall be new HDPE of first quality, compounded and manufactured specifically for producing HDPE geomembrane. Recycled materials or seconds shall not be used. Resin types shall not be mixed during manufacturing. The Geomembrane Resin shall meet the requirements in Table 9-1.

Table 9-1 Geomembrane Resin Requirements

Physical Characteristic	Units	Test Method	Requirement
Density (min.)	g/cm^3	ASTM D1505	0.932
Melt Index	g/10 min	ASTM D1238 Condition 190/2.16	≤ 1.0

9.1.3 Curtain Wall Panel

The GSE CurtainWall shall consist of 100 mil thick HDPE geomembrane with HDPE interlocks attached to each vertical end of the panel.

The geomembrane shall be manufactured from HDPE resin, in 100-mil thick sheets and conform to the physical requirements in Table 9-2. The Manufacturer shall use materials produced in North America or as accepted by the CM. The Manufacturer shall identify the percentage of processing aids, antioxidants, carbon black, and other additives. The geomembrane shall also meet the following requirements:

- Additives other than carbon black or pigment shall not exceed a combined maximum total of 1% by weight;
- Carbon black content shall not exceed 3% by weight,
- Total combined processing aids, antioxidants, carbon and other additives shall not exceed 3.5% by weight of finished geomembrane,
- All additives for UV protection, thermal stability, color, or processing agents must not "bloom" to the surface over time or inhibit welding,
- Finished product shall be free of blemishes, holes, pin holes, bubbles, blisters, excessive gels, undispersed resins, and carbon black, contamination by foreign matter, and nicks or cuts on edges, and
- Manufactured curtain wall sheets or panels shall be rolled for shipment.

Table 9-2 Properties and Quality Control Tests for Geomembrane Panel of the Curtain Wall

Physical Characteristic	Qualifier	Units	Test Method	Min Frequency	Requirement
Sheet Thickness (nominal)	min. avg. value	mil	ASTM D5199	5 per roll	100
	lowest individual reading	-	-		90
Top and Bottom Surfaces	-	-	Smooth		Top and Bottom Surfaces
Specific Gravity (Sheet)	lowest individual reading	-	ASTM D1505	See Note 1	0.94
Tensile Properties (each direction)			ASTM D6693 (Type IV)	See Note 2	
Strength at Yield	min. avg. value	lb/in	ASTM D6693	See Note 2	210
Elongation at Yield	min. avg. value	%	ASTM D6693	See Note 2	12

Table 9-2 Properties and Quality Control Tests for Geomembrane Panel of the Curtain Wall					
Physical Characteristic	**Qualifier**	**Units**	**Test Method**	**Min Frequency**	**Requirement**
Strength at Break	min. avg. value	lb/in	ASTM D6693	See Note 2	380
Elongation at Break	min. avg. value	%	ASTM D6693	See Note 2	700
Tear Resistance	min. avg. value	lb	ASTM D1004	See Note 2	70
Puncture Resistance	min. avg. value	lb	ASTM D4833	See Note 2	180
Carbon Black Content	Range	%	ASTM D1603 or D4218	See Note 2	2-3
Carbon Black Dispersion	minimum 8 of 10	category	ASTM D5596	See Note 2	Category 1 or 2
Oxidation Induction Time	min. avg. value	minutes	ASTM D3895 or D5885	See Note 2	100
UV Resistance	min. avg. value	% retained after 1,600 hours	GM11 and ASTM D5885	See Note 2	50
Notes: 1. Either 1 per 100,000 sf of sheet, or 1 per resin batch, whichever results in the greatest number of tests 2. One test per resin batch on typical sheet and seam.					

9.1.4 Curtain Wall Interlock

The curtain wall interlock shall consist of reversible HDPE Profile with multiple sealant chambers welded to the ends of adjacent panels. The tensile strength of the weld between the geomembrane and the interlock, and the tensile strength of the interlock shall both be stronger than the geomembrane panel. The interlocking section shall meet the following requirements:

- Minimum tensile strength at yield for the interlocking section shall be at least 400 pounds per linear inch.
- Shear strength of the wedge weld of geomembrane to interlock, or tensile strength of the connection, shall be at least 400 pounds per linear inch.

9.1.5 Hydrophilic Seal

A hydrophilic gasket shall be used at the curtain wall interlock. The seal shall be a hydrophilic elastomer profile that is capable of swelling in water to at least five times its volume within 72 hours. The seal shall have a delayed coating to prevent swelling during installation. The delayed coating shall dissolve in contact with water and not prevent long-term swelling of the hydrophilic seal.

9.2 SOURCE QUALITY CONTROL

9.2.1 Quality Control Tests and Certification

The Manufacturer shall perform source QC testing on the geomembrane at the manufacturing plant as indicated in Section 9.1.3 and Table 9-2. The objective of this testing shall be to confirm the Manufacturer published material characteristics and demonstrate the materials compliance with the specifications.

Manufacturer's tests shall demonstrate that the requirements specified in Sections 9.1.4 and 9.1.5 for Curtain Wall Interlock and the Hydrophilic Seal, respectively. The minimum testing frequency for source quality control tests shall be one test per every 500 lineal feet of joints.

The Manufacturer shall reject rolls for which QC requirements are not met, and shall certify the quality of the rolls shipped to the site. The physical property test results of the geomembrane shall be submitted to the CM prior to shipping in accordance with the frequency outlined in Table 6-2.

The CQA Project Director, in collaboration with the Engineer, shall review the documentation to check if the geomembrane conforms to the specifications.

9.2.2 Labeling

Manufacturer shall mark or tag all GSE CurtainWall panels with the following information:

- Manufacturer's name;
- Product identification,
- Lot number,
- Roll number,
- Roll dimensions, and
- Special handling requirements.

9.2.3 Warranty

The Manufacturer shall provide a warranty for the curtain wall material. Manufacturer's warranty shall include a minimum 5-year warranty for the materials against deterioration.

9.3 PURCHASE, DELIVERY, STORAGE AND HANDLING

9.3.1 Purchase Responsibility

The Owner will purchase the curtain walls from the Manufacturer (or Supplier) in sufficient quantity and store it at the mine site. The Manufacturer shall submit product data, QA/QC documents, and samples to the CM for the CQA Project Director's approval prior to shipping. The CM shall ensure that the CQA Project Director has approved the products, prior to authorizing the shipment.

9.3.2 Manufacturer QA/QC Documents

The Manufacturer (or Supplier) shall submit the following product data to the CM and product samples directly to the Engineer, and wait for the CQA Project Director's approval prior to shipping the curtain wall to the site:

- Resin Data for geomembrane used in the curtain wall:
  - ♦ Production date or dates;
  - ♦ Certification that the resin meets the product requirements,
  - ♦ Certification that all resin is from the same Manufacturer,
  - ♦ Copy of quality control certificates issued by the Manufacturer,
  - ♦ Test reports from the Manufacturer, and
  - ♦ Statement that no reclaimed polymer is added to resin during manufacture of geomembrane.
- Geomembrane used in the curtain wall:
  - ♦ Production date or dates;
  - ♦ Laboratory test results and certification that geomembrane used meets product requirements,
  - ♦ Certification that all geomembrane used is furnished by one supplier,
  - ♦ Certification that all rolls are manufactured from one resin type obtained from one supplier,
  - ♦ Copy of quality control certificates indicating compliance with the Specifications,
  - ♦ Test reports from the Manufacturer,
  - ♦ Statement that no reclaimed polymer is added to the resin, and
  - ♦ List of percentages/total of processing aids, antioxidants, and other additives other than carbon black added to or in the resin.
- Hydrophilic Seal Data:
  - ♦ Production date or dates;

- ♦ Laboratory certification that the hydrophilic seal meets the product requirements,
- ♦ Certification that all hydrophilic seal is furnished by one manufacturer and one supplier,
- ♦ Copy of quality control certificates issued by the Manufacturer,
- ♦ Test reports from the manufacturer, and
- ♦ Certification that the hydrophilic seal is chemically resistance to the material that it will be in contact.

- Curtain Wall Panels:
  - ♦ Manufacturer recommended curtain wall delivery, storage and handling instructions;
  - ♦ Manufacturer recommended curtain wall installation instructions, and
  - ♦ Sample Curtain Wall warranties for review by the CM.
- Product Samples (directly to the CQA Field Manager):
  - ♦ Two sets of sample curtain wall to be supplied for the work, with each sample being at least 6 inches wide and 2 inches long with interlock and hydrophilic seal.
  - ♦ One panel sample curtain wall to be supplied for the work, with the panel being at least 4 feet wide and 4 feet long with interlock and hydrophilic seal.

The CQA Project Director and CQA Field Manager shall review all QC certificates and test results for conformance with the Specifications and to confirm that the materials on-site correspond to the materials for which the documentation was prepared. Any discrepancies and non-conformance shall be reported to the CM with details of the discrepancies and nature of the materials actually on-site. Any geotextile that fails to meet the physical property requirements shall be not deployed.

9.3.3 Product Delivery, Storage and Handling

Delivery, storage and handling of the curtain wall shall conform to ASTM D4873-02 and the Manufacturer's recommendations. The Manufacturer shall label each GSE CuratinWall panel with the geomembrane lot and panel number, the interlock and hydrophilic seal lot numbers, and other information necessary to identify it for inventory and QC and CQA purposes.

9.3.3.1 Delivery

The Manufacturer shall deliver materials to a location to be determined by the CM only after the CM has accepted the required submittals from the Supplier. The CM acceptance shall be based on the CQA Project Director's review findings.

Upon receipt of the panels on site, the CM shall inspect the surfaces of all panels for defects and for damage. This inspection shall be made without moving panels unless defects or damages are seen or suspected. The CM will determine:

- Panels, or portions thereof, which should be rejected and removed from the site because they have severe flaws;
- Panels which include repairable flaws,
- Panels that are not properly labeled,

Unlabeled panels will not be used and will be returned to the Manufacturer and replaced by labeled panels.

The CM will ensure that any damaged protective covering is repaired immediately, and that the integrity and legibility of curtain panel labels are preserved.

9.3.3.2 Storage and Handling on Site

Upon receipt of delivery, the CM will store the curtain wall panels on site. Upon the Contractor's arrival on site, the Contractor shall inspect the curtain panels, inventory the materials, and accept responsibility for the future storage and handling of the materials.

Storage of geomembrane panels shall be subjected at all times to the following conditions:

- Protect from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, vandalism, excessive heat and cold, or any other deleterious condition;
- Store on a prepared surface,
- Not stack more than three rolls or three flats high, and
- Store hydrophilic seals in their original boxes in a dry location.

Handling of the geomembrane panels shall at all times be subject to the following conditions:

- Use cloth chokers, spreader bars and roll bars to load, move and deploy panels;
- No dragging of rolls or flats on the ground, and
- No folding of panels.

The CM and CQA Field Manager shall monitor all curtain wall handling operations and storage procedures to prevent damage.

9.4 GSE CURTAINWALL PANEL INSTALLATION

9.4.1 Deployment

The CQA Field Manager will continuously monitor and document the deployment of the GSE CurtainWall panels, document the time and location of material placement, note any damage to the materials, and record weather and subgrade conditions, to ensure compliance with the Specifications and GSE CurtainWall panel layout plan.

For any damage to the materials, the Specifications define the repair method to be used. The Contractor will complete all repairs in accordance with the Specifications.

9.4.2 Welding

QA for field welding between the geomembrane and GSE CurtainWall panels shall conform to the requirements of Geomembrane CQA, Section 6.0.

Seaming of the GSE CurtainWall panels shall be completed using the interlock and hydrophilic seal. Seaming operations must be fully monitored and documented by the CQA Field Manager to ensure conformance with the Specifications. Documentation shall include (at a minimum):

- Seam number based on a numbering system agreed to by the CQA Field Manager and Contractor;
- Elevation of ground surface at the location of each GSE CurtainWall panel seam,
- Elevation of trench bottom at the location of each GSE CurtainWall panel seam,
- Depth to trench bottom at the location of each GSE CurtainWall panel seam, and
- Length of hydrophilic seal installed and the depth at each GSE CurtainWall joint.

9.4.3 Nondestructive Seam Testing

The CQA Field Manager shall monitor and document each seam as in Section 9.4.2 to verify compliance with the Specifications and record seam defects and repairs. Documentation will include at least the date, time, location, and pass or fail for each test.

The CQA Field Manager shall review the documentation of the seam to visibly ensure full joint connection at the interlock seams. The portion of the trench containing the end panel with the interlock seam in question shall not be backfilled until the seam is approved by CQA Field Manager.

9.4.4 Repairs

Any part of a GSE CurtainWall panel with a flaw, that fails a nondestructive or destructive test, where destructive tests were cut, or where nondestructive tests left cuts or holes, must be repaired in accordance with the Specifications. The CQA Field Manager shall locate, record, and monitor and document all repairs to ensure that they are completed in accordance with the Specifications.

10.0 GROUT CQA

This section presents information regarding QA for grout mixing and installation. The grout is used to backfill the cutoff wall on both sides of the geomembrane for the purposes of anchoring the geomembrane and sealing the bottom of the geomembrane to competent bedrock. The design intent is to provide additional control of seepage from the tailings impoundment through the dam foundations.

10.1 GROUT MIX

The Contractor will mix grout in accordance with the Specifications using the batch plant provided by the Owner. The CQA Monitors shall monitor and document the mix operations as follows:

- Monitor and record the mix proportioning and mixing methods.
- Obtain grout samples for laboratory testing.
- Document the time and location of material mixing.
- Record weather conditions.

The CQA Field Manager shall review field reports and review the Contractor's QC test results to ensure conformance of the Specifications.

10.2 GROUT INSTALLATION

The CQA Field Manager or his or her designator will provide full time monitoring of the grout installation to ensure conformance with the Drawings and Specifications, and to ensure that a seal is provided at the bottom of the cutoff wall in accordance with the design intent to reduce seepage as much as practically possible.

Before the grout operations, the CQA Field Manager or his or her designator shall have completed geotechnical logging of all excavations as described in Section 3.2. The CQA Field Manager shall ensure that the sides and bottom of the excavation are clean and free of the following materials:

- Ice lenses;
- Loose soil debris,
- Fragments of fractured and blasted rock, and
- Organic and other deleterious materials.

The CQA Monitors shall continuously monitor and document the installation of the grout, document the time and location of material placement, and perform the following to ensure:

- Monitor areas where grout is to be installed;

- Verify placement conditions including weather and geomembrane installation,
- Document foundation and subgrade conditions, and
- Monitor curing and protection.

10.3 GROUT TESTING

Grout testing will consist of compression strength tests on grout cylinders measuring 4 inches in diameter and 8 inches long. The testing shall be conducted at frequencies of:

- 7-day strength and
- 28-day strength.

For each day of production, cast one set of three cylinders of grout for further compressive strength testing at the required frequencies. Measure the grout density at a frequency not less than once every 2 hours during the grouting operations. The cylinders will have a diameter of 4-inches and height of 8-inches or have a diameter to height ratio of 1:2.

11.0 CONTROLLED DENSITY FILL (CDF) CQA

This section presents information regarding QA for CDF mixing and installation. The CDF is used to backfill the trench on both sides of the curtain wall for purposes of anchoring the curtain wall and sealing the bottom of the curtain wall to competent bedrock. The design intent is to control seepage from the tailings impoundment through the right abutment and mill area.

11.1 CDF MIX

The Contractor will mix the CDF in accordance with the Specifications using the batch plant provided by the Owner. The CQA Monitors shall monitor and document the mix operations as follows:

- Monitor and record mix proportioning and mixing methods;
- Obtain samples for laboratory testing,
- Document the time and location of material mixing, and
- Record weather conditions.

The CQA Field Manager shall review field reports and review Contractor's QC test results to ensure conformance of the CDF quality with the Specifications.

11.2 CDF INSTALLATION

The CQA Field Manager or his or her designator will provide full time monitoring of the CDF installation to ensure conformance with the Drawings and Specifications, and to ensure that a seal is provided at the bottom of the curtain wall in accordance with the design intent to reduce seepage as much as practically possible.

Before the CDF placement, the CQA Field Manager or his or her designator will have completed geotechnical logging of all excavations as described in Section 3.2. The CQA Field Manager will ensure that the sides and bottom of the excavation are clean and free of the following materials:

- Ice lenses;
- Loose soil debris,
- Fragments of fractured and blasted rock, and
- Organic and other deleterious materials.

The CQA Monitors will continuously monitor and document the installation of the CDF, document the time and location of material placement, and perform the following to ensure:

- Monitor areas where CDF is to be installed;

- Verify placement conditions including weather and geomembrane installation,
- Document foundation and subgrade conditions, and
- Monitor curing and protection.

11.3 CDF TESTING

CDF testing will consist of compression strength and permeability tests on 4-inch by 8-inch cylinders. Strength testing and permeability testing will be conducted a minimum of once a day for each production day. Three cylinders will be obtained for each of the following tests.

- 7-day strength,
- 28-day strength, and
- Permeability.

A fourth cylinder will be obtained and will remain on site as a contingency cylinder in the event that a cylinder breaks en route to the laboratory or it is needed for additional testing.

12.0 INSTRUMENTATION CQA

This section presents information regarding CQA for the new instrumentation. Piezometers and thermistors that were installed for the future raises to closure include those listed in Table 12-1 below.

No additional monitoring points have been added for the future raises to closure. However, this CQA plan covers installation of instrumentation in the event that existing instrumentation requires replacement or additional monitoring points are identified.

Table 12-1 Future Raises to Closure Instrumentation

Boring ID*	Thermistor ID	Piezometer ID
SS-05-05	T-05-62	P-05-62
SS-07-05	T-05-63	P-05-63
SS-10-05	T-05-64	None
SS-11-05	T-05-65	P-05-65
SS-12-05	None	None
SS-15-05	T-05-67	P-05-67
SS-16-05	T-05-68	P-05-68
SS-17-05	None	P-05-69
West Riser	None	P-06-74**

12.1 MANUFACTURE

12.1.1 Manufacturer Qualifications

The CM will obtain all piezometers and thermistors of the vibrating wire type from Geokon Inc. or a manufacturer of similar experience in producing quality vibrating wire piezometers.

12.2 SOURCE QUALITY CONTROL

12.2.1 Quality Control Tests and Certifications

The Manufacturer will provide a quality control certification for each instrument verifying that it has been laboratory calibrated and meets the measurement criteria set forth in the Specifications. The Manufacturer's certification will at a minimum indicate the following for each instrument:

- Instrument model number, serial number, and operational pressure range;
- Conditions during calibration (date, temperature and pressure),
- Calibration standard control numbers,
- Applied pressure, gage readings, calculated pressures (before and after calibration) and percent error (before and after calibration),

- Gage factors,
- Pressure calculation equations used for instrument calibration,
- Factory zero readings including resistance, temperature, pressure and date, and
- Technician's name.

12.2.2 Labeling

Each instrument will be labeled with a unique identification number. Manufacturer's quality control documentation will be provided for each instrument and readily associated via the unique identification number.

12.2.3 Warranty

The Owner shall obtain a material warranty from the Manufacturer for a time period of at least 5 years. The warranty shall include coverage for the instrumentation materials and workmanship specifically provided under this project shall be free from any significant defects. The warranty shall also provide for the total and complete repair or replacement of the defect or defective materials upon written notification and demonstration by the Owner of the specific non-conformance.

Purchase, storage and installation of piezometers and thermistors is the responsibility of the Owner unless otherwise specified.

12.3 INSTRUMENTATION

12.3.1 Vibrating Wire Piezometers

The CM will approve the purchase of an acceptable vibrating wire piezometer from Geokon. The Owner will purchase the vibrating wire piezometers and ensure that they meet the minimum specification provided in Table 12-2.

Table 12-2 Vibrating Wire Piezometer Specifications

Standard Ranges	350, 700 kPa; 1, 2 and 3 Mpa
Over Range	2 × rated pressure
Resolution	0.025% F.S. (minimum)
Accuracy	±0.1% F.S.
Linearity[1]	<0.5% F.S. (±0.1% F.S. optional)
Temperature Range[2]	−20°C to +80°C

12.3.2 Vibrating Wire Readout Box

The CM will ensure that a compatible, properly functioning readout box is available for initial installation and ongoing monitoring of the instrumented locations. The readout box will function under a simple, single-button, operation under which each stored reading is identified by a reference number, ranging from 1 to 256, plus time, date and temperature. The readout box will allow for all readings to be transmitted to a host computer and imported into spreadsheet and database applications. The CM will

assure that the readout box adheres to the minimum specifications provided in Table 12-3.

Table 12-3 Instrumentation Readout Box Specifications	
Excitation Range	400 Hz to 6000 Hz, 5 Volt Square Wave
Resolution	0.1 µs
Timebase Accuracy	±50 ppm
Temperature Range	−10°C to +50°C

12.3.3 Thermistor Strings

The CM will approve the purchase of an acceptable thermistor strings from the manufacturer, Geokon. The Owner will purchase the thermistor strings and ensure that they meet the minimum specification provided in Table 12-4.

Table 12-4 Instrumentation Readout Box Specifications	
Standard Range	-50°C to 150°C
Resolution	0.1 ^{o}C
Accuracy	+/- 0.5^{o}C
Temperature Range	−20°C to +80°C

12.4 EQUIPMENT INSTALLATION

Instrumentation will be installed in accordance with the design plans and specifications. The CM shall ensure the following is completed to assure quality and conformance with the design:

- The Manufacturer's instruction manual is followed regarding:
  - ♦ Installation;
  - ♦ Field initiation, and
  - ♦ Calibration.
- Instrumentation is backfilled in place with a sand-bentonite mixture and graded sand to elevations and dimensions indicated on the drawings.
- For the piezometer connection to the instrumentation terminal box, ensure that:
  - ♦ Piezometer wires are threaded through to terminal box locations, and
  - ♦ Wiring to terminal panels and label wires, and terminal connections are connected.

12.4.1 Testing and acceptance

At the time of delivery from the Manufacturer, all instruments shall be inspected by the CM. The CM shall ensure that the instrumentation is stored in a clean, lighted, temperature-controlled, dry area with lock security.

The CM shall monitor and record results from all tests performed during installation. All instruments that fail required tests shall be rejected and replaced. All replacement instrumentation shall be tested before installation.