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Crandon Mining Company

7 N. BROWN ST., 3RD FLOOR
RHINELANDER, WI 54501-3161

January 30, 1997

Mr. Bill Tans
Wisconsin Department of Natural Resources
Bureau of Integrated Science Services
101 South Webster Street
P.O. Box 7921
Madison, WI 53707-7921

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Mr. David Ballman
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101

Dear Mr. Tans and Mr. Ballman:

Re: Crandon Project - Tailings Management Area Feasibility Report/Plan of Operation

Crandon Mining Company (CMC) is pleased to provide you with the report titled *Addendum No. 3 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation*.

Addendum No. 3 has been prepared on behalf of CMC by Foth & Van Dyke and Associates, Inc. As noted on the attached distribution list, CMC has distributed the information to appropriate state and federal agencies, to local officials, and to various interested parties. It is our understanding that the Wisconsin Department of Natural Resources (WDNR) and the U.S. Army Corps of Engineers (USCOE) will be responsible for distribution of the document to their appropriate staff members.

The primary purpose of Addendum No. 3 is to describe those details of the tailings management area (TMA) that have been modified as a result of providing a greater setback from Bur Oak Swamp. The issue of the setback distance from Bur Oak Swamp was the topic of much discussion between your two agencies and CMC at a number of meetings focusing on the TMA. In response to the setback issues that were raised, CMC developed a revised layout for the northwest portion of the TMA, increasing the setback distance. The modified footprint was staked out and observed in the field by personnel of your two departments and was addressed in discussions with your two agencies. CMC feels the facility modifications presented in this addendum resolve the Bur Oak Swamp setback issue.

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TMA Addendum No. 3 also consolidates into one document the modifications CMC has made to the facilities liner, cover, and leachate collection system design in response to questions and comments raised as part of your agencies review of the original May 1995 submittal. This information was previously documented in Addenda Nos. 1 and 2 previously submitted to your two agencies and others on February 21, 1996, and March 15, 1996. In addition, Addendum No. 3 provides additional detail on waste quantities and the project's earthwork balance, and addresses several outstanding issues raised at various TMA meetings that were not addressed through past submittals. Also, through this addendum CMC has addressed or is responding to the following items from the Wisconsin Department of Natural Resources (WDNR) July 31, 1995 letter pertaining to its preliminary review of the Crandon Project *Environmental Impact Report* (EIR). That letter included several staff comments pertaining to the project's *Tailings Management Area Feasibility Report/Plan of Operation* as it relates to the EIR.

Comment	Location Comment Addressed
42	See Response Below
126	See Response Below
128	See Response Below
129	See Response Below
130	See Response Below

Comment 42: What happens to the topsoil stockpile when additional TMA cells are developed?

Response 42: *Topsoil stripped during construction of TMA 1 not used for reclamation of berms, etc., will be stored in the topsoil stockpile to be located in the soil processing and construction staging area shown on Figure 6.12-1 of Addendum No. 3 to the May 1995 Tailings Management Area Feasibility Report/Plan of Operation. Topsoil removed from the TMA 2 area not used for berm reclamation will be placed on the same stockpile. Topsoil to reclaim TMA 1 will be taken from this stockpile.*

Topsoil from TMA 3 will be placed in a new topsoil stockpile to be located in the borrow area located north of TMA 2 and 4 as shown on Figure 6.12-3 of Addendum No. 3. Topsoil for the reclamation of TMA 2 will come from the topsoil stockpile located in the soil processing and construction staging area shown on Figure 6.12-1 and, if necessary, the topsoil stockpile located in the stockpile/borrow area north of TMA 2 and 4. Topsoil from the TMA 4 area not used for berm reclamation will be placed in the topsoil stockpile located to the north of TMA 2 and 4. The topsoil in this stockpile will be used to reclaim TMA 4 and the north borrow area during final TMA reclamation.

Comment 126: Figure 2-6, the ore body is shown as extending westwardly nearly to the west exhaust shaft, which would be located in section 30 of Lincoln Township. However, in figure 2-2, the ore body is shown as extending more than 2,000 feet into section 25 west of the west exhaust shaft in Nashville Township. Which is correct? Is the entire ore body planned for mining? Would ore be removed from lands beneath both townships?

Response 126: *Figure 2-2 is the correct representation of the extent of the ore body. The location of the west exhaust shaft is incorrectly shown on Figure 2-6. A revised Figure 2-6 will be included in a future update to the general project description (Section 2) of the May 1995 TMA report. The current mining plan involves ore removal from those portions of the ore body located in both the Towns of Lincoln and Nashville.*

Comments 128, 129, and 130 of the July 31, 1995 WDNR letter relate to CMC's initial proposal to use native clay in the TMA liner and cover system. Since WDNR's letter was written, CMC has modified its proposal to incorporate the use of a combined geosynthetic clay liner (GCL) and low permeability till layer in lieu of native clay for the TMA liner and final cover system. As a result of this modification, comments 128, 129, and 130 are no longer relevant.

If you or your staff have any questions regarding Addendum No. 3, please contact me at (715) 365-1450.

Sincerely,

D. E. Moe

Don Moe
Technical/Permitting Manager
Crandon Mining Company

DM:mld2

MLD2\93C049\GBAPP\42980\4000

Crandon Mining Company

7 N. BROWN ST., 3RD FLOOR
RHINELANDER, WI 54501-3161

April 29, 1997

Mr. Bill Tans
Wisconsin Department of Natural Resources
Bureau of Environmental Analysis and Review
P.O. Box 7921
Madison, WI 53707

Mr. David Ballman
Ecologist
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101

Dear Mr. Tans and Mr. Ballman:

Re: Crandon Project - *Addendum No. 3 to the Tailings Management Area Feasibility Report/Plan of Operation*

Crandon Mining Company (CMC) is pleased to submit the enclosed update to *Addendum No. 3 to the Tailings Management Area Feasibility Report/Plan of Operation* (Addendum No. 3).

The update has been prepared on behalf of CMC by Foth & Van Dyke and Associates, Inc. CMC has distributed the information to appropriate state and federal agencies, to local officials, and to various interested parties according to the current distribution list for Addendum No. 3. It is our understanding that the Wisconsin Department of Natural Resources (WDNR) and the U.S. Army Corps of Engineers (USCOE) will be responsible for distribution of the document to their appropriate staff members.

The pages contained in this update need to be inserted into Addendum No. 3 according to Items 1 through 3 on the attached reference list. This list will serve as a log and reference identifying changes made to Addendum No. 3 by CMC throughout the permitting process. If additional revisions are made, they will be added to the attached list in sequential order and the list will be forwarded with the changes.

If you or your staff have any questions regarding Addendum No. 3, please contact me at (715) 365-1450.

Sincerely,

D. E. Moe

Don Moe
Technical/Permitting Manager
Crandon Mining Company

DM:mld2
cc: Addendum No. 3 Distribution List

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Crandon Project Addendum No. 3 to the Tailings Management Area Feasibility Report/Plan of Operation
Log of Revisions and Additional Information

Entry Number	Date of Revision	Page(s)	Document Section Number	Description
1	4/29/97	2	Log of Updates	Insert after cover letter
2	4/29/97	—	Drawing 1	Added reference to updated Drawing 22
3	4/29/97	—	Drawing 2	Revised limits of disturbance
4	4/29/97	—	Drawing 3	Added limits of disturbance
5	4/29/97	—	Drawing 22	Added drawing

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Addendum No. 3 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation

93C049

Prepared for
Crandon Mining Company
7 North Brown Street, 3rd Floor
Rhinelander, Wisconsin 54501-3161

Prepared by
Foth & Van Dyke and Associates Inc.

January 1997

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Addendum No. 3 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation

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Appendices

1 Introduction

Crandon Mining Company (CMC) prepared a February 21, 1996 document titled "Addendum No. 1 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation" (Foth & Van Dyke, 1996a) which presented responses to a portion of the comments raised in the Wisconsin Department of Natural Resources (WDNR) January 4, 1996 TMA Completeness Determination Letter (WDNR, 1996). The addendum was provided to the WDNR at a February 22, 1996 TMA meeting with the WDNR, the U.S. Army Corps of Engineers (USCOE), and others. At this meeting, WDNR requested clarification of and additional information for some of the responses contained in Addendum No. 1.

The document titled "Addendum No. 2 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation" (Addendum No. 2) (Foth & Van Dyke, 1996b), submitted to the WDNR on April 4, 1996, was prepared both to clarify responses to additional information requested at the February 22, 1996 TMA meeting and to respond to the remaining comments contained within the WDNR's January 4, 1996 TMA Completeness Determination Letter (WDNR, 1996).

This document titled "Addendum No. 3 to the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation" (Addendum No. 3), has been prepared to document modifications to the facilities footprint in the vicinity of the Bur Oak Swamp, to consolidate modifications made to the facilities proposed liner, final cover and leachate management system design, and to address the remaining issues raised at the May 28, 1996 TMA meeting.

2 Purpose and Scope

The purpose of this document is to provide information required by NR 182, Wis. Admin. Code, and specified by the WDNR to make the May 1995 Crandon Project Tailings Management Area Feasibility Report/Plan of Operation (Feasibility Report) (Foth & Van Dyke, 1995a) complete. In addition, this document contains responses to requests from the WDNR and other agencies for specific additional information required for the WDNR to make a completeness and feasibility determination. In general, Addendum No. 3 contains the following:

- Redesign of the TMA footprint to provide additional setback from Bur Oak Swamp.
- Update of the TMA liner design, including changes to the soil component of the composite liner.
- Update of the leachate collection system (LCS) design, including changes to the LCS on the TMA cell base and the addition of a leachate drainage layer to the initial stage of interior cell sideslopes.
- Update of the HELP model to reflect the modifications made to the footprint, composite liner and LCS.
- Redesign of the surface water management facilities.
- Modifications to the site development and site phasing.
- Reassessment of the potential impacts to local drainage basins given the design modifications referenced above.
- Recalculation of the earthwork balance for the facility.
- Responses to issues raised by the WDNR in their January 4, 1996 TMA Completeness Determination letter (WDNR, 1996) and subsequent review meetings with the WDNR and other agencies.

3 TMA Redesign for Setback No. 2 Configuration

3.1 Background Information

The TMA Feasibility Report (Foth & Van Dyke, 1995a) provided, in Section 10.1, a detailed siting analysis used in evaluating alternative TMA sites. The siting criteria were used to perform a detailed comparative analysis of the advantages of Area 41 over Area 40. As a result of the agencies review of the Feasibility Report, additional information was requested to provide a broader discussion on the TMA siting evaluation process performed by CMC to select the TMA footprint with the least environmental impact. To address this specific comment, an update of the Feasibility Report was submitted to WDNR on February 19, 1996. The update included a new section, titled Section 10.2, TMA Location Within Area 41. Section 10.2 presented the process used to select the TMA footprint within Area 41 and a summary of a comparative impact assessment for alternative TMA footprints within Area 41.

On April 9, 1996, WDNR presented comments on CMC's siting evaluation and issued a letter on the proposed footprint in relationship to wetland F15 - Bur Oak Swamp. The WDNR concluded that the alternatives presented in CMC's siting analysis demonstrated that some additional setback from wetland F15 was necessary to protect this wetland. On April 22, 1996, the WDNR, USCOE, CMC, other agencies, and interested parties discussed the TMA footprint location and the potential impacts to wetland F15. As part of the discussion CMC indicated that it was likely setback alternatives could be developed which would achieve the WDNR's objective of minimizing disturbance to the wetland F15 watershed without requiring significant footprint modifications. As a follow-up to this TMA meeting, CMC identified an alternative setback proposal which CMC believed would address WDNR concerns regarding the proximity of the TMA limits of disturbance to wetland F15.

On May 14, 1996, the boundary of wetland F15 was delineated in the field as were the limits of disturbance for two TMA footprint alternatives: the F15 Setback Alternative No. 1, and the F15 Setback Alternative No. 2. The delineation procedures for wetlands involved an examination of the soils, vegetation, and the hydrology in accordance with the *1987 Corps of Engineers Wetland Delineation Manual* (USCOE, 1987). The field delineation provided a visual demonstration of the potential buffer areas between wetland F15 and the two TMA alternative footprints (see Figure 3.1-1).

The F15 Setback Alternative No. 1 was originally considered as part of the updated footprint alternatives analysis presented in Section 10.2 of the TMA Feasibility Report. The F15 Setback Alternative No. 2 is the alternative CMC developed following the April 9, 1996 TMA meeting to address WDNR concerns regarding the proximity of the TMA limits of disturbance to wetland F15. This alternative provides a minimum setback of approximately 200 feet from the F15 wetland.

On May 20, 1996, a field visit involving representatives from WDNR, USCOE, and other interested parties was conducted to review the relationship of wetland F15 to the limits of disturbance for the two TMA footprint alternatives. On May 21, 1996, a TMA meeting was held and included a discussion of the TMA footprint location. At that meeting, Mr. Christopher Carlson of the WDNR indicated that the proposed TMA footprint, described as the TMA North Alternative in Section 10.2 submitted to WDNR in February 1996, would need to be modified due to its proximity to wetland F15. Mr. Carlson went on to say that Setback Alternative No. 2

has the potential to be permittable. He further stated that Setback Alternative No. 2 appears to provide adequate separation for surface water management and impacts mitigation as related to wetland F15. Based on the discussion of the footprint issues during the May 21, 1996, TMA meeting, WDNR stated that additional information should be provided, in the form of an addendum to the Feasibility Report, presenting details of the revised TMA footprint for the purposes of feasibility determination.

3.2 Description of TMA Setback Alternative No. 2

3.2.1 Design Objectives

The TMA Setback Alternative No. 2 was proposed with the objective of providing additional setback from wetland F15, while minimizing additional disturbance due to footprint modifications elsewhere. To achieve this objective, TMA Setback Alternative No. 2 (also referred to as revised footprint) was developed by lowering the base grades in TMA cell 1, raising base grades in TMA cells 2, 3, and 4, and raising TMA berms by several feet. Also, additional separation to wetland F15 was accomplished by shifting the proposed runoff basins further from wetland F15 to maintain a minimum disturbance setback of 200 feet. The revised footprint retained the benefits of the initial design which avoided or minimized direct wetland disturbance and minimized disturbance within the Deep Hole and Duck Lake watersheds.

3.2.2 Revised Footprint

The revised footprint for the TMA and its surrounding facilities is shown on Figure 3.2-1. This footprint includes the TMA, TMA borrow area, reclaim pond, perimeter road, exterior drainage system, and runoff basins. The revised footprint avoids or minimizes direct and indirect disturbance to wetland F15 and other wetlands due to the construction of the TMA. The revised footprint results in a total of 344.8 acres of disturbance, including 22.8 acres of direct wetland disturbance. This compares to a total area of disturbance of 359 acres, including 21.9 acres of direct wetland disturbance in the original design. The revised footprint reduces total disturbance associated with the TMA and its surrounding facilities by approximately 14 acres. The increase in total wetland disturbance results primarily from raising the TMA berm and the incorporation of a perimeter access road in the design. The disturbance for the wetland F15 watershed would be approximately 63.1 acres, which is approximately 4.5 percent less than in the original proposal. The disturbance within the lake watersheds would be approximately 56.3 acres for Deep Hole Lake, 130.0 acres for Duck Lake, and 89.9 acres for Skunk Lake. Approximately 68.5 acres of the Hemlock Creek watershed would also be disturbed. When compared to the original footprint contained in the Feasibility Report (Foth & Van Dyke, 1995a) (see Figure 3.2-2 for originally proposed footprint), the disturbance within lake watersheds and the wetland F15 watershed are shown to be very similar to the original TMA layout with the greatest reduction in disturbance within the F15 watershed and a slight increase in the Hemlock Creek watershed due to the increased size of the borrow area.

The revised footprint involves a slight modification to the overall height of the TMA. The height of the TMA berms would increase by approximately 3.5 feet to maintain a cut/fill material balance. This change was accomplished while still minimizing or avoiding direct impact to wetlands. In addition, the revised footprint design minimized the quantity of materials required to be stored in the TMA staging/stockpile/borrow site north of TMA cells 2 and 4. As shown on Figure 3.2-1, the proposed borrow site configuration has been modified to address stockpiling and borrow needs of TMA cells 3 and 4. The adjustment of the borrow site final grades, as shown on Drawing 18, minimizes the hydraulic impact to wetland F15 by matching the original F15 drainage divide and discharging a portion of the borrow area's surface water through a new runoff basin (Basin 13) towards Hemlock Creek.

3.3 Direct Wetland Disturbance

As part of the revised TMA footprint modifications, the wetlands directly disturbed by the proposed construction will be slightly different from what has been previously described. Table 3.3-1 presents the updated direct wetland disturbance for the revised footprint. The updated wetland impacts take into consideration the increase in the TMA cell berm height and the addition of a perimeter road.

As described in Section 8 of this report, special construction techniques will be employed to protect wetlands located along the perimeter of the TMA footprint. Silt fences and cutoff wall installation (where required) will be installed to isolate wetlands from surface water and temporary dewatering disturbance and will hydraulically separate the wetlands from the TMA. The TMA perimeter road will then be installed with fill placed to also act as a barrier to these wetlands.

3.4 Comparison of Revised Footprint Impacts

Table 3.4-1 presents a summary of a comparison of the impacts of the revised footprint to the originally proposed TMA North footprint (Foth & Van Dyke, 1995a). The summary demonstrates that the revised footprint results in the least overall impact when compared to the original proposal.

Table 3.3-1

Acreages of Wetland Types Disturbed During TMA Construction

Wetland No.	Revised Footprint[1] Wetland Types: Emergent Marsh	Shrub Swamp	Deciduous Swamp	Coniferous Swamp	Bog	Total Area Impacted
F30[2]			0.02			0.02
F31			2.16	4.29		6.45
F32			0.68			0.68
F33			1.82			1.82
F66		2.54		4.64		7.18
F81	0.33					0.33
13			0.07			0.07
14			0.06			0.06
15			0.14			0.14
16			0.10			0.10
17			0.54			0.54
M3			1.82	1.60		3.42
M4			1.06			1.06
Unnamed (3)			0.89			0.89
Total	0.33	2.54	9.36	10.53		22.76

[1]Includes TMA, TMA borrow area, reclaim pond, perimeter road, exterior drainage system, and runoff basins.
[2]Wetland F30 is classified as a "streamside wetland" in Table 5-2 of the project's *Section 404 Permit Application Addendum 1* (Foth & Van Dyke, 1995d). The impacted portion of this wetland is dominated by deciduous swamp.

Prepared by: GAM
Checked by: PAE

Table 3.4-1

Summary of Revised Footprint Impacts

Parameter	Assessment of Revised Footprint Compared to TMA North Alternative
Historical, Archaeological, Cultural Resources	• Increased buffer to Bur Oak Swamp to a minimum of 200 feet.
Transportation and Utilities	• No significant difference.[1]
Climatology - Air Quality	• No significant difference.[1]
Geology - Soils	• No significant difference.
	• Requires slightly greater external borrow area (approximately 2 acres) and borrow materials (approximately 283,000 cubic yards).
Groundwater	• No change in average base separation to groundwater.
Surface Water	• 14 acres less total disturbance.
	• 4.5 percent less watershed disturbance in F15 wetland watershed.
	• Achievement of a better match of pre-development to post-reclamation runoff into wetland F15.
Biology	• Slightly greater disturbance of F66 wetland.
	• No difference in the number of state special concern species.
	• No difference in the number of state endangered species.
Wetlands	• An approximate 1-acre total increase in disturbance for wetlands M3, M4, and F66, combined.
Noise	• No significant difference since TMA berm height increase is only approximately 3.5 feet.[1]
Land Use - Zoning	• No significant difference.
Aesthetics	• No significant difference since the TMA berm height increase was only 3.5 feet.
Socioeconomics	• No significant difference.
Native American Communities	• No significant difference in the proximity to Native American communities.

[1]Setback Alternative No. 2 also includes a GCL/P40 till layer in lieu of off-site clay for the soil component of the liner and cover hydraulic barrier, therefore Setback Alternative No. 2 results in a significant reduction in off-site borrow material trucked to TMA site.

Prepared by: PAE
Checked by: JWS

4 Update of TMA Liner Design

4.1 Background

The January 4, 1996 completeness letter (WDNR, 1996) requested that an alternative to native clay for the soil component be evaluated for the TMA composite liner system. Based on a review of alternatives, CMC proposed using a geosynthetic clay liner (GCL) overlying a screened 12-inch thick, compacted till layer (P40 till). Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) provided general information concerning available GCLs, a performance assessment of GCLs, a discussion of the advantages of GCLs, an evaluation of the hydraulic equivalency of GCLs compared to compacted clay liners, and a comparison of GCLs to native clay liners. In addition, it was shown that a sufficient quantity of glacial till soils are available on-site to produce the required quantity of P40 till soil for a 12-inch thick layer. The discussion in Addendum No. 1 showed that a GCL in combination with a 12-inch compacted P40 till layer has many construction and environmental advantages over the originally proposed native clay. This section of Addendum No. 3 provides an update to the TMA composite liner design incorporating a GCL overlying a P40 till layer.

4.2 General Information Concerning Redesigned Composite Liner

Following is a listing of the updated TMA composite liner system components, which includes a reference to the section of the Feasibility Report (Foth & Van Dyke, 1995a) or Addendum No. 1 (Foth & Van Dyke, 1996a) where the design rationale and function are explained. This design proposes three types of liner systems, one for the base of the TMA cell and two for the interior slopes of these cells. The three types of liner systems have components from top to bottom as follows:

- Base liner system configuration (with leachate collection system) (refer to Drawing 29, Detail 1/29):
    - 18 inches riprap (refer to Section 6.3.5.1 of the Feasibility Report);
    - 6 inches of fines from the till processing (filter layer) (refer to Section 6.4.3.1 of the Feasibility Report);
    - 12 inches glacial till (filter layer) (refer to Section 6.4.3.1 of the Feasibility Report);
    - geotextile (filter) (refer to Section 6.4.3.1 of the Feasibility Report);
    - 24-inch granular soil drainage layer (refer to Section 6.4.3 of the Feasibility Report);
    - geotextile (cushioning) (refer to Section 6.3.5 of the Feasibility Report);
    - 60 mil high density polyethylene (HDPE) geomembrane (refer to Addendum No. 1 and No. 2 to the Feasibility Report);
    - GCL (refer to Addendum No. 1 to the Feasibility Report);

- 12 inches of P40 till soils (refer to Addendum No. 1 to the Feasibility Report); and

- prepared subgrade consisting of compacted till or native till soils (refer to Section 6.3.2 of the Feasibility Report).

• Interior slope configuration of composite liner with leachate collection system (refer to Drawing 29, Detail 3/29) for the initial stage of each cell only:

  - 18 inches of glacial till (protective layer);

  - geocomposite (a geonet with non-woven geotextiles heat bonded to its top and bottom surfaces) (refer to Section 5 of Addendum No. 3 to the Feasibility Report);

  - textured (both sides) 60 mil HDPE geomembrane;

  - GCL;

  - 12 inches of P40 till soils; and

  - prepared subgrade consisting of compacted till or native till soils.

Since it is anticipated that the coarser fraction of tailings will settle out closer to the 18-inch till layer on the sidewalls, calculations show that a 6-inch till fines layer is not required as a filter on the TMA cell sidewalls.

Interior slope composite liner without leachate drainage layer (refer to Drawing 29, Detail 4/29) for the second stage of each cell:

- 18 inches of riprap (only on the interior slopes where required during final tailings deposition);

- 18 inches of glacial till;

- geotextile (cushioning);

- textured (both sides) 60 mil HDPE geomembrane;

- GCL; and

- 12 inches P40 till soils.

The sections which follow describe the components of the composite liner which have been modified from the original Feasibility Report (Foth & Van Dyke, 1995a).

4.3 Alternative Soil Components for Composite Liner - P40 Till Layer

4.3.1 General

The P40 till layer is the initial portion of the soil liner component which will be placed and compacted directly on the prepared subgrade described in Section 6.3.2 of the Feasibility Report (Foth & Van Dyke 1995a). The following provides the specification for the P40 till layer.

4.3.2 P40 Till Layer

In Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a), CMC provided an analysis of the availability of on-site Late Wisconsinan Till (LWT) for manufacture of P40 till soils for the composite liner. This analysis showed that sufficient LWT is available to manufacture both the drainage layers and P40 till layers.

The specifications for the P40 till layer are as follows:

- Gradation:

P40 sieve (percent finer by weight)	-	100 percent
P200 sieve (percent finer by weight)	-	≥18 percent
C_u (coefficient of uniformity)	-	>6

- Unified Soil Classification: silty fine sand, clayey sand or sandy silt.

- Hydraulic conductivities estimated based on D_{10} size to range from $2x10^{-5}$ cm/sec (centimeters per second) to less than $1x10^{-6}$ cm/sec.

- Specifications for Construction:

  - place in two 6-inch lifts to a minimum thickness of 12 inches;

  - compact to 95 percent of the maximum dry density as determined by the Standard Proctor Test (ASTM D 698);

  - compact at a moisture content at or above the optimum moisture content determined by ASTM D 1557;

  - compact and roll smooth to achieve "intimate contact" with the GCL.

4.3.3 P40 Till Layer - Quality Assurance and Quality Control (QA/QC)

Appendix O of the Feasibility Report (Foth & Van Dyke, 1995a) contains the TMA Construction Quality Assurance Plan (CQA Plan) for the TMA cell construction. Appendix O has been modified to reflect the changes made to the TMA composite liner and leachate collection system and is incorporated into this Addendum No. 3 as Appendix A.

The Quality Assurance and Quality Control work required for the P40 till layer as proposed is similar to that required for a compacted clay liner (CCL) except that the P40 till layer is to function primarily as a backup layer to the low permeability GCL layer which is the main component providing the "composite action" with the overlying geomembrane.

4.4 Geosynthetic Clay Liner (GCL)

4.4.1 General

CMC was asked by WDNR to consider the use of a geosynthetic clay liner (GCL) as an alternative to the compacted clay liner (CCL) proposed in the May 1995 Feasibility Report (Foth & Van Dyke, 1995a). Consideration of the GCL was requested because of WDNR's concerns regarding the potential environmental impacts which could result from the excavation of approximately 743,000 cubic yards of clay from an off-site borrow source and the truck transportation of approximately 48,000 truckloads of clay to the TMA site over its life. Given the potential impacts of clay excavation and transport, GCLs are considered a more suitable alternative from the standpoint of environmental impacts.

Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) contains a review of available GCL materials, including a description of some of the GCLs commonly available in the United States. GCL manufacturers have a number of new products and variations of existing products under development (Koerner, 1996). The availability of GCL products is increasing as GCL use increases. CMC believes it is prudent to make the final selection of the GCL during the final Plan of Operation engineering for the TMA. This will allow CMC to select the best product available that meets the project and design requirements for the TMA cells. The purpose of this section is to provide sufficient information to show the proposed design is feasible. The following information is provided to demonstrate this feasibility.

- The available GCL is shown to be suitable for use, including:
  - equivalency to compacted clay liners;
  - material and interface shear strength;
  - stability issues on interior cell slopes, including the methods used to calculate stability;
  - compatibility with anticipated tailings pore water;

- Present manufacturer's quality control standards and frequency of testing;

- Update of CQA plan (Appendix A).

Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) did not select a specific available GCL. However, if the TMA were constructed today, the selected GCL would consist of a treated sodium bentonite sandwiched between two layers of needle punched non-woven geotextiles. CMC, for the reason stated above, does not wish to select a specific GCL, however, understands that a relatively detailed review of GCLs must be made to satisfy regulatory requirements. To satisfy these requirements, CMC has provided information concerning a GCL type available on the market today which meets project design needs. The list of available products from the *Geotechnical Fabrics Report 1997 Specifier's Guide* is provided in Appendix B. The potentially suitable GCLs are indicated in Appendix B with an asterisk (*).

4.4.2 Liner Equivalency

Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) provides a summary of installation and performance advantages of GCL as put forth by Dr. David Daniel in the U.S. Environmental Protection Agency's (USEPA) *Report of Workshop of Geosynthetic Clay Liners* (USEPA, 1993). Table 1 of Addendum No. 1 is a comparison of a GCL to a CCL which includes advantages and characteristics of each. In Addendum No. 1 it was determined that a geomembrane liner with a GCL backing has a smaller flow rate at least by a factor of 2.6 when compared with a geomembrane backed by a CCL (refer to Attachment 10 of Addendum No. 1 for calculations).

4.4.3 Mechanical Properties of GCL

GCL has been proposed to replace the clay component of the hydraulic barrier for both the liner and final cover systems. Since the GCL is a factory made product with bentonite, in comparison with clay, two additional aspects need to be evaluated. These are the internal strength of the GCL and the interface shear strength between the GCL and the soil on which it rests. While GCL is not likely to be of concern under compressive stresses under shear stress application as on the sideslopes there is a potential for decreased stability especially when the GCL is hydrated.

The method of evaluating the required interface and internal strength of the GCLs is the same as the one used to evaluate the required strength of the geomembrane interfaces (Section 6.3.4.2 of the Feasibility Report). Appendix C addresses the properties of GCL and its use in the TMA. The most critical part of the liner/final cover system from stability/stress conditions has been examined in this appendix and the minimum required strength has been identified.

4.4.4 Compatibility with Anticipated Tailings Pore Water

Addendum No. 2 to the Feasibility Report (Foth & Van Dyke, 1996b) provided information, including laboratory test data, for a contaminant resistant clay (CRC) called VOLCLAY® CRC manufactured by CETCO (Colloid Environmental Technology Company). VOLCLAY® CRC has been developed for containment of materials which contain high concentrations of salts and other ions. Laboratory test data from CETCO comparing VOLCLAY® CRC with other bentonites was provided in Table 1 of Addendum No. 2 (Foth & Van Dyke, 1996b). The data show that the VOLCLAY® CRC meets the performance requirements when prehydrated with a 1,000 ppm calcium chloride solution and permeated with a 10,000 ppm calcium chloride solution.

This section provides information from a case study where a contaminant resistant GCL was used and recommendations for a testing program for the GCL selected for use in the CMC TMA cells.

4.4.4.1 GCL Case Study

CETCO (1996) has published a case study relating to a containment system for a calcium carbonate sludge leachate from acidic mine drainage produced at a mine site in Northern California. The acidic mine drainage is collected at the site and treated through the addition of lime. The treatment process produces a calcium carbonate sludge removed by gravity settling in large effluent storage lagoons (CETCO, 1996). Table 4.4-1 contains sludge leachate data.

Table 4.4-1

Sludge Leachate Data from CETCO Case Study

Parameter	Concentrations (mg/l)[1] Sludge Leachate
pH	9.1
Conductivity	2,900 umho/cm
Total Suspended Solids	15
Total Dissolved Solids	3,013
Sulfate	1,843
Calcium	523
Magnesium	28
Aluminum	2

[1]mg/l unless otherwise indicated.

Source: CETCO (1996).

Checked by: PAE

Shackelford (1994) showed that the hydraulic conductivity of sodium bentonite can be adversely affected by permeation with solutions with high salt concentration or polyvalent cations such as calcium (Ca^{++}). Shackelford found that the effects to hydraulic conductivity tend to be much more severe when the first wetting liquid is the leachate or liquid containing the cations or salt solution. In terms of the CMC TMA cell GCL, the concentration of Ca^{++} ions are expected to be the main constituent of concern relative to compatibility of the GCL to the leachate. Preliminary results from CMC's waste characterization work show that the concentration of Ca^{++} in the CMC tailings pore water is in the same range as the Ca^{++} concentration in the sludge leachate from the CETCO case study (CETCO, 1996). Tailings pore water characteristics will be provided in the project's final waste characterization report. The CETCO case study compared the performance of VOLCLAY® CRC with ordinary bentonite (i.e., untreated bentonite) when using the leachate characterized in Table 4.4-1 as a permeant. The results of this comparison are shown in Table 4.4-2.

The data in Table 4.4-2 shows that the performance of VOLCLAY® CRC is not significantly impacted by initial exposure to a permeant high in Ca^{++}, in fact, the two key properties, free swell and hydraulic conductivity, exceed the acceptable values for GCL of 25 ml and $1x10^{-9}$ cm/sec, respectively (University of Texas, 1996). In review of this data, it can be concluded that the contaminant resistant bentonites available on the market today can be manufactured to be compatible with the tailings pore water expected from the TMA cells.

Table 4.4-2

Comparison of VOLCLAY® CRC to Ordinary Bentonite

Test Parameter	Ordinary Bentonite	VOLCLAY CRC®
Fluid Loss (API 13A/13B)	21 ml	10 ml
Free Swell (USP NF XVII)	13 ml	27 ml
Permeability (ASTM D 5084)	$1x10^{-8}$ cm/sec	$2x10^{-10}$ cm/sec

Source: CETCO, 1996.

Checked by: JWS

4.4.4.2 Future Testing Programs

The discussion of CRC in Addendum No. 2 to the Feasibility Report (Foth & Van Dyke, 1996b) outlined a three-tiered approach to establish which bentonite to use for an application with higher concentrations of soluble metallic cations. Since the concentration in the leachate of calcium in particular is expected to be greater than several hundred parts per million, second tier tests, fluid loss or top-loading filter press (TLFP), will be performed as part of the development of the project's Plan of Operation. If these test results are inconclusive or inconsistent, an extended permeability test will be performed as the third tier. A 30-day permeability test is typically sufficient for evaluating a GCL. Final selection of the GCL product to use for the TMA, including required testing, will occur during Plan of Operation preparation.

4.4.5 GCL Properties and Test Methods

4.4.5.1 General

The testing of GCLs includes manufacturing quality control testing, engineering design tests, and quality assurance/quality control testing completed during GCL construction. Since the GCL to be used for the TMA will be selected during final Plan of Operation development, the testing program outlined below may be modified as part of Plan of Operation development.

4.4.5.2 Manufacturing Quality Control (MQC)

It is understood that ASTM will adopt new standards for the certification of GCL properties. These standards are not yet published and the required values may vary depending on the specific GCL material and its properties. CETCO provided the ASTM test methods listed in Table 4.4-3 and the required values for CETCO's BENTOMAT "ST"®. It is understood that listed test methods and test frequency will become part of ASTM recommended MQC testing program for GCLs.

Table 4.4-3

Manufacturer's Recommended Quality Control Test Program for BENTOMAT "ST"®

Material Property	Test Method	Test Frequency, ft^2 (m^2)
Bentonite Swell Index[1]	ASTM D 5890	1 per 50 tonnes[6]
Bentonite Fluid Loss	ASTM D 5891	1 per 50 tonnes
Bentonite Mass/Area[2]	ASTM D 5261	40,000 ft^2 (4,000 m^2)
GCL Grab Strength[3]	ASTM D 4632	200,000 ft^2 (20,000 m^2)
GCL Grab Elongation	ASTM D 4632	200,000 ft^2 (20,000 m^2)
GCL Peel Strength	ASTM D 4632	40,000 ft^2 (4,000 m^2)
GCL Index Flux[4]	ASTM D 5887	Weekly
GCL Hydrated Internal Shear Strength[5]	ASTM D 5321	Periodic

[1] Bentonite property tests performed at CETCO's bentonite processing facility before shipment to CETCO's GCL production facilities.

[2] Bentonite mass/area reported at 0 percent moisture content. The reported value is equivalent to 0.95 psf at 20% moisture content, the GCL industry standard.

[3] All tensile testing is performed in the machine direction, with results as minimum average roll values unless otherwise indicated.

[4] Index Flux with deaired distilled water at 5 psi (35 kPa) confining pressure and 2 psi (15 kPa) head pressure. Reported value is equivalent to 925 gal/acre/day. This flux value is equivalent to a permeability of $5x10^{-9}$ cm/sec. This flux value should not be used for equivalency calculations. A flux test using gradients that represent field conditions must be performed to determine equivalency. The last 20 values may be reported from the end of the project date of the supplied GCL.

[5] Peak value measured at 200 psf (30 kPa) normal stress. Site-specific materials, GCL products, and test conditions must be used to verify internal and interface strength of the proposed design.

[6] One tonne is equivalent to 1.1 tons.

Source: CETCO (Colloid Environmental Technologies Company), May 1, 1996. Technical Data Sheet TR 404bm.

Checked by: REM

4.4.5.3 Engineering Design Tests

The engineering design tests required for a project are determined based on the specific conditions at the site and the specific products proposed for the GCL, geomembrane, geotextiles, geocomposite, and soils. Since all soil and geosynthetic products need to be selected prior to completing the engineering design tests, CMC proposes to complete these tests during the development of the project's final Plan of Operation. If the facility were constructed today, it is recommended that the engineering design tests specified in Table 4.4-4 be completed for the GCL. The type and frequency of these tests may be modified based on the GCL specified in the Plan of Operation.

Table 4.4-4

GCL Engineering Design Tests

Property	Test Method
GCL Permeability	ASTM D 5084 modified or GRI GCL-2
Direct Shear (top, bottom, and midplane)	ASTM D 5321
Creep Shear (top, bottom, and midplane)	ASTM D 5321 (modified)
Wide-Width Tensile Strength and Elongation	ASTM D 4595
GCL/P40 Till Compatibility	ASTM D 5084 modified using leachate

Prepared by: REM
Checked by: NXP

4.4.5.4 GCL Construction QA/QC

To account for the addition of the P40 till and GCL in the TMA composite liner, an update to the project's construction quality assurance plan (CQA plan) has been completed. This updated plan is included in Appendix A. The updated CQA plan contains the quality control and quality assurance requirements for both the P40 till and GCL.

4.5 Additional Construction Requirements

The addition of the GCL to the composite liner system results in the need to specify construction requirements which take into account the unique properties of GCLs. The following are recommendations for GCL transportation, storage, and installation which are generally adopted from guidelines in (University of Texas, 1996). These requirements will be incorporated into the plans and specifications for the construction of the initial stage of TMA cell 1.

- Transportation, Handling, and Storage:
  - GCL panels should be shipped in rolls by themselves with no other cargo and in protective wrappings so damage could not occur during transit.

- GCLs should be loaded and unloaded in a manner which does not cause damage to the GCL. Repair protective wrapping if damaged.

- Storage on-site should occur only for a short period of time. GCL should be stored off the ground and covered completely with a tarpaulin.

• Subgrade Preparation:

  - Subgrade must be free of gravel greater than 1 inch in diameter for angular stones and 2 inches in diameter for rounded stones.

  - Subgrade must be rolled smooth with a drum roller to the correct grade.

  - Moisture content of subgrade should be below optimum moisture for compaction if practicable.

• Installation:

  - GCL should be rolled onto the prepared subgrade. GCL panels should not be dragged onto the subgrade.

  - GCL can be installed in any direction on flat surface.

  - Install GCL panels with the length parallel to the slope (i.e., up and down the slope) for sloped surfaces.

  - Overlap GCL in accordance with the manufacturer's requirements, which is typically 6 to 9 inches.

  - Lay out only the quantity of GCL that can be covered before the end of the day to prevent premature wetting of the GCL by precipitation.

  - Apply additional bentonite to the GCL overlaps in accordance with the manufacturer's recommendations. Lay out GCL panels first, then roll back overlap and add bentonite as per the manufacturer's recommendations.

  - Cover and/or backfill GCL each day after installation. All backfill operation should be observed carefully since this is the most critical time for GCL puncture.

  - Perform required QA/QC activities prior to covering or backfilling.

• GCL Repair

  - Inspect and repair tears or rips in the GCL by covering with a patch of the same GCL material extending at least 12 inches in all directions beyond the extent of the damage.

  - Heat bond or tape GCL patch in place prior to backfilling.

- If bentonite is required at GCL seams, apply in accordance with the manufacturer's instructions on all patches.

Detailed information concerning the installation of the GCL and other geosynthetic components will be provided in the project's Plan of Operation report.

4.6 Required Strength Properties for Liner Materials

The TMA liner system as proposed consists of a number of layers of soil and geosynthetic materials. Since modifications have been made to the composite liner components (e.g., a GCL has been substituted for native clay), the stability of the liner system was reanalyzed and the minimum strength of the materials and the interfaces between the materials was reestablished. The calculation of the minimum required strength for the composite liner materials and interfaces necessary to achieve satisfactory stability conditions are provided in Appendix C which is an update to Appendix D of the Feasibility Report (Foth & Van Dyke, 1995a). The calculations in Appendix C determined that the required strength for a factor of safety of 1.2 is an angle of interfriction of 21.2°, which means that all materials (i.e., till, internal strength of the GCL and P40 material, etc.) should have a shear strength equal to or greater than 21.2°. In addition, all interfaces, i.e., till/geocomposite, geocomposite/geomembrane, geomembrane/GCL. GCL/P40 till, and P40 till/till must also have a friction angle equal to or greater than 21.2°.

5 Leachate Collection System Update

5.1 General

Sections 5.2 through 5.3 of Addendum No. 3 respond to a request from the Wisconsin Department of Natural Resources (WDNR) for additional information concerning the design of the leachate collection system (LCS) drainage layer on the interior slopes of the TMA cells. The discussion includes background information concerning drainage layer design concepts, a discussion of oxygen entry considerations, a discussion of the currently proposed drainage layer concept, a discussion of the availability and compatibility of the proposed drainage layer materials, and presents the design for the preferred option for sidewall drainage. In addition, the design modification of the LCS at the base of the facilities as proposed in Addendum No. 1 (Foth & Van Dyke, 1996a) are incorporated into the design presented herein.

Section 5.4 provides a response to a request from the WDNR for additional information concerning initial tailings placement in each of the four TMA cells, initial leachate collection system (LCS) operations, and the design criteria for the protective filters for the LCS drainage layer. It also addresses the physical filtration issues raised by WDNR.

5.2 Leachate Collection System Piping at the Facility Base

Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) proposed a design modification that incorporated additional LCS piping at the facility base. Addendum No. 1 showed that the additional LCS piping provides additional redundancy to the LCS system and modest improvements in LCS efficiency (refer to page 17 of Addendum No. 1 for additional details concerning leachate collection system efficiencies).

Drawing 16, Revised Piping Plan, has been updated to show that a system of 4-inch diameter perforated leachate collection pipes has been added to the base of each TMA cell at a regular spacing, typically approximately 260 feet apart. As shown on Drawing 16, the 4-inch diameter perforated pipes are laid in the drainage layer at an approximate 0.5 percent slope. As shown in Detail 7/29, a section of 6-inch leachate collection pipe connects the 4-inch pipes with the 6-inch perforated headers at the toe of the interior slope. Detail 7/29 also shows the temporary cleanout which is stubbed out for cleaning the 4-inch laterals upon completion of construction.

Attachment 7 of Addendum No. 1 (Foth & Van Dyke, 1996a) provides calculations which show that a 4-inch LCS pipe is properly sized to handle the peak daily flows as determined by the HELP model. The maximum flow capacity of a 4-inch pipe at 0.5 percent slope is about 80 gpm, the expected peak flow is approximately 5 gpm.

Calculations in Attachment 7 show that the maximum 1-foot head on the liner will not be exceeded with pipe installed at the base of the drain layer above the liner. Therefore trenches in the liner are not required to meet the performance requirements.

Appendix D contains a pipe strength analysis for the 4-inch LCS piping which shows that the pipe bedding (i.e., drainage layer) and the placement of bedding material around the pipe will result in a uniform loading on the pipe as well as reduction of load on the pipe due to soil-structure interaction of the bedding material completely surrounding the pipe and the strength of the tailings material itself.

5.3 Interior Berm Slopes - LCS

5.3.1 Background

The WDNR has raised concerns and questions regarding the extent of placement of the leachate collection system (LCS) drainage layer on the interior sidewalls of the proposed TMA cells. The discussion which follows provides a summary of CMC's work that addresses WDNR's questions and concerns.

As part of the initial TMA design process, performance objectives for the TMA were formulated. These performance objectives are listed in Section 6.1.1, Performance Objectives, of the Feasibility Report (Foth & Van Dyke, 1995a). Those performance objectives most closely related to the extent of the drainage layer on the sidewall are as follows:

- the design of a composite base liner system to control percolation rates . . ."

- "limiting oxygen entry into the tailings during the operating and post-closure period; and

Initially, in the Feasibility Report, these two performance objectives were used to evaluate alternatives relating to the extent of the drainage layer along the TMA sidewalls. Section 10.9.6 of the Feasibility Report (Foth & Van Dyke, 1995a), Need for Leachate Collection System, provides a comparison of the advantages and disadvantages of a full LCS which included a high permeability drainage layer along the entire sidewall to a partial LCS with a drainage layer partially up the sidewall of the initial constructed stage.

It is apparent from the Section 10.9.6 discussion that the main advantage of a drainage layer which extends partially up the interior sideslope is maintaining the tailings in a neutral state. WDNR responded to the May 1995 Feasibility Report design by requesting that CMC consider design measures which would increase the liner efficiency on the TMA cell sidewalls. WDNR's concern was outlined in Page 6, Comment 1 of the January 4, 1996 feasibility completeness determination letter, which states: "Provide a redesign of the facility extending the drainage layer up the side slopes of each cell at least to the first stage boundary to facilitate leachate drainage and collection." CMC responded to this request in *Addendum No. 1 to the Tailings Management Area Feasibility Report/Plan of Operation* (Addendum No. 1) (Foth & Van Dyke, 1996a). In summary, this response included the following conclusions:

- Extending the drainage system up the sidewalls is not necessary from a performance standpoint since the present design is capable of meeting the groundwater standards.

- To provide additional redundancy in the design, CMC proposed adding a geocomposite (drainage layer) which could be covered by a geomembrane on top of the composite liner. It was proposed to extend this halfway up the sideslope of the initial stage of each TMA cell (refer to Figure 6 of Addendum No. 1). This system would be stable under operational and closure conditions.

- The proposed design (as described in Addendum No. 1) would reduce the total percolation from the facility by about 50 percent and is generally equivalent in reducing percolation to extending a drainage layer halfway up the initial stage.

- Twenty years after closure, the percolation from the TMA is governed by the percolation through the final cover.

In developing its proposal, CMC believed the alternative presented in Addendum No. 1 provided a balance between the goals of limiting percolation from the facility and limiting oxygen entry. However, in subsequent meetings the WDNR restated their preference that additional redundancy be provided in the drainage layer on the interior sidewalls to further increase the LCS efficiency. WDNR has suggested that their preference is to have the drainage layer extend to the top of the initial stage of each TMA cell.

WDNR has also expressed concern regarding the potential for oxygen entry during the operations and long-term care periods. Section 6.2.7 of the Feasibility Report (Foth & Van Dyke, 1995a) contains an explanation of the three controls to be used during operation of the cells, two of which will control oxidation of the sulfide minerals and one of which will add additional neutralization potential to the tailings mass. The first two controls are part of the subaerial deposition process. These two controls, described in detail in Section 6.2.7.2, Operational Controls, of the Feasibility Report (Foth & Van Dyke, 1995a), can be summarized as follows:

- The planned tailings deposition will result in 50 to 75 percent of the tailings maintained below water; and

- The planned tailings deposition involves sequential tailings placement, meaning fresh, saturated tailings are deposited on a continuous basis, minimizing the time tailings are exposed to oxygen.

The final control is the natural calcium carbonate content of the tailings and the added neutralizing capacity provided by the addition of lime during the beneficiation process which results in an elevated pH of the process water used to slurry the tailings to the TMA. The three controls mentioned above accomplish the primary objective of the operations design which is to control acid formation.

Section 6.2.7.3, Available Measures to Control Initiated Sulfide Mineral Oxidation, of the Feasibility Report (Foth & Van Dyke, 1995a) describes methods available to control sulfide mineral oxidation in the unlikely event that oxidation is initiated. Section 6.6.8, Final Cover Performance in Preventing Oxygen Entry, of the Feasibility Report evaluates the three mechanisms by which oxygen entry could occur after final cover placement, i.e., diffusion, convective currents, and barometric pumping. Per the request of the WDNR, CMC is in the process of completing oxygen transport modeling related to the TMA. The objective of this work is to estimate the rate at which oxygen could enter the TMA at various stages in its operation, closure, and long-term management. A report on this work will be issued to the WDNR, USCOE, and others shortly.

5.3.2 Options for Extending the Drainage Layer

After further consideration, CMC agreed with WDNR's request to extend the drainage layer to the top of the initial stage of each TMA cell. CMC believes that although in its opinion this is not required to enable the facility to comply with state and federal environmental standards, it does provide additional redundancy and increases the efficiency of the LCS.

CMC has considered three options for materials for the drainage layer on the interior slopes. These are: 1) a geocomposite; or 2) a granular soil; 3) a geomembrane and geocomposite option. Detail 1/29 on Drawing 29 and Figures 5.3-1 and 5.3-2 graphically show the options. CMC desires to carry these three options into the final engineering design of the facility, and make the selection at the Plan of Operation stage of facility development. The three options considered are described below and a figure showing the configuration of each is provided. For presentation purposes, the geocomposite option has been designated as the preferred option and is shown in the design drawings, used in estimating construction quantities, used in completing HELP model calculations, etc.

5.3.2.1 Geocomposite Option

The geocomposite drainage layer option includes placing a geocomposite (a geonet with non-woven geotextiles heat bonded to its top and bottom surfaces) up the interior sideslope to approximately 1 foot below the maximum tailings elevation of the initial stage of each TMA cell (refer to Detail 1/29).

5.3.2.2 Granular Soil Option

The granular soil drainage layer option includes a granular soil layer extended up the interior sidewalls to approximately 1 foot below the maximum tailings elevation of the initial stage of each TMA cell (refer to Figure 5.3-1).

5.3.2.3 Geomembrane and Geocomposite Option

The geomembrane and geocomposite option consists of placing a geocomposite (a geonet with non-woven geotextiles heat bonded to its top and bottom surfaces) on top of the composite liner and then covering the geocomposite with a thermoplastic geomembrane (refer to Figure 5.3-2). The geomembrane would be lapped over the composite liner at the base of the cell for a length of no less than 5 feet. A woven geotextile would be placed over the upper geomembrane. The geomembrane is welded to the geomembrane component of the composite liner at the bench between the initial and the second stages of each cell.

5.3.3 LCS Material Compatibility

The three options proposed for the sidewall drainage in the TMA involve four different materials of construction as follows:

- soil component;
- non-woven geotextile component;
- geomembrane component; and
- geocomposite component.

The compatibility of these materials has been addressed in the Feasibility Report (Foth & Van Dyke, 1995a) and its subsequent addenda (Foth & Van Dyke, 1996a and b). Following are references where material compatibility has been addressed, along with a brief statement concerning the findings.

- Soil Component - The response to Page 3, Comment 19 in Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) addresses the compatibility of on-site soils with the tailings and leachate. Past studies have shown that on-site soil permeability, mineralogy, and chemical characteristics will only be slightly affected by tailings and/or leachate.

- Non-woven Geotextile Component - The response to Page 5, Comment 21 in Addendum No. 2 to the Feasibility Report (Foth & Van Dyke, 1996b) and the response to Page 3, Comment 18 in Addendum No. 1 (Foth & Van Dyke, 1996a) discuss the resin (polypropylene) which most likely will be used and the rationale for its selection. If the TMA were constructed today, polypropylene would be selected as the resin for the non-woven geotextiles, including the geotextiles used for filtration, separation and/or as cushioning layers as well as the geotextiles used in conjunction with the geonet (i.e., the geocomposite). Attachment 4 of Addendum No. 1 contains research findings concerning polypropylene compatibility which shows that acids, solvents, and solutions do not adversely impact this polymer.

- Geomembrane Component - If the TMA were constructed today, high density polyethylene (HDPE) would be selected as the geomembrane liner. The response to Page 3, Comment 18 in Addendum No. 1 and the response to Page 5, Comment 21 in Addendum No. 2 provide evidence that the proposed HDPE geomembrane is compatible with the expected characteristics of the tailings and leachate.

- Geocomposite Component - The majority of the geocomposites on the market today use high density polyethylene (HDPE) as a raw material for geonets. Polypropylene geotextiles are then heat bonded on the top and bottom of the geonet. Both of these materials are, as discussed above, compatible with the intended use.

5.3.4 Material Availability

The geosynthetics, including geotextiles, geomembranes, and geocomposites are available in the marketplace from numerous vendors. Appendix E contains an excerpt from the *Geotechnical Fabrics Report 1997 Specifier's Guide* to available products which includes geocomposite material specifications which could be used in the TMA. Potentially suitable geocomposites for the TMA are identified in Appendix E with an asterisk (*). The availability of the granular soil drainage layer was documented in the responses to Page 6, Comments 3 and 4 in Addendum No. 1 (Foth & Van Dyke, 1996a). Attachment 11 of Addendum No. 1 includes calculations of the availability of on-site till soils for manufacturing the drainage layer and P40 till layer required for the proposed TMA construction. Figure 10.4-1 is an earthwork balance flowchart showing that an adequate quantity of till is available for the drainage layer on the interior sideslopes. Figure 10.4-1 shows that approximately 612,200 in-place cubic yards of drainage layer material, including LCS and final cover, is required if the geocomposite is used as the drainage layer for the initial stage in all four TMA cells.

5.3.5 Preferred Option

As indicated earlier, CMC desires to carry the three drainage layer options discussed above into the Plan of Operation where the final selection will be made during the final engineering design of the initial cell. For the remainder of this addendum, the geocomposite will be considered as the preferred option for the berm slopes drainage layer.

The geocomposite drainage layer consists of a geonet, i.e., a net-like polymeric material formed from intersecting ribs integrally joined at the junctions, with non-woven geotextiles heat bonded to its top and bottom surfaces. Geocomposites are available from many manufacturers and, as with other geosynthetics, new products are constantly under development, resulting in frequent changes in product specifications. As such, CMC plans to make the final selection of the specific geocomposite during the development of the project's Plan of Operation.

The primary function of the geocomposite as the leachate collection medium of sidewalls is to convey liquid within the plane of its structure to the drainage layer at the base of the cell. The geocomposite properties related to this function are explained below.

- In plane flow rate (transmissivity) (ASTM D 4716) is the flow rate parallel to the long direction of the geocomposite measured at a gradient of 0.1 at 100 kPA (kilopascal), which is typically in the range of $4x10^{-3}$ m^2/sec (meter squared per second).

- Compressive strength (ASTM D 1621) is the cross-plane compressive strength of the geocomposite. The compressive strength is important since the geocomposite must resist yield or collapse under anticipated loadings which would impact the geocomposite's ability to conduct liquid.

5.4 Initial Tailings Filling Cycle and Drainage Layer Design Criteria Suitability

5.4.1 Background

The WDNR has raised the issue of potential physical clogging of the LCS filtration and drainage layers within the TMA cells. Based on our understanding of the issue, CMC is providing the following additional information to assist the WDNR in the completion of its review of this topic:

- a more detailed explanation of the initial tailings filling cycle;

- a more detailed explanation of the functioning and operation of the LCS during the initial filling process; and

- additional explanation concerning the suitability of the design criteria used for the protective filter design.

5.4.2 Initial Filling Cycle for Each TMA Cell

The initial filling of TMA 1 will come principally from accumulated precipitation, mine drainage and/or from water pumped from the construction well sited north of TMA 2. Upon completion of all construction including the placement of the waste rock as riprap, the cell bottom will be filled with water to a level approximately 3 to 5 vertical feet below the top of the riprap as shown on Figure 5.5-1. Initial placement of water at the onset of operation of each TMA cell is required to sustain the operation of the mill. If well water is pumped to the cell, the discharge pipeline will be laid horizontally on the riprap so the energy from the discharging water will be dissipated and erosion of the filter layers will not occur.

Initially, the tailings will be discharged subaqueously at a diffused velocity (i.e., deposited below the water placed in the cell). As described in Section 6.2.4 of the Feasibility Report (Foth & Van Dyke, 1995a), the tailings slurry will be discharged in a panel of adjacent spigots. The discharge panel will be moved progressively around the cell by closing spigots at one end of the panel while opening spigots at the other end of the panel. Each spigot assembly, which will be spaced at approximate 100 to 200-foot centers around the perimeter of the facility, will consist of a length of flexible rubber pipe which will extend from the tailings main header pipe surrounding the cell through a pinch valve to the desired discharge level (see Detail 3/29 on Drawing 29).

At the time of initial tailings placement the flexible rubber spigot pipes will extend to just at or below the water surface in the cell (Figure 5.5-2). As tailings placement continues, the elevation of the tailings at the perimeter of the cell will eventually extend above the surface of the water. At this time, since deposition will occur around the outside of the cell, a water pond will begin to form in the approximate center of the cell as shown in Figure 5.5-3. The tailings deposition will force the water to the center of the cell as shown in Figure 5.5-3, and deposition will become subaerial as described in Section 6.2.4, Tailings Distribution System, of the Feasibility Report (Foth & Van Dyke, 1995a).

As the tailings are deposited under water, they will tend to flow at a steeper angle at a diffused velocity with the denser particles settling faster. Tailings deposited on the beaches will be on a shallow slope with coarser particles near the perimeter of the cell and finer at the center of the cell below the deepest water in the pond. During the subaerial placement of tailings, the pond center is likely to contain the finest tailings particles.

Slurried tailings will initially flow onto the base of the cell and initially fill the voids in the riprap. The riprap will act to dissipate the discharge energy and to prevent erosion. Once the riprap voids are filled, subaqueous tailings beach development will occur, followed eventually by subaerial deposition which will result in consolidation of the subaqueously deposited tailings.

5.4.3 Initial Operation of the LCS

Initially the tailings will be deposited subaqueously as described above. Subaerial deposition then will occur until the riprap is completely covered with tailings, as shown in Figure 5.5-3. At this point, with subaerial deposition occurring and a water pond located near the center of the cell, pumping of leachate from the LCS using the pumps in the sideslope risers (SSR) will be started. The following describes the placement of tailings prior to initiation of LCS operation in each cell and conditions in the tailings during initial LCS operation.

- Since initial tailings placement will take place before leachate collection is initiated, a bed of settled tailings will be in contact with the riprap and till layer of the cell. The bed of settled tailings will also act as a quasi-filter as water is moved from the overlying tailings into the drainage layer.

- The profile of hydraulic conductivities through the tailings will likely be anisotropic with slightly lower permeabilities occurring near the cell center. The hydraulic conductivities of both fine and whole tailings will be in the 10^{-6} cm/sec range. The water flow path, once the sideslope riser (SSR) pump is turned on, will consist of both vertical and horizontal components with the water following the most permeable layer toward the lower heads in the drainage layer evacuated by the SSR pump.

- The initial maximum head in the leachate collection system should be equal to the pond elevation, which will decrease with time as the LCS drains the tailings to field capacity.

5.4.4 Suitability of Design Criteria for Protective Filter Design

The design of the protective filter system was performed using the NAVFAC DM-7.1 filter criteria. The calculation using this criteria was provided in Appendix F of the Feasibility Report (Foth & Van Dyke, 1995a). These criteria were first developed by Terzaghi based on experiments conducted by Bertram (1940) and later modified slightly after laboratory tests by the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation (Holtz and Kovacs, 1981). The Waterways Experiment Station, U.S. Army Corps of Engineers (1941, 1942) have investigated the validity of the criteria. The criteria have been used worldwide ever since. WDNR has questioned the validity of using these criteria for design of the protective filter for the LCS drainage layer of the TMA. CMC believes the design criteria are valid because the TMA drainage layer design is similar to the design and construction on which Terzaghi based his studies. Terzaghi worked out the protective filter design method based on studies he made in the design and construction of filters for large dams in South Africa (Parcher and Means, 1968). Filter design for dams is similar to that for the TMA in that flow occurs from fine grained medium to coarse grained medium as the coarse grained medium is drained. Also, the fact that the TMA drainage layer will be saturated and that pumping of the TMA leachate collection system will not commence until an appreciable depth of tailings has been placed and allowed to settle at the base of each cell, further supports the applicability of NAVFAC DM-7.1 filter criteria in the design of the protective filter system for the CMC tailings management area.

The protective filter design used for CMC's tailings management area meets Terzaghi's criteria for the design of filters (refer to Appendix F of the Feasibility Report) since the following conditions are met:

- The movement of particles from the tailings to the filter layers and from the filter layers to the drainage layer is restricted since the three criteria related to the gradation of the tailings and soils are met.
- The loss of head in the filters is not excessive and the permeability is sufficient to meet the drainage needs.
- Other gradation requirements for the filter (e.g., no sizes larger than 3 inches and particles passing #200 sieve less than 5 percent (NAVFAC, DM-7.1)) are met.

Since the protective filter design criteria is applicable to the TMA design and the design criteria are met, physical clogging of the filter layers is not considered an issue regarding operation of the TMA cells. As a result, CMC sees no need to perform a physical test of the proposed filtration system.

6 Final Cover System Update

6.1 General

As discussed in Section 5 of this addendum, CMC has selected a geosynthetic clay liner (GCL) underlain by 12 inches of on-site P40 till soil as the soil component for the TMA liner and capping systems. The selected final cover system consists of a multi-layered system, including the following components, from top to bottom:

- 6 inches - topsoil and vegetative layer;
- 36 inches - rooting layer;
- 12 inches - drainage layer;
- 60 mil HDPE geomembrane;
- geosynthetic clay liner (GCL);
- 12 inches - low permeability soil consisting of P40 till soil (P40 Till); and
- a grading layer of variable thickness.

The only modification made to the cover system proposed in the Feasibility Report (Foth & Van Dyke, 1996a) is the substitution of the GCL and 12 inches of P40 till for 1 foot of off-site native clay. The section which follows discusses these design modifications.

6.2 Advantages of GCL in the Final Cover

Addendum No. 1 to the Feasibility Report (Foth & Van Dyke, 1996a) contains a listing of some of the advantages of a GCL related to installation, performance, and environmental protection. Following is a list of advantages of GCLs specifically related to their use in cover systems. To a great extent, these advantages are as reported by Dr. David Daniel in USEPA (1993) and University of Texas at Austin (1996).

- Cap Hydraulic Equivalency - Attachment 10 of Addendum No. 1 (Foth & Van Dyke, 1996a) provided calculations which show that a composite cap with a GCL backing has a smaller flow rate at least by a factor of 2.6 when compared with composite cap systems using native clay as the soil component.

- Air Permeability - GCLs placed in contact with moist soils will hydrate to a moisture content greater than 100 percent within a few weeks (University of Texas at Austin, 1996). Hydrated bentonite effectively provides a very low air permeability which, in combination with the geomembrane in the cover, provides an effective barrier against air intrusion.

- Physical/Mechanical Issues

  - Freeze/Thaw Resistance - Available laboratory data indicates that GCLs do not undergo increases in hydraulic conductivities as a result of freeze/thaw (USEPA, 1993).

  - Wet/Dry Effects - Available laboratory data indicates that desiccation of wet GCL does not cause cracking and that GCLs do have self-healing properties, i.e., that they can swell around punctures or cracks (USEPA, 1993).

- Response to Differential Settlements - LaGatta (1992) and Boardman (1993) studied the effects of differential settlement on GCLs (Daniel, 1993). These tests showed that many GCLs can withstand large distortions (Δ/L up to 0.5, i.e., the ratio of the differential settlement (Δ) divided by the horizontal distance over which settlement occurs (L)) and tensile strains up to 10 to 15 percent without undergoing significant increases in hydraulic conductivity (Daniel, 1993). Available data indicates that GCLs can withstand much greater tensile deformation than compacted soils without cracking.

- Stability on Slopes - GCL interface shear strength is very dependent on the extent of hydration/water content and the type of GCL. The GCLs proposed for use in the TMA cover have adequate strength to remain stable on the 2 percent slopes proposed for the TMA final cover.

- Construction Issues

  - Effects of Subgrade and Backfill - GCLs have a capability of self-healing around punctures from small, sharp objects (Daniel, 1993), however, care must be taken during construction and especially during backfilling to prevent large rips or punctures from occurring. In addition, GCLs require a smooth subgrade free from gravel greater than 2 inches prior to installation.

  - Ease and Speed of Placement and Construction - GCLs are easier to place than soil liners and can be placed much more quickly than soil liners (Daniel, 1993).

  - Availability of Material - GCLs were selected as the soil component of the composite liner system primarily because large quantities of clay would be required from off-site sources. The environmental impacts of excavating the approximate 743,000 cubic yards of clay for liner/cap construction from off-site are considered by the WDNR to be significant. GCL, in combination with the P40 till layer, provides an alternative with the least potential impact.

  - Ease of Quality Assurance (QA) - The QA for GCL is much simpler and faster than for compacted clay liners, which translates into increased construction efficiencies and decreased costs.

The GCL in combination with 12 inches of compacted P40 till was selected for the final cover primarily because of the advantages of the GCL over the compacted clay liner, especially as related to environmental impacts, constructibility issues, and physical/mechanical issues. The 12-inch compacted P40 till layer is provided as an additional redundancy to the GCL which is considered the primary low permeability soil component of the composite cap. The P40 till layer provides a low permeability layer which can provide composite action with the geomembrane and thus provides redundancy in the soil component design.

7 HELP Model Update

7.1 Background

Section 6.7 of the Feasibility Report (Foth & Van Dyke, 1995a) explains how the HELP model analyses were used to estimate the quantity of percolation through the liner and cover systems during the operation, closure, and post-closure periods. Section 6.7.2 provides a HELP model program overview discussing the rationale for selecting model input data for evapotranspiration, precipitation, temperature, and solar radiation. Section 6.7.3 provides the rationale for selection of the various soil and geosynthetic material properties and the tailings properties which were estimated taking into account the depth at which each tailings layer was located.

The HELP models presented in the Feasibility Report (Foth & Van Dyke, 1995a) have been updated to reflect the modifications made to the TMA's composite liner, LCS, and final cover. Information concerning the redesign of the composite liner, LCS, and final cover are provided in Sections 4, 5, and 6 of this addendum, respectively.

7.2 Design Data

7.2.1 Primary Models

Three primary models were used for water balance determination of the selected TMA design. These three primary HELP model designs are hence referred to as Sideslope (drained), Sideslope (undrained), and Base HELP Models. Sideslope (drained) refers to the case where a geocomposite drainage layer is placed over the liner on the sideslope. Sideslope and Base HELP Models have the same final cover configuration. The Sideslope HELP Model bottom liner layer is sloped at 33 percent. The Base HELP Model has a different bottom liner configuration which is sloped at two percent from a high point in the center of each cell.

7.2.2 Submodels

The Sideslope and Base HELP Models have been further defined to better represent the primary configurations over time. The Sideslope HELP Models have been subdivided into several models to represent filling, closure, and post-closure scenarios. The Base HELP Model has also been divided into submodels representing the same time periods to more accurately represent the water balance at different time periods. The primary Sideslope and Base HELP Models have been divided into the following submodels:

- Stages I, III, V, and VII (first stage of TMA cell filling, referred to as initial stage) [Sideslope (undrained) does not pertain to these submodels];

- Stages II, IV, VI, and VIII (second stage of TMA cell filling) (referred to as second stages);

- Post-Closure Period With Lateral Drainage on the Sideslopes and Base (represents the case with active leachate removal);

- Post-Closure Period With Lateral Drainage on the Sideslope and No Lateral Drainage on the Base (represents the case when leachate removal is discontinued); and

- Post-Closure Period With Lateral Drainage on the Sideslope and No Lateral Drainage on Base, and the GCL and P40 till portion of the liner (represents an assumed case in which the geomembrane may no longer be functional after a very long period).

The first four scenarios listed above represent the TMA defined by construction, operation, and post-closure conditions. The fifth scenario represents a postulated condition where the geomembrane in the composite liner is assumed to degrade after 150 years. The postulated degradation scenario is very conservative and is included in the analysis to assess the performance of the TMA if such an unlikely condition were to occur. In fact, based on the December 1996 report prepared by GeoSyntec Consultants of Boca Raton, Florida, titled *Assessment of Long-Term Performance of the Proposed HDPE Geomembrane Liner and Cap at the Crandon Project TMA Facility* (GeoSyntec, 1996), ". . . the HDPE geomembrane liner and cap at the TMA facility should function as designed for a very long time (e.g., hundreds of years) without deterioration in performance." The GeoSyntec report is reproduced in Appendix F.

The Sideslope and Base HELP Models for the initial stages have been conducted based on the following generalized design specifications and soil characteristic input parameters:

<u>Sideslope (drained) HELP Model: Initial Stages</u>

- no cover;
- no surface water runoff;
- vertical profile of 22.5 feet of tailings (half of maximum tailings depth);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- lateral drainage through geocomposite above the liner;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-1.

<u>Base HELP Model: Initial Stages</u>

- no cover;
- no surface water runoff;
- vertical profile of 45 feet of tailings;
- 970-foot length of base drainage at two percent slope;
- lateral drainage above the liner;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-2.

The 4-inch diameter leachate collection system laterals with a typical 260-foot spacing at the cell base were not considered in the base second stages analyses. The more conservative 970-foot leachate collection system spacing was used in the Base HELP model's second stage.

Table 7.2-1

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (drained) - Initial Stages (I, III, V, VII)

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
1	Tailings	30	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5803
2	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5529
3	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
4	Till/ Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
5	Geocomposite/ Lateral Drainage	0.24	34	---	---	0.8500	0.0100	0.0050	33.0	0.0100
6	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
7	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
8	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-2

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Base - Initial Stages (I, III, V, VII)

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Tailings	60	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5803
2	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5529
3	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
4	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.5385
5	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.5338
6	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
7	Granular Soil/ Lateral Drainage	24	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
8	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
9	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
10	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

The Sideslope and Base HELP Models for second stages have been conducted based on the following generalized design specifications and soil characteristic input parameters:

Sideslope (drained) HELP Model: Second Stages

- no cover;
- no surface water runoff;
- vertical profile of 67.5 feet of tailings (average of tailings depth at ends);
- 135-foot length of drainage at 3H:1V slope;
- lateral drainage through till layer;
- lateral drainage through geocomposite;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-3.

Sideslope (undrained) HELP Model: Second Stages

- no cover;
- no surface water runoff;
- vertical profile of 22.5 feet of tailings (average of tailings depth at ends);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-4.

Base HELP Model: Second Stages

- no cover;
- no surface water runoff;
- vertical profile of 90 feet of tailings;
- 970-foot length of base drainage at two percent slope;
- lateral drainage above the liner;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-5.

Table 7.2-3

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (drained) - Second Stages (II, IV, VI, VIII)

Layer #	General Description	Thickness (Inches)	Classification HELP	Classification USDA	Classification USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Tailings	90	0	---	---	0.5803	0.5100	0.3400	4×10^{-6}	0.5803
2	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5529
3	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
4	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.5385
5	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.5338
6	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.5297
7	Tailings	120	0	---	---	0.5259	0.4915	0.3215	2.0×10^{-6}	0.5259
8	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
9	Geocomposite / Lateral Drainage	0.24	34	---	---	0.8500	0.0100	0.0050	33.0	0.0100
10	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---
11	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
12	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-4

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (undrained) - Second Stages (II, IV, VI, VIII)

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
1	Tailings	30	0	---	---	0.5803	0.5100	0.3400	4×10^{-6}	0.5803
2	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5529
3	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
4	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
5	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---
6	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
7	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-5

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Base - Second Stages (II, IV, VI, VIII)

Layer #	General Description	Thickness (Inches)	Classification HELP	Classification USDA	Classification USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Tailings	120	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5803
2	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5529
3	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
4	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.5385
5	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.5338
6	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.5297
7	Tailings	120	0	---	---	0.5259	0.4915	0.3215	2.0×10^{-6}	0.5259
8	Tailings	120	0	---	---	0.5226	0.4905	0.3205	1.5×10^{-6}	0.5226
9	Tailings	120	0	---	---	0.5195	0.4900	0.3200	1.0×10^{-6}	0.5195
10	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
11	Granular Soil/ Lateral Drainage	24	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
12	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
13	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
14	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

The Sideslope and Base HELP Models for Post-Closure Period with lateral drainage on the base have been conducted based on the following generalized design specifications and soil characteristic input parameters:

Sideslope (drained) HELP Model: Post-Closure Period With Lateral Drainage on the Base

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 67.5 feet of tailings (average of tailings depths at the end);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- lateral drainage through geocomposite;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-6.

Sideslope (undrained) HELP Model: Post-Closure Period With Lateral Drainage on the Base

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 22.5 feet of tailings (average of tailings depths at ends);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-7.

Base HELP Model: Post-Closure Period With Lateral Drainage on the Base

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 90 feet of tailings;
- 970-foot length of base drainage at 2 percent slope ;
- lateral drainage above the liner;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-8.

Table 7.2-6

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (drained) - Post-Closure Period With Lateral Drainage on the Base

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3231
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2443
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	90	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5129
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5200
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5216
11	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.5293
12	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.5127
13	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4976

Table 7.2-6 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	2.0×10^{-6}	0.5022
15	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2929
16	Geocomposite/ Lateral Layer	0.24	34	---	---	0.8500	0.0100	0.0050	33.0	0.0602
17	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---
18	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
19	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2248

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-7

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (undrained) - Post-Closure Period With Lateral Drainage on the Base

Layer #	General Description	Thickness (Inches)	Classification HELP	Classification USDA	Classification USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3231
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2443
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.5010
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	30	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5086
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.5359
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5445
11	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.3980
12	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---

Table 7.2-7 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
13	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
14	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.3073

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-8

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Base - Post-Closure Period With Lateral Drainage on the Base

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3231
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2443
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	3×10^{-5}	0.5010
7	Till/Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	240	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5186
9	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.5239
10	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.5270
11	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.5025
12	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4992
13	Tailings	120	0	---	---	0.5259	0.4915	0.3215	2.0×10^{-6}	0.5089

Table 7.2-8 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	1.5×10^{-6}	0.5216
15	Tailings	120	0	---	---	0.5195	0.4900	0.3200	1.0×10^{-6}	0.5194
16	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2960
17	Granular Soil/ Lateral Drainage	24	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0832
18	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
19	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
20	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2244

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

The Sideslope and Base HELP Models for the Post-Closure Period with lateral drainage on the sideslope and no lateral drainage on the base have been conducted based on the following generalized design specifications and soil characteristic input parameters:

<u>Sideslope (drained) HELP Model: Post-Closure Period With No Lateral Drainage on the Base</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 67.5 feet of tailings (average of tailings depths at the ends);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- lateral drainage through geocomposite;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-9.

<u>Sideslope (undrained) HELP Model: Post-Closure Period With No Lateral Drainage on the Base</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 22.5 feet of tailings (average of the depths at the ends);
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through the till layer;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-10.

<u>Base HELP Model: Post-Closure Period With No Lateral Drainage</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 90 feet of tailings;
- till and granular soil layer as vertical percolation layers;
- geomembrane;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-11.

Table 7.2-9

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (drained) - Post-Closure Period With No Lateral Drainage on the Base

Layer #	General Description	Thickness (Inches)	Classification HELP	Classification USDA	Classification USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3813
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2561
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0328
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	90	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.4817
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4739
11	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.4682
12	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.4636
13	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4594

Table 7.2-9 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	2.0×10^{-6}	0.4530
15	Till/ Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
16	Geocomposite/ Lateral Layer	0.24	34	---	---	0.8500	0.0100	0.0050	33.0	0.0100
17	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---
18	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
19	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.1973

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-10

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Sideslope (undrained) - Post-Closure Period With No Lateral Drainage on the Base

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3813
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2561
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0328
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	30	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.4817
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4740
11	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
12	Geomembrane	0.06	35	---	---	---	---	---	2×10^{-13}	---

Table 7.2-10 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
13	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
14	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2008

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-11

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model Base - Post-Closure Period With No Lateral Drainage

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3813
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2561
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0328
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	2×10^{-5}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	240	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4739
10	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.4682
11	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.4636
12	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4594
13	Tailings	120	0	---	---	0.5259	0.4915	0.3215	2.0×10^{-6}	0.4559

Table 7.2-11 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification			Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
			HELP	USDA	USCS					
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	1.5×10^{-6}	0.4530
15	Tailings	120	0	---	---	0.5195	0.4900	0.3200	1.0×10^{-6}	0.4503
16	Till	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
17	Granular Soil/ Vertical Percolation	24	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0320
18	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
19	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
20	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.1973

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

The Sideslope and Base HELP Models for Post-Closure Period with lateral drainage on the sideslope and no lateral drainage on base and a hypothesized lack of a geomembrane layer in the base after 150 years. The following generalized design specifications and soil characteristic input parameters represent this case:

<u>Sideslope (drained) HELP Model: Post-Closure Period With No Lateral Drainage on the Base and No Base Geomembrane</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 67.5 feet of tailings;
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through till layer;
- lateral drainage through geocomposite;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-12.

<u>Sideslope (undrained) HELP Model: Post-Closure Period With No Lateral Drainage on the Base and No Base Geomembrane</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 22.5 feet of tailings;
- 135-foot length of base drainage at 3H:1V slope;
- lateral drainage through the till layer;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-13.

<u>Base HELP Model: Post-Closure Period With No Lateral Drainage and No Base Geomembrane</u>

- final cover soils;
- 1,500-foot length of cover drainage at 2 percent slope;
- geomembrane;
- GCL;
- P40 till layer;
- vertical profile of 90 feet of tailings;
- GCL;
- P40 till layer; and
- soil and tailing layer data as provided in Table 7.2-14.

Table 7.2-12

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model
Sideslope (drained) - Post-Closure Period
With No Lateral Drainage on the Base and No Base Geomembrane

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3328
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2469
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0398
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	90	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.4817
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4739
11	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.4682
12	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.4636

Table 7.2-12 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
13	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4594
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	2.0×10^{-6}	0.4530
15	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
16	Geocomposite/ Lateral Layer	0.24	34	---	---	0.8500	0.0100	0.0050	33.0	0.0100
17	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
18	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.1886

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-13

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model
Sideslope (undrained) - Post-Closure Period
With No Lateral Drainage on the Base and No Base Geomembrane

Layer #	General Description	Thickness (Inches)	Classification HELP	Classification USDA	Classification USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3328
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2469
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0398
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/ Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	30	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5529	0.5066	0.3366	3.66×10^{-6}	0.4817
10	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4740
11	Till/Lateral Drainage	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
12	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	3×10^{-9}	0.7500
13	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.1886

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Table 7.2-14

Soil, Waste and Geosynthetic Characteristics Used for Water Balance Model
Base - Post-Closure Period With No Lateral Drainage on the Base and No Base Geomembrane

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
1	Topsoil	6	8	L	ML	0.4630	0.2320	0.1160	3.7×10^{-4}	0.3328
2	Till/ Rooting Zone	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2469
3	Granular Soil/ Lateral Drainage	12	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0398
4	Geomembrane	0.06	35	---	---	---	---	---	2.0×10^{-13}	---
5	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	2×10^{-5}	0.7500
6	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.2840
7	Till/Grading Layer	36	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
8	Tailings	240	0	---	---	0.5803	0.5100	0.3400	4.0×10^{-6}	0.5055
9	Tailings	120	0	---	---	0.5445	0.5033	0.3333	3.33×10^{-6}	0.4739
10	Tailings	120	0	---	---	0.5385	0.5000	0.3300	3.0×10^{-6}	0.4682
11	Tailings	120	0	---	---	0.5338	0.4965	0.3265	2.66×10^{-6}	0.4636
12	Tailings	120	0	---	---	0.5297	0.4930	0.3230	2.33×10^{-6}	0.4594
13	Tailings	120	0	---	---	0.5259	0.4915	0.3215	2.0×10^{-6}	0.4559

Table 7.2-14 *(Continued)*

Layer #	General Description	Thickness (Inches)	Classification HELP	USDA	USCS	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)	Initial Soil Water Content (vol/vol)
14	Tailings	120	0	---	---	0.5226	0.4905	0.3205	1.5×10^{-6}	0.4530
15	Tailings	120	0	---	---	0.5195	0.4900	0.3200	1.0×10^{-6}	0.4503
16	Till	18	10	SCL	SC	0.3980	0.2440	0.1360	1.2×10^{-4}	0.2440
17	Granular Soil/ Vertical Percolation	24	21	Gravel		0.3970	0.0320	0.0130	3.0×10^{-1}	0.0323
18	GCL[1]	0.24	17	---	---	0.7500	0.7470	0.4000	2×10^{-9}	0.7500
19	P40 Till	12	0	SiL	ML	0.5010	0.2840	0.1350	2×10^{-5}	0.1812

[1]Geosynthetic Clay Liner.

Prepared by: JBK
Checked by: NXP

Tables 7.2-1 through 7.2-5 show that the models do not include the ponding of the slurry water. The reason is that the purpose of the water balance is to primarily determine the leachate generation rates during the post-closure monitoring period and the percolation from the site. Since the simulation starts with the case of initial saturation of all tailing layers, the effect of the ponding (which is to keep the tailing layers saturated) will be indirectly accounted for if the percolation through the liner for the first year of simulation is assumed to be prevailing throughout the operation period of a cell. After conducting the HELP model runs in this fashion for the initial simulations described in the Feasibility Report (Foth & Van Dyke, 1995a), Dr. Paul Schroeder of the USCOE, Waterways Experiment Station, who is the primary author of HELP, was contacted for his comments on this approach. He suggested that the model may be operated to mimic the presence of ponding on the surface if the precipitation values were increased and surface runoff prevented. Accordingly, in the Feasibility Report (Foth & Van Dyke, 1995a), the submodels during the initial simulations for the open conditions were rerun with the precipitation values increased by a factor of two. The values obtained for leachate generation and percolation from the site for this case were found to be very similar to those obtained for the first year using actual precipitation values. This provided an interesting aspect of the simulation, that is, the percolation through the base liner was impacted more by the saturation and hence the head on the liner rather than the rainfall intensity during any year. For ponded conditions, since the rainfall variation will be neutralized by the relatively constant pond depths on the tailings surface, this result was not unexpected. Therefore, for the present analyses with the modified liner and sideslope drain systems, only the runs with the precipitation increased by a factor of two were conducted.

For all analyses, the geomembrane included both in the final cover and the base liner was considered to have one pinhole per acre due to manufacturing defects and four holes per acre due to installation defects. The contact between the membrane and the GCL component of the composite liner was taken as "good" discarding the option of "excellent" contact in order to obtain conservative (higher) values of percolation through the composite liner. Per Schroeder, et al. (1994), all the above conditions are either readily achievable by a good CQA program or are more conservative (producing larger percolation) than what will actually occur. Thus, by considering different geometry, appropriate material properties, and techniques for simulating operation scenarios, the HELP model should provide reasonably conservative values of leachate quantities and percolation from the site. These results are discussed in Section 7.3.

7.3 Summary of Results

7.3.1 Percolation Through the Liner and Leachate Generation

A summary of the results from the water balance study using the HELP models is shown in Table 7.3-1. The results shown pertain to the different submodels of both the Sideslope HELP Models and Base HELP Model. Thus different operation scenarios are covered; such as open case with doubled precipitation, closed case during initial post-closure period, closed case after the discontinuation of leachate removal and closed case after a very long time period when the geomembrane in the base liner is hypothetically assumed to be no longer effective. The percolation and leachate generation are given in terms of annual averages for the duration of the simulation period.

Table 7.3-1

Results of HELP Models Water Balance Analyses

HELP Model ID	Average Annual Totals for the Simulation Period					Peak Daily Values for Simulation Period			
	Water Balance Simulation Period Years	Percolation Through Cover[1] in/yr; (Percent)	Lateral Drainage/Leachate Collected[1] in/yr; (Percent)	Percolation Through Liner[1] in/yr; (Percent)	Average Head Across Liner (in)	Percolation Through Cover (in/day)	Lateral Drainage/Leachate Collected (in/day)	Percolation through Liner (in/day)	Average Head Across Liner (in)
Sideslope (drained) - Initial Stage (Table 7.2-1)	10	NA	14.71240 (23.37825)	0 (0)	0.006	NA	0.12109	0	0.018
Base - Initial Stage (Table 7.2-2)	10	NA	15.10629 (24.00415)	0.00019 (0.00030)	1.181	NA	0.09164	0.000001	2.614
Sideslope (drained) - Second Stage (Table 7.2-3)	10	NA	16.13380 (25.63687)	0 (0)	0.007	NA	0.12907	0	0.020
Sideslope (undrained) - Second Stage (Table 7.2-4)	10	NA	12.48061 (19.83190)	0.39887 (0.63381)	159.489	NA	0.04525	0.002628	288.000
Base - Second Stage (Table 7.2-5)	10	NA	16.20123 (25.74403)	0.00019 (0.00031)	1.266	NA	0.09074	0.000001	2.589
Sideslope (drained) - Final Cover (Table 7.2-6)	40	0.00010 (0.00034)	0.95497 (3.10485)	0 (0)	0	0.000004	0.09159	0	0.014
Sideslope (undrained) - Final Cover (Table 7.2-7)	40	0.00010 (0.00034)	0.50555 (1.64366)	0.00550 (0.1790)	3.266	0.000004	0.03780	0.001471	209.207
Base - Final Cover (Table 7.2-8)	40	0.00010 (0.00034)	1.33246 (4.33214)	0.00002 (0.00005)	0.104	0.000004	0.06450	0.000001	1.840
Sideslope (drained) - Final Cover (Table 7.2-9)	100	0.00010 (0.00034)	0.00011 (0.00036)	0 (0)	0	0.000005	0.00296	0	0.001
Sideslope (undrained) - Final Cover (Table 7.2-10)	100	0.00010 (0.00034)	0.00023 (0.00075)	0 (0.00001)	0	0.000005	0.00003	0	0.020

Table 7.3-1 *(Continued)*

HELP Model ID	Average Annual Totals for the Simulation Period					Peak Daily Values for Simulation Period			
	Water Balance Simulation Period Years	Percolation Through Cover[1] in/yr; (Percent)	Lateral Drainage/Leachate Collected[1] in/yr; (Percent)	Percolation Through Liner[1] in/yr; (Percent)	Average Head Across Liner (in)	Percolation Through Cover (in/day)	Lateral Drainage/Leachate Collected (in/day)	Percolation through Liner (in/day)	Average Head Across Liner (in)
Base - No Drainage (Table 7.2-11)	100	0.00010 (0.00034)	NA	0 (0.00001)	0.017	0.000005	NA	0	0.022
Sideslope (drained) - No Geomembrane at Base (Table 7.2-12)	100	0.00010 (0.00034)	0.00009 (0.00029)	0.00002 (0.00007)	0	0.000005	0.00296	0.000002	0.001
Sideslope (undrained) - No Geomembrane at Base (Table 7.2-13)	100	0.00010 (0.00034)	0.00001 (0.00002)	0.00023 (0.00075)	0	0.000005	0.00003	0.000111	0.020
Base - No Geomembrane at Base (Table 7.2-14)	100	0.00010 (0.00034)	NA	0.00016 (0.00051)	0	0.000005	NA	0.000106	0.010

[1]These values are given in inches/year as well as a percentage of precipitation, the latter within parentheses.

Notes:
- HELP Model ID - Soil properties are provided in respective table listed under the HELP Model ID.
- NA - Not applicable.
- Zero (0) represents values less than 0.0000005.

Prepared by: NXP
Checked by: JBK

For illustrative purposes, the yearly variation of percolation through the liner during the operation period for two cases, i.e., Sideslope Initial Stage, and Base Initial Stage are shown in Table 7.3-2 from the HELP model runs included in the May 1995 Feasibility Report (Foth & Van Dyke, 1995a). The percolation through the liner for Second Stage of the sideslope and base cases from the May 1995 Feasibility Report are shown in Table 7.3-3. Tables 7.3-2 and 7.3-3 show that the percolation quantities diminish through the initial years of operation. This is indicative of the gradual draining of the tailings and therefore does not account for the ponding on top of the cell. As described earlier in Section 7.2, one way to approximate the effects of ponding is to assume that the results for the first year (the maximum value) will continue to prevail throughout the time of ponding.

Table 7.3-2

Maximum Average Annual Percolation Through Liner During Operation Years of Initial Stages

Year	Sideslope (in/yr)	Base (in/yr)
1	0.187326	0.002003
2	0.149477	0.000396
3	0.112421	0.000441
4	0.044193	0.000342
5	0.005470	0.000290
6	0.013519	0.000327
7	0.006639	0.000404
8	0.012266	0.000315
9	0.017525	0.000489
10	0.005306	0.000491

Note: The data in this table does not pertain to the modified liner, LCS, and cover configurations described in Sections 4, 5, and 6. This data is from the initial HELP model runs presented in the May 1995 Feasibility Report and is presented here for illustrative purposes only. (Ref. Table 6.7-14 of the Feasibility Report.)

Prepared by: JBK
Checked by: MDF

Table 7.3-3

Maximum Average Annual Percolation Through Liner During Operation Years of Second Stages

Year	Sideslope (in/yr)	Base (in/yr)
1	0.817797	0.000816
2	0.844675	0.000956
3	0.788535	0.000956
4	0.683487	0.000959
5	0.554911	0.000751
6	0.687312	0.000356
7	0.666802	0.000505
8	0.659438	0.000288
9	0.818071	0.000356
10	0.731234	0.000604

Note: The data in this table does not pertain to the modified liner, LCS, and cover configurations described in Sections 4, 5, and 6. This data is from the initial HELP model runs presented in the May 1995 Feasibility Report and is presented here for illustrative purposes only. (Ref. Table 6.7-15 of the Feasibility Report.)

Prepared by: JBK
Checked by: MDF

A second, perhaps more appropriate method is to increase the rainfall data by a factor of two and thus create excess water on the top tailings layer (as described in Section 6.7.4.2 of the Feasibility Report (Foth & Van Dyke, 1995a)). The results of these analyses for the revised TMA design addressed in Addendum No. 3 in terms of averages through the simulation period are reproduced from Table 7.3-1 in Table 7.3-4. Yearly values of percolation through the liner during the operation period based on doubled precipitation for the initial stages and second stages are shown on Tables 7.3-5 and 7.3-6, respectively. These results do not show a diminishing trend of percolation through the liner with time during the operation period. Also, average head on the liner (Table 7.3-4) is similar to peak daily head. It can therefore be concluded that the effects of ponding can be approximated by the technique used, i.e., increased rainfall.

Table 7.3-4

HELP Model Results: Simulation of Ponding

HELP Model ID	Average Annual Totals for Model Duration				Peak Daily Values		
	Water Balance Duration Years	Lateral Drainage/Leachate Collected (in/yr)	Percolation Through Liner (in/yr)	Average Head Across Liner (in)	Lateral Drainage/Leachate Collected (in/day)	Percolation Through Liner (in/day)	Average Head Across Liner (in)
Sideslope (drained) - Initial Stage (Table 7.2-1)	10	14.71240	0	0.006	0.12109	0	0.018
Base - Initial Stage (Table 7.2-2)	10	15.10629	0.00019	1.181	0.09164	0.000001	2.614
Sideslope (drained) - Second Stage (Table 7.2-3)	10	16.13380	0	0.007	0.12907	0	0.020
Sideslope (undrained) - Second Stage (Table 7.2-4)	10	12.48061	0.398868	159.489	0.04525	0.002628	288.000
Base - Second Stage (Table 7.2-5)	10	16.20123	0.00019	1.266	0.09074	0.000001	2.589

Notes:
- Since HELP model output gives only six digits, values less than 0.0000005 are shown as 0.
- HELP Model ID - Soil properties are provided in the respective table listed under the HELP Model ID.

Prepared by: NXP
Checked by: JBK

Table 7.3-5

Percolation from TMA Based on Doubled Precipitation Values During Operation of Initial Stages

	Sideslope		
Year	Areas With Drainage (in/yr)	Areas Without Drainage (in/yr)	Base (in/yr)
1	0	NA	0.000328
2	0	NA	0.000167
3	0	NA	0.000126
4	0	NA	0.000285
5	0	NA	0.000181
6	0	NA	0.000144
7	0	NA	0.000205
8	0	NA	0.000107
9	0	NA	0.000056
10	0	NA	0.000282

Note: Since HELP model output gives only six digits, values less than 0.0000005 are shown as 0.
NA - Not Applicable.

Prepared by: NXP
Checked by: JBK

Table 7.3-6

Percolation from TMA Based on Doubled Precipitation Values During Operation of Second Stages

	Sideslope		
Year	Areas With Drainage (in/yr)	Areas Without Drainage (in/yr)	Base (in/yr)
1	0	0.359808	0.000387
2	0	0.493494	0.000217
3	0	0.491996	0.000168
4	0	0.424182	0.000247
5	0	0.425760	0.000168
6	0	0.501458	0.000160
7	0	0.280386	0.000188
8	0	0.104438	0.000137
9	0	0.311280	0.000107
10	0	0.595868	0.000168

Note: Since HELP model output gives only six digits, values less than 0.0000005 are shown as 0.

Prepared by: NXP
Checked by: JBK

Based on the above results it has been concluded that for the period when the TMA remains open and ponding takes place on top of the tailings, the leachate generation and percolation through the liner should be represented by HELP models using two times the average rainfall data (Tables 7.3-4, 7.3-5 and 7.3-6). After closure of each TMA cell when there will be no ponding and draining of the tailings is occurring, the results from the analyses with normal rainfall are appropriate (Table 7.3-1).

7.3.2 Total Percolation from the TMA

The results of HELP model studies show that the rate of percolation varies with time due to varying operation conditions (i.e. open, closed, geomembrane assumptions, etc.). Also, the rate of percolation varies due to changes in geometry (i.e., sideslope profile, base profile, tailing thickness, etc.). Therefore, in order to estimate the percolation rate through the liner with time, these conditions need to be considered.

Table 7.3-7 shows the estimated rates of percolation assuming the initial stage of a TMA cell will be operative for three years and the second stage for five years, including the consolidation period, before the cell is closed. The values in the table represent the maximum values from Tables 7.3-5 (0-3 yrs) and 7.3-6 (4-8 yrs).

Table 7.3-7

Percolation During Cell Filling

	Rate of Percolation (in/yr)	
	0-3 yrs	4-8 yrs
Sideslope With Geocomposite Drainage	0	0
Sideslopes With No Geocomposite	NA	0.595868
Base Area	0.000328	0.000387

NA - Not Applicable.
Note: Since HELP model output gives only six digits, values less than 0.0000005 are shown as 0.

Prepared by: NXP
Checked by: JWS

For the remaining periods the estimated percolation rates following placement of the cell final cover are as shown in Table 7.3-8.

Table 7.3-8

Post-Closure Percolation

Year from Placement of Cover	Sideslope: Areas With Drainage (in/yr)	Sideslope: Areas Without Drainage (in/yr)	Base (in/yr)
1	<0.0000005	0.217444	0.000171
2	<0.0000005	0.001019	0.000108
3	<0.0000005	0.000324	0.000028
4	<0.0000005	0.000255	0.000024
5	<0.0000005	0.000201	0.000022
6	<0.0000005	0.000160	0.000020
7	<0.0000005	0.000129	0.000018
8-15	<0.0000005	0.000057	0.000013
16-25	<0.0000005	0.000013	0.000009
26-35	<0.0000005	0.000004	0.000003
36-40	<0.0000005	0.000003	0.000002
41-50	<0.0000005	0.000003	0.000004
51-60	<0.0000005	0.000003	0.000004
61-70	<0.0000005	0.000003	0.000004
71-80	<0.0000005	0.000003	0.000004
81-90	<0.0000005	0.000003	0.000004
91-115	<0.0000005	0.000003	0.000004
116-140	<0.0000005	0.000003	0.000004
141	0.000005	0.012844	0.013495
142-165	<0.0000005	0.0001[1]	<0.0000005
166-175	<0.0000005	0.000104[1]	<0.0000005
176-185	<0.0000005	0.000098[1]	<0.0000005
186-195	<0.0000005	0.000117[1]	<0.0000005
196-205	<0.0000005	0.000099[1]	<0.0000005
206-215	<0.0000005	0.000089[1]	<0.0000005
216-220	<0.0000005	0.000137[1]	<0.0000005
221-230	<0.0000005	0.000110[1]	0.000103[1]
231-240	<0.0000005	0.000104[1]	0.000100[1]

[1]Percolation equal to infiltration through cover.

Prepared by: NXP
Checked by: SVD1

7.3.3 Leachate Generated

Tables 7.3-9 and 7.3-10 show leachate quantities for initial and second stage filling of a TMA cell, respectively. The results are shown for the case where ponding is simulated (i.e., two times precipitation). After closure of the unit, the estimated leachate quantities can be obtained from Table 7.3-1.

Table 7.3-9

Lateral Drainage/Leachate Collected[1] Initial Stages

Year	Sideslope (in/yr)		Base (in/yr)
	With Drainage	Without Drainage	
1	18.8906	NA	25.1527
2	14.8502	NA	13.8246
3	18.4672	NA	10.8828
4	13.3185	NA	21.9767
5	13.9488	NA	14.6985
6	16.5821	NA	11.9045
7	9.2977	NA	16.4137
8	7.9084	NA	9.0575
9	17.5957	NA	5.2628
10	16.2648	NA	21.8892

[1]Table based on results from HELP model runs with twice the normal mean monthly precipitation values.
NA = Not Applicable.

Prepared by: NXP
Checked by: JBK

Table 7.3-10

Lateral Drainage/Leachate Collected[1]
Second Stages

Year	Sideslope (in/yr) With Drainage	Sideslope (in/yr) Without Drainage	Base (in/yr)
1	30.7883	12.2066	29.3055
2	17.2812	13.4924	18.1662
3	11.7274	13.4363	14.5194
4	22.0404	13.0545	20.0920
5	14.3285	13.0280	14.4667
6	13.2855	13.5308	13.9150
7	16.1233	12.0379	16.0039
8	9.3325	9.7972	12.0786
9	5.3529	10.0708	9.7517
10	21.0778	14.1515	13.7133

[1]Table based on results from HELP model runs with twice the normal mean monthly precipitation values.

Prepared by: NXP
Checked by: JBK

The estimated leachate production rates for the initial and second stages shown in Tables 7.3-9 and 7.3-10 are not uniform, indicating that equilibrium has not been reached. To be conservative, the highest values should be used to estimate leachate quantities for sump and pump sizing. Accordingly, for the initial stage use 18.9 in/yr for sideslope areas and 25.2 in/yr for the base. For the second stage use 30.8 in/yr for sideslope with geocomposite drainage, 14.2 in/yr for areas without geocomposite, and 29.3 in/yr for the base.

7.4 HELP Model Input Parameters

As described in Section 6.7.2 of the Feasibility Report (Foth & Van Dyke 1995a), HELP model inputs can be grouped into the following three categories:

- Weather data;
- Soil data; and
- Design data.

Sections 6.7.2.1 through 6.7.2.5 of the Feasibility Report (Foth & Van Dyke, 1995a) describe how weather data inputs were arrived at. Such information should be considered to be of the highest caliber since it represents actual recorded data.

Section 6.7.3 and subsections 6.7.3.1 through 6.7.3.2.3 of the Feasibility Report (Foth & Van Dyke, 1995a) describe how the soil data inputs were obtained. The properties of the tailings, which may impact the rate at which leachate is collected and may also impact the estimated site percolation rates have been obtained from laboratory tests.

The sensitivity analyses completed by Peyton and Schroeder (1990) and Helmy Emam (1995) suggest that the quantity of percolation from a site will be impacted most by the hydraulic conductivities of the barrier layer (composite liner) and the drainage layer above it. For the Crandon Project, both of these items are "specified parameters". In other words, the values used in the HELP model runs are those which are specified in the design process and which will be verified in the field during construction. It should also be noted that CMC is proposing to perform post-installation leak testing of the geomembrane. This step, not a customary part of solid waste landfill construction, has been proposed as a result of CMC's recognition of the importance of achieving the effective hydraulic conductivity of the composite liner as part of the construction process. Post-installation leak testing will also help to verify the validity of the geomembrane manufacturing and installation defects quality control data used in the HELP model runs.

In view of the above discussion, CMC believes that the HELP model analyses completed for the TMA have been performed using scientifically supportable input data resulting in defensible output.

7.5 Verification of Percolation Through the TMA Liner

7.5.1 Background

During a review meeting, WDNR requested that CMC submit a comparison between the estimated percolation from the site using HELP model runs and those obtained using the Giroud-Bonaparte equations. Since the HELP model uses the Giroud-Bonaparte equations to characterize the flow through a small, albeit important, part of the material profile, the two methods are not entirely independent. The two methods are different in that while the Giroud-Bonaparte equations provide an estimated percolation rate for a given head on the liner for one set of values (i.e., sizes and distribution of defects of the geomembrane, membrane-substrate contact conditions, hydraulic conductivity and thickness of the underlying soil), the HELP model computes the head on the liner, percolation through the liner and lateral drainage simultaneously on a daily basis based on a water budget analysis. The output from the HELP model run shows the annual/monthly average head on the liner as well as annual/monthly percolation from the site. The output also gives the peak daily average head on the liner and peak percolation rate from the site. Thus, to compare the two methods, either the peak daily or the annual average percolation rates should be used.

7.5.2 Comparison of Average Annual Percolation

The percolation from the site is used as input into the project's solute transport model to evaluate compliance. Since the average annual values are used for this purpose, CMC believes the comparison of the estimated annual average percolation from the site using the HELP model and Giroud-Bonaparte equations is appropriate.

To complete this comparison, the average annual head on the liner as computed by the HELP model has been used in the calculations. Also, consistency regarding the sizes and distribution of defects of the geomembrane, membrane-substrate contact conditions, hydraulic conductivity, and thickness of the soil component have been maintained.

For the comparison, the properties of the liner system were fixed, making the "head on the liner" the only variable in the analysis. Different stages of the TMA construction and operation lead to different values of average annual heads, providing the basis for a good comparison of the two methods. The calculations performed are provided in Appendix G. The results of the calculations are summarized in Table 7.5-1 and show the following:

- For the eight cases of construction and operation considered, the range of annual average head was 0.000152 meters (0.006 inches) to 4.051 meters (159.5 inches), thus providing a comparison of calculated percolation rates over an extremely large range of heads.

- In general, the differences between the rates of percolation calculated using the two methods are very small.

- Except under two scenarios where the heads on the liner are very small, the HELP model predicts higher percolation through the liner than those predicted by the Giroud-Bonaparte equations.

- In the two cases where the HELP model predicts smaller percolation rates, the quantity of percolation is extremely small (less than 7.3×10^{-7} in/yr). This translates to less than 0.3 gallons per year from the area of the TMA where these conditions will prevail at any time during the construction and operation of the TMA.

In conclusion, the comparison shows that the results from the two methods are similar and that in all cases, with the exception of very low head conditions, the HELP model estimates are more conservative when compared to the Giroud-Bonaparte equations. For the very low head conditions the difference in the predictions of the two methods is insignificant.

Table 7.5-1

Comparison of HELP Model and Giroud-Bonaparte Equation Results

Case[1]	Head (inches)	Percolation Using Giroud-Bonaparte Equation (in/yr)	Percolation from HELP Model (in/yr)
1. Sideslope with geocomposite; initial stage	0.006	6.33×10^{-7}	3.8×10^{-8}
2. Base; initial stage	1.18	1.14×10^{-4}	1.9×10^{-4}
3. Sideslope with geocomposite; 2nd stage	0.007	7.29×10^{-7}	4.6×10^{-8}
4. Sideslope without geocomposite; 2nd stage	159.5	0.308	0.399
5. Base; 2nd stage	1.27	1.23×10^{-4}	1.9×10^{-4}
6. Base; early post-closure period	0.64	5.57×10^{-5}	1×10^{-4}
7. Base; leachate system shutoff	0.64	5.52×10^{-5}	1×10^{-4}
8. Sideslope without geocomposite; early post-closure period	3.27	4.37×10^{-4}	5.5×10^{-3}

[1] The first item designates location for which the percolation calculation is done. The second item designates the time period in which the calculation is performed. For example, "sideslope without geocomposite; 2nd stage" references that the percolation calculation was completed for the sideslope that by design does not have a drainage layer (geocomposite) and the period when the 2nd stage has been filled with tailings but before the cover is placed.

Prepared by: NXP
Checked by: PAE

8 Surface Water Control System

8.1 Background

This section presents a discussion of the updated surface water management system at the TMA during operations and after closure of the site. The modifications to the TMA facilities footprint discussed in Section 3 above result in the need to make slight changes in the planned surface water management structures.

Figures 6.12-1 through 6.12-5, originally presented in the May 1995 Feasibility Report (Foth & Van Dyke, 1995a), have been updated and Figures 6.12-2a, 6.12-3a, and 6.12-4a have been prepared to show the sequence of TMA stage development, including excavation, construction, and closure of the four TMA cells and their respective staging/borrow areas. These figures also show the locations of soil stockpiles and the planned surface water management system developed to minimize surface water erosion and surface water discharge impacts.

8.2 Stockpile Management and Erosion Control

8.2.1 General

Stockpiling of soils for future use and/or processing will occur during each stage of TMA construction. Removal of soil from stockpiles and/or processing will occur during the closure of each TMA cell. Addendum No. 2 to the Feasibility Report (Foth & Van Dyke, 1996b) established the general principles CMC will follow in the stockpiling of soils. These principles are repeated below along with additional details concerning stockpile management. Till processing activities for the first 24 years of the TMA life will be confined to the construction staging areas shown on Figures 6.12-1 through 6.12-4, and Figures 6.12-2a, 6.12-3a, and 6.12-4a. For the remaining 4 years of TMA operation and the 3 years of final reclamation, till processing activities will be located in the construction staging, stockpile, and borrow area to the north of TMA 2 and TMA 4. As a result, surface water management and erosion control features will be designed and constructed as permanent features. The final design of these features will be completed as part of the development of the project's Plan of Operation.

8.2.2 Principles of Surface Water Management and Erosion Control

The general principles for soil stockpiling as outlined in Addendum No. 2 (Foth & Van Dyke, 1996b) are:

- With the exception of the construction and closure of TMA 4, stockpiles will be located only in areas which have already been disturbed or that will be disturbed by future cell construction. Stockpiling during the construction and closure of TMA 4 will occur in the stockpile area to the north of TMA 4.

- Stockpiles will be located as close as practicable to either the construction staging area or the portion of the site in which the soil will be used.

- Stockpiles of soil for processing will be located in the construction staging area or adjacent to it throughout the first 24 years of TMA site life.

- Stockpiles will be confined to the smallest possible area. The size, shape, and construction of stockpiles will depend on the equipment used by the contractor, the soil type, and other factors.

- The management of surface water and the construction and/or installation of erosion control devices will be the first step in either stockpile construction or borrow area development.

The construction of surface water management features and installation of erosion control devices as the first step in construction or stockpiling activities will result in the minimization of soil erosion and the control of sedimentation in the disturbed area. Following is a list of the principles of erosion and sediment control from the *Wisconsin Construction Site Best Management Practices Handbook* (WDNR, 1989), and a description of what steps CMC will take to comply with these principles.

<u>Diversion of Surface Water Flow from Disturbed Areas</u>

The construction staging area and the soil and aggregate stockpile sites will be surrounded by a diversion berm and ditch system. This diversion berm and ditch system will typically be installed as shown on Figures 8.2-1, 8.2-2, and 8.2-3 and will be constructed around the site. Installation of the diversion berm and ditch will prevent stormwater run-on from entering disturbed areas and stormwater from leaving disturbed areas prior to being routed through a runoff basin. The diversion berm and ditch will be fertilized, seeded, and mulched following construction. Where stormwater flow velocity is high, erosion control devices such as riprap or erosion matting will be provided in the ditch or on the berm.

<u>Managing Overland Run-on</u>

The diversion berms/ditches mentioned above will prevent overland flow from undisturbed areas from entering areas disturbed by soil stockpiling processing or borrow activities. Overland flow will also be managed by locating the site, if possible, in higher areas of the topography so that overland flow is easier to manage.

<u>Trapping Sediment in Channelized Flow</u>

Figures 8.2-1 and 8.2-2 are a plan view and cross section of a typical stockpile area showing soil processing activities in progress. This typical stockpiling plan shows the following features designed to trap sediment originating from disturbed areas:

- rock berm (Figure 8.2-2), silt fence (Figure 8.2-4) or similar erosion control devices installed around all stockpiled soils or aggregates;

- internal drainage ditch to direct contact water to temporary runoff basins which will discharge surface water to the site's permanent surface water management system;

- exterior berm and ditch which directs contact water to temporary runoff basins; and

- temporary runoff basins to which all contact water drains which are designed to remove silt size particles for the 25-year, 24-hour rainfall event.

Establishing Permanent Drainageways

The drainageways around the construction staging area will be constructed as permanent drainageways since these drainageways will exist over approximately 24 years of the TMA cells life for the construction staging area. The drainageways around stockpiles areas will be designed and constructed in the same manner.

Trapping Sediment During Temporary Site Dewatering

Temporary dewatering activities required for the construction of TMA facilities will likely be limited to dewatering of areas of perched water associated with wetlands located within the TMA cell footprint. All water from wetland dewatering activities will be directed to a ditch which drains to a temporary runoff basin.

Preventing Tracking of Soil

Tracking of soil in and out of the stockpile and soil processing areas is minimized by providing a minimum 100-foot length of 2- to 3-inch clear stone at the entrance and exit (Figure 8.2-1). The clear stone will help reduce the quantity of soil tracked into and from the disturbed area.

Stabilizing Stockpile or Cut Areas

Stockpiles, cut areas, or other disturbed areas on which future activities will occur will be seeded with a temporary seed mixture immediately after borrow or stockpiling activities are completed for that year. Prior to seeding, the stockpile or cut slope surface will be "track walked" (i.e., driving a bulldozer up and down the slope to leave a pattern of imprints parallel to slope contours) to create a rough surface. Seed mixtures which conform to the State of Wisconsin Department of Transportation Standard Specification (WisDOT Specification) for road and bridge construction Section 630.3.3.42 (Borrow Pits and Waste Areas) will be applied as follows.

- 60 percent temporary species seeds consisting of oats and perennial rye grass.
- 40 percent permanent species seeds consisting of WisDOT Specification Seed Mixture No. 10.
- The borrow pit mixture listed above will be seeded at a rate of 1.5 pounds per 1,000 square feet (or approximately 65.3 pounds per acre).

Keeping Runoff Velocities Low

The design of both temporary and permanent erosion control facilities has as its goals keeping velocities of flowing water low. The measures proposed to accomplish this are as follows:

- provide riprap at inlets and outlets of temporary runoff basins and culverts, and in areas of the ditch where water velocities and/or volumes are high;
- provide energy dissipation (e.g., rock check dams) where velocities are high; and

- provide erosion control matting or other material to increase erosion resistance in ditches, when required.

Implementing a Sedimentation and Erosion Control Inspection and Maintenance Program

CMC has committed to a TMA operation inspection plan which includes routine inspection of the operating systems of the TMA and its appurtenant structures. The sedimentation and erosion control devices constructed for the TMA, construction staging area, and soil stockpiles are included in this program. Appendix P of the Feasibility Report (Foth & Van Dyke, 1995a) contains a typical assignment operation log for these inspections. This log includes inspection of surface water management and erosion control structures after each significant storm event. The Plan of Operation will include a more detailed inspection log for the facilities proposed for the initial phases.

8.3 Reevaluation of Surface Water Management Structures

Runoff basins for the updated TMA are as shown on Drawing 18. Runoff basins 10 and 11 were relocated to provide additional setback to wetland F15, but their capacity was maintained consistent with the original design. A presentation of the original design for the surface water management structures can be found in Section 6.9 of the Feasibility Report (Foth & Van Dyke, 1995a). Due to the revision of the TMA footprint as represented by Setback Alternative No. 2, the capacity of all surface water structures were reevaluated. The results are presented in Table 8.3-1. From a watershed area basis, the existing runoff basins watersheds did not significantly change as a result of the footprint modification. Since the existing runoff basin watersheds did not change appreciably, it was unnecessary to reanalyze the sizing of surface water management structures because under the original design, they were sized for the 100-year/24-hour storm event to achieve settling of the 10 micron particle size.

8.4 TMA Borrow Area Runoff Basin Design (Runoff Basin 13)

As discussed in Section 8.3, the originally designed runoff basin sizes have been unchanged. To better match the predevelopment overland flow to wetland F15, runoff basin 13 was added to the surface water management system. The addition of this runoff basin will result in those portions of the construction stockpile, borrow and staging areas, which are within the Hemlock Creek watershed, to drain into the Hemlock Creek watershed; and those portions of the construction stockpile/borrow area which are within the wetland F15/Skunk Lake watershed to drain into F15/Skunk Lake watershed. Runoff basin 13 is proposed to be located at the northeast toe of slope of the construction stockpile, borrow, and staging areas and to settle out 10 micron sized particles before runoff discharges to the Hemlock Creek watershed. In contrast to the other runoff basins, runoff basin 13 will include a wide weir outlet to allow the dispersion of runoff over a broader area to minimize erosion. Design calculations for runoff basin 13 are included in Appendix H.

The drainage area associated with runoff basin 13 consists of 12.4 acres of reclaimed upland. This basin will be long and narrow and have a 30-foot wide emergency spillway acting as the outlet. The spillway will be overtopped by 0.1 feet under a 100-year storm event so discharge rates down slope will have insignificant velocities and will not impact the downstream area. Details pertaining to runoff basin design are shown on Drawing 31. An updated summary of TMA runoff basin hydraulics is provided in Table 8.4-1, including the new runoff basin 13.

Table 8.3-1

Comparison of Original and Setback Alternative No. 2 TMA Runoff Basin Drainage Areas

Runoff Basin	Original Drainage Area/Watershed Area (acres)	Revised TMA Footprint Drainage Area (acres)	Percent Change (+/-)
8	73.1	73.6	+0.6%
9	55.4	57.9	+4.5%
10	54.3	53.1	-2.2%
11	83.6	83.1	-0.6%
12	37.8	36.7	-2.9%
13[1]	—	12.4	NA

[1]Basin added due to reconfiguration of the stockpile, borrow and construction staging area north of TMA cells 2 and 4.
NA = Not Applicable.

Prepared by: SRB
Checked by: PAE

Table 8.4-1

Runoff Basin Design Details for 100-Year, 24-Hour Event

Basin No.	Peak Inflow (cfs)	Peak Discharge (cfs)	Maximum Water Elevation (ft)	Storage Volume (ac-ft)	Required[1] Surface Area (sf)	Actual Water[2] Surface Area (sf)
Runoff Basin 8	52	9.8	1,628.7	3.14	19,870	42,690
Runoff Basin 9	45	7.8	1,694.3	2.47	15,915	82,764
Runoff Basin 10	42	9.2	1,650.9	2.12	18,720	35,284
Runoff Basin 11	60	10.5	1,656.7	3.60	21,400	39,204
Runoff Basin 12	31	8.5	1,701.0	1.42	17,300	32,234
Runoff Basin 13	18	3.0	1,623.4	0.53	6,100	14,810

[1] Based on silt-sized (10 micron) particle settling (2,030 sq ft per cfs out).
[2] Water surface area at maximum water level in the pond for the 100-year, 24-hour precipitation event.

Notes:
cfs - cubic feet per second
ac-ft - acre feet
sf - square feet
ft - feet

Prepared by: SRB
Checked by: PAE

9 Updated Waste Quantity Estimate and Waste Rock Placement in the TMA

9.1 General

The Feasibility Report (Foth & Van Dyke, 1995a) contains a detailed discussion of the estimated quantities of waste materials to be placed in the TMA facility. Section 9.2 provides an update to that information.

A planned update to Section 4.8 of CMC's *Mine Permit Application* will present a detailed waste rock management plan for the project. The objective of the plan is to define criteria and procedures that will be used during mine development to segregate waste rock into material that will not produce acid rock drainage and will leach only minute quantities of substances; and material that has the potential to produce acid rock drainage. The former material will be classified as Type I waste rock and the latter as Type II waste rock. The characteristics which distinguish Type I and Type II waste rock are described in the update to Section 3.5.5 of the project's May 1995 EIR.

Most of the Type I waste rock is expected to be generated during the pre-production development of the production shaft, internal ramps, underground maintenance shops, ventilation shafts and raises, and crosscuts to the orebody. Type I waste rock brought to the surface is proposed to be used as construction aggregate, road base and as fill in the fill material layer below the grading layer during final cover placement.

Type II waste rock will be principally generated during the advancement of lateral hangingwall drifts. These development drifts will be mined adjacent to the Crandon formation and will provide a means of access to the orebody through crosscuts. In addition, Type II waste rock will also be generated during development of other pre-production areas during periods when Type I and Type II materials will be mined concurrently. Type II waste rock will be hoisted to surface and temporarily stored in a lined facility north of the plant site. Type II waste rock hoisted to the surface will primarily be used as a construction material (e.g., riprap) within the lined area of the TMA cells. Type II waste rock not hoisted to the surface will be placed in mined out stopes underground and used as backfill.

The waste rock management plan will include a detailed presentation of the quantities of both Type I and Type II waste rock, the sequence of production, their sequence of use, temporary storage location, and temporary storage period. The method of placement in the TMA of Type II waste rock not used for construction purposes is presented in Section 9.3 of this report.

9.2 Updated Waste Quantity Estimate

This section provides an update of the estimate of waste materials projected to be placed in the TMA and an update of the estimated capacity of the TMA cells resulting from their reconfiguration.

Table 4.1-1 of the Feasibility Report (Foth & Van Dyke, 1995a) has been updated and included in this report as Table 9.2-1. Table 9.2-1 contains an updated estimate of the waste quantity expected for placement in the TMA cells. The zinc and copper tailings generation, and laboratory waste generation estimates have remained unchanged from the May 1995 Feasibility

Table 9.2-1

Updated TMA Estimated Waste Quantities

Waste Type	Density (PCF)	Annual (x 10^6) Tons	Generation (x 10^6) cy	Total Generation (x 10^6) Tons	Total Generation (x 10^6) cy
Zinc Tailings	97[1]	0.80	0.61	12.77	9.75
Copper Tailings	97[1]	0.80	0.61	9.63	7.36
Type II Waste Rock[2,3]	111.3	--	--	0.32 to 0.65	0.21 to 0.43
Type I Waste Rock[2,3]	111.3	--	--	0 to 0.61	0 to 0.41
Wastewater Treatment Plant Solids[4]	77.2[5]	--	--	0.10	0.10
Laboratory Waste[6]	NA	--	--	--	--
Demolition Waste	NA[7]			0.23	0.15
Total[8]				23.05 to 23.99	17.57 to 18.20
Reclaim Pond Solids[9]	77.2[5]	--	--	0.27	0.26

[1] 97 pcf (pounds per cubic foot) from SRK, December 1994, assumes subaerial deposition.
[2] Densities from EMC (1985), quantities from Foth & Van Dyke, 1995c.
[3] Current plans are to use all Type I and a portion of Type II waste rock as construction materials. The low end of the range of tons represents these conditions. In the event CMC decides not to use these materials for construction, their entire quantity will be placed in the TMA with the tailings. The upper end of the range of tons represents this condition.
[4] Wastewater treatment plant solids have been estimated at 100 tons for the total site life (CMC, 1996).
[5] Based on solids density for a similar treatment system.
[6] Laboratory waste will be approximately 10.0 tons/year or approximately 280 tons for the total site life.
[7] Density of demolition waste varies depending on the material.
[8] Total does not include contingency.
[9] Reclaim Pond Solids is not included in the waste quantities total as this is fine tailings carryover that has already been accounted for in the tailings quantities.

cy - cubic yards
x 10^6 - million
NA - not applicable
-- - not calculated since volume is very small or waste stream does not occur on an annual basis

Prepared by: REM
Checked by: PAE

Report. The waste quantity changes made in Table 9.2-1 relate to the Type I and Type II waste rock, wastewater treatment solids, and the inclusion of demolition wastes in TMA 4, Stage VIII volumes. Demolition waste expected to be placed in TMA 4, Stage VIII are listed in Section 11.4 of this report.

The design capacity of the reconfigured TMA is 20.57 million cubic yards. Based on the design capacity and the projected loading shown in Table 9.2-1, the contingency for the TMA ranges from 13 to 17 percent, which is similar to the original design.

9.3 Type II Waste Rock Placement in the TMA

During the operation period of the TMA, Type II waste rock will be hoisted to the surface and either used as construction material within the TMA as part of stage development, or placed directly into the TMA as it is hoisted to the surface. This section describes the methods to be used to place Type II waste rock in the TMA if it is not used for construction purposes.

Waste rock placement within the TMA will be performed using the general procedure outlined below regardless of the TMA cell being operated.

1. Type II waste rock will be transferred into trucks at the headframe waste rock bin for transport to the TMA.

2. A pad of waste rock will be developed out into the TMA beginning at the intersection of an outward TMA sidewall with the TMA center berm. Care will be taken to protect TMA sidewall liner and piping systems during pad placement.

3. As Type II waste rock is placed on the pad, it will be moved into and down the pad slope into tailings using a dozer.

4. As tailings are placed, the Type II waste rock will be covered with tailings.

5. As necessary, the pad will be raised using Type I waste rock near the sidewall to maintain the active tailings cell with a maximum 10 percent slope.

10 Earthwork Balance

10.1 General

Section 3 discussed the reconfiguration of the TMA footprint to address WDNR's concerns regarding potential impacts to wetland F15, known as the Bur Oak Swamp. This section discusses the updated earthwork balance for the revised footprint.

10.2 Goals

In the Feasibility Report (Foth & Van Dyke, 1995a), parameters for the earthwork design were as follows:

- Meet the requirements for the waste storage volume.
- Minimize or avoid environmental impacts outside the TMA footprint to the extent practicable by reducing the area or volume required for borrow and stockpiling.
- Maintain an adequate separation from groundwater.
- Balance the earthwork for TMA 1, Stage I to minimize double handling of soils.
- Provide an earthwork balance which will result in borrow outside the TMA footprint required only for final covering of TMA 4.

By adjusting the cell base elevations and berm height, CMC was able to meet the design goals and, at the same time, provide for the storage volume required.

10.3 Phased TMA Construction

Following is an overview of the phased construction of the TMA. Drawing 13, Site Sequencing Plan, can be referred to for details regarding cut/fill and other material quantities.

TMA 1, Stage I will be constructed from the soil excavated in Stage I. TMA 1, Stage I has been designed to provide a cut/fill balance. The soil required for the manufacture of the leachate collection system drainage layer and the P40 till layer will be excavated from Stage I and temporarily stockpiled for processing in the construction staging area as shown in Figure 6.12-1 of this report.

TMA 1, Stage II will be constructed using soil excavated from within the footprint of TMA 2. The soil required for the manufacture of the leachate collection system drainage layer and the P40 till layer will be excavated from TMA 2 and stockpiled for processing in the construction staging area within the TMA 2 footprint as shown in Figure 6.12-2.

TMA 2, Stage III will be constructed by excavating approximately 1.7 million cubic yards from within the TMA 2 footprint as shown on Figure 6.12-2a. A portion of the northwest facing TMA 2, Stage IV berm will be constructed as Stage III is built. Approximately 700,000 cubic yards of excess soil from Stage III excavation will be stockpiled in the stockpile/borrow area directly north of TMA 4. Soil for the manufacture of the drainage layer and P40 till layer will be excavated from the TMA 2 footprint and stockpiled for processing in the construction staging area as shown in Figure 6.12-2a.

TMA 2, Stage IV berms will be constructed by borrowing soil from TMA 3. Excavation within TMA 3 will also provide soil for final cover placement on TMA 1. Figure 6.12-3 shows TMA 1 final grades and the TMA 2, Stage IV constructed grades. Approximately 1.1 million cubic yards of soil materials will be placed in the TMA 1 final cover over the approximate two year closure period. The construction staging area shown in Figure 6.12-3 will continue to be used for stockpiling the soils required to manufacture the drainage layer and P40 till layer.

TMA 3, Stage V will be constructed by completing the excavation of TMA 3 to the required subbase grades and borrowing approximately 300,000 cubic yards from within the TMA 4 footprint. The soils required to manufacture the drainage layer and P40 till layer will be borrowed from TMA 4 and will be stockpiled in the construction staging area. Figure 6.12-3a depicts the proposed embankment configuration of TMA 3, Stage V and shows the location of the construction staging area.

TMA 3, Stage VI berms (Figure 6.12-4) will be constructed by borrowing soil from the TMA 4 footprint. Excavation within TMA 4 will also provide soil for the final covering of TMA 2. Figure 6.12-4 shows TMA 2 final grades which will require a total of approximately 1.4 million cubic yards of soil material to construct the TMA 2 final cover system. The soils required for manufacturing the drainage layer and P40 till layer for the TMA 2 final cover and the Stage VI liner system will be excavated from TMA 4 and stockpiled in the construction staging area for processing.

TMA 4, Stage VII will be constructed by completing the excavation required to bring TMA 4 to the required subbase grade elevation as shown on Figure 6.12-4a, which will result in a need to stockpile approximately 540,000 cubic yards in the stockpile/borrow area north of TMA 4. This stockpile will also serve as a source for the soil required to manufacture the drainage layer and P40 till layer for Stage VII. At this time, and for the remainder of the TMA life, the construction staging area will be located in the same area as the stockpile/borrow area.

TMA 4, Stage VIII berms will be constructed by borrowing soil materials stockpiled north of TMA 4. Soils required to manufacture the drainage and P40 till layers of Stage VIII will also come from this stockpile as shown on Figure 6.12-5. Soil required for TMA 3 final cover construction will come from the stockpile/borrow area. Figure 6.12-5 shows TMA 3 final grades which will require approximately 940,000 cubic yards of soil material including the soils required to manufacture the final cover drainage layer and P40 till layer.

TMA 4 will be closed using approximately 1 million cubic yards taken from the stockpile/borrow area north of TMA 4. Of the 1 million cubic yards, 100,000 cubic yards will be previously stockpiled soil and 900,000 cubic yards will be borrow material. The resulting final proposed configuration showing all four TMA cells closed is shown on Drawing 18 of this report.

10.4 Calculation Method

Drawing 13, Site Sequencing Plan, contains updated cut/fill volumes for site preparation of each stage of TMA construction. To make up for the loss of volume resulting from the relocation of the northwest boundary of TMA 2, the following changes were made to the shapes of the TMA cells:

- The net TMA lined area was decreased by approximately 5 acres due to the relocation of the northwest berm of TMA 2.
- The berms of the TMA cells were raised approximately 3.5 feet.
- The base grades of TMA cells 3 and 4 were raised approximately 3.5 feet.
- The base grade of TMA cell 1 was lowered approximately 7.1 feet.
- The base grade of TMA cell 2 was raised approximately 0.75 feet.

The above changes resulted in a cut and fill balance for TMA 1, Stage I earthwork. As in the original design, a borrow area is required for the soil needed to construct the TMA 4 final cover.

The Feasibility Report (Foth & Van Dyke, 1995a) provided a detailed accounting of the earthwork balance for the TMA construction, operation, and closure as required by NR 182.09, Wis. Admin. Code (Plan of Operation). The purpose of this section is to provide preliminary earthwork balance calculations as required by NR 182.08(2)(e)3, Wis. Admin. Code. The balance provided shows that an adequate volume of earth fill is available to construct the facility and the environmental impacts of the modification made will not result in adverse environmental consequences.

The earthwork quantities (cut/fill) and the quantities of soil and geosynthetic materials required for construction and closure of the TMA cells is provided in the tables on Drawing 13. The material added or modified in conjunction with the redesign of the liner and final cover are also included on Drawing 13. In general, the modifications made in this redesign are as follows:

- A 12-inch P40 till soil layer replaces 12 inches of off-site native clay (i.e., the low permeability soils) in the composite liner and composite cover.
- A geosynthetic clay liner (GCL) has been added to the soil component of the composite liner and final cover. The estimated quantity of GCL is provided in the tables.
- A geocomposite has been added to the sideslope of the initial stage of each cell (i.e., Stages I, III, V, and VII). The estimated quantity of geocomposite is provided in the table on Drawing 13.
- The thickness and quantity of the till protective layer on the base has been modified to 12 inches of unprocessed till overlain by a the 6-inch till filter layer.
- The drainage layer quantities have been recalculated to reflect the modifications made to the leachate collection system.
- As in the Feasibility Report (Foth & Van Dyke, 1995a), waste rock quantities used in construction are not included in the earthwork quantities.

Figure 10.4-1 has been prepared to provide a preliminary earthwork balance for the construction, operation, and closure of TMA 1 through TMA 4. Figure 10.4-1 shows the total cut in comparison with the total fill needs for TMA construction and closure. The processing of drainage layer and P40 till soils is taken into consideration and the quantity of each major soil layer on a per cell basis is presented. Figure 10.4-1 also shows the estimated quantity of the unused grain sizes of the till referred to in the Feasibility Report (Foth & Van Dyke, 1995a) as "by-products". These by-products of the manufacture of drainage layer and P40 till can be blended with excavated till and used as earth fill in applications where no till processing is required. In Figure 10.4-1 these by-products are added back into the unprocessed till, reducing the quantity of unprocessed till required. The quantities estimated in Figure 10.4-1 were an input used to estimate the size required for the stockpile, soil processing and construction staging area shown on Figures 6.12-1 through 6.12-5, including 6.12-2a, 6.12-3a, and 6.12-4a.

10.5 Earthwork Contingency

An earthwork contingency is necessary to account for uncertainties which are difficult to quantify for two components of the earthwork balance. The uncertainties are accounted for as follows:

- A 15 percent contingency has been added to the quantity of soil required to manufacture the drainage layer material for the LCS, and the final cover drainage layer. The contingency is needed primarily to account for the variability in the till properties. A contingency is not required for the manufacture of the P40 till soils since Figure 10.4-1 shows that approximately 199,900 cubic yards of excess fines are available assuming conservative average properties of the Late Wisconsinan Till (refer to Addendum No. 2, Attachment 11 (Foth & Van Dyke, 1996b)).

- A 15 percent contingency has been added to the quantity of unprocessed till required for the final cover grading layer. This contingency is needed to account for the settlement which may take place over the approximate 215 acres of tailings surface which will be covered.

The contingencies mentioned above are calculated in the footnote on Figure 10.4-1 as equalling approximately 450,000 cubic yards. So, if till properties were not as anticipated and/or if excessive settlement occurred in the tailings, an additional 450,000 cubic yards is available as borrow from the stockpile/borrow area north of TMA 4.

The total Type I mine waste rock which could be used in TMA cell construction is approximately 355,000 cubic yards. If Type I waste rock is used in TMA cell construction, the 15 percent contingency could be reduced to approximately 95,000 cubic yards, meaning that if conditions existed which resulted in a need to use the contingency amount, only 95,000 additional cubic yards would have to be borrowed from the stockpile/borrow area north of TMA 4. Including the Type I waste rock in the material balances simply lessens the borrow requirements for the project.

In addition, demolition waste (i.e., railroad ballast and sub-ballast, road gravel, broken concrete from on-site structures, etc.) could be used as part of the grading layer for the final cover for TMA 4. Using these materials in the final cover of TMA 4 would further reduce the need to borrow from the borrow area north of TMA 4.

11 Responses to Remaining Issues

11.1 General

During the process of reviewing the Feasibility Report (Foth & Van Dyke, 1995a), WDNR has periodically requested that items be clarified or additional information be provided. CMC has responded to these requests as they have been raised either through updates or addenda to the Feasibility Report or through written correspondence. In the following discussion CMC is responding to a series of agency requests for clarification and additional information.

11.2 Potential for Burrowing Animals to Impact the Integrity of the Final Cover System

Table 3.9-17 of the EIR (Foth & Van Dyke, 1995b) contains a list of mammal species documented in the study area that do or could burrow in search of food and/or shelter. Since the geomembrane will be covered with 4.5 feet of soil, mammals that generally burrow to a depth of less than 3 feet are not considered as posing a threat to the cap geomembrane. Using this criteria, four mammals found in the study area, the badger (*Taxidea taxus*), striped skunk (*Mephitis mephitis*), woodchuck (*Marmota monax*), and red fox (*Vulpes vulpes*), are reported to burrow greater than 3 feet (Jackson, 1961; Kurta, 1995). Following is a summary of the burrowing habits of these four mammals.

- Badger - Jackson (1961) reported that the badgers brooding nest is 24 to 30 inches in diameter placed 2 to 6 feet underground and can be 8 to 30 feet long. It is generally in a grassy area near the base of a hill or on an elevated plain. Jackson maintains that badgers are exceptional burrowers, however, coarse gravel or stone greatly impedes their progress.

- Striped skunk - The striped skunk typically dens in abandoned woodchuck holes (Kurta, 1995). However, if necessary it will construct a den which can be 18 to 50 feet long, and 3 to 4 feet deep (Jackson, 1961).

- Woodchuck - The woodchuck typically excavates 10- to 12-inch diameter entrance holes which sometimes drop straight down 2 feet into a subterranean system (Kurta, 1995). The burrow system ranges from 15 to 50 feet long, generally parallel to the surface, with multiple branching burrows. The passageways are generally 1 to 2 feet underground but in rare occasions can extend to 5 feet below the surface (Jackson, 1961).

- Red fox - The red fox almost always dens in an abandoned burrow of a woodchuck or other animal (Jackson, 1961). It is usually located in more open pasture land rather than heavy woodland. Dens are frequently located on higher slopes or the summits of hills. The red fox is not a natural burrowing animal, but will sometimes excavate its own burrow (Jackson, 1961). The burrow is generally 15 to 20 feet long, but can reach a length of 40 feet or more, and is at least 3 feet below the surface.

Based on the above information the burrowing habits of the four mammals are typically limited to the upper 4 feet of soil. The authors also mention that coarse layers (i.e., cohesionless deposits) are an impediment to burrowing for some species.

Landeen (1994) studied the impacts of animal intrusion in lysimeters lined with a 28 mil plastic liner. The purpose of this study was to resolve questions concerning potential effects of burrowing mammals relative to water storage in the soil. Three species of prolific burrowers were placed in the caged lysimeters to assess the impacts of burrowing activities on water storage in an arid environment at the Hanford site in the state of Washington. Landeen concluded the following from this field study.

- Geotextile layers were typically penetrated by burrowing activities.
- Data did not indicate any increase in long-term water storage in the soil as a direct result of animal burrowing activities.
- Animals burrowed to the bottom of the lysimeters but did not penetrate the plastic liners.

The relevance of Landeen's study to the Crandon Project is that burrowing activities do not appear to have a significant impact on the water storage in the upper soil horizons. Because of this and since it is unlikely that burrows will penetrate the geomembrane, CMC does not expect any increased potential for infiltration by air or water through the geomembrane.

The TMA final cover system components as proposed by CMC are listed in Section 6.1 of this report. Four-and-one-half feet of soil (54 inches) will overlie the geomembrane, including a granular soil drainage layer which will directly overlie the geomembrane. Based on the information presented above, this multi-layer covered system has a very low potential to be negatively impacted by animal burrowing activities because of the following:

- The four species in the study area with burrowing habits of concern typically do not burrow to depths greater than 4 feet.
- The drainage layer consisting of cohesionless granular soil will be a deterrent to deep burrowing mammals since it is not stable for tunneling.
- The final cover thickness over the geomembrane provides sufficient depth to protect the geomembrane from burrowing activities.
- The HDPE geomembrane is a deterrent to burrowing animals because of its strength, smooth surface, and thickness.
- Because HDPE is manufactured with petroleum based resins and contains no vegetable or animal sugars or starches, it is not known as a material attractive to burrowing animals.

11.3 Placement of Demolition Waste Material in the TMA Cells

Section 6.6.2.3, Final Tailings Deposition for TMA 4, of the Feasibility Report (Foth & Van Dyke, 1995a) indicates that demolition wastes may be used in combination with mine waste rock and/or on-site soils for the final grading layer of TMA 4. WDNR has asked CMC to clarify what types of demolition material will be placed in the TMA, when it will be placed, and at what location within the TMA the material will be placed.

The MPA (Foth & Van Dyke, 1995c) outlines the materials to be removed during site reclamation which will be disposed of on site. As described in the MPA, demolition material will be used to fill shafts at the plant site and may be placed in the TMA. The types and quantities of demolition materials that may be placed in the TMA based on the information presented in Table 5-12 (year 35) of the MPA are shown in Table 11.3-1.

Table 11.3-1

Demolition Waste That May be Placed in the TMA[1]

Source	Approximate Cubic Yards
Plant Site: Item 1a	
Stone Base	43,000
Concrete Rubble	12,400
Plant Site: Item 1b	
Soils from Lined Areas	52,000
Railroad Ballast and Subballast	6,600
Gravel Base Course	7,500
Tailings Pipeline: Item 2	
Lined Ditch Materials	8,600
Railroad Spur Line: Item 3	
Railroad Ballast and Subballast	16,000
Total	146,000

[1]Data from MPA Table 5-12 (Foth & Van Dyke, 1995c).

Prepared by: PAE
Checked by: JWS

If other demolition materials such as building rubble were available for placement in the TMA 4 fill material layer and it was decided they would be place there, they would need to meet the following requirements:

- be non-putrescible and non-organic in nature; and
- be free of sharp objects such as wire mesh or reinforcing steel.

The demolition material listed above will be used in lieu of or in conjunction with general earth fill in the grading layer above the tailings and below the final cover of TMA 4. Demolition material will be placed in the TMA as described in Section 6.6.2.2 of the Feasibility Report (Foth & Van Dyke, 1995a), by filling from the outside of the cell toward the inside to create a stable surface on which on-site soil can be placed. It is expected that the demolition material will be covered with a minimum of 3 feet of the soil grading layer. Given this minimum of 3 feet of on-site soil, the likelihood that the angular waste rock or demolition will come in close proximity to the soil component or geomembrane of the composite cap is very low.

When constructing the TMA grading layer, the demolition waste will be placed first. Once a stable surface over TMA 4 is obtained, the on-site soil grading layer will be placed and shaped to the 2 percent minimum slope required for the final cover to the approximate elevation of the bottom of the P40 till layer. The cell will be allowed to settle until the next construction season at which time the remainder of the final cover layers can be installed. As stated in the Feasibility Report (Foth & Van Dyke, 1995a), 4 to 6 feet of settlement is expected in the first 12 months after tailings placement with larger settlements likely occurring near the center of the cell. Any settlement areas will be returned to the correct grade prior to the placement of the composite cap.

11.4 Laboratory Waste Management

During the mine and mill operation period assay and metallurgical testing will be performed by CMC at an on-site laboratory. Any RCRA regulated wastes generated in the laboratory will be collected in a separate waste system, recovered and removed from the facility by a qualified contractor in compliance with RCRA requirements. Non RCRA regulated wastes generated from this testing will be handled as described below.

The assay laboratory is primarily a wet chemistry laboratory focusing on metal digestion analysis. Acid solutions generated during these analyses will be discharged to an acid neutralization basin prior to discharge to the tailings pump box for transport to the TMA.

The second laboratory is a metallurgical laboratory where geological and metallurgical samples are prepared for analysis. Waste produced from this laboratory will be collected and removed and sent where practicable to the SAG mill feed circuit. These quantities are relatively small and can be recycled in the circuit. This laboratory will also have an area where small bench scale testing can be performed on the flotation circuit. Waste produced from this area will consist principally of slurried ore and reagents. These wastes will be discharged to the tailings pump box for transport to the TMA.

Non RCRA regulated liquid laboratory wastes from mill process test programs will be discharged as sink water. These discharges will be pumped directly to the tailings pump box or will be collected at the laboratories in separate holding tanks and periodically pumped into the tailings pump box for transport to the TMA.

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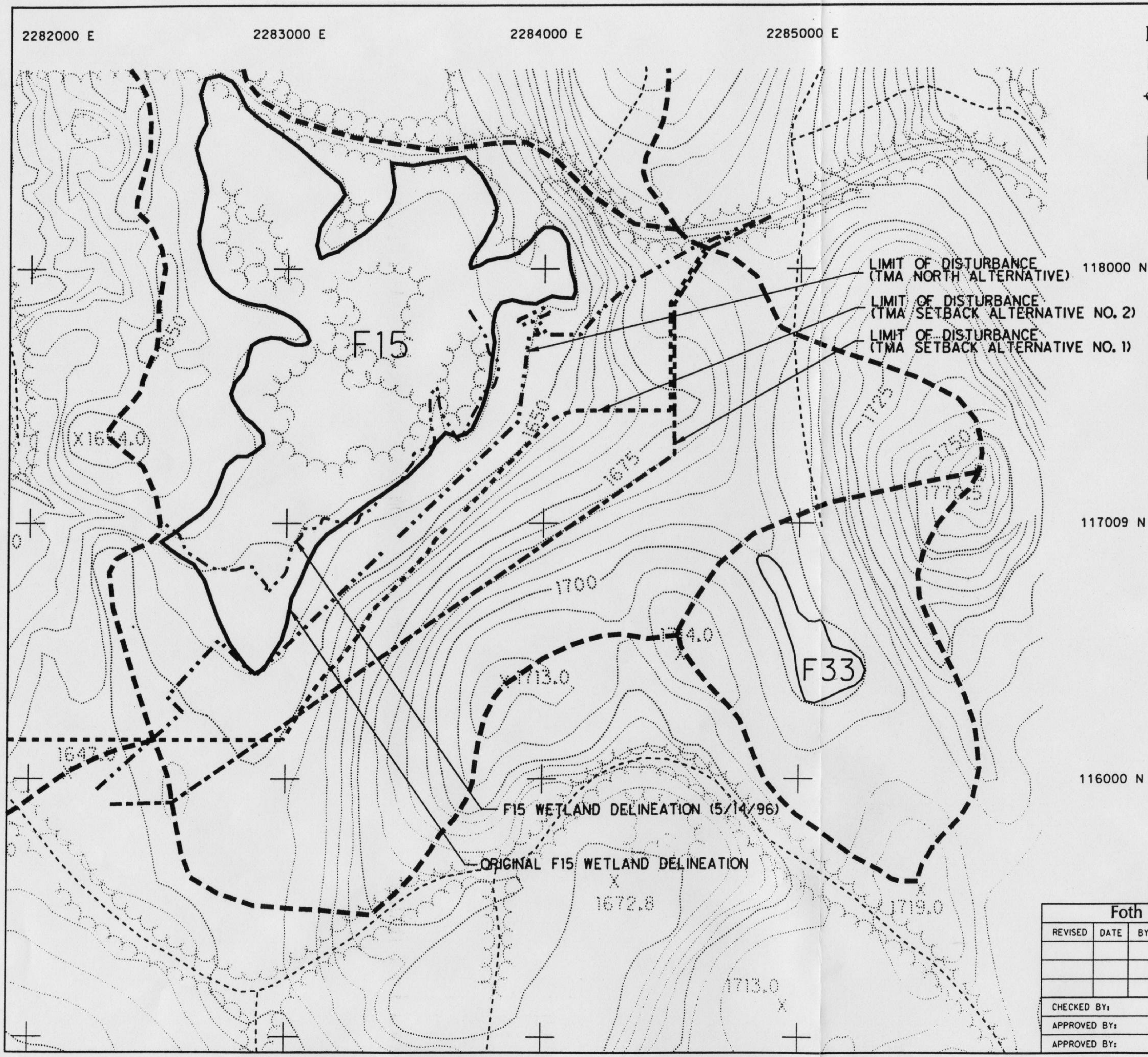

LEGEND

- STREAMS
- EXISTING ROAD
- 1675 EXISTING CONTOUR
- 1692.0 SPOT ELEVATION
- F15 WETLAND DELINEATION (ORIGINAL)
- F15 WETLAND DELINEATION (5/14/96)
- WATERSHED DELINEATION

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.

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REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE:	JAN. '97
APPROVED BY:	PAE	DATE:	JAN. '97
APPROVED BY:	GWS	DATE:	JAN. '97

Crandon Mining Company

FIGURE 3.1-1
TMA SETBACK ALTERNATIVES

Scale: 0 200' 400' Date: JANUARY, 1997
Prepared By: Foth & Van Dyke By: GAM

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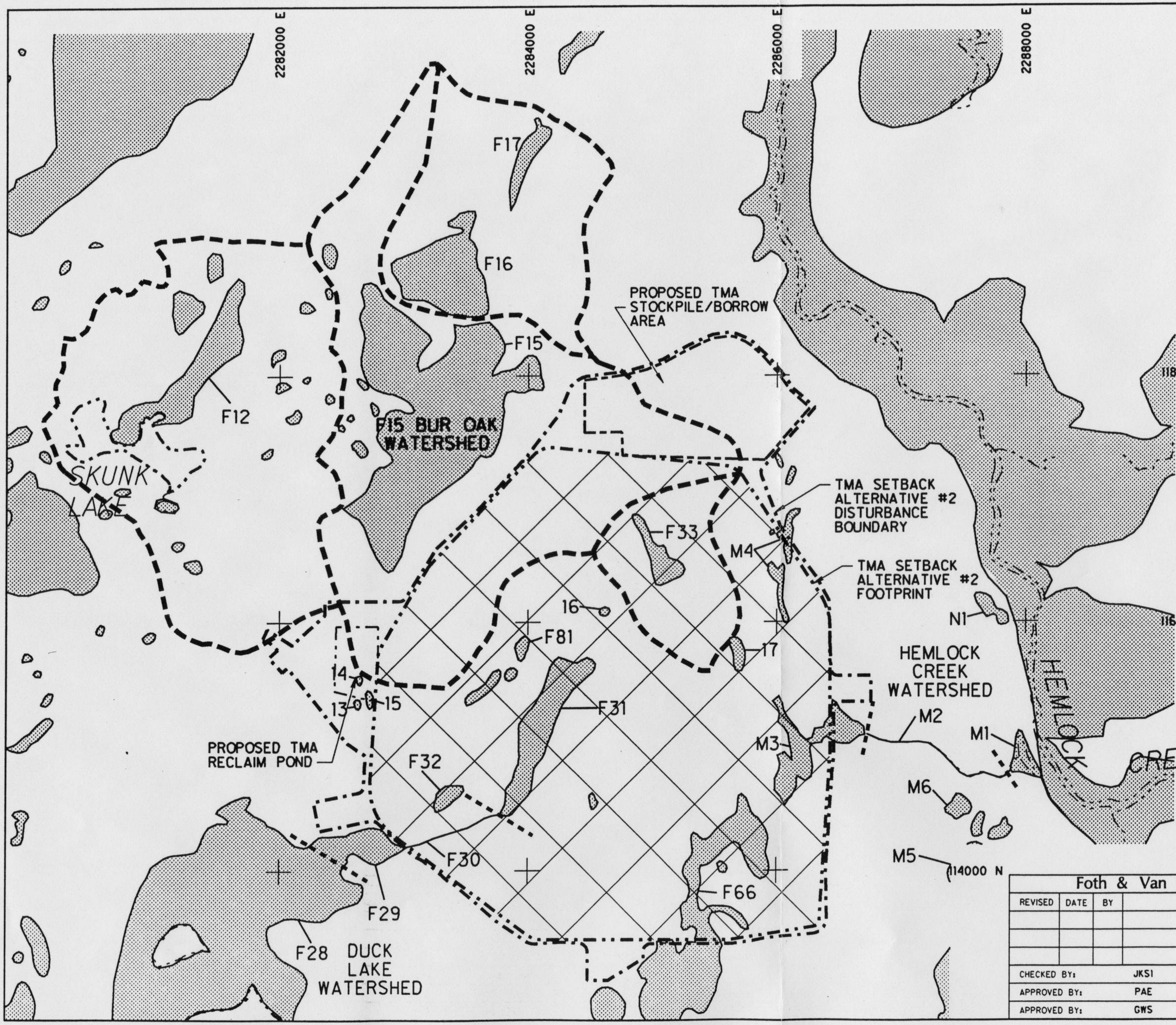

LEGEND

WETLAND

F15 WETLAND ID NUMBER

STREAMS

LAKES

WETLAND DIVIDER

WATERSHED BOUNDARY LINE

F15 SETBACK ALTERNATIVE TMA FOOTPRINT

F15 SETBACK ALTERNATIVE DISTURBANCE BOUNDARY

NOTES:

1. BASE MAP DIGITIZED FROM 1" = 1000' SCALE, MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.

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REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE: JAN, '97
APPROVED BY:	PAE	DATE: JAN, '97
APPROVED BY:	GWS	DATE: JAN, '97

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FIGURE 3.2-1

TMA SETBACK ALTERNATIVE #2

Scale: 0 400' 800' Date: JANUARY, 1997

Prepared By: Foth & Van Dyke By: GAM

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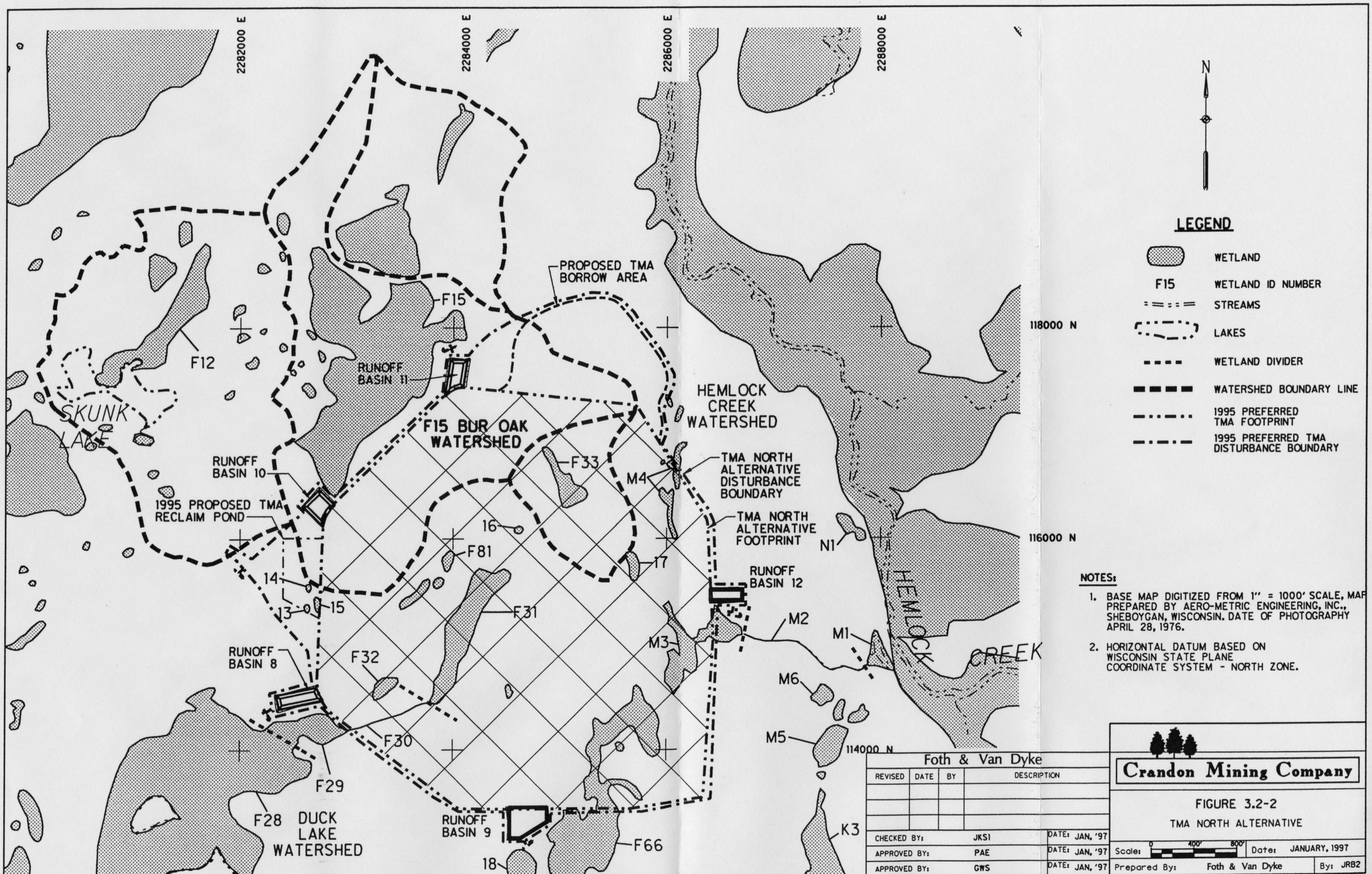

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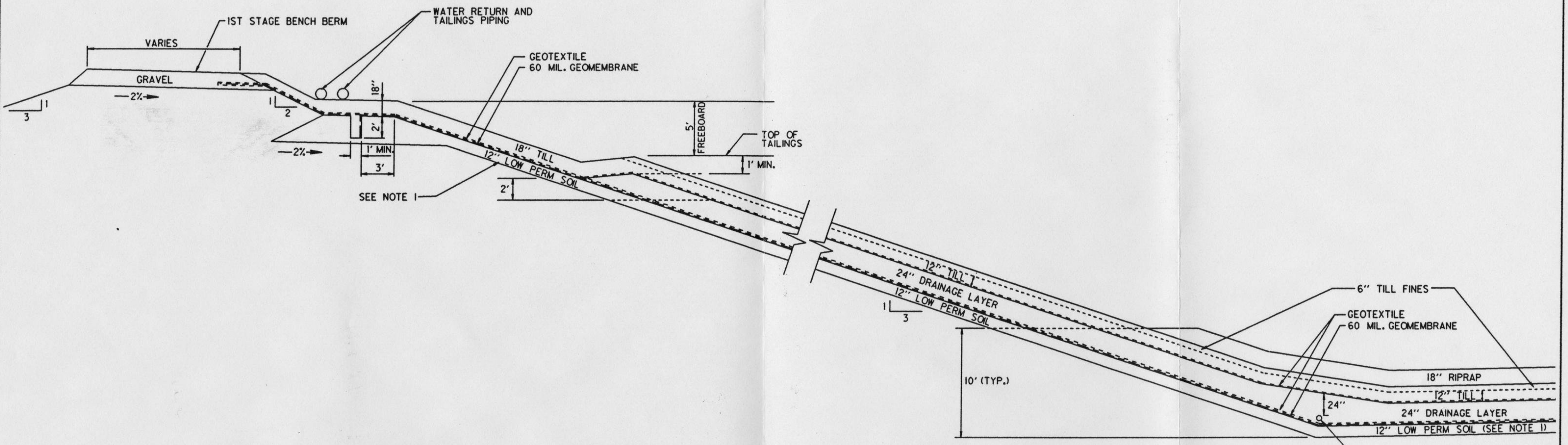

NOTE:

1. LOW PERMEABILITY SOIL CONSISTS OF A GEOSYNTHETIC CLAY LINER (GCL) AND 12 INCHES OF P40 TILL SOIL.

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REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE: JAN, '97
APPROVED BY:	REM	DATE: JAN, '97
APPROVED BY:	JWS	DATE: JAN, '97

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FIGURE 5.3-1

SIDEWALL GRANULAR DRAINAGE LAYER OPTION

Scale: NOT TO SCALE		Date: JANUARY, 1997	
Prepared By:	Foth & Van Dyke		By: JRB2

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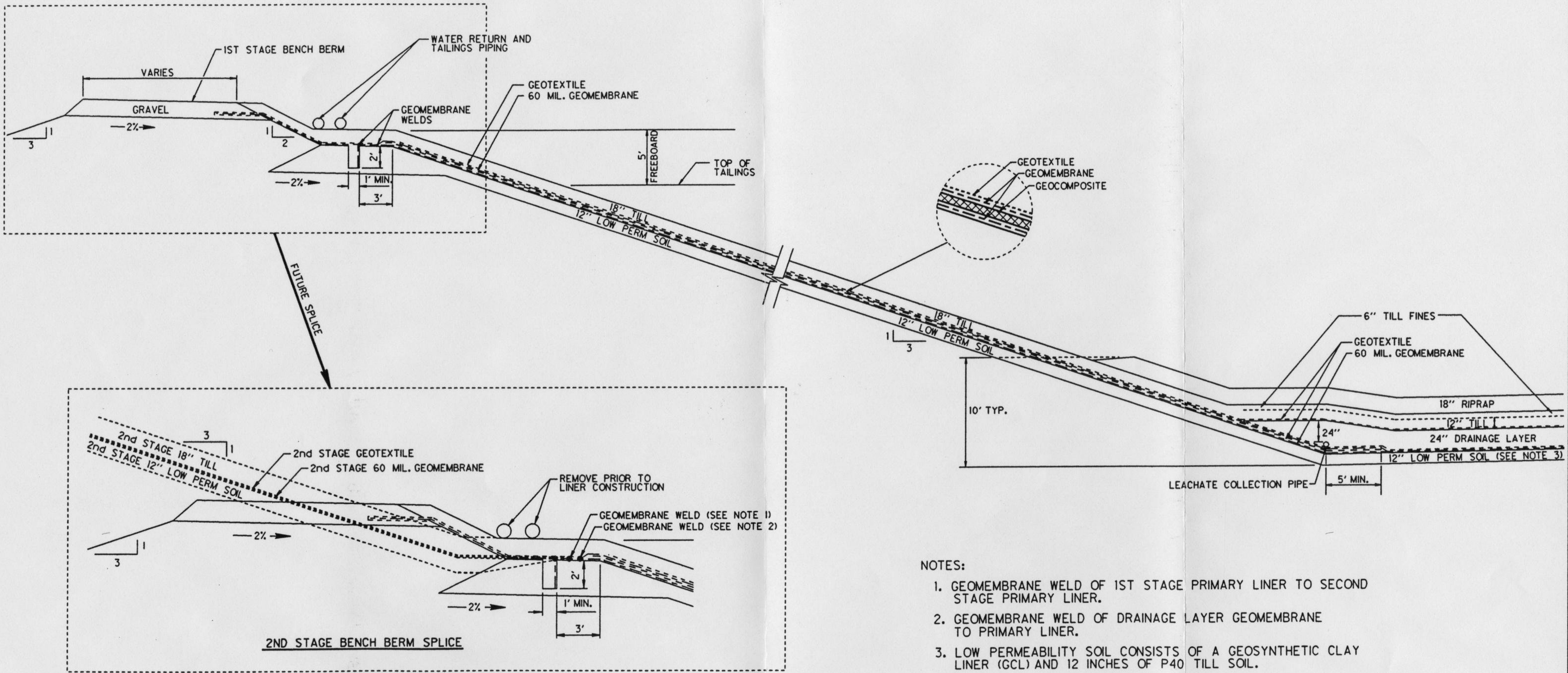

NOTES:

1. GEOMEMBRANE WELD OF 1ST STAGE PRIMARY LINER TO SECOND STAGE PRIMARY LINER.
2. GEOMEMBRANE WELD OF DRAINAGE LAYER GEOMEMBRANE TO PRIMARY LINER.
3. LOW PERMEABILITY SOIL CONSISTS OF A GEOSYNTHETIC CLAY LINER (GCL) AND 12 INCHES OF P40 TILL SOIL.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE: JAN, '97
APPROVED BY:	REM	DATE: JAN, '97
APPROVED BY:	JWS	DATE: JAN, '97

Crandon Mining Company

FIGURE 5.3-2

SIDEWALL GEOMEMBRANE/GEOCOMPOSITE DRAINAGE OPTION

Scale: NOT TO SCALE	Date: JANUARY, 1997		
Prepared By: Foth & Van Dyke			By: JRB2

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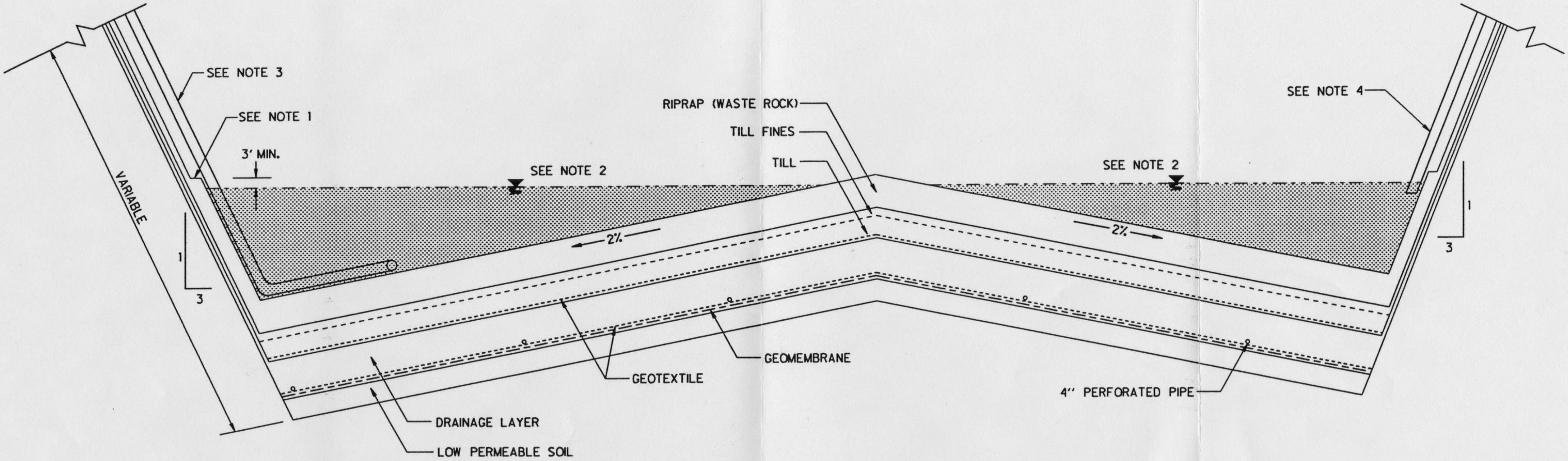
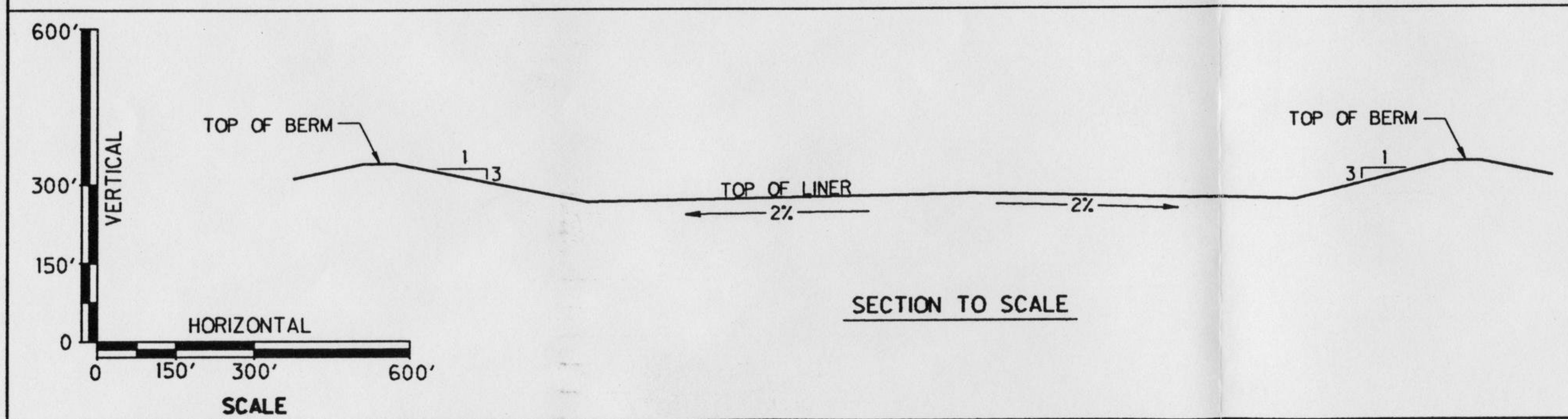


SCALE EXAGGERATION - 10 VERTICAL TO 1 HORIZONTAL

NOTES:

1. RIPRAP TO EXTEND TO A MAXIMUM OF APPROXIMATELY 12' FROM THE BASE.
2. INITIAL POND WATER LEVEL PRIOR TO TAILINGS PLACEMENT.
3. SPIGOT PIPE POSITION DURING INITIAL WATER DISCHARGE.
4. SPIGOT PIPE POSITION FOR START OF TAILING DEPOSITION.
5. LOW PERMEABILITY SOIL CONSISTS OF A GEOSYNTHETIC CLAY LINER (GCL) AND 12 INCHES OF P40 TILL SOIL.



SECTION TO SCALE

Foth & Van Dyke

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CHECKED BY:	JKS1	DATE: JAN, '97
APPROVED BY:	REM	DATE: JAN, '97
APPROVED BY:	GWS	DATE: JAN, '97

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FIGURE 5.5-1

INITIAL FILLING OF CELL WITH WATER

Scale: AS SHOWN		Date: JANUARY, 1997	
Prepared By: Foth & Van Dyke			By: JOW

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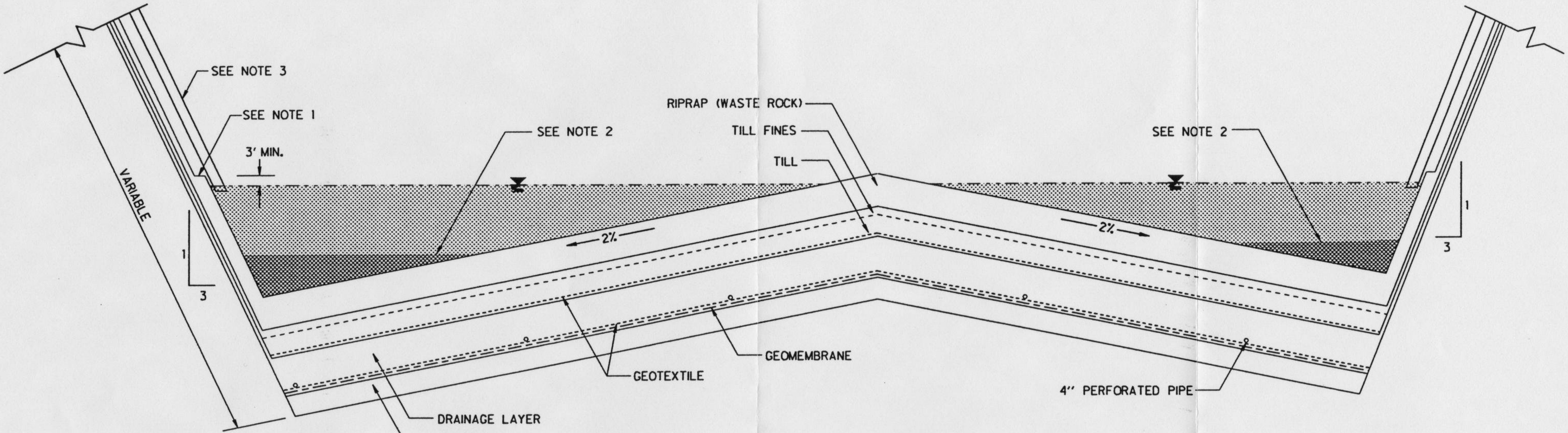
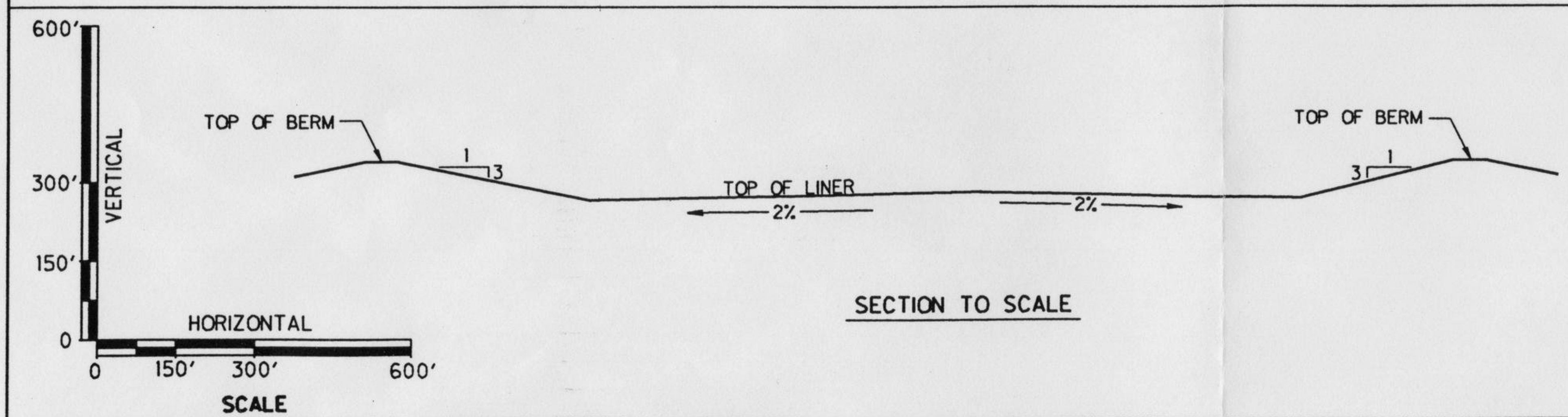


SCALE EXAGGERATION - 10 VERTICAL TO 1 HORIZONTAL

NOTES:

1. RIPRAP TO EXTEND TO A MAXIMUM OF APPROXIMATELY 12' FROM THE BASE.
2. SUBAQUEOUS TAILINGS FILLING WITH SIMULTANEOUS PROCESS WATER DECANT (NOT SHOWN).
3. TAILINGS SPIGOT PIPES TO BE CUT-OFF AS TAILINGS RISE.
4. LOW PERMEABILITY SOIL CONSISTS OF A GEOSYNTHETIC CLAY LINER (GCL) AND 12 INCHES OF P40 TILL SOIL.



SECTION TO SCALE

Foth & Van Dyke

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CHECKED BY:	JKS1	DATE:	JAN, '97
APPROVED BY:	REM	DATE:	JAN, '97
APPROVED BY:	GWS	DATE:	JAN, '97

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FIGURE 5.5-2

TAILINGS DEPOSITION PRIOR TO LEACHATE COLLECTION SYSTEM ACTIVATION

Scale: AS SHOWN	Date: JANUARY, 1997		
Prepared By: Foth & Van Dyke		By: JOW	

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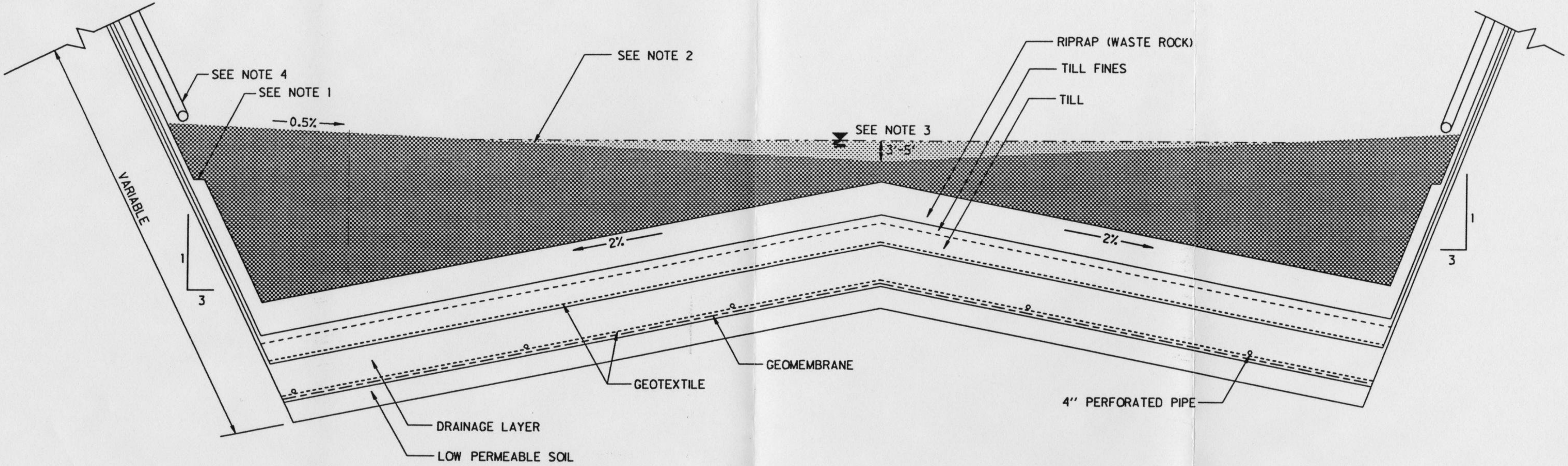
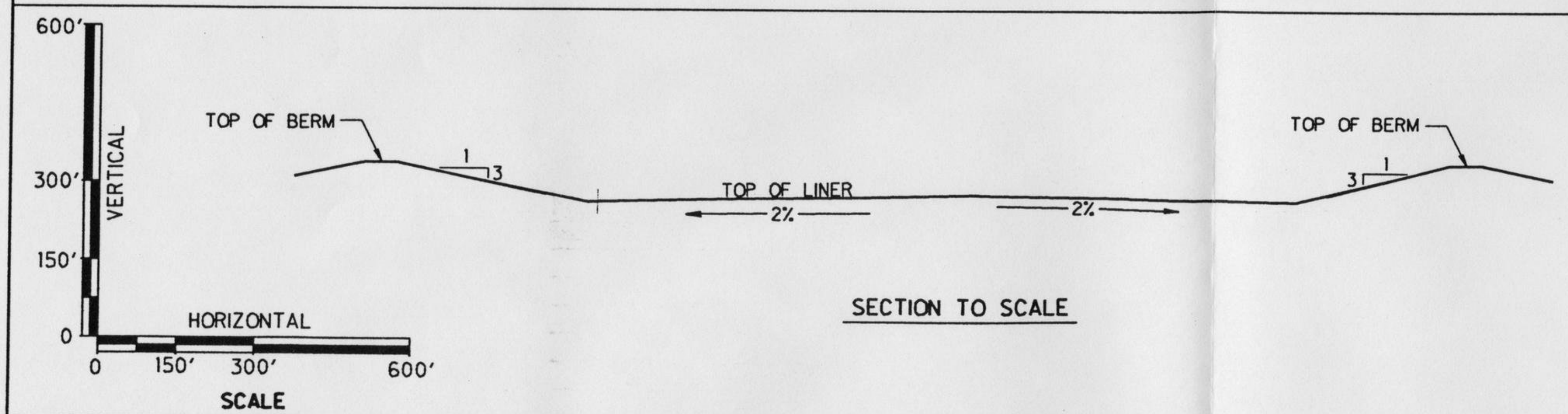


SCALE EXAGGERATION - 10 VERTICAL TO 1 HORIZONTAL

NOTES:

1. RIPRAP TO EXTEND TO A MAXIMUM OF APPROXIMATELY 12' FROM THE BASE.
2. SUBAERIAL TAILINGS PLACEMENT AFTER WATER POND IS FORCED TO THE CENTER OF THE CELL.
3. WATER DEPTH IN CENTER OF CELL IS 3 TO 5 FEET. DECANT STRUCTURE IN PLACE BUT NOT SHOWN.
4. SPIGOT PIPES CUT-OFF AS TAILINGS RISE. TAILINGS DEPOSITED AT APPROXIMATELY 0.5% ON BEACH SLOPES.
5. LOW PERMEABILITY SOIL CONSISTS OF A GEOSYNTHETIC CLAY LINER (GCL) AND 12 INCHES OF P40 TILL SOIL.



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APPROVED BY:	REM	DATE: JAN, '97
APPROVED BY:	GWS	DATE: JAN, '97

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FIGURE 5.5-3

TAILINGS FILLING AT ACTIVATION OF THE LEACHATE COLLECTION SYSTEM

Scale: AS SHOWN		Date: JANUARY, 1997	
Prepared By:	Foth & Van Dyke		By: JOW

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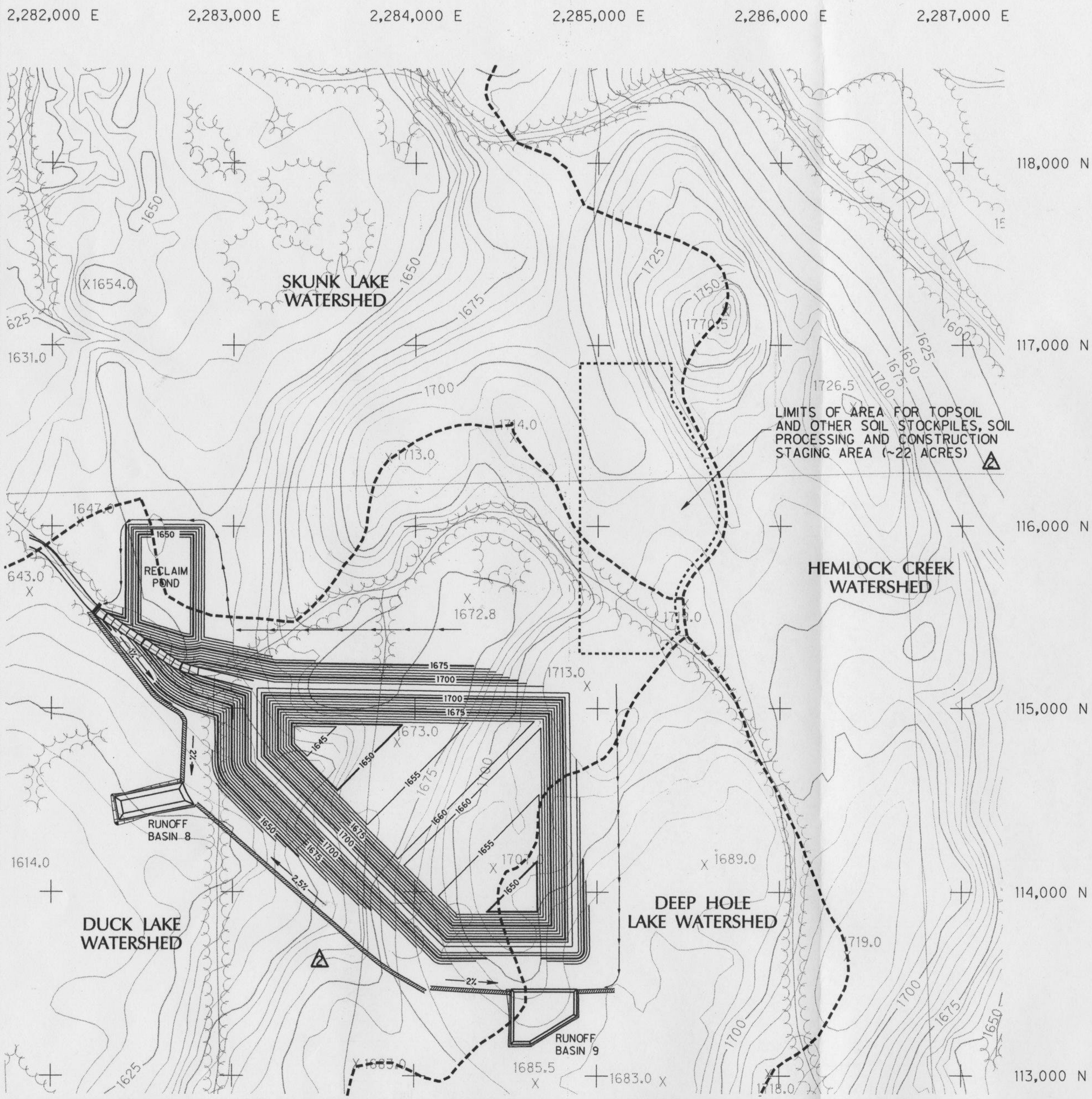

LEGEND

EXISTING ROAD

—1675— EXISTING CONTOUR

X 1692.0 SPOT ELEVATION

SECTION LINE

—1660— PROPOSED CONTOUR

PROPOSED TEMPORARY DRAINAGE FLOW

—2%— PROPOSED DRAINAGE DIVERSION BERM

2% PROPOSED SLOPE DIRECTION

WATERSHED BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.
6. CHANGED CONSTRUCTION STAGING AREA LOCATION AND ADDED STOCKPILE FOR SOIL PROCESSING.
7. CHANGED STOCKPILE, SOIL PROCESSING, AND CONSTRUCTION STAGING AREA LOCATIONS. REVISED BASE GRADES, FINAL GRADES AND SURFACE WATER DRAINAGE FOR ALL TMA CELLS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	3/13/96	REM	SEE NOTES 5 AND 6
2	1/22/97	REM	SEE NOTE 7

CHECKED BY:	JKS1	DATE:	MAY '95
APPROVED BY:	REM	DATE:	MAY '95
APPROVED BY:	GWS	DATE:	MAY '95

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FIGURE 6.12-1
TMA 1
STAGE I CONSTRUCTION

Scale: AS SHOWN	Date: MAY, 1995		
Prepared By: Foth & Van Dyke	By: GAM		

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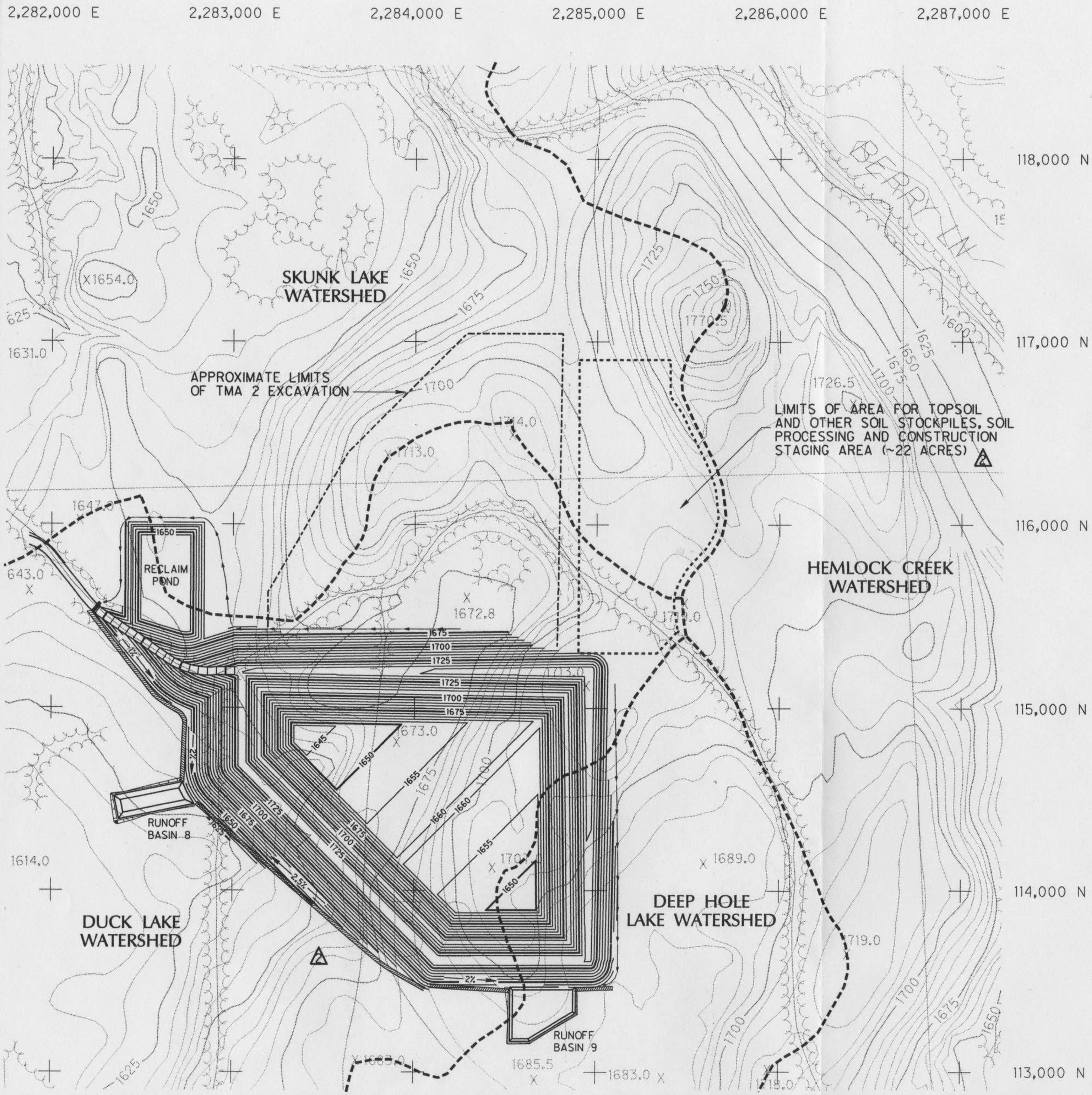

LEGEND

EXISTING ROAD

—1675— EXISTING CONTOUR

X 1692.0 SPOT ELEVATION

SECTION LINE

—1660— PROPOSED CONTOUR

PROPOSED TEMPORARY DRAINAGE FLOW

2% PROPOSED DRAINAGE DIVERSION BERM

2% PROPOSED SLOPE DIRECTION

WATERSHED BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.

2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.

3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.

4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.

5. (1) STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.

6. (1) CHANGED CONSTRUCTION STAGING AREA LOCATION AND ADDED STOCKPILE FOR SOIL PROCESSING.

7. (2) CHANGED STOCKPILE, SOIL PROCESSING, AND CONSTRUCTION STAGING AREA LOCATIONS. REVISED BASE GRADES, FINAL GRADES AND SURFACE WATER DRAINAGE FOR ALL TMA CELLS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	3/13/96	REM	SEE NOTES 5 AND 6
2	1/22/97	REM	SEE NOTE 7

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APPROVED BY:	REM	DATE: MAY '95
APPROVED BY:	GWS	DATE: MAY '95

Crandon Mining Company

FIGURE 6.12-2
TMA 1
STAGE II CONSTRUCTION

Scale: AS SHOWN | Date: MAY, 1995
Prepared By: Foth & Van Dyke | By: GAM

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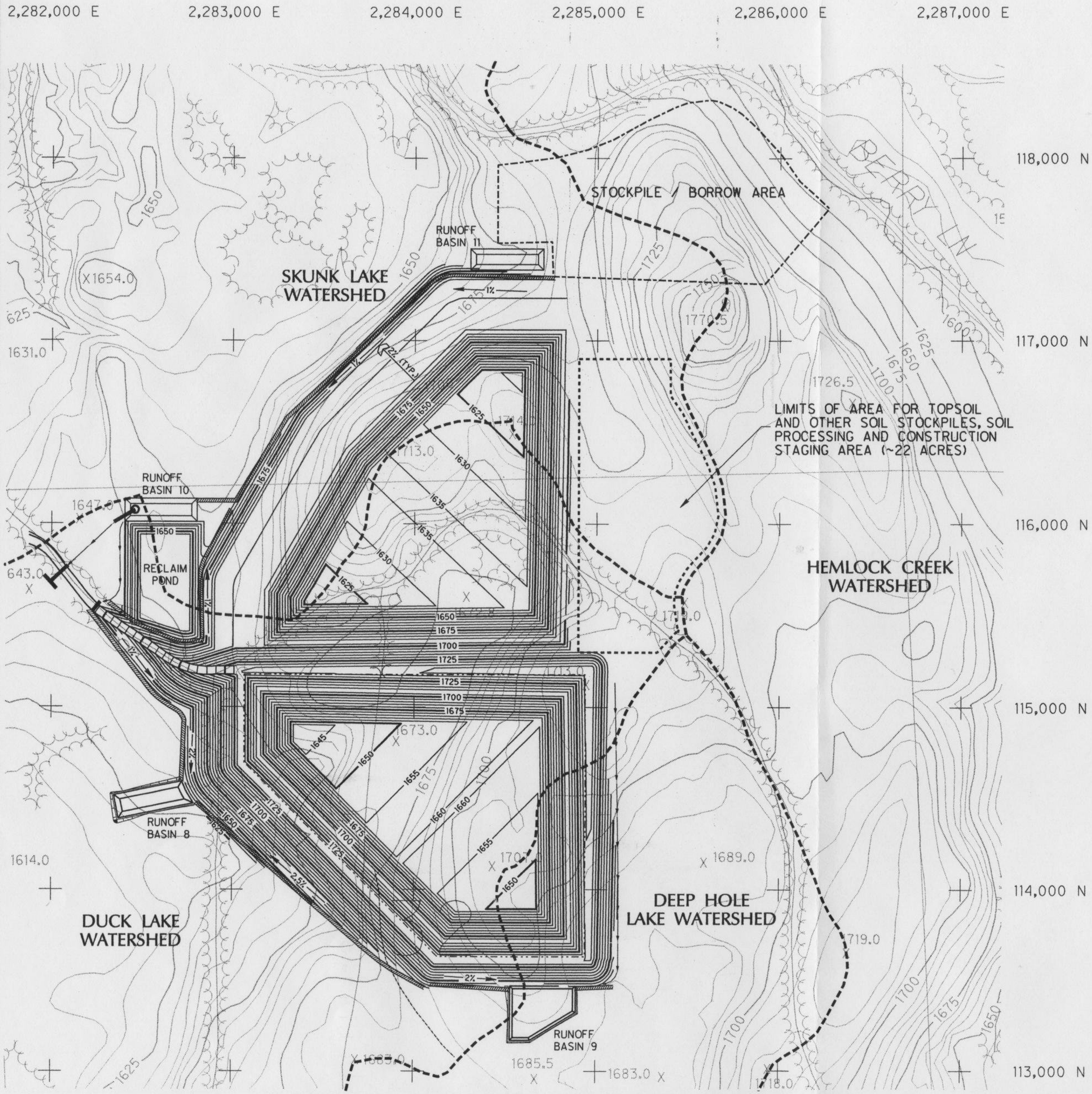

LEGEND

EXISTING ROAD

—1675— EXISTING CONTOUR

x1692.0 SPOT ELEVATION

SECTION LINE

—1660— PROPOSED CONTOUR

PROPOSED TEMPORARY DRAINAGE FLOW

2% PROPOSED DRAINAGE DIVERSION BERM

2% PROPOSED SLOPE DIRECTION

WATERSHED BOUNDARY

N

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

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APPROVED BY:	GWS	DATE: JAN. '97

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FIGURE 6.12-2A
TMA 2
STAGE III CONSTRUCTION

Scale: AS SHOWN		Date: JANUARY, 1997	
Prepared By:	Foth & Van Dyke		By: GAM

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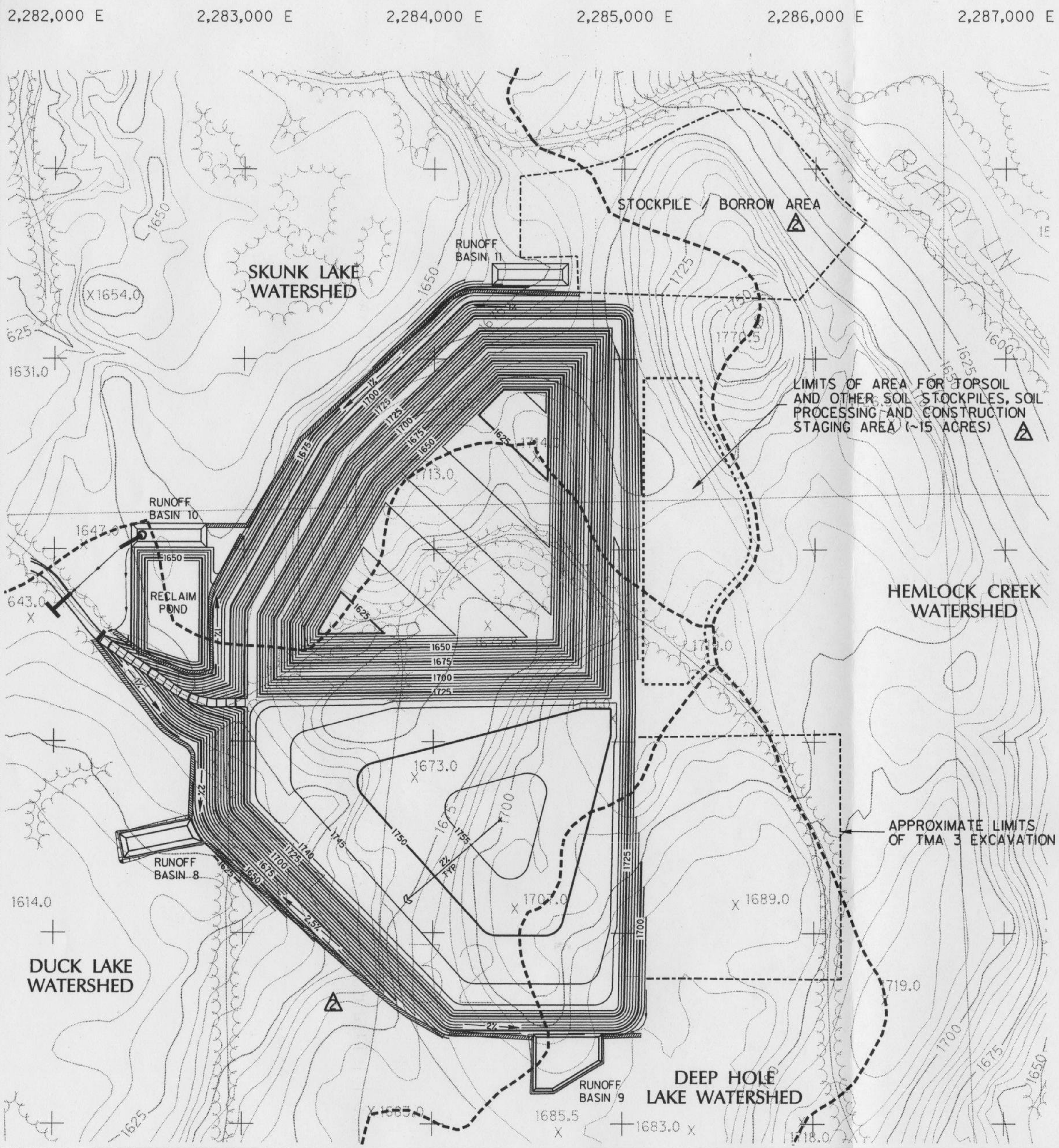

LEGEND

═══	EXISTING ROAD
—1675—	EXISTING CONTOUR
X 1692.0	SPOT ELEVATION
——	SECTION LINE
—1660—	PROPOSED CONTOUR
→	PROPOSED TEMPORARY DRAINAGE FLOW
2% →	PROPOSED DRAINAGE DIVERSION BERM
2% ⟫	PROPOSED SLOPE DIRECTION
- - - -	WATERSHED BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. (1) STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.
6. (1) CHANGED CONSTRUCTION STAGING AREA LOCATION AND ADDED STOCKPILE FOR SOIL PROCESSING.
7. (2) CHANGED STOCKPILE, SOIL PROCESSING, AND CONSTRUCTION STAGING AREA LOCATIONS. REVISED BASE GRADES, FINAL GRADES AND SURFACE WATER DRAINAGE FOR ALL TMA CELLS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

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1	3/13/96	REM	SEE NOTES 5 AND 6
2	1/22/97	REM	SEE NOTE 7

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APPROVED BY:	GWS	DATE: MAY '95

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FIGURE 6.12-3
TMA 2 STAGE IV CONSTRUCTION
TMA 1 FINAL GRADES

Scale: AS SHOWN		Date: MAY, 1995	
Prepared By: Foth & Van Dyke			By: GAM

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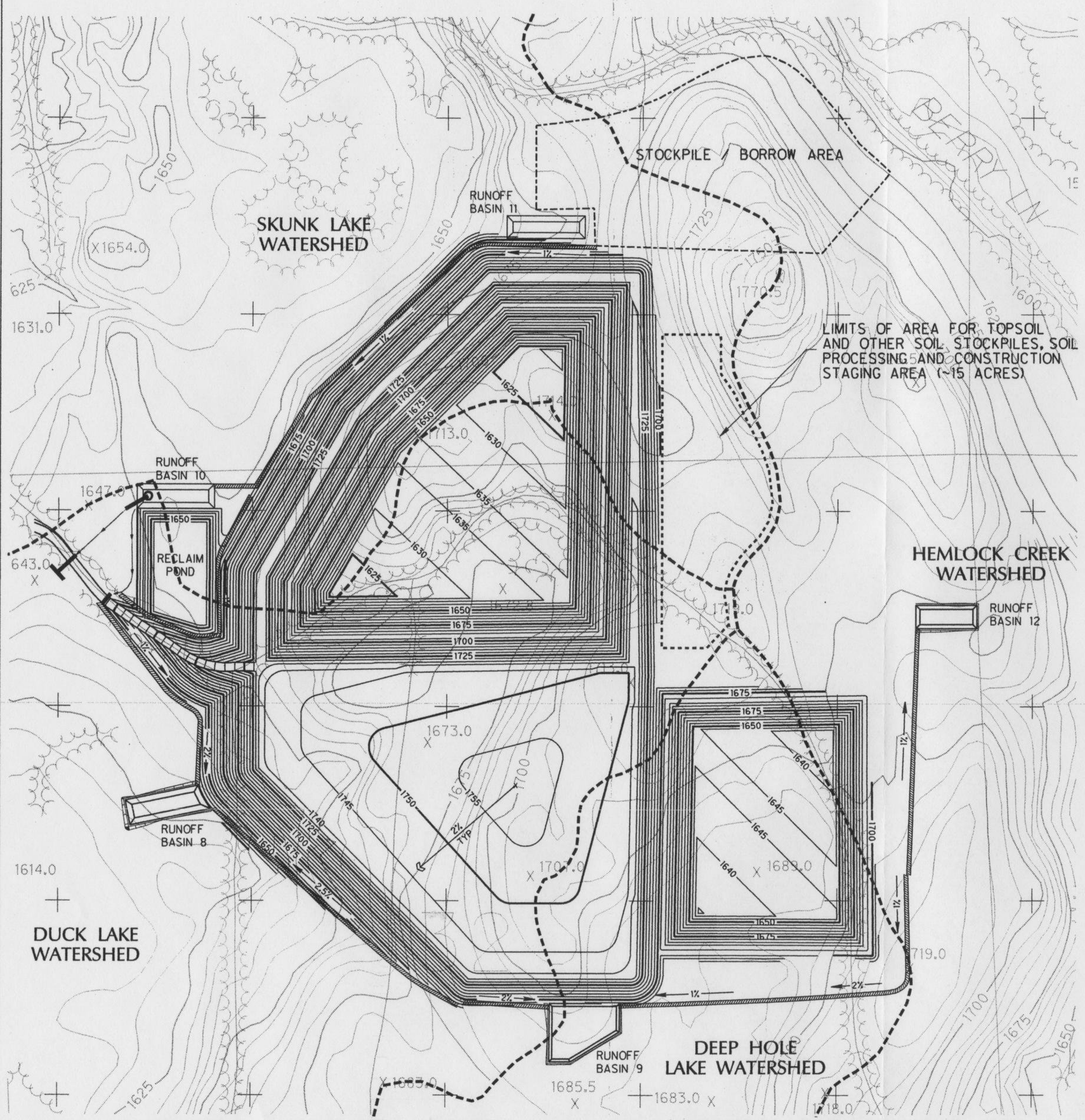
2,282,000 E | 2,283,000 E | 2,284,000 E | 2,285,000 E | 2,286,000 E | 2,287,000 E



LEGEND

	EXISTING ROAD
—1675—	EXISTING CONTOUR
x1692.0	SPOT ELEVATION
	SECTION LINE
—1660—	PROPOSED CONTOUR
	PROPOSED TEMPORARY DRAINAGE FLOW
2%	PROPOSED DRAINAGE DIVERSION BERM
2%	PROPOSED SLOPE DIRECTION
	WATERSHED BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
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3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

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APPROVED BY:	GWS	DATE:	JAN. '97

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FIGURE 6.12-3A
TMA 3
STAGE V CONSTRUCTION

Scale: AS SHOWN	Date: JANUARY, 1997		
Prepared By: Foth & Van Dyke	By: GAM		

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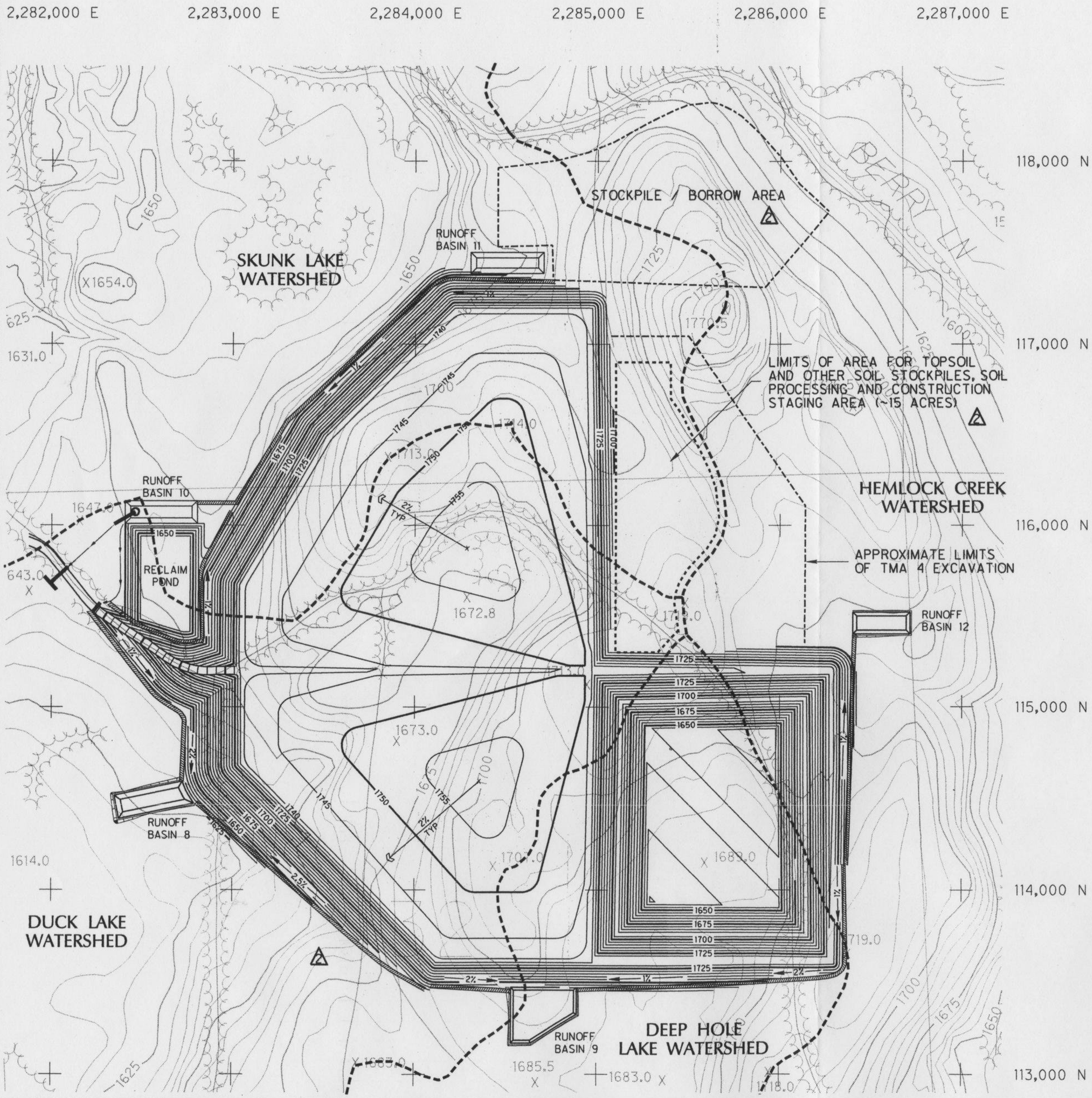

LEGEND

- EXISTING ROAD
- —1675— EXISTING CONTOUR
- X 1692.0 SPOT ELEVATION
- SECTION LINE
- —1660— PROPOSED CONTOUR
- PROPOSED TEMPORARY DRAINAGE FLOW
- PROPOSED DRAINAGE DIVERSION BERM
- 2% PROPOSED SLOPE DIRECTION
- WATERSHED BOUNDARY

NOTES:

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3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. (1) STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.
6. (1) CHANGED CONSTRUCTION STAGING AREA LOCATION AND ADDED STOCKPILE FOR SOIL PROCESSING.
7. (2) CHANGED STOCKPILE, SOIL PROCESSING, AND CONSTRUCTION STAGING AREA LOCATIONS. REVISED BASE GRADES, FINAL GRADES AND SURFACE WATER DRAINAGE FOR ALL TMA CELLS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

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1	3/13/96	REM	SEE NOTES 5 AND 6
2	1/22/97	REM	SEE NOTE 7

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APPROVED BY:	REM	DATE:	MAY '95
APPROVED BY:	GWS	DATE:	MAY '95

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FIGURE 6.12-4
TMA 3 STAGE VI CONSTRUCTION
TMA 2 FINAL GRADES

Scale:	AS SHOWN	Date:	MAY, 1995
Prepared By:	Foth & Van Dyke	By:	GAM

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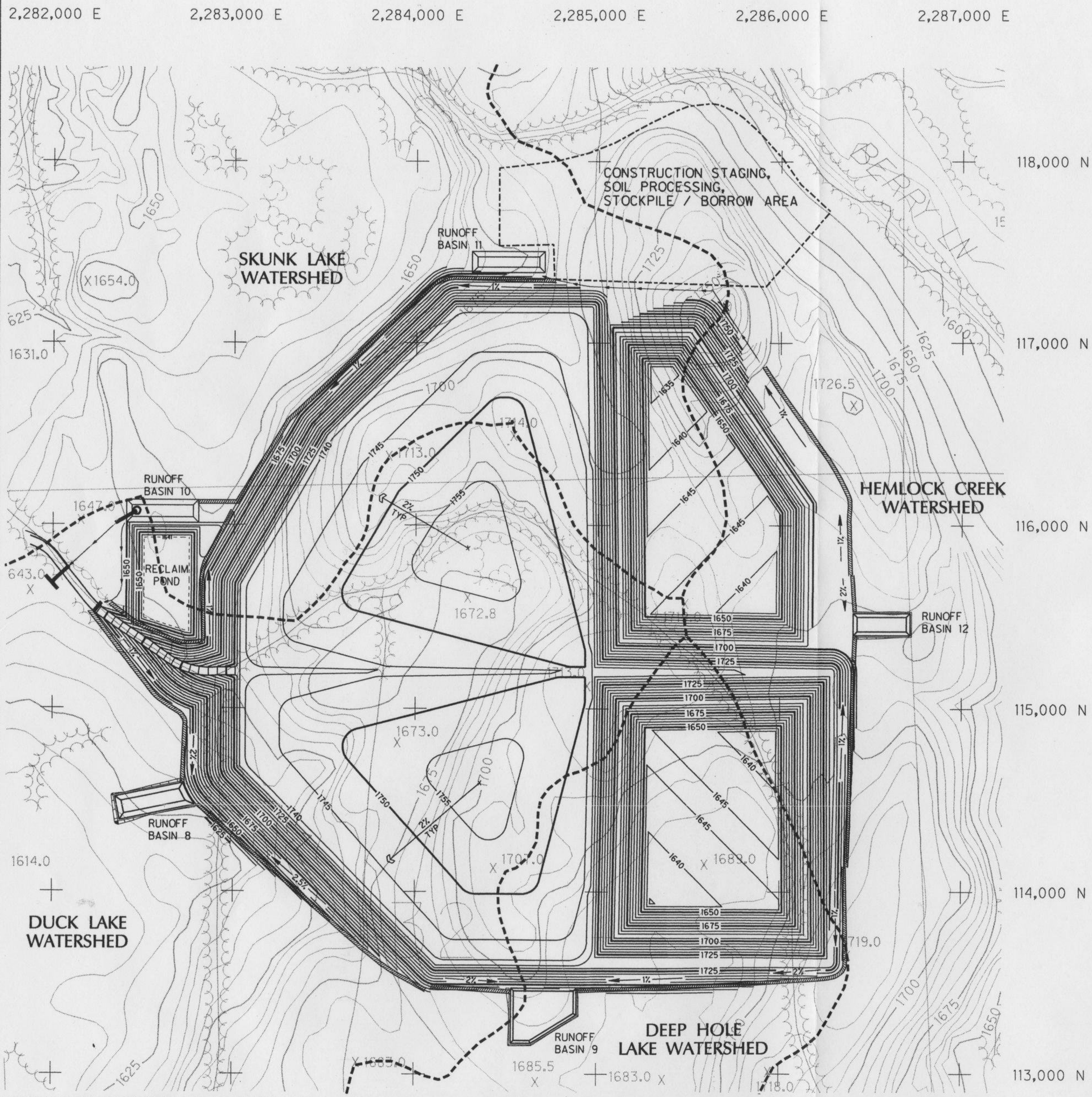

LEGEND

- EXISTING ROAD
- —1675— EXISTING CONTOUR
- X 1692.0 SPOT ELEVATION
- SECTION LINE
- —1660— PROPOSED CONTOUR
- PROPOSED TEMPORARY DRAINAGE FLOW
- 2% PROPOSED DRAINAGE DIVERSION BERM
- 2% PROPOSED SLOPE DIRECTION
- WATERSHED BOUNDARY

NOTES:

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2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

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CHECKED BY:	JKS1	DATE: JAN. '97
APPROVED BY:	REM	DATE: JAN. '97
APPROVED BY:	GWS	DATE: JAN. '97

Crandon Mining Company

FIGURE 6.12-4A
TMA 4
STAGE VII CONSTRUCTION

Scale: AS SHOWN	Date: JANUARY, 1997
Prepared By: Foth & Van Dyke	By: GAM

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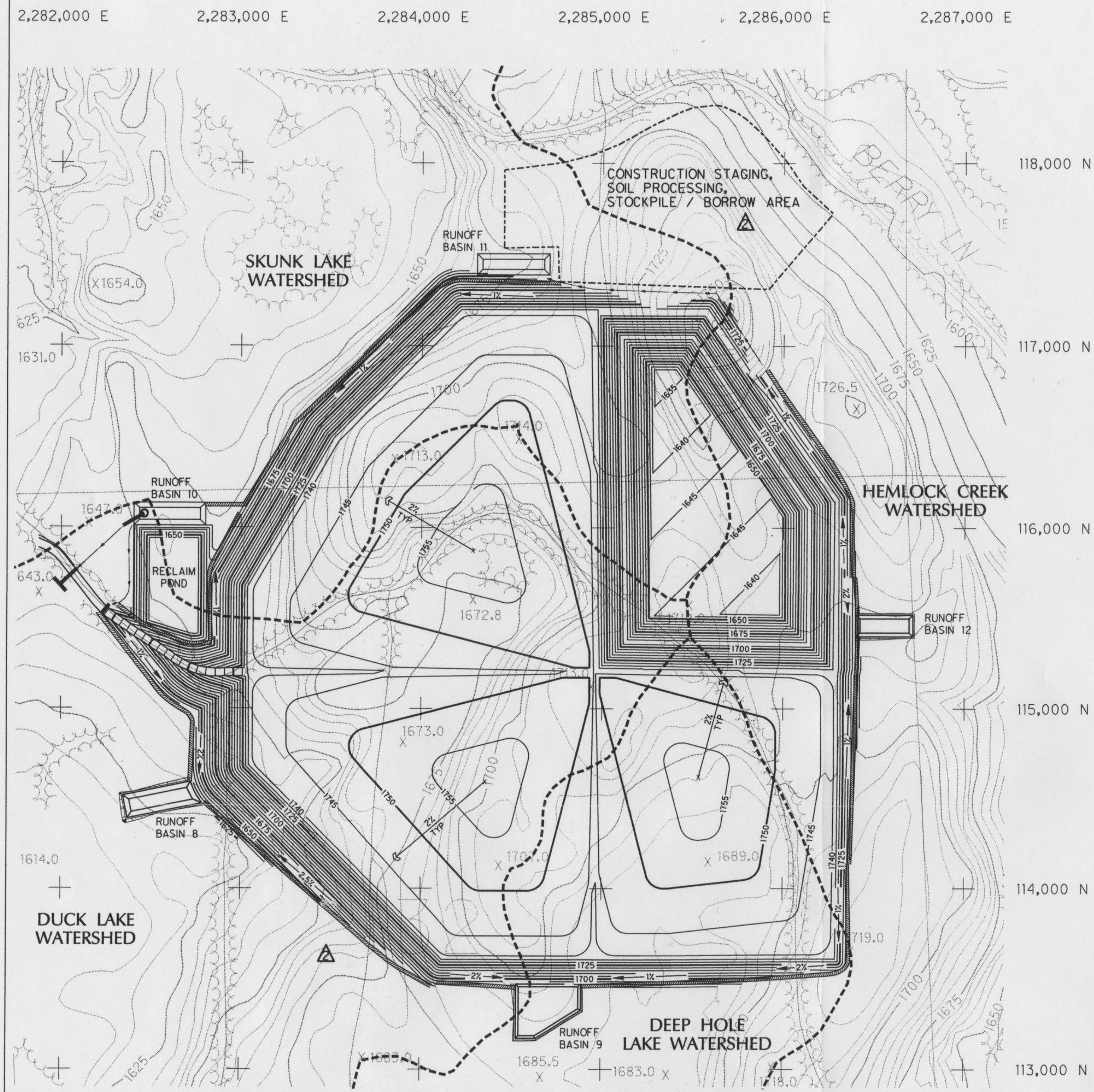

LEGEND

- EXISTING ROAD
- —1675— EXISTING CONTOUR
- X 1692.0 SPOT ELEVATION
- SECTION LINE
- —1660— PROPOSED CONTOUR
- PROPOSED TEMPORARY DRAINAGE FLOW
- 2% PROPOSED DRAINAGE DIVERSION BERM
- 2% PROPOSED SLOPE DIRECTION
- WATERSHED BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. STOCKPILE AREAS SIZE AND LOCATION ARE APPROXIMATE. THE CONTRACTOR MAY VARY LOCATION AND SIZE BASED ON LOGISTICS, CONSTRUCTION EQUIPMENT AVAILABILITY AND OTHER FACTORS.
6. CHANGED CONSTRUCTION STAGING AREA LOCATION AND ADDED STOCKPILE FOR SOIL PROCESSING.
7. CHANGED STOCKPILE, SOIL PROCESSING, AND CONSTRUCTION STAGING AREA LOCATIONS. REVISED BASE GRADES, FINAL GRADES AND SURFACE WATER DRAINAGE FOR ALL TMA CELLS.

0 300' 600'
SCALE

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	3/13/96	REM	SEE NOTES 5 AND 6
2	1/22/97	REM	SEE NOTE 7

CHECKED BY:	JKS1	DATE:	MAY '95
APPROVED BY:	REM	DATE:	MAY '95
APPROVED BY:	GWS	DATE:	MAY '95

Crandon Mining Company

FIGURE 6.12-5
TMA 4 STAGE VIII CONSTRUCTION
TMA 3 FINAL GRADES

Scale: AS SHOWN | Date: MAY, 1995
Prepared By: Foth & Van Dyke | By: GAM

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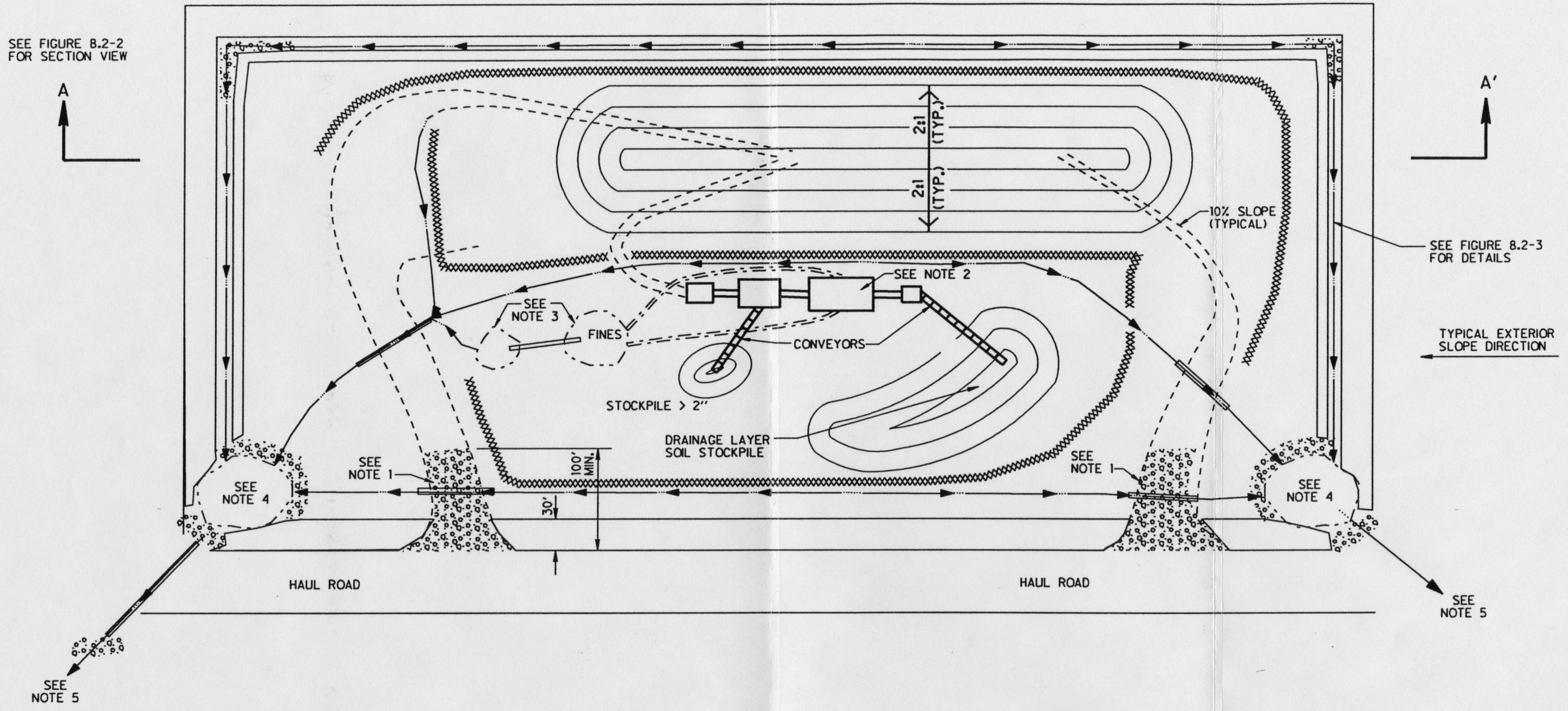

LEGEND

- RIPRAP
- CULVERT
- ROCK BERM (FIGURE 8.2-2) OR SILT FENCE (FIGURE 8.2-4)
- UNDISTURBED AREA
- DRAINAGE DITCH AND FLOW DIRECTION (SEE FIGURE 8.2-3)
- 2:1 SLOPE
- ACCESS ROAD
- RUNOFF BASIN

NOTES:

1. AT ENTRANCE AND EXIT TO STOCKPILING AND SOIL PROCESSING AREA PROVIDE A MINIMUM 100 FOOT LENGTH OF 2" TO 3" CLEAR STONE.
2. SOIL PROCESSING PLANT INCLUDING VIBRATING GRIZZLY FEEDER, SCREENING PLANT, JAW CRUSHER AND SCREEN AND WASH PLANT.
3. SETTLING PONDS WITH WATER RECIRCULATED TO WASH PLANT. EXCESS WATER, IF ANY, DISCHARGED TO SITE DITCHING.
4. TEMPORARY OR PERMANENT RUNOFF BASINS DESIGNED FOR 25 YEAR, 24 HOUR RAINFALL EVENT FROM STOCKPILE AREA.
5. DISCHARGE TO PERMANENT SITE DITCH WHICH FLOWS TO PERMANENT RUNOFF BASINS.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE: JAN, '97
APPROVED BY:	PAE	DATE: JAN, '97
APPROVED BY:	GWS	DATE: JAN, '97

Crandon Mining Company

FIGURE 8.2-1
TYPICAL STOCKPILE
AND SOIL PROCESSING LAYOUT

Scale: NOT TO SCALE	Date: JANUARY, 1997	
Prepared By: Foth & Van Dyke		By: JOW

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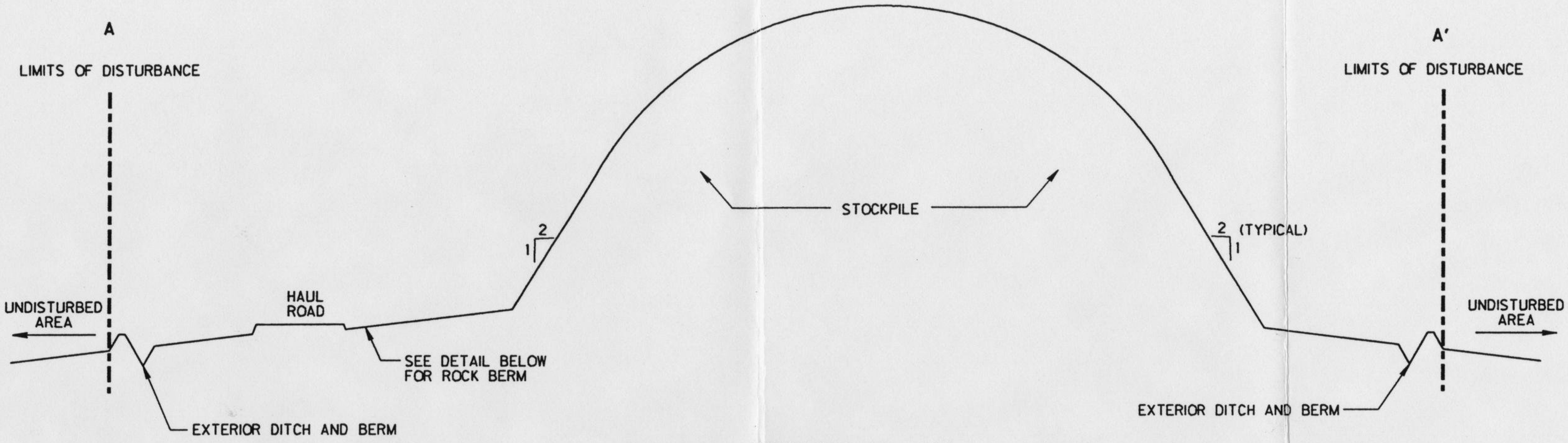
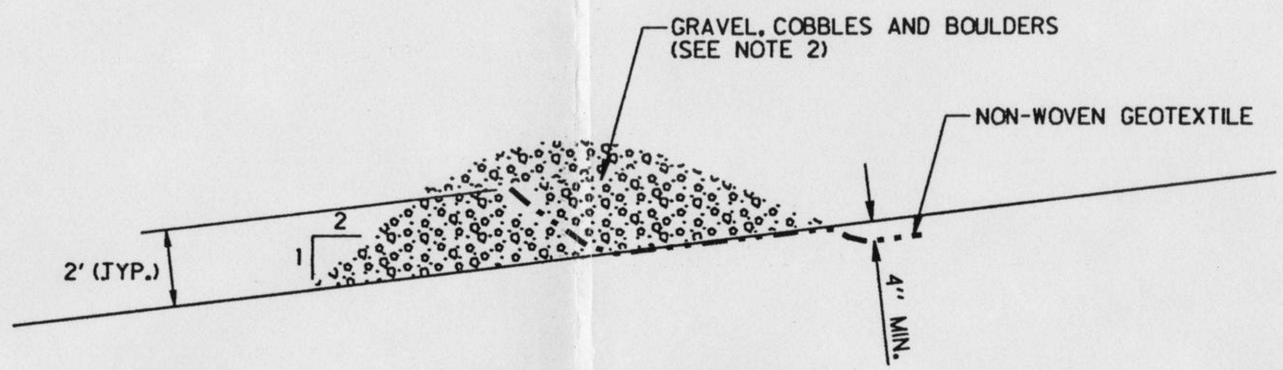


SECTION A - A'



TYPICAL ROCK BERM

NOTES:

1. TOE IN GEOTEXTILE A MINIMUM OF 4 INCHES BELOW THE EXISTING UP SLOPE GRADE.

2. MATERIAL COARSER THAN 2 INCH FROM SOIL PROCESSING.

3. SILT FENCE AS PER FIGURE 8.2-4 CAN BE SUBSTITUTED FOR THE ROCK BERM.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKSI	DATE: JAN, '97
APPROVED BY:	PAE	DATE: JAN, '97
APPROVED BY:	GWS	DATE: JAN, '97

Crandon Mining Company

FIGURE 8.2-2

CROSS SECTION A - A'

Scale: NOT TO SCALE		Date: JANUARY, 1997	
Prepared By: Foth & Van Dyke		By: JOW	

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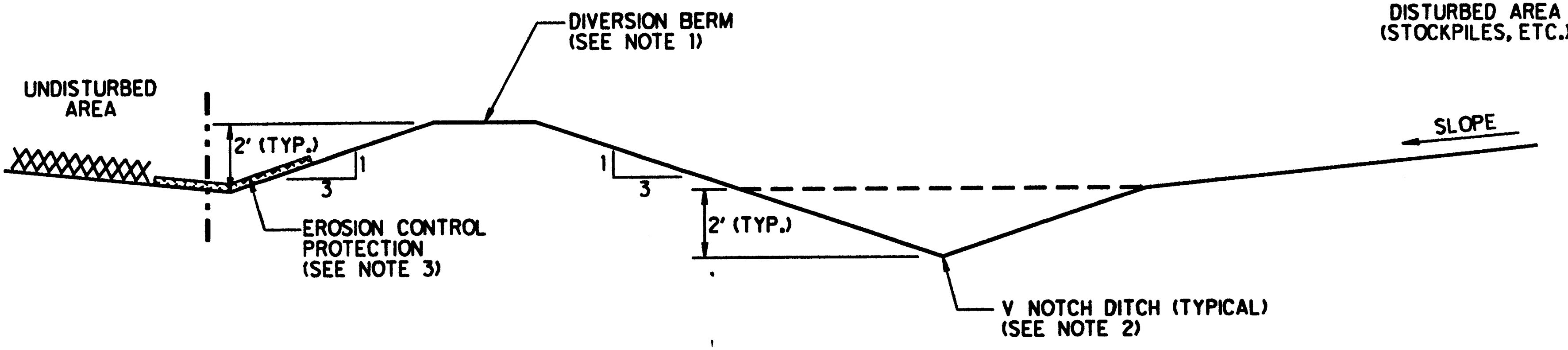


NOTES:

1. DIVERSION BERM AROUND THE STOCKPILE AREA TO BE CONSTRUCTED PRIOR TO STOCKPILING SOIL. DIVERSON BERM REQUIRED ONLY WHERE UNDISTURBED AREA SLOPES TOWARD THE STOCKPILE.

2. CONSTRUCT DIVERSION BERM WHERE REQUIRED WITH MATERIAL EXCAVATED FROM DITCH. PROVIDE EROSION CONTROL PROTECTION IN DITCH WHERE CALCULATIONS SHOW IT IS REQUIRED. FERTILIZE, SEED AND MULCH ENTIRE DITCH AND BERM.

3. PROVIDE EROSION CONTROL PROTECTION ON OUTER FACE OF DIVERSION BERM WHERE CALCULATIONS SHOW IT IS REQUIRED.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY: JKS1	DATE: JAN, '97	
APPROVED BY: PAE	DATE: JAN, '97	
APPROVED BY: GWS	DATE: JAN, '97	

Crandon Mining Company

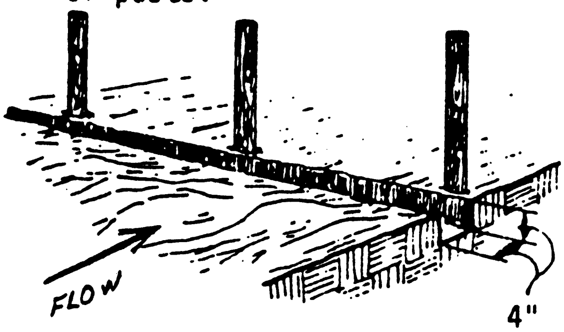
FIGURE 8.2-3

TYPICAL DIVERSION BERM
AND DITCH DETAIL

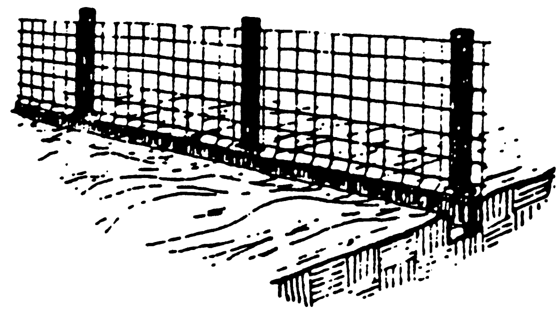
Scale: NOT TO SCALE	Date: JANUARY, 1997
Prepared By: Foth & Van Dyke	By: JOW

93C049

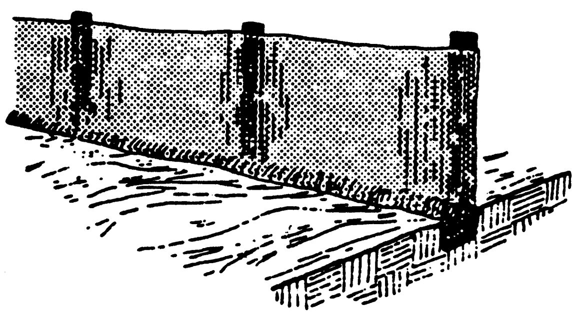
1. Set posts and excavate a 4"x4" trench upslope along the line of posts.



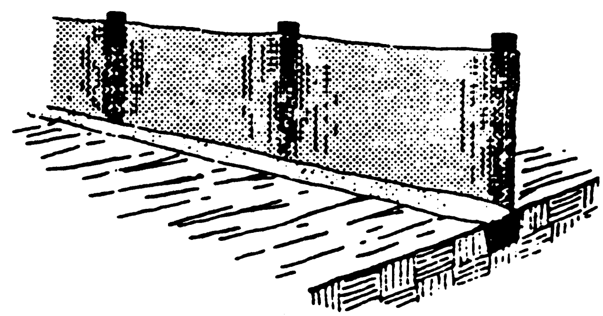
2. Staple wire fencing to the posts.



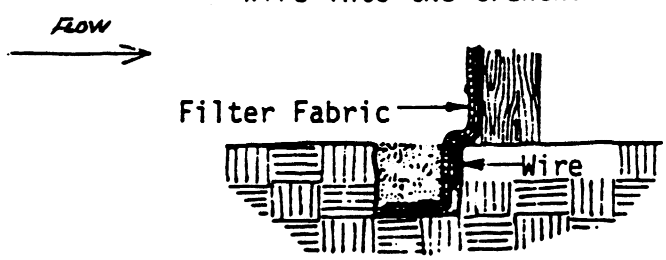
3. Attach the filter fabric to the wire fence and extend it into the trench.



4. Backfill and compact the excavated soil.



Extension of fabric and wire into the trench.



SOURCE:
ADAPTED FROM INSTALLATION OF STRAW AND FABRIC FILTER BARRIERS FOR SEDIMENT CONTROL, SHERWOOD AND WYANT

Foth & Van Dyke			
REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	JKS1	DATE: JAN. '97
APPROVED BY:	PAE	DATE: JAN. '97
APPROVED BY:	GWS	DATE: JAN. '97

Crandon Mining Company

FIGURE 8.2-4

TYPICAL CONSTRUCTION OF A SILT FENCE

Scale: NOT TO SCALE	Date: JANUARY, 1997
Prepared By: Foth & Van Dyke	By: JOW

93C049

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CUT (see Comment A)
(within TMA footprint)
10,363,900

BORROW[5, 6]
904,800[3,4]
Borrow with contingencies is:
904,800 + 447,400 = 1,352,200 cy

546,300 — 9,369,500 — 811,500 — 1,130,200 — 588,800

Till Sidewall and Base (see Comment B)

18" Till (sidewall)		
363,600/0.90	= 404,300	
12" Till (base)		
127,800/0.90	= 142,000	
491,400	546,300	

546,300 - 491,400 = 54,900

Unprocessed Till (see Comment C)

Berm Ramp Fill	175,000
Topsoil	176,900
Till - TMA Cells	5,421,600
Till - FC Grading Layer	1,040,600
Till - FC Rooting Layer	1,038,500
Till - FC Fill Material	1,516,900
	9,369,500
By-products	<588,800>

Drainage Layer (bottom)

	In-Place (cy)[1]	Fill (cy)
TMA1	73,200/0.315[2]	= 232,400
TMA 2	77,000/0.315	= 246,400
TMA 3	50,000/0.315	= 158,700
TMA 4	54,800/0.315	= 174,000
	255,600	811,500

Drainage Layer (final cover) (see Comment D)

	In-Place (cy)[1]	Fill (cy)
TMA1	88,500/0.315	= 281,000
TMA 2	110,000/0.315	= 349,200
TMA 3	75,900/0.315	= 241,000
TMA 4	81,600/0.315	= 259,000
	356,600	1,130,200

1,130,200 - 356,000 = 774,200

Fines
811,400 x 0.57[2] x 0.90[2] = 416,300
811,500 - 255,600 - 416,300 = 139,000

Fines
1,130,200 x 0.57 x 0.90 = 579,800
1,130,200 - 356,000 - 579,800 = 194,400

54,900 (see Comment E) — 139,600 — 194,400

"By-Products" (see Comment F)

>3"	54,900 cy
>3"	139,600 cy
>3"	194,400 cy
P40	199,900 cy
	588,800 cy

Base and Sideslope Liner and Base Fines

	Filter	LPS	Total
TMA1	18,300	93,800	112,100
TMA 2	19,400	117,600	137,000
TMA 3	12,500	80,900	93,400
TMA 4	13,700	87,000	100,700
	63,900	379,300	443,200

416,300 - 443,200 = <26,900>

Final Cover

	LPS
TMA1	87,500
TMA 2	109,600
TMA 3	75,100
TMA 4	80,800
	353,000

579,800 - 353,000 - 26,900 = 199,900

26,900

199,900 (see Comment G)

Notes

1. In-place cubic yards (cy) from Drawing No. 13.
2. Total percentage of till usable for drainage layer is conservatively calculated as follows:
   100% - 50% (<#40 sieve) - 8% (+1/2 inch) x 90% (less boulders) = 31.5%.
   Note, this is conservative since the gradation of the drainage layer allows up to 2 inch gravel to be used.
3. This balance assumes no use of Type I waste rock within the TMA cells.
4. Soil Balance Calculations

By-Products	<588,800>
Unprocessed Till	9,369,500
Sidewall and Base Till Cover	546,300
Liner Drainage Layer and LPS	811,500
Cap Drainage Layer and LPS	1,130,200
Total Fill Required	11,268,700
Available Cut	10,363,900
Borrow Required	904,800

5. Contingency Calculations
   Drainage Layer (processed soils)
   Final Cover 1,130,200
   Liner LCS 811,500
   1,941,700 x 0.15 = 291,300
   Final Cover Soil (grading layer only)
   Grading Layer, TMA 1-4
   1,040,600 x 0.15 = 156,100
   447,400
6. This earthwork balance is conservative since waste rock and demolition waste have not been considered as part of the available borrow materials of final cover and other uses.

Comments

A. Total cut from Table on Drawing No. 13.

B. Till screened to remove >3". Quantities from Drawing No. 13.

C. Quantity of unprocessed till required for construction and closure. Includes a reduction in the quantity by the amount of "by-products" from soil processing.

D. Drainage layer quantities from Drawing No. 13. Calculation of soil required to manufacture drainage layer from Addendum No. 2, Attachment 11 (Foth & Van Dyke, 1996b).

E. >3" by-products to be used as fill.

F. Total by-products available for use as fill.

G. Excess fines from processing (i.e., by-products) available for use as fill.

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	REM	DATE: JAN. '97
APPROVED BY:	PAE	DATE: JAN. '97
APPROVED BY:	GWS	DATE: JAN. '97

Crandon Mining Company

FIGURE 10.4-1
TMA TOTAL EARTHWORK
BALANCE FLOWCHART

Scale: NOT TO SCALE	Date: JANUARY, 1997		
Prepared By: Foth & Van Dyke	By: JRB2		

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Drawings for Addendum No. 3 to the TMA Feasibility Report/Plan of Operation

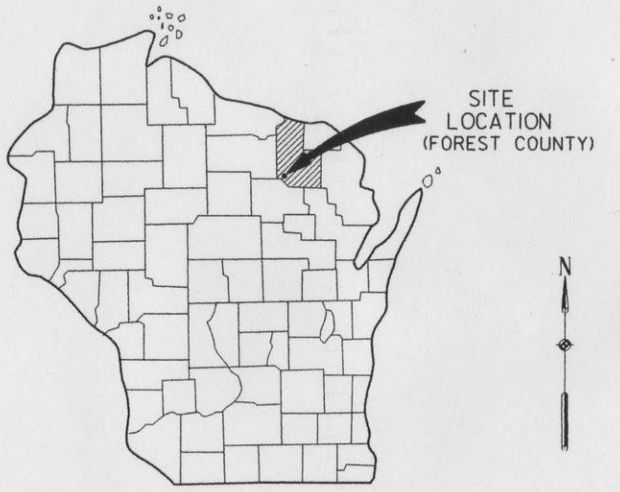
Crandon Mining Company

CRANDON PROJECT
ADDENDUM NO. 3 TO THE FEASIBILITY REPORT / PLAN OF OPERATION
TAILINGS MANAGEMENT AREA

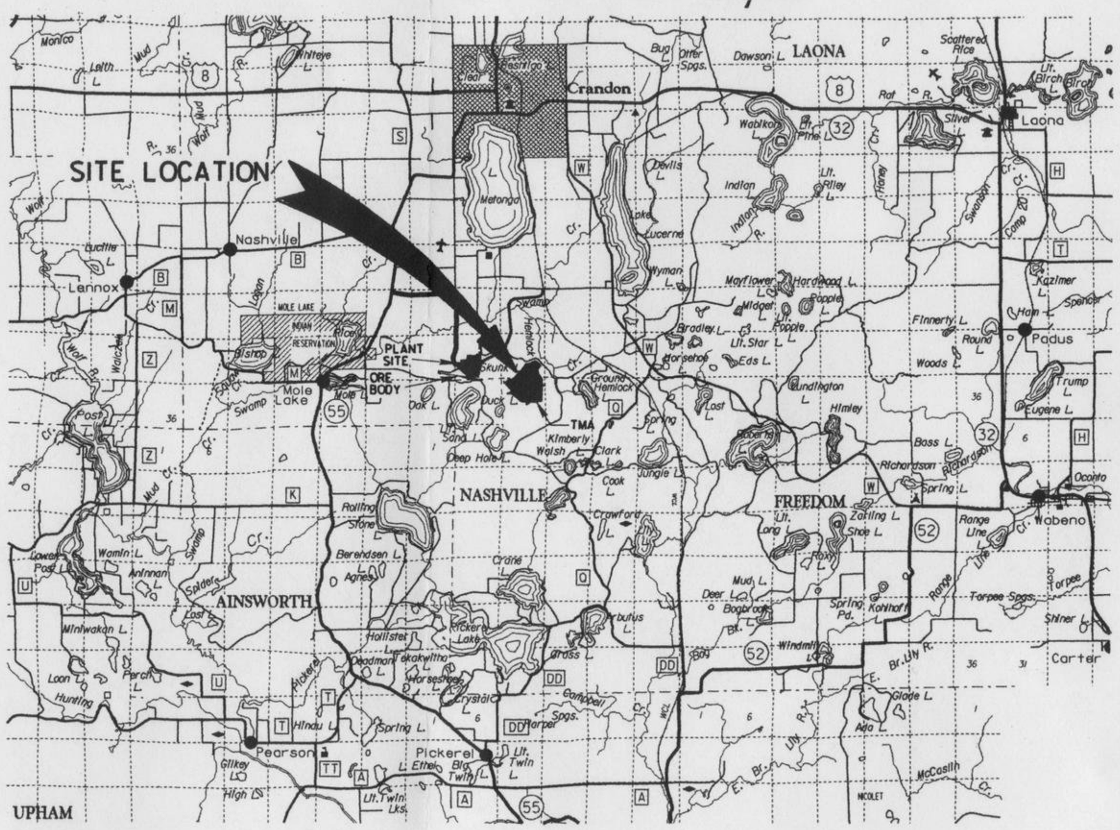
FOREST COUNTY, WISCONSIN

JANUARY, 1997

Foth & Van Dyke



COUNTY LOCATION MAP



SITE LOCATION MAP
SCALE: 1" = 10,000'

INDEX		
DRAWING NO.		DESCRIPTION
*	1	TITLE SHEET
	2	EXISTING CONDITIONS - CRANDON PROJECT
*	3	EXISTING CONDITIONS - TMA
	4	GROUNDWATER TABLE CONTOUR MAP OCTOBER, 1994
	5	GEOLOGIC CROSS SECTION LOCATION MAP
	6	GEOLOGIC CROSS SECTION F''-F'''
	7	GEOLOGIC CROSS SECTION G''-G'''
	8	GEOLOGIC CROSS SECTION H''-H'''
	9	GEOLOGIC CROSS SECTION J''-J'''
	10	GEOLOGIC CROSS SECTION K''-K'''
	11	GEOLOGIC CROSS SECTION L''-L'''
	12	GEOLOGIC CROSS SECTION O''-O'''
*	13	SITE SEQUENCING PLAN
*	14	SUBBASE GRADES
*	15	BASE GRADES
*	16	PIPING PLAN
*	17	TOP OF TILL GRADES AND TAILINGS DISTRIBUTION PIPING
*	18	FINAL GRADES
	19	TMA-1, STAGE I PHASING PLAN
	20	TMA-1, STAGE II PHASING PLAN
	21	TMA-1, FINAL GRADES AND TMA-2, BASE GRADES
	22	DRAINAGE PLAN
	23	ENGINEERING CROSS SECTIONS
	24	ENGINEERING CROSS SECTIONS
	25	ENGINEERING CROSS SECTIONS
	26	ENGINEERING CROSS SECTIONS
	27	ENGINEERING CROSS SECTIONS
*	28	DETAILS
*	29	DETAILS
*	30	DETAILS
*	31	DETAILS

* DENOTES REVISED DRAWINGS AND FIGURES FOR ADDENDUM NO. 3 INCLUDED AS PART OF THIS DRAWING SET.

WISCONSIN PROFESSIONAL ENGINEER
Nandakumaran Paruvakat
E-25845
Green Bay, Wis.

Nandakumaran Paruvakat
1-30-97

WISCONSIN PROFESSIONAL ENGINEER
PAUL A. EGELHOFF
E-26534
HOBART, WI

Paul A. Egelhoff
1-30-97

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

1

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LEGEND

- LAKES
- STREAMS
- EXISTING ROAD
- EXISTING CONTOUR
- SPOT ELEVATION
- SECTION LINE
- ORE BODY
- PROPOSED ACCESS ROAD
- PROPOSED HAUL ROAD
- PROPOSED RAILROAD SPUR
- PROPOSED FACILITIES
- PROJECT AREA

N

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. ORE BODY OUTLINE IS REPRESENTATIVE OF THE SUBCROP AT THE BASE OF THE OVERBURDEN.

SCALE
0 2000' 4000'

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	4/97	GWS	REVISED DISTURBANCE LIMIT

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE
EXISTING CONDITIONS - CRANDON PROJECT

SCALE AS SHOWN	STATE: WISCONSIN		COUNTY FOREST		
DRAWN BY GAM,JPRI	DATE 5/95	CHECKED BY	JKSI	DATE	5/95
APPROVED BY REM	DATE 5/95	APPROVED BY	NXP	DATE	5/95
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY	DEM	DATE	5/95
DRAWING NO. 2		SHEET 2 OF 31		REVISION NO. 1	

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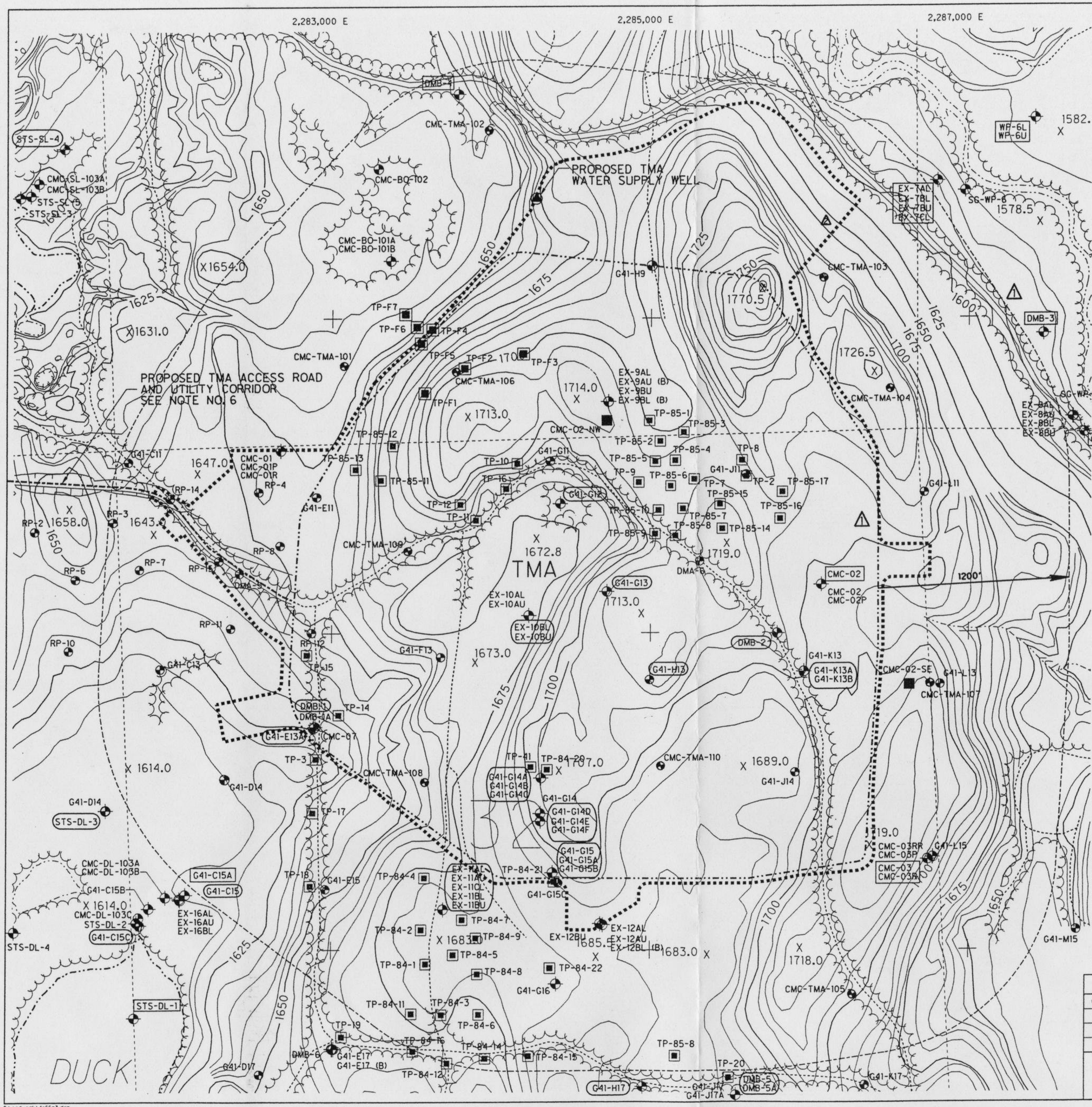

LEGEND

- LAKES
- STREAMS
- EXISTING ROAD
- —1675— EXISTING CONTOUR
- 1692.0 X SPOT ELEVATION
- SECTION LINE
- PERIMETER OF MINE WASTE SITE (AS PER NR182)
- COMPLIANCE BOUNDARY
- SOIL BORING LOCATION (SEE NOTE 5)
- MONITORING WELL LOCATION (SEE NOTE 5)
- WP-7L ABANDONED WELLS NUMBER AND LOCATION
- DMB-2 TEMPORARILY ABANDONED WELLS NUMBER AND LOCATION
- TEST PIT LOCATION (SEE NOTE 5)
- PROPOSED TMA WATER SUPPLY WELL LOCATION (SEE NOTE 5)
- OXYGEN BOREHOLES NUMBER AND LOCATION
- TMA DISTURBANCE BOUNDARY

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. SOIL BORING, TEST PIT, WATER SUPPLY WELL & MONITORING WELL NUMBERS ADJACENT TO SYMBOL.
6. UTILITY CORRIDOR INCLUDES TAILINGS PIPELINE, RECLAIM WATER PIPELINE, AND ELECTRICAL.
7. REVISED PERIMETER OF MINE WASTE SITE, AND COMPLIANCE BOUNDARY.

0 300' 600'
SCALE

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	1/22/97	REM	SEE NOTE 7
2	4/10/97	GWS	ADD TMA DISTURBANCE BOUNDARY

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

TITLE: EXISTING CONDITIONS - TMA

SCALE: AS SHOWN	STATE: WISCONSIN	COUNTY: FOREST	
DRAWN BY: GAM, JPRI	DATE: 5/95	CHECKED BY: JKSI	DATE: 5/95
APPROVED BY: REM	DATE: 5/95	APPROVED BY: NXP	DATE: 5/95
APPROVED BY: GWS	DATE: 5/95	CRANDON MINING COMPANY: DEM	DATE: 5/95
DRAWING NO.: 3		SHEET 3 OF 3	REVISION NO. 1

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SITE PREPARATION /OPERATION

CELL	STAGE	MINE WASTE VOLUME (CY)	EARTHWORK (CY) CUT (1)	EARTHWORK (CY) FILL	12" P40 TILL LAYER (CY)	GCL (SF)	GEOMEMBRANE (SF)	GEOTEXTILE (CUSHIONING) (SF)	GEOCOMPOSITE (SF)	24" DRAINAGE LAYER (CY)	GEOTEXTILE (FILTER) (SF)	12" (BASE) TILL LAYER (CY)	6" BASE FINES LAYER (CY)	18" SIDEWALL TILL LAYER (CY)
1	I	2,484,200	1,554,000	1,062,000	73,900	1,971,300	1,971,300	950,200	1,014,800	73,200	994,200	36,600	18,300	51,900
	II	2,153,600	100	1,761,300	19,900	522,600	522,600	522,600	N.A.	N.A.	N.A.	N.A.	N.A.	30,400
2	III	3,200,000	3,527,400	116,600	83,100	2,219,600	2,219,600	1,011,400	1,162,400	77,600	1,056,900	38,800	19,400	62,700
	IV	3,903,600	1,200	1,552,600	34,500	876,800	876,800	876,800	N.A.	N.A.	N.A.	N.A.	N.A.	49,600
3	V	1,605,300	2,009,400	12,800	54,000	1,430,500	1,430,500	656,700	768,600	50,000	691,600	25,000	12,500	40,700
	VI	2,649,800	5,900	693,000	26,900	725,700	725,700	725,700	N.A.	N.A.	N.A.	N.A.	N.A.	41,100
4	VII	1,727,700	3,248,000	100	59,900	1,588,200	1,588,200	708,600	874,000	54,800	747,600	27,400	13,700	42,900
	VIII	2,852,600	17,900	223,200	27,100	732,900	732,900	732,900	N.A.	N.A.	N.A.	N.A.	N.A.	43,700
TOTALS		20,576,800	10,363,900	5,421,600	379,300	10,067,600	10,067,600	6,184,900	3,819,800	255,600	3,490,300	127,800	63,900	363,000

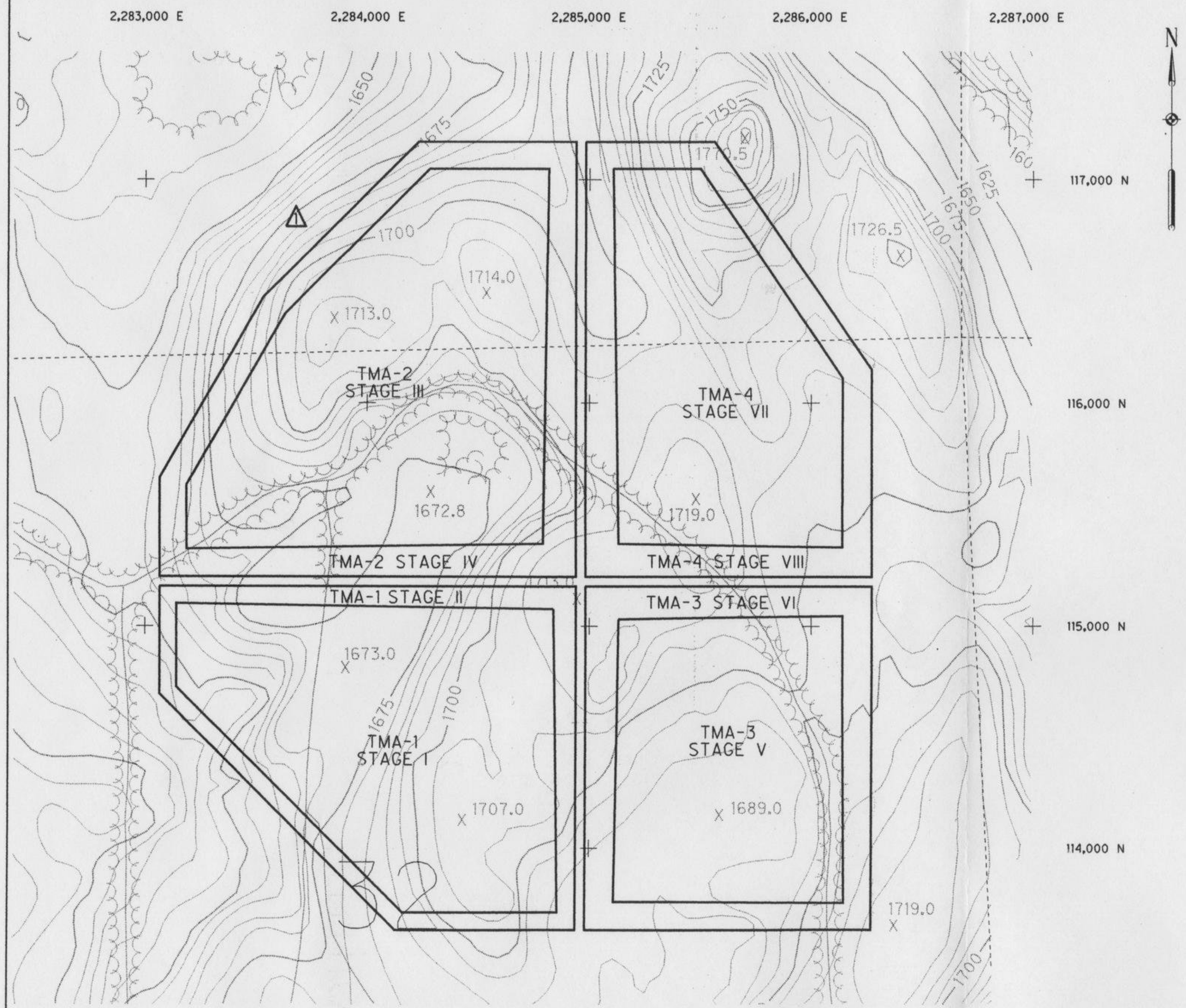
(1) INCLUDES TOPSOIL
(CY) - CUBIC YARDS
(SF) - SQUARE FEET
N.A. - NOT APPLICABLE
GCL - GEOSYNTHETIC CLAY LINER

NOTE:
1. RIPRAP IS INCLUDED IN THE WASTE QUANTITIES
2. ALL VALUES ARE INPLACE QUANTITIES

CLOSURE

CELL	STAGE	FILL MATERIAL (CY) (1)	36" SOIL GRADING (CY)	12" P40 TILL LAYER (CY)	GCL (SF)	GEOMEMBRANE (SF)	12" DRAINAGE LAYER (CY)	36" ROOTING LAYER (CY)	6" TOPSOIL LAYER (CY)
1	I	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
	II	373,600	258,900	87,500	2,420,400	2,420,400	88,500	258,000	44,300
2	III	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
	IV	507,100	321,000	109,600	2,980,500	2,980,500	110,000	322,200	53,700
3	V	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
	VI	307,900	221,800	75,100	2,076,300	2,076,300	75,900	220,200	38,000
4	VII	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
	VIII	328,300	238,900	80,800	2,234,200	2,234,200	81,600	238,100	40,900
TOTALS		1,516,900	1,040,600	353,000	9,711,400	9,711,400	356,000	1,038,500	176,900

(1) FILL MATERIAL MAY INCLUDE TAILINGS, WASTE ROCK, TILL SOIL OR OTHER MATERIALS.
(CY) - CUBIC YARDS
(SF) - SQUARE FEET
GCL - GEOSYNTHETIC CLAY LINER
N.A. - NOT APPLICABLE



NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. REVISED WASTE VOLUMES, CONSTRUCTION AND CLOSURE QUANTITIES, AND SITE CONFIGURATION.

LEGEND

EXISTING ROAD
— 1675 — EXISTING CONTOUR
1692.0 X EXISTING SPOT ELEVATION

SCALE
0 300' 600'

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	1/22/97	REM	SEE NOTE 5

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE
SITE SEQUENCING PLAN

SCALE AS SHOWN	STATE WISCONSIN	COUNTY FOREST	
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI	DATE 5/95
APPROVED BY REM	DATE 5/95	APPROVED BY NXP	DATE 5/95
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM	DATE 5/95
DRAWING NO. 13		SHEET 13 OF 31	REVISION NO. 1

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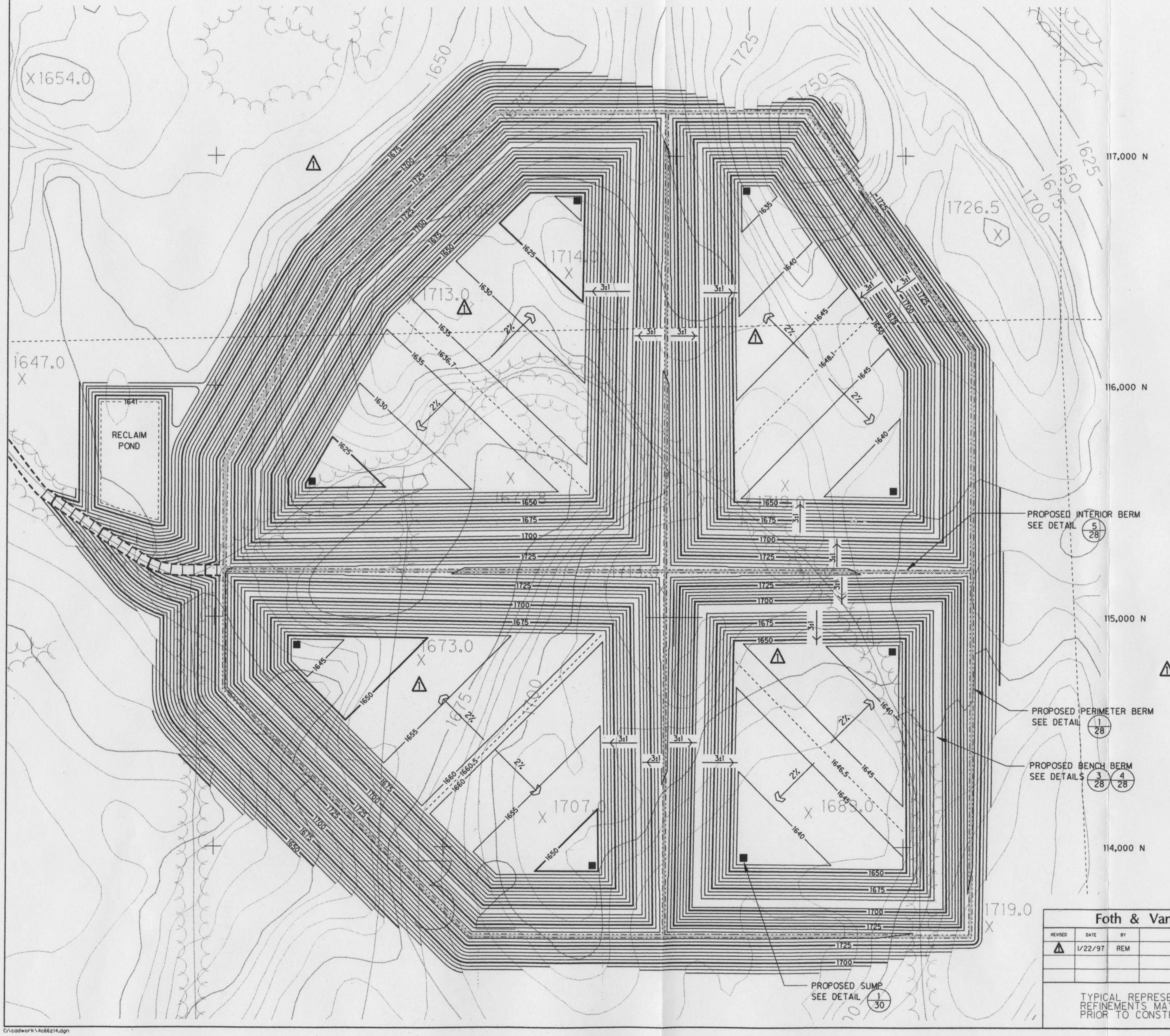

N

LEGEND

- STREAMS
- EXISTING ROAD
- —1675— EXISTING CONTOUR
- 1692.0 X SPOT ELEVATION
- EX-9BU EXISTING MONITORING WELL LOCATION AND NUMBER
- SECTION LINE
- 1725 PROPOSED CONTOUR (SUBBASE GRADE)
- 3:1 PROPOSED SLOPE RATIO (TYP.)
- 2% PROPOSED SLOPE (TYP.)
- PROPOSED SUMP LOCATION

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE SUBBASE GRADES. (BOTTOM OF 1' LOW PERMEABILITY SOIL LAYER).
6. REVISED SITE CONFIGURATION, AND SUBBASE GRADES.

SCALE

0 200' 400'

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	1/22/97	REM	SEE NOTE 6

TYPICAL REPRESENTATION:
REFINEMENTS MAY BE MADE
PRIOR TO CONSTRUCTION

TITLE

SUBBASE GRADES

SCALE AS SHOWN	STATE: WISCONSIN		COUNTY FOREST		
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI		DATE 5/95	
APPROVED BY REM	DATE 5/95	APPROVED BY NXP		DATE 5/95	
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM		DATE 5/95	
DRAWING NO. 14		SHEET 14 OF 31		REVISION NO. 1	

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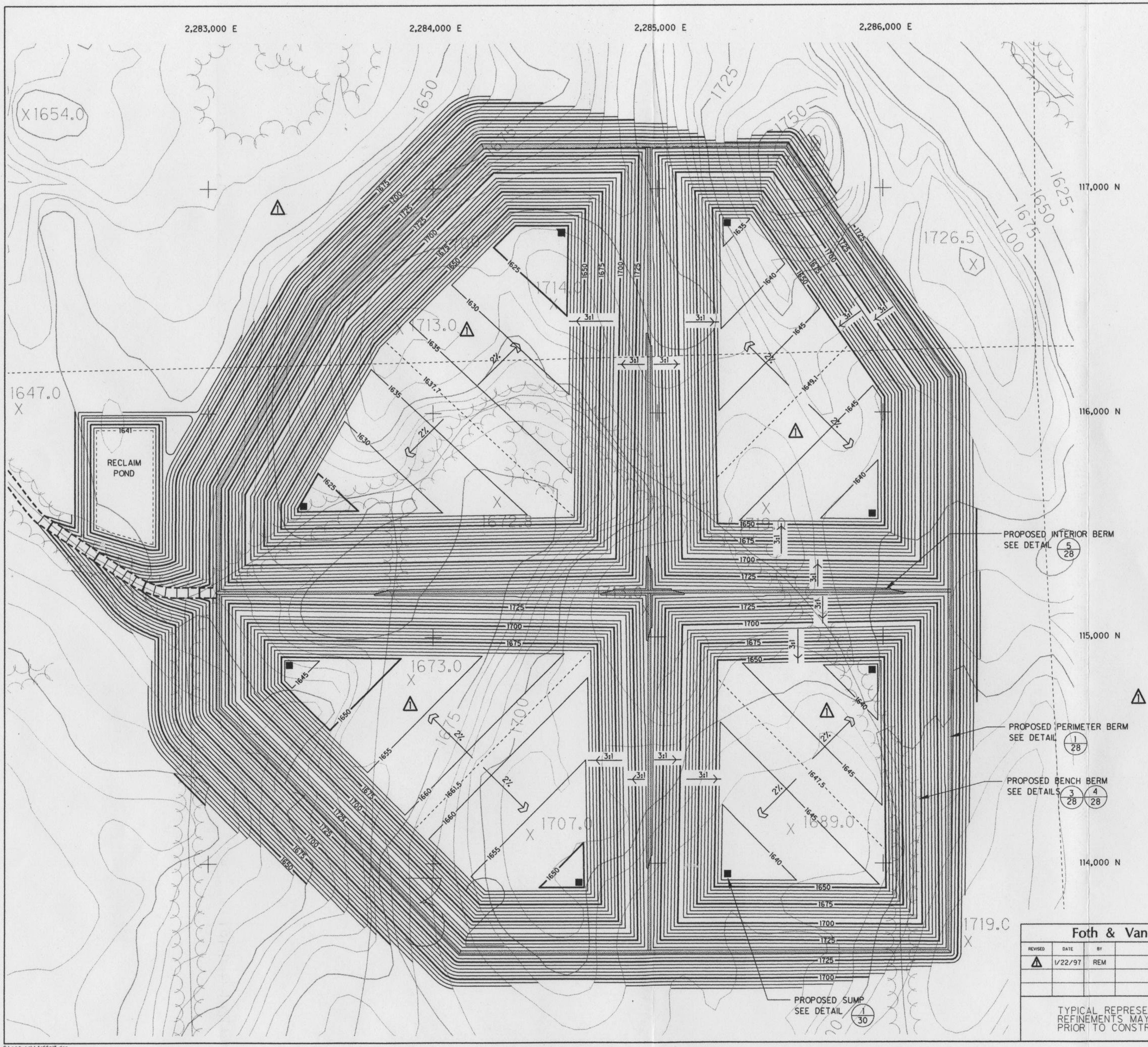

N

LEGEND

- LAKES
- STREAMS
- EXISTING ROAD
- —1675— EXISTING CONTOUR
- 1692.0 X SPOT ELEVATION
- SECTION LINE
- 1725 PROPOSED CONTOUR (SUBBASE GRADE)
- 3:1 PROPOSED SLOPE RATIO (TYP.)
- 2% PROPOSED SLOPE (TYP.)
- PROPOSED SUMP LOCATION

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE BASE GRADES. (TOP OF 1' LOW PERMEABILITY SOIL LAYER).
6. ⚠ REVISED SITE CONFIGURATION, AND BASE GRADES.

SCALE

0 200' 400'

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
⚠	1/22/97	REM	SEE NOTE 6

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

TITLE

BASE GRADES

SCALE AS SHOWN	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI	DATE 5/95		
APPROVED BY REM	DATE 5/95	APPROVED BY NXP	DATE 5/95		
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM	DATE 5/95		
DRAWING NO. 15		SHEET 15 OF 31	REVISION NO. 1		

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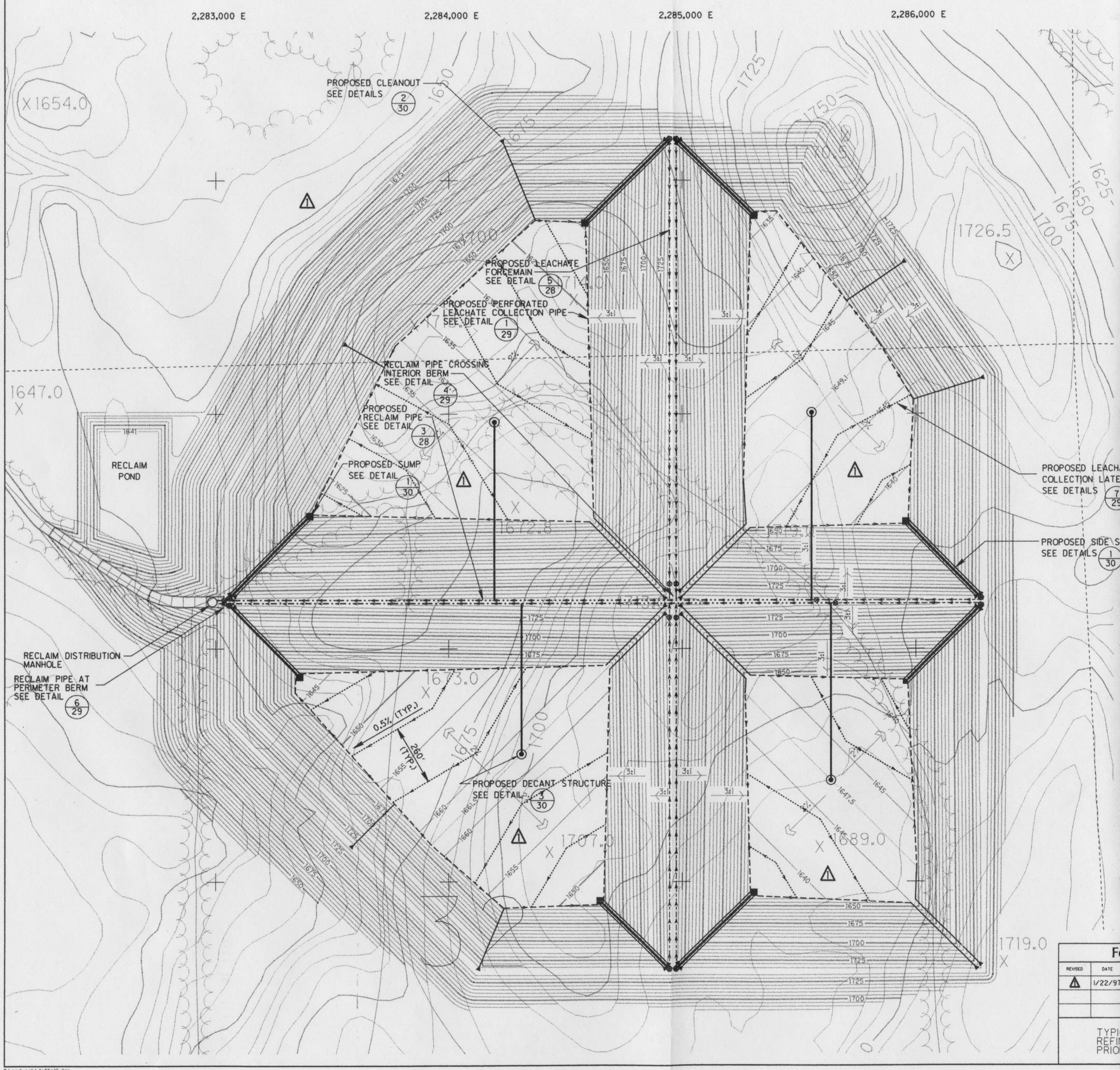

LEGEND

	EXISTING ROAD
—1675—	EXISTING CONTOUR
1692.0 X	SPOT ELEVATION
EX-9BU	MONITORING WELL LOCATION AND NUMBER
	SECTION LINE
1725	PROPOSED CONTOUR (BASE GRADE)
3:1	PROPOSED SLOPE RATIO (TYP.)
2%	PROPOSED SLOPE (TYP.)
	PROPOSED RECLAIM PIPE
	PROPOSED 4'' LEACHATE PIPE
	PROPOSED 6'' PERFORATED LEACHATE COLLECTION PIPE
	PROPOSED DECANT STRUCTURE
	PROPOSED VALVE
	PROPOSED SIDE SLOPE RISER
	PROPOSED CLEANOUT RISER
	PROPOSED SUMP LOCATION

N

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1''=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE BASE GRADES. (TOP OF 1' LOW PERMEABILITY SOIL LAYER).
6. REVISED SITE CONFIGURATION AND BASE GRADES. ADDED 4'' LCS PIPING

SCALE
0 200'' 400'

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
6	1/22/97	REM	SEE NOTE 6

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE
PIPING PLAN

SCALE AS SHOWN	STATE WISCONSIN	COUNTY FOREST	
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI	DATE 5/95
APPROVED BY REM	DATE 5/95	APPROVED BY NXP	DATE 5/95
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM	DATE 5/95
DRAWING NO. 16		SHEET 16 OF 31	REVISION NO. 1

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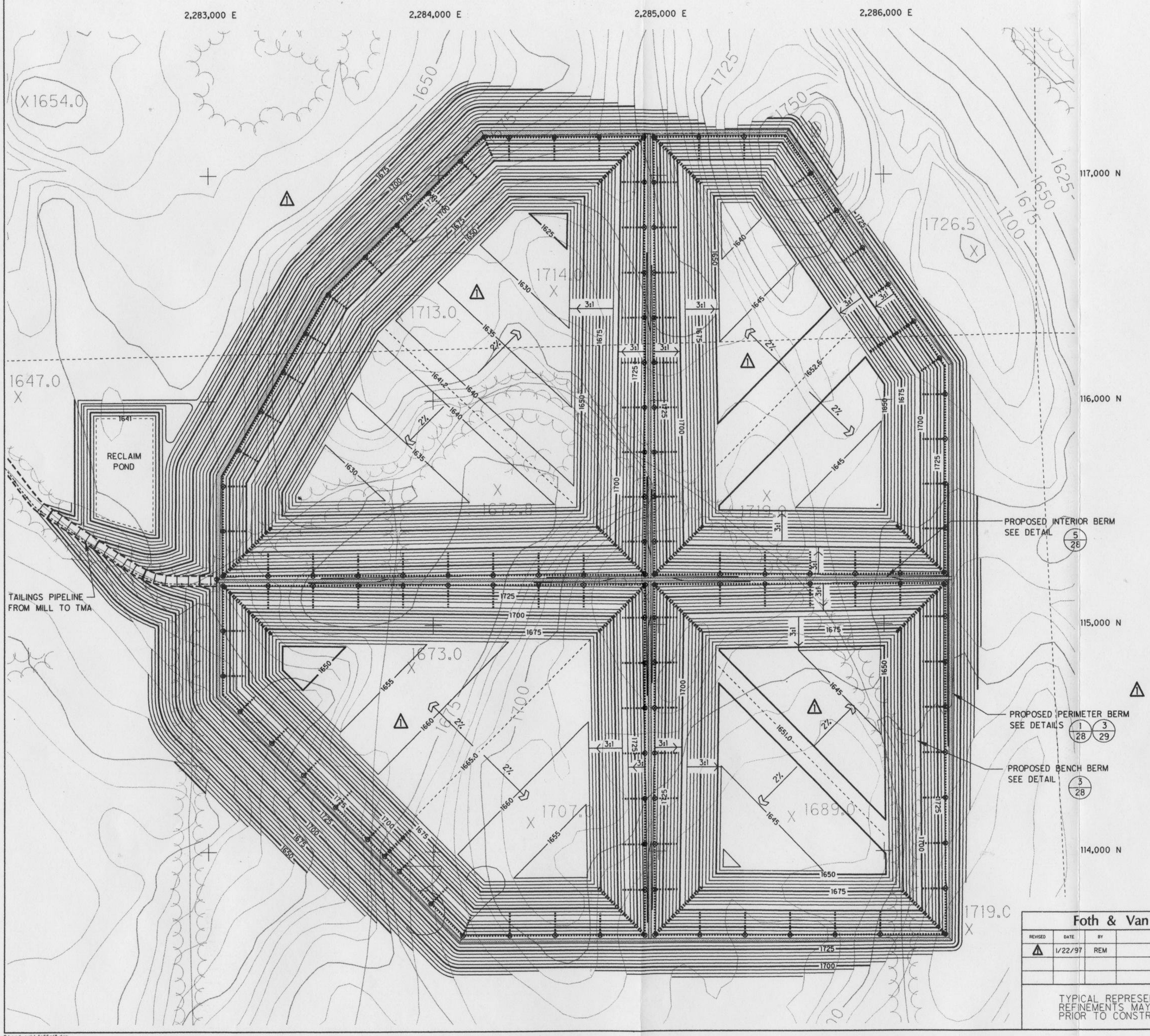

LEGEND

- LAKES
- STREAMS
- EXISTING ROAD
- —1675— EXISTING CONTOUR
- 1692.0 X SPOT ELEVATION
- SECTION LINE
- —1725— PROPOSED CONTOUR (TOP OF TILL GRADE)
- 3:1 PROPOSED SLOPE RATIO (TYP.)
- 2% PROPOSED SLOPE (TYP.)
- PROPOSED VALVE
- PROPOSED MASS DISCHARGE LINE
- PROPOSED TAILINGS DISCHARGE PIPELINE AND SPIGOT ARRANGEMENT

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE TOP OF TILL GRADES. (TOP OF 6" TILL FINES LAYER AT THE BASE)
6. △ REVISED SITE CONFIGURATION, AND TOP OF TILL GRADES.

SCALE 0 200' 400'

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
△	1/22/97	REM	SEE NOTE 6

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE
TOP OF TILL GRADES AND TAILINGS DISTRIBUTION PIPING

SCALE AS SHOWN	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI		DATE 5/95	
APPROVED BY REM	DATE 5/95	APPROVED BY NXP		DATE 5/95	
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM		DATE 5/95	
DRAWING NO. 17			SHEET 17 OF 31	REVISION NO. 1	

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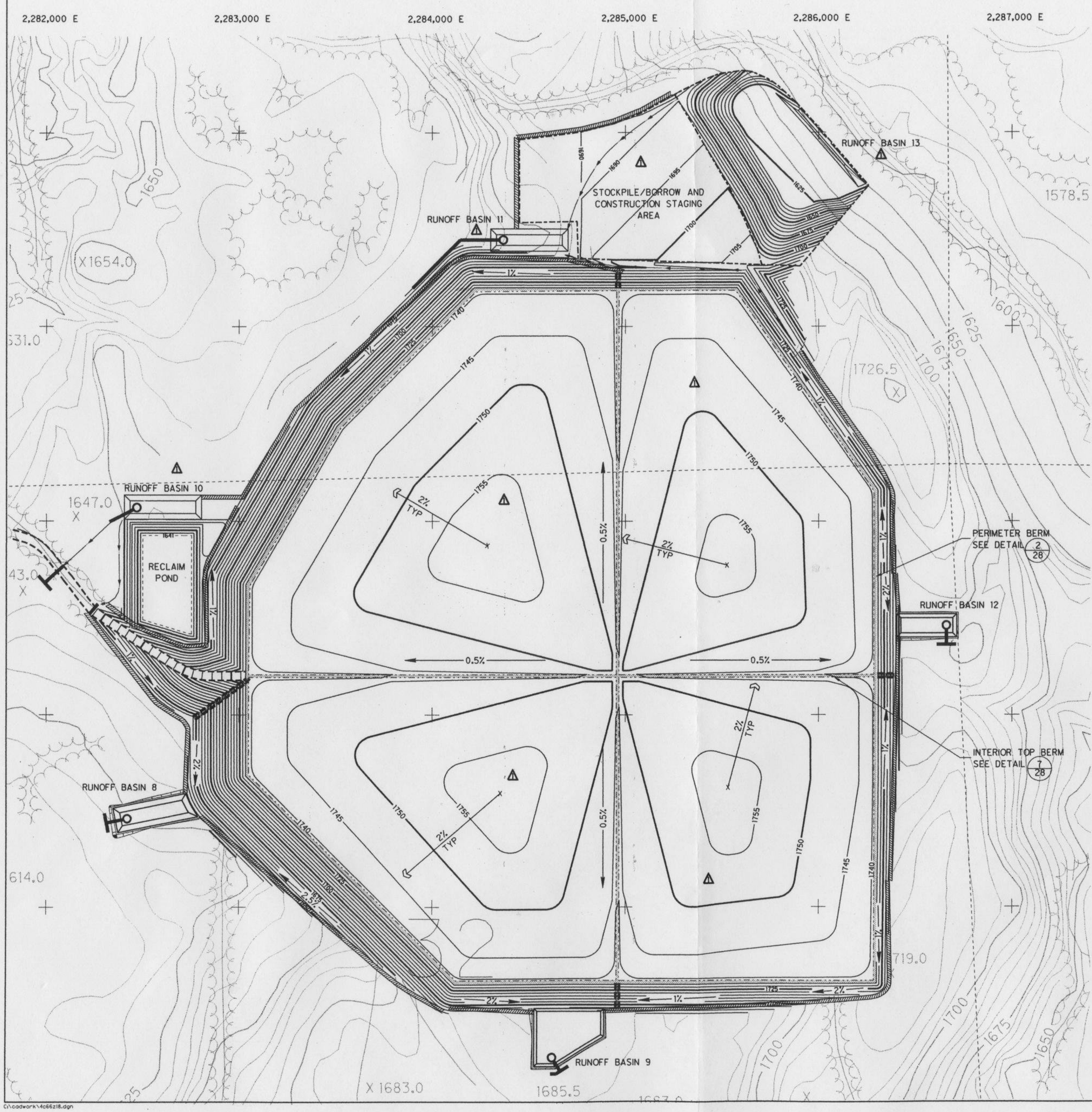

N

LEGEND

═══	EXISTING ROAD
—1675—	EXISTING CONTOUR
1692.0 X	SPOT ELEVATION
- - - - - - -	SECTION LINE
—1725—	PROPOSED CONTOUR (FINAL GRADE)
3:1	PROPOSED SLOPE RATIO (TYP.)
2%	PROPOSED SLOPE (TYP.)
	PROPOSED SPILLWAY
	PROPOSED BERM

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE FINAL GRADES. (TOP OF FINAL COVER).
6. △ REVISED SITE CONFIGURATION, FINAL GRADES, RUNOFF BASIN LOCATIONS, AND STOCKPILE/BORROW AREA.

SCALE
0 250' 500'

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
△	1/22/97	REM	SEE NOTE 6

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

TITLE: FINAL GRADES

SCALE AS SHOWN	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI		DATE 5/95	
APPROVED BY REM	DATE 5/95	APPROVED BY NXP		DATE 5/95	
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM		DATE 5/95	
DRAWING NO. 18		SHEET 18 OF 31		REVISION NO. 1	

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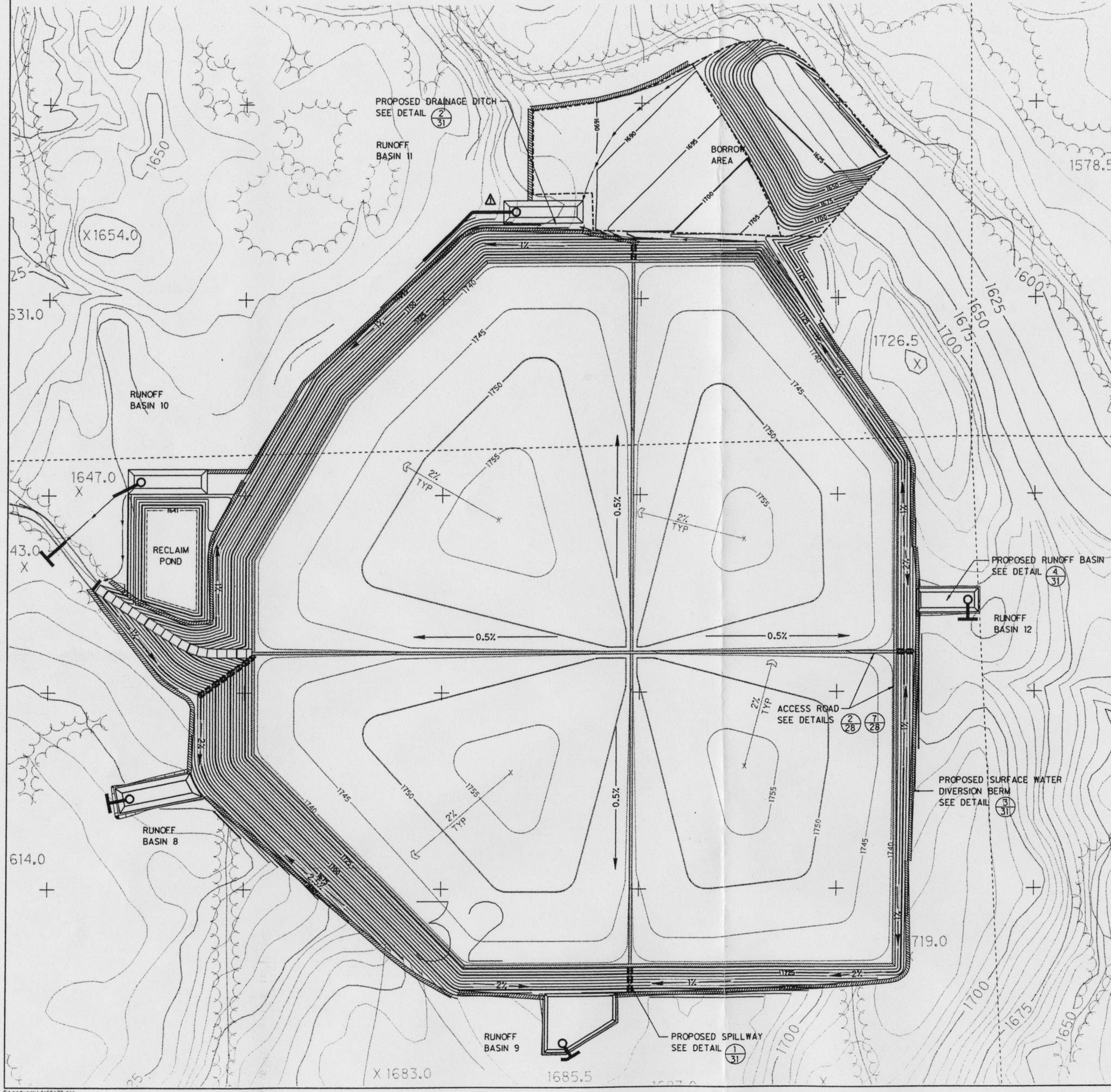

N

LEGEND

	EXISTING ROAD
—1675—	EXISTING CONTOUR
1692.0 X	SPOT ELEVATION
- - - - - -	SECTION LINE
—1725—	PROPOSED CONTOUR (FINAL GRADE)
3:1	PROPOSED SLOPE RATIO (TYP.)
2%	PROPOSED SLOPE (TYP.)
	PROPOSED BERM
	PROPOSED SPILLWAY
	PROPOSED DRAINAGE OUTLET
— 0.5% →	FLOW DIRECTION

NOTES:

1. TOPOGRAPHIC BASE MAP DIGITIZED FROM 1"=1000' SCALE, 5' CONTOUR INTERVAL MAP PREPARED BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN, WISCONSIN. DATE OF PHOTOGRAPHY APRIL 28, 1976.
2. HORIZONTAL DATUM BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM - NORTH ZONE.
3. VERTICAL DATUM BASED ON MEAN SEA LEVEL DATUM. CONTOUR INTERVAL IS FIVE FEET.
4. COUNTY AND TOWNSHIP LINES DIGITIZED FROM 7.5' SERIES USGS MAPS.
5. THIS DRAWING SHOWS CONTOURS DEPICTING SITE FINAL GRADES. (TOP OF FINAL COVER).
6. FOR EROSION CONTROL REFER TO THE CMC MPA, 1995 SUBMITTAL.
7. △ REVISED SITE CONFIGURATION, FINAL GRADES, RUNOFF BASIN LOCATIONS, AND STOCKPILE/BORROW AREA.

SCALE
0 250' 500'

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
△	4/10/97	GWS	SEE NOTE 7

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

TITLE

DRAINAGE PLAN

SCALE AS SHOWN	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI		DATE 5/95	
APPROVED BY REM	DATE 5/95	APPROVED BY NXP		DATE 5/95	
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM		DATE 5/95	
DRAWING NO. 22		SHEET 22 OF 31		REVISION NO.	

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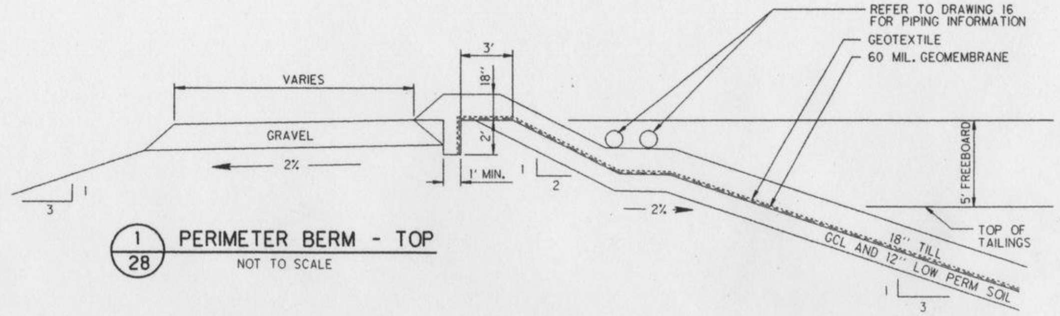
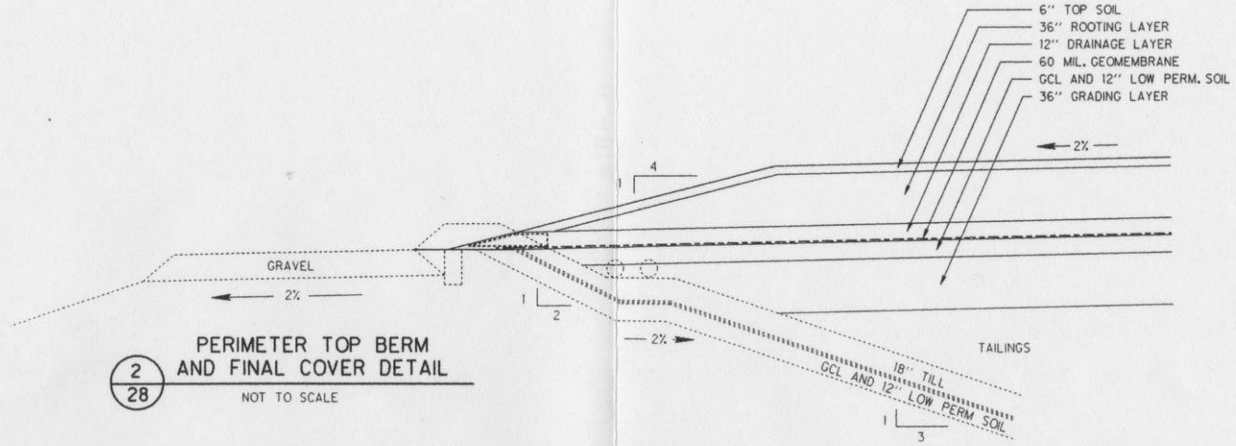
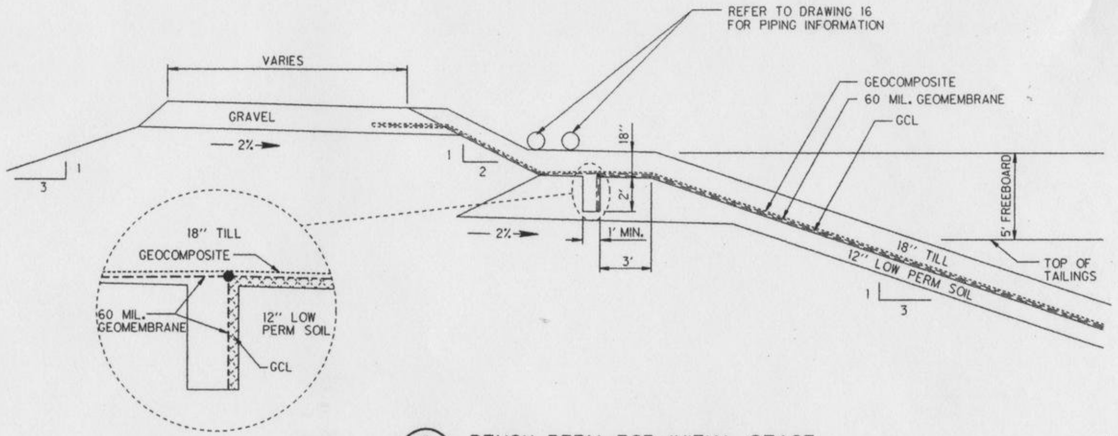


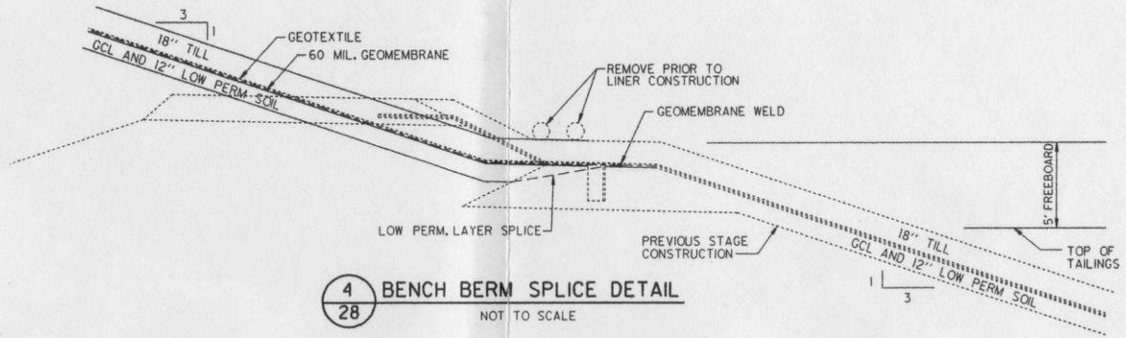
1/28 PERIMETER BERM - TOP
NOT TO SCALE



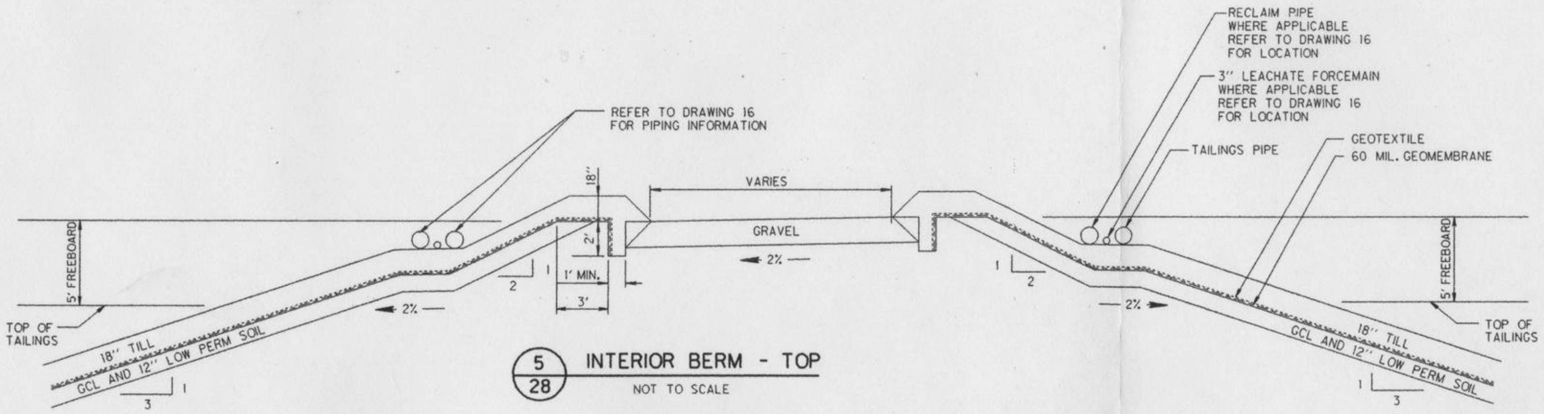
2/28 PERIMETER TOP BERM AND FINAL COVER DETAIL
NOT TO SCALE



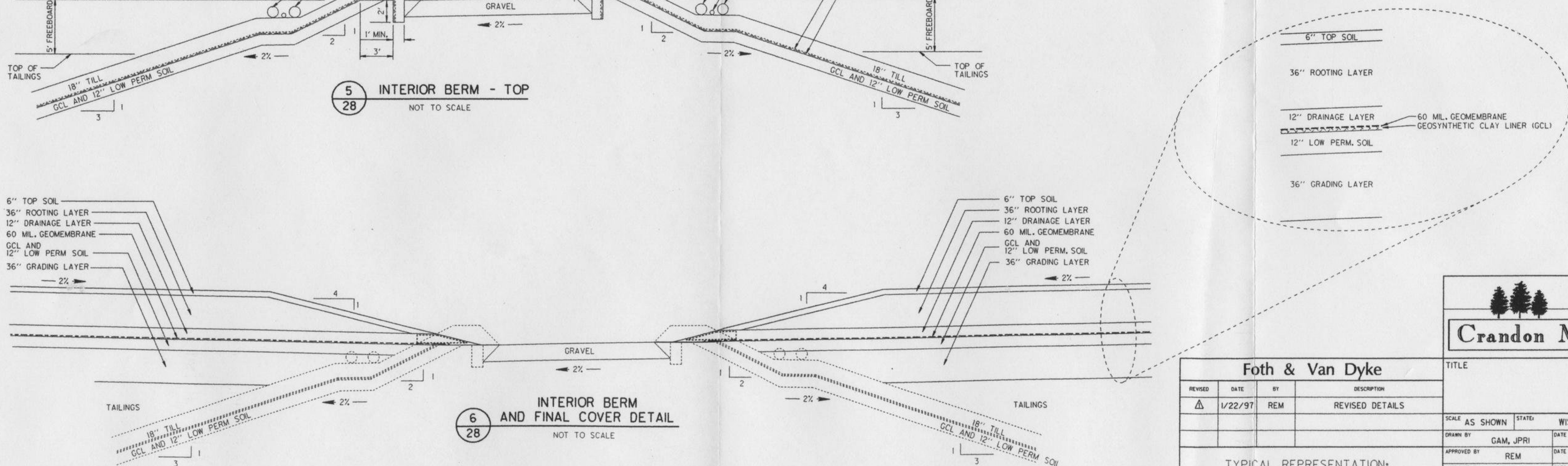
3/28 BENCH BERM FOR INITIAL STAGE
NOT TO SCALE



4/28 BENCH BERM SPLICE DETAIL
NOT TO SCALE



5/28 INTERIOR BERM - TOP
NOT TO SCALE



6/28 INTERIOR BERM AND FINAL COVER DETAIL
NOT TO SCALE

Crandon Mining Company

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	1/22/97	REM	REVISED DETAILS

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

TITLE
DETAILS

SCALE AS SHOWN	STATE: WISCONSIN	COUNTY FOREST	
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI	DATE 5/95
APPROVED BY REM	DATE 5/95	APPROVED BY NXP	DATE 5/95
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM	DATE 5/95
DRAWING NO. 28		SHEET 28 OF 31	REVISION NO. 1

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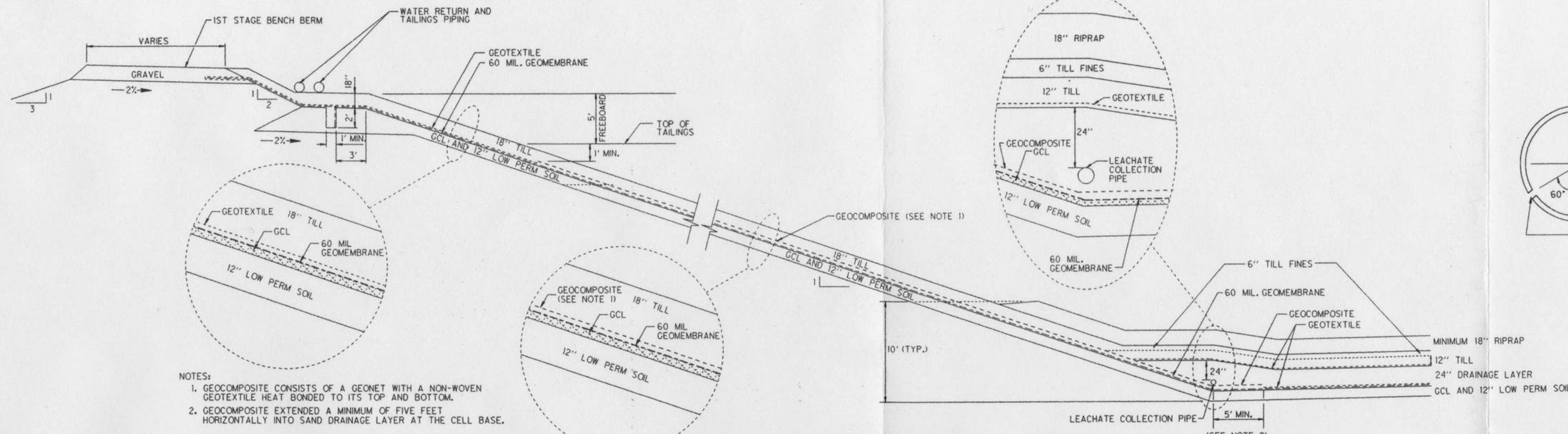
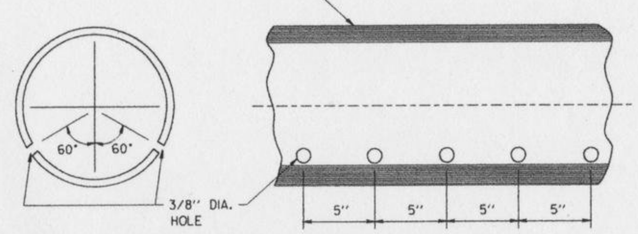
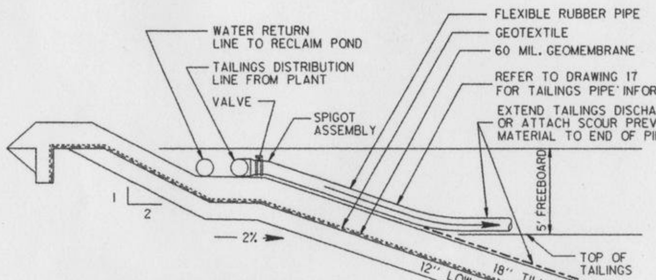


NOTES:
1. GEOCOMPOSITE CONSISTS OF A GEONET WITH A NON-WOVEN GEOTEXTILE HEAT BONDED TO ITS TOP AND BOTTOM.
2. GEOCOMPOSITE EXTENDED A MINIMUM OF FIVE FEET HORIZONTALLY INTO SAND DRAINAGE LAYER AT THE CELL BASE.

1/29 SIDEWALL GEOCOMPOSITE DRAINAGE LAYER - FIRST STAGE
NOT TO SCALE

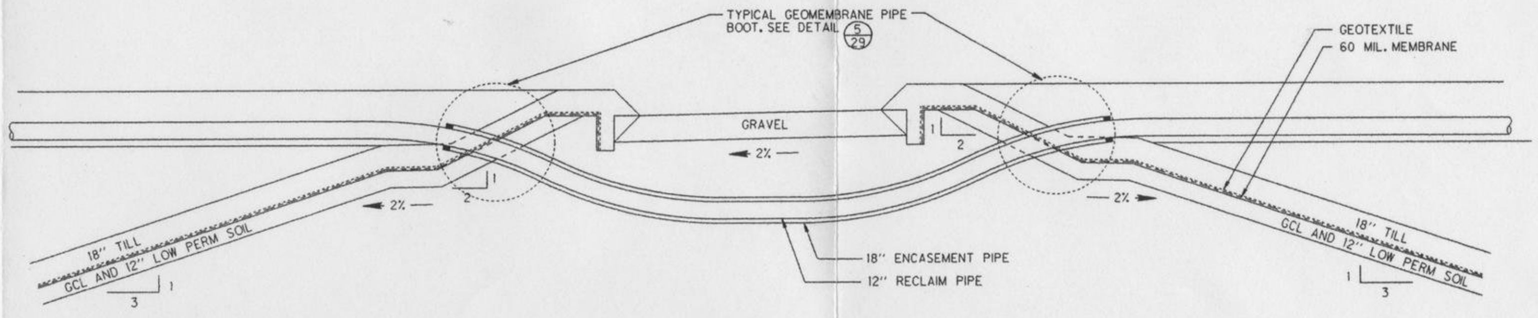


2/29 LEACHATE COLLECTION PIPE PERFORATION DETAIL
NOT TO SCALE

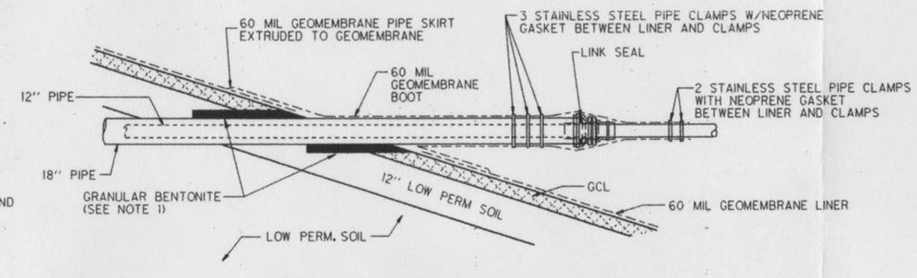


3/29 PERIMETER TOP BERM PIPING DETAIL
NOT TO SCALE

NOTE:
1. SPIGOT PIPE EXTENDS TO THE TOP OF THE TAILINGS AND IS CUT OFF OR EXTENDED AS REQUIRED TO DISCHARGE TAILINGS ON TO THE TAILINGS BEACHES.

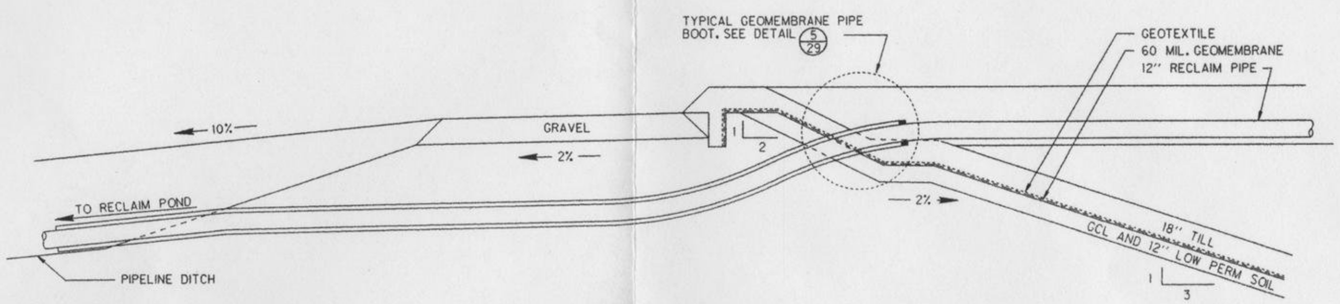


4/29 RECLAIM PIPE SECTION CROSSING INTERIOR BERM
NOT TO SCALE

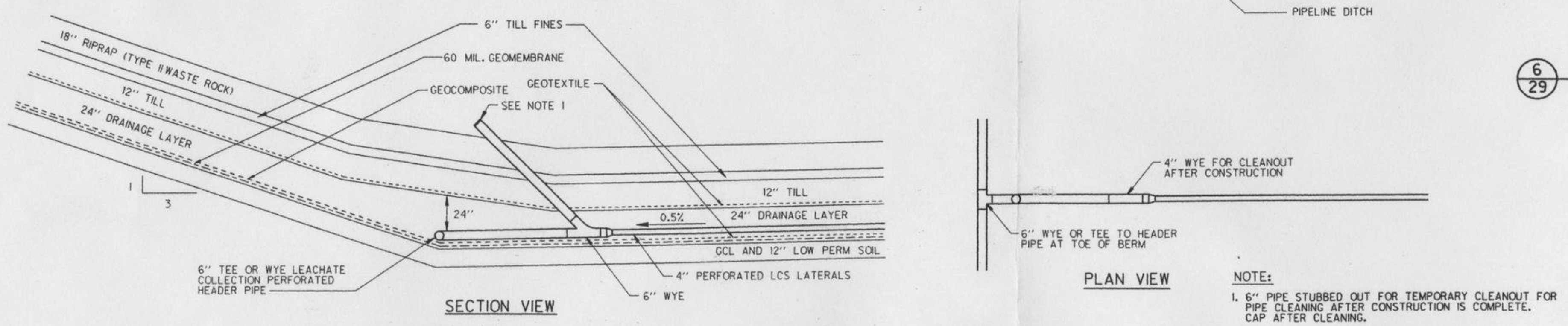


NOTE:
1. COMPACT GRANULAR BENTONITE AROUND PIPE, A MINIMUM ONE INCH THICK AND FOUR INCHES DEEP AROUND THE PIPE.

5/29 GEOMEMBRANE PIPE BOOT DETAIL
NOT TO SCALE



6/29 RECLAIM PIPE SECTION AT PERIMETER BERM
NOT TO SCALE



NOTE:
1. 6" PIPE STUBBED OUT FOR TEMPORARY CLEANOUT FOR PIPE CLEANING AFTER CONSTRUCTION IS COMPLETE. CAP AFTER CLEANING.

7/29 LEACHATE COLLECTION LATERAL DETAIL
NOT TO SCALE

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
⚠	1/22/97	REM	REVISED DETAILS

TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE: DETAILS

SCALE: AS SHOWN	STATE: WISCONSIN	COUNTY: FOREST	
DRAWN BY: GAM, JPRI	DATE: 5/95	CHECKED BY: JKSI	DATE: 5/95
APPROVED BY: REM	DATE: 5/95	APPROVED BY: NXP	DATE: 5/95
APPROVED BY: GWS	DATE: 5/95	CRANDON MINING COMPANY: DEM	DATE: 5/95
DRAWING NO.: 29		SHEET 29 OF 31	REVISION NO.: 1

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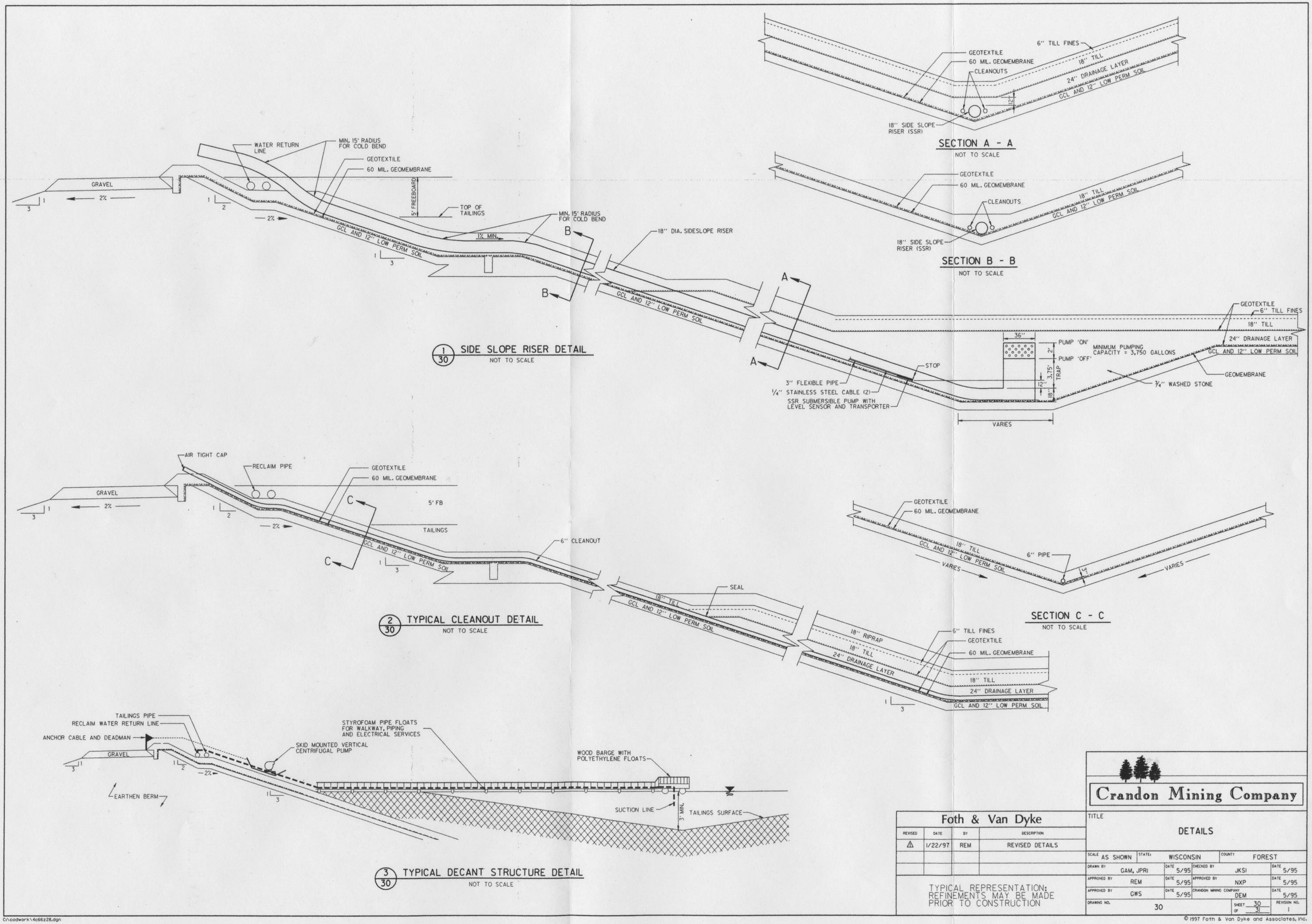
6" TILL FINES
GEOTEXTILE
60 MIL. GEOMEMBRANE
CLEANOUTS
18" TILL
24" DRAINAGE LAYER
GCL AND 12" LOW PERM SOIL
12"
18" SIDE SLOPE RISER (SSR)
SECTION A - A
NOT TO SCALE
GEOTEXTILE
60 MIL. GEOMEMBRANE
CLEANOUTS
18" TILL
GCL AND 12" LOW PERM SOIL
18" SIDE SLOPE RISER (SSR)
SECTION B - B
NOT TO SCALE
WATER RETURN LINE
MIN. 15' RADIUS FOR COLD BEND
GEOTEXTILE
60 MIL. GEOMEMBRANE
GRAVEL
2%
5' FREEBOARD
TOP OF TAILINGS
MIN. 15' RADIUS FOR COLD BEND
1½ MIN.
GCL AND 12" LOW PERM SOIL
18" DIA. SIDESLOPE RISER
B
A
GEOTEXTILE
6" TILL FINES
18" TILL
24" DRAINAGE LAYER
GCL AND 12" LOW PERM SOIL
36"
PUMP 'ON'
PUMP 'OFF'
2'
3.75' TRAP
12"
18"
MINIMUM PUMPING CAPACITY = 3,750 GALLONS
GEOMEMBRANE
¾" WASHED STONE
STOP
3" FLEXIBLE PIPE
¼" STAINLESS STEEL CABLE (2)
SSR SUBMERSIBLE PUMP WITH LEVEL SENSOR AND TRANSPORTER
VARIES
1/30 SIDE SLOPE RISER DETAIL
NOT TO SCALE
AIR TIGHT CAP
RECLAIM PIPE
GEOTEXTILE
60 MIL. GEOMEMBRANE
GRAVEL
2%
5' FB
TAILINGS
C
GCL AND 12" LOW PERM SOIL
6" CLEANOUT
18" TILL
SEAL
GEOTEXTILE
60 MIL. GEOMEMBRANE
18" TILL
GCL AND 12" LOW PERM SOIL
6" PIPE
VARIES
SECTION C - C
NOT TO SCALE
2/30 TYPICAL CLEANOUT DETAIL
NOT TO SCALE
18" RIPRAP
18" TILL
24" DRAINAGE LAYER
6" TILL FINES
GEOTEXTILE
60 MIL. GEOMEMBRANE
18" TILL
24" DRAINAGE LAYER
GCL AND 12" LOW PERM SOIL
TAILINGS PIPE
RECLAIM WATER RETURN LINE
ANCHOR CABLE AND DEADMAN
GRAVEL
2%
EARTHEN BERM
SKID MOUNTED VERTICAL CENTRIFUGAL PUMP
STYROFOAM PIPE FLOATS FOR WALKWAY, PIPING AND ELECTRICAL SERVICES
WOOD BARGE WITH POLYETHYLENE FLOATS
SUCTION LINE
3' MIN.
TAILINGS SURFACE
3/30 TYPICAL DECANT STRUCTURE DETAIL
NOT TO SCALE
Crandon Mining Company
Foth & Van Dyke
REVISED DATE BY DESCRIPTION
1 1/22/97 REM REVISED DETAILS
TYPICAL REPRESENTATION: REFINEMENTS MAY BE MADE PRIOR TO CONSTRUCTION
TITLE DETAILS
SCALE AS SHOWN STATE: WISCONSIN COUNTY FOREST
DRAWN BY GAM, JPRI DATE 5/95 CHECKED BY JKSI DATE 5/95
APPROVED BY REM DATE 5/95 APPROVED BY NXP DATE 5/95
APPROVED BY GWS DATE 5/95 CRANDON MINING COMPANY DEM DATE 5/95
DRAWING NO. 30 SHEET 30 OF 31 REVISION NO. 1
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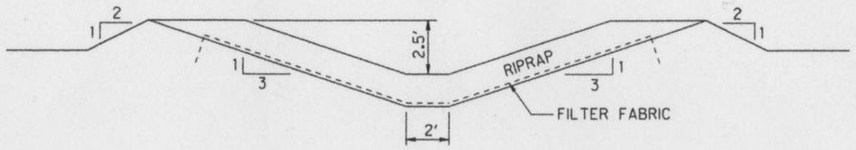
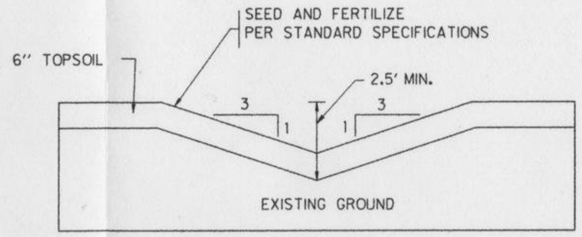
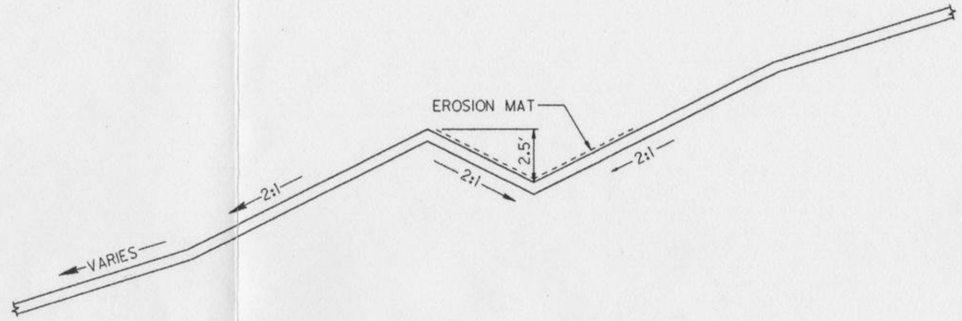


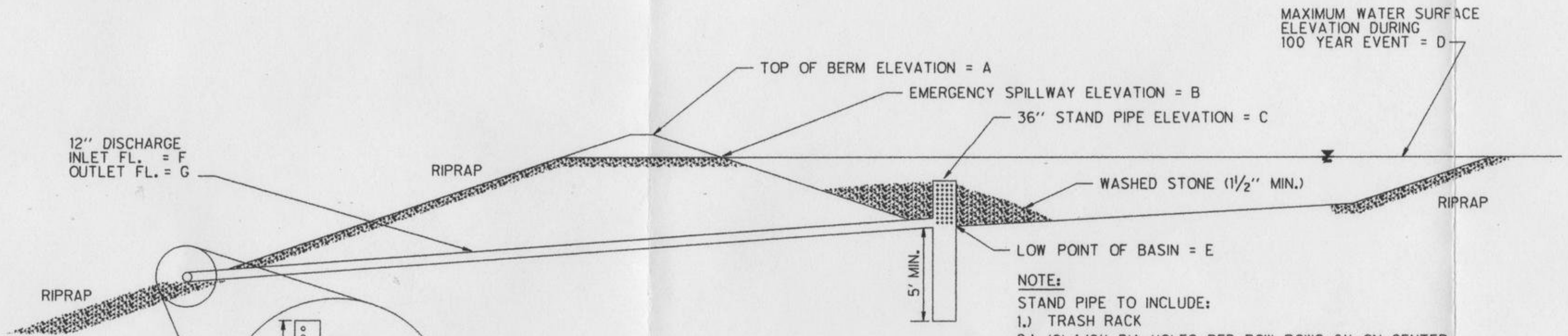
1/31 TYPICAL RIPRAPPED SPILLWAY
NOT TO SCALE



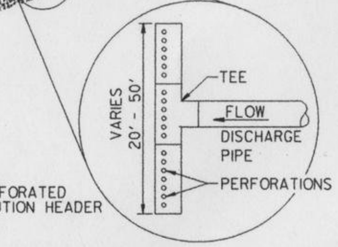
2/31 DRAINAGE DITCH DETAIL ''V'' NOTCH
NOT TO SCALE



3/31 TYPICAL SURFACE WATER DIVERSION BERM
NOT TO SCALE



NOTE:
STAND PIPE TO INCLUDE:
1.) TRASH RACK
2.) (9)-1/2'' DIA. HOLES PER ROW, ROWS 6'' ON CENTER
3.) WRAP WITH AKSD ENKADRAIN 9120 OR EQUAL



TYPICAL PERFORATED
TEE DISTRIBUTION HEADER

	A	B	C	D	E	F	G	
RUNOFF BASIN 8	1630.00	1628.80	1627.00	1628.72	1622.00	1622.00	1620.00	
RUNOFF BASIN 9	1695.00	1694.30	1694.00	1694.29	1690.00	1690.00	1689.00	
RUNOFF BASIN 10	1652.00	1650.90	1649.50	1650.90	1645.00	1645.00	1644.50	1
RUNOFF BASIN 11	1670.00	1668.72	1666.00	1668.72	1661.00	1661.00	1660.00	1
RUNOFF BASIN 12	1702.00	1701.10	1700.50	1701.04	1696.00	1696.00	1695.50	
RUNOFF BASIN 13	1624.00	1623.30	N.A.	1623.4	1619.00	N.A.	N.A.	1

N.A. - STAND PIPE AND DISCHARGE STRUCTURES ARE NOT APPLICABLE TO RUNOFF BASIN 13

4/31 TYPICAL SECTION - RUNOFF BASIN
NOT TO SCALE

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
1	1/22/97	GAM	ADDED BASIN 13, MOVED 10 AND 11

TYPICAL REPRESENTATION:
REFINEMENTS MAY BE MADE
PRIOR TO CONSTRUCTION

Crandon Mining Company

TITLE
DETAILS

SCALE AS SHOWN	STATE WISCONSIN	COUNTY FOREST	
DRAWN BY GAM, JPRI	DATE 5/95	CHECKED BY JKSI	DATE 5/95
APPROVED BY REM	DATE 5/95	APPROVED BY NXP	DATE 5/95
APPROVED BY GWS	DATE 5/95	CRANDON MINING COMPANY DEM	DATE 5/95
DRAWING NO. 31		SHEET 31 OF 31	REVISION NO. 1

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01/24/97 gam

Appendix A

Updated Appendix O from May 1995 Feasibility Report
Construction Quality Assurance Plan
Updated January 1997

Crandon Mining Company

Tailings Management Area (TMA) Construction Quality Assurance Plan Updated January 1997

Contents

Tables

Attachments

1 Introduction

1.1 Summary

The following sections define the Construction Quality Assurance (CQA) program for construction as required by § NR 182, Wis. Admin. Code. This plan is followed during construction to monitor and confirm that the construction features are constructed in accordance with the design and regulatory requirements.

1.2 Purpose and Scope

The purpose of the CQA plan is to provide minimum requirements for construction observation, testing, and documentation activities performed during construction. The purpose is to document that the constructed facility will meet or exceed all design requirements, specifications, regulatory and local approvals. The plan outlines the various organizations and their responsibilities involving the implementation and review of the various CQA activities. The plan also outlines sampling and testing programs to be carried out during the construction. The primary goal of the CQA plan is to provide a means of evaluating and controlling the quality of the constructed facility so that the intent of the design is met.

1.3 Applicable Units

This CQA plan applies to liner and final cover construction of Cells 1, 2, 3 and 4 of the Tailings Management Area (TMA) of Crandon Mining Company's (CMC) Crandon Project.

1.4 Design Summary

The liner design consists of three composite liner configurations, one for the base which includes a leachate drainage layer and one for the interior slopes with and without a leachate drainage layer. The three composite liner systems have the following components listed from top to bottom:

- Base Composite Liner Configuration

  - 18" riprap (Type II mine waste rock)
  - 6" Filter Layer (till fines)
  - 12" Glacial Till Layer (till fines)
  - Geotextile (filter)
  - 24" Gravel Drainage Layer
  - Geotextile (cushioning)
  - 60 mil HDPE Geomembrane
  - Geosynthetic Clay Liner (GCL)
  - 12" P40 Till Soil

- Interior Slope Composite Liner With Leachate Drainage Layer (first stage of each cell)

  - 18" Till Layer
  - Geocomposite (drainage layer)
  - 60 mil HDPE Geomembrane (textured on both sides)

- Geosynthetic Clay Liner (GCL)
- 12" P40 Till Soil Layer

- Interior Slope Composite Liner Without Leachate Drainage Layer (second stage of each cell)

  - 18" Riprap (only on the interior slopes where required during final tailings deposition)
  - 18" Glacial Till
  - Geotextile (cushioning)
  - 60 mil HDPE Geomembrane (textured on both sides)
  - Geosynthetic Clay Liner (GCL)
  - 12" P40 Till Soil Layer

The final cover design consists of a composite cover with the following components (listed from the top to the bottom):

- 6" Topsoil
- 36" Rooting Layer
- 12" Drainage Layer
- 60 mil HDPE Geomembrane
- Geosynthetic Clay Liner (GCL)
- 12" P40 Till Soil Layer
- Grading Layer (thickness varies)

Leachate collection system piping, reclaim water piping and tailings delivery and distribution piping will also be installed generally including the following:

- Leachate collection system piping

  - 6" perforated leachate collection system piping along the toe of the interior slope
  - leachate cleanout risers
  - sideslope risers
  - leachate transfer line piping
  - 4" perforated leachate collection system laterals across the cell base

- Reclaim water piping

  - reclaim suction line
  - reclaim force main to process water pond

- Tailings delivery piping

- Tailings distribution piping

  - mass discharge piping and fittings
  - spigotting piping and fittings

2 Responsibility and Authority

2.1 Permitting Agencies

The Wisconsin Department of Natural Resources (WDNR) has the regulatory authority for approval or denial of the development and operational permits required for the Crandon Project, TMA.

2.2 Facility Owner/Operator

CMC is responsible for the design, construction and operation of the facility in compliance with the regulatory requirements. CMC has the control of organizations charged with design, CQA, and construction activities. CMC will provide a construction manager who will be representing CMC in all construction related issues.

2.3 Design Engineer

Foth & Van Dyke and Associates Inc. (F&VD) has the primary responsibility for designing the facility to meet the design and operational requirements of WDNR and CMC. CMC is responsible for providing a design engineer to make necessary design changes if construction problems and/or material changes are identified in the field. CMC is also responsible for reporting and receiving approval from WDNR on any design modifications.

2.4 Construction Contractors

The various construction contractors are responsible for constructing the facility in strict compliance with the design criteria, plans and specifications, local and WDNR approvals. Construction contractors may implement their own quality control program for purposes of monitoring their related construction. The CQA program presented in this document provides the minimum standards for the acceptance of work.

2.5 Construction Quality Assurance Consultant

The CQA Consultant selected by CMC is independent from CMC's project manager, manufacturer(s) of materials used, and contractors. The CQA Consultant is responsible for observing and documenting all construction activities including all required quality control testing and for quality assurance inspections and tests. The CQA Consultant may engage the services of independent consultants to perform the following:

- Survey documentation.
- Soil laboratory testing.
- Geosynthetic laboratory testing.
- Testing and/or inspection of other construction materials.

Independent consultants will report directly to the CQA Consultant.

2.5.1 Construction Quality Assurance Officer

Prior to the start of construction, the CQA Consultant shall designate a person as the CQA Officer. The CQA Officer will be a professional engineer registered in the State of Wisconsin. The CQA Officer is responsible for supervising all the inspection and testing quality assurance/quality control (QA/QC) requirements of this section. The CQA Officer is also responsible for the preparation of a construction certification report following each construction phase.

The specific responsibilities for administering the QA/QC program are the responsibility of the CQA officer and include the following:

- Reviewing plans and specifications for clarity and completeness.
- Educating and training QA/QC personnel on requirements and procedures outlined in the CQA Plan.
- Scheduling and coordinating QA/QC activities.
- Supervising CQA field personnel.
- Confirming that QC data are accurately recorded and maintained.
- Confirming that the correct Quality Control (QC) procedures are used.
- Verifying that raw QA data are properly recorded, reduced, summarized and interpreted.
- Providing associated organizations with reports on QA/QC activities and results.
- Identifying non-conforming construction and verifying corrective measures.
- Preparing the Construction Document Report which confirms that the facility has been constructed in substantial compliance with design criteria, plans and specifications, local and WDNR approvals.

2.5.2 Construction Quality Assurance Monitor(s)

The Construction Quality Assurance Monitor(s) (CQAM), under the direct supervision of the CQA Officer, shall be present to perform inspections and testing during the following construction activities:

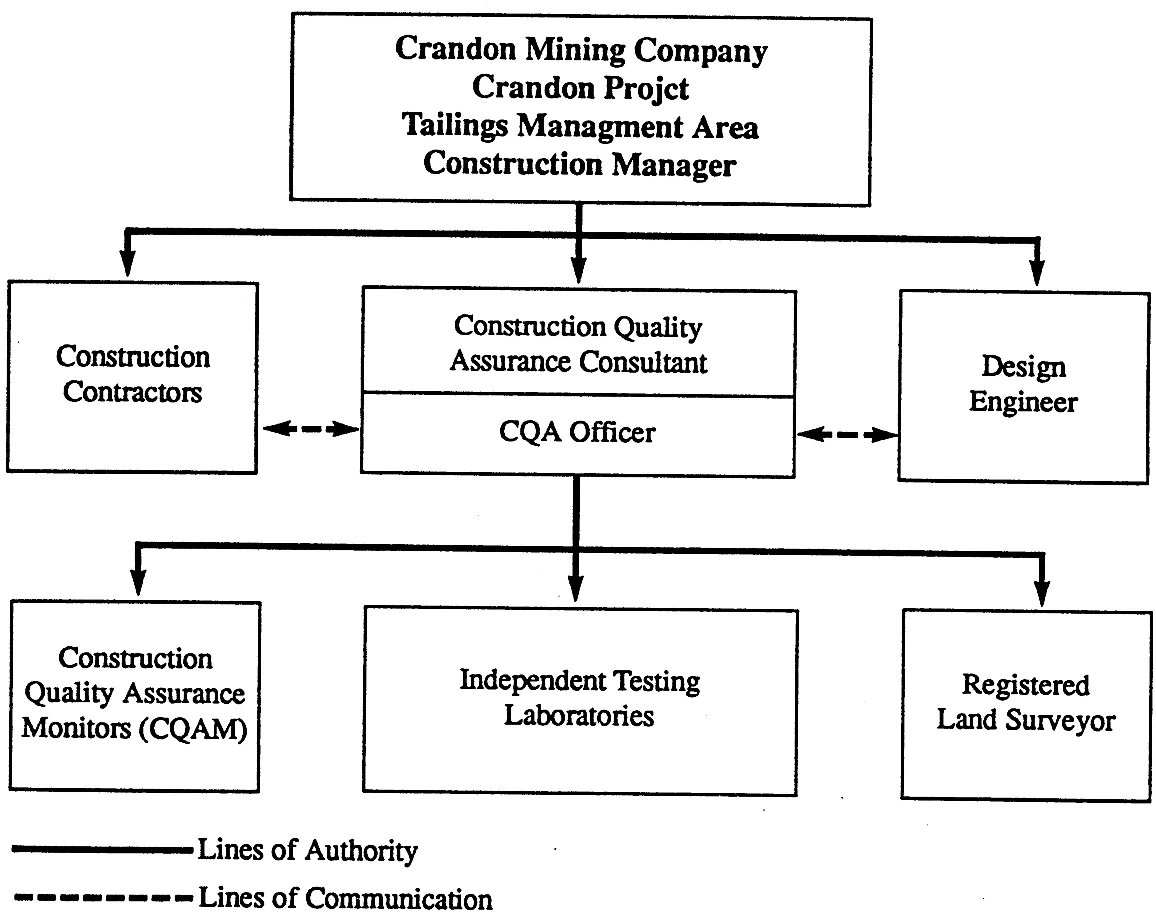
- Construction of the TMA berms and liner support soils
    - Placement of the P40 till soils.
    - Installation of GCLs, geomembranes, geotextiles, and geocomposites.
    - Placement of drainage layer soils.
    - Installation of leachate collection and transfer piping.
- Construction of final cover.
- Construction of surface water control features.
- Reclamation activities.

3 Lines of Communication/Project Meetings

3.1 Lines of Communication

The typical lines of communication necessary during the construction activities are illustrated in the flow chart below.

Lines of Communication for Crandon Mining Company



The CQA Consultant may engage independent consultants for completing the required survey documentation and quality control testing. Quality Control testing includes but is not limited to the following: laboratory soil testing, in-field geosynthetic and soil testing, and laboratory geosynthetic testing.

3.2 Project Meetings

Project meetings are held at regularly scheduled intervals during the course of the project to enhance communication between the organizations involved and to strengthen the responsibilities and authorities of each organization. The CQA Officer is responsible for coordinating and conducting these meetings.

3.2.1 Daily Progress Meetings

A progress meeting is held daily at the work area just prior to commencement or following completion of work. At a minimum, the meeting will be attended by the construction contractor and the QA/QC personnel. This meeting is documented by a member of the CQA Consultant's team. The purposes of the meeting are to:

- Review the previous day's activities and accomplishments.
- Review the work location and activities for the day.
- Identify the contractor's personnel and equipment assignments for the day.
- Discuss any potential construction problems.
- Identify non-conforming construction and determine appropriate corrective measures.

3.2.2 Meetings Regarding Non-Conforming Construction and/or Construction Problems

A special meeting will be held when a construction problem or non-conformance is present or is likely to occur. At a minimum, the meeting will be attended by the construction contractor and CQA Officer. This meeting is documented by the CQA Officer. The purpose of the meeting is to define and resolve problems or recurring work deficiencies.

3.2.3 Preconstruction Meeting

A preconstruction meeting is held prior to construction and prior to each substantial phase of construction, and shall be attended by all parties [contractors, CMC, CQA Officer (or designated CQAM), and Design Engineer]. The meeting is documented by a designated secretary, and minutes will be transmitted to all parties. The purposes of this meeting are to:

- Provide each party with all relevant QA/QC documents and supporting information.

- Familiarize each party with the site-specific QA/QC plan and its role relative to the design criteria, plans, and specifications.

- Determine any changes to the QA/QC plan that are needed to ensure that the facility will be constructed to meet or exceed the specified design.

- Review the responsibilities of each party.

- Review lines of authority and communication for each party.
- Discuss the established procedures or protocol for observations and tests including sampling strategies.
- Discuss the established procedures or protocol for handling construction non-conformance, repairs, and retesting.
- Review methods for documenting and reporting test data.
- Review methods for distributing and storing documents and reports.
- Review compliance of CQA plan with the WDNR and other regulatory approvals.

4 Construction Observation - Record Keeping

4.1 Daily Inspection Report

The CQAM(s) collects all of the samples and performs all of the Quality Control (QC) testing required by the CQA Plan. A daily inspection report is prepared by each inspector for each day of activity. The report contains, at a minimum, the following information:

- Date.
- Type of inspection.
- Summary of weather conditions.
- Summary of any meetings held and attendees.
- Equipment and personnel on the project.
- Summary of construction activities and locations.
- Description of off-site materials received.
- Calibration and recalibration of test equipment.
- Description of procedures used.
- Test locations, procedures, results and test data sheets.
- Summary of samples collected.
- Personnel involved in inspection and sampling activities.
- Signature of the inspector.
- Description of delays in construction activities.
- Detailed description of any problems or non-conforming construction.

4.2 Daily Summary Report

The CQA Officer or the CQAM, under the direct supervision of the CQA Officer, shall prepare a daily summary report which at a minimum contains the following:

- Date.
- Summary of weather conditions.
- Summary of location where construction is occurring.
- Contractors, equipment and personnel on the project.
- Summary of any meetings held and attendees.
- Description of all materials used and references or results of testing and documentation.
- Calibration and recalibration of test equipment.
- Daily inspection reports from each CQAM.
- Description of any construction not meeting the project requirements and how it was corrected.

4.3 Photographs

Photographs shall be obtained for all items of construction. A sufficient number of photographs shall be obtained to document the construction of each construction item (i.e., liners, covers, piping, diversion berms, geosynthetics, etc.). Each photograph shall be a dated 35 mm photograph and shall be recorded in a Photo Log (see Attachment No. 1).

Construction problems and non-conforming work shall be documented with photographs taken before and after the problem or non-conforming work is corrected.

4.4 Test Data Sheets

CQAM will record all test data results on the test data sheets provided in Attachment No. 1. Independent consultants engaged by the CQA Consultant shall submit their test results or data on forms acceptable to and approved by the CQA Consultant.

4.5 Document Control and Record Storage

4.5.1 Daily Records

The daily records maintained during construction activities include, but are not limited to the following:

- Daily inspection reports.
- Daily summary reports.
- Test data sheets from CQAM.
- Test data or documentation data sheets from independent consultants (if any).
- Field book maintained by each CQAM.

Daily records will be copied and forwarded to the CQA Officer on a daily basis.

4.5.2 Storage of Records

All document originals listed in 4.5.1 above will be stored in fireproof cabinets at the construction site. Copies of all documents will be on file at the CMC field office in Crandon, Wisconsin and the CQA Officer's office.

4.6 Construction Problem Identification and Design Changes

4.6.1 Construction Problem Identification

A Non-Conformance Notice is filled out when a deviation from the design plans and specifications, or non-conforming work, is observed by CQA personnel. An example of the form is in Attachment 1. The CQA Officer indicates the extent of work required to correct the deficiency and evaluates test results to determine if there were testing errors. These reports must be cross-referenced to laboratory test results, specific test data sheets, and daily summary reports and include all necessary information listed in Section 4.6.2 - Design Changes.

4.6.2 Design Changes

No changes in design will be made in the field without first completing and submitting a Field Modification form for approval by the Design Engineer and CMC. The CQA Officer and Design Engineer shall determine if WDNR approval is required for the proposed design change by contacting the WDNR representative. If WDNR approval is required the CQA Officer shall prepare the information required for a plan modification to the proposed design. If a plan modification submittal is not required, the CQA Officer shall complete the Field Modification form; examples of which are provided in Attachment 1. The Field Modification form shall include the following:

- Original design given by project plans and specifications.

- Reason(s) for required change in design.

- Proposed design changes including changes in materials, quantities, construction methods, and constructed performance.

- Supporting calculations.

4.6.3 Design Change Construction Documentation

Construction due to design changes shall be documented by the CQA Officer in the Daily Summary Report. All materials, quantities, and construction methods required for the design change shall be recorded. Documentation shall, in general, meet the requirements of Sections 5, 6 and 7.

5 Construction Observation - Testing and Verification

This section outlines minimum requirements for the testing and verification of the components of the liner and final cover system.

5.1 Survey Verification

Record surveys shall be performed by a professional land surveyor registered in the State of Wisconsin. At a minimum, the surveys shall document the following:

Liner and Leachate Collection System

- Subbase grades (bottom of a P40 till soils layer) on a 100-foot grid.
- Top of final surface on berm and embankment grades on a 200-foot grid.
- Top of P40 till soils on a 100-foot grid.
- Top of drainage layer on a 100-foot grid.
- Top of till soils on a 100-foot grid.
- Leachate pipe elevations at 25 foot interval.
- Leachate sump subbase grade and final grade elevation.
- Sideslope riser and cleanout location and elevations.

Cover System

- Top of final tailings or mine waste surface on 100-foot grid.
- Bottom of P40 till soils on 100-foot grid.
- Top of P40 till soils on 100-foot grid.
- Top of drainage layer on 100-foot grid.
- Top of rooting layer on 100-foot grid.
- Top of topsoil layer on 100-foot grid.

5.1.1 Tolerances

The vertical grading tolerance for each soil layer of liner and cover shall be at design grades to 0.1 foot above.

5.2 Thickness Verification

The CQAM(s) shall verify the thickness of the soil components on a 100-foot grid interval. The method of verification may include survey, use of settlement plates, hand augers or hand shoveling. Verification must be done in a manner which does not harm the geosynthetic components of the liner or cover.

5.2.1 Tolerances

Thickness of individual soil components must meet the following minimum tolerances:

Liner System	Minimum Thickness
1. P40 Till Soil Layer	12"
2. Drainage Layer	24"
3. Till Soil Layer	18" (on slope) 12" (on base only)
4. Fines from Till Processing	6" (on base only)
5. Leachate Pipe Bedding	6"
Cover System	
1. Grading Layer (thickness varies as required to attain a minimum 2% slope as per the drawings)	
2. P40 Till Soil Layer	12"
3. Drainage Layer	12"
4. Rooting Layer	36"
5. Topsoil Layer	6"

5.3 Soil Testing

The CQAM(s) shall collect samples of each soil component in accordance with Table 5-1. The samples will be sent to the selected soils laboratory for testing.

The CQAM(s) will also be responsible for all in-place testing in accordance with Table 5-1.

The following structures shall be constructed and sampled according to the CQA program in this section:

- Compacted subgrade and embankments.
- P40 Till soil layer.
- Geosynthetic Clay Liner (GCL).
- 60 mil HDPE Geomembrane.
- Drainage layer.
- Geotextile.
- Topsoil and rooting layer.

The material acceptability criteria for placed materials tested for quality assurance purposes shall be a maximum of ten (10) percent outliers. No outliers shall be accepted for workmanship acceptance (compaction, geomembrane installation, etc.).

Table 5-1

Construction Quality Assurance Testing Program for Soil Components
Crandon Project

Soil Component	Test/observation	Test Method	Min. Freq.	Acceptance Criteria
Liner System				
Liner Support Soils	In-place density	ASTM D2922	1/acre/ft	Percent Compaction Min. 95% (Standard Proctor, ASTM D698)
Drainage Layer (base)	Grain Size	ASTM D422	1/5000 cy	P200 <5%, C_u <4
	USCS Soil Class	ASTM D2487	1/5000 cy	GP/GW
	Hydraulic Conductivity[1]	ASTM D2434	1/10,000 cy	Min. 1 x 10^{-2} cm/sec
	Thickness	NA	200-foot grid	24" Min. LCS
Pipe Backfill	Grain Size	ASTM D422	1/1,000 LF	P200 <5%
Cover System				
Grading Layer	In-Place density	ASTM D2922	1/acre/ft	Percent Compaction Min. 95% (Standard Proctor, ASTM D698)
P40 Till Soil Layer	Grain Size	ASTM D422	1/acre/ft	P400 = 100%, P200 ≥18%, and Cu >6
	USCS Soil Class	ASTM D2487	1/acre/ft	ML or SM
	In-place Density	ASTM D2922	200-foot grid/ft	Min. 95% of ASTM D698
	In-place Moist. Cont.	ASTM D2922	200-foot grid/ft	See note 2.
	Stand. Proctor	ASTM D698	1/20,000 c.y.	NA
	Thickness	NA	100-foot grid	12" Min.
Drainage Layer	Grain size	ASTM D422	1/5000 cy	P200 <5%, C_u <4%
	USCS Soil Class	ASTM D2487	1/5000 cy	GP/GW
	Hydraulic Conductivity[1]	ASTM D2434	1/10,000 cy	Min. 1 x 10^{-2} cm/sec
	Thickness	NA	200-foot grid	12" Min. FCDL
Topsoil	pH, nitrogen, phosphorus and potassium, USCS classification		1/5 acres	NA

1. Recompacted Sample
2. -1% to +5% about the optimum moisture content as determined by ASTM D698.

LCS - Leachate Collection System
cm/sec - centimeters per second
cy - cubic yard
lf - lineal feet
C_u - Coefficient of Uniformity

FCDL - Final Cover Drainage Layer
P200 - percent passing the Number 200 Sieve
LL - liquid limit
PL - plastic limit
NA - not applicable

Prepared by: REM
Checked by: NXP

5.4 CQA Officer Inspection of Subgrade and Foundation

The CQA officer shall perform the following functions:

- Verify that the site conditions are, in general, as interpreted in the plans.
- Ensure that there are no moisture seeps. If *in situ* material provides insufficient strength (i.e., it is soft and yielding and ruts during proof rolling) the soft material shall be removed and replaced with compacted till materials. This material shall be compacted to achieve the density properties necessary listed in Table 5-1.
- Ensure that all trees, stumps, roots, boulders and debris are removed.
- Prohibit the placement of frozen soil. Prohibit the placement of soil onto frozen ground.
- Ensure that the foundation is constructed and graded to provide a smooth, workable surface on which to construct the liner.
- Ensure that the liner base and embankments are graded to the correct elevation and slope.

6 Construction Observation - Geosynthetics

The following section summarizes the quality assurance plan proposed for testing and monitoring of the GCL and geomembrane liner installation.

6.1 On-Site Quality Assurance - Geosynthetic Clay Liner (GCL)

6.1.1 GCL Rolls and Panels

Construction quality assurance monitoring for the rolls and panels include:

1. Monitoring and documenting the unloading of trucks delivering GCL rolls to the site.

2. Monitoring the handling and on-site storage of GCL rolls. GCL rolls should be stored in the designated storage area in accordance with the manufacturer's recommendations.

3. Recording the roll and batch numbers of GCL rolls delivered to the site.

4. Any protective wrapping that is damaged or stripped off must be repaired.

5. If rolls are to be stored for longer than 2 weeks, cover with a tarpaulin which is adequately secured.

6. Review of manufacturer's QA data for conformance with specifications.

7. Selecting samples from GCL rolls delivered to the site for off-site conformance testing. Conformance testing will be performed in accordance with the manufacturer's recommendations for the material selected. An example of a recommended conformance testing program for a type of GCL is outlined in Table 6-1. Samples shall be sent to a geosynthetics testing laboratory for material properties analysis.

8. Fixing a code number to samples and recording the manufacturer's roll numbers of the rolls from which samples are taken.

9. Labeling, packaging and shipping samples to an independent testing laboratory for conformance testing.

10. Interpreting laboratory test results in accordance with the specifications and accepting or rejecting delivered rolls based on results of laboratory testing.

11. Performance of visual review of GCL liner fabricated at the factory as it is unrolled and deployed at the job site for uniformity, damage, and imperfections, including holes, cracks, thin spots, tears, punctures, blisters, and foreign matter.

Table 6-1

Manufacturer's Recommended Quality Control Testing Program for BENTOMAT "ST"®

Material Property	Test Method	Test Frequency, ft^2 (m^2)
Bentonite Swell Index[1]	ASTM D 5890	1 per 50 tonnes[6]
Bentonite Fluid Loss	ASTM D 5891	1 per 50 tonnes
Bentonite Mass/Area[2]	ASTM D 5261	40,000 ft^2 (4,000 m^2)
GCL Grab Strength[3]	ASTM D 4632	200,000 ft^2 (20,000 m^2)
GCL Grab Elongation	ASTM D 4632	200,000 ft^2 (20,000 m^2)
GCL Peel Strength	ASTM D 4632	40,000 ft^2 (4,000 m^2)
GCL Index Flux[4]	ASTM D 5887	Weekly
GCL Hydrated Internal Shear Strength[5]	ASTM D 5321	Periodic

[1] Bentonite property tests performed at CETCO's bentonite processing facility before shipment to CETCO's GCL production facilities.
[2] Bentonite mass/area reported at 0 percent moisture content. The reported value is equivalent to 0.95 psf at 20% moisture content, the GCL industry standard.
[3] All tensile testing is performed in the machine direction, with results as minimum average roll values unless otherwise indicated.
[4] Index Flux with deaired distilled water at 5 psi (35 kPa) confining pressure and 2 psi (15 kPa) head pressure. Reported value is equivalent to 925 gal/acre/day. This flux value is equivalent to a permeability of 5×10^{-9} cm/sec. This flux value should not be used for equivalency calculations. A flux test using gradients that represent field conditions must be performed to determine equivalency. The last 20 values may be reported from the end of the project date of the supplied GCL.
[5] Peak value measured at 200 psf (30 kPa) normal stress. Site-specific materials, GCL products, and test conditions must be used to verify internal and interface strength of the proposed design.
[6] 1 tonne is equivalent to 1.1 tons.

Source: CETCO (Colloid Environmental Technologies Company), May 1, 1996. Technical Data Sheet TR 404bm.

6.1.2 GCL Panel Placement

Quality assurance monitoring for panel placement includes:

1. Obtaining a written acceptance of the subgrade by the GCL installer.

2. Evaluating and documenting weather conditions (*e.g.*, temperature, wind) and subgrade conditions for GCL placement and informing the CMC construction manager if requirements for weather conditions and subgrade are not met, so the CMC construction manager can decide to stop GCL placement.

3. Monitoring and documenting geomembrane placement as well as conditions of panels as placed.

    a. CQAM shall be present at all times during handling, placement, and covering of GCLs.

    b. Noting panel defects, tears or other deformities.

    c. Measuring as-delivered panel lengths and widths.

    d. Inspect subgrade to make sure it conforms to the project requirement, i.e., no rutting greater than 1 inch, no stones in subgrade greater than 1 inch and subgrade to the proper grade rolled smooth with a smooth drum roller.

    e. Verify that manufacturer's required panel overlap for GCL roll edges and ends is maintained (typically 6 to 9 inches for roll edges).

    f. Observe that the correct quantity of powdered bentonite is applied to the panel seams if required.

    g. On interior sideslope, anchor GCL at the top slope and roll the GCL down the slope.

    h. GCL should be free of wrinkles greater than 3 inches.

4. Repairs of GCL

    a. Document repair of all holes, tears, rips, or thin spots in the GCL.

    b. GCL should be patched with materials and as per the manufacturer's recommendation. Patches typically must extend at least 12 inches beyond the damaged area.

5. Covering

    a. GCL must be covered completely with the subsequent layer of the liner system at the end of each working day.

b. Monitor GCL for wrinkles during covering process.

6.2 On-Site Quality Assurance - Geomembrane

6.2.1 HDPE Geomembrane Rolls and Panels

Construction quality assurance monitoring for the rolls and panels include:

1. Monitoring and documenting the unloading of trucks delivering geomembrane rolls to the site.
2. Monitoring the handling and on-site storage of geomembrane rolls.
3. Recording the roll and batch numbers of geomembrane rolls delivered to the site.
4. Review of manufacturer's QA testing for conformance with specifications.
5. Selecting samples from geomembrane rolls delivered to the site for off-site conformance testing. Conformance testing will be performed as outlined in Table 6-2. Samples shall be sent to a geosynthetics testing laboratory for material properties analysis.
6. Fixing a code number to samples and recording the manufacturer's roll numbers of the rolls from which samples are taken.
7. Labeling, packaging and shipping samples to an independent testing laboratory for conformance testing.
8. Interpreting laboratory test results in accordance with the specifications and accepting or rejecting delivered rolls based on results of laboratory testing.
9. Performance of visual review of synthetic liner fabricated at the factory as it is unrolled and deployed at the job site for uniformity, damage, and imperfections, including holes, cracks, thin spots, tears, punctures, blisters, and foreign matter.

Table 6-2

60 Mil HDPE* Material Properties

Test	Procedure	Manufacturer Test Frequency	Third-Party Conformance Test Frequency
Density	ASTM D792 or ASTM D1505	(See note 1)	1each/100,000 ft^2
Thickness	ASTM D751		1 each/100,000 ft^2
Yield Strength	ASTM D638 NSF modified		1 each/100,000 ft^2
Yield Elongation	ASTM D638 NSF Modified		1 each/100,000 ft^2
Tensile Strength	ASTM D638 NSF Modified		1 each/100,000 ft^2
Tensile Elongation	ASTM D638 NSF Modified		1 each/100,000 ft^2
Modulus of Elasticity	ASTM D638 NSF Modified		NR
Carbon Black Content	ASTM D1603		NR
Carbon Black Dispersion	ASTM D3105 NSF Modified		NR
Environmental Stress Crack	ASTM D746		NR
Low Temperature Brittleness	ASTM D746		NR
Tear Resistance	ASTM D1004		NR
Dimensional Stability	ASTM D1204		NR
Puncture Resistance	FTMS 10/C or Method 2065		NR

NR - Third-Party Conformance Test not required.

Notes: 1. Manufacturers to provide manufacturer quality control data for all the tests listed for each roll.
2. In addition, the following test shall be performed for each resin that has been used in the manufacture of membranes: Melt Index, Carbon Black Content, and Carbon Black Dispersion.

*High Density Polyethylene (HDPE) is included as an example of material properties which will be provided when the geomembrane is selected in the final design.

6.2.2 Panel Placement

Quality assurance monitoring for panel placement includes:

1. Obtaining a written acceptance of the subgrade by the geomembrane installer.

2. Evaluating and documenting weather conditions (*e.g.*, temperature, wind) for geomembrane placement and informing the CMC construction manager if requirements for weather conditions are not met, so the CMC construction manager can decide to stop geomembrane placement.

3. Monitoring and documenting geomembrane placement as well as conditions of panels as placed.

   a. Noting panel defects, tears or other deformities.

   b. Measuring panel thicknesses at a minimum rate of five areas measured per roll.

   c. Measuring as-delivered panel lengths.

4. Recording the locations of installed panels and checking that the panels have been installed in accordance with the design plan.

   a. Assigning each panel a unique panel number and identifying that panel with the manufacturer's roll number.

   b. Recording panel numbers and locations on a panel layout diagram.

6.2.3 Geomembrane Field Seam Construction

Quality assurance monitoring and testing to be conducted for seam construction includes:

6.2.3.1 Fusion Welds

1. Monitoring trial fusion seams constructed prior to each seaming sequence to evaluate the seaming crew and equipment.

   a. Record machine temperature, ambient temperature, machine speed, seamer identification, machine number, date and time for all trial seams.

   b. Trial seams will be made at the beginning of each seaming period, at the discretion of the CQAM, at least once every four to five hours (minimum two (2) test seams each day) for each seaming apparatus used that day. Each seamer will make at least one (1) test each day. Six (6) specimens will be cut, and four (4) peel tests will be performed on the inside and outside tracks of the weld and two (2) shear tests will be performed. Field samples will be die cut to one-inch widths.

2. Evaluating and documenting trial seam test results in accordance with the specifications and accepting or rejecting seaming crews and/or equipment.

    a. Observing the performance of peel and shear tests on trial seam samples, shear tests must meet a minimum seam strength specified to pass and peel tests must have a film tearing bond failure and the minimum specified peal adhesion strength.

    b. Retain a portion of each trial seam.

3. Evaluating and documenting the suitability of weather conditions (*e.g.*, temperature, wind, humidity) for seaming and informing the CMC construction manager when weather conditions do not meet the specifications so the CMC construction manager can decide to stop geomembrane seaming.

4. Monitoring seam construction.

5. Assigning a seam number to each seam and recording seam construction data, including seam crew identification, machine number, date and time of seam construction, ambient temperature.

    a. Record the location of all seams on a seam layout diagram.

6. Confirming that the installer's field tensiometer has current calibration documentation. At a minimum, the field tensiometer shall have been calibrated within one year prior to start of project.

6.2.4 Seam Testing and Repair

Items included in the quality assurance for monitoring seam testing and repair include:

1. Monitoring and documenting non-destructive testing done to evaluate continuity of all seams.

    a. Observe seam pressure tests in accordance with project specifications.

    b. Observe air lancing tests in accordance with project specifications.

    c. Marking failed seams for repair.

    d. Document repair and retest of seam.

2. Selecting locations where geomembrane samples will be taken to conduct destructive testing.

    a. A minimum of one destructive test sample will be collected for every 500 lineal feet of field seam.

b. Locations of destructive test samples will be noted on a repair and sample location diagram.

3. Monitoring the cutting of samples by the geomembrane installer.

4. Assigning a unique number to each sample and recording sample locations and other pertinent observations made during sampling.

5. Monitoring the cutting of the sample in three parts: one for the geomembrane installer, one for archiving, and one for testing by the independent laboratory.

6. Monitoring and documenting the field seam destructive tests performed by the geomembrane installer.

7. Labeling, packaging and shipping samples to the independent laboratory for destructive testing.

8. Interpreting laboratory test results and accepting or rejecting seams based on independent laboratory test results.

9. Monitoring and documenting patching of holes caused by sampling.

10. Monitoring and documenting the non-destructive testing of the seams associated with seam repair.

11. Monitoring and documenting the repair of the rejected seams and the non-destructive testing of the seam repairs.

    a. Document passing seam tests between all destructive test locations.

    b. Record all seam repair locations.

12. Monitoring and documenting destructive testing related to seam repair.

    a. Monitoring and documenting one destructive seam sample for every 500 lineal feet of repaired seam as described above.

6.2.5 Defect Repairs

The following quality assurance monitoring and testing will be implemented to monitor defect repairs:

1. Performing systematic visual observation of the entire surface of the geomembrane to locate and document defects and indicate for each defect the type of repair that is required.

2. Monitoring and recording the repair of defects and the non-destructive testing of all repairs.

3. Recording the location and the nature of all defect repairs.

6.2.6 Anchor Trenches

Quality assurance associated with monitoring and testing of anchor trenches shall include the following:

1. Anchor trench excavation shall be monitored for proper depth and location.

2. Geomembrane panels extending into the anchor trench shall be monitored for complete seaming within the anchor trench.

3. Anchor trench backfill operations will be observed and documented.

    a. The length of the open trench shall not exceed the amount of liner to be placed in one day.

    b. The depth of a typical anchor trench shall be documented to conform to approved project drawings.

    c. Backfill shall be placed in thin lifts not to exceed one foot in loose thickness.

    d. Compaction of backfill using hand operated compaction equipment to a minimum of 95% of the maximum dry density as determined by the standard Proctor test (ASTM D 698) within -1% to +5% of optimum moisture content.

    e. Density tests will be performed at a minimum interval of 500 lineal feet of anchor trench.

6.3 Documentation and Reporting

Documenting and reporting methods will be implemented to systematically record results of on-site monitoring and on-site testing. Reporting forms will be used for roll and panel placement, trial weld construction, panel seaming, non-destructive seam testing and destructive seam testing. Unique identifying numbers will be assigned to each panel and seam and used to reference the panel and seam location and test results. Copies of quality assurance forms are included in Attachment 2.

A geomembrane installer's certificate of acceptance of the subgrade will be obtained prior to placement of geomembrane panels. A format for the certificate of acceptance is given in Attachment 2.

Panel location and seam location diagrams will be kept showing the location of all panels and seams, repairs and destructive sample test locations. These location diagrams will be updated on a daily basis and will be available for review by the construction manager.

A photo log will be created containing photos of all phases of the geomembrane liner installation, including deployment, seaming, testing, and anchor trench construction.

Copies of test results for all off-site laboratory testing will be forwarded to the on-site supervisor and will be made available to the construction manager. The laboratory test result documents will be maintained in a job file and submitted with the final certification report (see Section 7).

7 Construction Certification Report

7.1 Summary

Upon completion of the construction of each major phase and prior to placing in service, the CQA Officer shall submit a construction certification report to the agency. This report shall be prepared in accordance with Wisconsin Admin. Code §NR 182. The report shall contain, at a minimum, the following information:

- Based on the Owners/Contractors records and data collected through following the QA/QC measures outlined in Sections 5 and 6. The CQA Officer shall certify that the construction has been prepared and constructed in substantial conformance with the engineering plans and specifications.
- Daily summary reports.
- Detailed narrative describing the construction activities in chronological fashion.
- Analysis and discussion of all QA/QC testing performed with summaries of all test results.
- All raw data and test result reports performed during construction.
- Detailed description and documentation of all material and equipment types and specifications.
- Warranties, shop drawings and operating instructions for equipment and materials.
- Discussion of any construction material or equipment which deviated from the engineering plan, and reasons for deviation.
- Photographs documenting all aspects of facility construction.
- Record drawings containing:
  - Record subbase grade (bottom of clay liner) and/or bottom of clay cap elevations.
  - Record base grade (top of clay liner) elevations. This sheet shall also include a table summarizing the thickness of the clay liner at each survey location.
  - Locations/identification numbers for geomembrane panel layout (with size of panels provided in an accompanying table), seam type, repair location, destructive test location, and each panel and anchor trench location.
  - Thicknesses of granular drainage layers.
  - Locations of all soil tests and samples.
  - Cross-sections.

- As constructed details of TMA piping systems including:

  - leachate collection system piping.
  - reclaim return water piping.
  - tailings delivery piping.
  - tailings distribution system piping.

- Details of stormwater management and erosion control structures.

Attachment 1

Example of Forms

PHOTO LOG

Roll Number	Photo Number	Date Taken	Description

[32-15]94L011

Foth & Van Dyke

Client: ______________________ Scope I.D.: __________
Project: ______________________ Page: __________
Prepared by: ______________________ Date: __________

NON-CONFORMANCE NOTICE

DATE: ______________________ CONTRACT: ______________________
CONTRACTOR: ______________________ FOREMAN: ______________________
STREET: ______________________

- ☐ PIPELINE INSTALLATION FROM STATION __________ TO STATION __________
- ☐ COMPACTION FROM STATION __________ TO STATION __________ TYPE __________
- ☐ MANHOLE INSTALLATION AT STATION __________
- ☐ HYDRANT INSTALLATION AT STATION __________
- ☐ VALVE INSTALLATION AT STATION __________
- ☐ EXISTING WATERMAIN REPAIR __________
- ☐ EXISTING SEWER REPAIR __________
- ☐ SEWER LATERAL FOR __________
- ☐ WATER SERVICE FOR __________
- ☐ CATCH BASIN INSTALLATION AT STATION __________
- ☐ CURB AND GUTTER STATION __________ TO STATION __________
- ☐ SIDEWALK STATION __________ TO STATION __________
- ☐ CRUSHED AGGREGATE BASE STATION __________ TO STATION __________
- ☐ BACKFILL STATION __________ TO STATION __________
- ☐ OTHER __________

The above checked item(s) is not in conformance with the plans and specifications and will be deleted from future payment requests until the work is corrected.

COMMENT: ______________________

RESIDENT INSPECTOR: ______________________

DISTRIBUTION: ORIGINAL - CONTRACTOR CANARY - OWNER PINK - RESIDENT ENGINEER GOLD - INSPECTOR

Foth & Van Dyke

Client: ______________________ Scope I.D.: __________
Project: ______________________ Page: __________
Prepared by: ______________________ Date: __________
Checked by: ______________________ Date: __________

FIELD MODIFICATION

Addendum No.: __________

Description of Modification: ______________________

Basis for Modification: ______________________

ATTACHMENTS

Revised Drawing: ______________________

New Drawing: ______________________

Drawn by: ______________________ Date: __________
Checked by: ______________________ Date: __________
Approved by: ______________________ Date: __________

Copies to: ______________________

FORM EES012 (1/91)

Foth & Van Dyke	Client: ______ Scope I.D.: ______ Project: ______ Page: ______ Prepared by: ______ Date: ______

RESIDENT INSPECTION REPORT - SITE

Location: ______

Weather	Temp.		Sky	Rain	Snow	Project Site	
	Lo	Hi				Dry	Muddy

Contractors on Site:	Personnel (#)

Other Personnel on Site:	Purpose

Report of Observation of Work and Comments: ______

Additional Space on Back

FORM 171 (Rev. 9/90)

DENSITY TESTS OF COMPACTED FILL

Contractor: ______________________

Compaction Equipment: ______________________

____

Weather: ______________________

Method of Test:

☐ Nuclear Meter *(ASTM: D2922)*

☐ Sand Cone *(ASTM: D1556)*

Area/Location Tested: ______________________

Report Number: ______________________

Test No.	Test Location	Depth of Probe	Elevation	Proctor Used	Wet Density *(pcf)*	Moisture Content *(%)*	Dry Density *(pcf)*	Percent Compaction	Remarks

Proctor No.	Soil Classification	Max. Dry Density *(pcf)*	Moisture Content *(%)*	Compaction Spec. *(%)*

General Note:
Density test results are valid only at the locations and elevations tested.

Nuclear Meter Used: ______________________

Model: __________ Serial No.: __________

Standard Counts: ______________________

Density: __________ Moisture: __________

Foth & Van Dyke

Client: ______________________ Scope I.D.: __________

Project: ______________________ Page: __________

Prepared by: ______________________ Date: __________

Checked by: ______________________ Date: __________

Attachment 2

Geosynthetic CQA Forms

Foth & Van Dyke

PANEL PLACEMENT INFORMATION

PROJECT NO. ________

DATE ________

PAGE ____ OF ____

PANEL NUMBER	ROLL NUMBER	TIME	PANEL SIZE	PANEL LOCATION	SUBGRADE CONDITIONS	AMBIENT TEMP.	WEATHER/ WINDS	PANEL DAMAGE	QA MONITOR	COMMENTS

Foth & Van Dyke

NON-DESTRUCTIVE SEAM TEST INFORMATION

PROJECT NO. ______

DATE ______

PAGE ______ OF ______

TIME	SEAM NUMBER	TESTER INITIALS	AIR TESTING					PASS/FAIL	VACUUM BOX PASS/FAIL	QA MONITOR	COMMENTS
			PRESSURE (PSI)			TIME					
			START	END	DROP	START	END				

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PANEL SEAMING CHECKLIST

PROJECT NO. ________

DATE ________

PAGE ______ OF ______

START TIME	WEATHER/ WINDS	TEMPERATURE		SEAMER INITIALS	MACHINE NUMBER	TEMP. SETTING	SPEED SETTING	SEAM TYPE	SEAM NUMBER	PANEL NUMBERS	SEAM LENGTH	END TIME	QA MONITOR	COMMENTS
		AMBIENT	SHEET											

Foth & Van Dyke

TRIAL WELD INFORMATION

PROJECT NO. ____________

DATE ____________

PAGE ______ OF ______

TIME	WEATHER/ WINDS	ABBIENT TEMP.	SEAMER INITIALS	MACHINE NUMBER	EXTR. WELDS		FUSION WELDS			SEAM TEST		PASS/ FAIL	QA MONITOR	COMMENTS
					BARREL TEMP.	PREHEAT TEMP.	WEDGE TEMP.	SPEED SETTING	WHEEL TENSION SETTING	PEEL (LBS/INCH)	SHEAR (LBS/INCH)			

Foth & Van Dyke

DESTRUCTIVE SEAM TEST INFORMATION

PROJECT NO. ____________

DATE ____________

PAGE ______ OF ______

TIME	SAMPLE IDENTIFICATION	SEAM NUMBER	TESTER INITIALS	FIELD SEAM STRENGTH		PASS/ FAIL	DATE TO LAB PKG. SLIP NO.	LAB PASS/FAIL	QA MONITOR	COMMENTS
				PEEL (LBS/INCH)	SHEAR (LBS/INCH)					

Appendix B

GCL Information from *Geotechnical Fabrics Report* "*1997 Specifier's Guide*"

GEOSYNTHETIC CLAY LINERS

Product Name	GCL Dimensional Properties: Panel Size Roll Width/Length m/m (ft/ft)	GCL Dimensional Properties: Average Roll Weight kN (lb)	GCL Dimensional Properties: Bentonite Mass/Unit Area Exclusion of Glue Weight kN/m^2 (lb/ft^2)	GCL Hydraulic Properties: Flux at i = 1.0 [1] ASTM D 5084 [2] m/s (cm/sec)	Base Bentonite Properties: Swell Index USP-NF-XVII (min) ml/2g	Base Bentonite Properties: Fluid Loss API-13B (ml)	GCL Structural Components, Upper Geosynthetic: Type or Structure	GCL Structural Components, Upper Geosynthetic: Weight ASTM D 3776 g/m^2 (oz/yd^2)	GCL Structural Components, Lower Geosynthetic: Type or Structure	GCL Structural Components, Lower Geosynthetic: Weight ASTM D 3776 or Thickness ASTM D 5199 g/m^2 or mm (oz/yd^2 or mil)	GCL Tensile Properties (ASTM D 4595 for geotextiles as lower component; ASTM D 638 for geomembranes as lower component): Tensile Strength kN/m (lb/in)	GCL Tensile Properties: Tensile Elongation %	Manufacturer's Suggested Applications [3]

Colloid Environmental Technologies Co. (CETCO)

Product Name	Panel Size m/m (ft/ft)	Average Roll Weight kN (lb)	Bentonite Mass/Unit Area kN/m^2 (lb/ft^2)	Flux m/s (cm/sec)	Swell Index ml/2g	Fluid Loss (ml)	Upper Type or Structure	Upper Weight g/m^2 (oz/yd^2)	Lower Type or Structure	Lower Weight or Thickness	Tensile Strength kN/m (lb/in)	Tensile Elongation %	Manufacturer's Suggested Applications [3]
Claymax 200 R	4.2/45.7 (13.83/150)	1044 kg (2300)	3.7 [A] (0.75)	5×10^{-11} (5×10^{-9})	24	18	woven	108 (3.2)	nonwoven	203 (6.0)	13.1 [B] (75)	>10	LL
Claymax 500 SP	4.2/45.7 (13.83/150)	1044 kg (2300)	3.7 [A] (0.75)	5×10^{-11} (5×10^{-9})	24	18	woven	108 (3.2)	woven	108 (3.2)	26.3 [B] (150)	>10	LL
Claymax 600 SP	4.2/45.7 (13.83/150)	1044 kg (2300)	3.7 [A] (0.75)	5×10^{-12} (5×10^{-10})	24	18	woven	108 (3.2)	woven/FML composite	223 (6.6)	26.3 [B] (150)	>10	LC, SIC
Bentomat ST	4.6/45.7 (15/150)	1090 kg (2400)	3.7 [A] (0.75)	5×10^{-11} (5×10^{-9})	24	18	woven	112 (3.3)	nonwoven	203 (6.0)	15.8 [B] (90)	>10	LL, LC, SIC
Bentomat DN	4.6/45.7 (15/150)	1112 kg (2450)	3.7 [A] (0.75)	5×10^{-11} (5×10^{-9})	24	18	geotextile	203 (6.0)	nonwoven	203 (6.0)	26.3 [B] (150)	>10	LL

GSE Lining Technology Inc.

Product Name	Panel Size m/m (ft/ft)	Average Roll Weight kN (lb)	Bentonite Mass/Unit Area kN/m^2 (lb/ft^2)	Flux m/s (cm/sec)	Swell Index ml/2g	Fluid Loss (ml)	Upper Type or Structure	Upper Weight g/m^2 (oz/yd^2)	Lower Type or Structure	Lower Weight or Thickness	Tensile Strength kN/m (lb/in)	Tensile Elongation %	Manufacturer's Suggested Applications [3]
GSE GundSeal	5.3 [C]/61 [B] (17.5/200)	1900 (4200)	0.048 (1.0)	$<4 \times 10^{-14}$ ($<4 \times 10^{-12}$)	28	18	none	NA	smooth geomembrane	0.3–2.0 mm (12–80 mil)	varies w/ GM type	>10	caps, containment
GSE GundSeal Textured	5.3 [C]/62 [B] (17.5/170)	1900 (4200)	0.048 (1.0)	$<4 \times 10^{-14}$ ($<4 \times 10^{-12}$)	28	18	none	NA	textured geomembrane	0.75–2.0 mm (30–80 mil)	varies w/ GM type	>10	caps, containment, slope applications

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*

GSE GundSeal Geobond	5.3[C]/61[D] (17.5/200)	1900 (4200)	0.048 (1.0)	$<4 \times 10^{-14}$ ($<4 \times 10^{-12}$)	28	18	nonwoven	25 (0.75)	smooth or textured geomembrane	0.3–2.0 mm (12–80 mil)	varies w/ GM type	>10	caps, containment
GSE GundSeal Weldable	5.3[C]/62[D] (17.5/170)	1900 (4200)	0.048 (1.0)	$<4 \times 10^{-14}$ ($<4 \times 10^{-12}$)	28	18	nonwoven optional	if used 25 (0.75)	geomembrane w/ bentonite-free edges	0.75–2.0 mm (30–80 mil)	varies w/ GM type	>10	GM/clay composite lining applications (welded seams)

National Seal Co./Fluid Systems/Columbia Geosystems Ltd.

NSC FSI Bentofix Thermal-Lock NW	4.7/38.1 (15.5/125)	977 (2150)	0.0439 (0.90)	5×10^{-11} (5×10^{-9})	24	18	nonwoven	200 (6.0)	nonwoven	200 (6.0)	12.1 (70)	NP	LC, LL, SIC
NSC FSI Bentofix Thermal-Lock NS	4.7/38.1 (15.5/125)	977 (2150)	0.0439 (0.90)	5×10^{-11} (5×10^{-9})	24	18	nonwoven	105 (3.1)	woven	200 (6.0)	12.1 (70)	NP	LC, LL, SIC

Naue Fasertechnik GmbH & Co.

B4000	4.8/30 (15.7/98)	NP	4700 g/m²	2×10^{-11} (2×10^{-9})	25	18	PP nonwoven	300 (8.9)	PP scrim reinforced nonwoven	400 g/m² (10.3)	12 (DIN 53857)	50	SIC, WR, LC
D4000	4.8/30 (15.7/98)	NP	4700 g/m²	2×10^{-11} (2×10^{-9})	25	18	PP nonwoven	300 (8.9)	PP scrim reinforced nonwoven	350 g/m² (10.3)	12 (DIN 53857)	50	LL, LC
BIG 5000	4.8/30 (15.7/98)	NP	5000 g/m²	2×10^{-11} (2×10^{-9})	25	18	PP nonwoven	300 (8.9)	PP woven	200 g/m² (15.9)	8 (DIN 53857)	5	WR, LC, SIC
NSP 4900	4.8/30 (15.7/98)	NP	5600 g/m²	2×10^{-11} (2×10^{-9})	25	18	PP nonwoven	220 (6.5)	PP woven	110 g/m² (3.2)	6 (DIN 53857)	8	LL, LC, SIC

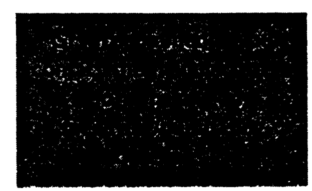
(1) Flux is defined as "flow rate/unit area," which can be converted to permeability using the equation:
Permeability = flux/hydraulic gradient

(2) Report result at a confining stress of 69 kN/m² (10psi) and 34 Kpa (5psi) head pressure

(3) CL = Canal liner
LL = Landfill liner
SIC = Surface impoundment cover
NP = Not provided by manufacturer
NA = Not applicable, per manufacturer

(A) Reported at 0 percent moisture content and kg/m²
(B) ASTM D 4632 Results (machine direction MARV)
(C) Available in 2.4 m (8 ft) widths
(D) Roll length depends upon geomembrane thickness.

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.



GEOSYNTHETIC CLAY LINERS

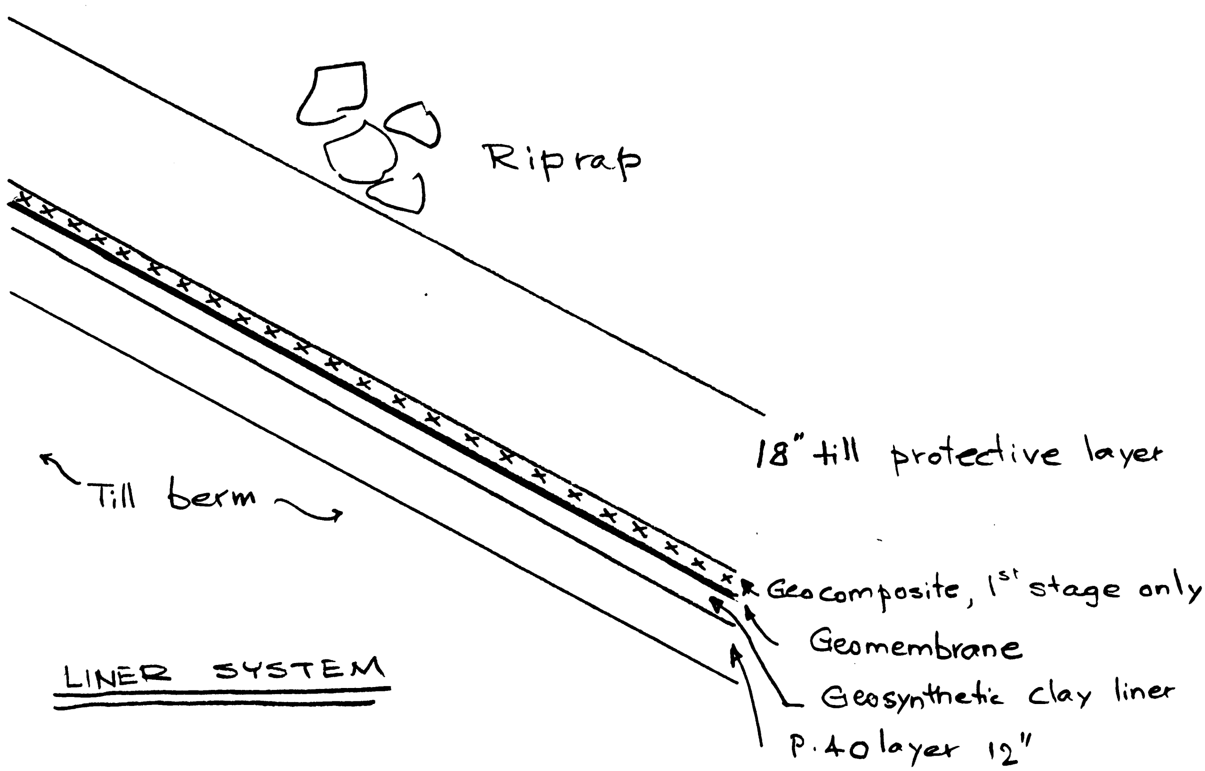
Appendix C

Required Strength Properties of Materials and Interfaces in the Liner System of the TMA Cells (January 1997 Update to Appendix D of the May 1995 Tailings Management Area Feasibility Report/Plan of Operation)

Foth & Van Dyke	Client: CMC — Scope I.D.: 93C049 Project: TMA — Page: 1 Prepared by: NXP — Date: 8/27/96 Checked by: BKirsch — Date: 8/28/96

REQUIRED STRENGTH PROPERTIES OF MATERIALS AND INTERFACES IN THE LINER SYSTEM OF THE TMAs.

Background: The liner system of the TMAs, as proposed, consist of a number of layers of different materials and thicknesses. These layers are placed parallel to the inside slope of the TMAs as shown in the sketch below



LINER SYSTEM

Because of the long slope lengths in relation to the thicknesses of the layers of materials with varying properties, the potential mode of failure of the cover system will be sliding of the materials along surfaces parallel to the slope surface. Such failures may occur along the interfaces of the different materials or through the materials themselves except the geomembrane and perhaps the geocomposite. Therefore, in order to obtain a safe design of the liner system one needs to determine the minimum required strength of the materials and the interfaces

Purpose: To calculate the minimum required strength for the liner materials and interfaces to achieve satisfactory stability conditions

Scope: Determine the mobilized shear strength for the material/interface through which failure is assumed, for a sliding block failure mode and factor of safety of 1.2.
Assume the thickness of cover soils to be 18 inches such that for deeper failures the analysis will be conservative.
Assume that ravelling or shallow failure within the protective layer is not critical

Conclusion: The required strength for a factor of safety of 1.2 is an angle of internal friction of 21.2°
This means that all materials (i.e., till, internal strength of GCL and P40 material) should have a shear strength equal to or greater than 21.2°
Also all interfaces, i.e till versus geocomposite, geocomposite versus geomembrane; geomembrane versus GCL. GCL versus P40 material; P40 material versus till must all have a friction angle equal to or greater than 21.2°

Foth & Van Dyke	
	Client: CMC — Scope I.D.: 93 C049
	Project: TMA — Page: ~~2/4~~ 3
	Prepared by: NXP — Date: 4-18-95
	Checked by: [signature] — Date: 5/3/95

Method of Analysis and Assumptions

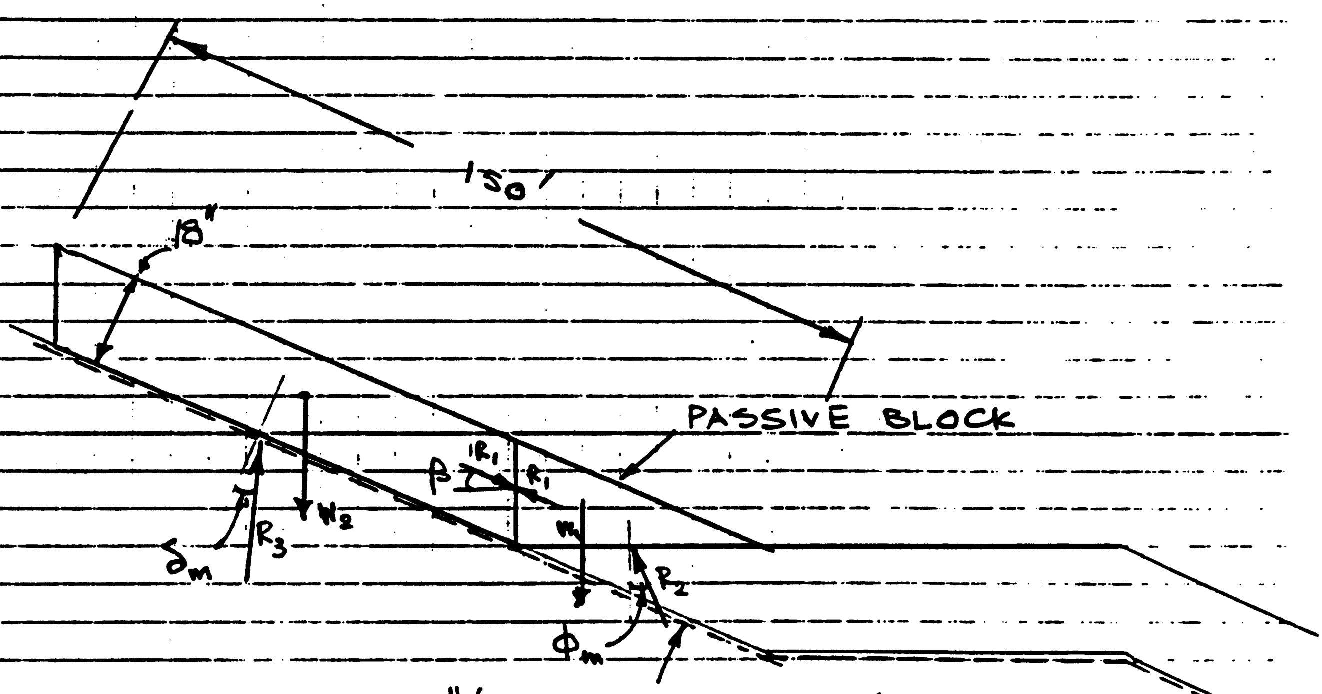
Assumptions – Slope Angle 18.43° (3H to 1V) ✓

Angle of internal friction of till = 32° ✓

Thickness of till cover = 1.5' ✓

Density of Till layer = 120 pcf. ✓

Maximum Length of Slope = 150' ✓

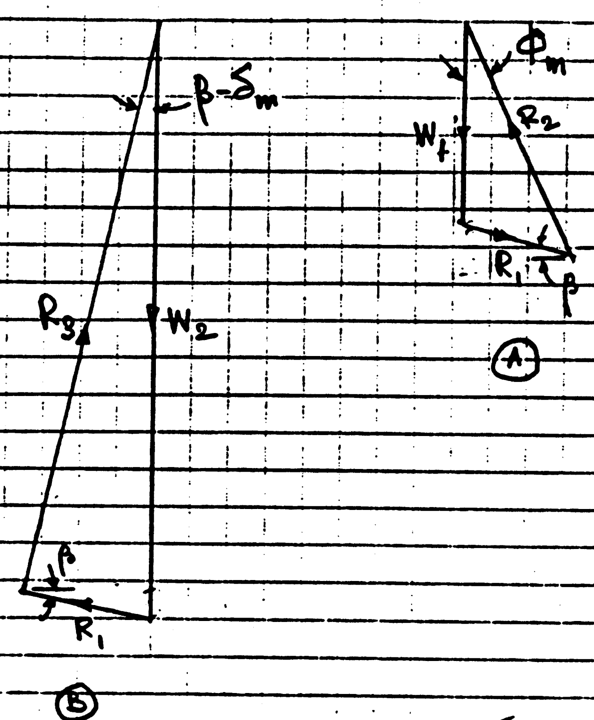


$W_2 = 145 \times 1.5 \times 120 = 26100$ lb/ft

$W_1 = \frac{4.5}{2} \times 1.5 \times 120 = 405$ lb/ft ✓

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: TMA	Page: 3/4
	Prepared by: NxP	Date: 4-18-9
	Checked by: [signature]	Date: 5/3/95

Force Polygons



From Force Polygon A

$$(W_1 + R_1 \sin\beta)\tan\phi_m = R_1 \cos\beta$$

$$R_1(\cos\beta - \sin\beta\tan\phi_m) = W_1\tan\phi_m$$

$$R_1 = \frac{W_1 \tan\phi_m}{\cos\beta - \sin\beta\tan\phi_m}$$

From Force Polygon B

$$\tan(\beta-\delta_m) = \frac{R_1\cos\beta}{W_2 - R_1\sin\beta}$$

$$\tan\phi_m = \frac{\tan\phi}{1.2} = \frac{\tan 32^\circ}{1.2}, \quad \phi_m = 27.51^\circ$$

$$R_1 = \frac{405(.521)}{\cos 18.43^\circ - \sin 18.43^\circ \tan 27.51^\circ} = \frac{210.9}{.95 - .165} = \underline{\underline{265.3}}$$

R_1

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
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	Prepared by: NXP	Date: 4/18/95
	Checked by: [signature]	Date: 5/3/95

$$\tan(\beta - \delta_m) = \frac{265.3\,(.95)}{26100 - 265.3\,(-.316)} = \frac{252.04}{26016}$$

$$= .0097$$

$$\therefore \beta - \delta_m \doteq .56^\circ$$

$$\therefore \delta_m = \beta - .56 = 18.43 - .56 = 17.87^\circ$$

$$\delta = \tan^{-1}(1.2 \tan 17.87^\circ)$$

$$= \underline{\underline{21.2^\circ}}$$

21.16°

Appendix D

4-Inch Perforated Pipe Strength Analysis

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: TMA - Pipe Crushing	Page: 1/10
	Prepared by: REM	Date: 7/17/96
	Checked by: KS	Date: 1/6/97

4" Perforated Leachate Collection Piping

Crushing Analysis

PURPOSE: TO SELECT THE PROPER 4" PE PIPE FOR THE leachate collection pipe laterals using the maximum loading conditions in TMA cells 1 - 4.

GIVEN: Pipe diameter (O.D.) = 4.500 in

- greatest depth of tailing = 108 feet (ie 108.22')
  γ = 97 lb/cF (Dd)
- 24" - 3" = 21" Stone over pipe
  γ = 130 lb/cF
- greatest depth of Final cover ~ 20 feet
  Av γ = 120 lb/cF
- Manufacturer's (Polypipe industries) maximum allowable PE pipe deflection for 4 in. PE pipe (SDR 17) is 5%
- $E_B = 5 \times 10^6$ psf (conservative value from Appendix "G" Attachment 1 to Feasibility Report, Foth and Van Dyke, 1995a
- Tailings W% = 34% (same reference as above)

SOLUTION: Calculate Pipe Deflection and Imposed Design / construction methods to limit deflection to less than the maximum allowable manufacturers recommendations

Determine Column Load on Collection Pipe

Tailings γ_d = 97 pcf $\gamma_{moist} = \gamma_d (1+w) = (97)(1.34) = 130$ lb/CF × 108 ft = 14040

Stone 130 lb/cF × 21" × 1 Ft / 24" – (227.5) = 230

Final cover 120 lb/cF × 20 Ft (WORST CASE) = 2400

Pipe = 0

Maximum Column load of collection pipe = 16,670

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: TMA - Pipe Crossing	Page: 2/10
	Prepared by: REM	Date: 7/17/96
	Checked by: KS	Date: 1/6/97

Determine Unit Weight on the Pipe

$16{,}670\ \text{lb/ft}^2 \times \frac{1}{130\ \text{ft}} = 128.23$ say $\underline{128\ \text{lb/ft}^3}$ ✓

Determine Marston Load $\underline{W_c}$, Per unit length where

$$W_C = C_C\, w\, B_C^{\,2}$$

where C_C = load coefficient which is a function of the projection ratio (p), settlement ratio (r_{sd}) and ratio of fill height (H) to pipe width (B_c) as per Figure 1 (below)

w = unit weight = 128 lb/FT3 ✓

B_c = outside diameter of pipe in feet

✓ $4.5\ \text{in} \times \frac{1\ \text{FT}}{12\ \text{in.}} = 0.375\ \text{FT}$ ✓

$H/B_c = \frac{130\ \text{ft}}{0.375\ \text{ft}} = 347$ ✓

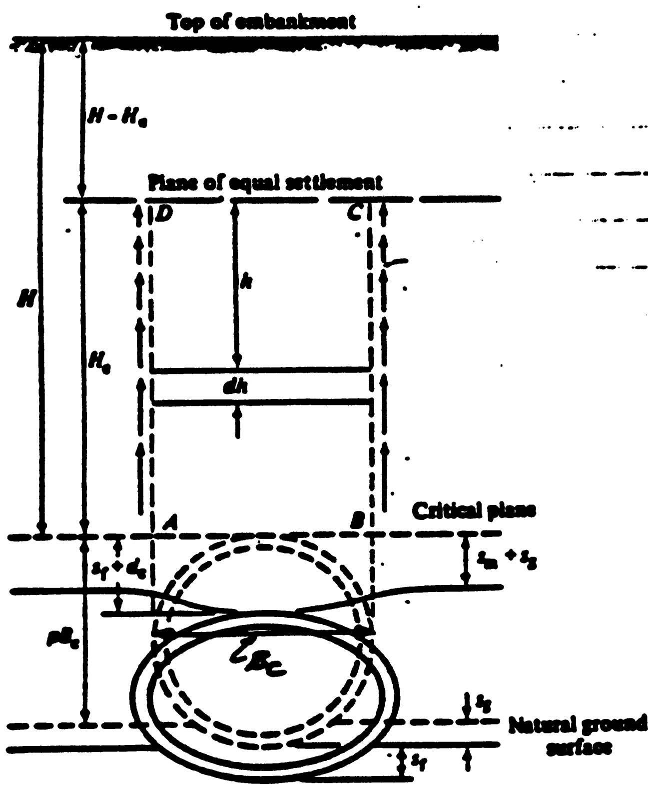

Figure Settlements that influence loads on positive projecting conduits (incomplete ditch condition). Key same as in Figure 26.12.

Source:

SPANGLER, M. G. & HANDY, R.L. (1982). Soil Engineering, Harper & Row, New York, NY

Foth & Van Dyke	Client: CMC — Scope I.D.: 93C049 Project: TMA - Pipe Crushing — Page: 3/10 Prepared by: REM — Date: 7/17/96 Checked by: KS — Date: 1/6/97

SOLUTION:
(CONTINUED)

VALUES OF C_c

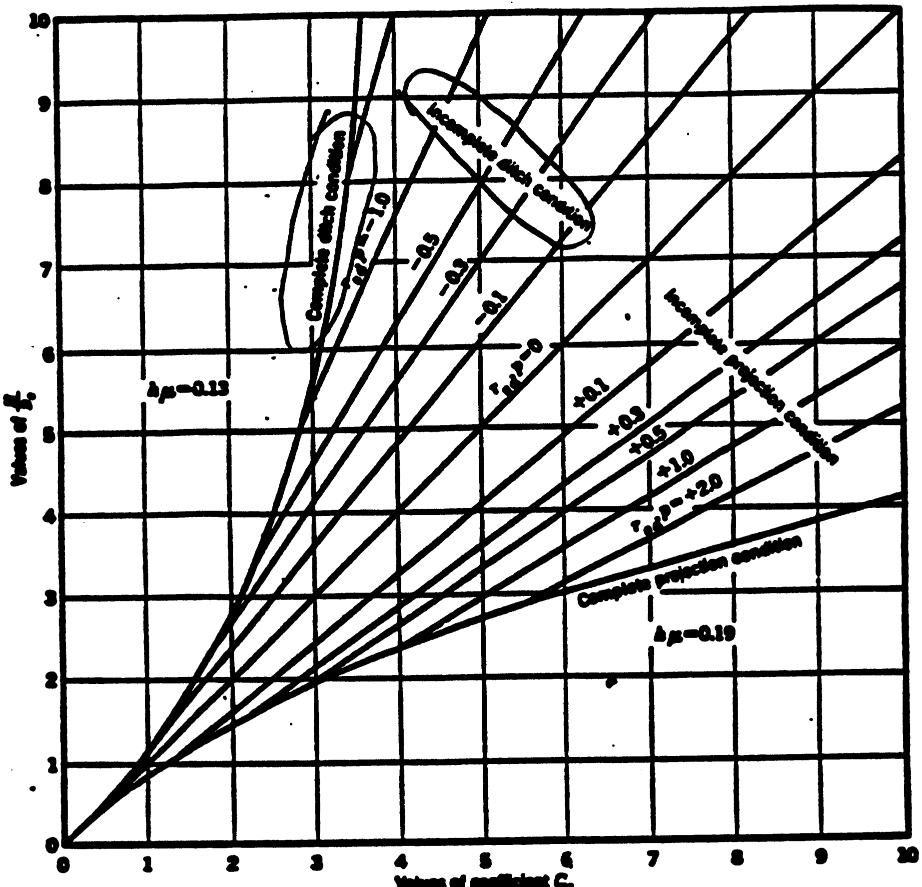

FIGURE #2

Diagram for coefficient C_c for positive projecting conduits.

SOURCE: GRAVITY SANITARY SEWER DESIGN AND CONSTRUCTION, "MANUALS & REPORTS ON ENGINEERING PRACTICE NO. 60," AMERICAN SOCIETY OF CIVIL ENGINEERS AND "MANUAL OF PRACTICE FD-5," WATER POLLUTION CONTROL FEDERATION. 1982.

4-a) NOTE: FOR FLEXIBLE PIPE UNDER MOST CONDITIONS THE PRODUCT $r_{sd} \cdot P$, is LESS THAN ZERO (NEGATIVE VALUE). FOR AN EMBANKMENT SITUATION SUCH AS A LANDFILL CONSTRUCTION, AN INCOMPLETE DITCH CONDITION EXISTS. BECAUSE $r_{sd} \cdot P$ is NEGATIVE THE total PRISM LOAD IS NOT APPLIED to THE PIPE.

Foth & Van Dyke

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Determine Ko which is the ratio of the lateral earth pressure to the vertical earth pressure - use $K_0 = 0.5$

Determine Kou given ϕ = internal friction angle of tailings = 10°

$\mu = \tan\phi = \tan 10^\circ = 0.176$

$K_{0\mu} = (0.5)(0.176) = 0.088$ say $\underline{0.09}$

$K_{0\mu} = 0.09$

Determine Ratio of C_c for $K_{0\mu} = 0.09$ for an Incomplete Ditch condition

GIVEN TABLE 1 Values of C_c in terms of H/B_c

· Since the incomplete ditch condition curves in Figure #2 (pg 3) are bounded by the complete ditch conditions curve the incomplete conditions curves for the $K_{0\mu} = 0.09$ can be determined by adjusting the values by the ratio of $K_\mu = 0.09$, $K_\mu = 0.013$ which is

$\frac{0.013}{0.09} = 1.44$ ... thus $C_c(1.44)$ will be applied for $K_{0\mu} = 0.09$

TABLE 1 Values* of C_c in Terms of H/B_c

Incomplete Projection Condition $K\mu = 0.19$ $r_{sd}p$	Equation	Incomplete Ditch Condition $K\mu = 0.13$ $r_{sd}p$	Equation
+0.1	$C_c = 1.23H/B_c - 0.02$	−0.1	$C_c = 0.82H/B_c + 0.05$
+0.3	$C_c = 1.39H/B_c - 0.05$	−0.3	$C_c = 0.69H/B_c + 0.11$
+0.5	$C_c = 1.50H/B_c - 0.07$	−0.5	$C_c = 0.61H/B_c + 0.20$
+0.7	$C_c = 1.59H/B_c - 0.09$	−0.7	$C_c = 0.55H/B_c + 0.25$
+1.0	$C_c = 1.69H/B_c - 0.12$	−1.0	$C_c = 0.47H/B_c + 0.40$
+2.0	$C_c = 1.93H/B_c - 0.17$		

Foth & Van Dyke

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Prepared by: RBM Date: 7/8/96
Checked by: J.S. Date: 1/6/97

TABLE #2 r_{sdp}, C_c, K_u/K_u FOR $K_u = 0.09$

$H/B_c = \frac{130\,ft}{0.375\,ft} = 346.67$ say $\underline{347\,ft/ft}$

r_{sdp}	C_c (from Table #1)	$C_c (K_u/K_u)$	Modified C_c Values
-0.1	$C_c = 0.82(347) + 0.05 =$	284.6(1.44)	409.8
-0.3	$C_c = 0.69(347) + 0.11 =$	239.5(1.44)	344.9
-0.5	$C_c = 0.61(347) + 0.20 =$	211.87(1.44)	305.1
-0.7	$C_c = 0.55(347) + 0.25 =$	191.1(1.44)	275.2
-1.0	$C_c = 0.47(347) + 0.40 =$	163.9(1.44)	235.4

Determine $r_{sd} = \dfrac{(S_m + S_g) - (S_f + d_c)}{S_m}$

where:

r_{sd} = settlement ratio

S_m = compression strain of the side column of height $p \cdot B_c$ where: p = projection ratio, B_c = width of pipe

S_g = settlement of the natural surface adjacent to the conduit

S_f = settlement of the conduit into its foundation

d_c = shortening of the vertical height of the conduit

For r_{sd} Assume $S_g = 0$ and $S_f = 0$

S_m = load strain = $\left(\dfrac{\text{Load of Prism}}{E_B \text{ of Stone}} (B_c) \right)$

Load of Prism = (unit weight)(height)(width) = L_p

$L_p = (128\ lb/CF)(130\ FT)(0.375\ FT) = 6240$ say $6200\ lb/FT$

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	Checked by: KS	Date: 1/6/97

E_B of stone $= 5 \times 10^6$ psf a conservative value from attachment 1 of Appendix G (F+VD Feasibility Report, 1995)

$$\therefore \quad S_m = \frac{6200\ lb/ft}{5 \times 10^6\ lb/ft^2} \cdot B_c = .00124\ B_c$$

$$d_c = (\%\ \text{of Deflection})(B_c)$$

% of deflection = Maximum recommended by manufacturer is 5% for SDR 17 pipe

$$r_{sd} = \frac{(S_m + S_g) - (S_t + d_c)}{S_m}$$

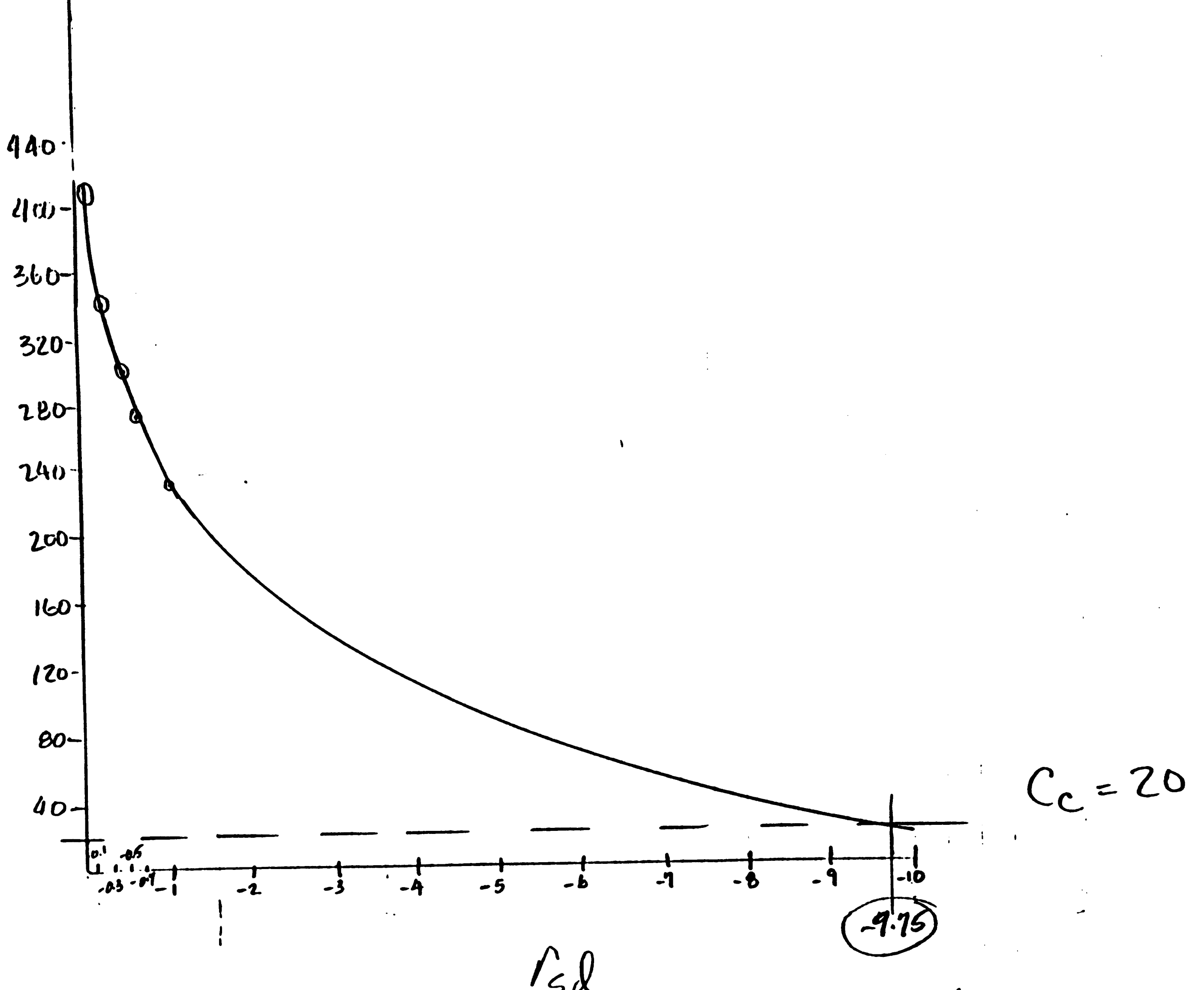
where $S_m = .00124(B_c)$
$S_g = 0$
$S_t = 0$
$d_c = 0.05\,B_c$

$$\therefore \quad \frac{\left[(.00124\,B_c) + 0\right] - \left[0 + 0.05\,B_c\right]}{0.005\,B_c}$$

$$r_{sd} = \frac{.00124\,B_c - 0.05\,B_c}{0.005\,B_c} = \frac{-4.876 \times 10^{-2}}{0.005} = -9.75$$

Foth & Van Dyke	Client: CMC Scope I.D.: 93C049 Project: TMA-Addendum NO. 3 Page: 7/10 Prepared by: REM Date: 7/18/96 Checked by: KS Date: 1/6/97

Figure 3 - Graphical Representation of TABLE 2 where C_c @ $\frac{K_{ou}}{K_u}$ Ratio for $K_u = 0.09$ Vs Γ_{sdp} (Assume $\rho = 1.0$)



Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: JMA - Addendum NO. 3	Page: 8/10
	Prepared by: REM	Date: 7/18/96
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Marson Load W_C

$$W_C = C_c\, W\, B_c^2$$

$C_C = 20$ from Figure 3

$W = 128$ lb/FT3 from pg 2

$B_C = .375$ FT

$$W_C = (20)(128)(.375)^2$$

$$W_C = 360\ \frac{\text{lbs}}{\text{lin Ft}} \times \frac{1\text{ Ft}}{12\text{ in}} = 30\text{ lb/lineal inch}$$

Modulus of Passive Soil Resistance $E' = 1000$ psi for dumped crushed rock from TABLE #3 (pg 8)

Solve for Deflection given

$$\Delta x = \frac{D_L \cdot K \cdot W_c}{\frac{2E}{3(SDR)^3} + 0.061\, E'}$$

where D_L - Lag Factor (1.25-1.5) use 1.5 (conservative)

K = Bedding class Factor use $K = 0.1$ (conservative)

$W_C = 30$ lbs/lineal inch

E = pipe modulus of elasticity = 30,000 psi (conservative)

$SDR = OD/T$ where OD - Outside pipe diameter, T - pipe wall thickness

$$SDR = \frac{4.5}{.265} = 16.98 \text{ say } 17$$

$E' = 1000$ psi from Table #3 (pg 8)

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: TMA-Addendum No. 3	Page: 9/10
	Prepared by: PJM	Date: 7/18/96
	Checked by: KS	Date: 1/6/97

TABLE #3

AVERAGE VALUES OF MODULUS OF SOIL REACTION, E'
(For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch			
	Dumped (2)	Slight, <85% Proctor, <40% relative density (3)	Moderate, 85%-95% Proctor, 40%-70% relative density (4)	High, >95% Proctor, >70% relative density (5)
Fine-grained Soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use E' = 0			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained Soils with Fines GM, GC, SM, SC[c] contains more than 12% fines	100	400	1,000	2,000
Coarse-grained Soils with Little or no Fines GW, GP, SW, SP[c] contains less than 12% fines	200	1,000	2,000	3,000
→ Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

E' FOR STONE BACKFILL

[a]ASTM Designation D 2487, USBR Designation E-3.
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e. GM-GC, GC-SC).
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m^3) (ASTM D 698, AASHTO T-99, USBR Designation E-11). 1 psi = 6.9 kPa.

SOURCE: "Soil Reaction for Buried Flexible Pipe" by Amster K. Howard, U.S. Bureau of Reclamation, Denver, Colorado. Reprinted with permission from American Society of Civil Engineers.

SOURCE: UNI-BELL PVC PIPE ASSOCIATION (1991)
HANDBOOK OF PVC PIPE: DESIGN AND CONSTRUCTION
DALLAS, TX p. 207

Foth & Van Dyke	Client: CMC	Scope I.D.: 93C049
	Project: TMA - Addendum NO. 3	Page: 10/10
	Prepared by: REM	Date: 7/15/96
	Checked by: KS	Date: 1/6/97

$$\Delta_x = \frac{(1.5)(0.1)(30)}{\frac{2(30{,}000)}{3(17)^3} + 0.061(1{,}000)} = \frac{4.5}{\frac{60000}{14739} + 61} =$$

$\Delta_x = 0.069$ say 0.07 inches ✓

Determine % deflection $= \frac{\Delta_x}{B_c} \times 100$

where $\Delta_x = 0.069$ inches $B_c = .375\,FT \times \frac{12\,in}{FT} = 4.5''$

$\frac{0.069}{4.5} \times 100 = 1.53\%$ ✓ okay since 5% is max.

Appendix E

Geocomposite (Geonet) Information from *Geotechnical Fabrics Report "1997 Specifier's Guide"*

GEONETS

Product Name	Structure [1]	Core/Net/Mesh Polymer Composition [2]	Geotextile Attached	Dimensional Properties: Width/Length m (ft)	Dimensional Properties: Core/Net/Mesh [3] Thickness ASTM D 5199 mm (mil)	Compressive Strength ASTM D 1621 kPa (psi)	In Plane Flow Rate ASTM D 4716 [4]: Gradient = 0.1 Pressure = 10 kPa (1.45 psi) m^2/s (gal/min/ft)	In Plane Flow Rate ASTM D 4716 [4]: Gradient = 0.1 Pressure = 100 kPa (14.5 psi) m^2/s (gal/min/ft)

Akzo Nobel Geosynthetics Co.

Product Name	Structure [1]	Core/Net/Mesh Polymer Composition [2]	Geotextile Attached	Width/Length m (ft)	Thickness mm (mil)	Compressive Strength kPa (psi)	Gradient = 0.1, Pressure = 10 kPa m^2/s (gal/min/ft)	Gradient = 0.1, Pressure = 100 kPa m^2/s (gal/min/ft)
Enkanet 4011	O/C	HDPE	nonwoven	2/97.6 (6.5/320)	9.40 (370)	NA	0.00896 (43.2)	0.00213 (10.2)
Enkanet 4211	O/C	HDPE	nonwoven (both sides)	2/97.6 (6.5/320)	8.13 (320)	NA	0.0046 (21.5)	0.00175 (8.4)
Enkanet 4015	O/C	HDPE	nonwoven	2/97.6 (6.5/320)	9.40 (370)	NA	0.0128 (61.8)	0.00394 (19.0)
Enkanet 4215	O/C	HDPE	nonwoven (both sides)	2/97.6 (6.5/320)	9.65 (380)	NA	0.00817 (39.4)	0.00354 (17.1)

Engineered Synthetic Products Inc.

Product Name	Structure [1]	Core/Net/Mesh Polymer Composition [2]	Geotextile Attached	Width/Length m (ft)	Thickness mm (mil)	Compressive Strength kPa (psi)	Gradient = 0.1, Pressure = 10 kPa m^2/s (gal/min/ft)	Gradient = 0.1, Pressure = 100 kPa m^2/s (gal/min/ft)
Transnet	GN	HDPE	none	4.3/91.5 (14/300)	5.5 (220)	NP	NP	3.4×10^{-3} (16.4)
Transnet TNT	OC	HDPE	nonwoven	4.3/varies (14/varies)	dependent on geotextile selection	NP	NP	dependent on geotextile selection

GSE Lining Technology Inc.

GSE HyperNet	GN	HDPE	none	4.3/91.5 (14/300)	5.5 (220)	240 (35)	5.0E-3 (24.2)	4.0E-3 (19.3)
* GSE FabriNet	GN	HDPE	nonwoven (3.5 oz. to 16 oz.)	4.3/varies (14/varies)	dependent on geotextile selection	240 (35)	dependent on geotextile selection	dependent on geotextile selection

National Seal Co./Fluid Systems/Columbia Geosystems Ltd.

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NSC Poly-Net FSI PN2000	GN	PE	none	2.3, 4.4/91.4 (7.54, 14.5/300)	4.06 (160)	NP	NP	1 x 10^{-4} (0.483)
NSC Poly-Net FSI PN3000	GN	PE	none	2.3, 4.4/91.4 (7.54, 14.5/300)	5.1 (200)	NP	NP	2 x 10^{-4} (0.97)
NSC Poly-Net FSI PN3000CN	GN	PE	none	2.3, 4.4/91.4 (7.54, 14.5/300)	5.1 (200)	NP	NP	2 x 10^{-4} (0.97)
* NSC Tex-Net FSI TN3002	O/C	PE	nonwoven	4.4/91.4 (14.5/300)	NP	NP	NP	3 x 10^{-5} (0.145)
* NSC Tex-Net FSI TN3002CN	O/C	PE	nonwoven	4.4/91.4 (14.5/300)	NP	NP	NP	3 x 10^{-5} (0.145)

(1) GN = Geonet
O/C = Other or composition
(2) PE = Polyethylene
HDPE =High density polyethylene
(3) If yes, specify if geotextile is woven (W) or nonwoven (NW)
(4) Thickness includes attached geotextile
(5) ASTM D 4716, seating time is 15 min and soil environment

NP = Not provided by manufacturer
NA = Not applicable, per manufacturer

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GFR recommends you contact the manufacturer(s) before making any specifying decisions.

GEONETS

Geotechnical Fabrics Report ■ December 1996

GEONETS

Product Name	Structure [1]	Core/Net/Mesh Polymer Composition [2]	Geotextile Attached	Dimensional Properties		Compressive Strength ASTM D 1621 kPa (psi)	In Plane Flow Rate ASTM D 4716 [4]	
				Width/Length m (ft)	Core/Net/Mesh [3] Thickness ASTM D 5199 mm (mil)		Gradient = 0.1 Pressure = 10 kPa (1.45 psi) m^2/s (gal/min/ft)	Gradient = 0.1 Pressure = 100 kPa (14.5 psi) m^2/s (gal/min/ft)

Tenax Corp.

Product Name	Structure	Polymer	Geotextile	Width/Length	Thickness	Compressive Strength	Flow Rate 10 kPa	Flow Rate 100 kPa
Tendrain 70	O/C	HDPE	nonwoven	2/61 (6.7/200)	8.8 (350)	>1436 (208)	$4.4E^{-3}$ (21.5)	$3.7E^{-3}$ (18)
Tendrain 100	O/C	HDPE	nonwoven	2/61 (6.7/200)	8.8 (350)	>1436 (208)	$4.4E^{-3}$ (21.5)	$3.7E^{-3}$ (18)
CE 2	GN	HDPE	none	2/55 (6.7/180)	5 (200)	NP	$2.7E^{-3}$ (13)	$2.6E^{-3}$ (12.5)
CE 3	GN	HDPE	none	2/55 (6.7/180)	4 (160)	NP	$1.9E^{-3}$ (9.2)	$1.8E^{-3}$ (8.7)
CE 9	GN	HDPE	none	2/55 (6.7/180)	5.5 (220)	NP	$3.1E^{-3}$ (15)	$3.0E^{-3}$ (14.5)
TNT 20606	O/C	HDPE	nonwoven	2/55 (6.7/180)	5.3 (210)	NP	NP	NP

WEBTEC Inc.

Product Name	Structure	Polymer	Geotextile	Width/Length	Thickness	Compressive Strength	Flow Rate 10 kPa	Flow Rate 100 kPa
TerraNet 160	GN	HDPE	none	2/55 (6.7/180)	4 (160)	NP	0.0019 (9.2)	0.0018 (8.7)

WEBTEC Inc.

TerraNet 200	GN	HDPE	none	2/55 (6.7/180)	5 (200)	NP	0.0027 (13)	0.0026 (12.5)
TerraNet 210	O/C	HDPE	nonwoven	2/55 (6.7/180)	5.3 (210)	NP	NP	NP
TerraNet 220	GN	HDPE	none	2/55 (6.7/180)	5.3 (220)	NP	0.0031 (15)	0.003 (14.5)

(1) GN = Geonet
O/C = Other or composition
(2) PE = Polyethylene
HDPE =High density polyethylene
(3) If yes, specify if geotextile is woven (W) or nonwoven (NW)
(4) Thickness includes attached geotextile
(5) ASTM D 4716, seating time is 15 min and soil environment

NP = Not provided by manufacturer
NA = Not applicable, per manufacturer

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GFR recommends you contact the [illegible] before [illegible]

GEONETS

Appendix F

Assessment of Long-Term Performance of the Proposed HDPE Geomembrane Liner and Cap at the Crandon Project TMA Facility December 1996 Report Prepared by GeoSyntec Consultants

Prepared for

Foth & Van Dyke
2737 South Ridge Road
P.O. Box 19012
Green Bay, WI 54307-9012

ASSESSMENT OF LONG-TERM PERFORMANCE OF THE PROPOSED HDPE GEOMEMBRANE LINER AND CAP AT THE CRANDON PROJECT TMA FACILITY

Prepared by

GEOSYNTEC CONSULTANTS
621 N.W. 53rd Street, Suite 650
Boca Raton, Florida 33487

Project #JP1032

18 December 1996

TABLE OF CONTENTS

6. CONCLUSIONS

REFERENCES

1. INTRODUCTION

1.1 Terms of Reference

This report has been prepared by Dr. J.P. Giroud and Mr. L.G. Tisinger, both of GeoSyntec Consultants (GeoSyntec), in accordance with a 12 November 1996 proposal written by Dr. Giroud and Mr. Tisinger to Mr. J. Sevick, of Foth and Van Dyke and Associates, Inc. (Foth and Van Dyke). This report has been reviewed by Dr. K. Badu-Tweneboah, also of GeoSyntec, in accordance with the peer review policy of the firm.

1.2 Purpose and Scope of this Report

This report has been prepared to assess the long-term durability of a 60-mil (1.5 mm) thick high density polyethylene (HDPE) geomembrane proposed for use to line and cap the Crandon Project Tailings Management Area (TMA). The facility is being constructed to contain tailings from zinc and copper mine operations. This report will present information on the durability of the proposed HDPE geomembrane based on data provided by Foth and Van Dyke on the expected site conditions.

1.3 Organization of Report

The remainder of this report is organized as follows:

- sources of information are discussed in Section 2;
- background on HDPE is discussed in Section 3;
- site conditions are discussed in Section 4;
- mechanisms of degradation of HDPE are discussed in Section 5; and
- conclusions are presented in Section 6.

2. SOURCES OF INFORMATION

The sources of information used in preparing this report included manufacturers of geomembranes, text books, technical reports, handbooks, technical papers, journal articles, and information provided by Foth and Van Dyke in the 19 November 1996 letter on the expected site conditions.

3. BACKGROUND ON HDPE GEOMEMBRANES

3.1 Introduction

A background on the molecular characteristics, or microstructure, of polyethylene and polyethylene materials is essential for understanding data on the long-term durability of HDPE geomembranes. Thus, this section presents a description of the microstructure of polyethylene and polyethylene materials.

3.2 Background on Polyethylene

Polyethylene (also called polyethene) is produced from an addition polymerization reaction in which individual molecules of ethylene (also called ethene) combine to form a long molecule [Brydson, 1982; Seymour and Carraher, 1981]. Ethylene is the monomer and polyethylene is the polymer. The resulting polyethylene molecule can be thought of as a chain in which the individual links of the chain have a simple molecular structure consisting of two carbon atoms bonded to each other and two hydrogen atoms bonded to each carbon atom. The carbon atoms, bonded to each other in a linear fashion, form the backbone of the polyethylene chain. There are two types of polyethylene materials: (i) low density non-linear polyethylenes formed in a high pressure process; and (ii) high density linear polyethylenes formed in a low pressure process [Martino, 1990]. Low density polyethylene materials are referred to as non-linear because they have many short and long branches stemming from the backbone of the polyethylene chain. These short- and long-branches prevent the close packing of individual molecules of polyethylene resulting in polyethylene material of a low density. In contrast, the molecules of linear polyethylene materials do not have a significant number of branches. This allows the close packing of molecules of polyethylene, resulting in a high density polyethylene material [Martino, 1990].

3.3 Characteristics of Non-Linear and Linear Polyethylene Materials

Non-linear polyethylene materials and linear polyethylene materials exhibit distinctly different properties. Non-linear polyethylene, or low density polyethylene (LDPE), materials have relatively low strength and high permeability to fluids compared to their high density counterparts, making LDPE an unsuitable material for geomembranes. Thus, LDPE will not be discussed in this report.

Conversely, linear polyethylene (HDPE) has relatively low permeability to fluids and is stronger than LDPE. As a result, HDPE is commonly used for geomembranes. Therefore, the structure of linear polyethylene materials is critical to the selection of HDPE for geomembranes and is discussed in greater detail below.

3.4 Structure of Linear Polyethylene Materials

As described in Section 3.2, linear polyethylene materials are manufactured under low pressure and are generally characterized by the close packing of the individual polyethylene molecules. In certain regions in linear polyethylene materials, the polyethylene molecules are highly ordered and densely packed. These regions are referred to as crystalline regions. The crystalline regions are connected by less organized polyethylene molecules, which form amorphous regions. Accordingly, polyethylene is generally referenced as having a semicrystalline structure. The amount of the crystalline regions in a given polyethylene material varies between 0 percent for a totally amorphous polyethylene material, to 100 percent for a totally crystalline polyethylene material [Tisinger and Giroud, 1993].

Highly crystalline linear polyethylene materials tend to be stiff and relatively inflexible [Brydson, 1982], making them undesirable for use as geomembranes, where a certain amount of flexibility is necessary. Therefore, to improve flexibility and to enhance environmental stress cracking resistance, linear polyethylene materials generally contain comonomers, such as butane, hexane, and heptane. Comonomers are additional reactants (typically 0.2% to 2.0% of the total reactants) to the monomer, ethylene. These comonomers result in relatively short branches on the linear polyethylene backbone; for example, butane produces a two-carbon long branch, and hexane, a four-carbon long branch. (It is noted that the branches of linear polyethylene materials contain two to four carbon atoms while the branches of LDPE can be tens or hundreds of carbon atoms long [Apse, 1989].) The branches in linear polyethylene are present in a very small proportion relative to the number of carbon atoms in the polyethylene backbone, e.g., one to ten branches per thousand carbon atoms of the backbone. The properties of linear polyethylene depend on the number and length of branches present on the polyethylene molecules. For example, the greater number of branches that are present, and the longer the branches, the more flexible a material will be.

4. ASPECTS OF THE SITE CONDITIONS THAT ARE RELEVANT TO HDPE PERFORMANCE

Aspects of the environment that may impact the performance of HDPE geomembranes include ultraviolet (UV) radiation, thermal energy, chemicals, oxygen, biological stresses, and mechanical stresses, some of which may be present at the TMA facility. The expected site conditions, described by Foth and Van Dyke in their 19 November 1996 letter to GeoSyntec have been interpreted with regard to their effect on geomembrane durability and are presented below.

4.1 Ultra Violet (UV) Radiation

Exposure of the geomembrane to UV radiation will be minimal because the geomembrane will be covered with soil immediately after completion of construction quality assurance activities on the geomembrane.

4.2 Heat Exposure and Climate Changes (Temperature Cycling)

The normal annual mean temperatures for the site are provided in Table 1.

Table 1. Normal Mean Monthly Weather Data for North Pelican Station[1]

Month	Temperature °F (°C)
January	10.2 (-12.1)
February	14.2 (-9.9)
March	26.0 (-3.3)
April	40.4 (4.7)
May	53.2 (11.8)
June	61.5 (16.4)
July	66.1 (18.9)
August	63.3 (17.4)
September	54.9 (12.7)
October	44.7 (7.1)
November	30.5 (-0.8)
December	15.5 (-9.2)

[1]Data represent 30-year average for the years 1961-1990.

Table 1 shows the average monthly temperatures for the location of the TMA facility. Inspection of Table 1 reveals that the difference between the lowest average temperature (January at 10.2°F (-12.1°C)) and the highest average temperature (July at 66.1°F (18.9°C)) is 56°F or 31°C. The effect of temperature is described in Section 5.1.1.1. It is noted that the liner will be covered with soil, therefore, it will likely be insulated from any extreme changes in the air temperature.

4.3 Concentration of Oxygen

Oxygen diffusion modeling has revealed that oxygen will not be present within the TMA; however, some oxygen may contact the geomembrane liner through the berm, and may also contact the cap through the cover soil.

4.4 Mechanical Stress

The facility has been designed so that any tensile stresses imparted by gravitational forces on the liner would be minimized.

4.5 Pore Water Quality (Chemicals)

The leachate generated in the TMA should have a pH that is neutral to slightly above neutral. However, a worst case leachate would have a pH that would be approximately 2.5. The leachate contains mostly salts, and some non-volatile organics. The leachate is not expected to contain volatile organic and semi-volatile organic constituents.

4.6 Microbiological Activities

Microbiological activity would be very unlikely because the soils in the area are tills, bedrock, and outwash, none having significant organic constituents.

4.7 Macrobiological Stress

Burrowing mammals exist in the vicinity of the TMA facility; however, the cap system of the cells will be covered by 4.5 ft (1.4 m) of soil materials which is thick enough to be a deterrent for most mammals. In addition, a drainage layer consisting of a 1 ft (0.3 m) thick layer of medium to coarse sand with gravel will be the first layer above the geomembrane, providing a deterrent for burrowing since it is coarse and cohesionless and will not be stable for tunneling.

5. MECHANISMS OF DEGRADATION OF HDPE GEOMEMBRANES

5.1 Action of Energy

5.1.1 Thermal Energy

5.1.1.1 Background Information

Extreme heat may cause molecular breakdown of polyethylene; however, the temperature must be approximately 830°F (450°C) before significant molecular breakdown occurs in HDPE compounds containing antioxidants [GeoSyntec Consultants, 1995]. (It should be noted that HDPE geomembranes contain high temperature and low temperature antioxidants. High temperature anti-oxidants provide protection for materials at extremely high temperatures, i.e., greater than the melting temperature, which is approximately 266°F (130°C). Low temperature antioxidants provide long-term protection at temperatures less than the melting temperature.) An extreme temperature, such as 830°F (450°C) (which would not occur from the sun), is well above the maximum expected temperature of an exposed geomembrane, i.e., approximately 160°F (70°C). Also, during seaming, the HDPE geomembrane will be exposed to an elevated temperature during a short period of time. To achieve a seam, the temperature of the geomembrane must be greater than the melting temperature for HDPE which is approximately 265°F (130°C). Typically, with geomembranes, the temperature of the geomembrane surface during seaming is 400 to 700°F (200 to 370°C), which is below 830°F (450°C). Therefore, heat alone is not expected to adversely impact the HDPE geomembrane.

Temperature cycling may result in the softening-hardening of the geomembrane, referring to the fact that HDPE becomes softer when heated and harder when cooled. However, this is an immediate effect that is only physical and is completely reversible. Therefore, temperature cycling should have no impact on the long-term performance of HDPE geomembranes.

Thermal expansion-contraction refers to the fact that the dimensions of HDPE geomembranes increase when temperature increases and decrease when temperature decreases. Giroud and Peggs [1990] have shown, on the basis of thermal expansion-contraction measurements, that thermal contraction of HDPE geomembranes in the field is less than 2%, even under an extreme outdoor temperature change (i.e., a change in geomembrane temperature from +160°F (+70°C) on a sunny summer day, to -40°F (-40°C) on a very cold winter night, a condition which is not likely to occur in the TMA facility, because the geomembrane will be insulated by the soil covering. A geomembrane or its seams can be affected if the contraction is greater than the geomembrane yield strain. Such a situation is described below.

At temperatures above 32°F (0°C), the yield strain of HDPE geomembranes is greater than 10%. The yield strain of an HDPE geomembrane decreases as temperature decreases. As indicated by Giroud [1994], for a typical HDPE geomembrane at a temperature of -40°F (-40°C), a yield strain of approximately 7% may be obtained, which is still greater than the strain of 2% that may be caused by thermal contraction. However, in extreme cases, a 2% strain due to thermal contraction may be combined with strains due to other causes, such as gravity on slopes, soil movements, and strain concentrations due to seam geometry. Consequently, under such extreme circumstances, the total strain in the vicinity of a seam may be close to the yield strain at low temperature. In addition, if the geomembrane is susceptible to cracking and contains crack initiation sites next to seams (which happens due to grinding as part of seam preparation), cracking may occur. The cracking mechanism described above has been observed a number of times. However, it is observed much less frequently than five to ten years ago, due to better seaming techniques (e.g., techniques such as fusion seaming that does not require grinding) and considerable progress in geomembrane resin selection (i.e., selections of resins that have a low susceptibility to cracking).

At the TMA facility, the conditions for stress cracking are not expected to exist because the geomembrane will be insulated by a soil covering. Thus, it is extremely unlikely that it will experience such temperatures at the TMA facility. Furthermore, it is important to note that thermal expansion-contraction is reversible, having no long-term effect on geomembranes.

Measures are typically taken during geomembrane installation to minimize the effects of thermal contraction-expansion. For example, an appropriate installation schedule (e.g., installing the geomembrane at relatively low ambient temperature, installing adjacent panels at similar ambient temperatures) can minimize thermal contraction of the geomembrane [Giroud and Peggs, 1990]. Also, it is a standard precaution during installation to remove the wrinkles which may result from thermal expansion and that are judged to be undesirable, i.e., those wrinkles that might fold over (or crease) during placement of the soil. The construction quality assurance plan should address the measures and precautions which can be taken to minimize the effects of thermal expansion-contraction.

5.1.1.2 Applicability to the Geomembrane Liner and Cap

From the above considerations it can be concluded that the geomembrane and cap of the TMA facility should not be expected to deteriorate under the action of thermal energy.

5.1.2 UV Radiation

5.1.2.1 Background Information

When UV radiation is absorbed by polyethylene materials, the molecules of the material break, forming highly reactive fragments called radicals. These radicals may recombine, bonding with fragments from adjacent molecules in a process called crosslinking (the by-products of the crosslinking process are referred to as crosslinks). Crosslinks have been found to occur only in the molecules of the amorphous regions of polyethylene materials. Because the polyethylene molecules are closely packed in the crystals, formation of fragments in the crystals is inhibited. As mentioned above, fragments are the precursors to crosslinks [Birkinshaw et al., 1989]. The radicals may also remain as molecular fragments, resulting in the breakdown of polyethylene molecules in a process called chain scission. However, crosslinking predominates.

UV radiation is not energetic enough to penetrate very far below the surface of polyethylene materials, therefore it is essentially a surface phenomenon. Even in the absence of carbon black, UV radiation does not penetrate far below the surface of the geomembrane.

5.1.2.2 Protection of HDPE from the Action of UV Radiation

To protect HDPE geomembranes from the effect of UV radiation, they are compounded with approximately 2.5 percent carbon black, an additive incorporated in the geomembrane during the manufacturing process. Carbon black screens UV radiation [Whitney, 1988], practically limiting the penetration of solar energy to a very shallow surface layer. There is considerable experience with the use of carbon black to protect polymers. The use of carbon black in polyethylene began in 1942 [Gilroy, 1985] for low density polyethylene (LDPE) used as telephone cable jackets, an application requiring constant outdoor exposure. Studies have indicated that carbon black must be finely divided and evenly dispersed throughout a material at a concentration of approximately 2.5% for optimum UV absorption [Whitney, 1988; Gilroy, 1985]. (In the geosynthetics industry, there are specifications for carbon black content and carbon black dispersion in geomembranes, e.g., National Sanitation Foundation (NSF) Standard 54, [NSF, 1991] for carbon black content: 2.0-3.0%, and for carbon black dispersion: A1, A2, or B1, where A1, A2, and B1 refer to standard patterns of dispersion.)

5.1.2.3 Applicability to the Geomembrane Liner and Cap

As discussed above, the action of UV radiation is only a surface phenomenon. In addition, the geomembrane and cap will be exposed to UV radiation only during facility construction and construction quality assurance activities. Therefore, it is not likely to adversely impact the performance of the geomembrane liner and cap. It is noted that UV radiation may also provide the energy that is required to initiate the oxidation process. This will be discussed in Section 5.3.

5.2 Reactivity and Interaction

5.2.1 Chemical Reactivity

5.2.1.1 Background Information

HDPE materials are chemically resistant due to two essential features. First, as all members of the polyethylene family, HDPE is essentially inert because it has no reactive sites. It does not react with most chemicals, which includes water and other inorganic chemicals, such as acids (pH < 7) (with the exception of oxidizing acids) or bases, (pH > 7). Second, as stated in Section 3, HDPE has a low permeability and, therefore, resists penetration by chemicals. However, under certain conditions, HDPE can chemically react and physically interact with chemicals that can adversely impact its performance. Such processes, or mechanisms, are discussed below.

5.2.1.2 Laboratory Experience

As discussed above, HDPE is very unreactive toward inorganic chemicals, such as those likely to be present in the material to be placed in the TMA. For example, in a study reported by Whyatt and Farnsworth [1990] in which an HDPE geomembrane was exposed for 120 days at 194°F (90°C) to a pH=14 solution containing metals, the geomembrane did not undergo chemically induced changes in properties. The authors of the study concluded that the geomembrane was resistant to chemical attack and should retain its ability to resist permeation. In another study, Haxo [USEPA, 1988] showed that both high density and low density polyethylene geomembranes are resistant to high pH (basic), low pH (acidic), and brine solutions. Soo et al. [1986] tested the effect of a low pH medium containing sulfuric acid and other compounds on HDPE and found that the HDPE did not undergo chemically induced changes in properties.

Finally, GeoSyntec has the following experience in chemical compatibility tests conducted on HDPE geomembranes using the EPA Method 9090 [USEPA, 1986]: more than 25 tests in basic media; and more than 20 tests in acidic media (which represents the worst-case leachate from the TMA facility as indicated in Section 4.5). In every case, the polyethylene geomembranes did not undergo chemical-induced changes in properties. It should be noted that these tests were conducted by an accredited laboratory, and that the results were accepted by regulatory agencies, resulting in permits for the waste disposal facilities.

5.2.1.3 Applicability to the HDPE Geomembrane Liner and Cap

The leachate from the TMA facility has the potential, in the worst case, to have a pH of less than 7 (i.e., has the potential to be acidic), thus the fact that HDPE geomembranes have high chemical compatibility with acidic media is applicable to the TMA facility.

5.2.2 Physical Interaction of HDPE and Chemicals

5.2.2.1 Background Information

Physical interaction of HDPE with a chemical occurs when HDPE, without experiencing change in the structure of its molecules, absorbs the chemical, which is usually organic. Organic chemicals can interact with HDPE because, like HDPE, they are nonpolar and, therefore, have similar intermolecular forces (cohesive forces) holding adjacent molecules together. Conversely, inorganic molecules are polar (i.e., are equivalent to dipoles from an electrical standpoint) and have very strong intermolecular forces holding them together. Therefore, they do not interact with polyethylene, and, as a result, the HDPE geomembrane liner and cap should not be impacted adversely by them.

Chemicals that can physically interact with HDPE are essentially non-polar and are the aromatic hydrocarbons (e.g., benzene, toluene, xylene, etc.), the chlorinated hydrocarbons (e.g., trichloroethylene, methylene chloride, etc.) and the aliphatic hydrocarbons (e.g., butane, pentane, hexane, etc.). The chemicals that physically interact with HDPE can permeate through an HDPE geomembrane. However, restricted by size, chemicals consisting of large molecules do not readily permeate HDPE. Small molecules, can permeate HDPE. (It is noted that water is small and, to a limited degree, can permeate HDPE. However, water is highly polar, and its permeation is restricted because it is incompatible with HDPE (i.e., HDPE is non-polar). Similarly, ionic chemicals (which are polar), such as salts, do not permeate HDPE materials because of their polarity and, in some cases, their size and polarity.) Finally, it should be noted that the effects of physical interaction are reversible, i.e., the effect is physical, not chemical, and should have no impact on the durability of the HDPE geomembrane.

5.2.2.2 Applicability to the Geomembrane Liner and Cap

As discussed in Section 4, the TMA facility leachate is not expected to have volatile organic chemicals. Therefore, the mechanisms of interaction of volatile organic chemicals with the HDPE geomembrane liner and cap are not relevant to the TMA facility.

5.2.3 Effect of Oxygen

5.2.3.1 Background Information on Oxidation

The predominant reaction of polyethylene materials and chemicals is oxidation. The following describes how polyethylene is oxidized and discusses the role of antioxidants.

5.2.3.2 Oxidation Mechanism

Energy supplied by (i) components of sunlight, i.e., infrared (heat) radiation and ultraviolet (UV) radiation (although UV radiation predominates) and (ii) high-energy radiation (i.e. radioactivity) may cause oxidation of HDPE, a chemical reaction of HDPE with oxygen. Oxidation is a step-wise process [Ciba-Geigy, 1987]. The polymer (such as polyethylene) first absorbs energy. This absorption excites the polymer molecules, causing them to break ("chain scission"), forming highly reactive fragments referred to as radicals. The radicals then react with oxygen, forming even more radicals. (It should be noted that oxygen is highly reactive for two reasons: (i) it is very small and can penetrate materials very easily; and (ii) it is a radical.) The process is terminated when the radicals either recombine or react with foreign materials, such as antioxidants (which will be discussed in Section 5.2.3.3), or when energy is no longer supplied.

5.2.3.3 Protection of HDPE Geomembranes from Oxidation

Because of the potential for energy from sunlight or high-energy radiation to cause oxidation that will reduce the performance of HDPE, HDPE materials are made resistant to the action of energy by antioxidants (sometimes called UV stabilizers), additives that are incorporated into the material at the manufacturing stage. Antioxidants owe their name to the fact that they prevent the oxidation process from developing. However, it is important to note that antioxidants act before oxygen reacts with radicals and, therefore, are effective in controlling the action of energy even in the absence of oxygen.

Antioxidants are complex chemicals that stop the oxidation process by reacting with radicals as soon as they are generated by the action of energy in the early stage of the oxidation process [Ciba-Geigy, 1987; Brydson, 1982; Kiss et al., 1990]. The reaction of antioxidants with radicals give neutral products, whereas, the reaction of oxygen with radicals of polyethylene give even more radicals. Antioxidants are typically present at a concentration of approximately 0.5% in HDPE materials. The use of antioxidants in polyethylene materials has been found to prevent or greatly delay the development of the oxidation process [Ciba-Geigy, 1987; Gray, 1990; Gray, 1991].

5.2.3.4 Oxidation Without a Significant Source of Energy

Experimental Results

Oxidation of HDPE samples with and without antioxidants has been found to occur to a limited extent in environments with very little energy and a limited supply of oxygen [Albertsson and Banhidi, 1980; Dolezel, 1967].

Abiotic oxidation and bio-oxidation tests have been reported by Albertsson and Banhidi [1980] (abiotic oxidation and bio-oxidation are described below). In these tests, small amounts of polyethylene molecules from HDPE were converted to carbon dioxide. If oxidation is allowed to proceed, molecules are broken down into smaller and smaller fragments, ultimately resulting in the formation of carbon dioxide. As a result, HDPE experienced a small loss in mass. As indicated by Albertsson and Banhidi [1980], only the polyethylene molecules with very low molecular weight were consumed, because they tend to reside on the surface of HDPE materials and are therefore the most accessible.

In the abiotic oxidation experiments, powdered HDPE was stored in water in the dark. Carbon dioxide was liberated, and the powdered HDPE sample underwent a mass loss at a rate of 0.048% per year.

In the bio-oxidation experiments, powdered HDPE was stored in biologically rich soils. Carbon dioxide was liberated, and the powdered HDPE sample underwent a mass loss at a rate of 0.073% per year.

Applicability to the Geomembrane Liner and Cap

The geomembrane will not be exposed to soils that are biologically rich, therefore the bio-oxidation experiments are not applicable to the geomembrane at the TMA facility. However, the abiotic oxidation experiments are applicable and will be discussed below.

5.2.3.5 Application of Results to the HDPE Geomembrane Liner and Cap

Calculated Rate of Mass Loss for the Geomembrane

Using the mass-loss rates provided in Section 5.2.3.4, the amount of HDPE lost from the geomembrane may be calculated. However, since the rate of oxidation is proportional to HDPE's exposed surface area [Wrigley, 1989], HDPE's surface-to-mass ratio must first be considered. The powdered HDPE sample in the foregoing study had a surface-to-mass ratio of 10.5 m^2/g [Albertsson and Banhidi, 1980], and a 60-mil (1.5-mm) thick geomembrane has a surface-to-mass ratio, calculated by the authors of this report, of 0.0014 m^2/g, considering the two faces of the geomembrane. The amount of the HDPE geomembrane's mass lost in one year due to abiotic-oxidation, would be 0.0000672%.

Accordingly, the mass lost would be:

- 0.00672% after 100 years;

- 0.01344% after 200 years;
- 0.02016% after 300 years; and
- 0.03360% after 500 years.

Even after 500 years, which is a very long time, this calculated mass loss is extremely small.

Effect of Mass Loss on the Performance of HDPE

As discussed above, Albertsson and Banhidi [1980] indicated that the polyethylene molecules consumed by abiotic oxidation (and also bio-oxidation) were of very low molecular weight, residing at or near the surface of the HDPE material. These molecules are preferentially attacked because they are accessible. Even though the molecules at the surface are attacked, the amount is very small, even after 500 years. Higher molecular weight molecules, which are part of the amorphous/crystalline structure of HDPE, are not accessible and are therefore not attacked. Molecules of the amorphous/crystalline structure of HDPE provide HDPE with its performance properties, whereas the low molecular weight molecules located at the surface of the HDPE material do not [Wrigley, 1989; Koerner et al., 1990]. Consequently, loss of the low molecular weight molecules should not impact the performance of HDPE geomembranes.

5.2.3.6 Applicability of the Action of Oxygen

In order for appreciable oxidation to occur, a significant supply of oxygen and a source of energy are needed. For the TMA facility, the geomembrane will only encounter such conditions during installation. However, with the antioxidants that are used in typical HDPE geomembranes, the amount of oxidation should be negligible, and should not impact the performance of the HDPE geomembrane at the TMA facility.

5.3 Micro-Biological Activities

Bacteria and fungi may attack materials by either accelerating hydrolysis (which does not occur in polyethylene materials) or by bio-oxidation. Bio-oxidation was discussed in Section 5.2.3.4 and was found not to impact the performance of HDPE.

5.4 Macro-Biological Activities

Wrigley [1987 and 1989] reports that HDPE materials have not been found to be food sources for larger organisms. In describing marine exposure tests, no significant attack occurred on HDPE, even from marine borers, which are very aggressive organisms. In contrast, Haxo and Haxo [1989] indicate that synthetic liners may be subject to gnawing from burrowing animals. (It should be noted that the authors of this report have no knowledge of any situation where animals have burrowed through an HDPE geomembrane.) However, in the case of the liner and cap at the TMA facility, burrowing should not be a factor since the design incorporates a drainage layer consisting of a 1 ft (0.3 m) thick layer of medium to coarse sand with gravel placed on the liner. This layer will provide a deterrent for burrowing since it is coarse and cohesionless and will not be stable for tunneling. Therefore, it is extremely unlikely that the geomembrane and cap will be subject to attack by macro-biological organisms.

5.5 Mechanical Stresses

All durability evaluations, such as those presented in the preceding sections, are based on the assumption that the geomembrane liner or cap of a waste containment facility has not been damaged by mechanical stresses (due to gravity and other causes) at any time during service life. This assumption can be made because of the following elements of the current state of practice:

- through theoretical analyses and performance evaluation of actual landfills, causes of stresses that may impact geomembrane liners and caps have been identified;

- design methods have been developed to eliminate or alleviate the stresses resulting from those potential causes; and

- construction methods and construction quality assurance procedures are available which ensure that liner systems are constructed in accordance with design and specifications.

It is understood that the TMA facility will be designed according to the state of practice and, therefore, the mechanical stresses imparted by gravity and other causes should be minimized to the point they do not adversely impact geomembrane durability.

5.6 Summary on Mechanisms of Degradation

The approach used in Section 5 consisted of systematically reviewing all potential failure mechanisms and evaluating their applicability to the TMA facility. From the discussions presented in Sections 5.1 and 5.5, it appears that HDPE geomembranes are extremely durable materials, but, as with every material, their performance may be impacted by exposure to some aggressive environments which include extreme heat, UV radiation, and aggressive chemicals. However, as indicated in Section 4, such conditions should not exist at the TMA facility, therefore, the performance of the HDPE geomembrane liner and cap is not expected to be significantly impacted by long-term exposure to the environmental conditions of the TMA facility. Therefore, the conclusions of a panel of experts presented below apply to the TMA facility.

A panel of polymer and geotechnical experts assembled by the United States Environmental Protection Agency (EPA) [Haxo and Haxo, 1988] to discuss the durability of geosynthetic materials used for lining waste disposal facilities made the following conclusions regarding the long-term durability of lining materials:

> *"The basic conditions to which polymeric FMLs and other components of a liner system are exposed in both MSW and hazardous waste landfills include comparatively low ambient temperatures, lack of light, moisture, aerobic and anaerobic atmospheres depending on the component of the liner system and the location within the fill, and low concentrations of dissolved constituents. Thus,*

polymeric materials placed in service in liner systems do not encounter the types of conditions that are normally considered to cause degradation of the base polymeric resins";

"The particular polymers used in the manufacture of products for the construction of landfill liner systems will not degrade in the environments they will encounter in landfills because of the lack of highly aggressive conditions that would cause degradation"; and

"The polymers that were discussed and first-grade compounds based on these polymers should maintain their integrity in landfill environments for considerable lengths of time, probably in terms of 100's of years".

Finally, it should be noted that, in 1995, the Geosynthetics Research Institute (GRI) embarked on a ten-year research program to assess the durability of HDPE geomembranes [GRI, 1995]. The purpose of the study is to provide quantitative lifetime predictions on geomembranes exposed to different environments (water, air, compressive stresses, and tension) at elevated temperature. The degradation data, if any, will be extrapolated using the Arrhenius equation (an equation used to extrapolate reaction time based on the effect of temperature on reaction rate) to predict the lifetime at the relevant temperature in a specific site. Preliminary results indicate that antioxidant depletion is 40 to 120 years, depending on exposure conditions (the program is only evaluating oxidation of HDPE geomembranes). However, the results are based on HDPE geomembranes that have been exposed to heat, UV radiation, or other aggressive environments, conditions which are different from those of the TMA facility, where the geomembrane and cap will be covered with soil. Thus, antioxidant depletion should not be a factor in the performance of the HDPE geomembrane at the TMA facility.

6. CONCLUSIONS

A comprehensive review of potential adverse effects that various environments may have on the durability of HDPE geomembranes has been presented in this report. Based on the facts presented, and the preceding discussions on the site conditions at the TMA facility, it is concluded that the proposed HDPE geomembrane can be expected to be used under conditions that should not adversely impact its long term performance. Specifically, the following conclusions are drawn:

- HDPE geomembranes are chemically resistant to the expected chemical constituents of the TMA facility;
- the performance of the HDPE geomembranes should not be impacted by exposure to the outdoors during installation as currently planned;
- outdoor temperature cycling should not adversely impact the performance of the liner and cap because they will be insulated by a soil cover;
- the HDPE geomembranes are resistant to attack by microorganisms and, because of the design of the facility, should not be subject to attack by macroorganisms; and
- the minimal mechanical stresses likely to be imparted on the liner and cap should not impact their long-term performance.

In conclusion, the HDPE geomembrane liner and cap at the TMA facility should function as designed for a very long time (e.g., hundreds of years) without deterioration in performance. This estimate is consistent with the EPA document discussed in Section 5, which concludes that a geomembrane durability in terms of hundreds of years can be expected.

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Appendix G

Leakage Rates Using Giroud-Bonaparte Equations

Foth & Van Dyke	Client: Crandon Mining Co — Scope I.D.: 93C049 Project: Crandon TMA — Page: 1 of 4 Prepared by: NXP — Date: 7-16-96 Checked by: PAG — Date: 7/17/96

LEAKAGE RATES USING GIROUD-BONAPARTE EQUATIONS

Purpose: To compare percolation through TMA liner using HELP model and Giroud-Bonaparte equations

Scope : For the cases of TMA conditions used in the HELP model studies, compare the results with calculated results using Giroud-Bonaparte equations. Employ the head on the liner as the average annal head obtained from the HELP model runs.

References: Schroeder, P.R et al (1994) "The Hydrologic Evaluation of Landfill Performance (HELP) model, Engineering Documentation for Version 3 EPA/600/R-94/168b, September 1994.

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Limitations : Giroud and Bonaparte equations as used here are obtained from Schroeder et al, who had used Giroud and Bonaparte (1989) and Giroud et al (1992) to develop the equations used in these calculations

Foth & Van Dyke	Client: Crandon Mining Co Scope I.D.: 93C049 Project: Crandon TMA Page: 2 of 4 Prepared by: NXP Date: 7-16-96 Checked by: [illegible] Date: 7/17/96

HELP model

Interfacial $$Q = K_s \, i_{av} \, n \, \pi R^2 \left(\eta_{20}/\eta_{15}\right) \quad — \text{①}$$

K_s = hyd. Con m/sec

i_{av} = hydraulic gradient

n = density of flaws /m²

R = Radius of wetted area

η_{20}, η_{15} viscosity at 20 and 15°

$$i_{av} = 1 + \frac{h_g}{2 T_s \ln \frac{R}{r_o}} \quad — \text{②}$$

h_g = total hydraulic head - m

T_s Thickness of controlling soil layer

r_o = radius of geomembrane flaw
= 0.0113 m for defects; 0.001 m for pinholes

For Good contact

$$R = 0.26 \, a_o^{0.05} \, h_g^{0.45} \, K_s^{-0.13} \quad — \text{③}$$

where a_o = Geomembrane flaw area .0001 m² for installation defects and 0.000000784 m² for pinholes

$K_s = 3 \times 10^{-11}$ m/sec for GCL

$$R = 0.26\,(.0001)^{.05} \times (3\times10^{-11})^{-.13}\; h_g^{.45} \quad \text{for defects}$$

$$= 3.828\; h_g^{0.45}\ \checkmark$$

$$R = 0.26\,(.000000784)^{.05}\,(3\times10^{-11})^{-.13}\; h_g^{.45} \quad \text{for pinholes}$$

$$= 3.003\; h_g^{0.45}\ \checkmark$$

$$i_{av} = 1 + \frac{h_g}{2(.0061)\ln\frac{R}{.0113}} \quad \text{for defects}$$

$$= 1 + \frac{h_g}{2(.0061)\ln\frac{R}{.001016}} \quad \text{for pinholes}$$

$$Q = K_s\, i_{av} \cdot n\, \pi R^2 \,(n_{20}/n_{15}) \qquad \underline{\text{defects}}$$

$$= 3\times10^{-11} \times \frac{1}{4046} \times 4 \times \pi \times \frac{.001}{.00114}\; i_{av} R^2 \quad m/s$$

1 acre = 4046 m^2

$$= \frac{3\times10^{-11}}{4046} \times \pi \times (.8772)\, 60\times1440\times365\times\frac{100}{2.54}\; i_{av} R^2 \times n$$

(n = 4) $= 1.015\times10^{-4}\; i_{av} R^2$ ✓ for defects

(n = 1) $2.537\times10^{-5}\; i_{av} R^2$ ✓ for pinholes

Foth & Van Dyke	Client: Crandon Mining Co — Scope I.D.: 93C049
	Project: Crandon TMA — Page: 4 of 4
	Prepared by: NXP — Date: 7-16-96
	Checked by: PAE — Date: 7/17/96

RESULTS

CASE	Av. Head on Liner m	Q from Giroud + Bonaparte in/yr			Q from HELP in/yr
		defect	pinholes	total	
1. Sideslope 1st stage	.0001524	5.48×10^{-7}	8.48×10^{-8}	6.33×10^{-7}	3.8×10^{-8}
2 Base 1st stage	.0.03	1.0×10^{-4}	1.35×10^{-5}	1.14×10^{-4}	1.9×10^{-4}
3 Sideslope 2nd stage (with Geocomposite)	.0001778	6.32×10^{-7}	9.7×10^{-8}	7.29×10^{-7}	4.6×10^{-8}
4. Sideslope 2nd stage (without Geocomposite)	4.051	.276	.032	.308	.399
5 Base 2nd stage	.0322	1.09×10^{-4}	1.46×10^{-5}	1.23×10^{-4}	1.9×10^{-4}
6. Base - Post Closure Monitoring Period	.0163	4.88×10^{-5}	6.9×10^{-6}	5.57×10^{-5}	1×10^{-4}
7 Base - Leachate System shutoff	.0162	4.84×10^{-5}	6.8×10^{-6}	5.52×10^{-5}	1×10^{-4}
8 Sideslope (without Geocomposite) Post Closure Monitoring Period	0.083	3.88×10^{-4}	4.85×10^{-5}	4.37×10^{-4}	5.5×10^{-3}

Appendix H

Runoff Basin 13
Surface Water Management Design Calculations Including TR-55 Hydrology and POND-2 Models

Crandon Mining Company

TMA Runoff Basin 13
QTR-55 Hydrology
POND-2 Hydrology

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 08-21-1996 13:48:30
Watershed file: --> TMA13 .WSD
Hydrograph file: --> TMA13 .HYD

CRANDON MINING COMPANY
FOTH AND VAN DYKE 93C049.54
BASIN 13 (NE: DISCHARGE TO HEMLOCK CREEK) 8/96
100 YEAR STORM EVENT

>>>> Input Parameters Used to Compute Hydrograph <<<<

Subarea Description	AREA (acres)	CN	Tc (hrs)	* Tt (hrs)	Precip. (in)	Runoff (in)	Ia/p input/used
STOCKPILE	12.40	60.0	0.20	0.00	5.00	1.30	.27 .30

* Travel time from subarea outfall to composite watershed outfall point.
Total area = 12.40 acres or 0.01938 sq.mi
Peak discharge = 18 cfs

>>>> Computer Modifications of Input Parameters <<<<<

Subarea Description	Input Values Tc (hr)	* Tt (hr)	Rounded Values Tc (hr)	* Tt (hr)	Ia/p Interpolated (Yes/No)	Ia/p Messages
STOCKPILE	0.20	0.00	**	**	No	--

* Travel time from subarea outfall to composite watershed outfall point.
** Tc & Tt are available in the hydrograph tables.

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 08-21-1996 13:48:30
Watershed file: --> TMA13 .WSD
Hydrograph file: --> TMA13 .HYD

CRANDON MINING COMPANY
FOTH AND VAN DYKE 93C049.54
BASIN 13 (NE: DISCHARGE TO HEMLOCK CREEK) 8/96
100 YEAR STORM EVENT

>>>> Summary of Subarea Times to Peak <<<<

Subarea	Peak Discharge at Composite Outfall (cfs)	Time to Peak at Composite Outfall (hrs)
STOCKPILE	18	12.2
Composite Watershed	18	12.2

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 08-21-1996 13:48:30
Watershed file: --> TMA13 .WSD
Hydrograph file: --> TMA13 .HYD

CRANDON MINING COMPANY
FOTH AND VAN DYKE 93C049.54
BASIN 13 (NE: DISCHARGE TO HEMLOCK CREEK) 8/96
100 YEAR STORM EVENT

Composite Hydrograph Summary (cfs)

Subarea Description	11.0 hr	11.3 hr	11.6 hr	11.9 hr	12.0 hr	12.1 hr	12.2 hr	12.3 hr	12.4 hr
STOCKPILE	0	0	0	1	5	14	18	13	7
Total (cfs)	0	0	0	1	5	14	18	13	7

Subarea Description	12.5 hr	12.6 hr	12.7 hr	12.8 hr	13.0 hr	13.2 hr	13.4 hr	13.6 hr	13.8 hr
STOCKPILE	5	4	3	3	2	2	2	2	2
Total (cfs)	5	4	3	3	2	2	2	2	2

Subarea Description	14.0 hr	14.3 hr	14.6 hr	15.0 hr	15.5 hr	16.0 hr	16.5 hr	17.0 hr	17.5 hr
STOCKPILE	1	1	1	1	1	1	1	1	1
Total (cfs)	1	1	1	1	1	1	1	1	1

Subarea Description	18.0 hr	19.0 hr	20.0 hr	22.0 hr	26.0 hr
STOCKPILE	1	1	1	0	0
Total (cfs)	1	1	1	0	0

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 08-21-1996 13:48:30
Watershed file: --> TMA13 .WSD
Hydrograph file: --> TMA13 .HYD

CRANDON MINING COMPANY
FOTH AND VAN DYKE 93C049.54
BASIN 13 (NE: DISCHARGE TO HEMLOCK CREEK) 8/96
100 YEAR STORM EVENT

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
11.0	0	14.8	1
11.1	0	14.9	1
11.2	0	15.0	1
11.3	0	15.1	1
11.4	0	15.2	1
11.5	0	15.3	1
11.6	0	15.4	1
11.7	0	15.5	1
11.8	1	15.6	1
11.9	1	15.7	1
12.0	5	15.8	1
12.1	14	15.9	1
12.2	18	16.0	1
12.3	13	16.1	1
12.4	7	16.2	1
12.5	5	16.3	1
12.6	4	16.4	1
12.7	3	16.5	1
12.8	3	16.6	1
12.9	2	16.7	1
13.0	2	16.8	1
13.1	2	16.9	1
13.2	2	17.0	1
13.3	2	17.1	1
13.4	2	17.2	1
13.5	2	17.3	1
13.6	2	17.4	1
13.7	2	17.5	1
13.8	2	17.6	1
13.9	2	17.7	1
14.0	1	17.8	1
14.1	1	17.9	1
14.2	1	18.0	1
14.3	1	18.1	1
14.4	1	18.2	1
14.5	1	18.3	1
14.6	1	18.4	1
14.7	1	18.5	1

TR-55 TABULAR HYDROGRAPH METHOD
Type II Distribution
(24 hr. Duration Storm)

Executed: 08-21-1996 13:48:30
Watershed file: --> TMA13 .WSD
Hydrograph file: --> TMA13 .HYD

CRANDON MINING COMPANY
FOTH AND VAN DYKE 93C049.54
BASIN 13 (NE: DISCHARGE TO HEMLOCK CREEK) 8/96
100 YEAR STORM EVENT

Time (hrs)	Flow (cfs)	Time (hrs)	Flow (cfs)
18.6	1	22.4	0
18.7	1	22.5	0
18.8	1	22.6	0
18.9	1	22.7	0
19.0	1	22.8	0
19.1	1	22.9	0
19.2	1	23.0	0
19.3	1	23.1	0
19.4	1	23.2	0
19.5	1	23.3	0
19.6	1	23.4	0
19.7	1	23.5	0
19.8	1	23.6	0
19.9	1	23.7	0
20.0	1	23.8	0
20.1	1	23.9	0
20.2	1	24.0	0
20.3	1	24.1	0
20.4	1	24.2	0
20.5	1	24.3	0
20.6	1	24.4	0
20.7	1	24.5	0
20.8	1	24.6	0
20.9	1	24.7	0
21.0	0	24.8	0
21.1	0	24.9	0
21.2	0	25.0	0
21.3	0	25.1	0
21.4	0	25.2	0
21.5	0	25.3	0
21.6	0	25.4	0
21.7	0	25.5	0
21.8	0	25.6	0
21.9	0	25.7	0
22.0	0	25.8	0
22.1	0	25.9	0
22.2	0		
22.3	0		

```
***************************************************
*                                                 *
*    CRANDON MINING COMPANY              8/96     *
*                 TMA SED. BASIN 13               *
*   FOTH AND VAN DYKE                 93C049.54   *
*            100-YR/24-HR STORM EVENT             *
*                                                 *
***************************************************
```

Inflow Hydrograph: TMA13 .HYD
Rating Table file: TMA13 .PND

----INITIAL CONDITIONS----
Elevation = 1618.99 ft
Outflow = 0.00 cfs
Storage = 0.00 ac-ft

GIVEN POND DATA			INTERMEDIATE ROUTING COMPUTATIONS	
ELEVATION (ft)	OUTFLOW (cfs)	STORAGE (ac-ft)	2S/t (cfs)	2S/t + 0 (cfs)
1618.99	0.0	0.000	0.0	0.0
1619.19	0.0	0.003	0.8	0.8
1619.39	0.0	0.007	1.6	1.6
1619.59	0.0	0.011	2.6	2.6
1619.79	0.0	0.016	3.8	3.8
1619.99	0.0	0.021	5.1	5.1
1620.19	0.0	0.027	6.6	6.6
1620.39	0.0	0.034	8.2	8.2
1620.59	0.0	0.041	10.0	10.0
1620.79	0.0	0.050	12.0	12.0
1620.99	0.0	0.059	14.2	14.2
1621.19	0.0	0.069	16.7	16.7
1621.39	0.0	0.080	19.3	19.3
1621.59	0.0	0.092	22.2	22.2
1621.79	0.0	0.104	25.3	25.3
1621.99	0.0	0.118	28.6	28.6
1622.19	0.0	0.168	40.8	40.8
1622.39	0.0	0.222	53.7	53.7
1622.59	0.0	0.278	67.3	67.3
1622.79	0.0	0.337	81.5	81.5
1622.99	0.0	0.398	96.2	96.2
1623.19	0.0	0.461	111.6	111.6
1623.39	2.4	0.527	127.6	130.0
1623.59	14.3	0.596	144.3	158.6
1623.79	31.9	0.668	161.6	193.5
1623.99	53.9	0.742	179.6	233.5

Time increment (t) = 0.100 hrs.

Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

INFLOW HYDROGRAPH — ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
11.000	0.00	-----	0.0	0.0	0.00	1618.99
11.100	0.00	0.0	0.0	0.0	0.00	1618.99
11.200	0.00	0.0	0.0	0.0	0.00	1618.99
11.300	0.00	0.0	0.0	0.0	0.00	1618.99
11.400	0.00	0.0	0.0	0.0	0.00	1618.99
11.500	0.00	0.0	0.0	0.0	0.00	1618.99
11.600	0.00	0.0	0.0	0.0	0.00	1618.99
11.700	0.00	0.0	0.0	0.0	0.00	1618.99
11.800	1.00	1.0	1.0	1.0	0.00	1619.25
11.900	1.00	2.0	3.0	3.0	0.00	1619.65
12.000	5.00	6.0	9.0	9.0	0.00	1620.48
12.100	14.00	19.0	28.0	28.0	0.00	1621.95
12.200	18.00	32.0	60.0	60.0	0.00	1622.48
12.300	13.00	31.0	91.0	91.0	0.00	1622.92
12.400	7.00	20.0	111.0	111.0	0.00	1623.18
12.500	5.00	12.0	120.0	123.0	1.49	1623.31
12.600	4.00	9.0	124.5	129.0	2.27	1623.38
12.700	3.00	7.0	125.5	131.5	3.01	1623.40
12.800	3.00	6.0	125.5	131.5	3.00	1623.40
12.900	2.00	5.0	125.3	130.5	2.58	1623.39
13.000	2.00	4.0	124.7	129.3	2.31	1623.38
13.100	2.00	4.0	124.2	128.7	2.23	1623.38
13.200	2.00	4.0	123.9	128.2	2.17	1623.37
13.300	2.00	4.0	123.6	127.9	2.12	1623.37
13.400	2.00	4.0	123.5	127.6	2.09	1623.36
13.500	2.00	4.0	123.3	127.5	2.07	1623.36
13.600	2.00	4.0	123.2	127.3	2.05	1623.36
13.700	2.00	4.0	123.2	127.2	2.04	1623.36
13.800	2.00	4.0	123.1	127.2	2.03	1623.36
13.900	2.00	4.0	123.1	127.1	2.02	1623.36
14.000	1.00	3.0	122.3	126.1	1.88	1623.35
14.100	1.00	2.0	121.0	124.3	1.65	1623.33
14.200	1.00	2.0	120.0	123.0	1.48	1623.31
14.300	1.00	2.0	119.3	122.0	1.36	1623.30
14.400	1.00	2.0	118.8	121.3	1.26	1623.30
14.500	1.00	2.0	118.4	120.8	1.20	1623.29
14.600	1.00	2.0	118.1	120.4	1.14	1623.29
14.700	1.00	2.0	117.9	120.1	1.11	1623.28
14.800	1.00	2.0	117.7	119.9	1.08	1623.28
14.900	1.00	2.0	117.6	119.7	1.06	1623.28
15.000	1.00	2.0	117.5	119.6	1.04	1623.28
15.100	1.00	2.0	117.4	119.5	1.03	1623.28
15.200	1.00	2.0	117.4	119.4	1.02	1623.28
15.300	1.00	2.0	117.4	119.4	1.02	1623.27
15.400	1.00	2.0	117.3	119.4	1.01	1623.27

POND-2 Version: 5.16 S/N: 1295130172
EXECUTED: 08-23-1996 08:23:18

Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

INFLOW HYDROGRAPH — ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
15.500	1.00	2.0	117.3	119.3	1.01	1623.27
15.600	1.00	2.0	117.3	119.3	1.01	1623.27
15.700	1.00	2.0	117.3	119.3	1.01	1623.27
15.800	1.00	2.0	117.3	119.3	1.00	1623.27
15.900	1.00	2.0	117.3	119.3	1.00	1623.27
16.000	1.00	2.0	117.3	119.3	1.00	1623.27
16.100	1.00	2.0	117.3	119.3	1.00	1623.27
16.200	1.00	2.0	117.3	119.3	1.00	1623.27
16.300	1.00	2.0	117.3	119.3	1.00	1623.27
16.400	1.00	2.0	117.3	119.3	1.00	1623.27
16.500	1.00	2.0	117.3	119.3	1.00	1623.27
16.600	1.00	2.0	117.3	119.3	1.00	1623.27
16.700	1.00	2.0	117.3	119.3	1.00	1623.27
16.800	1.00	2.0	117.3	119.3	1.00	1623.27
16.900	1.00	2.0	117.3	119.3	1.00	1623.27
17.000	1.00	2.0	117.3	119.3	1.00	1623.27
17.100	1.00	2.0	117.3	119.3	1.00	1623.27
17.200	1.00	2.0	117.3	119.3	1.00	1623.27
17.300	1.00	2.0	117.3	119.3	1.00	1623.27
17.400	1.00	2.0	117.3	119.3	1.00	1623.27
17.500	1.00	2.0	117.3	119.3	1.00	1623.27
17.600	1.00	2.0	117.3	119.3	1.00	1623.27
17.700	1.00	2.0	117.3	119.3	1.00	1623.27
17.800	1.00	2.0	117.3	119.3	1.00	1623.27
17.900	1.00	2.0	117.3	119.3	1.00	1623.27
18.000	1.00	2.0	117.3	119.3	1.00	1623.27
18.100	1.00	2.0	117.3	119.3	1.00	1623.27
18.200	1.00	2.0	117.3	119.3	1.00	1623.27
18.300	1.00	2.0	117.3	119.3	1.00	1623.27
18.400	1.00	2.0	117.3	119.3	1.00	1623.27
18.500	1.00	2.0	117.3	119.3	1.00	1623.27
18.600	1.00	2.0	117.3	119.3	1.00	1623.27
18.700	1.00	2.0	117.3	119.3	1.00	1623.27
18.800	1.00	2.0	117.3	119.3	1.00	1623.27
18.900	1.00	2.0	117.3	119.3	1.00	1623.27
19.000	1.00	2.0	117.3	119.3	1.00	1623.27
19.100	1.00	2.0	117.3	119.3	1.00	1623.27
19.200	1.00	2.0	117.3	119.3	1.00	1623.27
19.300	1.00	2.0	117.3	119.3	1.00	1623.27
19.400	1.00	2.0	117.3	119.3	1.00	1623.27
19.500	1.00	2.0	117.3	119.3	1.00	1623.27
19.600	1.00	2.0	117.3	119.3	1.00	1623.27
19.700	1.00	2.0	117.3	119.3	1.00	1623.27
19.800	1.00	2.0	117.3	119.3	1.00	1623.27
19.900	1.00	2.0	117.3	119.3	1.00	1623.27
20.000	1.00	2.0	117.3	119.3	1.00	1623.27

Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

INFLOW HYDROGRAPH — ROUTING COMPUTATIONS

TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
20.100	1.00	2.0	117.3	119.3	1.00	1623.27
20.200	1.00	2.0	117.3	119.3	1.00	1623.27
20.300	1.00	2.0	117.3	119.3	1.00	1623.27
20.400	1.00	2.0	117.3	119.3	1.00	1623.27
20.500	1.00	2.0	117.3	119.3	1.00	1623.27
20.600	1.00	2.0	117.3	119.3	1.00	1623.27
20.700	1.00	2.0	117.3	119.3	1.00	1623.27
20.800	1.00	2.0	117.3	119.3	1.00	1623.27
20.900	1.00	2.0	117.3	119.3	1.00	1623.27
21.000	0.00	1.0	116.5	118.3	0.87	1623.26
21.100	0.00	0.0	115.2	116.5	0.64	1623.24
21.200	0.00	0.0	114.3	115.2	0.48	1623.23
21.300	0.00	0.0	113.6	114.3	0.35	1623.22
21.400	0.00	0.0	113.1	113.6	0.26	1623.21
21.500	0.00	0.0	112.7	113.1	0.19	1623.21
21.600	0.00	0.0	112.4	112.7	0.14	1623.20
21.700	0.00	0.0	112.2	112.4	0.11	1623.20
21.800	0.00	0.0	112.0	112.2	0.08	1623.20
21.900	0.00	0.0	111.9	112.0	0.06	1623.19
22.000	0.00	0.0	111.8	111.9	0.04	1623.19
22.100	0.00	0.0	111.8	111.8	0.03	1623.19
22.200	0.00	0.0	111.7	111.8	0.02	1623.19
22.300	0.00	0.0	111.7	111.7	0.02	1623.19
22.400	0.00	0.0	111.7	111.7	0.01	1623.19
22.500	0.00	0.0	111.6	111.7	0.01	1623.19
22.600	0.00	0.0	111.6	111.6	0.01	1623.19
22.700	0.00	0.0	111.6	111.6	0.01	1623.19
22.800	0.00	0.0	111.6	111.6	0.00	1623.19
22.900	0.00	0.0	111.6	111.6	0.00	1623.19
23.000	0.00	0.0	111.6	111.6	0.00	1623.19
23.100	0.00	0.0	111.6	111.6	0.00	1623.19
23.200	0.00	0.0	111.6	111.6	0.00	1623.19
23.300	0.00	0.0	111.6	111.6	0.00	1623.19
23.400	0.00	0.0	111.6	111.6	0.00	1623.19
23.500	0.00	0.0	111.6	111.6	0.00	1623.19
23.600	0.00	0.0	111.6	111.6	0.00	1623.19
23.700	0.00	0.0	111.6	111.6	0.00	1623.19
23.800	0.00	0.0	111.6	111.6	0.00	1623.19
23.900	0.00	0.0	111.6	111.6	0.00	1623.19
24.000	0.00	0.0	111.6	111.6	0.00	1623.19
24.100	0.00	0.0	111.6	111.6	0.00	1623.19
24.200	0.00	0.0	111.6	111.6	0.00	1623.19
24.300	0.00	0.0	111.6	111.6	0.00	1623.19
24.400	0.00	0.0	111.6	111.6	0.00	1623.19
24.500	0.00	0.0	111.6	111.6	0.00	1623.19
24.600	0.00	0.0	111.6	111.6	0.00	1623.19

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Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

INFLOW HYDROGRAPH		ROUTING COMPUTATIONS				
TIME (hrs)	INFLOW (cfs)	I1+I2 (cfs)	2S/t - O (cfs)	2S/t + O (cfs)	OUTFLOW (cfs)	ELEVATION (ft)
24.700	0.00	0.0	111.6	111.6	0.00	1623.19
24.800	0.00	0.0	111.6	111.6	0.00	1623.19
24.900	0.00	0.0	111.6	111.6	0.00	1623.19
25.000	0.00	0.0	111.6	111.6	0.00	1623.19
25.100	0.00	0.0	111.6	111.6	0.00	1623.19
25.200	0.00	0.0	111.6	111.6	0.00	1623.19
25.300	0.00	0.0	111.6	111.6	0.00	1623.19
25.400	0.00	0.0	111.6	111.6	0.00	1623.19
25.500	0.00	0.0	111.6	111.6	0.00	1623.19
25.600	0.00	0.0	111.6	111.6	0.00	1623.19
25.700	0.00	0.0	111.6	111.6	0.00	1623.19
25.800	0.00	0.0	111.6	111.6	0.00	1623.19
25.900	0.00	0.0	111.6	111.6	0.00	1623.19

****************** SUMMARY OF ROUTING COMPUTATIONS ******************

Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

Starting Pond W.S. Elevation = 1618.99 ft

***** Summary of Peak Outflow and Peak Elevation *****

Peak Inflow = 18.00 cfs
Peak Outflow = 3.01 cfs
Peak Elevation = 1623.40 ft

***** Summary of Approximate Peak Storage *****

Initial Storage = 0.00 ac-ft
Peak Storage From Storm = 0.53 ac-ft

Total Storage in Pond = 0.53 ac-ft

POND-2 Version: 5.16 S/N: 1295130172

Pond File: TMA13 .PND
Inflow Hydrograph: TMA13 .HYD
Outflow Hydrograph: OUT .HYD

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08:23:18

Peak Inflow = 18.00 cfs
Peak Outflow = 3.01 cfs
Peak Elevation = 1623.40 ft

```
                                                                   Flow (cfs)
       0.0   2.0   4.0   6.0   8.0  10.0  12.0  14.0  16.0  18.0  20.0  22.0
       .------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-
       |
11.4 -|x
      |x
11.5 -|x
      |x
11.6 -|x
      |x
11.7 -|x
      |x*
11.8 -|x  *
      |x  *
11.9 -|x  *
      |x        *
12.0 -|x              *
      |x                           *
12.1 -|x                                         *
      |x                                                *
12.2 -|x                                                      *
      |x                                              *
12.3 -|x                                       *
      |x                              *
12.4 -|x                     *
      |   x               *
12.5 -|     x          *
      |       x      *
12.6 -|        x    *
      |         x *
12.7 -|          x
      |          x
12.8 -|          x
      |        *x
12.9 -|       * x
      |       *x
13.0 -|       *x
      |       *x
13.1 -|       *x
      |       *x
13.2 -|       *x
      |       x
13.3 -|       x
      |
   TIME
   (hrs)
```

* File: TMA13 .HYD Qmax = 18.0 cfs
x File: OUT .HYD Qmax = 3.0 cfs