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MISCELLANEOUS DETAILS AND ANALYSES
CRANDON PROJECT WASTE DISPOSAL SYSTEM
PROJECT REPORT 11

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

Report on

MISCELLANEOUS DETAILS AND ANALYSES
CRANDON PROJECT WASTE DISPOSAL SYSTEM
PROJECT REPORT 11

Submitted to:

Exxon Minerals Company
P. O. Box 813
Rhinelander, Wisconsin 54501

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September 1982

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Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

September 30, 1982

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F/N 181.6

Exxon Minerals Company
P. O. Box 813
Rhinelander, Wisconsin 54501

Attn: Mr. C. E. Fowler

RE: EXXON CRANDON PROJECT
WASTE DISPOSAL SYSTEM
CRANDON, WISCONSIN

Gentlemen:

We are pleased to present the final draft of our report "Miscellaneous Details and Analyses, Crandon Project Waste Disposal System, Project Report 11". This report presents the results of studies and analyses on various waste disposal system topics which have not been presented in prior project reports.

We appreciate the continuing opportunity to provide services to Exxon Minerals Company for the Crandon Project and extend our thanks to you and the Exxon staff for their excellent cooperation.

Very truly yours,

GOLDER ASSOCIATES

Gary H. Collison

Gary H. Collison, P.E.
Principal

GHC:dap

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1.0 INTRODUCTION

Exxon Minerals Company has retained Golder Associates to provide the preliminary engineering design for use in permitting the waste disposal system for their Crandon Mining Project in Forest County, Wisconsin. The purpose of this report is to present details and analyses for the selected alternative waste dispsoal site, system 41-114B, which have not been incorporated in other preliminary engineering design reports for the Project. The details and analyses included pertain to the site 41-114B waste disposal system and have been considered in the preliminary design.

This report is not intended to provide a review of the complete design nor provide a review of previous reports, studies, or analyses which have led to the discussions and presentations in this report. Where applicable, specific reference is made to other project reports or documents which will provide background information.

2.0 TAILINGS LIQUEFACTION

2.1 Liquefaction Potential

The liquefaction potential of both natural and man-made deposits of granular material depends on a number of factors. A brief review of the relative importance of various material properties and other factors in determining the susceptibility of cohesionless materials to liquefaction is presented below.

Grain size distribution is one of the most important characteristics of cohesionless materials with respect to liquefaction potential. Uniformly graded materials are more susceptible to liquefaction than well graded materials. For uniformly graded soils, fine sands tend to liquify more readily than gravelly soils, silts, or clays. Liquefaction potential is greatest for uniformly graded materials having a mean particle size (D_{50}) in the range of 0.075 to 0.2 mm. The size range of the Crandon tailings is estimated at between 0.004 and 0.03 mm (Ref. 7). Because of the fine grain size, the Crandon tailings are considerably less vulnerable to liquefaction than naturally occurring fine sands occurring at similar densities.

Liquefaction potential is strongly influenced by the magnitude of the in situ effective overburden pressure. For example, increasing the effective overburden pressure by lowering the phreatic surface within a deposit, would improve the stability of a deposit with respect to liquefaction. Conversely, reducing the effective overburden pressure would increase the susceptibility of the deposit to liquefaction.

The intensity and duration of the induced ground motions are of paramount importance in assessing liquefaction potential since they determine the induced shear stress and the number of significant stress cycles which the ground will experience.

2.2 Analysis of Stresses Created by Seismic Excitation

The most commonly used method of evaluating the liquefaction potential of granular materials subjected to seismic loading is based on empirical correlations of in situ materials characteristics and observed performance. A convenient parameter for describing the liquefaction potential of a granular deposit is the cyclic stress ratio which is the ratio of the average cyclic shear stress τ_h developed on a horizontal plane as a result of the seismic loading to the initial vertical effective overburden pressure σ_o'. The cyclic stress ratio developed in the field due to seismic excitation can readily be calculated from the relationship (Ref. 17):

$$\frac{(\tau_h)_{av}}{\sigma_o'} = 0.65 \; \frac{a_{max}}{g} \; \frac{\sigma_o}{\sigma_o'} \; r_d \qquad (2.1)$$

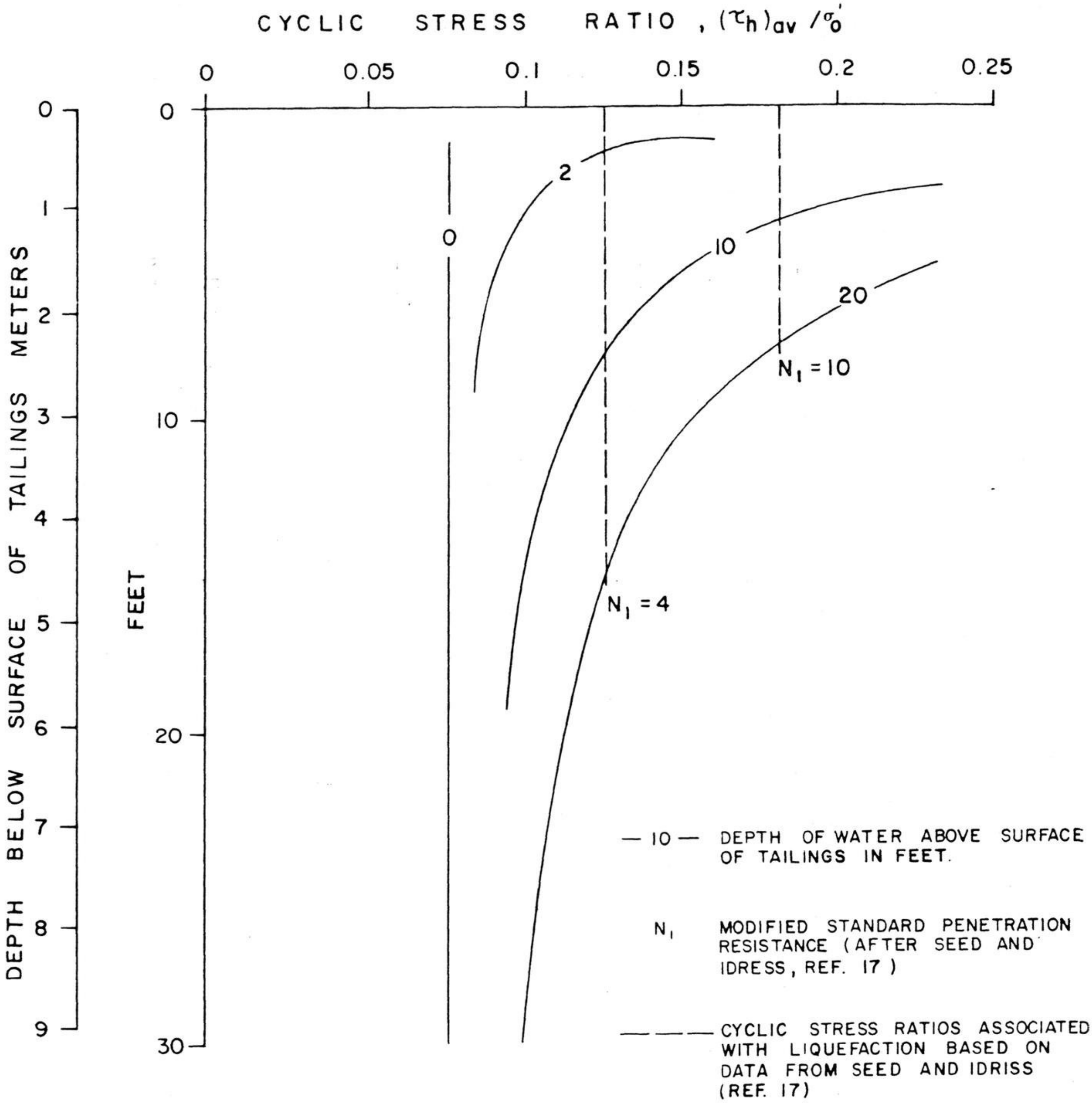
where
- a_{max} = maximum ground surface acceleration
- σ_o = total vertical overburden pressure
- σ_o' = initial vertical effective overburden pressure
- r_d = a stress reduction factor which ranges from 1 at ground surface to about 0.9 at 30 feet (9.1 m) depth.
- g = acceleration due to gravity

Seed and Idriss (Ref. 17) have correlated values of the cyclic stress ratio with a modified penetration resistance derived from blow counts (N values) recorded during Standard Penetration Tests (SPT) for sites where liquefaction has and has not occurred during earthquakes. The data presented by Seed and Idriss indicate that for silty sands having a mean particle size $D_{50} < 0.15$mm and a modified SPT penetration resistance (N_1) of 4 blows per foot, liquefaction could be initiated if the cyclic stress ratio exceeds 0.125 with a Magnitude 7.5 earthquake. For the same seismic event a cyclic stress ratio of 0.18 would be needed to induce liquefaction where the modified SPT penetration resistance (N_1) is 10 blows per foot. The work of Seed and Idriss (Ref. 17) is based on a Magnitude 7.5 earthquake which is a very severe event and provides a conservative upper limit to the estimated depth of the zone of potential liquefaction for the Crandon Project tailings.

2.3 Results of Liquefaction Analysis

Following the above principles, the potential maximum depth of liquefaction for the Crandon tailings can be estimated for a range of water depths above the surface of the tailings based on a ground surface acceleration of 0.06 g. This 0.06 g is the maximum acceleration expected at the Crandon site (Ref. 2). The results of the analyses are shown on Figure 2.1. These curves indicate that for a modified penetration resistance (N_1) of 4 blows per foot liquefaction should not occur unless the depth of water above the surface of the tailings is more than 2 feet (0.61 m) in which case the zone of liquefaction would be limited to the upper 1.5 feet (0.46 m) of tailings.

The range of modified penetration resistance from 4 to 10 blows per foot indicated in Figure 2.1 is equivalent to in situ SPT N values in the upper 15 feet (4.57 m) of tailings of between 3 and 7 blows per foot. This range is considered reasonable for freshly deposited tailings.



EXAMPLE: FOR $N_1 = 4$ AND DEPTH OF WATER 20 ft. (6.1m) ABOVE TAILINGS SURFACE, ZONE OF LIQUEFACTION EXTENDS 15 ft. (4.6m) BELOW TAILINGS SURFACE.

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Increasing the depth of water to 10 feet (3.0 m) would be sufficient to permit potential liquefaction to depths varying from about 3.5 feet (1.07 m) to 8 feet (2.44 m) below the surface of the tailings. The zone of liquefaction would extend to a maximum depth of 15 feet (4.57 m) below the surface of the tailings where the water depth is 20 feet (6.1 m).

The water depths assumed in the liquefaction analysis are probably somewhat greater than those which will occur in operating practice and free ponded water is anticipated to accumulate over only 20 percent of the surface area of the tailings ponds. Therefore, the risk of the development of extensive liquefaction zones is minimal and the effects of seismic excitation will be confined to shallow depths where confining pressures are small, the relative density is low and the depth of water exceeds about 5 feet (1.5 m). Even in the event that liquefaction zones develop, the materials will be contained in the tailings ponds.

3.0 FREEBOARD AND ROCK SLOPE PROTECTION

3.1 Requirements

Freeboard is defined in proposed NR 182 as "the height of the crest of the dam above the adjacent liquid surface within the impoundment". The design freeboard is defined as "the minimum freeboard which would occur during the design flood". Proposed NR 182 also requires in Section NR 182.11(q) that "sufficient freeboard measured from the inside crest shall be provided so as to contain the 100-year, 24-hour rainfall event and to prevent overtopping by waves during this design storm, or a minimum of 5 feet of freeboard shall be provided".

The design storm precipitation (100-year, 24-hour) for the Crandon Project area is 5.1 inches (130 mm) (Ref. 20). Proposed NR 182 is not specific on the design wind velocity, so the 100-year, one-hour wind velocity of 90 miles per hour (145 km/h) was selected for design (Ref. 18).

Overtopping is prevented by providing sufficient freeboard so that waves do not run-up the embankment slopes and overtop the pond crests. Wave run-up is a function of the wave height, embankment slope, and slope roughness. Wave height is a function of the water depth, wind velocity, wind duration, fetch, and wind direction.

In addition to overtopping, significant wave erosion must not be permitted to develop along the embankment slopes in order to maintain the integrity of the underdrain system, liner, and embankment. Protection against erosion by waves is provided by a slope protection such as rock, concrete, or similar wave energy resistive materials.

The type and amount of slope protection required around any pond is dependent on the size of the waves that will break on the slope, the embankment slope, and the material on the surface of the slope. The design of slope protection is intimately tied with the estimation of allowable freeboard. Freeboard heights may often be reduced by minimizing the amount of wave run-up that will occur on the slopes due to wind generated waves. Minimizing freeboard would also minimize the amount of slope protection needed.

3.2 Wind and Wave Height

The 90 mile per hour (145 km/h) design wind velocity is the maximum velocity sustained for one hour for a 100 year return period. This was obtained from isopach plots of annual velocity extremes for 30 feet (9.1 m) above the ground surface (Ref. 18). This value was not adjusted to account for reduction in wind velocity for heights lower than 30 feet (9.1 m) because the ponds are such large open areas.

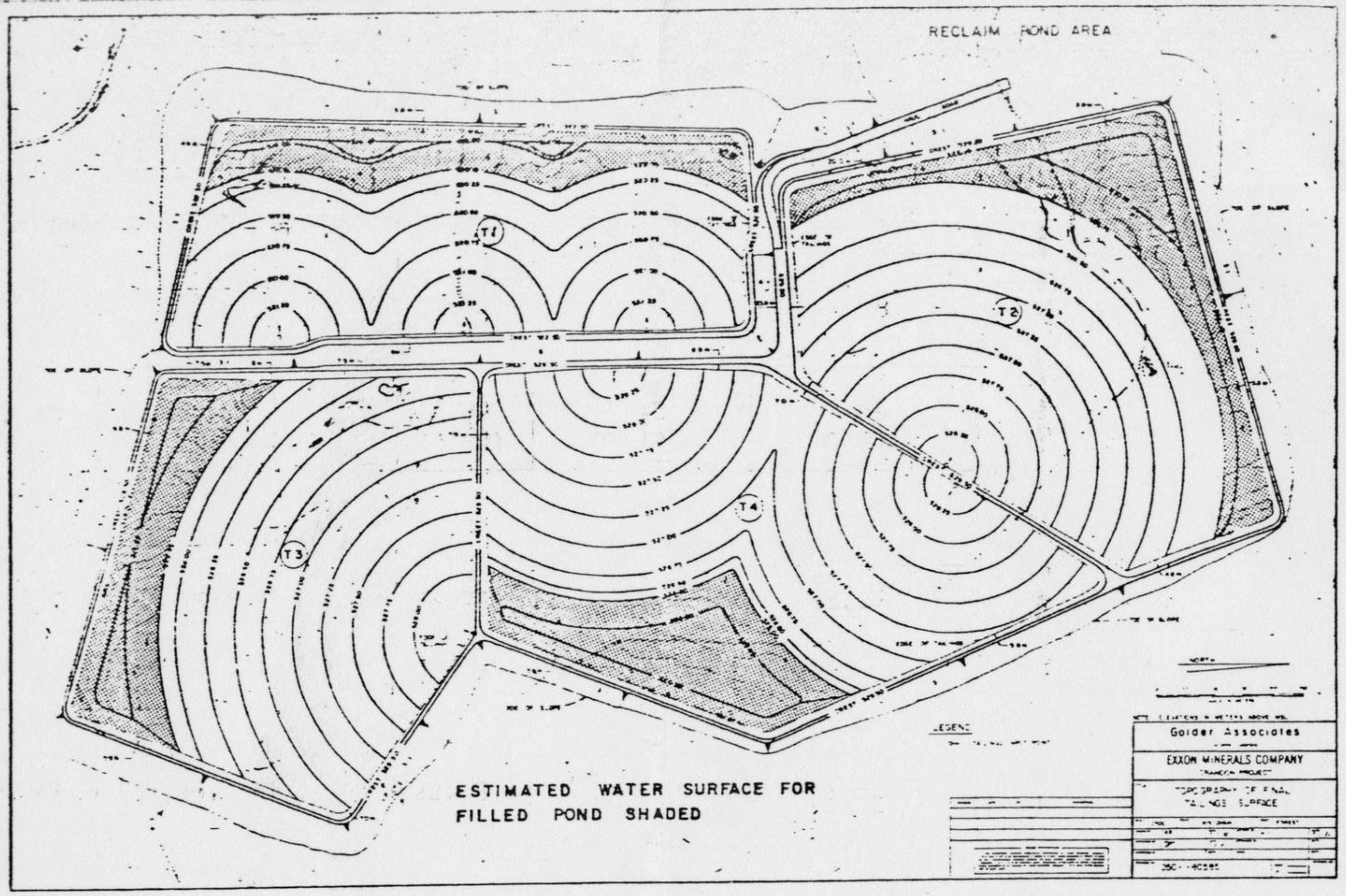
Wind direction is usually determined from wind rose data. Wind roses for the Rhinelander Airport and Central Wisconsin (Wausau) Airport indicate no prevailing wind direction could be assumed in the analyses. Therefore, the design wind was assumed to be in a direction which would produce maximum wave heights. This direction is assumed parallel to the maximum fetch.

The design fetch is the longest straight course length a wave can travel in the direction of the design wind. As previously noted, the maximum wind direction was taken in the same direction as the maximum fetch for each pond. The effective fetch is dependent on the width to length ratio of the water surface, resulting in a smaller fetch and smaller waves. The fetch and effective fetch for each pond are shown on Figure 3.1.

Analyses of wind generated waves are made in terms of a parameter called the "significant wave". The significant wave height corresponds to the average height of the highest third of the waves generated (Ref. 3). The design for this project is based on the maximum wave to minimize the potential for overtopping. The maximum wave has a height of about 1.87 times the significant wave height (Ref. 14). The relationship between the significant wave and other wave height characteristics is shown on Figure 3.1.

Significant wave heights were estimated for deep water conditions (water depth greater than one-half the wave length) (Ref. 14) which do not consider shoaling effects of shallow water and beaches. The shoaling effects of the 4.0 horizontal to 1.0 vertical side slopes are not significant (Ref. 14). The equations used to calculate the significant wave heights, wave period, and wave length are shown on Figure 3.1. The design wave height (maximum wave) is calculated from the significant wave height and the ratio of significant height to the maximum wave height is 1.87. The design wave heights are shown on Figure 3.1.

The design wave heights for the tailings ponds are based on the fetches from the approximate water surfaces shown on Figure 3.1. These areas represent about 20 percent of the pond surface areas when filled and are reasonable estimates for these facilities. The remaining surface of the ponds will be covered by a tailings beach sloping at an estimated 0.5 percent toward the ponded water area. As



RATIOS OF COMMONLY USED WAVE HEIGHT PARAMETERS TO SIGNIFICANT HEIGHT

Significant height (Ref. 3)	1.0
Average height (Ref. 3)	.6
Average height of highest 10% (Ref. 3)	1.3
Average height of highest 1% (Ref. 3)	1.7
Height of maximum wave (Ref. 14)	1.87

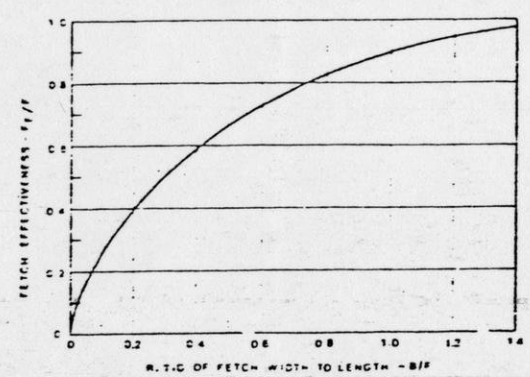


FIG. 14. Fetch width-length ratio vs. fetch effectiveness for rectangular fetches (after Saville 1954). Notes: for regular shaped reservoirs only; B = average reservoir width; F = straight line fetch; FF = effective fetch = fetch effectiveness × F. (FROM REF. 13)

POND	FETCH WIDTH B mi./(km)	FETCH LENGTH F mi./(km)	FETCH EFFECTIVE FF mi./(km)	WAVE PERIOD T sec.	WAVE LENGTH L ft./(m)	SIGNIFICANT WAVE HEIGHT Zw ft./(m)	DESIGN WAVE HEIGHT H ft./(m)
Tailings Pond 1	0.05 (0.08)	0.55 (0.88)	0.14 (0.22)	1.88	18.08 (5.51)	1.59 (0.48)	2.98 (0.91)
Tailings Pond 2	0.07 (0.11)	0.45 (0.72)	0.11 (0.18)	1.76	15.80 (4.82)	1.42 (0.43)	2.66 (0.81)
Tailings Pond 3	0.09 (0.14)	0.36 (0.59)	0.16 (0.26)	1.95	19.49 (5.94)	1.69 (0.52)	3.17 (0.97)
Tailings Pond 4	0.08 (0.13)	0.37 (0.59)	0.15 (0.24)	1.92	18.79 (5.73)	1.64 (0.50)	3.07 (0.94)
Reclaim Pond 1	0.17 (0.27)	0.26 (0.41)	0.19 (0.31)	2.05	21.46 (6.54)	1.83 (0.56)	3.43 (1.05)
Reclaim Pond 2	0.16 (0.25)	0.28 (0.45)	0.20 (0.31)	2.08	22.08 (6.73)	1.88 (0.57)	3.52 (1.07)
Tailings Pond 2 (filled with water)	0.42 (0.70)	0.43 (0.68)	0.39 (0.62)	2.50	32.09 (9.78)	2.58 (0.78)	4.82 (1.47)

Above calculations based on wind velocity (V) of 90 miles per hour (145km/h) using the following equations from Reference 12

English Units

$Zw = 0.034\ V^{1.06}\ FF^{0.47}$

$T = 0.45\ V^{0.44}\ FF^{0.28}$

$L = 5.12\ T^2$

Metric Units

$Zw = 0.005\ V^{1.06}\ FF^{0.47}$

$T = 0.32\ V^{0.44}\ FF^{0.28}$

$L = 1.56\ T^2$

Where:

Zw = Significant wave height, ft. (m)
T = Wave period, seconds
L = Wave length, ft. (m)
FF = Effective fetch, mi. (km)
V = Wind velocity, mph (km/h)

Note: H = 1.87 Zw from table shown at left.

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an upper bound to wave height, an additional calculation was made assuming that tailings pond 2 is completely covered by water. This example was selected because of its long fetch. The reclaim ponds will be filled with water and calculations were based on fetch measured across the ponds between the crests.

The wave heights calculated and reported on Figure 3.1 are generally in the 2 to 5 foot (0.61 to 1.5 m) range. These wave heights are used in analysis of wave run-up and slope protection which is discussed in the following subsection of this report.

3.3 Wave Run-up and Slope Protection

The amount of wave run-up will vary with different cover materials based upon energy dissipation. Steeper slopes, up to about 1.5 horizontal to 1.0 vertical, generate more run-up than do the flatter slopes. Smoother slopes generate considerably more wave run-up than rough open work slopes. The wave run-up can be reduced by using slope protection that absorbs the energy of the waves. The most efficient energy dissipators are rough structures with a high porosity.

The slopes around the reclaim ponds and the water storage sides of the tailings ponds will require some type of protection against erosion of wind generated waves. Some of the more common types could be concrete facing, soil stabilization, or rock. Concrete facing or soil stabilization would result in smooth slopes which would provide erosion protection but would increase the freeboard height because they allow higher wave run-up. This would necessitate higher embankments and larger areas of slope protection than needed for rough slope protection material such as rock.

Waste rock produced during mining the Crandon Project orebody will be used as underground backfill. However, during early stages of mine development a large volume of this rock, about 1.4 million cubic yards ($1.1 x 10^6$ m^3), will have to be brought to the surface because there will be no storage space in the mine. After the mill is in full operation a small amount of rock, about 13,000 cubic yards (10,000 m^3) per year, will be brought to the surface on a nearly continuous basis for permanent disposal. The rock which comes from the mine prior to mill operation is termed pre-production waste rock and will be 24 inches (610 mm) and smaller in size. The rock available after mill operation begins, production waste rock, will be 6 inches (152 mm) and smaller.

The wave run-up, above mean water surface, has been estimated from the charts shown on Figure 3.2 for the wave characteristics previously reported on Figure 3.1. These charts provide curves for smooth and rubble slopes. The wave run-up ratio versus wave steepness and embankment slope chart, as referenced, did not include curves for smooth slopes with $Z_w/L = 0.09$ nor curves for rubble mounds with $Z_w/L = 0.07$ to 0.09. To obtain this range the wave run-up ratio versus significant wave height to wave length ratio for 4.0 horizontal to 1.0 vertical slopes curves were extrapolated from the referenced chart. Though this is not an exact procedure, it is considered sufficient for the wave characteristics estimated for the proposed waste disposal ponds.

The estimated wave run-up for the reclaim and tailings ponds are also shown on Figure 3.2. It should be noted that the wave run-up factors from the chart on Figure 3.2

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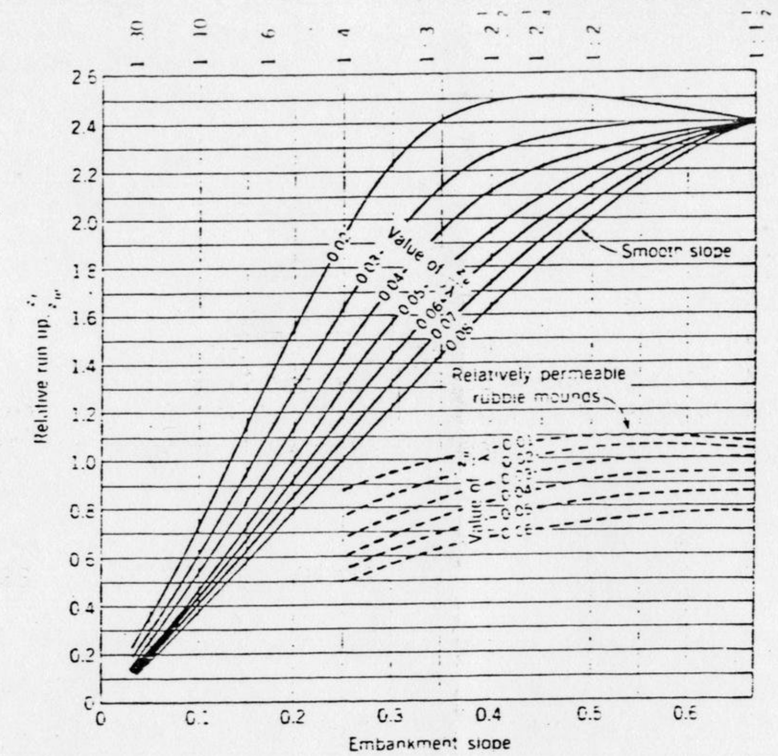
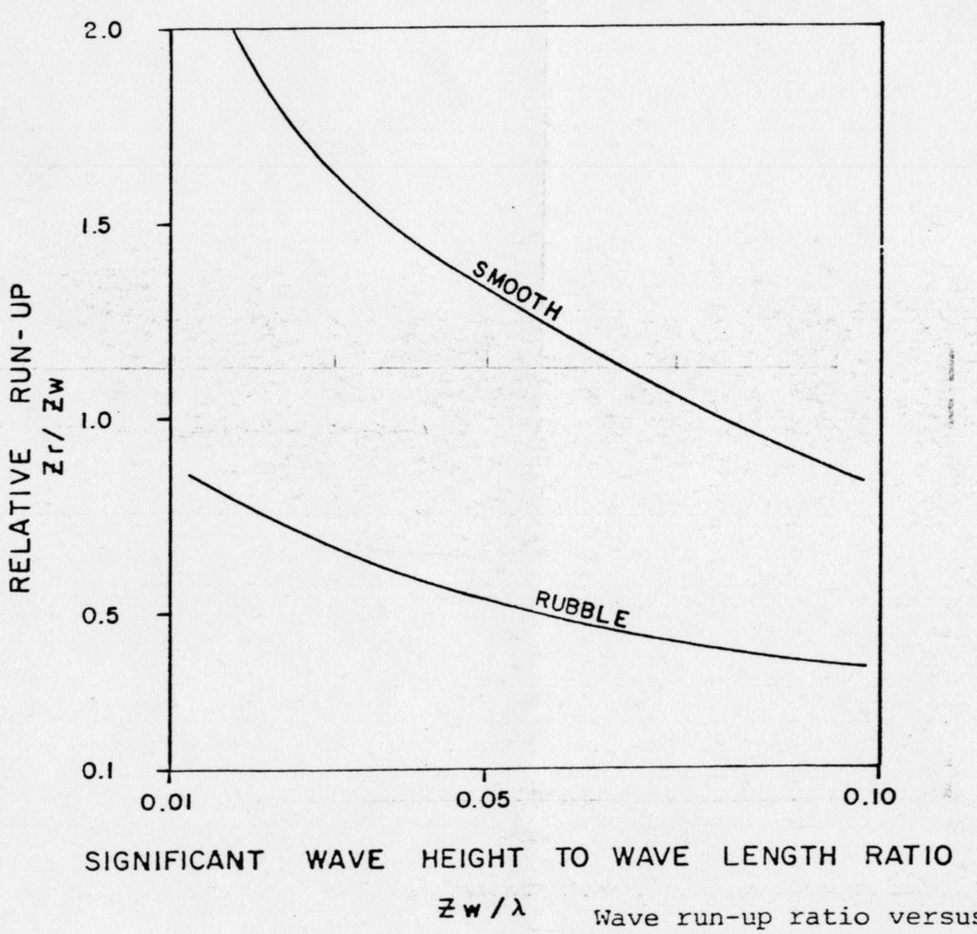


Wave run-up ratios vs. wave steepness and embankment slopes. (From Saville, McClendon and Cochran) (FROM REF. 15)



Wave run-up ratio versus significant wave height to wave length ratio for 4:1 slope extrapolated from above chart from Ref. 15

ESTIMATED WAVE RUN-UP HEIGHT FOR DESIGN

POND	DESIGN WAVE HEIGHT H ft./(m)	RATIO Zw/L	RUN-UP FACTORS, Zr/Zw: From Chart at Left			DESIGN RUN-UP ft./(m)
			Smooth	Rubble	Estimated for Design	
T1 - Tailings Pond 1	2.98 (0.91)	0.088	0.91	0.38	0.5	1.49 (0.45)
T2 - Tailings Pond 2	2.66 (0.81)	0.090	0.90	0.38	0.5	1.33 (0.41)
T3 - Tailings Pond 3	3.17 (0.98)	0.087	0.92	0.38	0.5	1.59 (0.49)
T4 - Tailings Pond 4	3.07 (0.94)	0.087	0.92	0.38	0.5	1.53 (0.47)
R1 - Reclaim Pond 1	3.43 (1.05)	0.085	0.94	0.39	0.5	1.71 (0.52)
R2 - Reclaim Pond 2	3.52 (1.07)	0.085	0.94	0.39	0.5	1.76 (0.53)
T2 - Tailings Pond 2 (filled with water)	4.82 (1.47)	0.080	0.99	0.41	0.5	2.41 (0.74)

Notes:
1. Design run-up = Hx Zr/Zw (H estimated for design)
2. L = Wave length from Figure 3.1 (L = λ in charts at left.)
3. Zw = Significant wave height from Figure 3.1

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are given for smooth or rubble slopes. The waste rock covering proposed is judged to have a roughness slightly lower than rubble slopes. The range of run-up factors was from about 0.38 for rubble slopes, to just below 1.0 for smooth slopes. A value of 0.5 was selected as being reasonable for waste rock covered slopes. Also, the wave run-up estimates on Figure 3.2 were made using the maximum wave height. This is a more conservative approach than using the significant wave height, which is customary.

As can be seen from the design wave run-up heights, they are all less than 1.8 feet (0.52 m), except for the upper bound condition which assumes tailings pond T2 to be completely covered with water. For this latter condition, the estimated wave run-up height is 2.41 feet (0.74 m). Thus, the design freeboard height must be at least 1.8 feet (0.55 m) above the water level resulting from the design storm.

3.4 Precipitation Storage Capacity

The pond crests are sloped toward the tailings ponds so that precipitation is collected over an area defined by the outside perimeter of the pond crests. Normal precipitation will be stored and discharged from the tailings ponds to the reclaim ponds with the decanted pond water. Normal precipitation will be accommodated within the normal operating levels of the tailings and reclaim ponds. The design storm water levels are added to the normal operating water levels to establish the level from which the "design freeboard", which must equal the estimated wave run-up, is measured. For purposes of discussion, the single term "freeboard" is used as the distance from the pond crest to the normal operating water level in the reclaim ponds and the normal operating water level for a filled tailings pond. Wave run-up and the design storm water must be accommodated within this freeboard.

The statistical rainfall data (Ref. 20) indicates the following frequencies and intensities:

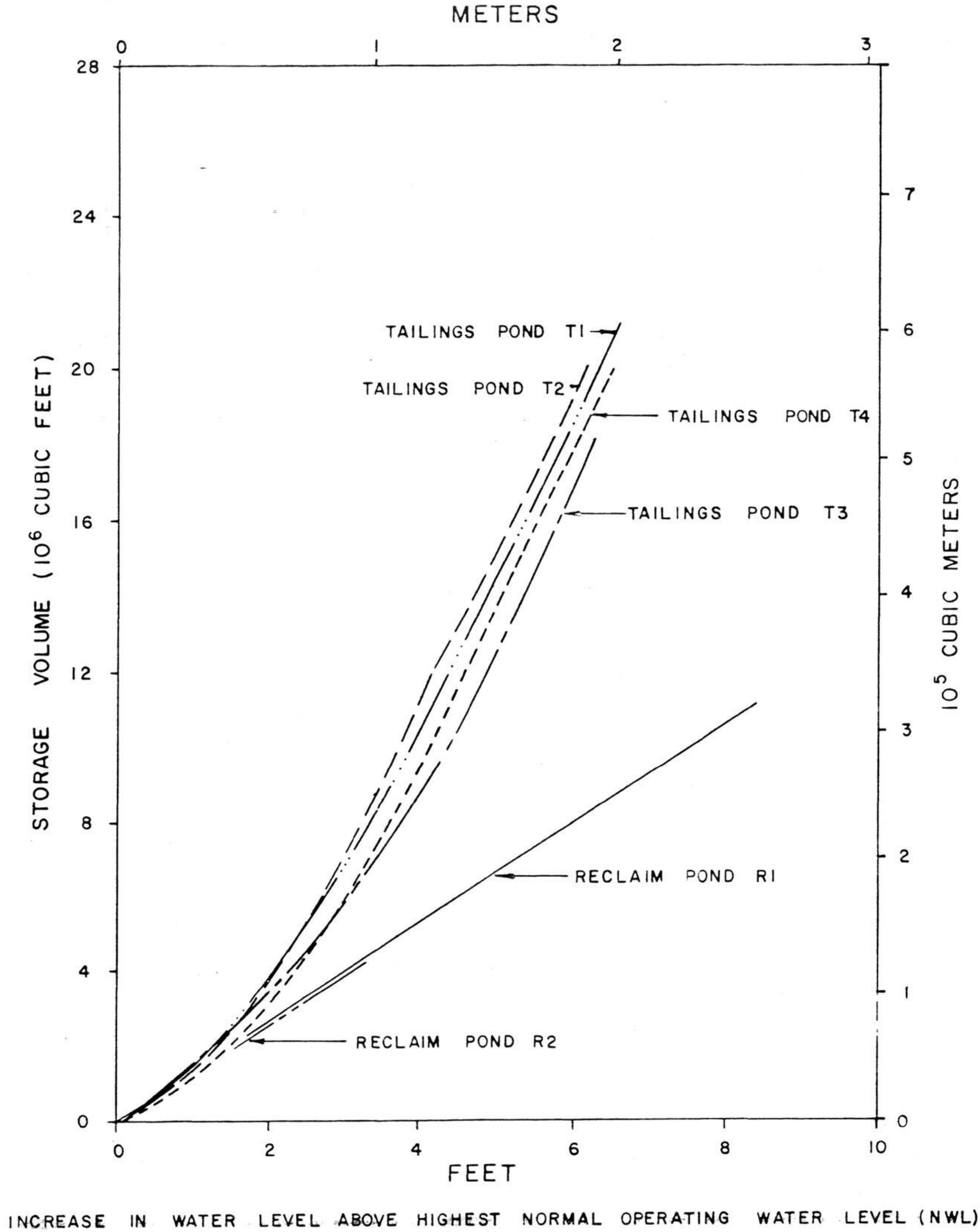
6 hour - 100 year = 3.8 inches (97 mm)
24 hour - 100 year = 5.1 inches (130 mm)
6 hour - Probable maximum precipitation (PMP) = 23.0 inches (584 mm)

Proposed NR 182 dictates the 5.1 inch (130 mm), 24 hour - 100 year event as the design storm. However, the Mine Safety and Health Administration (MSHA) requires that the PMP event of 23.0 inches (584 mm) be decanted from the largest tailings pond within ten days. For the Crandon Project this storm water will have to be decanted to the reclaim ponds, so they have been sized to retain the PMP plus the PMP water from tailings pond T4, the largest tailings pond, within their freeboard heights. For this case, the freeboard need not also contain the wave run-up height.

Additional storage volume has been provided in reclaim pond R1 for mine water in the event of a two week shut-down of the excess water treatment system. This volume is 6.74×10^6 cubic feet (1.91×10^5 m^3) which is equal to 2500 gallons per minute (0.158 m^3/s) for 14 days. This volume is equivalent to 8.45 feet (2.58 m) of storage height in reclaim pond R1. This storage height is provided within the freeboard, but not in addition to storm water or wave run-up height.

The tailings will slope downward from the input side of the ponds to the ponded water side of the ponds. Thus, the normal operating water levels in these ponds will be several feet lower than the highest point of the tailings. However, freeboard will also have to be provided above the tailings level to retain splashed water. A 3 foot (0.91 m) freeboard above the maximum tailings level has been provided in all tailings ponds. This additional height could be used for additional tailings storage if it is found during operation to be more freeboard than needed. Alternatively, the embankments could be reduced in height. With this amount of freeboard on the tailings input side, the available freeboard on the ponded water side has been estimated and compared against the height of wave run-up and height of water from the design storms. The height of water from the design storms accounts for a ponded water surface which will spread up the tailings beach. A series of curves showing the storage volume versus height of water above normal operating levels is shown on Figure 3.3. The water heights for the various storms and the freeboard provided are presented on Table 3.1. As can be seen from these figures, there is more than enough freeboard available in the tailings ponds to accomodate the the 24 hour - 100 year event plus wave run-up or the PMP event. There is even sufficient freeboard to accommodate these events in tailings pond T2 if it were covered by water up to the highest tailings storage level.

The various heights of the water from the design storms and the estimated wave run-up are presented in Table 3.1. Combinations of these events are also shown with the required freeboard above normal operating levels and the freeboard which has been provided in the preliminary engineering design. In summary, a freeboard of 3 feet (0.91 m) has been provided in all of the tailings ponds above the maximum tailings level and above normal operating water level in reclaim pond R2. The freeboard provided in reclaim pond R1 is 8.5 feet (2.59 m) to accomodate either



JOB NO. 786085	SCALE AS SHOWN	STORAGE VOLUME VS INCREASE IN WATER LEVEL
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TABLE 3.1

FREEBOARD AND STORM WATER LEVEL HEIGHTS

Pond	Freeboard Above Tailings ft./(m)	Freeboard Above NWL ft./(m)	Increase in Water Level Above NWL For Various Factors				
			PMP ft./(m)	Mine Water ft./(m)	100 yr.-24 hr. Design Storm ft./(m)	Wave Run-up ft./(m)	100 yr.-24 hour plus Run-up ft./(m)
T1-Tailings Pond 1	3.0 (0.91)	7.7 (2.35)	3.65 (1.11)	--	1.10 (0.34)	1.49 (0.45)	2.59 (0.79)
T2-Tailings Pond 2	3.0 (0.91)	11.0 (3.35)	4.00 (1.22)	--	1.25 (0.38)	1.33 (0.41)	2.58 (0.79)
T3-Tailings Pond 3	3.0 (0.91)	12.6 (3.65)	4.25 (1.30)	--	1.25 (0.38)	1.59 (0.49)	2.84 (0.87)
T4-Tailings Pond 4	3.0 (0.91)	9.5 (2.90)	4.40 (1.34)	--	1.80 (0.55)	1.53 (0.47)	3.33 (1.01)
R1-Reclaim Pond 1	--	8.5 (2.59)	8.45 (2.58)	5.10 (1.55)	0.50 (0.15)	1.71 (0.52)	2.21 (0.67)
R2-Reclaim Pond 2	--	3.0 (0.91)	2.95 (0.90)	--	0.50 (0.15)	1.76 (0.53)	2.26 (0.69)
T2-Tailings Pond 2 (filled with water)	3.0 (0.91)	3.0 (0.91)	1.92 (0.58)	--	0.43 (0.13)	2.41 (0.74)	2.84 (0.87)

Noted:
1. NWL - Normal operating water level (assumes full tailings ponds)
2. Tailings slope at 0.5 percent from input side to ponded water side.
3. PMP - Probable maximum precipitation is 23 inches (584 mm) in 6 hours.
4. 100 yr.-24 hr. design storm is 5.1 inches (130 mm).
5. Wave run-up from Figure 3.2.
6. PMP storage in ponds R1 and R2 includes PMP volume from T4 (87% in R1 and 13% in R2).

the two weeks of mine water in the event of a shut-down of the excess water treatment plant, or the PMP on pond R1 plus approximately 87% of the PMP routed off of tailings pond T4. The remaining 13% of the PMP volume from tailings pond T4 can be accommodated in Reclaim Pond R2 within its 3 foot (0.91 m) freeboard. For each pond the freeboard provided is sufficient to accommodate the various volumes of water and/or wave run-up from the design storms.

4.0 GLACIAL TILL FOR LINERS

The glacial till at the Crandon site is a heterogeneous mixture of silt, sand, gravel, cobbles and boulders having an in situ mass permeability of approximately $1x10^{-6}$ m/s ($3.3x10^{-6}$ ft./sec.) (Ref. 9). In order to keep leakage from the tailings ponds to an acceptable level it is necessary to install a relatively impermeable liner system.

The availability of glacial till from pond excavations makes it attractive for use as a liner provided the liner can be constructed to meet the permeability requirements associated with an acceptable level of leakage. Laboratory permeability tests on recompacted glacial till having a maximum particle size of 0.75 inches (19 mm) show that at Standard Proctor maximum dry density the permeability is on the order of $1x10^{-8}$ m/s ($3.3x10^{-8}$ ft./sec.) (Ref. 6). The laboratory testing program was extended to examine the influence of various quantities of bentonite additive on the permeability of compacted samples (Ref. 7). This work established that the permeability decreases with increasing bentonite content, but at a reducing rate, and that for bentonite contents in excess of 4 percent by weight the permeability approaches a value of approximately $5x10^{-10}$ m/s ($1.6x10^{-11}$ ft./sec.).

For preliminary design and conservatism in estimating liner costs, the proposed liner comprises a 6 inch (150 mm) thick layer of till mixed with about 8 percent, by weight, bentonite. It is envisaged that construction of a satisfactory liner would have to include processing of the natural material by crushing or screening to a maximum size of 0.75 inches (19 mm).

Laboratory grain size analyses on material finer than 3 inches (75 mm) indicate that up to 25 percent of the particles are larger than 0.75 inches (19 mm) (Ref. 6). The proportion of cobbles and boulders in the glacial till is subject to important spatial variations although examination of the till as exposed in the sides of excavated test pits suggest that the quantity of material coarser than 3 inches (75 mm) is typically about 5 percent. The maximum cobble size is estimated at 6 to 8 inches (150 to 200 mm), but occasional larger cobbles and boulders will be encountered.

Glacial till, moraines and boulder clays have been used in a number of major earth and rockfill dams to form low permeability cores and/or upstream blankets. Although seepage requirements for these structures are typically much less severe than those for the proposed tailings ponds, the available experience suggests that it is practical to place and compact such materials in layers up to 12 inches (300 mm) thick provided the maximum particle size does not exceed about one-half to two-thirds of the compacted thickness.

The effect of maximum particle size on the mass permeability of till/bentonite mixtures has not been established. The ability to construct the liner under rigorous quality control standards and achieve the desired permeability, uniformity of mix, and layer thickness is considered to be a primary factor in establishing the maximum aggregate size. A conservative maximum aggregate size of 0.75 inches (19 mm) is considered reasonable for preliminary engineering design and cost estimates. This size requirement can be met by crushing or screening the native glacial till.

During the final design process testing could be accomplished to re-evaluate the proposed maximum aggregate size and bentonite content to achieve the most economical combination which will provide the required quality low permeability liner.

5.0 FROST ACTION

5.1 Frost Susceptibility

In seasonal frost areas, ground freezing occurs during winters followed by thaw without development of permafrost. The problems involved are vertical or horizontal expansion of soil during freezing, and decrease of soil shear strength and rigidity during thawing. Soil heaving is caused by migration of supercooled water (which forms in void spaces below a critical size) within the soil to ice crystals which form in larger void spaces. This water freezes on contact with the crystals and the crystals grow. Crystal growth leads to formation of an ice lens. The ice lens grows in thickness in the direction of heat transfer and at the same time laterally, until the water source is cut off or the temperature in the soil is above the normal freezing point. Heaving occurs in the direction of least resistance.

The frost susceptibility of soil primarily depends on its gradation characteristics and density. Materials with small void spaces are more frost susceptible because of their greater capillary potential. The more pervious of these soils are the most susceptible because of their ability to transmit appreciable water through the void spaces. Thus, silts, silty sands, and clays with plasticity index of less than 12 percent have the highest frost heave potential. Clays of medium to high plasticity are susceptible to the formation of ice lenses, but significant heave only develops at very long, sustained freezing temperatures when there is enough time for water migration. Coarse soils with large pore sizes, such as well graded or poorly graded sands and gravels, have very low frost susceptibility. Increasing the density of the material may increase or decrease the frost susceptibility depending on the amount of fines it contains.

The most common method for the determination of frost susceptibility is based on the fines content of the soil. Table 5.1 gives the limiting values of particle sizes listed according to different sources. In the United States, the Casagrande criterion is very often followed in the design of pavements. According to the Casagrande criterion, the soils susceptible to frost action are:

(1) well graded soils containing more than 3 percent finer than 0.02 mm, and

(2) poorly graded soils containing more than 10 percent finer than 0.02 mm.

It should be noted that many of Casagrande's field observations were made in New Hampshire, where a frost penetration of about 3 feet (0.91 m) is probably representative of average conditions. In colder regions, where frost penetration of 5 to 7 feet (1.5 to 2.1 m) is common, frost heave theory predicts that a soil could have a higher percentage of fines than suggested by the Casagrande criterion and be equally frost susceptible. This is because the resulting larger heaving pressure would be counteracted by the greater overburden pressure above the frost front.

The U.S. Army Corps of Engineers has developed a classification divided into four groups, F1 through F4, for use as general guidance in estimating the relative frost susceptibility of soils. This classification system is shown as Table 5.2. The groups are listed in approximate order of increasing susceptibility to frost heave and/or weakening as a result of frost melting. There is some overlapping of frost susceptibility between groups.

TABLE 5.1

FROST SUSCEPTIBILITY BASED ON GRAIN SIZE

		Percentage by Grain Size (mm)					
Author	Fine Content	< 0.125	< 0.1	< 0.062	< 0.05	< 0.02	< 0.002
Beskow	Uniform	22-35		15-25			
	Nonuniform			33-50			
Kögler-Scheidig[a]	Uniform					3	
	Nonuniform					10	
Morton[a]					10		
Casagrande[c]	Uniform					3	
	Nonuniform					10	
Schaible	Frost susceptible		20			10	1
	Highly frost susceptible		40			20	6

[a] If free water is available, the frost susceptivity is classed by means of the permeability $k = 1.10^{6}$ to 1.10^{7} m/sec, highly frost susceptible, $k = 1.10^{7}$ to 1.10^{8} m/sec, frost susceptible, and $k < 1.10^{8}$ m/sec, not frost susceptible.

[b] Valid only for soils with particle diameter between 0.001 and 2.0 mm.

[c] According to Ducker not applicable, for vulcanic soils and for very uniform soils.

Table from Jessberger, H.L., "Frost Susceptibility Criteria", Highway Research Record No. 429, 1973, pp. 40-46.

TABLE 5.2

Frost Design Soil Classification

Frost Group	Kind of Soil	Percentage Finer than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	(a) Gravelly soils	Over 20	GM, GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC
	(c) Clays, PI >12	-	CL, CH
F4	(a) All silts	-	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	-	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	-	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

Groups are listed in approximate order of increasing frost susceptibility. Group F4 soils are of especially high frost susceptibility.

Table from U.S. Army Corps of Engineers, "Pavement Design For Frost Conditions", TM 5-818-2, Headquarters, Department of the Army, July 1965, p. 7.

In addition to the factors of grain size and density affecting the depth of frost penetration, other factors affect the depth of frost penetration and resulting heave. These are as follows:

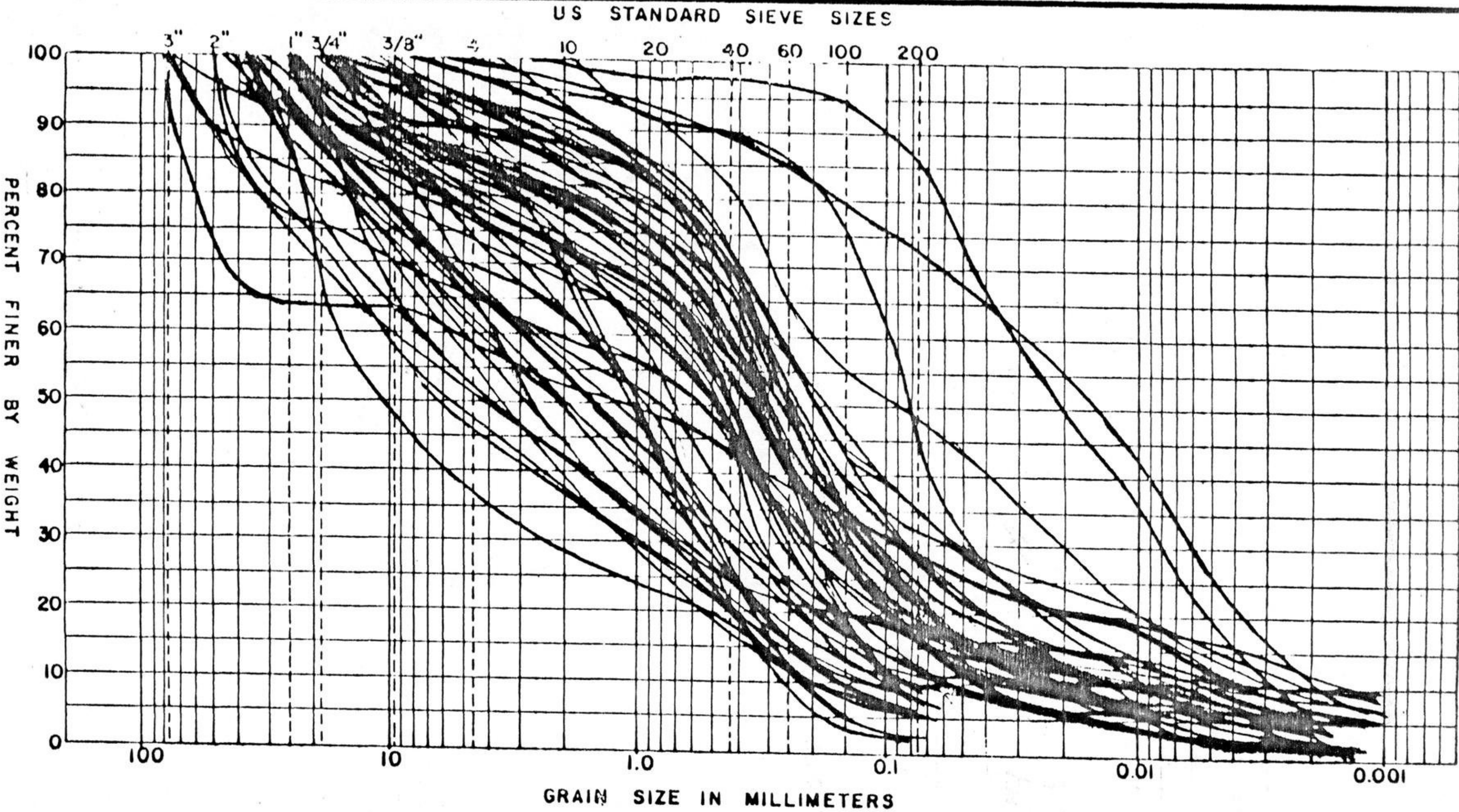
1. Freezing index (number of days times the average daily temperature in degrees F, minus 32°F)
2. Rate of change of temperature
3. Availability of water source
4. Initial degree of saturation

Without describing the specific effects of each of these factors, it is reasonable to assume that they will occur in combinations and ranges sufficient to cause frost penetration and resulting frost heave in frost susceptible soils in the Crandon Project area.

The measured depth of frost penetration in the north-central Wisconsin area has been reported by the Wisconsin Agriculture Reporting Service (Ref. 21) to be less than 2 feet (0.61 m). The Water Atlas of the United States (Ref. 5) suggests that the frost penetration in the Crandon Project area to be about 3 feet (0.91 m).

5.2 Frost Action Effects

The range of gradation of the glacial till samples tested from the Site 41 area is shown on Figure 5.1. In general, the glacial till soils fall into the Unified Soil Classification System group designations SM or SP-SM. As can be seen from Figure 5.1, the percentage of materials finer than 0.02mm ranges from near zero to 55 percent. However, most of the samples tested show less than 15 percent finer than 0.02mm. Therefore, on the average, the till soils should be classed as slightly frost susceptible, group F2 in Table 5.2, but some may exhibit more frost sus-



COBBLES	GRAVEL		SAND			FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO.	ELEV. OR DEPTH	W_n	W_L	W_p	I_p	DESCRIPTION OR CLASSIFICATION
-	-	-	-	-	-	Samples of Till from Site 41 (See Table C-2 in Appendix C of Reference 6 for listing of samples)

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GRAIN SIZE DISTRIBUTION

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FIGURE 5.1

ceptibility, and be classed as groups F3 and possibly F4. Thus, some frost heave is expected in the winter and some loss of strength is expected during Spring thaw. Such frost action is of little to no consequence to the integrity of the tailings and reclaim pond embankments.

The bentonite/till liner and top seal will have a sufficiently low permeability, on the order of 5×10^{-10} m/s (1.6×10^{-9} ft./sec.) that it should not be appreciably frost susceptible. In addition, this liner is covered by the 3 foot (0.91 m) thick underdrain system on the tailings pond bottom and the underdrain plus rock slope protection on most of the pond sides. The till/bentonite liner is covered by 2 feet (0.61 m) of granular cover on the reclaim pond bottom plus an additional 3 feet (0.91 m) of rock slope protection on the sides of the reclaim pond slopes. The top seal will be covered by 3 feet (0.91 m) of soil cover. These cover depths, the low susceptibility of the till/bentonite mixture to frost heave, and the anticipated operating water and tailings levels in the ponds indicate there will be almost no potential for frost heave damage to the till/bentonite liner or top seal.

The underdrain drain layer will have very little susceptibility to frost heave because it will not have material finer than 0.02mm. The filter layer above the drain will have the same potential as the glacial till since it is glacial till with the large cobbles removed. However, frost heave or loss of strength during thaw should not affect the integrity or filtering action of this layer.

The synthetic liner materials proposed (Hypalon or high density polyethelene) should not be affected by freeze-thaw cycles. In fact, the synthetic liner should retard the effects of freezing because of its very low permeability and slight insulating characteristics. Similarly, rock slope protection will not be susceptible to frost heave and will provide protection against frost heave by its weight and protection against frost penetration by providing a thick cover over the underlying materials.

6.0 EARTHWORK BULKING FACTOR

Earthwork bulking (swell or shrink) is the ratio of the compacted dry density to the in situ dry density for soils. It is estimated by comparing in situ density test data to laboratory compaction test data.

In situ dry density test results of the glacial till soil in the Crandon Project area ranged from 110 to 138 pounds per cubic foot (1762 to 2211 kg/m^3) (Ref. 6). Maximum dry densities for the same samples ranged from 124 to 137 pounds per cubic foot (1986 to 2195 kg/m^3) based on the Standard Proctor (ASTM D-698) test method (Ref. 6). The embankments for this project will be constructed of glacial till soils compacted to a minimum of 95 percent of the maximum dry density determined by the Standard Proctor compaction test. The average in-place density is expected to be higher than the minimum required; probably about 98 percent of maximum. Therefore, the average dry density of the compacted till is estimated to range from 121 to 132 pounds per cubic foot (1922 to 2115 kg/m^3).

As can be seen from the above figures, the in situ dry density test results bracket the estimated compacted dry density of the till. It must be noted that the in situ test results are from tests performed in test pits less than 20 feet (6.1 m) deep, but the tailings ponds will be excavated 50 to 65 feet (15 to 20 m) below the surface. Standard penetration test results from the borings in the Crandon Project area suggest that the till is somewhat denser below the upper 20 to 30 feet (6.1 to 9.1 m). Thus, the average in situ dry density may be slightly higher than that estimated from the test pit data. It is most likely that there will be some earthwork swell (increase in available volume) from excavation to fill. However, at this point in the preliminary design phase a bulking factor of 1.0 (no shrink or swell) has been used in earthwork balance calculations.

7.0 TILL PROCESSING

Glacial till will essentially be the only native soil used for construction of the waste disposal facilities. Although coarse grained stratified drift soil would be better suited for some aspects of construction, it does not occur close enough to the ground surface at Site 41 to be made available from pond excavations. The waste facility ponds have been designed so that they can be constructed almost totally from glacial till excavated from within the ponds and waste rock from the mine.

Gradation requirements for some of the materials to be employed in construction do not match the gradation of the native till. Therefore, processing, which could include crushing, screening, and/or washing of the till, will be necessary to produce the required gradations. The following is a list of materials to be processed from the glacial till. The word "waste" as used in this list and as used in discussing processing means those portions of the material which cannot be used or are left after processing of the specific material. These various waste soils will be used for other purposes as described. The size ranges for the materials listed below are provided for preliminary design and cost estimating. It is anticipated that these size ranges will be re-evaluated during final design.

1. Liner - Glacial till crushed to minus 19.1 mm (0.75 in.). No waste. To be mixed with bentonite.

2. Top Seal - Glacial till crushed to minus 19.1 mm (0.75 in.). No waste. To be mixed with bentonite.

3. Filter - Glacial till crushed or screened to minus 152 mm (6 in.). No waste.

4. Drain - Glacial till crushed and/or screened and washed to the following: minus 51 mm (2 in.), plus 0.42 mm (#40 U.S. Std. sieve). Based on till gradation curves, 48% is drain and 52% wet waste. Wet waste can be used as mine backfill make-up.

5. Transition material for the reclaim pond. Glacial till crushed and/or screened and washed to the following: plus 4.76 mm (#4 U.S. Std. sieve) with D_{85} 31.7 mm (1.25 in.). Based on till gradation curves, 20% is transition, 80% is wet waste. Waste can be used as mine backfill make-up or, if it can be dried enough, can be used as the protective cushion above the synthetic liner.

6. Protective cover for the reclaim pond above synthetic liner. Glacial till crushed finer than 4.76 mm (#4 U.S. Std. sieve).

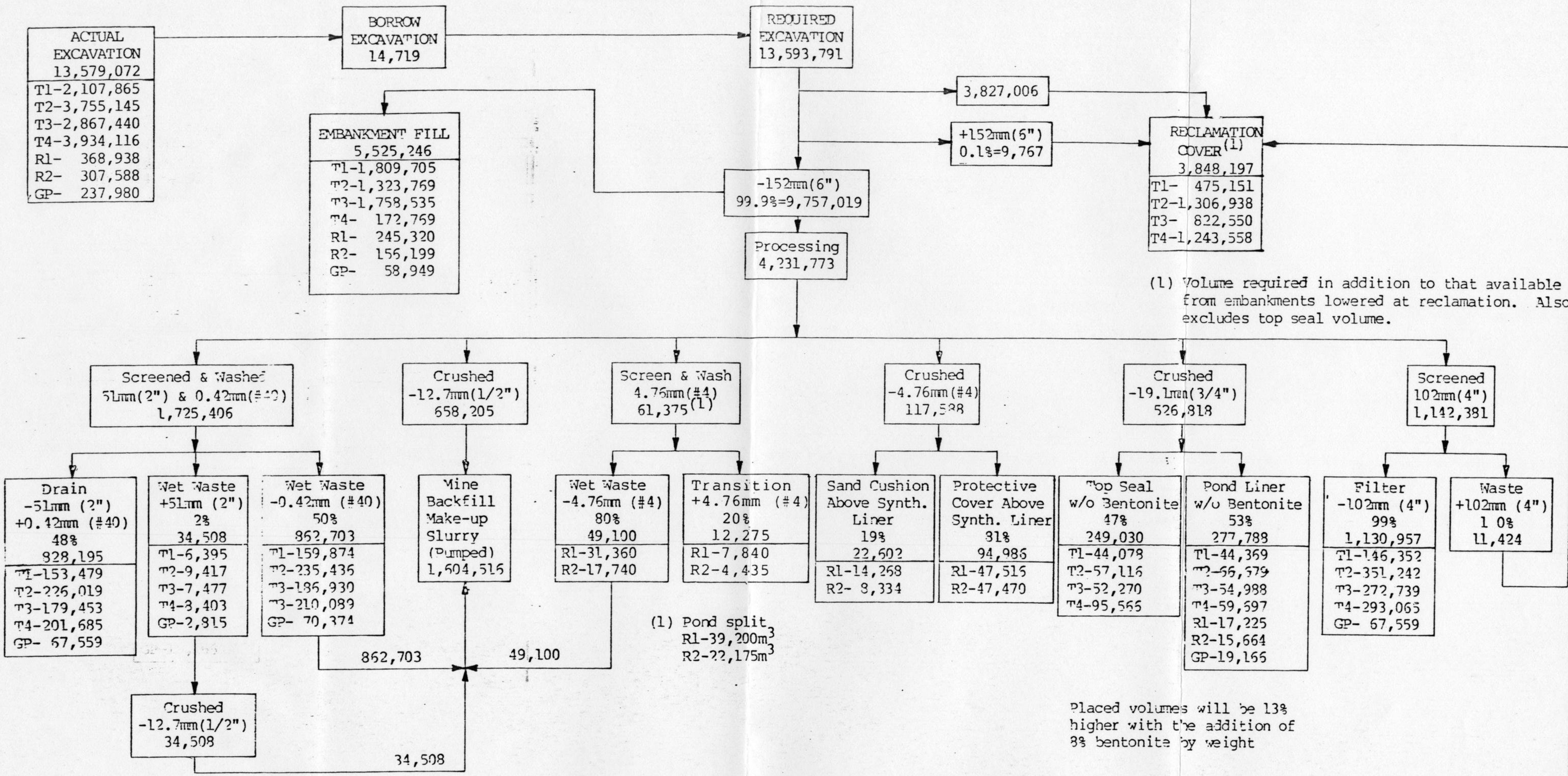
7. Sand cushion above synthetic liner for the reclaim pond. Glacial till crushed finer than 4.76 mm (#4 U.S. Std. sieve). (Assumes HDPE liner).

8. Sand cushion below synthetic liner - OFF SITE BORROW. Must be free of carbonate minerals. Same gradation as sand cushion above synthetic liner.

9. Mine backfill make-up - Glacial till crushed to minus 12.7 mm (0.5 in.). No waste. NOTE - Mine backfill can include all wet waste from processing of drain material and transition material.

In addition to the above processing requirements, it is suspected that the larger cobbles and boulders, larger than 6 inches (152 mm), will have to be scalped from the till prior to embankment construction. It is not possible with the existing data to accurately estimate the proportion of the till comprised by these large pieces. Therefore, a figure of 0.1 percent has been used as an allowance.

A flow chart of the excavation, embankment, fill, and processing of the glacial till is shown on Figure 7.1. The earthwork quantity estimates for system 41-114B are reflected in these figures. The latest estimate is within about 100,000 m^3 (131,000 cu. yds.) of balancing excavation and fill. Considering the level of detail to which the preliminary design has been made, this small imbalance (about 0.8 percent of the total excavation) is considered negligible. However, to make the figures show a balance, a 112,644 m^3 (147,324 cu. yds.) borrow has been shown.



Notes:
All quantities in cubic meters.
T1 to T4-Tailings Ponds 1 thru 4
R1 and R2-Reclaim Ponds 1 and 2
GP-Grading for pre-production waste rock

JOB NO. 786085	SCALE NONE	GLACIAL MATERIALS USE FLOW DIAGRAM
DRAWN CAB	DATE 8-16-82	
CHECKED	DWG. NO.	
Golder Associates		EXXON MINERALS COMPANY — FIGURE 7.1

8.0 EMBANKMENT TOE DRAINS

Toe drains at the downstream toe of an embankment slope are commonly required for earth dams and many tailings ponds. However, for the tailings and reclaim ponds for the Crandon Project, such toe drains are not necessary.

The till/bentonite liner and underdrain on the tailings pond embankment slopes and the till/bentonite liner overlain by a synthetic liner on the reclaim ponds preclude large volumes of seepage from entering the embankments. The maximum seepage rate from a tailings pond is estimated to be 4.0 gallons per minute (2.6×10^{-4} m^3/s) (see Section 13 of this report) which is far below the 1800 gallons per minute (0.11 m^3/s) per 100 acre (40 ha) pond estimated to cause mounding to the pond bottom and hence possible seepage at the embankment toe. Any seepage into the embankments will flow primarily downward by gravity until it reaches the groundwater level.

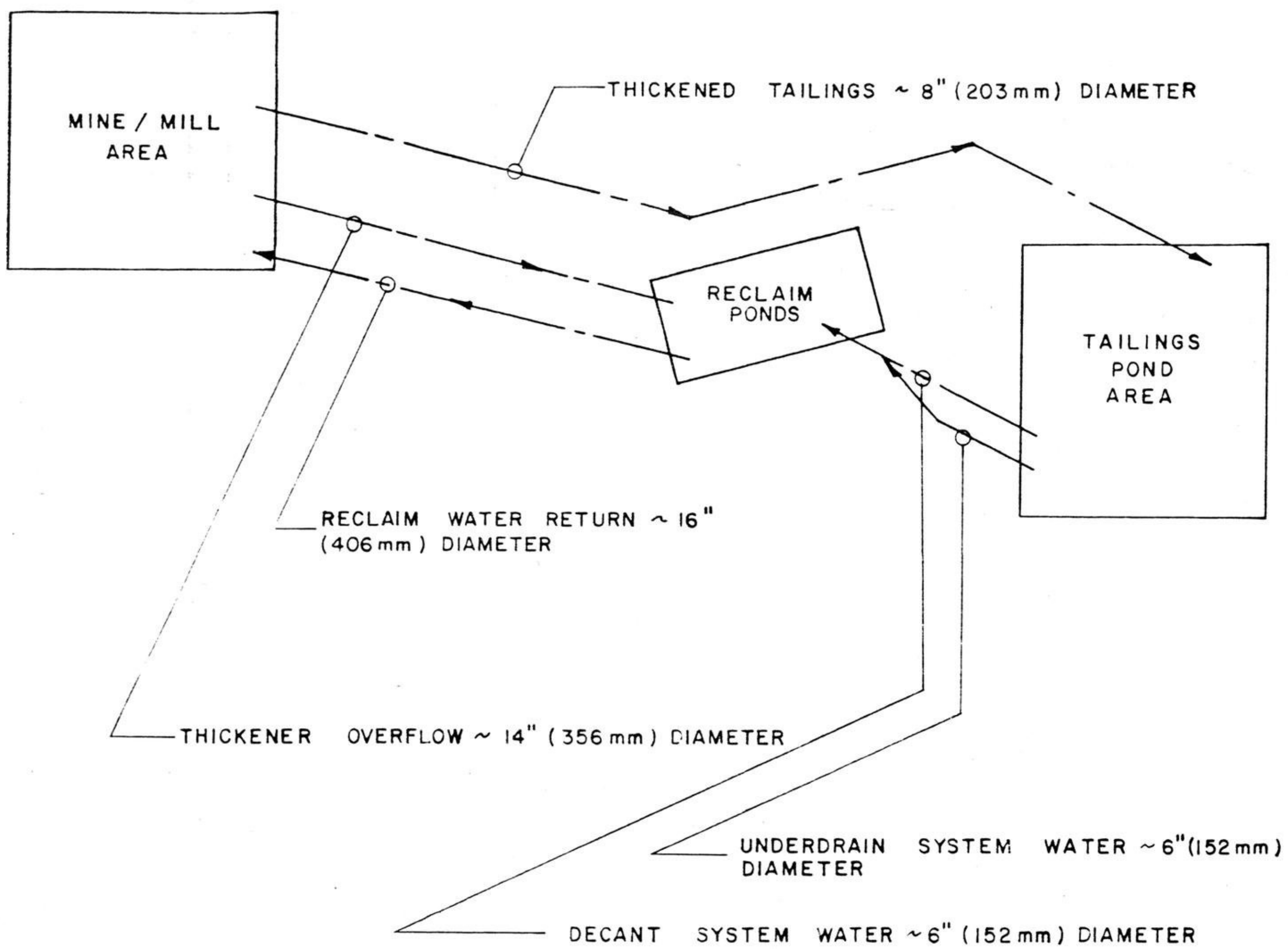
9.0 PIPELINES

The tailings materials will be pumped as a slurry to the waste disposal area at 50 percent solids, by weight, concentration. In the mill, these tailings are at a much lower concentration and will be thickened prior to pumping. In thickening, water is separated from tailings slurry concentration. This separated water is known as thickener overflow and will be pumped directly to the reclaim ponds. Water is returned to the mine/mill complex from the reclaim ponds.

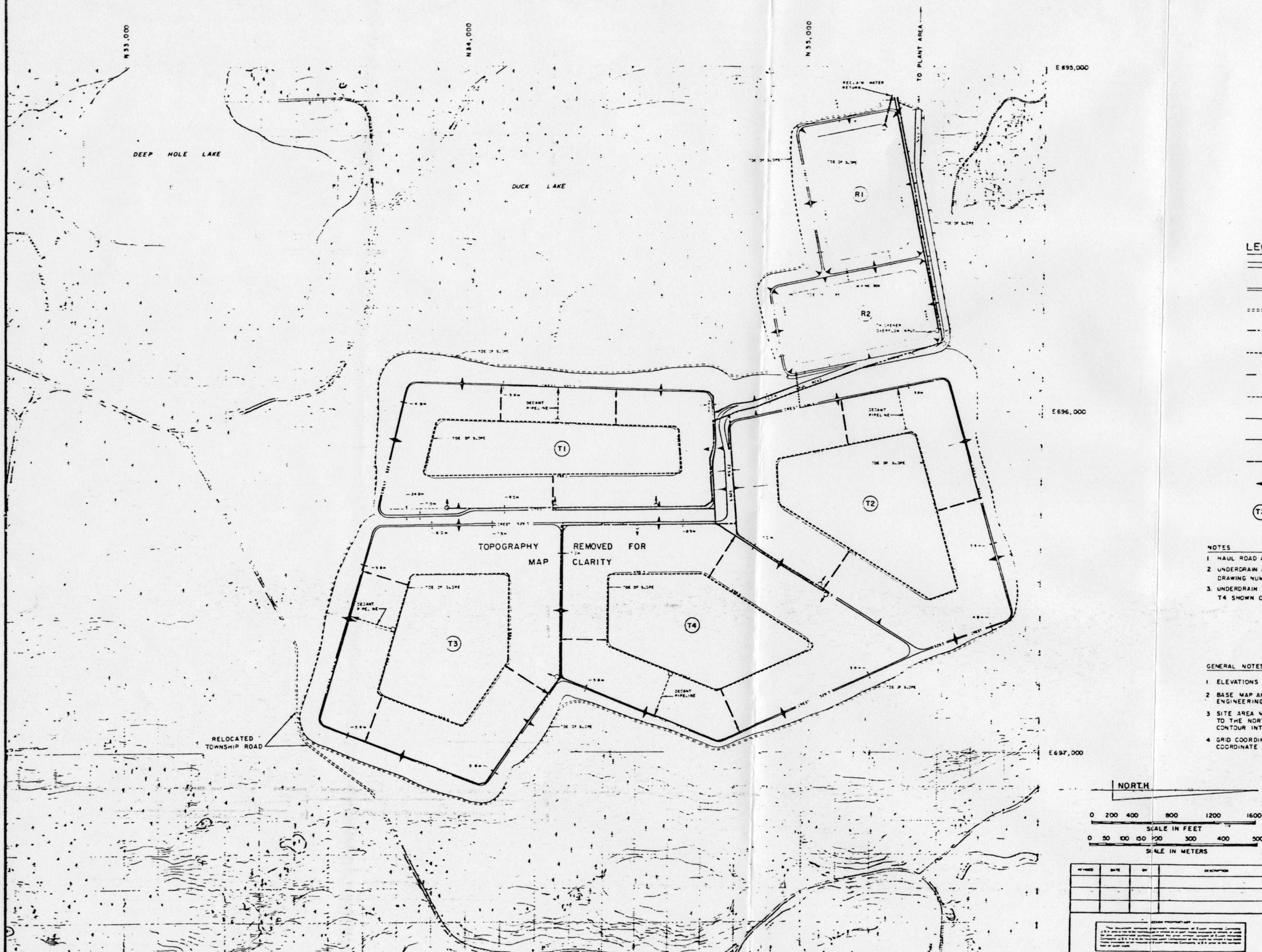
As the tailings settle, clarified water plus precipitation will be decanted from the tailings ponds and pumped to the reclaim ponds. Water collected in the tailings pond underdrain system will also be pumped to the reclaim ponds.

A schematic diagram of the pipelines and their approximate sizes is shown on Figure 9.1. The various alignments of the pipelines around the waste disposal and reclaim pond area are shown on Figure 9.2.

The pipelines will be located along the sides of the access/haul road from the mine/mill complex area to the reclaim pond area. Within the waste disposal and reclaim pond areas the pipelines will generally follow the pond crest alignments since they also serve as access roads around the waste disposal areas. The tailings slurry, decant water, and underdrain water pipelines will be placed along the inside edges of the crests where possible. Schematic diagrams of their location in cross section are presented in Section 10 of this report.



JOB NO. 786085	SCALE NONE	PIPELINE SIZES AND ALIGNMENT SCHEMATIC
DRAWN CAB	DATE 8-16-82	
CHECKED	DWG. NO.	
Golder Associates		EXXON MINERALS COMPANY — FIGURE 9.1



LEGEND

- HAUL ROAD
- CREST / ACCESS ROAD
- PERIMETER ACCESS ROAD (SECURITY FENCE OUTSIDE ACCESS ROAD)
- TAILINGS PIPELINE
- UNDERDRAIN COLLECTION PIPELINE
- UNDERDRAIN DISCHARGE PIPELINE
- DECANT PIPELINE FROM FLOATING PUMP
- DECANT AND UNDERDRAIN WATER PIPELINE
- THICKENER OVERFLOW
- RECLAIM WATER RETURN FROM FIXED PUMP
- TAILINGS INPUT POINT
- (T2) POND NUMBER

NOTES

1. HAUL ROAD AND CREST WIDTH DETAILS SHOWN ON DRAWING NUMBER 050-1-80597
2. UNDERDRAIN AND COLLECTION PIPE DETAILS FOR TAILINGS POND T1 SHOWN ON DRAWING NUMBER 050-1-80596.
3. UNDERDRAIN AND COLLECTION PIPE DETAILS FOR TAILINGS PONDS T2, T3, AND T4 SHOWN ON DRAWING NUMBER 050-1-80595.

GENERAL NOTES

1. ELEVATIONS IN METERS ABOVE MSL BASED ON 1929 ADJUSTMENT
2. BASE MAP AND AERIAL PHOTOGRAPHY (1976) BY AERO-METRIC ENGINEERING, INC.
3. SITE AREA MAPPING AT 1.0 METER CONTOUR INTERVAL. ADDITIONAL AREA TO THE NORTH AND WEST WAS ADDED AT THE AVAILABLE 2.0 METER CONTOUR INTERVAL TO PROVIDE ADDITIONAL COVERAGE.
4. GRID COORDINATES IN METERS BASED ON WISCONSIN STATE PLANE COORDINATE SYSTEM.

FIGURE 9.2

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CRANDON PROJECT

TITLE
SITE 41-114B
ACCESS ROADS AND PIPELINE ALIGNMENTS SCHEMATIC

SCALE AS SHOWN | STATE WISCONSIN | COUNTY FOREST
DRAWN BY CAB | DATE 9-17-82 | CHECKED BY | DATE 9/30/82
APPROVED BY | DATE 9/30/82

DRAWING NO. 050-1-80599 | 0

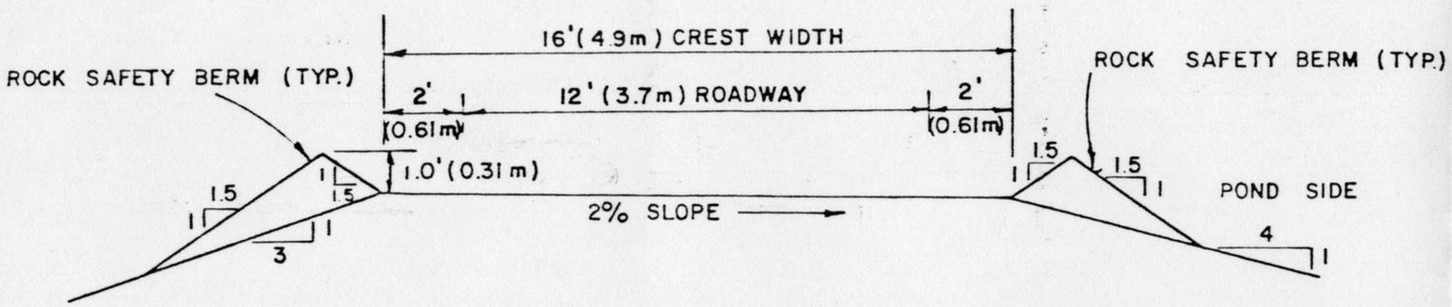
10.0 CREST AND ROAD DETAILS

10.1 Embankment Crests

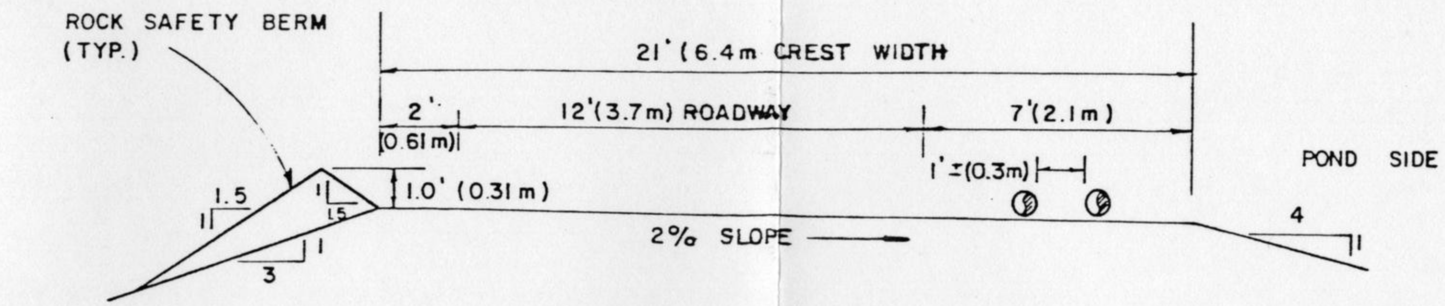
The embankment crests serve as access roads and pipeline corridors. The minimum crest width for vehicle access, assuming one-way traffic, is a 12 foot (3.7 m) roadway. In addition, safety berms, guard rails, or other obstructions with a height equal to that of a vehicle axle height are required by the Duluth, Minnesota regional office of the Mine Safety and Health Administration. Where pipelines are placed along the edge of the crest, they could also satisfy the requirements for a safety device. Safety berm height is presently estimated at one foot (0.31 m) for automobile or light truck vehicles. The minimum crest width for access, with no pipelines, has been set at 16 feet (4.9 m). The crests will have a stone surface to permit all weather use.

Pipelines along an embankment crest require space along both sides of the pipeline for access. The number of pipelines along a crest varies depending on the particular embankment. The pipeline alignments were shown on Figure 9.2 in Section 9. Typical sections of the varying crest width are shown on Figure 10.1. It should be noted that the water return lines from the decant and underdrain systems are assumed to be carried in one pipe in areas where they would parallel each other. However, these pipes are small enough that they could be carried in parallel without the need for increasing the crest width.

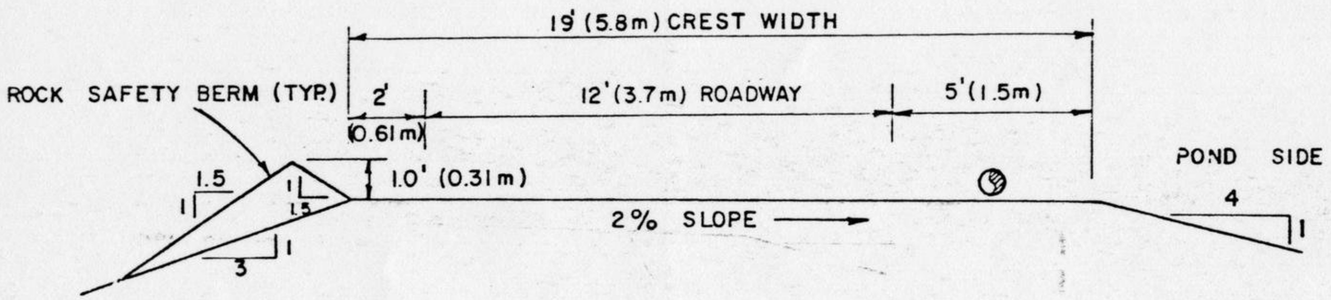
Crest widths for water retention dams are sometimes determined from a formula proposed by the United States Bureau of Reclamation (USBR) (Ref. 19) suggesting a width of 10 feet (3.1 m) plus one foot (0.3 m) for each 5 feet



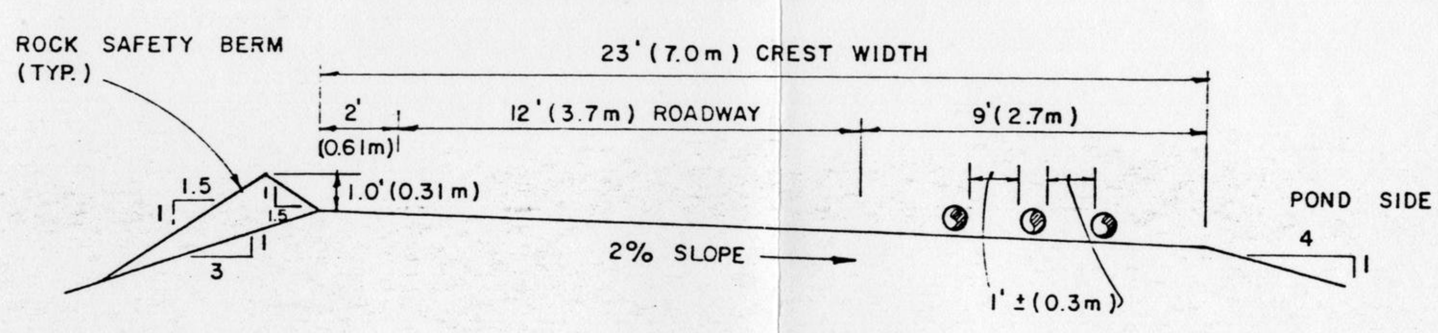
TYPICAL EMBANKMENT CREST
NO PIPELINES



TYPICAL EMBANKMENT CREST
TWO PIPELINES



TYPICAL EMBANKMENT CREST
ONE PIPELINE



TYPICAL EMBANKMENT CREST
THREE PIPELINES

JOB NO. 786085	SCALE 1:60	TYPICAL EMBANKMENT CREST SECTIONS
DRAWN CAB	DATE 8-16-82	
CHECKED	DWG. NO.	
Golder Associates		EXXON MINERALS COMPANY — FIGURE 10.1

(1.5 m) of embankment height. This criteria has not been applied to the determination of crest widths for this project. In some cases this formula suggests crests which would be too narrow and in some cases it would suggest crests which are considered too wide. The tailings ponds are not solely water retention structures and the analyses indicate that crests wider than those provided are not needed for stability. The reclaim ponds, which are water retention structures have crest widths in excess of those suggested by the USBR.

The crests of the tailings and reclaim ponds will be sloped to provide surface drainage into the ponds. Where safety berms or pipelines are located along the inside edge of the crest, small swales beneath the pipes or short gaps in the berms will be provided to permit runoff to drain into the ponds.

10.2 Haul Road

The haul road will be used by large trucks taking waste rock to the embankment/storage area. The size of these trucks requires a wider road than would otherwise be provided for normal maintenance and inspection vehicles. The haul road alignment within the waste disposal area is shown on Figure 9.2 in Section 9 of this report. The haul road will have a 2 foot (0.61 m) thick stone roadbed to permit all weather use for heavily loaded trucks. The surface will be sloped to drain.

The minimum two-way road width for a 50 ton (45×10^3 kg) capacity rock truck is 48 feet (14.6 m). Berms are required on the roadway edge of all fill slopes. The berm height, equal to the axle height of a 50 ton (45×10^3 kg) rock truck, is approximately 3.5 feet

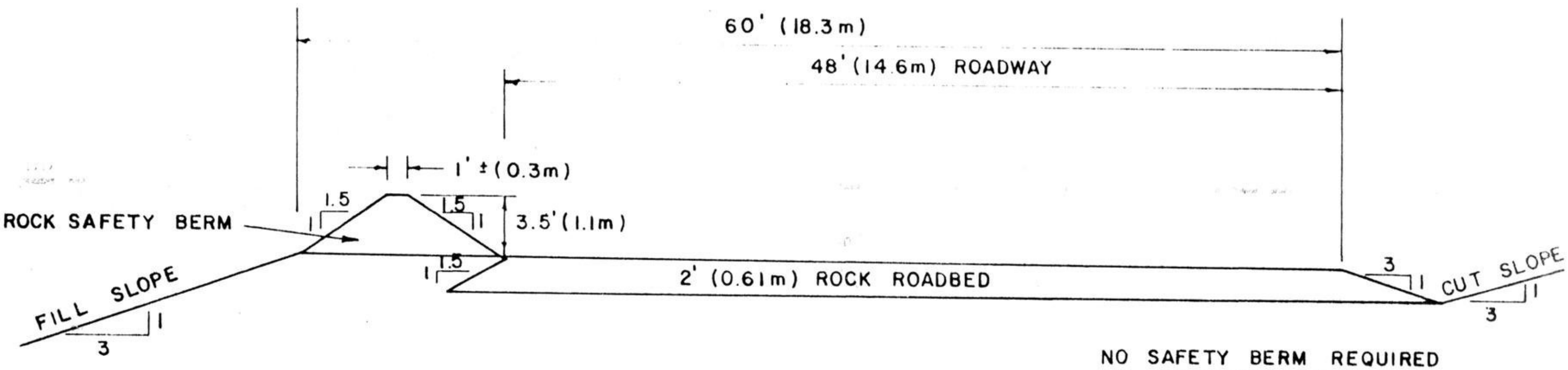
(1.1 m). To accommodate two-way traffic, berms, and pipelines, a nominal 66 foot (20 m) width has been provided for the haul road. Typical sections for the haul road are shown on Figure 10.2.

10.3 Perimeter Service Road and Security Fence

A service road around the perimeter of the waste disposal area will be provided. A security fence will be located outside of the perimeter service road. As the waste disposal area is developed in time, by the construction of the individual tailings ponds, the perimeter service road will be moved to coincide with the revised perimeter. The alignment of this road was shown on Figure 9.2 in Section 9.

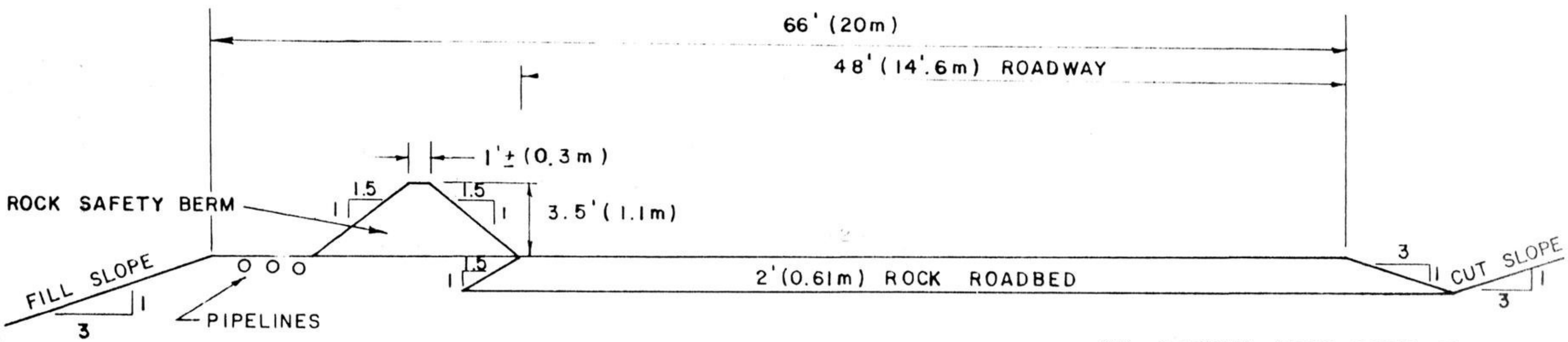
The purpose of this road is to provide access to the toe area of the embankment slopes for periodic inspection and maintenance. The road will be about 20 feet (6.1 m) wide which will permit passing room for two-way traffic. The road will approximately follow the contours of the natural ground. It will have a stone surface to permit all weather use.

In the southeast corner of the site, tailings pond T3 will cover part of an existing, unpaved township road. The affected portion of this road will be relocated outside of the waste facility. This relocation will be outside of the perimeter service road and security fence.



NO SAFETY BERM REQUIRED
ON CUT SLOPE SIDE

TYPICAL HAUL ROAD SECTION
NO ALLOWANCE FOR PIPELINES



NO SAFETY BERM REQUIRED
ON CUT SLOPE SIDE

TYPICAL HAUL ROAD SECTION
ALLOWANCE FOR THREE PIPELINES

Golder Associates

JOB NO. 786085
DRAWN CAB
CHECKED
SCALE 1:120
DATE 8-16-82
DWG. NO. —

EXXON MINERALS COMPANY

TYPICAL HAUL ROAD SECTIONS

FIGURE 10.2

11.0 EMBANKMENT SLOPE EROSION

Erosion potential from precipitation was determined for the 3.0 horizontal to 1.0 vertical exterior slopes proposed for the waste disposal system. This slope was selected from a maintenance and esthetic standpoint and, as will be discussed, will not tend to erode.

Soil erosion from surface water is primarily a function of the soil type and velocity of the water. The glacial till soil which will be used to construct the embankments is a fairly well graded granular material generally ranging in size from silt to coarse gravel. The maximum water velocity to minimize erosion on a granular material (sand and gravel) is about 4 feet per second (1.2 m/s). For properly grassed and maintained slopes this velocity may be as high as 6 feet per second (1.8 m/s) (Ref. 1):

The velocity of water flowing down a slope is a function of the depth of flow, the roughness of the surface, and the slope of the surface. For steady flow conditions the quantity of flow is described by the equation:

$$q = AV \tag{11.1}$$

where: q = discharge rate (L^3/T),
A = cross section area of the flow (L^2),
V = velocity of flow (L/T).

The velocity of flow is determined from Manning's equation:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{11.2}$$

where: V = velocity in feet per second,
n = roughness coefficient,
R = hydraulic radius, A/P (cross section area divided by wetted perimeter),

S = slope gradient (vertical divided by horizontal).

Combining these above equations we have:

$$q = \frac{1.49}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2} \qquad (11.3)$$

To estimate the velocity of sheet flow for the 3.0 horizontal to 1.0 vertical embankment slopes for the Crandon Project waste disposal ponds, a unit strip 1.0 feet (0.31 m) wide has been considered. The wetted perimeter is approximately 1.0 feet and the cross section area of the flow is 1.0 feet (0.31 m) times the depth of flow. The gradient is 0.33 for these slopes. A roughness coefficient of 0.035 has been selected for use in the analysis. This value is conservative since it assumes complete submergence of the grass cover.

The discharge rate of water along the slope has been estimated by the Rational Method for predicting rainfall runoff rates as expressed by the equation:

$$q = cia \qquad (11.4)$$

where:
- q = design peak runoff (discharge) rate in cubic feet per second,
- c = runoff coefficient,
- i = rainfall intensity for the design return period in inches per hour, and
- a = watershed area in acres.

The runoff coefficient has been selected as 1.0 which conservatively assumes all water is runoff (Ref. 1). The rainfall intensity for the 100 year, 1-hour storm in north-central Wisconsin was taken as 2.5 inches (64 mm)

(Ref. 20). The watershed area is that for a 1.0 foot (0.305 m) strip along a 100 foot (30.5 m) high embankment. This equals 300 square feet (27.9 m^2) which is equal to 6.9×10^{-3} acres (2.8×10^{-3} ha). Using these values the design peak runoff rate, q, is 1.7×10^{-2} cubic feet per second (4.0×10^{-3} m^3/s).

The depth of flow may now be calculated from equation 11.3 by solving for A since A = depth of flow for a unit strip. For conditions analyzed this depth is 0.15 inches (3.9 mm). Using this depth of flow in equation 11.1, the velocity of flow down the 3.0 horizontal to 1.0 vertical slope is 1.3 feet per second (0.40 m/s). This flow velocity is much lower than the 4 feet per second (1.2 m/s) to control erosion. Therefore, soil erosion should not be a problem with the 3.0 horizontal to 1.0 vertical exterior embankment slopes, nor a problem for the 4.0 horizontal to 1.0 vertical interior embankment slopes.

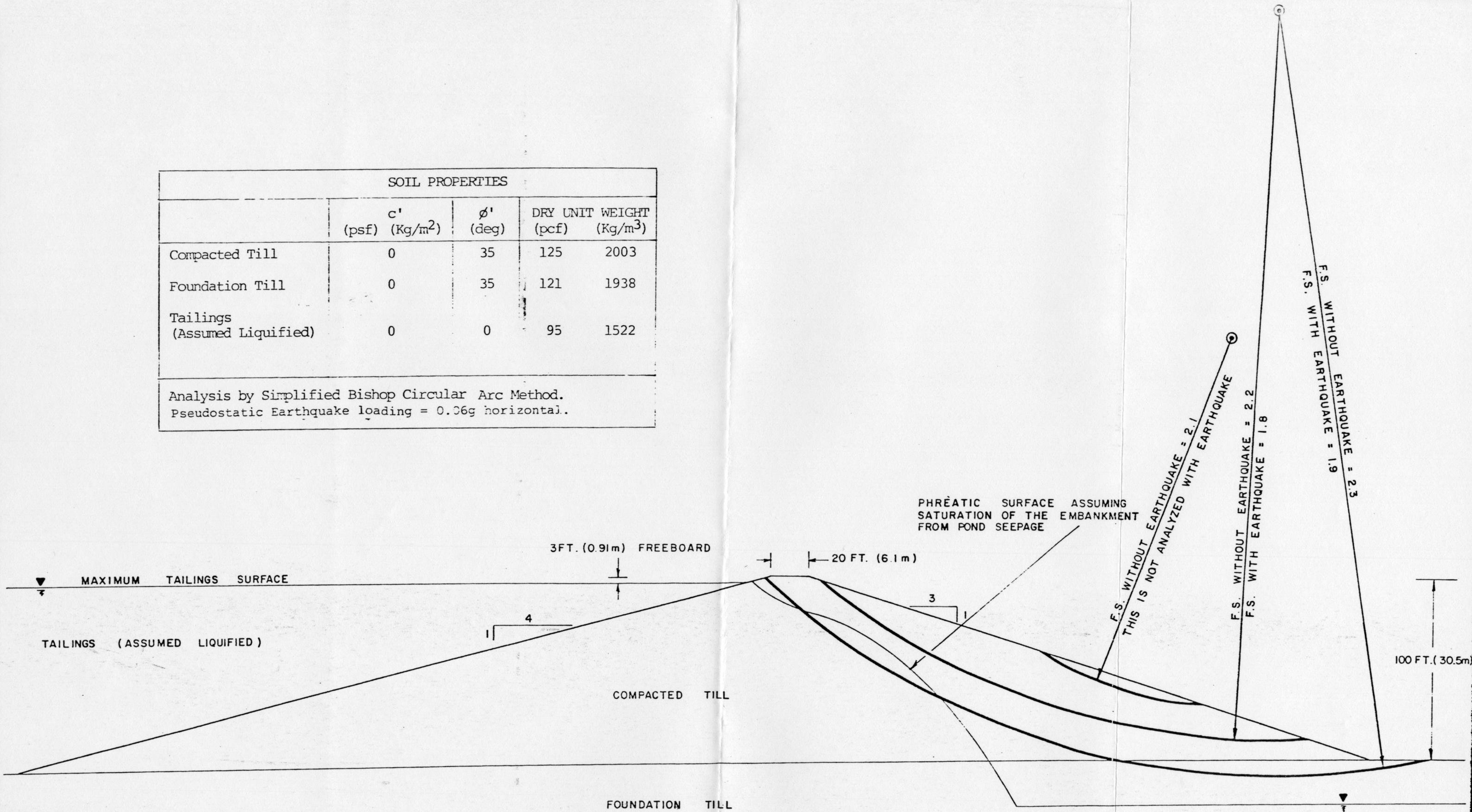
12.0 SLOPE STABILITY ANALYSIS

Embankment slope stability analyses were performed to determine the factor of safety with respect to potential slope failure. An embankment section 100 feet (30.48 m) high was analyzed. Since the analyses were performed before design of system 41-114B was complete, the maximum section of 105 feet (32 m) was not analyzed. However, safety factors from the analyses are sufficiently high that it is obvious that if the slightly higher embankment had been analyzed the results would have been equally satisfactory. The section analyzed has interior side slopes of 4.0 horizontal to 1.0 vertical and exterior side slopes of 3.0 horizontal to 1.0 vertical. The interior berms which tend to increase the stability of the slopes were added to system 41-114B after the analysis was made. Properties of the embankment and foundation materials were selected from the low end of the range of values determined by laboratory testing (Ref. 6). The soil properties used in the analyses are shown on Figure 12.1.

Two very conservative assumptions were made in the analyses. First, it was assumed that the tailings in the pond would be liquified and have no strength even though analyses (see Section 2 of this report) indicate the tailings mass is not likely to liquify. Second, it was assumed that seepage flow from the pond would be great enough to saturate a large portion of the embankment. A flow net analysis was then used to determine seepage forces in the embankment and foundation. This assumptions is very conservative since the estimated seepage from the pond will not be enough to saturate the surrounding embankment and foundation (Ref. 8).

SOIL PROPERTIES					
	c' (psf)	c' (Kg/m^2)	ϕ' (deg)	DRY UNIT WEIGHT (pcf)	DRY UNIT WEIGHT (Kg/m^3)
Compacted Till		0	35	125	2003
Foundation Till		0	35	121	1938
Tailings (Assumed Liquified)		0	0	95	1522

Analysis by Simplified Bishop Circular Arc Method.
Pseudostatic Earthquake loading = 0.06g horizontal.



JOB NO. 786085	SCALE 1:600	SLOPE STABILITY ANALYSIS	
DRAWN CAB	DATE 8-16-82		
CHECKED	DWG. NO.		
Golder Associates		EXXON MINERALS COMPANY	FIGURE 12.1

Embankment stability was analyzed for the static condition and with an earthquake loading. Both analyses included the two conservative assumptions previously described. A pseudo-static earthquake force of 0.06 g was applied in the analysis based on the possibility of a 6 percent horizontal ground acceleration (Ref. 2).

The 3.0 horizontal to 1.0 vertical downstream (outside) embankment slope was analyzed. The 4.0 horizontal to 1.0 vertical upstream (inside) embankment slope was not analyzed. Results of the downstream slope analyses were satisfactory and the flatter, upstream slope analyses would yield even higher safety factors.

A large number of potential circular arc failure surfaces were investigated with a computer using the Simplified Bishop Method. A hand check verified the accuracy of the computer solution. Analysis conditions and results are shown on Figure 12.1.

Results of the analysis show that without the earthquake force the minimum factor of safety against slope failure is 2.1. This is greater than the commonly considered minimum factor of safety of 1.5 (Ref. 4). The minimum factor of safety for any circular arc passing through the foundation was found to be 2.3. The minimum factor of safety including the earthquake loading was found to be 1.8. The minimum factor of safety for any circular arc passing through the foundation including the earthquake loading was 1.9. Both factors of safety are considerably above the minimum of 1.0 recommended in Reference 4 for earthquake loading.

Since the near-maximum section analyzed was found to be stable, analysis of any smaller embankment sections would also result in high safety factors. Also, since such high safety factors were obtained with the two very conservative assumptions that the tailings are liquified and have no strength and that much of the embankment is saturated, other analysis conditions with less conservative assumptions were not considered since they would result in higher safety factors. The analyses performed indicate stable embankment slopes for the proposed construction.

13.0 SEEPAGE HISTORY

13.1 Background

The seepage history for the 41-114B pond system was determined by adding together the seepage history of each tailings pond in the system. Seepage from each tailings pond was calculated using the methods presented in References 8 and 11. Seepage was calculated for each pond in three stages; 1) operation, 2) initial tailings desaturation, and 3) long term seepage. During the first two stages pond water in the underdrain system is assumed to be evacuated by pumping.

Seepage rates from the reclaim ponds have not been included in the system 41-114B seepage history because the estimated seepage rates are too small to be of any consequence. The estimated seepage rate during operation, based on a synthetic liner permeability of $1x10^{-14}$ m/s ($3.2x10^{-14}$ ft./sec.), for these two ponds combined is 0.2 gallons per minute ($1.2x10^{-5}$ m^3/s). Since these two ponds will be completely dismantled at reclamation there is no long term seepage from these ponds.

Each pond in the system will have a 6 inch (0.152 m) thick soil-bentonite liner over the pond bottom and side slopes. The design permeability of this liner is $5x10^{-10}$ m/s ($1.6x10^{-9}$ ft./sec.) (Ref. 7). The liner will be covered by a processed drain material 18 inches (0.457 m) thick. This drain material will be made of processed glacial till with particles smaller than the #40 sieve (0.42 mm) removed. Using Hazen's approximation and the anticipated grain size distribution of the drain, its permeability is estimated as $2.5x10^{-3}$ m/s ($8.2x10^{-3}$ ft./sec.). The estimated permeability of the tailings is $5x10^{-8}$ m/s ($1.6x10^{-7}$ ft./sec.) (Ref. 7).

After filling, the tailings surface in each pond will be reclaimed. The reclamation cover includes a 6 inch (152 mm) thick soil-bentonite cap with a design permeability of 5×10^{-10} m/s (1.6×10^{-9} ft./sec.). This cap is then covered with a 3 foot thick layer (0.91 m) of glacial soil which will support vegetation. The infiltration through the cap into the tailings has been conservatively estimated assuming that the entire glacial soil cover is saturated. The infiltration rate is reduced by an 11 week frozen period during which the infiltration rate is taken to be zero. Based on the above assumptions, the average annual infiltration through the cap has been estimated to be 3.4 inches per year (2.76×10^{-9} m/s) (Ref. 10).

After the reclamation cap is constructed, pumping of the underdrain system in each pond must continue until a substantial portion of the tailings desaturation has occurred. The period for which the pumping must continue is dependent on the peak seepage flow rate which is acceptable for the entire disposal system. Short pumping periods will result in high peak seepage flow rates for each pond and high peak flow rates for the system. Conversely, longer pumping periods result in lower peak seepage flow rates. The long term seepage flow rate (steady-state) is determined by the infiltration through the top seal, as further discussed in Section 13.4 of this report.

Assuming that 100 gallons per minute or less (6.3×10^{-3} m^3/s) is an acceptable peak seepage flow rate for the disposal system and the long term infiltration is 3.4 inches per year (2.76×10^{-9} m/s), the required time of pumping after the reclamation cap is constructed can be estimated by the procedures in Reference 8 with successive application of various pumping periods. For the 41-114B system, a pumping period of 17 years after copmletion of the reclamation cap for tailings ponds T2, T3, and T4 and a pumping period of 11 years for tailings pond T1 results in the seepage histories in Tables 13.1 through 13.4. These individual pond seepage histories and a composite seepage history for the 41-114B system are shown on Figure 13.1. The peak seepage rate for these conditions is 92.4 gallons per minute (5.83×10^{-3} m^3/s).

13.2 Pond Seepage During Operation

The seepage rate from a tailings pond during operation will increase slightly as the pond is filled. The rate of filling for each pond was estimated using the tailings production schedule provided by Exxon and the storage volume to area relation for each pond.

Using the estimated filling rate of each pond, the seepage rate from the tailings into the underdrain can be calculated by the following:

$$Q = k_t A \text{ (equation 7.3, Ref. 8)} \qquad (13.1)$$

where Q = seepage rate (L^3/T),
k_t = tailings permeability (L/T),
A = bottom area (L^2).

This approach assumes that little or no water is ponded over the tailings surface so that the gradient equals unity. It is also assumed that the entire tailings mass remains saturated until the reclamation cap is in place.

The seepage rate from the tailings is used to calculate the head (hydraulic thickness) in the underdrain. The method of determining the hydraulic thickness is explained

TABLE 13.1

TAILINGS POND T1 - SEEPAGE HISTORY

Time* (yrs.)	Q (gpm)	Q (m^3/s)	Comments
4	0	0	Pond startup
4.5	0	0	Initial tailings input @ t = 4.5 yrs.
5	1.4	.000089	
6	1.9	.000123	
7	2.4	.000150	
8	2.7	.000168	
9	3.0	.000187	Pond full
10	3.0	.000187	Reclamation cap complete
15	2.7	.000169	
20.8	2.6	.000166	Underdrain pump cut off
21.2	7.9	.000500	
21.6	13.4	.000845	
22	17.2	.001082	
22.5	20.3	.001280	
23	22.1	.001397	
23.5	23.3	.001467	
24	23.8	.001501	
24.5	24.0	.001517	Maximum seepage from pond
25	24.0	.001517	
25.5	23.7	.001495	
26	23.7	.001493	
--	14.1	.000892	Steady-state seepage

*Measured at the end of the year. Mine construction begins at start of year 1.

TABLE 13.2

TAILINGS POND T2 - SEEPAGE HISTORY

Time* (yrs.)	Q (gpm)	Q (m^3/s)	Comments
9	0	0	Pond startup
10	2.2	.000139	
11	2.6	.000164	
12	2.9	.000185	
13	3.2	.000205	
14	3.5	.000222	
15	3.8	.000239	
16	4.0	.000255	Pond full
17	4.0	.000255	Reclamation cap complete
27.6	3.6	.000225	
34	3.5	.000221	Underdrain pump cutoff
34.5	13.6	.000856	
35.1	21.0	.001324	
35.6	25.4	.001601	
36.2	28.0	.001764	
36.7	29.4	.001853	
37.3	30.1	.001900	Maximum seepage from pond
37.8	30.1	.001899	
38.4	30.0	.001890	
38.9	29.8	.001877	
39.5	29.4	.001857	
40.0	29.1	.001834	
40.6	28.7	.001812	
41.1	28.3	.001786	
41.7	27.9	.001763	
--	18.8	.001189	Steady-state seepage

*Measured at the end of the year. Mine construction begins at start of year 1.

TABLE 13.3

TAILINGS POND T3 - SEEPAGE HISTORY

Time* (yrs.)	Q (gpm)	Q (m^3/s)	Comments
16	0	0	Pond startup
17	1.9	.000123	
18	2.3	.000147	
19	2.7	.000169	
20	3.0	.000188	
21	3.2	.000205	
22	3.5	.000222	
23	3.8	.000237	Pond full
24	3.8	.000237	Reclamation cap complete
32.2	3.3	.000206	
41	3.2	.000205	Pump cut off
41.5	13.0	.000818	
42.1	20.1	.001268	
42.6	24.4	.001538	
43.2	26.8	.001692	
43.7	28.2	.001780	
44.3	28.8	.001818	
44.8	30.0	.001827	Maximum seepage from pond
45.4	28.9	.001824	
45.9	28.8	.001815	
--	17.2	.001088	Steady-state seepage

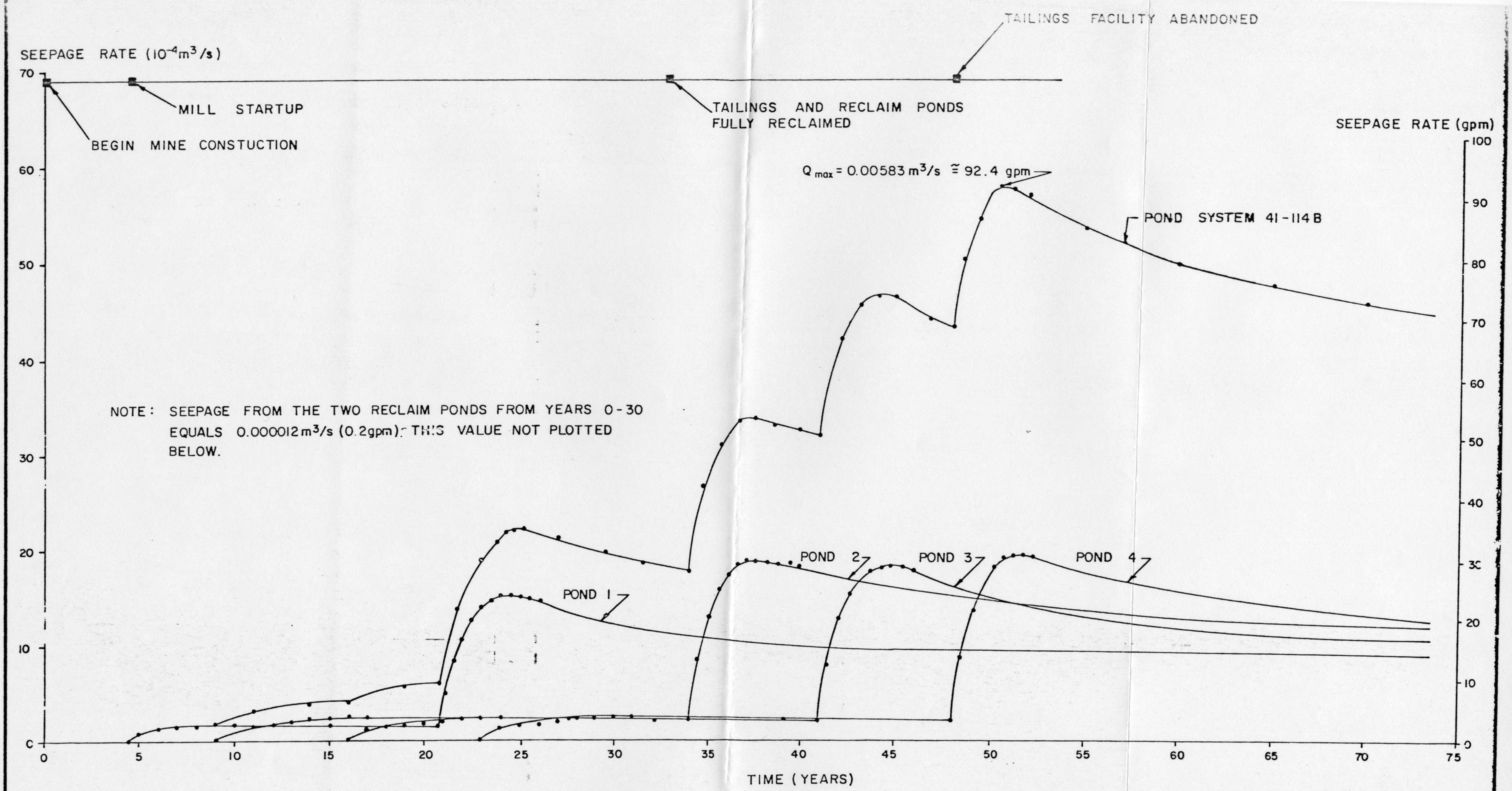
*Measures at the end of the year. Mine construction begins at start of year 1.

TABLE 13.4

TAILINGS POND T4 - SEEPAGE HISTORY

Time* (yrs.)	Q (gpm)	Q (m^3/s)	Comments
23	0	0	Pond startup
24	2.1	.000132	
25	2.5	.000158	
26	2.9	.000180	
27	3.2	.000200	
28	3.5	.000219	
29	3.7	.000236	
30	4.0	.000251	Pond full
31	4.0	.000251	Reclamation cap complete
39.2	3.6	.000226	
48	3.6	.000226	Pump cut off
48.5	14.2	.000898	
49.1	22.0	.001389	
49.6	26.5	.001672	
50.2	29.1	.001835	
50.7	30.6	.001928	
51.3	31.2	.001966	
51.8	31.2	.001970	Maximum seepage from pond
52.4	31.1	.001964	
52.9	30.9	.001950	
--	18.9	.001192	Steady-state seepage

*Measured at the end of the year. Mine construction begins at start of year 1.



JOB NO. 786085	SCALE AS SHOWN	SEEPAGE RATE HISTORIES
DRAWN CAB	DATE 8-16-82	
CHECKED	DWG. NO.	
Golder Associates		EXXON MINERALS COMPANY — FIGURE 13.1

in Reference 11. The hydraulic thickness in the bottom portion of the underdrain has a constant value on the order of 2 inches (0.05 m) during the filling of the pond. The hydraulic thickness in the slope portion of the underdrain increases as the pond is filled from near zero to a value less than 1 inch (0.025 m) depending on the exact pond configuration.

The seepage rate from the pond can be calculated using Equations 5.1 and 5.2 of Reference 8. The seepage rate will gradually increase as the pond is filled.

It is estimated that placement of the reclamation cap may take as long as one year after pond filling. Seepage from the pond during this one year period is expected to be unchanged from the seepage rate at the time the pond is filled.

13.3 Pond Seepage During Initial Tailings Desaturation

After the reclamation cap is in place, the tailings will begin to desaturate due to gravity drainage of water from the pore spaces of the tailings. Pumping from the underdrain must continue during this period to prevent a substantial increase in head on the liner and resultant increase of seepage out of the pond. After a substantial amount of desaturation has occurred, pumping can be discontinued but there will be an increase in pond seepage to a level not much greater than steady-state.

The rate of tailings desaturation was examined in detail in Appendix C of Reference 8. Two important conclusions from this work are that the time rate of tailings desaturation is roughly proportional to the depth of tailings, and that the desaturation rate is relatively insensitive to the infiltration rate through the cover seal. Of the two calculation methods presented in the referenced appendix, the equations of Method 1 were used to calculate the time history of seepage from the tailings. Having estimated the flow from the tailings into the underdrain and the hydraulic thickness, the resultant seepage rate from the pond has been calculated with the same equations used for these calculations during pond operation.

13.4 Long-Term Pond Seepage

When the tailings seepage is no longer pumped from the underdrain the flow from the tailings into the underdrain is greater than the flow from the underdrain out of the pond. Therefore, the piezometric level in the pond will rise and the excess water will be stored in the pore spaces of the underdrain and tailings. This higher piezometric level will increase the seepage rate from the pond. The piezometric level will increase until the flow from the pond is slightly greater than the flow into the underdrain. When this point is reached both inflow and outflow will gradually decrease until they reach steady-state and equal the infiltration through the reclamation cap.

The time history of the seepage rate after evacuation of water from the underdrain ceases has been calculated by applying a series of equations to a succession of time increments. Having estimated the rate of water inflow to the underdrain and the rate of seepage from the pond at the beginning of any time increment, the following equation is used to calculate the change, during the time increment, in the water volume stored in the soil pores:

$$\Delta s = (I-Q)\Delta t \qquad (13.2)$$

where I = inflow which includes infiltration plus gravity drainage from tailings (L^3/T),
Q = seepage from the pond (L^3/T),
Δs = change in water volume stored in pore spaces (L^3), and
Δt = time increment (T).

The total volume of water stored at the end of the time increment can be found using the storage value at the beginning of the increment and s.

The storage volume-height relation for the pond can be used with the pore storage to find the piezometric level in the pond at the end of the time increment. The seepage rate from the pond at the end of the increment can then be calculated by:

$$Q = K_L \frac{h+D_L}{D_L} A_b + K_L \frac{(h/2) D_L}{D_L} A_s + K_L (1.0)(A_t - A_b - A_s) \qquad (13.3)$$

where K_L = permeability of the liner (L/T),
h = piezometric level (L),
D_L = thickness of the liner (L),
A_b = area of the pond bottom (L^2),
A_s = area of the pond slopes below h (L^2), and
A_t = total area of pond covered with tailings (L^2).

Any number of desired time increments can be calculated in the above manner. The maximum rate of seepage from the pond will occur at the time when seepage from the pond first exceeds inflow. At all later times both the rate of seepage from the pond and rate of inflow to the pond will approach a value equal to the infiltration rate through the reclamation cap.

14.0 EMBANKMENT FAILURE FLOOD ANALYSIS

14.1 Purpose

The proposed Crandon Project slurry waste disposal ponds are designed with approximately 50% of their depth beneath the surrounding ground level. Anticipated mode of operation is to fill each of the tailings ponds with solid tailings, maintaining a clarificiation pool of water over approximately 20 percent of the pond surface area. Prior to closure, the largest clarification pool, which is in tailings pond T4, will be above surrounding ground level and will contain, with the PMP storm water, about $21x10^6$ cubic feet ($6x10^5$ m^3) of water. In addition, reclaim pond R1 with the PMP storm water will provide about $32x10^6$ cubic feet ($9x10^5$ m^3). Therefore, an analysis was performed to estimate the flood depths in the surrounding lakes which would result from a breach of either a tailings pond or a reclaim pond embankment. This analysis considers only the hydrologic/hydraulic impact of the released flood waters and not the resulting environmental impact.

14.2 Methodology

The proposed waste disposal system impoundments store relatively small volumes of water at shallow depths. Since the ponds are totally enclosed with no upstream drainage basin, there is no additional storm water volume to that which falls directly on the ponds. In the event of an embankment failure, there is little likelihood of hazard to human safety from the resulting flood, based on the following considerations.

1. The proposed disposal facility contains relatively small volumes of water stored in broad, shallow pools. This would result in low, gradual floodwaves in the event of a failure of the embankment.

2. There is very little development in the Project area. The homes around Hemlock Lake and Little Sand Lake are the only areas of concentrated development.

Based on the above outlined considerations regarding the storage volume and the hazard potential in the event of failure, a simple volume comparison approach was selected to assess the effects of embankment failure.

The volume of water stored in the slurry ponds prior to a hypothetical breach was taken as the design volume (full clarification pool) plus the water volume from the Probable Maximum Precipitation (PMP) event. The PMP corresponds to a total rainfall depth of 23 inches (584 mm) (Ref. 20). The volume of this storm event falling within each tailings pond was added to the volume of water in the clarification pool. These water volumes are presented in Table 14.1.

The reclaim ponds were assumed to store water to the normal operating level plus the PMP rainfall. Also, reclaim pond R1 was assumed to contain the PMP stormwater volume decanted from the largest tailings pond (pond T4) prior to failure. Table 14.1 lists the pertinent data regarding these storage volumes.

The surface watercourses surrounding the proposed waste disposal system are primarily lakes and gently sloping streams, as shown in Figure 14.1. As previously stated, the two main areas of development are Ground Hemlock and Little Sand Lakes. In order to determine the upper limit of depth increase in these and the other lakes due to a hypothetical embankment breach, the total design

TABLE 14.1

WASTE DISPOSAL POND VOLUMES

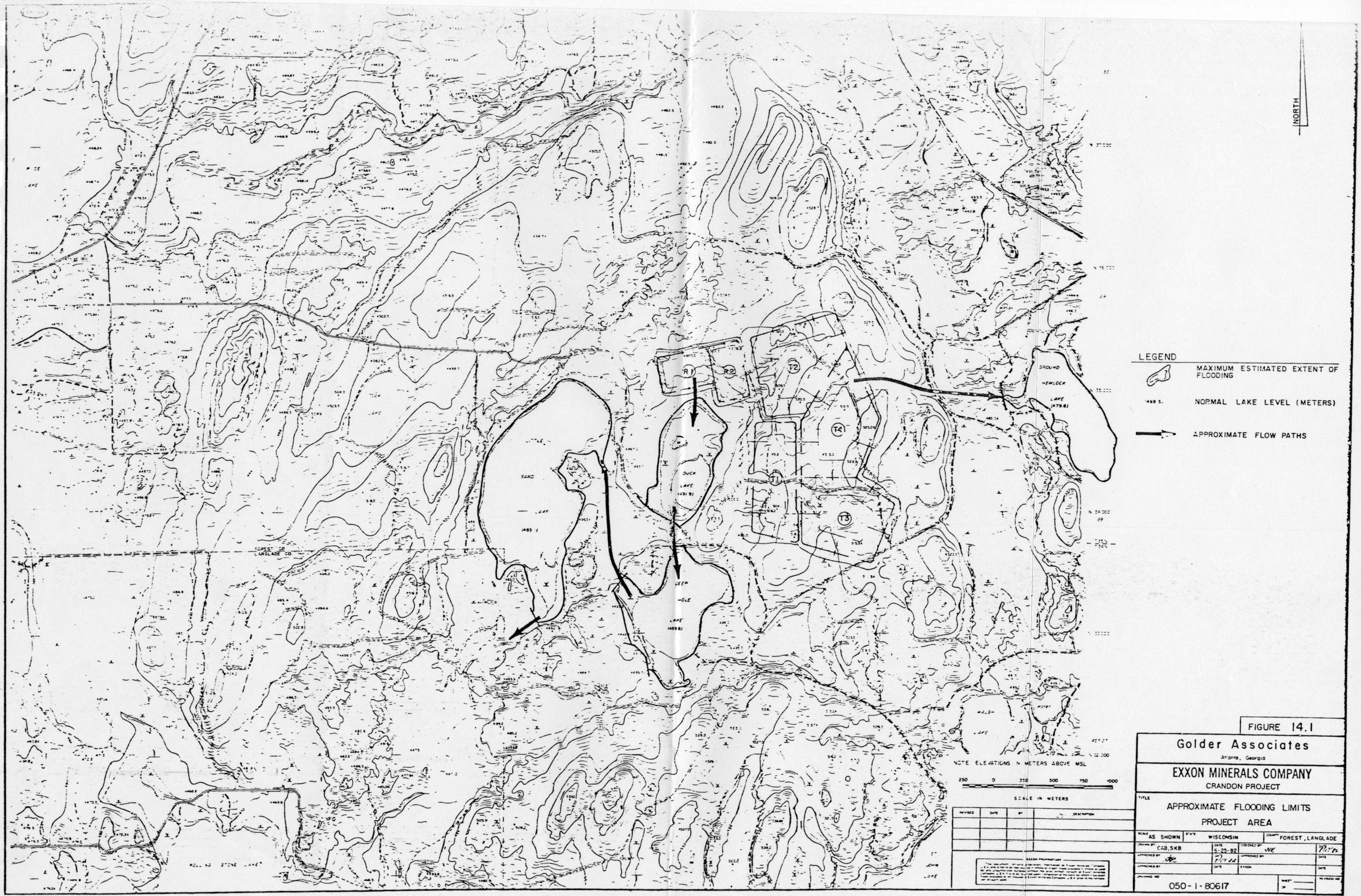
METRIC UNITS

Pond(1) Number	Design(2) Volume (m^3x10^5)	Surface(3) Area (ha)	PMP Volume (m^3x10^5)	Decant(4) Volume (m^3x10^5)	Total(5) Volume (m^3x10^5)
T1	1.83	40.0	2.34	--	4.17
T2	3.23	46.5	2.71	--	5.94
T3	2.62	42.7	2.49	--	5.11
T4	3.35	46.8	2.73	--	6.08
R1	5.90	13.5	0.78	2.73	9.41
R2	5.90	12.4	0.72	--	6.62

ENGLISH UNITS

Pond Number	Design Volume (ac.-ft.)	Surface Area (acres)	PMP Volume (ac.-ft.)	Decant Volume (ac.-ft.)	Total Volume (ac.-ft.)
T1	148	99	190	--	338
T2	262	115	220	--	482
T3	212	105	202	--	414
T4	272	116	222	--	493
R1	478	33	64	222	764
R2	478	31	59	--	537

Notes:
(1) T = Tailings slurry ponds, R = Reclaim water ponds.
(2) Design volume is volume of clarification pool at design normal water level (NWL).
(3) Surface area is total area draining into the pond, including crests and roads.
(4) Decant volume is assumed to be PMP volume from the largest pond, T4.
(5) Total volume used in breach analysis.



NORTH
LEGEND
MAXIMUM ESTIMATED EXTENT OF FLOODING
1489 5. NORMAL LAKE LEVEL (METERS)
APPROXIMATE FLOW PATHS
NOTE ELEVATIONS IN METERS ABOVE MSL
250 0 250 500 750 1000
SCALE IN METERS
SAND LAKE 1485.1
DUCK LAKE (1491.9)
DEEP HOLE LAKE (1489.8)
GROUND HEMLOCK LAKE (1479.8)
R1
R2
T1
T2
T3
T4
FOREST CO
LANGLADE CO
ROLLING STONE LAKE
FIGURE 14.1
Golder Associates
Atlanta, Georgia
EXXON MINERALS COMPANY
CRANDON PROJECT
TITLE
APPROXIMATE FLOODING LIMITS
PROJECT AREA
SCALE AS SHOWN
STATE WISCONSIN
COUNTY FOREST, LANGLADE
DRAWN BY CAB, SKB
DATE 5-25-82
CHECKED BY
APPROVED BY
DRAWING NO 050-1-80617

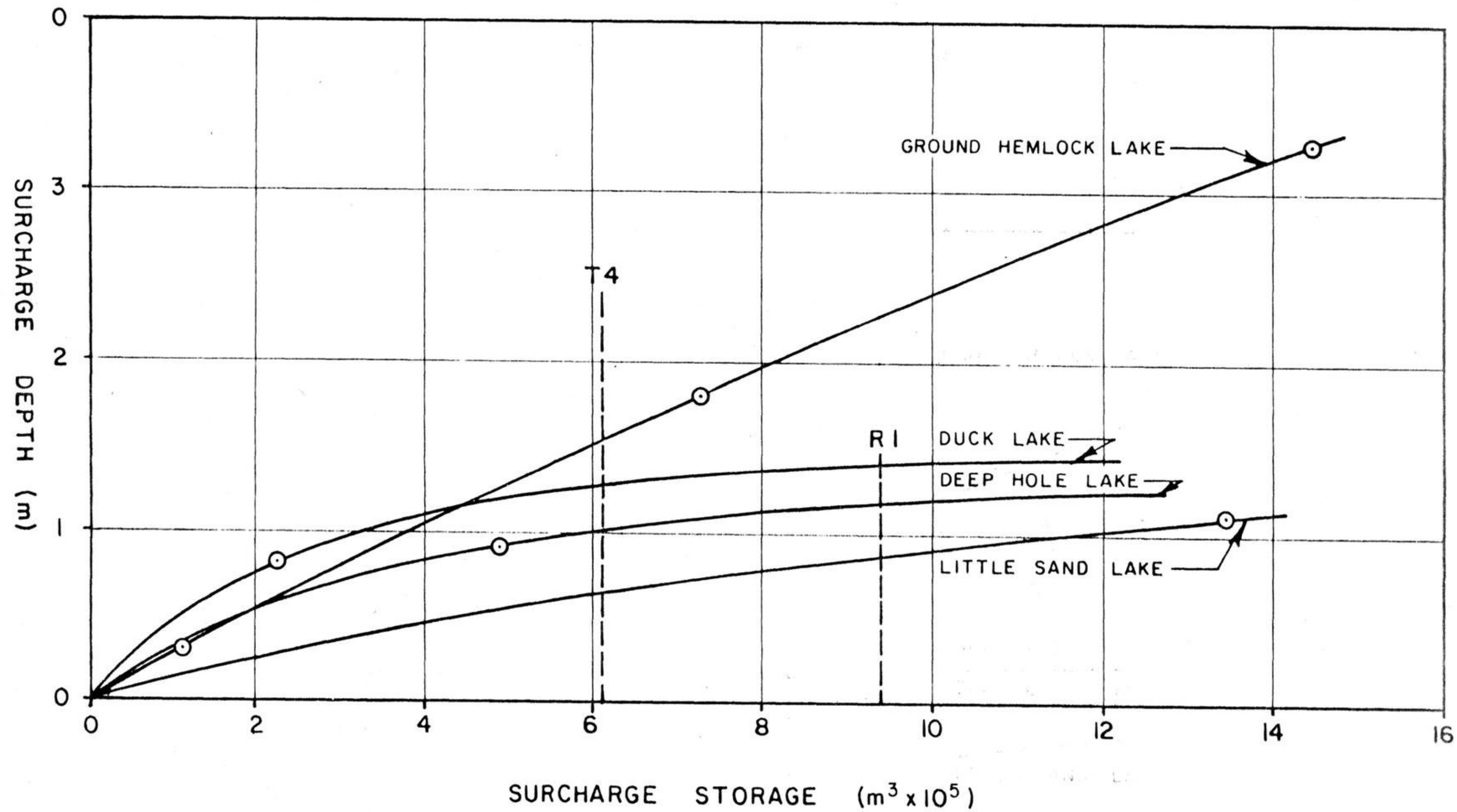
volume, plus storm and decant volumes of the largest tailings pond (T4) and reclaim pond (R1) was compared to the surcharge capacity of the lakes. This approach is conservative since it considers no outflow from the lake and assumes the entire waste system pond volume to be instantaneously transported to the lake.

In order to estimate the depth increase in the surrounding lakes the depth-storage relationship above normal water level for each lake was determined. Lakes such as Deep Hole Lake have several outlets at relatively shallow surcharge depths. However, the outflow characteristics of these outlets are a function of their geometry and vegetation. Therefore, the conservative approach was taken in which the surcharge depth-storage relationships assume no outflow through these points. The locations of these outlets are shown on Figure 14.1. The depth-storage relationships for each lake are shown on Figure 14.2. By entering these curves with a known flood volume, the maximum depth increase (surcharge depth) can be determined.

14.3 Results

Depth increases were estimated for Deep Hole, Duck, and Little Sand Lakes based on possible breach of reclaim pond R1 since R1 contains more water than R2 or tailings ponds T1 or T2. Depth increase for Ground Hemlock Lake was estimated for possible breach of tailings pond T4 which contains more water than tailings pond T3. By entering the surcharge stage-storage curves shown in Figure 14.2 with the appropriate hypothetical breach volume, a depth increase can be determined. A breach of tailings pond T4 into Ground Hemlock Lake would result in a maximum depth increase of 4.95 feet (1.51 m). A breach of reclaim water pond R1 into Deep Hole Lake would result in a maximum depth

NOTE:
SHAPE OF CURVES INFERRED FROM SURROUNDING TOPOGRAPHY



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DEPTH-STORAGE CURVES ABOVE NORMAL POOL

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FIGURE 14.2

increase of 3.84 feet (1.17 m). Similarly, a breach of R1 into Duck Lake and Little Sand Lake would result in depth increases of 4.59 feet (1.40 m) and 2.79 feet (0.85 m), respectively. The approximate surface areas of these lakes with these depth increases are shown in Figure 14.1. These depth increase values are considered to be conservatively high, especially for Ground Hemlock Lake, due to the assumption of no outflows or time delay of the inflowing floodwaters.

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