Geotechnical Engineering Program
Civil Engineering Department

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TAILINGS MANAGEMENT PLAN EVALUATION

DAWN MINING COMPANY

FORD, WASHINGTON

by

John D. Nelson and Steven R. Abt

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for

U.S. Nuclear Regulatory Commission
NMSS

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1. INTRODUCTION

This report presents a review of the proposed new tailings management plan for the Dawn Mining Company, Ford, Washington. At the present time the Dawn Mining Company is depositing tailings at the existing facility. However, in anticipation of the existing impoundment becoming filled, a 28-acre, membrane-lined, below-grade impoundment, is proposed for construction.

This report reviews the proposed tailings management plan as it relates to USNRC Regulatory Guide 3.11 and evaluates the potential impact of the proposed plan. Because below-grade disposal is considered a prime option as discussed in the Generic Environmental Impact Statement, and because to locate this impoundment elsewhere would lead to proliferation of sites (because of the existing impoundment) an extensive review of alternative tailings impoundment options was not considered necessary.

The long-term stability of the proposed tailings management plan has been reviewed to the extent possible. However, the designed reclamation plan is not complete and cannot be considered by itself without taking into account the reclamation of the existing facility. For that reason, it is recommended that the long-term stability of the proposed impoundment be considered along with the proposed reclamation plan for the existing facility when that plan becomes available.

This report was prepared in the Geotechnical Engineering Program, Civil Engineering Department, Colorado State University. Principal reviewers were Drs. Steven R. Abt and John D. Nelson.

2. DESCRIPTION OF SITE CHARACTERISTICS

The description of the site characteristics is taken primarily from the Environmental Impact Statement prepared by the Dawn Mining Company dated July 30, 1979. The site characteristics are presented in some detail in that document and general observations made during February 1980 indicated no particular disagreement with the descriptions as outlined in that document. For completeness, the site characteristics are described in less detail herein.

2.1 Topography

The Dawn Mill complex is located on a feature known as Walker's Prairie, which is a northeast-trending valley about two miles wide and fifteen miles long. The valley is bordered on the northwest by basalts and on the southeast by granite. The valley floor is a flat plain of glacial outwash and flood deposits cut by the meandering channel of Chamokane Creek. The valley floor elevations at the mill site range from 1750 to 1760 feet. Cut banks between stream level and the main valley floor terrace are steep. General vegetation in the area consists of fir and pine.

It is noted in the EIS that the only significant accretionary/avulsionary action in the area is downcutting by Chamokane Creek. Rates of downcutting and widening of meander zones were completed over the past 20,000 years. It is also noted, however, that Chamokane Creek cascades over a series of scenic small falls dropping about 50 feet in a span of 500 feet. This cascade is known as Chamokane Falls.

It may be questioned, therefore, whether the accretionary/avulsionary forces are operant in the area, at the present time, at the same rate as was active several thousand years ago. It should be expected that Chamokane Falls represents a stable base level in the area that may govern the rate of downcutting of Chamokane Creek at the present time. Nevertheless, it may be expected that rates of downcutting are less than or equal to the average rates computed in the Environmental Impact Statement. Thus, the resistant rock at Chamokane Falls should enhance the long-term stability of the site.

2.2 Geology

The site and regional geology of the area is described in Chapter 3 of the Environmental Impact Statement and in more detail in Appendix A of that document. In general, the region has been geologically quiet since the final recession of the Great Continental Glaciers about 20,000 years ago. In the last hundred years there has been a scattering of minor earthquakes, probably related to glacial unloading, with epicenters located less than 100 miles from the Dawn Mill site.

Walker's Prairie, the site of the Dawn operation, is believed to be an erosional feature free of potential seismic structures. The mill area is underlain by a granitic basement buried beneath thin remnants of Columbia River basalt in a thick accumulation of glacio-fluvial clays, sands, and bouldery gravels. Figure 2.1 shows an interpretive geologic profile of the materials underlying and adjacent to the mill site. The significant aspects of the geology with regards to the proposed project are:

- most of the excavation will involve gravel and fine sands.
- deeply weathered basalt bedrock may be encountered in the eastern part of the pit floor.
- a dense glacio-lacustrine clay underlies most of the project site and provides a base for any vertically infiltrating seepage. This main lower clay unit is believed to be influencing seepage patterns from the present tailings impoundment.

Soils in the project area are generally gravelly, loamy sands derived from glacial outwash deposits with little or no organic-rich topsoils.

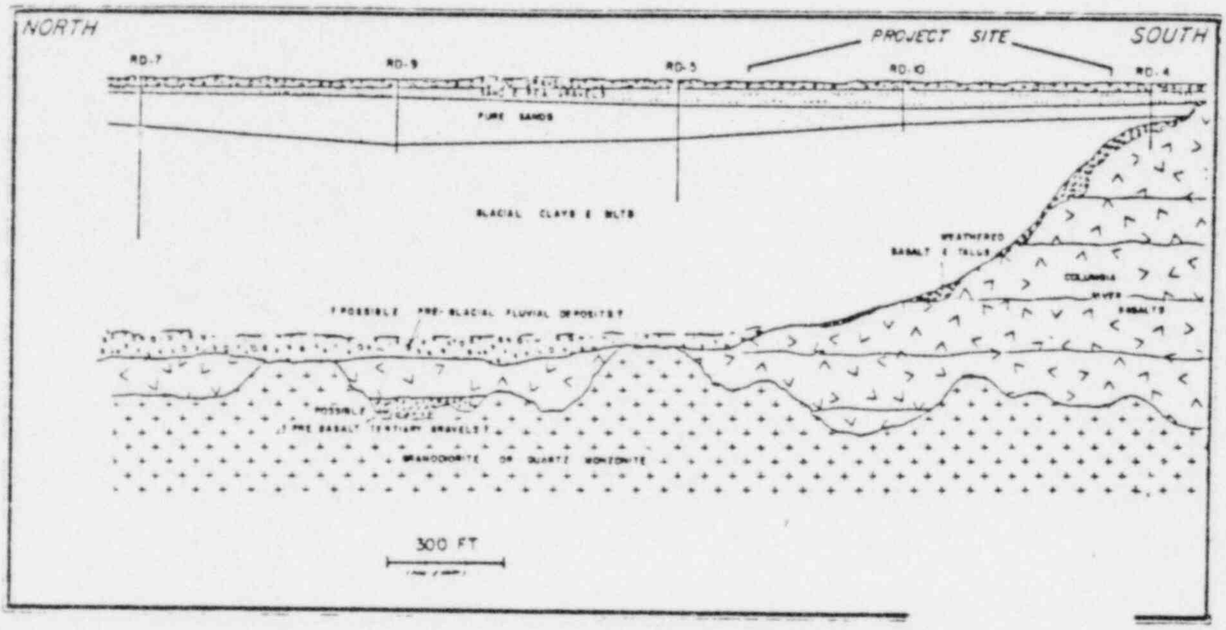


Figure 2.1. Interpretive Geologic Profile (Source: Reference 1)

2.3 Groundwater

The groundwater/surface water relationship in Walker's Prairie is complex and comprises several significant geohydrologic horizons. At the project site the uppermost groundwater zone occurs within and at the base of the highly permeable gravel/sand section composing the uppermost 100 feet of the valley fill. This water is perched on a dense silty blue/gray clay which serves as a base for vertical infiltration. The clay surface dips gently westward, inducing groundwater migration in that direction. Flows at this interface are said to be modest ranging up to a few gallons per minute.

Drill holes at the site of the proposed tailings impoundment for purposes of sub-soil investigation encountered groundwater in only one boring. The static water level in that hole was at a depth of 93.3 feet below the surface on October 17, 1979 (Ref. 2). Based on Dawn Mining Company data as noted in ref. 2, fluctuations in the groundwater table of up to 15 feet have been indicated in response to intense rainfall. Even with a fluctuation of 15 feet, normal groundwater levels would still be expected to remain beneath the maximum depth of excavation of the pit for the proposed tailings impoundment.

2.4 Hydrology

The design storm procedures outlined in the "Design of Small Dams" by the U.S. Bureau of Reclamation was used by the staff to compute the 36 hour, Probable Maximum Precipitation (PMP) General Storm, and the subsequent Probable Maximum Flood (PMF). The flood depth was derived in accordance with the PMF series specified in the USNRC Regulatory Guide 3.11. The PMF series analysis assumes that the impoundment must accept flood waters equivalent to 40 percent of the PMF followed in 3 to 5 days by the PMF, all of which was preceded or followed by a 100 year storm. The PMP was estimated to be 8.33 inches for the Dawn Mill site. The computations are presented in Section I, Appendix [illegible]

The USNRC PMF series yields equivalent storm depths of 3.4 inches, 8.4 inches and 4.4 inches for the 0.40 PMP, PMP and the 100 year storms respectively resulting in a total of 16.2 inches (1.35 feet) of precipitation. The PMF series thereby contributes in excess of 34 acre-feet of storm water to the impoundment. The proposed tailings pond at the Dawn Mill site covers an area of 25.5 acres which is adjacent to an existing tailings impoundment. Therefore, the area tributary to the impoundment is considered neglible. Because there is no tributary runoff, the design PMF is equal to the PMP.

The applicant utilized the U.S. Weather Bureau PMP computations of 1967 to generate a 72 hour PMP of 12 inches. However, instead of computing the 100 year storm depth and integrating this value into the USNRC PMF design procedure, the applicant apparently estimated the 100 year cumulative seasonal precipitation (October to March) of 25 inches for the project site. The total PMF precipitation depth reported was the summation of the PMP and the 100 year seasonal precipitation resulting in 37 inches (3.08 feet). The applicant estimated an inflow volume of 78.6 acre-feet of water into the impoundment.

Although the applicant did not follow the PMF series procedure as outlined in USNRC Regulatory Guide 3.11, the extreme conservatism resulted in a total precipitation of 2.25 times greater than a more traditional PMF series analysis. Therefore, the 37 inches of precipitation value generated by the applicant will be used throughout the remainder of this review.

2.5 Meteorology

The Dawn Mill site is located between the flatlands of the Columbia Basin to the west and the foothills of the Rocky Mountains to the east. The project site is situated such that there are no climatological records available. Therefore, the climatological data is based on the Spokane (25 miles to the southeast), Wellpinit (10 miles to the west southwest) and Chewelah (35 miles to the north northeast) weather stations.

The annual precipitation of the surrounding weather stations range from 16.5 inches to 19.3 inches. Approximately 70 percent of this total falls between the first of October and the end of March. During the October-March period, about half of the precipitation falls as snow. Throughout the remainder of the review presented in this report, the precipitation will be conservatively estimated at approximately 20 inches per annum for the Dawn project site.

The mean annual temperature for the area is about 47°F. The mean temperature during the winter months is about 28°F while the summer months average 66°F. Most of the air masses which reach the area consist of maritime Polar air brought in by prevailing westerly and south-westerly circulations. Occasionally, the area is over-ridden by dry continental Polar air masses from the north-east, resulting in high temperature/low humidity periods in the summer and/or sub-zero temperatures in the winter.

Annual high temperatures of 100°F have been recorded at the Dawn project site. Temperatures in the 80's and 90's are common during June, July and August. Annual low temperatures average approximately -7°F. However, extreme low temperatures have been recorded from -20°F to -40°F.

Prevailing surface winds are presented in Figure 2.2. Records from Spokane indicate that prevailing winds blow from the southwest and

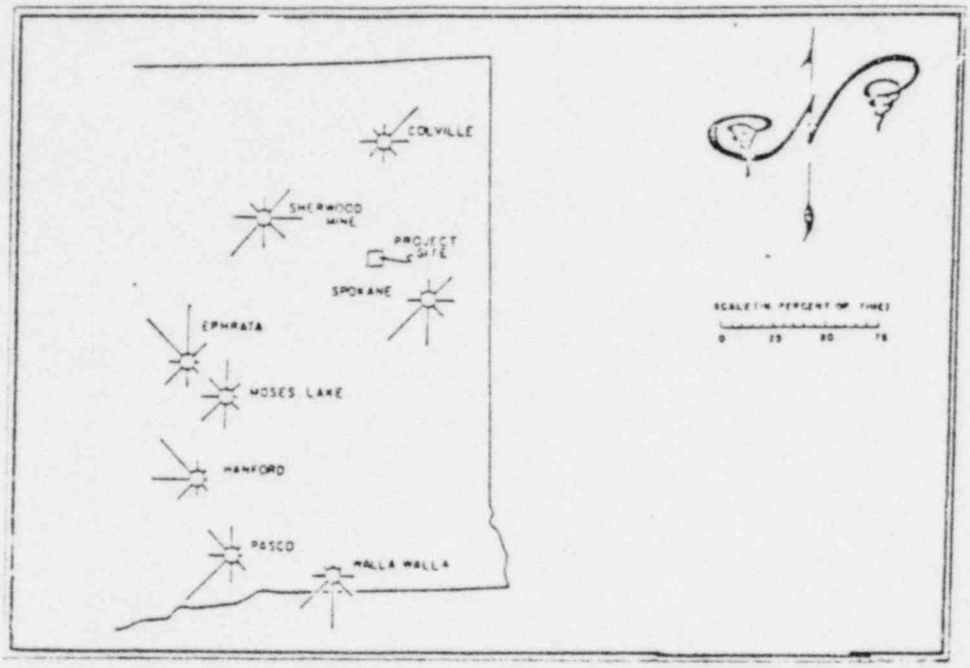


Figure 2.2. Surface Wind Roses for Washington Stations (Annual Average)

south-southwest at an average of 8.5 mph. During the winter months the air flow is commonly reversed, with winds out of the northwest.

The mean annual lake evaporation in the project area is approximately 38 inches per year. Class A pan evaporation is about 53 inches per year. An estimated monthly evaporation rate in inches per month is presented in Section III, Appendix D. It is estimated that the net annual evaporation is about 18 inches per year.

The mean annual relative humidity for Spokane is approximately 65 percent. Average vapor pressures are near 0.17 inches mid-winter and approximately 0.30 inches mid-summer.

Low-level temperature inversions commonly occur in northwestern Washington, active up to 20-30 percent of the late fall and winter months and increasing to 50-65 percent in summer and early fall. Periodic inversions during the winter months can result in stagnant surface conditions.

2.6 Seismicity

The area is stated to be an area of low seismic activity. Appendix B of the Environmental Impact Statement presents a tabulation of approximately 75 years of earthquake data. The greatest intensity of earthquakes listed therein are VI on the Modified Mercalli Scale, or magnitude 4.8 on the Richter Scale. In the Golder Report it is stated that a pseudo-static earthquake coefficient of 0.05g was utilized for design of the embankments. Data given in Bolt, 1977, indicates that for earthquakes of intensity VI on the Modified Mercalli Scale, maximum accelerations ranging from 0.005g up to 0.066g may be expected. In consideration of the fact that the earthquakes noted in Appendix B are for those within an area of 100 miles from the site and the fact that the site is located over a deep deposit of soil overlying bedrock, it is to be expected that the peak acceleration at this site will probably be less than 0.066g. Also, the pseudo-static earthquake coefficient used for stability analyses should be considerably lower than the maximum peak acceleration. Consequently, the use of a coefficient of 0.05g is considered to be realistic.

3. DESCRIPTION OF PROPOSED TAILINGS MANAGEMENT PLAN

3.1 General Project Description

At the present time three tailings disposal areas exist and tailings are being desposited therein. It is anticipated that the present tailings disposal capacity will be exceeded in a short period of time and an additional disposal system has been designed. Figure 3.1 shows a plan map of the existing tailings disposal areas and a proposed excavation site. The proposed tailings impoundment will consist of a specially excavated pit dug into the sand and gravel deposits to the south of the present tailings impoundment. Figures 3.2 and 3.3 indicate the cross-section of the proposed pit. The pit will be nominally 70 feet deep. The side slopes of the pit will be excavated on a slope of 3 horizontal to 1 vertical (3h:1v). Along the side of the pit adjacent to the existing tailings disposal facility, a 50 foot wide bench will be left to improve the overall stability. The other sides of the pit will be bounded by a 5 foot high and 30 foot wide perimeter dike constructed with materials removed from the excavation. Excess materials removed from the excavation will be stockpiled adjacent to the 5 foot high dike and other existing dikes for future use during reclamation.

The sides and bottom of the pit will be lined with 30 mil reinforced Hypalon to prevent seepage. An internal subdrain system will be placed on the bottom of the pit to accelerate removal of water from the tailings once the tailings disposal facility has been filled. A typical section through the pit showing details of the embankment are shown in Figure 3.4.

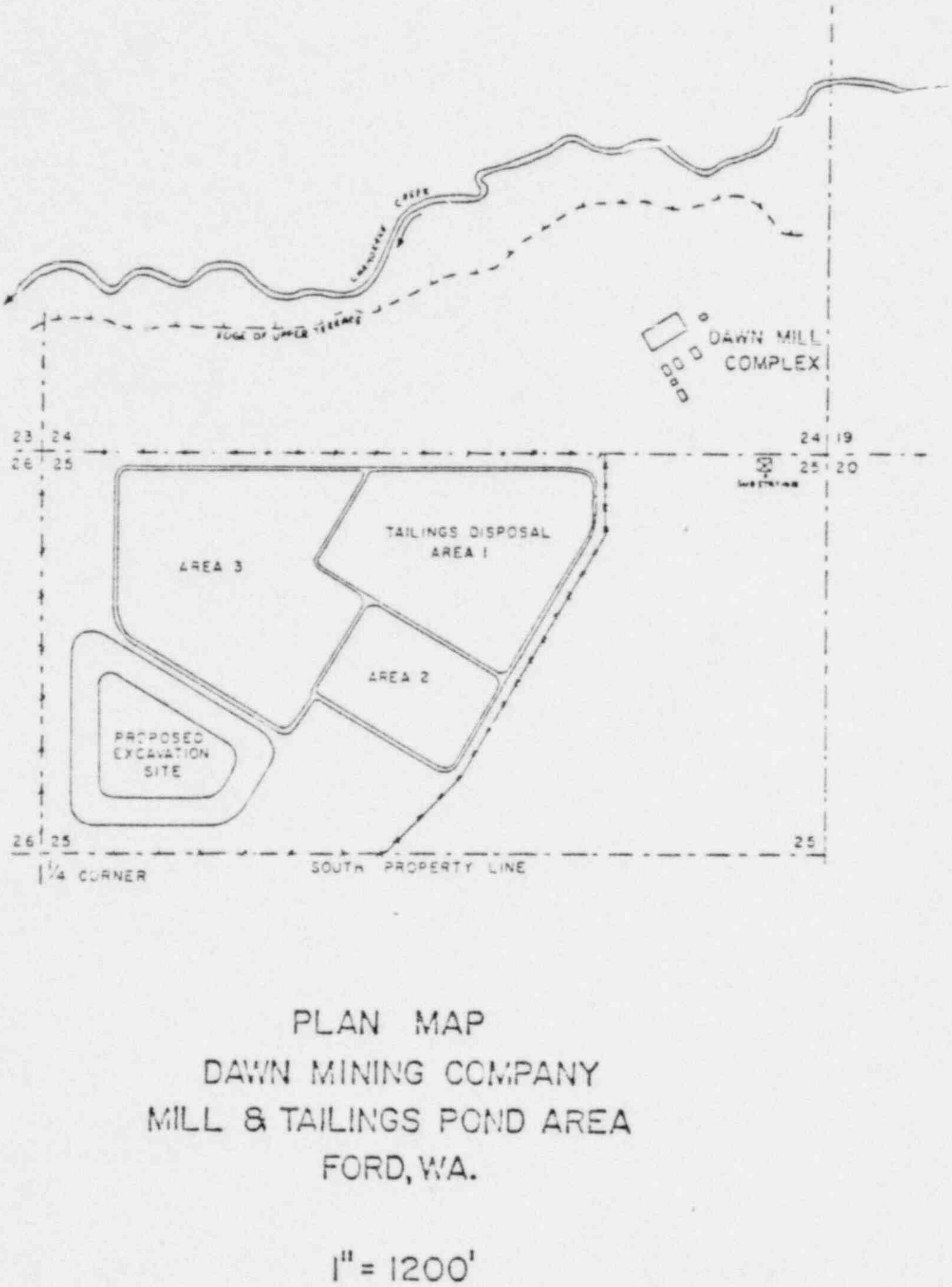


Figure 3.1. Plan Map (Source: Reference 1)

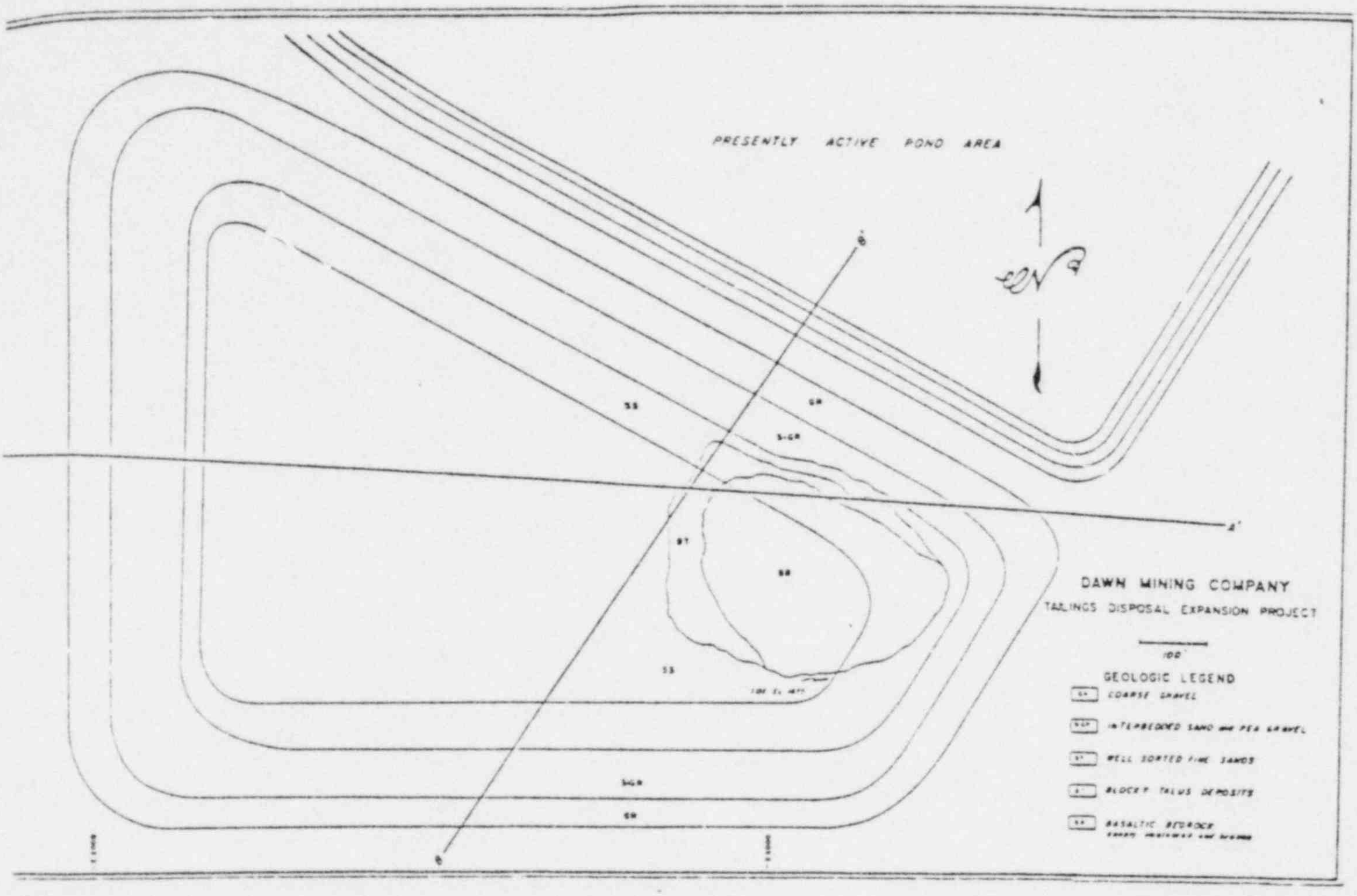


Figure 3.2. Tailings Disposal Expansion Project (Source: Reference 1)

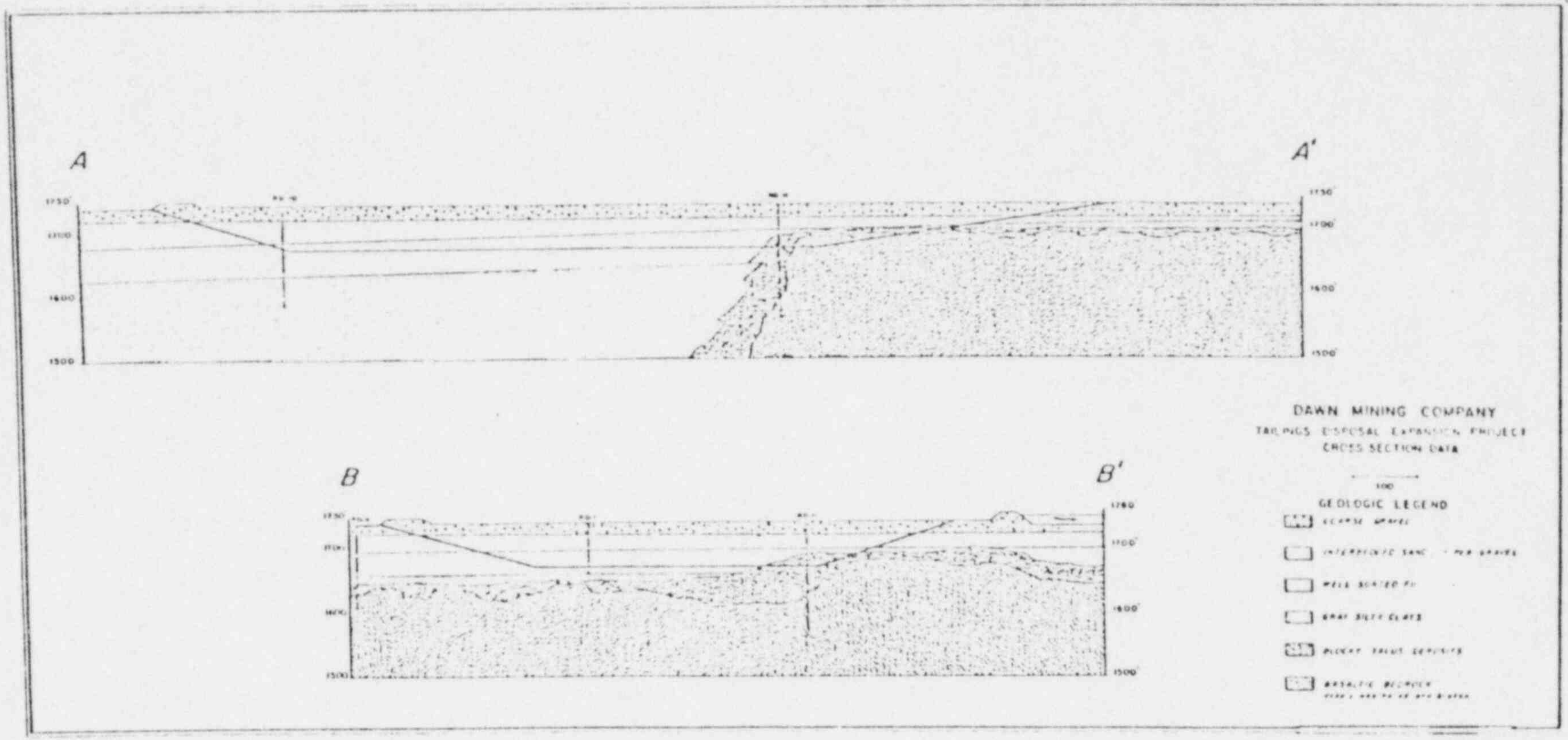


Figure 3.3. Cross-Sections of Proposed Pit (Source: Reference 1)

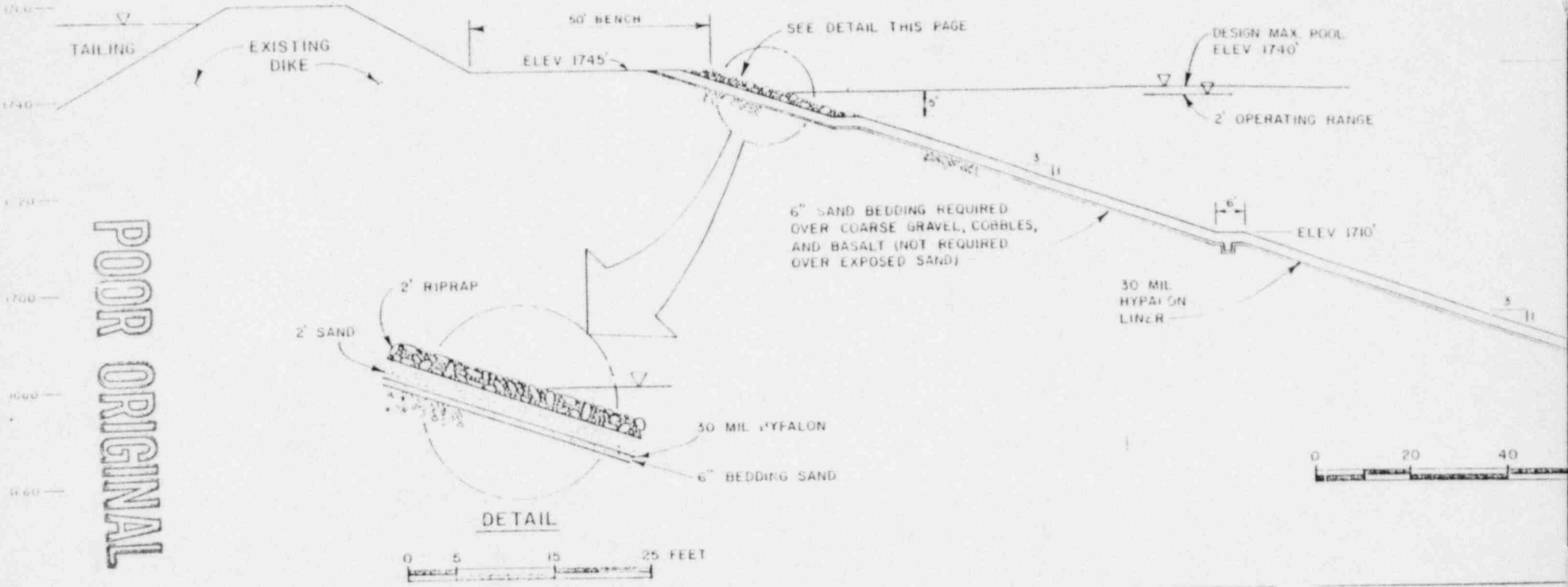
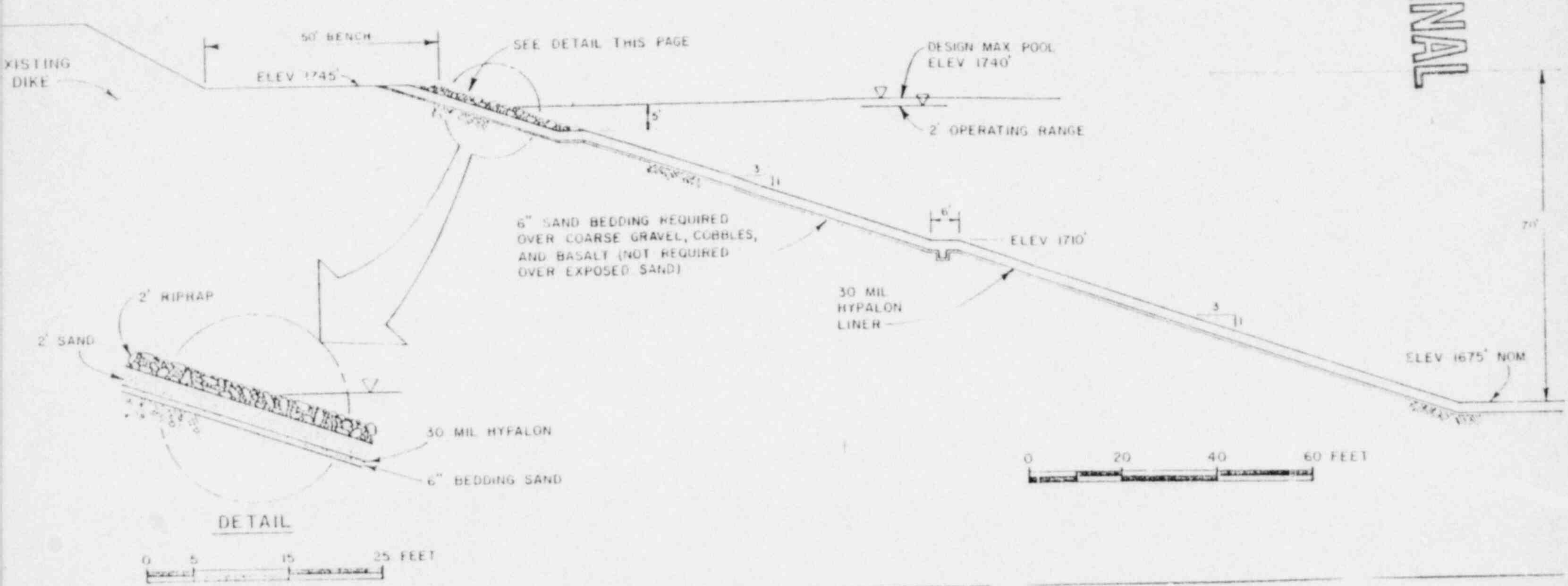


Figure 3.4. Typical Section of Pit (Source: Reference 2)



ure 3.4. Typical Section of Pit (Source: Reference 2)

3.2 General Disposal Operations

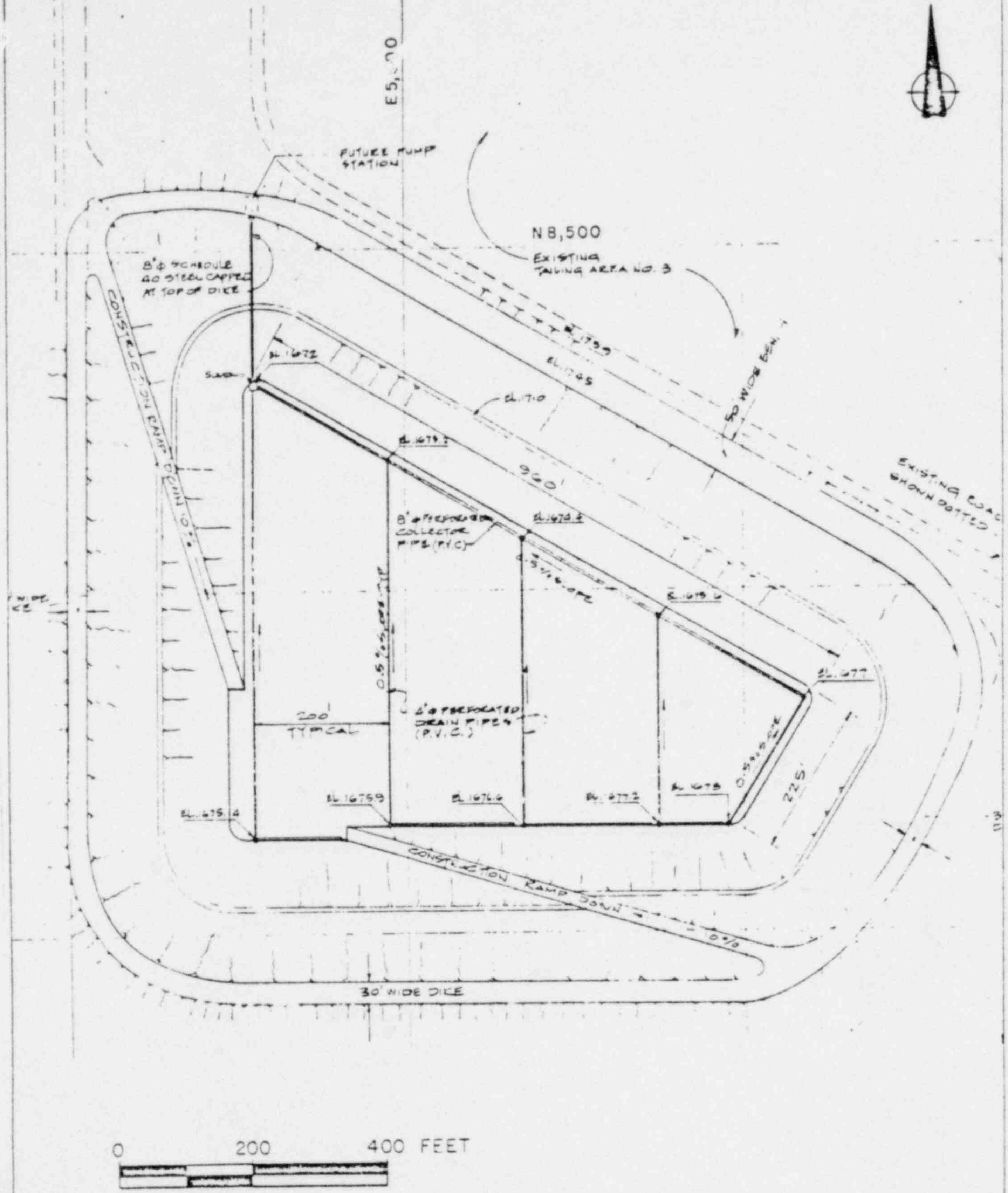
Tailings will leave the mill as a 30 to 50% solid slurry and will be pumped through a 6 inch PVC pipeline one-half mile to the proposed tailings disposal facility (Ref. 1). Tailings components are derived from several points in the mill circuit, but the principal exit point for the leached solid residues is from the Number 4 thickener underflow.

Tailings slurry will flow into the disposal facility at a rate of approximately 0.95 acre-feet per day (Ref. 1). Based on a 365 day per year operation, nearly 374 acre-feet of slurry will be deposited into the disposal facility annually. The pond will be allowed to fill with no decantation of the solution until the pond reaches the maximum operating level of from elevation 1738 feet to elevation 1740 feet. Since a Hypalong liner will be installed to control seepage, the only loss from the disposal facility will be to evaporation processes. The methods, locations and rate of decanting has not been addressed and cannot be evaluated. It is anticipated that an amendment will be filed after operations begin, specifically addressing decanting design and operation.

Throughout operations, a minimum freeboard of 5 feet will be maintained to manage any unexpected influxes of slurries or precipitation. A complete freeboard analysis will be discussed in Section 4.6.2 of this report.

3.3 Embankment and Pit

The general layout of the pit is shown in Fig. 3.5. As shown in Fig. 3.4, the slopes of the pit will typically be 3h:1v and the upper



REFERENCE: KILBORN/NUS INC
NO.7419-13-004
DATE: 14 SEPT. 79

POOR ORIGINAL

Figure 3.5. Plot Plan and Layout of Subdrain System (Source: Reference 2)

10 feet (vertical distance) of the slope will be covered with riprap to protect against wave action.

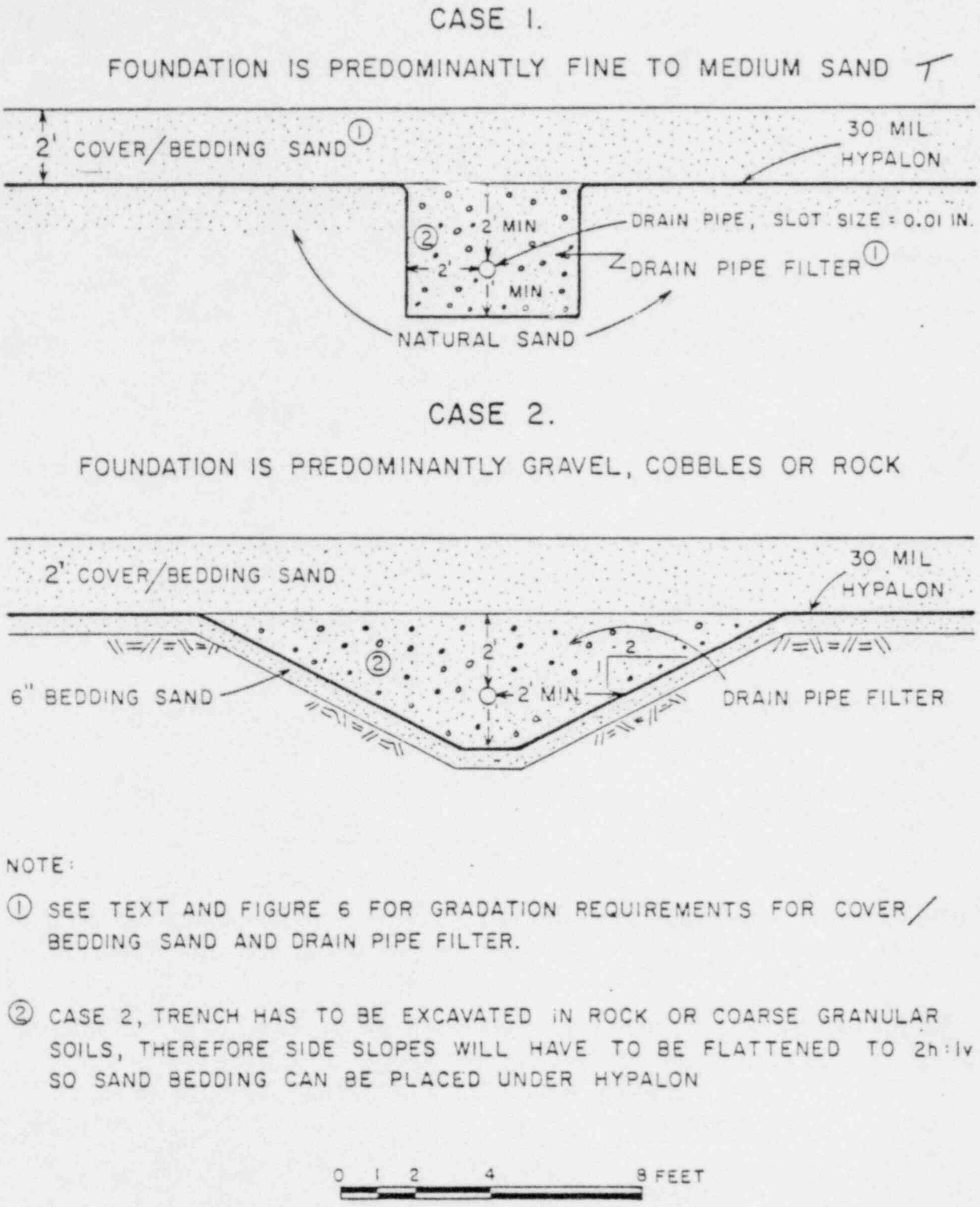
The 5 foot high perimeter dike will be constructed from the near surface coarse gravel sand and cobble material at the site and will also be lined with Hypalon and riprapped (Ref. 2). The Golder Report recommends that materials be compacted to at least 95% of the maximum density as specified by the modified compaction test, ASTM D1557. They recommend that the soil be placed in lifts of 12 inches or less, loose thickness. Slopes of all constructed dikes will be no steeper than 3h:1v.

All materials out of which the embankment pit will be excavated consist of natural granular materials of glacio-fluvial origin.

3.4 Seepage Control System

To minimize seepage from the impoundment, a 30 mil Hypalon liner will be placed on the bottom and along the sides of the pit. Details of placement of the liner are shown in Fig. 3.4. Six inches of bedding sand will be placed underneath the liner to minimize punctures from underlying gravel and cobbles. Two feet of sand will be placed on top of the liner for protection. Along the upper edges of the impoundment two feet of riprap will be placed to provide slope protection for the bedding sand from wave action.

The layout of the subdrains is shown in Fig. 3.5. Cross-sections of the drains are shown in Fig. 3.6. The drains consist of slotted drain pipe surrounded by filter material and covered by the cover sand protecting the liner. Calculations presented in the Geotechnical Design Report by Golder Associates (Ref. 2) indicate that the drains will



NOTE:

① SEE TEXT AND FIGURE 6 FOR GRADATION REQUIREMENTS FOR COVER/BEDDING SAND AND DRAIN PIPE FILTER.

② CASE 2, TRENCH HAS TO BE EXCAVATED IN ROCK OR COARSE GRANULAR SOILS, THEREFORE SIDE SLOPES WILL HAVE TO BE FLATTENED TO 2h:1v SO SAND BEDDING CAN BE PLACED UNDER HYPALON

Figure 3.6. Drainpipe Installation Details (Source: Reference 2)

be capable of carrying seepage in excess of the full quantity of water delivered to the tailings through the slurry pipeline.

4. EVALUATION OF PROPOSED TAILINGS MANAGEMENT PLAN

4.1 Slope Stability

4.1.1 Soil Shear Strength Parameters

Because the materials through which the tailings disposal pit will be excavated are non-cohesive granular soils, the shear strengths were established by correlation with standard penetration test data as presented in Lambe and Whitman, 1969. Considering the Standard Penetration Test data presented in the boring logs for the three bore holes advanced by Golder Associates (Ref. 2) the lowest blow count observed in the upper 70 feet was 9 blows per foot with many values greater than 20 blows per foot. The average of all Standard Penetration results was 20 blows per foot.

A correlation between friction angle and penetration resistance as presented in Peck, Hansen, and Thornburn (Ref. 8) indicates that for a blow count of 20 an angle of internal friction of approximately 33 degrees would be appropriate. Golder and Associates assumed an angle of internal friction of 32 degrees which corresponds with a blow count of approximately 16 blows per foot.

However, of the 26 Standard Penetration Test results, a total of 9 penetration tests indicated blow counts lower than 16. Furthermore, there is no indication in the Golder Report (Ref. 2) that corrections were made to the Standard Penetration Test results for the effects of overburden pressure (see Ref. 8).

If the blow counts are corrected according to procedures indicated in Peck, Hanson, and Thornburn (Ref. 8) the average blow count for all values observed is 15. These corrections have been made on the boring logs and are included in Appendix A.

Some variation is seen to exist from one hole to the other. For example, the average of all blow counts in Borehole 1 is 21, whereas the average blow count for all values observed in Borehole 2 is only 9, and the average blow count observed in Borehole 3 is only 13.

The angle of internal friction corresponding to Standard Penetration Test resistances of 9 or 13 is less than 30 degrees as indicated in Peck, Hanson and Thornburn (Ref. 8).

The values of blow count indicated by the correlation given in Peck, Hanson, and Thornburn (Ref. 8) have been shown to be conservative on the basis of experience by many investigators in the field. Consequently, it is possible that Golder Associates may have some data to indicate that a value of 32 degrees is realistic. However, this is not borne out by correlation of the corrected blow counts. Consequently, it must be assumed at this point that the angle of internal friction of 32 degrees used in the stability analysis is unconservative and a lower value should be utilized unless the results of shear strength testing indicate otherwise.

4.1.2 Stability Analyses

Slope stability analyses were conducted by the reviewers utilizing computer program STABL2. A total of ten potential failure surfaces were generated and the lowest factor of safety observed for those trial failure surfaces was 1.95. All other values were greater than 2. Results of these computations and the cross-section utilized is shown in Appendix A. The shear strength parameters used in these analyses were the same as those used by Golder Associates (Ref. 2).

The phreatic surface utilized in the stability analyses was the same as that used by Golder Associates. As discussed in the previous section describing the groundwater conditions at the site, this phreatic surface is considered to be realistic and even under fluctuations due to heavy rains it is not expected that the phreatic surface would rise above the bottom of the pit. The use of more realistic shear strength values could cause these values of factor of safety to be somewhat lower.

4.1.3 General Recommendations Concerning Slope Stability

In general, the computed factors of safety for the stability of the pit walls is greater than a value of 1.5 required by USNRC Regulatory Guide 3.11. However, the shear strength values that were utilized in arriving at these factors of safety were unconservative and did not appear to have been corrected for overburden stress. Consequently, it is recommended that corrected values of blow count be utilized to arrive at more correct values of angle of internal friction. Alternatively, the results of shear strength testing such as direct shear tests could be used to indicate the reasonableness of the value that actually was utilized.

4.2 Settlement

In the request for additional information and as addressed in the responses to NRC questions dated July, 1980 (Ref. 4), it was requested that the applicant conduct settlement analyses to indicate the potential for differential settlement that could result due to differences in the subsoil conditions from one end of the pit to the other. One end is underlain by basalt whereas the remainder of the disposal area is underlain by a stiff clay. The analyses that were conducted and presented in Reference 4 take into account settlement of the sand only.

These settlement computations were checked by the reviewers using approximate relationships. The computations are shown in Appendix B.

The results indicated in Appendix B compare favorably with the computed results indicated by Golder Associates in Reference 4. However, none of the computations presented by the applicant take into account the deep layer of glacial clays underlying the site. Because one end of the site is underlain by basalt with little or no clay present there, the total settlement of the clay could manifest itself as differential settlement across the site.

Considering the difference between the unit weight of the tailings and that of the existing sand (i.e., 108.5 - 97.4 pcf = 11 pcf), the total excess load that will be applied is approximately 11 x 70 = 0.35 tons per square foot. If the clay is highly overconsolidated, it is expected that this low additional loading would cause little or no settlement. However, there is no data presented to indicate what the compressibility of the clay is. It is recommended, therefore, that additional laboratory or field data be presented to show that, in fact, the compressibility of the clay is low enough to cause elastic and plastic settlement of the large thickness of clay to be negligible.

4.3 Liquefaction Potential

On page 7 of Reference 2, it is stated that because the water table is low, because the fine sands are relatively dense, and because the earthquake potential at the site is low, liquefaction is considered unlikely at this site. Generally, liquefaction is of concern only for loose soils when the water table is near the surface. Consequently, because of the very low water table and the fact that overlying soils are coarse grained, the reviewers agree that the liquefaction potential at the site is low. This is believed to be true in spite of the fact that many corrected blow counts in the fine sand are as low as 6 or 7 indicating a loose soil.

4.4 Slope Protection

In Fig. 4 of the Golder Report (Ref. 2) and as shown in Fig. 3.4 of this report, riprap slope protection will be provided at the top edge of the pit. In response to the NRC question No. 4 (Ref. 4) it is stated that this riprap and Hypalon liner will be extended up onto the dikes. However, Fig. 3.4 does not indicate that the riprap on the liner will extend beyond the top of the pit. This point should be made clear in the specifications if, in fact, the liner and riprap is to extend onto the face of the dikes.

In Reference 2 it is stated that basalt from the disposal pit in combination with the coarse-grained, near-surface materials of the site may be used for riprap. The only specifications regarding riprap size is that cobbles, boulders, or rock fragments up to 12 to 18" in diameter would be satisfactory for use in the riprap. Sherard et al. (Ref. 7, Table 8.1:1) indicates that for wave heights of zero to two feet, the minimum average rock size (D_{50}) should be 10 inches. Furthermore, it should be provided that filter criteria is met between the riprap and the sand bedding material. It is recommended that more detailed specifications for the riprap be provided that indicate a minimum average grain size (D_{50}) of 10 inches, that the riprap be well graded, and that filter criteria should be met between the sand bedding material and the riprap.

4.5 Seepage Control System

4.5.1 Integrity of the Liner

In the response to NRC question No. 1 (Ref. 4) shear stresses acting on the liner were computed to be 15.5 psi. It is believed that an error exists in the computation shown in the response to question No. 1. In Appendix C, computations of shear stresses are provided which indicate that the shear stresses should, in actuality, be 5 psi. Nevertheless, the shear stress computed in Appendix C is lower than that reported in Ref. 4 and the factor of safety would therefore be even greater than that indicated in Ref. 4.

4.5.2 Underdrain System and Filter Criteria

The underdrain system shown in Fig. 3.6 indicates various zones of sand and bedding material and tailings in contact with one another. Computations to check filter criteria are shown in Appendix C. The grain size distribution curves shown in Ref. 2 were stated to have been designed on the basis of Corps of Engineer criteria, whereas the criteria shown in Appendix C were taken from the Pit Slope Manual (Ref. 10). It is shown that for both the bedding and the drain pipe filter rules 2 and 4 have not been met. It is recommended, therefore, that the design gradation shown in the specifications for the bedding and filter material be revised so as to ensure conformance with rules 2 and 4.

4.6 Hydrologic Considerations

4.6.1 Water Balance Analysis

A water balance analysis was conducted to insure that the applicant's tailings pond management scheme will function as proposed. In performing this analysis, the assumptions that were made are as follows:

- Annual precipitation is approximately 20 inches per annum
- The total evaporation rate is 38 inches per year (Ref. 1). Monthly evaporation rates are depicted in Section III, Appendix D.
- Seepage will not occur out of the pit (Ref. 1)
- The slurry inflow into the pond is 0.95 acre-feet per day, (Ref. 4). It was assumed the mill will operate 365 days per year.
- The pond will accept all slurry discharges until filled, after which decanting will be required.
- The maximum operating level of the tailings pond is elevation 1740.

Since the applicant did not provide either a stage-volume curve or a stage-surface area curve to perform the water balance analysis, curves were constructed based on information presented in Reference 4. Both the stage-volume and stage-surface area curves are shown in Appendix D.

The water balance analysis was performed in a conservative manner. The analysis began in January, at a time when pond evaporation would be a minimum. The result depicted in Appendix D indicates that the applicant can discharge slurry into the pond approximately 2.75 years before decanting operations are required. This value agrees with the applicant's analysis presented in Reference 4.

Further analysis of the tailings pond management scheme, in terms of the water inflow-outflow characteristics, cannot be evaluated because the question of how and where decanting will occur has not been addressed. Also, a refined estimate of the percentage of slurry solid would be needed to further refine the water balance analysis. Presently, the applicant reports the solids are from 30-50 percent of the slurry (Ref. 1).

4.6.2 Freeboard

The applicant estimated the PMF to contribute 37 inches (3.08 feet) of water to the 25.5 acre impoundment resulting in a volume of approximately 78.6 acre-feet. The hydrological analysis is presented in Section 2.4. Based upon an approximate impoundment fetch length of 1200 feet and an estimated maximum wind velocity of 100 mph, the maximum wave height and subsequent embankment runup is calculated to be approximately 2 feet (Ref. 6,7).

The maximum pond operating level has been established at elevation 1740 with an impoundment crest elevation 1745. The proposed pond operating freeboard is 5 feet (see Dawn Mining Company drawing 7419-13-002). Combining the PMF depth with the potential wave height results in a required freeboard of 5.08 feet which is necessary to retain any potential overtopping of the embankment crest.

The upper ten feet of the embankment (i.e., elevation 1735 to 1745) will be riprapped to insure embankment protection from wave action as shown in Fig. 3.4. Consequently, wind and wave damage should be minimal. Because of the wave protection and the extreme conservatism with which the PMF was estimated, it is recommended that the proposed 5 feet of freeboard be considered adequate.

4.7 Construction

Recommendations are presented in Ref. 2 for details of the embankment and pit construction and placement of materials. However, no official construction specifications have been drawn up. It is recommended that a set of construction specifications be prepared and submitted for approval prior to startup of construction operations. With regard to the construction specifications noted in Ref. 2, the following comments are offered:

It is recommended in Ref. 2 that the embankments be compacted to at least 95% of the maximum density as specified by the Modified Compaction Test, ASTM D1557. The reviewers believe that 95% of the modified maximum density is not necessary and stability can be achieved with lower densities. To specify such high densities could lead to problems with construction of the embankments and in construction control.

It is also recommended in Ref. 2 that the soil should be placed in loose lifts 12 inches or less in thickness. It is believed that a lift thickness of 12 inches is too great and will not provide for uniform density across the lift. The reviewers recommend that this value should be decreased to approximately 9 inches or less.

It should be specified that quality control and specification compliance tests be submitted to NRC. NRC inspection of the embankment construction is recommended during the following stages of construction:

1. When pit excavation is nearing completion.
2. When foundation treatment for the embankment has been completed and prior to placement of compacted fill. (The specifications should require that all organic matter be stripped from the site prior to construction of dikes around the pit.)
3. At an early stage of perimeter dike construction.
4. During placement of the liner system and when the perimeter dike is nearly completed.
5. At completion of the pit and embankment construction, and after placement of the liner.

5. REVIEW OF LONG-TERM STABILITY OF PROPOSED RECLAMATION PLAN

5.1 Technical Criteria

An evaulation of the long-term stability of the uranium mill tailings disposal plan was performed. The reclmation plan of the proposed disposal site must be incorporated into the entire tailings disposal area currently in operation and cannot be reviewed completely on the basis of existing information. Nevertheless, the following evaluation indicates the adequacy of the proposed plan from the viewpoint of long-term stability. The long-term stability and proposed reclamation plan was evaluated in accordance with the list of failure modes presented in Table 5.1. Because of the subsurface disposal plan proposed, several of the potential failure mechanisms indicated in Table 5.1 have minor or no consequence with regards to the long-term stability.

5.2 Proposed Reclamation and Long-Term Stability Plan

Dawn Mining Company developed the following program for reclamation and long-term stabilization of tailings disposal facilities. The following steps have been proposed to stabilize the tailings pond:

1. Tailings will be allowed to dewater for a period of one to three years to allow heavy equipment to work on the tailings surface. Interim measures (sprinkling or wood chip cover) will be taken to control dusting.
2. The tailings surface will be graded to enhance drainage.
3. A layer of clay two feet thick will be placed and compacted over the tailings surface.
4. An additional layer of fill 8 foot thick consisting of sand and gravel will be placed overlying the clay.
5. No topsoil will be added to this cover since the area surrounding the project site has minimum natural "A" horizon soil development.

Table 5.1 Complete list of failure modes considered in assessment of long-term stability. (Ref. 11)

A. Failure Modes Associated with Impoundment Elements

1. CAP
    - a) Differential settlement
    - b) Gullying
    - c) Water sheet erosion
    - d) Wind erosion
    - e) Flooding
    - f) Chemical attack
    - g) Shrinkage

2. LINERS
    - a) Differential settlement
    - b) Subsidence of subsoil and rock
    - c) Chemical attack
    - d) Physical penetration

3. EMBANKMENT
    - a) Differential settlement
    - b) Slope failure
    - c) Gullying
    - d) Water sheet erosion
    - e) Wind erosion
    - f) Flooding
    - g) Weathering and chemical attack

4. REVEGETATION
    - a) Fire
    - b) Climatic change

5. WATER DIVERSION STRUCTURES
    - a) Slope failure
    - b) Obstruction

B. Failure Modes Associated with Natural Phenomena

1. Earthquakes
2. Floods
3. Windstorms
4. Tornadoes
5. Glaciation
6. Fire and Pestilence

6. The cover over the tailings will be graded and contoured so as to eliminate the possibility of ponding of precipitation over the area. In addition, the slopes of the capped layer will be reduced to a slope of 5h:lv by the addition of fill materials along the periphery.
7. The entire area will be seeded and fertilized to stabilize the cover. It is stated in the EIS (Ref. 1) that natural reforestation will ensue fairly rapidly as evidenced by the trees presently growing on the abandoned tailings berm around the areas presently in operation.
8. Revegetation effort will be monitored for success and remedial measures will be taken to ensure coverage of the area.

5.3 Evaluation of Proposed Reclamation Plan

5.3.1 Failure Modes Associated with Impoundment Elements

5.3.1.1 Cap

The cap will consist of 10 feet of cover. The first 2 feet immediately above the tailings surface will comprise a clay layer which will be placed and compacted. An 8 feet thick layer of sand and gravel will be placed over the clay. The cover over the tailings will be graded and contoured to facilitate drainage away from the impoundment area to reduce or minimize sheet or gully erosion. Cover will be placed at a maximum slope of 5h:lv. Due to the materials being used to comprise the cap, it is anticipated that the effects due to the differential settlement and chemical attack will be minimal. However, over long-term periods it is expected that the clay cover could become dessicated and crack. No consideration should be given, therefore, to radon attenuation by the clay.

5.3.1.2 Liners

The entire disposal area floor and sideslope surface will be lined with a fabric-reinforced, 30 mil synthetic rubber "Hypalon" liner. It is proposed that the liner membrane extend up the dike to a level 5 feet above the original ground surface. A 2 foot thick cover layer of stabilized sand will be placed over the membrane. After the tailings are placed into the facility, the tailings will be dewatered for from one to three years through decanting and evaporative processes. Assuming proper placement of the liner and satisfactory dewatering of the tailings, all free water in the tailings should be removed in a short term period. It is anticipated that differential settlement or other adverse affects to the liner should be minimal. However, if disruption of the liner does occur over long-term periods it is not expected that sufficient water will be available to cause excessive seepage.

5.3.1.3 Embankments

The proposed impoundment is designed to be constructed below the surface of the natural ground level. The embankment crest at elevation 1745 and the top of the reclaimed tailings surface at elevation 1740 are below the minimum natural ground elevation of 1750. In view of the subsurface disposal plan, it is not anticipated that adverse effects due to slope failure or cracking of the embankment would be significant. It is anticipated that after the completion of dewatering and initiation of the cap placement, the embankment will not experience any adverse effects due to water or wind erosion, flooding, or chemical attack.

5.3.1.4 Revegetation

It is proposed that the reclaimed area will be seeded and fertilized to stabilize the cover. However, due to the nature of the sand and gravel materials to be placed in conjunction with the low summer season precipitation it is anticipated that reforestation and revegetation of the cap will be difficult. Furthermore, vegetative development may require maintenance for an extensive period of time. It is suggested, therefore, that an alternative plan to vegetation be considered such as riprap or other more stable means of long-term protection.

5.3.2 Failure Modes Associated with Natural Phenomena

5.3.2.1 Earthquakes

The embankment for the tailings impoundment was designed for a pseudostatic earthquake coefficient of 0.05g. In the low seismic area in which the impoundment is located, the coefficient is considered adequate as discussed previously. The embankment is, therefore, expected to be stable for expected earthquake loading during operation. Also, as discussed previously the liquefaction potential is low. Since the impoundment is constructed below the ground surface and surrounded by safety dikes, the potential for dispersion of the tailings due to earthquake and/or liquefaction is considered to be minimal.

5.3.2.2 Floods

The Dawn Mill site lies within the drainage basin of the Chamokane Creek, the principal surface stream of Walker's Prairie Basin. Although the tailings disposal site is located near the creek, the disposal site has been located upon a terrace and a stockpile dike has been constructed around the entire disposal area. The impoundment has been isolated from the remainder of the watershed and adverse effects due to flooding are not considered significant. Since all the tailings will be disposed of below the natural ground level and covered with a 10 foot cap, adverse effects due to flooding over long-term periods are considered to be minimal. It is anticipated that any runoff from the impoundment area, will be handled by proper contouring of the surface. This point, however, must be considered in conjunction with reclamation of the entire impoundment.

5.3.2.3 Wind Storms, Tornadoes, Glaciation, Fire and Chemicals

The effects of wind storms, tornadoes, glaciation, fire and pestilence were shown to be negligible in the long-term report by Nelson and Shepherd (Ref. 11).

6. RECOMMENDED LICENSE CONDITIONS

The following text outlines items that have not been addressed adequately. It is believed that license conditions should be imposed to provide for resolution of these items.

6.1 Slope Stability

The assumed shear strength values are not consistent with correlations for corrected blow count values. Additional analyses should be conducted to indicate that adequate factors of safety will exist for shear strength values consistent with the lower corrected blow count values. Alternatively, laboratory or field shear strength testing data should be provided to indicate that the value of shear strength parameters are realistic.

6.2 Settlement Analyses

Field or laboratory data should be provided to indicate that the compressibility of the underlying deep deposits of clay are sufficiently low so that the loading imposed by the difference in unit weights between that of the tailings and the in-situ density of the sands (i.e., 108.5 - 97.4 pcf) will not cause excessive differential settlement. If this cannot be demonstrated, the design of the liner system should be revised to preclude rupture of the liner due to differential settlement.

6.3 Slope Protection

Specifications should be provided for the riprap material. The riprap should have a minimum average grain size (D_{50}) of 10 inches, it should be well graded, and it should meet filter criteria with the bedding sand.

6.4 Seepage Control System

The specified grain size distributions for the bedding sand and drain pipe filter should be adjusted so that filter criteria are met. (See Appendix C).

6.5 Decanting Operation

Operational procedures should be defined to indicate how decanting will be accomplished after the initial period of 2.75 years. These procedures should also indicate the method of handling the decant water and disposal thereof.

6.6 Construction

6.6.1 Specifications

Construction specifications should be written and approved prior to construction. In the specifications lift thicknesses should be decreased to 9 inches or less.

6.6.2 Inspections

Inspections by the NRC should be provided for at the intervals noted in Section 4.6 of this report. The NRC should be notified at least six weeks prior to each construction feature to provide adequate time for on-site inspections in accordance with those intervals.

6.6.3 Reports

Contractors should submit to NRC within six months after completion of each stage of construction, as-built drawings showing construction details of the liner system, embankment, foundation and subsoil profiles prior to embankment construction, and a construction report summarizing the following:

1. Compaction control test results
2. Classification of all soils used in the embankment
3. Construction equipment and procedure
4. Unexpected conditions and problems encountered in construction, and methods employed to resolve these problems.

REFERENCES

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2. Golder Associates (1979), "Geotechnical Design Data, Proposed Tailings Disposal Facility Expansion, Dawn Mining Company, Ford, Washington.

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6. Creager, W. P., Justin, J. D., and Hinds, J. (1945), Engineering for Dams, John Wiley and Sons, New York.

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10. CANMET (1977), "Pit Slope Manual," Energy, Mines and Resources, Canada, Ottawa, Canada.

11. Nelson, John D., and Shepherd, Thomas A., (1978), "Evaluation of Long-Term Stability of Uranium Mill Tailings Disposal Alternatives," for Argonne National Laboratories, Contract No. 31-109-38-4199.

APPENDIX A

Stability Computations

APPENDIX A

Part I - Boring Logs Showing Corrected Values of Blow Counts

Sheet 1 of 3

RECORD OF BOREHOLE 1

Figure A-

LOCATION See Figure A-1
DATUM DMC A-2
DATE 9/18 to 9/19-79
BORING METHOD Hollow Auger
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	$N_{corr.}$
0	Loose brown fine to coarse gravel and cobbles with fine to coarse sand and trace silt (GP)					1741						
			☒	SPT	12							17
5	Stiff brown clay (CL)											
	Compact brown clayey fine sand and silt (SC)											
10			■	SS	24		3.9	82				
	Dense brown fine to coarse sand and gravel with some silt (SP/GP)		☒	SPT	34		3.0					37
15												
20			☒	SS	39		5.9			.010		
25			☒	SPT	34		3.2					31
30			■	SS	97		2.7	116				
	Grading thin clay/silt layers from 30 ft. to 40 ft.											

REMARKS
SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5 FT

Golder Associates

DRAWN _____
CHECKED REV ___

Sheet 2 of 3

RECORD OF BOREHOLE 1

A-3

Figure A-2B

LOCATION See Figure A-1

DATUM DMC

DATE 9/18 to 9/19 -79

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN

BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	Ncor.
35			☒	SPT	47	1706	2.0					40
40			■	SS	93				RAPID			
	Compact to dense brown fine to medium sand with trace silt (SP)											
45			☒	SPT	34		7.8					29
50			■	SS	41							
55			☒	SPT	21		7.6					16
60			■	SS	43		7.9	84	.0004			
65	Grading with clayey silty layers at 65 ft.		☒	SPT	10		2.4			.0004		6

REMARKS SPT = Standard Penetration Test and Jar Sample

SS = 3" Split Spoon with Brass Sleeves

☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE 1 IN TO 5 FT

Golder Associates

DRAWN ______

CHECKED REV

Sheet 3 of 3

RECORD OF BOREHOLE 1

Figure A-2C

LOCATION See Figure A-1 A-4
DATUM DMC
DATE 9/18 to 9/19-79
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN
BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	Ncor;
70			■	SS	60	1671						
75	Grading clean fine sand		☒	SPT	23		8.9					13
80			■	SS	72		8.2	89				
									RAPID			
85			☒	SPT	35		8.4					18
90			☒	SS	34		7.8	89				
											▼ 93'4" 10-17	
95			☒	SPT	19		24.2					9
									.0004			
100	Boring completed 98'8" 9-19-78											

Navg = 21
0

REMARKS SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5FT

Golder Associates

DRAWN _ _ _ _ _ _
CHECKED REV

Sheet 1 of 3

RECORD OF BOREHOLE 2

A-5

Figure A-2D

LOCATION See Figure A-1
DATUM DMC
DATE 9-21-79
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.
BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	
0	Loose brown fine to coarse gravel and cobbles with some fine to medium sand, trace silt (GP)					1745						
5												
10												
15												
20	Dense brown fine to coarse sand and gravel with trace silt. (SP/GP)		☒	SS	33		2.4			.02		
25			☐	SPT	12							//
30			■	SS	28		4.3	100				
	Compact to dense brown fine to medium sand (SP)											

REMARKS
SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeve
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5FT

Golder Associates

DRAWN ______
CHECKED REV ___

Sheet 2 of 3

RECORD OF BOREHOLE 2

Figure A-2E

LOCATION See Figure A-1

DATUM DMC A-6

DATE 9-21-79

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN

BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	
35	(Cont'd)		☒	SPT	18		4.8			.032		15
	Compact to dense brown fine to medium sand (SP)											
40			■	SS	38		4.9	101				
										.009		
45			☒	SPT	9		7.1			.017		7
50			■	SS	15							
55			☒	SPT	12		8.5					8
60			■	SS	20		6.1	93				
										.003		
65			☒	SPT	14		7.7					8

REMARKS SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
= No Recovery = Disturbed Sample = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5 FT

Golder Associates

DRAWN
CHECKED REV

Sheet 3 of 3

RECORD OF BOREHOLE 2
A-7

Figure A-2F

LOCATION See Figure A-1
DATUM DMC
DATE 9-21-79
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN
BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	
70			■	SS	23	1675	13.6					
75	Grading with thin layers of clay/silt from 75'		☒	SPT	11							6
80			☐	SS	35							
	Talus Material, large boulders (Basalt?)								.0004			
85	Boring completed 84 ft. 9-21-79 Groundwater not encountered.											$N_{avg} = 9$

REMARKS SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5FT

Golder Associates

DRAWN _ _ _ _ _
CHECKED REV

Sheet 1 of 3

RECORD OF BOREHOLE 3
A-8

Figure A-2G

LOCATION See Figure A-1
DATUM DMC
DATE 9-24-79/9-25-79
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.
BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	Ncor
0	Loose brown cobbles, fine to coarse gravel with some fine to coarse sand and trace silt (GP)					1741						
5												
10												
15			☐	SPT	25		3.2					27
20			☒	SS	55		3.2					
25			☐	SPT	29		2.1					26
30			■	SS	32		7.5	104				
	Dense brown fine to coarse sand, fine to coarse gravel layers & trace silt (SP-GP)											

REMARKS SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5 FT

Golder Associates

DRAWN
CHECKED REV

Sheet 2 of 3

RECORD OF BOREHOLE 3 A-9

Figure A-2H

LOCATION See Figure A-1 DATUM DMC DATE 9-24-79/9-25-79

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	N cor.
35			☒	SPT	18	1706	4.5					15
40			■	SS	33		6.2	100				
									RAPID			
45			☒	SPT	19		6.7					14
50			■	SS	26		12.0	98				
	Compact to dense brown fine to medium sand (SP) (Thin brown clay layers present at 61' and 80'-85')											
55			☒	SPT	13		3.6					8
60			☒	SS	34		25.3					
65			☒	SPT	13		5.1					8
									.003			

REMARKS SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

VERTICAL SCALE
1 IN. TO 5 FT

Golder Associates

DRAWN ______
CHECKED REV

Sheet 3 of 3

RECORD OF BOREHOLE 3
A-10

Figure A-21

LOCATION See Figure A-1
DATUM DMC
DATE 9-24-79/9-25-79
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN
BORING METHOD Hollow Auger

SOIL PROFILE			SAMPLES									
DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	FIELD PERM. (CM/SEC)	PERM. D_{10}^2 (CM/SEC)	WATER LEVEL	N_{cor}
						1671						
70	(Cont'd) Compact to dense, brown fine to medium sand (SP) (Thin brown clay layers present at 61' and 80'-85')		■	SS	31							
75			☒	SPT	12		9.3					7
80			■	SS	27							
85			☒	SPT	13		4.8					7
90			■	SS	36							
									.0025			
95			☒	SPT	14		9.3					7
100	Groundwater not encountered.		☒	SPT	15							7
	Boring completed 101½' 9-25-79											$N_{ave} \simeq$ ⑬

REMARKS
SPT = Standard Penetration Test and Jar Sample
SS = 3" Split Spoon with Brass Sleeves
☐ = No Recovery ☒ = Disturbed Sample ■ = Relatively Undisturbed Sample

OVERALL $N_{ave} = 15$

VERTICAL SCALE
1 IN. TO 5 FT

Golder Associates

DRAWN
CHECKED REV

4

APPENDIX A

Part II - Slope Stability Analysis

DAWN MINING COMPANY

This analysis utilized the program STABL2. A total of 10 potential failure surfaces were generated from each of three initiation points. The three initiation points included: 1) the toe of the proposed pit, 2) the crest of the proposed pit, and 3) a point midway between these two points on the pit slope. The left termination point was taken as the crest of the proposed facility and the right termination point as a point on the existing tailing surface. Minimum elevation of surface development was taken as 75', a distance of 25' below the bottom of the proposed pit. Length of segments defining the surfaces was taken as 15'. Limits for surface initiation angles were randomly chosen by the computer.

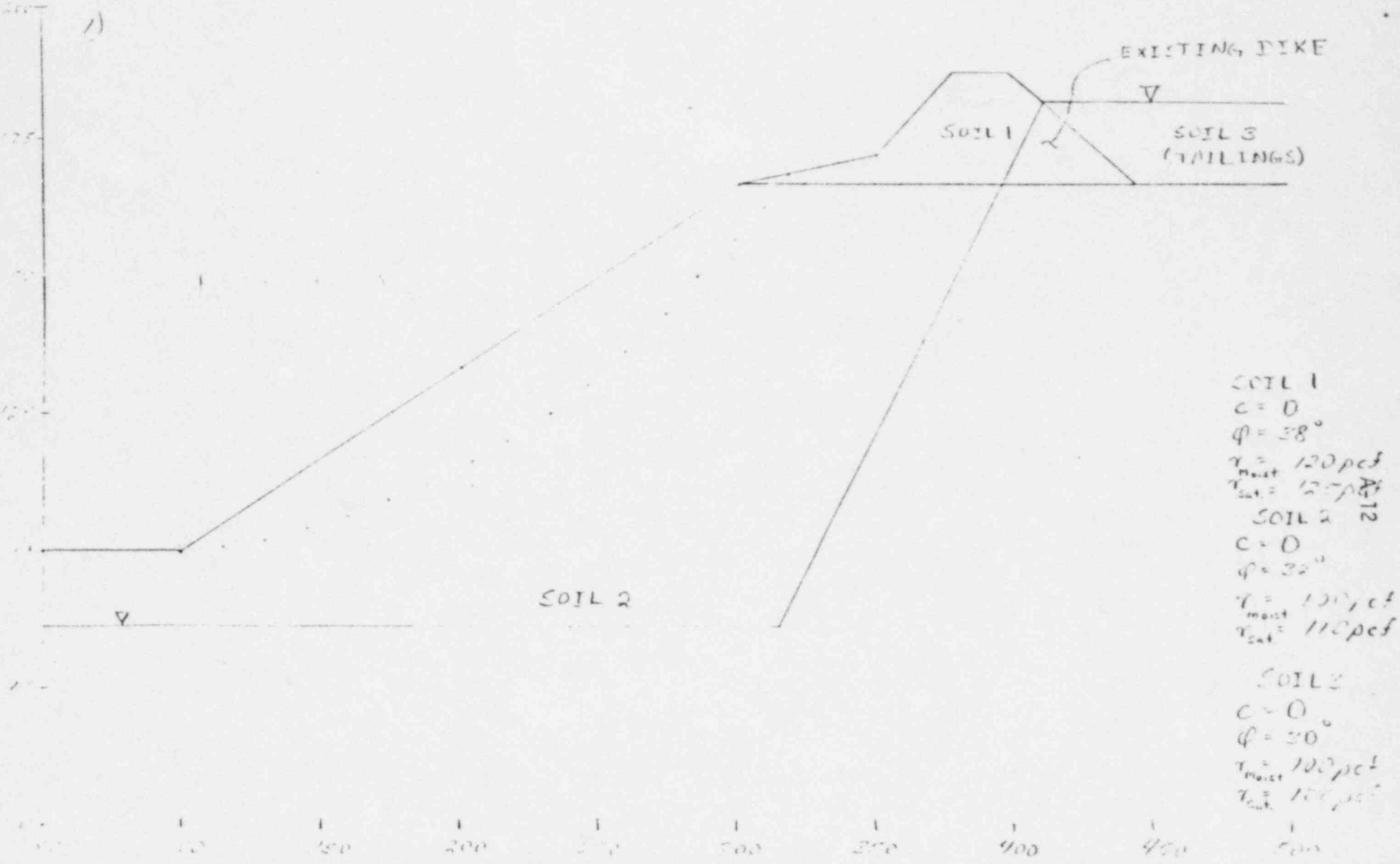
Factors of Safety (for the ten most critical of the trial failure surfaces examined)

1. 1.950
2. 2.016
3. 2.102
4. 2.166
5. 2.168
6. 2.207
7. 2.211
8. 2.215
9. 2.217
10. 2.245*

*Note: Initiation point coordinates: x = 200.5, y = 133.5 (located on pit slope) F.S. = 2.25.

Initiation point coordinates for the remaining 9 failure surfaces: x = 100.0, y = 100.0 (located at the toe of the proposed pit) $F.S._{avg}$ = 2.14.

1)



SOIL 1
c = 0
φ = 38°
γ_{moist} = 120 pcf
$\gamma_{sat.}$ = 125 pcf

A-12

SOIL 2
c = 0
φ = 32°
γ_{moist} = 100 pcf
$\gamma_{sat.}$ = 110 pcf

SOIL 3
c = 0
φ = 30°
γ_{moist} = 100 pcf
$\gamma_{sat.}$ = [illegible] pcf

DAWN MINING COMPANY Scale 1" = 50'

APPENDIX B

Settlement Computations

DAWN $_{B-1}$

JDN
18 Aug 80

SETTLEMENT COMPUTATIONS

$$\rho = \frac{\Delta q_s \times 12}{N}\left(\frac{B}{B+1}\right)^2 \quad - \quad \text{Eq. 14.21}$$

LAMBE & WHITMAN

IF B LARGE

$$\rho \approx \frac{12\Delta q_s}{N}$$

$$\Delta\gamma' = 108 - 97 \approx 10 \text{ pcf.}$$

$$\Delta q_s = 10 \times 70 = 700 \text{ psf} = 0.35 \text{ T/ft}^2$$

$$\rho \approx \frac{12 \times .35}{\underset{\nwarrow \underline{N_{corr}}}{7}} = \underline{\underline{0.6''}} \quad - \text{OK.}$$

HOWEVER, NO DATA ON COMPRESSIBILITY OF CLAY

POOR ORIGINAL

APPENDIX C

Filter Criteria and Shear Stress Imposed on Liner

Filter Criteria Check (D&N Mining)

Design Info. - Pit Slope Manual (Energy, Mines, and Resources Canada)
Chapter 9 Waste Embankments (p. 55-57)

Tailings - Bedding Sand Interface

F - Bedding Sand
B - Tailings

Rule #1

$\frac{D_{15F}}{D_{85B}} < 5$ $\frac{.4}{.24} = 1.7$ OK

Rule #2

$\frac{D_{15F}}{D_{15B}} > 5 \text{ and } < 20$ $\frac{.09}{.004} = 50$ not ok

↳ 40 (Bureau)

Rule #3

$\frac{D_{50F}}{D_{50B}} < 25$ $\frac{1.0}{.044} =$ [illegible]   OK

Rule #4

$\frac{D_{85F}}{D_{15F}} < 5$ $\frac{2.4}{.088} = 27.2$ not ok

COARSE
FINE

Rule #5

filter contains < 5% passing #200   OK

Rule 6 & 7 not applicable

Bedding Sand - Drain Pipe Filter Interface

F - Drain Pipe Filter
B - Bedding Sand

Rule #1 $\frac{2.5}{.43} = 5.8$ marginal

Rule #2 $\frac{.21}{.14} = .7$ not OK

Rule #3 $\frac{2.5}{.12} = 20.0$ marginal

Rule #4 $\frac{2.5}{.2} = 22.?$ not OK

Rule #5   filter contains < 5% passing #200   OK

Rule #6 $\frac{D_{85F}}{\text{Maximum Opening of Pipe Drain}} \geq 2$ $\frac{2.4\text{mm}}{.254\text{mm}} = 9.4$ OK

FINE

Rule #7 not applicable

POOR ORIGINAL

DAWN PROJECT. JDN
C-2 18 AUG 80

COMPUTATION OF SHEAR STRESS ON LINER

$T = W \sin\theta$ NEGLECTING SIDE FORCES

$$\tau = \frac{T}{b/\sin\theta} = \frac{T\sin\theta}{b}$$

$$\tau = \frac{W\sin^2\theta}{b} = \frac{\gamma_t h\, b\, \sin^2\theta}{b}$$

$$= \gamma_t h \sin^2\theta$$

$$\tau = \frac{108.5 \times 65}{144} \sin^2 18.4°$$

$$= \underline{\underline{5\ psi}}.$$

POOR ORIGINAL

APPENDIX D

Hydrologic Computations

D-1

P M F 72 [illegible] [illegible] Storm 1 sq mile = 640 acres

1 mile = 5280 ft

General maximum 6-hour point rainfall

([illegible] Storm West of the 105 meridian Zone B) = 3.5"

SPOKANE

For extending the 6-hr local storm value to a period of 36 hrs multiply

3.5 times the [illegible]

1. a. A = 25.5 acres

A = 0.040 square miles

b. Length of longest water course

L ≅ 1200' = 0.23 miles

c. Difference in elevation from the highest point in the basin

to the basin outlet

Δh = 1745' - 1370' = 15'

2. [illegible] Storm [illegible] -- PMP

3. Six hour local storm extended to a period of 36 hr

Zone B, West of 105° meridian

I = 2 [illegible] increments

Rainfall = 3.5" (for area reduction [illegible]

4. General site conditions

a. Antecedent moisture condition AMC = II

b. [illegible] soil [illegible] II

c. [illegible] = 95

5. Triangular Hydrograph Information

a. $L = 0.23$ miles

$\Delta h = 75'$

b. T_c = Time of concentration

$$T_c = \left[\frac{11.9\, L^3}{H} \right]^{0.385} = 0.090 \text{ hr.}$$

No correction is needed for T_c because CN is > 80

$D = 2$ hrs.

$$T_p = \frac{D}{2} + 0.6\, T_c = 1.05 \text{ hrs.}$$

$T_b = 2.67\, T_p = 2.80$ hr

$T_r = T_b - T_p = 2.80 - 1.05 = 1.75$ hr

$$q_i = \frac{484\, A\, Q}{T_p}$$

For $Q = 1''$ of rainfall

$$q_i = \frac{484 \times 0.040 \times 1}{1.05} = 18.44 \text{ cfs}$$

$q_i \cong 19$ cfs

6 hr - General Storm - P.M.P (extended to 36 hr)

1	2	3	4	5	6	7
Duration (hrs)	% 6 hr Rainfall	Cumulative Rainfall (in)	Incremental Rainfall (in)	Design Rainfall (in)	Adj Design Rainfall (in)	P.M.P Cum. Des Rainfall (in)
0	-	-	-	-	-	-
2	0.48	1.68	1.68	0.24	0.24	0.24
4	0.77	2.70	1.02	0.28	0.28	0.52
6	1.00	3.50	0.80	0.28	0.28	0.80
8	1.18	4.13	0.63	0.35	0.35	1.15
10	1.36	4.76	0.63	0.39	0.39	1.54
12	1.53	5.36	0.60	0.45	0.45	1.99
14	1.66	5.81	0.45	0.60	0.60	2.59
16	1.77	6.20	0.39	0.63	0.63	3.22
18	1.87	6.55	0.35	0.80	0.80	4.02
20	1.95	6.83	0.28	1.68	1.68	5.70
22	2.03	7.11	0.28	1.02	1.02	6.72
24	2.10	7.35	0.24	0.63	0.63	7.35
26	2.16	7.56	0.21	0.21	0.21	7.56
28	2.22	7.77	0.21	0.21	0.21	7.77
30	2.28	7.98	0.21	0.21	0.21	7.98
32	2.32	8.12	0.14	0.14	0.14	8.12
34	2.35	8.23	0.11	0.11	0.11	8.23
36	2.38	8.33	0.10	0.10	0.10	8.33

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[illegible]

[illegible]

1	8	9	10	11	12	13	14
Durations (hrs)	Q Cumulative Runoff (in)	Increm. Runoff (in)	q_i (cfs)	q_p (cfs)	Begin Hydrograph, hrs	Peak Hydrograph hrs	End Hydrograph hrs
0	–	-	-	-	-	-	-
2	0.04	0.04	19	0.76	0	1.05	2.80
4	0.19	0.15	19	2.85	2	3.05	4.80
6	0.40	0.21	19	3.99	4	5.05	6.80
8	0.70	0.30	19	5.70	6	7.05	8.80
10	1.05	0.35	19	6.65	8	9.05	10.80
12	1.49	0.44	19	8.36	10	11.05	12.80
14	2.08	0.59	19	11.21	12	13.05	14.80
16	2.66	0.58	19	11.02	14	15.05	16.80
18	3.45	0.79	19	15.01	16	17.05	18.80
20	5.12	1.67	19	31.73	18	19.05	20.80
22	6.13	1.01	19	19.19	20	21.05	22.80
24	6.75	0.62	19	11.78	22	23.05	24.80
26	6.95	0.20	19	3.80	24	25.05	26.80
28	7.18	0.23	19	4.37	26	27.05	28.80
30	7.40	0.22	19	4.18	28	29.05	30.80
32	7.53	0.13	19	2.47	30	31.05	32.80
34	7.64	0.11	19	2.09	32	33.05	34.80
36	7.73	0.09	19	1.71	34	35.05	36.80

II. a) Elevation - area Curve

From Appendix 2 - Dawn Mining Company Report

— Top Surface Area = 1'055.000 ft^2 = 24.22 Acr Elev. = 1740 ft.

Bottom Surf. Area = 366.300 ft^2 = 8.41 Acr Elev = 1675 ft.

See Figure No 1

b) Elevation - Volume Curve

Elevation (ft)	Surface Area (Acres)	Average Volume (Acre-ft)
1675	8.41	0
1680	9.00	43.53
1685	9.55	89.91
1690	10.20	139.29
1695	10.80	191.79
1700	11.60	247.79
1705	12.30	307.54
1710	13.15	371.17
1715	14.20	434.80
1720	15.40	508.80
1725	17.00	589.80
1730	18.80	679.30
1735	21.30	779.55
1740	24.22	893.35

See Figure No 2

POOR ORIGINAL

Elevation vs Surface Area

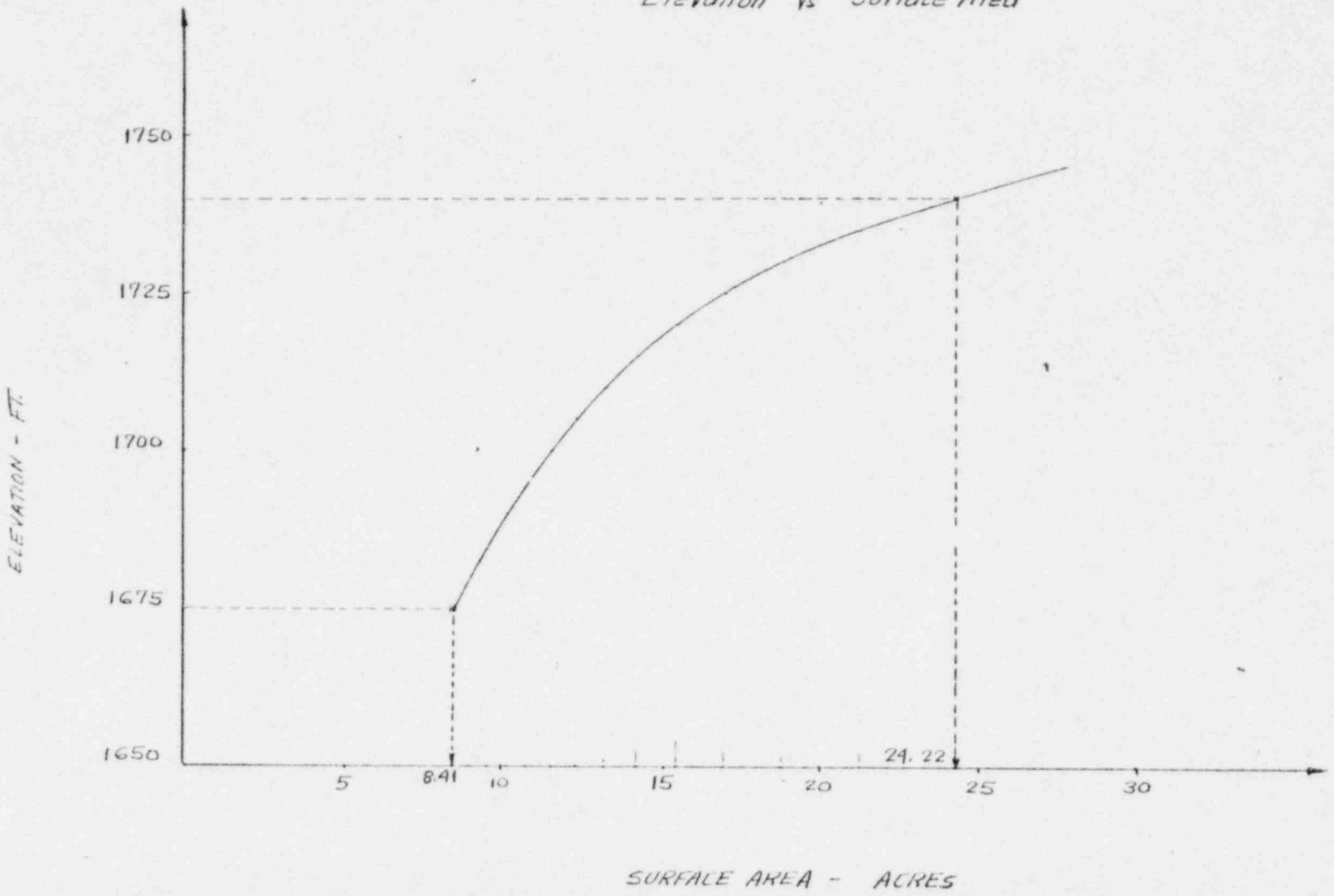

FIGURE No 1

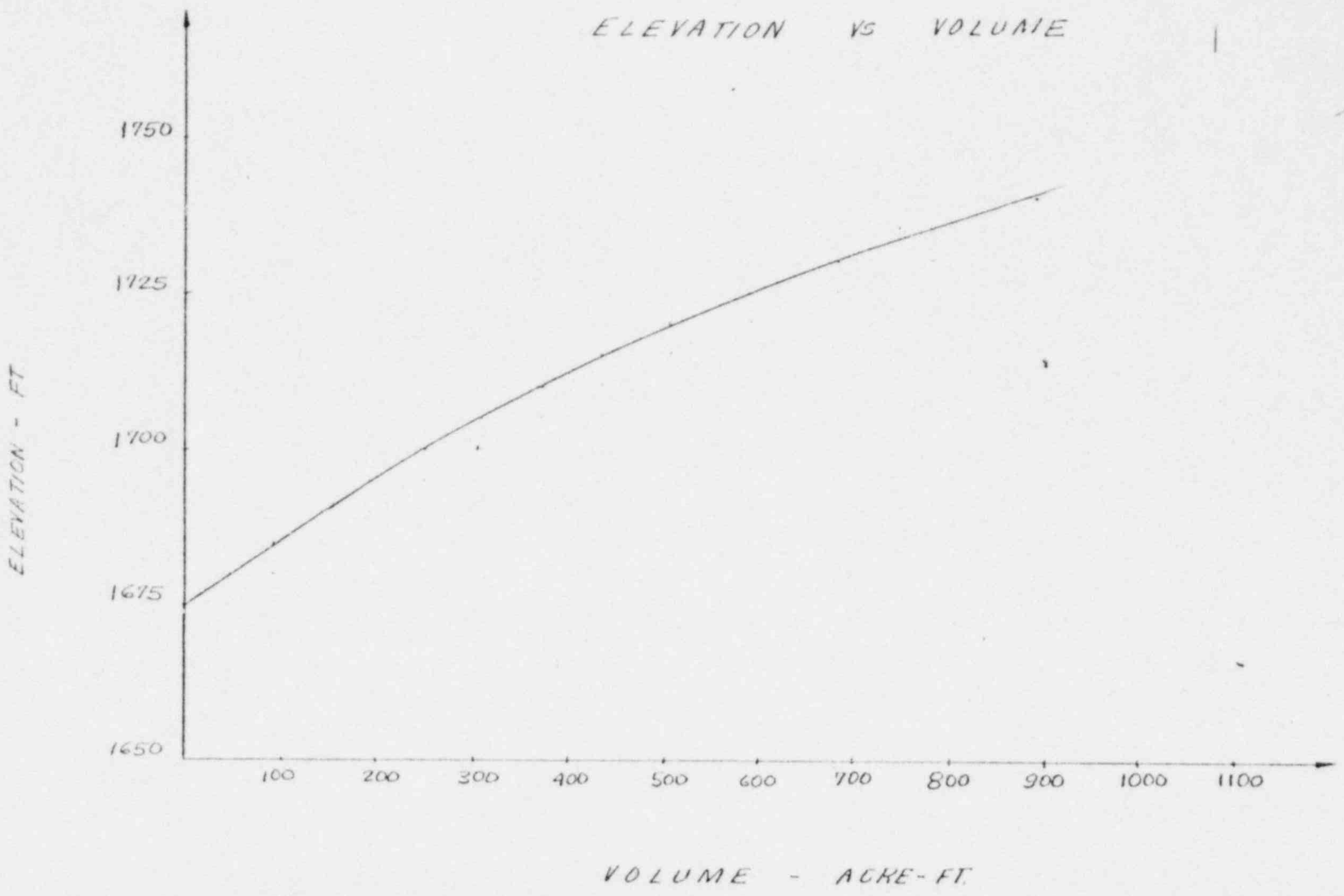
ELEVATION VS VOLUME
ELEVATION - FT.
1750
1725
1700
1675
1650
100
200
300
400
500
600
700
800
900
1000
1100
VOLUME - ACRE-FT.

FIGURE No 2

III. Inflow - Outflow Water Balance

Normal tailings inflow = 0.95 acre-feet / day
Mean annual lake evaporation = 38" / year

	Jan	Feb	Mar	Apr	Mav	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Avg. T°F)	26.8	31.7	39.0	48.0	56.1	62.5	70.6	68.5	60.3	49.0	36.1	30.8
Av. Evap (in)	–	0.15"	1.44"	3.04"	4.48"	5.62"	7.07"	6.69"	5.24"	3.23"	0.95"	–
Av. Precip (in)	2.10"	1.65"	1.54"	0.97"	1.36"	1.29"	0.39"	0.54"	0.87"	1.59"	1.95"	2.25"
Tailings inf (Acr.ft)	29.45	26.6	29.45	28.5	29.45	28.5	29.45	29.45	28.5	29.45	28.5	29.45

		T. Inflow Acr. ft	Precip. Acre-ft	Evap. Acre-ft	Outflow Acre-ft.
1981	JAN	29.45	1.54	–	29.45
	FEB	56.05	1.26	0.11	57.20
	MAR	86.65	1.22	1.14	86.73
	APR	115.23	0.80	2.50	113.53
	MAY	142.98	1.16	3.82	140.32
	JUN	168.82	1.13	4.94	165.01
	JUL	194.46	0.35	6.39	188.42
	AUG	217.87	0.50	6.23	212.14
	SEP	240.64	0.83	5.02	236.45
	OCT	265.90	1.57	3.18	264.29
	NOV	292.79	1.97	0.96	293.80
	DEC	323.25	2.35	–	325.60

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Acre-ft

		T. inflow	Precip	Evap	Outflow
1982	JAN	355.05	2.26	–	357.31
	FEB	383.91	1.84	0.17	385.58
	MAR	415.03	1.78	1.66	415.15
	APR	443.65	1.16	3.63	441.18
	MAY	470.63	1.68	5.52	466.79
	JUN	495.29	1.63	7.11	489.81
	JUL	519.26	0.51	9.19	510.58
	AUG	540.03	0.72	8.93	531.82
	SEP	560.32	1.19	7.17	554.34
	OCT	583.79	2.24	4.54	581.49
	NOV	609.99	2.83	1.38	611.44
	DEC	640.89	3.38	–	644.27
1983	JAN	673.72	3.27	–	676.99
	FEB	703.59	2.67	0.24	706.02
	MAR	735.47	2.59	2.42	735.64
	APR	764.14	1.69	5.30	760.53
	MAY	789.98	2.44	8.05	784.37
	JUN	812.87	2.38	10.38	804.87
	JUL	834.32	0.74	13.38	821.68
	AUG	851.13	1.04	12.90	839.27
	SEP	867.77	1.71	10.29	859.19
	OCT	888.64	3.19	6.49	885.34 ✓ o.k.