MONTANORE PROJECT
SANDERS AND LINCOLN COUNTIES, MONTANA
INTERIM TAILINGS IMPOUNDMENT
ENGINEERING REPORT

Noranda Minerals Corp.
Montanore Project
101 Woodland Road
P.O. Box 1486
Libby, Montana 59923

by:

Morrison Knudsen Corporation
Environmental Services Group
180 Howard Street
San Francisco, California 94105

July 1990

Chapter

FIGURES

APPENDICES

CHAPTER 1
SUMMARY AND CONCLUSIONS

Noranda Minerals Corp. is planning the development of the Montanore Project located in Sanders and Lincoln Counties, Montana, for mining and milling copper-silver ore. Mill tailings will be produced at a maximum rate of 20,000 tons per day for 18 years and will total 120,000,000 tons. The tailings will be stored in an impoundment located on Little Cherry Creek in the Kootenai National Forest. This report presents the results of interim engineering and design studies performed by Morrison-Knudsen Engineers, Inc. (MKE) as of June 1990 for the tailings impoundment.

This report was prepared to provide supplemental information to the regulatory agencies during the draft EIS review period. A report on final design of the tailings impoundment will be issued after the 1990 geotechnical investigation and laboratory testing of tailings have been completed. Results presented in this report will have to be confirmed and, if necessary, modified to reflect the results of the exploration and laboratory test data. The available data on the tailings impoundment site geotechnical conditions are presented in a March 1990 geotechnical report prepared by MKE.

The scope of work for this report consists of (1) hydrologic studies, (2) stability analyses of the starter dam and final tailings embankment, (3) embankment dam design, (4) design of a diversion system and (5) design of the reclaimed impoundment.

The proposed project consists of a tailings impoundment formed by constructing a tailings retention dam, a seepage collection dam, a diversion dam and a diversion channel. For operations start-up, construction will be completed for the (1) tailings-retention starter dam, (2) diversion dam (3) diversion channel, (4) seepage collection dam and (5) a portion of a rockfill toe dike. Operational stages will consist of constructing the south saddle dam during years 4 and 5 and the north saddle dam during year 13. The toe dike will be enlarged throughout operations. The dam will be raised to a maximum height of about

370 feet in the downstream direction by spigotting and compacting coarse tailings resulting from a two-stage cycloning operation.

The results of flood routing studies reported in 1989 were revised to reflect the modified diversion system design. The local storm Probable Maximum Flood (resulting from the 6-hour Probable Maximum Precipitation, PMP) flow of 5230 cfs was routed through the diversion pond and channel. The diversion system will intercept runoff from about 50 percent of the total impoundment watershed area. The tailings impoundment was sized to contain runoff resulting from the 24-hour general storm PMP plus snowmelt.

Erosion protection for the diversion channel, consisting of riprap and gabion, was designed for a smaller flood event since erosion damage will not cause failure of the tailings retention dams. Therefore, erosion protection was designed for the flood with a peak inflow of 360 cfs resulting from the 100-year, 6-hour precipitation.

Stability analyses were performed for both the starter dam and final tailings dam. Stability was analyzed for the end-of-construction condition for the starter dam and steady-state seepage and seismic loading for both the starter and final dams. For the dam designs presented in this report, the results of the stability analyses indicate that the calculated factors of safety equal or exceed the minimum acceptable values. The analyses will be finalized based on the results of the 1990 geotechnical exploration.

Embankment construction materials will consist of required excavation materials, borrow materials from within and adjacent to the impoundment, mine waste rock and tailings. Materials from required excavations, primarily from the diversion channel, will be used to construct the starter dam and diversion dam. Mine waste rock will be used to construct the toe dike. Mine waste rock also will be screened to provide both riprap and riprap bedding materials. Crushed mine waste rock will be further crushed, screened and washed to provide both filter and drain materials to control seepage within the embankment dams.

Coarse tailings for dam construction will be produced by double cycloning the mill feed. The results of cyclone analyses indicate that the tailings will contain about 8 percent fines and the recovery will be about 43 percent of the mill feed. The second stage underflow will be spigotted onto the face of the dam to increase impoundment capacity. The fine tailings from the first stage cyclone overflow will be deposited in the tailings pond; the second stage overflow will be returned to the feed of the first stage cyclones.

As indicated in the March 1990 geotechnical report, artesian groundwater conditions were observed in the impoundment site; the maximum observed head above the ground surface was 24 feet. In order to prevent excessive pressures from developing within the foundation, which would decrease stability, a pressure relief system is likely to be required. This relief system, consisting of wells and local trenches, would be installed at start-up. If pressure heads increase during the operational life of the impoundment, the pressure relief system would be expanded. A preliminary design study of the pressure relief system is included herein. Final evaluation of foundation conditions and pressure relief requirements will be made based on the results of the 1990 exploration program.

An instrumentation system is planned for the impoundment. Pneumatic piezometers will be used to measure pore water pressures within the tailings dam and south saddle dam. Also, monitor wells will be installed in conjunction with the pressure relief well system. As the impoundment size increases, additional piezometers and monitor wells will be installed. To measure seepage and flow from the spigotting operation, a V-notch weir will be installed at the downstream toe of the dam.

The 4000-foot long diversion channel is located on the south side of the impoundment and was sized to pass the full PMF. The channel excavation will have a trapezoidal cross-section with a bottom width of 20 feet. The maximum excavation depth will be about 100 feet. Approximately 1.9 million c.y. of earth and weathered rock materials will be excavated from the diversion channel and pond. As indicated above, materials removed from the excavation will be used for dam construction. The diversion channel will discharge into a natural stream channel that flows into Libby Creek.

A plan and section of the reclaimed tailings impoundment are presented. The reclamation scheme is the same as that presented in the 1989 preliminary engineering report.

The impoundment design presented in this report was prepared to provide information to the agencies during the draft EIS review period. The design reflects our current thinking as of June 1990. Noranda intends to undertake a supplemental geotechnical investigation during the summer of 1990. Based on the results of that investigation and on tailings testing currently in progress, the design of the impoundment structures will be finalized, and plans and specifications will be prepared.

CHAPTER 2
INTRODUCTION

2.1 BACKGROUND

Noranda Minerals Corp. is developing the Montanore Project located in Sanders and Lincoln Counties, Montana, for mining and milling copper-silver ore. As a part of the project, a tailings impoundment is proposed on Little Cherry Creek, in the Kootenai National Forest (Figure 1). The tailings impoundment site is located about 14 miles south of Libby, east of the Cabinet Mountains.

Final design studies for the tailings impoundment are in progress. This interim report presents the results of tailings impoundment design studies performed by Morrison-Knudsen Engineers, Inc. (MKE) as of June 1990. This report was prepared to provide supplemental information to the regulatory agencies during the draft EIS review period. A report on final design of the tailings impoundment will be issued after the 1990 geotechnical investigation and laboratory testing of tailings have been completed. Results presented in this report will have to be confirmed and, if necessary, modified to reflect the results of the exploration and laboratory testing data.

Available data on the tailings impoundment site geotechnical conditions are presented in the Geotechnical Report, Tailings Impoundment Site, March 1990, prepared by MKE. That report presents the results of both the 1988 and 1989 geotechnical investigations and includes seismicity, seismic design parameters, field investigation results and laboratory test data.

2.2 PURPOSE AND SCOPE

The purpose of this study is to present the results of interim engineering studies for the Little Cherry tailings impoundment. The scope of work for the final design studies consists of the following:

- Conduct a field investigation that includes the following activities:
  - Seismic refraction surveys
  - Exploratory drilling
  - Test pit excavations
  - Laboratory testing

- Perform the following final design studies:
  - Stability analyses
  - Tailings settlement analysis
  - Diversion channel design
  - Dam and dike design
  - Reclamation plan
  - Construction cost estimates

- Prepare the following technical reports:
  - Geotechnical Report
  - Tailings Impoundment Engineering Reports

- Attend meetings with Noranda Minerals Corporation and regulatory agencies.

The approach to the work is discussed in the Design Basis Memorandum in Appendix A.

Since preparation of this report precedes the 1990 investigation and completion of tailings testing, the results of all final design studies cannot be included in this report. The tailings settlement analysis, final design of the foundation pressure relief system and part of the stability analyses, including dynamic stability, will not be included in this report. A final design report that includes the results of all engineering studies will be issued after the completion of the field and laboratory studies.

2.3 AUTHORIZATION

Noranda Minerals Corp. authorized the work for this phase of the project in April 1989.

CHAPTER 3
PROJECT DESCRIPTION

3.1 SITE DESCRIPTION

The Little Cherry Creek tailings impoundment is located approximately 14 miles south of Libby, Montana (Figure 1). The site lies in a heavily forested valley which drains into Libby Creek to the east. Approximately 40 percent has been clear-cut by previous logging operations. Elevations range from 3,240 feet at the toe of the seepage collection dam to 5,400 feet at the highest point in the Little Cherry Creek watershed area. Access to the site is via U.S. Forest Service roads, about 12 miles from U.S. Route 2. In August 1989, a one-mile access road was cleared in the south portion of the tailings impoundment site, north of the proposed diversion channel. Additional site and subsurface conditions information is contained in the Geotechnical Report (MKE, 1990).

3.2 DESIGN CRITERIA AND TAILINGS PRODUCTION

The Little Cherry tailings impoundment is designed to contain approximately 120,000,000 tons (111,000,000 c.y.) of material anticipated over the 18-year operational life of the mine. General design criteria are presented in the Design Basis Memorandum (DBM), Appendix A.

The starter stage of the impoundment will include construction of the starter dam, diversion dam and channel, and partial construction of the rockfill toe dike. After completion of the starter stage, subsequent tailings dam construction will proceed by the "downstream" method, using cycloned tailings sands. The tailings slurry of 60 percent solids will be conveyed by pipeline from the mill to a cyclone plant at the damsite. Tailings production rates over the 18-year operational life are presented in the Design Basis Memorandum in Appendix A. The mill tailings gradation is presented in Table 3-1.

TABLE 3-1
MILL TAILINGS GRADATION

Tyler Sieve No.	Particle Size (Microns)	Cumulative % Passing Sieve
65	208	99.5
100	147	90.2
150	104	69.4
200	74	52.1
270	52	35.9
400	37	25.0

Several factors influence impoundment location and design. First, the tailings impoundment dam is located as far upstream as possible from the confluence of Little Cherry Creek and Libby Creek to minimize the potential effect that impoundment seepage could have on Libby Creek. Second, it is desirable to keep the toe dike upstream of a narrow gorge on Little Cherry Creek to facilitate construction and to minimize waste rock requirements. Third, the impoundment dam crest alignments are positioned to take full advantage of the Little Cherry basin topography, thereby minimizing earthfill dam borrow and runoff storage requirements while maximizing available tailings storage capacity.

The required tailings dam crest elevation is based on providing storage of the following:

- Tailings
- Tailings effluent for 20 days at 8,000 gpm (about 700 acre-feet)
- Design flood (see Chapter 4)
- Minimum freeboard of three feet above the peak flood water surface.

A diversion dam and channel are included to divert flow from the Little Cherry Creek watershed, thereby decreasing the required design flood storage volume in the tailings impoundment.

Seepage collection ditches are located along the toe of the downstream slope to convey seepage and runoff to the seepage collection impoundment. The seepage collection dam and impoundment are sized to store the maximum combination of the following parameters over the operational life of the dam:

- Runoff resulting from 100-year, 24-hour storm
- Underflow water
- Seepage from the impoundment

Also included in sizing the seepage collection dam and impoundment is a minimum three feet of freeboard above the maximum water surface.

3.3 GENERAL ARRANGEMENT OF IMPOUNDMENT STRUCTURES

Alignments and locations of dams, channels and impoundments are shown on Figure 2. The diversion dam is located on the southwest side of the tailings impoundment to divert runoff that would otherwise drain into the tailings impoundment. This runoff is diverted into the diversion channel that is located south of the tailings impoundment. Diverted runoff flows from west to east and is discharged into an existing drainage flowing southwest of the site and ultimately into Libby Creek. The north and south saddle dams are located approximately along the ridges that form up the Little Cherry Creek valley. The tailings dam is located approximately across the valley to form the impoundment. Crest elevations of the diversion dam, north and south saddle dams, and the final tailings dam (prior to reclamation) are 3,710 feet. The seepage collection impoundment and dam is located downstream of the tailings dam (to the east) to intercept runoff and seepage that emerges from the tailings dam.

CHAPTER 4
HYDROLOGIC STUDIES

4.1 DESIGN FLOOD CRITERIA

The design flood criteria are based on the U.S. Forest Service and U.S. Corps of Engineers (1977) criteria (see Design Basis Memorandum, Appendix A). The designation of the tailings impoundment design flood is based on size and hazard potential classifications. The tailings retention dam will be raised incrementally to increase impoundment storage capacity. Size classification is determined by either storage or dam stage height, whichever gives the larger size category. For this project, dam stage heights control the size classification.

U. S. Highway 2 is located about 5.7 river miles downstream of the Little Cherry damsite; the nearest dwelling is located about 5.4 miles downstream of the site. Libby is the closest town to the impoundment site and is located about 14 miles downstream (north) of the site. Because of the potential for downstream damage and resulting clean-up, the U.S. Forest Service and the Montana Department of State Lands consider the impoundment site to have moderate to high hazard potential (see Design Basis Memorandum, Appendix A).

Based on dam stage size and hazard potential classifications, the Forest Service and Department of State Lands designated the following design flood criteria:

For containment: 24-hour general storm
Probable Maximum Precipitation (PMP)

For diversion: 72-hour general storm
Probable Maximum Flood (PMF)

The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region (U.S. Corps of Engineers, 1977). The PMF is derived from the PMP, which is defined as theoretically the depth of precipitation for a given duration that approaches the upper limit of what the atmosphere can produce over a given area at a particular location at a certain time of the year (U.S. Department of Commerce, Hydrometeorological Report No. 43, 1966).

Because thunderstorm events should also be considered for small watersheds, the local storm PMF (resulting from the 6-hour PMP) was also considered for diversion. The more critical of the two diversion floods was used for diversion system design. In addition, a low-flow channel within the main diversion channel was designed for a 100-year storm event.

Based on engineering practice, the minimum embankment dam freeboard (above the peak flood water surface) was taken to be 3 feet (U.S. Bureau of Reclamation, "Design of Small Dams," 1987).

4.2 METHODOLOGY

The Corps of Engineers' "HEC-1 Flood Hydrograph Package" computer program was used for the hydrologic studies. The "Hydrometeorological Report No. 43, Probable Maximum Precipitation - Northwest States" (Hydromet 43), developed by the U.S. Department of Commerce, Weather Bureau (1966) was used as the basis for estimating the local storm and general storm PMP's.

An unlimited snowpack was assumed to be available for snowmelt during the 24-hour and 72-hour general storm PMP's. Data on snowpack and snow-water equivalents are not available; thus, snowmelt estimates were based on the Corps of Engineers' "Runoff from Snowmelt" (1960). Since forest cover is less than 80 percent of the watershed, the snowmelt equation developed for open and partly forested areas was used. Percent forest cover was estimated from July 1988 aerial photographs. Dew point temperatures and wind speeds during the PMP were developed from procedures described in Hydromet 43.

The general storm PMF was based on the general-storm type 24-hour and 72-hour PMP plus snowmelt; the local storm PMF was based on the 6-hour local storm PMP. Unit hydrographs were developed following the Soil Conservation Service (SCS) method, as described in the U.S. Bureau of Reclamation "Design of Small Dams" (1987). Infiltration and retention losses were also estimated following SCS hydrologic soil group and curve number (CN) procedures. These estimates were based on soil conditions and vegetal cover as determined from the aerial photographs and information provided in available reports (U.S. Department of Agriculture, 1984). Hydrologic Soil Group B was used in the analysis. Basin lag time was calculated by using procedures described in "Design of Small Dams." The antecedent moisture conditions (AMC) for the PMF estimates were based on AMC III conditions.

A diversion dam and diversion channel are required to divert the watershed runoff around the Little Cherry tailings impoundment. To determine the size of the channel and diversion dam height, the design flood (described in Section 4.1) was routed through the diversion pond by using HEC-1 procedures.

4.3 WATERSHED CHARACTERISTICS

The Little Cherry watershed, tailings impoundment and diversion system are shown on Figure B-1, Appendix B. The tailings retention dam will be constructed across Little Cherry Creek. A diversion dam will be constructed at the upstream end of the tailings impoundment and a channel will divert runoff around the south side of the impoundment. A small catchment area south of the diversion channel also contributes runoff into the channel.

Table 4.1 summarizes the drainage area characteristics of the Little Cherry watershed. As shown in Table 4.1, the diversion dam reduces the tailings impoundment watershed area by about 50 percent.

TABLE 4.1

DRAINAGE AREA CHARACTERISTICS

Tailings Impoundment Drainage Area (square miles)	0.88
Diversion Impoundment Drainage Area (square miles)	0.90
Total Drainage Area (square miles)	1.78
Small Catchment Drainage Area (square miles)	0.20
Mean Basin Elevation (feet)	3860
Lag Time for Diversion Impoundment Watershed (hours)	0.32
Lag Time for Small Catchment (hours)	0.25
Curve Number (AMC III)	73
Percent Forest Cover	76

4.4 PROBABLE MAXIMUM PRECIPITATION, SNOWMELT AND NET RUNOFF

The 24-hour general storm PMP with snowmelt was used to compute runoff into the tailings impoundment. Both the 72-hour general storm PMP with snowmelt and the 6-hour local storm PMP were considered for the diversion system. Estimates of snowmelt during the 24-hour and the 72-hour general storm PMP are necessary because of the relatively high mean basis elevation of the area. In addition, based on Hydromet 43, the most critical general storm PMP was determined to occur in June. The spring melt season for this area occurs primarily from spring to early summer. Estimates of snowmelt during the 6-hour local storm PMP were not necessary, since this storm event is associated with summer-autumn thunderstorms. Precipitation, snowmelt and net runoff for the Little Cherry watershed are summarized in Table 4.2 below.

TABLE 4.2
PRECIPITATION, SNOWMELT AND NET RUNOFF

Precipitation Event	Precipitation (inches)	Precipitation & Snowmelt (inches)	Net Runoff (inches)
6-hour Local Storm PMP	11.7	N/A	8.2
24-hour General Storm PMP	11.9	15.8	9.8
72-hour General Storm PMP	17.0	24.4	10.2

4.5 DESIGN FLOODS

The tailings impoundment was sized to completely contain runoff resulting from the 24-hour general storm PMP plus snowmelt. Since all runoff upstream of the diversion dam will be routed around the tailings impoundment, only the tailings impoundment drainage area (see Table 4.1) was used to estimate the required inflow volume for containment. The total inflow volume was calculated as the sum of (1) direct precipitation (11.9 inches) on the pond surface and (2) net runoff (9.8 inches) from the surrounding topography. Because pond elevation and thus pond area increase with time (see Figure 10), the volume contribution due to direct precipitation also increases with time. Depending on pond surface area, the total inflow volume was estimated to range from about 460 to 540 acre-feet for the starter and final impoundments, respectively (see Appendix B, Figure B-2).

For the diversion dam and channel, the PMF hydrographs for both the 72-hour general storm plus snowmelt and the 6-hour local storm were computed using HEC-1 procedures. Peak discharges for the 72-hour PMF and the 6-hour PMF are approximately 2510 cfs and 5230 cfs, respectively. Since the larger more critical peak discharge is produced by the 6-hour PMF, this flood event was used as the design flood for diversion system design. Computer printouts of the design flood calculations for the 6-hour PMF are presented in Appendix B.

4.6 DIVERSION FLOOD ROUTING

The 6-hour local storm PMF was routed through the diversion impoundment and channel. Computer printouts of the flood routing calculations, diversion pond area-capacity curves, discharge rating curve and hydrographs are presented in Appendix B.

Table 4.3 summarizes the flood routing results for the diversion pond. The diversion pond was assumed to be full (water surface at El. 3680) at the beginning of the design storm. Peak outflow from the diversion impoundment is about 4230 cfs. Combined with the inflow from the catchment area south of the diversion channel, total peak discharge into the diversion channel is about 5060 cfs. The peak design flood flow velocity in the diversion channel was computed to be about 18 feet per second.

TABLE 4.3
LITTLE CHERRY DIVERSION SYSTEM
RESULTS OF PMF ROUTING

	6-hr Local PMF[b]
Dam Crest Elevation (feet)	3710
Diversion Channel Crest Elevation (feet)	3680
Initial W. S. El. (feet)	3680
Maximum W. S. El. (feet)	3691.4
Minimum Freeboard (feet)[a]	18.6
Peak Inflow (cfs)	5230
Peak Outflow (cfs)	4230
Peak Diversion Channel Flow[c]	5060

Notes:
[a] Minimum freeboard equals dam crest elevation minus maximum water surface elevation.
[b] Design flood is the flood resulting from the 6-hour local storm PMP event.
[c] Includes outflow from diversion impoundment and inflow from small catchment south of channel.

4.7 100-YEAR FLOOD AND ROUTING RESULTS

The diversion channel has been designed for the full PMF. However, a low-flow channel within the main diversion channel has been designed for a 100-year storm event.

The 100-year storm for the low-flow channel within the main diversion channel was based on rainfall from "NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume I - Montana" (1973). The antecedent moisture conditions (AMC) were based on AMC III conditions. Inflow from the small catchment south of the diversion channel was also included.

Results of the flood routing are summarized in Table 4.4. The diversion pond was assumed to be full (water surface at El. 3680) at the beginning of the storm. Peak outflow from the diversion impoundment is about 140 cfs. Adding the flow from the small catchment area south of the diversion channel, the total peak outflow from the diversion channel is about 170 cfs. Computer printouts of the flood routing calculations and hydrographs are presented in Appendix B.

TABLE 4.4
LITTLE CHERRY DIVERSION SYSTEM
RESULTS OF 100-YEAR FLOOD ROUTING

	100-yr, 6-hr Flood[a]
Dam Crest Elevation (feet)	3710
Diversion Channel Crest Elevation (feet)	3680
Initial W.S. El. (feet)	3680
Maximum W.S. El. (feet)	3681.7
Minimum Freeboard (feet)[b]	28.3
Peak Inflow (cfs)	360
Peak Outflow (cfs)	140
Peak Diversion Channel Flow[c]	170

Note:

(a) Flood is based on a 100-year, 6-hour storm event.

(b) Minimum freeboard equals dam crest elevation minus maximum water surface elevation.

(c) Includes outflow from impoundment and inflow from small catchment south of channel.

CHAPTER 5
STABILITY ANALYSES

5.1 SOIL PARAMETERS

The soil parameters used in the slope stability analysis are summarized in Table 5.1. Clayey soil foundation strength parameters were determined by testing undisturbed samples. Total strength parameters were based on the results of unconsolidated-undrained (UU) triaxial compression tests and effective strength parameters were determined from consolidated-undrained (CU) triaxial tests with pore water pressure measurements (MKE, Geotechnical Report, 1990). These values will be confirmed or modified based on the results of further sampling and testing, to be performed in the summer of 1990. Effective strength parameters for compacted clayey borrow samples for embankment core zones were determined from the results of CU tests with pore water pressure measurements. These samples were compacted to 95 percent of the maximum dry density and at optimum moisture content as determined by ASTM D698. This compactive effort would be specified for clayey embankment soils.

Strength parameters of granular foundation soils and embankment transition and shell zones were based on typical values for similar materials. Published data were used to estimate the strength parameters of rockfill (Leps, 1970).

The strength parameters and unit weights of coarse cycloned tailings were based on typical values of similar tailings (Vick, 1983). These values will be confirmed or modified based on the results of triaxial compression tests on compacted tailings samples, which are currently in progress. For the purpose of stability analyses, the fine pond tailings were conservatively assumed to have no strength.

5.2 CRITERIA AND LOADING CONDITIONS

The starter dam was checked for static and seismic stability at two locations (see Appendix C). Stability analysis of the final embankment is in progress.

TABLE 5.1
SOIL PARAMETERS FOR STABILITY ANALYSES

Stability Soil No.	Description	Material	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion[a] (psf)	Friction Angle[a] (deg.)
1	Core	Gravelly sandy clay or sandy clayey silt	130	133	2000[b] 200 1000[c]	0[b] 32 10[c]
2	Transition	Silty sandy gravel and cobbles	127	129	0	35
3	Shell	Weathered bedrock, mixture of cobbles, gravel, sand and clayey soil	130	138	0	35
4	Foundation-Upper Zone	Gravelly sandy clay and sandy clayey silt	130	132	1750[b] 0 1000[c]	0[b] 33 17[c]
5	Foundation-Lower Zone	Gravels, cobbles and boulders in sandy clayey silty matrix	125	130	0	35
6	Rockfill (Toe Dike)	Mine waste rock	140	145	0	42
7	Coarse Tailings	Fine sand	110	123	0	33
8	Fine Tailings	Fine sandy silt	-	110	0	0

[a] Parameters apply to all load conditions unless otherwise noted.

[b] Parameters for end-of-construction condition.

[c] Parameters for seismic condition.

The design criteria and loading conditions are presented in Table 5.2 for the cases considered in the slope stability analyses. The minimum acceptable factors of safety used in the analyses are as recommended by the Corps of Engineers (1970).

TABLE 5.2
DESIGN CRITERIA AND LOAD CONDITIONS

Case	Load Condition	Embankment Stage	Slope	Minimum Acceptable Factor of Safety
1A,B	End-of-Construction	Starter Dam	Upstream and Downstream	1.3
2A,B	Steady-State Seepage	Starter and Final Dams	Downstream	1.5
3	Design Flood	Final Dam	Downstream	1.4
4A,B	Seismic*	Starter and Final Dams	Downstream	1.0

* Seismic Coefficient = 0.10g

The stability of the starter dam was checked for the end-of-construction condition before tailings are deposited. This condition is the most critical static case for the upstream slope since subsequent deposition of fine tailings into the pond will stabilize the slope.

The stability of the downstream slopes of the starter dam was analyzed for steady-state seepage and seismic loading. The assumed piezometric surfaces in the dam and foundation are shown on Figure C-8, Appendix C. Pressure relief wells would be used to reduce uplift at the downstream toes of the starter and final dam stages. A conservative phreatic surface will also be used to check that the stability of the downstream slope would be adequate for the design flood condition, Case 3 (see Chapter 4). A 1000-foot wide beach with a 1.0 percent slope would normally prevent the pond shoreline from reaching the embankment dam.

Seismic stability was evaluated by the "pseudo-static" method. Based on a review of site seismicity, a seismic coefficient of 0.10g was used in the stability analyses (MKE, Geotechnical Report, 1990). In the "pseudo-static" method of stability analysis, the effects of an earthquake on a potential slide mass are represented by an equivalent static horizontal force determined as the product of a seismic coefficient and the weight of the potential slide mass (see Design Basis Memorandum, Appendix A).

5.3 METHOD AND RESULTS

The computer programs STABL (Siegel, 1975) and UTEXAS2 (Edris, 1987) were used to calculate the factors of safety for circular failure surfaces. Hand calculations were performed using the infinite slope method. The results of the interim stability analyses are presented in Table 5.3 and are compared with the Corps of Engineers (1970) criteria for minimum acceptable factors of safety. The critical failure surfaces for each loading condition are shown in Appendix C.

As shown in Table 5.3, the calculated factors of safety equal or exceed the minimum acceptable values for all cases analyzed. Due to clayey soil foundation conditions, the results of the interim stability analyses indicate that the starter dam would require upstream and downstream slopes that are flatter than originally estimated in the preliminary engineering report, in order to provide adequate stability for the end-of-construction condition. Additional soil strength data will be obtained in 1990 and used in final analyses to confirm or modify the dam slopes.

TABLE 5.3

RESULTS OF INTERIM STABILITY ANALYSES
(IN PROGRESS)

Case	Load Condition	Embankment Stage	Slope	Minimum Acceptable Factor of Safety[a]	Calculated Factor of Safety[f]
1A	End-of-Construction	Starter Dam[b]	Upstream	1.3	1.3
1B	End-of-Construction	Starter Dam	Downstream	1.3	1.3
2A	Steady-State-Seepage	Starter Dam	Downstream	1.5	1.8
2B	Steady-State-Seepage	Final Dam[c]	Downstream	1.5	To be determined
3	Design Flood[d]	Final Dam	Downstream	1.4	To be determined
4A	Seismic[e]	Starter Dam	Downstream	1.0	1.2
4B	Seismic[e]	Final Dam	Downstream	1.0	To be determined

Notes:

(a) From Corps. of Engineers (1970)

(b) Crest El. 3500

(c) Crest El. 3710

(d) 24-hour PMP plus Snowmelt (see Chapter 4)

(e) Seismic Coefficient = 0.10g

(f) Lowest Factor of Safety of the two cross-sections

CHAPTER 6
EMBANKMENT DAMS

6.1 GENERAL DESCRIPTION OF DAMS

General dimensions and features of the embankment dams are presented in Table 6.1 below:

TABLE 6.1
SUMMARY OF DAM DIMENSIONS

Item	Crest El. (feet)	Crest Length (feet)	Sideslopes (H:V)	Max. Height (feet)
Diversion Dam	3,710	1,030	2.5:1	95
Starter Dam	3,500	2,780	2.5:1 above El. 3450 4:1 below El. 3450	125
South Saddle Dam	3,710	3,740	2:1	130
Seepage Collection Dam	3,325	320	2.5:1	70
North Saddle Dam	3,710	1,630	2.5:1	40
Toe Dike	Varies	8,060	2:1	80
Tailings Dam	3,710	6,100	2:1 U/S, 3:1 D/S	370

6.1.1 Starter Stage

A. Starter Dam

The starter dam is designed to impound tailings during the first two years of operation. The starter dam will have upstream and downstream slopes of 2.5(H):1(V) above El. 3450 and 4(H):1(V) below El. 3450, and will have a 30-foot wide crest at El. 3500. The total volume to construct the dam will be approximately 1,680,000 c.y.

B. Diversion Dam

The diversion dam will be located in the Little Cherry Creek drainage at the southwest corner of the impoundment and will divert flow away from the proposed impoundment. The diversion dam will have upstream and downstream slopes of 2.5(H):1(V) and a 30-foot wide crest at El. 3710. The upstream face will be covered with a 2-foot thick riprap layer to protect against erosion. The water impounded by the diversion dam will be directed into the diversion channel, which will convey the flow south of the impoundment and eventually into Libby Creek. Total material volume required to construct the dam is approximately 372,000 c.y.

C. Seepage Collection Dam

The seepage collection dam will be located in the Little Cherry Creek drainage downstream (east) of the tailings impoundment approximately 2700 feet upstream of the confluence of Little Cherry Creek and Libby Creek. The seepage collection dam will have 2.5(H):1(V) slopes and a 30-foot wide crest at El. 3325. The upstream face will be covered with a 2-foot-thick riprap layer to protect against erosion. An emergency spillway will be provided to prevent overtopping of the dam. Total volume to construct the dam is approximately 53,000 c.y. The impoundment behind the dam will have a storage capacity of about 100-acre-feet, sufficient to store the runoff from a 100-year 24-hour storm, the seepage through the tailings dam and the water from underflow spigotting on the downstream slope of the tailings dam.

D. Toe Dike

The toe dike will be located at the downstream toe of the tailings dam and will retain sand tailings used to construct the tailings dam. The toe dike will be constructed of mine waste rock and will have 2(H):1(V) slopes and a 20-foot wide crest. Toe dike construction will begin during the starter dam construction and continue during operations. The starter portion will be constructed to grades and alignments shown on Figure 3 from waste rock obtained during the evaluation and predevelopment stages. Toe dike construction for the starter stage will require approximately 171,000 c.y.

6.1.2 Operational Stage

A. Main Tailings Dam

The main tailings dam will consist of the earthfill starter dam, rockfill toe dike and coarse tailings fill (cycloned sand). Slopes of the coarse tailings fill will be 2(H):1(V) upstream and 3(H):1(V) downstream. The final crest will be 30 feet wide and at El. 3710. The crest will move in the downstream direction as sands are spigotted on the downstream face. Total volume of tailings sand required to construct the dam is approximately 27,200,000 c.y.

B. South Saddle Dam

The south saddle dam will be located on the south side of the impoundment and is an extension of the tailings retention dam. The south saddle dam is zoned to allow staged construction of the dam as the pond surface rises. The dam will have 2(H):1(V) upstream and downstream slopes. A transition of the downstream slope from 2(H):1(V) to 3(H):1(V) is located on the south abutment where the south saddle dam connects with the tailings dam (see Figure 2). Total volume to construct the south saddle dam is approximately 1,930,000 c.y.

C. North Saddle Dam

The north saddle dam will be located at the northwest corner of the impoundment. The north saddle dam has a 30-foot wide crest at El. 3710 and slopes of 2.5(H):1(V). North saddle dam construction will require approximately 166,000 c.y.

D. Toe Dike Enlargement

During the operational life of the project, additional waste rock generated will be used to construct the toe dike to final grades and alignments shown on Figure 4. An additional 353,000 c.y. will be required to construct the toe dike from starter to final grades, bringing the total toe dike waste rock volume to approximately 524,000 c.y.

6.2 SITE PREPARATION

Areas to be cleared of vegetation and stripped of salvageable soil containing organic matter include the following:

- Embankment foundations
- Required excavations
- Tailings pond
- Borrow areas

The areas of each structure or excavation, percent of land covered by forest and estimated volume of soil stripping are presented in Table 6.2.

Soil materials will be salvaged from all disturbed areas, with the exception of slopes over 50 percent and soil storage areas. Based on the results of the test pit excavations, the depth of soil salvage is anticipated to be 1 to 2 feet in the impoundment area; however the actual depth of salvageable soils may vary depending on site conditions. Soil salvage depths of different soil types have

TABLE 6.2
SUMMARY OF SITE PREPARATION

Structure	Footprint Foundation Area (Ac)	Forest Area[4] (Ac)	Percent Forest Area %	Estimated[5] Soil Salvage Volume cy
Starter Dam	31	6	19	
Toe Dike	30	13	43	
Final Dam[1]	171	41	24	275,900
Diversion Dam	6	6	100	9,700
Seepage Collection Dam	2	2	100	3,200
Diversion Channel & Pond	42	42	100	67,800
Seepage Collection Pond	11	7	64	17,700
South Saddle Dam	30	30	100	48,400
North Saddle Dam	7	5	71	11,300
Impoundment[2]	400	195	49	645,300
Borrow Area A[3]	145	55	38	
Borrow Area B	58	13	22	93,600
Borrow Area C	65	3	5	104,900
Borrow Area D	142	113	80	229,100
Total				1,506,900

---

[1] Includes starter dam and toe dike.

[2] Excludes foundation area of impoundment dams.

[3] Stripping volume included in impoundment.

[4] Base map prepared by U.S. Borax and air photo provided by Noranda.

[5] Average depth of stripping - 1.00 feet.

been estimated and presented in the Permit Application, (Noranda, 1989). An average soil salvage depth of 1.00 foot, based on test pit exploration by MKE, was used for computing the volumes in Table 6.2. The permit application indicates that the estimated volume of salvageable soil based on various excavtion depths of different soil types for the tailings impoundment and seepage collection pond is 1,700,000 cubic yards. The permit application shows an estimated volume of about 64,000 cubic yards for the diversion channel. The soil will be stockpiled in various locations shown on Figure 2 to be used for cover during reclamation.

Soil will be salvaged in two lifts in portions of the tailings impoundment area. The deeper second lift will have a higher percentage of coarse fragments. This material will be used for erosion protection on the downstream face of the impoundment embankments during the reclamation phase.

Prior to soil salvage, commercial trees will be harvested in all areas to be disturbed. Non-merchantable trees, shrubs, and slash will be dozerpiled into windrows (using a brush blade to minimize soil accumulation) and burned. All requirements of the Montana Slash Disposal Law will be observed.

6.2.1 Dam Foundations

After stripping the embankment foundation, irregularities in the surface will be leveled by grading. Depressions will be filled with compacted soils of the same characteristics as the overlying embankment zone. Any loose or soft materials encountered during grading will be removed from the dam foundation and replaced with compacted fill. The entire dam foundation will be scarified and compacted by six passes of a sheepsfoot roller. All embankment subgrade areas will be graded to drain to prevent ponding during foundation preparation operations.

If a coarse granular soil is exposed on the surface under the core zone, the clayey soil (core) will be extended into the foundation below the pervious layer. Depth of excavation will have to be determined in the field.

Areas downstream of the starter dam will be cleared, stripped and prepared in stages prior to placement of the blanket drain and coarse tailings sands.

The seepage collection dam may be founded on exposed bedrock in the Little Cherry Creek channel. In core foundation areas where hard bedrock is exposed, the foundation will be broomed and cleaned with air and water jets. Open joints, fractures and bedding planes will be cleaned out with air and water jets and backfilled with sand-cement slush grout. Depressions and irregularities will be filled with dental concrete.

If soft rock is exposed under the core zone, the rock will be excavated to a smooth surface. The core zone will be placed directly after the rock is excavated to prevent deterioration of the exposed surface.

6.2.2 Impoundment Area

A. Tailings Impoundment

The tailings impoundment area will be cleared and stripped in stages during the operational life of the impoundment. Trees will be harvested and stumps, where soil is stripped, will be removed within the impoundment area. Approximately 49 percent of the impoundment area (excluding dam foundations) is covered by forest. Soil borrow area within the tailings impoundment covers approximately 145 acres (36 percent of the tailings pond).

A majority (approximately 80 percent) of the test pit logs and laboratory classification test results of soils in the upper 5 feet indicate a gravelly sandy silty clayey soil within the impoundment. Grain size analyses of the samples generally show 50 to 70 percent passing the No. 200 sieve (0.074 mm). However, the results of the geotechnical investigations also indicate that some silty sand and gravel fluvial outwash deposits occur in the impoundment site. If soils consisting of sandy gravels and cobbles are exposed during stripping of the tailings pond area and borrow excavation operations within the tailings pond, these areas will be covered with a 3-foot thick layer of compacted clayey soil to minimize infiltration of water into the foundation.

B. Seepage Collection Impoundment

Site preparation for the seepage collection impoundment will include tree harvesting, stump removal and stripping of the organic topsoil. The slopes within the pond area average approximately 2.75(H):1(V). The area of the impoundment is approximately 10.4 acres.

Test pits in the area encountered gravelly sandy silt to clayey silt soils. Seepage water was not encountered in any of the test pits; however, water may be encountered along the bottom of the natural drainage.

Clayey silty soils for the seepage collection dam will be obtained by excavating materials within the seepage collection pond area. The excavated slopes will be flattened to a gradient no steeper than 3(H):1(V).

If bedrock with open joints or soils consisting of sandy gravels and cobbles are exposed during grading operations, these areas will be covered with a 3-foot thick layer of compacted clayey soil to minimize infiltration of water into the foundation.

6.3 EMBANKMENT DAM ZONES

The embankment will be zoned earthfill structures. A summary of materials, sources, placement and compaction requirements is presented in Table 6.3. A summary of estimated quantities for each earthfill zone and borrow source is shown in Table 6.4 Descriptions of the various embankment zones follow:

TABLE 6.3

CONSTRUCTION MATERIAL SUMMARY

ZONE	DESCRIPTION	MATERIAL	SOURCE	LIFT THICKNESS (in.)	COMPACTION
1	Core	Gravelly sandy clay or sandy clayey silt	Borrow Area A and seepage pond area	6 - 8	95% ASTM D698
2	Transition Zone	Silty sandy gravel and cobbles, 6-inch maximum size	Borrow Area B	12	4 passes of 10-ton vibratory roller
3	Shell	Weathered bedrock, mixture of cobbles, gravel sand and clayey soil, 24-inch maximum size	Required Excavations, Borrow Areas C and D	24	4 passes of 10-ton vibratory roller
4	Cycloned Sand	8% finer than 0.074 mm	Cyclone underflow	8 (parallel to slope)	95% ASTM D698
5	Rockfill	24 inches maximum size	Mine waste rock	24	4 passes of 10-ton vibratory roller or D8 Dozer[a]
6	Filter	Sand, 1/2-inch maximum size and not more than 5% finer than 0.074 mm	Processed from mine waste or import	12	4 passes of D8 Dozer[a]
7	Blanket Drain	Gravel, 3-inch maximum size and not more than 2% finer than 0.074 mm	Processed from mine waste or import	12	4 passes of D8 Dozer[a]
	Riprap	Well-graded, 7-in. avg. size	Mine waste (run of mine)	24	None required

(a) Or equivalent dozer.

F:\NORANDA\IERS6.T63

TABLE 6.4

ESTIMATED EMBANKMENT CONSTRUCTION QUANTITIES

Fill	Zone 1: Core (c.y.)	Zone 2: Transition Zone (c.y.)	Zone 3: Shell Material (c.y.)	Zone 4: Coarse Tailings (c.y.)	Zone 5: Rockfill (c.y.)	Zone 6: Filter (c.y.)	Zone 7: Blanket Drain (c.y.)	Riprap (c.y.)	Total Embankment
Starter Stage									
Starter Dam	261,000	70,000	1,261,000	0	0	46,000	42,000	0	1,680,000
Toe Dike	0	0	0	0	171,000	18,000	18,000	0	207,000
Main Dam Drains	0	0	0	0	2,000[a]	53,000	43,000	0	98,000
Diversion Dam	88,000	30,000	247,000	0	0	0	0	7,000	372,000
Seepage Collection Dam	45,000	0	0	0	0	6,000	0	2,000	53,000
Totals	394,000	100,000	1,508,00	0	173,000	123,000	103,000	9,000	2,410,000
Operation Stage									
Toe Dike	0	0	0	0	353,000	28,000	28,000	0	409,000
Final Dam	0	0	0	27,200,000	10,000[a]	302,000	218,000	0	27,730,000
South Saddle Dam	387,000	123,000	1,282,000	0	0	138,000	0	0	1,930,000
North Saddle Dam	29,000	12,000	125,000	0	0	0	0	0	166,000
Totals	416,000	135,000	1,407,000	27,200,000	363,000	468,000	246,000	0	30,235,000

Note:

(a) This volume is crushed rock in trunk drains.

F:\NORANDA\IERS6.T64

- Zone 1: Core material consisting of gravelly sandy clay or gravelly sandy clayey silt will be obtained from Borrow Area A within the tailings impoundment and from the seepage pond area. Maximum particle size will be 3 inches. Clusters of open gravels will not be allowed in the zone 1 fill. The material will be placed in 6- to 8-inch thick lifts and compacted to a dry density equal to at least 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor compaction). Laboratory tests of Borrow Area A soils indicate a maximum dry density of about 125 pounds per cubic foot (pcf) and optimum moisture content of about 10 percent. Laboratory test data show that the natural water contents in Borrow Area A varies widely, from 6 to 22 percent. A laboratory test of a sample from the seepage collection pond shows a natural moisture content of 34 percent. Therefore, moisture conditioning will be required to bring the soils within minus 1 percent to plus 2 percent of the optimum moisture content.

- Zone 2: Transition zone material will consist of silty sandy gravel and cobbles obtained from Borrow Area B above the impoundment tailings pond. Maximum particle size will be 6 inches. Clusters of open gravels or cobbles will not be allowed in Zone 2. The material will be placed in 12-inch thick lifts and compacted with 4 passes of a 10-ton vibratory roller.

- Zone 3: Shell materials, consisting of weathered bedrock and a mixture of cobbles, gravel, sand and clayey soil, will be obtained from required excavations and from Borrow Areas C and D outside the impoundment. Maximum particle size will be 24 inches. This material will be selectively placed so as to avoid open clusters of cobbles and boulders against transition and filter zones. The coarser materials will be routed to the exterior slopes for erosion protection. The material will be placed in 24-inch thick lifts and compacted with 4 passes of a 10-ton vibratory roller.

- Zone 4: The cycloned sand will contain 8 percent fines and will be obtained from a double-cycloning operation. As the sands are spigotted onto the downstream slope, a bulldozer will be used to spread and compact the sands. The approximate lift thickness will be about 8 inches parallel to the downstream slope and the sands will be compacted to at least 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor Compaction). Laboratory test results indicate a maximum dry density of about 95 pcf and an optimum moisture content of about 19 percent.

- Zone 5: Rockfill will consist of run of mine waste rock. The lift thickness will be adjusted to accommodate the rockfill materials, but based on the run-of-mine waste rock gradation shown in the Design Basis Memorandum (Appendix A), it is anticipated that lift thickness will be 24 inches. A 10-ton smooth drum vibratory roller will be used to compact the rockfill in the initial stages of the toe dike. A D8 dozer or equivalent will be used to compact subsequent stages of the toe dike rockfill. Four passes of either the roller or dozer will be needed for compaction.

- Zone 6: Filter materials will consist of gravelly sand with a maximum particle size of 1/2-inch, not gap-graded, and with no more than 3 percent finer than the No. 200 Sieve. Gradation requirements are presented below. The material could be crushed, screened and washed from mine waste rock. Alternatively, the filter material could be imported from commercial sources in Libby. The filter material will be placed in 12-inch thick lifts and compacted by 4 passes of a D8 dozer.

Filter Material Gradation Band

Sieve Size	% Passing
1/2"	100
3/8"	97-100
No. 4	85-100
No. 8	65-92
No. 16	36-76
No. 30	12-50
No. 50	2-20
No. 100	0-7
No. 200	0-3

- Zone 7: Gravel drain material will be processed to a maximum particle size of 3 inches from mine waste rock or the gravel will be imported. The material will not be gap-graded and not more than 2 percent will pass the No. 200 Sieve. Gradation requirements are presented below. The material will be placed in 12-inch thick lifts and compacted by 4 passes of a D8 dozer.

Drain Material Gradation Band

Sieve Size	% Passing
3"	100
2"	90-100
1-1/2"	81-100
1"	66-100
3/4"	55-100
1/2"	40-83
3/8"	30-70
No. 4	2-40
No. 8	0-12
No. 16	0-5
No. 200	0-2

- Riprap: Riprap will consist of run of mine waste rock. The material will have a 7-inch average (D_{50}) size and a 24-inch maximum size. The material will be placed by dumping. Riprap will be used for upstream slope protection on the diversion and seepage collection dams. In addition, processed riprap will be used to line the diversion channel. Riprap requirements for the diversion channel are discussed in Chapter 7.

6.4 AVAILABILITY OF CONSTRUCTION MATERIALS

Earthfill will be obtained from the diversion channel and seepage collection pond excavation and from borrow areas (Figure 2). A summary of the availability of construction materials from each borrow area and required excavations and required earthfill volumes is presented in Table 6.5. The depth of excavation in the borrow areas is planned to be less than 20 feet.

A majority of the shell material for constructing the starter impoundment structures will be obtained from required excavations. Additional shell material, if needed, will be obtained from Borrow Area D. Core material will be obtained from Borrow Area A and the seepage collection pond. Transition material will be obtained from Borrow Area B. Further testing and evaluation of the borrow areas will be made in 1990; the final borrow plan will be based on the results of these studies.

Earthfill material for construction of the south and north saddle dams during operation stage will be obtained from Borrow Areas A, B, C and D. The majority (approximately 75 percent) of the earthfill requirement during operation is shell material.

During operations, the tailings retention dam will be constructed primarily of cycloned coarse tailings (underflow). The fine tailings (overflow) will be deposited in the tailings pond. The tailings production schedule was provided by Noranda and is presented in the Design Basis Memorandum in Appendix A. The average in-place densities of coarse and fine tailings were estimated to be 100

TABLE 6.5

AVAILABILITY OF CONSTRUCTION MATERIALS

Material	Required Volume Starter (cy)	Operation (cy)	Total (cy)	Source	Available Volume (cy)	Area (acres)
Core	394,000	416,000	810,000	Borrow Area A	1,164,000	145
				Seepage Collection Pond		
Transition Zone	100,000	135,000	235,000	Borrow Area B	561,000	58
Shell	1,508,000	1,407,000	2,915,000	Diversion Channel & Pond	1,710,000	39
				Borrow Area C	734,000	65
				Borrow Area D	1,605,000	142
Cycloned Sand	0	27,200,000	27,200,000	Coarse Tailings	30,889,000	-
Rockfill	173,000	363,000	536,000	Mine waste rock	See Table 6.7	-
Filter	123,000	468,000	591,000	Mine waste rock	See Table 6.7	-
Drain	103,000	246,000	349,000	Mine waste rock	See Table 6.7	-
Riprap	9,000	0	9,000	Mine waste rock	See Table 6.7	-

pcf and 70 pcf, respectively, based on past experience and published data (Vick, 1983). These values will be confirmed or modified based on the results of laboratory tests on tailings now in progress. The estimated amount of coarse and fine tailings in tons and cubic yards produced each year is presented in Table 6.6. Approximately 30,889,000 cubic yards of coarse tailings and 79,724,000 cubic yards of pond tailings will have been produced by the end of milling operation (year 18).

As directed by Noranda, availability of sand for tailings dam construction was assumed to be 80 percent of the time that tailings are produced. Both coarse and fine tailings will be deposited in the pond during the remaining 20 percent of the time. The pond tailings in Table 6.6 include the fine tailings produced during 80 percent of the time plus the total mill tailings produced during the remaining 20 percent of the time.

Drain and filter material will be obtained by crushing, screening and washing mine waste rock or the material will be imported from a commercial source. Riprap will be obtained from mine waste, required rock excavations, and oversized cobbles and boulders.

TABLE 6.6
ANNUAL PRODUCTION OF SANDS AND POND TAILINGS

	Coarse Tailings		Pond Tailings	
Year	**(tons)**	**(c.y.)**	**(tons)**	**(c.y.)**
1	1,266,000	943,500	2,506,000	2,551,700
2	1,913,000	1,416,700	3,587,000	2,652,400
3-17	2,431,000	1,802,000	4,569,000	4,650,000
18	2,022,000	1,499,200	3,706,000	3,770,500
Totals	41,666,000	30,889,400	78,334,000	79,724,600

6.4.1 Required Excavations

Required excavations include the diversion channel and pond, and the seepage collection pond. The diversion channel excavation will provide approximately 1.8 million cubic yards of shell material and the seepage collection pond excavation will provide about 45,000 cubic yards of Zone 1 material for the starter stage construction. Clayey soils from the diversion channel will be routed to Zone 1. Additional shell material, if needed for starter stage construction, will be obtained from Borrow Area D. A 10 percent loss due to shrinkage was estimated for calculating the fill volume for the excavated material. Shell material will be needed for the starter and diversion dams. This material will consist of weathered bedrock, and a mixture of cobbles, gravel, sand and silty clayey soil. Clayey silty Zone 1 material for the seepage collection dam will be obtained by excavation in the seepage collection pond. Required excavations may also provide core and transition materials if suitable soils are found.

Dozers with rippers will be needed to excavate these materials. Hard zones of bedrock may be encountered in the diversion channel excavation and probably will require limited blasting. Maximum size of the shell material will be 24 inches. Oversized hard, durable rock will be used for riprap.

6.4.2 Borrow Areas

Four borrow areas shown on Figure 2 are proposed to provide construction materials during starter and operation stages. The available volume of soil material and acreage of each borrow area are presented in Table 6.5. A description of each borrow area is given below:

- Borrow Area A: This borrow area is located in the western (upper) part of the impoundment. Borrow Area A will provide core material for the starter and diversion dams during the starter stage and south saddle dam during the operation stage. Estimated quantity of available core material is approximately 1,164,000 cubic yards.

The borrow area covers approximately 145 acres below the final pond level at El. 3700. Test pit logs and laboratory testing show the upper 4 to 11 feet of soils to be a gravelly sandy clay material. Based on visual estimates, approximately 10 percent cobbles and 5 percent boulders were encountered during test pit excavation in the borrow area. Oversized material larger than 6 inches will be raked out during excavation or placement of the fill and used as shell material or riprap.

- Borrow Area B: Borrow Area B is located west of and above the tailings pond. This borrow area will provide transition zone material for the starter and diversion dams during the starter stage and south saddle dam during the operation stage. Estimated quantity of available transition zone material from Borrow Area B is approximately 561,000 cubic yards. The borrow area covers approximately 58 acres.

  Test pit logs and laboratory testing indicate the upper ten feet of soils to consist of silty sand and silty sand gravel. Test pit logs indicate the volume of cobbles and boulders are approximately 15 and 10 percent, respectively, of the borrow material. Oversized material will be removed and used for shell material or riprap. Additional test pits will be excavated during the summer of 1990 in the borrow area to confirm the available quantity and soil type.

- Borrow Areas C and D: Borrow Areas C and D will provide shell material for embankment construction during the operation stage. Estimated material quantity available in Borrow Areas C and D are approximately 734,000 and 1,605,000 cubic yards, respectively. During operation stage, approximately 1,407,000 cubic yards of shell material will be needed for the south and north saddle dams. The total areas of Borrow Areas C and D are 65 and 142 acres, respectively. If clayey soils are found, this material will be used in the core zones of the embankment dams.

A test pit exploration and laboratory testing program are proposed for the borrow areas during the summer of 1990. The exploration will be used to identify the soil types and confirm the quantity and suitability of available materials.

6.4.3 Waste Rock

Rock for drain, filter and riprap will be obtained from mine waste rock. The estimated quantity of rock available and required amount in cubic yards is presented in Table 6.7. Rock is designated as crushed, run of mine and rock stored in mine. Table 6.7 shows that the required volumes can be obtained from crushed rock and run of mine rock. Table 6.8 shows the anticipated gradation of both crushed and run of mine waste rock.

Approximately 235,000 cubic yards of filter, drain and riprap will be needed for the starter stage construction. A majority of filter and drain rock will be obtained from crushed mine waste rock. Approximately 171,000 cubic yards of mine waste rock will be used to construct the toe dike during the starter stage.

During operations, approximately 714,000 cubic yards of filter and blanket drain material will be needed for construction. Filter and drain materials will be needed during years 4 and 5 for construction of the south saddle dam. In addition, the main dam blanket drain and filters will be placed in stages ahead of placement of the coarse tailings. Rockfill to be used for enlargement of the toe dike during operations is approximately 353,000 cubic yards. Placement of rockfill into the toe dike will be done in stages as rock becomes available.

Crushing, screening and washing of mine waste rock to obtain filter and drain materials could be done near the impoundment. Tailings pond water may be used for washing. Sediments from the washing process would be deposited into the tailings pond.

TABLE 6.7
MINE WASTE ROCK SCHEDULE

Year[2]	Project Stage	In-Place Tons (X1000)	Cum. Tons (X1000)	Available Cum. Volume[1] (c.y. X1000)	Required Volume (c.y. X1000)	Rock Designation
1	Evaluation	192	192	101.7	0	Crushed
2		187	379	200.7	0	Crushed
3	Preproduction Development	587	966	511.6	0	Run of Mine
4		535	1501	794.9		Run of Mine
5	Initial Production	0	1501	794.9	425 (Starter Stage)	--
6		664	2165	1146.6		Run of Mine
7		1091	3256	1724.4	1,077 (Operation Stage)	Run of Mine
8	Full Production	537	3793	2008.8		Stored in Mine
9		485	4278	2265.7		Stored in Mine
10		75	4353	2305.4		Stored in Mine
11-17		75/yr.	4878	2583.5		Stored in Mine
≥18		0	4878	2583.5		Stored in Mine
					Total: 1,502	

[1] The dry unit weight of all mine waste rockfill placed and compacted in embankments is estimated to be 140 pcf.

[2] This schedule is for production of mine waste rock only. Tailings storage begins at year 5, which is year 1 in Figures 10 and 17.

TABLE 6.8
WASTE ROCK GRADATIONS

Crushed Rock		Run of Mine Rock	
Size (in.)	**% Passing**	**Size (in.)**	**% Passing**
10	100	24	100
8	90	16	90
7	80	13	80
6	70	10	70
5	60	9	60
4	50	7	50
3	40	5	40
2	30	3	30
1-1/2	20	2	20
-3/4	10	1	10

6.4.4 Tailings

During operations, cycloned coarse tailings (underflow) will provide the construction material for the main tailings impoundment dam. A two stage cycloning process will be used to produce sands for downstream construction of the main tailings dam. The amount of sand is based on a tailings feed consisting of 20 percent solids (pulp density) and 80 percent availability of sands during mill operations. The results of cyclone analyses and grain size gradation curves of the first and second stages underflow are presented in Appendix D. The overflow from second stage will be injected into the mill feed before the tailings enter the first stage. Twenty-six-inch diameter cyclones will be used in the first and second stages. The number of cyclones required for the first stage varies from 5 to 9 during years 1 through 18. The number of cyclones for the second stage varies from 3 to 5 for the 18 years of operation.

The analyses show that the sand recovery from the second stage would be about 43 percent of the mill feed and would contain about 8 percent fines (material finer than 74 microns). Table 6.6 shows the amount of coarse and fine tailings in tons and cubic yards produced each year. The total amount of sand produced is approximately 41,666,000 tons.

Based on an in-place density of 100 pcf, approximately 30,889,000 cubic yards of sands will be available at the end of year 18. Approximately 27,200,000 cubic yards of coarse tailings are required for dam construction. For a starter dam with a crest at El. 3500, the results of the material balance studies show that sufficient sands could be produced to construct the tailings dam to its maximum 370-foot height (El. 3710) at about two years before completion of mill operation. The remaining sands will be used to construct a berm along the crest of the retention dam for reclamation.

As indicated by Noranda, coarse tailings production has been assumed to continue during winter months. During winter, tailings sand would be stockpiled or placed on the embankment if weather permits.

The amount of water required for the cycloning process in addition to the water contained in the tailings slurry from the mill is listed on Table 6.9. Flow charts of the process water cycloning requirements are included in Appendix D.

TABLE 6.9
SUMMARY OF TAILINGS UNDERFLOW

Year	Mill Feed TPH	1st Stage[1] Feed TPH	2nd Stage U/Flow TPH	% Fines U/Flow	% Recovery of Mill Feed	Additional Water Needed for Cycloning (gpm)
1	479	562	200.9	7.5	41.9	503
2	699	810.3	303.9	8.3	43.5	694
3-17	889	1030.0	385.9	8.3	43.4	884
18	728	844.8	321.2	8.6	44.1	706

[1] 1st stage feed is mill feed plus the second stage overflow

6.5 SEEPAGE CONTROL

Seepage through the embankments will be controlled by conventional filter and drain zones, as described below:

- Starter dam: A blanket of clean gravel protected by sand filter layers will be provided under the downstream shell of the dam, as shown on Figure 6.

- Seepage collection dam: This dam will be constructed using local low-permeability materials (Zone 1). The dam will include chimney and blanket sand filter-drains to control and route seepage to the downstream toe.

- South saddle dam: Chimney and blanket sand filter-drains, protected by a transition zone, will be provided downstream of the core and under the downstream shell.

- Diversion dam: During the initial years of the impoundment, seepage through the diversion dam will flow from the diversion pond. As the tailings impoundment fills, the decant pond level and tailings level will eventually reach and exceed the diversion pond level, reversing the direction of seepage. Therefore, the diversion dam has been designed with a central core and progressively coarser outer zones to facilitate drainage toward both embankment toes.

- Tailings dam: A blanket drain will be placed on the foundation as the tailings dam is being raised. The drain will consist of processed clean gravel and will be 2 feet thick in the valley bottom and 1 foot thick at higher elevations (see Figure 4). To prevent piping of the tailings and foundation soils into the gravel drain, 1-foot-thick sand filter blankets will be located above and below the blanket drain (see Figure 8). Filters will also be placed on the upstream face of the toe dike to prevent piping of tailings sand into the rockfill. In addition, because seepage will tend to collect along the bottom of the main drainages, two high-capacity trunk drains will be constructed along the drainages (see Figures 4 & 7). The trunk drains will consist of a 4-foot-thick layer of crushed mine rock completely enveloped by two filter layers.

Seepage through the tailings dam will be routed via ditches to the seepage collection pond. The water in the pond will be pumped back to the tailings impoundment. Any fines that collect against the upstream face of the toe dike will be removed during operations.

- North saddle dam: This low saddle dike includes a transition zone downstream of the core and a coarse downstream shell.

6.6 CONTROL OF FOUNDATION UPLIFT PRESSURES

The available geotechnical data indicate that groundwater within the dam foundation area flows through soil strata that appear to be extensively confined by overlying lower-permeability soils. Artesian pressures have been observed in some of the exploratory boreholes drilled in the impoundment area, particularly along Little Cherry Creek (MKE, 1990). The gradual filling of the tailings impoundment, with associated higher water level, will likely result in gradually increasing pore water pressures in the impoundment foundation soils. Where low permeability soils under the dam and downstream of it overlie more pervious soils or rock, the low permeability soils may act as a confining layer. If sufficiently extensive, such a layer would impede dissipation of impoundment-induced pore pressures in the foundation, possibly resulting in excessive pressures at the toe of the dam. Excessive pressures in the dam foundation would decrease dam stability and could cause sliding or heaving at the toe of the dam. Therefore, it is anticipated that foundation pore pressure relief may be necessary along the toe of the dam.

A preliminary study of alternative pore pressure relief systems was made and the results of the study are summarized in a letter report entitled "Tailings Dam Foundation Pressure Relief System, Preliminary Design," which is included in Appendix E. The main conclusions are as follows:

- The pressure relief system should be designed and installed based on the observational approach. With this approach, a pressure relief system of modest size would be initially installed and a well-planned monitoring system would be implemented. Foundation pore pressures would be regularly monitored and foundation conditions would be inspected visually. Excessive pore pressures and distress that develop in localized areas of the foundation would be immediately treated by installing relief wells and possibly by placing a tailings fill surcharge over the distress areas. This work would be part of the regular maintenance of the impoundment.

- Alternative pressure relief systems that were evaluated included (1) drainage trenches and (2) lines of relief wells. The relief wells were found to be more economical and better suited to the observational approach than the drainage trenches. Therefore, it was recommended that further design efforts be concentrated on developing a relief well system, possibly supplemented with local drainage trenches if appropriate in areas of suitable topography.

- An initial pressure relief system could include a line of 230-foot-deep relief wells at 100- to 200-foot spacing along the toe of the starter dam and a similar line of monitoring wells installed about 300 feet downstream of the relief wells. A plan of this system is shown on Sketch 4 of Appendix E. The relief and monitoring well spacings and depths are preliminary and should be confirmed or modified based on the results of additional field investigations as described below.

- Additional field explorations and analysis will be required before the design of the pressure relief system can be finalized. The explorations will include drilling, packer tests and pump tests along the proposed relief well alignment in order to (1) determine depth of confining stratum in the dam foundation, (2) determine aquifer thickness and characteristics, and (3) confirm or modify the proposed pressure relief system location and dimensions. The final analysis and design will be based on the results of the additional field investigation.

6.7 INSTRUMENTATION

The proposed dam instrumentation is shown on Figures 9A and 9B and will consist of the following:

1. Pneumatic Piezometers: Pneumatic piezometers will be placed at selected locations of the main tailings dam and south saddle dam to measure pore pressures at the base of the dams.

   Four evenly spaced transverse sections in the main tailings dam will be provided with two to three pneumatic piezometers. A transverse section is located near each abutment of the starter dam and a third in the maximum section. The fourth section is located in the right-third of the starter dam.

   Piezometers will be located near the downstream toe of the starter dam and near the upstream toe of the rockfill toe dike. Additional piezometers may be installed halfway between the upstream and downstream piezometers.

   Two pneumatic piezometers will be installed in the maximum section of the south saddle dam. One piezometer will be located near the downstream side of the core foundation contact and the other at the downstream toe of the dam.

All pneumatic piezometers will be installed at the foundation contact with the bottom layer of the blanket drain. The piezometers and pressure lines will be installed in an 18-inch deep trench to protect the piezometers from damage during construction of the blanket drain.

The pneumatic piezometers are operated by fluid or air pressure acting on a diaphragm. A pressure line is the feed and the return line is connected to a detector in a readout unit. The readout will be on a bourdon gauge or a digital display. Each transverse section will be connected to a central readout station. The end of the feed line and readout unit will be placed in a protective enclosure to protect the equipment from damage during construction and vandalism.

2. <u>Seepage Measurement Weir</u>: All seepage emerging at the downstream toe of the main tailings dam will be collected in ditches and routes to a seepage measure weir location in Little Cherry Creek. The quantity of water flow will be metered over a 12-inch, 90-degree V-notch. The flow will consist of seepage and water decanted from the tailings underflow slurry. The decanted flow can be estimated and subtracted from the total flow to determine the seepage rate.

3. <u>Pressure Relief Wells</u>: Pressure relief wells may have to be installed in the foundation beneath the main tailings dam, as described in Section 6.6. The depth and location of the relief wells are to be determined. A typical relief well detail is shown on Figure 9B. Water from the relief wells will be discharged into the blanket drain.

4. <u>Monitoring Wells</u>: Monitoring wells will be installed in the tailings dam foundation downstream of the relief wells to measure the foundation piezometric pressures. The monitoring wells will become relief wells once they are covered by the embankment. A permanent line of monitoring wells will be installed along the downstream toe of the final tailings dam.

The monitoring wells are similar to the relief wells. A removable cap and pressure gauge, instead of a tee, will be placed at the well collar.

6.8 RECLAIMED IMPOUNDMENT

Following the end of milling operations, the tailings impoundment will, be reclaimed. A reclamation plan, based on Montana Department of State Lands regulations, will be implemented, on a continuing basis, to maintain the vegetative cover, soil stability, drainage and safety conditions appropriate to the subsequent use of land. The intent of the proposed reclamation plan is to minimize potential long-term erosion and maximize stability of the tailings embankment dam. A description of the construction considerations during reclamation is presented in Chapter 9 of the Preliminary Engineering Report (MKE, 1989). A plan of the reclaimed impoundment, sections and details are shown on Figures 15 and 16.

A coarse tailings berm will be formed along the south and east sides of the impoundment dam crest. The impoundment surface will be graded downward from the berms toward the northwest at a slope of 0.5 to 1.0 percent, along the natural slope of the tailings surface. Two drainage swales will be graded on the tailings surface, one beginning at the southeastern portion of the impoundment and the other at the diversion dam. Surface water will drain from the impoundment to the Bear Creek drainage. The small north saddle embankment will be graded to allow drainage from the impoundment. Drainage ditches sloping toward the northwest corner of the impoundment will be excavated along the north and west edge of the impoundment. The ditches will intercept off-pond runoff to minimize the amount of water infiltrating the pond.

The final tailings pond surface and downstream face of the embankment dam will be covered with 18- and 24-inch thick layers of the stockpiled subsoil, respectively. Soils with a higher percentage of coarse fragments will be placed on the downstream face of the embankment dams. The soil layer will be vegetated with grass to minimize erosion due to wind and surface runoff.

6.9 GENERAL CONSTRUCTION SCHEDULE

The bar schedule shown on Figure 17 outlines the main construction activities from start-up through impoundment reclamation. The activities are briefly discussed below.

1) Mobilization and Site Preparation

Upon completion of mobilization, site preparation work will commence. Site preparation includes removal of all trees and shrubs within the impoundment area footprint (including the footprint of all dams), the diversion channel and impoundment, the seepage collection impoundment, seepage collection ditches, borrow areas located outside the impoundment, cyclone plant, and all required access and haul roads.

2) Foundation Preparation for Starter Stage Dams

Foundation preparation will begin with the diversion dam and starter dams. Either concurrent to or immediately following the above work, foundation preparation for the seepage collection dam and toe dike will commence.

3) Construction of the Diversion Dam and Channel

Excavation of the diversion channel will commence. Excavated material will be used for construction of the diversion and starter dams and/or stockpiled as required.

4) Opening of Borrow Areas

Borrow areas will be opened and production of fill materials will commence. Materials for different earthfill dam zones will be either hauled directly and placed at its final destination or stockpiled for later placement.

5) Construction of Starter Dam, Toe Dike and Seepage Dam

Once the borrow areas have been opened and material is available, construction of the starter dam, toe dike and seepage collection dam will begin.

Upon completion of the seepage collection and starter dams, seepage collection ditches will be constructed to channel seepage from the starter dam once tailings placement begins.

6) Commencement of Tailings Disposal

As the mine becomes operational, tailings slurry will be produced at the mill and conveyed through pipelines approximately six miles to the cyclones at the impoundment site. From the cyclone plant, the sand tailings will be conveyed to the starter dam crest to begin construction of the tailings dam. Fine tailings will be pumped to the impoundment for storage.

7) Construction of South Saddle Dam

As the tailings impoundment begins to rise, construction of the south saddle dam will commence. The south saddle dam has to be started before the tailings impoundment surface reaches El. 3590, which is anticipated to occur during year six of the operational life of the impoundment.

8) Construction of the North Saddle Dam

The north saddle dam should be constructed before the tailings impoundment rises to El. 3670, which is anticipated to occur during year fourteen of the operational life of the impoundment.

9) Completion of Impoundment and Reclamation

When the tailings dam crest reaches El. 3710 and tailings dam construction is completed, both coarse and fine tailings will be pumped into the impoundment. At the completion of the operational life of the impoundment, the top of the impoundment area will be graded as shown on Figure 15 to promote drainage to the northwest and seeded.

CHAPTER 7
DIVERSION CHANNEL

7.1 GENERAL DESCRIPTION

The tailings impoundment will be developed by constructing a dam across Little Cherry Creek. The watershed area above the proposed tailings dam is 1.78 square miles. A permanent diversion system consisting of a dam at the upstream end of the impoundment and a diversion channel will be constructed to route the creek around the south side of the impoundment and reduce the watershed area tributary to the tailings impoundment; the diversion channel will lead to a natural stream channel. The diversion system is shown on Figure 11.

As explained in Chapter 4, the diversion system reduces the tailings impoundment drainage area to 0.88 square miles. The drainage area tributary to the diversion channel is 1.10 square miles, of which 0.90 square miles is the catchment of the diversion pond and 0.20 square miles is the watershed intercepted along the channel excavation.

In addition to the permanent diversion system, temporary diversion ditches will be excavated around the tailings impoundment to minimize runoff into the impoundment during operation. As the impoundment fills, new ditches will be excavated farther uphill.

7.2 DIVERSION CHANNEL SYSTEM

7.2.1 Diversion Pond

The diversion pond, formed by a 95-foot-high diversion dam, will route Little Cherry Creek into the diversion channel. The pond volume between the diversion channel crest elevation and the dam crest is 317 acre-feet. This volume includes about 170,000 cubic yards of impoundment excavation. The excavation, shown on Figure 11, will yield borrow material for the diversion dam and improve the hydraulic approach to the diversion channel inlet. In addition, by increasing the pond volume, the excavation will increase the regulating effect of the pond. The area-capacity curves for the pond are shown in Appendix B, Figure B-3.

7.2.2 Diversion Channel

As indicated in Chapter 4, the diversion channel was designed to pass the 6-hour local storm PMF. Results of the PMF routing are shown in Table 4.3. The computed PMF maximum water level at the inlet is at El. 3691.4, leaving 18.6 feet of freeboard below the crest of the diversion dam.

The diversion channel is about 4,000 feet long with a 2.4 percent gradient. The cross-sectional area is trapezoidal, with an excavated bottom width of 20 feet and 2(H):1(V) sideslopes (Figure 12). Benches are provided on the slopes at 30-foot intervals to facilitate access and maintenance. The channel slope and cross-sectional area are adequate to pass the PMF with a minimum of 2 feet freeboard.

The excavated channel will be lined with riprap for erosion protection. The riprap (Figure 12) will form a low-flow channel within the excavated channel and is designed to resist erosion from the routed 100-year flood event. Larger floods could cause some erosion damage and require maintenance, but would not threaten the tailings dam (Figure 13). The riprap consists of a 2-foot-thick rock layer underlain by a 6-inch gravel bedding. The rock will be well-graded with a minimum average (D_{50}) size of 7 inches. Suitable riprap can be obtained by screening run-of-mine waste rock through a 3-inch grizzly. The bedding will be a well-graded sandy gravel. The undersize of the riprap grizzly operation is likely to be adequate as gravel bedding. Suitable riprap and bedding gradation ranges are presented in Table 7.1. Approximately 12,000 c.y. of riprap and 3,500 c.y. of bedding will be required.

Most of the diversion channel will be excavated into weathered rock. Where fine-grained soils are exposed in the channel subgrade, the proposed gravel bedding would not provide adequate filter protection. Therefore, wherever such soils are encountered in the subgrade during excavation, a filter fabric will be placed over the subgrade before placing the gravel bedding.

TABLE 7.1
RIPRAP AND BEDDING GRADATIONS

Riprap		Bedding	
Size (in.)	**% Passing**	**Size (in.)**	**% Passing**
24	100	3	100
12	50 - 100	1-1/2	45 - 85
8	30 - 70	3/4	10 - 45
6	15 - 40	3/8	0 - 25
3	0 - 10	No. 4	0 - 15

The excavated slopes above the edge of the riprap will be protected by mulching and seeding with native vegetation. Additional erosion protection, with riprap or gabion mattresses, will be provided where streams flow over excavated slopes.

7.2.3 Outlet Structure

The outlet section of the channel consists of a drop structure with a stair-step configuration leading to a natural stream channel (Figure 14). The drop structure will be constructed using 3 x 3-foot gabions to form the steps and 12-inch thick gabion mattresses to provide erosion protection for the invert and slopes. Design flood flow velocities were based on the 100-year event.

Rock for the gabions will grade from 4 inches to 8 inches in size and will be obtained by processing mine waste rock. Approximately 1,500 c.y. of rock will be required.

The natural stream channel downstream of the diversion channel is heavily timbered. The current condition should provide considerable erosion protection against the diverted flows. Therefore, initially, erosion protection is not planned for the stream channel. Erosion protection will be constructed if erosion is observed during operations. Such protection would consist of rockfill check dams to dissipate flow energy in the areas where damage occurs.

CHAPTER 8
MONITORING

8.1 GENERAL

A program will be established to monitor the performance of the impoundment facilities. The program includes periodic visual inspections, reviews by engineers experienced in tailings dam design and construction, and measuring embankment seepage and foundation piezometric pressures.

8.2 VISUAL INSPECTIONS

Weekly visual inspections will be made to observe and document the conditions of the embankment dams, diversion channel and abutments. A checklist will be prepared as a guide for systematically inspecting the site facilities. The checklist will include the following:

- Freeboard Adequacy
- Tailings Beach Width
- Embankment Dams and Abutments:
  - Cracking, sloughing, depressions and erosion
  - Changing trends in seepage quantities, turbid seepage (piping) and wet spots on abutments and on downstream faces of embankments
  - Conditions of piezometer terminals and standpipe piezometers
- Diversion Channel:
  - Cracking, sliding and erosion
  - Condition of riprap
  - Debris and vegetation in channel

The checklist will be completed by designated operations personnel trained in the inspection and maintenance of tailings dams. The inspectors will report their findings to management. The visual inspection records will also be reviewed annually by tailings dam engineers. After impoundment closure, the frequency of inspections will be gradually decreased.

8.3 INSTRUMENT READINGS

The pore water pressures in the tailings dam foundation and downstream of the dam will be measured at monitoring wells during operation and for several years after closure of the impoundment. In addition, as described in Section 6.7, the pore pressures within the dam blanket drains will be measured with pneumatic piezometers along four transverse sections of the tailings dam and one section of the south saddle dam. The leads from these piezometers will be extended to read-out terminals at the downstream toes of the dams. A V-notch weir will be located at the downstream toe of the tailings dam to monitor seepage rates.

The seepage rate and the water pressures in monitoring wells and piezometers will be measured weekly during operations. The monitoring data will be evaluated annually by a tailings dam engineer to confirm that the dam is performing as planned and to make recommendations for remedial actions if foundation pressures are higher than anticipated. This data will be used to determine the necessity of installing additional pressure relief wells to reduce high foundation pore pressures.

8.4 CONTINGENCY PLAN

With the construction of any type of impoundment structure, there is always the possibility, however remote, of partial or total failure. Therefore, a contingency plan is required to (1) identify the type of incident, (2) formulate the required remedial action and (3) identify the authorities to be contacted.

Actual remedial actions would be determined from the specific conditions of the failure incident. The plant manager or his designate will be contacted in the event that any unusual condition is observed. The plant manager will notify the tailings dam engineer. If, after consultation with the engineer, the condition is considered to be sufficiently serious, the plant manager will notify the U.S. Forest Service in Libby and the Department of State Lands in Helena.

After initial remedial actions have been identified and implemented, an engineering analysis will be conducted to determine the cause of the problem and procedures for long-term stabilization.

CHAPTER 9
REFERENCES

1. Administrative Rules of Montana, Chapter 4, Reclamation, Part 3 - Metal Mine Reclamation and Sub-Chapter 1, Rules and Regulations Governing the Montana Metal Mine Reclamation Act, 1 July 1980.

2. Chen-Northern, Montana Project, Hydrology Investigation Report, 1989.

3. Edris, E.V., Jr. and S.G. Wright, "User's Guide: UTEXAS2 Slope Stability Package," University of Texas at Austin, Department of Civil Engineering and Department of the Army, Waterways Experiment Station, Corps of Engineers, August 1987.

4. Fetter, Jr., C. W., "Applied Hydrogeolgy," Charles E. Merrill Publishing Company, 1980.

5. Hydrometrics, Tailing Impoundment Seepage Analysis, ASARCO Rock Creek Project, June 1987.

6. Lawrence Livermore Laboratory, Geoscience Data Base, Handbook for Modeling a Nuclear Waste Repository, Vol. 1, NUREG/CR-0912, December 1979.

7. Leps, T. M., "Review of Shearing Strength of Rockfill," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 96, SM4, July 1970, pp. 1159-1170.

8. "Liquefaction of Soils During Earthquakes," Committee on Earthquake Engineering, National Academy of Sciences, 1985.

9. Morrison-Knudsen Engineers, Inc., Montanore Project, Geotechnical report Tailings Impoundment Site, 2 Volumes, March 1990.

10. Morrison-Knudsen Engineers, Inc. (MKE), Montana Project, Preliminary Engineering Report, 1989.

11. Noranda, "Application for a Hard Rock Operating Permit and Proposed Plan of Operation," Montana Project, Lincoln and Sanders Counties, Montana, Volume 2, March 1989.

12. Seed, H.B., "The Selection of Design Earthquakes for Critical Structures," Bulletin of the Seismological Society of America, Vol. 72, No. 6, December 1982.

13. Seed, H.B., "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, GT2, February 1979.

14. Seed, H.B. and I.M. Idriss, "Ground Motions and Soil Liquefaction During Earthquakes," Earthquake Engineering Research Institute, 1982.

15. Seed, H.B. and G.R. Martin, "The Seismic Coefficient in Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 92, SM3, 1966.

16. Siegel, R.A., "STABL User Manual," Joint Highway Research Project, Purdue University, School of Civil Engineering, and Indiana Department of Highways, June 1975.

17. Siegel, R.A., "STABL User Manual," Joint Highway Research Project, Report JHRP-75-9, June 1988.

18. U.S. Corps of Engineers, "Engineering and Design Stability of Earth and Rock-Fill Dams," EM 1110-2-1902, April 1970, and Change 1, 17 February 1982.

19. U.S. Corps of Engineers, Manual EM-1110-2-1406, "Runoff from Snowmelt," 1960.

20. U.S. Corps. of Engineers, "Recommended Guidelines for Safety Inspection of Dams," National Program of Inspection of Dams, 1977.

21. U.S. Department of Agriculture, "Soils Information," U.S. Forest Service, 1984.

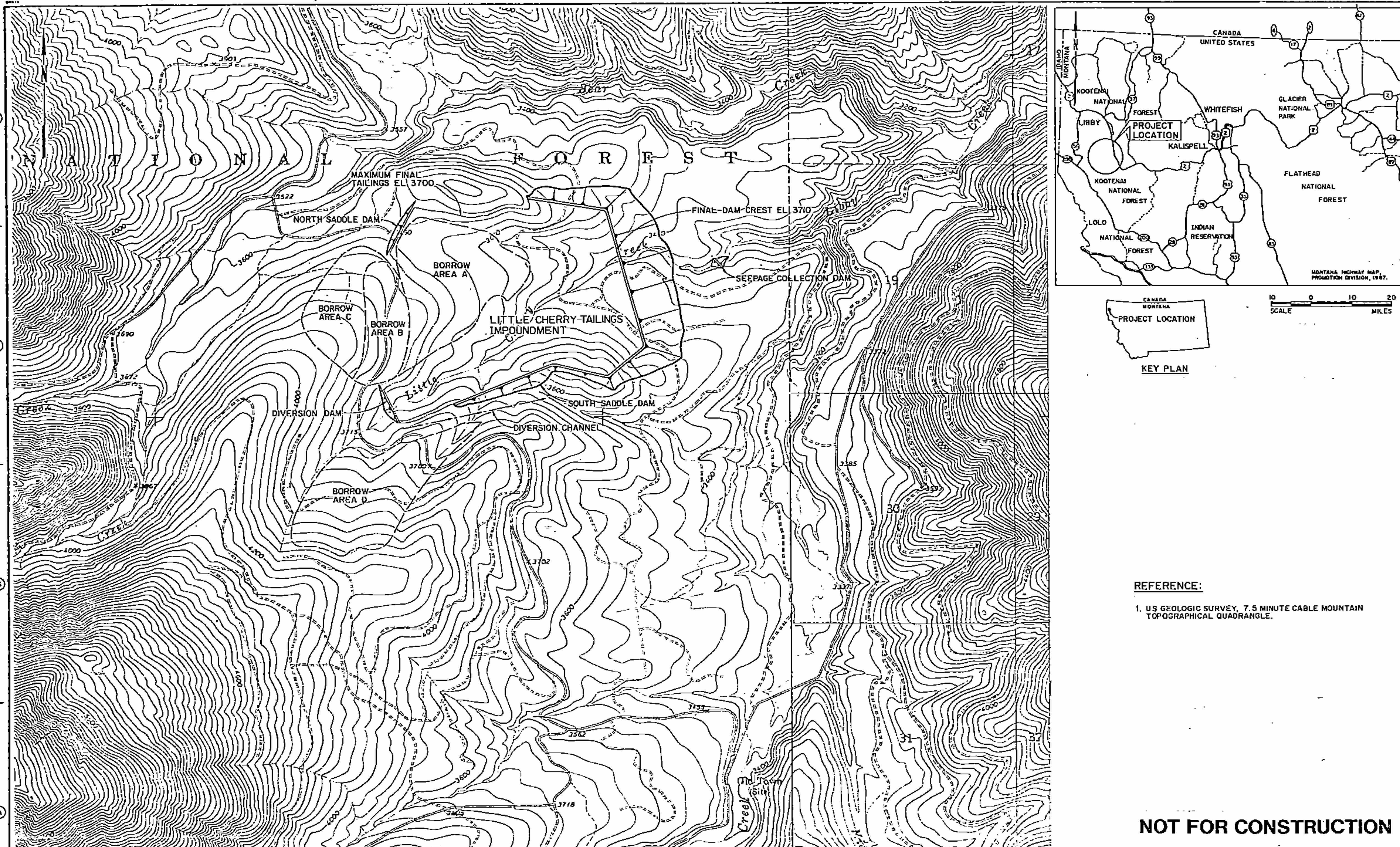
22. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, "NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume I-Montana," 1973.

23. U.S. Department of Commerce, Weather Bureau, "Hydrometeorological Report No. 43, Probable Maximum Precipitation, Northwest States," 1966.

24. U.S. Department of the Interior, Bureau of Reclamation, "Design of Small Dams," 1987.

25. Vick, S.G., "Planning, Design and Analysis of Tailings Dams," John Wiley & Sons, 1983.

FIGURES



REFERENCE:

1. US GEOLOGIC SURVEY, 7.5 MINUTE CABLE MOUNTAIN TOPOGRAPHICAL QUADRANGLE.

NOT FOR CONSTRUCTION

SCALE 1000 0 1000 2000 FEET

NO.	DATE	REVISIONS	BY	CHK	APPD

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

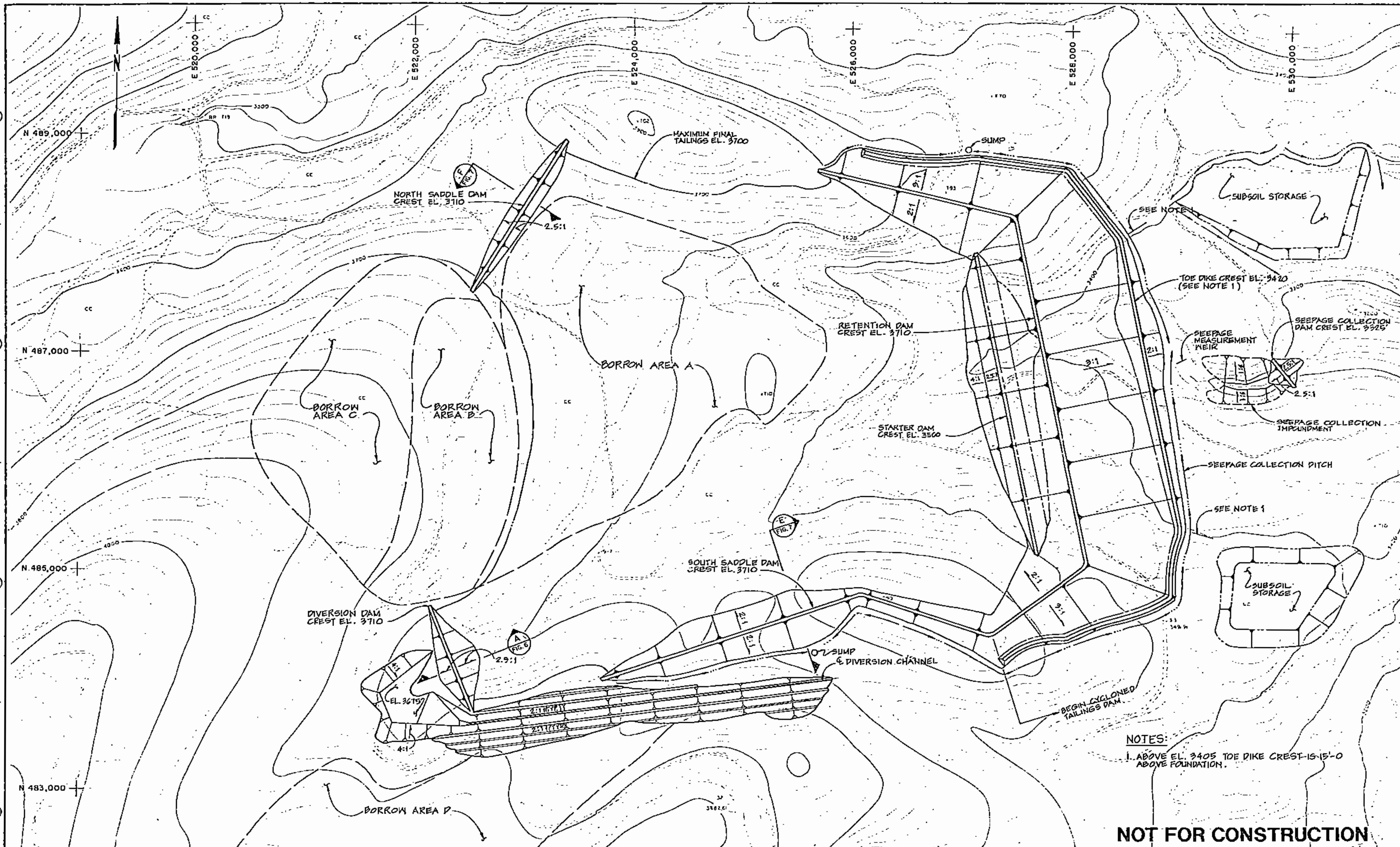
DESIGNED JCP | DRAWN CCR | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

LOCATION MAP

FIGURE 1



NOTES:

1. ABOVE EL. 3405 TOE DIKE CREST IS 15'-0 ABOVE FOUNDATION.

NOT FOR CONSTRUCTION

400 0 400 800
SCALE FEET

NO	DATE	REVISIONS	BY	CHK	APP'D

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

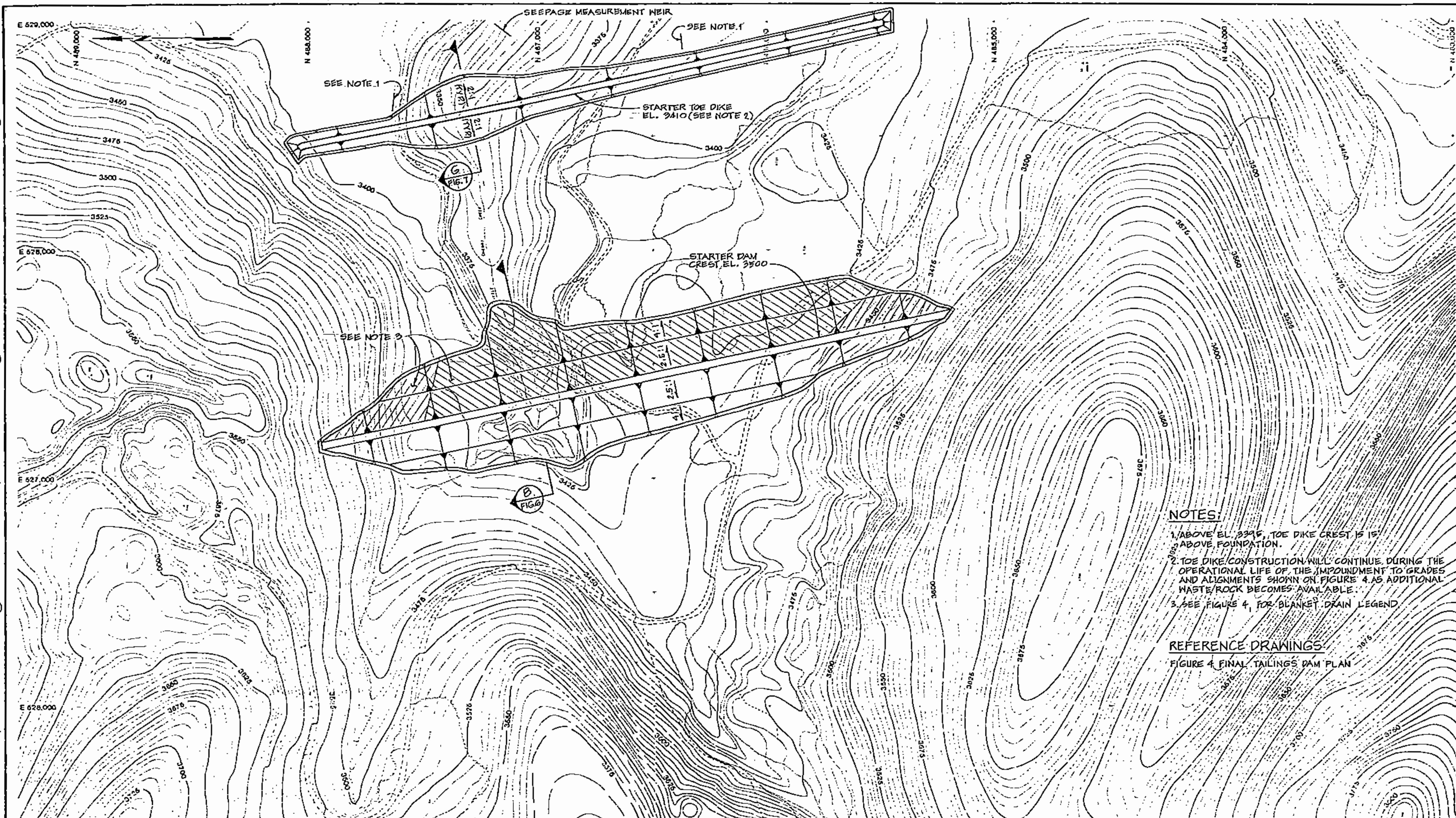
DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

SITE PLAN

FIGURE 2



NOTES:

1. ABOVE EL. 3395, TOE DIKE CREST IS 15' ABOVE FOUNDATION.
2. TOE DIKE CONSTRUCTION WILL CONTINUE DURING THE OPERATIONAL LIFE OF THE IMPOUNDMENT TO GRADES AND ALIGNMENTS SHOWN ON FIGURE 4 AS ADDITIONAL WASTE ROCK BECOMES AVAILABLE.
3. SEE FIGURE 4, FOR BLANKET DRAIN LEGEND.

REFERENCE DRAWINGS:

FIGURE 4 FINAL TAILINGS DAM PLAN

NOT FOR CONSTRUCTION

200 0 200 400
SCALE FEET

NO.	DATE	REVISIONS	BY	CHK	APPD

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP	DRAWN JMM	CHECKED MPF	RECOMMENDED
DATE JUNE 1990			APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

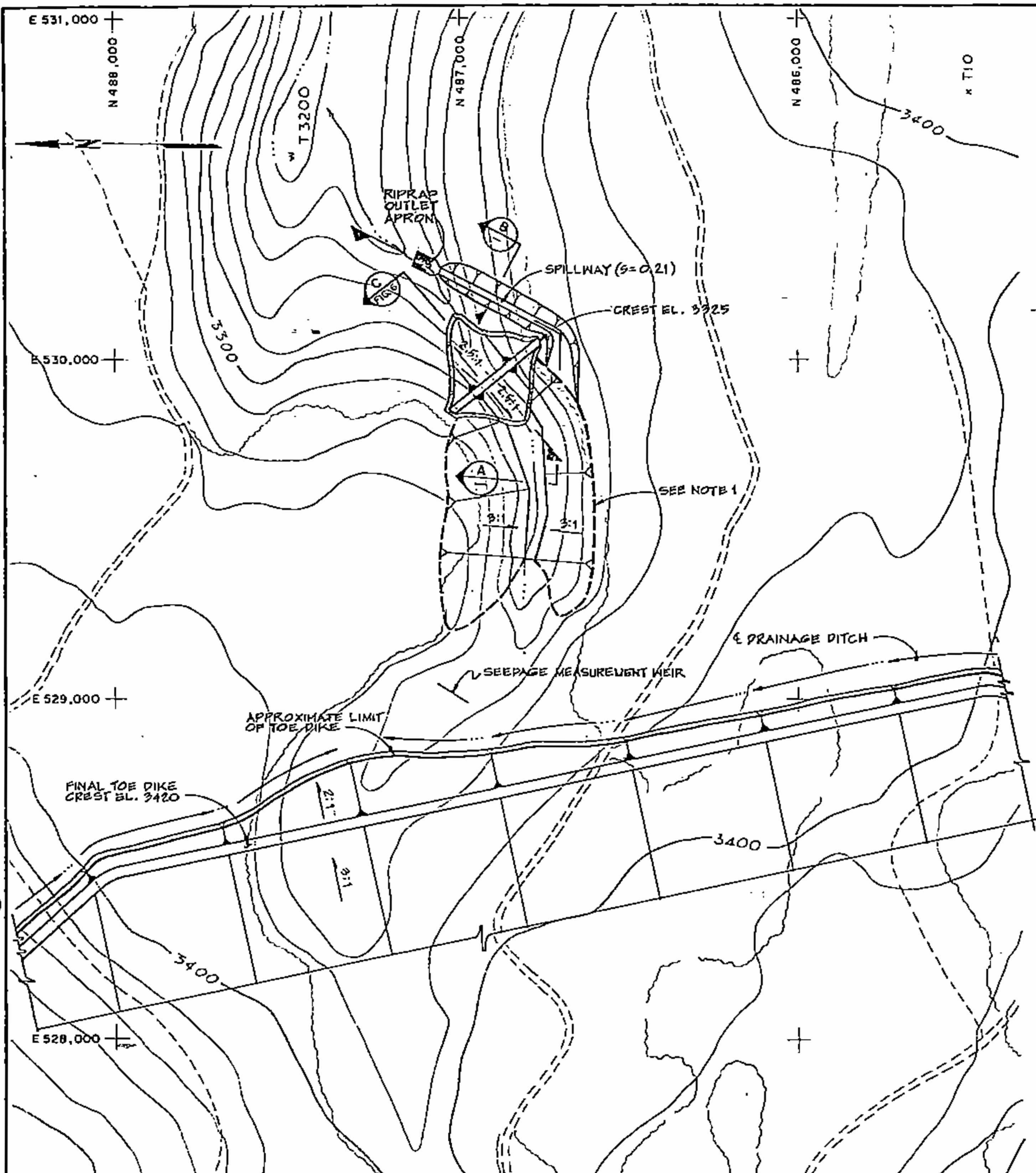
STARTER DAM AND TOE DIKE PLAN

FIGURE 3

NOT FOR CONSTRUCTION

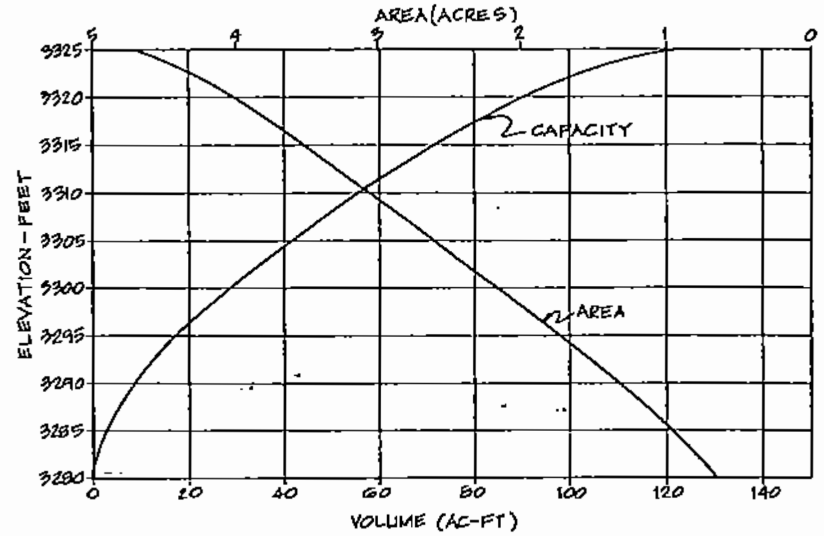
SCALE 200 0 200 400 FEET	
MORRISON-KNUDSEN ENGINEERS, INC. A MORRISON KNUDSEN COMPANY 180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105	DESIGNED JCP DRAWN CCR CHECKED MPF RECOMMENDED DATE JUNE 1990 APPROVED
NORANDA MINERALS CORPORATION	
MONTANORE PROJECT	FINAL TAILINGS DAM PLAN
FIGURE 4	

NO	DATE	REVISIONS	BY	CHK	APPD

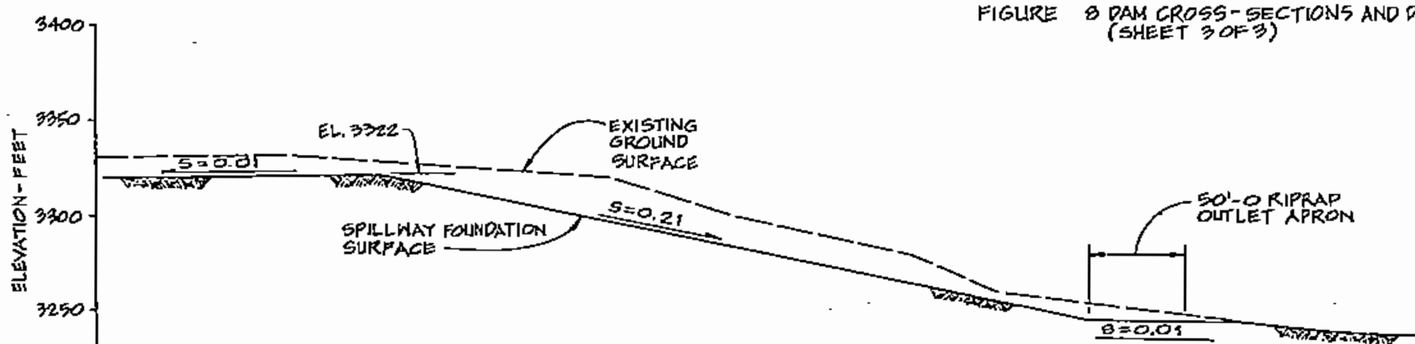


PLAN

200 0 200 400
SCALE FEET

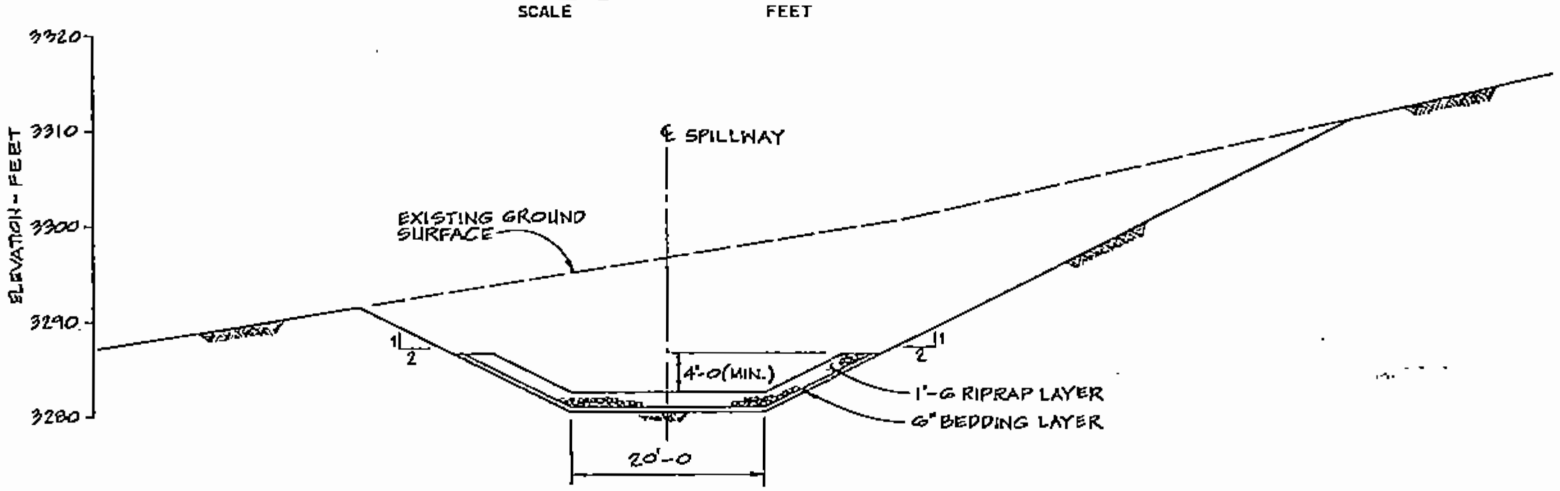


SEEPAGE COLLECTION IMPOUNDMENT AREA-CAPACITY CURVES (SEE NOTE 2)



EMERGENCY SPILLWAY
SECTION A

50 0 50 100
SCALE FEET



SPILLWAY
SECTION B

10 0 10 20
SCALE FEET

NOTES:

1. BORROW MATERIAL FOR SEEPAGE COLLECTION DAM CONSTRUCTION MAY BE OBTAINED FROM THIS AREA AS APPROVED BY ENGINEER. CUT SLOPES SHOWN ARE PRELIMINARY AND WILL BE DETERMINED IN THE FIELD BASED ON MATERIAL REQUIRMENTS AND AVAILABILITY.
2. AREA-CAPACITY CURVES SHOWN REPRESENT EXISTING GROUND SURFACE CONDITIONS AND ARE APPROXIMATE.

REFERENCE DRAWINGS:

FIGURE 2 SITE PLAN
FIGURE 6 DAM CROSS-SECTIONS AND DETAILS (SHEET 1 OF 3)
FIGURE 8 DAM CROSS-SECTIONS AND DETAILS (SHEET 3 OF 3)

NOT FOR CONSTRUCTION

NO.	DATE	REVISIONS	BY	CHK	APPD

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP	DRAWN JMM	CHECKED MPF	RECOMMENDED
DATE JUNE 1990			APPROVED

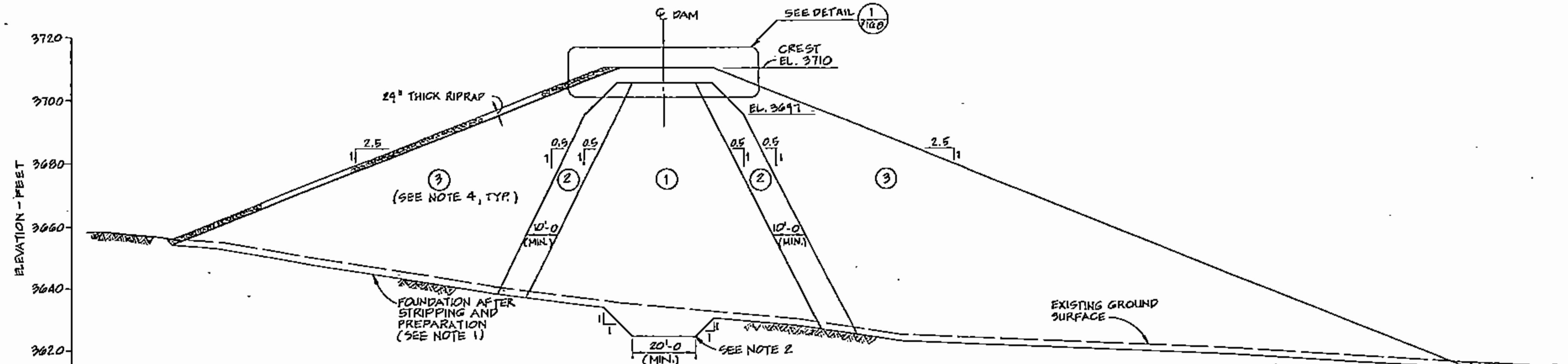
NORANDA MINERALS CORPORATION

MONTANORE PROJECT
SEEPAGE COLLECTION IMPOUNDMENT AND DAM PLAN AND AREA-CAPACITY CURVES

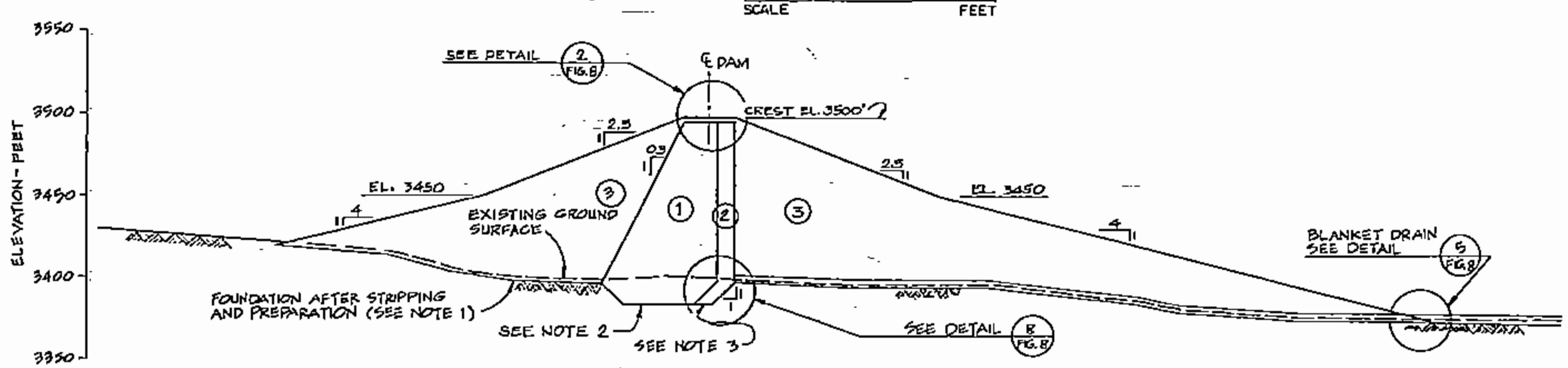
FIGURE 5

NOTES:

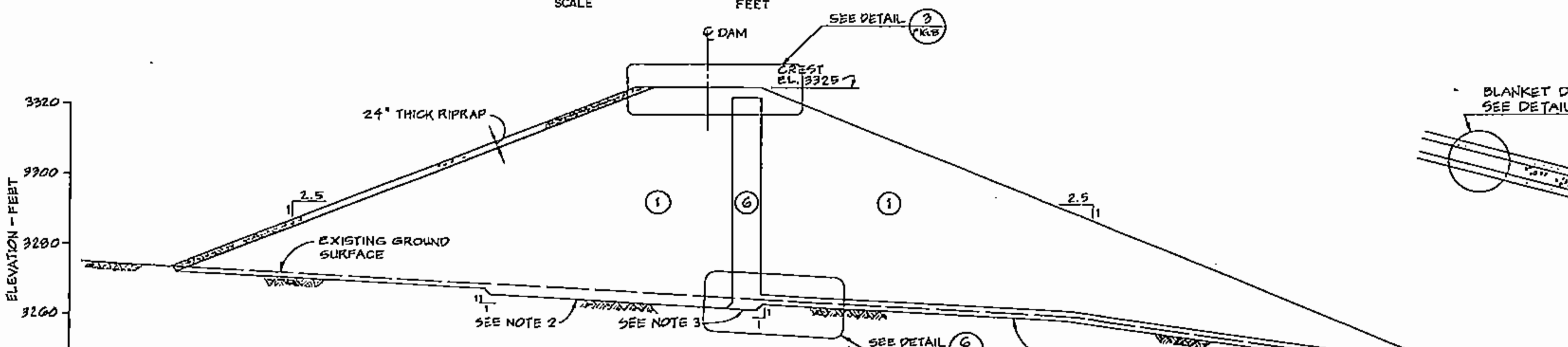
1. STRIPPING AND FOUNDATION PREPARATION SHALL BE DONE AS REQUIRED BY THE ENGINEER.
2. A. CORE TRENCH SHALL EXTEND THROUGH PERVIOUS SOILS AS REQUIRED BY THE ENGINEER.
   B. WHERE CORE IS FOUNDED ON BEDROCK, FOUNDATION SHALL BE CLEANED WITH AIR/WATER JETS.
   C. SLUSH GROUT SHALL BE APPLIED TO JOINTS AND DEPRESSIONS FILLED WITH DENTAL CONCRETE AS REQUIRED BY ENGINEER.
3. ZONE 2 (OR ZONE 6) TRANSITION SHALL EXTEND INTO CORE TRENCH IN AREAS OF COARSE FOUNDATION SOILS AS REQUIRED BY ENGINEER.
4. COARSE MATERIALS SHALL BE ROUTED TO OUTSIDE SLOPES.
5. FOR TRUNK DRAIN NO. 1, t = 40 FEET. FOR TRUNK DRAIN NO. 2, t = 10 FEET.



DIVERSION DAM
SECTION A FIG.2



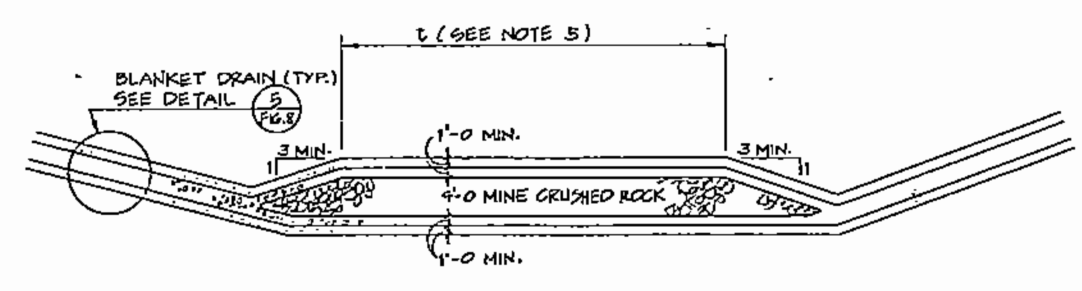
STARTER DAM
SECTION B FIG.3



SEEPAGE COLLECTION DAM
SECTION C FIG.5

LEGEND:

ZONE	DESCRIPTION
1	GRAVELLY SANDY CLAY OR SANDY CLAYEY SILT
2	SILTY SANDY GRAVEL AND COBBLES
3	WEATHERED BEDROCK, MIXTURE OF COBBLES, GRAVEL, SAND AND CLAYEY SOIL
4	CYCLONED TAILINGS SAND
5	RUN OF MINE WASTE ROCK
6	SAND FILTER
7	GRAVEL DRAIN



TYPICAL TRUNK DRAIN
DETAIL 10 FIG.9

NOT FOR CONSTRUCTION

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED

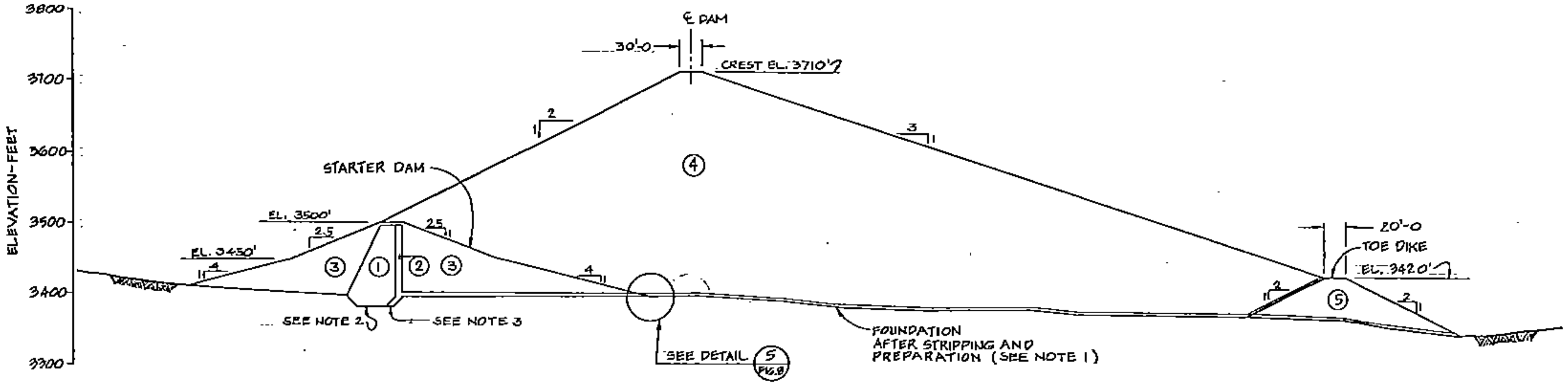
NORANDA MINERALS CORPORATION

MONTANORE PROJECT
DAM CROSS-SECTIONS AND DETAILS
(SHEET 1 OF 3)

FIGURE 6

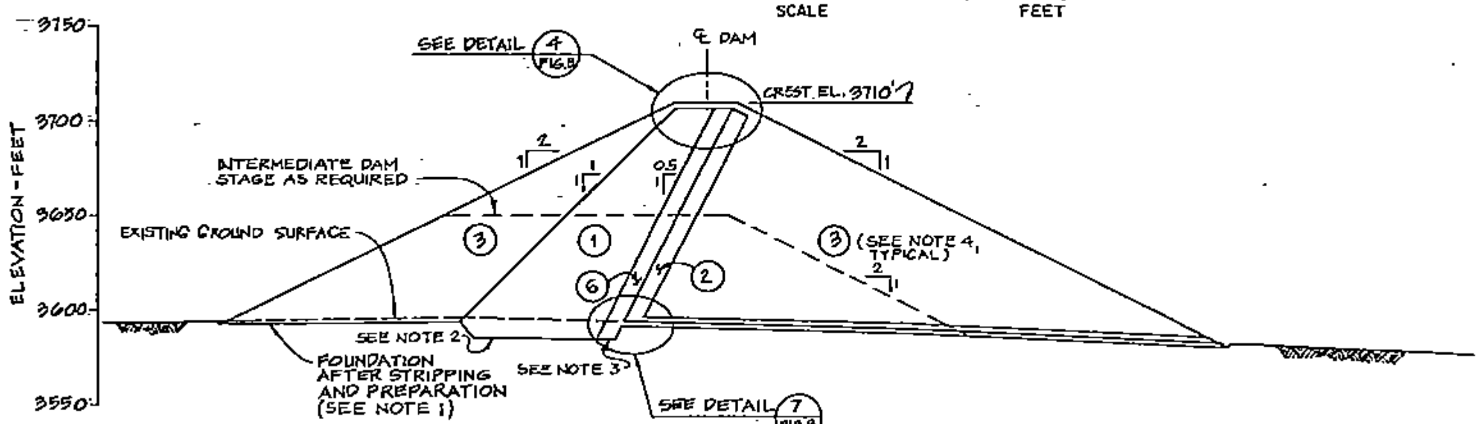
NOTE:

FOR NOTES AND LEGEND SEE FIGURE 6



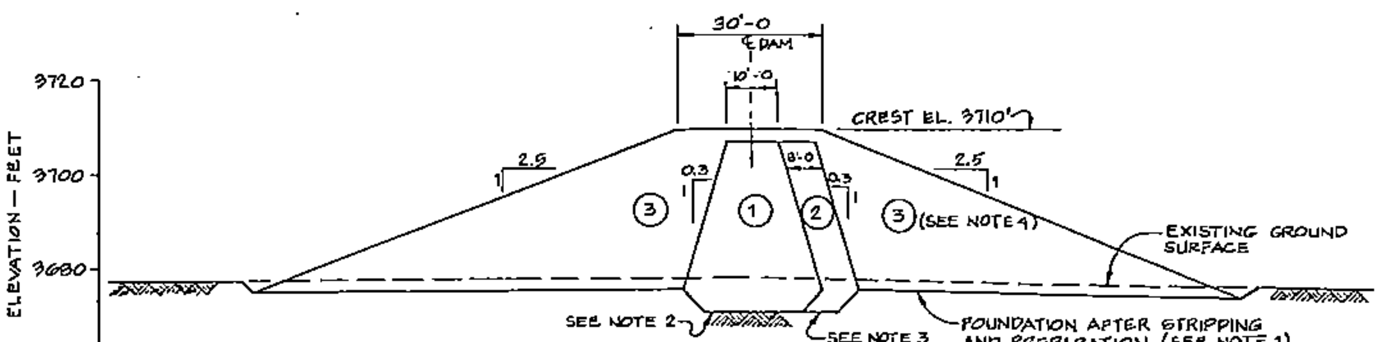
TAILINGS RETENTION DAM
SECTION D (FIG. 4)

SCALE 100 0 100 200 FEET



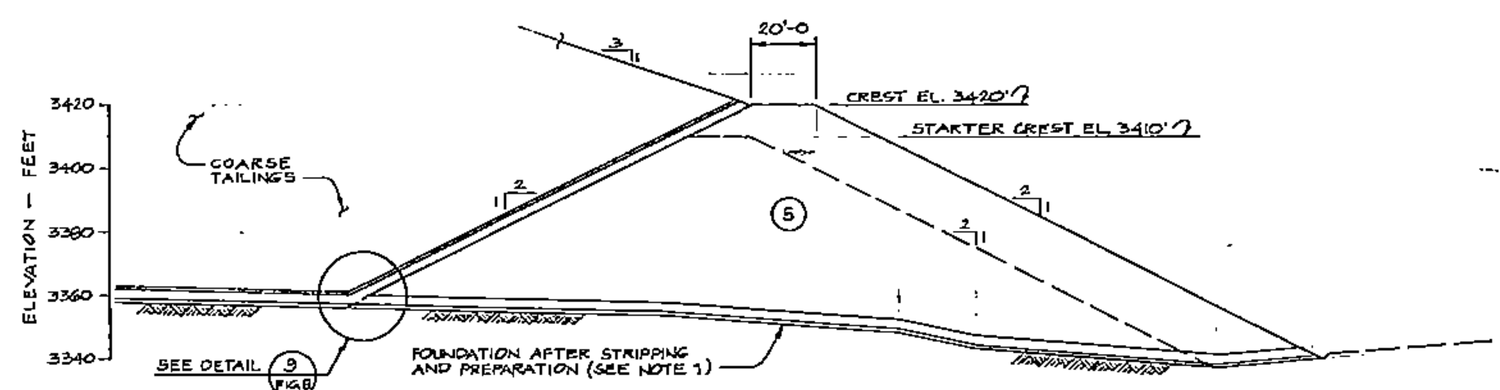
SOUTH SADDLE DAM
SECTION E (FIG.2)

SCALE 50 0 50 100 FEET



NORTH SADDLE DAM
SECTION F (FIG.2)

SCALE 20 0 20 40 FEET



TOE DIKE SECTION G (FIG.3,4)
SCALE: 1" = 30'-0

NOT FOR CONSTRUCTION

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

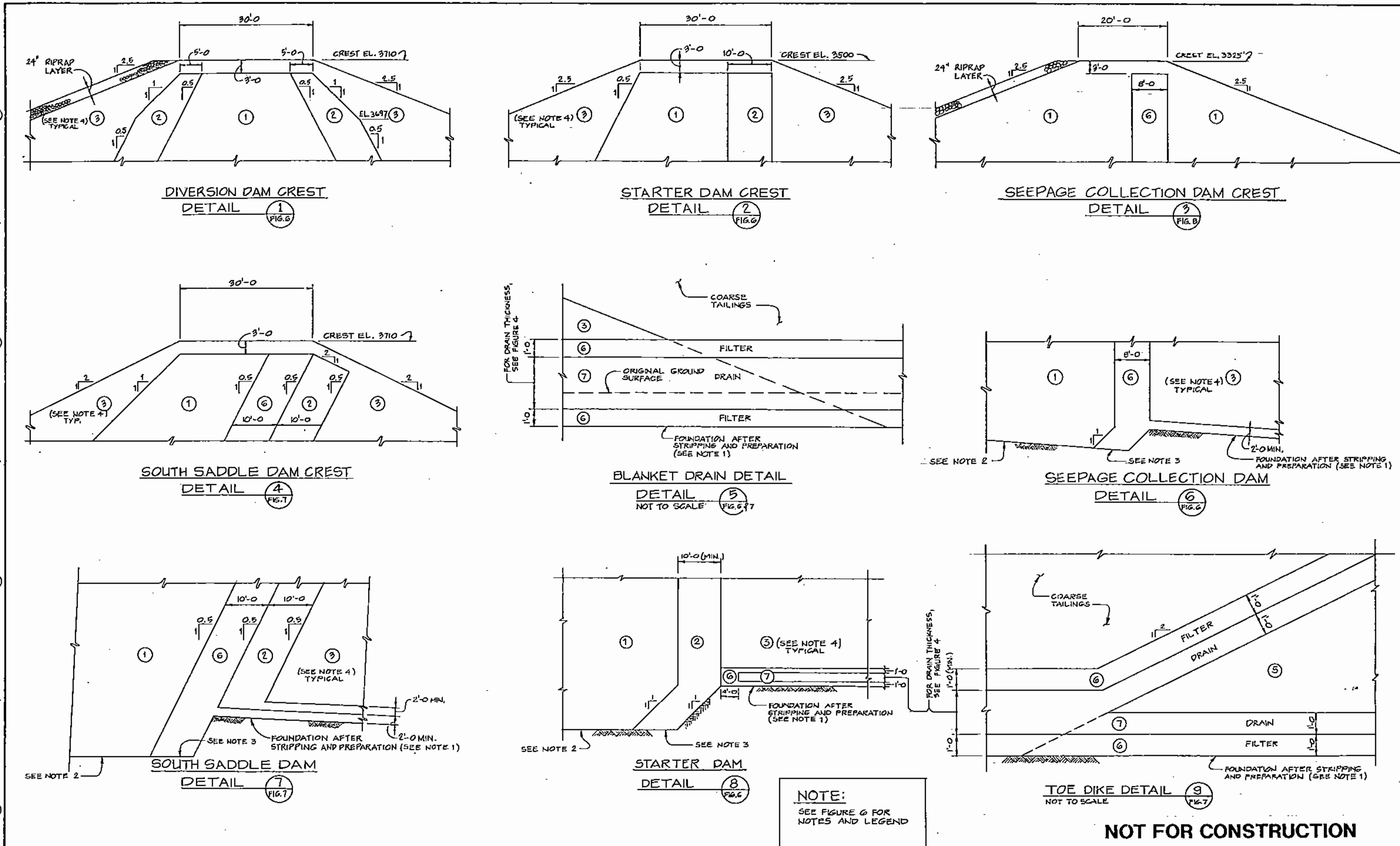
DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NO. | DATE | REVISIONS | BY | CHK | APPD

NORANDA MINERALS CORPORATION

MONTANORE PROJECT
DAM CROSS-SECTIONS AND DETAILS
(SHEET 2 OF 3)

FIGURE 7



NOTE:
SEE FIGURE 6 FOR NOTES AND LEGEND

NOT FOR CONSTRUCTION

SCALE 10 0 10 20 FEET

NO. | DATE | REVISIONS | BY | CHK. | APPD

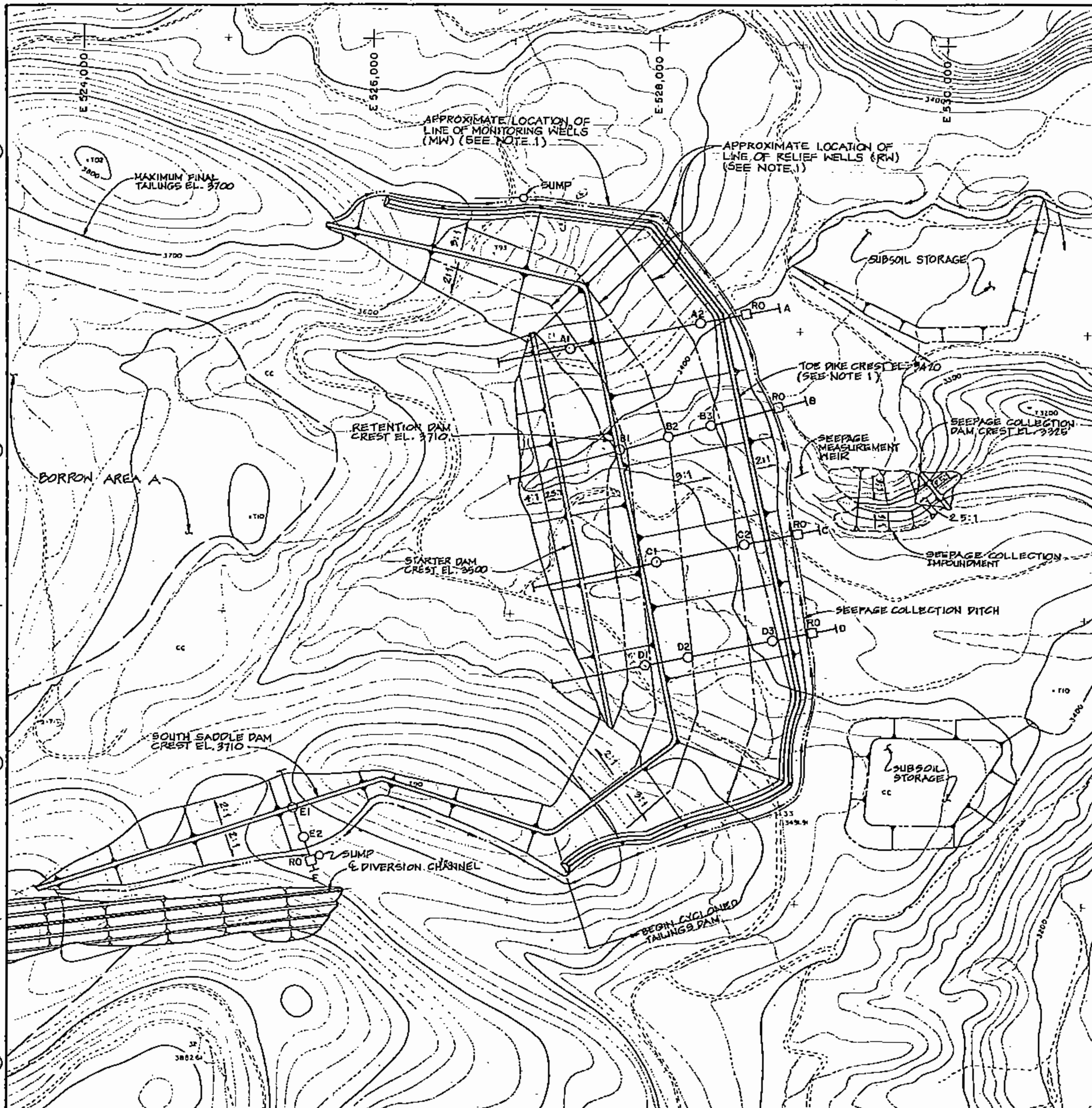
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT
DAM CROSS-SECTIONS AND DETAILS
(SHEET 3 OF 3)

FIGURE 8



NOTES:

1. DEPTHS AND LOCATIONS OF RELIEF WELLS TO BE DETERMINED.

REFERENCE DRAWINGS:

LEGEND:

A—A	TRANVERSE SECTIONS OF INSTRUMENTATION
A1 ⊙	LOCATION OF PNEUMATIC PIEZOMETERS
RO □	PNEUMATIC PIEZOMETER READOUT STATION
RW	RELIEF WELL
MW	MONITORING WELL

NOT FOR CONSTRUCTION

400 0 400 800
SCALE FEET

MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED VNP | DRAWN AOM | CHECKED APR | RECOMMENDED

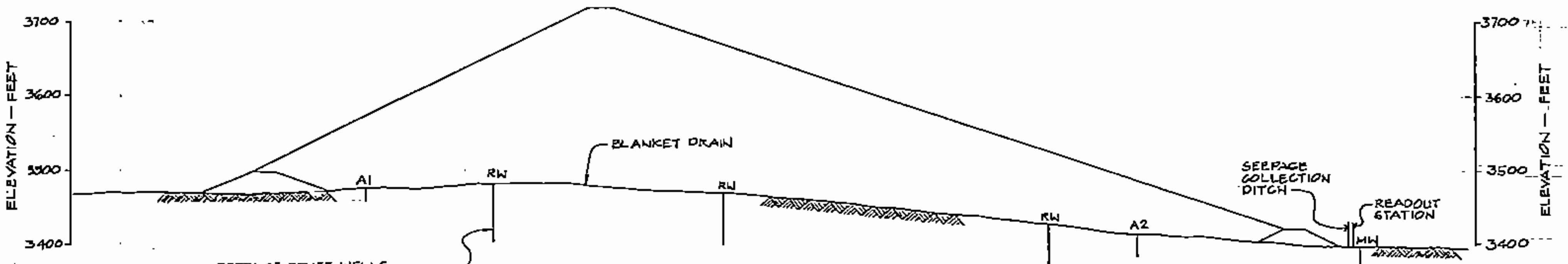
NORANDA MINERALS CORPORATION

MONTANORE PROJECT

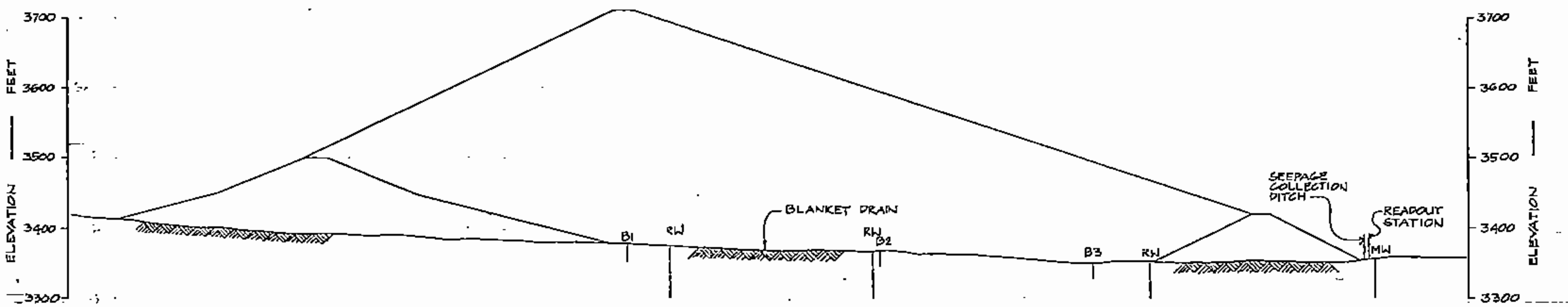
RELIEF WELL AND INSTRUMENTATION PLAN

SHEET 1 OF 2

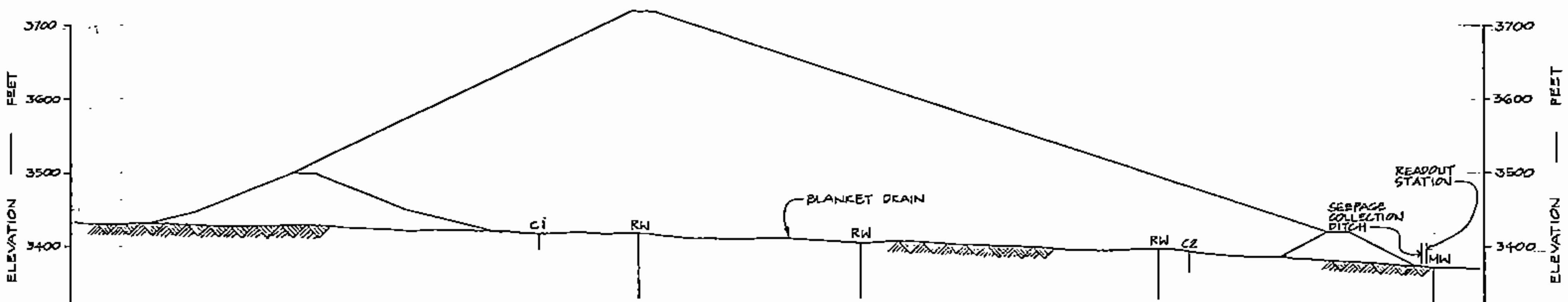
FIGURE 9A



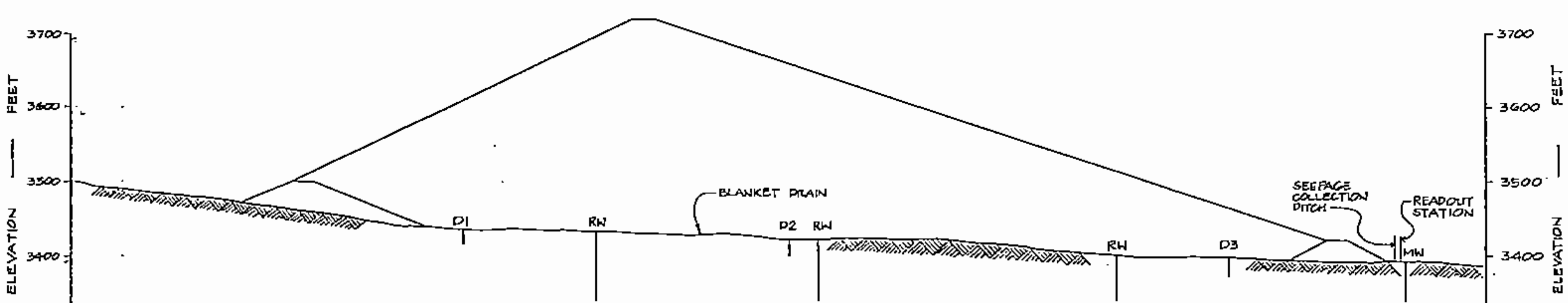
SECTION A



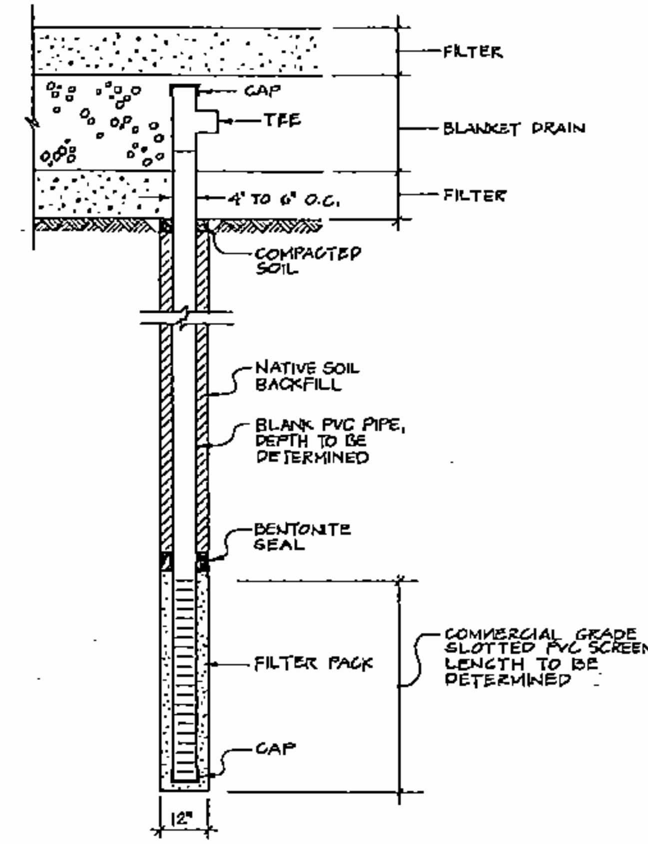
SECTION B



SECTION C

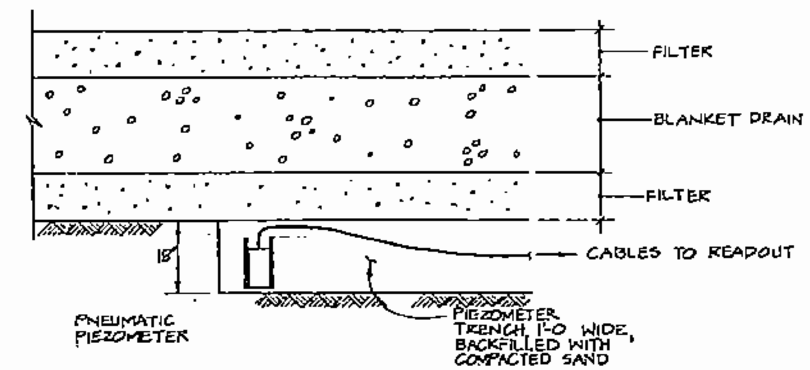


SECTION D

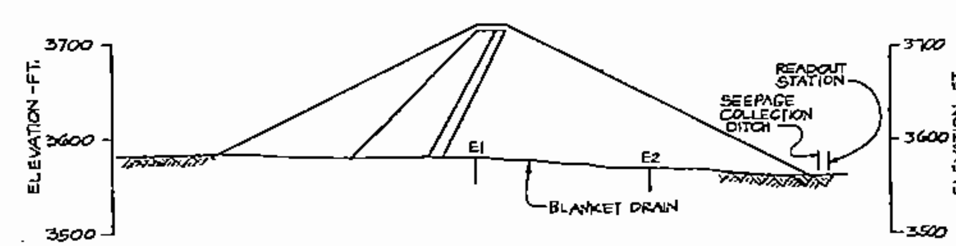


TYPICAL RELIEF WELL INSTALLATION

NOT TO SCALE



TYPICAL PNEUMATIC PIEZOMETER INSTALLATION



SECTION E

NOT FOR CONSTRUCTION

100 0 100 200
SCALE FEET

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED VNP	DRAWN AOM	CHECKED APR	RECOMMENDED
DATE JULY 1990			APPROVED

NO. DATE REVISIONS BY CHK APPD

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

TRANSVERSE SECTIONS AND DETAILS

SHEET 2 OF 2 REV.

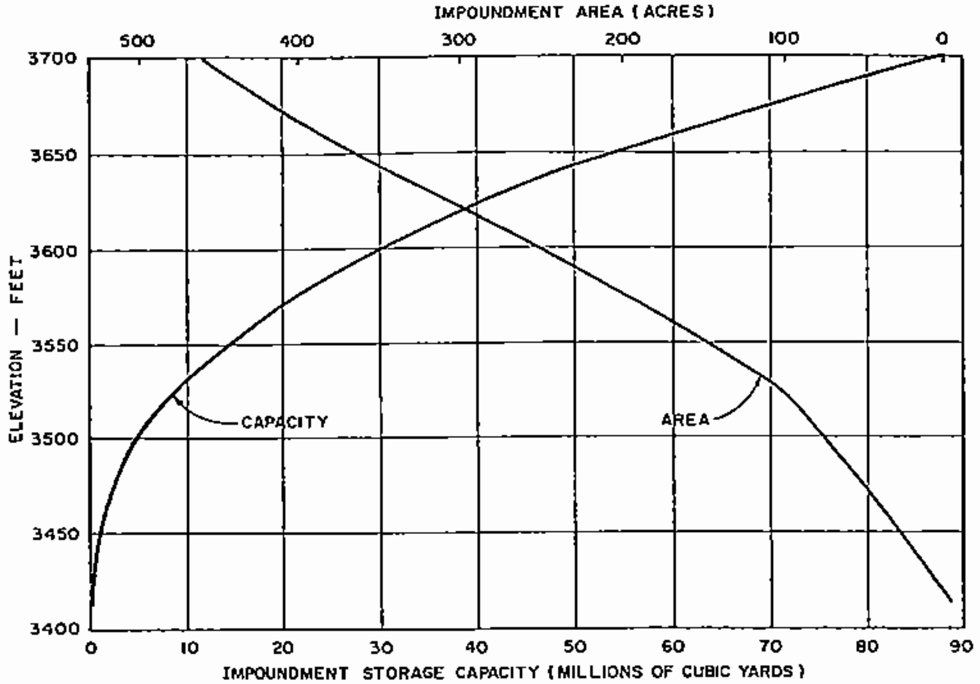
FIGURE 9B

NOTES:

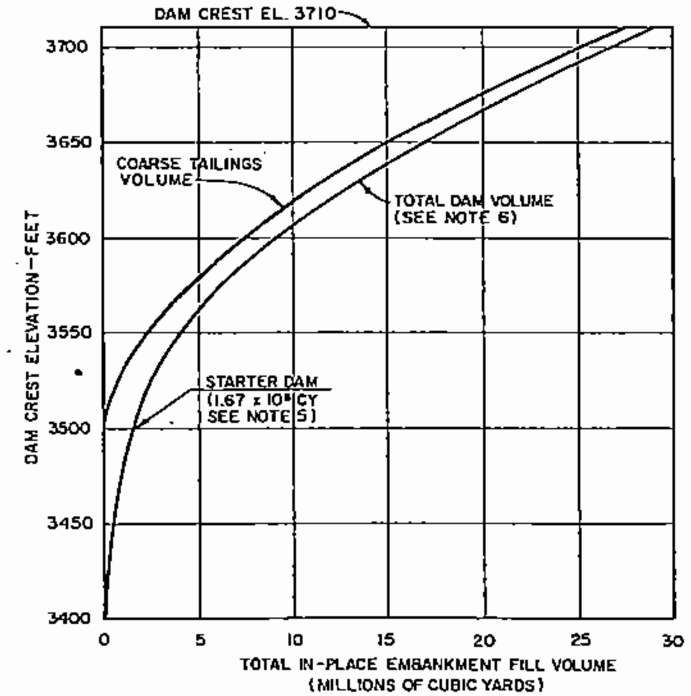
1. AREA-CAPACITY CURVES DO NOT INCLUDE ADDITIONAL STORAGE RESULTING FROM BORROW MATERIAL EXCAVATION WITHIN THE IMPOUNDMENT.
2. IN-PLACE TAILINGS VOLUME BASED ON 100 POUNDS PER CUBIC FOOT (PCF) FOR COARSE TAILINGS AND 70 PCF FOR FINE TAILINGS.
3. IN-PLACE TAILINGS VS. TIME BASED ON A MAXIMUM PRODUCTION RATE OF 20,000± DRY TONS OF TAILINGS PER DAY.
4. MINIMUM REQUIRED CREST ELEVATION BASED ON 3-FOOT FREEBOARD, STORAGE OF TAILINGS EFFLUENT FOR 20 DAYS AT 8000 GPM (707AC-FT), AND STORAGE OF DESIGN FLOOD.
5. TOTAL DAM VOLUME INCLUDES STARTER DAM, TOE DIKE AND COARSE TAILINGS FILL.
6. AREA CAPACITY AND STAGING CURVES ASSUME A HORIZONTAL TAILINGS SURFACE. ACTUAL TAILINGS SURFACE CONDITIONS WILL VARY DURING CONSTRUCTION.
7. DAM CREST ELEVATION - VS - TIME CURVE ASSUMES THAT 80 PERCENT OF TAILINGS SAND IS PLACED IN THE DAM.

REFERENCES:

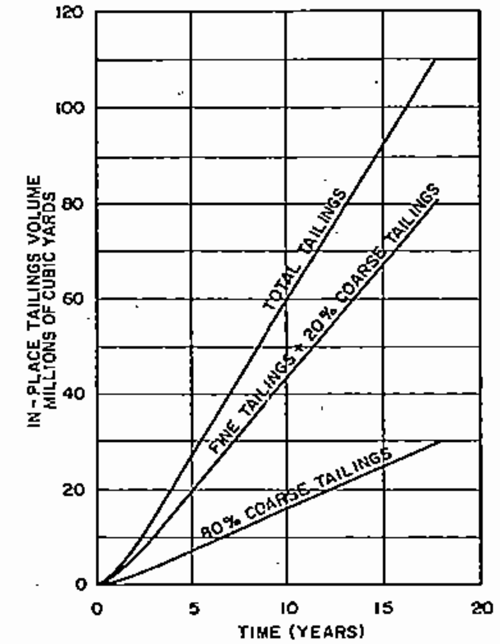
FIGURE 2–SITE PLAN



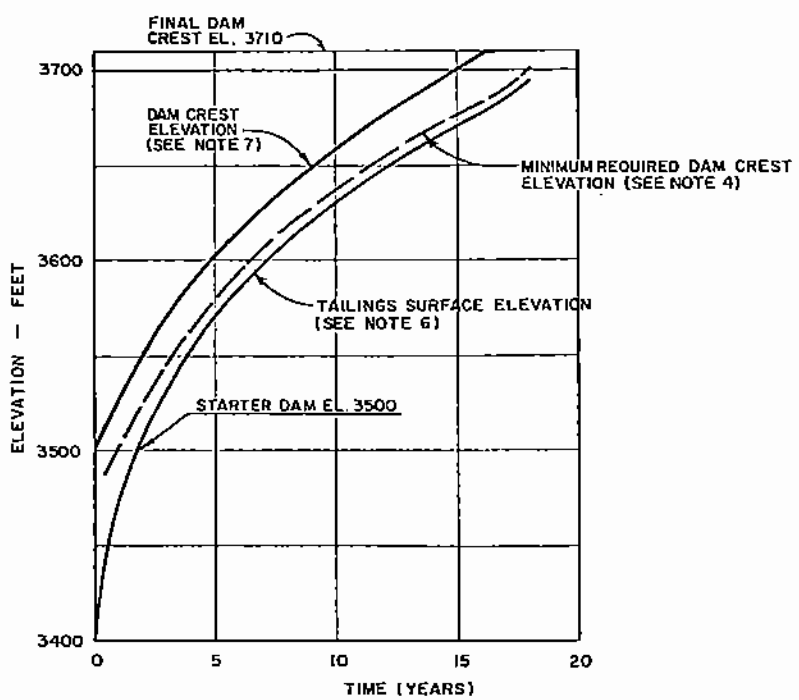
(A) IMPOUNDMENT AREA–CAPACITY



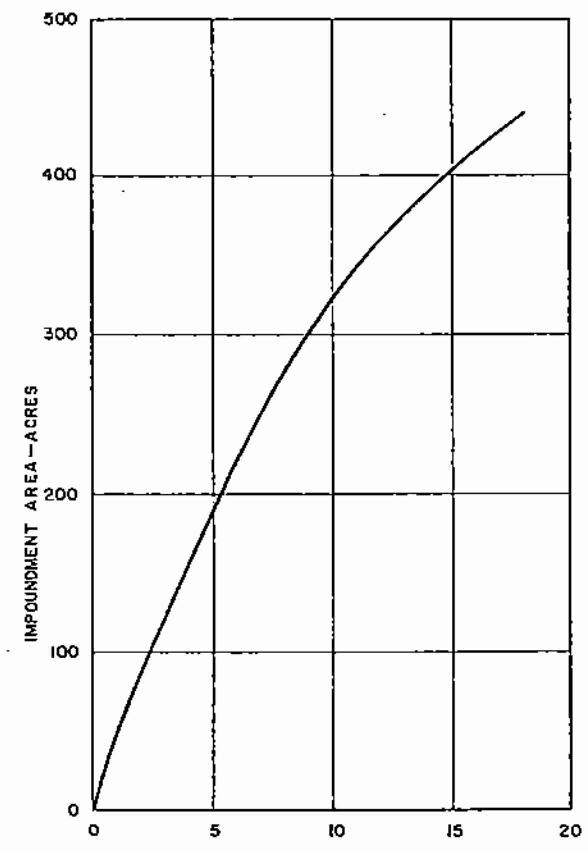
(B) DAM CREST ELEVATION VS. FILL VOLUME
(SEE NOTE 2)



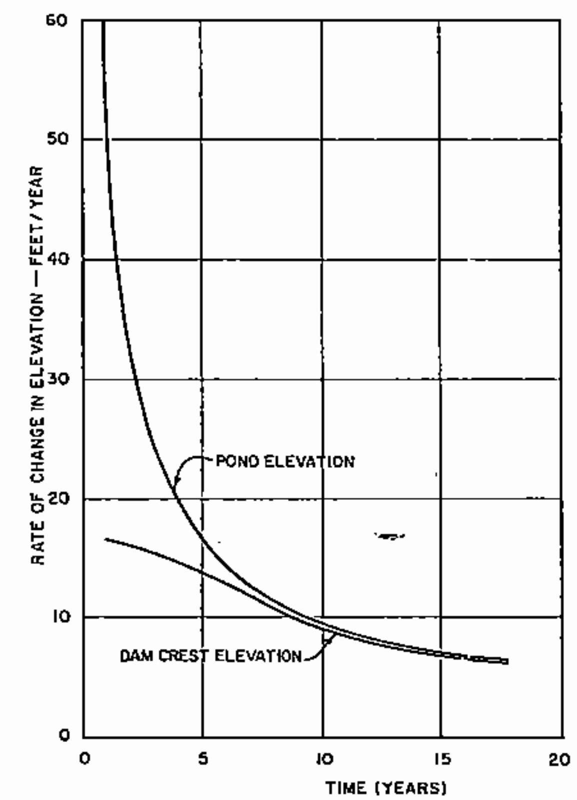
(C) IN-PLACE TAILINGS VOLUME VS. TIME
(SEE NOTES 2 AND 3)



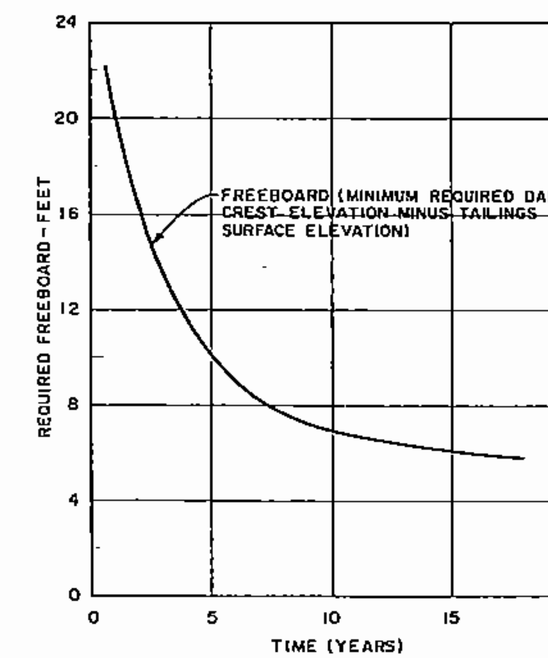
(D) TAILINGS SURFACE ELEVATION AND MINIMUM REQUIRED DAM CREST ELEVATION VS. TIME



(E) IMPOUNDMENT AREA VS. TIME



(F) RATE OF CHANGE OF POND AND DAM CREST ELEVATION
(SEE NOTE 7)



(G) REQUIRED IMPOUNDMENT FREEBOARD VS. TIME
(SEE NOTE 6)

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP | DRAWN CCR | CHECKED MPF/VNP | RECOMMENDED
DATE JUNE 1990 | APPROVED

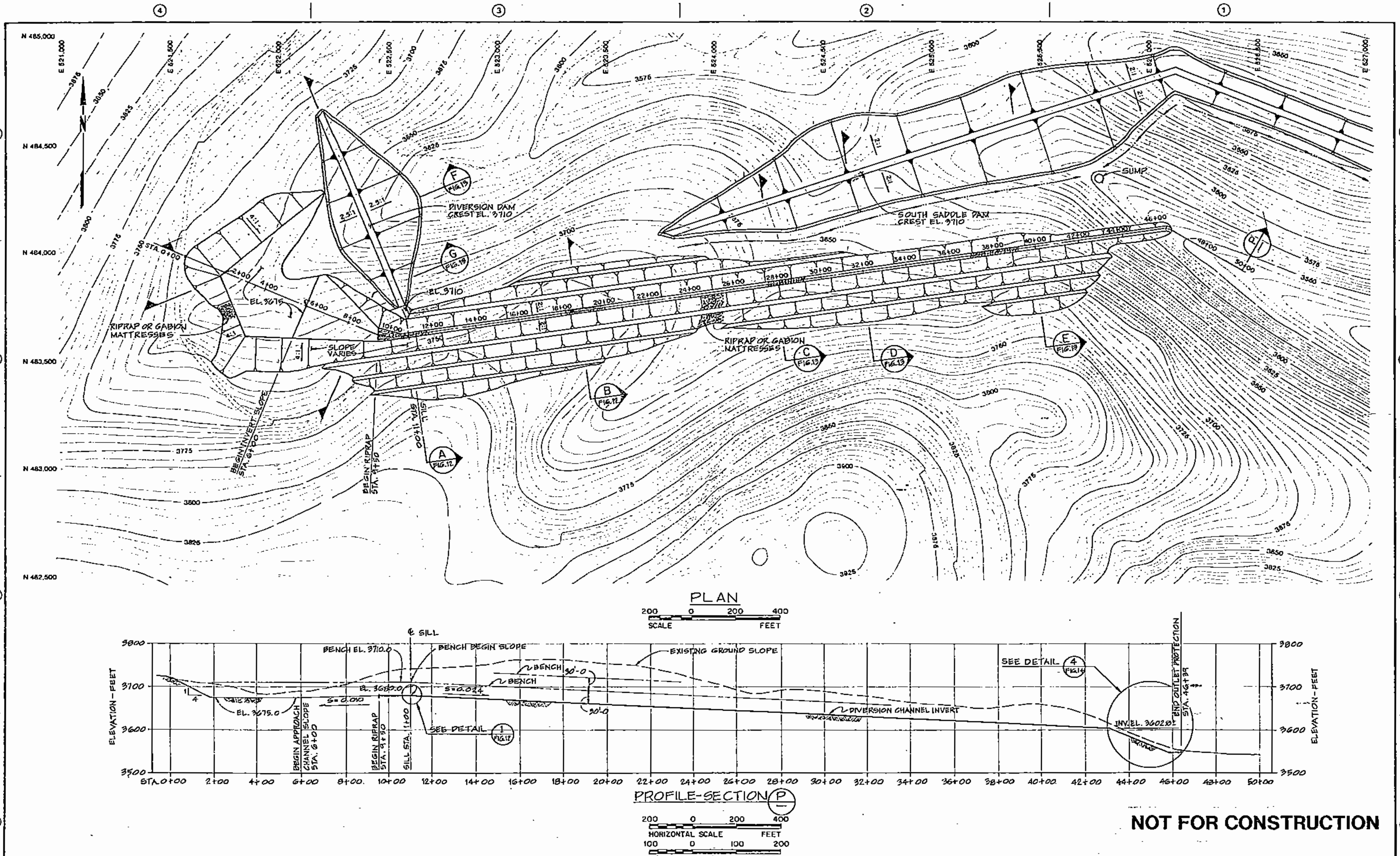
NO. | DATE | REVISIONS | BY | CHK | APPD

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

AREA-CAPACITY AND STAGING CURVES

FIGURE 10



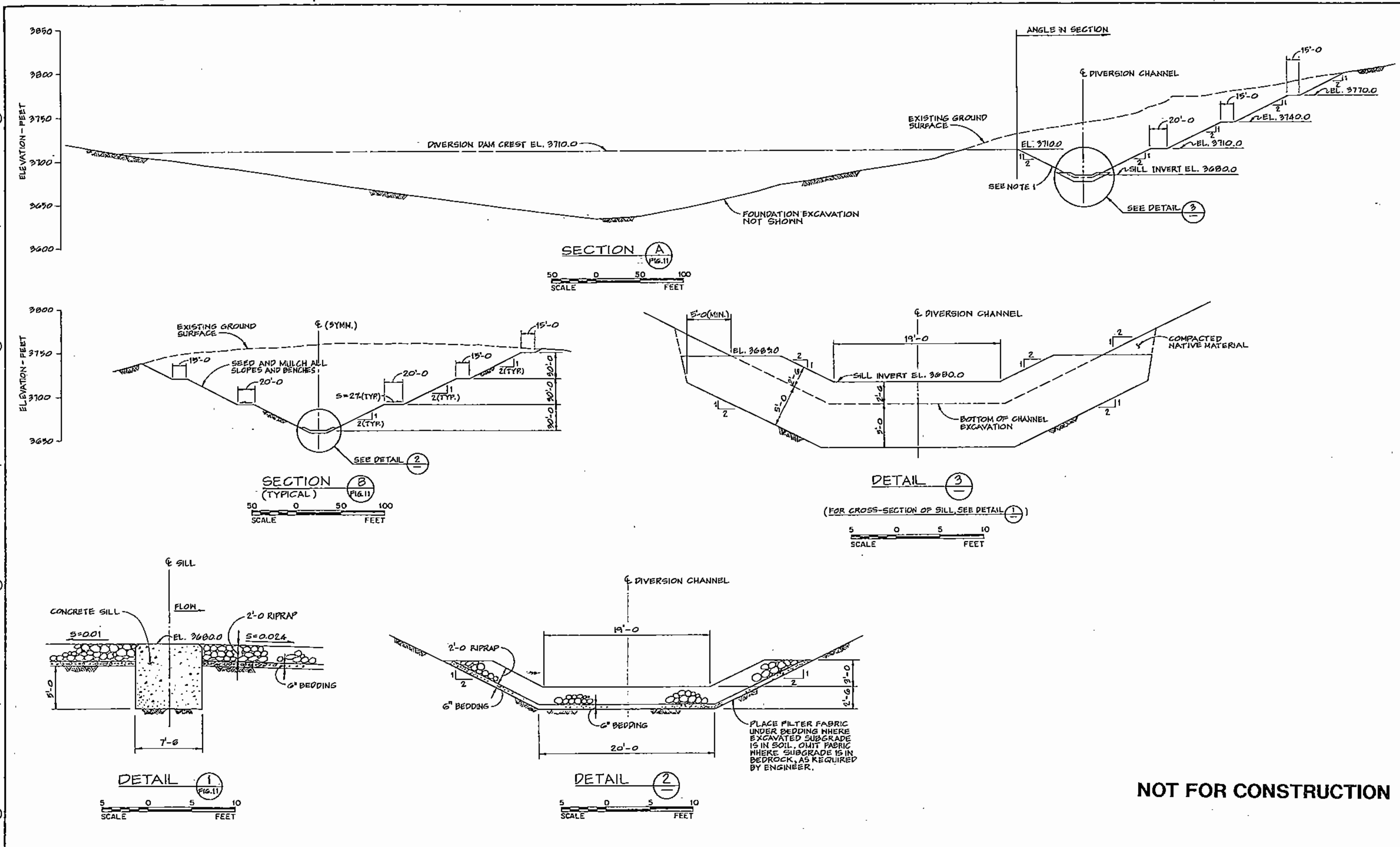
MORRISON-KNUDSEN ENGINEERS, INC. A MORRISON KNUDSEN COMPANY 180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105	
DESIGNED APR	DRAWN JMM
CHECKED MPF	RECOMMENDED
DATE JUNE 1990	APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

DIVERSION DAM AND CHANNEL PLAN AND PROFILE

FIGURE 11



NOT FOR CONSTRUCTION

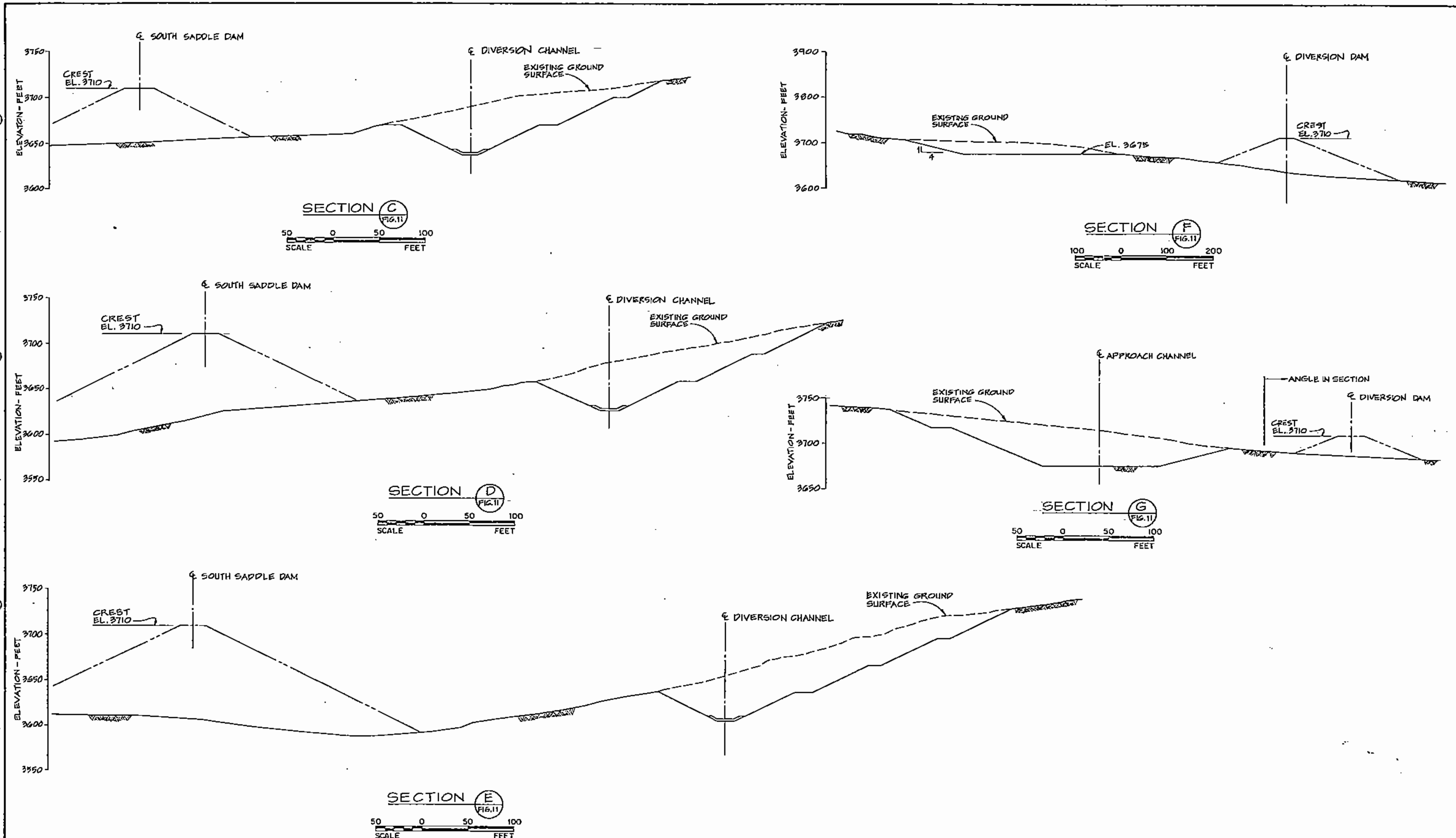
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED APR | DRAWN JMM | CHECKED MPF | RECOMMENDED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT
DIVERSION CHANNEL
SECTIONS AND DETAILS
(SHEET 1 OF 2)

FIGURE 12



NOT FOR CONSTRUCTION

NO.	DATE	REVISIONS	BY	CHK	APPD

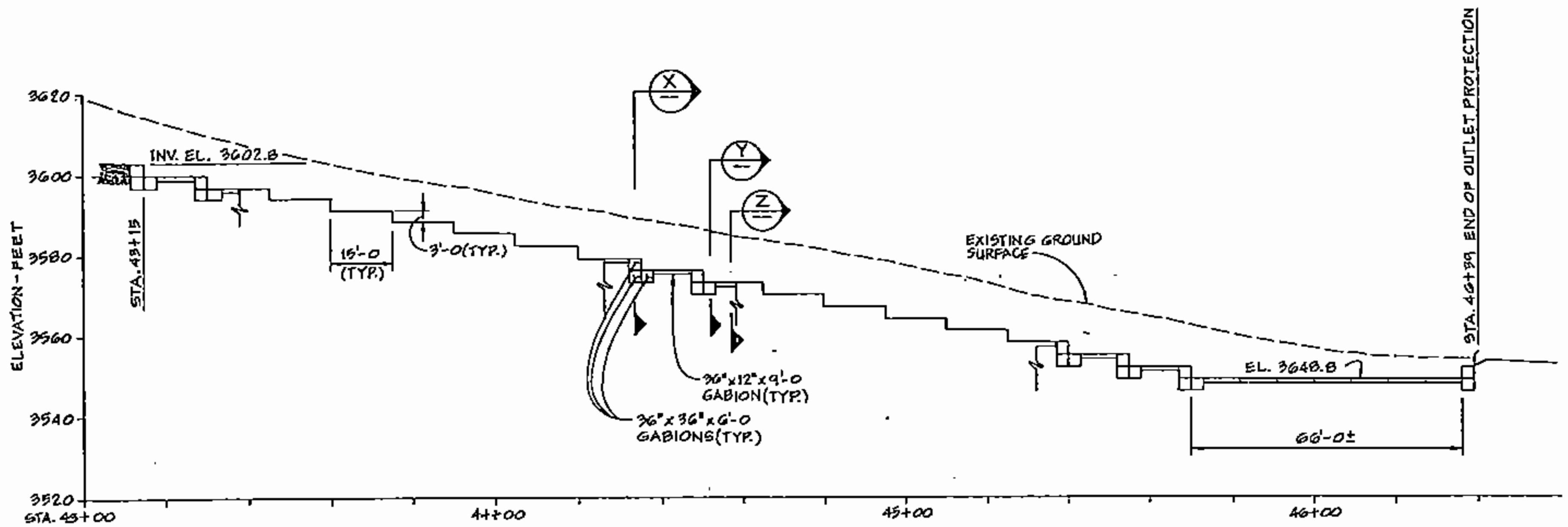
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED APR | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

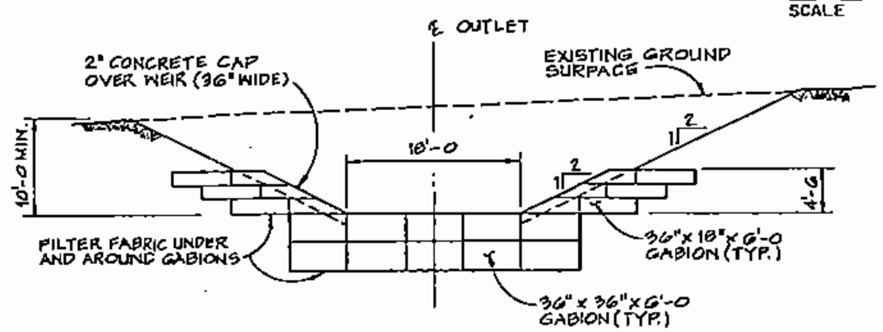
MONTANORE PROJECT
DIVERSION CHANNEL
SECTIONS AND DETAILS
(SHEET 2 OF 2)

FIGURE 13



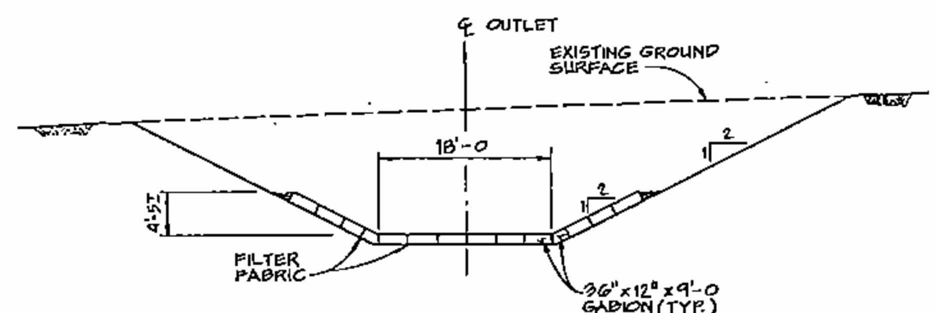
DETAIL 4 (FIG. 11)

20 0 20 40
SCALE FEET



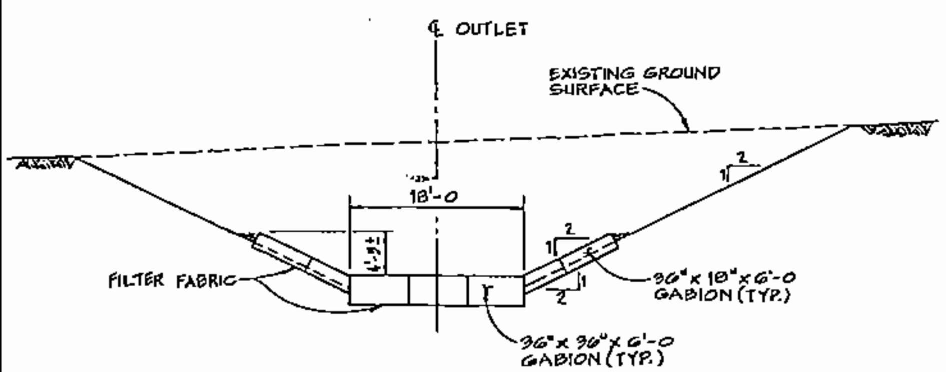
SECTION X

10 0 10 20
SCALE FEET



SECTION Z

10 0 10 20
SCALE FEET



SECTION Y

10 0 10 20
SCALE FEET

NOT FOR CONSTRUCTION

NO.	DATE	REVISIONS	BY	CHK	APPD

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

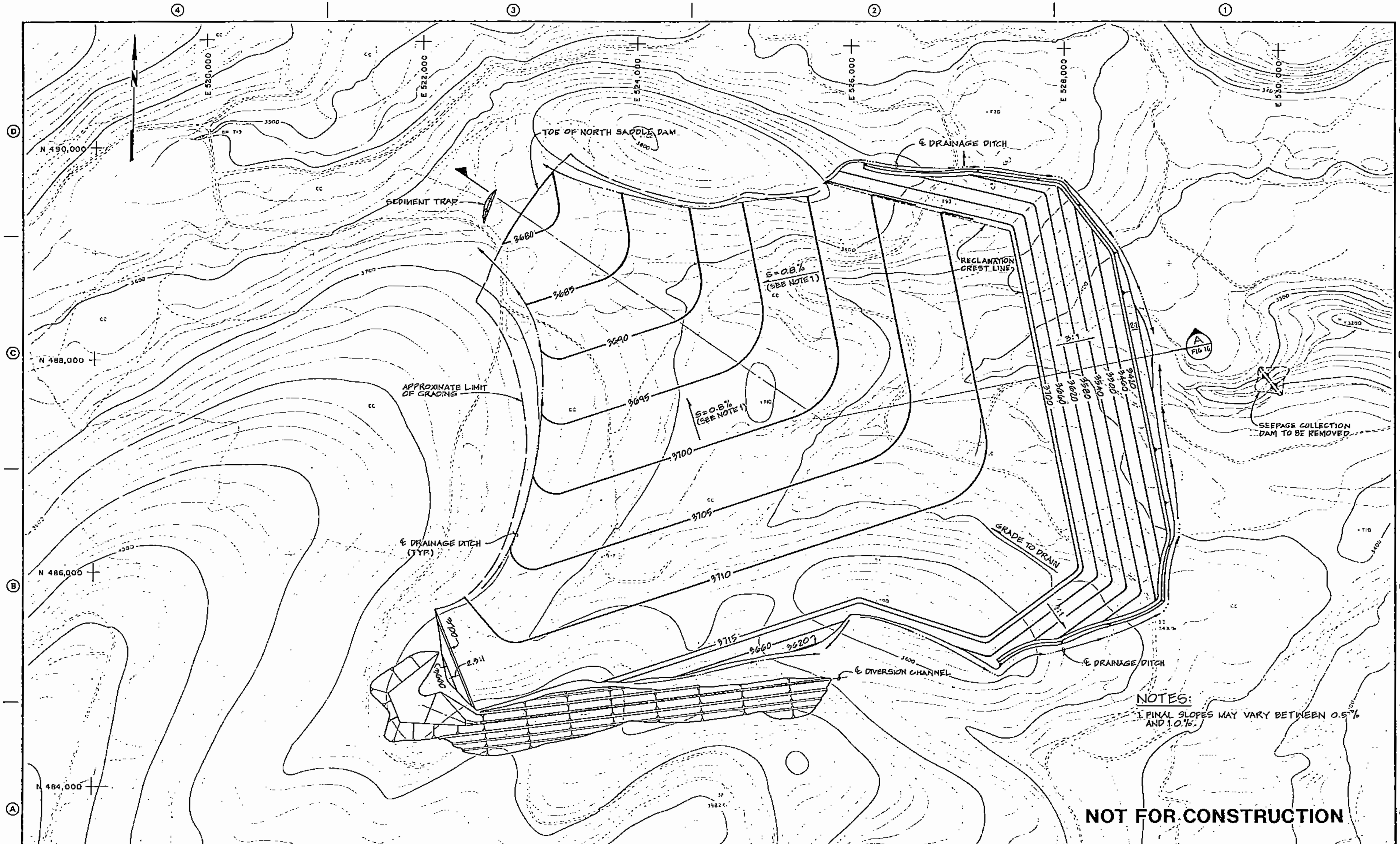
DESIGNED APR | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT

DIVERSION CHANNEL OUTLET SECTIONS AND DETAILS

FIGURE 14



400 0 400 800
SCALE FEET

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

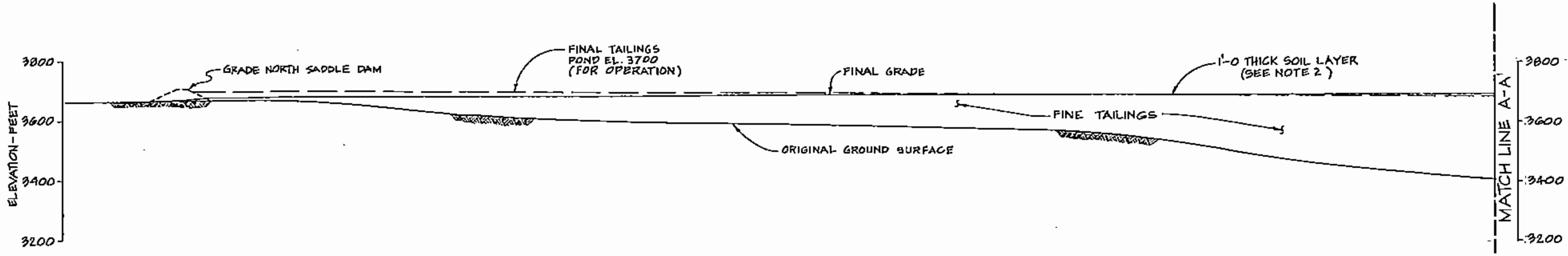
DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED | APPROVED

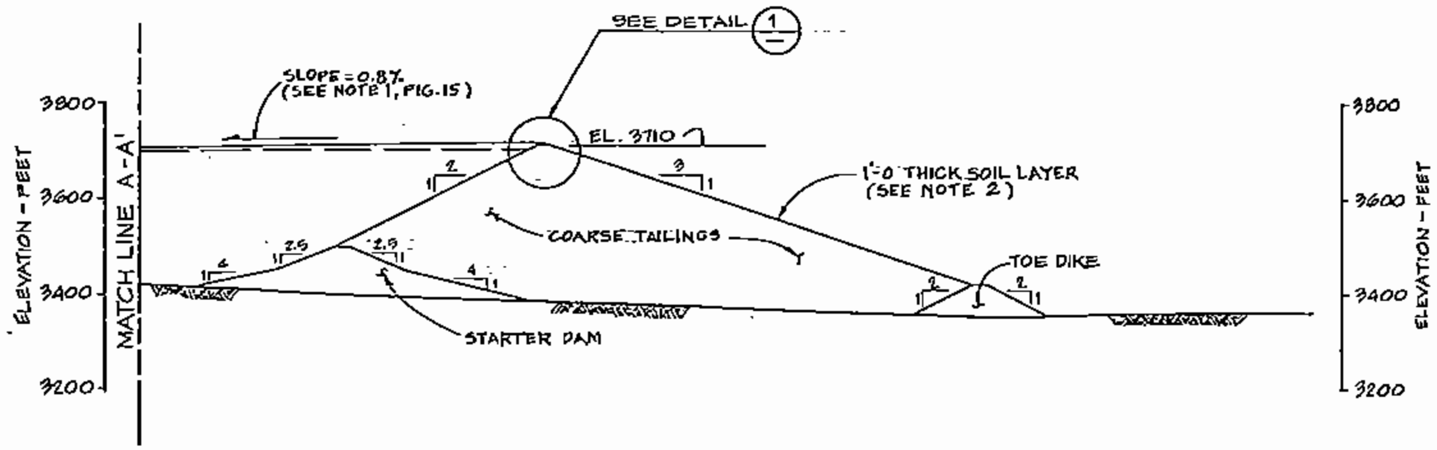
NORANDA MINERALS CORPORATION

MONTANORE PROJECT

RECLAMATION PLAN

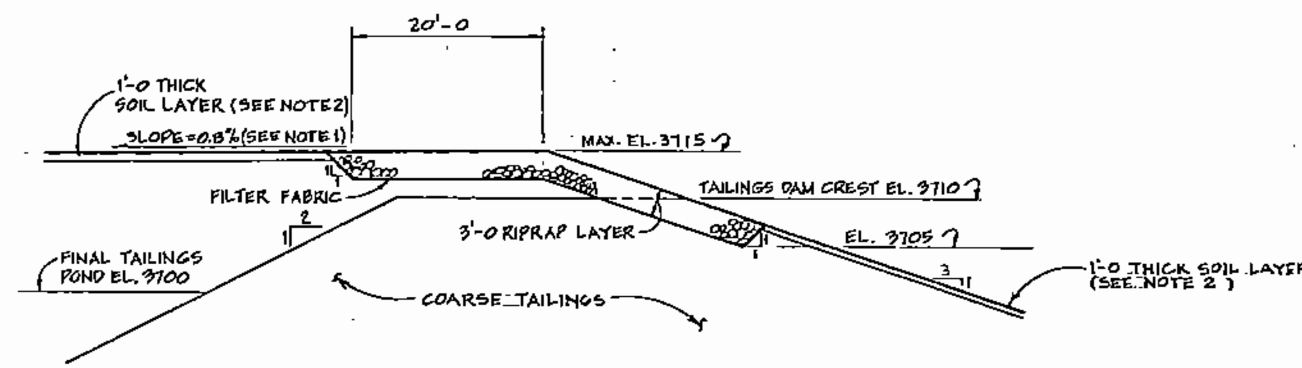
FIGURE 15





TAILINGS IMPOUNDMENT RECLAMATION
SECTION A (FIG.15)

200 0 200 400
SCALE FEET



TAILINGS DAM CREST AFTER RECLAMATION
DETAIL 1

10 0 10 20
SCALE FEET

NOTES:

1. FINAL SLOPES MAY VARY BETWEEN 0.5% AND 1.0%.
2. SURFACE TO BE SEEDED WITH GRASS.

NOT FOR CONSTRUCTION

NO.	DATE	REVISIONS	BY	CHK	APPD

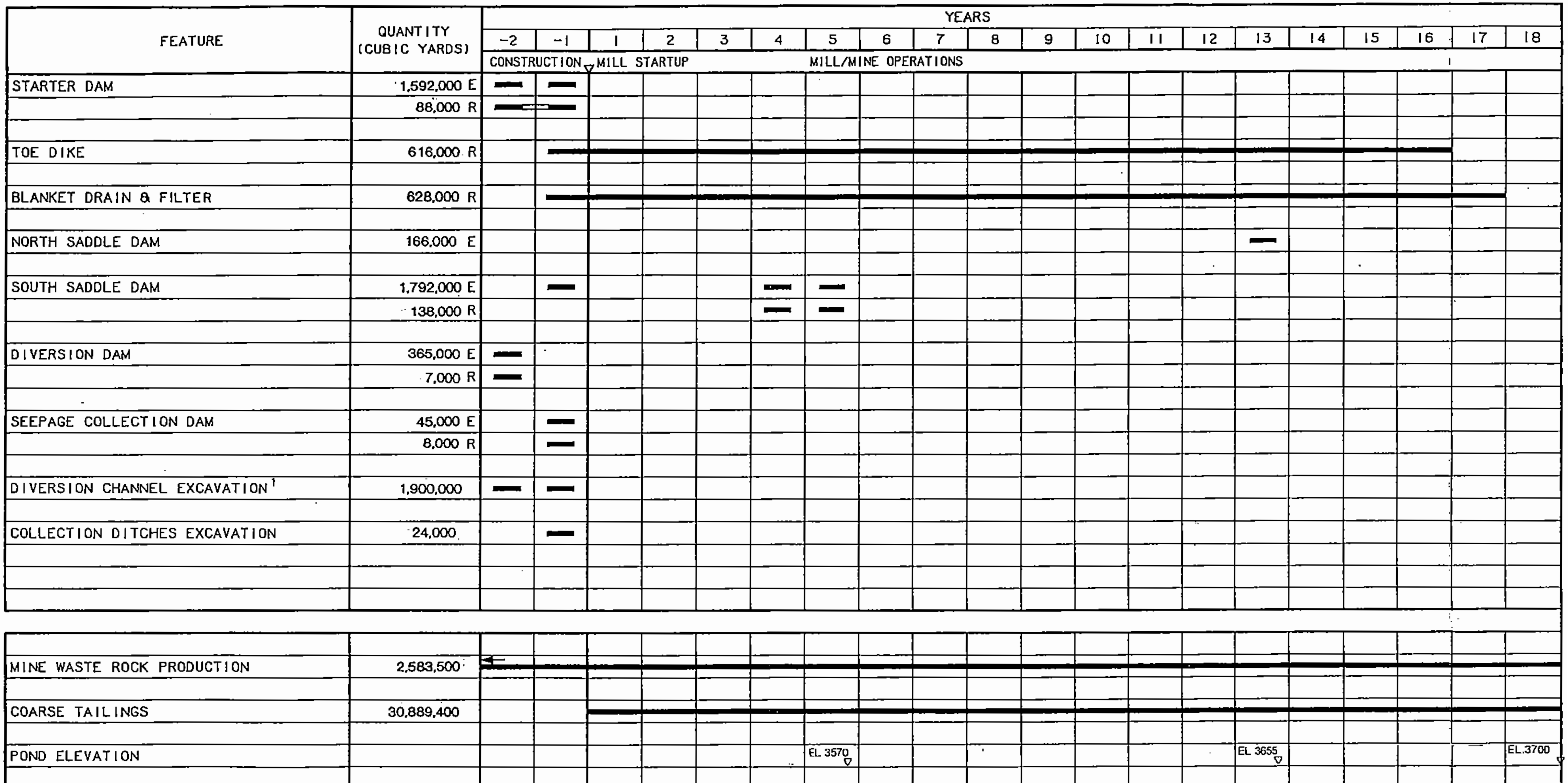
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED JCP | DRAWN JMM | CHECKED MPF | RECOMMENDED
DATE JUNE 1990 | APPROVED

NORANDA MINERALS CORPORATION

MONTANORE PROJECT
RECLAMATION SECTION AND DETAIL

SHEET OF REV.
FIGURE 16



NOTES: ▬ CONSTRUCTION ACTIVITY

▭ FLOAT

E-EARTHFILL MATERIALS (ZONE 1,2,3,4)

R-ROCKFILL MATERIALS (ZONES 5 TO 7 AND RIPRAP)

1-INCLUDES EXCAVATION OF DIVERSION DAM IMPOUNDMENT

MONTANORE PROJECT

IMPOUNDMENT CONSTRUCTION SCHEDULE

FIGURE 17

8802901GB000900

APPENDIX A
DESIGN BASIS MEMORANDUM

Noranda Minerals Corp.
Montanore Project
Design Basis Memorandum No. 8029-201-R1
General Final Design Criteria

CONTENTS

Revision Number	Reviewed	Submitted Proj. Mgr.	Approved Ch. Eng.	Issue Date
1	[signatures] Vincent Pascucci	M. Forest	E. S. Smith	6/13/90

Approved by Noranda Minerals Corp. (Name/Date)______________________

1.0 INTRODUCTION

Noranda Minerals Corporation is currently planning the further development of the Montanore Project located between Libby and Noxon in Sanders and Lincoln Counties, Montana, for mining and milling copper - silver ore. This Design Basis Memorandum describes the scope of work, design criteria and work approach for final design of the Little Cherry impoundment, to store tailings resulting from milling operations.

Structures required for tailings impoundment development include the following:

- Tailings dam, consisting of starter dam and toe dike.
- Two saddle dams
- Diversion dam
- Diversion channel
- Seepage collection dam

A geotechnical investigation, which is part of the final design, supplements the 1988 geotechnical data base developed during the preliminary design stage.

2.0 PROJECT GOAL

The goal of the engineering work and geotechnical investigation outlined in this Design Basis Memorandum is to finalize the design for the Little Cherry tailings impoundment. To accomplish this goal, the scope of work described in Section 3.0 has been developed to (1) better define geotechnical conditions at the impoundment site, (2) prepare a final design and (3) provide construction cost estimates. The design will form the basis for which contract plans and specifications can subsequently be prepared.

3.0 SCOPE OF WORK

The scope of work for final design consists of the following tasks:

- Conduct a field investigation that includes the following activities:
  - Seismic refraction surveys
  - Exploratory drilling
  - Test pit excavations
  - Laboratory testing

- Perform the following final design studies:
  - Stability analyses
  - Tailings settlement analysis
  - Diversion channel design
  - Dam and dike design
  - Reclamation plan
  - Construction cost estimates

- Prepare the following technical reports:
  - Geotechnical Report
  - Tailings Impoundment Engineering Reports

- Attend meetings with Noranda Minerals Corporation and regulatory agencies.

4.0 DESIGN CRITERIA

4.1 Tailings Production

During the operational stages of the impoundment, the embankment dam will be constructed by the downstream method using cycloned tailings sand. A two-stage cycloning process will be required to produce a sand that will be sufficiently clean for drainage and construction purposes. The anticipated gradation of the tailings feed (based on flotation test results) is shown in Table 4-1 below:

TABLE 4-1
TAILINGS FEED GRADATION

Tyler Sieve No.	Particle Size (Microns)	Cumulative % Passing Sieve
65	208	99.5
100	147	90.2
150	104	69.4
200	74	52.1
270	52	35.9
400	37	25.0

For tailings impoundment sizing, the tailings production rate will be as shown in Table 4-2 below:

TABLE 4-2
TAILINGS PRODUCTION

Year of Mill Operation	Tailings Production
1	3,772,000 tons
2	5,500,000 tons
3 to 17	7,000,000 tons per year
18	5,728,000 tons
Total	120,000,000 tons

The mill will operate 328 days per year. Availability of sand for tailings dam construction will be assumed to be 80% of the time that tailings are produced. Coarse and fine tailings will be deposited in the pond during the 20% of the time that sand would not be available for dam construction.

Coarse and fine tailings production curves and impoundment staging curves will be developed. Cyclone analyses will be performed to determine the percent split of coarse and fine tailings. Dry unit weights of tailings will be determined by laboratory testing of test samples (Subsection 5.2.E). For preliminary planning, the dry unit weight of the coarse tailings will be 100 pcf and for the fine tailings, 70 pcf will be used.

4.2 Stability

The following stability analyses will be performed:

- Conventional static and seismic stability analyses
- Dynamic response and deformation analyses
- Liquefaction analyses.

Conventional stability analyses will be performed for the starter and final stages of the tailings dam. The starter dam will be evaluated for the end-of-construction, steady-state seepage and seismic conditions. The final dam will be evaluated for steady-state seepage, seismic and flood conditions. The analyses will be performed by using the strength parameters determined from the results of the laboratory testing program (Subsection 5.2.E). The tailings in the impoundment will be conservatively assumed to behave as a dense fluid and, therefore, will be assigned zero shear strength.

The minimum acceptable factors of safety for use in design are shown in Table 4-3 and are as recommended by the Corps of Engineers (Ref. 1).

TABLE 4-3
STABILITY CRITERIA

Load Condition	Embankment Stage	Slope	Minimum Acceptable Factor of Safety
End-of-Construction (before tailings deposition)	Starter Dam	Upstream & Downstream	1.3
Long-term (full tailings pond)	Starter and Final Dams	Downstream	1.5
Flood Surcharge Condition	Final Dam	Downstream	1.4
Seismic (pseudo-static)	Starter and Final Dams	Downstream	1.0

The project site is located close to the boundaries of Seismic Zones 1, 2 and 3 (see Figure 1). Based on a review of site seismicity during preliminary design, a seismic coefficient of 0.10g will be used in pseudo-static stability computations as recommended by the Corps of Engineers for Seismic Zone 3 (Ref. 1).

In the "pseudo-static" method of stability analysis, the effects of an earthquake on a potential slide mass are represented by an equivalent static horizontal force determined as the product of a seismic coefficient and the weight of the potential slide mass. The use of the maximum ground acceleration as the seismic coefficient would produce an equivalent static horizontal force equal to the maximum transient inertia force developed on the mass during the design earthquake. However, the length of time for which the force acts is an important factor in the development of deformations. Therefore, the use of the maximum transient force as an equivalent static force would be unduly conservative (Ref. 2) and the recommended seismic coefficient of 0.10g should be used.

A Maximum Credible Earthquake (MCE) event for use in both seismic deformation and liquefaction analyses was determined based on magnitudes of historical earthquakes and correlations between magnitude and length of active faults (potential activity during Pleistocene or Holocene). The MCE is defined as the largest rationally conceivable event that could occur in the tectonic environment in which the project is located (Ref. 3). For impoundment design, the MCE was determined to be as follows (Ref. 4):

- Source: Bull Lake Fault
- Distance: 20 km
- Magnitude: 7.0
- Peak Bedrock Acceleration: 0.22g
- Duration of Significant Shaking: 27 seconds

Due to the large size of the ultimate dam (nearly 400 feet high) and the potential for a strong nearby earthquake, an analysis will be conducted to confirm that embankment deformations resulting from the design earthquake will be acceptable. Of particular concern is that there should not be an excessive reduction of embankment freeboard due to seismically-induced deformations. The analysis approach is described (Subsection 5.3.A.2).

Foundation liquefaction analyses will be performed. Liquefaction is defined as the rapid build-up of pore-water pressures and the resulting loss of soil strength caused by seismic shaking (Ref. 5). Soil deposits that are particularly susceptible to liquefaction consist of loose, saturated sands. Liquefaction potential of sandy foundation soils due to the MCE event will be evaluated by methods outlined in Refs. 6 and 7.

4.3 Design Flood

The designation of the tailings impoundment design flood is based on size and hazard potential classifications. The tailings retention dam will be raised incrementally to increase impoundment storage capacity. Size classification is determined by either impoundment storage or dam stage height, whichever gives the larger size category. For this project, dam stage heights control size classification.

The distance, in river miles, to the nearest dwelling and to U.S. Highway 2 from the Little Cherry impoundment site is 5.4 and 5.7, respectively. The regulatory agencies consider the impoundment site to have moderate to high hazard potential.

Based on dam stage size and hazard potential classifications, the agencies designated the following design flood criteria:

- For containment: 24-hour general storm Probable Maximum Precipitation (PMP)
- For diversion: 72-hour general storm Probable Maximum Flood (PMF)

Because thunderstorm events should also be considered for small watersheds, the local storm PMF (resulting from the 6-hour PMP) was also considered for diversion. The results of hydrologic analyses show that this flood is the more critical of the two diversion floods and will therefore be used for diversion system design (Ref. 8).

For interim stages less than 100 feet high that would be present for short term (less than 5 years), the containment flood will be calculated from the 24-hour general storm 1/2 PMP.

The minimum embankment dam freeboard (above the peak flood water surface) will be 3 feet.

4.4 Channel Design

A. Diversion Channel - The 6-hour local storm PMF inflow hydrograph will be routed through the diversion pond and the diversion channel. The U.S. Army Corps of Engineers HEC-1 computer program will be used for this purpose. The peak flood inflow into the diversion pond was computed to be 5230 cfs. (Ref. 8).

The diversion channel will have adequate longitudinal slope and cross-sectional area to pass the design flow with a minimum of 2 feet of freeboard. The channel will have a trapezoidal cross-section. The bottom width, side slopes and channel gradient will be designed to pass the design flood flow and to minimize erosion potential.

To protect against erosion, portions of the channel in soil or weathered rock will be lined with riprap. The riprap design will be based on the 100-year flood event. Methods outlined in Refs. 9 and 10 will be used to select the median (D50-min) rock size. Manning's roughness coefficient will be calculated using the U.S. Army Corps of Engineers method (Ref. 10).

Gradation and thickness of the riprap lining will be based on the Corps of Engineers method (Ref. 10) with consideration given to economic use of available materials. A filter fabric and a bedding layer will be provided beneath the riprap to prevent the migration of finer subgrade materials through riprap materials. Filter criteria for the bedding will be as indicated in Ref. 11.

B. Outlet - The outlet section of the channel will consist of a stair-step configuration between the end of the diversion channel and the natural stream channel. Erosion protection of the outlet section will consist of gabion mattresses. Design flood flow velocities will be based on the 100-year event.

C. Stream Channel - Initially, erosion protection is not planned for the stream channel downstream of the diversion channel outlet. Erosion protection will be constructed if erosion is observed during operations. Such protection would consist of rockfill check dams to dissipate flow energy. The stream channel erosion protection will be based on the 100-year flood. If needed, check dams will be constructed of well-graded, durable rockfill that will retain sediment. The rockfill gradation will be designed to be stable under the design flood. The check dams will be keyed into the stream channel.

The spacing of the rockfill check dams will be based on check dam height, channel gradient and the expected final slope of the channel after sediment has accumulated behind the dams. For a final sediment deposit slope, the height and spacing of the check dams will be calculated to minimize the total required rockfill volume.

The check dams will be considered as broad-crested weirs and the discharge relationship will be based on weir flow formulas. Rockfill aprons will be constructed, if necessary, to dissipate energy of the flow over the check dams. The lengths of the aprons will be based on the heights of the check dams and channel slope.

4.5 Waste Rock Production

The mine waste rock available for dam construction is shown in Table 4-4 below:

TABLE 4-4
MINE WASTE ROCK SCHEDULE

Year	Project Stage	In-Place Tons (X1000)	Cum. Tons (X1000)	Rock Designation
1	Evaluation	192	192	Crushed
2	"	187	379	Crushed
3	Preproduction Development	587	966	Run of Mine
4	"	535	1501	Run of Mine
5	Initial Production	0	1501	--
6	"	664	2165	Run of Mine
7	"	1091	3256	Run of Mine
8	Full Production	537	3793	Stored in Mine
9	"	485	4278	Stored in Mine
10	"	75	4353	Stored in Mine
11-17	"	75/yr.	4878	Stored in Mine
≥18	"	0	4878	Stored in Mine

Table 4-5 shows the anticipated gradations of both crushed and run of mine waste rock. If rock stored in the mine is needed for construction, it will have to be crushed for transportation to a loading area via conveyor.

TABLE 4-5
WASTE ROCK GRADATIONS

Crushed Rock		Run of Mine Rock	
Size (in.)	% Passing	Size (in.)	% Passing
10	100	24	100
8	90	16	90
7	80	13	80
6	70	10	70
5	60	9	60
4	50	7	50
3	40	5	40
2	30	3	30
1-1/2	20	2	20
3/4	10	1	10

The dry unit weight of all mine waste rockfill placed and compacted in embankments is estimated to be 140 pcf.

5.0 APPROACH

5.1 General

This section presents the approach for final design of the Little Cherry Tailings impoundment site. The section is divided into the following tasks as outlined in Section 3.0:

- Geotechnical Investigation
- Final Design Engineering
- Technical Reports
- Review Meetings

5.2 Geotechnical Investigation

A. General - A field investigation program was undertaken during the summer of 1989 to supplement the 1988 geotechnical data base developed for the permit application. The program consisted of seismic refraction surveys, drilling and test pit excavations. The locations of the 1988 and 1989 seismic lines, drill holes and test pits are shown on the Exploration Plan, Figure 2.

Due to access limitations during the initial investigation program in July and August 1988, drilling could not be performed along the diversion channel alignment. Therefore, a large part of the 1989 investigation was focused on the diversion channel.

For access to the site, existing tracks and roads were used to the maximum possible extent. However, for access to drill sites, close coordination with the Forest Service was required for timber cutting and construction activities. All tracks and sites for drilling and test pits were restored as required by the Forest Service.

B. Seismic Refraction Surveys - The subsurface investigation began with a seismic survey of the impoundment site. Seismic refraction surveys were performed along the diversion channel alignment to estimate depths to bedrock and rippability of the materials to be excavated. Seismic velocities also will be useful to estimate excavation equipment requirements. Seismic refraction surveys also were performed in the dam foundation to provide supplemental data on depths to bedrock. Seismic refraction surveys were performed along 35 lines in the impoundment site.

C. Drilling - Drilling was done in the impoundment site to explore dam foundation conditions and materials in the diversion channel excavation. For the 1989 investigations, 20 borings totaling 2265 linear feet were drilled. A summary of the drilling program is shown in Table 5-1 below.

TABLE 5-1

SUMMARY OF DRILLING PROGRAM

Area	Boring No.	Total Depth (feet)
Tailings Dam	DH-1	100
	DH-2	110
	DH-3	367
	DH-3A	25
South Saddle Dam	DH-4	40
North Saddle Dam	DH-5	197
Tailings Impoundment	DH-6	60
Diversion Channel	DH-7	73
	DH-8	98
	DH-9	82
	DH-10	87
Diversion Dam Abutment	DH-11	192
Tailings Dam	DH-12	110
South Saddle Dam	DH-13	80
	DH-14	60
Tailings Impoundment	DH-15	120
	DH-17	80
	DH-18	65
Tailings Dam	DH-19	105
	DH-20	214
	TOTAL =	2265

Eighteen borings were cased with PVC pipe to monitor groundwater levels in the dam foundation, proposed diversion channel excavation and tailings impoundment. Field permeability tests were performed in borings to supplement permeability values measured previously. Tests in overburden were performed by using constant head open-end casing tests conforming to procedures in the U.S. Bureau of Reclamation's "Earth Manual", Method E-18 (Ref. 12). Water pressure tests in bedrock were conducted by using a single pneumatic packer. Permeability tests also were performed in borings drilled in the diversion channel site. The results of these tests will indicate the magnitude of anticipated groundwater flow into the channel excavation. This information will be used to estimate groundwater control measures.

Standard Penetration Tests (SPT's) were performed in the borings at approximately 5-foot intervals to a depth of about 25 feet and at selected depths below 25 feet. The results of the SPT's will be used to characterize foundation materials for use in stability and liquefaction analyses (Subsection 5.3. A.).

Undisturbed clayey soil samples were collected during the drilling program using Shelby tube samplers. These samples will be used for strength testing as described in Subsection 5.2.E below.

D. Test Pits - Twenty-seven backhoe test pits were excavated in the proposed borrow area and diversion channel excavation to confirm the suitability and condition of the soils for use as construction materials. Also, test pits were excavated in the impoundment site to identify any areas of permeable materials that may need to be blanketed with clayey soils. Samples from the test pits were taken for laboratory testing.

A backhoe with a 20-foot reach was used for final design exploration. All test pit sites were restored as required by the U.S. Forest Service.

Field density tests to measure soil densities were performed in 6 shallow test pits excavated in the proposed dam foundation.

E. Laboratory Testing - A laboratory testing program was performed to obtain engineering parameters of the soils at the tailings impoundment site. A tailings testing program is currently in progress. The test data will be evaluated for use in dam design and stability analyses.

Testing of soil samples (collected from the drilling and test pit excavation program) and tailings samples follows American Society for Testing and Materials (ASTM) procedures and consists of the following:

- Grain size analyses - ASTM D422
- Natural moisture contents - ASTM D2216
- Plasticity index characteristics - ASTM D4318
- In situ dry densities - ASTM D2937
- Moisture/density characteristics - ASTM D698
- Relative Density - ASTM D4253 and D4254
- Triaxial compression - ASTM D2850 (unconsolidated-undrained tests)
- Consolidation - ASTM D2435

Grain size and plasticity index tests were performed to classify foundation soils and potential borrow materials. Natural moisture contents were measured to determine requirements for construction moisture content conditioning of borrow materials. Dry densities of undisturbed soil samples were determined for use in stability analyses. Moisture/density tests were performed to determine maximum densities and optimum water contents for embankment construction. One relative density determination will be performed on a sand tailings sample for use in liquefaction and stability analyses (Sub-section 5.3.A).

Strength parameters will be used in final design stability analyses. Unconsolidated-undrained (UU) triaxial compression tests were performed on undisturbed samples of clayey foundation soils. Strength parameters determined from these tests will be used in the stability analysis for the end-of-construction condition (see Section 4.2). Consolidated-undrained (CU) triaxial compression tests with pore pressure measurements were performed on undisturbed clayey foundation samples and on compacted clayey borrow area soils. Results from these tests will be used to analyze long-term, pseudo-static (earthquake) and flood surcharge conditions. The borrow area soils were compacted to densities and moisture contents that are representative of those expected in the earthfill dams (i.e., 95% of the maximum dry density and at optimum moisture content as determined by ASTM D698).

Since the dam will be constructed primarily of sand tailings, CU triaxial compression test will be performed on tailings samples obtained from grinding rock samples. The sand tailings sample gradation will be based on the results of cyclone analyses. Sample density will be based on moisture-density test data.

To estimate settlement of the fine tailings (Subsection 5.3.B), consolidation tests will be performed on fine tailings samples. A void ratio-effective stress relationship will be determined from consolidation test results. This relationship will be used to estimate fine tailings density to confirm pond storage calculations. The fine tailings gradation will be based on the results of cyclone analyses.

5.3 Final Design Engineering

A. Stability Analyses - The following analyses will be performed:

- Limit equilibrium static and seismic stability analyses
- Dynamic response and deformation analyses
- Liquefaction analyses

A.1 Limit Equilibrium Analyses - Static and seismic stability analyses will be performed for the initial and final dam stages using the Bishop and infinite slope methods of analysis. The computer program STABL (Ref. 13) or other program will be used to determine the factor of safety for circular sliding surfaces. For cohesionless materials, calculations also will be performed using the infinite slope method. The minimum factor of safety calculated for each loading condition will be compared to the minimum acceptable values given in Section 4.2. Stability will be evaluated for an estimated phreatic surface within the dam and for an estimated foundation piezometric surface. The foundation piezometric surface will be based on the results of the relief well system design studies (Subsection 5.3.D.2).

A.2 Dynamic Response and Deformation Analyses - The peak bedrock acceleration (0.22g) determined for permit application studies (Ref. 4) will be used in the dynamic response and deformation analysis.

The predominant period of the site due to design earthquake motion was estimated to be 0.32-second from Seed, Idriss and Kiefer (Ref. 14). The duration of significant ground shaking at the site was estimated to be 27 seconds by procedures described in Bolt (Ref. 15).

An earthquake acceleration time-history record will be selected based on peak acceleration, predominant period and site conditions that are similar to those for the project site. To compute dynamic response of the dam, the record will be scaled to the peak ground acceleration determined for this project. Response spectra will be computed to verify that the record is appropriately conservative.

The dynamic response of the dam (maximum crest acceleration) due to the design ground motion will be computed by using the computer program SHAKE (Ref. 16) for a one-dimensional seismic wave propagation analysis. Shear modulus parameters of tailings and foundation materials for use in the analysis will be determined from published literature. Earthquake induced deformations will be estimated using the procedures developed by Makdisi and Seed (Ref. 17).

A.3 Liquefaction Analyses - Foundation liquefaction analyses will be performed. Shear stresses caused by seismic shaking (MCE acceleration) will be computed during the dynamic response analysis described in Subsection 5.3.A.2 above. The resistance of soils to liquefaction will be based on Standard Penetration Test data obtained during drilling (Ref. 18). The effect of silt content in soils on liquefaction potential will be evaluated (Ref. 7). Factors of safety against potential liquefaction (resistance to liquefaction/seismic shear stress) will be computed.

B. Tailings Settlement Analysis - Settlement of the fine tailings will be calculated by using conventional or, if needed, one-dimensional finite strain consolidation of fine-grained material (Ref. 19). Estimates of settlement with time will be obtained from the results of laboratory consolidation tests and procedures that relate percent consolidation to time (Ref. 20). Settlement analysis results will be used to estimate post-reclamation settlement of the impoundment surface and the need to overbuild (with coarse tailings or other fill) to compensate for settlement.

C. Diversion Channel Design - The design of the diversion channel will be based on the results of the geotechnical investigation, which may indicate the need to revise the channel location or modify the channel invert profile.

The design flood to determine the diversion dam height and diversion channel size will be based on the 6-hour local storm PMP event. The 6-hour PMF into the diversion pond (Ref. 8) has a computed peak discharge of 5230 cfs. This flood will be routed through the diversion pond by using HEC-1 procedures. The design flood to calculate flow depths and discharge rates in the diversion channel will be based on the outflow from the diversion pond plus the 6-hour PMF from the small catchment area south of the diversion channel.

Where the channel invert will be constructed in soil or soft rock, riprap protection will be designed to resist erosion from the routed 100-year flood flows. The required thickness and gradation of the riprap layer will be determined from the results of hydraulic analyses. Rock for riprap will be obtained from mine waste rock. Filter requirements between the riprap and foundation soils will be determined. Flow energy dissipation measures at the downstream end of the channel will be designed to minimize potential scour. Channel side slopes will be analyzed to confirm that they will have adequate stability.

The following diversion channel drawings will be prepared:

- Plan and Profile
- Channel Sections and Details
- Outlet Sections and Details

D. Dam and Dike Design

D.1 Embankments - The earthfill embankment structures required for the Little Cherry tailings impoundment will consist of the following:

- Tailings dam, including starter dam and toe dike
- Two saddle dams
- Diversion dam
- Seepage collection dam

The design of the structures developed during the preliminary stage will be used as a starting point for final design. Material availability will be checked and construction quantities will be estimated. Construction quantities will be used to compute mass-balance relationships.

The embankment dams will be constructed of (1) required excavation material, e.g. diversion channel, (2) materials excavated from the impoundment, (3) evaluation adit and mine waste rock and (4) tailings sands. The embankment dam zone configurations will be designed to take economical advantage of material availability.

A major effort of this work item will be directed towards designing the filter and drain system of the dam. Mine waste rock will be processed (by crushing, screening and washing) to provide materials for filters and drains. To prevent piping (internal erosion) of embankment materials, the gradations of filters and drains will be designed on the basis of established filter criteria (Ref. 21). Finger drains or blanket drains will be designed to conduct seepage through the dam and from the foundation to the downstream toe. Required thickness of the drain will be based on seepage volume and permeability of the drain materials (based on the D15 size of the drain material, Ref. 22).

Construction requirements for the embankment zones, including material gradations, placement and compaction will be determined. These requirements will form the basis for preparing construction specifications.

The ultimate impoundment will be sized to store 120 million tons of tailings. The starter impoundment will be sized to store tailings resulting from at least the first year of production. For capacity calculations, the tailings density will be based on results of the laboratory soil testing plus experience with similar tailings materials. Area-capacity curves and tailings production rates will be used to develop construction staging curves.

Analyses will be performed for various cyclone combinations to determine fines content and recovery of sands from the cyclone underflow. The analyses will be performed for pulp densities of 20 and 30% for the feed. The feed gradation will be as shown in Table 4-1. Based on the tailings production shown in Table 4-2 and a 328-day per year mill operation, the feed to the cyclones follows:

Year	Feed (Tons Per Hour)
1	479
2	699
3 to 17	889
18	728

Two stage cycloning will be needed to obtain sand tailings (cyclone underflow) clean enough for dam construction. The cyclone combination to be adopted will be that which yields the cleanest sand product and which also satisfies dam volume requirements. Laboratory testing is being performed to determine permeabilities of selected underflow gradations.

Final design of the tailings dam will include an instrumentation plan to measure piezometric pressures and seepage. A plan showing the seepage measurement weir and piezometer installation details will be prepared.

The following drawings will be produced:

- Impoundment Site Plan - Starter Stage
- Impoundment Site Plan - Final Stage
- Dam Sections and Details - Starter Stage
- Dam Sections and Details - Final Stage
- Instrumentation Plan and Installation Details
- Construction Staging Curves

This arrangement of drawings will expedite future preparation of construction drawings for the starter stage.

D.2 Pressure Relief Well System - The groundwater levels measured in borings in the dam foundation indicate that artesian pressures exist in the foundation. The highest pressure measured was 24 feet above the ground surface at Boring DH-15. In order to prevent excessive pressures from developing in the foundation during impoundment operations, a pressure relief well system will be required.

Designing the pressure relief well system requires determination of the spacing, depth and diameter of the wells. Parameters required for the analysis are foundation permeability, thickness of soil strata and hydraulic head. Permeability will be obtained from the results of constant head tests and pump tests.

The maximum allowable head between relief wells will be based on the results of stability analyses (Subsection 5.3.A.1). As described in Subsection 5.3.A.1, various foundation pressure heads will be used in stability analysis calculations and the maximum allowable pressure head in the foundation will be determined.

The relief well system will be installed during the life of the impoundment. If pressure heads increase during the expansion of the tailings impoundment, additional wells will be installed.

Typical well details will be prepared. The wells will have a screen and filter pack to prevent piping of the foundation soils. Filter criteria are specified in Ref. 22.

E. Reclamation Plan - A final impoundment reclamation scheme will be presented. Montana regulations require that the land be returned to comparable utility that existed prior to impoundment construction (Ref. 23). The reclamation plan will basically consist of (1) soil stripping and stockpiling at the beginning of each impoundment construction stage, (2) placing geofabric and coarse tailings or waste rock in depression areas, (3) deposition of coarse tailings along the upstream side of the crest during the final years of operation, (4) capping the final tailings surface and downstream slope of the tailings dam with soil, including the stockpiled soil, and (5) seeding the cap and downstream slope of the dam. The crest of the tailings dam will be built up to prevent rainfall and snowmelt runoff from flowing over the downstream slope of the dam. The north saddle dam will be graded so runoff from the tailings surface will flow through the north saddle into Bear Creek.

The following drawings will be produced:

- Impoundment Reclamation Plan
- Impoundment Reclamation Sections and Details

F. Construction Cost Estimates

F.1 Initial Estimate - For planning purposes, an initial construction cost estimate of the starter, operation and reclamation stages of the impoundment was prepared. Quantities were estimated from preliminary drawings prepared for the permit application. Standard unit costs adjusted for the project area were used for most line items. Preliminary estimates were made for major cost items such as the diversion channel excavation. For minor items, where design details have not yet been developed to permit quantity estimation, lump sum allowances were included in the estimate. Unit costs and subtotals of each impoundment structure were presented in the estimate.

Only civil construction items were included in the cost estimates; mechanical equipment (e.g., cyclones and pumps) and pipelines were not included. A construction materials flow diagram was included to show the source and placement of earth and rock material. A conceptual impoundment construction schedule for the life of the project was included in the estimate. To assist Noranda in their planning, a cash flow forecast table was also presented.

F.2. Final Estimate - After design studies have been completed, construction cost estimates will be prepared for the (1) starter stage, (2) civil work during operation and (3) reclamation of the tailings impoundment. Costs will be estimated by assuming that the work will be performed by an independent contractor. As Noranda directed, only civil construction items will be estimated; mechanical equipment (e.g., cyclones and pumps) and pipelines will not be included in the estimate. Also, operational costs for raising the dam by cycloning tailings will be excluded.

Supporting documentation will show the estimated quantity and labor, material construction equipment and subcontract cost, if applicable, for each line item in the estimate. Unit cost details will be shown where appropriate. The estimate will be accompanied by a narrative describing basic assumptions and criteria used in the development of estimated costs. Studies will be performed to establish crew composition, productivity and schedules for all significant construction activities.

Hourly construction craft labor rates (union or non-union) will be agreed on with Noranda prior to preparation of the estimate. Material and permanent equipment costs will be based on vendor quotations or experience from prior projects, as appropriate.

Allowances for material loss, damage and waste will be identified. Standard construction equipment rental and operating costs will be adjusted for project location and working conditions for use in the estimates.

Contractor mark-up, including administration and overhead costs, contingency and profit will be calculated separately and distributed to the various estimate line items in proportion to direct labor costs.

Contingency allowances and inflation rates to be included in the estimate will be discussed and agreed upon with Noranda to ensure conformance with established policy.

Estimates will be prepared using the computerized estimating package (CAES). Estimated costs will be sorted and presented in an easy readable format.

A construction schedule will be prepared for the starter stage based on the final design and cost estimate, and will identify all relevant procurement and construction tasks. Schedule tasks will be selected that have easily identifiable start and finish events to enable objective monitoring of progress. Cash flow tables will be produced from cost estimates and task activities.

5.4 Technical Reports

The following reports will be prepared:

- Geotechnical Report
- Tailings Impoundment Engineering Reports

The Geotechnical Report (Ref. 24) was prepared and contains the following information:

- Site exploration plan, showing geologic data and locations of drill holes, test pits and seismic refraction lines

- Maps showing bedrock contours, soil depth contours and potentiometric surface contours

- Geologic sections

- Results of seismic refraction surveys

- Drill hole and test pit logs

- Laboratory test results

For the purpose of providing design information to the regulatory agencies during the draft EIS review period, an interim design report will be issued. Since preparation of this report will precede the completion of tailings laboratory testing and the exploration program planned for the summer of 1990, the results of all design studies cannot be included in this report. The tailings settlement analysis, final design of the foundation pressure relief system and part of the stability analyses, including dynamic stability, will not be included in the interim report. Stability analyses of the starter dam and final tailings dam will be included in the interim report. A final design report that includes the results of all engineering studies will be issued subsequently.

The technical reports will be organized in formats agreed upon with Noranda. Each report will begin with a summary section that gives a complete overview of the work completed and results. All support information and data will be contained in readily referenced appendices.

5.5 Review Meetings

As the work progresses, periodic meetings will be held with Noranda, the Montana Department of State Lands and the U.S. Forest Service. The meetings will be used to discuss results of investigations and any critical project decisions.

6.0 REFERENCES

1. U. S. Corps of Engineers, "Engineering and Design, Stability of Earth and Rock-fill Dams," EM 1110-2-1902, April 1970, and Change 1, 17 February 1982.

2. Seed, H. B. and Martin, G. R., "The Seismic Coefficient in Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 92, SM3, 1966

3. Seed, H.B., "The Selection of Design Earthquakes for Critical Structures," Bulletin of the Seismological Society of America, Vol. 72, No. 6, December 1982.

4. Morrison-Knudsen Engineers, Inc., Geotechnical Report, February 1989.

5. Seed, H. B., "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 105, GT2, February 1979.

6. Seed, H. B., and Idriss, I. M., Ground Motions and Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, 1982.

7. Liquefaction of Soils During Earthquakes, Committee on Earthquake Engineering, National Academy of Sciences, 1985.

8. Morrison-Knudsen Engineers, Inc., Tailings Impoundment Preliminary Engineering Report, February 1989.

9. U.S. Department of Transportation, Federal Highway Administration, "Design of Stable Channels with Flexible Linings," Hydraulic Engineering Circular No. 15, October 1975.

10. U.S. Corps of Engineers, Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July 1, 1970.

11. Cedergren, H.R. Seepage, Drainage, and Flow Nets, J. Wiley & Sons, Inc., 1967.

12. U.S. Bureau of Reclamation, Earth Manual, 1974.

13. Siegel, R. A., "STABL User Manual," Joint Highway Research Project, Report JHRP-75-9, June 1988.

14. Seed, H.B., Idriss, I.M., and Kiefer, F.W. "Characteristics of Rock Motions during Earthquakes," Journal of Soil Mechanics and Foundations, American Society of Civil Engineers, Vol. 95, SM5, 1969.

15. Bolt, B.A., "Duration of Strong Ground Motion," Proc. 5th World Conference on Earthquake Engineering, Rome, Vol. 2, 1973.

16. Schnabel, B., Lysmer, J., and Seed, H.B., "SHAKE User Manual" Earthquake Engineering Research Center, Report No. EERC 72-12, December 1972.

17. Makdisi, F.I. and Seed, H.B., "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments," Earthquake Engineering Research Center, Report No. UCB/EERC - 77/19, August 1977.

18. Seed, H.B., Idriss, I.M., and Arango, I., "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 109, No. 3, March 1983.

19. Cargill, K.W., "Consolidation of Soft Layers by Finite Strain Analysis," Miscellaneous Paper GL-82-3, U.S. Army Engineer Waterways Experiment Station, March 1982.

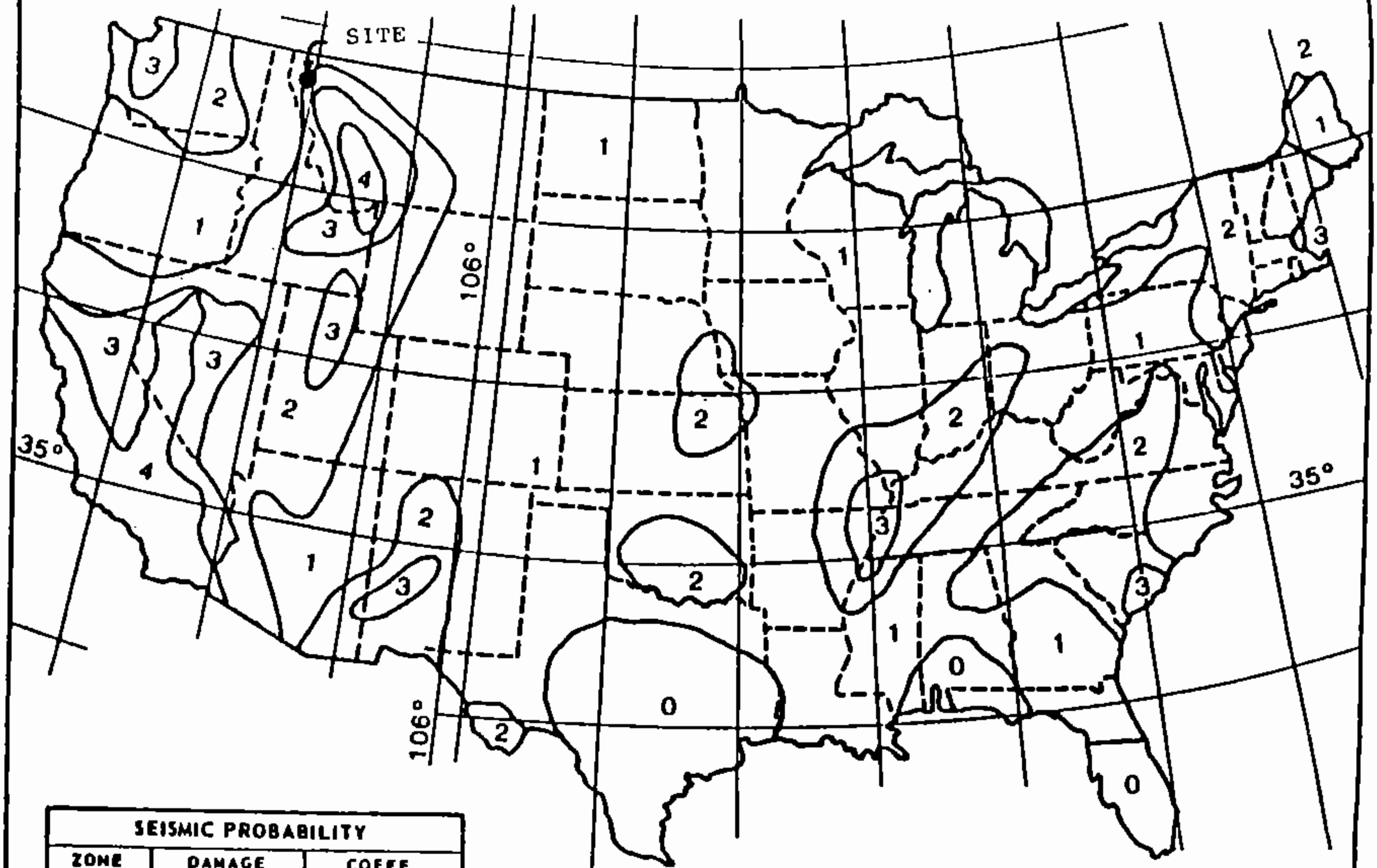
20. Cargill, K.W., "Prediction of Consolidation of Very Soft Soil," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 110, No. 6, June 1984.

21. Sherard, J.L. and Dunnigan, L.P., "Filters and Leakage Control in Embankment Dams," Seepage and Leakage from Dams and Impoundments, American Society of Civil Engineers, May 5, 1985.

22. Sherard, J.L., Dunnigan, L.P., and Talbot, J.R., "Basic Properties of Sand and Gravel Filters," Journal of Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 110, No. 6, June 1984.

23. Administrative Rules of Montana, Chapter 4, Reclamation, Part 3 - Metal Mine Reclamation and Sub-Chapter 1, Rules and Regulations Governing the Montana Metal Mine Reclamation Act, 1 July 1980.

24. Morrison-Knudsen Engineers, Inc., "Geotechnical Report, Tailings Impoundment Site," Volumes 1 and 2, March 1990.



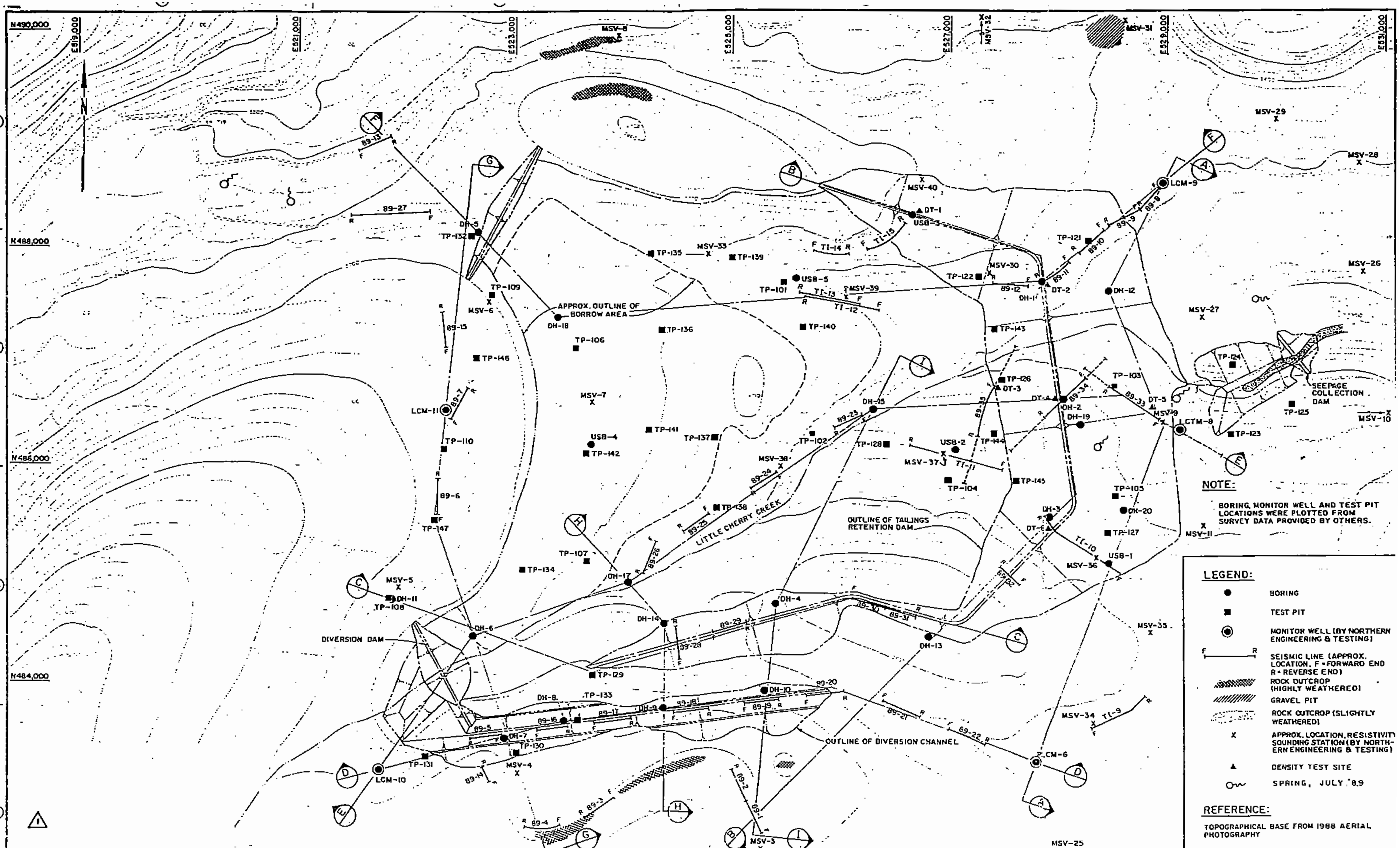
SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

SEISMIC ZONE MAP OF THE CONTIGUOUS STATES

U.S. Corps of Engineers. "Engineering and Design, Stability of Earth and Rock-Fill Dams", EM 1110-2-1902,

April 1970, and Change 1, 17 February 1982.

FIGURE 1



400 0 400 800
SCALE FEET

NO.	DATE	REVISIONS	BY	CHK	APPD
1	6/13/91	Removed Note		MPJ	

MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

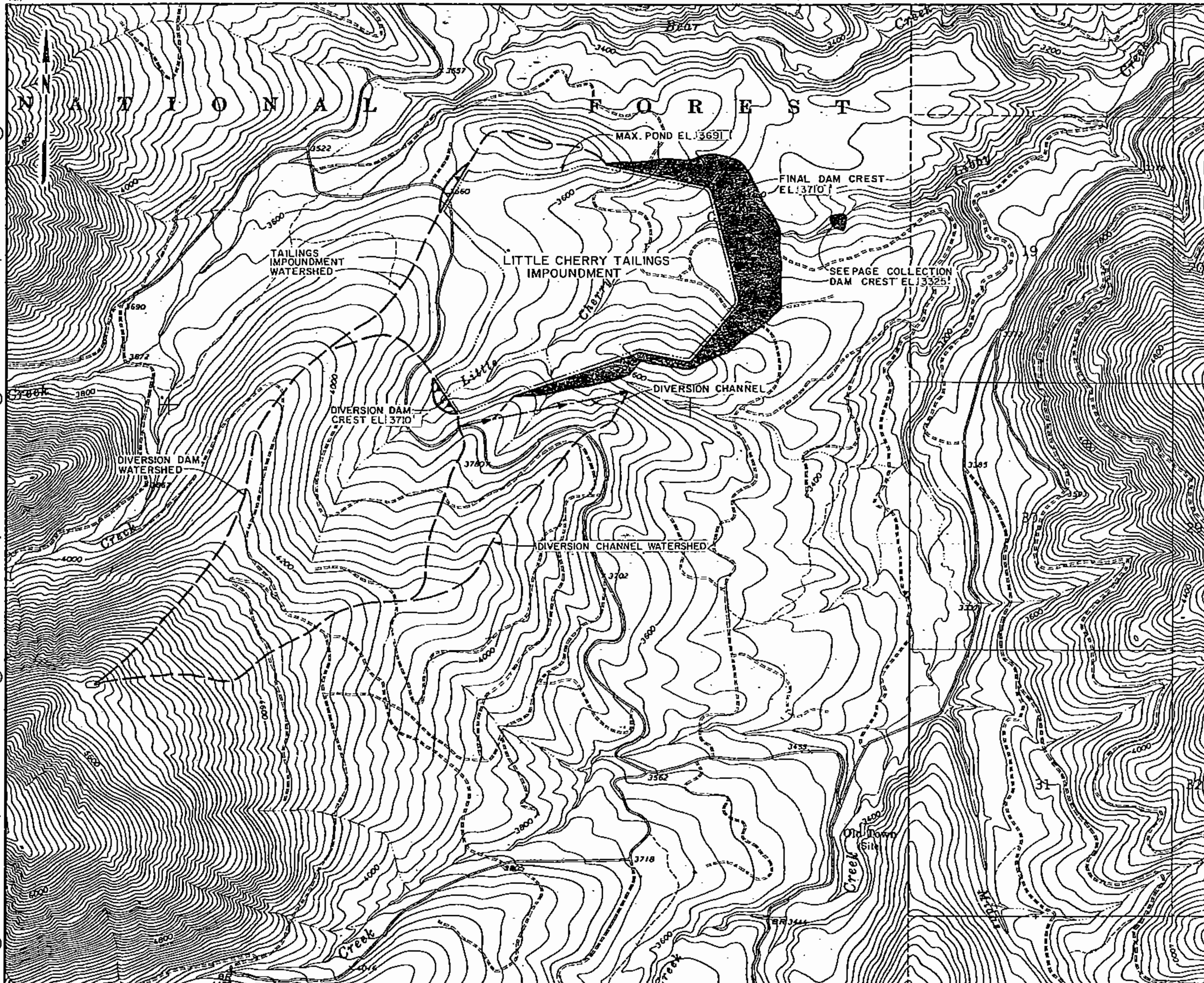
DESIGNED DJU | DRAWN RBC | CHECKED NPF | RECOMMENDED
DATE OCTOBER 1988 | APPROVED

NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY SITE
EXPLORATION PLAN

FIGURE 2

APPENDIX B
HYDROLOGIC ANALYSES



REFERENCE:

U.S. GEOLOGICAL SURVEY, 7.5 MINUTE CABLE MOUNTAIN TOPOGRAPHICAL QUADRANGLE

CONCEPTUAL–NOT FOR CONSTRUCTION

1000 0 1000 2000
SCALE FEET

NO.	DATE	REVISIONS	BY	CHK	APPD

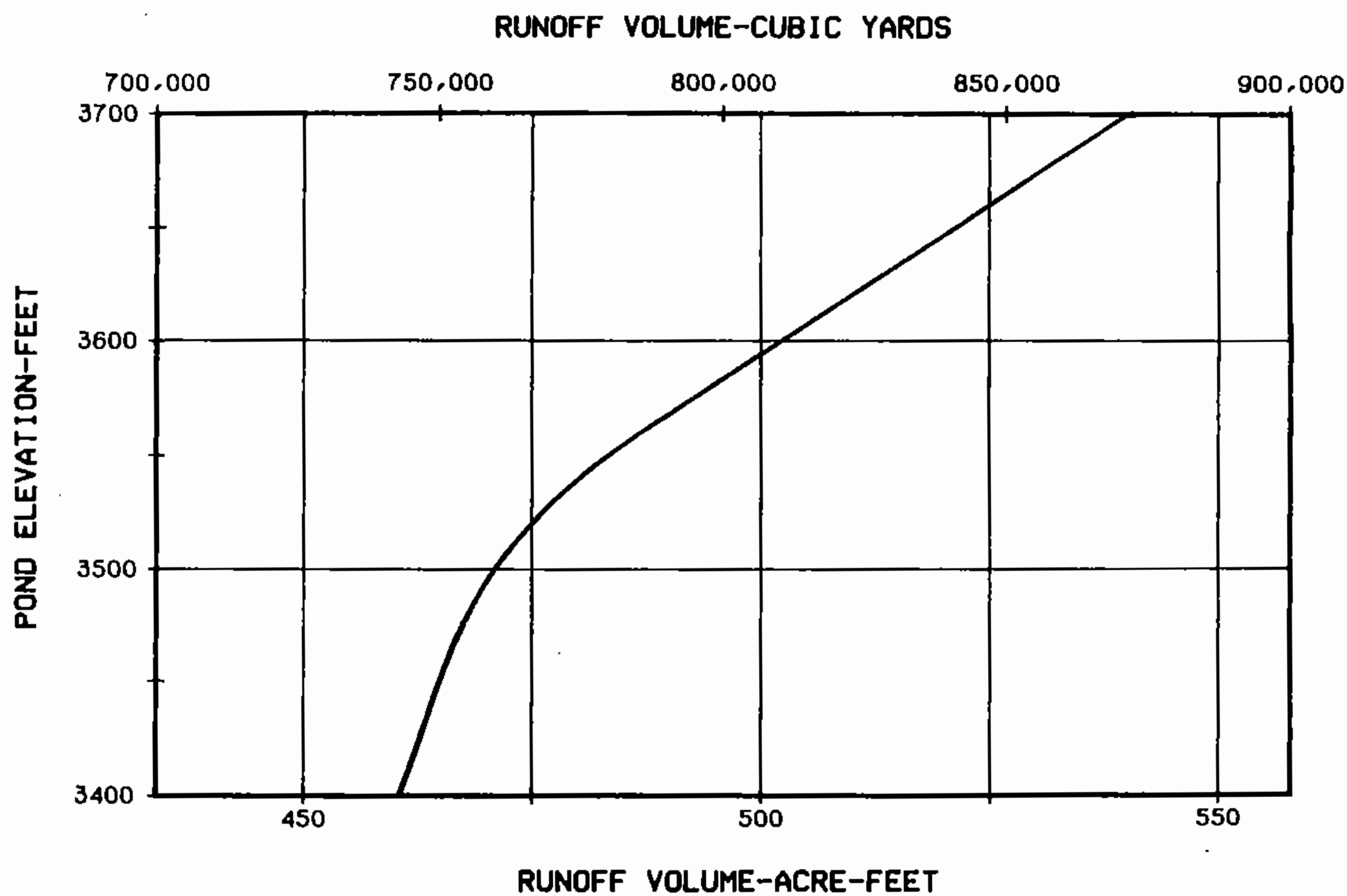
MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED VNP | DRAWN AMC | CHECKED MPF | RECOMMENDED
DATE DECEMBER 1988 | APPROVED

MONTANORE PROJECT
IMPOUNDMENT WATERSHED AREAS

MKE NO. | SHEET OF | REV.

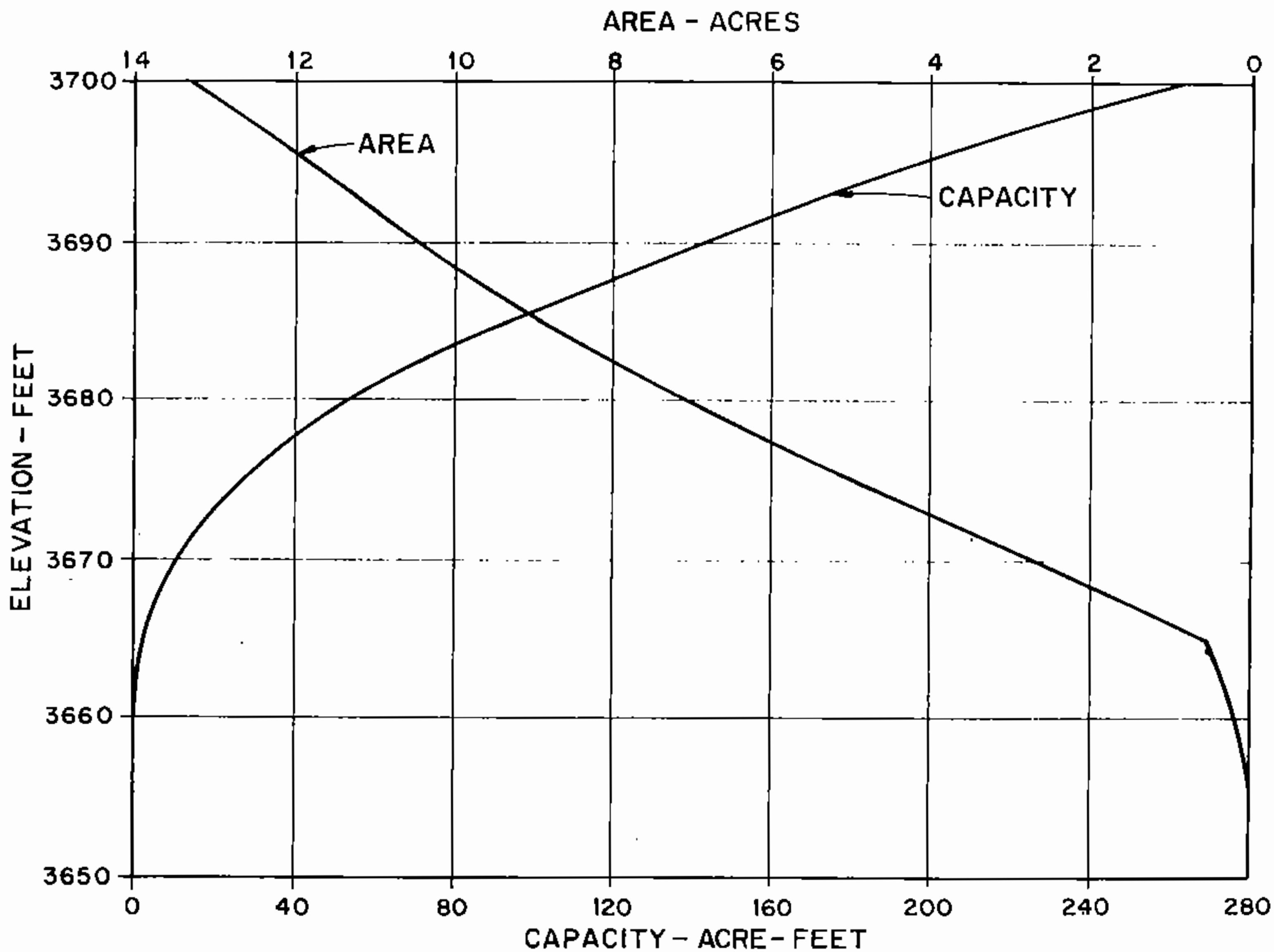
FIGURE B-1



MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

	MAN		CADS		MPF
DATE	DECEMBER 1988				

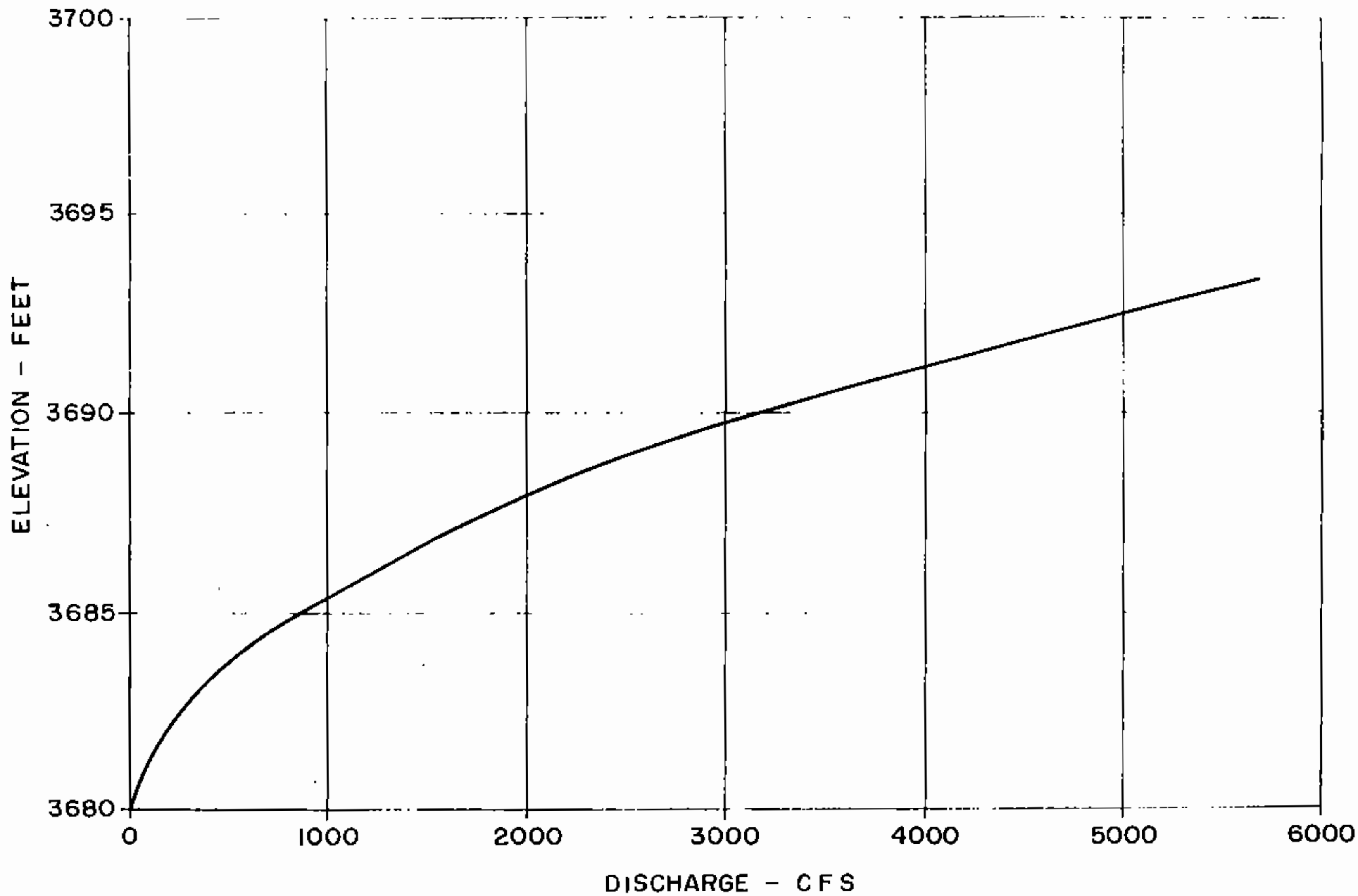
MONTANORE PROJECT
LITTLE CHERRY TAILINGS IMPOUNDMENT
RUNOFF VOLUME
FIGURE B-2



MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED ACC	DRAWN CCR	CHECKED MPF	RECOMMENDED
DATE JUNE 1990		APPROVED	

MONTANORE PROJECT
DIVERSION IMPOUNDMENT
AREA - CAPACITY CURVES
FIGURE B-3



MORRISON-KNUDSEN ENGINEERS, INC.

180 HOWARD STREET SAN FRANCISCO, CALIFORNIA 94105

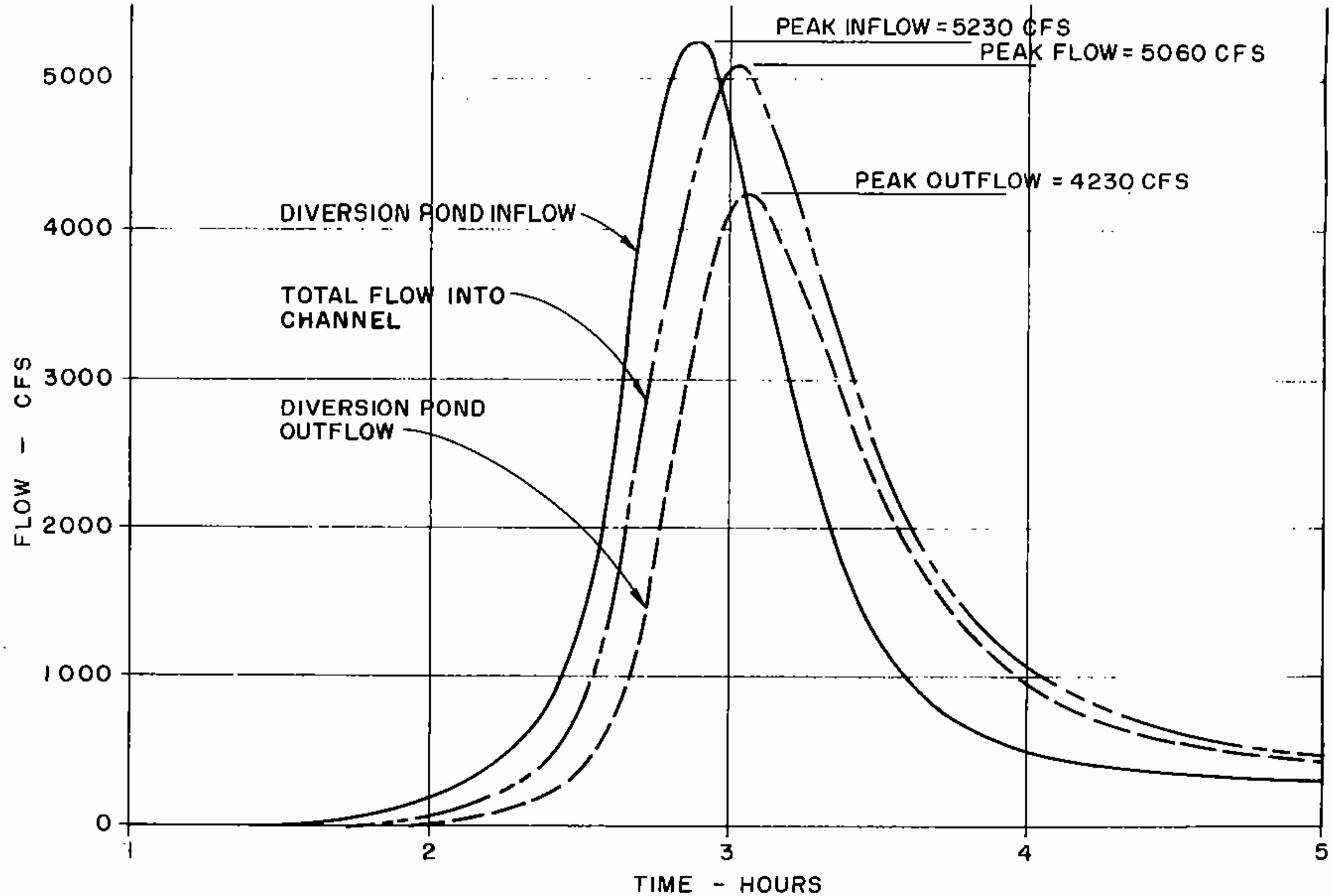
DESIGNED ACC	DRAWN CCR	CHECKED MPF	RECOMMENDED
DATE JUNE 1990		APPROVED	

MONTANORE PROJECT

DIVERSION CHANNEL

DISCHARGE RATING CURVE

FIGURE B-4



MORRISON-KNUDSEN ENGINEERS, INC.

180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

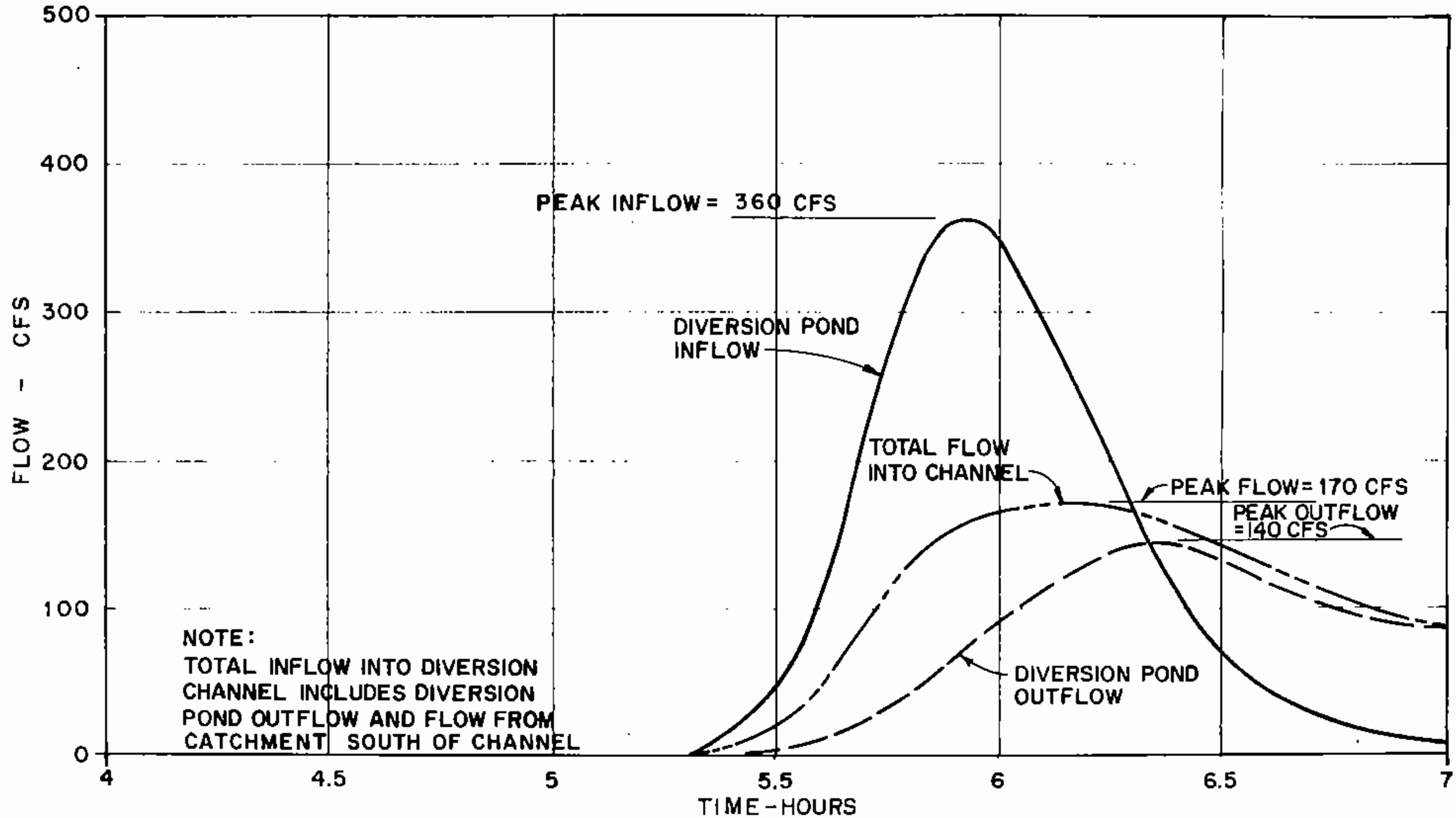
DESIGNED ACC	DRAWN BH	CHECKED MPF	RECOMMENDED
DATE JUNE 1990		APPROVED	

MONTANORE PROJECT

LITTLE CHERRY CREEK DIVERSION

6-HOUR LOCAL STORM PMF

FIGURE B-5



MORRISON-KNUDSEN ENGINEERS, INC. 180 HOWARD STREET SAN FRANCISCO, CALIFORNIA 94105			
DESIGNED ACC	DRAWN CCR	CHECKED MPF	RECOMMENDED
DATE JUNE 1990		APPROVED	

MONTANORE PROJECT

LITTLE CHERRY CREEK DIVERSION 100-YEAR, 6-HOUR FLOOD

FIGURE B-6

6-HOUR LOCAL STORM PMF
(HEC-1 Output)

```
1                                                   ****
                           FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
              U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                                    ****




            THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3


1                                            HEC-1 INPUT                                               PAGE  1

        LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

          1           ID   MONTANORE PROJECT  LITTLE CHERRY DIVERSION SITE
          2           ID    THUNDERSTORM PMF
          3           IT       5                      80
          4           IO       2       0       0

          5           KK       1 POND INFLOW
          6           BA     0.9
          7           PB       0
          8           PI     .02     .02     .02     .02     .03     .03     .03     .03     .05     .05
          9           PI     .05     .05     .08     .08     .10     .10     .12     .12     .14     .14
         10           PI     .15     .15     .16     .16     .24     .32     .40     .48     .72    1.36
         11           PI     2.0     .88     .64     .40     .32     .24     .10     .10     .10     .07
         12           PI     .07     .07     .07     .07     .07     .06     .06     .06     .05     .05
         13           PI     .05     .05     .05     .05     .05     .05     .05     .05     .05     .05
         14           PI     .04     .04     .04     .04     .04     .04     .02     .02     .02     .02
         15           PI     .02     .02
         16           LS       0      73       0
         17           UD     .32

         18           KK       1 ROUTING THROUGH POND
         19           RS       1    ELEV    3680
         20           SA       0     0.5     5.0     9.1    13.3    15.0
         21           SE    3655    3665    3675    3685    3700    3710
         22           SS    3680
         23           SQ       0       0     114     341     662    1076    1586    2194    2904    3721
         24           SQ    5686    5690
         25           SE    3660    3680  3681.5  3682.9  3684.2  3685.6  3686.8  3688.2  3689.5  3690.7
         26           SE  3693.3    3710

         27           KK       2 INCREMENTAL FLOW
         28           BA     0.2
         29           PB       0
         30           PI     .02     .02     .02     .02     .03     .03     .03     .03     .05     .05
         31           PI     .05     .05     .08     .08     .10     .10     .12     .12     .14     .14
         32           PI     .15     .15     .16     .16     .24     .32     .40     .48     .72    1.36
         33           PI     2.0     .88     .64     .40     .32     .24     .10     .10     .10     .07
         34           PI     .07     .07     .07     .07     .07     .06     .06     .06     .05     .05
         35           PI     .05     .05     .05     .05     .05     .05     .05     .05     .05     .05
         36           PI     .04     .04     .04     .04     .04     .04     .02     .02     .02     .02
         37           PI     .02     .02
         38           LS       0      73       0
         39           UD     .25

         40           KK       2  COMBINED FLOW
         41           HC       2
         42           ZZ
1                                                   ****
```

```
                         FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
          U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                                  ****


                   MONTANORE PROJECT  LITTLE CHERRY DIVERSION SITE
                    THUNDERSTORM PMF

  4 IO        OUTPUT CONTROL VARIABLES
                    IPRNT           2  PRINT CONTROL
                    IPLOT           0  PLOT CONTROL
                    QSCAL          0.  HYDROGRAPH PLOT SCALE

   IT         HYDROGRAPH TIME DATA
                     NMIN           5  MINUTES IN COMPUTATION INTERVAL
                    IDATE      1    0  STARTING DATE
                    ITIME        0000  STARTING TIME
                       NQ          80  NUMBER OF HYDROGRAPH ORDINATES
                   NDDATE      1    0  ENDING DATE
                   NDTIME        0635  ENDING TIME

                COMPUTATION INTERVAL      .08 HOURS
                     TOTAL TIME BASE     6.58 HOURS

        ENGLISH UNITS


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


         **************
         *            *
  5 KK   *          1 *    POND INFLOW
         *            *
         **************

              SUBBASIN RUNOFF DATA

  6 BA         SUBBASIN CHARACTERISTICS
                    TAREA         .90  SUBBASIN AREA

               PRECIPITATION DATA

  7 PB              STORM       11.76  BASIN TOTAL PRECIPITATION

  8 PI          INCREMENTAL PRECIPITATION PATTERN
                     .02       .02       .02       .02       .03       .03       .03       .03       .05       .05
                     .05       .05       .08       .08       .10       .10       .12       .12       .14       .14
                     .15       .15       .16       .16       .24       .32       .40       .48       .72      1.36
                    2.00       .88       .64       .40       .32       .24       .10       .10       .10       .07
                     .07       .07       .07       .07       .07       .06       .06       .06       .05       .05
                     .05       .05       .05       .05       .05       .05       .05       .05       .05       .05
                     .04       .04       .04       .04       .04       .04       .02       .02       .02       .02
                     .02       .02

 16 LS         SCS LOSS RATE
                    STRTL         .74  INITIAL ABSTRACTION
                   CRVNBR       73.00  CURVE NUMBER
                    RTIMP         .00  PERCENT IMPERVIOUS AREA

 17 UD         SCS DIMENSIONLESS UNITGRAPH
                     TLAG         .32  LAG
```

***

UNIT HYDROGRAPH
21 END-OF-PERIOD ORDINATES

153.	489.	968.	1192.	1152.	954.	657.	437.	304.	211.
144.	99.	67.	46.	32.	22.	15.	11.	8.	5.
2.									

HYDROGRAPH AT STATION 1

DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q		DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1	0000	1	.00	.00	.00	0.	*	1	0320	41	.07	.01	.06	2107.
1	0005	2	.02	.02	.00	0.	*	1	0325	42	.07	.01	.06	1652.
1	0010	3	.02	.02	.00	0.	*	1	0330	43	.07	.01	.06	1301.
1	0015	4	.02	.02	.00	0.	*	1	0335	44	.07	.01	.06	1046.
1	0020	5	.02	.02	.00	0.	*	1	0340	45	.07	.01	.06	865.
1	0025	6	.03	.03	.00	0.	*	1	0345	46	.07	.01	.06	735.
1	0030	7	.03	.03	.00	0.	*	1	0350	47	.06	.00	.06	645.
1	0035	8	.03	.03	.00	0.	*	1	0355	48	.06	.00	.06	580.
1	0040	9	.03	.03	.00	0.	*	1	0400	49	.06	.00	.06	529.
1	0045	10	.05	.05	.00	0.	*	1	0405	50	.05	.00	.05	487.
1	0050	11	.05	.05	.00	0.	*	1	0410	51	.05	.00	.05	450.
1	0055	12	.05	.05	.00	0.	*	1	0415	52	.05	.00	.05	416.
1	0100	13	.05	.05	.00	0.	*	1	0420	53	.05	.00	.05	388.
1	0105	14	.08	.08	.00	0.	*	1	0425	54	.05	.00	.05	368.
1	0110	15	.08	.08	.00	0.	*	1	0430	55	.05	.00	.05	353.
1	0115	16	.10	.10	.00	0.	*	1	0435	56	.05	.00	.05	343.
1	0120	17	.10	.10	.00	0.	*	1	0440	57	.05	.00	.05	337.
1	0125	18	.12	.11	.01	1.	*	1	0445	58	.05	.00	.05	332.
1	0130	19	.12	.11	.01	4.	*	1	0450	59	.05	.00	.05	330.
1	0135	20	.14	.12	.02	14.	*	1	0455	60	.05	.00	.05	328.
1	0140	21	.14	.11	.03	33.	*	1	0500	61	.05	.00	.05	327.
1	0145	22	.15	.11	.04	62.	*	1	0505	62	.04	.00	.04	325.
1	0150	23	.15	.10	.05	100.	*	1	0510	63	.04	.00	.04	320.
1	0155	24	.16	.10	.06	146.	*	1	0515	64	.04	.00	.04	311.
1	0200	25	.16	.10	.06	196.	*	1	0520	65	.04	.00	.04	299.
1	0205	26	.24	.13	.11	255.	*	1	0525	66	.04	.00	.04	289.
1	0210	27	.32	.16	.16	334.	*	1	0530	67	.04	.00	.04	280.
1	0215	28	.40	.17	.23	454.	*	1	0535	68	.02	.00	.02	271.
1	0220	29	.48	.18	.30	634.	*	1	0540	69	.02	.00	.02	258.
1	0225	30	.72	.22	.50	902.	*	1	0545	70	.02	.00	.02	237.
1	0230	31	1.36	.32	1.04	1350.	*	1	0550	71	.02	.00	.02	212.
1	0235	32	2.00	.31	1.69	2161.	*	1	0555	72	.02	.00	.02	190.
1	0240	33	.88	.10	.78	3309.	*	1	0600	73	.02	.00	.02	171.
1	0245	34	.64	.07	.57	4488.	*	1	0605	74	.00	.00	.00	155.
1	0250	35	.40	.04	.36	5165.	*	1	0610	75	.00	.00	.00	137.
1	0255	36	.32	.03	.29	5233.	*	1	0615	76	.00	.00	.00	113.
1	0300	37	.24	.02	.22	4802.	*	1	0620	77	.00	.00	.00	87.
1	0305	38	.10	.01	.09	4081.	*	1	0625	78	.00	.00	.00	62.
1	0310	39	.10	.01	.09	3342.	*	1	0630	79	.00	.00	.00	43.
1	0315	40	.10	.01	.09	2673.	*	1	0635	80	.00	.00	.00	29.

TOTAL RAINFALL = 11.76, TOTAL LOSS = 3.51, TOTAL EXCESS = 8.25

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	6.58-HR
+ (CFS)	(HR)					
		(CFS)				
+ 5233.	2.92		798.	727.	727.	727.

```
                 (INCHES)     8.240        8.240        8.240         8.240
                  (AC-FT)      396.         396.         396.          396.

                  CUMULATIVE AREA =      .90 SQ MI



*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *          1 *     ROUTING THROUGH POND
          *            *
          **************

              HYDROGRAPH ROUTING DATA

 19 RS          STORAGE ROUTING
                      NSTPS           1  NUMBER OF SUBREACHES
                       ITYP        ELEV  TYPE OF INITIAL CONDITION
                     RSVRIC     3680.00  INITIAL CONDITION
                          X         .00 WORKING R AND D COEFFICIENT

 20 SA                 AREA          .0          .5         5.0         9.1        13.3        15.0

 21 SE          ELEVATION       3655.00     3665.00     3675.00     3685.00     3700.00     3710.00

 23 SQ          DISCHARGE            0.          0.        114.        341.        662.       1076.       1586.       2194.       2904.       3721.
                                  5686.       5690.

 25 SE          ELEVATION       3660.00     3680.00     3681.50     3682.90     3684.20     3685.60     3686.80     3688.20     3689.50     3690.70
                                3693.30     3710.00

 22 SS          SPILLWAY
                       CREL     3680.00  SPILLWAY CREST ELEVATION
                      SPWID         .00  SPILLWAY WIDTH
                       COQW         .00  WEIR COEFFICIENT
                       EXPW        1.50  EXPONENT OF HEAD

                                                         ***

                                                  COMPUTED STORAGE-ELEVATION DATA

          STORAGE        .00       1.67      25.27      94.76     261.76     403.18
        ELEVATION    3655.00    3665.00    3675.00    3685.00    3700.00    3710.00

                                                  COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

          STORAGE        .00        .21       1.67      25.27      54.89      65.70      76.66      87.62      94.76     100.26
          OUTFLOW        .00        .00        .00        .00        .00     114.00     341.00     662.00     898.56    1076.00
        ELEVATION    3655.00    3660.00    3665.00    3675.00    3680.00    3681.50    3682.90    3684.20    3685.00    3685.60

          STORAGE     111.55     125.19     138.33     150.85     179.35     261.76     403.18
          OUTFLOW    1586.00    2194.00    2904.00    3721.00    5686.00    5687.60    5690.00
        ELEVATION    3686.80    3688.20    3689.50    3690.70    3693.30    3700.00    3710.00

*********************************************************************************************************************************

                                                   HYDROGRAPH AT STATION        1

*********************************************************************************************************************************
                                             *                                              *
 DA MON HRMN ORD   OUTFLOW   STORAGE    STAGE * DA MON HRMN ORD   OUTFLOW   STORAGE    STAGE * DA MON HRMN ORD   OUTFLOW   STORAGE    STAGE
                                             *                                              *
```

1	0000	1	0.	54.9	3680.0	*	1	0215	28	84.	62.9	3681.1	*	1	0430	55	562.	84.2	3683.8
1	0005	2	0.	54.9	3680.0	*	1	0220	29	118.	65.9	3681.5	*	1	0435	56	523.	82.9	3683.6
1	0010	3	0.	54.9	3680.0	*	1	0225	30	205.	70.1	3682.1	*	1	0440	57	489.	81.7	3683.5
1	0015	4	0.	54.9	3680.0	*	1	0230	31	327.	76.0	3682.8	*	1	0445	58	461.	80.8	3683.4
1	0020	5	0.	54.9	3680.0	*	1	0235	32	584.	85.0	3683.9	*	1	0450	59	437.	79.9	3683.3
1	0025	6	0.	54.9	3680.0	*	1	0240	33	1013.	98.3	3685.4	*	1	0455	60	417.	79.3	3683.2
1	0030	7	0.	54.9	3680.0	*	1	0245	34	1766.	115.6	3687.2	*	1	0500	61	401.	78.7	3683.1
1	0035	8	0.	54.9	3680.0	*	1	0250	35	2649.	133.6	3689.0	*	1	0505	62	387.	78.2	3683.1
1	0040	9	0.	54.9	3680.0	*	1	0255	36	3542.	148.1	3690.4	*	1	0510	63	375.	77.8	3683.0
1	0045	10	0.	54.9	3680.0	*	1	0300	37	4100.	156.3	3691.2	*	1	0515	64	364.	77.5	3683.0
1	0050	11	0.	54.9	3680.0	*	1	0305	38	4231.	158.2	3691.4	*	1	0520	65	353.	77.1	3683.0
1	0055	12	0.	54.9	3680.0	*	1	0310	39	4032.	155.4	3691.1	*	1	0525	66	343.	76.7	3682.9
1	0100	13	0.	54.9	3680.0	*	1	0315	40	3642.	149.6	3690.6	*	1	0530	67	334.	76.3	3682.9
1	0105	14	0.	54.9	3680.0	*	1	0320	41	3183.	142.6	3689.9	*	1	0535	68	326.	76.0	3682.8
1	0110	15	0.	54.9	3680.0	*	1	0325	42	2734.	135.2	3689.2	*	1	0540	69	318.	75.6	3682.8
1	0115	16	0.	54.9	3680.0	*	1	0330	43	2339.	127.9	3688.5	*	1	0545	70	309.	75.1	3682.7
1	0120	17	0.	54.9	3680.0	*	1	0335	44	2007.	121.0	3687.8	*	1	0550	71	298.	74.6	3682.6
1	0125	18	0.	54.9	3680.0	*	1	0340	45	1727.	114.7	3687.1	*	1	0555	72	285.	73.9	3682.6
1	0130	19	0.	54.9	3680.0	*	1	0345	46	1479.	109.2	3686.5	*	1	0600	73	271.	73.3	3682.5
1	0135	20	1.	55.0	3680.0	*	1	0350	47	1267.	104.5	3686.0	*	1	0605	74	256.	72.6	3682.4
1	0140	21	2.	55.1	3680.0	*	1	0355	48	1090.	100.6	3685.6	*	1	0610	75	242.	71.9	3682.3
1	0145	22	6.	55.4	3680.1	*	1	0400	49	980.	97.3	3685.3	*	1	0615	76	226.	71.1	3682.2
1	0150	23	11.	55.9	3680.1	*	1	0405	50	885.	94.3	3685.0	*	1	0620	77	209.	70.3	3682.1
1	0155	24	19.	56.7	3680.2	*	1	0410	51	800.	91.8	3684.7	*	1	0625	78	192.	69.4	3682.0
1	0200	25	29.	57.7	3680.4	*	1	0415	52	725.	89.5	3684.4	*	1	0630	79	173.	68.6	3681.9
1	0205	26	43.	59.0	3680.6	*	1	0420	53	659.	87.5	3684.2	*	1	0635	80	155.	67.7	3681.8
1	0210	27	61.	60.7	3680.8	*	1	0425	54	607.	85.8	3684.0	*						
						*							*						

****

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	6.58-HR
+ (CFS)	(HR)					
		(CFS)				
+ 4231.	3.08		772.	703.	703.	703.
		(INCHES)	7.974	7.974	7.974	7.974
		(AC-FT)	383.	383.	383.	383.

PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
		6-HR	24-HR	72-HR	6.58-HR
+ (AC-FT)	(HR)				
158.	3.08	83.	81.	81.	81.

PEAK STAGE	TIME	MAXIMUM AVERAGE STAGE			
		6-HR	24-HR	72-HR	6.58-HR
+ (FEET)	(HR)				
3691.37	3.08	3683.47	3683.16	3683.16	3683.16

CUMULATIVE AREA = .90 SQ MI

*** ***

```
          **************
          *            *
  27 KK   *       2    *     INCREMENTAL FLOW
          *            *
          **************

             SUBBASIN RUNOFF DATA

  28 BA        SUBBASIN CHARACTERISTICS
                     TAREA        .20  SUBBASIN AREA
```

PRECIPITATION DATA

29 PB STORM 11.76 BASIN TOTAL PRECIPITATION

30 PI INCREMENTAL PRECIPITATION PATTERN

.02	.02	.02	.02	.03	.03	.03	.03	.05	.05
.05	.05	.08	.08	.10	.10	.12	.12	.14	.14
.15	.15	.16	.16	.24	.32	.40	.48	.72	1.36
2.00	.88	.64	.40	.32	.24	.10	.10	.10	.07
.07	.07	.07	.07	.07	.06	.06	.06	.05	.05
.05	.05	.05	.05	.05	.05	.05	.05	.05	.05
.04	.04	.04	.04	.04	.04	.02	.02	.02	.02
.02	.02								

38 LS SCS LOSS RATE

STRTL .74 INITIAL ABSTRACTION

CRVNBR 73.00 CURVE NUMBER

RTIMP .00 PERCENT IMPERVIOUS AREA

39 UD SCS DIMENSIONLESS UNITGRAPH

TLAG .25 LAG

***

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

UNIT HYDROGRAPH

17 END-OF-PERIOD ORDINATES

59.	201.	320.	320.	249.	149.	93.	60.	37.	23.
15.	9.	6.	4.	3.	1.	0.			

****

HYDROGRAPH AT STATION 2

****

DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1	0000	1	.00	.00	.00	0.	*	1	0320	41	.07	.01	.06	339.
1	0005	2	.02	.02	.00	0.	*	1	0325	42	.07	.01	.06	259.
1	0010	3	.02	.02	.00	0.	*	1	0330	43	.07	.01	.06	204.
1	0015	4	.02	.02	.00	0.	*	1	0335	44	.07	.01	.06	167.
1	0020	5	.02	.02	.00	0.	*	1	0340	45	.07	.01	.06	142.
1	0025	6	.03	.03	.00	0.	*	1	0345	46	.07	.01	.06	126.
1	0030	7	.03	.03	.00	0.	*	1	0350	47	.06	.00	.06	115.
1	0035	8	.03	.03	.00	0.	*	1	0355	48	.06	.00	.06	107.
1	0040	9	.03	.03	.00	0.	*	1	0400	49	.06	.00	.06	100.
1	0045	10	.05	.05	.00	0.	*	1	0405	50	.05	.00	.05	94.
1	0050	11	.05	.05	.00	0.	*	1	0410	51	.05	.00	.05	89.
1	0055	12	.05	.05	.00	0.	*	1	0415	52	.05	.00	.05	84.
1	0100	13	.05	.05	.00	0.	*	1	0420	53	.05	.00	.05	80.
1	0105	14	.08	.08	.00	0.	*	1	0425	54	.05	.00	.05	77.
1	0110	15	.08	.08	.00	0.	*	1	0430	55	.05	.00	.05	75.
1	0115	16	.10	.10	.00	0.	*	1	0435	56	.05	.00	.05	74.
1	0120	17	.10	.10	.00	0.	*	1	0440	57	.05	.00	.05	73.
1	0125	18	.12	.11	.01	0.	*	1	0445	58	.05	.00	.05	73.
1	0130	19	.12	.11	.01	2.	*	1	0450	59	.05	.00	.05	73.
1	0135	20	.14	.12	.02	5.	*	1	0455	60	.05	.00	.05	72.
1	0140	21	.14	.11	.03	12.	*	1	0500	61	.05	.00	.05	72.
1	0145	22	.15	.11	.04	20.	*	1	0505	62	.04	.00	.04	72.
1	0150	23	.15	.10	.05	31.	*	1	0510	63	.04	.00	.04	70.
1	0155	24	.16	.10	.06	43.	*	1	0515	64	.04	.00	.04	67.
1	0200	25	.16	.10	.06	55.	*	1	0520	65	.04	.00	.04	64.
1	0205	26	.24	.13	.11	70.	*	1	0525	66	.04	.00	.04	62.

1	0210	27	.32	.16	.16	92.	*	1	0530	67	.04	.00	.04	60.
1	0215	28	.40	.17	.23	129.	*	1	0535	68	.02	.00	.02	58.
1	0220	29	.48	.18	.30	182.	*	1	0540	69	.02	.00	.02	54.
1	0225	30	.72	.22	.50	261.	*	1	0545	70	.02	.00	.02	48.
1	0230	31	1.36	.32	1.04	399.	*	1	0550	71	.02	.00	.02	41.
1	0235	32	2.00	.31	1.69	662.	*	1	0555	72	.02	.00	.02	37.
1	0240	33	.88	.10	.78	1012.	*	1	0600	73	.02	.00	.02	34.
1	0245	34	.64	.07	.57	1269.	*	1	0605	74	.00	.00	.00	31.
1	0250	35	.40	.04	.36	1310.	*	1	0610	75	.00	.00	.00	26.
1	0255	36	.32	.03	.29	1177.	*	1	0615	76	.00	.00	.00	19.
1	0300	37	.24	.02	.22	964.	*	1	0620	77	.00	.00	.00	13.
1	0305	38	.10	.01	.09	769.	*	1	0625	78	.00	.00	.00	8.
1	0310	39	.10	.01	.09	596.	*	1	0630	79	.00	.00	.00	5.
1	0315	40	.10	.01	.09	450.	*	1	0635	80	.00	.00	.00	3.
							*							

****

TOTAL RAINFALL = 11.76, TOTAL LOSS = 3.51, TOTAL EXCESS = 8.25

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	6.58-HR
+	(CFS)	(HR)					
			(CFS)				
+	1310.	2.83		177.	162.	162.	162.
			(INCHES)	8.247	8.247	8.247	8.247
			(AC-FT)	88.	88.	88.	88.

CUMULATIVE AREA = .20 SQ MI

*** ***

```
            **************
            *            *
  40 KK     *          2 *      COMBINED FLOW
            *            *
            **************
```

41 HC HYDROGRAPH COMBINATION

ICOMP 2 NUMBER OF HYDROGRAPHS TO COMBINE

***

****

HYDROGRAPH AT STATION 2

SUM OF 2 HYDROGRAPHS

****

DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW
1		0000	1	0.	*	1		0140	21	14.	*	1		0320	41	3522.	*	1		0500	61	473.
1		0005	2	0.	*	1		0145	22	26.	*	1		0325	42	2993.	*	1		0505	62	459.
1		0010	3	0.	*	1		0150	23	42.	*	1		0330	43	2543.	*	1		0510	63	445.
1		0015	4	0.	*	1		0155	24	62.	*	1		0335	44	2174.	*	1		0515	64	431.
1		0020	5	0.	*	1		0200	25	85.	*	1		0340	45	1869.	*	1		0520	65	417.
1		0025	6	0.	*	1		0205	26	113.	*	1		0345	46	1605.	*	1		0525	66	404.
1		0030	7	0.	*	1		0210	27	153.	*	1		0350	47	1382.	*	1		0530	67	395.
1		0035	8	0.	*	1		0215	28	213.	*	1		0355	48	1197.	*	1		0535	68	385.
1		0040	9	0.	*	1		0220	29	301.	*	1		0400	49	1079.	*	1		0540	69	372.
1		0045	10	0.	*	1		0225	30	466.	*	1		0405	50	979.	*	1		0545	70	356.
1		0050	11	0.	*	1		0230	31	726.	*	1		0410	51	888.	*	1		0550	71	339.

1	0055	12	0.	*	1	0235	32	1246.	*	1	0415	52	808.	*	1	0555	72	321.
1	0100	13	0.	*	1	0240	33	2025.	*	1	0420	53	738.	*	1	0600	73	305.
1	0105	14	0.	*	1	0245	34	3034.	*	1	0425	54	684.	*	1	0605	74	287.
1	0110	15	0.	*	1	0250	35	3960.	*	1	0430	55	637.	*	1	0610	75	268.
1	0115	16	0.	*	1	0255	36	4719.	*	1	0435	56	597.	*	1	0615	76	246.
1	0120	17	0.	*	1	0300	37	5063.	*	1	0440	57	562.	*	1	0620	77	222.
1	0125	18	0.	*	1	0305	38	5000.	*	1	0445	58	534.	*	1	0625	78	199.
1	0130	19	2.	*	1	0310	39	4627.	*	1	0450	59	510.	*	1	0630	79	178.
1	0135	20	6.	*	1	0315	40	4092.	*	1	0455	60	490.	*	1	0635	80	158.
				*					*					*				

****

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	6.58-HR
+ (CFS)	(HR)					
		(CFS)				
+ 5063.	3.00		949.	865.	865.	865.
		(INCHES)	8.024	8.024	8.024	8.024
		(AC-FT)	471.	471.	471.	471.

CUMULATIVE AREA = 1.10 SQ MI

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+										
	HYDROGRAPH AT									
+		1	5233.	2.92	798.	727.	727.	.90		
	ROUTED TO									
+		1	4231.	3.08	772.	703.	703.	.90		
+									3691.37	3.08
	HYDROGRAPH AT									
+		2	1310.	2.83	177.	162.	162.	.20		
	2 COMBINED AT									
+		2	5063.	3.00	949.	865.	865.	1.10		

1 HEC-1 INPUT

LINE ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

100-YEAR, 6-HOUR FLOOD
(HEC-1 Output)

1

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985

U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3

1 HEC-1 INPUT

```
LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1         ID   MONTANORE PROJECT  LITTLE CHERRY DIVERSION SITE
  2         ID    100-YR, 6-HR STORM
  3         ID    DISTRIBUTION =6,5,4,3,2,1
  4         IT       5                     100
  5         IO       2       0       0

  6         KK       1 POND INFLOW
  7         BA     0.9
  8         PB       0
  9         PI    .010    .010    .010    .010    .010    .010    .011    .011    .011    .011
 10         PI    .011    .011    .011    .011    .011    .011    .011    .011    .011    .011
 11         PI    .011    .011    .011    .011    .011    .011    .011    .011    .011    .011
 12         PI    .011    .011    .011    .011    .011    .011    .013    .013    .013    .013
 13         PI    .013    .013    .013    .013    .014    .014    .014    .014    .015    .015
 14         PI    .016    .016    .016    .016    .016    .016    .016    .016    .016    .016
 15         PI    .044    .058    .073    .088    .131    .248    .365    .161    .117    .073
 16         PI    .058    .044
 17         PI
 18         PI
 19         LS       0      73       0
 20         UD     .32

 21         KK       1 ROUTING THROUGH POND
 22         RS       1    ELEV    3680
 23         SA     0.0     0.5     5.0     9.1    13.3    15.0
 24         SE    3655    3665    3675    3685    3700    3710
 25         SS    3680
 26         SQ       0       0     114     341     662    1076    1586    2194    2904    3721
 27         SQ    5686    5690
 28         SE    3660    3680  3681.5  3682.9  3684.2  3685.6  3686.8  3688.2  3689.5  3690.7
 29         SE  3693.3    3710

 30         KK       2 INCREMENTAL FLOW
 31         BA     0.2
 32         PB       0
 33         PI    .010    .010    .010    .010    .010    .010    .011    .011    .011    .011
 34         PI    .011    .011    .011    .011    .011    .011    .011    .011    .011    .011
 35         PI    .011    .011    .011    .011    .011    .011    .011    .011    .011    .011
 36         PI    .011    .011    .011    .011    .011    .011    .013    .013    .013    .013
 37         PI    .013    .013    .013    .013    .014    .014    .014    .014    .015    .015
 38         PI    .016    .016    .016    .016    .016    .016    .016    .016    .016    .016
 39         PI    .044    .058    .073    .088    .131    .248    .365    .161    .117    .073
 40         PI    .058    .044
 41         PI
 42         PI
 43         LS       0      73       0
 44         UD     .25
```

```
          45          KK       2  COMBINED FLOW
          46          HC       2
          47          ZZ
1                                                          ****
                                  FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
               U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                                           ****



                         MONTANORE PROJECT  LITTLE CHERRY DIVERSION SITE
                          100-YR, 6-HR STORM
                          DISTRIBUTION =6,5,4,3,2,1

    5 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           2  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

      IT          HYDROGRAPH TIME DATA
                        NMIN           5  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1    0  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         100  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1    0  ENDING DATE
                      NDTIME        0815  ENDING TIME

                 COMPUTATION INTERVAL     .08 HOURS
                     TOTAL TIME BASE    8.25 HOURS

             ENGLISH UNITS


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


               **************
               *            *
    6 KK       *        1   *      POND INFLOW
               *            *
               **************

                  SUBBASIN RUNOFF DATA

    7 BA          SUBBASIN CHARACTERISTICS
                       TAREA         .90  SUBBASIN AREA

                  PRECIPITATION DATA

    8 PB               STORM        2.20  BASIN TOTAL PRECIPITATION

    9 PI            INCREMENTAL PRECIPITATION PATTERN
                         .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                         .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                         .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                         .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                         .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                         .02       .02       .02       .02       .02       .02       .02       .02       .02       .02
                         .04       .06       .07       .09       .13       .25       .37       .16       .12       .07
                         .06       .04

   19 LS          SCS LOSS RATE
                       STRTL         .74  INITIAL ABSTRACTION
```

CRVNBR 73.00 CURVE NUMBER
RTIMP .00 PERCENT IMPERVIOUS AREA

20 UD SCS DIMENSIONLESS UNITGRAPH
TLAG .32 LAG

***

UNIT HYDROGRAPH
21 END-OF-PERIOD ORDINATES

153.	489.	968.	1192.	1152.	954.	657.	437.	304.	211.
144.	99.	67.	46.	32.	22.	15.	11.	8.	5.
2.									

****

HYDROGRAPH AT STATION 1

****

DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1		0000	1	.00	.00	.00	0.	*	1		0410	51	.01	.01	.00	0.
1		0005	2	.01	.01	.00	0.	*	1		0415	52	.02	.02	.00	0.
1		0010	3	.01	.01	.00	0.	*	1		0420	53	.02	.02	.00	0.
1		0015	4	.01	.01	.00	0.	*	1		0425	54	.02	.02	.00	0.
1		0020	5	.01	.01	.00	0.	*	1		0430	55	.02	.02	.00	0.
1		0025	6	.01	.01	.00	0.	*	1		0435	56	.02	.02	.00	0.
1		0030	7	.01	.01	.00	0.	*	1		0440	57	.02	.02	.00	0.
1		0035	8	.01	.01	.00	0.	*	1		0445	58	.02	.02	.00	0.
1		0040	9	.01	.01	.00	0.	*	1		0450	59	.02	.02	.00	0.
1		0045	10	.01	.01	.00	0.	*	1		0455	60	.02	.02	.00	0.
1		0050	11	.01	.01	.00	0.	*	1		0500	61	.02	.02	.00	0.
1		0055	12	.01	.01	.00	0.	*	1		0505	62	.04	.04	.00	0.
1		0100	13	.01	.01	.00	0.	*	1		0510	63	.06	.06	.00	1.
1		0105	14	.01	.01	.00	0.	*	1		0515	64	.07	.07	.01	2.
1		0110	15	.01	.01	.00	0.	*	1		0520	65	.09	.08	.01	7.
1		0115	16	.01	.01	.00	0.	*	1		0525	66	.13	.11	.02	16.
1		0120	17	.01	.01	.00	0.	*	1		0530	67	.25	.19	.06	37.
1		0125	18	.01	.01	.00	0.	*	1		0535	68	.37	.24	.12	86.
1		0130	19	.01	.01	.00	0.	*	1		0540	69	.16	.10	.06	166.
1		0135	20	.01	.01	.00	0.	*	1		0545	70	.12	.07	.05	262.
1		0140	21	.01	.01	.00	0.	*	1		0550	71	.07	.04	.03	331.
1		0145	22	.01	.01	.00	0.	*	1		0555	72	.06	.03	.03	360.
1		0150	23	.01	.01	.00	0.	*	1		0600	73	.04	.02	.02	350.
1		0155	24	.01	.01	.00	0.	*	1		0605	74	.00	.00	.00	311.
1		0200	25	.01	.01	.00	0.	*	1		0610	75	.00	.00	.00	259.
1		0205	26	.01	.01	.00	0.	*	1		0615	76	.00	.00	.00	204.
1		0210	27	.01	.01	.00	0.	*	1		0620	77	.00	.00	.00	151.
1		0215	28	.01	.01	.00	0.	*	1		0625	78	.00	.00	.00	107.
1		0220	29	.01	.01	.00	0.	*	1		0630	79	.00	.00	.00	73.
1		0225	30	.01	.01	.00	0.	*	1		0635	80	.00	.00	.00	50.
1		0230	31	.01	.01	.00	0.	*	1		0640	81	.00	.00	.00	34.
1		0235	32	.01	.01	.00	0.	*	1		0645	82	.00	.00	.00	24.
1		0240	33	.01	.01	.00	0.	*	1		0650	83	.00	.00	.00	16.
1		0245	34	.01	.01	.00	0.	*	1		0655	84	.00	.00	.00	11.
1		0250	35	.01	.01	.00	0.	*	1		0700	85	.00	.00	.00	8.
1		0255	36	.01	.01	.00	0.	*	1		0705	86	.00	.00	.00	5.
1		0300	37	.01	.01	.00	0.	*	1		0710	87	.00	.00	.00	4.
1		0305	38	.01	.01	.00	0.	*	1		0715	88	.00	.00	.00	2.
1		0310	39	.01	.01	.00	0.	*	1		0720	89	.00	.00	.00	1.
1		0315	40	.01	.01	.00	0.	*	1		0725	90	.00	.00	.00	1.
1		0320	41	.01	.01	.00	0.	*	1		0730	91	.00	.00	.00	0.
1		0325	42	.01	.01	.00	0.	*	1		0735	92	.00	.00	.00	0.
1		0330	43	.01	.01	.00	0.	*	1		0740	93	.00	.00	.00	0.
1		0335	44	.01	.01	.00	0.	*	1		0745	94	.00	.00	.00	0.

```
   1    0340   45    .01    .01    .00        0.     *     1    0750   95    .00    .00    .00        0.
   1    0345   46    .01    .01    .00        0.     *     1    0755   96    .00    .00    .00        0.
   1    0350   47    .01    .01    .00        0.     *     1    0800   97    .00    .00    .00        0.
   1    0355   48    .01    .01    .00        0.     *     1    0805   98    .00    .00    .00        0.
   1    0400   49    .01    .01    .00        0.     *     1    0810   99    .00    .00    .00        0.
   1    0405   50    .01    .01    .00        0.     *     1    0815  100    .00    .00    .00        0.
                                                  *
*****************************************************************************************************************

   TOTAL RAINFALL =    2.20, TOTAL LOSS =    1.79, TOTAL EXCESS =     .41

  PEAK FLOW     TIME                        MAXIMUM AVERAGE FLOW
                                     6-HR       24-HR       72-HR      8.25-HR
 +   (CFS)      (HR)
                          (CFS)
 +     360.     5.92                  40.         29.         29.         29.
                        (INCHES)     .413        .413        .413        .413
                         (AC-FT)      20.         20.         20.         20.

                        CUMULATIVE AREA =     .90 SQ MI



*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
   21 KK    *         1  *      ROUTING THROUGH POND
            *            *
            **************

               HYDROGRAPH ROUTING DATA

   22 RS         STORAGE ROUTING
                      NSTPS            1  NUMBER OF SUBREACHES
                       ITYP         ELEV  TYPE OF INITIAL CONDITION
                     RSVRIC      3680.00  INITIAL CONDITION
                          X          .00 WORKING R AND D COEFFICIENT

   23 SA               AREA         .0         .5        5.0        9.1       13.3       15.0

   24 SE          ELEVATION    3655.00    3665.00    3675.00    3685.00    3700.00    3710.00

   26 SQ          DISCHARGE         0.         0.       114.       341.       662.      1076.      1586.      2194.      2904.      3721.
                                 5686.      5690.

   28 SE          ELEVATION    3660.00    3680.00    3681.50    3682.90    3684.20    3685.60    3686.80    3688.20    3689.50    3690.70
                               3693.30    3710.00

   25 SS          SPILLWAY
                       CREL      3680.00  SPILLWAY CREST ELEVATION
                      SPWID          .00  SPILLWAY WIDTH
                       COQW          .00  WEIR COEFFICIENT
                       EXPW         1.50  EXPONENT OF HEAD

                                                  ***

                                             COMPUTED STORAGE-ELEVATION DATA

            STORAGE        .00       1.67      25.27      94.76     261.76     403.18
          ELEVATION    3655.00    3665.00    3675.00    3685.00    3700.00    3710.00

                                          COMPUTED STORAGE-OUTFLOW-ELEVATION DATA
```

STORAGE	.00	.21	1.67	25.27	54.89	65.70	76.66	87.62	94.76	100.26
OUTFLOW	.00	.00	.00	.00	.00	114.00	341.00	662.00	898.56	1076.00
ELEVATION	3655.00	3660.00	3665.00	3675.00	3680.00	3681.50	3682.90	3684.20	3685.00	3685.60
STORAGE	111.55	125.19	138.33	150.85	179.35	261.76	403.18			
OUTFLOW	1586.00	2194.00	2904.00	3721.00	5686.00	5687.60	5690.00			
ELEVATION	3686.80	3688.20	3689.50	3690.70	3693.30	3700.00	3710.00			

HYDROGRAPH AT STATION 1

DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1		0000	1	0.	54.9	3680.0	1		0250	35	0.	54.9	3680.0	1		0540	69	15.	56.4	3680.2
1		0005	2	0.	54.9	3680.0	1		0255	36	0.	54.9	3680.0	1		0545	70	29.	57.7	3680.4
1		0010	3	0.	54.9	3680.0	1		0300	37	0.	54.9	3680.0	1		0550	71	48.	59.4	3680.6
1		0015	4	0.	54.9	3680.0	1		0305	38	0.	54.9	3680.0	1		0555	72	69.	61.4	3680.9
1		0020	5	0.	54.9	3680.0	1		0310	39	0.	54.9	3680.0	1		0600	73	89.	63.3	3681.2
1		0025	6	0.	54.9	3680.0	1		0315	40	0.	54.9	3680.0	1		0605	74	106.	64.9	3681.4
1		0030	7	0.	54.9	3680.0	1		0320	41	0.	54.9	3680.0	1		0610	75	122.	66.1	3681.6
1		0035	8	0.	54.9	3680.0	1		0325	42	0.	54.9	3680.0	1		0615	76	137.	66.8	3681.6
1		0040	9	0.	54.9	3680.0	1		0330	43	0.	54.9	3680.0	1		0620	77	142.	67.1	3681.7
1		0045	10	0.	54.9	3680.0	1		0335	44	0.	54.9	3680.0	1		0625	78	141.	67.0	3681.7
1		0050	11	0.	54.9	3680.0	1		0340	45	0.	54.9	3680.0	1		0630	79	134.	66.7	3681.6
1		0055	12	0.	54.9	3680.0	1		0345	46	0.	54.9	3680.0	1		0635	80	124.	66.2	3681.6
1		0100	13	0.	54.9	3680.0	1		0350	47	0.	54.9	3680.0	1		0640	81	114.	65.7	3681.5
1		0105	14	0.	54.9	3680.0	1		0355	48	0.	54.9	3680.0	1		0645	82	108.	65.1	3681.4
1		0110	15	0.	54.9	3680.0	1		0400	49	0.	54.9	3680.0	1		0650	83	102.	64.5	3681.3
1		0115	16	0.	54.9	3680.0	1		0405	50	0.	54.9	3680.0	1		0655	84	95.	63.9	3681.3
1		0120	17	0.	54.9	3680.0	1		0410	51	0.	54.9	3680.0	1		0700	85	89.	63.4	3681.2
1		0125	18	0.	54.9	3680.0	1		0415	52	0.	54.9	3680.0	1		0705	86	84.	62.8	3681.1
1		0130	19	0.	54.9	3680.0	1		0420	53	0.	54.9	3680.0	1		0710	87	78.	62.3	3681.0
1		0135	20	0.	54.9	3680.0	1		0425	54	0.	54.9	3680.0	1		0715	88	73.	61.8	3681.0
1		0140	21	0.	54.9	3680.0	1		0430	55	0.	54.9	3680.0	1		0720	89	68.	61.3	3680.9
1		0145	22	0.	54.9	3680.0	1		0435	56	0.	54.9	3680.0	1		0725	90	63.	60.9	3680.8
1		0150	23	0.	54.9	3680.0	1		0440	57	0.	54.9	3680.0	1		0730	91	59.	60.5	3680.8
1		0155	24	0.	54.9	3680.0	1		0445	58	0.	54.9	3680.0	1		0735	92	55.	60.1	3680.7
1		0200	25	0.	54.9	3680.0	1		0450	59	0.	54.9	3680.0	1		0740	93	51.	59.7	3680.7
1		0205	26	0.	54.9	3680.0	1		0455	60	0.	54.9	3680.0	1		0745	94	47.	59.4	3680.6
1		0210	27	0.	54.9	3680.0	1		0500	61	0.	54.9	3680.0	1		0750	95	44.	59.1	3680.6
1		0215	28	0.	54.9	3680.0	1		0505	62	0.	54.9	3680.0	1		0755	96	41.	58.8	3680.5
1		0220	29	0.	54.9	3680.0	1		0510	63	0.	54.9	3680.0	1		0800	97	38.	58.5	3680.5
1		0225	30	0.	54.9	3680.0	1		0515	64	0.	54.9	3680.0	1		0805	98	35.	58.2	3680.5
1		0230	31	0.	54.9	3680.0	1		0520	65	0.	54.9	3680.0	1		0810	99	33.	58.0	3680.4
1		0235	32	0.	54.9	3680.0	1		0525	66	1.	55.0	3680.0	1		0815	100	31.	57.8	3680.4
1		0240	33	0.	54.9	3680.0	1		0530	67	3.	55.2	3680.0							
1		0245	34	0.	54.9	3680.0	1		0535	68	7.	55.6	3680.1							

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	8.25-HR
+ (CFS)	(HR)					
		(CFS)				
+ 142.	6.33		34.	25.	25.	25.
		(INCHES)	.353	.353	.353	.353
		(AC-FT)	17.	17.	17.	17.

PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
		6-HR	24-HR	72-HR	8.25-HR
+ (AC-FT)	(HR)				
67.	6.33	58.	57.	57.	57.

```
 PEAK STAGE    TIME                          MAXIMUM AVERAGE STAGE
                                    6-HR      24-HR      72-HR     8.25-HR
+  (FEET)      (HR)
  3681.67      6.33              3680.44    3680.32    3680.32    3680.32

                     CUMULATIVE AREA =      .90 SQ MI



*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
  30 KK   *        2   *     INCREMENTAL FLOW
          *            *
          **************

             SUBBASIN RUNOFF DATA

  31 BA        SUBBASIN CHARACTERISTICS
                    TAREA        .20  SUBBASIN AREA

               PRECIPITATION DATA

  32 PB             STORM       2.20  BASIN TOTAL PRECIPITATION

  33 PI          INCREMENTAL PRECIPITATION PATTERN
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .02      .02      .02      .02      .02      .02      .02      .02      .02      .02
                    .04      .06      .07      .09      .13      .25      .37      .16      .12      .07
                    .06      .04

  43 LS        SCS LOSS RATE
                    STRTL        .74  INITIAL ABSTRACTION
                   CRVNBR      73.00  CURVE NUMBER
                    RTIMP        .00  PERCENT IMPERVIOUS AREA

  44 UD        SCS DIMENSIONLESS UNITGRAPH
                     TLAG        .25  LAG

                                                                   ***

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

                                                          UNIT HYDROGRAPH
                                                     17 END-OF-PERIOD ORDINATES
             59.       201.       320.       320.       249.       149.       93.       60.       37.       23.
             15.         9.         6.         4.         3.         1.        0.

*******************************************************************************************************************************

                                                    HYDROGRAPH AT STATION        2

*******************************************************************************************************************************
                                                                   *
      DA MON HRMN  ORD    RAIN    LOSS  EXCESS     COMP Q          *    DA MON HRMN  ORD    RAIN    LOSS  EXCESS     COMP Q
                                                                   *
       1     0000    1     .00     .00     .00         0.          *     1     0410   51     .01     .01     .00         0.
       1     0005    2     .01     .01     .00         0.          *     1     0415   52     .02     .02     .00         0.
       1     0010    3     .01     .01     .00         0.          *     1     0420   53     .02     .02     .00         0.
```

1	0015	4	.01	.01	.00	0.	*	1	0425	54	.02	.02	.00	0.
1	0020	5	.01	.01	.00	0.	*	1	0430	55	.02	.02	.00	0.
1	0025	6	.01	.01	.00	0.	*	1	0435	56	.02	.02	.00	0.
1	0030	7	.01	.01	.00	0.	*	1	0440	57	.02	.02	.00	0.
1	0035	8	.01	.01	.00	0.	*	1	0445	58	.02	.02	.00	0.
1	0040	9	.01	.01	.00	0.	*	1	0450	59	.02	.02	.00	0.
1	0045	10	.01	.01	.00	0.	*	1	0455	60	.02	.02	.00	0.
1	0050	11	.01	.01	.00	0.	*	1	0500	61	.02	.02	.00	0.
1	0055	12	.01	.01	.00	0.	*	1	0505	62	.04	.04	.00	0.
1	0100	13	.01	.01	.00	0.	*	1	0510	63	.06	.06	.00	0.
1	0105	14	.01	.01	.00	0.	*	1	0515	64	.07	.07	.01	1.
1	0110	15	.01	.01	.00	0.	*	1	0520	65	.09	.08	.01	2.
1	0115	16	.01	.01	.00	0.	*	1	0525	66	.13	.11	.02	6.
1	0120	17	.01	.01	.00	0.	*	1	0530	67	.25	.19	.06	13.
1	0125	18	.01	.01	.00	0.	*	1	0535	68	.37	.24	.12	30.
1	0130	19	.01	.01	.00	0.	*	1	0540	69	.16	.10	.06	56.
1	0135	20	.01	.01	.00	0.	*	1	0545	70	.12	.07	.05	80.
1	0140	21	.01	.01	.00	0.	*	1	0550	71	.07	.04	.03	90.
1	0145	22	.01	.01	.00	0.	*	1	0555	72	.06	.03	.03	87.
1	0150	23	.01	.01	.00	0.	*	1	0600	73	.04	.02	.02	75.
1	0155	24	.01	.01	.00	0.	*	1	0605	74	.00	.00	.00	62.
1	0200	25	.01	.01	.00	0.	*	1	0610	75	.00	.00	.00	48.
1	0205	26	.01	.01	.00	0.	*	1	0615	76	.00	.00	.00	33.
1	0210	27	.01	.01	.00	0.	*	1	0620	77	.00	.00	.00	22.
1	0215	28	.01	.01	.00	0.	*	1	0625	78	.00	.00	.00	14.
1	0220	29	.01	.01	.00	0.	*	1	0630	79	.00	.00	.00	9.
1	0225	30	.01	.01	.00	0.	*	1	0635	80	.00	.00	.00	5.
1	0230	31	.01	.01	.00	0.	*	1	0640	81	.00	.00	.00	3.
1	0235	32	.01	.01	.00	0.	*	1	0645	82	.00	.00	.00	2.
1	0240	33	.01	.01	.00	0.	*	1	0650	83	.00	.00	.00	1.
1	0245	34	.01	.01	.00	0.	*	1	0655	84	.00	.00	.00	1.
1	0250	35	.01	.01	.00	0.	*	1	0700	85	.00	.00	.00	0.
1	0255	36	.01	.01	.00	0.	*	1	0705	86	.00	.00	.00	0.
1	0300	37	.01	.01	.00	0.	*	1	0710	87	.00	.00	.00	0.
1	0305	38	.01	.01	.00	0.	*	1	0715	88	.00	.00	.00	0.
1	0310	39	.01	.01	.00	0.	*	1	0720	89	.00	.00	.00	0.
1	0315	40	.01	.01	.00	0.	*	1	0725	90	.00	.00	.00	0.
1	0320	41	.01	.01	.00	0.	*	1	0730	91	.00	.00	.00	0.
1	0325	42	.01	.01	.00	0.	*	1	0735	92	.00	.00	.00	0.
1	0330	43	.01	.01	.00	0.	*	1	0740	93	.00	.00	.00	0.
1	0335	44	.01	.01	.00	0.	*	1	0745	94	.00	.00	.00	0.
1	0340	45	.01	.01	.00	0.	*	1	0750	95	.00	.00	.00	0.
1	0345	46	.01	.01	.00	0.	*	1	0755	96	.00	.00	.00	0.
1	0350	47	.01	.01	.00	0.	*	1	0800	97	.00	.00	.00	0.
1	0355	48	.01	.01	.00	0.	*	1	0805	98	.00	.00	.00	0.
1	0400	49	.01	.01	.00	0.	*	1	0810	99	.00	.00	.00	0.
1	0405	50	.01	.01	.00	0.	*	1	0815	100	.00	.00	.00	0.
							*							

TOTAL RAINFALL = 2.20, TOTAL LOSS = 1.79, TOTAL EXCESS = .41

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	8.25-HR
+ (CFS)	(HR)					
		(CFS)				
+ 90.	5.83		9.	6.	6.	6.
		(INCHES)	.413	.413	.413	.413
		(AC-FT)	4.	4.	4.	4.

CUMULATIVE AREA = .20 SQ MI

```
          **************
          *            *
 45 KK    *          2 *     COMBINED FLOW
          *            *
          **************

 46 HC        HYDROGRAPH COMBINATION
                  ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE
```

***

HYDROGRAPH AT STATION 2
SUM OF 2 HYDROGRAPHS

DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW
1		0000	1	0.	1		0205	26	0.	1		0410	51	0.	1		0615	76	170.
1		0005	2	0.	1		0210	27	0.	1		0415	52	0.	1		0620	77	164.
1		0010	3	0.	1		0215	28	0.	1		0420	53	0.	1		0625	78	154.
1		0015	4	0.	1		0220	29	0.	1		0425	54	0.	1		0630	79	142.
1		0020	5	0.	1		0225	30	0.	1		0430	55	0.	1		0635	80	130.
1		0025	6	0.	1		0230	31	0.	1		0435	56	0.	1		0640	81	117.
1		0030	7	0.	1		0235	32	0.	1		0440	57	0.	1		0645	82	110.
1		0035	8	0.	1		0240	33	0.	1		0445	58	0.	1		0650	83	103.
1		0040	9	0.	1		0245	34	0.	1		0450	59	0.	1		0655	84	96.
1		0045	10	0.	1		0250	35	0.	1		0455	60	0.	1		0700	85	90.
1		0050	11	0.	1		0255	36	0.	1		0500	61	0.	1		0705	86	84.
1		0055	12	0.	1		0300	37	0.	1		0505	62	0.	1		0710	87	78.
1		0100	13	0.	1		0305	38	0.	1		0510	63	0.	1		0715	88	73.
1		0105	14	0.	1		0310	39	0.	1		0515	64	1.	1		0720	89	68.
1		0110	15	0.	1		0315	40	0.	1		0520	65	3.	1		0725	90	63.
1		0115	16	0.	1		0320	41	0.	1		0525	66	7.	1		0730	91	59.
1		0120	17	0.	1		0325	42	0.	1		0530	67	16.	1		0735	92	55.
1		0125	18	0.	1		0330	43	0.	1		0535	68	37.	1		0740	93	51.
1		0130	19	0.	1		0335	44	0.	1		0540	69	72.	1		0745	94	47.
1		0135	20	0.	1		0340	45	0.	1		0545	70	109.	1		0750	95	44.
1		0140	21	0.	1		0345	46	0.	1		0550	71	138.	1		0755	96	41.
1		0145	22	0.	1		0350	47	0.	1		0555	72	156.	1		0800	97	38.
1		0150	23	0.	1		0355	48	0.	1		0600	73	164.	1		0805	98	35.
1		0155	24	0.	1		0400	49	0.	1		0605	74	168.	1		0810	99	33.
1		0200	25	0.	1		0405	50	0.	1		0610	75	170.	1		0815	100	31.

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	8.25-HR
+ (CFS)	(HR)					
		(CFS)				
+ 170.	6.25		43.	31.	31.	31.
		(INCHES)	.364	.364	.364	.364
		(AC-FT)	21.	21.	21.	21.

CUMULATIVE AREA = 1.10 SQ MI

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

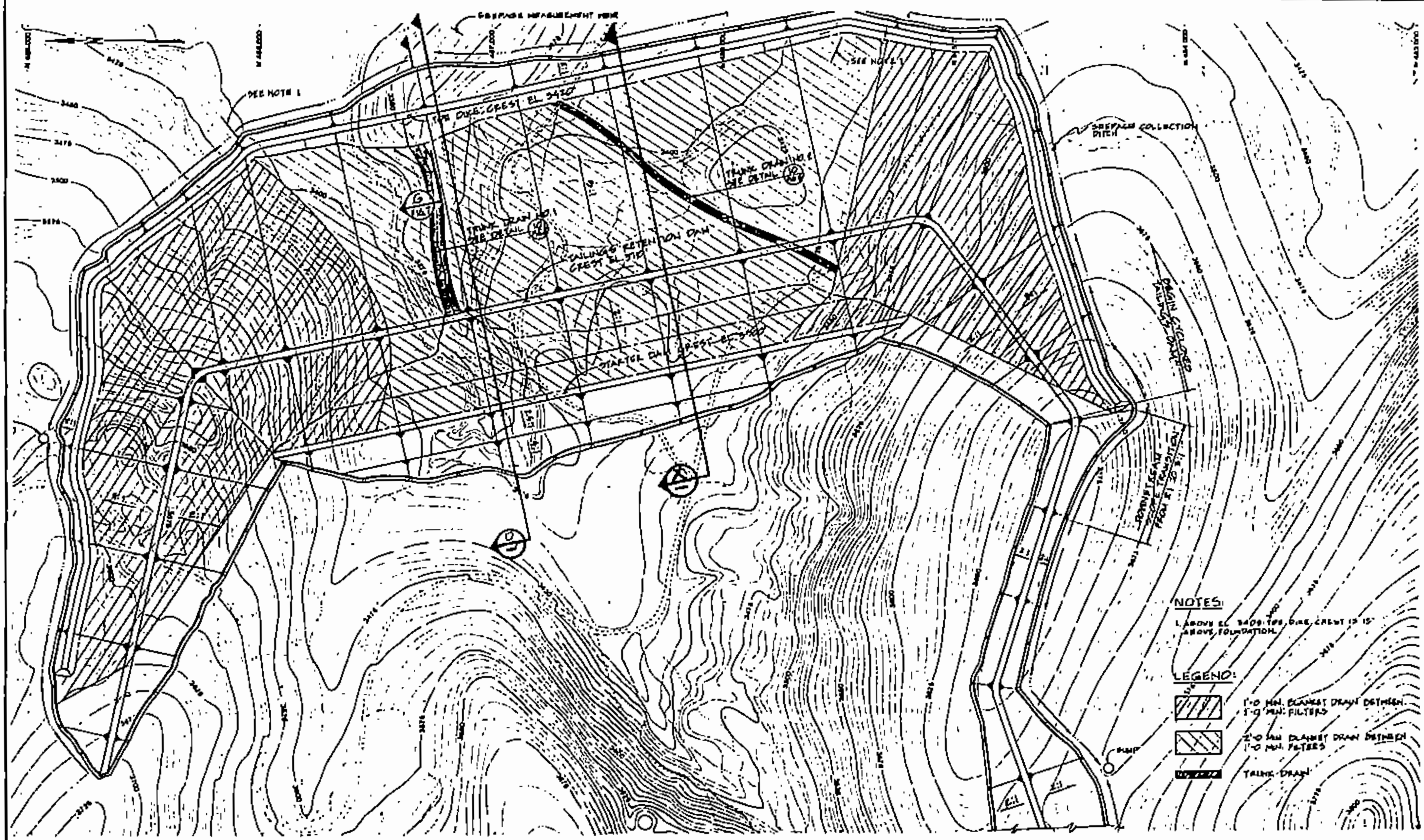
PEAK TIME OF AVERAGE FLOW FOR MAXIMUM PERIOD BASIN MAXIMUM TIME OF

OPERATION	STATION	FLOW	PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	STAGE	MAX STAGE
+									
HYDROGRAPH AT									
+	1	360.	5.92	40.	29.	29.	.90		
ROUTED TO									
+	1	142.	6.33	34.	25.	25.	.90		
+								3681.67	6.33
HYDROGRAPH AT									
+	2	90.	5.83	9.	6.	6.	.20		
2 COMBINED AT									
+	2	170.	6.25	43.	31.	31.	1.10		

1 HEC-1 INPUT

LINE ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

APPENDIX C
STABILITY ANALYSES
(To Be Completed)



NO	DATE	REVISIONS	BY	CHK	APP'D

MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED	DRAWN	CHECKED	RECOMMENDED
DATE		APPROVED	

NORANDA MINERALS CORPORATION

MONTANORE PROJECT
SLOPE STABILITY ANALYSIS
LOCATION OF SECTIONS

SHEET OF | REV

FIGURE C-1

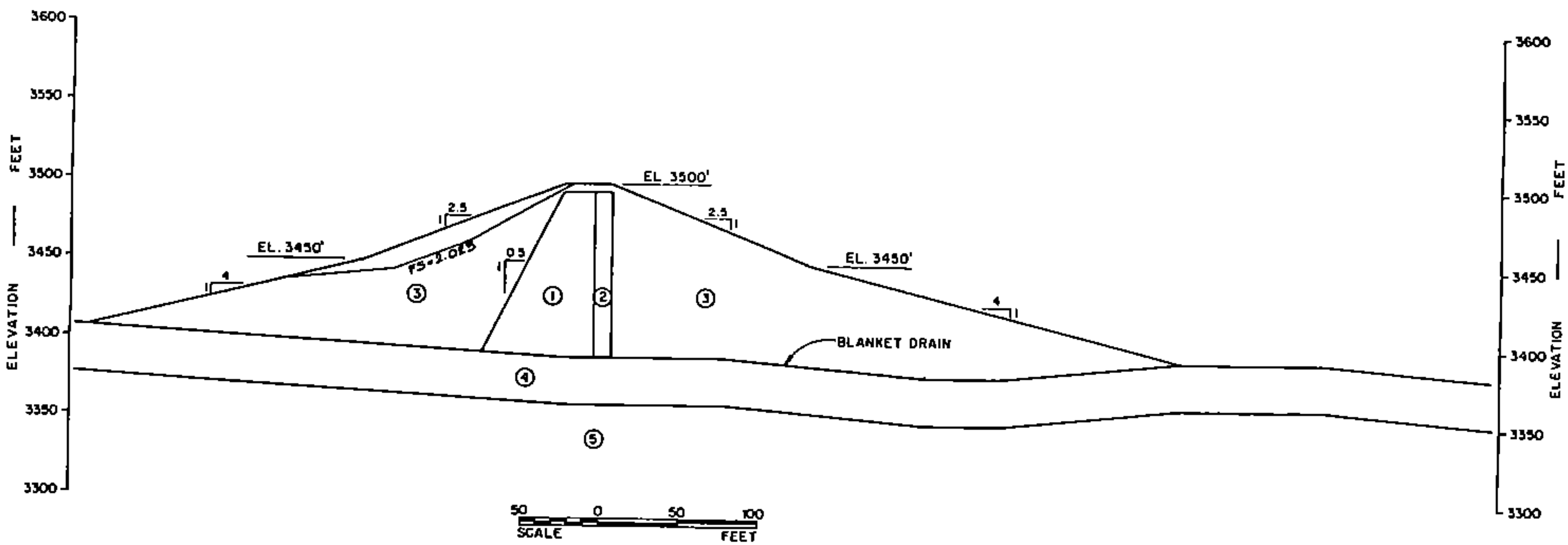


FIGURE C-3

CASE IA
STARTER DAM-UPSTREAM SLOPE
SECTION D
END OF CONSTRUCTION

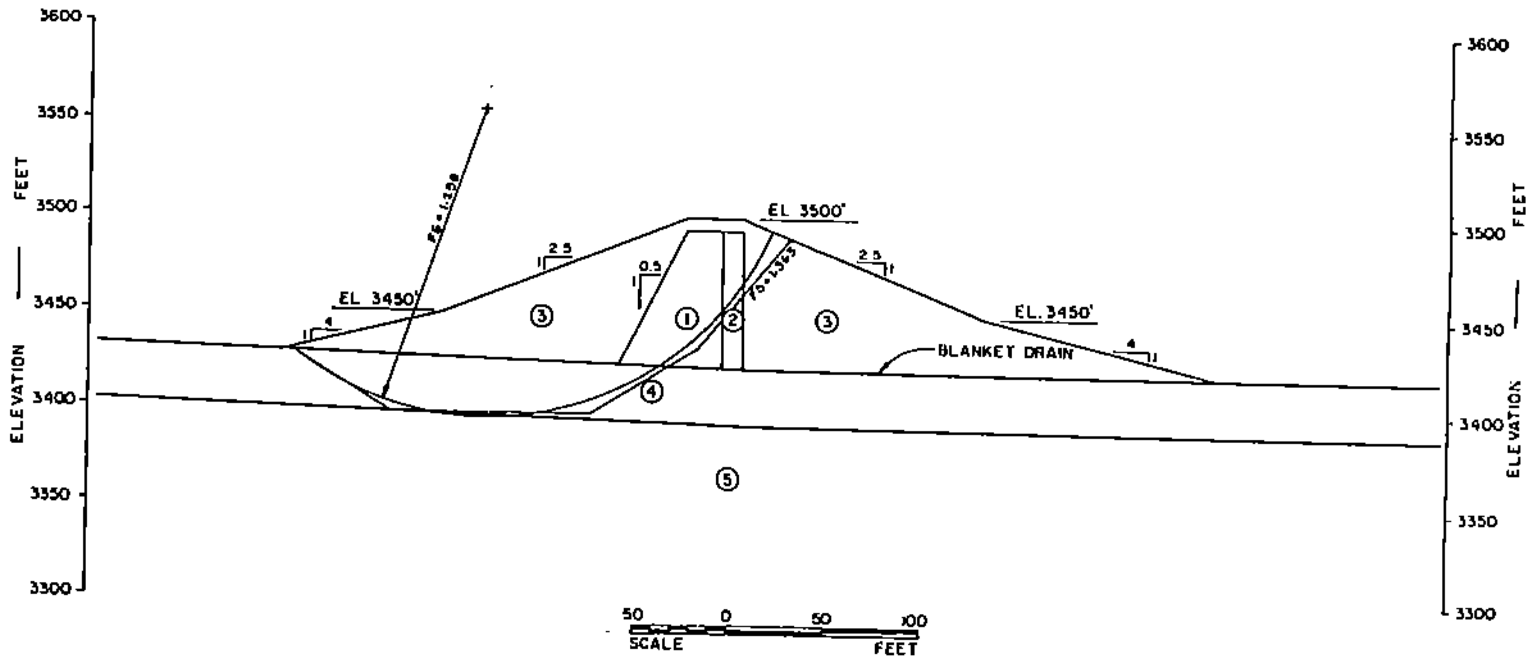


FIGURE C-4

CASE IA
STARTER DAM-UPSTREAM SLOPE
SECTION X
END OF CONSTRUCTION

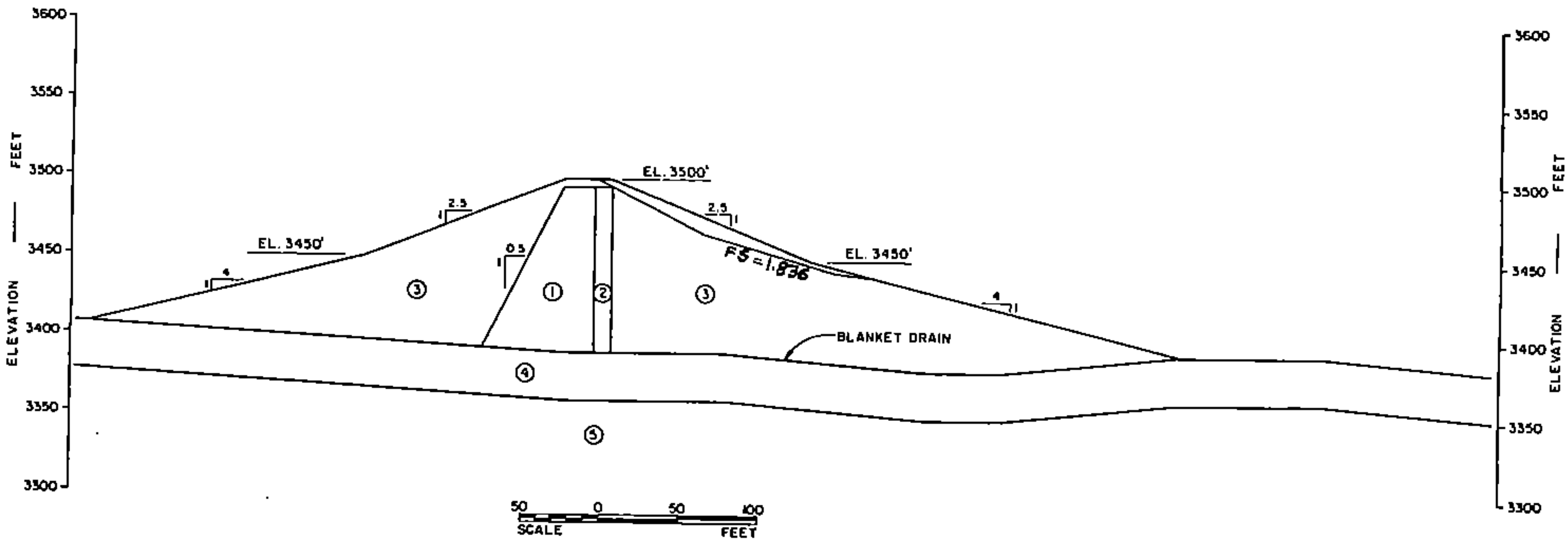


FIGURE C-5

CASE IB
STARTER DAM-DOWNSTREAM SLOPE
SECTION D
END OF CONSTRUCTION

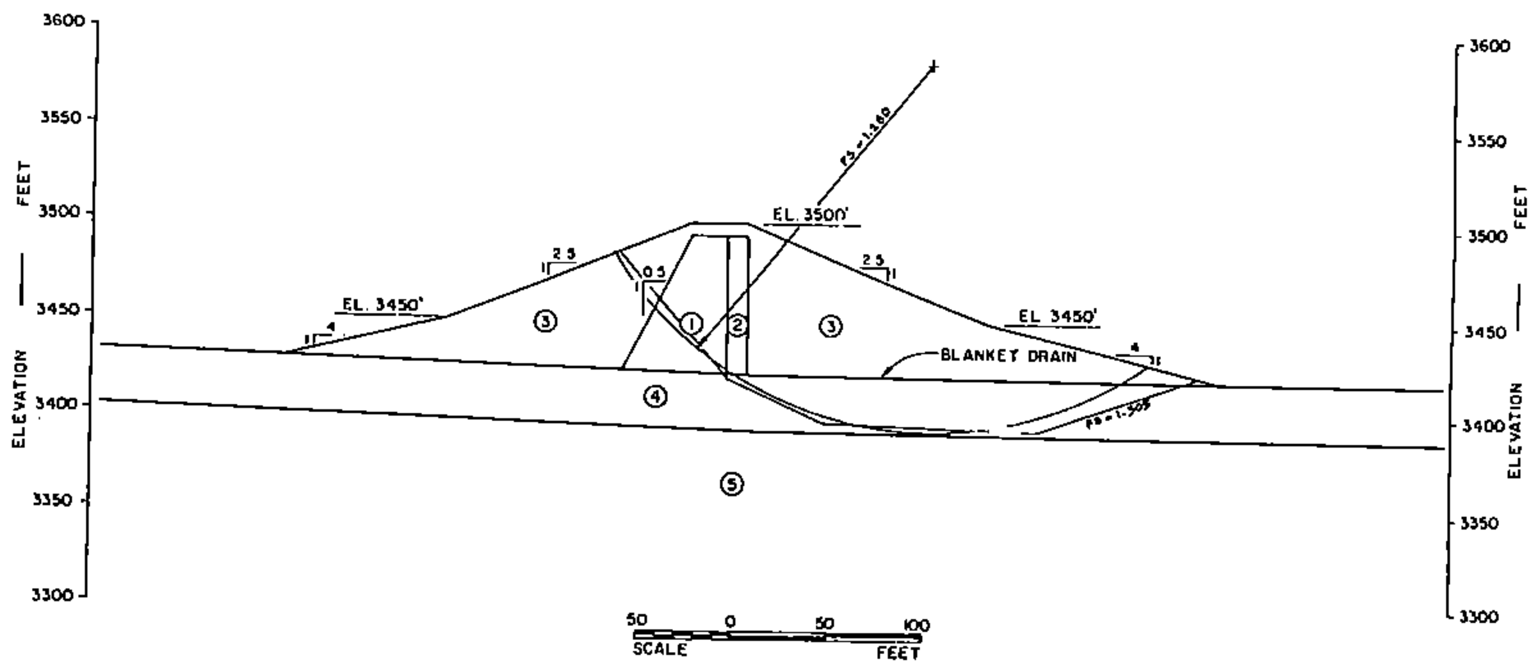


FIGURE C-6

CASE IB
STARTER DAM-DOWNSTREAM SLOPE
SECTION X
END OF CONSTRUCTION

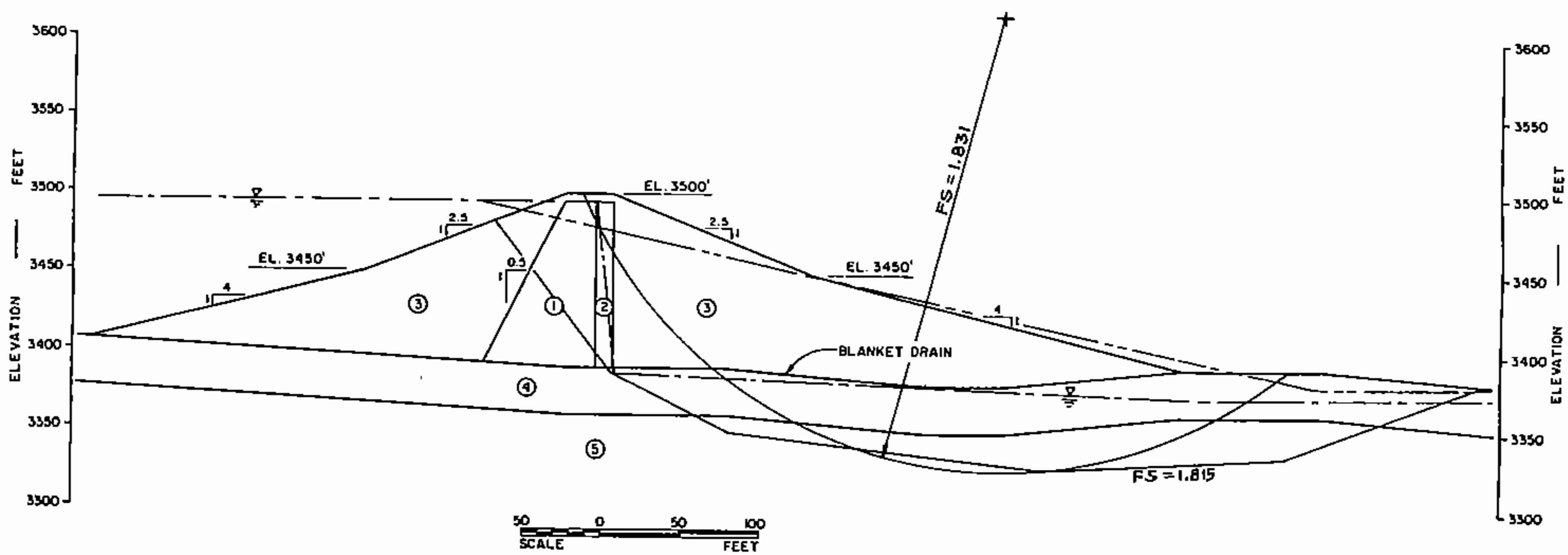


FIGURE C-7

CASE 2A

STARTER DAM-DOWNSTREAM SLOPE

SECTION D

STEADY STATE SEEPAGE

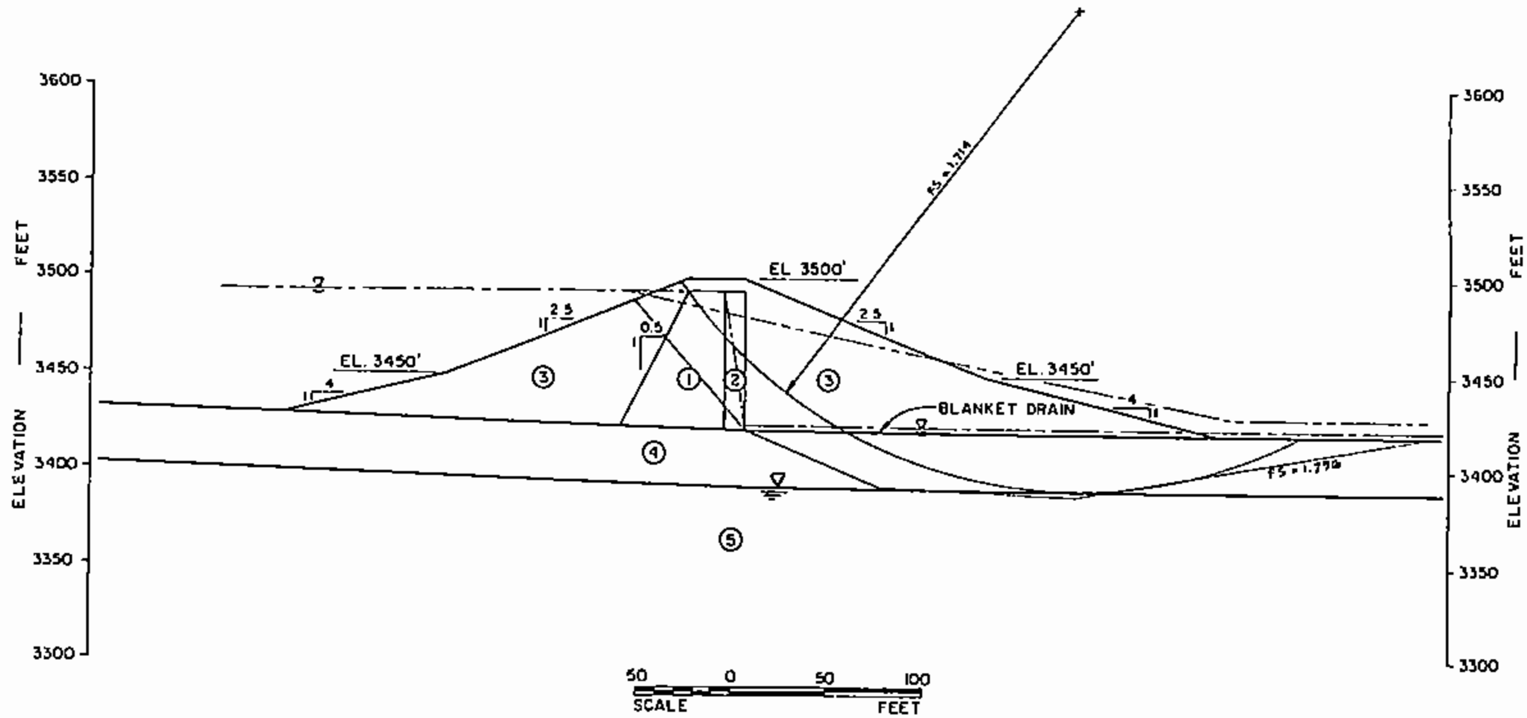


FIGURE C-8

CASE 2A

STARTER DAM-DOWNSTREAM SLOPE

SECTION X

STEADY STATE SEEPAGE

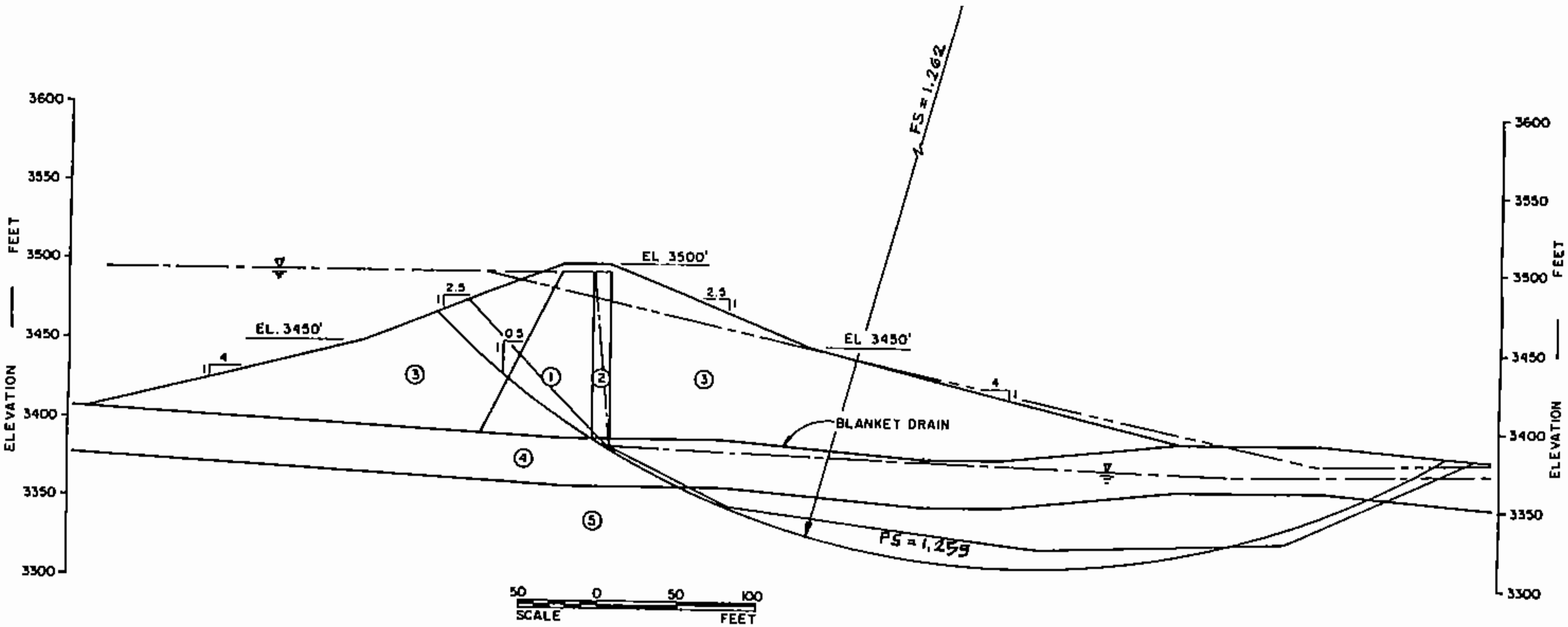


FIGURE C-13

CASE 4A
STARTER DAM-DOWNSTREAM SLOPE
SECTION D
SEISMIC LOADING (K=0.10)

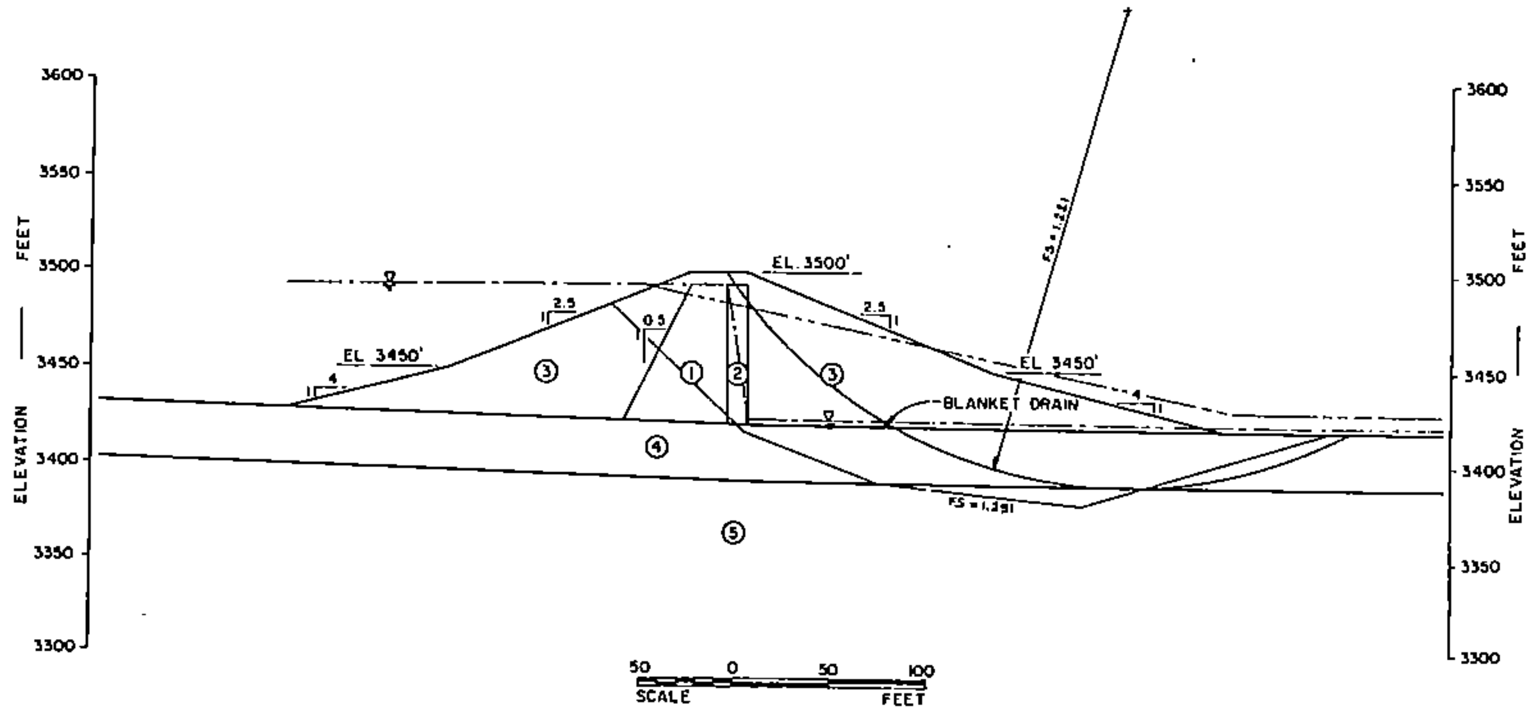


FIGURE C-14

CASE 4A
STARTER DAM-DOWNSTREAM SLOPE
SECTION X
SEISMIC LOADING (K=0.10)

APPENDIX D
CYCLONE ANALYSES

KREBS ENGINEERS | 1205 CHRYSLER DRIVE MENLO PARK, CALIFORNIA 94025-9926 TEL: (415) 325-0751 TELEX: WUI 348403 or RCA 278769 FAX: (415) 326-7048

FAX

TO: COMPANY: Morrison-Knudson
FAX NO: 442-7673
ATTENTION: Mr. Vince Pascucci

FROM: KREBS ENGINEERS - MENLO PARK, CA USA
FAX NO: (415) 326-7048
SENDER: Mr. Patrick Turner DATE: May 31, 1990

Total pages including this page: 10

Reference: Tailings Project For Noranda

Dear Vince:

Attached are mass balance sheets for 3 two stage cyclone systems. The only difference between the three sets of balances is the new feed tonnage.

The overflow from the second stage of cyclones is recirculated back to the primary cyclones. The primary cyclone overflow is discharged and the underflow is diluted and fed to the secondary cyclones. The secondary cyclone underflow is the product.

In all three cases the secondary cyclone underflow is 91 - 92% plus 200 mesh. Please note that in all three cases the -400 mesh in the product is less than 1%. In our experience, we have found that the percolation rate is more directly related to the amount of -400 mesh rather than the amount of -200 mesh. In other words the -200 mesh specification could be relaxed while maintaining a strict -400 mesh specification. This would allow us to make a finer cut and increase the recovery.

For both stages we are recommending our D26B cyclones. This cyclone has been used for this application at many mines around the world. We could look at using smaller cyclones to increase recovery if the -200 mesh specification can be relaxed.

As we discussed on the phone, I believe the feed distribution is too heavily weighted in the 100 x 270 mesh fraction. Listed below are cyclone overflow products from 3 different mines. These distributions have been plotted on the attached log/log paper. From looking at many sets of data, Krebs Engineers has come up with a standard distribution curve on the log/log paper. This allows us to come up with a complete distribution even if we have only one data point. Typically we work off of the P80 specification. This is the micron size where 80% of the cyclone overflow is finer than that size.

KREBS CYCLONE PROBLEM ANALYSIS

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7048

SHEET STAGE 1
DATE 31-May-90
BY Jon N

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow

First Stage, Year 1. (Feed includes secondary cyclone overflow.)

NUMBER, MODEL KREBS CYCLONE 5 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.0 in.	15 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	565.0	278.1	286.9
STPH LIQUID	2192.5	2069.6	122.9
STPH PULP	2757.5	2347.7	409.8
% SOLIDS WT	20.5	11.8	70.0
S.G. PULP	1.144	1.079	1.757
% SOLIDS VOL	9.0	4.9	47.3
U.S. GPM PULP	9627	8695	932

REFERENCE: 53 4.0 64

MICRON	FEED CUM % +	FEED IND % +	FEED STPH	OVERFLOW CUM % +	OVERFLOW IND % +	OVERFLOW STPH	UNDERFLOW CUM % +	UNDERFLOW IND % +	UNDERFLOW STPH	RECOVERY
300	0.1	0.1	0.5	0.0	0.0	0.0	0.2	0.2	0.5	100.0
275	0.2	0.1	0.5	0.0	0.0	0.0	0.3	0.2	0.5	100.0
250	0.3	0.1	0.5	0.0	0.0	0.0	0.5	0.2	0.5	100.0
212	0.4	0.2	1.0	0.0	0.0	0.0	0.8	0.3	1.0	100.0
150	8.4	8.0	45.0	0.1	0.1	0.2	16.4	15.6	44.8	99.6
106	27.9	19.5	110.4	2.6	2.6	7.1	52.4	36.0	103.2	93.5
75	48.0	20.1	113.7	15.7	13.0	36.2	79.4	27.0	77.5	68.1
53	66.3	18.2	103.1	39.3	23.6	65.6	92.5	13.1	37.5	36.4
37	77.2	10.9	61.8	57.3	18.0	50.1	96.6	4.1	11.8	19.0
-37	100.0	22.8	128.6	100.0	42.7	118.9	100.0	3.4	9.7	7.6
			565.0			278.1			286.9	50.8

KREBS CYCLONE PROBLEM ANALYSIS

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7048

SHEET STAGE 2
DATE 31-May-90
BY Jon N

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow
2nd Stage, Year 1

NUMBER, MODEL KREBS CYCLONE 3 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.5 in.	11 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	286.9	86.0	200.9
STPH LIQUID	1097.9	1019.8	78.1
STPH PULP	1384.8	1105.8	279.0
% SOLIDS WT	20.7	7.8	72.0
S.G. PULP	1.146	1.050	1.798
% SOLIDS VOL	9.1	3.1	49.7
U.S. GPM PULP	4827	4206	621

REFERENCE 53 4.0 70

MICRON	FEED			OVERFLOW			UNDERFLOW			RECOVERY
	CUM %+	IND %+	STPH	CUM %+	IND %+	STPH	CUM %+	IND %+	STPH	
300	0.2	0.2	0.5	0.0	0.0	0.0	0.2	0.2	0.5	100.0
275	0.3	0.2	0.5	0.0	0.0	0.0	0.5	0.2	0.5	100.0
250	0.5	0.2	0.5	0.0	0.0	0.0	0.7	0.2	0.5	100.0
212	0.8	0.3	1.0	0.0	0.0	0.0	1.2	0.5	1.0	100.0
150	16.4	15.6	44.8	0.5	0.5	0.4	23.3	22.1	44.3	99.1
106	52.4	36.0	103.2	13.0	12.5	10.7	69.3	46.1	92.5	89.6
75	79.4	27.0	77.5	48.8	35.9	30.8	92.5	23.2	46.6	60.2
53	92.5	13.1	37.5	78.5	29.7	25.5	98.5	6.0	12.0	32.0
37	96.6	4.1	11.8	89.7	11.2	9.6	99.6	1.1	2.1	18.1
-37	100.0	3.4	9.7	100.0	10.3	8.9	100.0	0.4	0.9	8.9
			286.9			86.0			200.9	70.0

KREBS CYCLONE PROBLEM ANALYSIS

SHEET STAGE 1
DATE 31-May-90
BY Jon N

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7048

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow

First Stage. Year 2. (Feed includes secondary cyclone overflow.)

NUMBER, MODEL KREBS CYCLONE 7 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.0 in.	16 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	810.3	395.1	415.2
STPH LIQUID	3271.1	3093.2	178.0
STPH PULP	4081.5	3488.3	593.2
% SOLIDS WT	19.9	11.3	70.0
S.G. PULP	1.139	1.075	1.757
% SOLIDS VOL	8.7	4.7	47.3
U.S. GPM PULP	14313	12964	1349

REFERENCE: 53 4.0 63

MICRON	FEED CUM % +	FEED IND % +	FEED STPH	OVERFLOW CUM % +	OVERFLOW IND % +	OVERFLOW STPH	UNDERFLOW CUM % +	UNDERFLOW IND % +	UNDERFLOW STPH	RECOVERY
300	0.1	0.1	0.7	0.0	0.0	0.0	0.2	0.2	0.7	100.0
275	0.2	0.1	0.7	0.0	0.0	0.0	0.3	0.2	0.7	100.0
250	0.3	0.1	0.7	0.0	0.0	0.0	0.5	0.2	0.7	100.0
212	0.4	0.2	1.4	0.0	0.0	0.0	0.8	0.3	1.4	100.0
150	8.5	8.1	65.4	0.1	0.1	0.2	16.5	15.7	65.1	99.6
106	27.7	19.2	156.0	2.3	2.2	8.9	51.9	35.4	147.1	94.3
75	47.2	19.5	157.8	14.2	11.9	47.2	78.6	26.6	110.6	70.1
53	65.6	18.4	149.4	37.7	23.5	92.8	92.2	13.6	56.6	37.9
37	76.8	11.2	90.5	56.1	18.4	72.6	96.5	4.3	17.8	19.7
-37	100.0	23.2	187.9	100.0	43.9	173.4	100.0	3.5	14.5	7.7
			810.3			395.1			415.2	51.2

KREBS CYCLONE PROBLEM ANALYSIS

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 326-0751 FAX: (415) 326-7046

SHEET STAGE 2
DATE 31-May-90
BY Jon N

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow
2nd Stage, Year 2.

NUMBER, MODEL KREBS CYCLONE 4 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.5 in.	14 PSI

SPECIFIC GRAVITY: SOLIDS 2.80 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	415.2	111.3	303.9
STPH LIQUID	1678.0	1559.8	118.2
STPH PULP	2093.2	1671.1	422.0
% SOLIDS WT	19.8	6.7	72.0
S.G. PULP	1.139	1.043	1.796
% SOLIDS VOL	8.7	2.7	49.7
U.S. GPM PULP	7341	6402	939

REFERENCE 53 4.0 66

MICRON	FEED			OVERFLOW			UNDERFLOW			RECOVERY
	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	
300	0.2	0.2	0.7	0.0	0.0	0.0	0.2	0.2	0.7	100.0
275	0.3	0.2	0.7	0.0	0.0	0.0	0.5	0.2	0.7	100.0
250	0.5	0.2	0.7	0.0	0.0	0.0	0.7	0.2	0.7	100.0
212	0.8	0.3	1.4	0.0	0.0	0.0	1.2	0.5	1.4	100.0
150	16.5	15.7	65.1	0.3	0.3	0.3	22.5	21.3	64.8	99.5
106	51.9	35.4	147.1	9.8	9.5	10.6	67.4	44.9	136.5	92.8
75	78.6	26.6	110.6	42.9	33.1	36.9	91.7	24.3	73.8	66.7
53	92.2	13.6	56.6	75.4	32.4	36.1	98.4	6.7	20.4	36.1
37	96.5	4.3	17.8	88.2	12.8	14.3	99.6	1.2	3.6	19.9
-37	100.0	3.5	14.5	100.0	11.8	13.2	100.0	0.4	1.3	9.3
			416.2			111.3			303.9	73.2

KREBS CYCLONE PROBLEM ANALYSIS

SHEET STAGE 1
DATE 31-May-90
BY Jon N

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7046

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow

First Stage. Years 3 to 17. (Feed includes secondary cyclone overflow.)

NUMBER, MODEL KREBS CYCLONE 9 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.0 in.	16 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	1030.0	503.1	527.0
STPH LIQUID	4152.3	3926.4	225.8
STPH PULP	5182.3	4429.5	752.8
% SOLIDS WT	19.9	11.4	70.0
S.G. PULP	1.139	1.075	1.757
% SOLIDS VOL	8.7	4.7	47.3
U.S. GPM PULP	18171	16459	1712

REFERENCE: 53 4.0 63

MICRON	FEED			OVERFLOW			UNDERFLOW			RECOVERY
	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	
300	0.1	0.1	0.9	0.0	0.0	0.0	0.2	0.2	0.9	100.0
275	0.2	0.1	0.9	0.0	0.0	0.0	0.3	0.2	0.9	100.0
250	0.3	0.1	0.9	0.0	0.0	0.0	0.5	0.2	0.9	100.0
212	0.4	0.2	1.8	0.0	0.0	0.0	0.8	0.3	1.8	100.0
150	8.5	8.1	83.1	0.1	0.1	0.3	16.6	15.7	82.8	99.6
106	27.7	19.2	198.2	2.3	2.3	11.4	52.0	35.5	186.8	94.2
75	47.2	19.5	200.4	14.3	12.0	60.2	78.6	26.6	140.2	70.0
53	65.6	18.4	189.8	37.8	23.5	118.1	92.2	13.6	71.6	37.7
37	76.8	11.2	115.0	56.2	18.4	92.4	96.5	4.3	22.6	19.6
-37	100.0	23.2	239.0	100.0	43.8	220.6	100.0	3.5	18.5	7.7
			1030.0			503.1			527.0	51.2

KREBS CYCLONE PROBLEM ANALYSIS

SHEET STAGE 2
DATE 31-May-90
BY Jon N

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7048

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow
2nd Stage, ~~Years~~ 3 to 17.

NUMBER, MODEL KREBS CYCLONE 5 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.5 in.	14 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	527.0	141.0	385.9
STPH LIQUID	2125.8	1975.8	150.1
STPH PULP	2652.8	2116.8	536.0
% SOLIDS WT	19.9	6.7	72.0
S.G. PULP	1.139	1.043	1.796
% SOLIDS VOL	8.7	2.7	49.7
U.S. GPM PULP	9302	8110	1193

REFERENCE 53 4.0 66

MICRON	FEED CUM % +	FEED IND % +	FEED STPH	OVERFLOW CUM % +	OVERFLOW IND % +	OVERFLOW STPH	UNDERFLOW CUM % +	UNDERFLOW IND % +	UNDERFLOW STPH	RECOVERY
300	0.2	0.2	0.9	0.0	0.0	0.0	0.2	0.2	0.9	100.0
275	0.3	0.2	0.9	0.0	0.0	0.0	0.5	0.2	0.9	100.0
250	0.5	0.2	0.9	0.0	0.0	0.0	0.7	0.2	0.9	100.0
212	0.8	0.3	1.8	0.0	0.0	0.0	1.2	0.5	1.8	100.0
150	16.6	15.7	82.8	0.3	0.3	0.4	22.5	21.3	82.4	99.5
106	52.0	35.5	186.8	9.8	9.5	13.3	67.5	45.0	173.5	92.9
75	78.6	26.6	140.2	42.8	33.0	46.6	91.7	24.3	93.6	66.8
53	92.2	13.6	71.6	75.3	32.5	45.8	98.4	6.7	25.9	36.1
37	96.5	4.3	22.6	88.1	12.8	18.1	99.6	1.2	4.5	19.8
-37	100.0	3.5	18.5	100.0	11.9	16.8	100.0	0.4	1.7	9.1
			527.0			141.0			385.9	73.2

KREBS CYCLONE PROBLEM ANALYSIS

1205 Chrysler Drive, Menlo Park, CA 94025-9925
TEL: (415) 325-0751 FAX: (415) 326-7048

SHEET STAGE 1
DATE 01-Jun-90
BY Jon N

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow
First Stage, Year 18. (Feed includes secondary cyclone overflow.)

NUMBER, MODEL KREBS CYCLONE 7 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.0 in.	17 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	844.8	406.8	438.1
STPH LIQUID	3395.2	3207.4	187.7
STPH PULP	4240.0	3614.2	625.8
% SOLIDS WT	19.9	11.3	70.0
S.G. PULP	1.140	1.074	1.757
% SOLIDS VOL	8.7	4.7	47.3
U.S. GPM PULP	14862	13439	1423

REFERENCE: 53 4.0 62

MICRON	FEED			OVERFLOW			UNDERFLOW			RECOVERY
	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	CUM % +	IND % +	STPH	
300	0.1	0.1	0.7	0.0	0.0	0.0	0.2	0.2	0.7	100.0
275	0.2	0.1	0.7	0.0	0.0	0.0	0.3	0.2	0.7	100.0
250	0.3	0.1	0.7	0.0	0.0	0.0	0.5	0.2	0.7	100.0
212	0.4	0.2	1.5	0.0	0.0	0.0	0.8	0.3	1.5	100.0
150	8.5	8.1	68.0	0.1	0.1	0.2	16.3	15.5	67.8	99.7
106	27.7	19.2	161.9	2.1	2.1	8.4	51.4	35.1	153.6	94.8
75	47.1	19.4	164.2	13.6	11.5	46.6	78.2	26.8	117.6	71.6
53	65.6	18.5	156.6	37.0	23.5	95.5	92.2	14.0	61.1	39.0
37	76.8	11.2	94.6	55.6	18.6	75.5	96.5	4.4	19.1	20.2
-37	100.0	23.2	195.8	100.0	44.4	180.6	100.0	3.5	15.2	7.8
			844.8			406.8			438.1	51.9

KREBS CYCLONE PROBLEM ANALYSIS

1205 Chrysler Drive, Menlo Park, CA 94025-9928
TEL: (415) 325-0751 FAX: (415) 326-7048

SHEET STAGE 2
DATE 01-Jun-90
BY Jon N

CLIENT Morrison-Knudson Engineers, Inc.

PROBLEM Recover +200 mesh tailings to the cyclone underflow

Second Stage, Year 18.

NUMBER, MODEL KREBS CYCLONE 5 Operating Krebs D26B Cyclone

	INLET	VORTEX FINDER	APEX	PRESSURE
ORIFICES:	45.00 sq.in.	10.00 in.	3.5 in.	14.5 PSI

SPECIFIC GRAVITY: SOLIDS 2.60 LIQUID 1.00

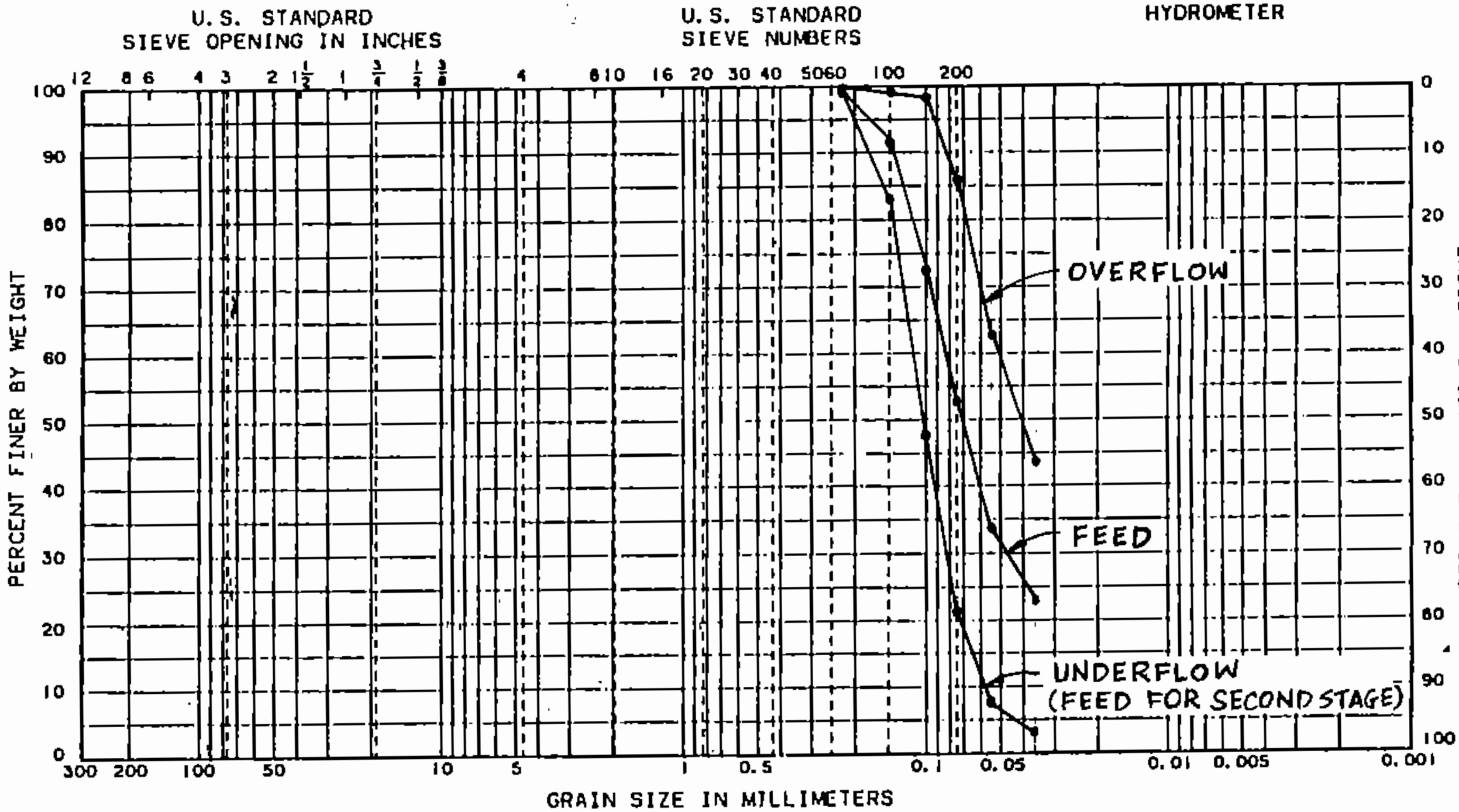
	FEED	OVERFLOW	UNDERFLOW
STPH SOLIDS	438.1	116.8	321.2
STPH LIQUID	1737.7	1612.8	124.9
STPH PULP	2175.8	1729.6	446.2
% SOLIDS WT	20.1	6.8	72.0
S.G. PULP	1.141	1.043	1.796
% SOLIDS VOL	8.8	2.7	49.7
U.S. GPM PULP	7615	6623	993

REFERENCE 53 4.0 65

MICRON	FEED CUM % +	FEED IND % +	FEED STPH	OVERFLOW CUM % +	OVERFLOW IND % +	OVERFLOW STPH	UNDERFLOW CUM % +	UNDERFLOW IND % +	UNDERFLOW STPH	RECOVERY
300	0.2	0.2	0.7	0.0	0.0	0.0	0.2	0.2	0.7	100.0
275	0.3	0.2	0.7	0.0	0.0	0.0	0.5	0.2	0.7	100.0
250	0.5	0.2	0.7	0.0	0.0	0.0	0.7	0.2	0.7	100.0
212	0.8	0.3	1.5	0.0	0.0	0.0	1.1	0.5	1.5	100.0
150	16.3	15.5	67.8	0.3	0.3	0.3	22.1	21.0	67.5	99.5
106	51.4	35.1	153.6	9.3	9.0	10.5	66.7	44.5	143.1	93.2
75	78.2	26.8	117.6	42.0	32.7	38.2	91.4	24.7	79.3	67.5
53	92.2	14.0	61.1	75.1	33.1	38.7	98.4	7.0	22.4	36.7
37	96.5	4.4	19.1	88.2	13.0	15.2	99.6	1.2	3.9	20.2
-37	100.0	3.5	15.2	100.0	11.8	13.8	100.0	0.4	1.4	9.3
			438.1			116.8			321.2	73.3

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY

GRAIN SIZE ANALYSIS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

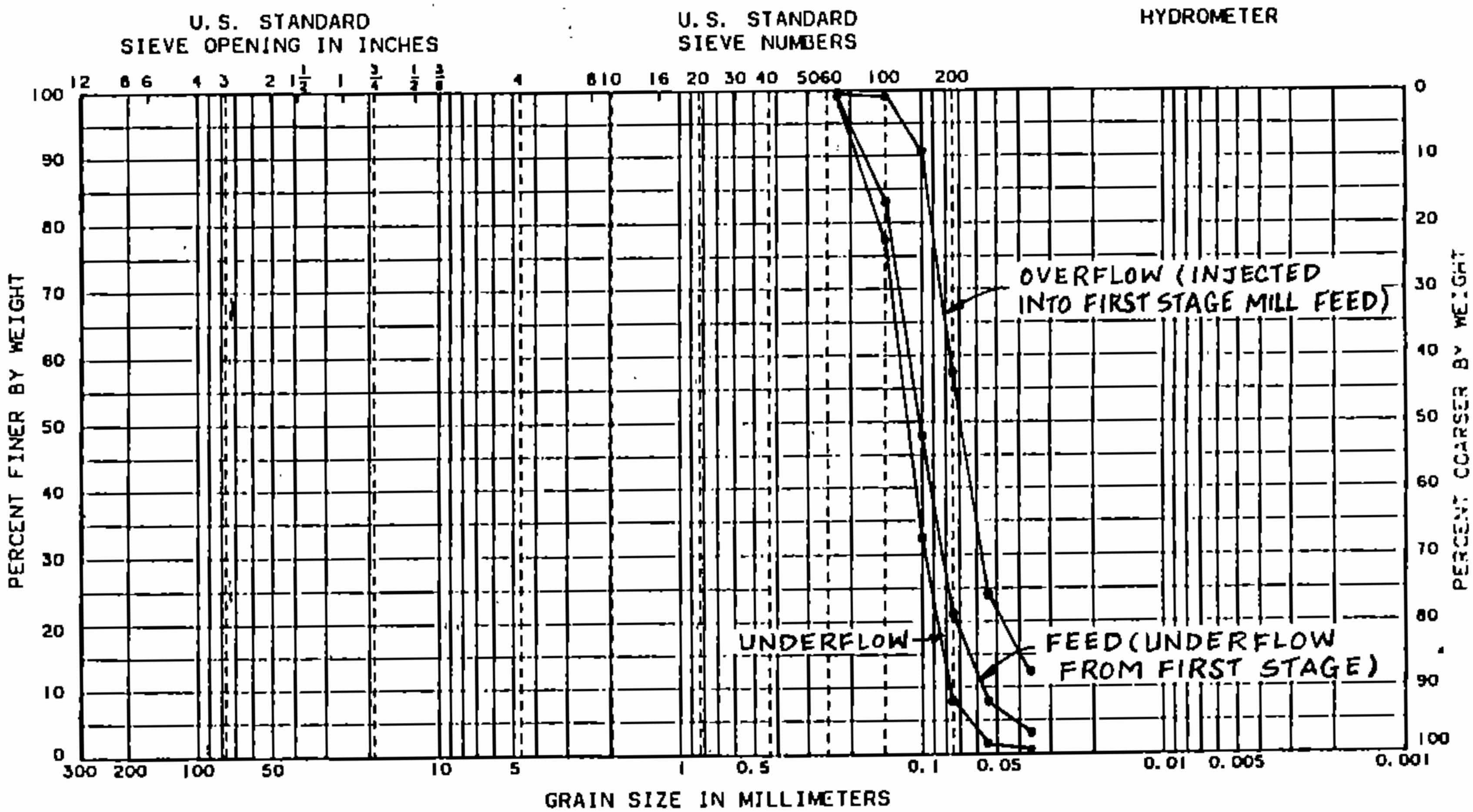
SAMPLE NO.	ELEV. OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

PROJECT MONTANORE
JOB NO. 8029-02
AREA FIRST STAGE
HOLE NO. YEARS 3-17
DATE JULY 1990
FIGURE D-1

CAB110

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY

GRAIN SIZE ANALYSIS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV. OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

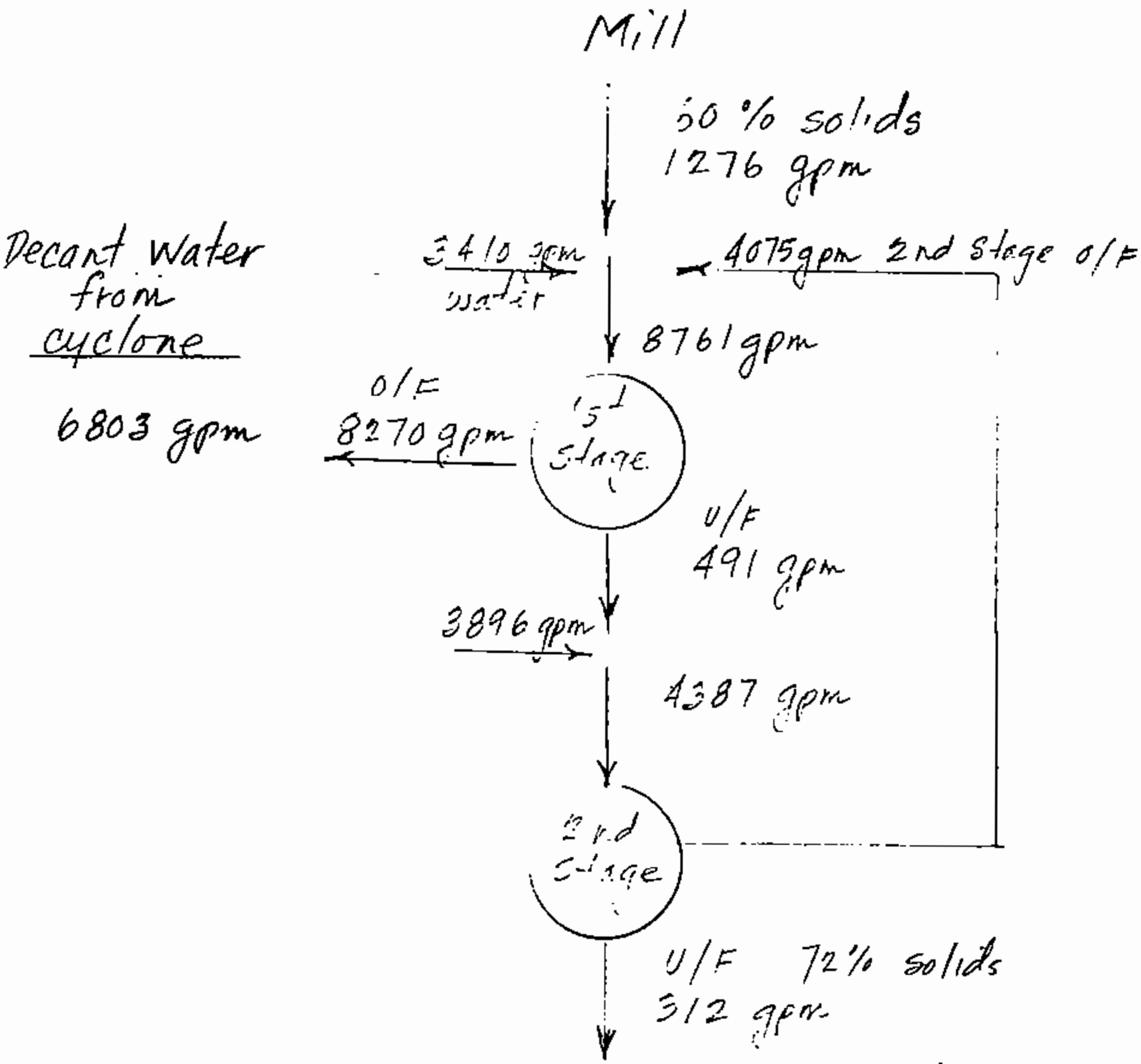
PROJECT MONTANORE
JOB NO. 8029-02
AREA SECOND STAGE
HOLE NO. YEARS 3-17
DATE JULY 1990

FIGURE D-2

CAB110

CYCLONE WATER BALANCE

Year 1 - Alternative 9

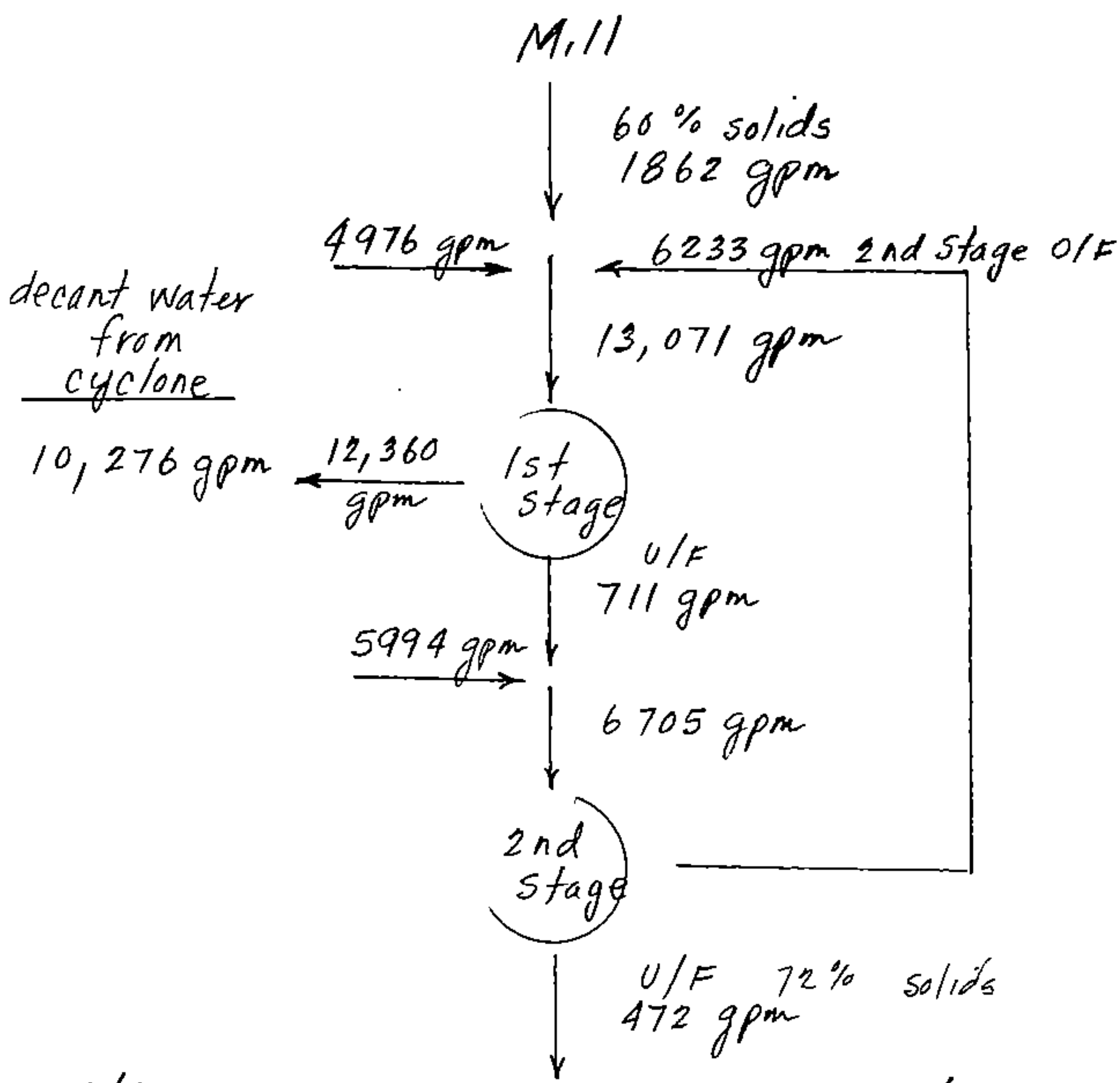


O/F available	req'd	U/F available
6803 gpm	7306 gpm	0 gpm

additional water needed 503 gpm

CYCLONE WATER BALANCE

Year 2 - Alternative 9

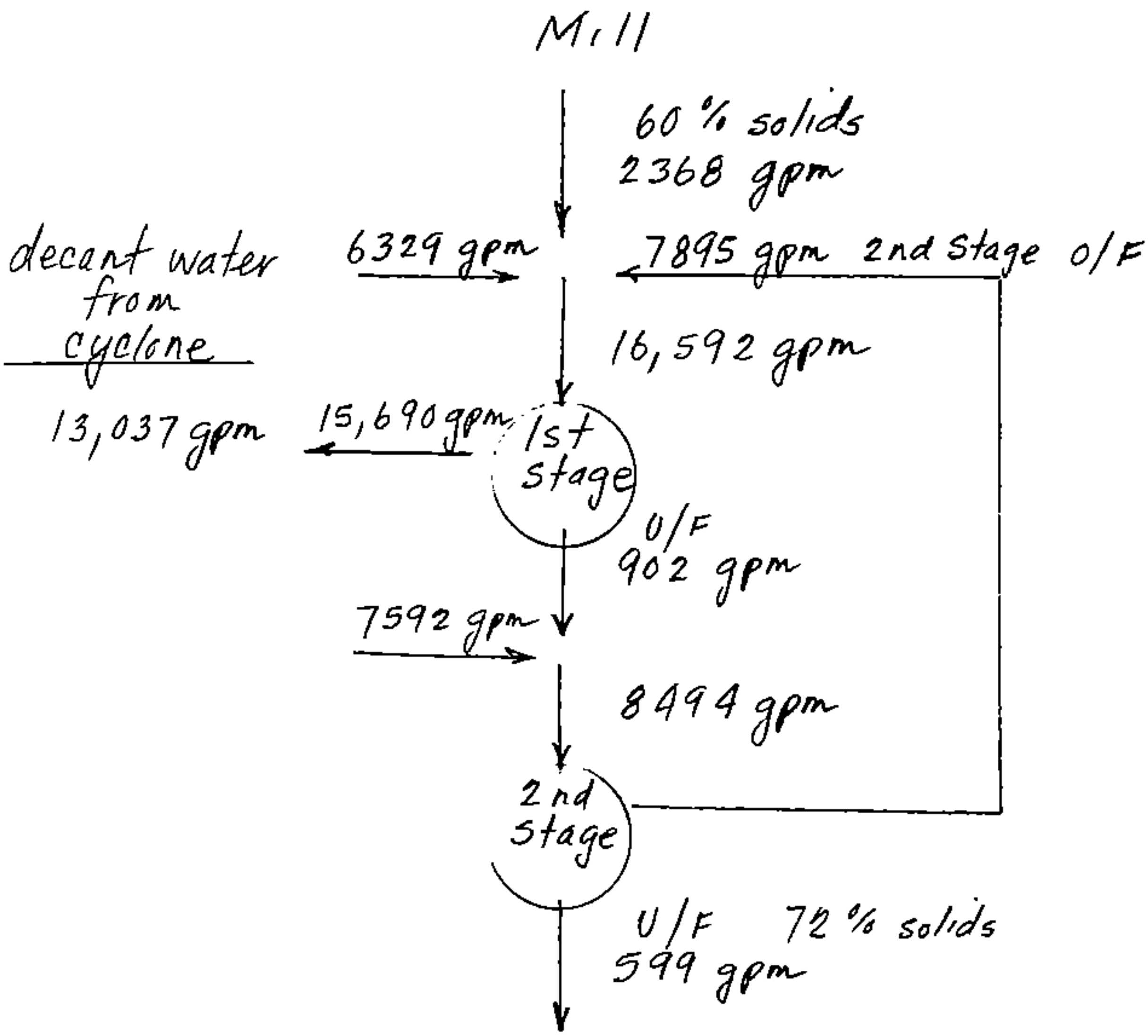


O/F available	req'd	U/F available
10,276 gpm	10,970 gpm	0 gpm

additional water needed 694 gpm

CYCLONE WATER BALANCE

Year 3-17 - Alternative 9



O/F available	req'd	U/F available
13,037 gpm	13,921 gpm	0 gpm

additional water needed 884 gpm

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Sheet 1 4

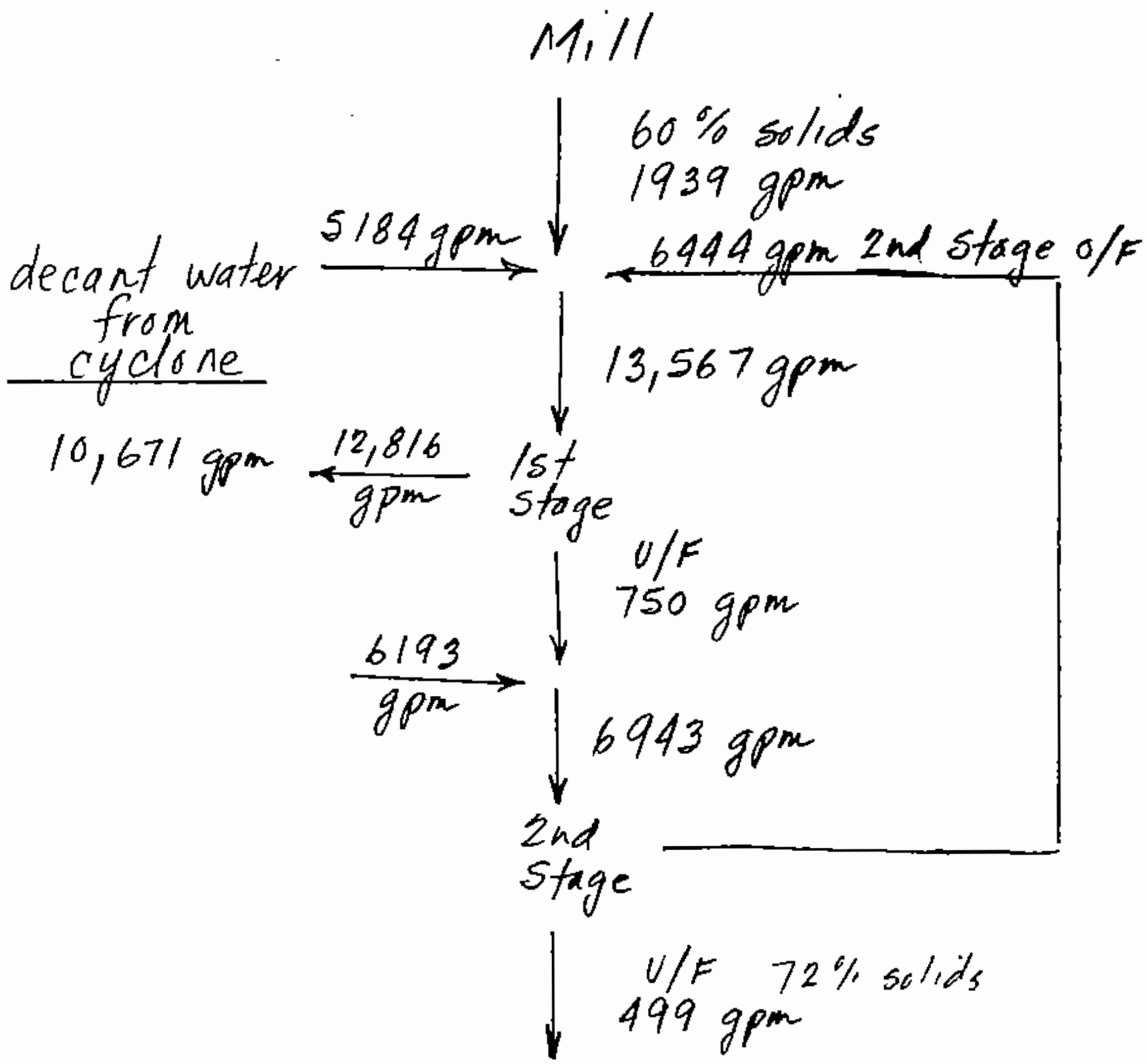
Project ______ Contract No. ______ File No. ______

Feature ______ Designed ______ Date ______

Item ______ Checked ______ Date ______

CYCLONE WATER BALANCE

Year 18 - Alternative 9



O/F available	req'd	U/F available
10,671 gpm	11,377 gpm	0 gpm

additional water needed = 706 gpm

APPENDIX E
TAILINGS DAM FOUNDATION PRESSURE RELIEF SYSTEM

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

HEADQUARTERS OFFICE
180 HOWARD STREET
SAN FRANCISCO CALIFORNIA U.S.A. 94105
TELEX (WUI) 677058 (ITT) 470040 (RCA) 278362 (WUD) 34376
PHONE (415) 442-7300 FAX (415) 442-7405

8029-010
30 April 1990

Noranda Minerals Corp.
Montanore Project
101 Woodland Road
P.O. Box 1486
Libby, MT 59923

Attention: Mr. Joe Scheuering
Project Manager

Subject: Tailings Dam Foundation Pressure Relief System

Gentlemen:

The attached report summarizes the results of MKE's preliminary engineering studies for the tailings dam foundation pressure relief system.

As indicated in our March 1990 Geotechnical Report, artesian pressures were found in borings drilled in the impoundment site. To prevent excessive pressures from developing in the proposed dam foundation that would decrease stability, a pressure relief system will be required. The system will be installed at start-up and expanded during the operational life of the impoundment.

This report presents preliminary designs for both drainage trench and relief well systems. Monitoring programs are discussed and cost data are presented. The report also presents conclusions and recommendations on the preferred approach for pressure relief and recommendations for further work to verify design layouts.

We are available to discuss this report with Noranda. If you have any questions or comments, please call us.

Sincerely,

Alberto Pujol

M. P. Forrest

A. Pujol-Rius
Project Engineer

M. P. Forrest
Project Manager

Attachment

cc: Vern Coffin, Toronto, Ontario
Ron Bradburn, Kelowna, B.C.

30APRTDF.NOR

MONTANORE PROJECT
TAILINGS DAM FOUNDATION PRESSURE RELIEF SYSTEM
PRELIMINARY DESIGN

PURPOSE

A review of the foundation conditions for the proposed Montanore Project tailings dam was performed in order to (1) evaluate the need for a pressure relief system in the dam foundation, (2) develop a preliminary design of the pressure relief system and (3) prepare recommendations for additional field work for verification of design.

BACKGROUND

A dam for impounding copper-silver tailings is planned across Little Cherry Creek approximately one mile west of the confluence with Libby Creek, in Lincoln County, Montana. Based on preliminary engineering studies, a 120-foot-high zoned earth and rockfill starter dam will be constructed with a crest at El. 3500 to provide tailings storage for the first year of mill operation. The dam will be gradually raised using cycloned tailings underflow as embankment material. The tailings overflow will be stored in the impoundment behind the dam. The ultimate dam crest will be at about El. 3700, at which the dam will reach a height of approximately 380 feet. It is estimated that the impoundment will be filled with tailings after 18 years of mine operation. Once the impoundment is full, it will be reclaimed by dewatering the pond and regrading, capping and revegetating the impoundment surface. The tailings dam design is presented in MKE's February 1989 Preliminary Engineering Report.

Geotechnical conditions in the impoundment are described in MKE's March 1990 Geotechnical Report. Bedrock at the site consists of highly fractured and weathered metamorphosed sedimentary rocks, primarily metamorphosed siltstone and argillite. A few outcrops exist along the edges of the impoundment. Generally, the depth to rock varies from 15 to 80 feet; however, geotechnical investigations indicate that there is a buried bedrock channel deeper than 367 feet that crosses the proposed dam foundation.

The impoundment soils mainly consist of dense mixtures of clay, silt, sand, gravel, cobbles and boulders. These soils are believed to be of glacial origin. In portions of the impoundment, surficial deposits of stiff silt and clay without a significant fraction of coarse-grained soils have been observed. These materials are presumed to be of alluvial or lacustrine origin. It is known that an ancestral lake occupied a large area east of the site. Lake-deposited silt sediments exist with a thickness of over 100 feet along Libby Creek. The maximum lake level has been hypothesized to be roughly at El. 3550, which lies above the dam foundation but below the dam crest. In the tailings impoundment area, the boundary between glacial sediments and lake deposits has not been determined. If such boundary exists, it probably occurs as a complex interfingering of glacial and lake deposits over a large transitional area, rather than as a well-defined boundary surface.

Artesian pressures have been observed in some of the exploratory boreholes drilled in the impoundment area, particularly along Little Cherry Creek. However, in most of the boreholes that indicated artesian pressures, the confining stratum could not be identified.

STATEMENT OF PROBLEM

The gradual filling of the tailings impoundment, with associated higher water level, will result in gradually higher pore water pressures in the impoundment foundation soils. Where low permeability soils under the dam and downstream of it overlie more pervious soils or rock, the low permeability soils may act as a confining layer. If sufficiently extensive, such a layer would impede dissipation of impoundment-induced pore pressures in the foundation, possibly resulting in excessive pressures at the toe of the dam. Excessive pressures in the dam foundation would decrease dam stability and could cause sliding and/or heaving at the toe of the dam. Ultimately, they could render the foundation unsuitable for further embankment construction.

THEORETICAL CONSIDERATIONS

In qualitative terms, the following conditions must exist for excessive impoundment-induced pressures to develop at the toe of the dam:

(1) The impoundment piezometric head must be transmitted with little head loss to the impoundment foundation soils. The head can be transmitted through the slimes, as well as directly through those soils which are submerged by ponded water but not blanketed by slimes (along the tail and sides of the impoundment). With regard to the impoundment soils covered by slimes, if the permeability of the slimes is much lower than that of the underlying soils, a significant portion of the head would be lost through the slimes.

(2) The foundation soils must transmit the piezometric head horizontally to the toe of the dam without significant head loss. In order for this to be possible, the soil strata that transmit the piezometric head must (a) extend under the dam to provide horizontal hydraulic continuity, and (b) be overlain by lower-permeability materials that impede seepage and dissipation of pore pressures into the drainage blanket at the base of the dam. Dissipation of piezometric head will be further impeded if the strata that transmit the head grade in the downstream direction into lower-permeability materials so that no efficient downstream drainage can occur.

(3) The thickness of the layer overlying the pressurized strata must be relatively small compared to the excess water pressures, so that the upward hydraulic gradient will cause a significant reduction in effective stresses, resulting in loss of shear strength and unacceptable reduction of the safety factor of the dam.

In the extreme, if the above conditions are present and pressure relief is not provided, the loss of shear strength could cause sliding at the toe of the dam. Uplift of the confining layer by the pressurized strata could cause heaving, boils and resulting degradation of the soil conditions downstream of the dam.

Soil improvement (drainage and densification) would then be necessary before the soils can be accepted as the foundation for further expansion of the tailings embankment.

EVALUATION OF AVAILABLE DATA

Based on the above discussion, the available data on impoundment foundation conditions were evaluated to answer the following questions:

1. Is there a confining stratum extending over the entire reservoir area?

2. Is there a confining stratum extending over the dam foundation area?

3. Can the slimes be relied upon to blanket the reservoir and minimize foundation pore pressures?

These questions are addressed below:

1. Is there a confining stratum extending over the entire reservoir area?

Some of the boring logs in the reservoir area show low-permeability soils overlying coarser, more permeable materials (e.g., boring DH-17, USB-2 and USB-5). However, other logs do not provide any indication of a confining layer (DH-4, DH-14, DH-18, USB-4). With respect to test pits, some logs show upper silt layers underlain by gravel, (TP-134, -141, -139, -128, and -144) but others show sandy soils at the ground surface (TP-101 and -102), and others show low-permeability soils extending the full depth of the test pits (TP-106, -107, -135 to -138, -140, -142 and -104).

Based on the above, it appears that confining layers, while extensive, are not continuous over the entire reservoir area. Therefore, it must be anticipated that water stored in the tailings impoundment will have access to the full column of soils underlying the area. The impoundment design cannot rely on the hypothetical existence of a natural clay or silt layer blanketing the bottom of the tailings reservoir for controlling pore water pressures at the toe of the dam.

2. Is there a confining stratum extending over the dam foundation area?

The available boring logs indicate that confining strata exist throughout the portion of the dam foundation south of Little Cherry Creek. Borings DH-2 and DH-19 encountered 20 to 25 feet of silty soils overlying coarse grained soils. Boring LCTM-8 revealed several layers of water-bearing gravel overlain by silty soils. Boring DH-3 and DH-20 encountered silt and clay to depths of 30 and 45 feet, respectively, underlain by gravels, cobbles and boulders in a silt matrix. Thinner confining strata were also found on the right abutment at boring USB-1 (to a depth of 18 feet) and DH-13 (to a depth of 5 feet).

North of the creek, the existing information is less definitive. A possible confining layer was noted from 16 to 22 feet in boring USB-3. Boring DH-1 revealed low-permeability soils extending to bedrock at 77 feet. Gravel strata containing perched water levels were observed in boring LCM-9, interbedded with lower-permeability silty soils. Boring DH-12 encountered silty soils to bedrock at 65 feet. Artesian pressures were observed in the hole after it was completed 46 feet into bedrock.

Artesian pressures above ground level have been measured in holes DH-2, -3, -12, -19, and -20 in the dam foundation and in PLCM-6, south of the right abutment. Groundwater confinement was noted after drilling into bedrock at borings DH-2, -12, -19 and PLCM-6, and within soil at borings DH-3 and -20. Two springs have been mapped within the dam foundation area south of Little Cherry Creek. One spring has been mapped north of the creek about 800 feet downstream of the dam.

The data summarized above indicate that confining strata cover most of the dam foundation. At some locations the full soil column may act as a confining stratum for groundwater in bedrock fractures. The degree of lateral continuity of the confining strata is uncertain and should be investigated further, although the numerous observations of artesian conditions indicate that it is considerable. The observed springs suggest that the confined, relatively permeable strata may pinch out eastward or grade into less permeable soils, perhaps due to a lateral transition from glacial deposits to lakebed sediments. The springs are manifestations of drainage to the ground surface and provide some degree of pressure relief.

3. Can the slimes be relied upon to blanket the reservoir and minimize foundation pore pressures?

The slimes in the reservoir could be considered to form a low permeability blanket over the natural impoundment soils. However, in order for the blanket to be effective, the ponded water (decant pond) must be substantially confined within the tailings so that direct contact between free water and impoundment soils is minimized. In addition, the permeability of the slimes has to be significantly lower than that of the foundation soils. Lastly, tailings consolidation must not result in significant additional amounts of water being introduced into the foundation.

With regard to the location of the decant pond, it is not anticipated that it will be possible to confine it to a location entirely within previously deposited slimes. The Preliminary Engineering Report indicates that the decant pond could reach a volume of about 700-acre-feet. If located at the reservoir tail, such a pond would probably cover many acres of natural soils (not blanketed by slimes) to a depth of several feet.

In regard to the slimes permeability and consolidation properties, published data indicate copper slimes permeabilities in the range of 10^{-5} to $5x10^{-7}$ cm/sec. The permeability of impoundment soils has been measured to average $2x10^{-5}$ cm/sec at the boring locations. However, borehole pressure tests primarily measure horizontal permeability which, because of stratification, tends to be higher than the vertical permeability. The vertical permeability of the foundation soils is relevant because seepage under a slimes blanket would be close to vertical in the immediate vicinity of the slimes. It is expected that the vertical permeability of the impoundment soils would be of the same order of magnitude as that of the slimes.

It is therefore concluded that while the slimes may reduce foundation pressures and seepage flow from the impoundment, they cannot be completely relied upon to control pore pressures at the toe of the dam.

In summary, the evaluation of available data indicate that impoundment seepage is likely to occur into soil strata that, within the dam foundation, appear to be extensively confined by overlying lower-permeability soils. Therefore, pressure relief will be required along the toe of the dam. However, the field exploration data do not fully define the lateral continuity and depth of the potential confining layers in the dam foundation or the thickness of the aquifer. Further investigations will be required to optimize the design of seepage control measures.

PRELIMINARY DESIGN OF PRESSURE RELIEF SYSTEM

Preliminary designs of alternative pressure relief systems were developed based on the available data. Conservative assumptions were made to supplement the existing data. Two alternative systems were considered:

(1) Drainage trench: A drainage trench would be excavated along the downstream toe of the starter dam and would be backfilled with drain material. The preliminary plan location is shown on Sketch 1. The material from the trench excavation could be placed in the starter dam embankment, although it would probably have to be dried before it could be compacted in the fill. The trench would be lined with filter fabric to prevent the adjacent soil from piping into the drain material. Crushed mine waste rock could probably be used as drain material, provided that a heavy-duty fabric is used. The trench would parallel the dam crest. Two spur trenches would be constructed to discharge the seepage from the trench by gravity into the seepage collection dam.

(2) Relief wells: A line of relief wells would be drilled along the same alignment as the trench. The well casings would be open at the collar and would discharge into the drainage blanket of the tailings dam. Alternatively, a manifold pipe could be used to collect the well

discharge, but it would have to be designed for the load imposed by the tailings dam.

The main advantage of the drainage trench is that it would provide continuous pressure relief in the direction parallel to the dam crest. This is desirable since the foundation conditions involve complex interfingering and interlayering of different materials. Relief wells would be unlikely to pierce all layers and lenses that may become pressurized by the impoundment head. Additionally, the trench provides a large surface area available for drainage across the foundation soils, a desirable characteristic since the soils are fairly tight.

The main advantage of the relief wells is that they would be installed to a much greater depth than the trench, thereby allowing deep pressure relief. This is a significant advantage since deep artesian pressures, indicative of deep confinement, have been observed at several boreholes. Deep confining strata could not be penetrated by the drainage trench.

In either case, a monitoring system consisting of piezometers and survey monuments would be needed downstream of the pressure relief trench or wells to verify the effectiveness of the system or to determine the need for additional relief measures. The role of monitoring is discussed later in this report.

The geometric model used for the analysis of the drainage trench is illustrated in Sketch 2. A confined aquifer is assumed to be recharged by a line source with head equal to the elevation of the decant pond in the tailings impoundment. The simple model used conservatively assumes that if pressure relief were not provided, the head in the confined aquifer would equal that of the line source, i.e., no leakage or dissipation of pressures occurs through the confining stratum and no other drainage feature exists in the downstream direction (Ref. 1). The data required to evaluate pressure relief due to the drainage trench include (1) the thickness of the confining stratum, (2) the thickness of the confined aquifer, (3) the distance between the line source and the trench, and (4) the head at the line source.

Based on the exploration data, the thickness of the confining stratum was estimated to average 30 feet. The thickness of the confined aquifer cannot be estimated from the available data since, within the depths explored, the permeability of bedrock appears to equal or exceed that of the overburden. A sensitivity analysis was performed using assumed aquifer thicknesses of 200 and 400 feet.

Several combinations of values of head at line source and distance between line source and trench were analyzed to provide a general estimate for the range of results. Referring to Sketch No. 2, line source No. 1 is for a starter impoundment containing virtually no tailings but with a water level at El. 3460. The water would result from mine startup and storm runoff. Line source No. 2 is for a full starter impoundment; the location of the assumed source is 400 feet upstream of the dam toe to account for the tailings covering the bottom of the impoundment. The 400-foot distance is about 50% of the distance to the reservoir tail. The full distance to the tail was not used because the decant pond would probably submerge natural ground closer to the dam near the abutments. Lastly, line source No. 3 represents the full ultimate impoundment just before reclamation.

The analysis yields the maximum head in the confined aquifer downstream of the drainage trench. For this preliminary evaluation, the maximum allowable head was estimated to be 10 feet above the ground surface. For final design, verification or modification of this estimate by slope stability analysis will be required.

The results of the analysis indicate that a 40-foot-deep trench (i.e., 10-foot-deep penetration below the assumed 30-foot-deep confining stratum) would be required to relieve pore-pressures if a 200-foot-thick aquifer is assumed. If a 400-foot-thick aquifer is assumed, a 50-foot-deep trench (providing 20-foot-deep penetration into the aquifer) would be required. A conceptual cross-section of the 50-foot trench is shown on Sketch No. 3. Estimated order-of-magnitude flows from the trench are 100 to 200 gpm. However, actual flows could differ from the estimated values.

A similar analysis was performed assuming gravity drainage wells instead of a trench (Ref. 2). Results for assumed aquifer depths of 200 feet and 400 feet and for alternative well depths and radii are summarized below:

Assumed Aquifer Thickness (Ft)	Well Depth (Ft)	Required Well Spacing (Ft)		Estimated Well Flow (GPM per Well)
		Well Radius= 0.5 Ft	Well Radius= 1.0 Ft	
200	150	70	85	1 - 2
	180	90	110	2 - 3
	230	130	150	3 - 4
400	150	10	15	0.5 - 1
	180	22	29	1 - 2
	230	45	55	2 - 3
	330	85	100	4 - 5

The table shows that the assumed aquifer thickness significantly affects the required well depth and spacing.

COSTS

Estimated order-of-magnitude costs are $250 per linear foot of drainage trench and $50 per linear foot of relief well. The cost of the drainage trench and spur trenches shown on Sketch 1 would be in the order of $1.5 million to $1.7 million. The cost of a comparable relief well system consisting of a 5,000-foot-long line of 230-foot-deep relief wells at 45 feet spacing would be about $1.3 million. The above costs do not include the cost of the downstream monitoring wells, which would be necessary for either alternative.

DISCUSSION OF DESIGNS

It is believed that the preliminary designs described above provide a conservative scope of the pressure relief system required for this site. The main simplifying assumptions made (i.e., that a continuous confining stratum exists over the entire dam foundation and that the only drainage available to the confined aquifer is that provided by the pressure relief system) are conservative. However, as indicated above, the size of the pressure relief system also depends on the thickness of the confined aquifer. Further field investigation will be required to define aquifer thickness.

Regardless of which pressure relief system is finally selected, monitoring of foundation pore pressures downstream of the system will be essential to verify that excessive pore pressures do not develop. Monitoring will be accomplished primarily by direct measurement of water heads at monitoring wells installed downstream of the pressure relief system, supplemented by surveying of surface markers on the foundation and visual observation of foundation conditions. When opened to the atmosphere or to the drainage blanket of the dam, the monitoring wells will become additional relief wells.

Because the impoundment will be raised gradually over an extended period of time, the observational approach can be used (Ref. 3). In this approach, a pressure relief system would be initially installed, but its size would be smaller than the conservative dimensions described above. A system to monitor foundation pore pressures and heave would also be installed, and a continuous monitoring program would be implemented to record the behavior of the dam foundation under the existing loading conditions. Analysis of the data would show whether the installed system is sufficient or additional pressure relief is required. Once excessive pressures are detected, whether by piezometer readings or by observation of sand boils or sloughing at the toe of the dam, the distressed areas would have to be immediately treated by installing additional relief wells and possibly by placing suitable fill over them to provide additional overburden weight. Prompt implementation of remedial measures to relieve localized distress would be a maintenance activity.

A pressure relief system based on relief wells would be better suited to permit the implementation of the observational approach than the drainage trench. An initial system could be installed with a reasonably wide spacing between wells, and the system could then be readily expanded by installing additional wells as required.

A drainage trench would be less well suited for this approach since, in any case, the trench must fully penetrate the confining stratum and extend a minimum depth into the confined aquifer. Because the average depth to the bottom of the confining stratum in the dam foundation area is estimated to be about 30 feet, a minimum average trench depth of 40 feet would be required. The cost of installation of the trench would be substantially higher than that of the initial relief well system. If a drainage trench were installed, additional pressure relief in areas where required would have to be provided by relief wells since deepening of the trench is unlikely to be practical.

Based on the theoretical design results described above, we estimate that an initial pressure relief system could consist of (1) a line of 230-foot-deep 12-inch diameter relief wells located parallel to the downstream toe of the starter dam with spacing between wells of about 100 to 200 feet and (2) a line of monitoring wells parallel to the line of relief wells and located about 300 feet downstream of it. This concept is illustrated on Sketch No. 4. The monitoring well spacing would be 200 feet and the wells would be staggered with respect to the relief wells, as shown on Sketch No. 4. The monitoring wells would be constructed to the same dimensions as the relief wells. Once the toe of the tailings dam covers the monitoring wells, the wells would be left open into the drainage blanket and would serve as additional relief wells. A new line of monitoring wells would then have to be installed farther downstream. The location of this line and the well spacing would be determined based on the foundation performance at that time. The ultimate dam foundation could have a total of 4 or 5 lines of wells, as shown on Sketch No. 5. Estimated order-of-magnitude flows from the final pressure relief system are 100 to 300 gpm. However, actual flows could differ from the estimated values. The approximate cost of the initial pressure relief system, including the monitoring wells, is estimated to be about $600,000.

The approach proposed above provides considerable flexibility to make changes based on the actual conditions encountered. If based on a well-planned monitoring program, it should provide adequate control of foundation pore pressures.

The relief and monitoring well spacings indicated above are preliminary and should be confirmed or modified based on the results of additional field investigations.

CONCLUSIONS AND RECOMMENDATIONS

1. Available data suggest that seepage from the impoundment will flow into soil strata that, within the dam foundation, appear to be confined by overlying lower-permeability soils. It is anticipated that foundation pore pressure relief will be necessary along the toe of the dam. However, the lateral continuity and depth of the potential confining layers in the dam foundation, as well as the thickness of the confined strata, need to be further explored.

2. Field data indicate that natural silt or clay layers, while extensive, are not continuous over the entire reservoir area. Therefore, the impoundment design should not rely on the existence of a natural clay or silt layer blanketing the bottom of the tailings impoundment for controlling pore water pressures at the toe of the dam.

3. Likewise, the slimes in the impoundment will not form an effective blanket that can be relied upon for controlling pore pressures at the toe of the dam.

4. It is recommended that the pressure relief system be designed and installed based on the observational approach. With this approach, a pressure relief system of modest size would be initially installed and a well-planned monitoring system would be implemented. Foundation pore pressures would be regularly monitored and foundation conditions would be inspected visually and by surveying surface monuments. Because the initial pressure relief system would be of a smaller scale than indicated by conservative, simplified analyses, excessive pore pressures and distress could develop in localized areas of the foundation. These areas would be immediately treated by installing relief wells and possibly by placing a tailings fill surcharge over them. This work would be part of the regular maintenance of the impoundment.

5. Alternative pressure relief systems include (1) drainage trenches and (2) lines of relief wells. The relief wells would be more economical and better suited to the observational approach than the drainage trench. Therefore, further design efforts should be concentrated on developing a relief well system, with local drainage trenches.

6. Based on simplified analyses, the initial pressure relief system should include a line of 230-foot-deep relief wells at 100- to 200-foot spacing along the toe of the starter dam and a similar line of monitoring wells installed about 300 feet downstream of the relief wells. The order-of-magnitude cost of this well system is $600,000.

7. Additional drilling, pressure tests and pump tests should be performed along the proposed relief well alignment in order to (1) determine depth of confining stratum, (2) determine aquifer thickness and characteristics, and (3) confirm or modify the proposed pressure relief system location and dimensions. The exploratory borings could be drilled and cased to meet relief well specifications so that the borings may be incorporated in the final system.

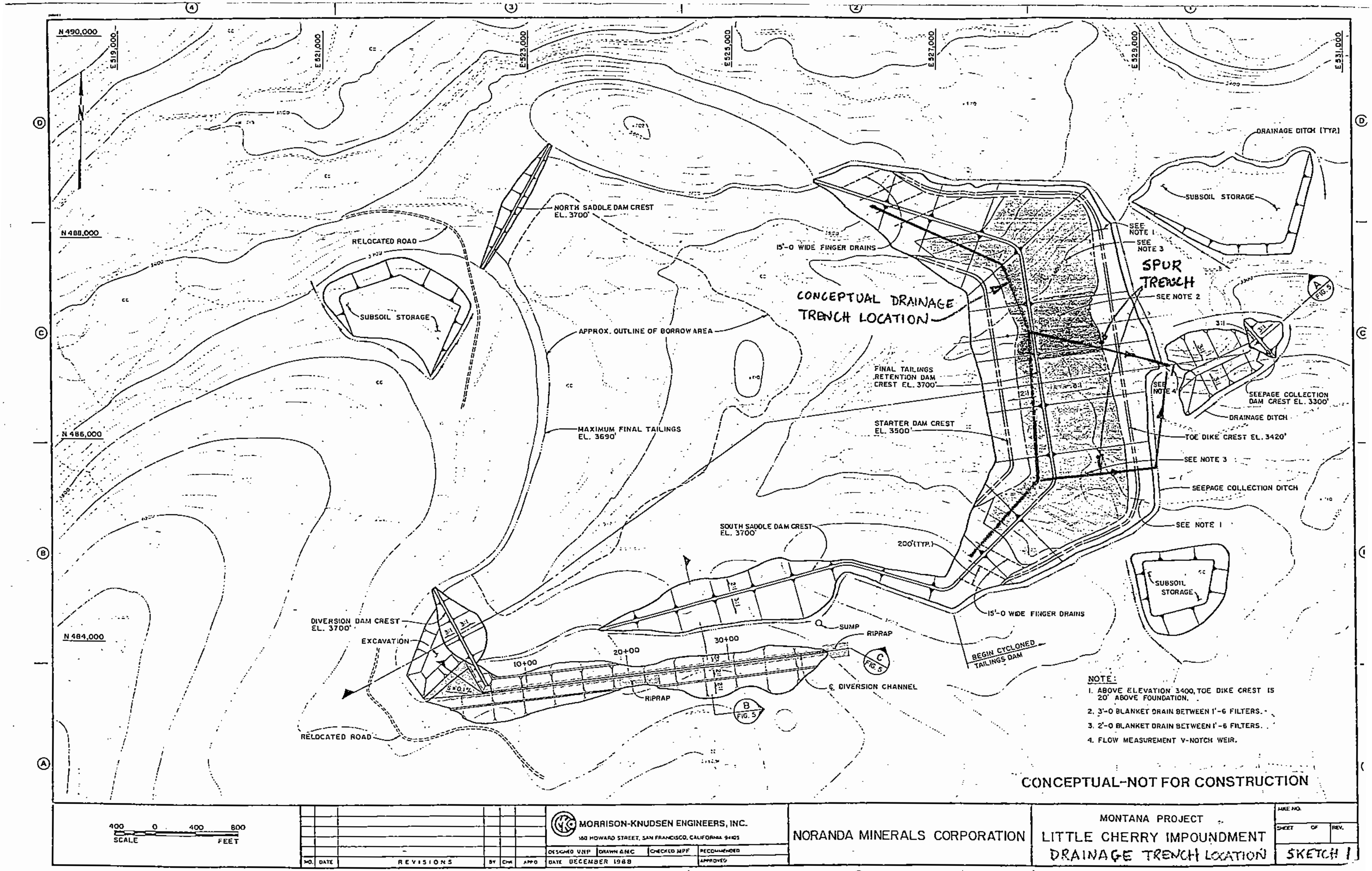
8. Additional analysis will be required to finalize design. The analysis will be based on the results of the additional field investigation.

<u>REFERENCES</u>

1. Mansur, C.I. and R.I. Kaufman, "Dewatering," in G.A. Leonards, Ed., "<u>Foundation Engineering</u>," McGraw-Hill, 1962.

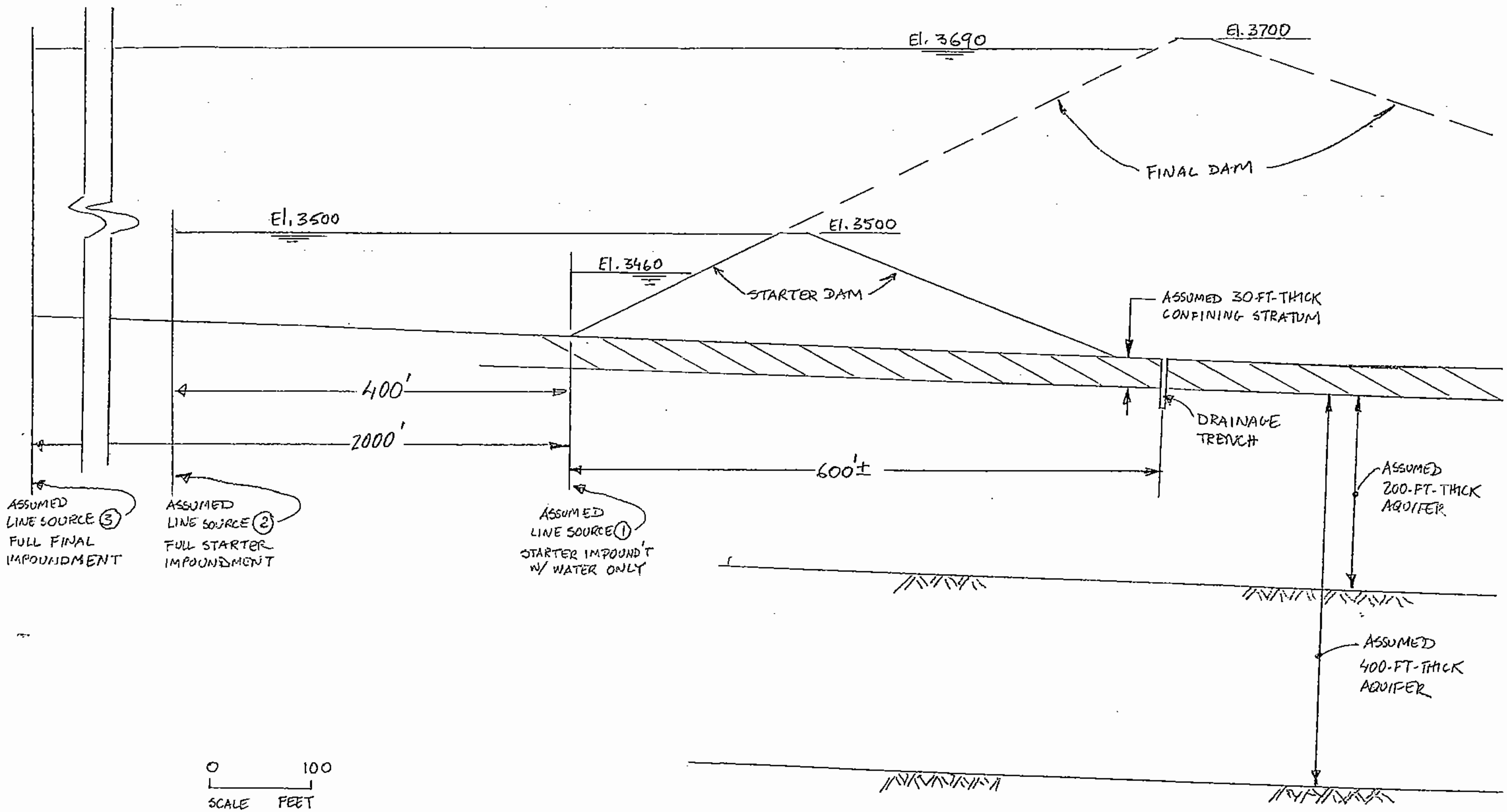
2. Turnbull, W.J. and C.I. Mansur, "Design of Control Measures for Dams and Levees," <u>ASCE Transactions</u>, Vol. 126, Part I, 1961, pp. 1486-1521.

3. Smith, E.S., "Planning Tailing Dam Location, Construction and Operation," <u>Mine Planning and Development</u>, First International Symposium on Mine Planning and Development, Beijing, China, Miller-Freeman Publishers, 1980.


N 490,000
N 488,000
N 486,000
N 484,000
E 519,000
E 521,000
E 523,000
E 525,000
E 527,000
E 529,000
E 531,000
NORTH SADDLE DAM CREST EL. 3700'
RELOCATED ROAD
SUBSOIL STORAGE
APPROX. OUTLINE OF BORROW AREA
MAXIMUM FINAL TAILINGS EL. 3690'
15'-0 WIDE FINGER DRAINS
CONCEPTUAL DRAINAGE TRENCH LOCATION
SPUR TRENCH
SEE NOTE 1
SEE NOTE 3
SEE NOTE 2
SEE NOTE 4
DRAINAGE DITCH (TYP.)
FINAL TAILINGS RETENTION DAM CREST EL. 3700'
STARTER DAM CREST EL. 3500'
SEEPAGE COLLECTION DAM CREST EL. 3300'
DRAINAGE DITCH
TOE DIKE CREST EL. 3420'
SEEPAGE COLLECTION DITCH
SOUTH SADDLE DAM CREST EL. 3700'
200'(TYP.)
DIVERSION DAM CREST EL. 3700'
EXCAVATION
10+00
20+00
30+00
SUMP
RIPRAP
FIG. 5
℄ DIVERSION CHANNEL
BEGIN CYCLONED TAILINGS DAM
NOTE:
1. ABOVE ELEVATION 3400, TOE DIKE CREST IS 20' ABOVE FOUNDATION.
2. 3'-0 BLANKET DRAIN BETWEEN 1'-6 FILTERS.
3. 2'-0 BLANKET DRAIN BETWEEN 1'-6 FILTERS.
4. FLOW MEASUREMENT V-NOTCH WEIR.
CONCEPTUAL-NOT FOR CONSTRUCTION
400 0 400 800
SCALE FEET
NO. DATE REVISIONS BY CHK APPD
MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105
DESIGNED VNP DRAWN AMC CHECKED MPF RECOMMENDED APPROVED
DATE DECEMBER 1988
NORANDA MINERALS CORPORATION
MONTANA PROJECT
LITTLE CHERRY IMPOUNDMENT
DRAINAGE TRENCH LOCATION
SHEET OF REV.
SKETCH 1

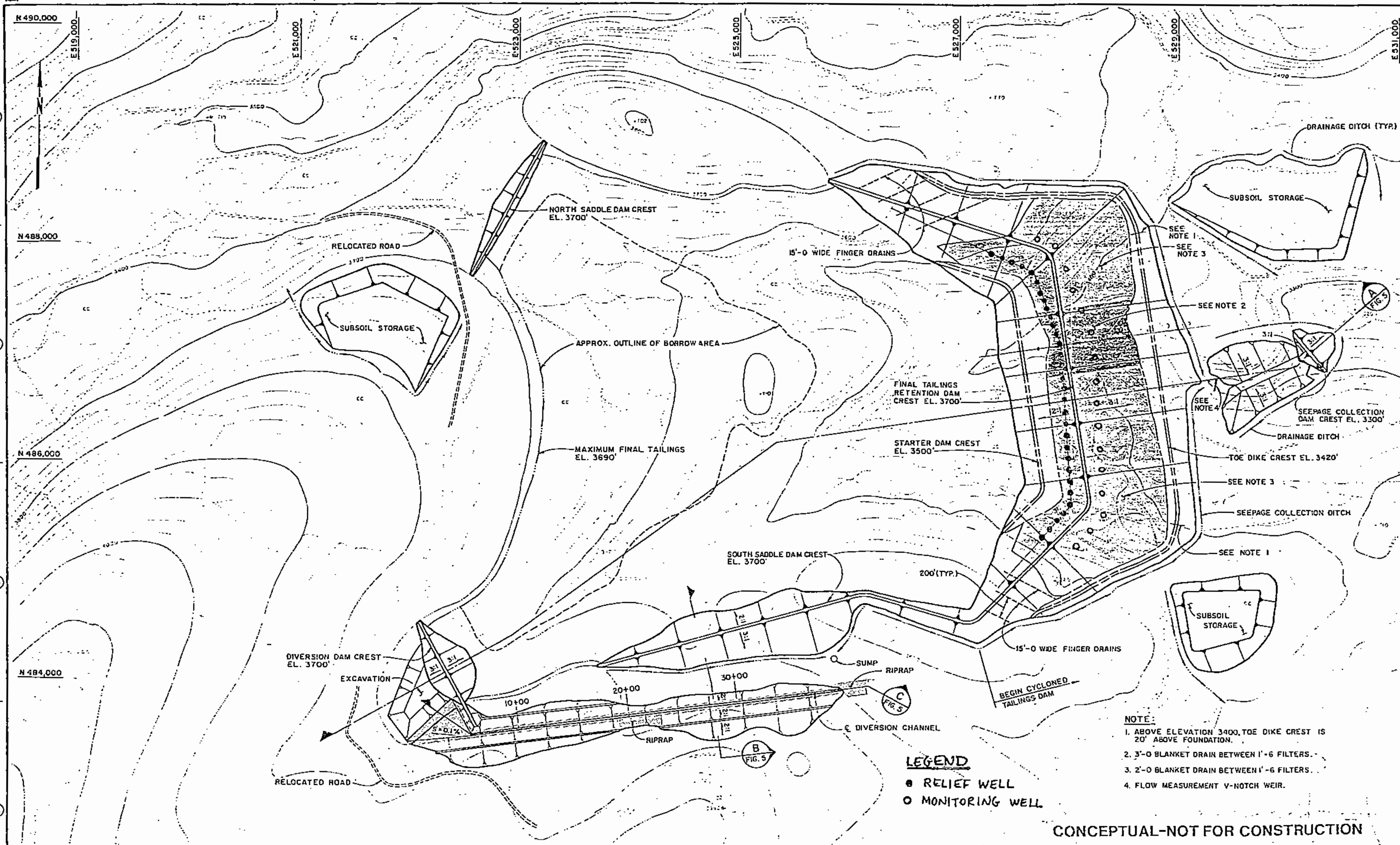
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

			Sheet
Project	NORANDA - MONTANORE	Contract No.	File No.
Feature	TAILINGS IMPOUNDMENT	Designed APR	Date 3-2-90
Item	PRESSURE RELIEF SYSTEM	Checked	Date



CROSS-SECTION FOR ANALYSIS OF PRESSURE RELIEF SYSTEM

SKETCH 2



400 0 400 800
SCALE FEET

NO.	DATE	REVISIONS	BY	CHK	APPD

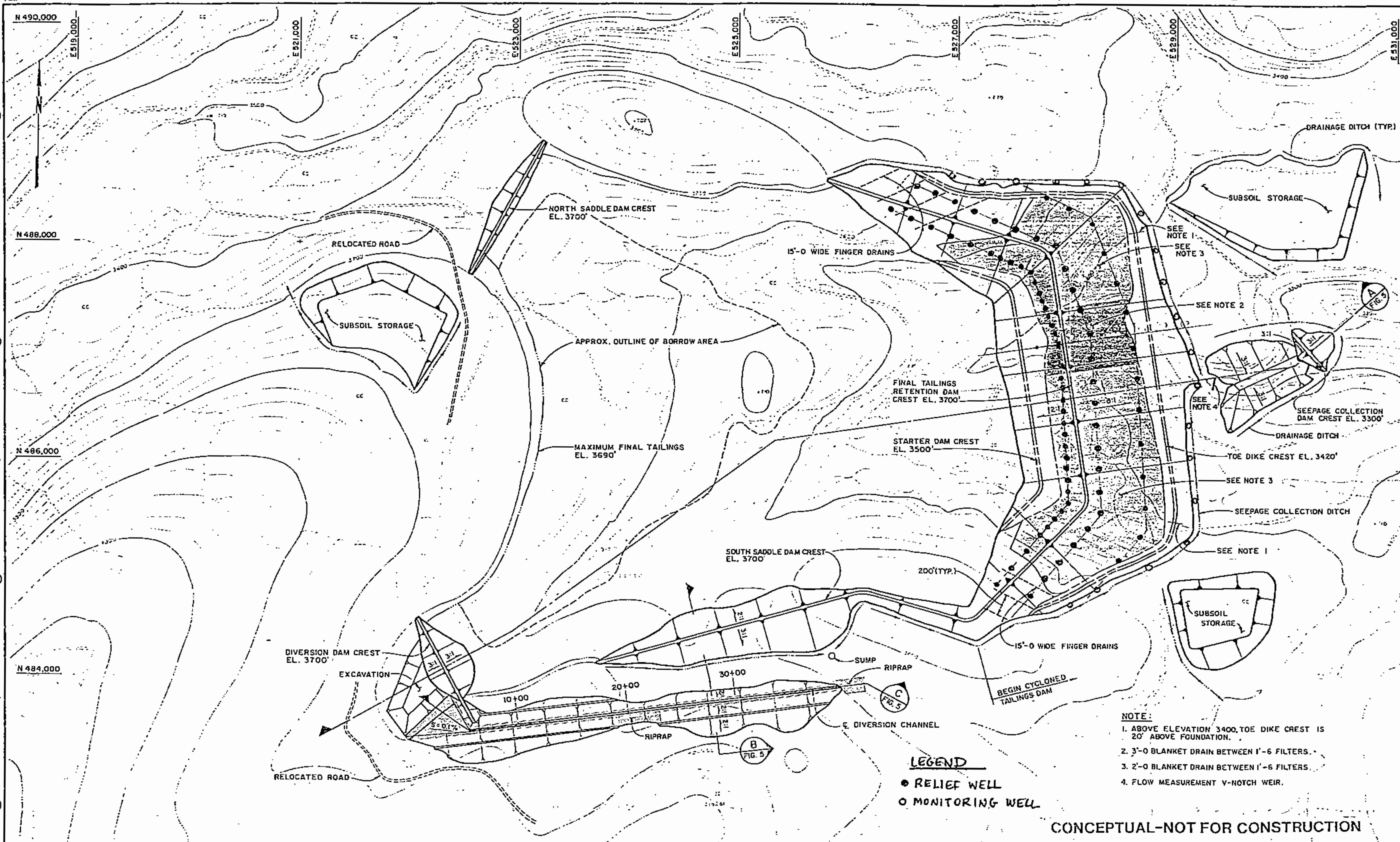
MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED VNP | DRAWN AMC | CHECKED MPF | RECOMMENDED
DATE DECEMBER 1988 | APPROVED

NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY IMPOUNDMENT
RELIEF WELLS
INITIAL INSTALLATION

SKETCH 4



400 0 400 800
SCALE FEET

NO.	DATE	REVISIONS	BY	CHK	APPD

MORRISON-KNUDSEN ENGINEERS, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED VNP | DRAWN AMC | CHECKED MPF | RECOMMENDED
DATE DECEMBER 1988 | APPROVED

NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY IMPOUNDMENT
RELIEF WELLS
ULTIMATE INSTALLATION

SKETCH 5