MONTANORE PROJECT
GEOTECHNICAL REPORT
TAILINGS IMPOUNDMENT SITE
VOLUME 1 OF 2

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

by:
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March 1990

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26 March 1990

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Attention: Mr. Joe Scheuering
Project Manager

Subject: Montanore Project
Geotechnical Report
Tailings Impoundment Site

Gentlemen:

Morrison-Knudsen Engineers, Inc. (MKE) is pleased to submit this geotechnical report for the Montanore Project.

This report presents the results of both the 1988 and 1989 geotechnical investigations for the proposed tailings impoundment in the Little Cherry site.

The geotechnical investigations consisted of (1) geologic mapping, (2) seismic refraction surveys, (3) drilling, (4) test pit excavations, (5) in situ field density testing, and (6) laboratory testing. Seismic design criteria and a description of subsurface conditions (bedrock, soils and groundwater) are presented. This report supersedes the December 1989 interim geotechnical report, as laboratory soils testing has been completed.

If you have any questions about this geotechnical report, please call me at (415) 442-7593.

Sincerely,

M. P. Forrest

M. P. Forrest
Project Manager

MPF/sww

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MONTANORE PROJECT

GEOTECHNICAL REPORT

TAILINGS IMPOUNDMENT SITE

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CHAPTER 1
SUMMARY AND CONCLUSIONS

Noranda Minerals Corp. is currently planning the development of the Montanore Project located between Libby and Noxon in Sanders and Lincoln Counties, Montana, for mining and milling copper-silver ore. As part of this project, an impoundment located in the Little Cherry Creek valley will be required to store tailings resulting from milling operations. Morrison-Knudsen Engineers, Inc. (MKE) has been authorized to perform a geotechnical investigation at the proposed tailings impoundment site.

This geotechnical report presents the results of MKE's studies. Evaluations of soils, bedrock and groundwater are included. This report supersedes the December 1989 interim geotechnical report as laboratory testing of impoundment soils has been completed.

Surficial deposits at the lower elevations of the eastern slopes in the project area include alluvial flood plain deposits, colluvium and glaciolacustrine and glaciofluvial deposits. The glaciofluvial deposits consist of sands and gravels; glaciolacustrine deposits consist primarily of gravelly clayey silt with some varved clays.

The bedrock at the site is Precambrian meta-sedimentary rock of the Belt Supergroup that consists primarily of weathered and fractured argillite, siltite and quartzite. Surficial glacial soils cover much of project area at the lower elevations. After the uplift of the Cabinet Mountains, an extensive period of glaciation and erosion followed. Though continental glaciers may not have extended to the project area, ice sheets moving south from Canada during the glacial stages dammed north flowing rivers, creating huge lakes. Sediments from one of these lakes reached the project site (Chen-Northern, 1989).

The site is in the northernmost end of the Intermountain Seismic Belt, which is characterized by moderate to large magnitude earthquakes with shallow focal depths. The largest events known to have occurred in the belt are the magnitude

7.5 Hebgen Lake, Montana (1959), and the magnitude 7.3 Borah Peak, Idaho (1983), earthquakes, both approximately 300 miles from the site. Although there have been no large historic earthquakes in the site region, the frequency of small to moderate events and the presence of long faults of undetermined activity suggest the potential for a large earthquake. Algermissen et al. (1982) assigned a maximum earthquake of magnitude 6.5 to the area, but it was not associated with a specific fault.

Maximum credible earthquakes (MCE) were established for potentially active seismic sources: the Rainy Creek and Bull Lake faults, the Flathead Lake seismic zone, and a random local earthquake. The most significant event for impoundment design is a magnitude 7 earthquake located on the Bull Lake fault, 22 km from the site. At the project site, the anticipated peak rock acceleration is 0.22g and duration of strong motion is 27 seconds for this earthquake.

In the summers of 1988 and 1989, field explorations were performed to determine depths to bedrock, characteristics of the bedrock and soil, and groundwater conditions. Exploration included geologic mapping, seismic refraction surveys, drilling, test pit excavations, permeability tests and field density tests.
A total of 42 seismic lines, totaling 16,445 linear feet, were surveyed to provide data on depth to bedrock and soil conditions. Piezometers were installed in 21 of the 25 borings to monitor groundwater. A total of 43 test pits were excavated to depths ranging from 5 to 17 feet. Selected test pit and boring samples were tested for moisture content, dry density, grain size, plasticity characteristics, compaction and specific gravity. Unconsolidated undrained (UU) and consolidated undrained (CU) triaxial compression tests were performed on undisturbed clayey foundation soils to determine strength parameters for use in stability analyses. In addition, CU tests were performed on borrow soils compacted to the density and moisture content that would be required for embankment dam construction.

Results of the field exploration show that bedrock generally consists of low-grade metamorphosed siltstone (siltite) and argillite, with some interbedded quartzite. Bedrock cores are generally moderately to highly weathered and fractured.

The bedrock surface is generally reflected by the topography. In the southeastern part of the site, a buried bedrock channel deeper than 367 feet extends through the proposed damsite. The results of the exploration indicate that bedrock is less than 50 feet deep in both abutments of the proposed tailings retention dam. In the proposed excavation for the diversion channel, depths to bedrock in test pits and borings range from 3 to 41 feet.

Soil thicknesses are estimated to generally be less than 100 to 150 feet thick. The site soils primarily consist of dense mixtures of clay, silt, sand, gravel and cobbles. Boulders and cobbles within a matrix of finer soils were encountered in many borings and test pits in the impoundment site.

Groundwater in the impoundment site is both confined and perched. Artesian flow was noted in several borings in the proposed dam foundation. Pressure heads as high as 24 feet above the ground surface were measured. In the proposed diversion channel, the highest reported water level was 73 feet below the ground surface.

Permeabilities of soils and bedrock vary widely. Silty sandy gravels typically have permeabilities on the order of 10^{-5} to 10^{-4} cm/sec. Weathered rock has permeabilities ranging from about 10^{-6} to 10^{-4} cm/sec. Low permeability silty clayey soils act as confining zones where they overlie more pervious soils or rock.

Groundwater data indicate that artesian conditions exist in the proposed dam foundation. In order to prevent excessive pressures from developing in the foundation, which would decrease stability, a pressure relief system will be required. This relief system, consisting of wells and possibly drainage trenches, will be installed at start-up and expanded during the operational life of the impoundment. Foundation piezometric pressures will be measured during operations. If pressure heads increase during the expansion of the impoundment, additional wells will be installed. The pressure relief system will be designed during the engineering phase of work.

Prior to placement of embankment fill, the foundation will be cleared of vegetation and stripped of soil containing organic matter. This soil will be stockpiled for later reclamation activities. Any loose or soft materials will be removed from the dam foundation and replaced with compacted fill. Based on the results of the test pit excavations, the depth of stripping is anticipated to be between 1 and 3 feet in the dam foundation; however, the actual depth of stripping will be determined during construction.

The diversion channel will be excavated in weathered rock and soil materials. The results of seismic refraction surveys and drilling indicate that these materials generally can be excavated by ripping; however, limited blasting will probably be needed in hard, unweathered rock, such as in quartzite zones. Groundwater levels measured in the channel area indicate that the channel invert should be above groundwater. Therefore, the excavation would generally be in dry conditions, but local seepages from perched water should be expected. For preliminary layouts, slopes in weathered rock will be 1:1; in soils, the slopes will be 2(H):1(V). These slopes will be confirmed by performing stability analyses during final design studies. Because of the presence of slickensided joints, and talc, chlorite and clay mineral fillings in bedrock fractures, local flattening of channel excavation slopes in rock may be required to prevent instability.

On-site materials for construction of the earthfill dam embankments will consist of soil and weathered rock removed from the diversion channel and other required excavations. Supplemental materials will be obtained from a borrow area located in the western portion of the impoundment. Borrow volume requirements will be determined during final design. Soil borrow materials will consist of silty clay and silty sandy gravel mixed with cobbles and boulders. Due to the wide range of natural moisture contents in the proposed borrow materials, moisture conditioning of the soils will be needed for embankment construction.

Within the potential borrow area in the western part of the impoundment, seepage was observed in two test pits at depths of 10 and 11 feet. The borrow excavations will have to be planned so that they do not extend below groundwater levels. Also, if sandy gravelly soils are exposed during borrow excavations, these areas will have to be covered with a 3-foot thick layer of compacted clayey soil to minimize seepage from the impoundment.

A further exploration program is planned to confirm groundwater conditions for design of the relief well system. Pumping tests are planned for that field program.

CHAPTER 2
INTRODUCTION

2.1 BACKGROUND

Noranda Minerals Corp. is developing the Montanore Project, a copper-silver mining and milling operation. The project is in Sanders and Lincoln Counties, Montana as shown on Figure 1. As a part of this project, a tailings impoundment is proposed on Little Cherry Creek, in the Kootenai National Forest, and is shown on Figure 2.

In 1988, a preliminary field investigation was conducted for preparation of an operations permit application. Further field exploration was conducted in 1989 for final design.

2.2 PURPOSE AND SCOPE

The purpose of this report is to present the results of the 1988 and 1989 geotechnical investigations conducted by MKE. This final report supersedes the December 1989 interim report, as laboratory tests have been completed. The report provides support for engineering studies and design of the proposed tailings impoundment.

The scope of work for the geotechnical investigations includes the following tasks:

- Conduct field investigations consisting of:
  - Data review
  - Reconnaissance
  - Geologic mapping
  - Seismicity evaluation
  - Seismic refraction survey

- Test pit excavations
- Exploratory drilling
- Field density testing
- Laboratory testing

- o Provide seismic design ground motion parameters
- o Evaluate subsurface conditions.

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

2.3 AUTHORIZATION

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

CHAPTER 3
GEOLOGIC SETTING

The primary geological feature of the area is the Cabinet Mountains, a rugged range with altitudes of more than 7,000 feet. Cirques with high walls at the heads of the valleys were formed during the last Pleistocene glaciation. Many of the cirque basins contain lakes that are fed by snowmelt.

The project area is underlain by Precambrian meta-sedimentary rocks of the Belt Supergroup. The rocks form a north-trending structure bounded on the east and west by high-angle faults. The rock structure in general is tilted northward. The bedrock consists primarily of argillite, siltite and quartzite, with some carbonate horizons. Bedrock is exposed over most of the wilderness area except for talus and rock slides that cover many of the steep slopes. Surficial soils related to glaciation cover much of project area at the lower elevations (U.S. Geological Survey and U.S. Bureau of Mines, 1981).

An extensive period of glaciation and erosion during the Pleistocene Epoch followed uplift of the Cabinet Mountains. Glacial striations on ridges indicate that ice (probably alpine or mountain glaciers) once covered all but the highest peaks of the range. Continental glaciers may not have covered the project area. However, the effect of the ice sheets moving south from Canada during the Pleistocene glacial stages was to dam the north flowing rivers, creating huge lakes. Glacial Lake Missoula formed in this manner in the Clark Fork drainage west of the Cabinet Mountains. The water backed up into southern Montana some 250 miles, reached depths of over 2,000 feet, and rose to an elevation of 4,150 feet (Montana Bureau of Mines and Geology, 1962). Ice also blocked the north flowing Kootenai and other local rivers, creating lakes along the eastern side of the Cabinet Mountains. Sediments from one of these lakes reach an elevation of 3,550 feet in the project area, which appears to be slightly lower than that of Glacial Lake Missoula (Chen-Northern, 1989).

Surficial deposits at the lower elevations of the eastern slopes in the project area include alluvial flood plain deposits (silts, sands, gravels and cobbles), colluvium (slope wash boulders and soils), glaciolacustrine gravelly clayey silt with some varved clays, and glaciofluvial sands and gravels.

CHAPTER 4
SEISMICITY

4.1 GENERAL

This chapter provides background information on seismicity of the region surrounding the Montanore Project. The chapter also includes data used to define a project design earthquake, including ground motions expected at the site, along with the procedures used to obtain these parameters.

4.2 REGIONAL SEISMICITY

The site region lies near the northernmost end of the Intermountain Seismic Belt (ISB), a north-south oriented zone of seismic activity that includes the Wasatch Front in central Utah, the Teton-Yellowstone area of Wyoming and parts of the northern Rockies in western Montana. The ISB is characterized by moderate to large magnitude earthquakes with shallow focal depths. Throughout the ISB, including the project site, the predominant type of faulting is normal, along north to northwest oriented faults.

Table 4.1 summarizes significant earthquakes in the region and includes the two largest events known to have occurred in the ISB: the magnitude (M) 7.5 Hebgen Lake, Montana, earthquake of 1959 and the M 7.3 Borah Peak, Idaho, earthquake of 1983. Both these earthquakes were approximately 300 miles from the site so that ground motions felt in the site region were mild. No significant damage occurred in Libby during either earthquake (Stover, 1985).

Swarms of moderate to small earthquakes have occurred repeatedly near Flathead Lake, as shown on Figure 3A. Despite the long history of intense activity, magnitudes reported by the National Oceanic and Atmospheric Administration (NOAA) have never exceeded 5.0. However, Qamar and Stickney (1983) estimate magnitudes of 5.5 for earthquakes in 1945 and 1952 based on unusually large areas where ground motions were felt.

TABLE 4.1

SIGNIFICANT EARTHQUAKES IN SITE REGION(1)

Date	Location	Magnitude(2)	Maximum Intensity(3)	Approx. Distance to Project Site (miles)
2-25-71	S.E. of Libby, MT	---	IV	14
6-26-64	Marion, MT	4.7	IV	21
3-12-18	Lake Pend Oreille, ID	---	IV	27
4-15-52	Whitepine, MT	---	IV	30
8-16-60	Sandpoint, ID	---	IV	47
7-10-30	Missoula, MT	---	V	48
12-19-57	Wallace, ID	5.0	VI	48
11-28-26	Wallace, ID	---	V	48
5- 9-44	Wallace, ID	---	IV	48
6- 8-54	Wallace, ID	---	V	50
9-23-61	Wallace, ID	---	IV	51
11-01-42	WA/ID border	---	VI	52
9-23-45	Flathead Lake, MT	5.5(5)	VII	65
2- 4-75	Creston/Kalispell, MT(8)	4.6	VI	66
3-31-52	Big Fork, MT	5.5(5)	VII	70
7-31-69 to 10-30-70	Canada (9 events)(4)	5.0 to 5.3	VI	116 to 128
12-20-72	Canada	5.1	VI	116
7-16-36	Milton-Freewater, OR	5.75(5)	VII	205
8-17-59	Hebgen Lake, MT	7.5(6)	X, V(7)	293
10-28-83	Borah Peak, ID	7.3(6)	IX, V(7)	310

Notes:

(1) Sources of data: NOAA data file and Qamar and Stickney (1983). Earthquakes with M less than 5 and more than 50 miles from the site are not included (except the 1975 Creston/Kalispell earthquake).
(2) Magnitudes are body wave magnitudes (m_b) except as otherwise noted.
(3) Epicentral Modified Mercalli Intensity.
(4) Swarm of 9 earthquakes with M greater than 5.
(5) Estimated magnitude.
(6) Surface wave magnitude (M_s).
(7) Reported local intensity at Libby, Montana.
(8) Representative event; numerous other earthquakes with M less than 5 have occurred near Flathead Lake.

Other clusters of earthquakes have occurred near Wallace, Idaho, 50 miles south of the site, and in Canada, more than 115 miles to the north. In both areas, the maximum magnitudes have not exceeded 5.3. Many of the events originating from the Wallace area are rock bursts related to mining operations in the Coeur d'Alene mining district (Qamar and Stickney, 1983). Seismicity within a 125-mile radius of the project site is shown on Figure 3B, prepared by the National Geophysical Data Center of NOAA.

Although there have been no large historic earthquakes in the site region, the frequency of small to moderate events and the presence of long faults of undetermined activity indicate that a large earthquake is possible. Algermissen et al. (1982) attributed a maximum magnitude of 7.3 to the Flathead Lake zone and areas to the southeast.

4.3 LOCAL SEISMICITY

The area from Bitterroot Valley north into Canada and from Flathead Lake west to the Washington-Idaho border is characterized by low seismicity. There is no record of a moderate earthquake in the area. However, there have been scattered small earthquakes in the Idaho panhandle in the vicinity of Lake Pend Oreille and a few small earthquakes between Libby and Kalispell. As shown in Table 4.1, the largest and most significant of these was a M 4.7 earthquake in 1964, 21 miles east of the project site. The closest event occurred about 14 miles east of the site in 1971, but was small. No earthquakes exceeding magnitude 4.5 have occurred in northwest Montana in the last five years (M.C. Stickney, personal communication, 1988).

A notable lack of seismicity has been associated with geologic structures such as the Hope, Bull Lake, and Rainy Creek faults (Figure 3). Nevertheless, uncertainties in knowledge of fault activity and a rather short historical earthquake record led Algermissen et al. (1982) to select a maximum earthquake of magnitude 6.5 for the area, which is greater than the largest observed event (M = 5.0). This earthquake is not assigned to any specific fault.

4.4 FAULTS

Witkind (1975) identified and classified faults in the region that show evidence of movement in the last 20 million years. Of those faults, only two in the site area, the Rainy Creek and Bull Lake faults, show evidence of displacing Pleistocene deposits. This places the age of fault movement at less than 1.5 to 2 million years. Orientations and locations of the faults are indicated on Figure 3.

- Rainy Creek Fault: Witkind (1975) classified this fault as active, based on a report that it ruptured the ground surface during an earthquake in 1964. However, the Corps of Engineers performed further field investigations of the Rainy Creek fault in a study for Libby Reregulating dam (Corps of Engineers, 1978, cited in Camp, Dresser & McKee, Inc., 1988). A conclusion of this study was that the reported fault scarp was not tectonic in origin and that it was related to slumping of poorly compacted tailings and overburden. There is now doubt whether this fault should be considered potentially active.

- Bull Lake Fault: Witkind (1975) classified this fault as late Quaternary, less than about 700,000 years old, and therefore potentially active.

- Faults Bounding Libby Valley: Witkind (1975) evaluated these possible faults as late Cenozoic (older than several million years) and questioned their existence. Since there is little evidence for their activity, these faults are not considered to be potential earthquake sources.

- Hope Fault: This fault is about 70 miles long and extends southeastward from Lake Pend Oreille, along the Clark Fork River past Thompson Falls. Witkind (1975) found it to be older than Pleistocene, more than 1.5 to 2 million years old. Therefore, it is not considered to be a potential earthquake source.

- Flathead Lake Seismic Zone: Earthquake activity near Flathead Lake is treated here as a seismic zone in which epicenters do not clearly coincide with known faults. There are numerous candidate faults in the area, both normal and strike slip, but their behavior and individual potential have not been resolved. A conservative distance of 40 miles from the site has been assumed to account for possible branching faults, such as the Big Draw fault, shown on Figure 3A.

4.5 MAXIMUM CREDIBLE EARTHQUAKES

The maximum credible earthquake (MCE) is defined as the largest rationally conceivable seismic event that could occur in the tectonic environment in which the project is located (Seed, 1982). MCE's were established for three potential sources: the Rainy Creek and Bull Lake faults, the Flathead Lake seismic zone, and a random local earthquake. MCE magnitudes for Bull Lake and Rainy Creek faults were calculated using the formula of Slemmons (1982) for normal faults:

$$M_s = 0.809 + 1.341 \log L$$

where L is the entire length of the longest fault segment in meters. Maximum earthquake magnitudes for the Flathead Lake zone and for random local earthquakes were adopted from Algermissen et al. (1982). MCE magnitudes are shown in Table 4.2.

4.6 SEISMIC DESIGN CRITERIA

Peak rock accelerations at the project site were calculated for each of the four potential seismic sources using current attenuation functions (Campbell, 1981; Joyner and Boore, 1982; and Idriss, 1985). Table 4.2 summarizes the MCE magnitudes, source distances, and resulting rock accelerations. As shown in Table 4.2, a random, local earthquake gives the largest estimate of peak acceleration. However, although the MCE on the Bull Lake fault yields an acceleration 5% lower than that from a local source, because of its larger magnitude, it could produce shaking of longer duration (i.e., more significant number of stress cycles). For purposes of the design earthquake, rounding the

Bull Lake MCE to magnitude 7.0 is appropriate. Thus, the design mean peak rock acceleration produced by an MCE of magnitude 7.0 is 0.22g.

The maximum duration of significant shaking for the design earthquake was based on Bolt's (1973) relation of duration and magnitude within a 25 km radius of an earthquake. The duration is taken as the time elapsed between the first and last acceleration pulses that exceed 0.05g and 1 hz. The duration of shaking for design is 27 seconds.

The predominant period of the earthquake motion at the project site was determined by procedures described in Seed, Idriss and Kiefer (1969). From their relations of observed predominant period, distance and magnitude, an average expected value was determined. For sites within 40 km of the seismic source, the predominant period is not sensitive to distance. The predominant period of the design earthquake at the site for design is 0.32-second.

The design earthquake ground motion parameters are summarized below:

- Source: Bull Lake Fault
- Magnitude: 7.0
- Distance from site: 20 km
- Mean Peak Rock Acceleration: 0.22g
- Duration: 27 seconds
- Predominant Period: 0.32-second

TABLE 4.2
PEAK GROUND ACCELERATION FOR
VARIOUS SOURCE MAXIMUM CREDIBLE EARTHQUAKES

Earthquake Source	Site Distance (km)	Fault Length[1] (km)	MCE Magnitude	Mean Peak Ground Acceleration (g)			
				C[2]	J + B[3]	I[4]	Average
Bull Lake Fault	20	30.6	6.8	0.18	0.19	0.21	0.20
Rainy Creek Fault	24	23.3	6.7	0.15	0.15	0.17	0.16
Flathead Zone	65	---	7.3[5]	0.09	0.06	0.09	0.08
Random Local	15	---	6.5[5]	0.19	0.21	0.23	0.21

Notes:

(1) Fault lengths are scaled from map shown in Witkind (1977).

(2) Campbell (1981) attenuation.

(3) Joyner and Boore (1982) attenuation

(4) Idriss (1985) attenuation

(5) Algermissen et al. (1982)

CHAPTER 5
FIELD EXPLORATION AND LABORATORY TESTING

5.1 FIELD EXPLORATION

A. General

In July and August 1988, a preliminary field exploration was performed at the Little Cherry tailings impoundment site. For final design studies, additional exploration was conducted between August and October 1989. The field exploration programs consisted of geologic mapping, seismic refraction surveys, drilling, test pit excavations and in-situ density tests. A summary of the field exploration programs is presented in Table 5.1 at the end of this chapter.

In July 1988, Chen-Northern drilled 5 wells, totaling 370 linear feet, in the Little Cherry impoundment site. Aquifer testing was performed in the wells and the test results are presented in Chen-Northern's 1989 report. Also, Chen-Northern conducted 25 resistivity surveys at the impoundment site. Chen-Northern's exploratory work supplements MKE's field exploration of the Little Cherry impoundment site. The locations of the wells and resistivity surveys are shown on Figure 4.

B. Geologic Mapping

Geologic mapping was performed at the impoundment site. The mapping was done to identify bedrock outcrops, landslides, rockslide areas, avalanche chutes and springs. The geologic data are shown on the exploration plan, Figure 4.

C. Seismic Refraction Surveys

In July 1988, 7 seismic refraction lines were surveyed in the project area to obtain information on depth to bedrock, depths to soil layers, and rippability of site materials. In July and August 1989, 35 additional seismic refraction lines were surveyed in the impoundment area. The line designations, number of seismic lines and total footage are presented in Table 5.1.

Seismic lines were each 325 feet long with geophone spacings of 25 feet; occasionally additional offset shotpoints were used to increase the depth of information obtained. A twelve-channel signal-enhancement seismograph was used to record compressional wave (P-wave) velocities, and explosives or a 10-pound sledge hammer was used as an energy source. The approximate seismic line locations are shown on Figure 4 and a summary of the results of the seismic refraction surveys is presented in Appendix B.

D. Drilling

In July and August 1988, five borings (USB-1 to USB-5) ranging in depth from 25 to 81 feet and totalling 307 linear feet were drilled under the direction of MKE. The boring designations, number of borings and total footage drilled for the tailings impoundment site are presented in Table 5.1. To minimize ground disturbance, preliminary exploration borings were located in readily accessible areas as shown on Figure 4.

The borings were drilled by Chen-Northern of Great Falls, Montana, using a Mobile B-53 rotary drill rig. Borings were drilled to determine depth to bedrock, characteristics of bedrock and soils, and the depth to groundwater. The borings were generally drilled with a 2-15/16-inch tricone bit and casing in the soils. Bedrock was cored by using NQ diamond bits and double-tube core barrels. Standard Penetration Test samplers (2-inch O.D., ASTM D1586) and 2.5-inch O.D. samplers were used to obtain drive samples where soil conditions permitted. Thin-walled steel Shelby tubes were used to obtain undisturbed soil samples, but sample recovery was poor and the tubes were generally bent and creased because of gravels and cobbles. Approximate unconfined compressive strengths of cohesive soil samples were measured with a pocket penetrometer. To monitor groundwater levels, Borings USB-1, 2 and 5 were cased with 1-inch I.D. PVC pipes (bottom sections of the pipes are slotted).

Between August and October 1989, twenty additional borings (DH-1 to DH-20) were drilled at the impoundment site. The borings range in depth from 25 to 367 feet, and total 2265 linear feet. These borings, except DH-15, were drilled by J.D.

Welsh and Associates of Boise, Idaho, using an Acker ADII truck-mounted drill rig and a skid-mounted Longyear 28 drill rig. Borings were drilled to provide supplemental data on depth to bedrock, characteristics of the bedrock and the soils and the depth to groundwater. Where possible, the borings were started by using a hollow stem auger. Where auger drilling was not possible or when auger refusal occurred, the soil was cored with HQ or NQ diamond bits and triple-tube core barrels. Standard Penetration Test samplers and 2.5-inch O.D. samplers were used to obtain drive samples where soil conditions permitted. Since the Longyear 28 drill rig was not equipped with a hammer or cathead, drive samples could not be obtained from Borings DH-9 and DH-10. Shelby tubes were used to obtain undisturbed clayey soil samples, but they were limited to the south part of the proposed main dam foundation due to the gravelly soils found in the remainder of the site.

To monitor groundwater levels, all borings, except DH-3A, DH-5 and DH-15, were cased with 1-inch I.D. PVC pipe. DH-15 was cased with 2-inch I.D. PVC pipe. This boring was drilled by Ruen Drilling, of Clark Fork, Idaho, using a truck-mounted Schramm air rotary drill rig.

Boring logs and monitoring well construction details are presented in Appendix C. Water level data is presented in Table C.1, Appendix C. Locations of all borings, except DH-3A, were surveyed by others.

Field tests were performed in the tailings impoundment site to determine permeability values of foundation materials for seepage analyses. The tests were performed in soils using constant head open-end casing tests conforming to procedures in the U.S. Bureau of Reclamation's "Earth Manual" (1974), Method E-18. Water pressure tests were performed in bedrock using a single pneumatic packer. Results of the permeability tests are presented in Table C.2, Appendix C, and are discussed in Section 6.2.D.

E. Test Pits

In July and August 1988, 10 test pits (TP-101 to TP-110), ranging in depth from 9 to 16 feet, were excavated in the impoundment to characterize the soil materials and to estimate the amount of boulders in the soil matrix. Test pits were located in readily accessible areas (Figure 4). Sorlie Excavating of Trout Creek, Montana, excavated the test pits with a Case 580D backhoe equipped with a 2.0-foot wide bucket. Soil samples were taken from the test pits and transported to Chen-Northern in Great Falls for laboratory testing. Approximate unconfined compressive strengths of cohesive soils were measured with a pocket penetrometer.

In July 1989, 27 additional test pits (TP-121 to TP-147), ranging in depth from 5 to 17 feet, were excavated in the impoundment site to evaluate borrow materials and impoundment excavation conditions. The test pits were excavated by Sorlie Excavating using a Cat 215 track-mounted backhoe with a 3.5-foot wide bucket. Soil samples taken from the test pits were transported to Chen-Northern in Great Falls, Montana, for laboratory testing. Approximate unconfined compressive strengths of cohesive soils were measured with a pocket penetrometer.

Test pit locations are shown on Figure 4 and test pit logs are presented in Appendix D.

F. Density Tests

Density tests were performed in six 5- to 6-foot deep test pits (DT-1 to DT-6) excavated in the proposed tailings dam foundation. These tests were performed by using the sand-cone method conforming to ASTM D1556 and the nuclear densometer method conforming to ASTM D2922. This work was performed concurrently with the test pit excavation program. A summary of test results is presented in Appendix E.

5.2 LABORATORY TESTING

Laboratory tests were performed to obtain engineering parameters of the soils at the tailings impoundment site. Moisture content (ASTM D2216), dry density, grain size (ASTM D422), Atterberg limits (ASTM D4318), moisture-density (ASTM D698), and specific gravity (ASTM D854) were determined for selected test pit and boring samples obtained during both the 1988 and 1989 explorations. Consolidated undrained (CU) triaxial compression tests with pore pressure measurements and unconsolidated undrained (UU) triaxial compression tests were performed on undisturbed Shelby tube samples of clayey foundation soils. CU tests were also performed on clayey soil borrow materials that were compacted to 95% of the maximum dry density at optimum moisture content as determined by ASTM D698. Strength parameters determined from these tests will be used in stability analyses. The types and number of tests at the tailings impoundment site are summarized in Table 5.2. Laboratory test results are presented in Appendix F and are summarized in Table F-1, Appendix F.

TABLE 5.1. SUMMARY OF FIELD EXPLORATION

	SEISMIC LINES			BORINGS			TEST PITS	
Year	Designation	Number of Lines	Total Coverage (Ft.)	Designation	Number of Borings	Total Footage	Designation	Number of Test Pits
1988	TI-9 to 15	7	4,525	USB-1 to 5	5	307	TP-101 to 110	10
1989	89-1 to 35	35	11,920	DH-1 to 20	20*	2,265	TP-121 to 147, DT-1 to 6	33
Total		42	16,445		25	2,572		43

*DH-16 was not drilled; DH-3A was added.

TABLE 5.2. SUMMARY OF LABORATORY TESTS

Year	Sample Source	Moisture Content ASTM D2216	Dry Density	Sieve Analysis ASTM D422	Sieve and Hydrometer ASTM D422	Atterberg Limits ASTM D4318	Compaction ASTM D698	Specific Gravity ASTM D854	UU Triaxial	CU Triaxial
1988	Test Pit	5		5	1	4	2	2		
	Boring	3	1	7		1				
1989	Test Pit	22		14	6	10	3	3		3**
	Density Test Pit			5	1	3				
	Boring	16	6	25	7	15			5	5
Total		46	7	56	15	33	5	5	5	8

**Recompacted samples.

CHAPTER 6
SITE AND SUBSURFACE CONDITIONS

6.1 SITE CONDITIONS

The site is located in a valley that drains northeast into Libby Creek. The highest point in the watershed is at El. 5,400. Portions of the site are clear-cut and others are heavily timbered. Access to the site is via Forest Service roads, about 12 miles from U. S. Route 2. In August 1989, a 1-mile road was cleared in the south portion of the tailings retention damsite just above the proposed diversion channel.

As shown on Figure 4, isolated outcrops of highly weathered, fractured and jointed meta-sedimentary rock are located on the north faces of ridges, north and south of Little Cherry Creek. Elsewhere, the ridges are covered with colluvium and glacial outwash. Most of Little Cherry Creek is 50 feet or more above bedrock. Near the proposed seepage collection damsite, the creek has cut through the surficial material, exposing less weathered meta-sedimentary rock.

At the proposed damsite, the valley walls and creek channel are covered with colluvium, glacial outwash and lake bed sediments. The colluvium overlies the higher slopes of the valley and consists of coarse slopewash material. The glacial outwash is gravelly and contains a major clay and silt fraction. The lake bed sediments are mostly fine-grained clayey silt and varved clays with some gravel. These lake bed soils were reported to be below about El. 3,550, which is the highest level that lake bed sediments have been reported (Chen-Northern, 1989). Within the impoundment, colluvium and glacial outwash may overlie these sediments, indicating a later period of deposition.

Groundwater is transmitted through the more pervious layers of colluvial and glacial outwash and also through fractures in bedrock. Where groundwater is confined by the relatively impervious sediments, aquifers under artesian pressure result. Springs were observed during the 1989 field investigation and occur in areas of the proposed main dam and west of the north saddle dam, as shown on Figure 4.

6.2 SUBSURFACE CONDITIONS

A. General

The exploration plan showing both the 1988 and 1989 field programs is presented on Figure 4. Contour maps of the bedrock surface, soils isopach and piezometric surface are presented on Figure 5, 6 and 7, respectively. Geologic sections are shown on Figures 8 through 15. The sections show generalized soil types, bedrock profiles, permeability data, groundwater levels and seismic velocities. The information shown on the contour maps and geologic sections (Figures 5 through 15) was interpreted from the results of the exploration. Actual locations of bedrock and piezometric levels could vary from those indicated on the figures.

B. Bedrock

Bedrock observed in the borings generally consists of low-grade metamorphosed siltstone (siltite) and argillite. In DH-4, DH-10 and DH-13, the bedrock also contains interbedded quartzite. In general, the quartzite is hard, whereas the siltstone and argillite vary from hard to soft. The siltite and argillite are easily broken along thin, brown and black laminae. Bedrock cores obtained from the borings are generally moderately to highly weathered, to a soil-like texture in places. Some cores from Borings DH-10 and DH-13 are only slightly weathered. The weathered and fractured upper section of the bedrock in many cores did not exhibit a clear contact with the overlying dense, bouldery soils.

The rock is iron- and manganese-oxide stained. Between the proposed diversion dam and right abutment of the main dam, occasional vugs, generally less than 1/3 inch, though as large as 1-1/2 inches, were noted in Borings DH-6, DH-7 and DH-8.

Planar fractures with smooth to slightly rough surfaces occur throughout the meta-sedimentary rock. Fracturing observed in the cores was usually moderate (3 to 5 fractures per foot) to intense (more than 10 fractures per foot), but some zones were slightly fractured (less than 1 fracture per foot). Fractures are both parallel and oblique to bedding. However, within the siltite and argillite, fractures most commonly occur along laminae. Some fractures intermittently contain thin talc, chlorite, silt or soft clay filling. Slickensided fractures were noted in many cores. Core recovery averaged 70%, but frequently was 100%. The rock quality is very poor to fair (generally with Rock Quality Designations of 0% to 30%), reflecting the weathered and soft condition of the rock.

With borehole, test pit, and outcrop data as controls, an approximate bedrock surface was estimated, as shown on the bedrock contour map (Figure 5). The bedrock surface tends to be reflected by the topography, with the creek location in the west approximating the pre-glacial channel. However, in the eastern part of the site, a deep bedrock channel extends through the proposed damsite. Its existence, first suspected based on the results of seismic refraction survey line TI-10, was confirmed by Boring DH-3, where bedrock is deeper than 367 feet. Boring DH-20 further delineated the channel by reaching bedrock in the northern flank of the channel at 205 feet below ground level.

The results of the field explorations indicate that bedrock occurs at shallow depths (less than 50 feet) beneath both abutments of the proposed tailings retention dam. Except in Well LCM-9, bedrock was not reported in the borings drilled in the valley bottom during the 1988 field investigation, although bedrock may have been reached in Well LCTM-8 at about 114 feet. In the 1989 exploration, in the main dam area, bedrock was encountered in all borings, except in DH-3. Bedrock was encountered in the proposed diversion channel location at depths varying from 3 to 41 feet. Rock was noted at greater depths just northwest of the diversion damsite (180 feet in DH-11) and at the north saddle dam (145 feet in DH-5). Rock conditions in these areas resemble those in other boreholes: poor to fair quality, moderately to intensely fractured and weathered.

C. Soils

1. **Soil Distribution** - Primarily dense silty sandy and gravelly soils and stiff silty clayey soils were observed in the borings and test pits over most of the site (see Figures 8 to 15). The various soil types are intermixed with each other; the soil gradations change both laterally and with depth. In the tailings retention dam foundation, zones of sandy clayey silty materials were found. Boulders and cobbles in a matrix of finer soils were encountered throughout the site such as in Borings USB-2, USB-4, DH-3 and DH-20, and in Wells LCTM-8 and LCM-11. The amount of boulders in the test pits was visually estimated to range from about 5 to 10% of the soil volume.

The estimated soil thicknesses in the site are shown on the isopach map (Figure 6). Over most of the impoundment site, the soil is less than 100 to 150 feet thick. In the bedrock channel, however, the soil thickness is greater than 367 feet at Boring DH-3. Soils in the channel consist of clayey sandy silt with cobbles and boulders. The results of the exploration indicate that along most of the proposed diversion channel and main dam abutments, soils are generally less than 50 feet thick. Through part of the proposed seepage collection damsite, bedrock is exposed in Little Cherry Creek.

The results of the seismic refraction surveys show that soils have compressional wave (P-wave) velocities ranging from about 1,200 to 9,000 feet per second (fps). Near surface soils have lower velocities; higher velocities of deeper materials indicate increasing soil densities. Measured rock velocities range from about 7,000 to 15,000 fps, and reflect the degree of weathering. Highly weathered, soft rock would have velocities at the low end of this range.

2. **Soil Properties** - Laboratory test results of samples from the dam foundation and impoundment site indicate that the fines content ranges widely (see Figures F-1 to F-3, Appendix F). Sandy gravel with 7% silt was found in Test Pit TP-105. This test pit was probably located in a fluvial outwash deposit. The fine portions of the samples generally classify as low to medium plasticity clays, with liquid limits between 20 and 42; plasticity indices generally are between 3 and 22 (see Figure F-4, Appendix F).

Natural moisture contents of samples vary widely, from 2 to 34%. Figure F-5, Appendix F, shows sample moisture contents plotted against fines contents. The figure shows a general trend of increasing moisture with increasing fines content. The moisture content (34%) of a silty clay sample from Test Pit TP-124 was determined to be close to the liquid limit (33%) indicating that this soil would have very little strength in a remolded condition. However, this sample was the only one observed with a moisture content nearly equal to the liquid limit.

At the proposed damsite, six 5- to 6-foot deep test pits (DT-1 to DT-6) were excavated to determine the in situ density of the foundation material. Soils encountered in these test pits consisted of gravelly sandy clayey silt. Seven field density tests were performed by nuclear densometer (ASTM D2922) and sand cone (ASTM D1556) methods . Results of these tests show that moisture contents and dry densities range from 12 to 24% and 95 to 124 pcf, respectively, averaging 16% and 113 pcf.

The results of 5 moisture-density tests (ASTM D698) on clayey and silty soils show maximum dry densities between 117 and 126 pounds per cubic foot (pcf) and optimum moisture contents between 9 and 13%. Due to the wide range of natural moisture contents, moisture conditioning of the soils will be needed for embankment dam construction.

Five unconsolidated undrained (UU) triaxial compression tests were performed on undisturbed clayey soil samples from the foundation of the tailings dam. The test results indicate that the undrained shear strength ranges from 1200 to 3300 pounds per square foot (psf) (see Figure F-6, Appendix F). These results are plotted against the natural moisture contents (Figure F-7, Appendix F). As expected, the figure shows that the lowest strength was for the sample with the highest water content. Insufficient data exist to indicate any further correlation between undrained strength and moisture content.

The undrained shear strength and plasticity index test data can be used to determine whether the clayey soil samples are over-consolidated. For normally-consolidated clays, ratios of undrained shear strength to effective overburden pressure (c/p') can be correlated with plasticity index (P.I.) values by the following empirical relation (Terzaghi and Peck, 1967):

$$c/p' = 0.11 + 0.0037(P.I.) \qquad (1)$$

Using Equation (1) for the maximum P.I. value of 22, c/p'= 0.19. Figure F-7 shows that the c/p' values for selected soil samples within 23 feet of the ground surface range from 0.6 to 4.4 and exceed the c/p' ratio for normally-consolidated clays. Therefore, the test results indicate that the soil samples are over-consolidated. Over-consolidation could have resulted from desiccation or from overburden that was subsequently eroded.

Five consolidated undrained (CU) triaxial compression tests with pore pressure measurements were performed on undisturbed clayey soil samples from the dam foundation. Effective-stress paths were plotted to obtain the strength parameters from these tests (Figure F-8A, Appendix F). A Mohr Envelope was plotted based on the effective stress path results (Figure F-8B, Appendix F). The test results indicate that the effective angle of internal friction is 33 degrees. The test results also indicate an effective cohesion (c'). However, due to potential fissures in over-consolidated clayey soils, an effective cohesion should not be relied upon and, therefore, c'=0.

Three CU tests with pore pressure measurements were performed on sandy silty clay test pit samples from the proposed borrow area in the upper part of the impoundment site. These samples were compacted to 95% of the maximum dry density and at the optimum moisture content as determined by ASTM D698. This density and moisture content would be specified for embankment dam construction. Stress-path plots and the related Mohr Envelope (Figures F-9A and F-9B, Appendix F) show that the effective cohesion is 290 psf and the effective angle of internal friction is 32 degrees.

D. Groundwater

Groundwater in the Little Cherry site is recharged primarily from the hillside in the west. The general flow direction in the impoundment site is from west to east, corresponding approximately to the site topography. Portions of this groundwater, which flow beneath the impoundment site, emerge as springs around the proposed tailings damsite and downstream near the confluence of Little Cherry and Libby Creeks (Chen-Northern, 1989).

A summary of 1989 piezometer readings is presented in Table 6.1. The piezometric levels in the damsite ranged from 34 feet below ground level (GL) to 11 feet (4.8 psi) above GL, except in Borings DH-4 and DH-13 in the right abutment, where groundwater was not observed. In the proposed diversion channel, the water level was 73 feet below GL in Boring DH-9 and 85 feet below GL in DH-10; water was not encountered in Borings DH-7 and DH-8. The water level was 140 feet below GL in Boring DH-5, near the proposed north saddle dam. In the impoundment site, the piezometric levels ranged from 50 feet below GL to 24 feet above GL (10.4 psi). In the left abutment of the proposed diversion dam, groundwater was measured at a depth of 105 feet in DH-11. Seepage was observed at depths of 4.5 to 11 feet in six test pits during excavation: TP-101,-102,-104,-110,-137 and -144.

Based on the results of field investigations, groundwater at the proposed impoundment site occurs (1) as perched water in the soil deposits, and (2) as a more extensive aquifer in bedrock and the overlying soils, which is subject to varying degrees of confinement. Figure 7 shows the piezometric surface contours derived from water level measurements from the piezometers. Along Little Cherry Creek, the piezometric surface gradient averages about 4% to the northeast.

The results of the field permeability tests are summarized in Table 6.2. As indicated in the table, the highest permeabilities occur in the sand and gravel with low fines content. Sand and gravel permeability values typically were measured to range from $2x10^{-6}$ to $2x10^{-3}$ cm/sec and average about 10^{-5} to 10^{-4} cm/sec. The permeabilities in the weathered rock generally were measured to range from less than 10^{-6} to $2x10^{-4}$ cm/sec. Both of these materials are water bearing. In the clayey sandy silty soils, which have high fines content, measured permeabilities are generally less than $7x10^{-6}$ cm/sec. These silty soils act as confining zones where they overlie more pervious soils or rock. Where silty soils underlie more pervious soils, perched conditions exist.

In the north and south abutments and in the diversion channel site, where surficial deposits are thin and bedrock is near the ground surface, the piezometric levels generally are below the soil-bedrock contact. The piezometric levels in Borings DH-9, DH-10 and USB-3 are below this contact; groundwater was not encountered in the piezometers installed in bedrock at DH-4, 7, 8 and 13.

The main aquifer is generally confined. Artesian flow conditions were observed in some of the piezometers in the valley bottom, as in Borings DH-6 and DH-15, downstream from the diversion channel at PLCM-6, and in the proposed dam foundation at DH-2, DH-3, DH-12 and DH-19. Pressure heads as high as 22 to 24 feet above GL (9.5 to 10.4 psi) were measured in DH-6 (diversion dam area) and DH-15 (impoundment area) as reported by Noranda in November 1989. In the tailings retention dam area, pressure heads range from 2 feet (DH-3 and DH-12) to about 10 feet above GL (DH-2 and DH-19).

Perched water occurs throughout the soils of the Little Cherry site, as indicated by decreasing water levels during drilling, as noted in DH-11 and DH-12. Perched water conditions are attributed to the interfingering of relatively impervious clayey silt within more pervious sediments.

TABLE 6.1

SUMMARY OF BORING AND WELL DATA[a]

Boring No.	Auger Depth (ft)	Total Depth (ft)	Depth to Bedrock (ft)	Depth to Static Water (ft)[b]	Screen Interval (ft)	Sand Backfill Interval (ft)[c]	Sand Backfill Interval in Soil (S) or Rock (R)	Elevation of Ground Surface
DH-1	10	100	77	20	80 - 100	75 to BH	R/S	3445.6
DH-2	10	110	54	9 AGL	90 - 110	78 to BH	R	3414.4
DH-3	0	367	-	2 AGL	140 - 160 223 - 243	130 to 180 213 to 320	S	3435.4
DH-3A	25	25	-	Dry	-	-	-	3435±
DH-4	0	40	9	Dry	20 - 40	21 to BH	R	3609.5
DH-5	5	197	145	140	-	-	-	3674.4
DH-6	5	60	40	22 AGL	40 - 60	36 to BH	R/S	3593.7
DH-7	44	73	34	Dry	53 - 73	47 to BH	R	3769.3
DH-8	15	98	15	Dry	78 - 98	73 to BH	R	3745.9
DH-9	0	82	41	73	62 - 82	51 to BH	R	3677.7
DH-10	0	87	18	85	67 - 87	57 to BH	R	3657.5
DH-11	15	192	180	105	170 - 190	165 to 190	R/S	3760.4
DH-12	5.5	110	65	2 AGL	90 - 110	84 to BH	R	3393.2

(a) All depths are measured from ground level.

(b) AGL = above ground level; water levels were measured between Sept. and Nov. 1989, except water levels in USB-3 and USB-4, which were measured at the time of drilling (July and August 1988).

(c) BH = Bottom of Hole.

TABLE 6.1

SUMMARY OF BORING AND WELL DATA[(a)]

(Continued)

Boring No.	Auger Depth (ft)	Total Depth (ft)	Depth to Bedrock (ft)	Depth to Static Water (ft)[(b)]	Screen Interval (ft)	Sand Backfill Interval (ft)[(c)]	Sand Backfill Interval in Soil (S) or Rock (R)	Elevation of Ground Surface
DH-13	6.5	80	16	Dry	60 - 80	56 to BH	R	3682.5
DH-14	0	60	30	17	40 - 60	35.5 to BH	R	3574.7
DH-15	0	120	-	24 AGL	93 - 113	80 to 113	S	3440.5
DH-17	26	80	65	23	60 - 80	59 to BH	R/S	3541.8
DH-18	4.5	65	-	50	45 - 65	40 to BH	S	3659.2
DH-19	0	105	60	11 AGL	85 - 105	76 to BH	R	3413.4
DH-20	25	214	205	9	144 - 154	136 to 154	S	3428.4
USB-1	0	81	-	22	61 - 81	49 to BH	S	3462.6
USB-2	18.5	81	-	0	61 - 81	55 to BH	S	3440.9
USB-3	35	70	27	34	-	-	-	3580.7
USB-4	0	25	-	Dry	-	-	-	3646.6
USB-5	0	50	-	26	30 - 50	25 to BH	S	3516.0
LCTM-8	0	140	(?)	11	33 - 93	26 to 94	S	3380.5
LCM-9	0	72	65	8	57 - 67	56 to 64	S	3420.6
LCM-10	0	40	-	7	25 - 40	23 to BH	S	3740.2
LCM-11	0	63	-	6	57 - 62	55 to BH	S	3756.2
PLCM-6	0	56	50	(A) 2	8 - 18	7 to 33	S	3445.1
				(B) 7 AGL	48 - 53	46 to 56	R/S	

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TABLE 6.2

SUMMARY OF FIELD PERMEABILITY TEST RESULTS

Boring or Well	Material	Type of Test	Permeability (10^{-6} cm/sec)
USB-3	Weathered rock	Packer Test	<1 to 29
USB-4	Silty sandy gravel, cobbles and boulders	Constant Head	100 to 1800
USB-5	Gravelly silty sand	Constant Head	14 to 70
	Silty clay-clayey silt	Constant Head	3 to 7
LCTM-8*	Silty sandy gravel, cobbles and boulders	Pump Test	130
LCM-9*	Silty gravel with boulders	Slug Test	90
LCM-10*	Gravel, silt and sand	Slug Test	2
LCM-11*	Silty sandy gravel, cobbles and boulders	Slug Test	24
PLCM-6* (shallow)	Silty sandy gravel	Slug Test	1900
PLCM-6* (deep)	Silty sandy gravel	Flow Test (Constant Drawdown)	4500 to 6900
DH-1	Gravelly sandy silt	Constant Head	<1 to 6
	Weathered rock	Packer Test	2
DH-2	Weathered rock	Packer Test	16 to 234
DH-4	Weathered rock	Packer Test	14 to 20
DH-5	Clayey sandy silt, cobbles and boulders	Constant Head	<1 to 28
	Weathered rock	Packer Test	<1 to 2

<u>Notes</u>:

* From Chen-Northern (1989)

TABLE 6.2

SUMMARY OF FIELD PERMEABILITY TEST RESULTS (CONTINUED)

Boring or Well	Material	Type of Test	Permeability (10^{-6} cm/sec)
DH-6	Clayey sandy silt, gravel and cobbles	Constant Head	<1
DH-7	Weathered rock	Packer Test	23 to 79
DH-8	Weathered rock	Packer Test	4 to 7
DH-9	Weathered rock	Packer Test	72 to 115
DH-10	Weathered rock	Packer Test	131 to 721
DH-11	Cobbles, gravel, sand, silt, and clay	Packer Test	7
	Weathered rock	Packer Test	2
DH-13	Weathered rock	Packer Test	2 to 146
DH-14	Weathered rock	Packer Test	12 to 46
DH-17	Cobbles, gravel, and clayey sandy silt	Constant Head	<1
	Weathered rock	Packer Test	<1
DH-18	Clayey sandy silt	Constant Head	<1 to 2
DH-19	Weathered rock	Packer Test	87 to 196
DH-20	Gravels, cobbles, and boulders in clayey sandy silt matrix	Constant Head	<1 to 2
	Weathered rock	Packer Test	<1 to 2

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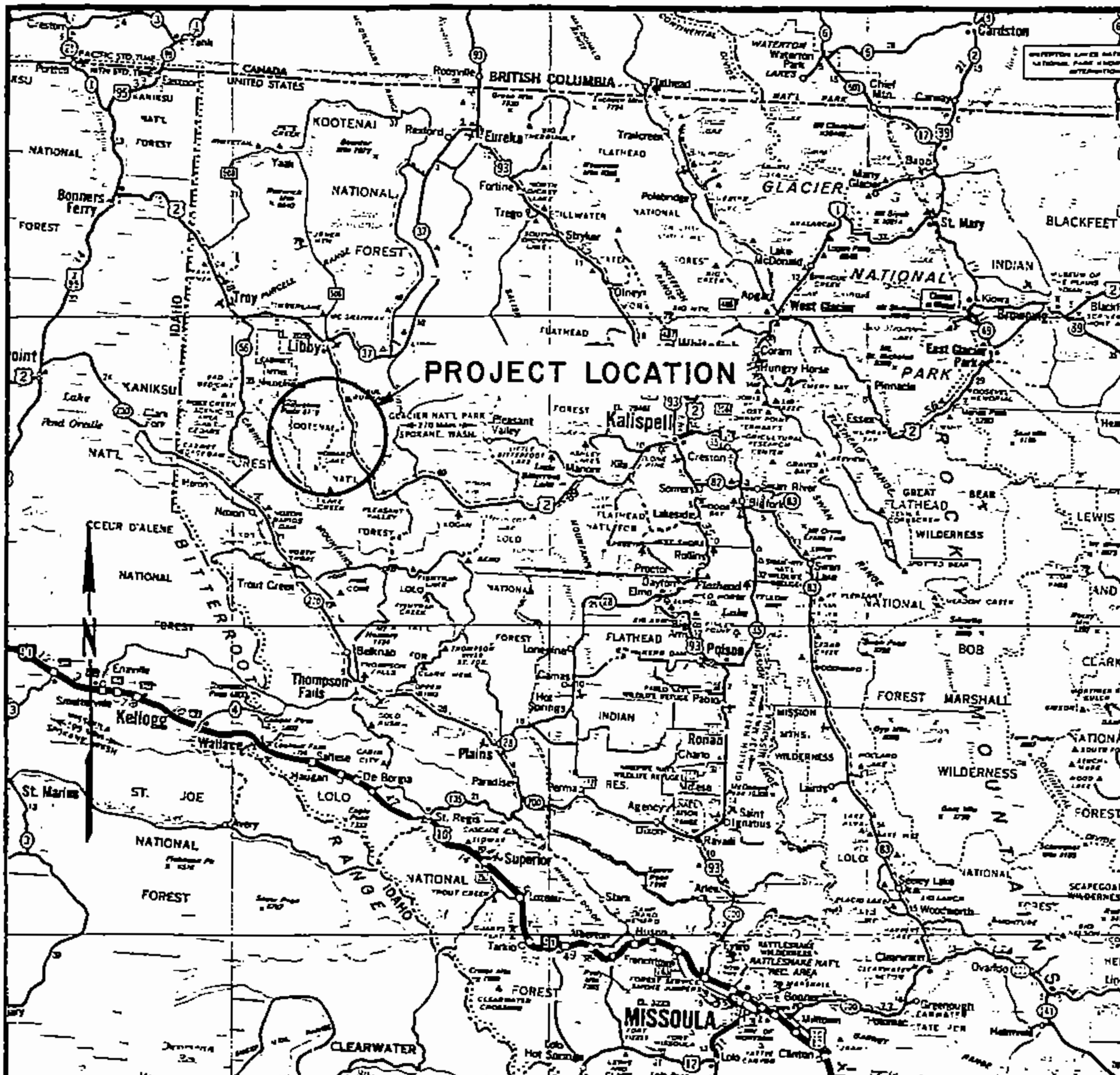
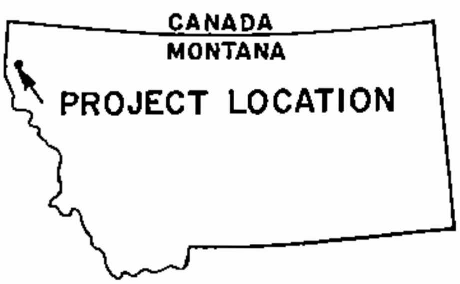




KEY PLAN

25 0 25 50

SCALE MILES

MORRISON-KNUDSEN ENGINEERS, INC.

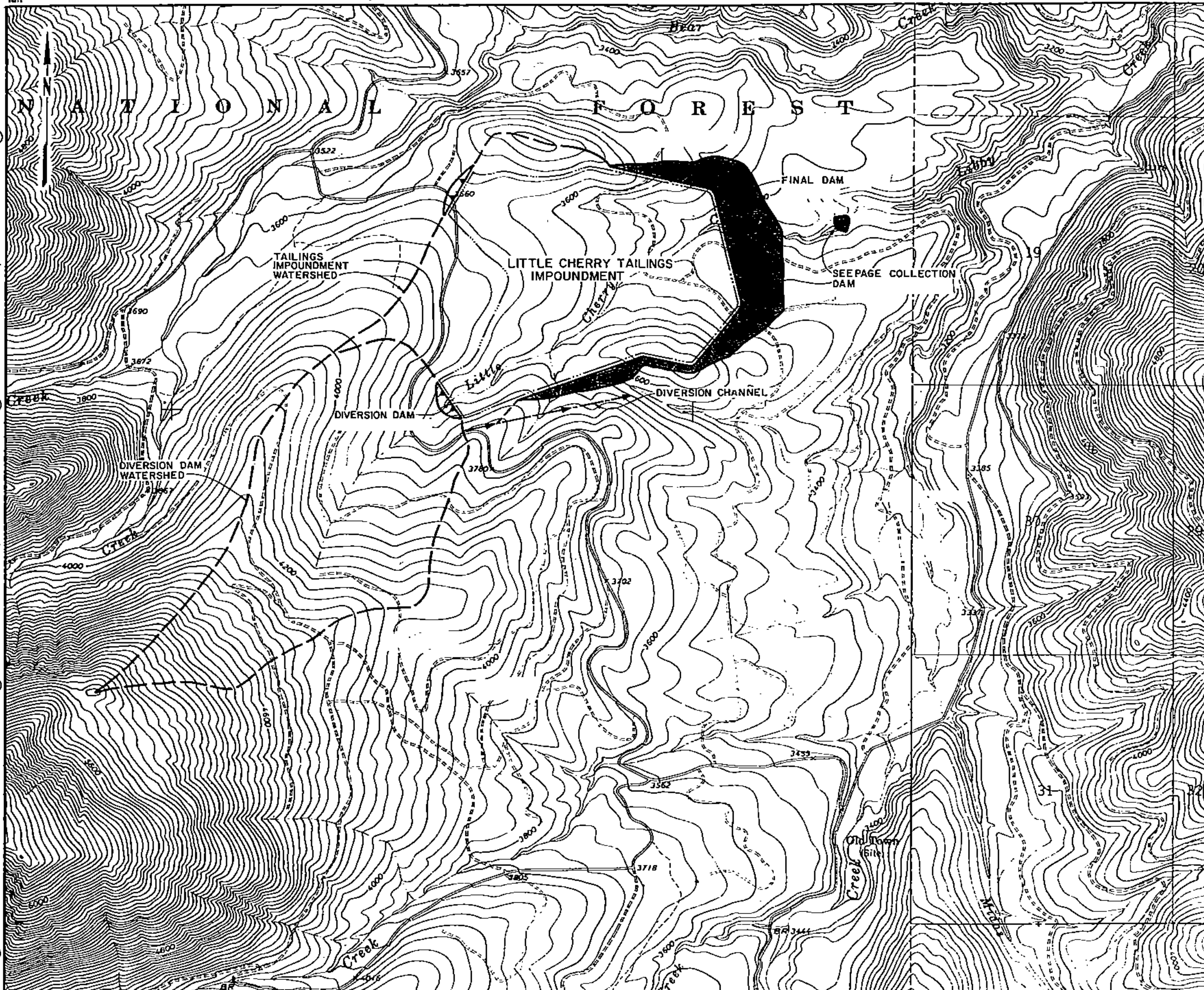
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED RB	DRAWN RB	CHECKED MPF	RECOMMENDED
DATE OCTOBER 1988			APPROVED

MONTANA PROJECT

PROJECT LOCATION MAP

FIGURE 1



REFERENCE:

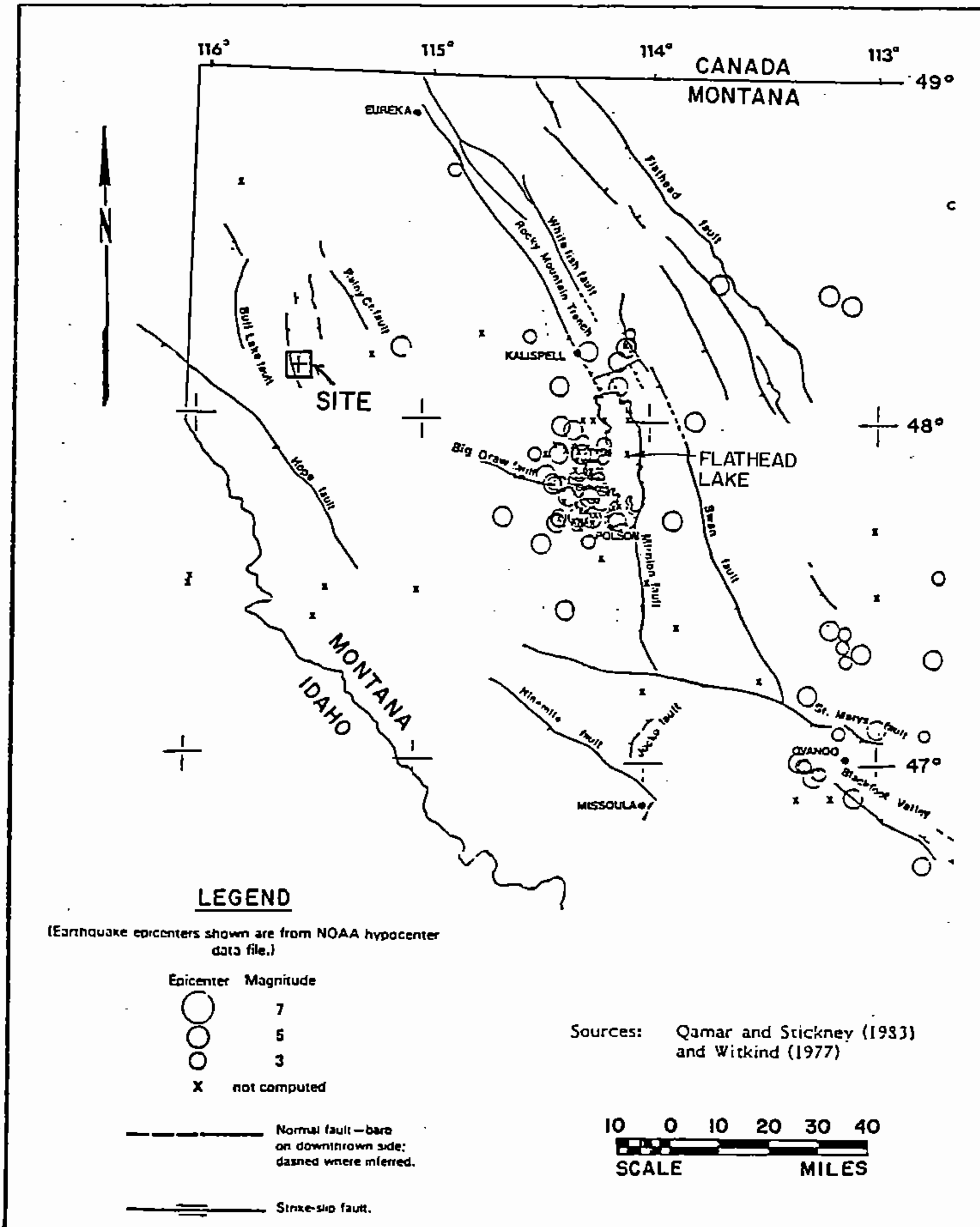
U.S. GEOLOGICAL SURVEY, 7.5 MINUTE CABLE MOUNTAIN TOPOGRAPHICAL QUADRANGLE

CONCEPTUAL-NOT FOR CONSTRUCTION

1000 0 1000 2000 SCALE FEET	MORRISON-KNUDSEN ENGINEERS, INC. 180 HOWARD STREET SAN FRANCISCO CALIFORNIA 94105 DESIGNED CFS DRAWN AMC CHECKED MPF RECOMMENDED DATE DECEMBER 1989 APPROVED	NORANDA MINERALS CORPORATION	MONTANA PROJECT VICINITY PLAN & WATERSHED AREAS

NO DATE REVISIONS BY CHK APP'D

FIGURE 2



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180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

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DATE OCTOBER 1988			APPROVED

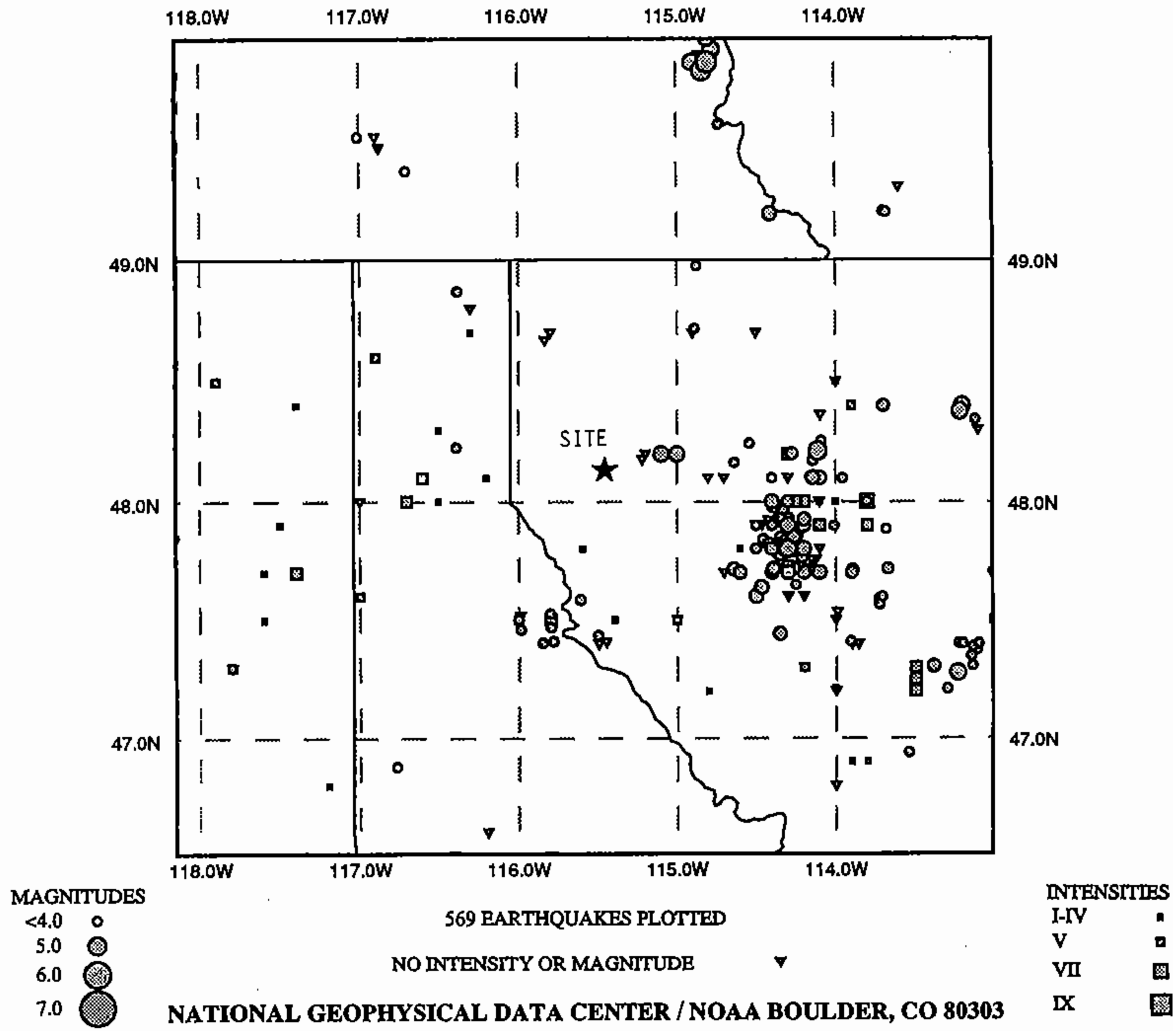
MONTANA PROJECT

HISTORICAL EARTHQUAKES IN MONTANA, 1869-1979

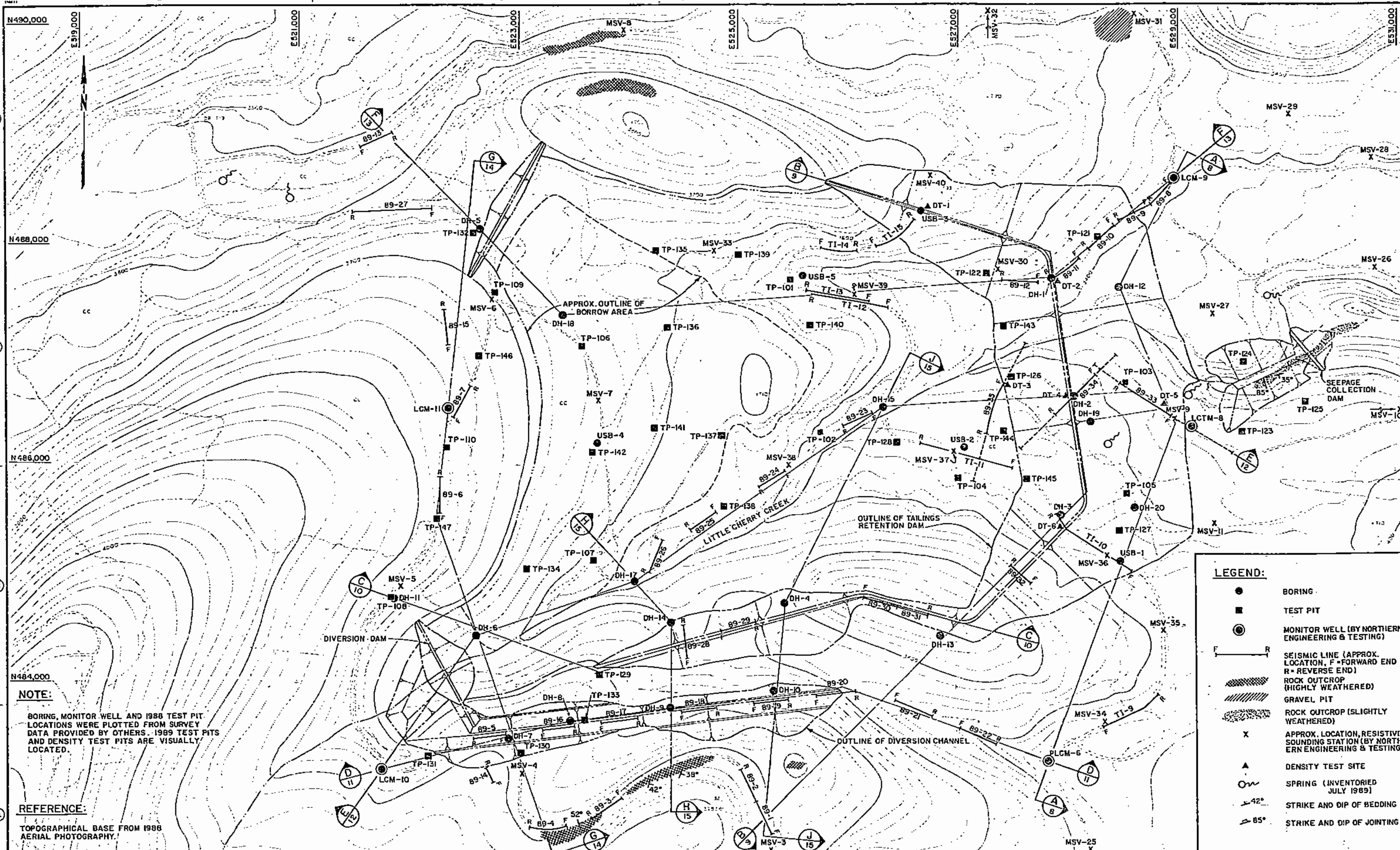
FIGURE 3A

FIGURE 3B

MONTANORE SEISMICITY



NATIONAL GEOPHYSICAL DATA CENTER / NOAA BOULDER, CO 80303



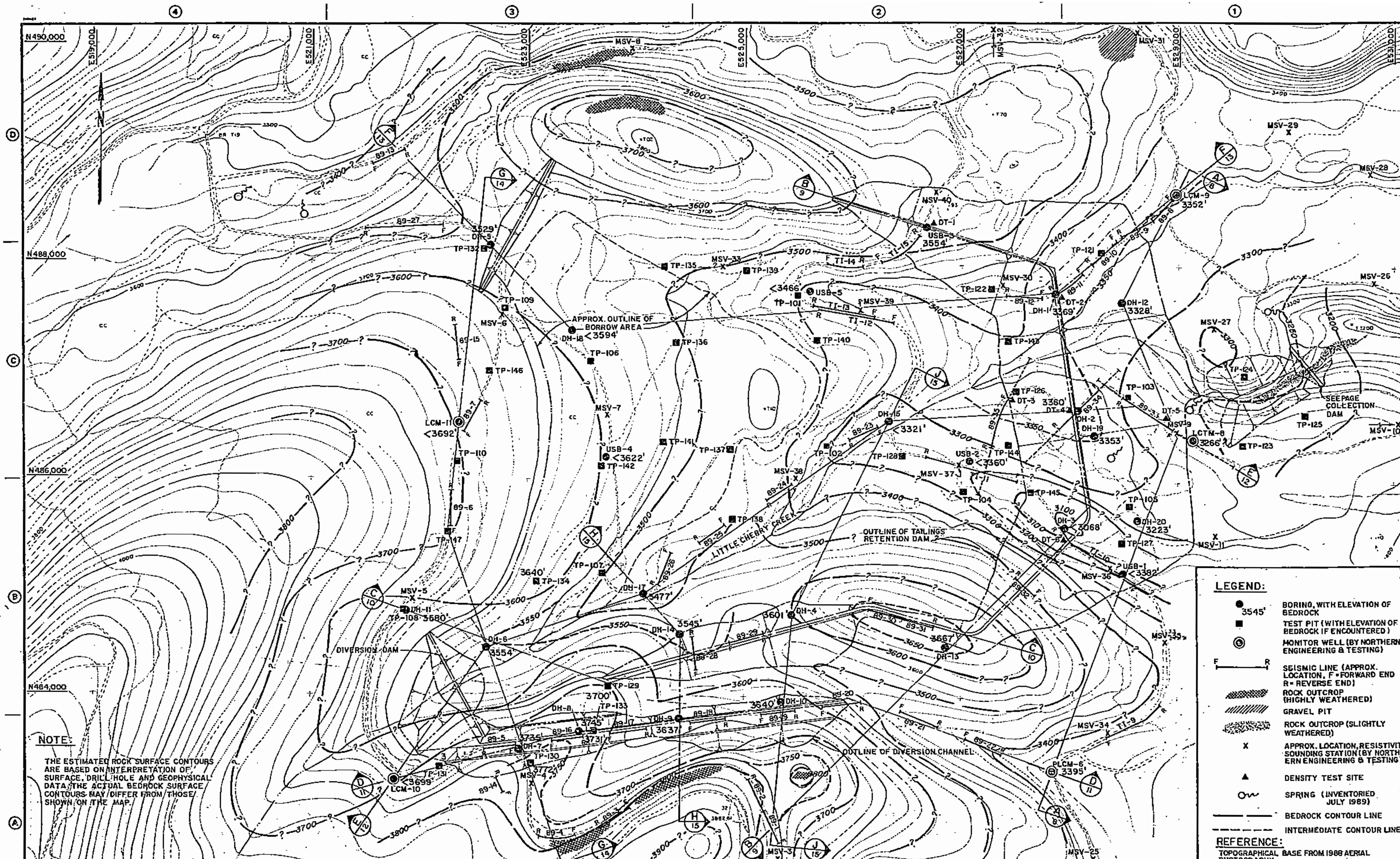
SCALE 400 0 400 800 FEET	
MORRISON-KNUDSEN ENGINEERS, INC. 180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105	NORANDA MINERALS CORPORATION
DESIGNED CFS · DRAWN RBC · CHECKED MPF, JTK · RECOMMENDED · APPROVED · DATE NOVEMBER 1989	
NO · DATE · REVISIONS · BY · CHK · APPD.	

MONTANA PROJECT

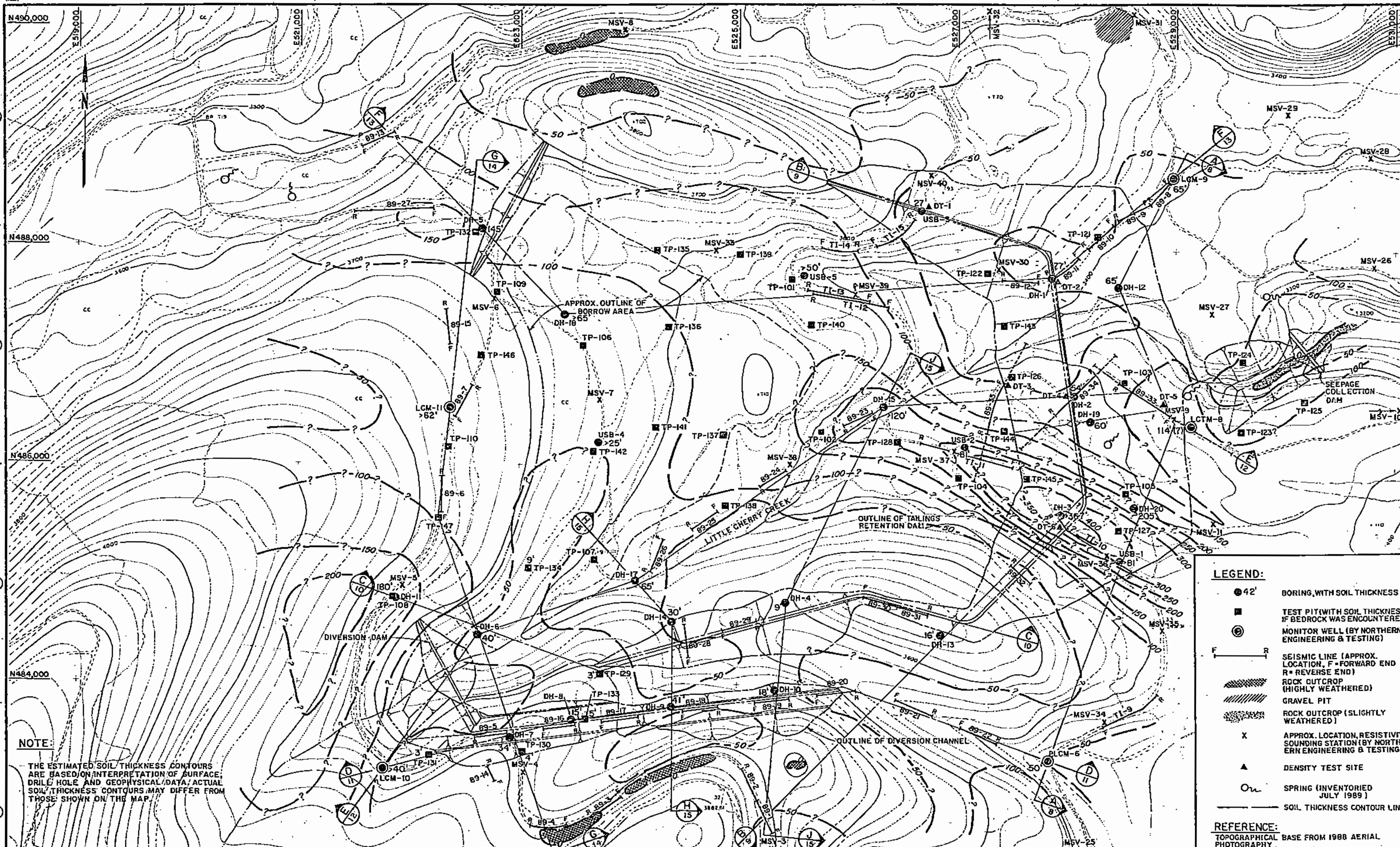
LITTLE CHERRY SITE

EXPLORATION PLAN

FIGURE 4



SCALE 400 0 400 800 FEET	
NO. DATE REVISIONS BY CHK. APPD.	
MORRISON-KNUDSEN ENGINEERS, INC. 180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105	DESIGNED CFS · DRAWN RBC · CHECKED PYL · RECOMMENDED · DATE NOVEMBER 1989 · APPROVED
NORANDA MINERALS CORPORATION	
MONTANA PROJECT	LITTLE CHERRY SITE BEDROCK CONTOUR MAP
FIGURE 5	SHEET OF REV.



NO.	DATE	REVISIONS	BY	CHK.	APPD.

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180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

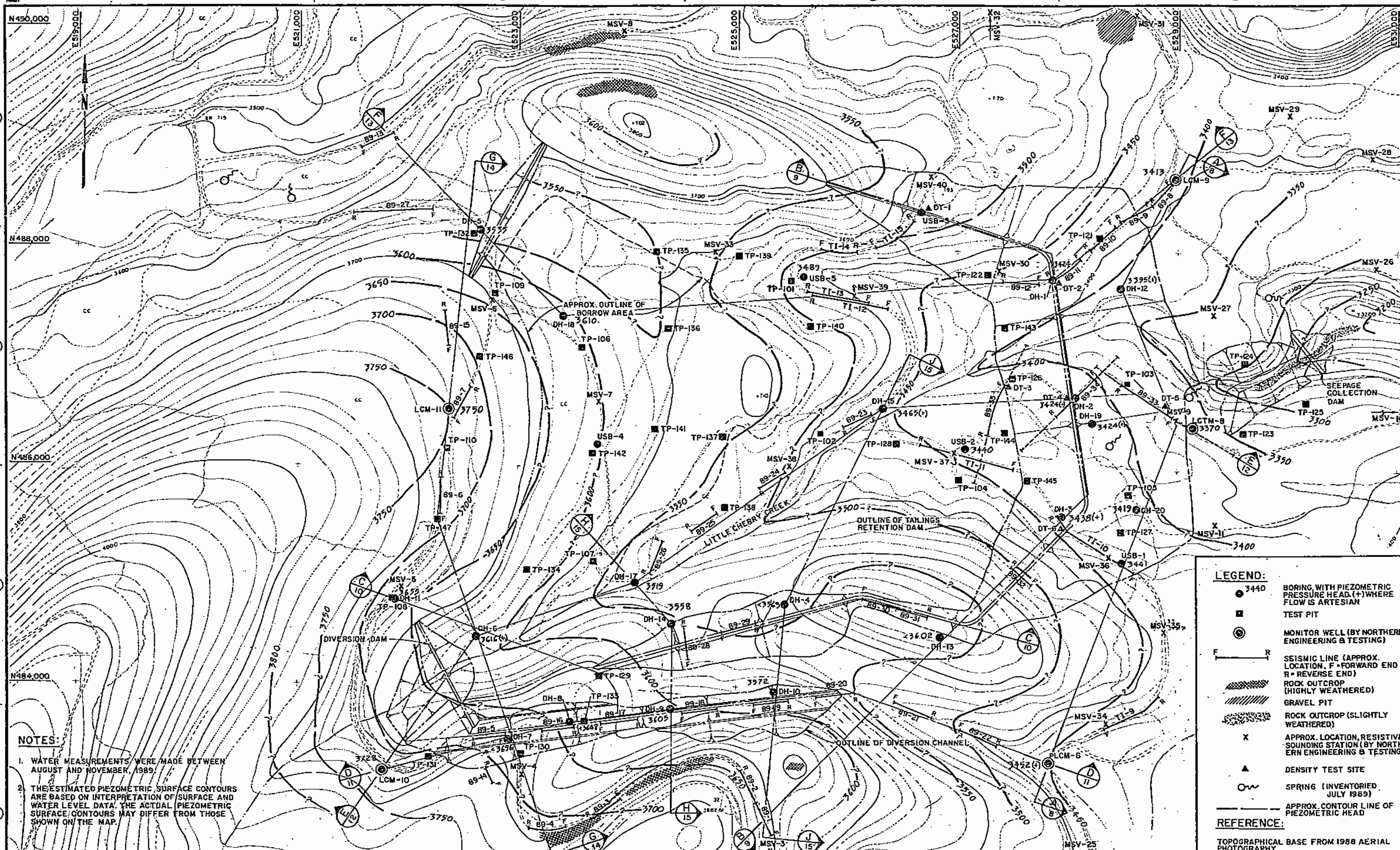
DESIGNED CFS | DRAWN RBC | CHECKED PYL | RECOMMENDED
DATE NOVEMBER 1989 | APPROVED

NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY SITE
SOILS ISOPACH MAP

FILE NO | SHEET OF | REV.

FIGURE 6



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NO.	DATE	REVISIONS	BY	CHK.	APPD.

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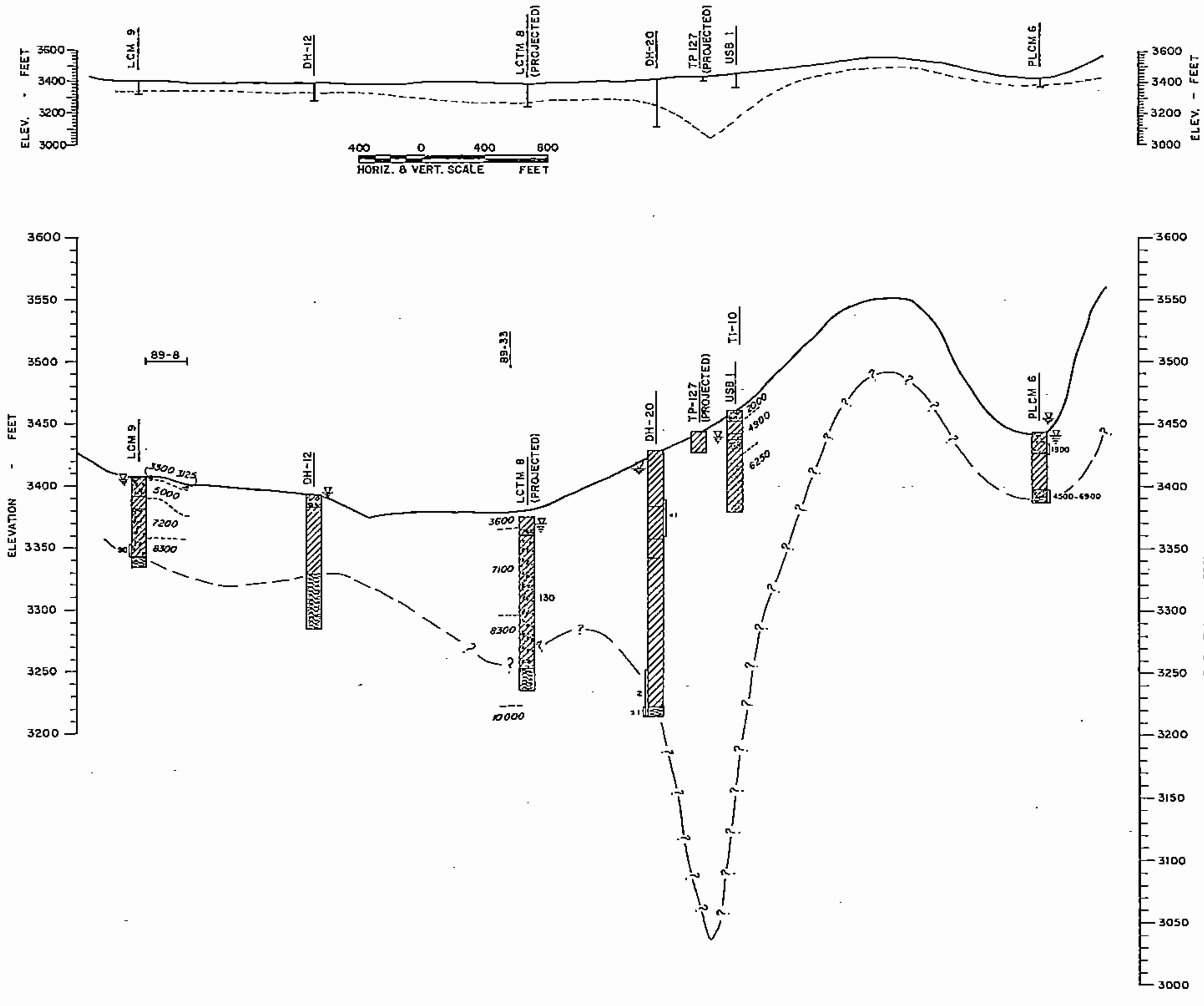
NORANDA MINERALS CORPORATION

MONTANA PROJECT

LITTLE CHERRY SITE
PIEZOMETRIC SURFACE MAP

FIGURE 7



SECTION (A) FIG. 4

50 0 50 100 VERT. SCALE FEET

400 0 400 800 HORIZ. SCALE FEET

NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

89-10 SEISMIC LINE LONGITUDINAL TO SECTION

89-11 SEISMIC LINE TRANSVERSE TO SECTION

DH-1 BORING

ML, ML-CL, CL*

PIEZOMETRIC SURFACE

SW, SP, GW, GP*

27 PERMEABILITY VALUES (NORMALIZED TO 10^{-6} CM/SEC)

ML-SM, SM*

GM, GM-GP*

COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)

WEATHERED BEDROCK

3500/4000 SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)

TP-101 TEST PIT

ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

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DATE NOV. 1989

NORANDA MINERALS CORPORATION

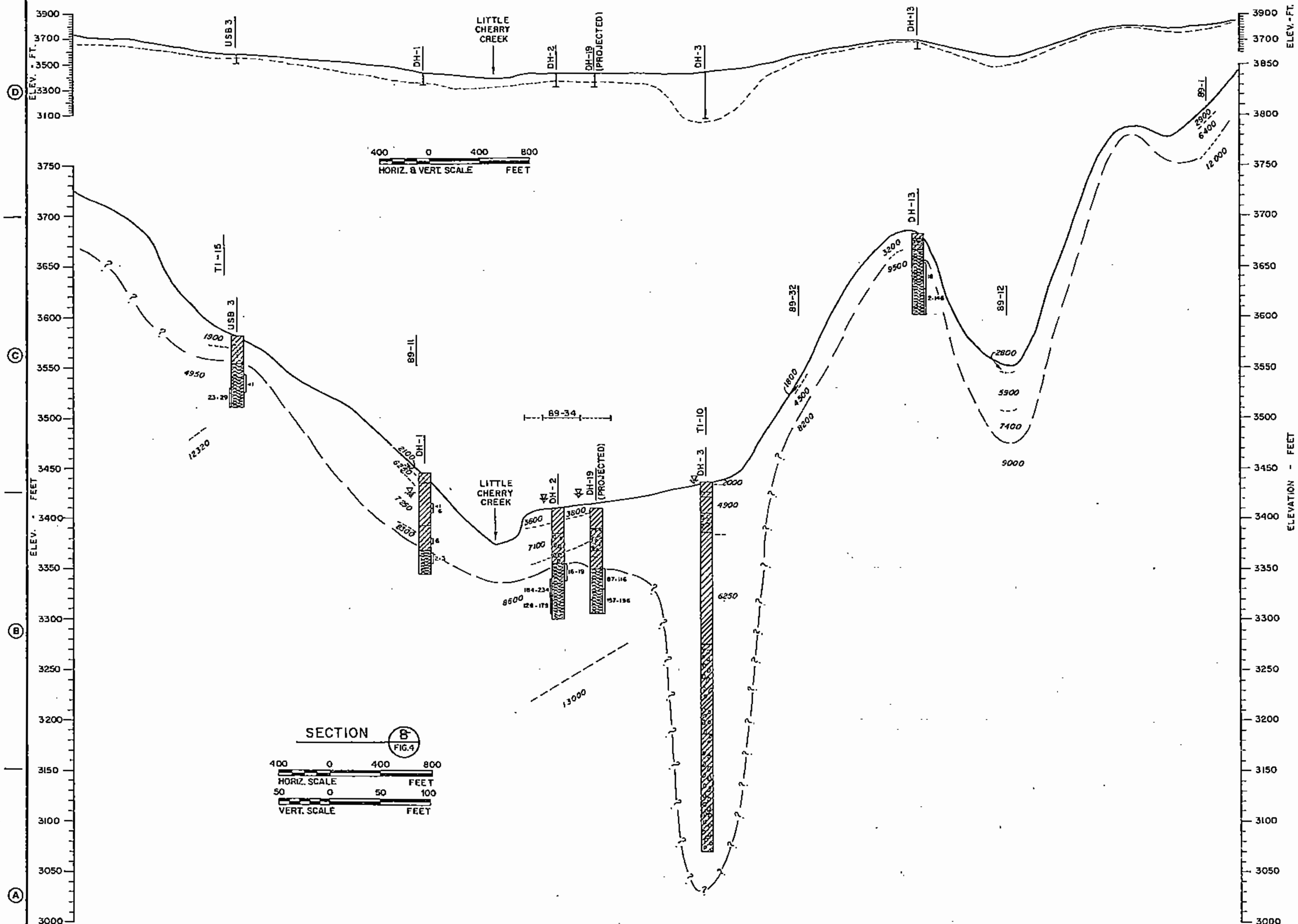
MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION A

MKE NO. | SHEET OF | REV.

FIGURE 8

NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.



LEGEND:

89-10 SEISMIC LINE LONGITUDINAL TO SECTION

89-11 SEISMIC LINE TRANSVERSE TO SECTION

DH-1 BORING

ML, ML-CL, CL *

PIEZOMETRIC SURFACE

SW, SP, GW, GP *

27 PERMEABILITY VALUES (NORMALIZED TO 10^{-6} CM/SEC)

ML-SM, SM *

GM, GM-GP *

COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)

WEATHERED BEDROCK

2300/4900 SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)

TP-10 TEST PIT

ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

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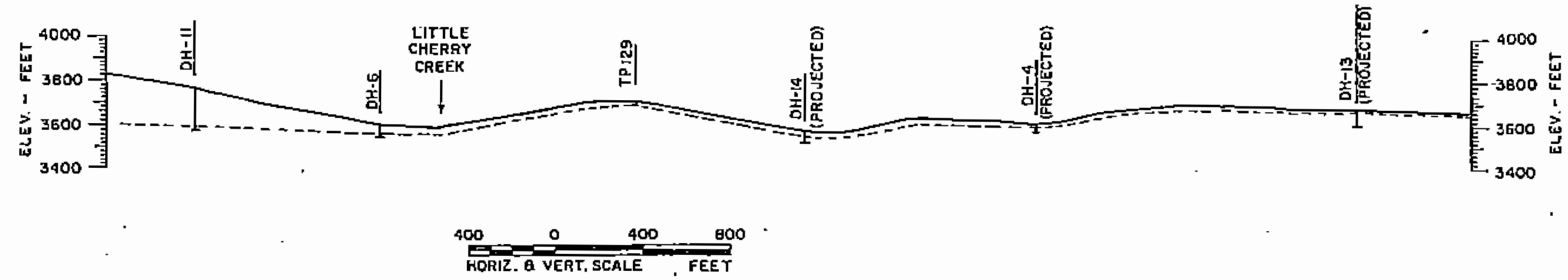
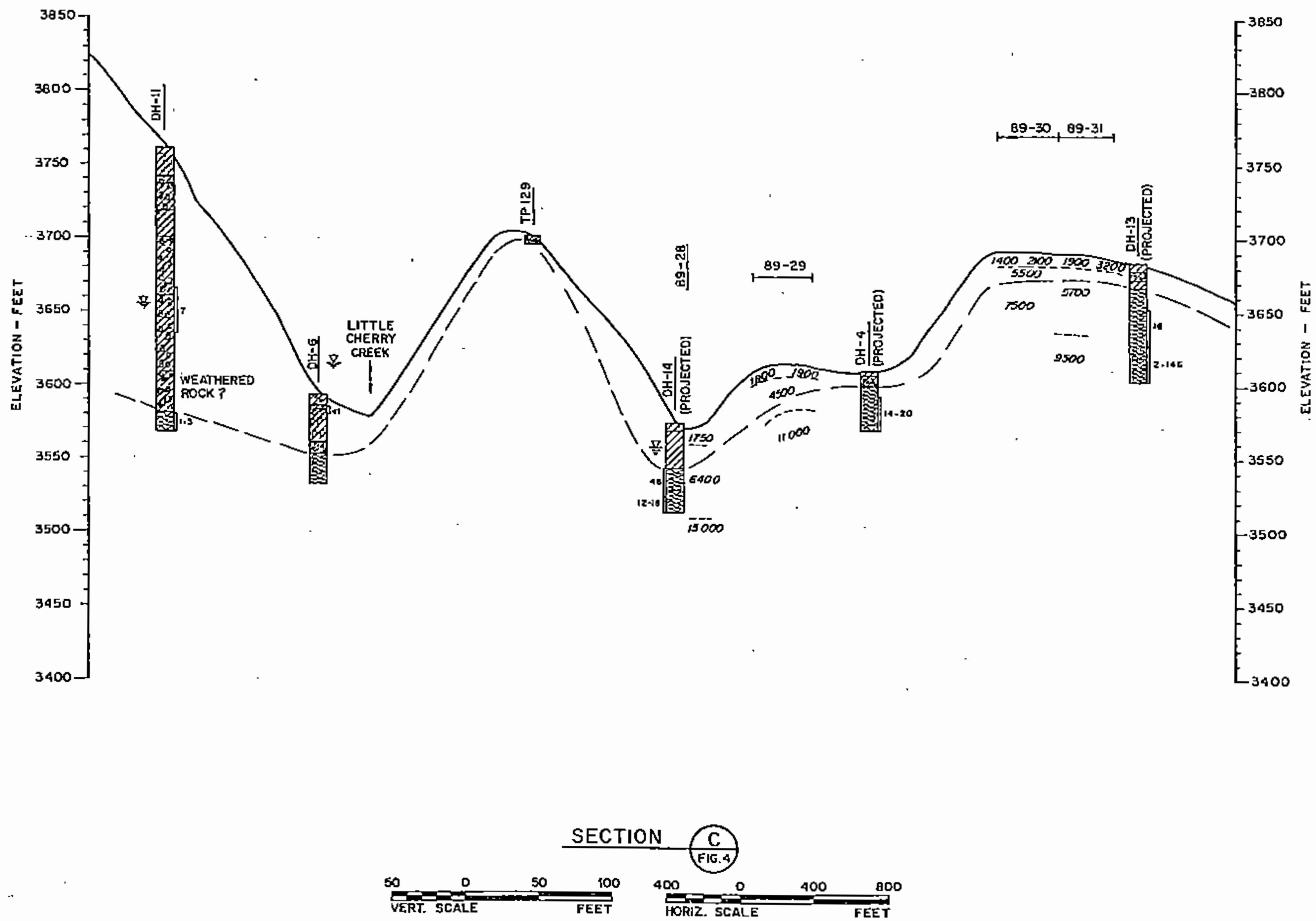
NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY SITE GEOLOGIC SECTION B

MKE NO. SHEET OF REV.

FIGURE 9



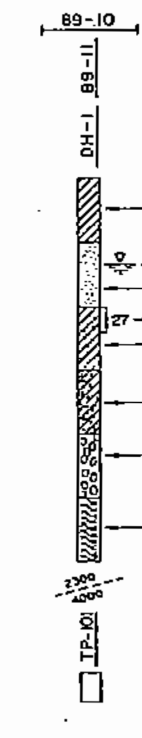


SECTION C (FIG. 4)

NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:



SEISMIC LINE LONGITUDINAL TO SECTION

SEISMIC LINE TRANSVERSE TO SECTION

BORING

ML, ML-CL, CL *

PIEZOMETRIC SURFACE

SW, SP, GW, GP *

PERMEABILITY VALUES (NORMALIZED TO 10^{-6} CM/SEC)

ML-SM, SM *

GM, GM-GP *

COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)

WEATHERED BEDROCK

SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)

TEST PIT

ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

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180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

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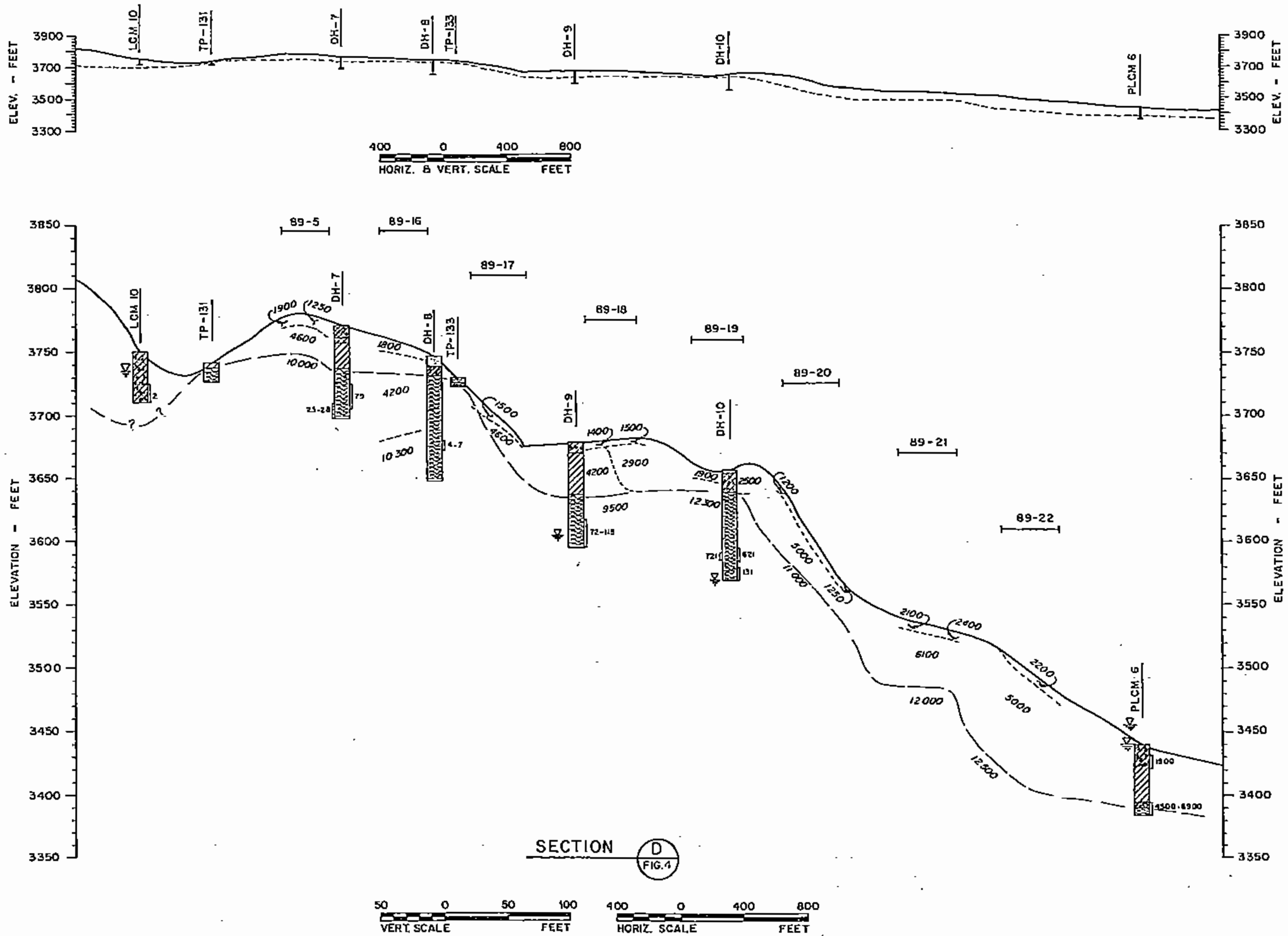
MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION C

MKE NO. | SHEET OF | REV.

FIGURE 10



NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

- 89-10 — SEISMIC LINE LONGITUDINAL TO SECTION
- 89-11 — SEISMIC LINE TRANSVERSE TO SECTION
- DH-1 — BORING
- ML, ML-CL, CL*
- PIEZOMETRIC SURFACE
- SW, SP, GW, GP*
- 27 — PERMEABILITY VALUES (NORMALIZED TO 10-6 CM/SEC)
- ML-SM, SM*
- GM, GM-GP*
- COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)
- WEATHERED BEDROCK
- 2300 / 4000 — SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)
- TP-101 — TEST PIT
- ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

NO.	DATE	REVISIONS	BY	CHK.	APPD.

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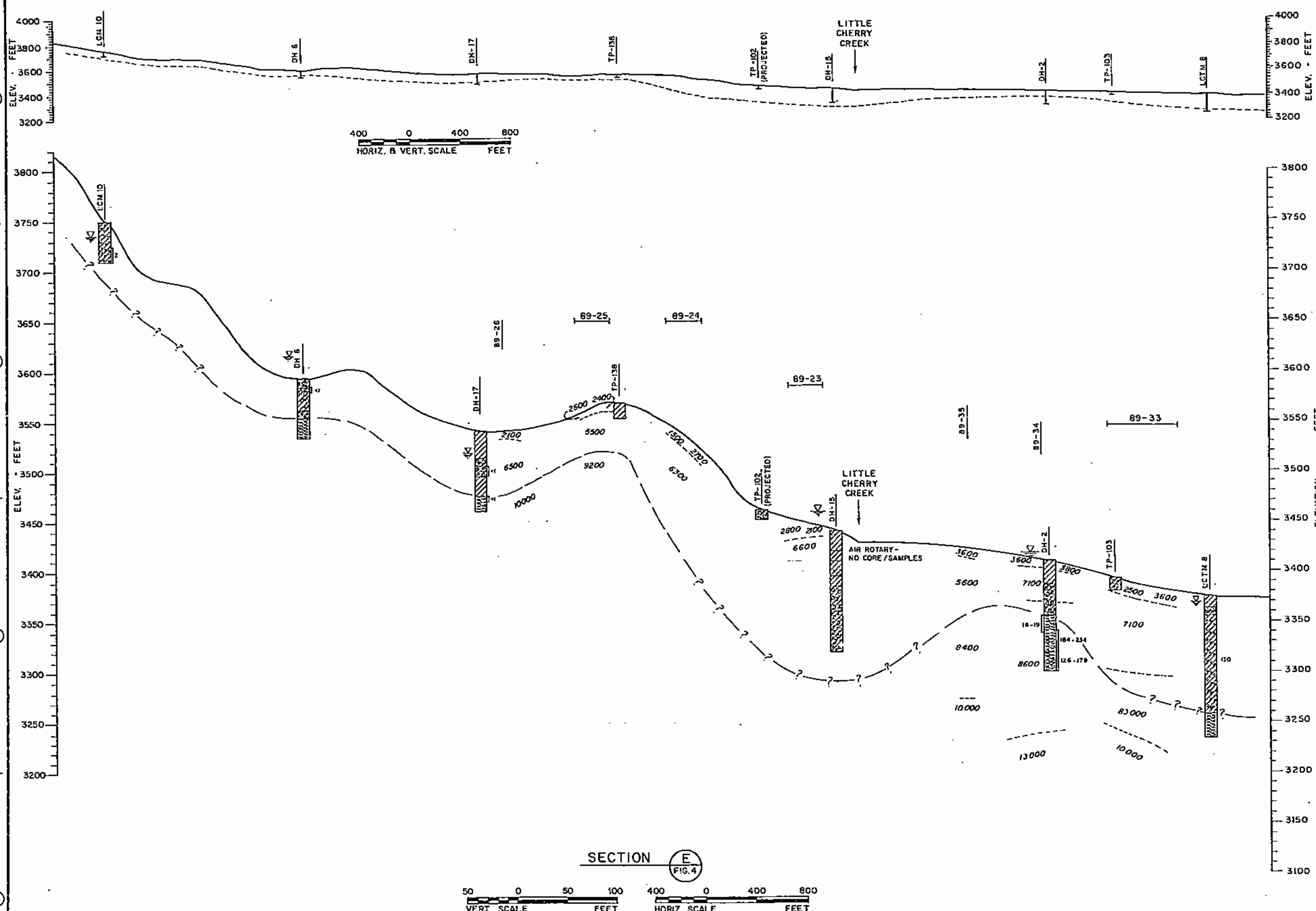
NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION D

MKE NO.
SHEET OF REV.
FIGURE 11



NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

89-10 SEISMIC LINE LONGITUDINAL TO SECTION

89-11 SEISMIC LINE TRANSVERSE TO SECTION

DH-1 BORING

- ML, ML-CL, CL*
- PIEZOMETRIC SURFACE
- SW, SP, GW, GP*
- 27 PERMEABILITY VALUES (NORMALIZED TO 10^{-4} CM/SEC)
- ML-SM, SM*
- GM, GM-GP*
- COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)
- WEATHERED BEDROCK

SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)

TP-101 TEST PIT

ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

NO.	DATE	REVISIONS	BY	CHK.	APPD.

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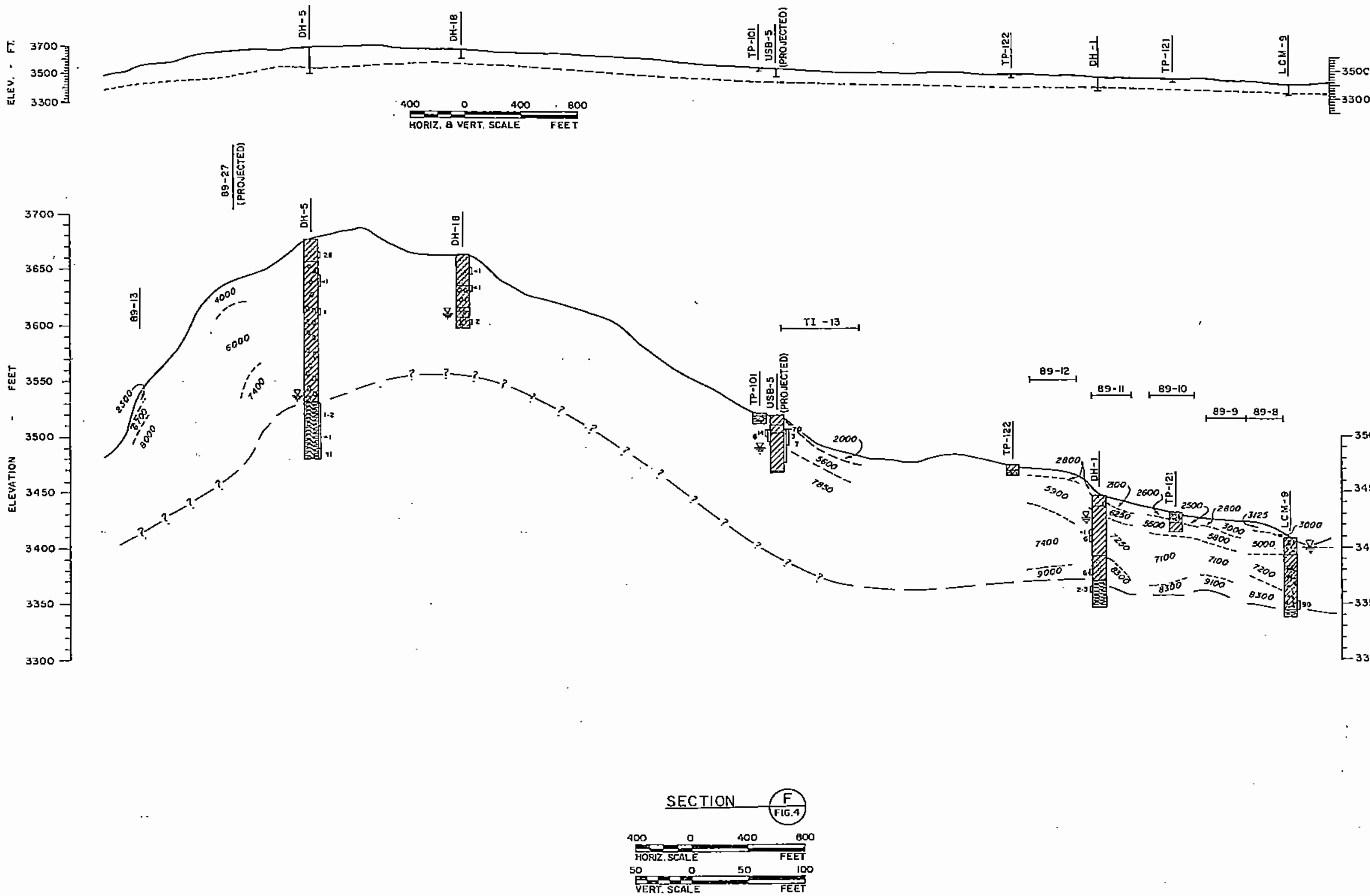
MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION E

MKE NO.
SHEET OF REV.
FIGURE 12



NOTES:

1. THE ESTIMATED BEDROCK SURFACE IS BASED ON INTERPRETATION OF DRILL HOLE AND GEOPHYSICAL DATA. THE ACTUAL BEDROCK SURFACE MAY DIFFER FROM THAT SHOWN ON THIS SECTION.
2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

- 89-10 — SEISMIC LINE LONGITUDINAL TO SECTION
- 89-11 — SEISMIC LINE TRANSVERSE TO SECTION
- DH-1 — BORING
- ML, ML-CL, CL *
- PIEZOMETRIC SURFACE
- SW, SP, GW, GP *
- 27 — PERMEABILITY VALUES (NORMALIZED TO 10-6 CM/SEC)
- ML-SM, SM *
- GM, GM-GP *
- COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)
- WEATHERED BEDROCK
- 3300 / 4000 — SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)
- TP-101 — TEST PIT
- ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

NO.	DATE	REVISIONS	BY	CHK.	APPD

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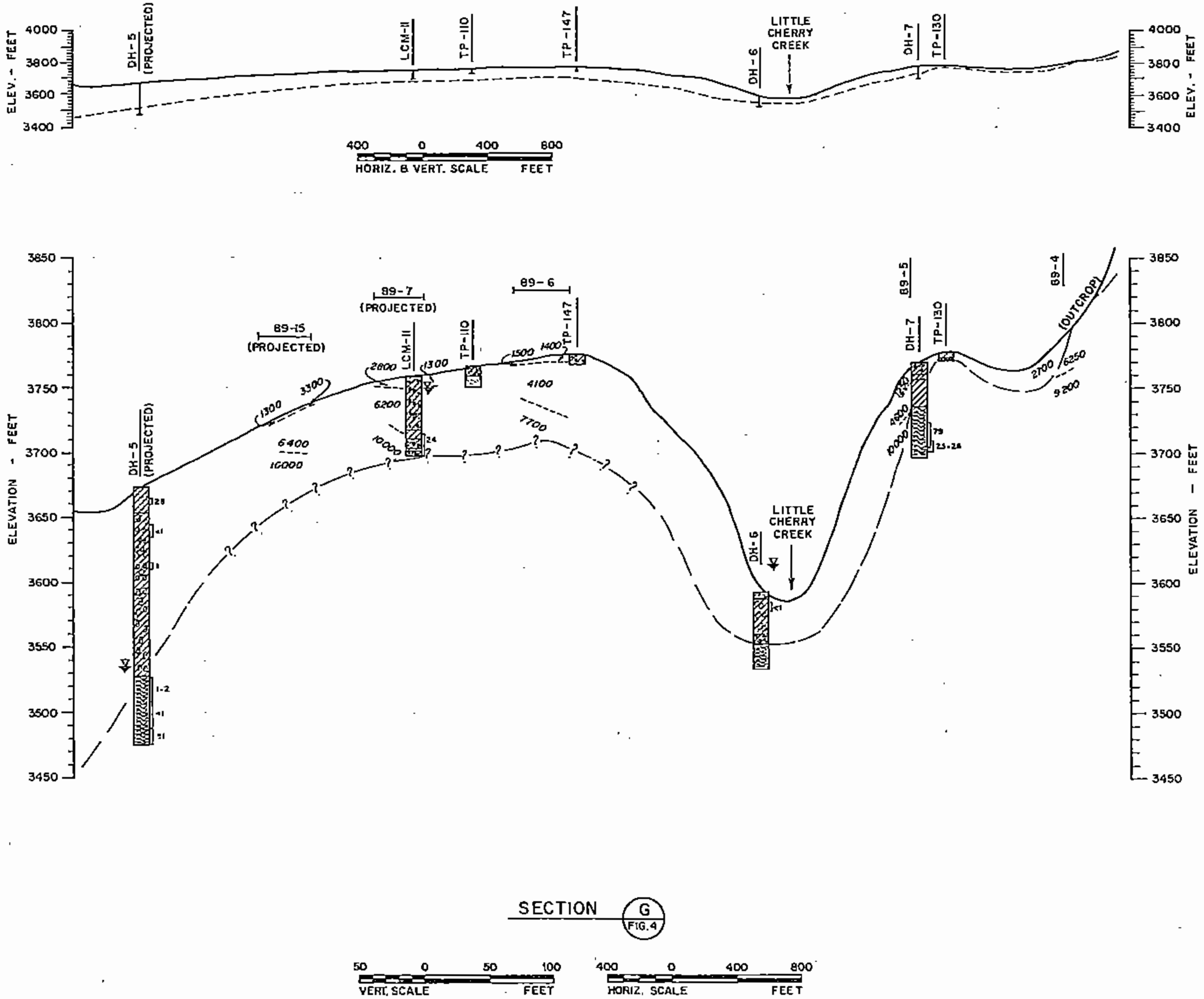
MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION F

FILE NO.
SHEET OF | REV.
FIGURE 13



NOTES:

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3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

- 89-10 — SEISMIC LINE LONGITUDINAL TO SECTION
- 89-11 — SEISMIC LINE TRANSVERSE TO SECTION
- DH-1 — BORING
- ML, ML-CL, CL*
- PIEZOMETRIC SURFACE
- SW, SP, GW, GP*
- 27 — PERMEABILITY VALUES (NORMALIZED TO 10-4CM/SEC)
- ML-SM, SM*
- GM, GM-GP*
- COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)
- WEATHERED BEDROCK
- 2300/4000 — SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)
- TP-101 — TEST PIT
- ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

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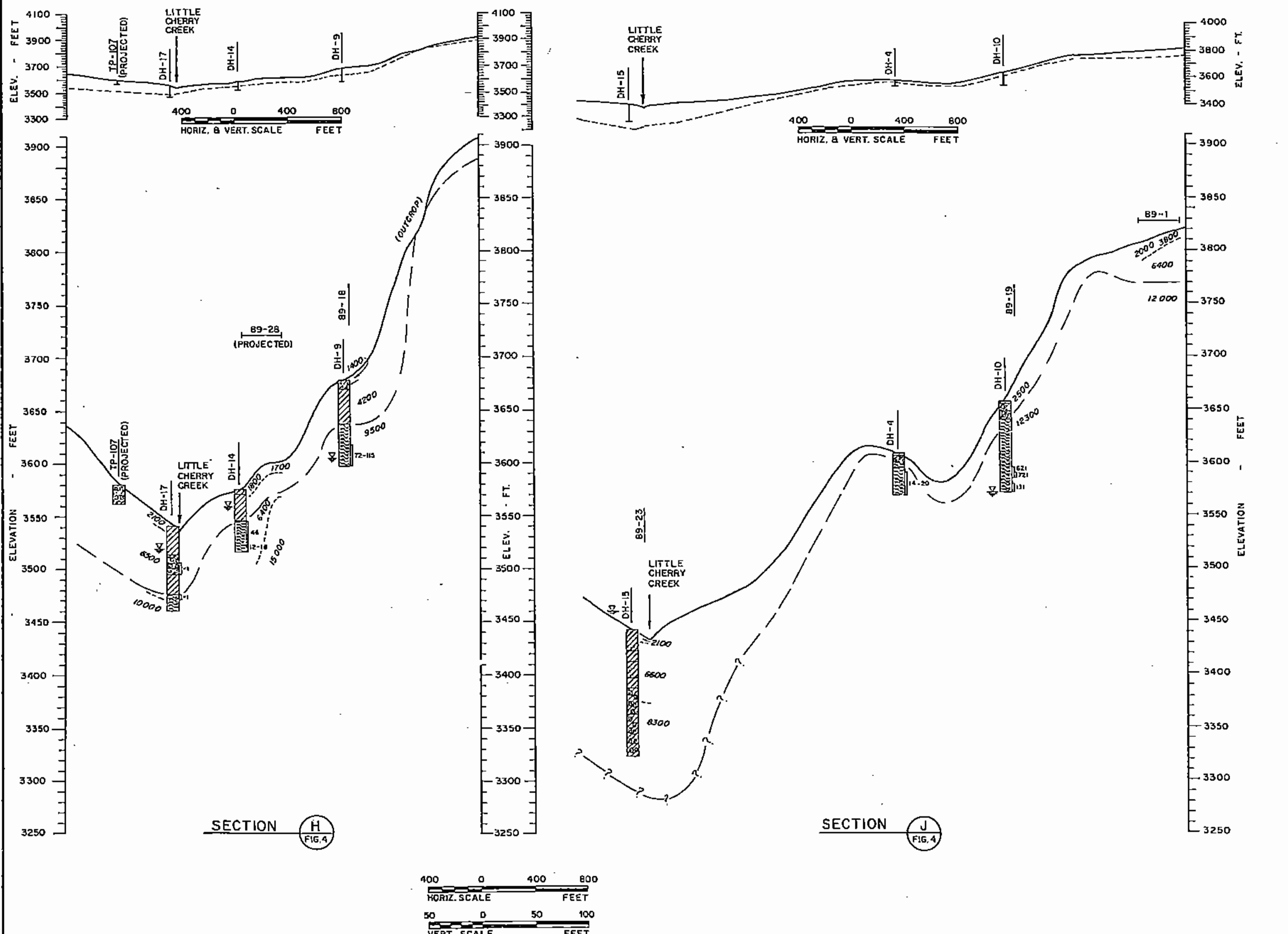
MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTION G

FIGURE 14



NOTES:

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2. FOR SPECIFIC GEOTECHNICAL INFORMATION, REFER TO APPENDIX B FOR SEISMIC REFRACTION SURVEY RESULTS; APPENDIX C FOR DRILL LOGS, WATER LEVEL DATA AND PERMEABILITY TEST DATA; AND APPENDIX D FOR TEST PIT LOGS.
3. PIEZOMETRIC HEAD MEASUREMENTS WERE TAKEN BETWEEN AUGUST AND NOVEMBER 1989.
4. TWO PIEZOMETERS WERE INSTALLED IN PLCM-6 AND DH-3.

LEGEND:

89-10 — SEISMIC LINE LONGITUDINAL TO SECTION

89-11 — SEISMIC LINE TRANSVERSE TO SECTION

DH-1 — BORING

ML, ML-CL, CL*

PIEZOMETRIC SURFACE

SW, SP, GW, GP*

27 — PERMEABILITY VALUES (NORMALIZED TO 10^{-6} CM/SEC)

ML-SM, SM*

GM, GM-GP*

COBBLES AND/OR BOULDERS (IN COMBINATION WITH ANY OF THE SOILS DESIGNATED ABOVE)

WEATHERED BEDROCK

2300 / 4000 — SEISMIC VELOCITY (FT/SEC) SHOWING APPROX. VELOCITY INTERFACES)

TP-101 — TEST PIT

ESTIMATED BEDROCK PROFILE INTERPRETED FROM BORING OR SEISMIC REFRACTION DATA, QUERIED WHERE INFERRED

* UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL. CLASSIFICATION IS FOR THE MATRIX. MATERIAL MAY ALSO CONTAIN BOULDERS AND COBBLES.

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DATE NOV. 1989 | APPROVED

NORANDA MINERALS CORPORATION

MONTANA PROJECT
LITTLE CHERRY SITE
GEOLOGIC SECTIONS H & J

MKE NO. | SHEET OF | REV.

FIGURE 15