Homathko Development

Generation Engineering Feasibility Interim Report

B.C.Hydro
ectric Generation Projects Division

March 1984

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Technical information – The technical information contained in these documents, including geotechnical investigations, hydrotechnical studies, project layouts, and capacity and energy analyses, is historical and would require updating to reflect current engineering practices, environmental information, regulatory requirements, and design methodologies.

This report references the Taseko-Tatlayoko Diversion and Nemaia Dam project concepts. BC Hydro is not considering these concepts as part of the potential project design in the investigative work under consideration today.

These documents are provided to support an understanding of BC Hydro's historical planning work and must be read in that context.

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HOMATHKO DEVELOPMENT

GENERATION

ENGINEERING FEASIBILITY

INTERIM REPORT

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This report contains information available as of March, 1984. By September 1984, all Homathko Development studies and data collection programs had been suspended. Some information in this report may be incomplete and is subject to amendment as a result of future studies.

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SYNOPSIS

SYNOPSIS

The Homathko River has been recognized for many years as having the potential for hydroelectric development. The river falls 827 m in its course from Tatlayoko Lake through the Coast Range Mountains to Bute Inlet.

An engineering feasibility study of developing the hydroelectric generation potential began late in 1981. However, because of budget restraints the feasibility study was halted early in 1984 before it could be completed. The work done, recommendations for the completion of the feasibility study and costs and benefits for a scheme of development have been documented.

The development of the Homathko River would be comprised of a small storage project near the outlet of Tatlayoko Lake, generation projects at Nude Canyon and Waddington Canyon and a substantial storage and generation project on Mosley Creek, a tributary to the Homathko River (Fig. 3-1). Powerhouses for the Mosley Creek and Nude Canyon projects would be adjacent to one another in an underground cavern on Mosley Creek, 1 km upstream of its confluence with the Homathko River. These powerhouses would be served by long tunnels from the Mosley Creek and Nude Canyon dams respectively. The powerhouse for the Waddington Canyon Project would be located underground at Waddington Canyon with a tailrace tunnel discharging into the Homathko River 2.7 km downstream of the canyon. The collector station for the transmission of power from all the projects would be located on the surface above the Mosley/Nude powerhouse.

Access to the Waddington Canyon, Mosley Creek and Nude Canyon projects would be by road from Bute Inlet. Access to the Tatlayoko Storage Project would be by road from the community of Tatla Lake.

The principle project data are:

	Waddington Canyon	Mosley Creek	Nude Canyon	Tatlayoko Storage	Total
Installed Capacity (MW)	420	290	185	-	895
Number of Units	2	2	1	-	5
Average Streamflow (m^3/s)	139.1	49.2	42.2	19.4	-
Gross Head (m)	207.3	434.5	309.5	-	951.3
Generation Construction Cost (Oct. '83 $ Millions)	685	726	306	75	1793
Average Annual Energy (avg MW)	242	168	110	-	520
(GW.h)	2121	1473	964	-	4558
Generation Construction Cost per Unit of Capacity ($/kW)	1631	2504	1654	-	2003
Generation Construction Cost per Unit of Average Energy at Site ($/avg kW)	2831	4321	2782	-	3448
Construction Period (years)	6 3/4	6 3/4	5 3/4	4	-

The Waddington Canyon Project would require the storage provided by the Mosley Creek Project for the most economical operation. The Nude Canyon Project would require the storage provided by the Tatlayoko Storage Project for the most economical operation. Although the Tatlayoko Storage Project provides some benefit at the Waddington Canyon Project, the Nude Canyon Project would also have to be constructed to make the Tatlayoko Storage Project economical. Mosley Creek Project would not be dependent on any other project in the development.

In the above tabulation the cost of the access road from Bute Inlet to the Mosley Creek Dam is allocated to the Mosley Creek Project because

Mosley would be the first project to be developed. Costs of the Waddington Canyon and Nude Canyon projects were estimated assuming prior construction of the Mosley Creek project access road.

Although the feasibility study was not completed, the interim results are encouraging. A relatively small program of field investigations and office studies would enable the feasibility study to be completed. Particular attention should be paid to the collection of hydrometeorological data and the study of the glaciermelt contribution to streamflows. The network of data collection stations designed for this study should be re-activated at least 10 years prior to the start of further studies to provide an adequate data base for future studies.

REPORT

SECTION 1.0 - INTRODUCTION

1.1 BACKGROUND

The Homathko River has been recognized for many years as having potential for hydroelectric development. The Water Rights Branch of the Department of Lands examined a development of the Taseko, Chilko and Homathko rivers in 1940 and their report was revised in 1948.[1] In 1958 the B.C. Power Commission retained consultants to undertake a feasibility study of the hydroelectric potential of the Homathko River.[2] In 1972 the B.C. Energy Board conducted a preliminary feasibility study of the Homathko Development.[3] In 1982 B.C. Hydro completed an overview study of the hydroelectric potential of the Homathko River.[4]

1.2 TERMS OF REFERENCE

Under Assignment 481-093, dated 3 October 1981, the Hydroelectric Generation Projects Division (HGPD) commenced feasibility studies of the Homathko River. The assignment limited studies to the Homathko drainage basin. Diversion of water from the Taseko or Chilko rivers was not considered.

Under the same assignment studies were undertaken by HGPD for B.C. Hydro's Environmental and Socio-Economic Services Department. Close liaison between engineering and environmental studies was maintained throughout the studies.

In 1982 fiscal restraint reduced the exploration program. In 1983 the exploration program was cancelled and in January 1984 all studies were suspended and documentation began. This report is a documentation of studies completed and includes recommendations for work to be done to complete the feasibility study. This report also presents costs and benefits of the development for use in generation planning studies.

1.3 SCOPE OF WORK

The feasibility studies were to include field investigations and office studies sufficient to:

1. Examine the technical and economic feasibility of the four projects identified in the overview study.

2. Determine the types of dams and their approximate locations.

3. Carry out a preliminary optimization of dam height, installed capacity and reservoir drawdown.

4. Determine potential energy output and prepare capital cost estimates.

5. Locate possible sources of construction materials and carry out a preliminary study of project access.

6. Provide information on project impacts for environmental and socio-economic studies.

When studies were halted, some were further advanced than others and, as would be expected, there were some inconsistencies in project data arrived at by the various disciplines working on the development. However, these inconsistencies, some of which may have a significant effect on individual items, are minor with respect to the overall cost and benefit of the development. In particular, detailed hydraulic calculations which could alter some of the hydraulic facilities have not been reviewed with the project layout engineers and have not been incorporated in this report. As part of the documentation of the studies all working files from each discipline are to be gathered together and stored under the direction of the Supervisor of the Administrative Services Section of the Hydroelectric Generation Services Department.

Core from the drill holes is to be stored in core shacks to be maintained at the Nude Canyon, Mosley Creek and Waddington Canyon projects.

SECTION 2.0 - GENERAL INFORMATION

2.1 AREA LOCATION AND DESCRIPTION

The development area, shown on Fig. 2-1, lies in the Coast Mountains north of Bute Inlet and is approximately 270 air km northwest of Vancouver.

The Homathko River rises on the western edge of the Fraser Plateau and flows south to Tatlayoko Lake. The river then cuts a southwesterly course through the extremely rugged Coast Range Mountains to its mouth at Bute Inlet. The Homathko drainage basin varies considerably in its physical features. North of Tatlayoko Lake the region is semi-arid. The lower portion of the river is in a wet, heavily timbered coastal valley.

In its course from Tatlayoko Lake through the Coast Range Mountains to the head of Bute Inlet, the Homathko River drops 827 m in 130 km. The steepest section of the river is between Tatlayoko Lake and Waddington Canyon, a narrow rock gorge with almost vertical walls, situated about 55 km upstream of the river's mouth.

Below Waddington Canyon, the river gradient lessens and the river meanders along a wide, relatively flat bottomed valley to the head of Bute Inlet.

Mosley Creek, which is the principal tributary of the Homathko River, starts in Bluff Lake and flows southward in a broad flat-bottomed valley. The valley is well suited for a reservoir, and also provides a favourable location for a damsite near its lower end. From this point to its confluence with the Homathko River, Mosley Creek becomes a swift-flowing, turbulent river in a rocky canyon.

Tatlayoko Lake, near the head of the Homathko River at El. 827, lies in a trench 27 km long on the edge of the Fraser Plateau, parallel to and approximately 16 km west of Chilko Lake. Tatlayoko Lake serves as a natural reservoir for the upper Homathko watershed.

2.2 AERIAL PHOTOGRAPHY, TOPOGRAPHIC MAPPING AND SURVEY INFORMATION

(a) Aerial Photography

There is a large amount of aerial photography covering various parts of the development area. An index of the potentially useful photography was prepared and is held on file by the Photogrammetry Section, Civil Systems Department, Computer and Management Systems Division, General Corporate Services. The index consists of a series of 1:250 000 and 1:50 000 scale map sheets marked in colour to show photo coverage.

(b) Topographic Mapping

Topographic mapping at 1:10 000 scale was prepared for the development area (Fig. 2-2).

Topographic mapping at 1:2500 scale was prepared for the project sites (Fig. 2-3) and used for project layout studies.

Initially, the 1:2500 scale topographic mapping was prepared from aerial photography taken in 1976 and 1979. Fieldwork in 1982 showed that there were unacceptable errors in the mapping resulting, in part, from the photography being at too small a scale. New aerial photography was taken in 1983 and new 1:2500 scale mapping prepared. However, work had to continue using the "old" mapping. When drawings were prepared using the "old" mapping, a note was made on the drawings indicating the preliminary nature of the topography. All "old" mapping was destroyed.

A digital terrain model was compiled during the preparation of the mapping. The digital data for the 1:2500 scale mapping is available on the Intergraph Computer Assisted Drafting (CAD) system. The CAD system can plot topography and planimetry simultaneously at any scale.

A summary report on all topographic mapping was prepared.[5]

(c) <u>Survey Information</u>

Field surveys were carried out:

1. for control for aerial photography;
2. in support of the geotechnical exploration program; and
3. in support of hydrological and environmental studies.

Reports[6,7] were prepared for all survey work.

SECTION 3.0 - DEVELOPMENT SCHEMES

3.1 DESCRIPTION

The topography in the Homathko River basin provides a "natural" choice for the scheme of development which is shown on Figs. 3-1 and 3-2. The development would consist of a storage project near the outlet of Tatlayoko Lake, generation projects at Nude Canyon and Waddington Canyon and a storage and generation project on Mosley Creek, a tributary to the Homathko River. Reservoir storage and flooded areas are illustrated on Fig. 3-3. The principal project data are given in Table 3-1.

The dam for the Tatlayoko Storage Project would be located at either the Nostetuko Axis or the Bowl Lake Axis and, using the natural storage in Tatlayoko Lake, would provide flow regulation for the Nude Canyon Project. The Nostetuko Axis is 1 km downstream of the confluence of the Homathko and Nostetuko rivers. The Bowl Lake Axis is another 5 km further downstream. The Nostetuko Axis would provide a storage reservoir only, whereas the Bowl Lake Axis would develop a gross head of about 75 m for power generation. Other axes, upstream of the Nostetuko Axis, were considered but were not examined in detail because of less favourable foundation conditions. Past reports have suggested a dam across the outlet of Tatlayoko Lake and this location should be examined in greater detail. In the proposed scheme, channel improvements at the south end of Tatlayoko Lake would enable the necessary storage to be obtained by drawing down the lake. The overview study suggested a scheme which raised the lake to obtain storage. Further studies, layouts and cost estimates are required to determine the optimum operating range; environmental costs would be included in these comparisons.

The Nude Canyon Dam would be located 2 km downstream of the confluence of Nude Creek and the Homathko River. The dam and an 8 km power tunnel would develop a gross head of 309.5 m on the Homathko River above its confluence with Mosley Creek. The Nude Canyon Reservoir would have very little live storage and the project would be essentially a run-of-river project. The powerhouse for the Nude Canyon Project would be located underground on Mosley Creek 1 km upstream of its confluence with the Homathko River.

The Mosley Creek Powerhouse would be adjacent to the Nude Canyon Powerhouse. The Mosley Creek Dam would be 9 km upstream from the powerhouse and, like the Nude Canyon Dam, would be connected to the powerhouse by a long tunnel. The Mosley Creek Reservoir would provide the largest storage in the Homathko Development and provide flow regulation for the Mosley Creek and Waddington Canyon projects. The project would develop a gross head of 434.5 m.

The Waddington Canyon Project would be located 1 km upstream of the confluence of the Homathko River with Klattasine Creek. The powerhouse would be located underground close to the dam and have a 2.7 km long tailrace tunnel. A gross head of 207.3 m would be developed. There would be little live storage available in the reservoir and, like the Nude Canyon Project, Waddington would be essentially a run-of-river project. Longer tailrace tunnels, which would increase the energy output of Waddington, were under consideration but would result in very low flows in respectively longer sections of the Homathko river immediately below the Waddington.

Homathko feasibility studies were halted before completion and the development scheme and project layouts shown in this report require further review.

Two other schemes of development were examined and discarded because of higher costs - dams at, or 3 km upstream of, Tragedy Canyon instead of

Waddington Canyon, and dams at Great Canyon and Reliance Creek instead of Nude Canyon.

The power studies examined the benefit of constructing the Mosley Creek Project without any other development; also the benefit of constructing the Mosley Creek and Waddington Canyon projects without the Nude Canyon and Tatlayoko Storage projects was examined. These alternatives are discussed in Section 4.0.

3.2 SEQUENCE OF DEVELOPMENT

The Mosley Creek Project would be developed first because it provides most of the storage for the Homathko Development. Mosley is the only generation project in the development that does not rely on another project for flow regulation. The Waddington Canyon Project would be the second project to be developed. Waddington is the most economically attractive project but it does require development of storage at Mosley for its most economical operation.

The Nude Canyon and Tatlayoko Storage projects would be built simultaneously as the final stage of the Homathko Development. Nude requires the storage at Tatlayoko for the most economical operation. The Tatlayoko Storage Project does provide some additional benefits at Waddington but not enough to justify its construction prior to the Nude Canyon Project.

Thus, the Homathko Development has the flexibility of being able to be developed in 3-stages in capacity increments of 290 MW, 420 MW and 185 MW for Mosley, Waddington and Nude respectively. However, the total capacity of 895 MW could be brought on line in the same year if required.

3.3 AREA ACCESS

The rugged terrain of the Homathko area makes road construction difficult and expensive. All potential road access routes were flown by helicopter and possible alignments were noted on 1:50 000 scale maps. Type of terrain, probable tunnel locations and probable bridge sites were also identified.

The routes were based on a maximum 6 percent grade and a minimum 250 m horizontal radius. A cost estimate was prepared for each section of road and the most economical routes, shown on Fig. 3-1, were chosen for presentation in this report.

(a) Waddington Canyon Project

Materials and equipment would be shipped by barge to the head of Bute Inlet. Presently a logging road runs from Bute Inlet to within 1 km of Waddington Canyon and, with some improvements, this road would provide the permanent access to the project.

(b) Mosley Creek Project

Permanent access would be by road from Waddington Canyon to the Mosley/Nude Powerhouse and to the crest of the Mosley Dam. If the Mosley Creek Project were built before the Waddington Canyon Project, the section of road between Bute Inlet and Waddington Canyon would have to be constructed, because equipment and materials for Mosley would be shipped by barge to Bute Inlet.

A temporary access road would be constructed from the community of Tatla Lake along the Mosley Valley to the damsite to provide an alternative access during construction. However, this construction access would be flooded later by the Mosley Reservoir.

(c) Nude Canyon Project

Access to the Mosley/Nude Powerhouse would be from Bute Inlet, using the already completed Bute-Mosley access road.

The construction of a permanent two lane access road to the Nude Canyon damsite would be expensive. It is currently proposed to enlarge the upstream 4 km of power tunnel to provide a one-lane construction access to the damsite. This portion of the power tunnel would be linked by a short access tunnel to a surface road from the powerhouse area. A 4-wheel drive type road would also be built for construction access and would be retained to provide vehicular access to the damsite when the power tunnel is in use.

A 4-wheel drive type road from the Tatlayoko Storage Project to the damsite would also be built for construction access; it would be flooded when the reservoir is filled.

(d) Tatlayoko Storage Project

Access to either the Nostetuko Axis or the Bowl Lake Axis would be by gravel road from the community of Tatla Lake. Presently, a gravel road runs from Tatla Lake to the north end of Tatlayoko Lake and a logging road continues to the south end of Tatlayoko Lake. Improvements would be made to the logging road and some new road construction, including a permanent bridge over the Nostetuko River, would complete the project access.

Area access was a subject raised several times at public meetings, held during the course of the studies. Although there might be some period during construction when a temporary road link from the Coast to the Interior would exist, there would not be a permanent access road from the Coast to the Interior.

3.4 GENERATING AND COLLECTOR SWITCHING STATIONS

The Homathko transmission collector station would be located on the surface at the site of the Mosley and Nude underground powerhouses. Mosley generating station and Nude generating station plant outputs would be transmitted to the surface collector station by 13.8 kV isolated phase bus ducts in lead shafts from the powerhouse. Waddington generating station plant output would be transmitted to the collector station by a 12 km long 230 kV transmission line. If constructed, Bowl Lake generating station plant output would be transmitted to the collector station by a 36 km long 69 kV transmission line.

The switching station control centre would be connected to the powerhouse control rooms and the dams of each of the generating stations and also to a remote System Control Centre.

Fig. 3-3 shows a single line diagram of generating and collector switching stations.

SECTION 4.0 - POWER AND PROJECT SELECTION STUDIES

4.1 GENERAL

When studies on Homathko were suspended and documentation of work to date was started, the power and project selection studies were not finished. However, sufficient work had been done on power and project selection to choose a scheme of development and project layouts for presentation in this report. Optimization of the scheme was not completed and the scheme represents the "best guess" based on studies carried out by that time.

The power studies completed to date were documented in a separate memorandum[8] and a summary is provided in the following sections.

4.2 STUDY PARAMETERS

Parameters for selecting the scheme of development were developed by System Engineering Division. The parameters used in these studies are presented in a document entitled "Evaluation of Power Benefits for Project Optimization Studies, June 1983", a copy of which is in the appendices of the memorandum on power studies.

The principle parameters used for the power and project selection studies for the Homathko Development were:

1. Value of energy - $0.0215/kW.h. (A discount rate of 4 percent net of inflation was used to establish this figure.)

2. Period of flow record - October 1940 to September 1975 (power flows are discussed further in Section 6.5).

3. Annual Cost = Total Generation Construction Cost x 0.058. (This factor includes corporate overhead, interest during construction, annual interest charges, depreciation and annual operating and maintenance costs but excludes water rental fees, capacity charges, grants and taxes during plant operation.)

4. Reservoir storage was operated such that the average energy production in each month matched, as closely as possible, the shape of the forecast annual load demand.

4.3 METHODOLOGY

Power studies determined the long-term average energy production of the Homathko Development. The project selection studies determined the optimum dam height and generating capacity at each project.

A range of dam heights was chosen for each project, and for each dam height a range of generating capacities was chosen. One project layout and cost estimate was prepared for each project. The estimates were adjusted for varying dam height and generating capacity to give a cost estimate for each of the variations for each project. The energy produced by each variation was to have been calculated with the assistance of a computer, but studies were halted before this stage was completed. The net benefits for each variation would have been calculated and the point at which net benefits were a maximum would have been chosen as the optimum point for design.

Three alternative degrees of development of the hydroelectric potential in the Homathko River basin were being examined:

Alternative I - Mosley Creek Project alone
Alternative II - Mosley Creek and Waddington Canyon projects

Alternative III - Mosley Creek, Waddington Canyon, Nude Canyon and Tatlayoko Storage projects

Other degrees of development, such as Waddington Canyon Project alone, were not examined.

An iterative process was being used to find the optimum project configuration (normal maximum reservoir level and maximum turbine discharge) for each alternative development. For example, the steps for Alternative II were:

1. Choose a project configuration for Waddington and optimize Mosley.

2. Fix Mosley at the optimum project configuration from Step 1 and optimize Waddington.

3. Fix Waddington at the optimum project configuration from Step 2 and optimize Mosley again.

This process converged quickly to the optimum for the alternative.

4.4 RESULTS

Varying dam heights and installed capacities were being examined for the Mosley Creek and Nude Canyon Projects. No variations were examined for the Waddington Canyon or Tatlayoko Storage projects. The following results are, therefore, preliminary.

1. Alternative III, with projects at Mosley Creek, Waddington Canyon, Nude Canyon and Tatlayoko Storage (Nostetuko Axis) would have the lowest unit energy cost. Table 4-1 summarizes the power studies and Table 4-2 presents the power and costs of these projects.

Fig. 4-1 illustrates the annual pattern of energy production, flows and reservoir levels.

2. Alternative II, with projects at Waddington Canyon and Mosley Creek would have a slightly higher unit energy cost than Alternative III. The marginal cost of adding the Tatlayoko Storage Project to Alternative II was greater than the marginal benefits gained at the Waddington Canyon Project. However, if the Nude Canyon Project is added to the development, the Tatlayoko Storage Project would be required to maximize the benefits from the Nude Canyon Project.

3. Alternative I, the Mosley Creek Project alone, would have the highest unit energy cost, primarily because of the high cost of project access.

4. Energy from a power project at the Bowl Lake Axis alternative of the Tatlayoko Storage Project would cost about twice as much as energy from the Homathko development scheme presented herein.

5. Alternative III, as presented, would result in rather poor load shaping throughout the year, as shown on Fig. 4-1. With 50 m of reservoir drawdown at Mosley, instead of 30 m, much better load shaping could be achieved with little change in average energy production.

SECTION 5.0 - GEOTECHNICAL INVESTIGATIONS

5.1 GENERAL

Detailed results of the geotechnical investigations carried out as part of the feasibility study of the Homathko Development are reported separately.[9] This section provides a summary of those investigations which concentrated on the development scheme described in Section 3.0 of this report.

5.2 REGIONAL GEOLOGY

(a) Bedrock Geology

Fig. 5-1 is a regional geological plan of the Homathko Development area. The development is located within both the Coast Plutonic Complex and the Tyaughton Belt geological provinces.

The Coast Plutonic Complex is a northwesterly-trending belt of mostly intrusive rocks of Mesozoic age (230 to 65 million years old) and intermediate composition (grandiorite, quartz diorite, or diorite). These have intruded a variety of older sedimentary and volcanic rocks, remnants of which are preserved as metamorphic roof pendants. Waddington Canyon and Mosley Creek projects are underlain by Coast intrusive rocks, and Nude Canyon Project is underlain mostly by metasedimentary/metavolcanic roof pendant rocks.

The Tyaughton Belt rocks consist of several sedimentary/volcanic rock groups which are also of Mesozoic age (230 to 65 million years old). Rock types include breccias, tuffs, agglomerates, volcanic sandstones, siltstones, shales, greywackes, and minor limestones.

The sedimentary/volcanic rock groups have been extensively intruded by stocks, dykes and sills of the Coast Plutonic Complex.

Faulting is the region's dominant structural feature, particularly in Tyaughton Belt rocks. Tyaughton Belt faults can be classified into three groups. These are:

1. Thrust faults which dip southwesterly at shallow angles, and predate intrusion of the Coast Plutonic Complex.

2. Right lateral strike-slip faults which trend northwesterly. Some of these predate and some postdate the Coast Plutonic Complex.

3. Normal faults which trend northeasterly. Most of these postdate the Coast Plutonic Complex and offset faults in the other two groups.

Faulting within the Coast Plutonic Complex is less obvious, but the presence of major northwesterly and northeasterly-trending linears suggests both Tyaughton Belt fault groups 2 and 3 are present there.

All faults in the region appear to have been inactive since mid-Tertiary times (10 to 5 million years ago).

Other structural features in the region are only prominent locally, and are usually related to faulting.

(b) Glacial Geology

During Pleistocene times, between 2 million and 8 thousand years ago, the Homathko Development area was extensively glaciated several times. The latest episode of glaciation began when local alpine glaciers developed into valley glaciers which eventually coalesced to form part of the Cordilleran ice sheet. Ice flowed towards the Pacific and towards the interior plateau from a glacial divide that trended northeasterly through the vicinity of Mt. Waddington. Many glaciers and icefields are still present, and glacial and periglacial processes have significantly affected present topography and the distribution of overburden. Higher elevations have been stripped of most overburden, valleys have been widened and deepened, and unconsolidated deposits are almost all associated with glacial features such as moraines, or glaciofluvial outwash plains.

(c) Regional Seismicity

Based on the faulting in the area and the recorded seismic history, a maximum credible earthquake for the area of M6.5 was selected. A maximum horizontal acceleration of 0.2 g was chosen for feasibility stage design of major structures at all sites. For design of ancillary structures that may have to conform to National Building Code requirements, an acceleration of 0.1 g was used.

5.3 FOUNDATION INVESTIGATIONS

(a) Waddington Canyon Project

(i) Introduction

Foundation investigations were carried out by B.C. Hydro during 1982 and 1983, and consisted of:

1. Reconnaissance geological mapping (1:10 000 scale) of the damsite, reservoir, and tailrace tunnel areas.

2. Detailed geological mapping (1:2500 scale) of the damsite area.

3. A gravity survey of lower Klattasine Creek.

4. Three drill holes near the damsite with a total length of approximately 275 m.

Fig. 5-2 is a geological plan of the damsite area; geological sections are shown on Fig. 5-3.

(ii) Rock Types

The damsite area straddles a major, northwesterly-trending contact between granitic plutonic rocks to the northeast and a high-grade metamorphic "migmatite complex" of interlayered gneisses and granitic rocks to the southwest. The contact is gradational over about 700 m. Unfoliated, to slightly foliated, biotite granite is located immediately to the northeast of the proposed axis. A transitional zone consisting of about 70 percent granite and 30 percent granitic gneiss extends from the dam axis to Klattasine Creek. Further southwesterly, migmatitic or granitic gneiss predominates.

(iii) Joints

There are two prominent "master" joint sets. One set, designated "Set A", is parallel to foliation in the migmatite complex and strikes northwesterly and dips steeply southwesterly. A second set, designated "Set B", strikes northeasterly almost perpendicular to "Set A" and dips subvertically. Parallel to both sets are prominent linear gullies up to 10 m wide and 10 m deep, which are typically 100 to 150 m apart. Most gullies parallel to Set A are caused by differential erosion of closely-spaced foliation slips. Most gullies parallel to Set B are caused by differential erosion of fractured andesite dykes which have been intruded along the joints.

(iv) Faults

Major faults were not found near the proposed dam axis but a sizable fault probably underlies Klattasine Creek. The fault is not exposed but it is inferred to strike northwesterly and dip subvertically. The fault is subparallel to foliation in the migmatite complex and to Set A joints. The inferred width of the fault is 30 to 40 m. Tentative contours of the bedrock surface above the fault trace are shown in Fig. 5-2. These contours are based only on interpretation of 1983 gravity survey data and are preliminary.

(v) Rock Permeability

1982 drill holes were tested to determine permeability. Drill hole intersections with Set A and Set B joint sets were tested; all reliable tests gave permeability coefficients of zero or less than 10^{-4} cm/s. Features intersected by drill holes to date, therefore, are almost impermeable at depth. However, minor rust-staining along a few joints is evident in the drill core samples and indicates that those joints are probably open to percolating groundwater.

(vi) Geology - Area of Proposed Axis

At the Waddington Canyon axis, the Homathko River is underlain by hard, strong, interlayered granite and granitic gneiss. There appear to be no weak zones or shears beneath the river channel.

Near the axis, the bedrock surface beneath the channel is at about El. 130, which is about 30 m below present river level. The channel is infilled with coarse gravel. About 18 m of this gravel appears to have been deposited when a debris flow came down Klattasine Creek and blocked the Homathko River sometime between 1972 and 1974 (Subsection 5.4).

Both abutments are precipitous walls of fresh hard, strong granite or granitic gneiss with almost no overburden or vegetation cover.

Surface linears (a linear is a surface depression extending a reasonable distance) A5 and A7 cross the axis. A hole drilled across linear A5 intersected two or three zones of moderately jointed, slightly altered, permeable, and weathered rock which is moderately hard and competent. Drilling also indicates that a band of soft, biotite-rich gneiss which underlies linear A7 on surface does not persist at depth. Dyke B5 also crosses the axis. Drilling indicates the dyke is moderately jointed but hard and competent at depth.

(vii) Geology - Proposed Tailrace Tunnel Alignments

Some features of regional geology discovered during a reconnaissance of the area are as follows:

1. Rocks along the longest proposed tunnel alignment from 0 to about 6.5 km south of Klattasine Creek are expected to be foliated granitic intrusive rocks interlayered with gneiss. From 6.5 to 12 km south of Klattasine Creek, rocks along the alignment are expected to be unfoliated intrusive rocks mostly of intermediate composition (granodiorite, quartz monzonite, or quartz diorite). All rock types are hard and competent.

2. Three major east-west linears which cross the longer alignment are probably faults inferred to be 10 to 40 m wide. Rock near them is probably closely jointed and/or sheared, and less competent and more permeable than rock elsewhere.

3. A fault which probably underlies Klattasine Creek is infilled with overburden to an unknown depth. Bedrock surface contours shown on Fig. 5-2 indicate about 50 m of bedrock cover above the tailrace tunnel, but these contours are based only on interpretation of the 1983 gravity survey and are preliminary.

(viii) Geotechnical Factors Affecting the Project

1. The Waddington Canyon damsite is underlain by hard, strong, impermeable granite and granitic gneiss which would provide a suitable foundation for any type of dam.

2. The linears, which were intersected by drill holes, are either dykes or narrow zones of slightly weathered and fractured rock, both of which are competent at depth. However, not all the linears were intersected at depth by drill holes and further drilling would be necessary to do so.

3. The depth of overburden beneath Klattasine Creek is unknown. The 2.7 km tailrace tunnel alignment is probably in the order of 50 m below the bedrock surface where it passes beneath the creek, but this could only be confirmed by drilling.

4. The proposed tailrace tunnel alignments are expected to be almost entirely in hard, strong granitic intrusives or gneiss, requiring little or no support. The longer proposed alignment would pass beneath two or three major

linears which are probably faults. Fault zones could require support and/or lining. These probable fault zones, especially the one beneath Klattasine Creek, could be quite permeable and substantial water inflows could be encountered when tunnelling across them.

(b) Mosley Creek Project

(i) Introduction

Foundation investigations were carried out by B.C. Hydro during 1982 and 1983 and consisted of:

1. Geological mapping (1:2500 scale) of the axis area.

2. Geological mapping (1:10 000 scale) of the proposed power tunnel route.

3. Four diamond drill holes with a total length of 439 m near the proposed axis.

Fig. 5-4 is a geological plan of the Mosley Creek damsite area. Geological sections are shown on Fig. 5-5.

(ii) Rock Types

The Mosley Creek project area is within the Coast Plutonic Complex. Rocks around the proposed axis are all of igneous origin.

More than 90 percent of the rock exposed in the area is fresh, hard, strong, medium-grained quartz diorite. Alteration is very minor and confined to a few narrow zones of fractured rock. The quartz diorite is typically unjointed to sparsely jointed.

The quartz diorite has been intruded by numerous dark grey diorite dykes, from a few centimetres to about 10 m thick. Most dykes are 1 to 4 m thick. The dykes are jointed and less competent than the quartz diorite, but are still fairly hard and strong.

(iii) Rock Permeability

About 20 selected intervals, within holes drilled around the proposed axis during 1982, were tested to determine permeability. Intervals were selected to test jointed dykes, zones of altered, weathered rock, and other possibly permeable features.

One zone of broken, weathered rock had a permeability coefficient of 1.7×10^{-4} cm/s. Almost all other intervals tested gave K values between zero (most) and 10^{-5} cm/s. Rocks around the axis are therefore essentially impermeable.

(iv) Linears

Three prominent parallel linears (L1, L2 and L3, Fig. 5-4), which trend parallel to Mosley Creek, lie near Axis M3 on the right bank. Geological features which underlie them could cross the axis. Their parallelism suggests a common origin. Surface mapping gave no indication of their origin, so three holes (HM1, HM2 and HM4) were drilled to intersect them at depth. Their origin is still in doubt. They are probably caused by differential erosion of either underlying zones of altered, jointed, but still strong quartz diorite or less competent, possibly sheared, underlying diorite dykes. Other less likely causes are tension cracks from an ancient rockslide, glacial lateral meltwater channels, or "Sakung" features (i.e. cracks resulting from differential isostatic rebound of a recently deglaciated valley floor).

(v) Geology - Area of Proposed Axis

The inclined drill hole HM-3 was drilled beneath Mosley Creek about 30 m upstream of the axis and penetrated 25 m of boulder gravel. The inferred depth of coarse gravel beneath Mosley Creek at the axis is 15 to 20 m.

Very hard, strong, quartz diorite is exposed on both abutments. The quartz diorite is intruded by several diorite dykes 4 to 10 m wide. Most dykes strike parallel to the axis and dip steeply upstream. The dykes are fairly hard and competent with no indication of shearing but some dykes have been eroded preferentially to form linear gullies.

Linears L1, L2, and L3 do not cross the axis, but extensions of the geological feature(s) which underlie the linears might cross the axis.

(vi) Power Tunnel Route

The proposed Mosley power tunnel would probably intersect hard, strong impermeable quartz diorite for its entire 9 km length. The quartz diorite would be classed as "good" to "very good", for tunnelling purposes (Council of Scientific and Industrial Research (C.S.I.R.), Pretoria, South Africa, tunnelling rock classification system).

Four kilometres from its upstream end, the power tunnel would pass beneath a prism of interlayered andesite and schist which could extend down to the tunnel. The andesite/schist unit is weaker than the quartz diorite, and is probably somewhat permeable.

About five linears that cross the proposed tunnel alignment are evident on the surface and could intersect the tunnel. Three linears are healed breccia zones less than 5 m wide which should be competent at depth. The other two linears could be caused by faults, fracture zones, or eroded dykes.

(vii) Nude-Mosley Powerhouse Area

The proposed underground powerhouse location is on the left bank of Mosley Creek near the Mosley Creek-Tiedemann Creek-Homathko River confluence area about 9 km downstream from the Mosley Creek dam axis (Fig. 5-6).

The left bank lower slope is covered by trees and coarse talus, with blocks to 15 m maximum dimension; it shows no signs of instability. The upper slope is mostly bare rock bluffs.

A terrace of glacial till, at the foot of the slope upstream of the creek which drains Lowwa Lake is probably the remainder of the lateral moraine from a glacier which came down the Mosley Creek valley.

The slopes are underlain by hard, strong, sparsely to moderately jointed foliated granodiorite. Joints generally strike westerly to northwesterly and dip steeply. Foliation strikes northwesterly with variable steep dips.

Near the powerhouse site, Mosley Creek appears to be underlain by a fault, which is continuous over many kilometres.

(viii) Geotechnical Factors Affecting the Project

1. The Mosley Creek damsite appears to be underlain by very hard, strong, impermeable intrusive bedrock, which would provide a suitable foundation for any type of dam.

2. The causes of the three prominent linears on the right bank near the axis are still not known. If they are underlain by faults, highly jointed zones, or cracks, they could pose seepage or differential settlement problems. If they are slide scarps (unlikely) the "bedrock" exposed on the lower right abutment would actually be several large, loose rock blocks.

3. Linears L1 to L3 are subparallel to the proposed right bank diversion tunnel and could adversely affect tunnelling conditions if they intersect the tunnel.

4. The proposed 8.4 km long power tunnel would most likely intersect very hard, strong intrusive rocks, of good to very good quality for tunnelling purposes over its entire length. Some of the four or five linears which intersect the proposed tunnel alignment could be faults or fractured dykes which could require local rock support or even tunnel lining.

5. The underground powerhouse shown in this report is likely to be located in hard, strong quartz diorite of very good quality. However, there is a linear trending down Lowwa Creek which may be a fault and may intersect the powerhouse site.

(c) Nude Canyon Project

(i) Introduction

Foundation investigations were carried out by B.C. Hydro during 1982 and 1983 and consisted of:

1. 1:10 000 scale geological reconnaissance mapping of the proposed power tunnel route.

2. 1:2500 scale geological mapping of the area around the proposed damsite.

3. Three drill holes near the damsite with a total length of about 425 m.

Fig. 5-7 is a geological plan of the area around the proposed damsite, and Fig. 5-8 is a geological section near the axis.

(ii) Rock Types

The area is underlain by a complex of metamorphic rocks which include metamorphosed volcanic rocks (chloritized meta-andesites) and metasedimentary rocks (schistose quartzite, skarn-like rocks, and biotite gneiss). All "metamorphic complex" rocks are moderately to very hard and strong. The quartzites and most metavolcanics are slightly foliated and part preferentially along foliation planes. Other metamorphic rocks appear to be nearly mechanically isotropic.

The "metamorphic complex" rocks around the site have been intruded and altered pervasively by numerous dykes, sills, and irregular to lenticular bodies of diorite. The intrusive diorite bodies are all quite hard, strong and competent. Most are sill-like or lenticular with maximum dimensions parallel to the metamorphic foliation.

Foliation at the damsite strikes consistently northwesterly at about 45° to the river reach and dips 30° to 55° to the southwest (downstream).

(iii) Rock Permeability

Twenty selected intervals within the 1982 drill holes were tested for permeability. Zones of sparsely to moderately jointed or fractured intrusive or "metamorphic complex" rocks were usually tight (K = 0). More closely jointed zones (defect spacing less than 10 cm) generally had K values of less than 2 x 10^{-5} cm/s. A few narrow zones of closely-spaced rusty joints and fractures had K values of about 2 x 10^{-4} cm/s. The damsite would be essentially impermeable. A few fracture zones would require treatment.

(iv) Faults

Most faults around the site are slips parallel to foliation planes. They are typically zones of broken rock less than 1 m thick with a few parallel gouge seams less than 1 cm thick. Most slips were observed in single drill hole intersections and it has still to be determined whether the slips are continuous.

There are three or four parallel linears on the left bank, above the proposed axis. The linears form cracks where they intersect bare rock faces, and they are probably faults. They strike subparallel to the Homathko River and appear to dip downslope (westerly) at 50° to 70°. The linear or fault nearest the axis (fault NC1) crosses the axis at about El. 1000, which is well above maximum reservoir level.

(v) Power Tunnel Route

Most of the proposed power tunnel's 7 km length would probably be in hard, strong, quartz diorite. The quartz diorite would be classed as "good" to "very good" for tunnelling purposes (CSIR tunnelling rock classification system).

Several prominent linear features cross and could intersect the proposed tunnel alignment. Some are shear zones up to 10 m wide on surface.

(vi) Avalanches

Abutments on both sides rise continuously to the snowfield and mountain peaks above. There are several avalanche tracks in the immediate site area (Fig. 5-7). Avalanche tracks are evident on the right canyon slope about 250 m and 460 m upstream of the dam axis and about 550 m downstream of the axis. Another avalanche track occurs on the left canyon slope about 200 m downstream of the dam axis.

(vii) Geotechnical Factors Affecting the Project

1. Rocks which would underlie a dam at Nude Canyon appear to be strong, competent, and almost impermeable. They are foliated, however, and could contain continuous shears parallel to their foliation.

2. The avalanche/debris flow areas on both banks between Bellamy Creek and the dam axis could cause problems during construction and operation of a dam.

3. The proposed 7 km long power tunnel on the right bank would intersect rock of good to very good quality for tunnelling purposes. Some of the four or five faults or dykes which intersect the proposed alignment could require local rock support or lining.

(d) Tatlayoko Storage Project

(i) Introduction

Several potential axes for the Tatlayoko Storage Project were investigated (Fig. 3-1). Axis T1 would be located at the outlet of Tatlayoko Lake; Axis T2 approximately 1 km downstream of the lake outlet; Axis T3, the Nostetuko Axis, approximately 5 km downstream of the lake outlet; Axis T4 approximtely 4 km downstream of the lake outlet and just below the confluence of the Homathko and Nostetuko rivers; and the Bowl Lake Axis approximately 11 km downstream of the lake outlet.

Foundation investigations were carried out by B.C. Hydro during 1982 and 1983 and consisted of:

1. Geological mapping (1:10 000 scale) of the Tatlayoko Storage Project area.

2. Geological mapping (1:2500 scale) of all potential Tatlayoko Storage Project axes.

3. Seismic and gravity surveys in the area of axes T1, T2, T3 and T4; a preliminary gravity survey in the Bowl Lake area.

4. One drill hole at each of axes T1, T2 and T3. Total length of drill holes was 196 m. Drilling was not done at the Bowl Lake Axis or Axis T4.

A geological plan and section of the proposed Nostetuko Axis (T3) are shown in Figs. 5-9 and 5-10, respectively. Bedrock surface contours of the proposed Nostetuko Axis are shown in Fig. 5-11. A geological plan and section of the proposed Bowl Lake Axis are shown on Figs. 5-12 and 5-13, respectively.

(ii) General Geology

Geological interpretation is hampered by lack of outcrop in key areas, absence of marker horizons, widespread faulting, suspected rapid facies changes, and continuous gradations between some rock types. The interpretation shown on the plans and sections is preliminary.

The project study area is underlain by Upper Triassic sediments, volcanics and volcanic-derived sediments, Cretaceous pyroclastics, and a few Late Cretaceous diorite stocks with related dykes. Any metamorphism present is low grade.

Faults of two ages are present. One older fault, the Ottarasko Fault which appears to separate two ages of Triassic sedimentary sequences, is part of a northwesterly trending fault group which crosses the southern portion of Tatlayoko Lake. A group of much younger northeasterly trending faults of probable Tertiary age parallel the Homathko River valley. The younger faults form linears with associated talus and fault scarps. Displacements along the younger faults appear to have been mainly vertical, but displacement of dykes in the Bowl Lake area indicates some horizontal displacements as well. The axial planes of several minor anticlines and synclines in the Bowl Lake Axis area are parallel to the younger faults.

(iii) Nostetuko Axis (T3)

The potential dam axis is located in an area of structural complexity. The Ottarasko Fault is inferred to lie parallel to and about 100 m upstream of the axis. At least three younger faults are inferred to intersect the dam axis beneath the valley floor floodplain.

Based on limited geophysical surveys and a single drill hole, the fluvial deposits are estimated to vary from 30 to 55 m in depth over a buried channel width of about 250 m. Across most of this width, bedrock surface is indicated to lie generally at El. 760. Drill Hole HT3 intersected about 35 m of dense interlayered fluvial sands and gravels before reaching shale bedrock. Some sandy layers in this deposit contained up to 30 percent fines, and material near the bottom appeared to be much coarser than near the surface. Overburden on the right bank appears to vary from about 30 m at the bottom of the slope to very little near dam crest elevation.

About 800 to 1300 m south of the Homathko River at Axis T3, there is a topographic saddle with lowest level at about El. 815, and a width of about 240 m near dam crest level. Overburden depth appears to vary from zero at the lowest point to about 20 m at the extreme left of this saddle area.

The left abutment bedrock of the main dam and that in the saddle dam area is of moderately competent pyroclastic rocks. These pyroclastics are underlain by sedimentary rocks, predominantly shale, which form the bedrock surface of the infilled river channel. Bedrock underlying the extensive overburden cover on the right abutment is formed from medium grained, volcanic-derived sediments.

Gravity and seismic surveys were also carried out along the Homathko River channel between the drill holes completed at Axes T1, T2 and T3. The gravity survey infers the presence of a bedrock high, of about El. 800, at the confluence of the Homathko and Nostetuko rivers. If such a high bedrock level were proven, overburden depth in the river channel immediately downstream from the river confluence could be as little as 5 m. Further investigations were planned for an Axis T4 selected about 150 m downstream from the confluence where the valley width at dam crest level is about 600 m. Ground surface across most of Axis T4 is at approximately El. 810. It is probable that an infilled, ancient Nostetuko River channel is present under the left portion of this axis.

(iv) Bowl Lake Axis

Moderately competent arkose and non-swelling dark shales form the left abutment, and similar shales with minor interlayered arkose of the older Triassic sequence form the right abutment. At least three younger faults intersect the dam axis within the Homathko River valley floor. An older fault is inferred to lie subparallel to the axis about 150 m upstream.

Near the axis, the maximum overburden depth indicated by a gravity survey is about 30 m, but no hole was drilled to confirm this.

An extensive sand and gravel alluvial terrace lies above normal river level on the left riverbank mostly downstream of the dam axis. This heavily forested terrace extends a maximum of about 120 m away from the left riverbank, with the river confined between this terrace and the bedrock canyon wall on the right. This terrace extends from slightly upstream of the dam axis to about 500 m downstream.

A proposed saddle dam area on the northwesterly extension of the axis is underlain mainly by moderately competent arkose and moderately competent shale in an area of apparent structural complexity.

The saddle dam axis lies between two moderately competent, tuffaceous rock outcrops which are about 60 m apart. Ground level is at about El. 820, approximately 10 m below dam crest level. It is currently believed that there may be up to 40 m of overburden in this narrow saddle area. A second topographic low occurs about 100 m northwest of the saddle dam area, with ground at El. 840, approximately 10 m above proposed reservoir level.

(v) Other Axes

The depth of the fluvial deposits at Axis T1, based on geophysical surveys and one drill hole, appears to vary between 25 m and 60 m. Located on Axis T1, Drill Hole HT1 penetrated to bedrock through 55 m of dense fluvial gravel and sand with little or no fines. Some loss of drilling fluid circulation indicates that the gravels are very permeable. Virtually no overburden is present over bedrock on the higher parts of the dam abutments.

From limited geophysical surveys and a single drill hole at Axis T2, the fluvial deposits are estimated to vary from about 40 to 65 m in depth. In Drill Hole T2 bedrock was encountered at 55 m depth. The overburden sequence consists of 35 m of dense fluvial sands with less than 10 percent fines, 12 m layer of loose silty sand with about 50 percent fines, and an underlying moderately dense layer of sandy gravel with almost no fines. A thin veneer of overburden partially obscures each abutment to about El. 845.

(vi) Geotechnical Factors Affecting Tatlayoko Storage Project

1. The bedrock which underlies all axes is of moderately sound arkose and tuff, but includes minor amounts of soft, incompetent shales. Potentially significant faults apparently occur at all axes. However, the bedrock is of a quality suitable for supporting a gravity dam.

2. All axes are believed to be underlain by gravels which are probably permeable and would require cut-off trenches or extensive blanketing to control seepage.

3. Poorly graded uniform fine sands and sandy silts, which underlie Axis T2 and could locally underlie all other axes, may be slightly compressible and would probably have low shear strengths.

5.4 CONSTRUCTION MATERIALS

(a) Introduction

The 1982 construction materials investigation was based on airphoto terrain analysis. The limited field program employed helicopter reconnaissance and ground traverses to confirm the airphoto interpretation. The 1983 program extended the results of the 1982 program and concentrated on materials required for the type of dam chosen for each project:

Waddington Canyon Project	-	Concrete arch dam
Mosley Creek Project	-	Concrete-faced rockfill dam
Nude Canyon Project	-	Concrete arch dam
Tatlayoko Storage Project (all axes)	-	Embankment dam

A surficial mapping program was carried out to identify potential construction material sources for the four projects.

Potential borrow areas are shown on Fig. 5-14. Table 5-1 summarizes approximate borrow volumes and haul distances for the various projects.

(b) Waddington Canyon Project

The requirements for concrete aggregate and granular embankment material could be satisfied by the cobble, gravel and sand deposit of Area 6. This material would require crushing and processing to make suitable concrete aggregate but is located immediately downstream (0.5 to 3.5 km) of the proposed site.

The large steep lateral moraines along Klattasine Creek are from 2 to 6 km from the damsite and could provide impervious material; however, initial test results indicate that the till properties are technically marginal and borrow operations would be complicated by the gravel and colluvium overlying the material.

(c) Mosley Creek Project

The project requirements for sand and gravel for use as concrete aggregate and granular embankment material could be satisfied by the Area 3 deposit located between 2.5 and 4.5 km from the site on the left bank of Mosley Creek. Should additional material be required, the borrow operations could be extended upstream towards the fluvial fans of Scimitar, Five Fingers and Mercator creeks.

The sandy silt overburden in Area 3 is the closest source for impervious material. This material has marginal properties and its use should be limited to temporary cofferdams. An alternative source would be the moraines around Tellot Glacier and Ephemeron Lake. These also appear to have marginal material properties and the haul road would be difficult to construct.

Investigations for rockfill were not carried out. It was assumed that suitable rockfill could be quarried close to the damsite.

(d) Nude Canyon Project

The project requirements for sand and gravel for use as concrete aggregate and granular embankment material could be satisfied by combining the material from the Doran Creek/Homathko River confluence area, the Tiedemann Creek fan and from the fluvial plain at the confluence of Mosley and Tiedemann creeks and the Homathko River. These sources are between 6 and 8 km from the damsite.

Impervious material, if required, may be satisfied by the clayey silt deposit in Area 7, located on Nude Creek approximately 8 to 10 km from the proposed site. This material may be difficult to obtain because a 15 m thick layer of coarse glaciofluvial material overlies the area; also access would be difficult.

(e) Tatlayoko Storage Project

The project requirements for sand and gravel, for use as concrete aggregate and granular embankment material, could easily be satisfied from Area 8. The haul distance depends on the axis chosen, but would not be more than 5 km.

The requirements for impervious material could be satisfied by either the glacial till in Area 9 or by the purple-brown silt of the Rafferty Creek fan (southeast corner of Area 8). The haul distance from Area 9 varies between 4 and 9 km depending on the axis chosen and the haul distance from the Rafferty Creek fan source would not exceed 5 km for any of the proposed axes.

5.5 DEBRIS FLOWS

Debris flows are a relatively common phenomena in the Homathko drainage basin. In 1982 a field reconnaissance was undertaken to document existing debris flows and to identify any other potential debris flows. These have been described in Report Geo 18/83[10] and are briefly summarized below.

A moraine dam impounding a large glacial lake (Nostetuko Lake, Fig. 3-1) was breached in the early hours of 19 July 1983, spilling some 6 x $10^6 m^3$ of water into the headwaters of Nostetuko River. It appears that a glacier above the lake "calved" and caused a wave which overtopped the moraine dam. A channel about 38 m deep was eroded through the moraine within about 5 hours. Trees on the valley floor downstream were knocked down and extensive erosion and deposition took place.

A similar event occurred between 1971 and 1973 at Klattasine Lake. A moraine dam containing Klattasine Lake failed and the resulting debris flow down Klattasine Creek temporarily blocked the Homathko River at its exit from Waddington Canyon. A comparison of 1983 and 1958 surveys shows that the riverbed in 1983 was about 18 m higher than it was in 1958. The effects of this debris flow can be identified by changed riverbed levels as far as 14 km downstream of the confluence of Klattasine Creek and the Homathko River. Klattasine Lake outlet has been eroded to bedrock and the event could not be repeated. However, the presence of unstable slopes along Klattasine Creek suggests the possibility of debris flows occurring in the future.

In the Homathko drainage basin, alpine glacial lakes have a relatively small volume (less than 10 x $10^6 m^3$ of water). One ice-dammed lake (named MT Lake), located on a western tributary to Nostetuko River, caused a sudden flood, or "jøkulhlaup" (Icelandic for "glacier burst"), in 1982 and 1983. This may be an annual event in this area. However if the glacier changes shape, then the drainage pattern of the lake could change.

There are several moraine-dammed lakes in the Homathko drainage basin. In many cases the discharge of entrapped water is by seepage through pervious zones of the moraine material.

Table 5-2 lists creeks that have moraine dammed lakes in their drainage basins, approximate water volumes of the lakes, and the likelihood of moraine dam failures.

The moraine failure assessment considers the following:

1. moraine material;

2. size and slopes of moraine dams;

3. depth, velocity and duration of overflow;

4. likelihood of glacier "calving", or overburden sliding into the lake;

5. possibility of piping failure as a result of large seepage flows.

No area of major concern was identified. However, because of the hazard which could be created by a sudden rise in river flows during investigation, construction or operation of the Homathko Development, tributaries where such an event might occur (Nostetuko River, Tiedemann

Creek and Whitesaddle Creek areas) should be examined on a regular basis whenever personnel are working in these areas.

SECTION 6.0 - HYDROLOGY

6.1 GENERAL

The hydrologic studies and investigations undertaken for the feasibility study of the Homathko Development were designed to meet the requirements of the scheme of development described in Section 3.0.

The hydrologic regime of the Homathko River basin is complicated by extensive glaciation and extreme variation in precipitation. The B.C. Energy Board Report[3] and the Overview Study[4] identified the need to establish a comprehensive network of climatological and hydrometric stations in the Homathko River basin in order to estimate power flows and design floods with confidence. The recommendations were not carried out until the assignment for the feasibility study was issued. There remained, therefore, a poor hydrometeorological data base at the start of the feasibility study.

Hydrometric stations and climatological stations were constructed in 1982 and 1983. Data collected at hydrometric stations include continuous water levels, continuous water temperatures, occasional discharge measurements and occasional suspended sediment samples. Data collected at climatological stations include continuous air temperatures, precipitation and occasional snow depths. Continuous relative humidity, solar radiation and wind data were collected at various locations in the basin. In addition to data collected at hydrometric and climatological stations, suspended sediment samples, water temperatures and stage discharge data were collected in 1983 for engineering and environmental hydrology studies at several locations along the upper and lower Homathko River.

The details and preliminary analyses of the data collected are presented in separate reports on Fluvial Morphology,[11] Watershed Hydrology,[12] and Sediment Regime.[13]

6.2 CLIMATE

The climate of the Homathko River basin changes rapidly from a dry interior continental climate to a wet, west coast, marine climate. There are extreme horizontal and vertical temperature and precipitation gradients within each project basin as a result of the topographic relief. The entire basin is exceptionally rugged with significant areas covered by glaciers and permanent ice fields.

When studies started in 1982 the only existing climatological stations (shown in Table 6-1) were low elevation stations located in coastal and interior basins. The only long-term climate station located within the Homathko River basin was the Atmospheric Environment Service (AES) station 1088010 Tatlayoko Lake. This is an interior station which has an average annual precipitation of 412 mm and an average annual temperature of 3.9°C. Mean monthly temperature and precipitation data for two coastal stations (Powell River and Campbell River) and for two interior stations (Tatlayoko Lake and Kleena Kleene) are shown in Table 6-2. The coastal stations have much higher precipitation (1094 to 1581 mm) and warmer temperatures (10.5 to 8.7°C). There were no high elevation stations in the basin that could be used to determine the temperature and precipitation gradients. A climatological network (described in Section 6.3) was established in 1982/83 to collect meteorological data within the Homathko River basin.

Examination of hydrometric data at Water Survey of Canada (WSC) streamflow stations indicates the Homathko River basin is subject to typical coastal fall and winter storms. The coastal mountains (El. >1000 m) provide a barrier to the movement of warm moist air masses from the Pacific Ocean and cause high orographic precipitation in the lower Homathko River basin. Hydrometric data at Tragedy Canyon indicates that the occasional winter storm penetrates the mountain barrier and moves into the upper Homathko River basin. Occasionally, in the winter, cold arctic air masses move from the interior to the coast.

6.3 DATA COLLECTION NETWORK

Records of hydrological and meteorological data (streamflow, suspended sediment, precipitation, air temperature, etc.) because of their temporal as well as spatial variability, gain in validity and significance with the length of continuance. Several years of data are required to define average monthly or annual values within acceptable limits, and even longer periods of record are necessary to adequately define the extremes of variability. Historical data, if not recorded, must be reconstituted by correlation with records elsewhere and used, regardless of the accuracy of such reconstituted records. Intermittent records are of marginal value in reconstituting records.

The power generating potential of hydroelectric plants is directly proportional to the water supply, and an over (under) estimation of this supply as a result of short-term or non-existent flow records could result in an over (under) estimation of the value of the project. Long-term hydrological and meteorological data are also essential to reliably determine flood frequency relationships for the risk/cost analysis and economic selection of diversion design floods, and to reliably estimate the Probable Maximum Flood for the design of spillways and to ensure the ultimate safety of the project. Lack of long-term data results in less reliable estimates for these design capacities, and safety factors must be increased to compensate for this uncertainty at the expense of project economics.

Long-term streamflow and meteorological data are required for the economical design of hydroelectric projects, in particular in optimizing the generating capacity and dam height, and in the competent design of spillways and discharge facilities. Increased accuracy obtained in spillway design alone could produce significant capital savings.

Simply adopting a conservative design approach, however, is not satisfactory when dealing with environmental issues such as the effect of flow regulation on downstream flow, sediment, ice and water temperature, and the resulting influence on the morphology and ecology of the river. To adequately assess these concerns long-term continuous records providing a reliable estimate of the average conditions as well as the extremes of variability are essential.

Hydrometeorological records are used to carry out the following analyses:

1. Streamflow analysis for water supply (generating capacity), flood frequency (diversion works) and reservoir operation.

2. Glaciology as it affects the long-term water supply.

3. Hydrometeorological analysis to determine the Probable Maximum Flood (spillway capacity).

4. Suspended sediment and bedload analysis to determine reservoir life, changes in fluvial morphology and effects of the project on fisheries.

5. Ice regime analysis for the design of cofferdams, tailraces and spillways.

6. Thermal regime analysis to determine project impacts on fisheries and need for selective withdrawal ports.

The B.C. Energy Board Report recommended that "expansion of the meteorological networks and snowcourse networks in such sparsely gauged basins as the Homathko, Skeena, Stikine and Liard should be actively pursued in conjunction with the expansion of the stream gauging network. In light of possible development in basins with many permanent snowfields (i.e. Homathko) a better understanding of the mass balance of glaciers should be sought".

After consultation with federal and provincial meteorologists, the Provincial Waste Management Branch (WMB), Air Studies Section, (formerly the Air Studies Branch) designed a climatological network based on World Meteorological Organization (WMO) standards to provide air temperature, precipitation and snowcourse data. The network provided climate stations in each project basin at low, mid and high elevation locations. The climate network was constructed in 1982 and 1983 by WMB personnel and was serviced by WMB during 1982/83 and 1983/84. Due to government cut-backs B.C. Hydro personnel were required to service the network after 31 March 1984.

Three automatic weather stations were installed by B.C. Hydro in the Homathko River basin in 1983 to obtain wind, air temperature and relative humidity data. The watchman at B.C. Hydro's Tatlayoko exploration camp recorded daily air temperatures, percent cloud cover, and relative humidity data at 0800 and 2000 hour for the period November 1982 to March 1984. Solarimeter data was also collected at three sites.

Ten point snowcourse measurements were obtained at Tiedemann Glacier, Nostetuko River, and Tatlayoko Lake.

Hydrometric data (water levels, discharge, water temperature and suspended sediment samples) were obtained at six sites within the Homathko River basin by Water Survey of Canada under a cost sharing agreement with B.C. Hydro.

The network of hydrometeorological stations is shown on Fig. 6-1. Table 6-3 gives the period of record and type of data collected at each station.

In September 1984 all data collection for the Homathko Development was suspended. It is expected that by March 1985 B.C. Hydro will have decommissioned the data collection network and withdrawn from all cost sharing agreements to collect data for the Homathko Development.

The minimum period for which concurrent data are required for studies of the Homathko Development is considered to be 10 years. Table 6-3 presents the period of record available at each station in the data collection network and it can be seen that a complete set of concurrent data are only available from 1983. It took 2 years from the issue of the Homathko assignment to complete the installation of the data collection network. Considering the installation time and the very limited period of concurrent data now available, a period of 10 years would be appropriate for data collection prior to the resumption of feasibility studies.

A hydrometeorological watershed model capable of modelling runoff from rainfall, snowmelt and glaciermelt could identify key climate stations and indicate redundant stations which could be eliminated from the network. A reduction in the number of climate stations would reduce data collection costs.

6.4 STREAMFLOW

Daily flow data for the Homathko River basin are recorded by Water Survey of Canada (WSC) at the following streamflow stations:

Station No.	Station Name	Project Site
08GD004	Homathko River at the Mouth	
08GD005	Homathko River below Nude Creek	Nude Canyon
08GD006	Homathko River at Tragedy Canyon	Waddington Canyon
08GD007	Mosley Creek near Dumbell Lake	Mosley Creek
08GD008	Homathko River at Inlet to Tatlayoko Lake	
08GD009	Homathko River below Nostetuko River	Tatlayoko Storage

The locations of these stations are shown on Fig. 6-1.

The hydrometric stations in the Homathko River basin have varying lengths of record. The longest record is at WSC Station 08GD004, Homathko River at the Mouth, which is mainly continuous since 1957, except for the period 1960 to 1970 when the recording gauge was operative during the open water season only.

Station 08GD009, Homathko River below Nostetuko River, was established in July 1983 but, due to equipment malfunction, no data was recorded. Station 08GD008, Homathko River at the Inlet to Tatlayoko Lake, was operational during the open water season from 1968 to 1975. For the other stations, continuous open water discharge records are available for the period 1960 to 1970. The period of record of each hydrometric station is given in Table 6-3.

The significance of the tidal effect on the accuracy of the streamflow records at Station 08GD004, Homathko River at the Mouth, was assessed. The 1973 hydrographs derived from water level records affected by tides and from water level records not affected by tides were compared, and only minor discrepancies were noted. Therefore, it is concluded that the adjustments made to offset the tidal effect reasonably estimate the streamflows at this station. A more detailed description of the assessment of tidal effects at this station is included in the Watershed Hydrology Report.

The Tatlayoko Lake Storage Project is located within the dry interior continental climatic zone with estimated unit runoff at the outlet of Tatlayoko Lake, below Ottarasko Creek and below Nostetuko River of 0.007, 0.012 and 0.015 $m^3/s/km^2$, respectively. The new WSC Station 08GD009, Homathko River below Nostetuko River, constructed in 1983 will provide the necessary hydrometric data to check and adjust the above unit runoff amounts. The hydrometric data at WSC streamflow stations on the Homathko River at Tragedy Canyon, Nude Creek and Mosley Creek, with unit runoffs of 0.033, 0.022 and 0.031 $m^3/s/km^2$, are representative of the inflow to the proposed Waddington Canyon, Nude Canyon and Mosley Creek projects respectively. The projects are located between the dry interior continental climatic zone and the wet coastal marine climatic zone. WSC Station 08GD004, Homathko River at the Mouth, with a unit runoff of 0.046 $m^3/s/km^2$ is in the wet coastal marine climatic zone and streamflows are 92 percent higher than streamflows recorded at WSC Station 08GD006, Homathko River at Tragedy Canyon.

6.5 POWERFLOW

Monthly streamflow values for the Nude Canyon, Mosley Creek and Waddington Canyon projects were generated for the period 1928 to 1980 from streamflow data synthesized from the WSC Station 08GD004, Homathko River at the Mouth. Since this station was only operational from 1957, missing data (mostly in the winter months) were derived from correlations with long-term WSC Station 08MG005, Lillooet River near Pemberton. For periods when Lillooet River data were not available, missing data were derived from WSC Station 08MA002, Chilko River near Redstone.

Monthly streamflow values for the Tatlayoko Storage Project at the Nostetuko and Bowl Lake axes were assumed to be 0.46 of the Nude Dam streamflows, based on spot discharge measurement and drainage area ratios. The new WSC Station 08GD009, Homathko River below Nostetuko River, will permit better predictions of powerflows in future studies.

Power studies started before much of the work on streamflows was done. The period 1940 to 1975 was used for all power studies. Table 6-4 shows the average monthly streamflows for this period of record.

Probability exceedence curves for the natural flow regime are shown on Fig. 6-2 and for the regulated regime on Fig. 6-3.

6.6 DESIGN FLOODS

Design floods for diversion and spillway facilities were based on Regional Flood Frequency Curves given in the B.C. Energy Board report (Volume 5, Appendix III). These design floods are shown in Table 3-1.

Probable Maximum Floods (PMFs) were used as the design flood for spillways. A comparison of the spillway design floods with PMFs derived from detailed hydrometeorological studies on other B.C. Hydro projects in similar hydrologic zones indicated that the design floods are reasonable estimates.

Design floods for diversion facilities are discussed in the relevant project sections (7.3(c), 8.3(c), 9.3(c), 10.3(c) and 11.3(c)).

It must be noted that the estimated design floods do not account for floods and flood waves generated by the discharge from ice dammed lakes, moraine dammed lakes, debris torrents, glacier outbursts, calving glaciers or slides and avalanches. The hazard potential of existing moraine dammed lakes and ice dammed lakes was discussed in Section 5.5. A review of hourly hydrometric data from Water Survey of Canada would permit the identification of such phenomena and permit an assessment of the significance of such events with respect to the planning and design of hydroelectric facilities.

6.7 RESERVOIR SEDIMENTATION

Suspended sediment samples were collected at Water Survey of Canada (WSC) cableways by WSC and B.C. Hydro personnel in 1982 and 1983. Suspended sediment rating curves were derived and long-term suspended sediment loads were estimated for each of the project sites.

Reservoir life is considered to be the period from construction up to the time when deposits encroach on live storage. The following reservoir life estimates are preliminary, as they are based on only 2 years of suspended sediment data. Better data on reservoir volumes and a longer period of record of sediment loads are required to verify these preliminary estimates.

Reservoir life estimates are shown in Table 6-5. Since independent bedload calculations were not made, bedloads were estimated as a percentage of the estimated long-term suspended sediment loads. A range of bedloads was used to establish the sensitivity of reservoir life to bedload estimates. A 100 percent increase in the estimated bedloads resulted in only a 20 percent decrease in reservoir life. Details of the reservoir life calculations are presented in the Preliminary Sediment Regime Study.[13]

The Tatlayoko Storage Project with the dam axis downstream of the Nostetuko River or at Bowl Lake would create a large reservoir in Tatlayoko Lake and a small forebay reservoir at the dam, linked by a narrow channel (see Fig. 10-1). Sediment loads at the Tatlayoko Storage Project were estimated by prorating on a drainage area basis the suspended sediment loads calculated for Station 08GD005, Homathko River below Nude Creek, where suspended sediment samples were collected. Sediments primarily from Ottarasko Creek and Nostetuko River would be deposited near the mouth of these tributaries, in the small forebay. It was assumed that sediments entering Tatlayoko Lake would be trapped. With the dam axis downstream of Nostetuko River, the reservoir life of the forebay would be in the order of 75 years. With the dam axis at

Bowl Lake, the reservoir life would be in the order of 750 years. The reservoir life could be doubled for both cases described above if Ottarasko Creek were diverted into the Tatlayoko Lake portion of the reservoir. The adequacy of suspended sediment data collected at the new Station 08GD009, Homathko River below Nostetuko River should be assessed.

The Nude Reservoir would have a life of about 230 years, assuming the Tatlayoko Storage Project would have been constructed.

The Waddington Reservoir would have a life of about 400 years. If the Tatlayoko Storage and Nude Canyon projects were not constructed, the estimated reservoir life would be about 270 years, assuming the Mosley Creek Project would have been constructed.

The 19 July 1983 Nostetuko flood could have created an over estimate of the long-term sediment load for stations on the Homathko River. Hence, the above reservoir life estimates might increase as data from further sediment sampling is included in the calculations.

The Mosley Reservoir would have a reservoir life of about 1300 years.

6.8 GLACIOLOGY

Thirty-six percent of the drainage basin for the Waddington Reservoir is covered by glaciers and the hydrology of the basin is significantly affected by the glaciers. However, when the drainage areas for the Mosley Creek, Nude Canyon and Tatlayoko Storage projects are included, only 13.5 percent of the total drainage area above the Waddington Dam is covered by glaciers. Tables 6-6, 6-7 and 6-8 give the percentage of sub-basins covered by glaciers.

In order to assess the significance of glaciers on water supply for the hydroelectric development of the Homathko river basin, the Glaciology Division of the National Hydrology Research Institute (NHRI), was consulted in 1980. Because of the hydrological significance of glaciers in the basin, a field program was undertaken by NHRI and B.C. Hydro to obtain mass balance data on Tiedemann and Bench glaciers in 1981, 1982 and 1983. The mass balance data was prorated (by NHRI) for the drainage area above the Waddington Canyon Project in order to quantify the glaciermelt contribution to streamflow and assess the significance of glaciers on this project. The results are tabulated in Table 6-9. The net annual reduction in glacier mass resulted in the summer flows (April to September) at Waddington Canyon being augmented by 18.6 percent, 20.9 percent and 11.7 percent in 1981, 1982 and 1983 respectively.

The variation in annual glaciermelt contribution may be the result of different meteorological conditions (air temperature, insolation, albedo, etc.); physical conditions; volcanic dusting (Mt. St. Helens); or even in the prorating of glaciermelt based on mass balance measurements from only two glaciers (Tiedemann and Bench). Glaciologists have indicated that generally, glaciers have been retreating (melting) since the late 19th Century and augmenting streamflows.

Based on the U.S. Environmental Protection Agency's (EPA) climatic forecast of a "greenhouse effect", it appears that, in the future, there may be an increase in the glaciermelt contribution compared to that computed above. It should be noted, however, that a decrease in glaciermelt could result from a colder climate or when glaciers have retreated to higher altitudes (an equilibrium position) or glaciers have been depleted. The glaciermelt contribution is important in the planning and design of the hydroelectric facilities and a hydrometeorological watershed model capable of modelling glaciermelt is required to verify the glaciermelt contribution.

6.9 ICE REGIME

A review of WSC data and 1982/83 and 1983/84 ice observations by B.C. Hydro, Water Survey of Canada, and Waste Management Branch personnel indicates that complete ice cover of the Homathko River occurs infrequently. Cold weather between November and March could create a complete ice cover; however, the warm moist winter storms that occur during this period have a tendency to deter the formation and persistence of ice.

During the winter an ice cover forms on the small lakes in the Homathko River basin. During extended cold periods an ice cover forms on portions of Tatlayoko Lake. It is anticipated that under regulation, ice cover might form on the reservoirs during extended cold periods. The higher flows during the winter months would probably reduce the extent and duration of ice cover on the river.

SECTION 7.0 - WADDINGTON CANYON PROJECT

7.1 DESCRIPTION OF SITE

The Waddington Canyon Project is located near the downstream end of Waddington Canyon, a short distance upstream of Klattasine Creek confluence and about 50 km upstream from Bute Inlet. Waddington Canyon extends about 1 to 1.5 km upstream from its mouth before widening significantly. The river flows through canyons for most of the distance upstream to the Mosley Creek and Tiedemann Creek confluences, with only occasional valley widenings. Logging road access exists from Bute Inlet to about 1 km downstream of Waddington Canyon.

Klattasine Creek joins the Homathko River from the south about 350 to 600 m downstream of the Waddington Canyon mouth. Alluvial fan deposits, largely contributed by debris flows from the sudden failure of a moraine dammed glacial lake, extend over a distance of several kilometres downstream from the site.

Specifically, the dam would be located about 650 m upstream of the canyon outlet and the underground powerhouse would be located within the left abutment rock ridge which lies between Waddington Canyon and Klattasine Creek. A 2.7 km long tailrace tunnel from the powerhouse would pass under Klattasine Creek and exit on the left bank of the river about 4.5 km from the canyon outlet.

In the lower portion of Waddington Canyon the river is confined between precipitous rock walls which rise almost vertically from beneath deep alluvial deposits. With river levels of about El. 155 to El. 160, the canyon width at the base of these rock walls varies from 15 to 70 m. The right canyon wall rises over great heights to alpine slopes above. However, the left abutment is limited in height to the top of a rock ridge at about El. 350 near the canyon outlet. At the dam axis this ridge is about El. 425 approximately 250 m away from the river. At distances of 1000 m upstream of the canyon outlet and 250 m away from the river it rises to El. 650. The line of this ridge is broken by an abrupt cliff of about 60 m height which occurs just downstream of the dam axis and the proposed underground powerhouse location.

At the proposed dam axis the left abutment, irregular in shape, has an average inclination of about 60° from horizontal. The right abutment is smoother and flatter with an average inclination of about 45°. The abutments are slightly convex in places.

Downstream of the canyon the valley broadens out to widths of 600 to 1400 m, through which the Homathko River meanders in braided channels.

7.2 DEVELOPMENT OF INTERIM PROJECT ARRANGEMENT

(a) Tailrace Concept

A paramount consideration in siting the power facilities was to avoid, or minimize, construction and maintenance costs associated with the present debris flow alluvium, or possible future alluvium deposition, at the confluence and downstream of Klattasine Creek. In all earlier studies the powerhouse tailrace was located in the immediate canyon area. To continue with such a location would have involved expensive and uncertain river channel improvements in terms of tailwater conditions and maintenance requirements. In addition, a significant portion of the head indicated to be available prior to the debris slide would have been foregone.

Initially, a right bank underground powerhouse location with a lengthy tailrace channel was considered. The tailrace channel would have been excavated into the right bank river terrace deposits to exit into the Homathko River downstream of the Klattasine Creek confluence. However, this concept was set aside after only rudimentary study when it became evident that tailwater uncertainties and channel maintenance requirements, while alleviated, would remain significant concerns, and that the incremental costs of tailrace channel and additional power tunnel and/or penstocks could not be justified by the modest head benefits.

Analyses were then made of extending the hydraulic conduit to various downstream locations to increase available head for power generation while minimizing the impact of the Klattasine Creek debris flows on tailwater levels. For this study a long tailrace tunnel, with powerhouse adjacent to the dam, was selected instead of a power tunnel because the conduit would be required to pass under Klattasine Creek and an inferred underlying fault. A high pressure power tunnel at low-level was not considered to be as technically and economically sound as a low pressure tailrace tunnel. Analyses indicated large net benefits obtainable by adopting an extended tailrace tunnel concept. A 2.7 km long tailrace tunnel, exiting 4.5 km downstream of the canyon outlet, was selected as a base scheme. This location appears to be a practical minimum distance downstream from the canyon, recognizing the configuration of the river channel and the extent of change to riverbed elevation due to Klattasine Creek alluvial deposits. At this location the tailwater level is about 41 m lower than that existing at the canyon outlet, which represents an increase in gross head from about 166 m to 207.3 m. There would be some impact on salmon spawning with the majority of natural flows diverted from the river channel between the dam and this tailrace portal tunnel.

As discussed in Subsection 7.4, analyses indicate that net benefits, excluding allowance for impacts on the salmon fishery, increase with a much longer tailrace tunnel.

(b) Upstream Damsite and Power Tunnel

A comparison was made between a dam located at Tragedy Canyon with a 7.6 km long power tunnel and power facilities located at Waddington Canyon, and the arrangement with dam and all facilities located at Waddington Canyon. An elementary cost analysis indicated the cost of the power tunnel far exceeded potential dam savings, without accounting for hydraulic tunnel losses, even though the dam height could be reduced from 200 m at Waddington Canyon to 70 m at Tragedy Canyon.

(c) Dam Type

The shape of the canyon walls allows for a favourable crest to height ratio of 1.8, and, combined with apparently very strong abutment rock, leads to the selection of an arch dam for this site. An arch dam has been put forward as the proposed dam type in all past studies, including those by Sandwell/ElectroWatt for the B.C. Power Commission. Their preliminary design was completed sufficiently to define concrete volume for acceptable stress levels. More than adequate quantities of concrete aggregate appear to be available nearby. On the other hand, suitable impervious core material for an embankment dam alternative has not been found in quantity to date. Accordingly, an arch dam was selected for preliminary studies, recognizing that alternative dam types would be investigated in the future.

(d) Reservoir Level

Estimates of arch dam costs for alternative reservoir levels were based on the USBR formulae[14] adjusted to conform to comparable precedents. These precedents included the earlier arch dam design by ElectroWatt and the modern, more efficient designs for Contra and Daba Klamm arch dams. Appropriate axes were selected to suit topographic conditions for the individual reservoir levels and dam heights, with the axes for the highest dams being located furthest upstream to take advantage of increasing upstream heights of the rock ridge.

These preliminary studies indicated that the optimum Waddington maximum normal reservoir level could be slightly under El. 325. However, pending more detailed analyses this elevation was adopted for present development considerations.

For full development of head and energy, the Waddington Reservoir would need to back water up to the Mosley/Nude generating station. With a maximum normal reservoir level at El. 325, the Waddington Reservoir would flood 1.2 km up Mosley Creek to the Tiedemann Creek confluence. Here a powerhouse site could be conveniently located to house units for both the Mosley Creek and Nude Canyon projects. With minor channel improvements in the reach immediately upstream of Tiedemann Creek only 0.5 m maximum backwater loss would be experienced.

Lowering the reservoir level by 5, 10 or 15 m would result in a direct loss of head or very major channel improvements since there is no convenient powerhouse site further downstream. The value of the lost head and energy with any major lowering of the reservoir appears to be considerably greater than cost reductions from lowering the Waddington Canyon dam height.

A comparative cost analysis was also made of Waddington reservoir levels El. 325, El. 350, El. 375 and El. 400. For all reservoir levels except El. 400 a common Mosley/Nude powerhouse could be considered, with only adjustment of tailwater level. However, for any reservoir level greater than El. 375, because of topography limitations, separate powerhouses would be required for each of the projects. In this study, the reduced energy production from the combined Mosley and Nude powerplants with increased Waddington reservoir levels was more than compensated for by increased energy production from the Waddington powerplant. This net increase in generation would result in more effective use of the head at Waddington than the upstream powerplants, because of local inflows downstream of the Mosley Creek and Nude Canyon projects. However, when considering the value of these incremental energy benefits in conjunction with incremental power facility costs and incremental dam costs, a distinct cost advantage was offered by the lowest dam, reservoir level El. 325, which would develop all head between the upstream and downstream projects. This result was most sensitive to the incremental arch dam costs which predominate in the benefit/cost comparisons.

7.3 INTERIM PROJECT ARRANGEMENT

(a) General

The interim arrangement is shown on Figs. 7-1, 7-2 and 7-3 and would comprise a concrete arch dam, with intermediate level outlets in the dam (service spillway); an auxiliary tunnel spillway; a plunge pool downstream of the dam; and power facilities with individual power intakes and penstocks, an underground powerhouse and a long tailrace tunnel. The reservoir would be operated on a "run-of-river" basis, with only 1.5 m of pondage drawdown assumed below maximum normal reservoir level El. 325.

(b) Dam

The 200 m high double curvature concrete arch dam would have a developed crest length of about 330 m. Weathered rock would be removed from the foundation. Additional rock excavation would be required to reduce the convexity of the right abutment and to trim the abutments to a regular "V" shape to avoid high stress concentrations.

The abutment rock is hard, competent interlayered granite and granite gneiss which would require a minimum of foundation preparation. The peripheral rock area in contact with the arch dam would be consolidation grouted to a depth of 10 m. A grout curtain would be provided along the upstream periphery of the dam to a maximum depth of 65 m, varying to 30 m minimum depth under the upper portions of the foundation. A system of drain holes and abutment drainage adits, spaced at about 35 m vertically, would be provided downstream of the grout curtain to control hydrostatic pressures underneath the dam and in the abutments.

Based on limited drill hole information the base, of the arch dam is assumed to be at El. 128, beneath approximately 30 m of alluvium. More detailed exploratory drilling will be required to confirm the elevation of bedrock in the canyon for future design studies.

(c) Diversion

Diversion during construction would be through a single 12.0 m diameter, shotcrete-lined, D-shaped tunnel 460 m long located within the right abutment. Design flow capacity of this tunnel would be 935 m^3/s based on a flood of estimated 10-year frequency. The 10-year return period for the design flood was considered appropriate because the main dam would be a concrete arch and would not suffer substantial damage if the cofferdam were overtopped. Maximum flow velocity in the tunnel would be 8.3 m/s. The closure gate would be 9.0 m wide x 16.0 m high. Shotcrete rock support of the tunnel walls and soffit was assumed, except near the tunnel portals where concrete lining would be provided. However, in future studies an unlined diversion tunnel should be studied considering indications that this tunnel could be excavated through excellent, massive bedrock. This tunnel would be plugged permanently after gate closure and commencement of reservoir filling. To provide flexibility during diversion gate closure, in the event of unanticipated closure difficulties, a small gated sluice, later to be plugged with concrete, would be provided at a suitable level in the dam base.

The Homathko River carries a significant sediment load through Waddington Canyon and scour around the diversion tunnel closure gate could be expected. Consideration should be given to providing a steel lining on the invert and bottom 0.5 m of the walls at the gate section and for about 15 m downstream. A gated sluice in the base of the concrete arch dam would allow for inspection of the diversion tunnel.

Both cofferdams would be constructed on top of the river alluvium, with only minor excavation for levelling a working area.

The upstream cofferdam, with crest El. 174.6, would consist of concrete arch sections placed on a concrete base, with a concrete cutoff wall installed through the alluvium to bedrock underneath the concrete base. The maximum height of this arch cofferdam above its base would be about 20 m; its crest length would be 80 m. Initially, a small temporary sand/gravel cofferdam with upstream sealing would be required to divert the low winter flows away from the work area and through the diversion tunnel.

The downstream cofferdam, with crest El. 166.6, consisting of a small fill dam and concrete core wall would also be placed over a concrete cutoff wall through the deep alluvium.

(d) Discharge Facilities

(i) Intermediate Level Outlets (Service Spillway)

A battery of five outlets would be provided in the arch dam to control reservoir levels and releases during reservoir filling and powerplant shutdowns, to release normal flood spills, and for emergency reservoir evacuation purposes. These five outlets would be capable of discharging 1330 m^3/s at maximum flood level El. 328, with 3 m of flood surcharge above maximum normal reservoir level. This capacity would be approximately that required to pass a flood with an 80-year return period.

Each outlet would consist of a jet flow gate 3000 mm in diameter for flow regulation, and a 3000 mm guard gate of the ring follower type. These gates would be located in a gate house cantilevered out from the downstream face of the dam. Bulkhead gate guides would be provided on the upstream face of the dam to lower a bulkhead gate for servicing the guard gate. The bell mouth inlet would be protected by a coarse trashrack to prevent large debris from entering the steel-lined conduit.

The outlets would be arranged in pairs at El. 230 and El. 210, with a single outlet at El. 190. They would be oriented laterally and vertically to spread the impact locations and to extend the jet throws as far as possible from the dam.

(ii) Plunge Pool and Weir

The energy of releases from the intermediate level outlets (service spillway) would be largely dissipated in an unlined plunge pool 45 m wide x 35 m deep extended to a point about 200 m downstream of the arch dam toe. To form this plunge pool with its invert at El. 128, a maximum 30 m of alluvium downstream of the arch dam excavation would have to be removed. In addition rock trimming of the channel sides would be required.

Downstream slope protection and a positive control for plunge pool water levels would be provided by a thick mass concrete slab placed over the sloping alluvium excavation, with the underlying alluvium heavily grouted. Special protection would not be provided for the side walls and floor of the plunge pool.

(iii) Auxiliary Spillway

The auxiliary spillway would be utilized only when flood releases exceed the discharge capability of the service spillway. It would have a maximum discharge capability of 2500 m^3/s at the maximum flood level of El. 328.

The components of this spillway are located in the left abutment and are comprised of a curved approach channel, a control structure with two radial gates 12.0 m wide x 12.0 m high, and a free-flow, modified D-shaped, 60 m long spillway tunnel which varies from an upstream section 16.0 m wide x 16.7 m high to a more steeply inclined 12.5 m diameter section (Figs. 7-1 and 7-2). Aeration slots would be provided downstream of the radial gates. Auxiliary spillway releases would be thrown by a terminal flip bucket, with a maximum velocity of about 40 m/s, to an impact area downstream of the plunge basin weir.

In selecting a single tunnel spillway for auxiliary spill purposes at this site, it is recognized that:

1. This facility in its auxiliary role would be seldom used, and then possibly for only limited periods.

2. If required auxiliary spillway releases had to be curtailed, excess flood flows could be passed over the crest of the arch dam without risk to the structure.

(e) Power Facilities

The power generation facilities, shown on Fig. 7-3, would comprise separate power intakes and penstocks serving two units with a combined capacity of 420 MW, and a single tailrace tunnel extending 2.7 km to rejoin the Homathko River about 4.5 km downstream of Waddington Canyon. The annual capacity factor would be 58 percent.

The power intakes would be sited on the left bank close to the arch dam, with the invert set at El. 309.5 for sufficient submergence to avoid air entrainment under minimum reservoir conditions. For this "run-of-river" project a pondage drawdown of only 1.5 m has been assumed. The intake structure would house an operating gate 4.0 m wide x 5.6 m high and an upstream bulkhead gate for each intake. The bellmouth intake would be provided with a set of trashracks which would be cleaned from the gatehouse deck by a trashrack cleaning machine.

Following a short transition from the intake gate, power flows would be conveyed through an upper bend, inclined shaft and lower bend via a concrete-lined 4.8 m diameter penstock, 260 m long, and a steel-lined 4.4 m diameter section extending from the turbine inlet to a point 50 m upstream of the penstock coupling chamber. Maximum velocities in the penstocks would be 6.8 and 8.0 m/s in the concrete and steel-lined sections, respectively.

The underground powerhouse, shown on Fig. 7-3, would contain two units and a service/control bay in the main cavern, with transformers and switching equipment in a secondary cavern on the downstream side of the main cavern. The control building would be located at the end of the powerhouse adjacent to the service bay. The main cavern would be 85 m long with a span of 21.5 m, and a maximum depth from roof soffit to draft tube base of about 46 m. The transformer gallery would be 60 m long with a width of 15 m and a height of about 17 m. For the indicated high rock quality no special rock support appears necessary, and only suspended ceilings would be provided to safeguard equipment and personnel from minor spalling of rock. A single 5.0 m diameter unlined high voltage CGI bus lead shaft, 210 m long, would connect to the surface CGI bus terminal equipment building. A drainage and air supply tunnel would be provided upstream of the main cavern, and an air vent 165 m long would be extended to the surface from the draft tube gallery. Access would be provided to the service bay by a 10 m wide x 7.5 m high x 670 m long D-shaped tunnel, which would curve up an 8 percent grade to exit near the Waddington Canyon outlet.

The two Francis type hydraulic turbines would each be rated at 213 200 kW under a net head of 193.3 m, and would operate with a synchronous speed of 200 rpm. The turbine throat diameter would be about 3720 mm. The minimum setting of the turbine distributor centreline would be 2.25 m below the minimum tailwater level with only one unit operating at half output and with no spill. (Subsequent hydraulic studies indicate that the tailwater level for turbine setting could be up to 2 m higher than that shown on Fig. 7-3.)

An average power flow of 137.7 m^3/s (average runoff minus spill) was adopted based on Overview Report data, because power flow studies were incomplete. This value would be subject to minor revision during the next phase of feasibility studies. The maximum combined discharge of both turbines at rated output would be 244.8 m^3/s. For conduit size optimization purposes, a weighted average power flow of 171.6 m^3/s has been adopted, consistent with an annual plant capacity factor of 58 percent.

The powerhouse would be equipped with an overhead bridge crane with a main hoist capacity of 550 t, and an auxiliary hoist capacity of 20 tons.

The two generators would be of the umbrella type, each equipped with a single combined thrust and guide bearing. They would be rated 210 MW (221.1 MVA, 0.95 pf) with a voltage of 13.8 kV. Two 3-phase banks of 13.8/230 kV generator transformers and 230 kV CGI switchgear would be located in the transformer gallery.

Powerhouse, dam and switchgear gallery operations would be controllable from the plant control room underground, from the Homathko Collector Switching Station and from the Systems Control Centre at Burnaby Mountain.

Dam station service power would be provided by a 13.8 kV transmission line fed from the powerhouse generator bus. Diesel generators would provide standby power.

The turbine draft tubes would extend under the transformer gallery to a surge chamber/manifold. Hoistway and storage provisions would be made in the surge manifold for installing and removing the draft tube gates. A 4.0 m diameter air shaft 165 m long would extend to the surface to vent air during changes in the surge chamber level. The surge chamber was sized to accommodate sudden shutdown of one unit with one continuing to operate, and the sudden startup of one unit with one unit operating. For the load acceptance case it was found that full load acceptance on one unit gave higher maximum water levels than full load acceptance on two units simultaneously because of the higher initial water levels. The surge chamber, with reduced freeboard, would accommodate full load acceptance on one unit with the other unit operating during a one in 100-year flood. Surge chamber dimensions were not optimized.

The single tailrace tunnel would extend 2.7 km from the surge chamber manifold, and would be unlined over most of its length. Standard drill and blast methods were assumed for construction. Nominal rock bolting, mesh and shotcrete support were assumed. The lined portions of the tunnel consist of a concrete collar at the outlet portal and a concrete lining where the tunnel would pass through the inferred Klattasine Fault. The tailrace tunnel would carry full, slightly pressurized flow with a D-shape section 10.0 m in diameter. Maximum velocity with both turbines operating at capacity would be 2.7 m/s, and the weighted average tunnel velocity would be 1.9 m/s. A preliminary optimization analysis considering incremental tunnel costs and incremental hydraulic loss benefits, with varying diameters, was used as the basis for tunnel size selection.

At the tunnel outlet, a tailrace channel about 300 m in length would be required to convey the power flows to the Homathko River. Maximum excavation depth in the river alluvial deposits would be approximately 30 m near the portal; elsewhere channel excavation would vary between 10 and 15 m.

(f) Site Access

(i) Special Construction Access

A special construction tunnel 6.5 m wide x 5.0 m high has been included in the interim project layout to provide access for construction of the two cofferdams and for excavation of the dam foundations and plunge pool area. This access tunnel, 750 m in length, would be located in the right abutment with its entrance at the canyon outlet. It would closely parallel the canyon wall, cross over the diversion tunnel near its outlet, exit in the plunge pool area, and continue between the diversion tunnel and canyon wall to exit at the upstream cofferdam. This construction access would be particularly expensive, and it is possible that future studies would show that a surface access into the canyon might be more economical.

(ii) Permanent Site Access

The permanent access road from Bute Inlet on the right bank of the Homathko River immediately downstream of the canyon would branch to provide low level access to the powerhouse and high level access to the dam crest and various left bank facilities. Access to the powerhouse would be via a bridge 80 m long across the canyon outlet to the left bank entrance of the powerhouse access tunnel. High level access to the dam crest would be developed in open cut, largely in rock, up the less precipitous portion of the right canyon slope. From intersection with the dam crest the permanent access road would continue to the Mosley-Nude powerhouse area. Access across the dam would lead to the spillway and power intakes and the CGI bus terminal equipment building. An elevator would provide access from the CGI bus terminal equipment building to the powerhouse. Another elevator would provide access from the top of the dam to the dam abutment galleries and outlet equipment.

7.4 ALTERNATIVE PROJECT ARRANGEMENTS

Studies indicate that a much longer tailrace tunnel would provide greater net benefits than the scheme presented in Section 7.3. Tailrace tunnels varying in length from 2.7 km to 16 km were studied, and an alternative tailrace tunnel length of 13.2 km was selected as the most attractive.

An unlined, D-shaped tailrace tunnel of 10.0 m diameter, excavated by drill and blast methods, was used for these comparisons. (A larger diameter tunnel may be required for the longer tunnels.) The alternative tunnel routes are shown on Fig. 7-4.

The 13.2 km alternative would develop 76 m more gross head than the 2.7 km alternative, permitting an increase in installed capacity of 120 MW to a total of 540 MW, for a 60 percent annual capacity factor installation.

The two Francis type hydraulic turbines would each be rated at 274 100 kW under a net head of 236.5 m, and would operate with a synchronous speed of 225 rpm. The turbine throat diameter would be 3640 mm, slightly smaller than that for the 2.7 km alternative.

With an average power flow of 137.7 m^3/s, the maximum discharge of both turbines would be 259.7 m^3/s, and the weighted average power flow would be 170.8 m^3/s based on a 60 percent annual capacity factor. Maximum and weighted average tunnel velocities would be 2.9 and 1.9 m/s, respectively.

Dimensions for the powerhouse would be similar to the 2.7 km alternative, since the increased synchronous speed tends to almost completely compensate for increased generator capacity. Accordingly, the overall powerhouse arrangement and location would be similar to those described in Section 7.3, except for the following:

1. Because of an 80 m deeper setting the access tunnel to the service bay would be increased by 1000 m to a new length of about 1700 m.

2. The high voltage lead shaft, elevator shaft and vent shafts would be increased in length by about 80 m.

3. The length of penstocks would be increased by about 90 m.

All other components of the project general arrangement would be identical to those described for the 2.7 km alternative.

Net benefits for the 13.2 km alternative tailrace tunnel excavated by drill and blast methods is indicated to be about $90 million over the project life. However, studies of tunnel excavation by tunnel boring machine (TBM) methods indicate the potential for increased net benefits over the project life. For a TBM tunnel of 8.8 m diameter, hydraulically equal to the 10.0 m diameter D-shaped tunnel excavated by drill and blast methods, studies indicate a potential net benefit compared to the 2.7 km alternative of about $190 million. The optimum TBM tunnel diameter could be somewhat larger with a further increase in net benefits.

7.5 CONSTRUCTION CONSIDERATIONS AND COST ESTIMATES

(a) Construction Schedule

The proposed construction schedule (Fig. 7-5) indicates a construction period of 6 3/4 years from award of the first contract at the start of Year 1 to the in-service date of the first 210 MW unit in the fall of Year 7; the second unit would follow by approximately 4 months. Project approval would be required at least 6 months prior to award of the first contract.

River diversion would take place early in Year 3, and the reservoir would be filled in the summer of Year 7.

The critical path for construction of the project would be generally along the following activities: development of camp and site access - construction access tunnel - diversion tunnel - dam abutment and foundation excavation - dam concrete - reservoir filling.

For the first unit to be ready for testing when the reservoir is filled, powerplant construction would need to be concurrent with dam construction. The overall sequence for powerplant construction is: powerhouse access tunnel - powerhouse/penstocks excavation - concrete placement - equipment installation.

Tailrace tunnel and manifold excavation would be coordinated with powerhouse construction. The spillway tunnel and headworks would be constructed at the same time as the dam, to be ready for reservoir filling.

Conventional methods of construction and production rates similar to those achieved on other hydroelectric projects were assumed for the preparation of the construction schedule. A winter shutdown was assumed for most activities, except for underground work.

(b) Site Facilities

The Waddington Canyon site is characterized by the very steep canyon walls making dam construction access difficult and costly.

Construction roads are described in Section 7.3(f)(i). Concrete for the dam would be placed from a cableway system. This cableway would also be necessary to move equipment used for excavation of the dam abutments.

Most of the construction facilities such as labour camp, shops, storage areas, would be located on the level area on the right bank of the Homathko River, downstream of the dam near the Klattasine Creek confluence. The extent of potential debris flows in Klattasine Creek should be examined before locating construction facilities in this area.

(c) <u>Capital Cost Estimate</u>

The total construction cost of the Waddington Canyon Project, excluding corporate overhead and interest during construction, is estimated to be $685.0 million, expressed in October 1983 dollars as detailed in Table 7-1. Expenditures by fiscal years are shown in Table 7-2.

This cost does not include any allowance for constructing an access road from Bute Inlet, as it is included in the cost of the Mosley Creek Project which would be constructed before the Waddington Canyon Project.

This estimate does not include any allowance for project licencing and environmental or socio-economic costs. Engineering study costs prior to final design are also excluded.

Section 7.3(e) suggests that only nominal rock support would be required in the underground powerhouse. However, a conservative approach was taken in the cost estimate and rock support similar to that required for Mica powerhouse was included.

This estimate includes the cost of the 230 kV transmission line to the collector switching station at the Mosley/Nude powerhouse and 50 percent of the cost of the collector switching station.

SECTION 8.0 - MOSLEY CREEK PROJECT

8.1 DESCRIPTION OF SITE

The Mosley Creek damsite is located in a canyon 28 km downstream from Twist Lake and 10 km upstream of the confluence of Mosley Creek and the Homathko River. The Mosley Creek powerhouse site is located 1 km upstream from this river confluence, and immediately upstream of where Tiedemann Creek enters Mosley Creek. As shown on the location plan, Fig. 8-1, a power tunnel and penstock of about 9 km length would provide a nearly direct connection between the power intake at the dam and the underground powerhouse.

Mosley Creek flows generally southward from Twist Lake between, on the east, Mount Success and, on the west Mounts Tellot, Tiedemann and Waddington. About 26 km downstream of Twist Lake the creek leaves a broad "U" shaped glacial valley with a moderate gradient of 0.3 percent, and enters an incised steep-walled canyon with a gradient of about 3 percent. This steeper gradient continues down the canyon for about 2.4 km to where the creek passes through a prominent "Z" bend.

Three axes have been considered as damsites, at distances of 250 m, 900 m and 2100 m downstream from the head of canyon. These axes have been designated during current feasibility studies as Axes M1, M2 and M3, respectively. Axis M1 is at approximately the same location as selected for the B.C. Power Commission in 1958 and, with minor difference, is in the area adopted for the layout presented in the B.C. Hydro Overview Report. Axis M2 was also investigated by the B.C. Power Commission, and considered during the course of their feasibility studies. Axis M3 lies just upstream of the "Z" bend and uses this feature for the project arrangement.

At Axis M1, the right abutment is precipitous in places with an average slope of 55° from the horizontal, and the left abutment is much flatter with an average slope of about 20°. Creek level is at El. 688 approximately, and from exploratory drilling lowest bedrock level appears to be between El. 660 and El. 670. Exploratory drilling in 1957 of the comparatively flat left abutment indicates a shallow overburden cover; however, drilling at that time in and close to the creek channel indicates alluvium with boulders and cobbles to depths of 15 to 20 m. Bedrock is composed of hard strong quartz diorite with dykes parallel to the prominent cliffs.

At Axis M2, both abutments rise at an angle of about 30° from the horizontal. Creek level is at about El. 680; and the depth of alluvium in the creek exceeds 25 m. Limited, somewhat inconclusive subsurface investigations were carried out at this site in 1957. Exploration indicates that the abutments are covered by 10 to 15 m of gravel and colluvium. On the riverbank a hole was drilled through coarse gravel to a depth of 21 m without encountering bedrock. Drill cores elsewhere indicated bedrock to be hard, strong sparsely jointed quartz diorite intruded by a few andesite dykes.

At Axis M3, the right abutment has an average slope of 30°, with a slope of up to 50° over the lowest 50 m height; the left abutment averages about 45° with precipitous portions. River level is approximately El. 642, and the depth of alluvium over bedrock appears to be about 20 m based on one 1982 drill hole. Overburden appears to remain at this depth upstream of the axis, and tends to reduce in depth downstream to essentially no alluvium near the "Z" bend.

Several avalanche tracks are evident along the left bank. The avalanche track closest to Axis M3 is about 300 m upstream.

The tailrace channel of the Mosley powerhouse would lie in the southerly flowing reach of Mosley Creek immediately upstream of the Tiedemann

Creek confluence. The underground powerhouse would be located under a gently sloping, densely forested slope. The lower 100 m of elevation rises at a slope of slightly less than 15°, with that above rising at about 30°. Rock outcrops have not, so far, been encountered on this slope, except for the extensive outcrops at river level and at considerable heights above. An average 5 m of overburden depth over this gentle, forested slope is currently assumed. There is no evidence of slide or avalanche activity in the powerhouse area.

8.2 DEVELOPMENT OF INTERIM PROJECT ARRANGEMENT

(a) General

The 1958 report for the B.C. Power Commission describes three alternative dam arrangements for the Axis M1 area. These alternative dam types were rockfill with an impervious core, rockfill with a concrete facing, and a concrete arch dam. Cost-comparison favoured selection of a rockfill dam, and uncertainty about availability of suitable core material placed emphasis on a concrete-faced rockfill arrangement.

For the B.C. Hydro Overview Report, a concrete gravity dam was selected for a layout developed about 60 m upstream of Axis M1.

All the above layouts were prepared for a maximum normal reservoir level of El. 2560 feet (El. 780 m).

(b) Damsite Selection

Based on an initial selection of a concrete-faced gravel dam, preliminary layouts were prepared at locations close to Axes M1, M2 and M3. With Axis M3 the power tunnel would be about 1.6 km shorter than with Axis M1. Dam heights at Axes M1, M2 and M3 would be about 125, 134 and 164 m, respectively, with maximum normal reserovir level El. 780. Axis M3 was found to be the most economic alternative.

In this preliminary study it was assumed that diversion and spillway costs would be similar for the alternatives, and that only nominal stripping would be required under the embankments. This latter assumption tended to favour the upstream layouts since significant depths of alluvium over extensive areas were known to exist under the upstream embankments.

In addition to economic preference, a layout could be developed at Axis M3 largely clear of interference from avalanche activity. Axes M1 and M2 lie under extensive avalanche tracks down the left side of the valley.

Accordingly, foundation drilling investigations in 1982 were concentrated in the vicinity of Axis M3, and all further layout studies were confined to this area.

(c) Dam Type

Initially the layout at Axis M3 was based on a concrete-faced gravel dam. However a concrete-faced rockfill dam, with steeper upstream and downstream slopes, was adopted to minimize exposure of the diversion tunnel intakes to potential avalanches down an existing track about 300 m upstream of the current dam axis. With the rockfill layout the dam axis was also moved slightly downstream from the original Axis M3.

An impervious core rockfill dam was not considered further since recent exploration continues to indicate the absence of any significant quantities of suitable impervious material.

A rollcrete embankment was given brief consideration; however, it was set aside at this time since the technology and construction methods remain to be proven for dams of the height proposed.

(d) Reservoir Level

On completion of a general arrangement for a concrete-faced rockfill embankment with maximum normal reservoir level El. 780, arrangement variations for reservoir levels of El. 760 and El. 800 were determined. Following cost analysis and results from preliminary power studies, a maximum normal reservoir level of El. 760 was adopted as being close to, and slightly below, the indicated optimum development.

When completing the feasibility studies a revised optimization study should be carried out with smaller reservoir level increments including levels below El. 760.

(e) Power Tunnel

Alternative power tunnel diameters were studied assuming use of a tunnel boring machine (TBM), or drill and blast methods. These studies indicated that tunnel costs with TBM excavation would be considerably less than by drill and blast methods for the range of tunnel diameters under consideration. Although the rock along the tunnel route is expected to be very hard, investigations have indicated that there would be no unusual penalty in TBM excavation up to at least 8 m diameter tunnels.

In these studies, and in selecting an optimum TBM tunnel diameter, the value of reduced hydraulic losses with TBM versus drill and blast methods, and with increasing tunnel diameters was included in the cost-benefit evaluation. For these studies an annual capital cost factor of 5.8 percent and a value of 21 mills/kW·h (including incremental capacity value), at site, were adopted.

(f) Powerhouse and Penstocks

Powerhouse arrangements during earlier studies indicated a preference for a surface powerhouse. Currently, preliminary comparison indicates an underground powerhouse would be more economic, primarily because of the shorter pressure penstocks.

In selecting the location of the underground powerhouse, increasing economy was found in locating the powerhouse as far as possible under the hillside away from Mosley Creek. Maximum rock cover was arbitrarily established by limiting the low voltage lead shafts to a maximum height of 150 m.

A detailed comparison of surface and underground powerhouses should be made when feasibility studies are resumed.

Alternative penstock arrangements were also studied, described as follows:

1. A high level bifurcation downstream of the surge shaft with high level butterfly shutoff valves and individual inclined and horizontal penstocks to the two turbines.

2. A similar arrangement except the high level butterfly valves were replaced by spherical shutoff valves immediately upstream of the turbines.

3. A single penstock from downstream of the surge shaft via an inclined shaft and horizontal tunnel to immediately upstream of the powerhouse where a bifurcation and spherical shutoff valves would be located.

Arrangement 1 was found to be marginally more economic than Arrangement 2. However, Arrangement 3 was found to be markedly more economic than either of the first two alternatives. Accordingly, this arrangement has been adopted for the Interim Project Arrangement presented in the next section.

In this analysis the value of varying hydraulic losses with different arrangements was accounted for in conjunction with varying capital costs. An annual capital cost factor of 5.8 percent and a value of 20 mills/kW·h were adopted for this study.

8.3 INTERIM PROJECT ARRANGEMENT

(a) General

The interim arrangement is shown on Figs. 8-1 and 8-2 and would comprise a rockfill dam with concrete facing in the vicinity of Axis M3; facilities for river diversion during construction; a pair of spillway tunnels; and power facilities in the left bank consisting of a power intake, a power tunnel with surge shaft, a penstock with bifurcation and turbine shutoff valves, an underground powerhouse, and a tailrace tunnel. The reservoir would have a 30 m drawdown depth to provide flow regulation for the Mosley Creek and Waddington Canyon powerhouses. Power studies showed that the Mosley generating station would be limited in operation to 9 months per year. Thus while the annual plant capacity factor would be about 62 percent, the effective capacity factor for weighting of power conduit losses would be about 83 percent.

(b) Dam

A rockfill embankment with an upstream concrete face was selected. The upstream and downstream slopes would be 1.4H:1V and the dam crest would be at El. 767.5 with a parapet wall providing freeboard of 9 m above maximum normal reservoir level and 6 m above maximum flood level. The embankment has been moved as much as possible in a downstream direction, just short of spilling down into the river channel at the "Z" bend. Bedrock level under the dam is indicated to be at El. 620, with the resulting maximum dam height being about 148 m. The crest length of the embankment would be 270 m.

The concrete facing would vary in thickness from 0.3 m at the crest to 0.75 m at its lowest level, and would be reinforced throughout, with steel continuous through some of the joints.

The concrete facing would be placed on a bedding zone of sand and gravel, graded and compacted to form a dense base. The bedding zone would be protected by a layer of shotcrete to avoid ravelling of the surface. Perimeter and vertical joints would be sealed with two or three water stops and sealant, depending on location. The concrete facing would be connected to a perimeter slab cast on sound bedrock. This slab would vary in width from 4.0 m at the crest to 7.5 m at the lowest point, would be reinforced throughout and anchored into unweathered bedrock. A grout curtain would be installed through the perimeter slab.

Alluvium would be excavated from under the upstream two-thirds of the embankment, including preparation for the perimeter slab area; however, alluvium would be left in place under the remainder of the embankment. All overburden would be removed from the abutments before fill placement; however, stripping of weathered rock would only be required for the perimeter slab foundation.

An impervious blanket of silty-sand (obtained from the Tiedemann Creek area) with a sand/gravel cover would be placed over the concrete facing to a level equal to one-third of maximum dam height. The slope of this blanket would be 2.5H:1V. This blanket would be placed to give additional sealing and slab protection in the lower part of the canyon where repairs, if required, would be very difficult.

(c) Diversion

Diversion during construction would be through two 5.0 m diameter D-shaped tunnels in the right abutment, each about 550 m in length. Closure gates would be 4.0 m wide x 7.7 m high. The tunnels would be permanently plugged with concrete following start of reservoir filling. In future studies utilization of one tunnel for low level outlets should be considered. Maximum velocity in the diversion tunnels would be 8.7 m/s with a flow of 340 m^3/s, based on a flood of 30-year frequency.

The upstream cofferdam would consist of an 11.0 m high sheet pile cellular structure filled with sand and gravel with crest at El. 660.0. A concrete cutoff wall would be constructed through the alluvium to bedrock directly in front of and sealed to the cellular cofferdam. This type of cofferdam was selected primarily so that the diversion tunnel inlets could be located close to the dam, to minimize risk of blockage from avalanches which have occurred a short distance upstream.

A small, 5 m high downstream cofferdam would be constructed of dumped rockfill and a semi-impervious silty-sand blanket.

(d) Spillway

Two concrete-lined spillway tunnels would be provided in the right abutment. The tunnels would have submerged intakes and free-flow downstream of the gates. The spillways would be used for release of flood waters, operational releases and for partial, emergency reservoir evacuation purposes. No other outlet or spillway would be provided. The design capacity for passing the PMF with a 3 m flood surcharge would be 1560 m^3/s.

Each concrete-lined spillway tunnel would be 5.8 m in diameter and about 330 m long. (Subsequent hydraulic calculations indicate that the tunnel diameter could be as large as 8.0 m.) An 80 m high tower would contain 4.0 m wide x 8.0 m high operating and bulkhead gates. A bellmouth entrance would be provided at the upstream end of these tunnels. The intake invert would be at El. 690 and the maximum head on the sill would be 73 m. A coarse trashrack and stoplog guides would be provided upstream of the bellmouth entrance.

Air would be introduced into the tunnels via an airshaft in the tower, and aeration devices would be provided in the tunnel to minimize the risk of cavitation damage under the design free-flow conditions.

The tunnels would curve gently from intake level to about El. 635 at the downstream portal. Here, flip buckets designed for maximum throw and dispersion would direct the discharge down the channel below the "Z" bend. Maximum flow velocity at the bucket would be about 40 m/s.

(e) Power Facilities

The power generation facilities, shown on Figs. 8-3 and 8-4, would comprise a power intake, an 8400 m long power tunnel with surge shaft, a single penstock 760 m long branching to two unit penstocks, an underground powerhouse and a 390 m long tailrace tunnel. The combined capacity of the two units would be 290 MW. The annual capacity factor would be 58 percent.

The power intake would be sited on the left bank close to the rockfill dam, with a short approach channel within the reservoir to accommodate minimum level flows. The invert would be set at El. 718 to avoid air entrainment with minimum reservoir level El. 730. The intake structure would house an operating gate 4.3 m wide x 6.5 m high and an upstream bulkhead gate. The bellmouth intake would be provided with a set of trashracks which could be cleaned from the gate house deck.

The power tunnel would be excavated by a tunnel boring machine and would be 5.5 m diameter. The maximum flow velocity would be 3.6 m/s, and the weighted average flow velocity would be 2.3 m/s. Apart from a short section of concrete lining at the intake, it is anticipated that no lining would be required except for occasional use of shotcrete and rockbolting. The tunnel would extend 45 m downstream from the surge tunnel to the penstock concrete transition. Between these points a 200 m long adit 5.5 m in diameter would be required to provide construction access to the power tunnel. On completion of all tunnel work, the adit would be sealed by a concrete plug with a watertight door for maintenance and inspection purposes.

The shotcrete-lined surge shaft would be located to one side of the power tunnel, connected by short lengths of 4.5 m diameter surge tunnel and riser shaft. A simple surge shaft, 12 m in diameter, would rise 115 m to a concrete collar with cover at El. 801. The top of the surge shaft would extend above grade, with 5 m of overburden assumed to overlie bedrock. This surge shaft was designed for full load rejection and for a 50 percent load demand (one unit 100 percent). (Subsequent hydraulic calculations indicated that the invert of the surge tank should be 10 m lower and the soffit of the power tunnel should be changed accordingly.)

A rock trap approximately 20 m long would be provided immediately upstream of the concrete penstock transition, to prevent movement of rock spalls down the penstock.

The concrete-lined penstock, 3.5 m in diameter, would extend for 430 m from the upper transition and vertical bend down a steeply inclined shaft to a lower combined vertical and horizontal bend. The penstock would continue with steel lining at 3.2 m diameter over an additional distance of 325 m to the bifurcation.

The steel-lined bifurcation would have an inlet diameter of 3.2 m and two outlets of 2.1 m diameter. A 2.1 m diameter spherical turbine inlet valve would be installed at the end of each penstock, and a short section of 2.1 m diameter penstock would complete closure to the turbine inlet of the same diameter. Maximum design head on the spherical valves and turbine scroll cases, including 30 percent transient pressure rise, would be 570 m.

The underground powerhouse, shown on Fig. 8.4, would contain two units, a control bay at the west end of the powerhouse, and a service/erection bay on the east side. The cavern including service and control bays would be about 62 m long x 18.5 m wide, with a maximum depth from roof soffit to draft tube base of 39 m.

Special provisions were included in the design to allow for the future addition of a single Nude Canyon generating unit. These special provisions include:

1. Extension of the powerhouse excavation, from generator floor level to cavern soffit, by 10 m beyond the service bay in an easterly direction.

2. Widening of the roof arch by 1.6 m, to a total span of 20 m to suit Nude Canyon requirements.

3. Increasing the span of the overhead bridge crane, also by 1.6 m.

With these extensions, it is believed that the cavern could be extended, and other underground excavation completed, for the Nude Canyon generating unit by careful excavation techniques which would minimize interruption of service of the Mosley Creek units.

For the indicated high quality of the rock no special concrete arch support appears necessary, and only suspended ceilings would be provided to safeguard equipment and personnel from minor spalling of rock.

Two 4.0 m diameter unlined low voltage bus lead shafts, 150 m long, would connect to the surface bus terminal equipment buildings where the bus would connect to the generator transformers. This building would be located next to the Homathko collector switching station. An elevator would provide access between the collector switching station, bus terminal buildings and the powerhouse generator floor. A 2 m diameter air shaft of 125 m length would also provide a vent to the surface from the draft tube gallery. Access would be provided to the south side of the service/erection bay by a 10 m wide x 7.5 m high D-shaped, 350 m long tunnel. This tunnel would slope up an 8 percent grade on the east side of the tailrace tunnel to emerge on the creek bank well above the tailrace tunnel.

The two Francis type hydraulic turbines would each be rated at 141 100 kW under a net head of 372.1 m, and would operate with a synchronous speed of 450 rpm. The anticipated maximum output of each turbine would be 147 200 kW at a maximum net head of 382.7 m. The turbine throat diameter would be 2050 mm. The setting of the turbine distributor centreline would be 6.25 m below minimum tailwater level, defined for this plant as that with only one unit operating at half discharge and with no spill.

An average power flow of 50.1 m^3/s (average runoff minus spill) was adopted based on Overview Report data. This value could be subject to minor revision during the next phase of feasibility studies. The maximum discharge of both turbines at maximum output would be 85.0 m^3/s. For conduit size optimization purposes, a weighted average power flow of 55.3 m^3/s was adopted.

The powerhouse would be equipped with an overhead bridge crane of 400 t main hoist capacity, and 20 t auxiliary hoist capacity.

The two generators would be of the umbrella type, each equipped with a single combined thrust and guide bearing. They would be rated 145 MW (152.6 MVA, 0.95 pf) at a voltage of 13.8 kV. The generator terminal equipment would be connected by isolated phase 13.8 kV bus in the bus lead shafts to the two 3-phase banks of 13.8/230 kV generator transformers on the surface. The transformers would be connected by 230 kV CGI bus to the Mosley Creek section of the Collector Station 230 kV CGI switchgear.

Powerhouse, dam and switchgear operations would be controlled from the plant control room underground, from the Homathko Collector Switching Station located above, and from the System Control Centre at Burnaby Mountain.

Dam station service power would be provided by a 12 kV transmission line from the collector switching station. Diesel generators would provide standby power.

Draft tube flows would pass into a common manifold and gate chamber before conveyance through a single 4 m wide x 9 m high tailrace tunnel of about 390 m length. Hoistway and gate storage provisions would be made in the manifold for installing and removing the draft tube gates. The tailrace tunnel would be unlined except for a short concrete collar at the portal. Maximum flow velocity with both units operating at capacity would be 3.3 m/s; and the weighted average flow velocity would be about 2.8 m/s.

The creek channel for a distance of about 280 m downstream from the tunnel portal would be widened and deepened to reduce the backwater loss between the tailrace tunnel and the Waddington Canyon reservoir. With maximum powerflows from the combined Mosley and Nude powerplants the backwater above Waddington maximum normal reservoir level El. 325 would be 0.5 m to the tunnel portal. With 1.5 m pondage drawdown of the Waddington reservoir, the minimum tailwater level at the tailrace tunnel portal would be El. 324.4 with one unit operating at half discharge. The volume of excavation to obtain the above tailwater levels is estimated to be about 15 000 m^3 of which approximately half is anticipated to be bedrock.

(f) Site Access

As shown on Fig. 8.3, the permanent access road from the Waddington Canyon Project would approach the Mosley Creek powerhouse area along the left side of Tiedemann Creek. It would join the Mosley Creek powerhouse to damsite access road on the right bank of Mosley Creek in the vicinity of a short span bridge over the creek just 80 m upstream of the tailrace tunnel. From the portal area of the powerhouse access tunnel, a local access road with an average gradient of 8 percent would be required up the slope above the powerhouse to access the Homathko Collector Switching Station and the power tunnel portal area.

8.4 CONSTRUCTION CONSIDERATIONS AND COST ESTIMATES

(a) Construction Schedule

The proposed construction schedule (Fig. 8-5) indicates a construction period of 6 3/4 years from award of the first contract at the start of Year 1 to the in-service date for the first 145 MW unit in the fall of Year 7; the second unit would follow by approximately 4 months. Project approval would be required at least 6 months prior to award of the first contract.

River diversion would take place early in Year 4, and the reservoir would be filled in the summer of Year 7.

The first 2 years of the schedule would be used primarily to construct access roads to the project location. The main activities governing the overall duration of project construction are in the following sequence: project access roads - labour camp and site preparation - diversion tunnels - cofferdams - rockfill dam and concrete facing - reservoir filling.

Construction of the following features would take place concurrently with dam construction for the plant to be operational when the reservoir is filled: spillway tunnels, power tunnel and penstock, powerhouse and switchyard including generating equipment installation.

For the preparation of the construction schedule conventional methods of construction and production rates similar to those achieved on other hydroelectric projects were assumed, except for the power tunnel. A tunnel boring machine was assumed for the design and construction of the 8.4 km long power tunnel. A winter shutdown was assumed for most activities except for underground work.

(b) Site Facilities

The powerplant would be located 10 km, by road, from the damsite. The main construction facilities would probably be installed in the Tiedemann Creek area near the powerhouse, with other facilities near the damsite.

(c) Construction Cost Estimate

The total construction cost of the Mosley Creek Project, excluding corporate overhead and interest during construction, is estimated to be $726.8 million, expressed in October 1983 dollars as detailed in Table 8-1. Expenditures by fiscal years are shown in Table 8-2.

This estimate does not include any allowance for project licencing and environmental or socio-economic costs. Engineering study costs prior to final design are also excluded.

It should be noted that the full cost of constructing a permanent access road from Bute Inlet (approx. $150 million) is included in this capital cost estimate. Also, 50 percent of the cost of the collector switching station is included.

Section 8.3(e) suggests that only nominal rock support would be required in the underground powerhouse. However, a conservative approach was taken in the cost estimate and rock support similar to that required for the Mica powerhouse was included.

SECTION 9.0 - NUDE CANYON PROJECT

9.1 DESCRIPTION OF SITE

The Nude Canyon damsite is located on the Homathko River, approximately 9.6 km upstream from the confluence of Mosley Creek and the Homathko River. Nude Creek is located a further 2.3 km upstream of the damsite. Nude Canyon lies almost equidistant from Success Mountain on the right and Reliance Mountain on the left.

The Nude Canyon powerhouse site is located on Mosley Creek 1 km upstream from the confluence with the Homathko River. Underground space at this site would be shared with the generation facilities for the Mosley Creek Project. As shown on the location plan, Fig. 9.3, a power tunnel and penstock of about 7.6 km overall length would provide a nearly direct connection between the power intake at the dam and the powerhouse, with a dog-leg offset of about 0.7 km.

The damsite, shown on Fig. 9-1, is located at the bottom of a V-shaped gorge approximately 400 m downstream of Bellamy Creek which flows into the Homathko River from the right. At this site river level varies between El. 550 and El. 555 depending on flows and location. The left abutment, with an average slope of about 48°, is sparsely treed. Above the proposed crest level this slope is irregular in shape and moderately weathered. The relatively smooth right abutment has an average slope of about 40°. Overburden consists of minor talus deposits on the lower slopes and a narrow gravel bar in the river channel. About 200 m downstream from the tentative dam axis the riverbed widens on the right to form an embayment partially infilled with sand and gravel. At the dam axis it is estimated that the alluvium depth would be a maximum of 5 m.

For description of the powerhouse site refer to the Mosley Canyon Project, Section 8.1 and Fig. 9-3.

9.2 DEVELOPMENT OF INTERIM PROJECT ARRANGEMENT

(a) Dam Type

The B.C. Hydro Overview Study selected an earthfill embankment with an impervious core. A dam height of 210 m for a maximum normal reservoir level of El. 744 m was selected as suitable for the scheme involving diversion of Taseko Lake into the Homathko River watershed. Average flows at the damsite would have been more than doubled with the Taseko Diversion.

It now appears unlikely that suitable core material will be found within economic distance of the damsite. Furthermore, sand and gravel embankment shell material would have to be imported from the Tiedemann Creek area over a lengthy haul route involving, in part, an expensive access tunnel.

Accordingly, the only embankment type which seems practical for this damsite would be a rockfill dam with a concrete face. This type of dam would have been well beyond precedence for the dam heights originally considered.

With the frequent spacing of avalanche tracks in the immediate site area, it was recognized that avalanche hazards would exist at a number of locations over an embankment dam and its attendant construction diversion facilities. As well, permanent operating facilities such as the power intake and chute spillway could be at risk throughout the project life.

With these considerations a concrete arch dam was selected for interim feasibility project layout purposes. The valley shape with a crest width to height ratio of 2.5 to 3.0 indicates that a concrete arch dam would be a suitable choice for development of this difficult site. In addition, imported construction material - concrete aggregate - would be minimized in selection of this dam type.

(b) Reservoir Level and Dam Height

An initial general arrangement for an arch dam, 215 m high, was prepared for a maximum normal reservoir level at El. 756. This level was selected since it would provide for complete head development to the tailwater of a possible Bowl Lake generating station. However, in a cost-benefit analysis, it was found that this reservoir level and dam height were clearly well beyond economic limits. It was recognized that development of all the available head might only be justified with diversion of flows from Taseko Lake.

Following a study of incremental costs versus incremental benefits, a reservoir level of El. 635 was selected as being close to optimum development. In this optimization study the incremental costs of the arch dam predominated, with incremental costs of generating capacity and penstocks being a minor component. An annual capital cost factor of 5.8 percent and an energy value of 21 mills/kW·h were adopted in this analysis.

In determining arch dam concrete volumes for the various dam heights considered in the optimization study, the most efficient arch dam axis varied from a location 140 m to a location 360 m downstream from Bellamy Creek for reservoir levels of El. 756 to El. 635 respectively.

(c) Access to Damsite

In the the Overview Study an independent permanent access road, primarily by tunnel, was to be provided to the damsite from the vicinity of the Mosley Creek confluence. Early indication of what then appeared to be a major cost to provide this permanent access, suggested the possibility of utilizing a portion of the power tunnel, by enlargement, for construction access and ultimate "semi-permanent" access, to realize significant project cost savings. In using the term "semi-permanent" it was considered that for any major repairs at the damsite or equipment movements to or from the damsite, the reservoir could be lowered and access through the tunnel effected through a bulkhead in the adit plug and through the power intake.

For this purpose a 300 m long adit to the power tunnel from a ravine approximately 4.5 km downstream from the damsite is currently indicated, with the power tunnel enlarged to 7.0 m wide x 6.5 m high for access purposes. The downstream portion of the power tunnel would be 5.5 m diameter and excavated by tunnel boring machine.

A 4-wheel drive type road would be built for construction access and would be retained to provide vehicular access to the damsite when the power tunnel is in use.

(d) Powerhouse and Penstock

Since it is proposed that a common Mosley-Nude powerhouse be developed, the studies described in Section 8.2(f) leading to the selected arrangement apply in general to the Nude Canyon Project.

9.3 INTERIM PROJECT ARRANGEMENT

(a) General

The interim arrangement is shown on Figs. 9-1 and 9-2 and would comprise a concrete arch dam, with low level outlets in the dam (service spillway) and an auxiliary spillway on the dam crest; a plunge pool downstream of the dam; and power facilities on the right side of the river consisting of a power intake, a power tunnel with surge shaft, a penstock with turbine inlet valve, an underground powerhouse, and a tailrace tunnel. The reservoir would be operated on a "run-of-river" basis, with only 1.5 m of pondage drawdown assumed below maximum normal reservoir level El. 635.

(b) Dam

The 98 m high double curvature concrete arch dam would have a developed crest length of about 280 m. The abutments would require a minimum of foundation preparations. Shaping of the abutments for the arch foundations would be limited to generally shallow excavation to unweathered rock, with additional excavation of the left abutment convexity to obtain a more regular "V" shape to avoid high stress concentrations. Overburden depth and depth of weathered rock is expected to be less than 2 to 3 m.

The peripheral rock area in contact with the arch dam would be consolidation grouted to a depth of 10 m. In addition, a grout curtain would be provided along the upstream periphery of the dam to a maximum depth of about 30 m. A system of drain holes and abutment drainage adits would be provided downstream of the grout curtain to control hydrostatic pressures underneath the dam and in the abutments.

(c) Diversion

Diversion during construction would be through a single 6.2 m diameter D-shaped tunnel 255 m long, located within the right abutment. Design flow capacity of this tunnel would be 230 m^3/s based on a flood of estimated 10-year frequency. With the competent rock it is expected that tunnel lining would not be required except at the portal structures. It was assumed that shotcrete lining would be required between the concrete-lined portal sections. Maximum velocity in the tunnel would be 8.2 m/s. This tunnel would be sealed permanently with a concrete plug behind a 4.0 m wide x 8.8 m high diversion closure gate after commencement of reservoir filling. To provide flexibility during diversion gate closure, a small gated sluice, subsequently filled with concrete, would be provided at a low level in the arch dam.

The 15 m high upstream cofferdam, with crest at El. 561.5, would be constructed in the dry with a central impervious core placed on bedrock and with rockfill shells placed on the riverbed alluvium. The maximum depth of alluvium is expected to be about 5 m. To facilitate construction of this cofferdam, a low first-stage embankment of dumped rockfill/gravel with an impervious blanket would be placed to accomplish initial diversion during a period of low flows.

The downstream cofferdam would be a very low embankment with an impervious blanket to provide a partial seal to bedrock.

(d) Discharge Facilities

(i) Low Level Outlets (Service Spillway)

Two low level outlets through the arch dam would be provided to control reservoir levels and releases during reservoir filling, to release normal flood spills and for emergency reservoir evacuation purposes. These outlets would be capable of discharging 360 m^3/s at maximum flood level El. 638. This capacity would be approximately that of a 300-year return period flood.

Releases would be controlled by 2700 mm diameter jet flow gates angled upwards to throw the jets as far as possible from the dam. A guard gate of the ring follower type would be provided immediately upstream of each regulating gate, and provision would be made on the upstream face of the dam for lowering a bulkhead gate to permit servicing of the guard gates.

The regulating and guard gates would be located in a housing cantilevered from the downstream face of the dam. The bellmouth inlet would be protected by a coarse trashrack to prevent large material from entering the steel-lined conduit.

(ii) Auxiliary Spillway

The spillway would be located on the crest of the dam. There would be four bays, two each side of facilities for the low level outlet bulkhead gates. The bulkhead gates would be operated by a 20 t gantry crane. Releases would be controlled by four radial gates 6.3 m wide x 3.5 m high, with crest level at El. 632. The discharge capability of this facility would be 775 m^3/s at maximum flood level El. 638.

Water released down the ogee slope would be broken up by splitter blocks which would cause considerable dispersion and energy dissipation in the air and reduce dynamic pressures in the plunge pool.

(iii) Plunge Pool and Weir

The energy of releases from the arch dam discharge facilities would be largely dissipated in a plunge pool with depths varying between 9 and 14 m depending on flow rate. Water levels in this pool would be controlled by a concrete gravity weir, approximately 10 m high and 30 m wide, located 120 m downstream of the dam.

The plunge basin would be lined with reinforced concrete secured by anchors to bedrock for a distance of 70 m downstream from the dam to accommodate the impact of discharges from the crest spillway.

Some bedrock erosion would be anticipated in the plunge pool downstream of the concrete lining at areas of impact from low level outlet operation. With an upwards deflection angle of 30° the jets would fall between 90 and 100 m downstream of the dam. These outlets would be oriented to provide for best possible cushioning of the jets in the plunge pool.

(e) Power Facilities

The power generating facilities, shown on Figs. 9-3 and 9-4, would comprise a power intake, a 7070 m long power tunnel with surge shaft, a single 500 m long penstock with turbine shut-off valve, underground powerhouse, and a 390 m long tailrace tunnel. The capacity of the single unit would be 185 MW, giving an annual plant capacity factor of 59 percent.

The power intake would be sited on the right bank approximately 100 m upstream of the arch dam. The invert would be set at El. 600 to provide ample submergence for prevention of air entrainment when the reservoir is drawn down by 1.5 m below maximum normal reservoir level El. 635. The intake structure would house an operating gate 3.3 m wide x 5.7 m high and an upstream bulkhead gate. The bellmouth intake would be provided with a set of trashracks which could be cleaned from the gate house deck. This structure would be designed to permit limited traffic through the structure to the enlarged power tunnel beyond, should semi-permanent access to the damsite via the power tunnel ever be called upon after commissioning of this project.

The upstream 4.0 km of the power tunnel would be excavated by drill and blast methods to a D-shaped section 7.0 m wide x 6.5 m high for construction and semi-permanent access purposes. A 300 m long adit of the same dimensions would extend from a nearby ravine to surface access road leading down to the Mosley-Nude powerhouse area. This adit would be sealed by a concrete plug and watertight bulkhead prior to reservoir filling.

The downstream 3070 m of this tunnel would be excavated by tunnel boring machine to a diameter of 5.5 m. This diameter is slightly greater than the indicated optimum size from cost-benefit studies, but the diameter was rounded up arbitrarily to equal the Mosley Creek power tunnel size for possible construction advantages. A 260 m long construction adit of the same diameter would extend into the power tunnel downstream from the surge shaft. This adit would be plugged at its interface with the power tunnel by a concrete plug and watertight bulkhead.

Considering the competent bedrock anticipated over the power tunnel length, the tunnel is assumed to be unlined between the concrete lining at the intake and the concrete transition to the penstock, except for occasional use of shotcrete and rock bolting. A rock trap approximately 20 m long would be provided immediately upstream of the concrete-lined penstock transition, to prevent movement of rock spalls into the penstock.

The maximum flow velocity would be 1.9 and 3.2 m/s in the enlarged upstream and in the downstream TBM sections, respectively. With weighted average power flow, the respective velocities would be 1.3 to 2.2 m/s.

The unlined surge shaft would be located to one side of the power tunnel about 50 m upstream of the penstock transition. A simple surge shaft, 6 m in diameter, would rise 115 m from tunnel invert level to a concrete collar with cover at El. 698. The surge shaft would be connected to the power tunnel by a short length of 4.5 m diameter surge tunnel. The top of the surge shaft would extend a minimum of 3 m above bedrock, which is assumed to be overlain by 5 m of overburden. For the dimensions described, this surge shaft was designed for full load rejection and for a 50 percent load demand. Load demand downsurge was originally determined for a two-unit Nude powerhouse, and should 100 percent load demand now be required for the single unit, either the power tunnel invert at the surge shaft would be lowered or the portion of the surge shaft below normal surge tank water levels corresponding to rated discharges would require some enlargement.

The concrete-lined penstock, 3.5 m in diameter, would extend from the upper transition and vertical bend down a steeply inclined shaft to a lower vertical bend over a distance of about 335 m.

From the lower bend, the penstock would continue with steel lining at 3.2 m diameter to a short penstock transition which would reduce the diameter to 2.9 m. A spherical turbine inlet valve and a short closure section would connect the penstock to the 2.9 m diameter turbine spiral case inlet. The overall length of steel-lined conduit would be 165 m. Maximum design head on the turbine inlet valve and turbine spiral case would be 410 m, including 30 percent transient pressure rise from turbine shutdown. Maximum penstock velocities would be 7.8 and 9.3 m/s in the concrete and steel-lined sections, respectively.

The underground powerhouse, shown on Fig. 9-4, would contain a single unit located beside the two Mosley units. The service/erection bay and the control bay was assumed to be operational when Nude Canyon construction commences. The additional length of the powerhouse would be 29 m and includes a 10 m extension of the Mosley service bay for construction of the Nude generating facilities. The span of this portion of the underground cavern would be 20 m and the maximum depth from roof soffit to draft tube base would be 40 m. For description of special provisions made for extension of the Mosley powerhouse refer to Section 8.3(e).

One 4.0 m diameter unlined low voltage bus lead shaft, 150 m long, would connect to the surface bus terminal equipment building which would house the main transformers and switching equipment. All access between the service/control bays and the terminal equipment building would be common to the Mosley powerhouse. A 2.0 m diameter air vent of 125 m length would be provided to the surface from the independent draft tube gallery.

The Francis type hydraulic turbine would be rated at 187 800 kW under a net head of 280.6 m, and would operate with a synchronous speed of 300 rpm. The turbine throat diameter would be about 2800 mm. The minimum setting of the turbine distributor centreline would be 4.4 m below minimum tailwater level, based on this unit operating at 1/2 output and no spill from the Mosley Creek reservoir.

An average power flow of 41.8 m^3/s (average runoff minus spill) has been adopted from the Overview Report data, and could be subject to minor revision during the next phase of feasibility studies. The maximum turbine discharge at rated output would be 75.0 m^3/s. For conduit size optimization purposes, a weighted average power flow of 51.8 m^3/s was adopted based on an annual plant capacity factor of 60 percent.

The Nude unit would be installed and maintained by the 400 t overhead bridge crane with auxiliary 20 t hoist, installed for the Mosley units.

The generator would be of the umbrella type with a single combined thrust and guide bearing. It would be rated 185 MW (194.7 MVA, 0.95 pf) at a voltage of 13.8 kV. The generator terminal equipment would be connected by isolated phase 13.8 kV bus in the bus lead shaft to the 3-phase 13.8/230 kV generator transformer on the surface outside the Homathko Collector Switching Station. The transformer would be connected by 230 kV CGI bus to the Nude Canyon section of the Collector Station 230 kV CGI switchgear.

Operations of the Nude unit and facilities at the Nude Canyon damsite would be controlled from the Mosley-Nude control room underground, the Homathko Collector Switching Station, and from the System Control Centre at Burnaby Mountain.

Dam station service power would be provided by a 12 kV transmission line from the collector switching station. Diesel generators would provide standby power.

Turbine flows would pass from the draft tube through a gate chamber and transition section to a 4 m wide x 9 m high tailrace tunnel of 390 m length. The draft tube gate hoist and gate storage provisions would be as provided for the Mosley units, with the gate chamber being interconnected. Maximum flow velocity in the tailrace tunnel would be 2.9 m/s, and the weighted average flow velocity would be 2.6 m/s.

Channel improvements downstream of the tunnel outlet are described in Section 8.3(e).

(f) Site Access

The Mosley-Nude powerhouse access is shown on Fig. 9-3, and is described generally in Section 8.3(f). In addition to powerhouse and collector station access which would have been developed for the Mosley Canyon Project, a branch road would be provided to the power tunnel adit and another branch road including 300 m of tunnel would be extended 3.7 km up the Homathko River to the portal of the intermediate power tunnel adit, considered for construction and semi-permanent access to the damsite.

9.4 CONSTRUCTION CONSIDERATIONS AND COST ESTIMATES

(a) Construction Schedule

The proposed construction schedule (Fig. 9-5) indicates a construction period of 5 3/4 years from award of the first contract at the start of Year 1 to the in-service date for the 185 MW generating unit in the fall of Year 6. Project approval would be required at least 6 months prior to award of the first contract.

River diversion would take place early in Year 4, and the reservoir would be filled in the summer of Year 6.

The first 2 years of construction would be used to provide access to the damsite. The main activities governing the overall duration of the schedule are in the following sequence: dam access road and tunnel - diversion tunnel and damsite facilities - cofferdams - dam abutment and foundation excavation - dam concrete and spillway gates installation - reservoir filling.

Extension of the Mosley powerhouse to accommodate the Nude unit would commence in Year 3 for the unit to be ready for testing when the reservoir is filled. The downstream portion of the power tunnel and penstock would be constructed at the same time as the powerhouse.

Conventional methods of construction and production rates similar to those achieved on other hydroelectric projects were assumed for preparation of the construction schedule, except for the power tunnel. It was assumed that a tunnel boring machine would be used in the downstream 3 km section of power tunnel, which is of the same diameter as the Mosley power tunnel. Conventional drill and blast techniques were assumed for the 4 km upstream section which would be enlarged for construction access. A winter shutdown was assumed for most activities, except for underground work.

(b) Site Facilities

The Nude Project is characterized by the difficulty of access to the damsite. For construction of the powerhouse and power tunnel, the same camp and work areas would be used as for construction of the Mosley powerhouse, namely the Tiedemann Creek area.

Aggregate for dam concrete would be either crushed rock produced at the damsite or aggregate hauled from the Tiedemann Creek area to a batch plant located at the damsite. Transportation of pre-mixed concrete from the powerhouse area to the dam could be considered.

It was assumed that the labour force working at the damsite would be transported from the camp area near the powerhouse. The alternative of installing a temporary camp at the damsite to reduce travel time should be considered.

Dam concrete would be placed using a cableway. Construction access roads would be constructed on the right bank from the dam crest level to the diversion tunnel inlet and outlet levels, but there would be no roads on the left bank. The cableway would be used to handle equipment for most of the abutment excavation work. Until construction of a cableway, helicopter access would be required for preparatory work on the left bank.

(c) Construction Cost Estimate

The total construction cost of the Nude Canyon Project, excluding corporate overhead and interest during construction, is estimated to be $305.8 million, expressed in October 1983 dollars as detailed in Table 9-1. Expenditures by fiscal year are shown in Table 9-2. A permanent two lane access road to the dam would add approximately $60 million to the total capital cost.

This estimate does not include any allowance for project licencing, environmental or socio-economic costs. Engineering study costs prior to final design are also excluded.

SECTION 10.0 - TATLAYOKO STORAGE PROJECT-NOSTETUKO AXIS

10.1 DESCRIPTION OF SITE

(a) General

The project area covering all alternatives considered for development of storage on Tatlayoko Lake extends from the Tatlayoko Lake outlet down the Homathko River for 11 km to the vicinity of Bowl Lake. In this reach the Homathko River occupies a broad valley infilled with flood plain deposits varying from silty sands to boulder gravels. Major tributaries entering the Homathko River in this reach are Ottarasko Creek, with confluence 2.7 km downstream of the lake outlet, and Nostetuko Creek with confluence 3.6 km further downstream. Each of these tributaries contributes about the same annual average runoff as the natural runoff from Tatlayoko Lake.

From limited water level records it is estimated that the natural range of lake levels varies from a probable high of El. 827.5 to a minimum no-flow level of about El. 826.2. The surface area of the lake is approximately 4060 ha.

Alternative storage damsites investigated include Axis T1, located at the Tatlayoko Lake outlet, Axis T2 located approximately 1100 m further downstream; and Axis T3 (the Nostetuko Axis) located 4.7 km downstream from the lake outlet and about 800 m downstream from Nostetuko Creek.

Axis T1 was selected for all early studies and was adopted for presentation in the Overview Study Report.

An alternative axis was examined at Bowl Lake which is approximately 11 km downstream of Tatlayoko Lake outlet. The Bowl Lake Axis is described in Section 11.0.

(b) Axis T1

The Tatlayoko Lake site, Axis T1, is crossed by the Homathko River at the extreme left flank, and by Ottarasko Creek at the extreme right flank. Here the valley is about 900 m wide between rock abutments, with ground level varying from El. 826 on the banks of the Homathko River to El. 842 at the base of the abutment near Ottarasko Creek. The average surface of the floodplain is about El. 830. The right abutment rises steeply to a knoll at about El. 950; and on the left, the abutment also rises steeply to a terrace at about El. 850.

(c) Axis T2

This axis is crossed by the Homathko River at about mid-axis and by Ottarasko Creek at the extreme right flank. The valley is slightly narrower here than at Axis T1, with about 750 m between the steeply rising right and gently rising left abutments. A knoll of bedrock forms the abutment on the right with a top elevation at about El. 860. The left abutment rises to a bedrock knoll at about El. 900. The ground level on the floodplain varies from about El. 820 near the Homathko River to about El. 825 near Ottarasko Creek and about El. 835 at the left flank.

(d) Axis T3 (Nostetuko Axis)

At the Nostetuko Axis the Homathko River lies against a steep rock abutment on the left which rises to about El. 930. The right abutment, largely masked by overburden, rises gently at a slope of about 20° from the horizontal to above El. 1000. River level is at approximately El. 795 with the present channel being 40 to 50 m wide. At dam crest level the river valley is about 360 m wide.

10.2 DEVELOPMENT OF INTERIM PROJECT ARRANGEMENT

(a) General

In the Overview Study development of storage at Tatlayoko Lake was based on raising the lake level to El. 836 m, approximately 9 m above normal level, with an embankment dam and release works in the vicinity of Axis T1. Ottarasko Creek would have been diverted into the lake.

(b) Alternative Dam Axes

The extent of seepage cutoff under and adjacent to the alternative axes was investigated. Recognizing the large area and high cost of deep cutoff walls at Axis T1, there was a tendency to favour the downstream axes where the extents of cutoff area are much smaller.

Axis T3 was favoured over Axis T2 for the following reasons:

1. Flows from the Nostetuko River into the reservoir would increase the flows regulated by the Tatlayoko Storage Project.

2. The single drill hole at Axis T2 indicated the presence of a significant layer of soft, likely compressible material in the riverbed alluvium.

Accordingly, layout studies were concentrated on development of Axis T3.

Subsequent to receiving the results of geophysical investigations along the river channel, a preliminary layout was prepared for Axis T4, which is about 150 m downstream of the confluence of the Homathko and Nostetuko rivers. The purpose of this layout was to aid in decision-making on the next phase of subsurface investigations. It was recommended that further geophysical survey work and drilling be carried out in the vicinity of Axis T4, as well as in the Axis T3 area.

(c) Alternative Reservoir Levels

Two alternative maximum normal reservoir levels were studied:

1. Maximum normal (raised) lake level would be El. 836, with 10 m of drawdown. No outlet channel improvements would be required.

2. Maximum normal lake level El. 827.5 m, with 10.5 m of drawdown. Channel improvements at Tatlayoko Lake outlet to achieve these lower levels would be required.

In Alternative 2 above, maximum lake level El. 827.5 was selected as the apparent historical maximum lake level, based on an assessment of limited lake level data. The drawdown of 10.5 m was arbitrarily adopted to provide about the same live storage as available with Alternative 1.

Comparative embankment dam layouts at Axis T3 for the two alternative maximum normal lake levels were prepared in conjunction with a study of channel improvements at the outlet of Tatlayoko Lake. In cost-comparison the saving from reducing the dam height by 8.5 m exceeded by a significant margin the cost of channel excavation at the lake outlet. In this study no account was taken of possible social and environmental implications.

With the indicated cost advantage, studies of the Nostetuko Axis were developed further with maximum normal lake level El. 827.5.

(d) Alternative Dam Types

Initially because of apparent unavailability of impervious material in the nearby region of the sites, gravel embankments with upstream concrete facing were studied for the alternative dam heights for the Nostetuko Axis (as well as for the Bowl Lake Axis). It was quickly apparent that the cost of this embankment type would be excessive because of the large excavation required for the concrete facing base slab and generally under the upstream two-thirds of the embankment foundation.

Review of impervious material availability indicated that suitable till material was present along the lakeshore about 5 km upstream from the lake outlet and approximately 15 km from the damsite by road. Accordingly, an embankment design with an impervious core and gravel shells, with a cutoff wall to bedrock under the core, was adopted for alternative dam height studies and for the Interim Project Arrangement.

10.3 INTERIM PROJECT ARRANGEMENT

(a) General

The interim arrangement is shown on Figs. 10-1 and 10-2 and would comprise an earthfill embankment dam located approximately at Axis T3; facilities for river diversion during construction; low-level outlet works; and a free-crest spillway located in a saddle area about 900 m south of the damsite. The reservoir would be operated over a range of 10.5 m to provide flow regulation for the downstream projects.

(b) Dam

An earthfill embankment dam with impervious core and sand/gravel shells would be constructed with upstream and downstream slopes of 2.25H:1V and 2.0H:1V, respectively. With crest at El. 835.5, 8 m of freeboard would be provided above maximum normal lake level El. 827.5, with this freeboard being reduced by 3 m during passage of the Probable Maximum Flood (PMF). Maximum dam height would be 40 m above the alluvial foundations, and the dam crest length would be 360 m.

Foundation preparations would consist of removal of organic matter and general leveling to facilitate construction. A concrete cutoff wall of about 40 m maximum depth would be installed through the alluvium to bedrock.

The impervious core would be located centrally within the embankment, and would be provided with transitions, filter and drainage zones between core and pervious shells. A special zone of core material mixed with bentonite would provide a tight seal to the cutoff wall below. Riprap would be placed over the drawdown zone on the upstream slope, and slope protection would be provided downstream. The cofferdams would be integral with the embankment dam.

(c) Diversion

Diversion during construction would be through a single 6.0 m diameter D-shaped tunnel located in the left abutment with a length of about 310 m. Shotcrete lining would be provided except at the upstream and downstream portals, and centrally as provision for outlet works, where concrete lining would be provided. On the upstream side a 60 m long approach channel would be excavated through the alluvium to the portal. A similar exit channel 170 m long would be required. The upstream channel in particular, considering permanent operation of this facility, would be stabilized against erosion by shotcrete and mesh protection. The diversion design discharges would be 215 m^3/s based on a flood of 30-year frequency.

The upstream cofferdam would be approximately 18 m high constructed of dumped rockfill with an impervious blanket. A smaller cofferdam similarly constructed would be provided downstream. The impervious blanket would be removed from the downstream cofferdam prior to reservoir filling. A line of drainage wells would be provided both upstream and downstream within the cofferdammed area to control seepage during embankment foundation preparation.

Closure of the diversion tunnel would be preceeded by placement of a coarse trashrack at the entrance portal. The closure gate would be lowered down a gate shaft located near the dam axis at the left abutment. With the gate in place the low level outlet works would then be completed.

(d) Low Level Outlet

A concrete plug containing a 2.6 m diameter steel-lined conduit would be constructed immediately downstream of the diversion closure gate, against the concrete liner installed prior to diversion. At the downstream end of this plug a 2.6 m diameter hollow cone valve would control releases through the outlet. The capacity of this valve would be 75 m^3/s under minimum reservoir level El. 817. This discharge was chosen to be equal to the maximum flow rate of the Nude Canyon turbine.

The diversion closure gate would be used as a guard gate and for servicing of the outlet valve. Downstream of the valve the tunnel would be protected by a steel liner over a distance of about 25 m to a jet deflector. The gate shaft would be about 37 m deep and would contain the diversion closure gate slot, an air shaft and manway access. Construction and maintenance access would be from the downstream end of the tunnel.

(e) Spillway

The free-crest spillway would be constructed in a saddle approximately 900 m south of the Homathko River. The low ogee weir, with crest El. 828, would provide 0.5 m freeboard with maximum normal reservoir level El. 827.5. Head on the weir would be 2.5 m during passage of the PMF, estimated to be 565 m^3/s.

The 75 m wide weir would have a 400 m long approach channel. Downstream of the weir a short protective concrete apron would be provided to ease the flows onto an unlined bedrock channel. Flows from this free-crest spillway would be returned to the Homathko River channel at a point about 1000 m downstream of the dam.

The weir crest would be surmounted by a bridge which would carry the main access road to the damsite.

(f) Lake Outlet Canal

As shown on Fig. 10-3, the river channel would be altered to permit lowering of the lake by excavation of a 1200 m long canal from Tatlayoko Lake outlet. To facilitate construction the canal inlet would be located to the right of the natural outlet to remain generally clear of the river channel, with only one crossing of a meander. The entire excavation is expected to be in pervious alluvium, with side slopes of 3H:1V assumed, above a 2 m bottom width.

With an inlet invert El. 813.3, the canal would be capable of passing 30 m^3/s at a velocity of 0.6 m/s with minimum lake level El. 817. This design velocity was selected to ensure formation of an ice cover during periods of low temperature operation. However, the design capacity may have to be increased if greater releases are required for power generation at the Nude Canyon Project.

(g) Site Access

Access to the site would be via a bridge across the spillway and a gravel road to the left abutment of the dam.

10.4 CONSTRUCTION CONSIDERATIONS AND COST ESTIMATES

(a) Construction Schedule

The proposed construction schedule (Fig. 10-4) shows a construction period of 4 years from award of the first contract in the spring of Year 1, to completion of the low level outlet facilities late in Year 4.

The first year would be used mainly to improve and construct access roads and prepare the site. River diversion would take place in the spring of Year 3, and the Tatlayoko Storage Project would be fully operational by the end of Year 4.

Construction of the cut-off wall below the dam embankment is critical to the overall schedule and it appears that it should continue during the winter months, whereas all other activities would be discontinued in winter. The sequence of construction of the cutoff wall in conjunction with embankment placement should be studied in more detail as well as increasing excavation in the dam core area to reduce the cut-off wall area.

Overburden material from the Tatlayoko outlet channel excavation was assumed to be usable as fill for the dam shell.

It should be noted that with the proposed scheme, no water can be released from Tatlayako Lake after closure of the diversion gate, until such time as either the low-level outlet valve in the diversion tunnel is operational, (approximately 3 months) or the Lake level has increased to the spillway crest elevation.

(b) Site Facilities

Location of construction facilities and spoil areas should not pose any problems as the terrain is relatively level.

(c) Construction Cost Estimate

The total construction cost of the Tatlayoko Storage Project (Nostetuko Axis), excluding corporate overhead and interest during construction, is estimated to be $75.3 million, expressed in October 1983 dollars as detailed in Table 10-1. Expenditures by fiscal year are shown in Table 10-2.

This estimate does not include any allowance for project licencing and environmental or socio-economic costs. Engineering study costs prior to final design are also excluded.

SECTION 11.0 - TATLAYOKO STORAGE PROJECT - BOWL LAKE AXIS

11.1 DESCRIPTION OF SITE

The Bowl Lake Axis is located 11 km downstream from the Tatlayoko Lake outlet and 6 km downstream from the Nostetuko Axis, as shown on the location plan of Fig. 11-1. The Bowl Lake site is under consideration as a storage and power generation project, alternative to a storage only project at the Nostetuko site.

The Bowl Lake Axis lies in a canyon about 700 m due south of Bowl Lake. In addition to the main canyon site, a topographic low occurs between rock outcrops on the right abutment about 600 m west of Bowl Lake. The Stonsayako River enters the Homathko River from the southeast about 900 m downstream from the axis.

At the main dam axis the river is confined in a canyon between rock slopes from river level, at about El. 755, to about El. 850 on the right abutment and to heights in excess of El. 1000 on the left. Along the dam axis the river width is about 80 m, and the canyon width at the dam crest level is about 430 m. The right abutment has an average slope of about 30° from the horizontal; the left abutment slope rises initially at about 40° before flattening to an average slope of about 30°.

11.2 DEVELOPMENT OF INTERIM PROJECT ARRANGEMENT

(a) General

For the B.C. Hydro Overview Report, a Bowl Lake Project was presented which was based on a storage dam at the Tatlayoko Lake outlet, with a long power tunnel to a powerhouse located in the vicinity of Bowl Lake. This project was considered only as a possible generation component in the Scheme of Development with diversion of flows from Taseko Lake into Tatlayoko Lake.

An initial preliminary cost comparison indicated the possibility that a comprehensive development near Bowl Lake could provide economic energy generation, in addition to providing flow regulation for the downstream projects. Accordingly, layout studies at the Bowl Lake Axis were developed in more detail for comparison with the "storage only" Nostetuko Axis.

(b) Alternative Reservoir Levels

As described in Subsection 10.2(c) for the Nostetuko damsite, two alternative reservoir levels were studied for the Bowl Lake site. It was also concluded for this site that there is economy in selecting a maximum normal lake level El. 827.5, with a reduced dam height and river channel improvements at the lake outlet, instead of the previously selected maximum normal lake level El. 836 without river channel improvements.

Reservoir level and flow conditions for the Bowl Lake Axis alternative have been taken as equal to those for the Nostetuko Axis.

(c) Alternative Dam Types

Similarly, as described in Subsection 10.2(d) for the Nostetuko Axis, a concrete-faced gravel dam layout was studied initially, but this dam type was subsequently set aside when it was proven more economic to select an embankment dam with impervious core and gravel shells with a cutoff wall through the alluvium.

11.3 INTERIM PROJECT ARRANGEMENT

(a) General

The interim arrangement is shown on Figs. 11-1 and 11-2 and would comprise an earthfill embankment dam; facilities for river diversion during construction; outlet works and a free-crest spillway in the saddle dam area; and power facilities comprising a power intake, a power tunnel and penstock, and a surface powerhouse. The reservoir would be operated over a range of 10.5 m to provide flow regulation for the downstream projects.

(b) Dam

An earthfill dam with impervious core and sand/gravel shells would be constructed with upstream and downstream slopes of 2.25H:1V and 2.0H:1V, respectively. Crest level would be at El. 835.5 to provide 8 m and 5 m of freeboard above maximum normal and maximum flood reservoir levels, respectively. Maximum dam height would be about 80 m above the alluvial foundations, and the crest width would be approximately 420 m.

Foundation preparations would consist of removing organic matter, excavation of a 10 m deep core trench, and general leveling to facilitate construction. A concrete cutoff wall of about 35 m maximum depth would be installed from the core trench through alluvium to bedrock.

The impervious core would be located centrally within the embankment, and would be provided with transitions, filter and drainage zones between core and pervious shells. A special zone of core material mixed with bentonite would provide a tight seal to the cutoff wall below. On the bedrock abutments preparation for the core zone would include removal of weathered rock, blanket grouting, placing dental concrete where necessary, and a single line grout curtain which would be extended below the alluvium for sealing to the cutoff below.

Riprap would be placed over the drawdown zone on the upstream slope, and slope protection would be provided downstream. The cofferdams would be integral with the embankment dam.

(c) Diversion

Diversion during construction would be through a single 6.3 m diameter D-shaped tunnel located in the right abutment with a length of about 500 m. Shotcrete lining would be provided on the walls and soffit except at the upstream and downstream portals where the tunnel would be concrete-lined. On the upstream side a 90 m long approach channel would be excavated through the alluvium to the portal. A similar exit channel 50 m long would be required. The diversion design discharge would be 215 m^3/s based on a flood of 30-year frequency. This tunnel would be sealed with a concrete plug prior to reservoir filling.

The upstream cofferdam would be approximately 24 m high constructed of rockfill and an impervious blanket, placed directly on the alluvium. A 10 m high downstream cofferdam would be constructed similarly. The impervious blanket would be removed from the downstream cofferdam prior to reservoir filling. A line of drainage wells would be provided both upstream and downstream within the cofferdammed area to control seepage during embankment foundation preparation.

(d) Spillway

A free-crest spillway would be constructed in the saddle approximately 800 m northwest of the embankment dam. The low ogee weir, with a maximum height of 11 m, would be 70 m wide with crest El. 828.5 to provide 1.0 m freeboard with maximum normal reservoir level El. 827.5. Head on the weir would be 2.0 m during passage of the PMF, estimated to be 565 m^3/s.

Downstream of the weir a short protective concrete apron would be provided to guide the flows onto an unlined bedrock channel. Spillway flows would be returned to the Homathko River channel 1500 m downstream of the embankment dam.

The weir crest would be surmounted by a bridge to provide access to the outlet works.

(e) Outlet Works

The outlet works structure would be located on the northwest end of the free-crest spillway, with a gravity wall completing closure to bedrock on the right flank of the saddle.

Outlet works would be provided to allow flow releases as required for power generation at the downstream projects. The capacity of the outlets would be 75 m^3/s under minimum reservoir level El. 817, equal to the maximum flow rate of the Nude Canyon turbine. It is assumed that this flow rate would have to be provided with the Bowl Lake generating unit shutdown.

The outlet works would comprise two sluices with fixed wheel gates 2.5 m wide x 2.2 m high, with invert El. 812.5. Maximum head on these gates would be 18 m. Provision would be made for lowering bulkhead gates in a separate slot within the intake structure.

(f) Power Facilities

The power generation facilities, shown on Fig. 11-2, would comprise a power intake, a 310 m long power tunnel and penstock, and a surface powerhouse. The capacity of the single unit would be 22 MW, based on an annual plant capacity factor of about 60 percent.

The power intake would be sited on the right bank approximately 30 m upstream of the embankment dam axis. The invert would be set at El. 810 to provide ample submergence when the reservoir is drawn down to minimum reservoir level El. 817. The intake structure would house an operating fixed wheel gate 2.3 m wide x 3.2 m high and an upstream bulkhead gate. The bellmouth intake would be provided with a set of trashracks which could be cleaned from the gate house deck. Access to the free-standing tower would be via a single span bridge 25 m long.

The power conduit would comprise a 4.0 m diameter D-shape, shotcrete-lined tunnel about 170 m long; followed by a 3.4 m diameter concrete-lined inclined shaft, which includes an upper and lower bend; and a horizontal 3.1 m diameter steel-lined penstock connected to the turbine spiral case. The maximum flow velocity in the above penstock sections would be 2.4, 3.8 and 4.6 m/s, respectively. The maximum design head on the steel liner and turbine scroll case would be 99 m, including 30 percent transient pressure rise due to turbine shutdown.

The surface powerhouse would contain a single unit and a service/control bay, with an overall length of 23 m. The span of the powerhouse would be 15 m, and the overall height from roof to draft tube base excavation would be 26 m.

The Francis type hydraulic turbine would be rated at 20 400 kW with a rated net head of 68.7 m, and would operate with a synchronous speed of 300 rpm. The turbine would be capable of developing 22 300 kW under a maximum net head of 72.9 m. The turbine throat diameter would be 1980 mm, and the minimum setting of the turbine distributor centreline would be 2.0 m below minimum tailwater level.

An average power flow of 20.7 m^3/s (average runoff minus spill) has been assumed. The maximum turbine discharge with full reservoir would be 34.4 m^3/s, and a weighted average power flow of 23.6 m^3/s has been adopted based on an annual plant capacity factor of about 60 percent.

An overhead bridge crane of 100 t capacity, and an auxiliary hoist of 20 t capacity would be provided for unit installation and maintenance.

A 20-ton gantry crane would be located on the tailrace deck for operation of the draft tube gates.

The generator would be rated 22 MW (23.2 MVA, 0.95 pf) at a voltage of 13.8 kV. A 3-phase 13.8/69 kV generator transformer and 69 kV switchgear would be located adjacent to the back wall of the powerhouse.

The damsite areas would be provided with station service supply from the powerhouse, with standby diesel generation.

The control room in the powerhouse would be interconnected by the transmission line carrier for control from the Homathko Collector Switching Station, and from the remote Systems Control Centre.

(g) Site Access

As shown on Fig. 11-1 site access would be provided from the west end of Tatlayoko Lake by a 13 km long permanent access road. From a point on the left bank this access road would branch to the crest of the embankment dam, with the main road continuing down to the powerhouse across a short bridge. This powerhouse access road would branch to the saddle dam area where it would cross the spillway to the outlet works control structure.

11.4 CONSTRUCTION CONSIDERATIONS AND COST ESTIMATES

(a) Construction Schedule

The proposed construction schedule (Fig. 11-4) shows a construction period of 5 years from award of the first contract at the start of Year 1, to the in-service date for the 22 MW generating unit at the end of Year 5.

The first year would be mainly used to improve and construct access roads to the site. River diversion would take place early in Year 3, and operation of the Tatlayako Storage Project could commence in the fall of Year 5.

The main activities governing the overall duration of project construction are in the following sequence: project access roads, site preparation, diversion tunnel, cofferdams, foundation excavation and concrete cutoff wall, embankment construction, Tatlayako channel excavation, impounding. The spillway would be constructed prior to impounding.

Construction of the powerhouse and power conduit would start early in Year 3.

Conventional methods of construction and production rates similar to those achieved on other hydroelectric projects were assumed in the preparation of the construction schedule. A winter shutdown was assumed for most outdoor activities.

It was assumed that overburden material from the Tatlayako outlet channel excavation would be used as fill for the dam shell, except for the final "plug" excavation.

(b) Site Facilities

Most of the construction facilities would be located in the relatively flat area near Bowl Lake, which will be within the reservoir area.

It was assumed that borrow areas for fill and aggregate would be located in the vicinity of the site, except for till material which would be hauled from a source identified on the east shore of Tatlayako Lake.

(c) Construction Cost Estimate

The total construction cost of the Bowl Lake Project, excluding corporate overhead and interest during construction, is estimated to be $157.6 million, expressed in October 1983 dollars as detailed in Table 11-1.

This estimate does not include any allowance for project licencing and environmental or socio-economic costs. Engineering study costs prior to final design are also excluded.

SECTION 12.0 - CONCLUSIONS

The feasibility study of the Homathko Development was terminated prior to completion, when the 1983 field exploration program was cancelled. It is not possible at this stage to draw firm conclusions on the feasibility of the development. However, investigations and studies to-date are encouraging and a relatively small program of field investigations and office studies would enable the feasibility study to be completed.

Studies to date indicate that a four-project development would be the best hydroelectric scheme for the Homathko River. The installed capacity, average energy production and construction cost would be as follows:

Project	Installed Capacity (MW)	Average Energy Production (Avg MW)	Generation Construction Cost ($\$ \times 10^6$)
Waddington Canyon	420	242	685
Mosley Creek	290	168	727
Nude Canyon	185	110	306
Tatlayoko Storage	-	-	75
TOTAL	895	520	1793

The Bowl Lake Axis alternative for the Tatlayoko Storage Project should not be considered further.

The hydrometeorological data base at the start of the feasibility study was poor. The network of data collection stations installed during the study would have provided an adequate data base for future studies. However, the present data base is still poor.

SECTION 13.0 - RECOMMENDATIONS FOR COMPLETION OF FEASIBILITY STUDY

13.1 GENERAL

B.C. Hydro's present system for preparing budgets requires that a budget be prepared 7 months prior to approval by the Board of Directors. New funds are generally available on 1 April which is the beginning of the fiscal year. It would be necessary to issue an assignment for completion of the feasibility study in April, a little more than 2 years prior to completion of the study. Funds should be available in the first year for literature review, hydrologic studies, power studies, field reconnaissance, permit applications and planning for drilling. Layout studies, drilling and further power studies would take place in the second year and the feasibility report would be issued early in the third year. Fig. 13-1 shows the schedule recommended for future studies of the Homathko Development. Funds should be available for data collection 10 years prior to the start of further studies.

13.2 DATA COLLECTION

The following recommendations relate to the collection of hydrometeorological data:

1. Hydrometric data collection (continuous water levels, continuous water temperatures, occasional discharge measurements and occasional suspended sediment samples) at the six Water Survey of Canada streamflow stations established in the Homathko River basin should commence 10 years prior to the start of further studies.

2. Glacier mass balance measurements on Tiedemann and Bench glaciers should commence 10 years prior to the start of further studies to determine the glaciermelt contribution to streamflows.

3. Climatological data collection (air temperature, precipitation, wind, solar radiation, cloud cover, and relative humidity) at the locations established during these studies should commence 10 years prior to the start of further studies.

4. A detailed suspended sediment sampling program should commence 5 years prior to the start of further studies. The program would require four 10-day field trips per year to obtain data representative of the sediment regime.

5. Hydrometeorological data should be closely monitored to determine the accuracy and adequacy of the data and ensure that only valid data is collected.

6. A hydrometeorological watershed model capable of modelling runoff from rainfall, snowmelt and glaciermelt should be established at the time data collection is resumed. This model should be used to identify key climate stations to be maintained as index stations and indicate redundant stations which could be eliminated.

7. The hydrometeorological model should be used to verify glaciermelt contribution to streamflow.

13.3 HYDROLOGIC STUDIES

It is recommended that the following hydrologic studies be undertaken during the first year of the 2-year period necessary for the completion of feasibility studies.

1. Synthesize the probable maximum floods for each project using the hydrometeorological watershed model to be established during data collection.

2. Investigate the origin of flash floods to permit the assessment of the significance of such events to the design of the facilities.

3. Synthesize historical streamflows using the hydrometeorological watershed model for comparison with powerflows generated by correlation with streamflow stations located in other watersheds. Determine the significance of any difference and the effect on the project. Estimate the effect of warmer or cooler climate on the glacial contribution to streamflows.

4. Perform a multiple correlation analysis with air temperature and precipitation as independent variables to determine if streamflows are better represented by this technique than simple regression of streamflow stations from neighbouring watersheds. If streamflows are significantly different, then historical streamflows should be resynthesized using a multiple correlation method to more accurately estimate powerflows.

5. Determine the sensitivity of reservoir lives to variations in the suspended sediment estimates. It is recommended that this item be investigated prior to the start of the detailed suspended sediment sampling program. Digital terrain model data should be used to verify estimated reservoir volumes for the small forebay created downstream of Tatlayoko Lake.

6. Determine if the 19 July 1983 debris flow in the Nostetuko River created an overestimation of long-term sediment load at stations on the Homathko River.

7. Consider the diversion of Ottarasko Creek into Tatlayoko Lake so that the sediment load from Ottarasko Creek can be stored in

Tatlayoko Lake and not in the smaller forebay of the dam at the Nostetuko Axis.

8. Investigate the distribution of sediment deposition in the Waddington Reservoir. Sediments from Tiedemann and Tellot glaciers might affect the location of the Mosley/Nude powerhouse facilities.

9. The bedload volume deposited in Waddington Canyon, as identified by 1957 and 1982 drill holes, should be quantified as a check on the bedload calculations.

10. A literature review on reservoir sedimentation and sediment yield from glaciated basins should be undertaken to determine the potential significance of glaciers on sediment yield.

13.4 POWER STUDIES

When studies of the Homathko Development resume, the economic parameters will have changed and an improved data base might have modified the powerflows. Power studies should be completely redone.

The sensitivity of energy production to glaciermelt contribution should be determined. Glaciermelt contribution between April and September may increase or decrease streamflows by up to 20 percent.

13.5 PROJECT ARRANGEMENT STUDIES

(a) Waddington Canyon Project

When feasibility studies are resumed a high priority should be given to completing an analysis of costs and benefits in developing a 13.2 km tailrace tunnel arrangement to a comparable level of detail to that for the 2.7 km arrangement. This analysis should include consideration of the transient hydraulic conditions. The use of surge tanks, including air cushioned tanks at mid or quarter points should be considered. In addition, the implications of dewatering an additional 16 km of river channel and its impact on the salmon fishery should be assessed relative to the incremental net benefits which can be obtained with the 13.2 km tunnel development.

There will also be some impact on the salmon fishery with the 2.7 km long tailrace tunnel. This impact should be assessed in conjunction with the incremental impacts associated with the long tailrace tunnel.

An alternative arrangement with a long power tunnel should be reconsidered, to determine if there would be any overall cost advantage with reduced access tunnel length and surface powerhouse versus the increased costs of a low level, high pressure tunnel passing under Klattasine Creek, and in other possible areas requiring special lining provisions.

Alternative types of dams and axes within Waddington Canyon should be investigated. In particular, considering the availability of sand and gravel nearby and the apparent lack of significant impervious material in the area, a sand/gravel embankment dam with an upstream concrete face should be investigated. It appears unlikely that a rockfill dam with a concrete face could be competitive with the gravel dam. However, considering excavation requirements for a chute spillway along the canyon wall, serving at least in part as a dam quarry, this dam type merits future study.

In conjunction with an embankment dam layout, special consideration should be given to the optimum use of low level outlets in one of the diversion tunnels in conjunction with an auxiliary chute spillway, to determine if an intermediate level outlet tunnel could be avoided.

For the arch dam, auxiliary spillway arrangements which should be considered as alternatives include a "morning glory" type spillway tunnel, a high level outlet works with submerged entrances, a gated crest spillway on the arch dam, and sluiceways through the arch dam.

The discharge facilities for Waddington concentrate a tremendous energy in a very confined space. The potential for scour is significant and is an important factor in the layout of the facilities. Further studies of the scour and energy dissipation problems should be done.

(b) Mosley Creek Project

In addition to the concrete-faced rockfill dam described in Section 8.3, a sand and gravel embankment with a concrete facing should be reconsidered. Granular materials appear to be abundant in the area and should offer economy by comparison with rockfill. The rockfill embankment was selected to minimize risk of diversion tunnel blockage from avalanches. A review of avalanche risks in the area about 300 m upstream of the dam axis should be made to determine if this should be a factor in defining dam type at this site.

A concrete arch dam arrangement should be considered as an alternative, especially since hydraulic discharge facilities as well as diversion facilities, could be accommodated more readily with this dam type than with embankment dam arangements. In addition, an arch dam arrangement could be developed that would avoid the area of potential avalanche activity.

Depending on improvement in the "state of the art" for rollcrete gravity dam construction, future studies may find this technique to be technically acceptable and economically attractive at this site. Spillway and outlets could be readily accommodated with such a gravity dam layout.

The hydraulic discharge facilities as presented in Section 8.3 had not been developed to a satisfactory level of completion when studies were terminated. Utilization of one of the diversion tunnels for low level outlets is an obvious consideration. Such a low level outlet would serve at least in part to satisfy reservoir drawdown criteria and would permit some reduction in capacity of the spillway tunnels.

Consideration should be given to providing only one diversion tunnel.

A surface chute spillway arrangement should be developed in conjunction with the use of the low level outlets, and if necessary an intermediate level outlet. The chute spillway could be located on the right flank of the embankment dam to discharge in the same area as the tunnel spillways.

A combination of a submerged intake tunnel spillway, as presented, with a surface intake tunnel spillway with or without low level outlets also merits future study.

Spillway and outlet arrangements for an arch dam would include, with low level outlets through the dam, consideration of:

1. Intermediate level outlet gates, or sluices, through the dam.

2. An auxiliary chute spillway on the right abutment of the dam with a service spillway through the arch dam.

The alternative of locating the generator transformers in an underground gallery should be reviewed. With this arrangement the use of 230 kV CGI bus in the lead shaft and 230 kV switchgear in the powerhouse would improve plant reliability by providing more options for generator unit switching.

Consideration should be given to using a single 5 m diameter bus lead shaft for the two sets of bus leads instead of two separate 4 m diameter lead shafts.

A surface powerhouse arrangement merits a more detailed assessment in future studies.

(c) Nude Canyon Project

Alternative cofferdam concepts should be evaluated since local sources of impervious material, such as from the Nude Creek area, could be expensive to develop because of difficult access.

Considering limited availability of impervious material and extent of haul to obtain granular material, in addition to extremely difficult access in the site area, only one embankment dam type merits future study as an alternative to the concrete arch dam. This embankment should comprise a rockfill structure, using rockfill from spillway chute excavation and a nearby quarry, with an upstream concrete facing. With this dam type consideration would have to be given to selection of spillway and outlet tunnels to avoid potential hazards of avalanches which could occur on either flank of the embankment dam.

A concrete gravity dam layout and a massive form of concrete buttress dam should also be studied when feasibility studies are resumed. Depending on the "state of the art" in roller compacted concrete construction at that time, this method could be economically attractive for a gravity dam arrangement. Spillway and outlets could be readily accommodated with either concrete gravity or buttress dam layouts, using a conventional stilling basin.

As an alternative to the crest spillway it would be possible to locate a chute spillway on the right abutment to take advantage of the contours above the embankment. With the chute spillway headworks on the arch dam crest near the abutment, excavation for the chute could be minimal. With this alternative there could be some danger to the spillway from rockfalls or avalanches.

Provision of independent construction and permanent access to the Nude Canyon damsite, instead of the semi-permanent access described for the Interim Project Arrangement, merits a more detailed study in the future. In addition to comparing capital costs of alternative road and tunnel, the value of increased hydraulic losses with attendant loss of energy production and reduction in installed capacity, as well as increased size of surge shaft, would have to be assessed against the independent access to the Nude Canyon damsite.

As recommended for Mosley, the alternative of locating the generator transformers in an underground gallery should be reviewed.

As recommended for Waddington, scour from the spilling of water should be investigated.

Alternative methods of access to the damsite should be assessed.

In combination with Mosley Creek Project a surface powerhouse arrangement merits a more detailed assessment in future studies.

(d) Tatlayoko Storage Project

Engineering and environmental studies should determine the optimum normal maximum reservoir level.

Further consideration should be given to a low dam arrangement, maximum normal reservoir level El. 827.5, in the vicinity of Axis T1. Considering the reduced head conditions, with drawdown below normal lake levels, seepage losses from an economic viewpoint would not be a significant concern. Cutoff of seepage under and around structures in the improved channel and adjacent flanks would be designed solely for structure safety and prevention of piping from the pervious materials into the downstream river channel. This alternative should also consider increasing the regulation of the Homathko River by diverting Ottarasko Creek into Tatlayoko Lake.

Savings with such an upstream low dam arrangement would have to be balanced against inability to regulate the Nostetuko River flows at Axis T1.

Specific subsurface information should be obtained for the Axis T4 area. If the site appears to be technically feasible and bedrock is as high as expected a general arrangement should be prepared to determine if a reduction in cutoff requirements would compensate for increased crest length and embankment volume.

The Bowl Lake Axis should not be considered further.

The size of the dam core excavation and the cut-off wall for Nostetuko Dam should be optimized.

Consideration should be given to reducing the standard of the access road.

(e) Homathko Collector Switching Station

Consideration should be given to using only 500 kV switchgear instead of both 230 kV and 500 kV switchgear in the Collector Switching Station located above the Mosley/Nude powerhouse.

Consideration should be given to a scheme which would have 500 kV switching stations at both Waddington and Mosley/Nude powerhouses.

13.6 GEOTECHNICAL INVESTIGATIONS

(a) Foundation Investigations

(i) Waddington Canyon Project

1. Several linears around the proposed axis should be intersected by drill holes to determine their nature at depth.

2. The depths to bedrock beneath Klattasine Creek indicated by the 1983 gravity survey should be checked by drill holes to confirm the amount of bedrock cover over the proposed tailrace tunnel.

3. The tailrace tunnel alignments should be mapped at 1:10 000 scale to examine several linears in more detail and to obtain more information about bedrock structure.

(ii) Mosley Creek Project

1. More drill holes are required to determine the nature of the right bank linears.

2. A drill hole is required to determine the maximum depth to bedrock beneath Mosley Creek.

(iii) Nude Canyon Project

1. Holes should be drilled on both abutments to examine the rock at depth. The continuity of foliation slips, and whether or not the left abutment is underlain at a shallow depth by fault "NC1" should be established.

2. A hole should be drilled near the proposed axis to determine overburden depth beneath the river.

(iv) Tatlayoko Storage Project

1. A seismic survey should be carried out and additional holes should be drilled around the Nostetuko Axis to determine overburden depth.

2. Holes should be drilled near the Nostetuko Axis to confirm the nature of faults inferred to cross it.

3. A hole should be drilled at the proposed Nostetuko Saddle Dam axis to determine depth to bedrock and bedrock type.

(b) Construction Materials

The apparent shortage of suitable impervious material for all the project sites, except Tatlayoko Storage Project, will require further investigation, including sampling and testing to determine technical feasibility.

Although granular material for both concrete aggregate and granular fill is generally available within reasonable distance of each project site, it is recommended that further investigations, including bulk field gradation determination and petrographics, be carried out to confirm technical feasibility.

13.7 CONSTRUCTION COST ESTIMATES

Approximately 25 percent of the total construction cost would be for tunnels. Tunnelling methods, particularly the use of a tunnel boring machine, should be reviewed in detail before the completion of the feasibility study.

The level of contingencies on the construction cost estimates was chosen on the basis of the level of study (Feasibility). It is recommended that contingencies be reviewed by a team of discipline specialists who should give consideration to the nature of potential problems on each project and not simply the level of study. The review could be modelled on that carried out by the Stikine/Iskut project group.[15]

Rock support in tunnels, shafts and underground powerhouses should be reviewed in detail.

ACKNOWLEDGEMENTS

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This report was reviewed by functional Section Supervisors, Department Managers and the Manager of Development and Design.

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TABLES

TABLE 3-1

HOMATHKO DEVELOPMENT
PRINCIPAL PROJECT DATA

Item	Waddington Canyon	Mosley Creek	Nude Canyon	Tatlayoko Storage: Nostetuko Axis	Tatlayoko Storage: Bowl Lake Axis
Dam					
Type	Conc. Arch	Conc. Faced Rockfill	Conc. Arch	Earthfill	Earthfill
Maximum height (m)	200	148	98	40	80
Crest length (m)	330	370	280	360	420
Spillway					
Design Capacity m^3/s	3830	1560	1135	565	565
Diversion					
Design capacity (m^3/s)	935	340	230	215	215
Reservoir					
Minimum level (m)	323.5	730.0	633.5	817.5	817.5
Maximum flood level (m)	328.0	763.0	638.0	830.5	830.5
Maximum normal level (m)	325.0	760.0	635.0	827.5	827.5
Maximum normal surface area (ha)	782.0	2943.0	162.3	3830.0	4240.0
Power Facilities					
Total plant capacity (MW)	420	290	185	-	22
Units, no. and capacity (MW)	2-210	2-145	1-185	-	1-22
Units rotating speed (rpm)	200	450	300	-	300
Maximum plant discharge (m^3/s)	247	85	75	-	34
Maximum normal gross head (m)	207.3	434.5	309.5	-	74.5

TABLE 4-1

HOMATHKO DEVELOPMENT
SUMMARY OF POWER STUDIES

Project	Maximum Normal Reservoir Level (m)	Head*[1] (m)				Discharge (m^3/s)			Average Plant Efficiency*[1]	Average Energy (Avg MW)
		Average Reservoir Level	Average Tailwater Level	Average Head Loss	Average Net Head	Average Inflow	Average Spill	Average Power Flow		
Mosley Creek Project	760	748	324	31	393	49	-	49	0.89	168
Waddington Canyon Project	325	324	118	5	201	140	2	138	0.89	242
Nude Canyon Project	635	634	324	10	300	42	-	42	0.89	110

*[1] These data were not taken directly from the power studies, but were estimated in order to provide the reader with a general appreciation of the factors involved in the power studies.

TABLE 4-2

HOMATHKO DEVELOPMENT
POWER AND COSTS

Project	Installed Capacity (MW)	Average Energy (Avg MW)	Generation Construction Cost*1 ($ Millions)	Generation Construction Cost*1 per Unit of Capacity ($/kW)	Generation Construction Cost*1 per Unit of Average Energy at Site ($/Avg kW)
Waddington Canyon	420	242	685	1631	2831
Mosley Creek	290	168	727	2507	4327
Nude Canyon	185	110	306	2060	3464
Tatlayoko Storage	-	-	75		
TOTAL	895	520	1793	2003	3448

*1 See Tables 7-1, 8-1, 9-1 and 10-1 for details of construction costs.

TABLE 5-1

HOMATHKO DEVELOPMENT
SUMMARY OF POTENTIAL BORROW AREAS

Project	Material Type	Potential Source	Approximate Haul Distance (km)	Estimated Volume ($\times 10^6 m^3$)	Remarks
Tatlayoko Storage	Granular	Area 8	0-5	29	- suitable as concrete aggregate or embankment material
	Impervious	Rafferty Creek Fan (S-E Area 8)	2-3	3.5	- low plasticity, fine sandy silt, no coarse fraction
	Impervious	Area 9	4-9	4	- medium plasticity, well graded glacial till; haul distance depends on axis chosen
Nude Canyon	Granular	Area 4	8	10	- will require crushing and processing
	Granular	Doran Creek	6	2.5	- will require crushing and processing
	Impervious	Area 7	8-10	-	- medium plasticity, clayey silt, silty sand and gravel overburden (10-15 m); difficult haul route
Mosley Creek	Granular	Area 3	2.5-4.5	2	- variable sandy silt overburden (1-2 m) and substantial quantities are available upstream
	Impervious	Tellot Glacier Moraines	10-15	7.5	- non-plastic silty moraines; difficult haul route
Waddington Canyon	Granular	Area 6	0.5-3.5	8	- will require crushing and processing
	Impervious	Klattasine Creek	2-6	-	- steep, difficult to excavate cliffs with variable overburden (10-100 m)

TABLE 5-2

HOMATHKO DEVELOPMENT POTENTIAL DEBRIS FLOWS

Location	Size (m)	Maximum Volume (m^3)	Material Damming Lake	Comments
Small lake on Tiedemann Creek	10 x 5 x 1	50	ice and moraine	Small lake, if moraine breached, small flood.
Tiedemann Lake	500 x 300 x 10	8×10^5	moraine	Medium chance of washout. No surface outlet, discharge by seepage.
Scimitar Creek	-	750		Small lakes on lateral moraine.
Whitesaddle Creek				
Top	200 x 540 x 20	1×10^6	moraine	No exit creeks, drainage by seep through gravel and boulders, blocks 5 cm to 2 m size.
Second Lake	100 x 600 x 30	1×10^6	avalanche debris	If voids plug with silt lakes may overflow.
Third Lake	150 x 600 x 1	5×10^4	avalanche	Very shallow.
Bottom Lake	150 x 700 x 20	1×10^6	avalanche	Shallow.
Middle Fork - Nostetuko				
Nostetuko Lake	750 x 350 x 35	5×10^6	moraine	Moraine overtopped at 2:00 a.m., 19 July 1983. Lake breached dam and emptied at once, flooding the creek.
	100 x 50 x 10	2×10^4	moraine	
Frobisher Lake	200 x 400 x 5	2×10^5	moraine overlying rock	

TABLE 5-2 - (Cont'd)

Location	Size (m)	Maximum Volume (m^3)	Material Damming Lake	Comments
West Fork - Nostetuko				
Iceberg Lake	750 x 500 x 20	5×10^6	rock and moraine	Dammed by about 5 m of moraine above rock. If breached, only about one-quarter of the lake volume will escape.
MT Lake	500 x 200 x 30	0.5×10^6	ice	Lake emptied about 12:00 noon on 8 August 1983 and about the same date the previous year. Probably an annual event. Outlet under glacier.
Gillman	very small			No danger, very small.
Doran	< 10 x 10			No lakes on main creek but a number of lakes on edge of valley, high. Very small, not important.
Ottarasko				
South Fork	50 x 15 x 3	1000	moraine	Small lake.
	-		moraine	Remnants of a large lake which failed in past (tree covered, at least 300 ybp).
North Fork	no lakes			
Valleau				
South Fork	very small (10 x 10 x 1)		moraine	No danger.
North Fork	very small (10 x 10 x 1)		moraine	No danger.

TABLE 5-2 - (Cont'd)

Location	Maximum Size (m)	Volume (m^3)	Material Damming Lake	Comments
Razor				
East Fork	10 x 20 x 2	2×10^2	moraine	Too small to be of consequence.
	10 x 20 x 2	2×10^2	moraine	Too small to be of consequence.
	120 x 50 x 15	5×10^4	moraine	Larger lake, possible to flood but danger not very high.
	700 x 200 x 5	5×10^5	moraine	No outlet, underground passage, may break but danger not high.
West Fork	1000 x 200 x 20	2×10^6	half rock, half avalanche at outlet	Named Buckhorn Lake (met station, cabin), very little danger of failure
Hell Raising Creek				
North Fork	200 round		moraine	Shallow, flat outlet, no flooding danger.
	200 x 300 x 1	3×10^4	moraine	Shallow, flat outlet, no flooding danger.
South Fork	400 x 300 x 10	6×10^5	rock	Waterfall over rock lip 100 m high, spectacular, Small danger.
Sand Creek	puddle		moraine	No flood potential, very small lake.
Crazy Creek				
North Fork	100 x 10 x 1	5×10^2	moraine	Very small, flat outlet, no danger.
Scimitar Creek				
North Fork, pocket valley	100 x 50 x 2	5×10^3	moraine	Long flat outlet, no danger.
Tellot Creek	900 x 300 x 10	1×10^6	moraine	Moraine quite long - low danger (Ephemeron Lake)

TABLE 5-2 - (Cont'd)

Location	Maximum Size (m)	Maximum Volume (m^3)	Material Damming Lake	Comments
Five Fingers			moraine	Perhaps there was a larger lake which emptied. All trees have regrown.
Klattasine Creek	50 x 50 x 5	6×10^3	rock	Lake has washed out moraine and has eroded outlet down to bedrock.
Quartz North Arm	600 x 500 x 5	5×10^4	avalanche	Outlet flat, some bedrock. Could fail but not much volume.
Nude	500 x 50 x 5	5×10^3	moraine	Outlet flat, no danger of breaching.

Note: Twist, South Fork Crazy, Mercator, Five Fingers and Jamison creeks were examined but they had no lakes or they had lakes of no consequence.

TABLE 6-1

HOMATHKO DEVELOPMENT
COASTAL AND INTERIOR CLIMATE STATIONS*1

Station No.	Station Name	Latitude North °	'	Longitude West °	'	Elevation (m)	Record Period	Data Type	Annual*2 Precip. (mm)	Temp. (°C)
Coastal Stations										
1064375	Knight Inlet	51	06	125	35	9	1962-1966	T,P	-	-
1066JFQ	Philips Arm	50	33	125	22	2	1973-1977	T,P	-	-
1046390	Powell River	49	53	125	34	54	1924-Present	T,P	1094	10.5
1021260	Campbell River	50	01	125	18	79	1936-1969	T,P	1581	8.7
1021261	Campbell River A	49	57	125	16	105	1965-Present	T,P	1406	8.2
1068677	Wannock River	51	41	127	15	8	1976-Present	T,P	-	-
1060840	Bella Coola	52	20	126	38	3	1895-1939	T,P	-	-
	Bella Coola	52	22	126	41	18	1939-Present	T,P	1614	7.5
1065670	Ocean Falls	52	21	127	41	5	1924-Present	T,P	4387	8.1
Interior Stations										
1088010	Tatlayoko Lake	51	39	124	23	847	1928-Present	T,P	412	3.9
1088C09	Tatla Lake (BCFS)	51	54	124	36	945	1973-Present	T,P	-	-
1084350	Kleena Kleene	52	03	124	54	823	1933-1940	P	-	-
	Kleena Kleene	51	59	124	56	899	1942-1968	T,P	334	1.6
1086556	Puntzi Mtn.	52	10	124	12	1372	1959-1961	T,P		
	Puntzi Mtn.	52	07	124	05	910	1968-1979	T,P	318	1.5
1080431	Anahim Lake	52	21	125	02	1097	1954-1967	T,P	-	-
	Anahim Lake	52	28	125	18	1064	1975-1980	T,P	-	-
1080284	Alexis Creek	52	05	123	17	800	1973-Present	T,P	-	-
1080289	Alexis/Tautki Creek	52	33	123	11	1219	1961-Present	T,P	-	-
1080870	Big Creek	51	58	122	48	661	1893-1900	T,P	-	-
	Big Creek	51	43	123	02	1128	1904-1942	T,P	336	2.0
1084490	Lajoie Dam	50	50	122	52	686	1963-1980	T,P	540	5.4
1086090	Pemberton Meadows	50	27	122	56	223	1912-1967	T,P	990	7.0
1086083	Pemberton BCFS	50	19	122	49	218	1969-Present	T,P	1187	7.2
109QR57	Wineglass Ranch	51	51	122	39	488	1973-Present	T,P	309	5.9
1084731	Lunch Lake	51	50	124	28	1012	1980-Present	T,P	-	-
1085208	Mosley Creek/ Sand Creek	51	36	125	00	760	1980-Present	T,P	-	-
104375	Bluff lake	51	46	124	43	914	1981-Present	T	-	-

Note: T = Temperatures; P = Precipitation

*1 Established prior to the start of the feasibility study.

*2 30-year (1951-1980) mean obtained from "Canadian Climate Normals, 1951-1980, Temperature and Precipitation, British Columbia", Atmospheric Environment Service, Environment Canada.

TABLE 6-2

HOMATHKO DEVELOPMENT
MEAN MONTHLY TEMPERATURE AND PRECIPITATION DATA

Location:	Coastal Stations				Interior Stations			
Station:	Powell River		Campbell River		Tatlayoko Lake		Kleena Kleene	
Station No.:	1046390		1021260		1088010		1084350	
Record Period:	1924-Present		1936-1969		1928-Present		1933-1968	
Elevation (m):	52		79		853		899	
	Temp (°C)	Precip (mm)	Temp (°C)	Precip (mm)	Temp (°C)	Precip (mm)	Temp (°C)	Precip (mm)
January	3.2	146.3	1.1	222.6	-8.5	48.9	-13.1	42.2
February	4.9	101.2	3.4	172.5	-3.6	30.0	-7.3	17.5
March	6.0	88.5	4.5	154.8	-0.8	22.0	-3.4	13.8
April	9.1	54.0	7.3	77.4	4.0	17.4	2.4	18.3
May	12.9	54.5	11.3	53.0	8.3	20.3	7.2	22.9
June	15.9	56.4	14.2	49.1	11.5	29.0	10.8	32.1
July	18.3	40.4	16.9	40.0	14.0	26.7	13.2	23.1
August	18.2	57.4	16.7	55.0	13.7	30.8	12.7	33.1
September	15.3	63.6	13.5	77.1	10.2	27.3	8.9	22.1
October	10.9	122.5	8.8	162.8	5.4	47.3	3.5	27.9
November	6.6	149.4	4.6	239.7	-1.6	54.2	-4.5	36.0
December	4.6	160.0	2.5	276.5	-5.8	58.3	-10.7	45.2
Average	10.5	1094.2	8.7	1580.5	3.9	412.2	1.6	334.2

Source: "Canadian Climate Normals, 1951-1980, Temperature and Precipitation, British Columbia", Atmospheric Environment Service, Environment Canada.

TABLE 6-3

HOMATHKO DEVELOPMENT
HOMATHKO RIVER BASIN HYDROMETEOROLOGICAL NETWORK

Station No.	Station Name	Latitude North °	'	Longitude West °	'	Elevation (m)	Record Period	Data Type
Water Survey of Canada - Hydrometric Stations								
08GD004	Homathko River at the Mouth	51	00	124	55	2	1957-1981 1982-Present	Q Q,SS,WT
08GD006	Homathko River at Tragedy Canyon	51	16	124	53	250	1960-1969 1982-Present	Q Q,SS,WT
08GD005	Homathko River below Nude Creek	51	21	124	46	570	1960-1970 1982-Present	Q Q,SS,WT
08GD009	Homathko River below Nostetuko River	51	25	124	30	780	1983-Present	Q,SS,WT
08GD008	Homathko River at Tatlayoko Lake Inlet	51	40	124	24	-	1968-1975 1982-Present	Q Q,SS,WT
08GD007	Mosley Creek near Dumbell Lake	51	25	124	56	690	1960-1970 1982-Present	Q Q,SS,WT
Federal Climatological Stations								
1088010	Tatlayoko Lake	51	39	124	23	847	1928-Present	T,P
1088009	Tatla Lake (BCFS)	51	54	124	36	945	1973-Present	T,P
1084731	Lunch Lake	51	50	124	28	1012	1980-Present	T,P
1085208	Mosley Creek/ Sand Creek	51	36	125	00	760	1980-Present	T,P
B.C. Hydro Climatological Stations								
104035	Blackhorn	51	35	124	46	1700	1983-Present	T,P
104375	Bluff Lake	51	46	124	43	914	1981-Present	T
104043	Bowl Lake	51	24	124	36	739	1983-Present	T,P
104040	Doran	51	16	124	38	1905	1983-Present	T,P
104039	Five Fingers	51	31	124	54	1783	1983-Present	T,P
104030	Gillman Creek	51	16	124	49	1734	1982-Present	T,P,SC
104026	Hell Raving	51	43	125	05	1798	1982-Present	T,P,SC
104047	Jamison	51	36	124	33	2038	1983-Present	T,P
104029	Lowwa Lake	51	18	124	50	337	1982-Present	T,P,SC
104034	Middle Lake	51	42	124	53	815	1983-Present	T,P
104036	Mt. Juno	51	34	125	13	2028	1983-Present	T,P
104045	Nostetuko-2	51	14	124	27	1487	1982-Present	T,P,SC
104041	Nude	51	28	124	47	1730	1983-Present	T,P
104042	Ottarasko	51	28	124	40	1899	1983-Present	T,P
104048	Potato	51	38	124	19	1963	1983-Present	T,P,SC
104038	Radiant	51	27	125	11	1943	1983-Present	T,P
104044	Reliance	51	23	124	39	1463	1983-Present	T,P
104037	Scimitar	51	19	125	02	821	1983-Present	T,P
104045	Stikelan	51	20	124	21	1882	1983-Present	T,P
104031	Stonsayako	51	20	124	35	1920	1982-Present	T,P
104033	Tatlayoko S.	51	27	124	28	853	1982-Present	T,P
104049	Tatlayoko Wind	51	38	124	22	968	1983-Present	W
104028	Tiedemann	51	21	124	59	1432	1982-Present	T,P,SC
104027	Twist Creek	51	37	125	12	1676	1982-Present	T,P,SC
BCH #4	Tatlayoko Camp	51	27	124	26	870	1982-1984	T,C,RH,S

Continued...

TABLE 6-3 - (Cont'd)

HOMATHKO DEVELOPMENT
HOMATHKO RIVER BASIN HYDROMETEOROLOGICAL NETWORK

Station No.	Station Name	Latitude North °	'	Longitude West °	'	Elevation (m)	Record Period	Data Type
B.C. Hydro Automatic Weather Stations								
BCH #1	Brew Creek/Homathko	51	06	125	01	30	1983-Present	W,T,RH
BCH #2	Mosley Creek/Homathko	51	19	124	50	410	1983-Present	W,T,RH
BCH #3	Fosters Ranch	51	36	125	00	760	1983-Present	W,T,RH,S
B.C. Forest Service Climatological Stations								
0508	Scar Creek	51	11	125	02	95	1982-Present	T,P,S
Snow Courses (10 Point)								
3A17	Tiedemann Glacier	51	21	124	58	1400	1981-Present	WE,SD
3A22	Nostetuko River	51	13	124	25	1600	1982-Present	WE,SD
3A13	Tatlayoko Lake	51	36	124	20	1710	1952-Present	WE,SD

T = Temperatures; P = Precipitation; W = Wind; RH = Relative Humidity; S = Solar Radiation, SC = Snow Course; C = Cloud Cover; Q = Daily Discharge; SS = Suspended Sediment; WT = Water Temperature: WE = Water Equivalent; SD = Snow Depth.

TABLE 6-4

HOMATHKO DEVELOPMENT
AVERAGE MONTHLY STREAMFLOWS

Month	Waddington Canyon Project (m^3/s)	Mosley Creek Project (m^3/s)	Nude Canyon Project (m^3/s)	Tatlayoko Storage Project (Nostetuko Axis) (m^3/s)
January	30.9	11.8	14.1	6.5
February	33.1	10.9	10.7	4.9
March	28.1	9.2	11.1	5.1
April	45.3	13.8	12.0	5.5
May	130.5	48.1	42.4	19.5
June	287.3	102.4	87.5	40.3
July	375.5	136.9	104.2	47.9
August	333.6	116.4	92.3	42.5
September	195.2	68.0	57.9	26.6
October	115.5	39.4	36.2	16.7
November	55.1	18.3	23.5	10.8
December	40.8	15.6	14.3	6.6
Annual Average	139.1	49.2	42.2	19.4

*1 For 35-year period of record from 1940 to 1975, including correlated data.

TABLE 6-5

HOMATHKO DEVELOPMENT

RESERVOIR LIFE ESTIMATES

Project	Reservoir Life (years)
Tatlayoko Storage*1	
- Nostetuko Axis	75
- Bowl Lake Axis	750
Nude Canyon	230
Mosley Creek	1300
Waddington Canyon	
- With Tatlayoko, Nude, Mosley	400
- With Mosley only	270

*1 The reservoir life of both alternatives would be doubled if Ottarasko Creek were to be diverted into the Tatlayoko Lake portion of the reservoir.

TABLE 6-6

HOMATHKO DEVELOPMENT
UPPER HOMATHKO RIVER DRAINAGE BASIN
AREAS COVERED BY GLACIERS

Sub-basin	River Distance (km)	Sub-basin Area (km^2)	Accumulated Area to this station (km^2)	Sub-basin Glacier Area (km^2)	(%)	Accumulated Glacier Area to this station (km^2)	(%)
Tatlayoko Lake Basin u/s of Ottarasko Creek	97.5	1061	1061	18	1.7	18	1.7
Ottarasko Creek Basin	97.2	252	1313	18	7.0	36	2.7
Nostetuko River Basin	95.5	258	1571	45	17.5	81	5.2
Sub-basin u/s of Nude Creek	72.5	232	1803	42	18.1	123	6.8
Nude Dam d/s of Bellamy Creek	70.5	206	2009	33	16.0	156	7.8
Sub-basin u/s of Doran Creek	64.5	30	2039	1	3.3	157	7.7
Doran Creek	63.0	185	2224	44	23.6	201	8.9
Mosley Creek at Homathko River	60.5	1899	4123	351	18.5	552	13.4
WSC Station 08GD006 at Tragedy Canyon	54.0	110	4233	16	14.5	568	13.4
Waddington Dam	48.4	40	4273	-	-	568	13.3
Sub-basin							
d/s of Mosley Dam		299					
d/s of Nude Dam		254					
u/s of Waddington Dam		543	-	196	36.1	-	-

Notes:

1. Areas were planimetered from 1:50 000 maps.
2. River distance was measured from 1:50 000 maps.

TABLE 6-7

HOMATHKO DEVELOPMENT
MOSLEY CREEK DRAINAGE BASIN
AREAS COVERED BY GLACIERS

Sub-basin	River Distance (km)	Sub-basin Area (km^2)	Accumulated Area to this station (km^2)	Sub-basin Glacier Area (km^2)	Sub-basin Glacier Area (%)	Accumulated Glacier Area (km^2)	Accumulated Glacier Area (%)
Outlet of Middle Lake	55.5	671	671	15	2.2	15	2.2
d/s of Crazy Creek	34.0	509	1180	38	7.5	53	4.5
Mosley Dam (and WSC 08GD007)	11.8	420	1600	163	38.8	216	13.5
u/s of Tellot Creek	7.3	25	1625	4	16.0	220	13.5
Tellot Creek Basin	7.4	125	1750	41	32.8	261	14.9
Tiedemann Creek Basin	1.0	148.5	1899	90	60.4	351	18.5
Homathko River Confluence	0	0.5	1899	-	-	351	18.5

Notes:

1. Areas were planimetered from 1:50 000 maps.
2. River distance was measured from 1:50 000 maps.

TABLE 6-8
HOMATHKO DEVELOPMENT
LOWER HOMATHKO RIVER DRAINAGE BASIN
AREAS COVERED BY GLACIERS

Sub-basin	River Distance (km)	Sub-basin Area (km^2)	Accumulated Area to this station (km^2)	Sub-basin Glacier Area (km^2)	(%)	Accumulated Glacier Area (km^2)	(%)
Above WSC Station 08GD006 at Tragedy Canyon	54	-	4233	-	-	568	13.3
Waddington Dam	48.4	40	4272	-	-	568	13.3
Below Klattasine Creek	48.0	70	4342	17	24	585	13.5
Below XLH 170 (local)	46.5	7	4350	-	-	585	13.4
Above Coola Creek (XLH 154)	43.0	63	4413	15	23.6	600	13.6
Below XLH 150 (Waddington Glacier)	41.8	87	4500	54	62.7	654	14.5
Below XLH 140 (Bridge)	37.6	34	4534	3	8.7	657	14.5
Scar Creek Basin	37.5	170	4704	55	32.7	712	15.1
Below XLH 120	33.0	20	4724	-	-	712	15.0
Whitemantle Creek Basin	32.0	167	4891	82	49.3	794	16.2
Below XLH 110	31.5	2	4893	-	-	794	16.2
Below XLH 90	25.6	47	4940	4	9.0	798	16.2
Brew Creek Basin	25.0	199	5139	91	45.4	889	17.3
Below XLH 70	20.0	46	5185	3	6.5	892	17.2
Below Jewakwa River	16.5	283	5468	180	63.6	1072	19.6
Below Heakamie River (XLH 50)	15.3	272	5740	163	60.3	1235	21.5
Below Smith Creek	13.0	48	5788	9	18.1	1244	21.4
Below XLH 30	10.5	17	5805	0.4	2.4	1244.4	21.4
Below XLH 20	7.0	20	5825	-	-	1244.4	21.4
Below XLH 10 (WSC Cableway)	5	5	5830	-	-	1244.4	21.3
Bute Inlet	0	78	5908	6	7.2	1250.4	21.2

Notes:
1. Areas were planimetered from 1:50 000 maps.
2. River distance was measured from 1:50 000 maps.
3. Lower Homathko River basin (1636 km^2) has 682.4 km^2 of glaciers (41.7 %).

TABLE 6-9

HOMATHKO DEVELOPMENT
ESTIMATED GLACIERMELT CONTRIBUTION TO STREAMFLOW
AT WATER SURVEY OF CANADA STATION 08GD006
HOMATHKO RIVER AT TRAGEDY CANYON

	1981	1982	1983
Glaciermelt Volume ($10^6 m^3$)*1	586	741	372
Mean Melt Rate - MR (6 melt months)*2 (m^3/s)	37.16	47.0	23.6
Average Streamflow - Q Tragedy Canyon (6 melt months)*2 (m^3/s)	200.0	224.9	201.5
Glaciermelt Contribution to Summer Streamflow	18.6%	20.9%	11.7%

Sample Calculation for 1981:

$$MR = \frac{\text{Glacier Melt Volume}}{\text{6 melt months}} = \frac{586 \times 10^6 m^3}{(365/2)24 \times 60 \times 60 \text{ s}} = 37.16 \text{ m}^3/\text{s}$$

$$\% \text{ of Streamflow} = \left(\frac{MR}{Q}\right) 100\% = \left(\frac{37.16}{200}\right) 100\% = 18.6\%$$

*1 Glaciermelt volume is from NHRI Mass Balance Data. It is the net annual reduction in glacier mass i.e. the amount the galciers retreated each year.

*2 6 melt months are April to September inclusive.

TABLE 7-1

HOMATHKO DEVELOPMENT
WADDINGTON CANYON PROJECT
(420 MW - RES. EL. 325 m)
ESTIMATE OF CONSTRUCTION COSTS
($ Millions)

	Feature	
1.	Land and Rights	1.7
2.	Flowage	3.7
3.	Area Access	0.0
4.	Site Clearing	0.5
5.	Site Access	17.1
6.	Diversion Tunnels	21.7
7.	Cofferdams and Unwatering	5.7
8.	Concrete Arch Dam	136.5
9.	Plunge Pool	16.5
10.	Spillway Tunnel	42.9
11.	Power Intake	6.5
12.	Penstocks	19.8
13.	Powerhouse and Switchgear	
	- Civil	99.7
	- Mechanical	24.4
	- Electrical	47.3
14.	Tailrace Tunnel	48.0
15.	Construction Services	29.5
	DIRECT CONSTRUCTION COST	521.5
16.	Contingencies	95.6
17.	Project Final Design and Administration	61.7
18.	Construction Insurance and Bonds	6.2
	TOTAL CONSTRUCTION COST	685.0

Notes

1. Costs were estimated in October 1982 dollars and escalated to October 1983 by an increase of 7 1/2 percent.
2. Construction Services are 6 percent of the Direct Construction Cost.
3. Contingencies are 10 percent on mechanical and electrical items and 20 percent on other items.
4. Environmental Contingencies are not included.
5. Final Design and Construction Supervision is 10 percent of Direct Cost plus Contingencies. (Study Costs and Investigations are not included.)
6. Construction Insurance and Bonds are 1 percent of Direct Cost plus Contingencies.
7. Costs of Licencing are not included.
8. Interest during construction and corporate overhead costs are not included.

TABLE 7-2

HOMATHKO DEVELOPMENT
WADDINGTON CANYON PROJECT
ESTIMATE OF CAPITAL EXPENDITURES BY FISCAL YEARS
($ Millions)

Fiscal Year (1 April-31 March)								Total Expenditure $Oct. '83
0	1	2	3	4	5	6	7	
3.5	26.9	34.5	35.6	99.1	228.8	197.0	46.6	685.0

Award of first contract

Unit 1 in-service

Unit 2 in-service

TABLE 8-1

HOMATHKO DEVELOPMENT
MOSLEY CREEK PROJECT
(290 MW - RES. EL. 760 m)
ESTIMATE OF CONSTRUCTION COST
($ Millions)

Feature	
1. Land and Rights	4.4
2. Flowage	3.2
3. Area Access	117.7
4. Site Clearing	0.6
5. Site Access	5.9
6. Diversion Tunnels	14.6
7. Cofferdams and Unwatering	3.5
8. Concrete Faced Rockfill Dam	101.7
9. Spillway (Tunnels)	40.8
10. Power Intakes	8.8
11. Power Tunnel and Surge Tank	60.8
12. Penstocks	25.8
13. Powerhouse Access Tunnel	5.6
14. Powerhouse and Switchgear	
- Civil	68.1
- Mechanical	18.7
- Electrical	39.5
15. Construction Services	31.2
DIRECT CONSTRUCTION COST	550.9
16. Contingencies	103.9
17. Project Final Design and Administration	65.5
18. Construction Insurance and Bonds	6.5
TOTAL CONSTRUCTION COST	726.8

Notes

1. Costs were estimated in October 1982 dollars and escalated to October 1983 by an increase of 7 1/2 percent.
2. Construction Services are 6 percent of the Direct Construction Cost.
3. Contingencies are 10 percent on mechanical and electrical items and 20 percent on other items.
4. Environmental Contingencies are not included.
5. Final Design and Construction Supervision is 10 percent of Direct Cost plus Contingencies. (Study Costs and Investigations are not included.)
6. Construction Insurance and Bonds are 1 percent of Direct Cost plus Contingencies.
7. Costs of Licencing are not included.
8. Interest during construction and corporate overhead costs are not included.

TABLE 8-2

HOMATHKO DEVELOPMENT
MOSLEY CREEK PROJECT
ESTIMATE OF CAPITAL EXPENDITURES BY FISCAL YEARS
($ Millions)

Fiscal Year (1 April-31 March)								Total Expenditure $Oct. '83
0	1	2	3	4	5	6	7	
3.4	48.5	101.7	63.0	76.3	203.2	172.5	58.2	726.8

Award of first contract

Unit 1 in-service

Unit 2 in-service

TABLE 9-1

HOMATHKO DEVELOPMENT
NUDE CANYON PROJECT*[1]
(185 MW - RES. EL. 635 m)
ESTIMATE OF CONSTRUCTION COST
($ Millions)

Feature	
1. Land and Rights	0.8
2. Flowage	3.1
3. Area Access	20.7
4. Site Clearing	0.3
5. Site Access	4.3
6. Diversion Tunnel/Outlet Control	5.9
7. Cofferdams and Unwatering	1.1
8. Concrete Arch Dam and Spillway	50.5
9. Plunge Pool	6.7
10. Power Intake	4.8
11. Power Tunnel and Surge Tank	48.3
12. Penstock	13.8
13. Powerhouse and Switchgear	
- Civil	29.1
- Mechanical	11.1
- Electrical	18.7
14. Construction Services	15.6
DIRECT CONSTRUCTION COST	232.4
15. Contingencies	42.8
16. Project Final Design and Administration	27.5
17. Construction Insurance and Bonds	2.8
TOTAL CONSTRUCTION COST	305.5

Notes

1. Costs were estimated in October 1982 dollars and escalated to October 1983 by an increase of 7 1/2 percent.
2. Construction Services are 6 percent of the Direct Construction Cost.
3. Contingencies are 10 percent on mechanical and electrical items and 20 percent on other items.
4. Environmental Contingencies are not included.
5. Final Design and Construction Supervision is 10 percent of Direct Cost plus Contingencies. (Study Costs and Investigations are not included.)
6. Construction Insurance and Bonds are 1 percent of Direct Cost plus Contingencies.
7. Costs of Licencing are not included.
8. Interest during construction and corporate overhead costs are not included.

TABLE 9-2

HOMATHKO DEVELOPMENT
NUDE CANYON PROJECT
ESTIMATE OF CAPITAL EXPENDITURES BY FISCAL YEARS
($ Millions)

Fiscal Year (1 April-31 March)							Total Expenditure $Oct. '83
0	1	2	3	4	5	6	
2.2	12.8	77.5	33.0	71.2	81.6	27.2	305.5

Award of first contract

Unit 1 in-service

TABLE 10-1

HOMATHKO DEVELOPMENT
TATLAYOKO STORAGE PROJECT - NOSTETUKO AXIS
(RES. EL. 827.5 m)
ESTIMATE OF CONSTRUCTION COST
($ Millions)

	Feature	
1.	Land and Rights	0.4
2.	Flowage	0.3
3.	Area Access	4.0
4.	Site Clearing	0.2
5.	Site Access	1.9
6.	Diversion Tunnel/Outlet Control	13.6
7.	Cofferdams and Unwatering	2.2
8.	Embankment Dam	21.5
9.	Tatlayoko Channel	3.4
10.	Spillway	5.9
11.	Construction Services	3.2
	DIRECT CONSTRUCTION COST	56.6
12.	Contingencies	11.2
13.	Project Final Design and Administration	6.8
14.	Construction Insurance and Bonds	0.7
	TOTAL CONSTRUCTION COST	75.3

Notes

1. Costs were estimated in October 1982 dollars and escalated to October 1983 by an increase of 7 1/2 percent.
2. Construction Services are 6 percent of the Direct Construction Cost.
3. Contingencies are 10 percent on mechanical and electrical items and 20 percent on other items.
4. Environmental Contingencies are not included.
5. Final Design and Construction Supervision is 10 percent of Direct Cost plus Contingencies. (Study Costs and Investigations are not included.)
6. Construction Insurance and Bonds are 1 percent of Direct Cost plus Contingencies.
7. Costs of Licencing are not included.
8. Interest during construction and corporate overhead costs are not included.

TABLE 10-2

HOMATHKO DEVELOPMENT
TATLAYOKO STORAGE PROJECT (NOSTETUKO AXIS)
ESTIMATE OF CAPITAL EXPENDITURES BY FISCAL YEARS
($ Millions)

Fiscal Year (1 April-31 March)					Total Expenditure $Oct. '83
0	1	2	3	4	
0.6	9.7	30.4	16.8	17.8	75.3

Award of first contract (year 0)

Storage operational (year 4)

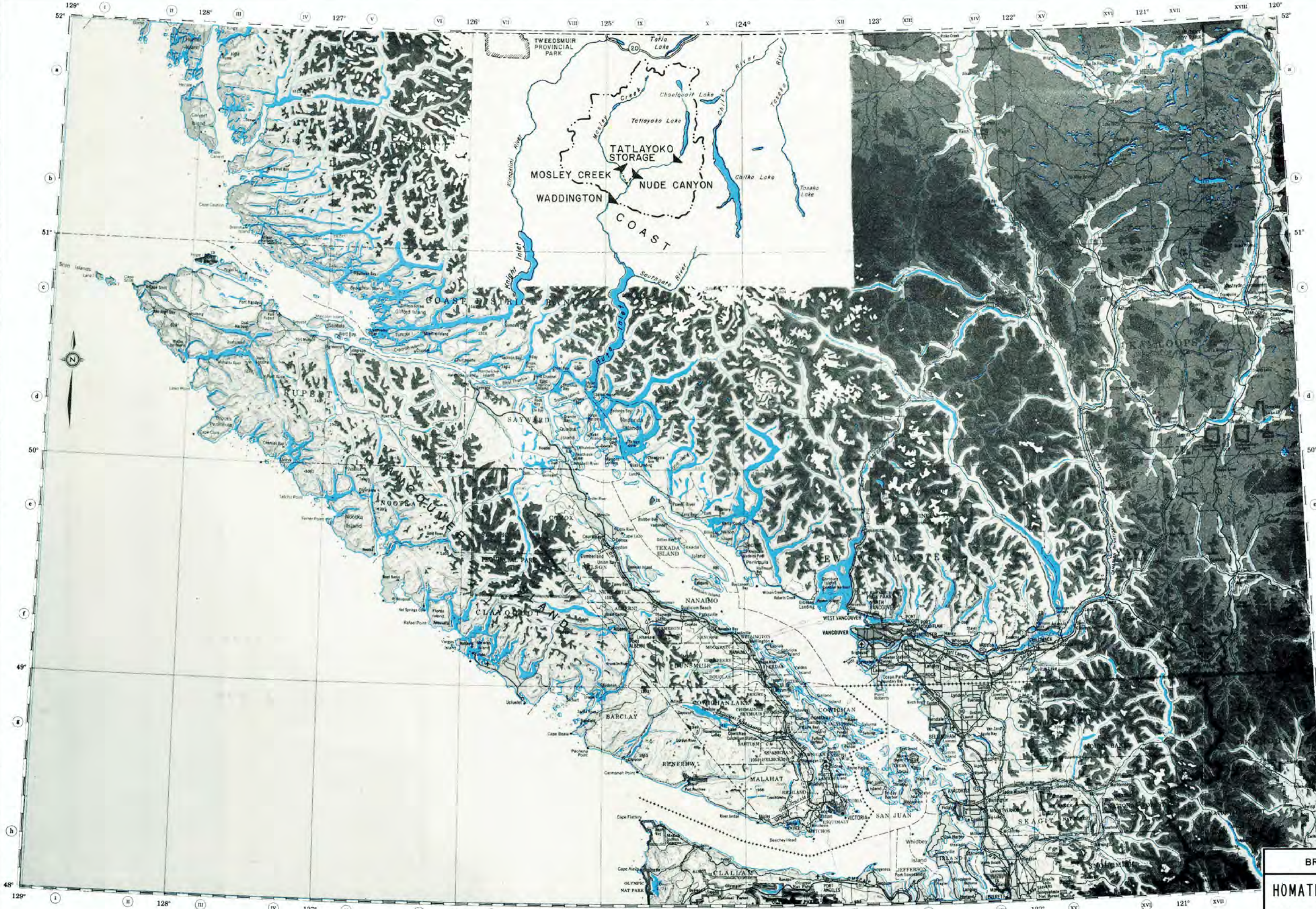
TABLE 11-1

HOMATHKO DEVELOPMENT
TATLAYOKO STORAGE PROJECT - BOWL LAKE AXIS
(22 MW - RES. EL. 760 m)
ESTIMATE OF CONSTRUCTION COST
($ Millions)

	Feature		
1.	Land and Rights		1.0
2.	Flowage		0.9
3.	Area Access		12.1
4.	Site Clearing		0.4
5.	Site Access		1.9
6.	Diversion Tunnel/Outlet Control		8.1
7.	Cofferdams and Unwatering		3.3
8.	Embankment Dam		45.1
9.	Tatlayoko Channel		3.5
10.	Spillway		6.0
11.	Power Intake		3.1
12.	Power Tunnel		5.2
13.	Powerhouse and Switchgear	- Civil	5.1
		- Mechanical	3.8
		- Electrical	13.6
14.	Construction Services		6.8
	DIRECT CONSTRUCTION COST		119.9
15.	Contingencies		22.1
16.	Project Final Design and Administration		14.2
17.	Construction Insurance and Bonds		1.4
	TOTAL CONSTRUCTION COST		157.6

Notes

1. Costs were estimated in October 1982 dollars and escalated to October 1983 by an increase of 7 1/2 percent.
2. Construction Services are 6 percent of the Direct Construction Cost.
3. Contingencies are 10 percent on mechanical and electrical items and 20 percent on other items.
4. Environmental Contingencies are not included.
5. Final Design and Construction Supervision is 10 percent of Direct Cost plus Contingencies. (Study Costs and Investigations are not included.)
6. Construction Insurance and Bonds are 1 percent of Direct Cost plus Contingencies.
7. Costs of Licencing are not included.
8. Interest during construction and corporate overhead costs are not included.
9. Includes the Bowl Lake portion of collector switching station electrical costs.
10. Cost of 69 kV line to collector station included.



LEGEND

— · · — *Outline of Drainage Basin*

▼ *Damsite*

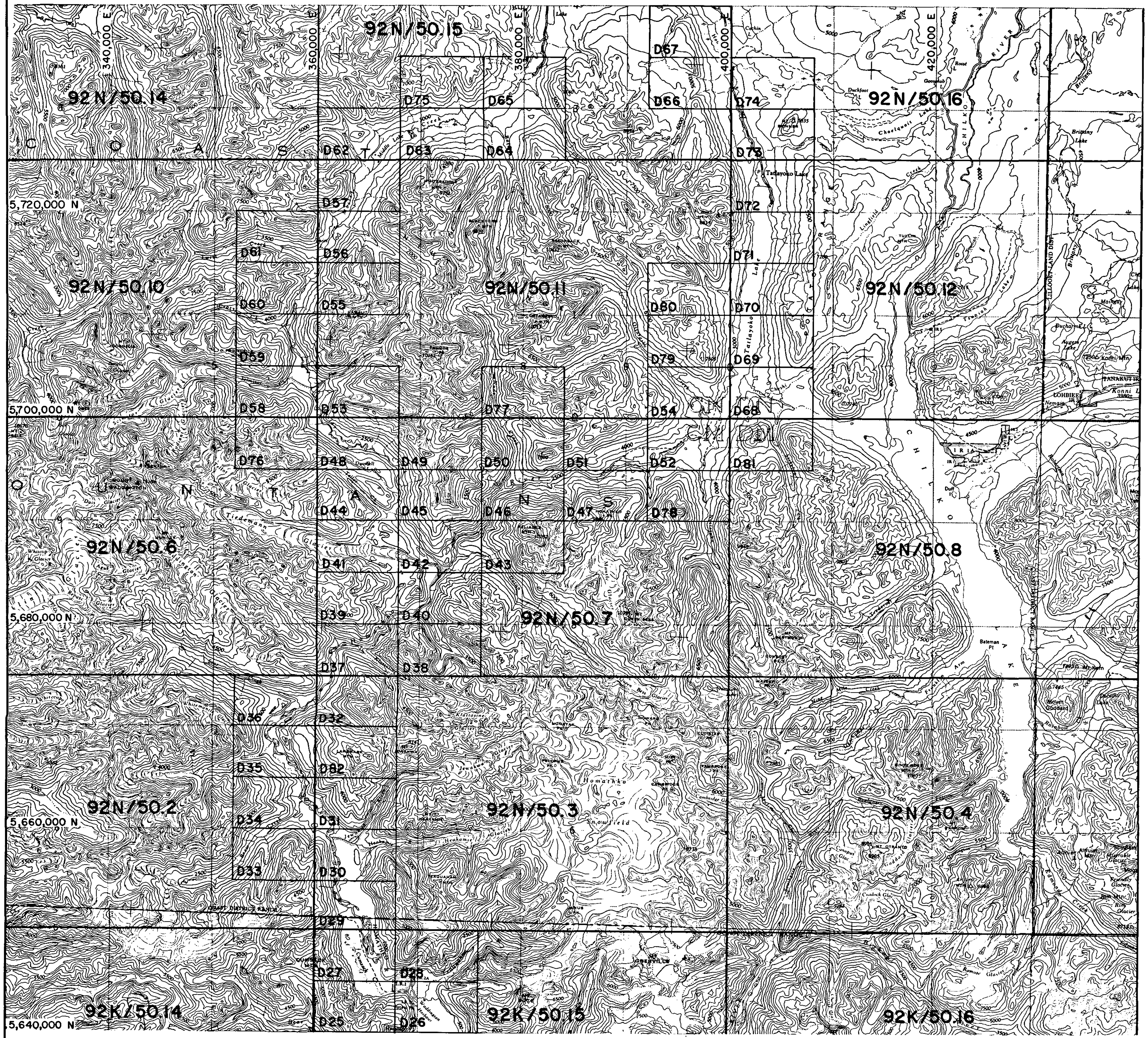
Scale : 10 0 10 20 30 40 Miles

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT

AREA PLAN

DATE MAR 1984	FIG. 2-1	R
DWN D.J.	DWG No. 704 - C14 - D1	



ALL DWG. NUMBERS SHOWN ARE PRECEDED BY 704 - C11-_____.

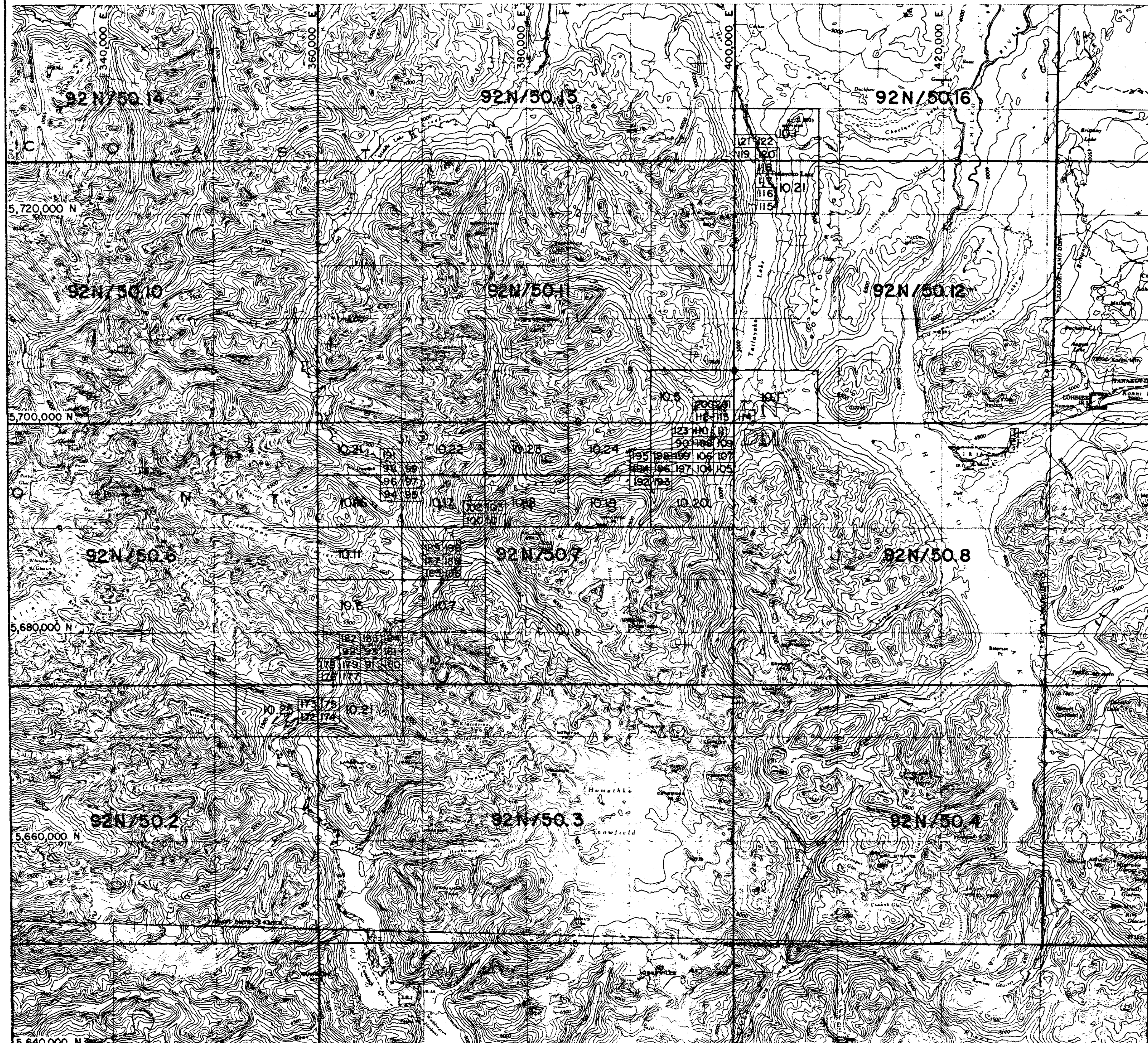
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

INDEX OF 1:10,000 SCALE TOPOGRAPHIC MAPPING

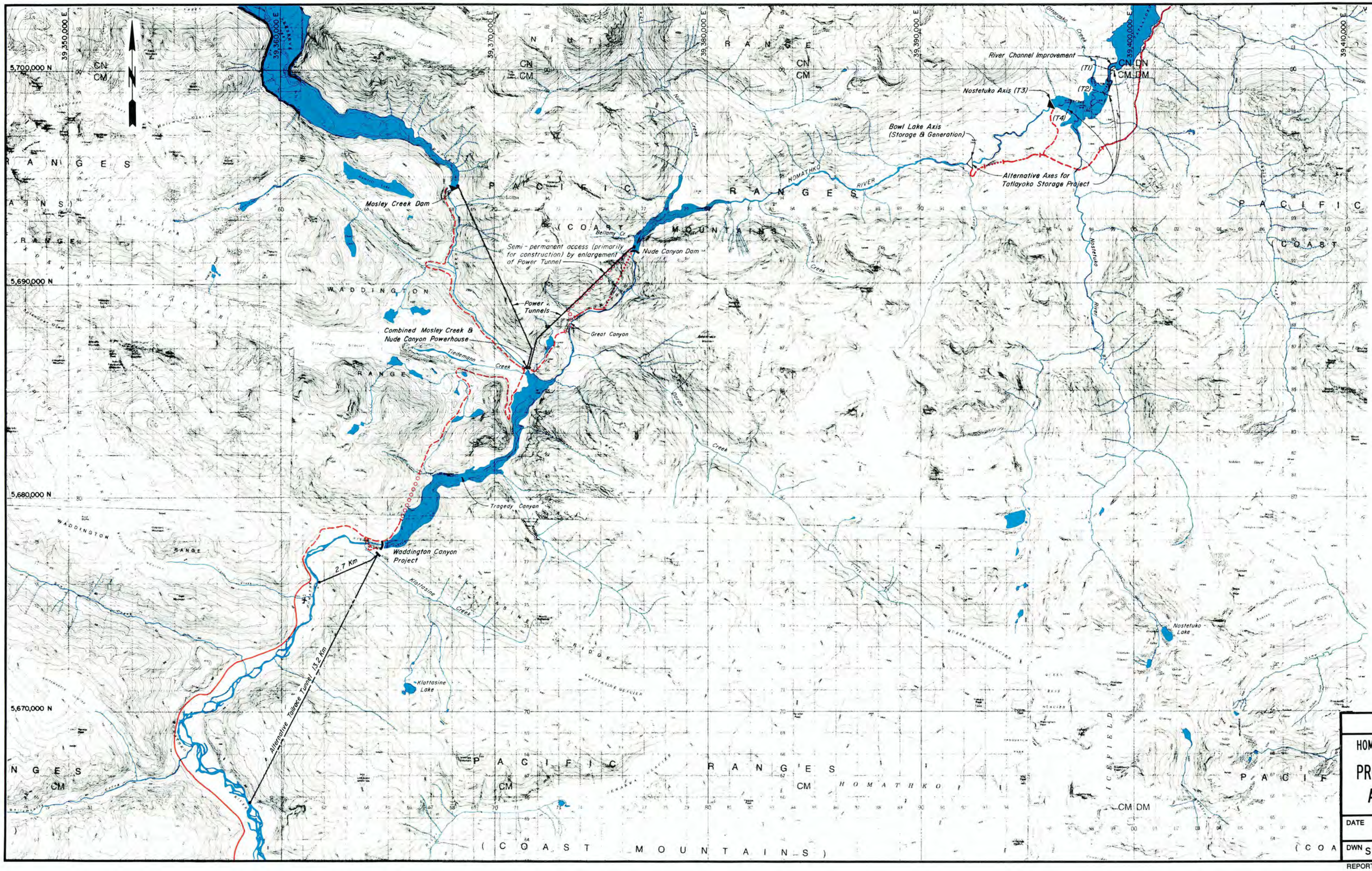
DATE MAR 1984	FIG. 2-2
DWN SJF	DWG No. 704 - C14 - B427

REPORT No. H1688



ALL DWG. NUMBERS SHOWN ARE PRECEDED BY 704 - C11 - D___.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT	
INDEX OF 1:2,500 SCALE TOPOGRAPHIC MAPPING	
DATE MAR 1984	FIG. 2-3 M
DWN SJF	DWG No. 704 - C14 - B428



NOTES:

1. IF PERMANENT ACCESS TO NUDE CANYON DAMSITE REQUIRED (NOT ASSUMED), ENLARGEMENT OF THE NUDE CANYON POWER TUNNEL WOULD BE DELETED.
2. CONTOURS ARE IN FEET.
3. TEMPORARY ROADS FOR CONSTRUCTION ACCESS ARE NOT SHOWN.

LEGEND:

PERMANENT ACCESS PROPOSED

- EXISTING ROAD
- NEW ROAD
- NEW TUNNEL

POSSIBLE ALTERNATIVE PERMANENT ACCESS

- NEW ROAD
- NEW TUNNEL

Scale: 1 0 1 2 3 4 5 6 Kilometres

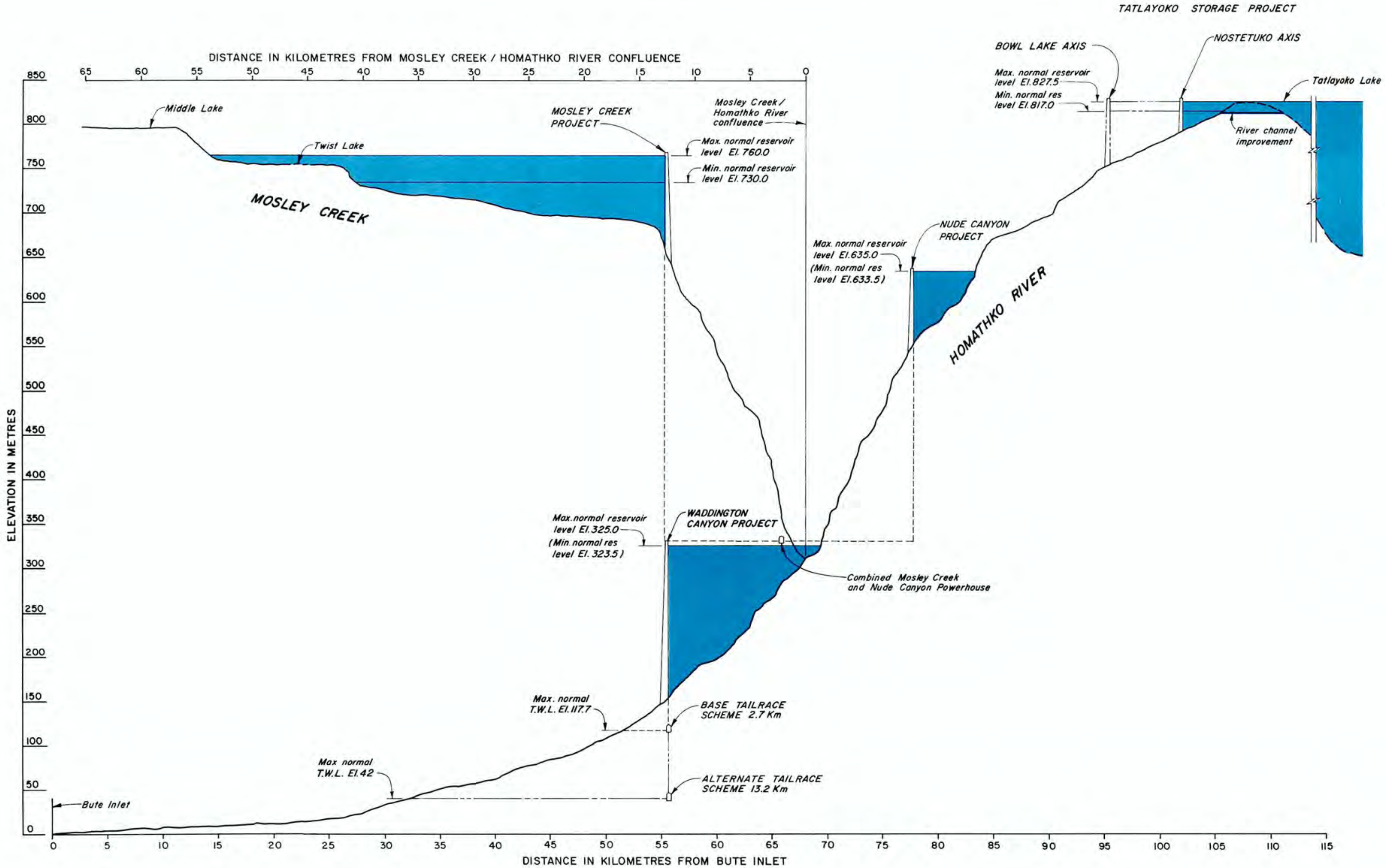
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT- INTERIM FEASIBILITY REPORT

PROJECT LOCATIONS, RESERVOIRS AND AREA ACCESS

DATE MAR 1984	FIG. 3-1	R
DWN SJF	DWG No. 704-C14-U429	

REPORT No. H1688



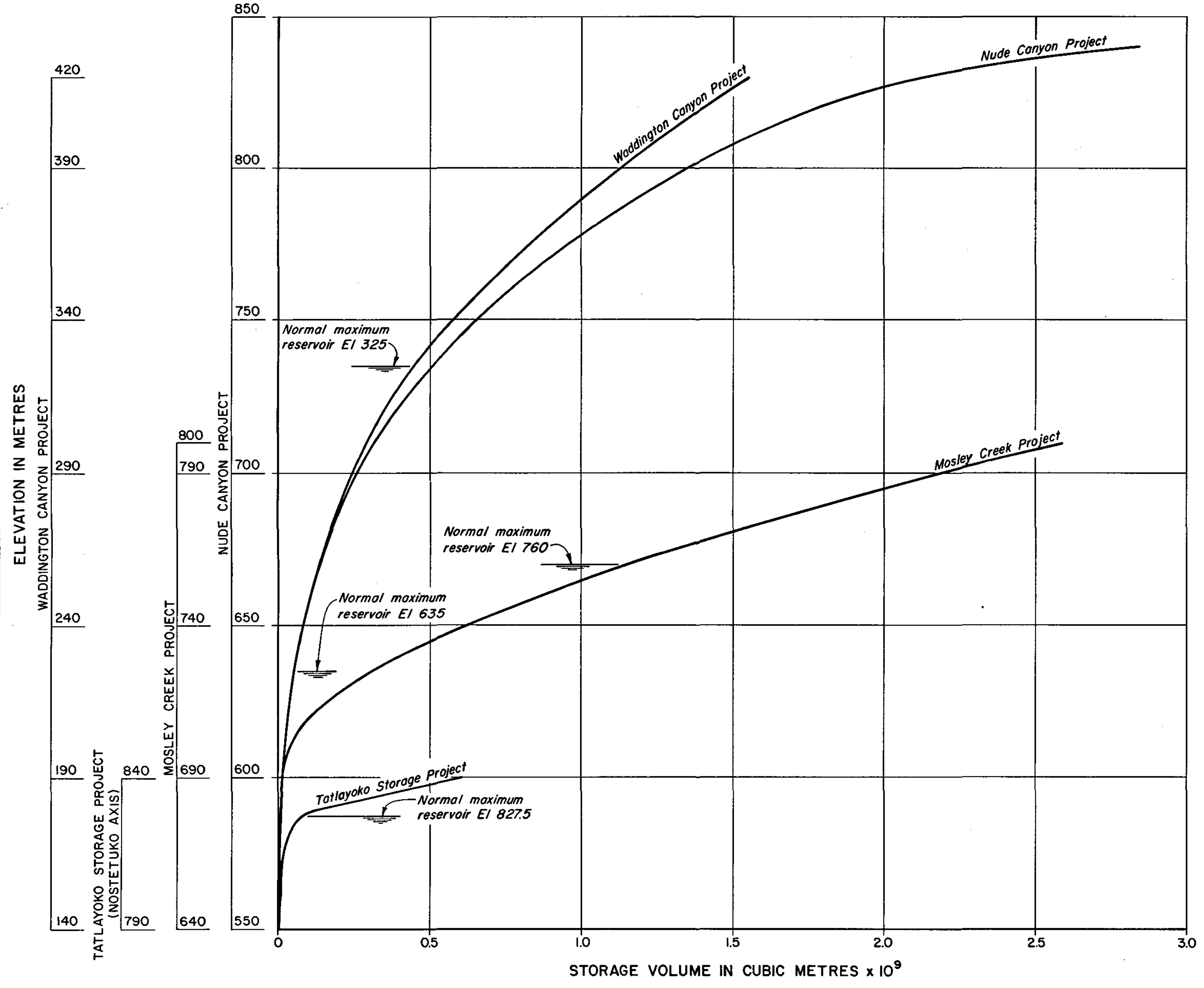
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

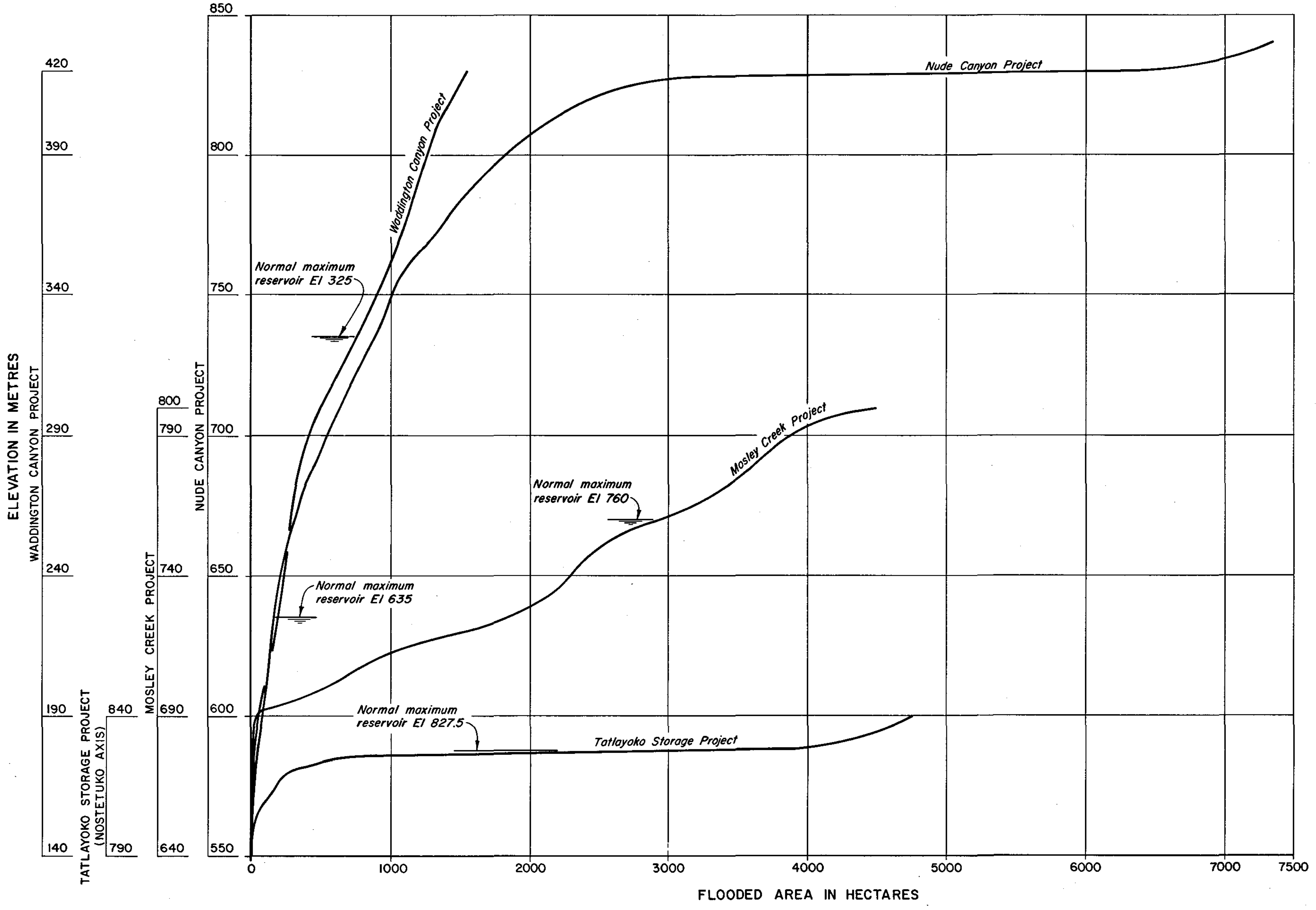
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

RIVER PROFILES SHOWING THE PROPOSED SCHEME OF DEVELOPMENT

DATE MAR 1984	FIG. 3-2	R
DWN SJF	DWG No. 704-C14-D430	

REPORT No. H1688





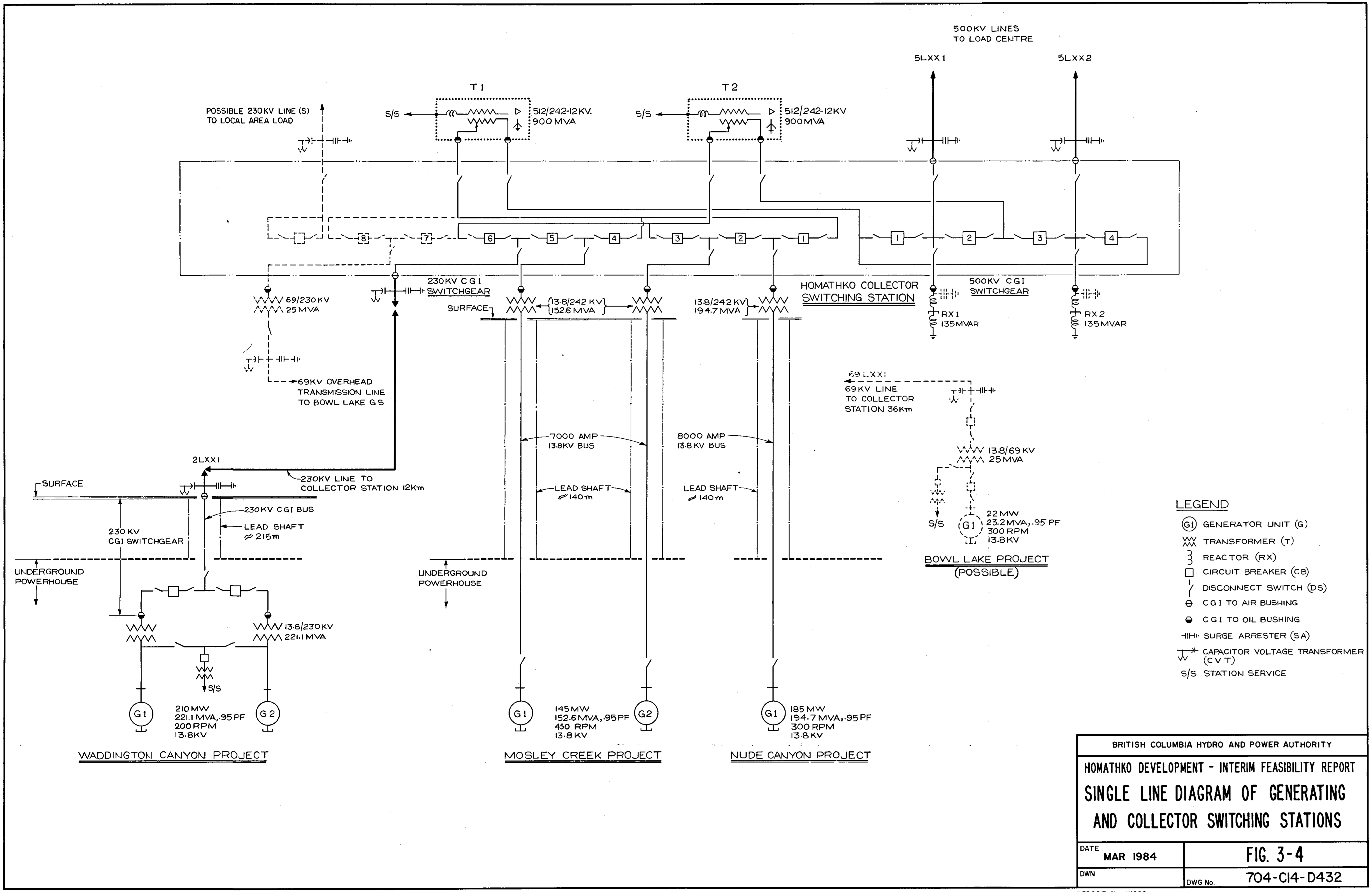
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

RESERVOIR STORAGE & FLOODED AREAS

DATE MAR 1984	FIG. 3-3
DWN DBG	DWG No. 704-C14-U481

REPORT No. H1688



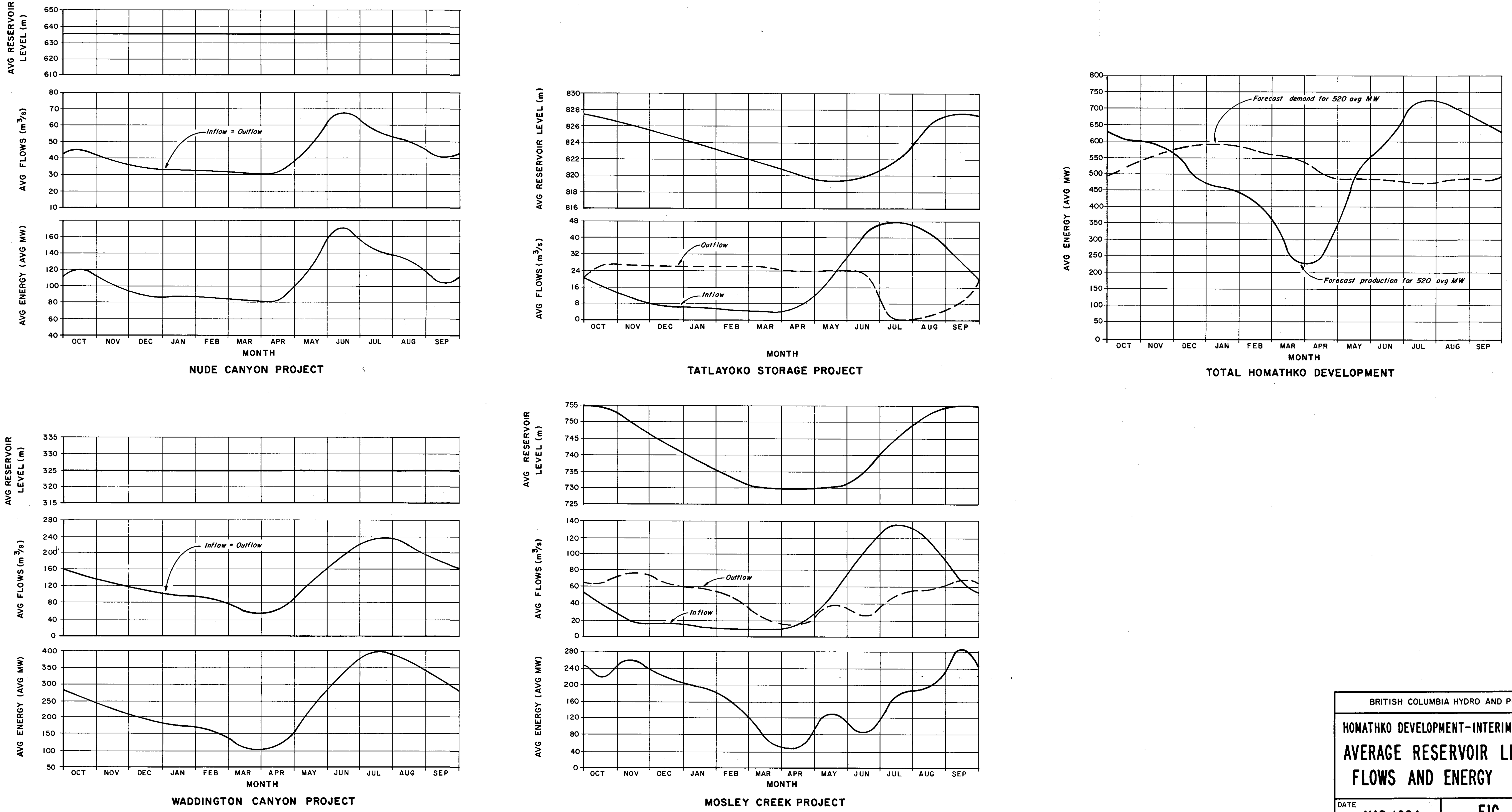
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

SINGLE LINE DIAGRAM OF GENERATING AND COLLECTOR SWITCHING STATIONS

DATE MAR 1984	FIG. 3-4
DWN	DWG No. 704-C14-D432

REPORT No. H1688



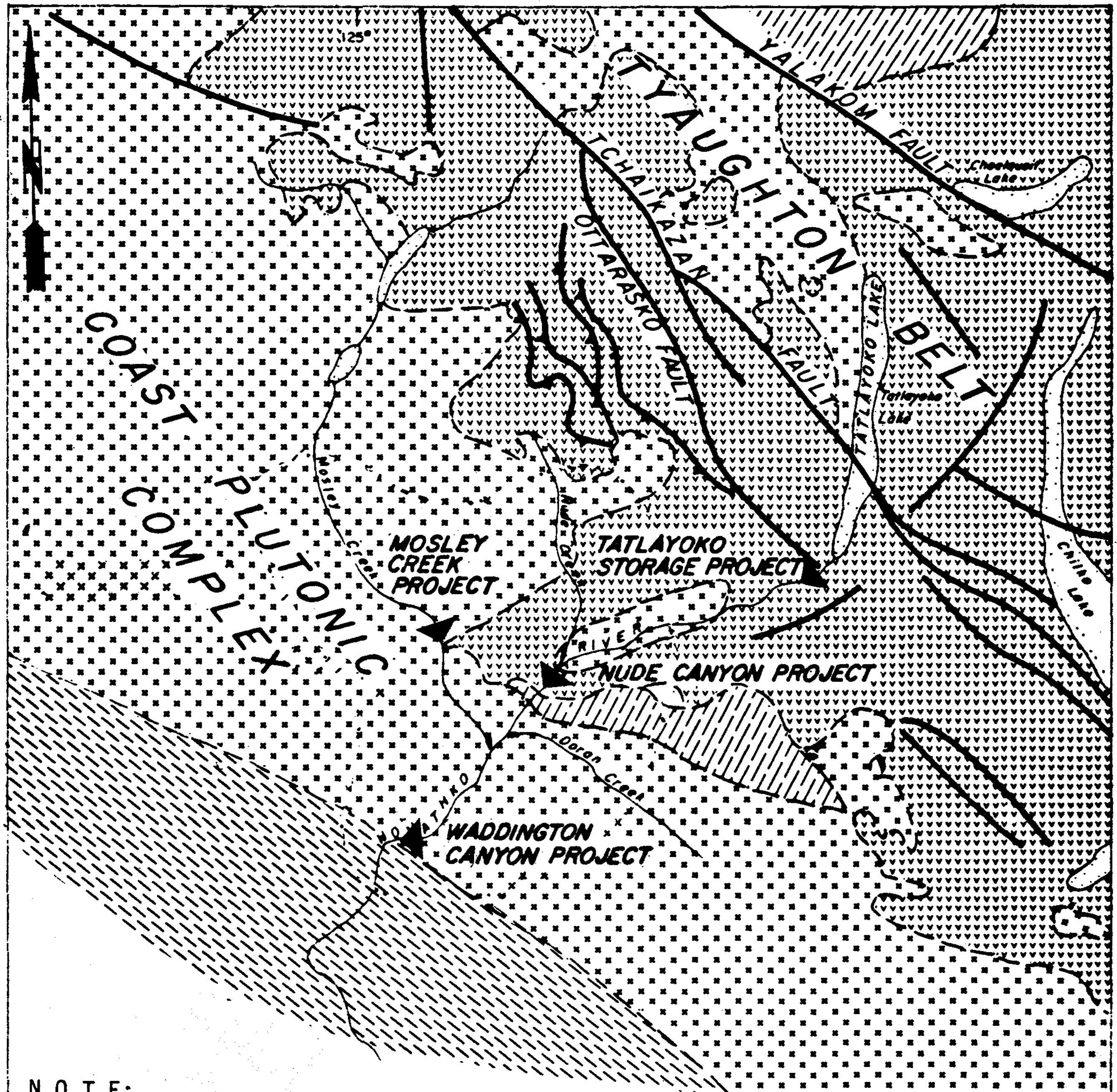
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

AVERAGE RESERVOIR LEVELS FLOWS AND ENERGY

DATE MAR 1984	FIG. 4-1
DWN BY	DWG No. 704-C14-U480

REPORT No H1688



NOTE:

1. ADAPTED FROM 1982 REGIONAL GEOLOGY REPORT BY DOLMAGE CAMPBELL & ASSOC.

Scale: 10 0 10 20 Km

LEGEND:

- VOLCANIC & CLASTIC SEDIMENTARY ROCKS
- COAST RANGE INTRUSIVES
- GNEISS COMPLEX
- MIGMATITE
- FAULT
- PROPOSED DAMSITE

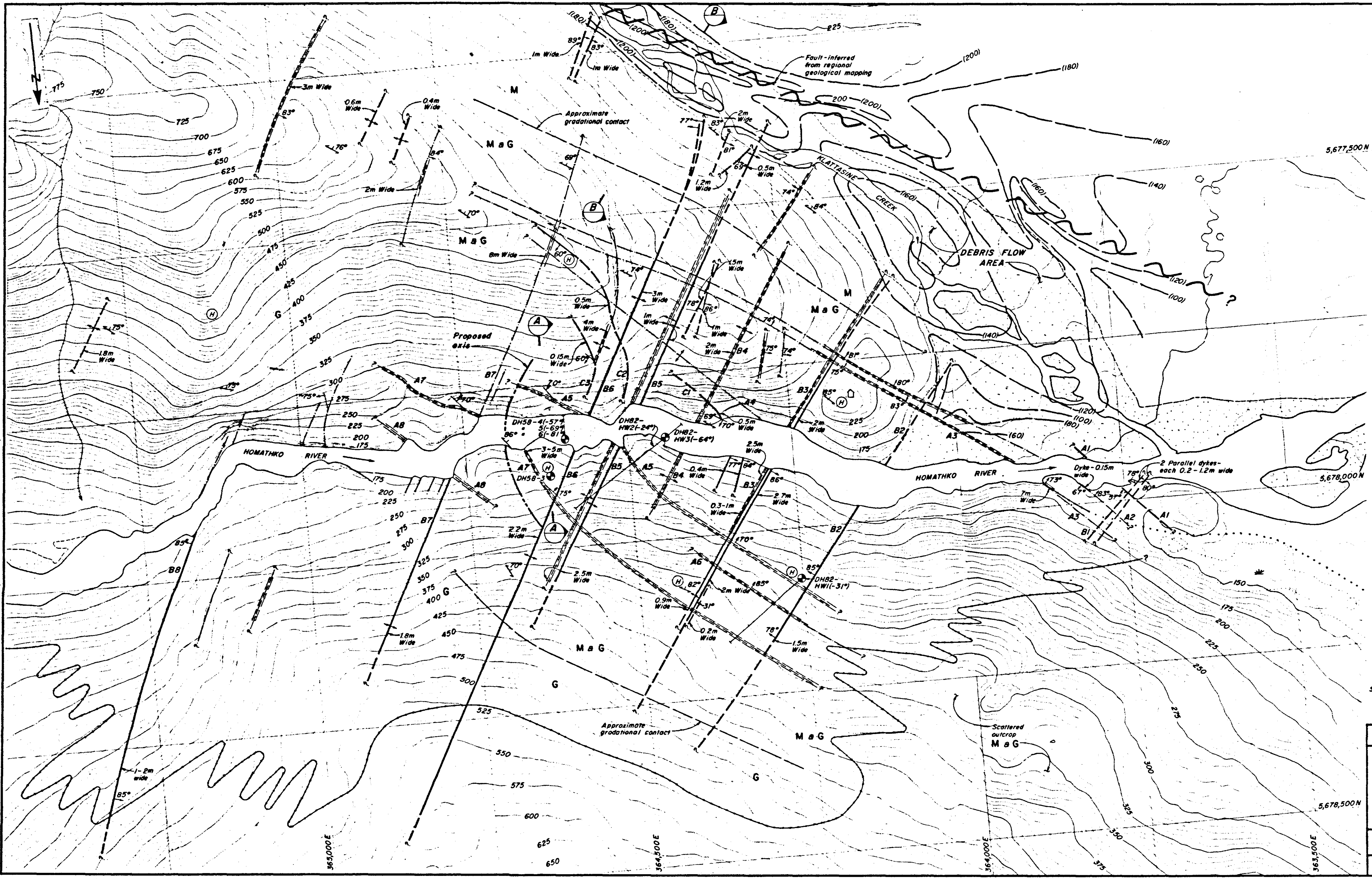
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

HOMATHKO PROJECT AREA REGIONAL BEDROCK GEOLOGY

DATE MAR 1984	FIG. 5-1
DWN K L	DWG. No. 704-C14-A433

REPORT No. H1688



NOTES:

1. FOR GEOLOGICAL SECTION, SEE FIG. 5-3.
2. TOPOGRAPHY IS PRELIMINARY.
3. ESTIMATED BEDROCK SURFACE CONTOURS ARE FROM 1983 GRAVITY SURVEY AND ARE PRELIMINARY. ALSO, THEY ARE REFERENCED TO TOPOGRAPHY SURVEYED DURING 1983 AND DO NOT FIT EXACTLY THE PRELIMINARY TOPOGRAPHY CONTOURS SHOWN.
4. LOCATION OF ALFRED WADDINGTON HISTORIC WAGON ROAD SHOWN IS FROM 1983 HERITAGE INVENTORY ASSESSMENT REPORT AND IS APPROXIMATE ONLY.

LEGEND:

SOIL AND ROCK TYPES

- DEBRIS FLOW
- OVERBURDEN - MOSTLY BOULDER GRAVEL (RE-WORKED SLIDE DEBRIS)
- M MIGMATITE COMPLEX
- G GRANITE
- M & G INTERLAYERED MIGMATITE COMPLEX AND GRANITE
- BASIC DYKE (OBSERVED, INFERRED)

STRUCTURE

- FOLIATION (INCLINED, VERTICAL)
- PROMINENT SINGLE CRACK (JOINT FACE)
- CLOSELY SPACED PARALLEL JOINTS
- GEOLOGICAL CONTACT - GRADATIONAL
- BEDROCK SURFACE CONTOURS
- A4 LINEAR FEATURE REFERRED TO IN TABLE 3 IN TEXT

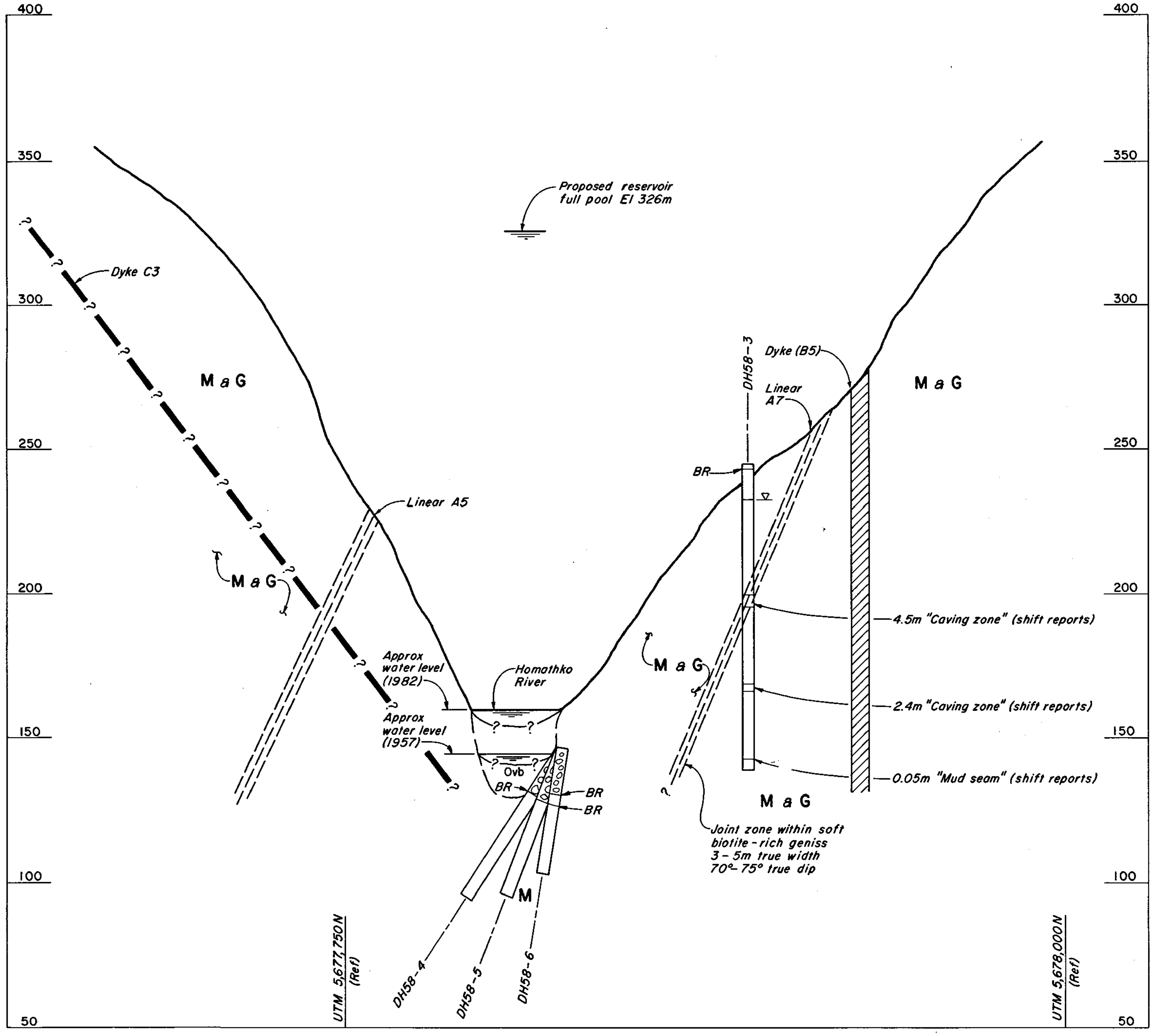
GENERAL SYMBOLS

- DIAMOND DRILL HOLE (INCLINED, VERTICAL)
- (H) HELIPAD
- CORE SHACK
- SWAMP
- ALFRED WADDINGTON HISTORIC WAGON ROAD (OBSERVED, INFERRED)

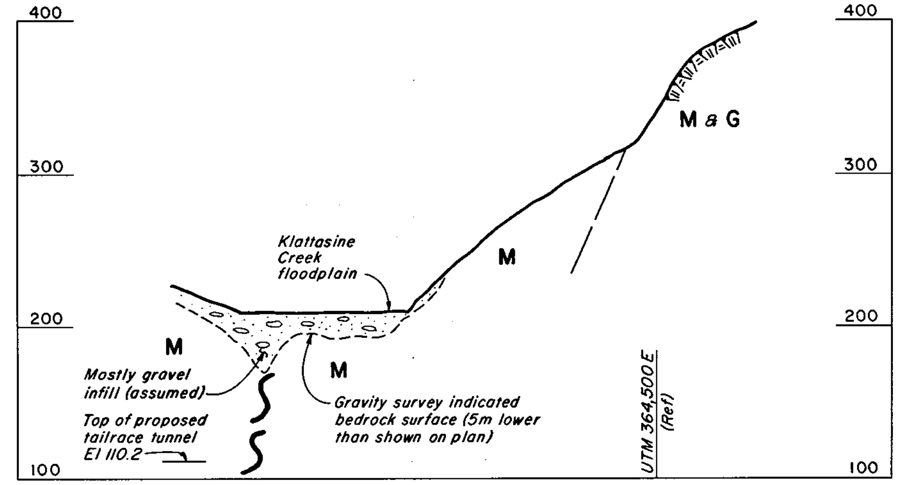
Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY		
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT		
WADDINGTON CANYON PROJECT GEOLOGICAL PLAN		M
DATE MAR 1984	FIG. 5-2	R
DWN KL	DWG No 704-C14-U434	

REPORT No. H1688



SECTION (A)
Scale: A



SECTION (B)
Scale: B

NOTES:

1. DRILL COLLAR ELEVATIONS FOR HOLES DH58-3-6 DETERMINED FROM 1958 SURVEY (B.C. POWER COMMISSION DWG. 704-X34469).
2. TOPOGRAPHY IS PRELIMINARY.
3. 1958 DRILL HOLE DETAILS ARE DERIVED FROM DRILLERS SHIFT REPORTS. DETAILED LOGS ARE NOT AVAILABLE.

LEGEND:

SOIL TYPES

- GRAVEL
- Ovb — OVERBURDEN - UNDIFFERENTIATED

ROCK TYPES

- ANDESITE DYKE (NARROW, WIDER)
- G — FOLIATED GRANITE
- M — INTERBANDED FOLIATED GRANITE AND MIGMATITE

STRUCTURAL INFORMATION

- GEOLOGICAL CONTACT
- FAULT
- JOINT ZONE - CLOSELY SPACED, PARALLEL JOINTS

DRILL HOLE DATA

- REFERENCE ELEVATION (DRILL DECK)
- BR — BEDROCK SURFACE
- R — DEPTH OF RUSTY FRACTURES
- C — GEOLOGICAL CONTACT (SHARP, PLANAR UNLESS NOTED OTHERWISE)
- WATER LEVEL
- CL — CORE LOSS
- F(36°) — FOLIATION (TRUE ANGLE WITH CORE AXIS)

Scale: B 50 0 50 100 150 200 Metres

Scale: A 20 0 20 40 60 80 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

WADDINGTON CANYON PROJECT GEOLOGICAL SECTIONS

DATE MAR 1984	FIG. 5-3	R
DWN KL	DWG No. 704-C14-D435	

REPORT No. H1688

NOTES:

1. TOPOGRAPHY IS PRELIMINARY.
2. FOR GEOLOGICAL SECTIONS SEE FIG. 5-5

LEGEND:

SOIL AND ROCK TYPES

- TALUS
- Ovb — OVERBURDEN - UNDIFFERENTIATED
- QD — QUARTZ DIORITE
- ANDESITE DYKE (OBSERVED, INFERRED)

GEOLOGIC SYMBOLS

- JOINT, (INCLINED, VERTICAL)
- FOLIATION (INCLINED, VERTICAL)
- LINEAR
- TENSION CRACK
- SLIDE SCARP
- OUTCROP BOUNDARY
- LIMITS OF AREA OF CONTINUOUS OR GOOD INTERMITENT OUTCROP EXAMINED
- DYKE REFERRED TO IN TEST

GENERAL SYMBOLS

- DIAMOND DRILL HOLE (INCLINED, VERTICAL)
- HELICOPTER PAD
- CORE SHACK
- CUT AND SURVEYED GRIDE LINE ENDS
- PROMINENT AVALANCHE TRACK

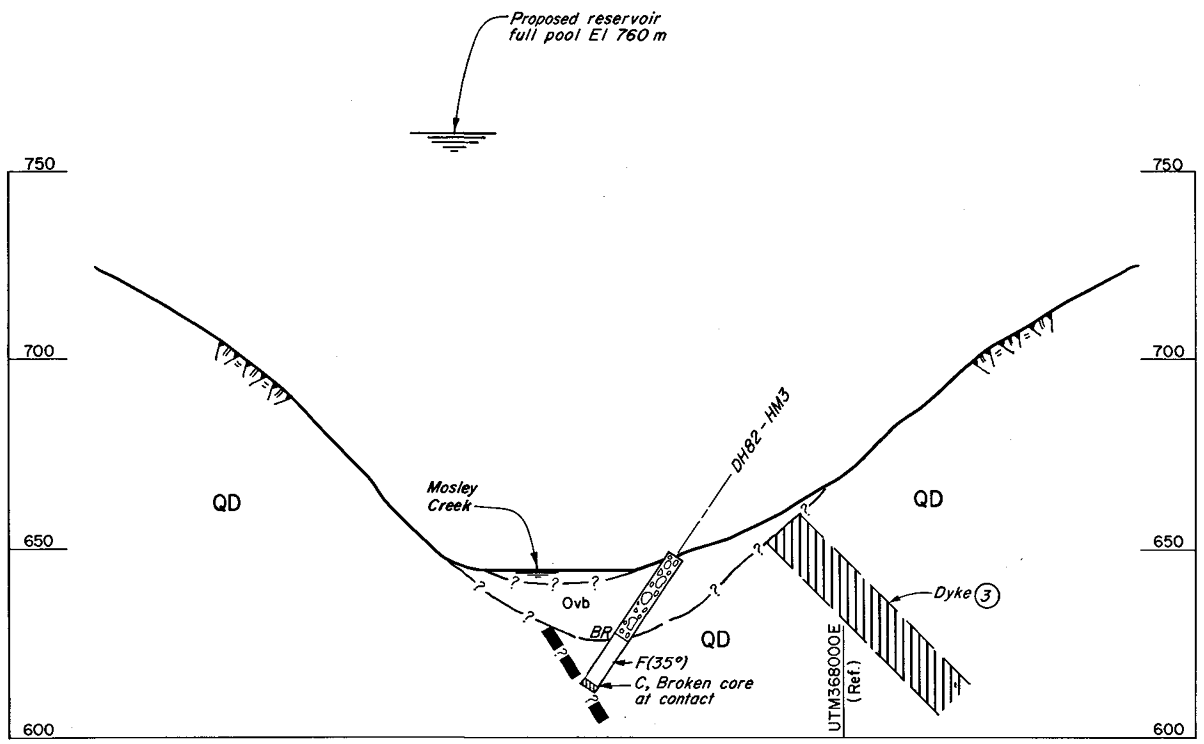
Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

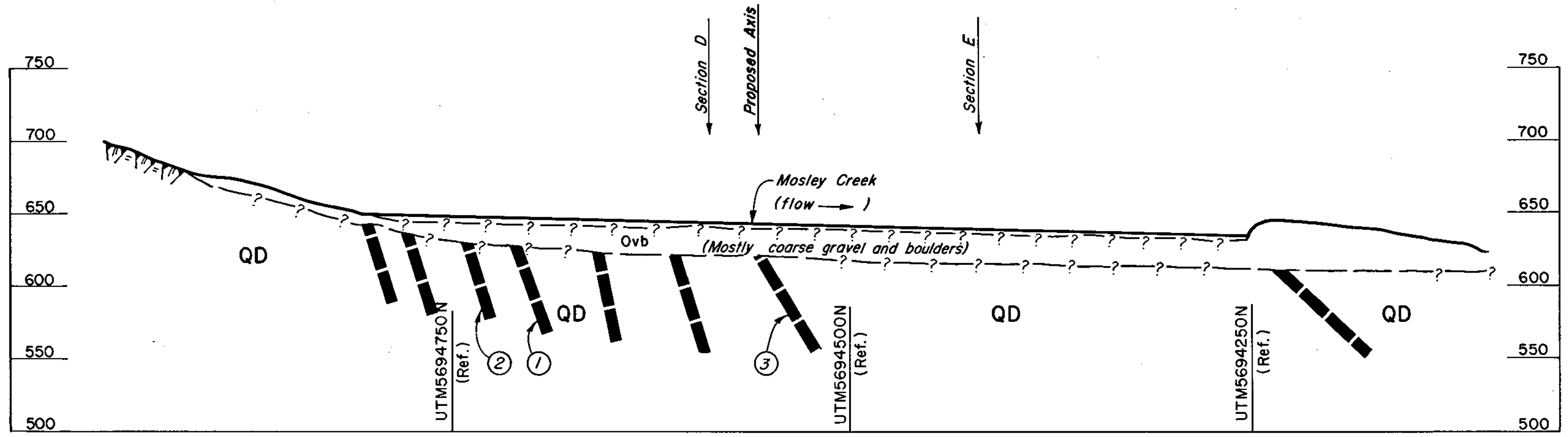
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

MOSLEY CREEK PROJECT GEOLOGICAL PLAN

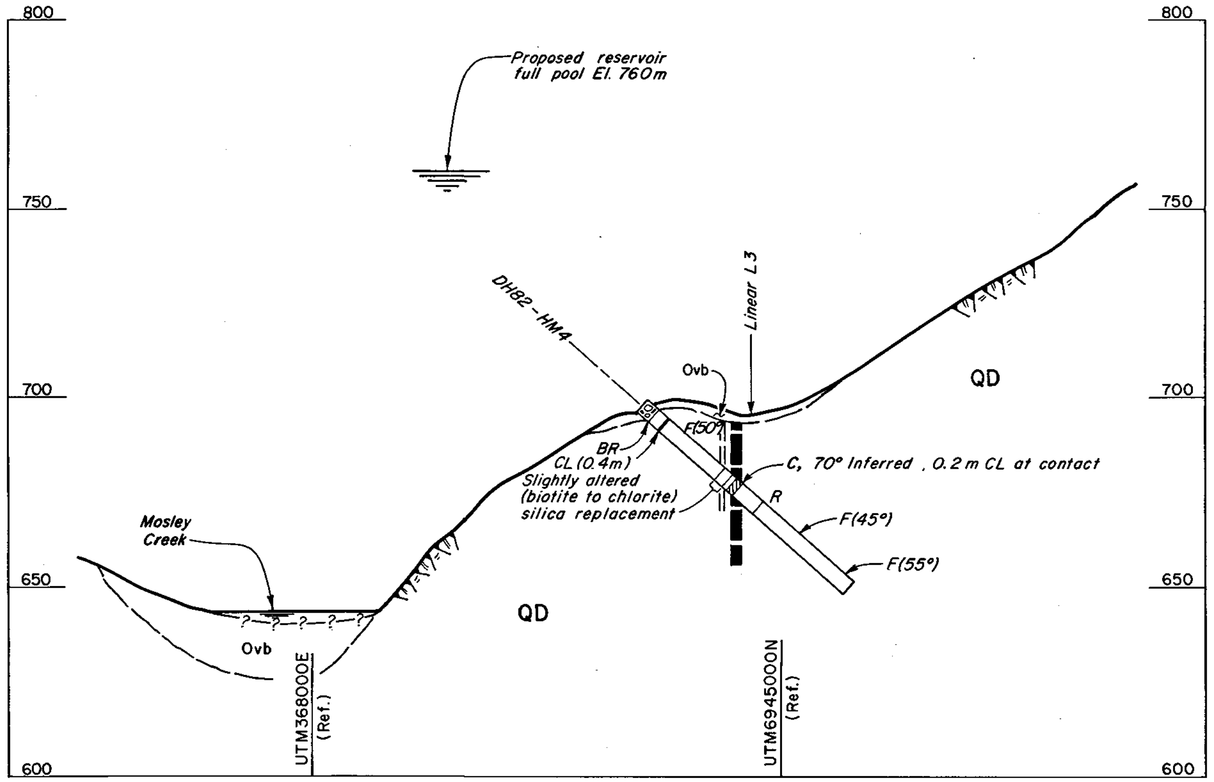
DATE MAR 1984	FIG. 5-4
DWN KL	DWG No 704-C14-D436



SECTION D
Scale A



SECTION F
Scale B



SECTION E
Scale A

LEGEND:

SOIL TYPES

- GRAVEL
- Ovb OVERBURDEN - UNDIFFERENTIATED

ROCK TYPES

- QD QUARTZ DIORITE
- ANDESITE DYKE (LARGE, SMALLER)

STRUCTURAL INFORMATION

GEOLOGICAL CONTACT

- ——— DEFINED
- — — — APPROXIMATE
- ?——? INFERRED

DRILL HOLE DATA

- ZERO POINT (DRILL DECK)
- BR BEDROCK SURFACE
- R DEPTH OF RUSTY FRACTURES
- C GEOLOGICAL CONTACT (SHARP AND PLANAR UNLESS NOTED OTHERWISE)
- BROKEN ZONE
- ▽ WATER LEVEL
- CL CORE LOSS
- F(130°) FOLIATION (TRUE ANGLE WITH CORE AXIS)
- DYKE

NOTES:

1. TOPOGRAPHY IS PRELIMINARY.

Scale: B 50 0 50 100 150 200 Metres

Scale: A 20 0 20 40 60 80 Metres

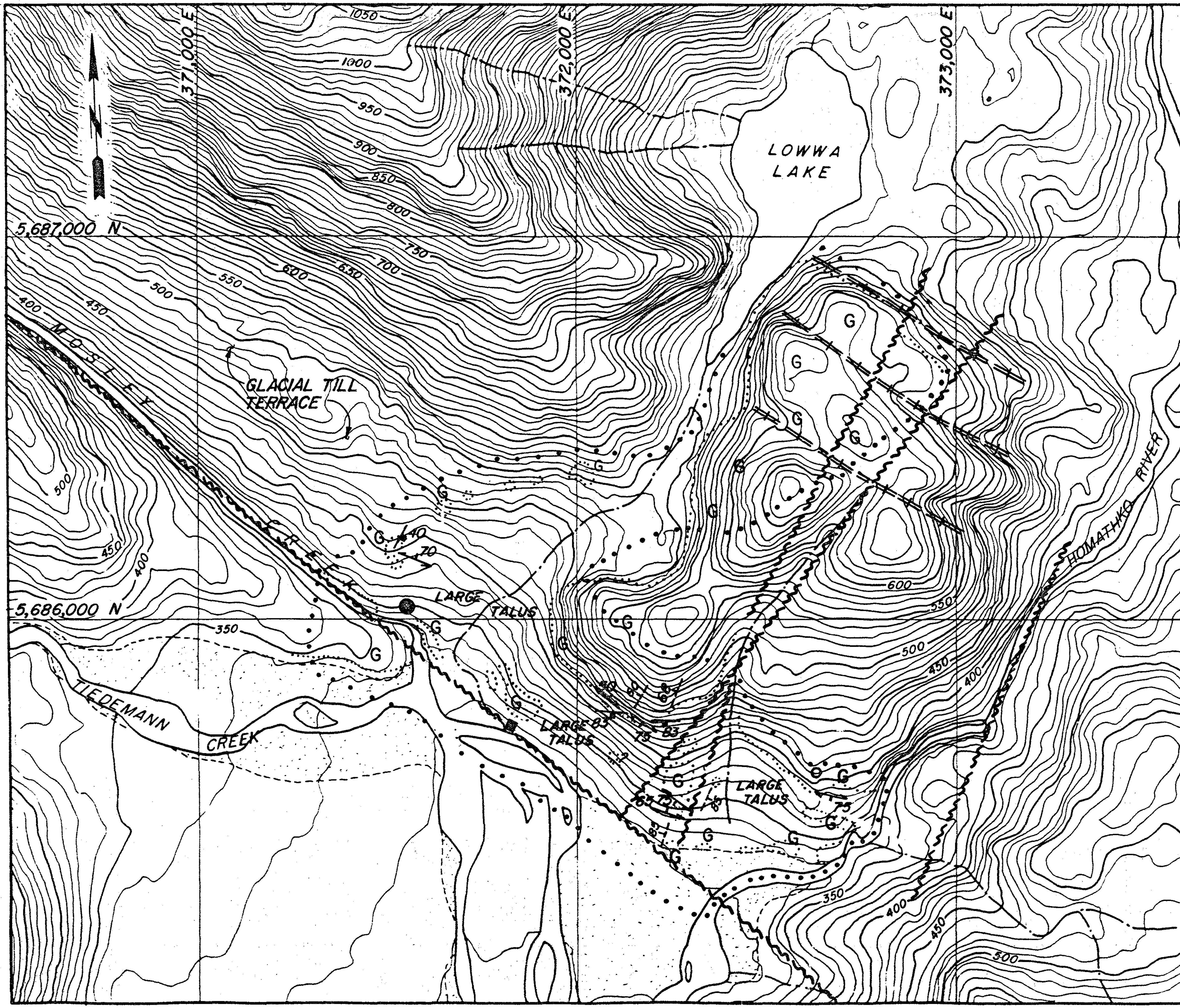
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

MOSLEY CREEK PROJECT GEOLOGICAL SECTIONS

DATE MAR 1984	FIG. 5-5	R
DWN KL	DWG No. 704-C14-D437	

REPORT No. H1688



NOTES:

1. GEOLOGICAL INTERPRETATION IS BASED ON LIMITED INFORMATION AND IS PRELIMINARY.

2. TOPOGRAPHY IS PRELIMINARY

LEGEND:

WEATHER TOWER (APPROXIMATE LOCATION)

OLD TRAPPER'S CABIN

PROPOSED POWERHOUSE SITE (APPROXIMATE)

FAULT, ASSUMED

LINEAR

50 FOLIATION

60 FRACTURE

40 JOINTING

G OUTCROP-ALL GRANDIORITE

LIMIT OF MAPPING

Scale: 200 0 200 400

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

NUDE / MOSLEY POWERHOUSE GEOLOGICAL PLAN

M

DATE MAR 1984	FIG. 5 - 6
DWN SJF	DWG No 704-C14-B438

REPORT No H1688

NOTES:

1. FOR GEOLOGICAL SECTION, SEE FIG. 5-8
2. TOPOGRAPHY IS PRELIMINARY.

LEGEND:

ROCK TYPES

M	METAMORPHIC COMPLEX - PHYLLITES, SCHISTS, GNEISSES, AND METAVOLCANICS - TYPICALLY PERMEATED EXTENSIVELY AND FURTHER ALTERED BY IRREGULAR TO SILL-LIKE BODIES OF GRANITIC INTRUSIVE ROCKS
Gn	GNEISSIC
Sch	SCHISTOSE
In	INTRUSIVE ROCKS - MOSTLY QUARTZ DIORITE - LARGER BODIES
	INTRUSIVE ROCKS - SMALLER "STOCKWORK" (SHOWN ONLY WHERE CLEARLY TRACEABLE)

GEOLOGICAL SYMBOLS

FOLIATION - GNEISSIC BANDING OR SCHISTOSITY (INCLINED, VERTICAL) W = WEAK FOLIATION

GEOLOGICAL CONTACT (OBSERVED, INFERRED)

FAULT (OBSERVED, INFERRED)

OUTCROP BOUNDARY

LIMITS OF AREA OF CONTINUOUS OUTCROP, OR EXTENSIVE INTERMITTANT OUTCROP, EXAMINED

GENERAL

DIAMOND DRILL HOLE (INCLINED, VERTICAL)

HELIPAD

CORE SHACK

GLACIAL STRIAE

EDGE OF LINEAR DRAW

SURVEY STATION

PROMINENT AVALANCHE TRACK

Scale: 50 0 50 100 150 200 Metres

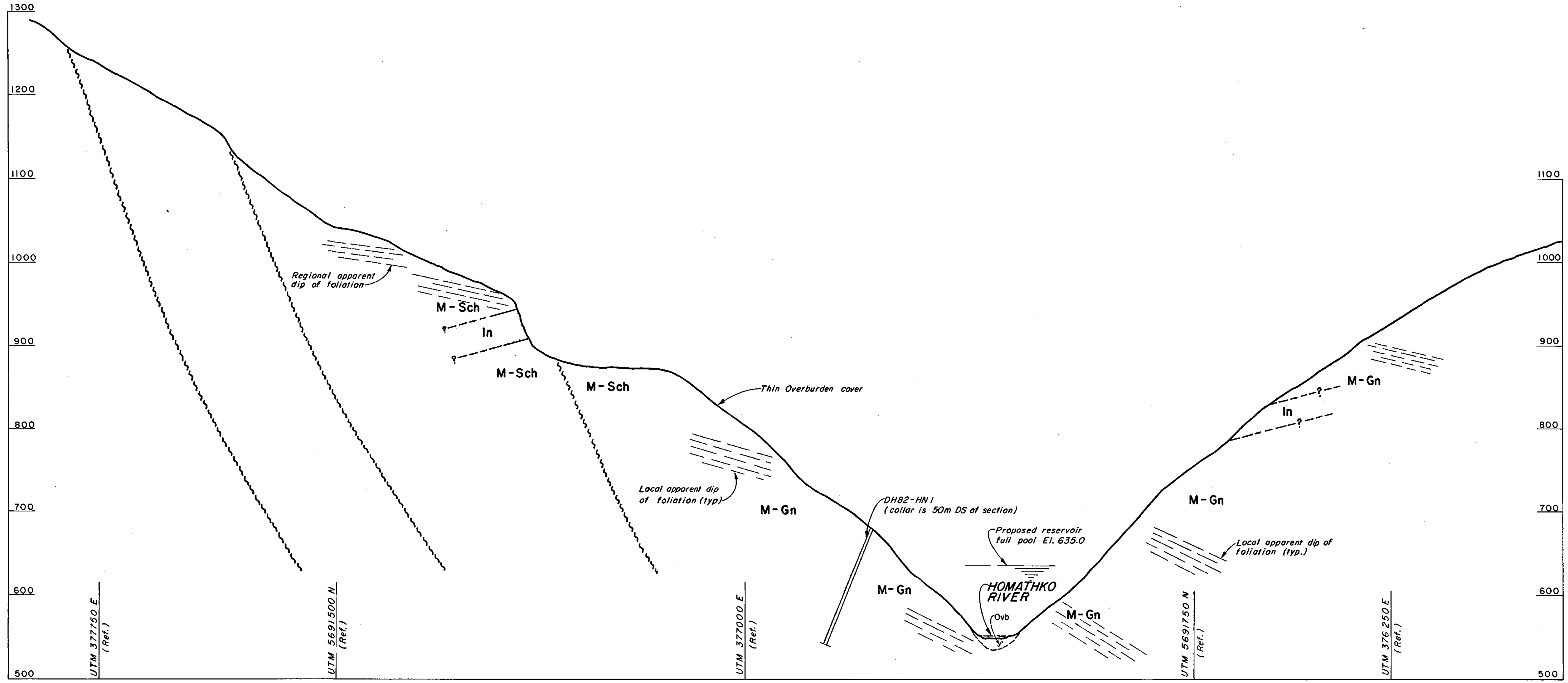
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

NUDE CANYON PROJECT
GEOLOGICAL PLAN

DATE MAR 1984	FIG. 5-7 R
DWN SJF	DWG No. 704-C14-U439

REPORT No. H1688



SECTION (A)

(Coincides with proposed Dam Axis)

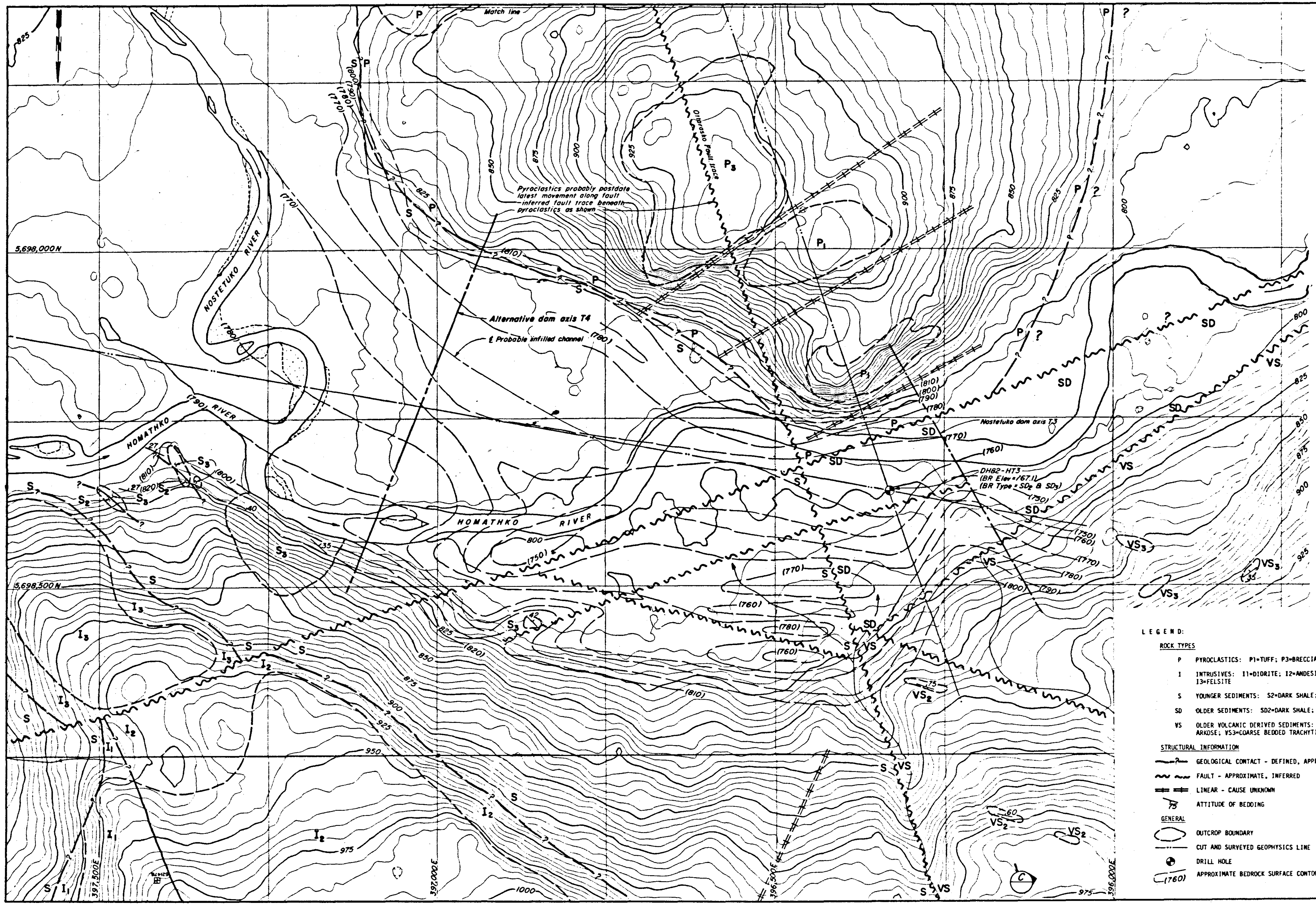
Scale: 50 0 50 100 150 200 Metres

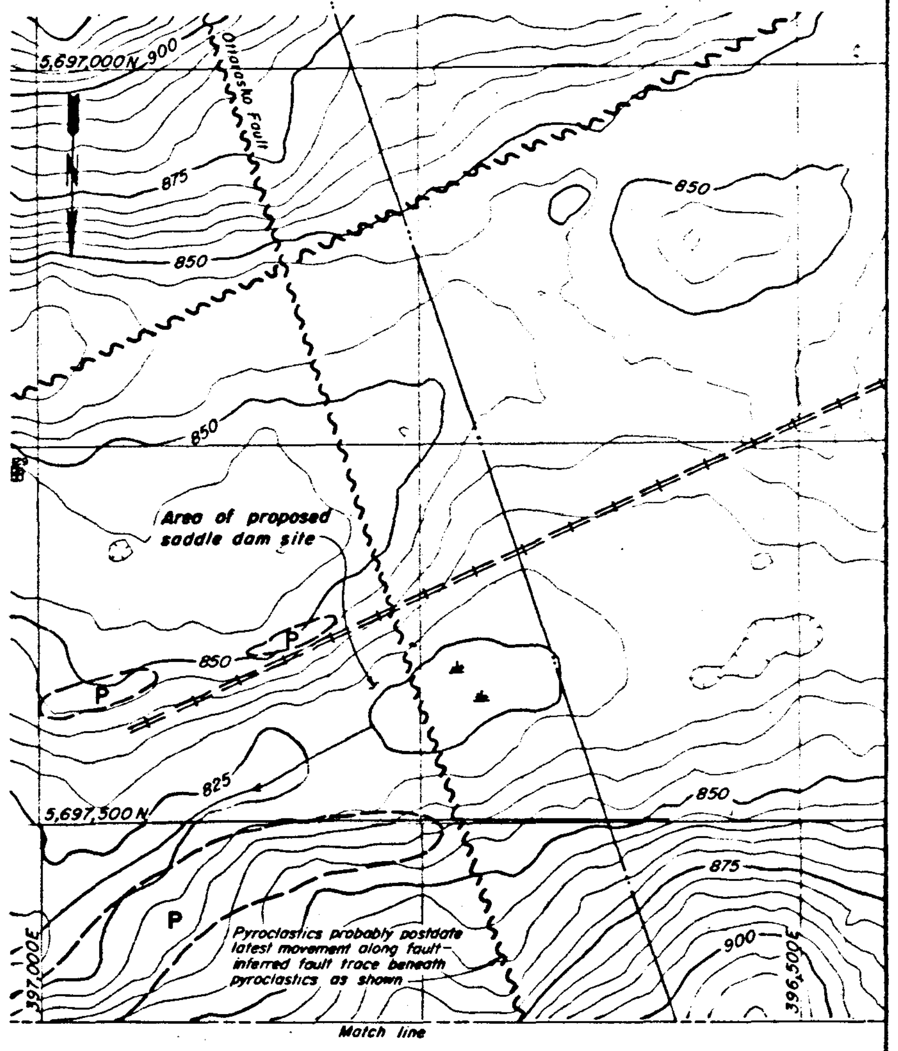
NOTES:

1. SEE GEOLOGICAL PLAN, FIG. 5-7 FOR LEGEND.
2. TOPOGRAPHY IS PRELIMINARY.
3. GEOLOGICAL INTERPRETATION IS BASED ON LIMITED INFORMATION AND IS PRELIMINARY.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT	
NUDE CANYON PROJECT GEOLOGICAL SECTION	
DATE MAR 1984	FIG. 5 - 8
DWN CIC	DWG No. 704-C14-D440

REPORT No. H1688





NOTES:

1. TOPOGRAPHY IS PRELIMINARY.
2. GEOLOGICAL INTERPRETATION IS PRELIMINARY.
3. THE ROCK TYPES LISTED ARE AN ABBREVIATION OF A LEGEND COVERING A LARGE AREA.
4. BEDROCK SURFACE CONTOURS ARE INTERPRETED FROM LIMITED GEOPHYSICAL DATA AND ARE PRELIMINARY.

LEGEND:

ROCK TYPES

- P PYROCLASTICS: P1=TUFF; P3=BRECCIA
- I INTRUSIVES: I1=DIORITE; I2=ANDESITE ALTERATION; I3=FELSITE
- S YOUNGER SEDIMENTS: S2=DARK SHALE; S3=ARKOSE
- SD OLDER SEDIMENTS: SD2=DARK SHALE; SD3=ARKOSE
- VS OLDER VOLCANIC DERIVED SEDIMENTS: VS2=TUFFACEOUS ARKOSE; VS3=COARSE BEDDED TRACHYTIC RUBBLE

STRUCTURAL INFORMATION

- GEOLOGICAL CONTACT - DEFINED, APPROXIMATE
- FAULT - APPROXIMATE, INFERRED
- LINEAR - CAUSE UNKNOWN
- 75 ATTITUDE OF BEDDING

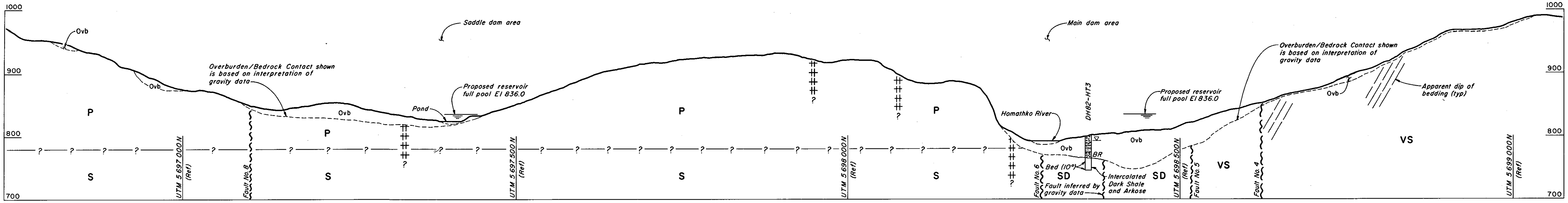
GENERAL

- OUTCROP BOUNDARY
- CUT AND SURVEYED GEOPHYSICS LINE
- DRILL HOLE
- (760) APPROXIMATE BEDROCK SURFACE CONTOUR

Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT TATLAYOKO STORAGE PROJECT NOSTETUKO AXIS GEOLOGICAL PLAN	
DATE MAR 1984	FIG. 5-9
DWN KL	DWG No 704-C14-U441

REPORT No H1688



SECTION C

(Coincides approximately with proposed axis T3)

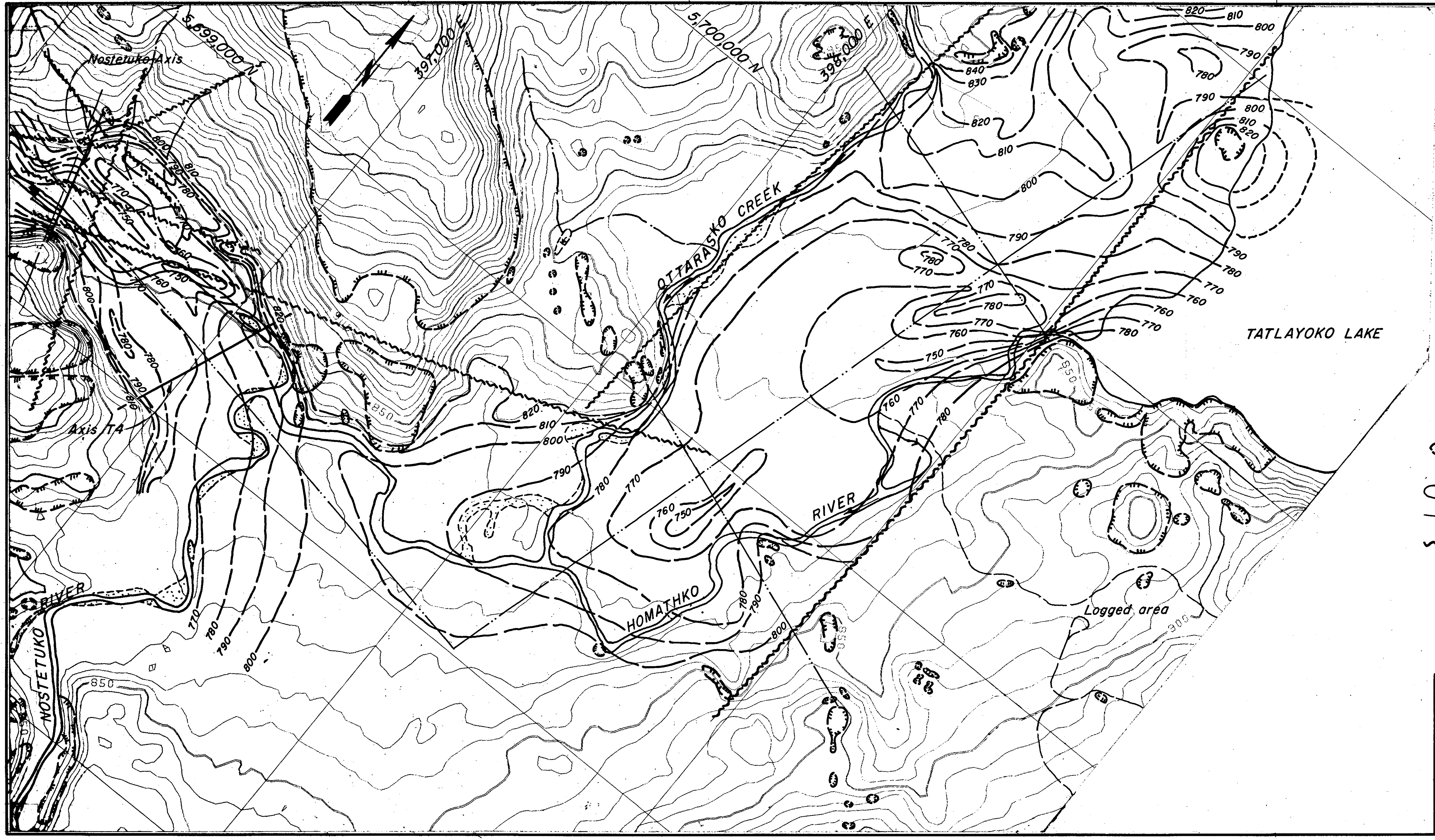
NOTES:

1. FOR GEOLOGICAL PLAN, SEE FIG. 5-9
2. FOR LEGEND SEE FIG. 5-9
3. TOPOGRAPHY IS FROM SURVEYED GEOPHYSICAL LINES.
4. GEOLOGICAL INTERPRETATION IS BASED ON LIMITED INFORMATION AND IS PRELIMINARY.

Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT TATLAYOKO STORAGE PROJECT NOSTETUKO AXIS GEOLOGICAL SECTION	
DATE MAR 1984	FIG. 5-10
DWN BE	DWG No 704-C14-U442

REPORT No. H1688



NOTES:

1. BEDROCK SURFACE CONTOURS ARE BASED ON LIMITED INFORMATION AND ARE PRELIMINARY.

2. TOPOGRAPHY IS PRELIMINARY.

LEGEND:

GENERALIZED OUTCROP BOUNDARY

750 BEDROCK CONTOURS

CUT AND SURVEYED LINES

FAULT: INFERRED

Scale: 0 100 200 300 400 500 Metres

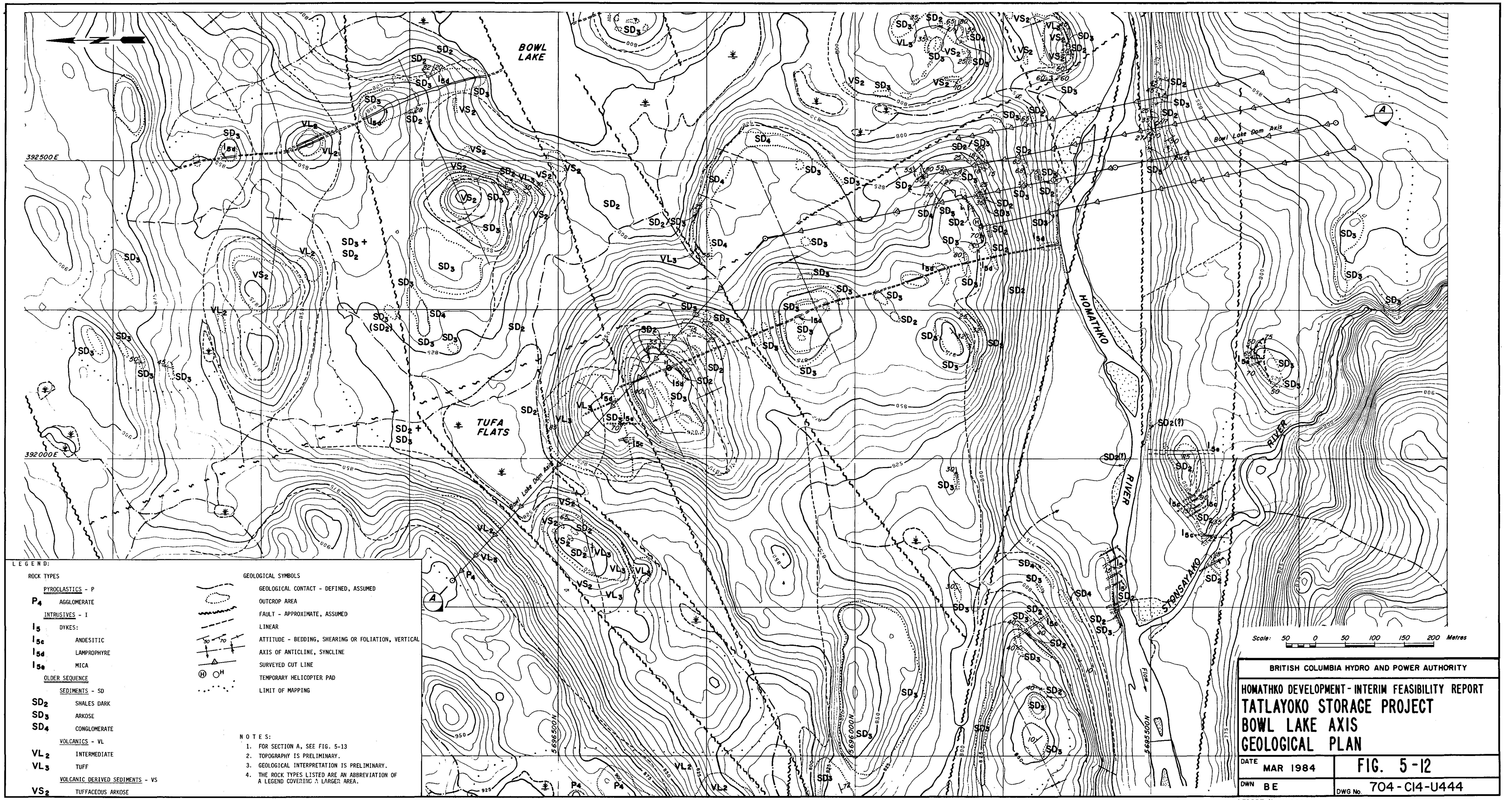
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT
NOSTETUKO AXIS
BEDROCK SURFACE CONTOURS

DATE MAR 1984	FIG. 5-11
DWN SJF	DWG. No. 704-C14-C443

REPORT No. H1688



LEGEND:

ROCK TYPES

PYROCLASTICS - P

P_4 AGGLOMERATE

INTRUSIVES - I

I_5 DYKES:

I_{5c} ANDESITIC

I_{5d} LAMPROPHYRE

I_{5e} MICA

OLDER SEQUENCE

SEDIMENTS - SD

SD_2 SHALES DARK

SD_3 ARKOSE

SD_4 CONGLOMERATE

VOLCANICS - VL

VL_2 INTERMEDIATE

VL_3 TUFF

VOLCANIC DERIVED SEDIMENTS - VS

VS_2 TUFFACEOUS ARKOSE

GEOLOGICAL SYMBOLS

GEOLOGICAL CONTACT - DEFINED, ASSUMED

OUTCROP AREA

FAULT - APPROXIMATE, ASSUMED

LINEAR

ATTITUDE - BEDDING, SHEARING OR FOLIATION, VERTICAL

AXIS OF ANTICLINE, SYNCLINE

SURVEYED CUT LINE

TEMPORARY HELICOPTER PAD

LIMIT OF MAPPING

NOTES:

1. FOR SECTION A, SEE FIG. 5-13
2. TOPOGRAPHY IS PRELIMINARY.
3. GEOLOGICAL INTERPRETATION IS PRELIMINARY.
4. THE ROCK TYPES LISTED ARE AN ABBREVIATION OF A LEGEND COVERING A LARGER AREA.

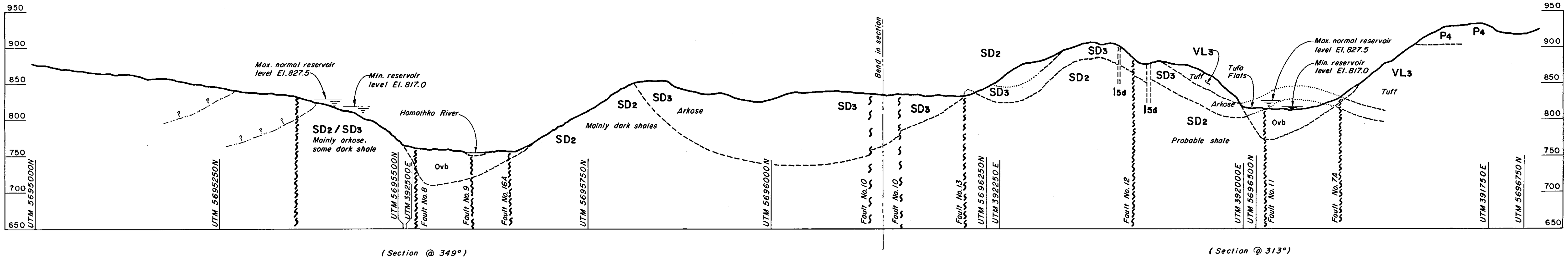
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT BOWL LAKE AXIS GEOLOGICAL PLAN

DATE MAR 1984	FIG. 5-12
DWN BE	DWG No. 704-C14-U444

REPORT No. H1688



SECTION (A)

(Coincides with Bowl Lake Dam Axis)

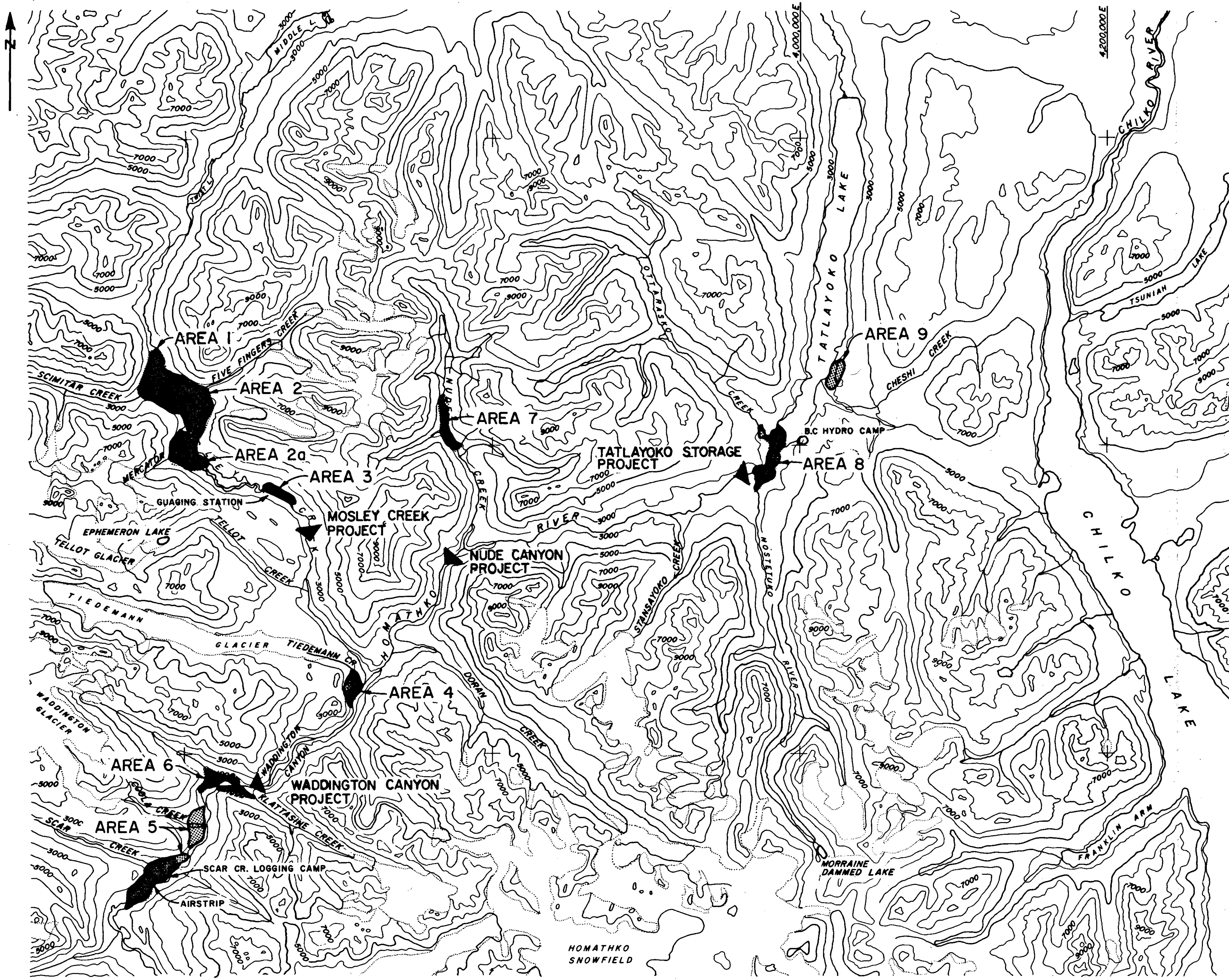
NOTES:

1. FOR GEOLOGICAL PLAN, SEE FIG. 5-12
2. SEE FIG. 5-12 FOR LEGEND.
3. GEOLOGICAL INTERPRETATION IS BASED ON LIMITED INFORMATION AND IS PRELIMINARY.

Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT TATLAYOKO STORAGE PROJECT BOWL LAKE AXIS GEOLOGICAL SECTION	
DATE MAR 1984	FIG. 5 - 13
DWN B E	DWG No. 704 - C14 - U445

REPORT No. H1688



POTENTIAL BORROW AREA SUMMARY

AREA	MATERIAL TYPE
AREA 1	GRANULAR
AREA 2, 2a	GRANULAR
AREA 3	GRANULAR
AREA 4	GRANULAR
AREA 5	GRANULAR
AREA 6	GRANULAR
AREA 7	CLAY
AREA 8	GRANULAR
AREA 9	GLACIAL TILL

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO PROJECT - INTERIM FEASIBILITY REPORT

CONSTRUCTION MATERIALS INVESTIGATIONS

SUMMARY OF POTENTIAL BORROW AREAS

DATE MAR 1984	FIG. 5 - 14	R	
DWN DJ	DWG No. 704-C14-B446		

REPORT No. H1688

LEGEND:

WATER SURVEY OF CANADA - HYDROMETRIC STATIONS

- 08GD004 - HOMATHKO R. AT THE MOUTH
- 08GD007 - MOSLEY CREEK NEAR DUMBELL LAKE
- 08GD005 - HOMATHKO R. BELOW NUDE CREEK
- 08GD006 - HOMATHKO R. AT TRAGEDY CANYON
- 08GD008 - HOMATHKO R. AT INLET OF TATLAYOKO LAKE
- 08GD009 - HOMATHKO R. BELOW NOSTETUKO RIVER
- 08MA003 - TASEKO R. AT OUTLET OF TASEKO LAKE

CLIMATOLOGICAL STATIONS

- 23 B.C. HYDRO (@ HIGH, MID, & LOW ELEVATIONS) 23-PRECIPITATION, 23-AIR TEMPERATURE, 1-WIND VELOCITY AND WIND DIRECTION, 11-SNOW COURSE (5 POINT)
- 4 FEDERAL, PRECIPITATION, AIR TEMPERATURE
- 1 B.C. FOREST SERVICE, PRECIPITATION, AIR TEMPERATURE
- A 3 B.C. HYDRO AUTOMATIC WEATHER STATION, AIR TEMPERATURE, WIND VELOCITY AND WIND DIRECTION, RELATIVE HUMIDITY
- R 3 B.C. HYDRO SOLARIMETER, SOLAR RADIATION
- T 1 B.C. HYDRO TATLAYOKO CAMP OBSERVER, AIR TEMPERATURE, RELATIVE HUMIDITY, CLOUD COVER

SNOW COURSES (10 POINT)

- 3A17 TIEDEMANN GLACIER
- 3A22 NOSTETUKO RIVER
- 3A13 TATLAYOKO LAKE

PROJECT SITES

- TATLAYOKO STORAGE PROJECT
- NUDE CANYON PROJECT
- MOSLEY CREEK PROJECT
- WADDINGTON CANYON PROJECT

Scale: 5 0 5 10 15 20 km

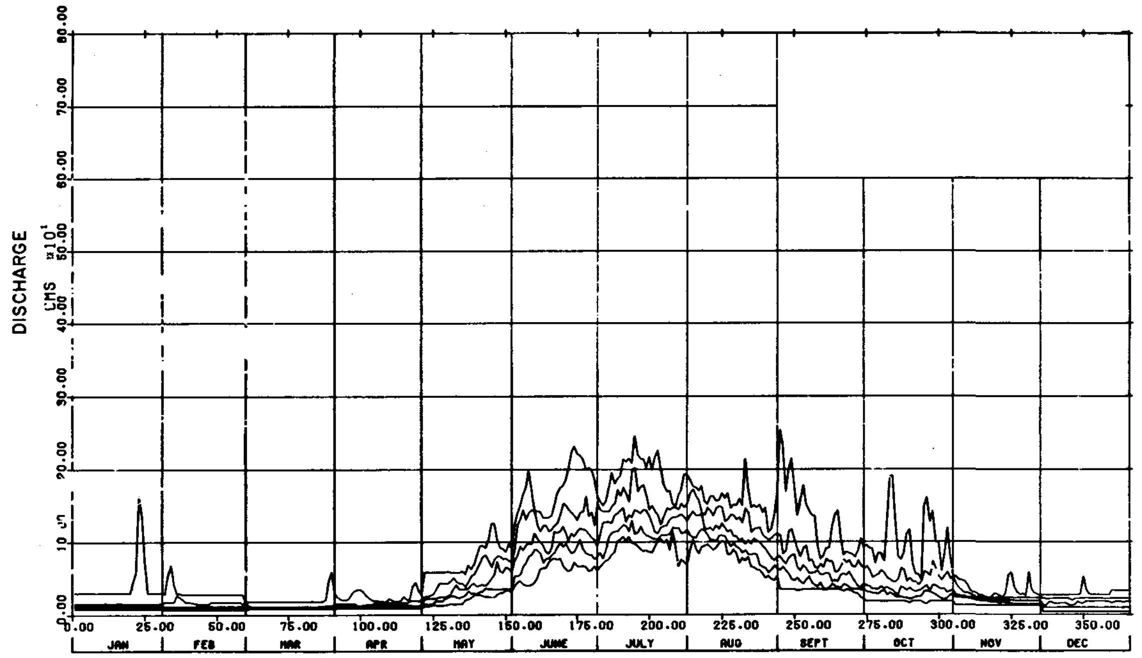
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

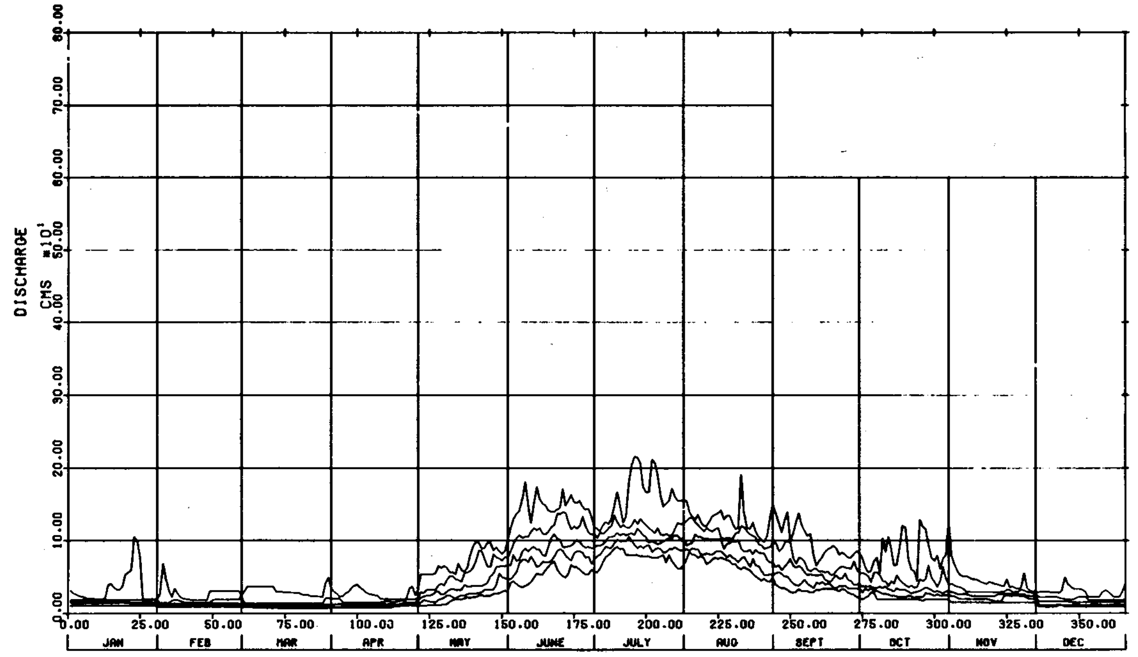
HOMATHKO RIVER DRAINAGE BASIN HYDROMETEOROLOGICAL NETWORK AREA LOCATION PLAN

MAR 1984	FIG. 6-1
DWN KL	DWG No. 704 - C14 - D303

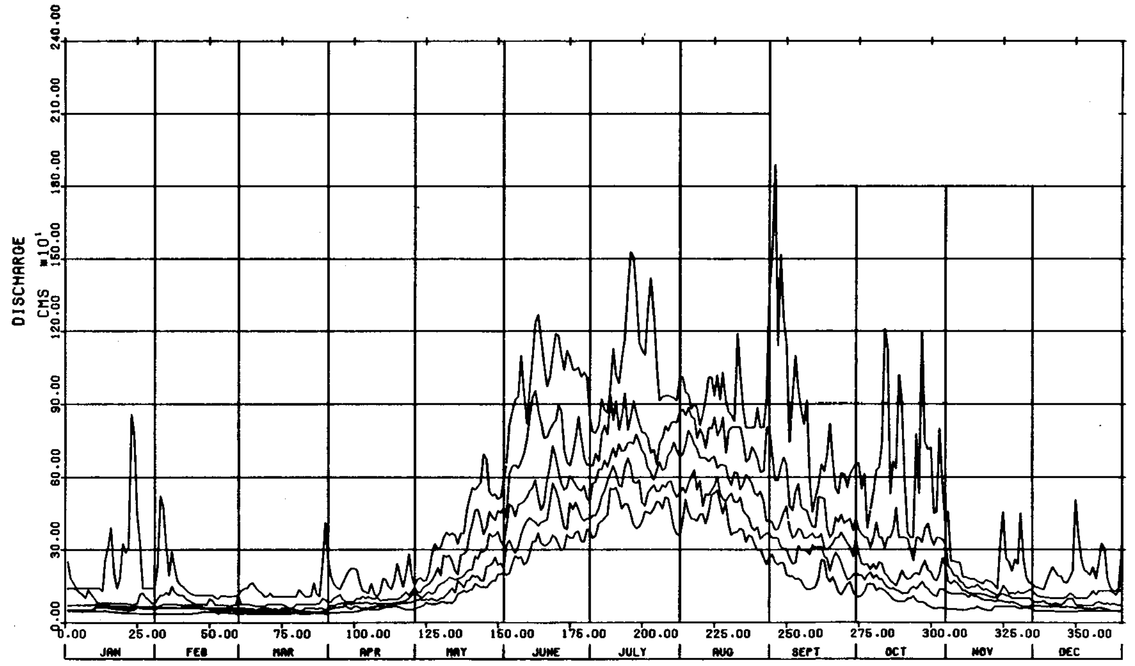
REPORT No. H1688



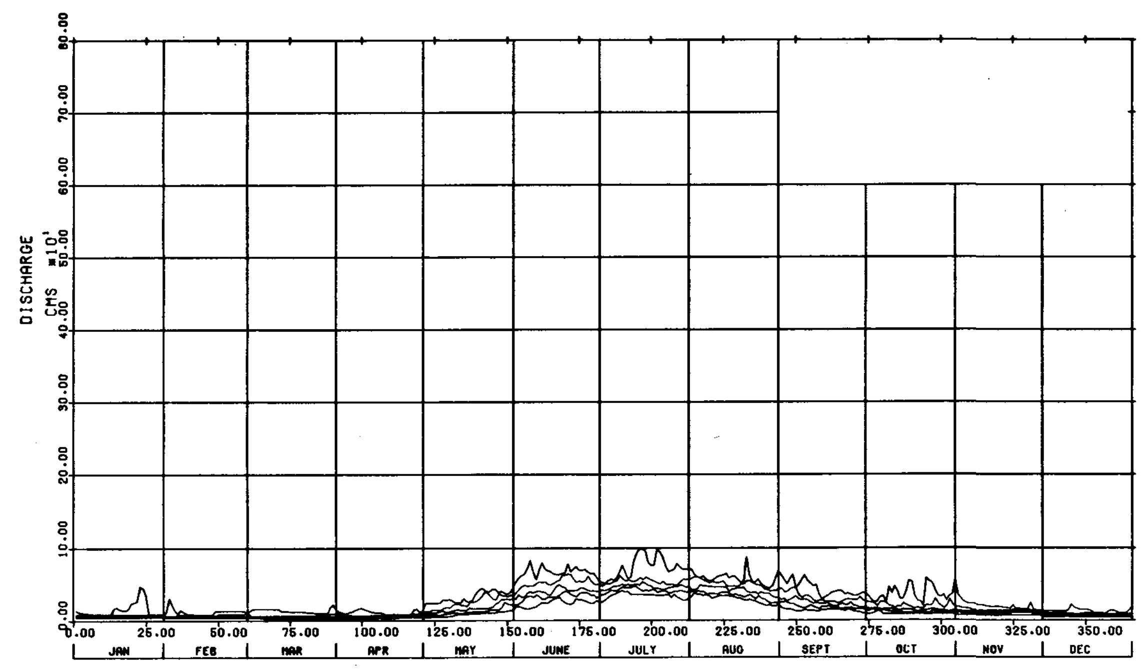
MOSLEY CREEK PROJECT



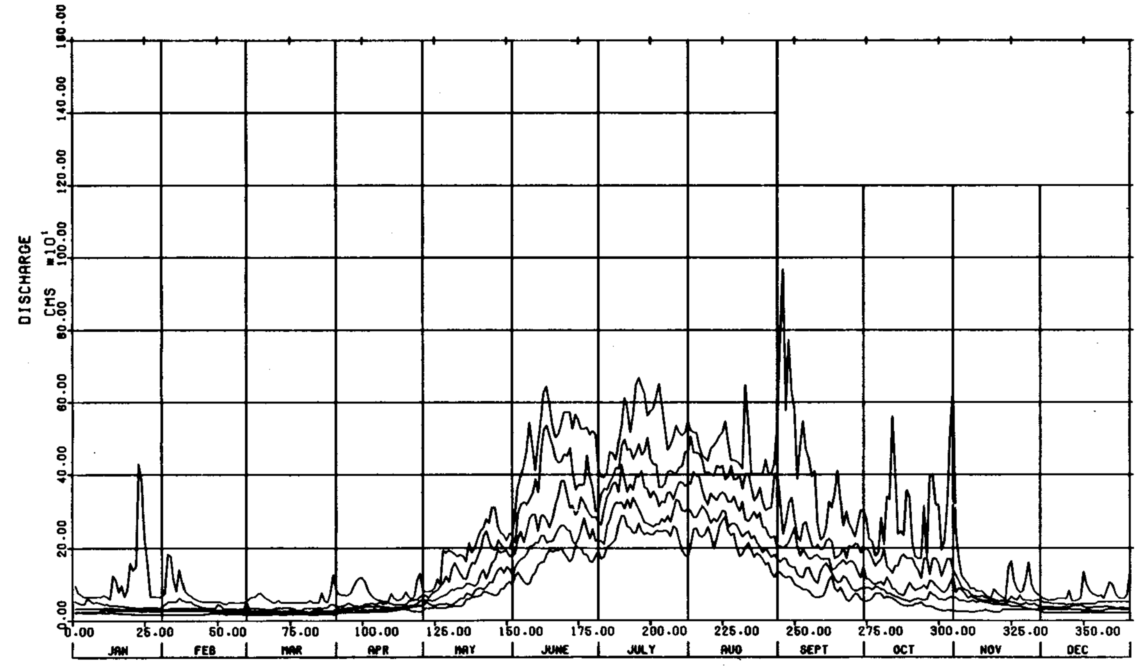
NUDE CANYON PROJECT



HOMATHKO RIVER AT THE MOUTH



TATLAYOKO STORAGE PROJECT (NOSTETUKO AXIS)



WADDINGTON CANYON PROJECT

NOTES:

1. PLOTS REPRESENT MEAN DAILY FLOWS.
2. HIGHEST PLOT LINE- PROBABILITY OF EXCEEDENCE IS 0% (MAXIMUM MEAN DAILY FLOWS).
3. SECOND HIGHEST PLOT LINE-- PROBABILITY OF EXCEEDENCE IS 25%.
4. THIRD HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 50%.
5. FOURTH HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 75%.
6. LOWEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 100% (MINIMUM MEAN DAILY FLOWS).

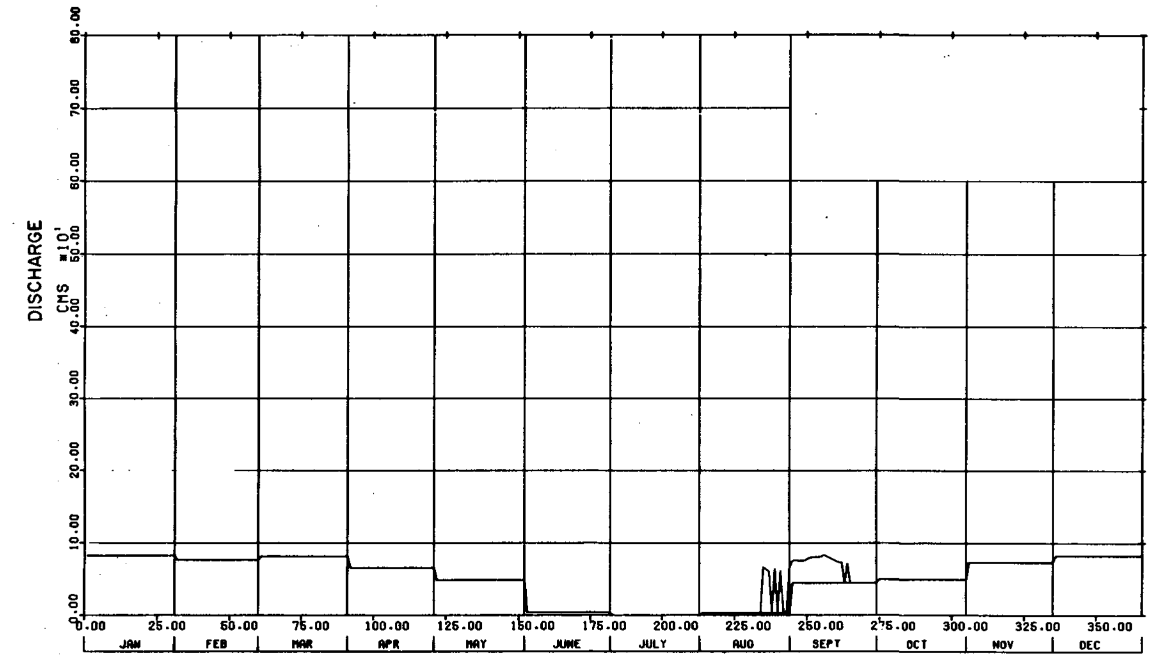
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT- INTERIM FEASIBILITY REPORT

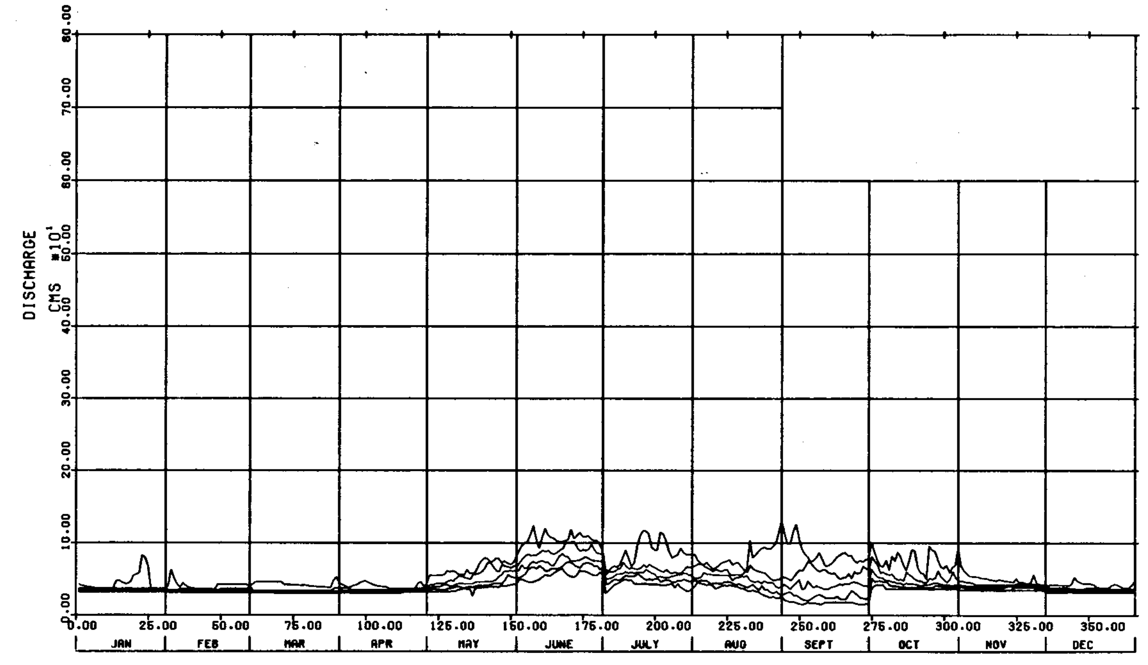
NATURAL FLOW REGIME PROBABILITY EXCEEDENCE CURVES (1960 - 1970)

DATE MAR 1984	FIG. 6-2
DWN SJF	DWG No 704-C14-C447

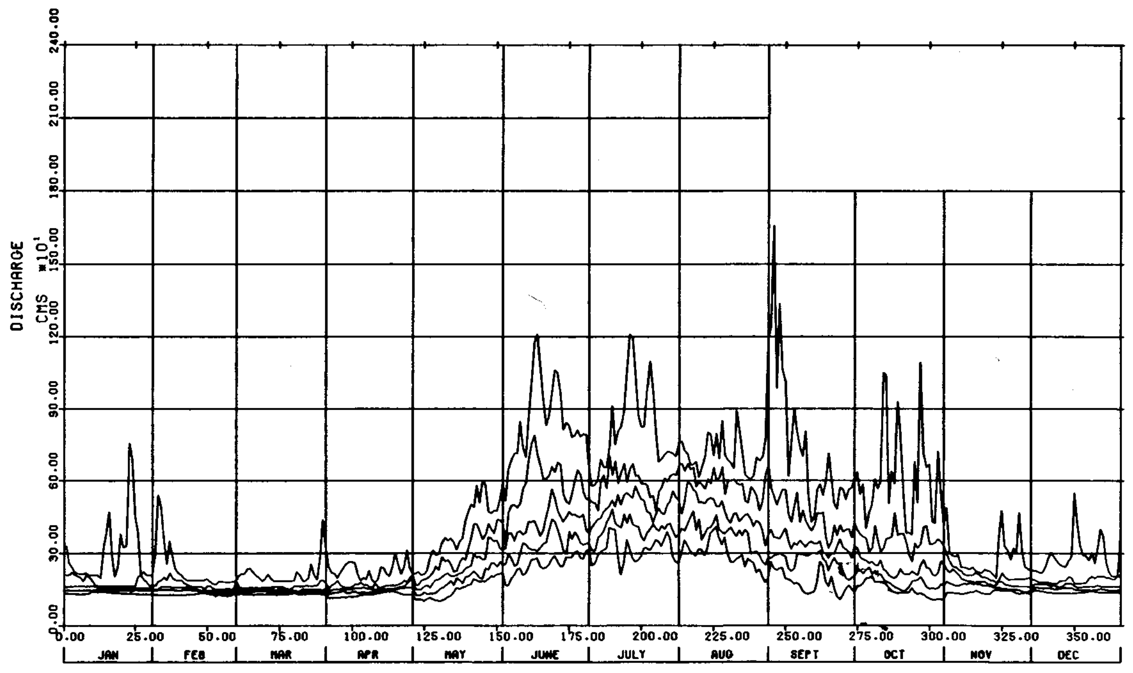
REPORT No H1688



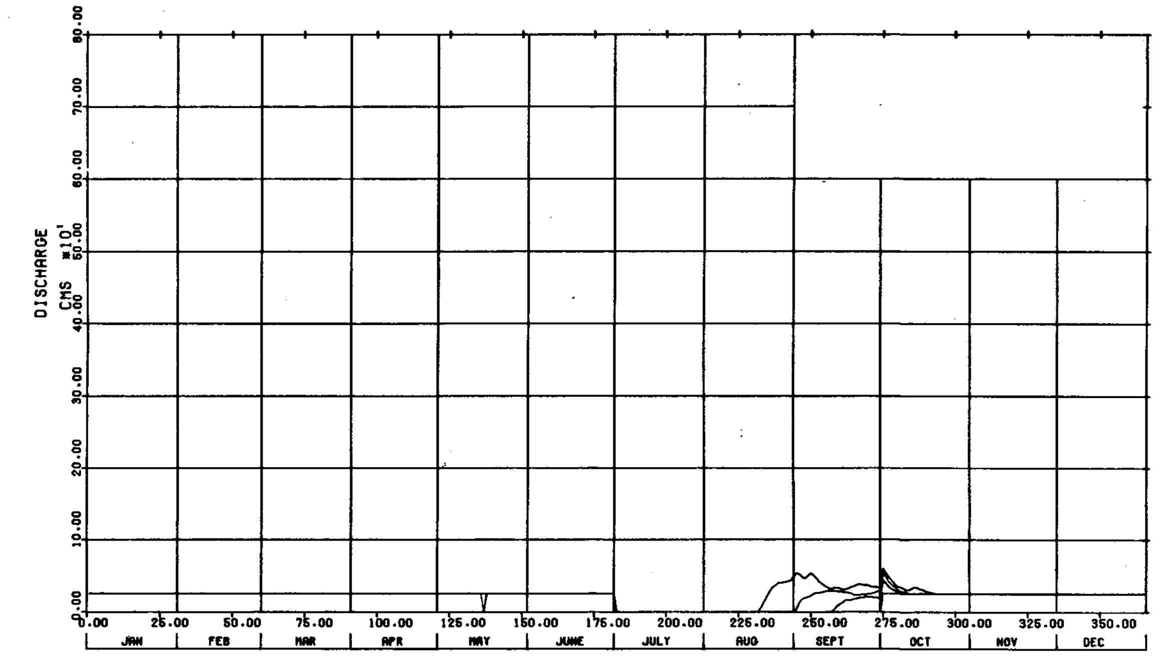
MOSLEY CREEK PROJECT



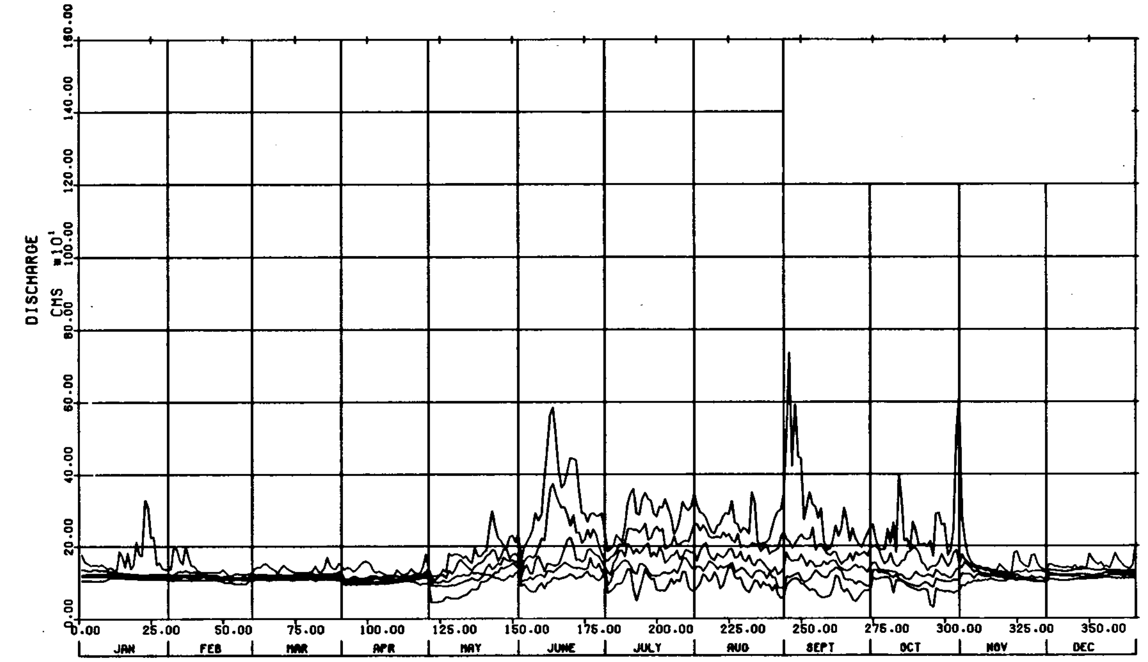
NUDE CANYON PROJECT



HOMATHKO RIVER AT THE MOUTH



TATLAYOKO STORAGE PROJECT (NOSTETUKO AXIS)



WADDINGTON CANYON PROJECT

NOTES:

1. PLOTS REPRESENT MEAN DAILY FLOWS.
2. HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 0% (MAXIMUM MEAN DAILY FLOWS).
3. SECOND HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 25%.
4. THIRD HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 50%.
5. FOURTH HIGHEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 75%.
6. LOWEST PLOT LINE -- PROBABILITY OF EXCEEDENCE IS 100%. (MINIMUM MEAN DAILY FLOWS).

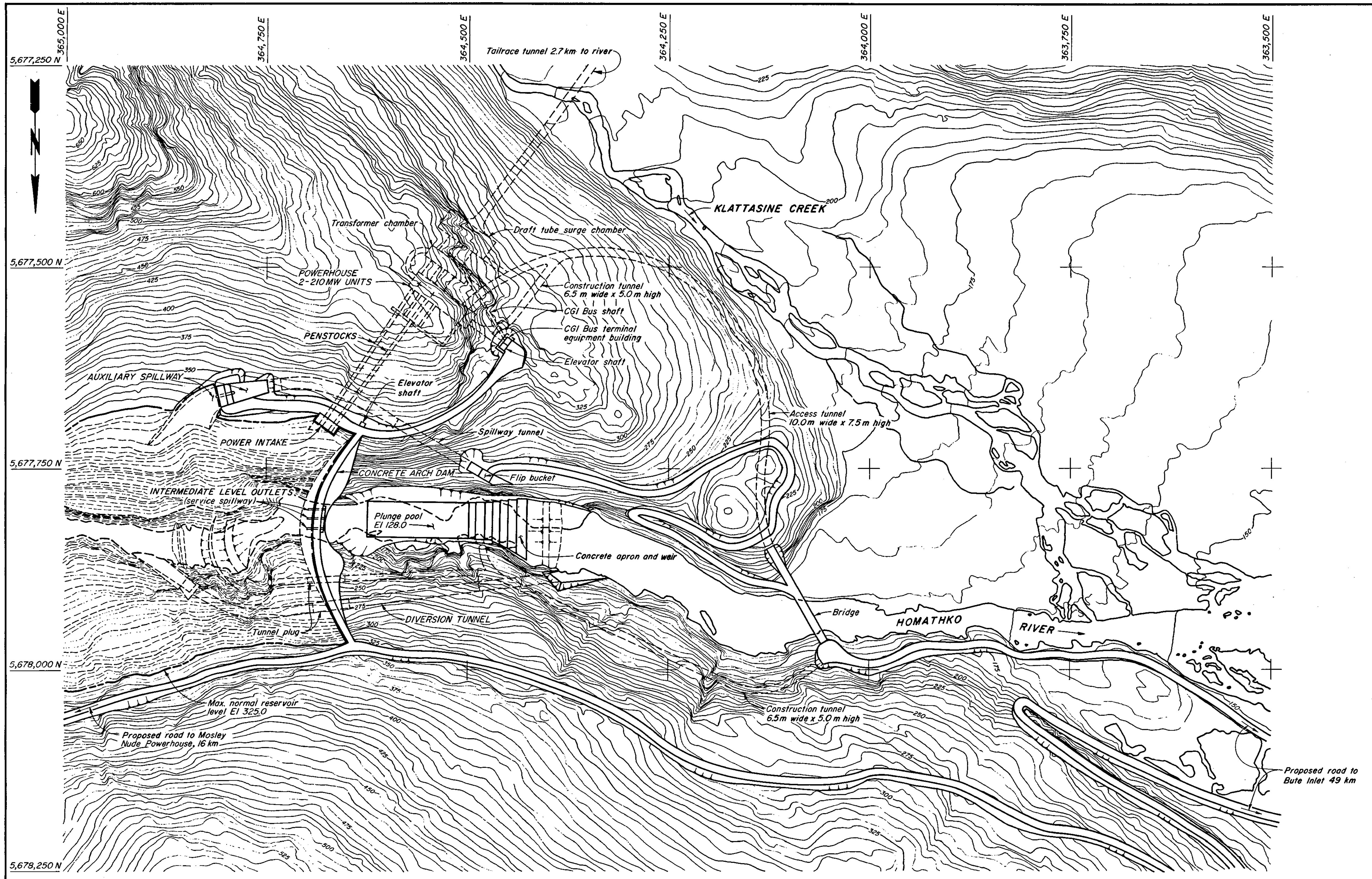
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

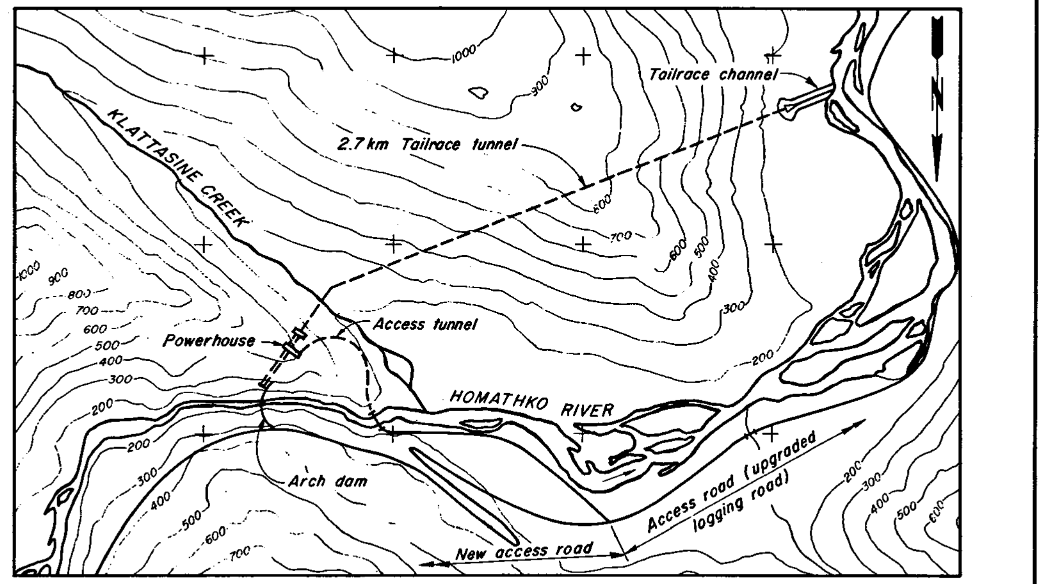
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

REGULATED FLOW REGIME PROBABILITY EXCEEDENCE CURVES (1960 - 1970)

DATE MAR 1984	FIG. 6-3
DWN SJF	DWG No 704-C14-C448

REPORT No H1688



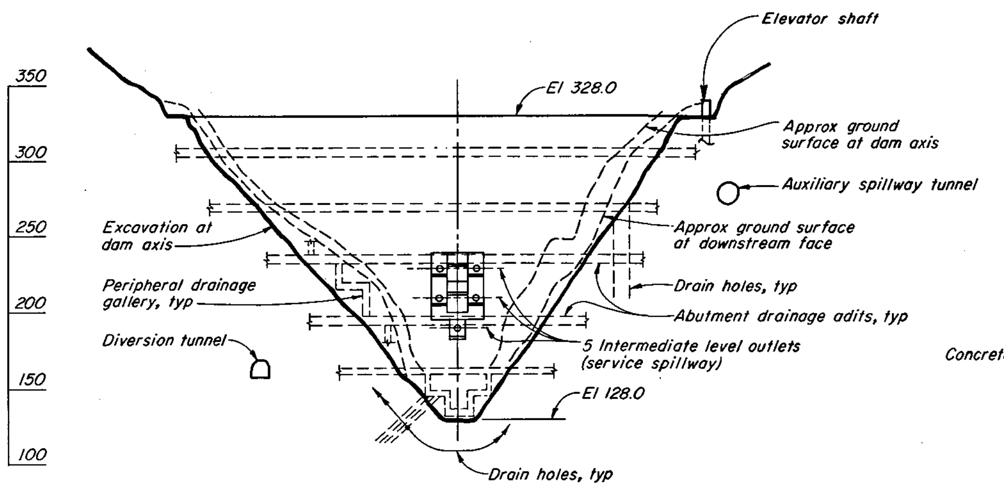


LOCATION PLAN

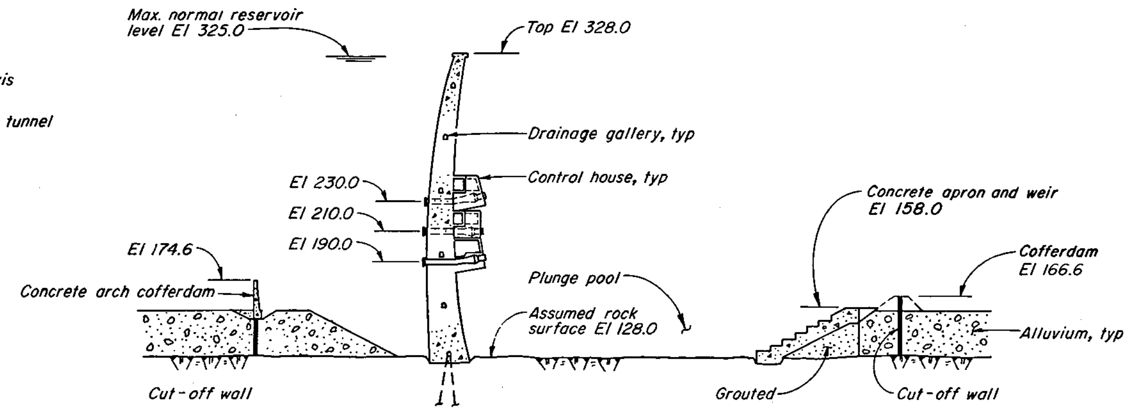
Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT	
WADDINGTON CANYON PROJECT GENERAL ARRANGEMENT	
DATE MAR 1984	FIG. 7-1
DWN FM	DWG No. 704-C14-U449

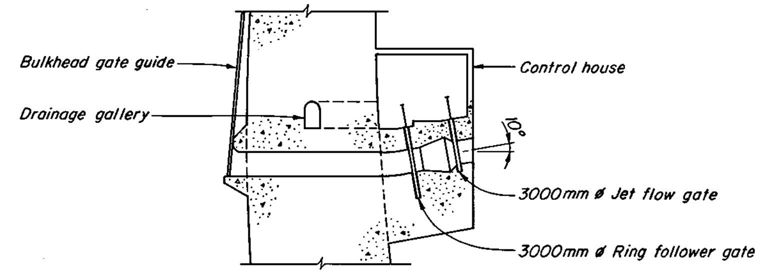
REPORT No H1688



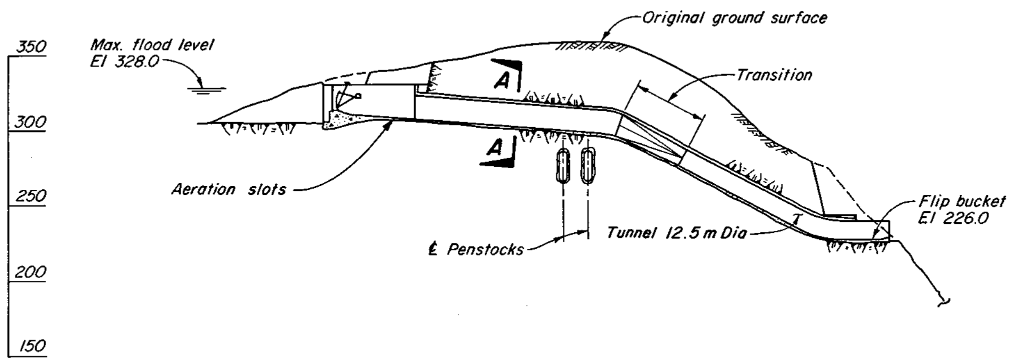
DEVELOPED DOWNSTREAM ELEVATION
Scale A



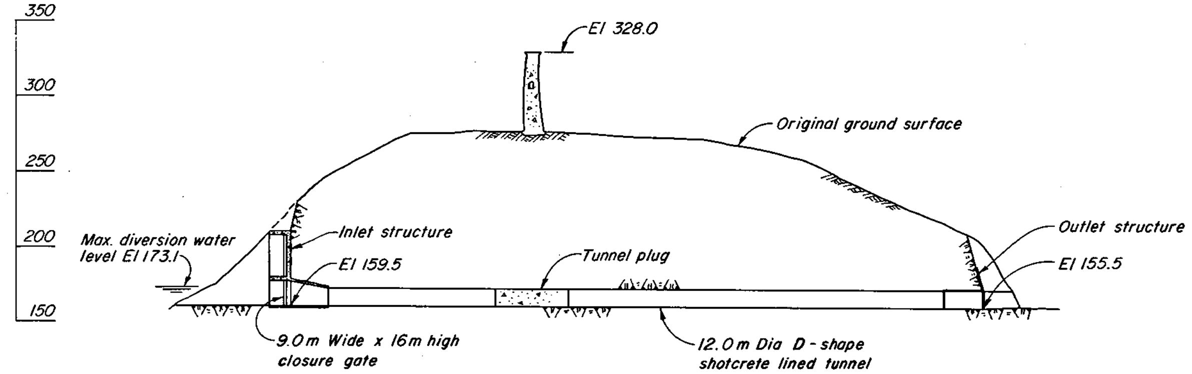
SECTION THROUGH CONCRETE ARCH COFFERDAM, DAM AND PLUNGE POOL
Scale A



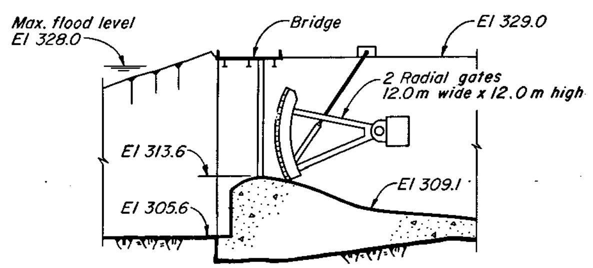
SECTION THROUGH OUTLET VALVES
Scale B



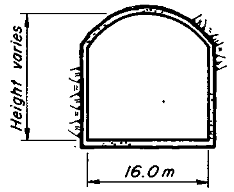
SECTION THROUGH AUXILIARY SPILLWAY CHUTE AND TUNNEL
Scale A



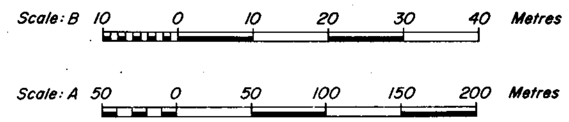
SECTION THROUGH DIVERSION TUNNEL
Scale A



SECTION THROUGH AUXILIARY SPILLWAY
Scale B



SECTION A-A
Scale B



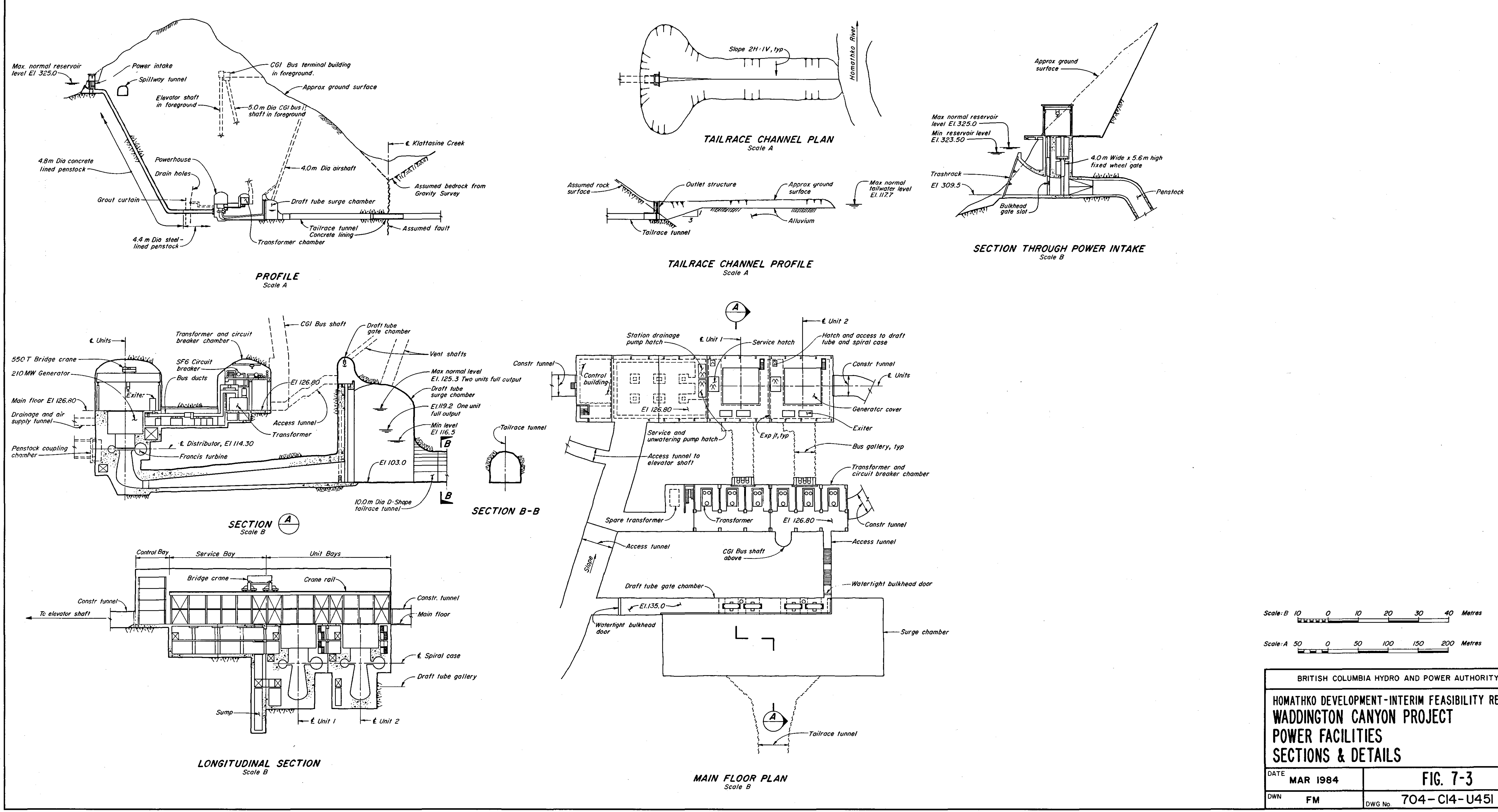
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

WADDINGTON CANYON PROJECT
DAM ELEVATION AND SECTIONS

DATE MAR 1984	FIG. 7-2
DWN FM	DWG No. 704-C14-D450

REPORT No. H1688



BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

WADDINGTON CANYON PROJECT
POWER FACILITIES
SECTIONS & DETAILS

DATE MAR 1984	FIG. 7-3
DWN FM	DWG No. 704-C14-U451

REPORT No H1688

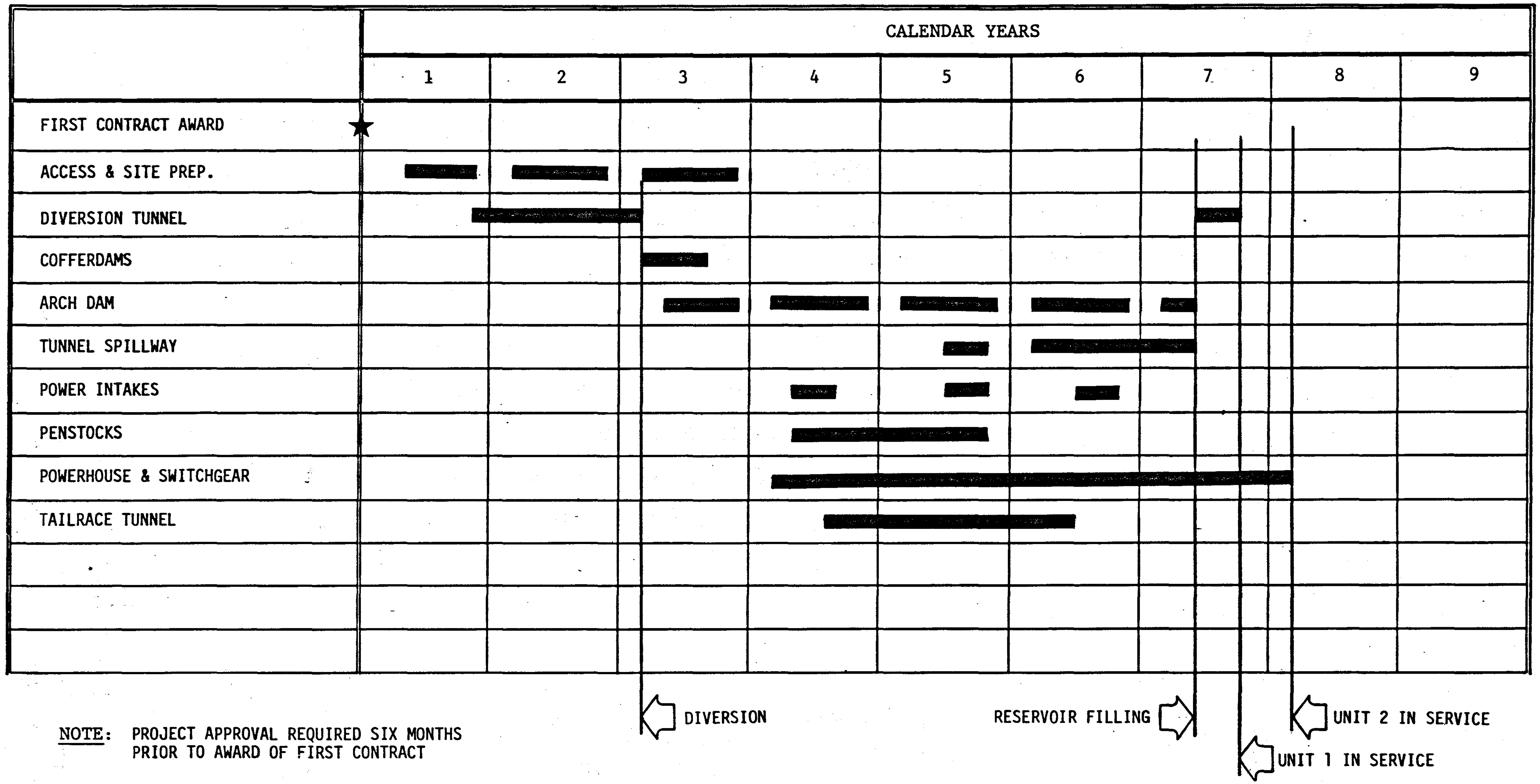
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

WADDINGTON CANYON PROJECT
ALTERNATIVE PROJECT ARRANGEMENT

DATE MAR 1984	FIG. 7-4	
DWN FM	DWG. No	704-C14-D452

REPORT NO. H1688



NOTE: PROJECT APPROVAL REQUIRED SIX MONTHS PRIOR TO AWARD OF FIRST CONTRACT

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

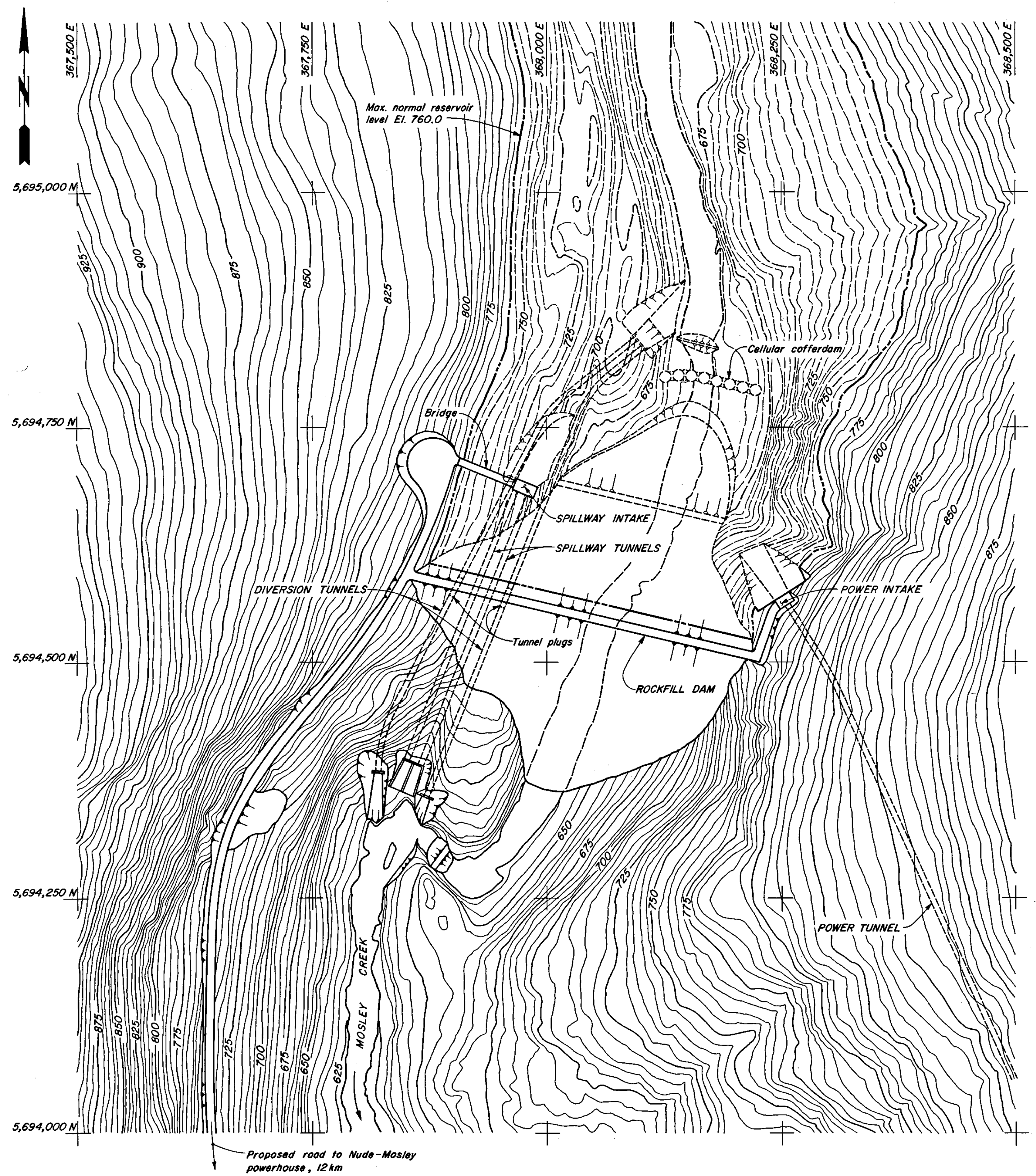
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

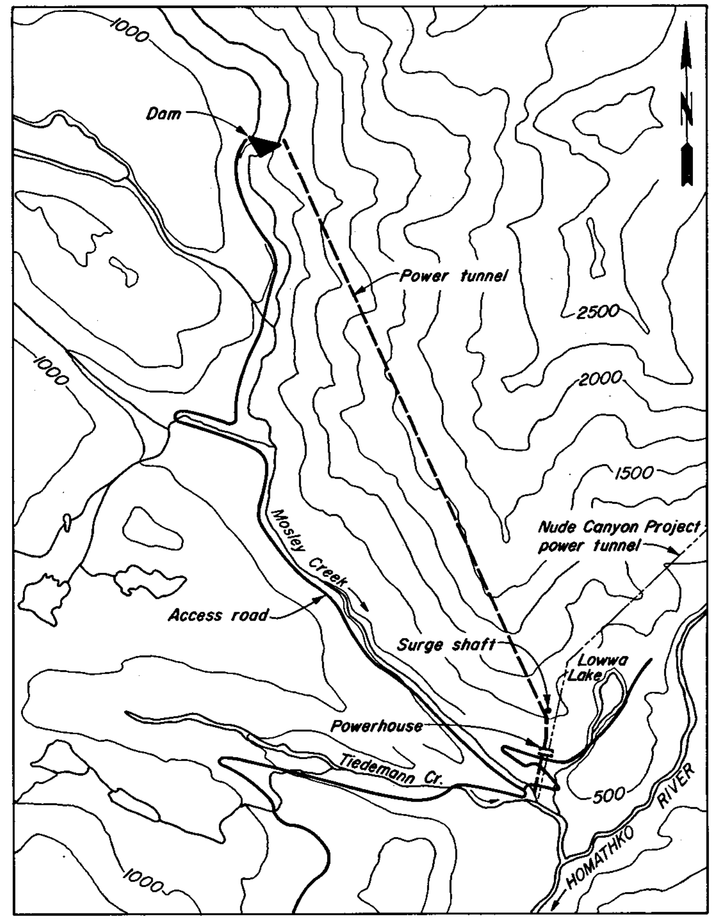
WADDINGTON CANYON PROJECT

CONSTRUCTION SCHEDULE

DATE MAR 1984	FIG. 7-5
DWN A.SZ.	DWG. No. 704-C14-B453

REPORT No H1688





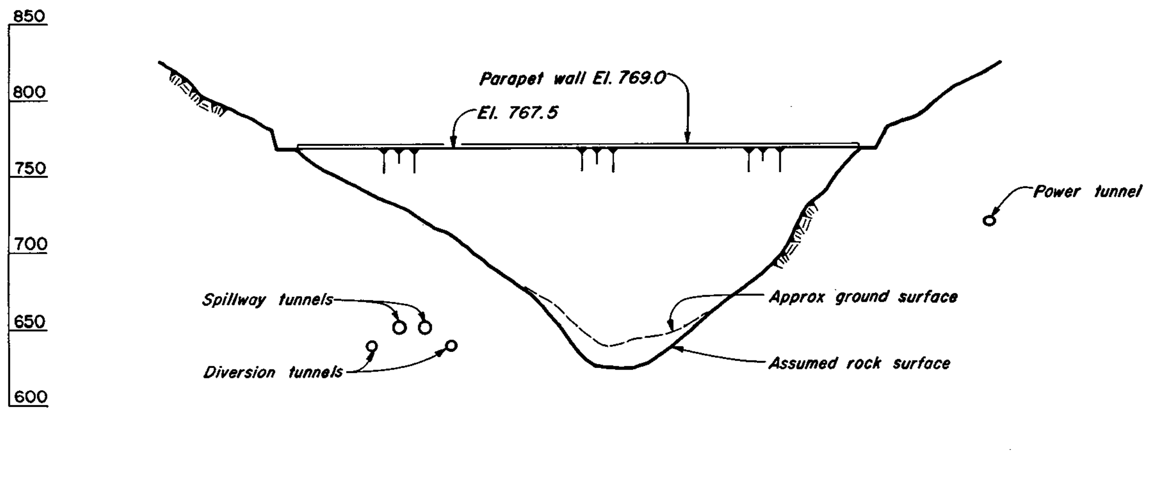
LOCATION PLAN
Scale A

Scale A 0 1000 2000 3000 4000 5000 Metres

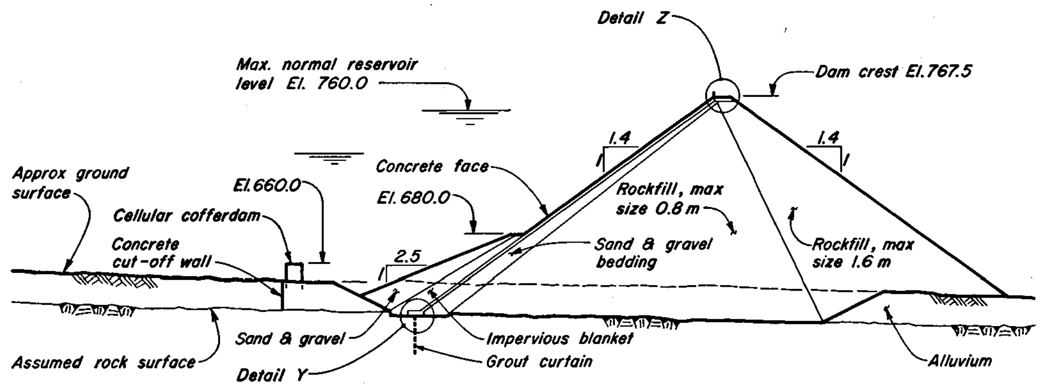
Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT	
MOSLEY CREEK PROJECT GENERAL ARRANGEMENT	
DATE MAR 1984	FIG. 8-1
DWN GB	DWG No. 704-C14-D454

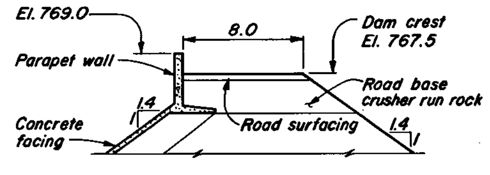
REPORT No. H1688



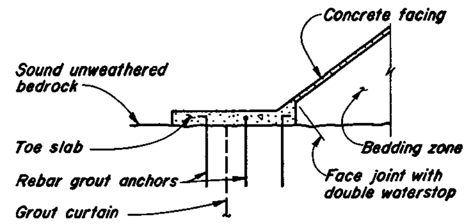
DOWNSTREAM ELEVATION
Scale A



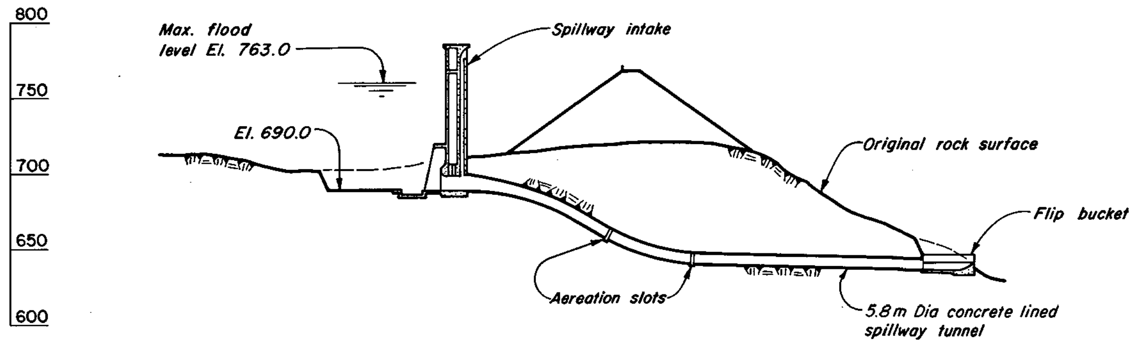
SECTION THROUGH ROCKFILL DAM
Scale A



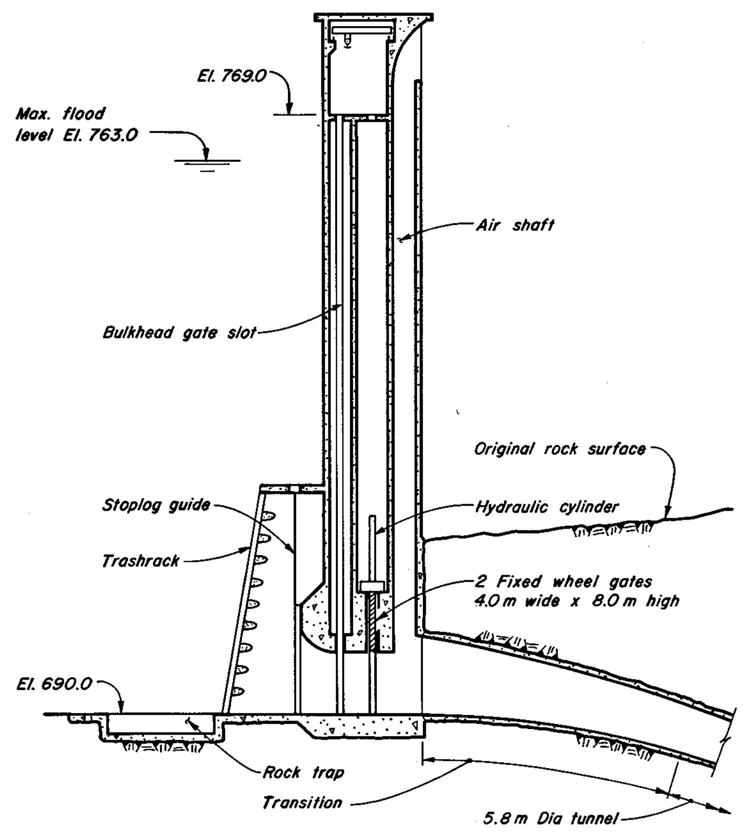
DETAIL Z
NTS



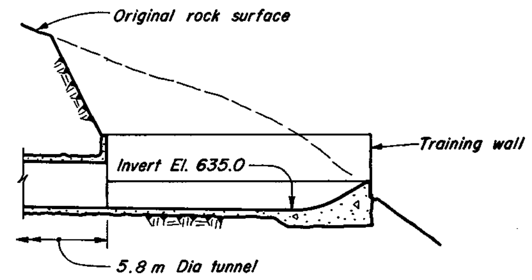
DETAIL Y
NTS



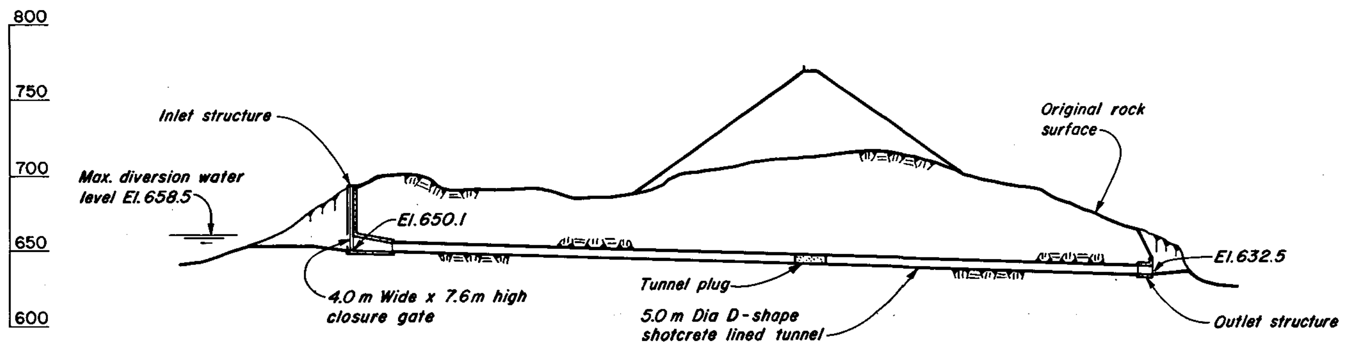
SECTION THROUGH SPILLWAY
Scale A



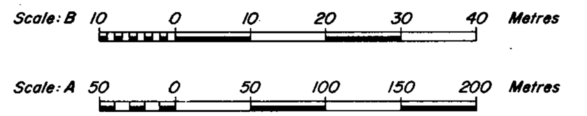
SECTION THROUGH SPILLWAY INTAKE
Scale B



SECTION THROUGH FLIP BUCKET
Scale B



SECTION THROUGH DIVERSION TUNNEL
Scale A



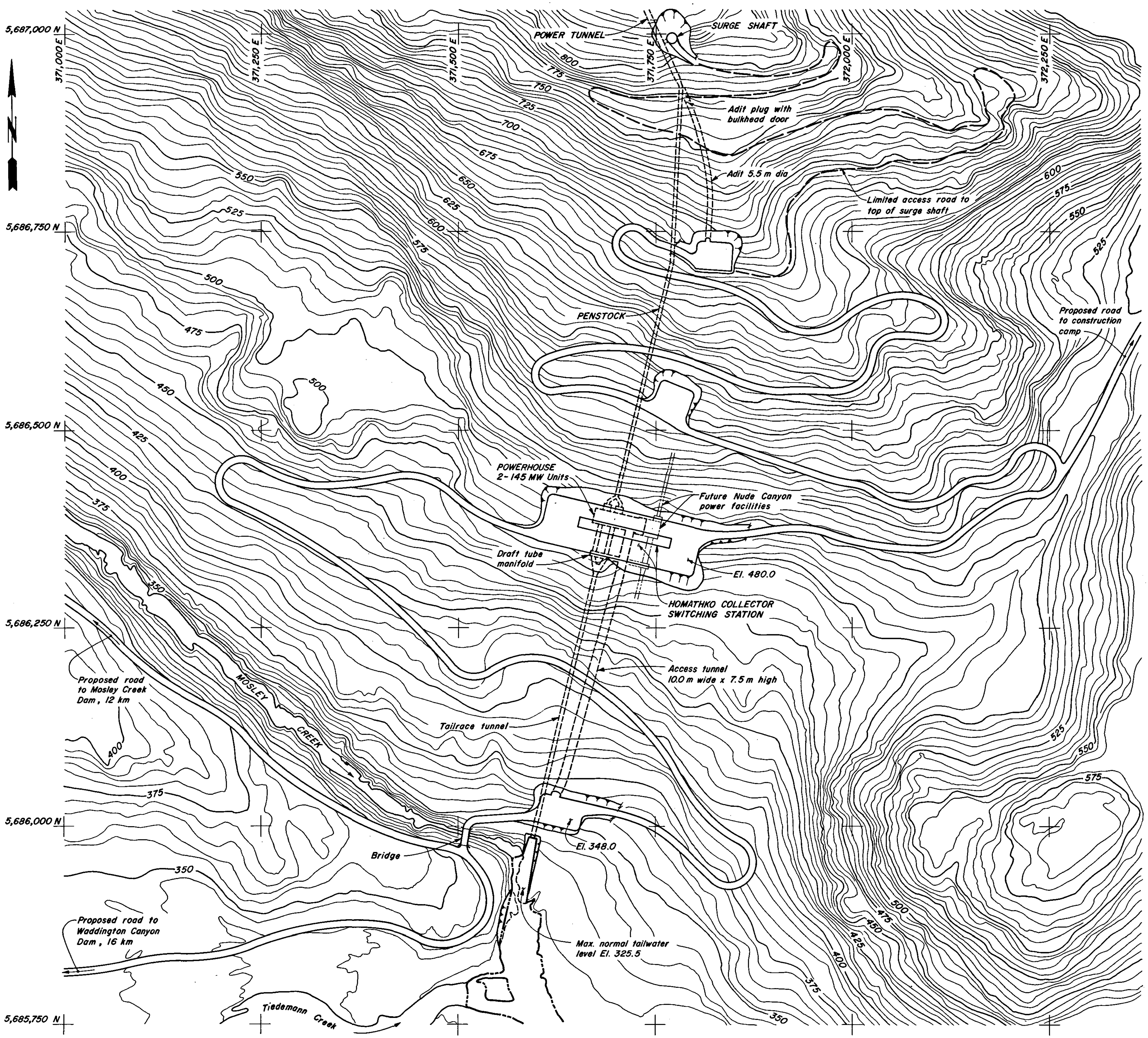
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

MOSLEY CREEK PROJECT
DAM ELEVATION, SECTIONS AND DETAILS

DATE MAR 1984	FIG. 8-2
DWN GB	DWG No. 704 - C14 - D455

REPORT No. H1688



Scale: 50 0 50 100 150 200 Metres

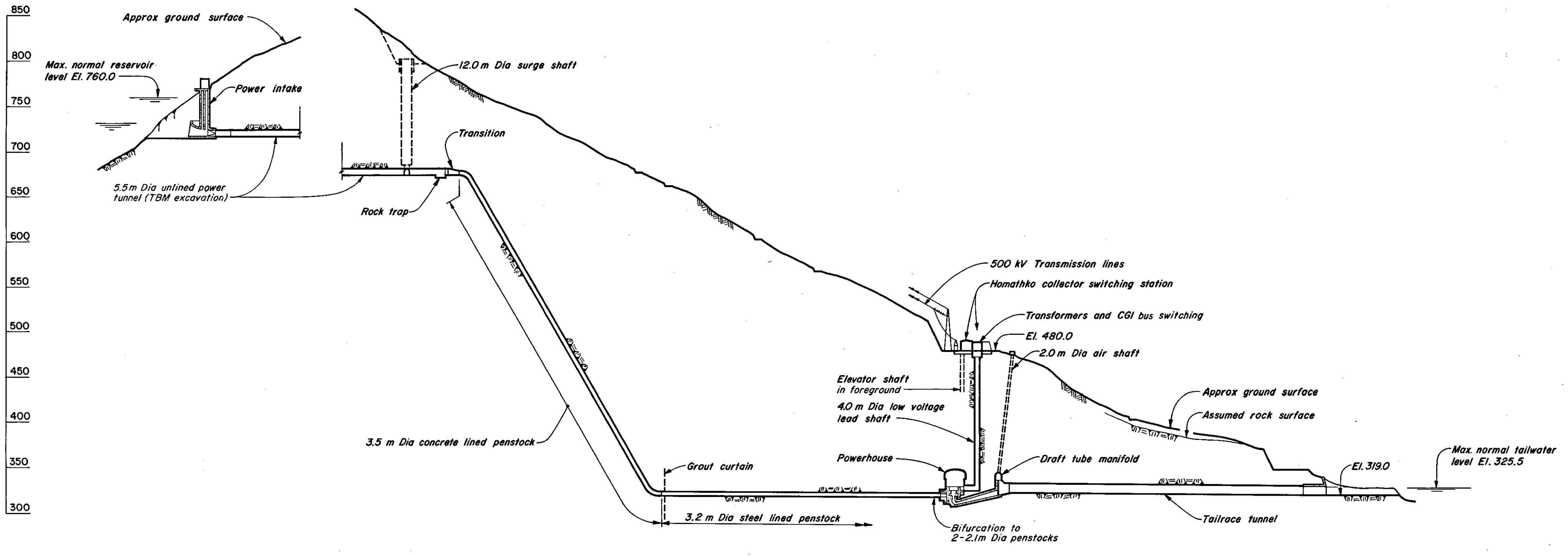
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

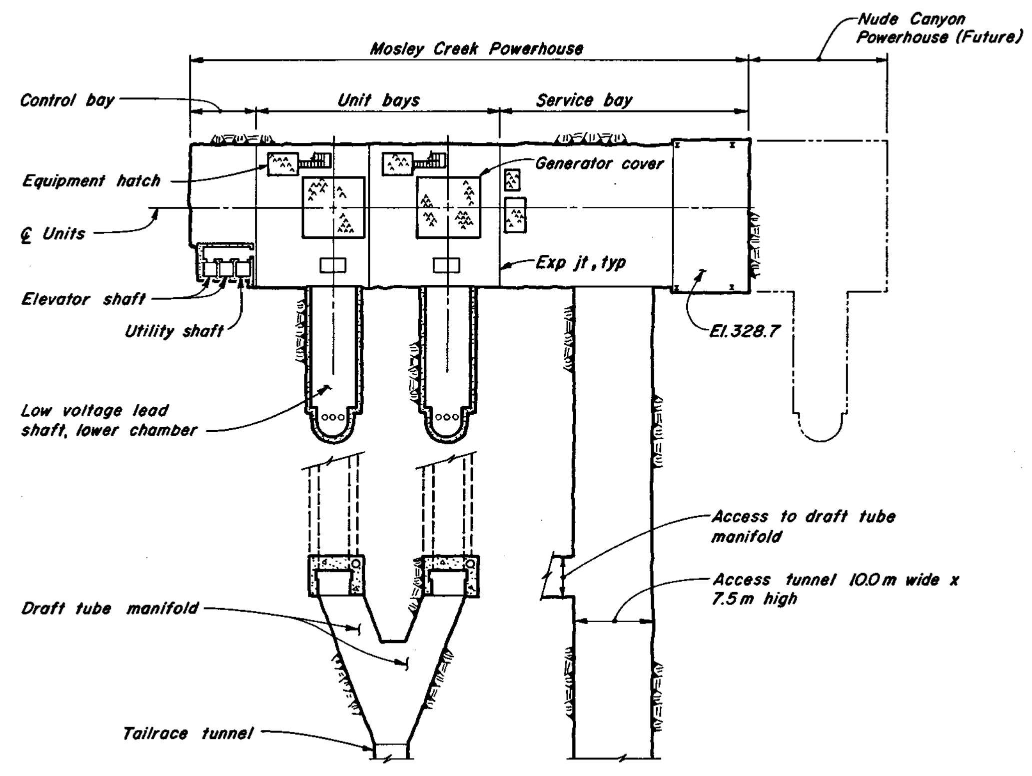
MOSLEY CREEK PROJECT
POWER FACILITIES
GENERAL ARRANGEMENT

DATE MAR 1984	FIG. 8-3
DWN GB	DWG No. 704-C14-D456

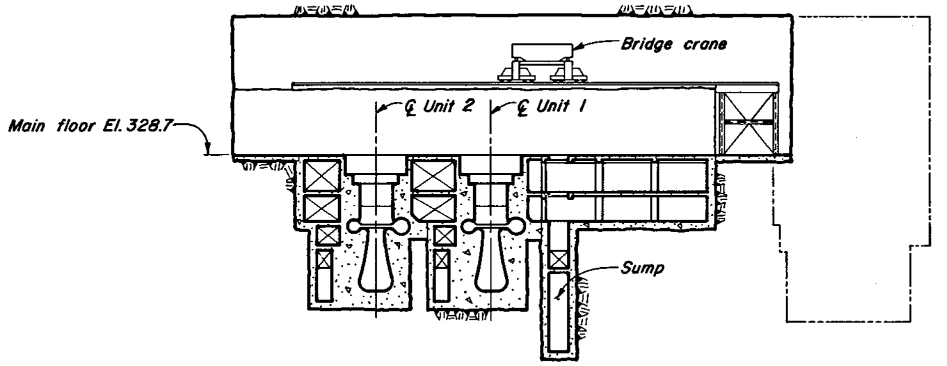
REPORT No. H1688



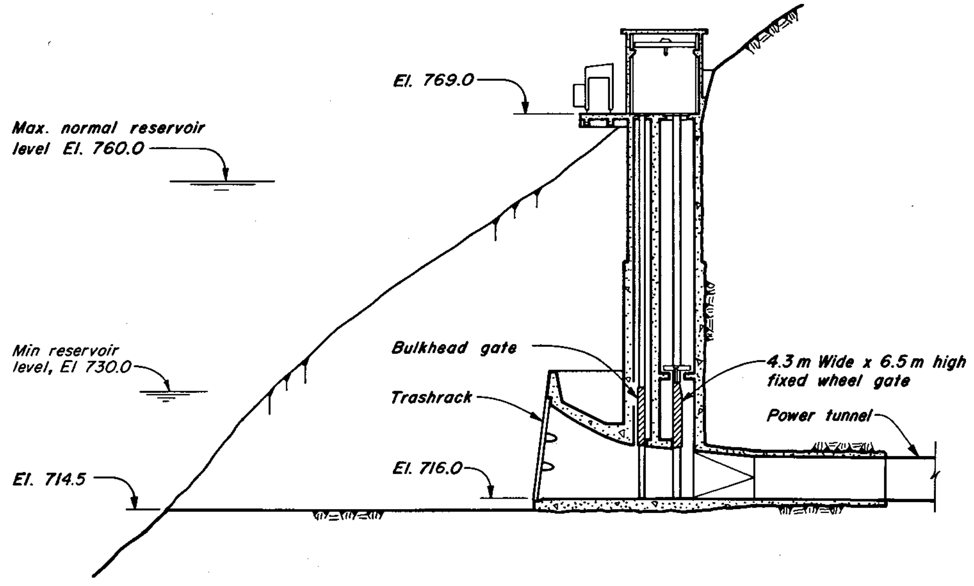
PROFILE
Scale A



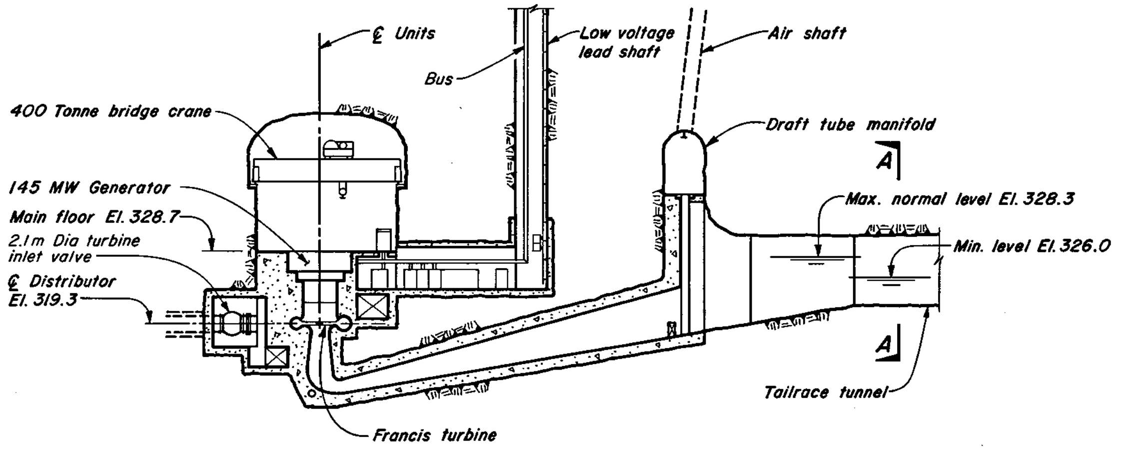
MAIN FLOOR PLAN
Scale B



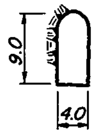
LONGITUDINAL SECTION
Scale B



SECTION THROUGH POWER INTAKE
Scale B



TRANSVERSE SECTION
Scale B



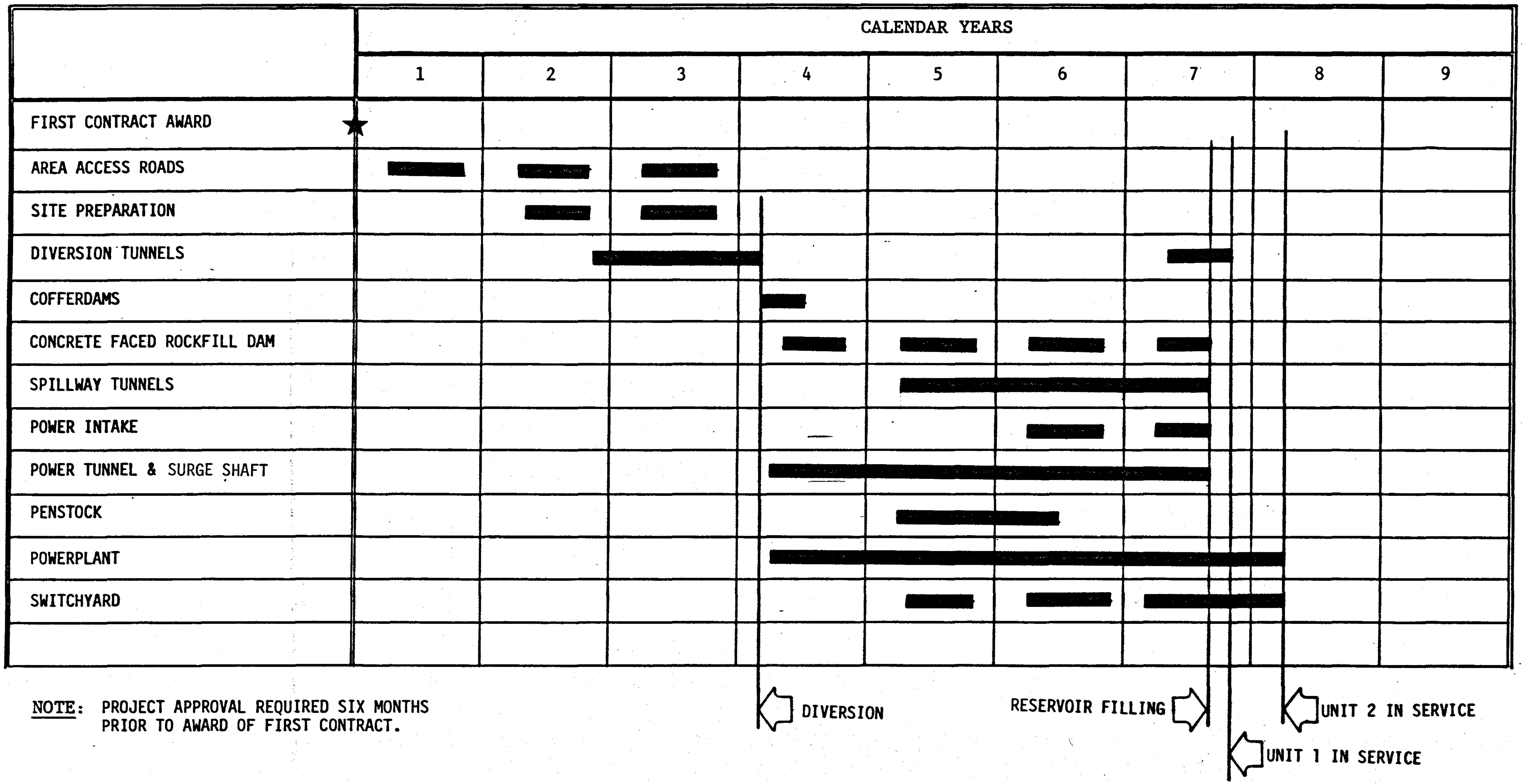
SECTION A-A
Scale B

Scale: B 10 0 10 20 30 40 Metres

Scale: A 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT MOSLEY CREEK PROJECT POWER FACILITIES PLAN AND SECTIONS	
DATE MAR 1984	FIG. 8-4
DWN GB	DWG No. 704-C14-U457

REPORT No H1688



<u>NOTE</u>: PROJECT APPROVAL REQUIRED SIX MONTHS PRIOR TO AWARD OF FIRST CONTRACT.

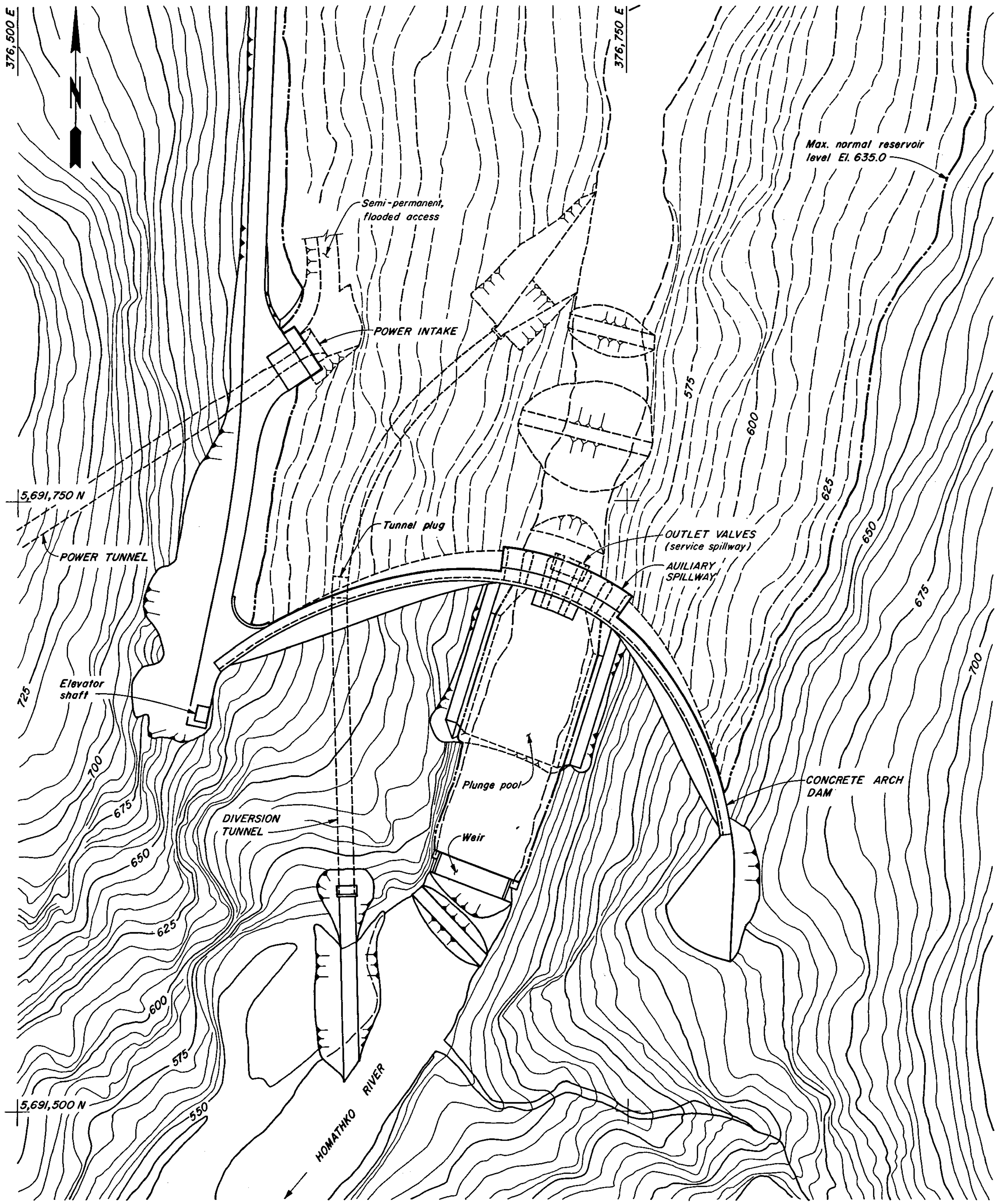
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

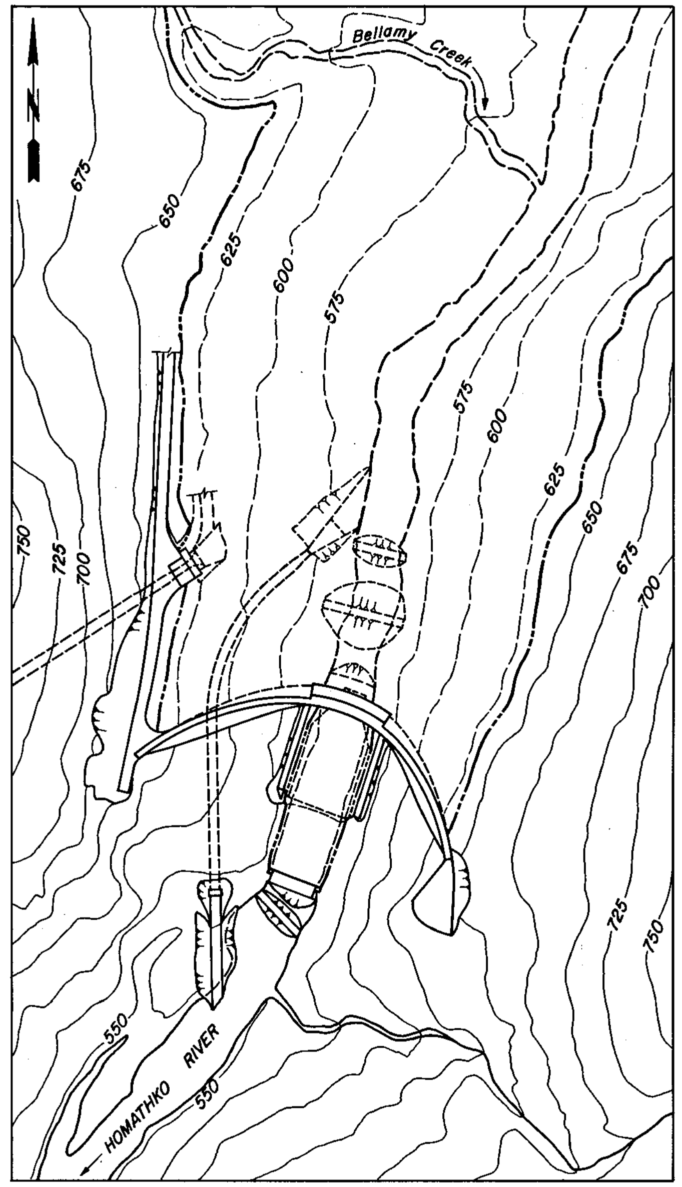
MOSLEY CREEK PROJECT
CONSTRUCTION SCHEDULE

DATE MAR 1984	FIG. 8-5
DWN A. SZ.	DWG. No. 704-C14-B458

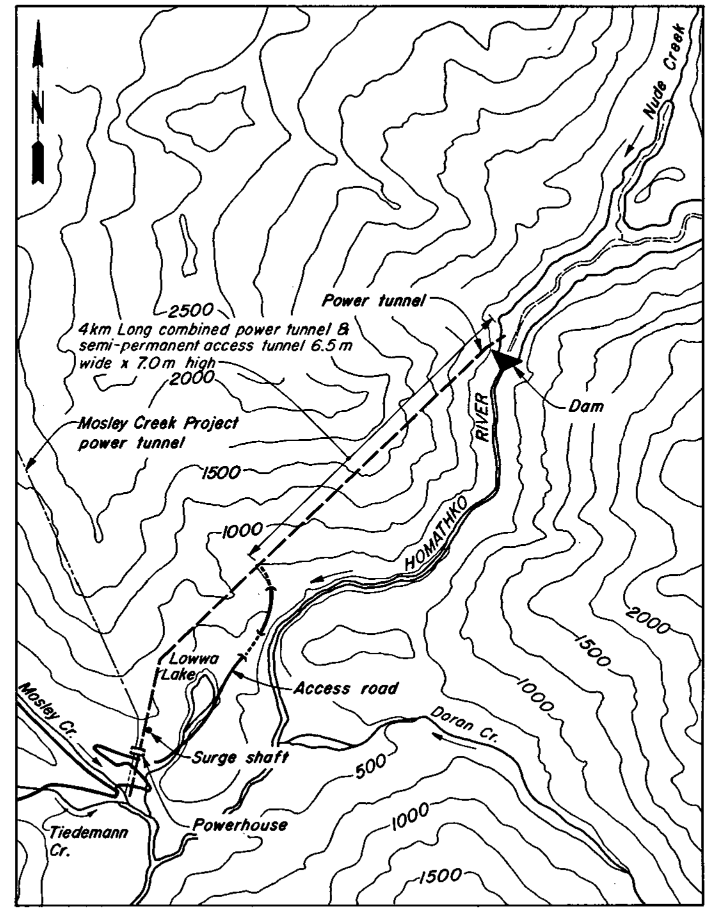
REPORT No H1688



DAMSITE ARRANGEMENT
Scale A



PROJECT SITE PLAN
Scale B



LOCATION PLAN
Scale C

Note:
Project Site Plan is drawn to the same scale as the other Project Site Plans for comparison purposes.

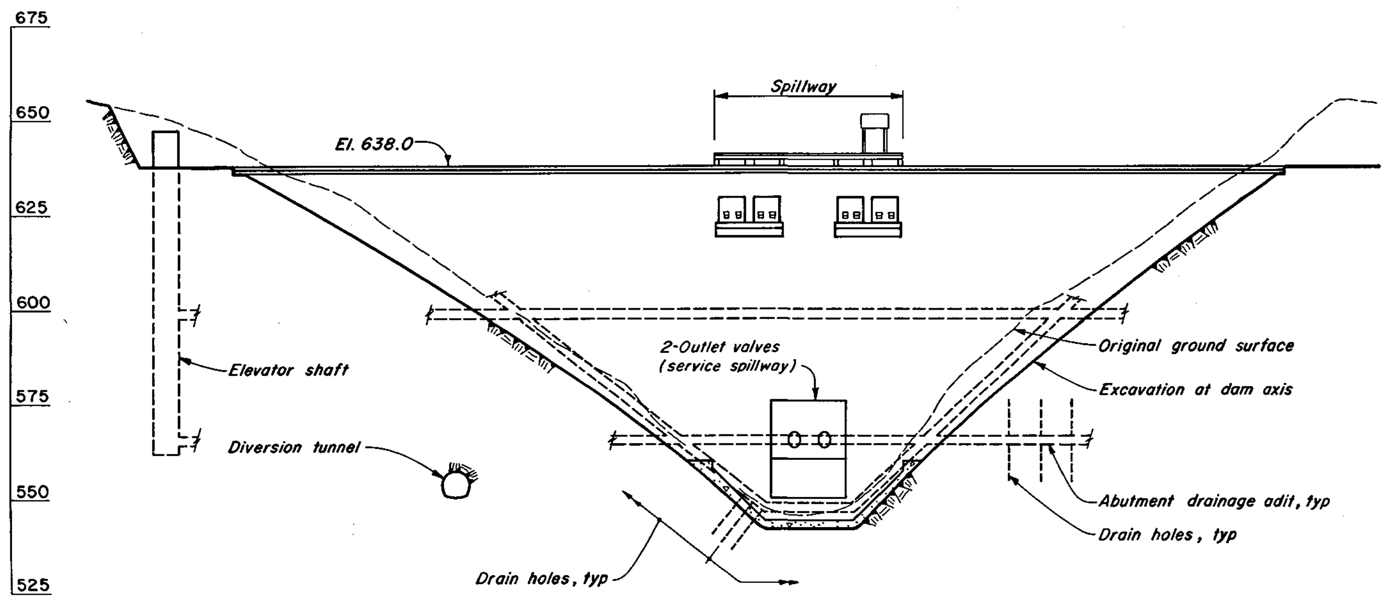
Scale: C 0 1000 2000 3000 4000 5000 Metres

Scale: B 50 0 50 100 150 200 Metres

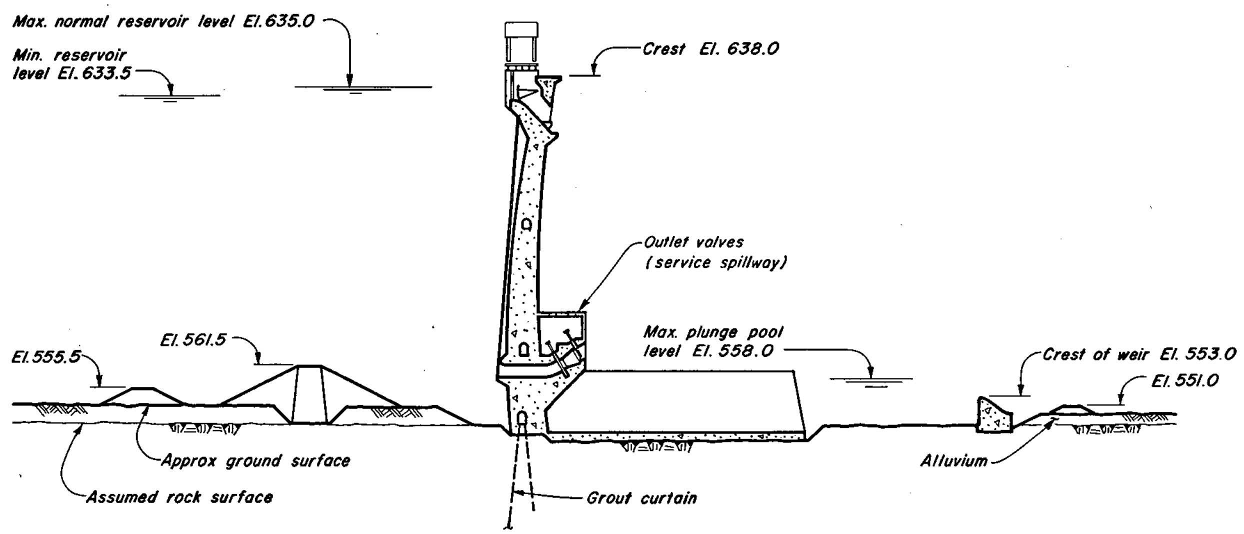
Scale: A 20 0 20 40 60 80 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT	
NUDE CANYON PROJECT GENERAL ARRANGEMENT	
DATE MAR 1984	FIG. 9-1
DWN GB	DWG No. 704-C14-D459

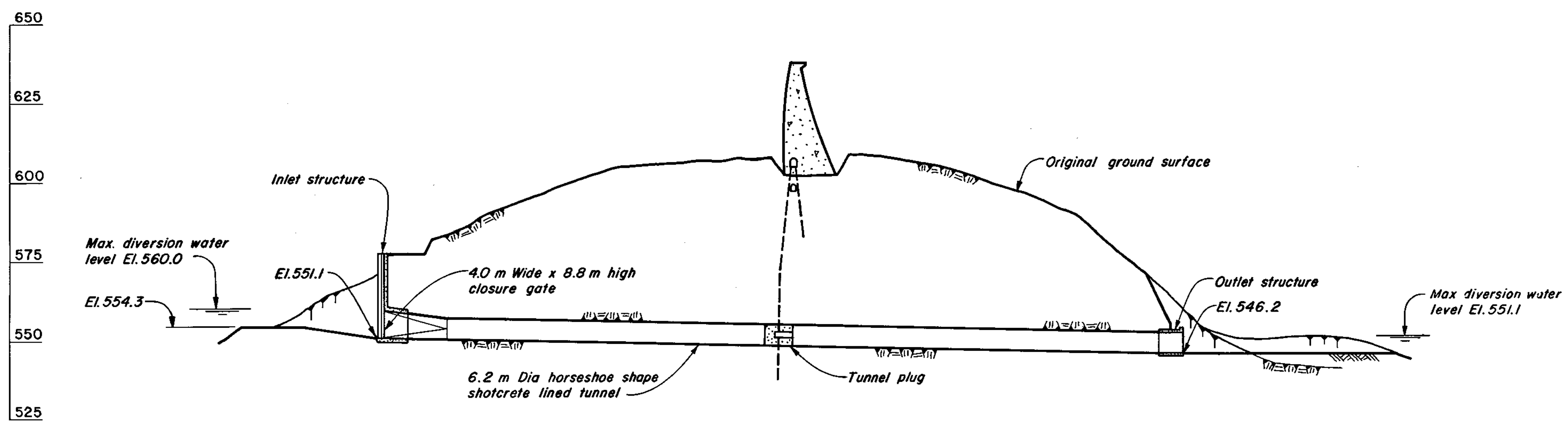
REPORT No. H1688



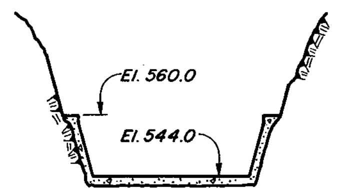
DEVELOPED DOWNSTREAM ELEVATION
Scale A



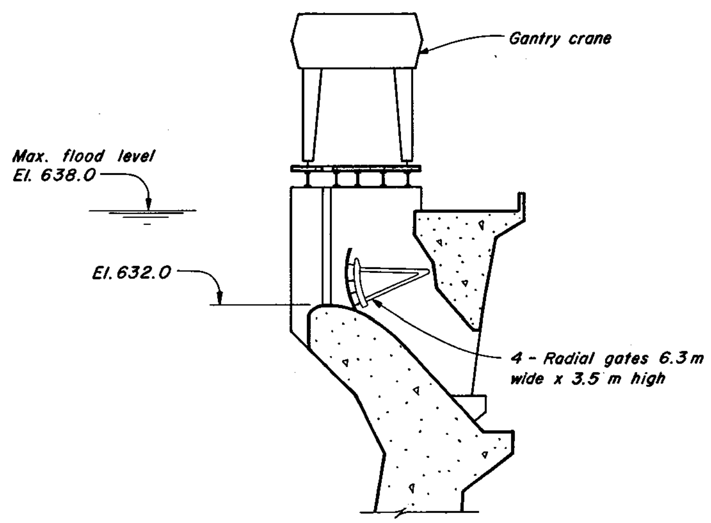
SECTION THROUGH CONCRETE ARCH DAM
Scale A



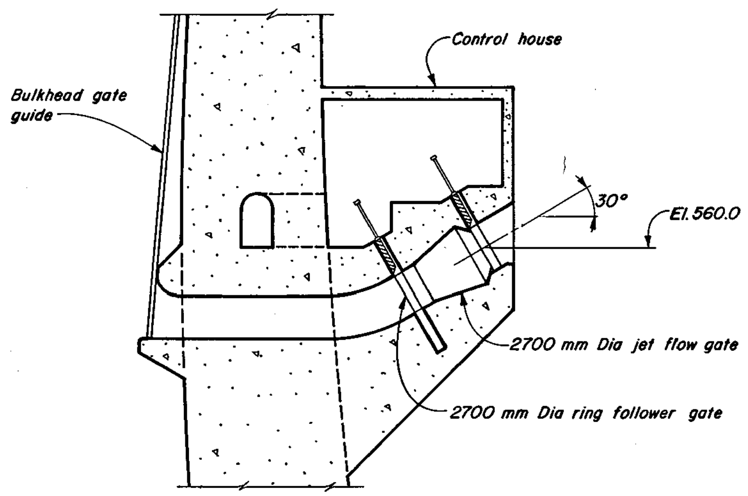
SECTION THROUGH DIVERSION TUNNEL
Scale A



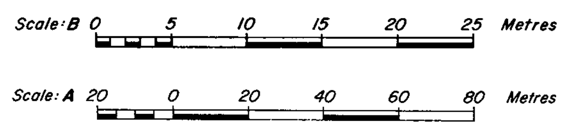
SECTION THROUGH PLUNGE POOL
Scale A



SECTION THROUGH AUXILIARY SPILLWAY
Scale B



SECTION THROUGH OUTLET VALVES
Scale B



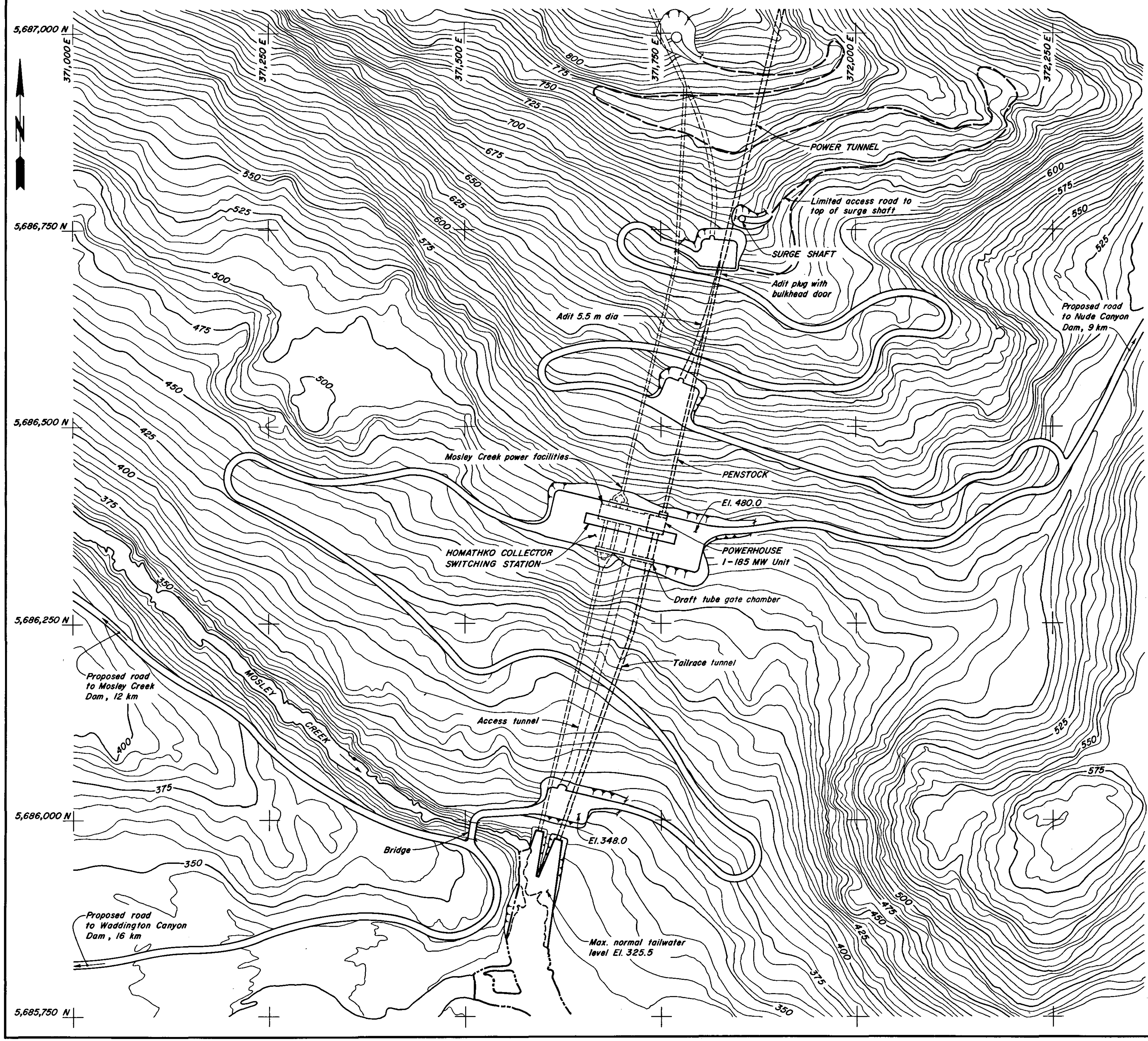
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

NUDE CANYON PROJECT
DAM ELEVATION AND SECTIONS

DATE MAR 1984	FIG. 9-2
DWN GB	DWG No. 704-C14-D460

REPORT No. H1688



Scale: 50 0 50 100 150 200 Metres

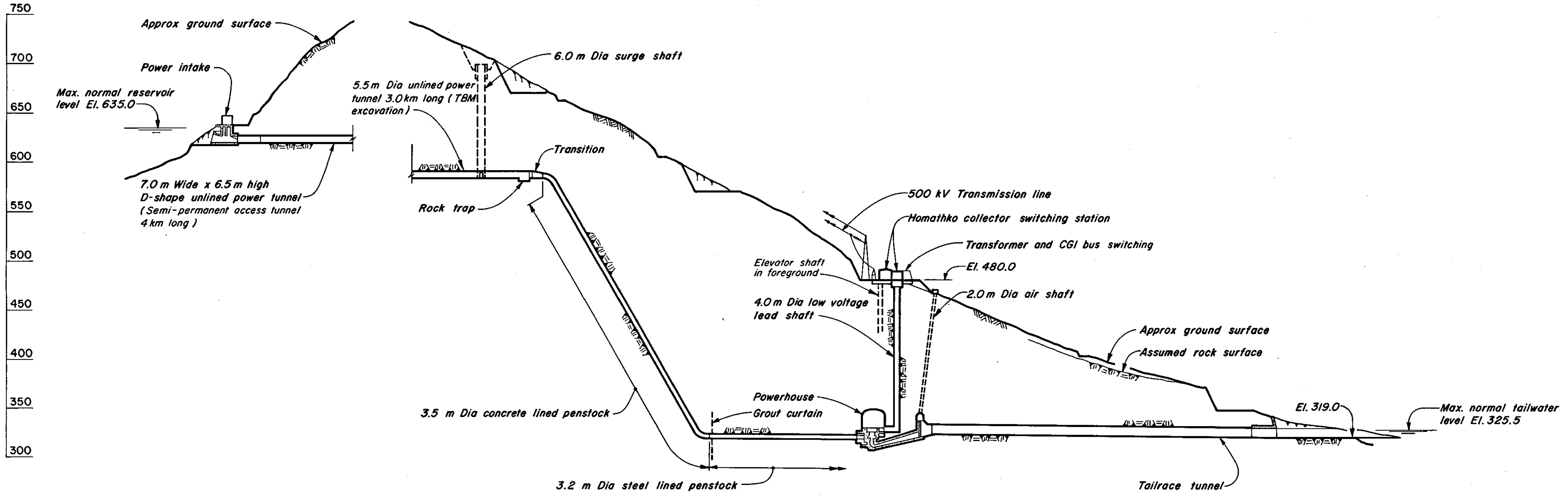
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

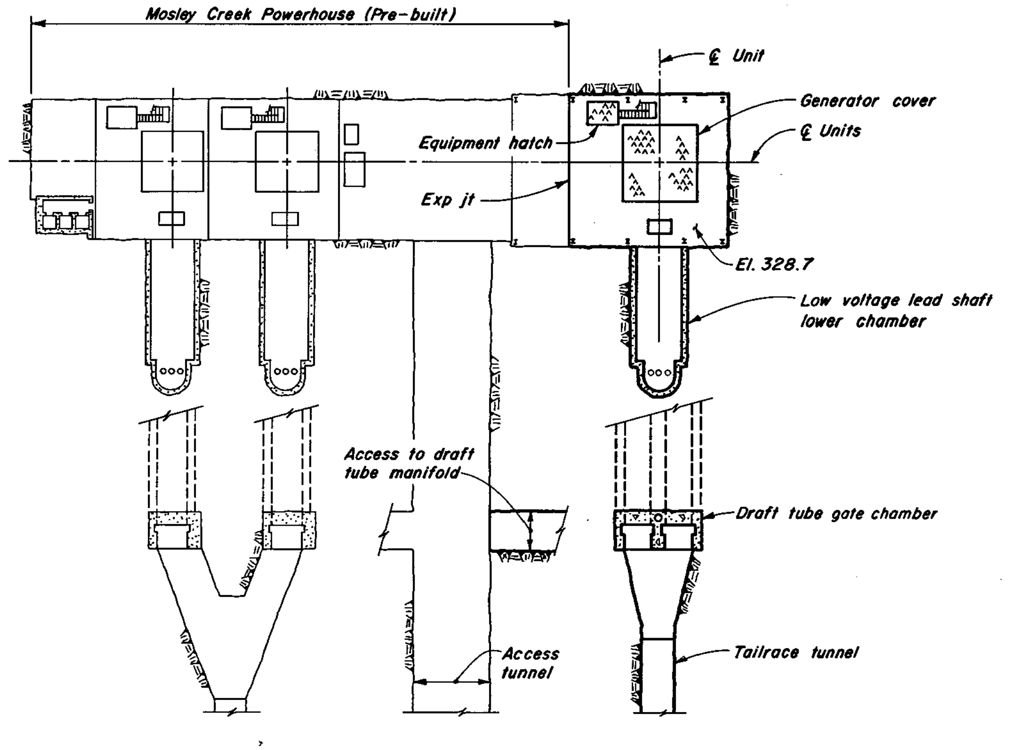
NUDE CANYON PROJECT
POWER FACILITIES
GENERAL ARRANGEMENT

DATE MAR 1984	FIG. 9-3
DWN GB	DWG No. 704-C14-D461

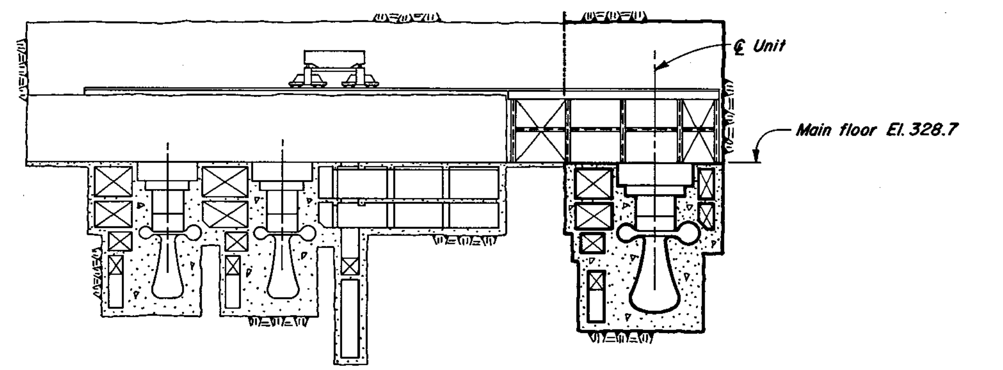
REPORT No. H1688



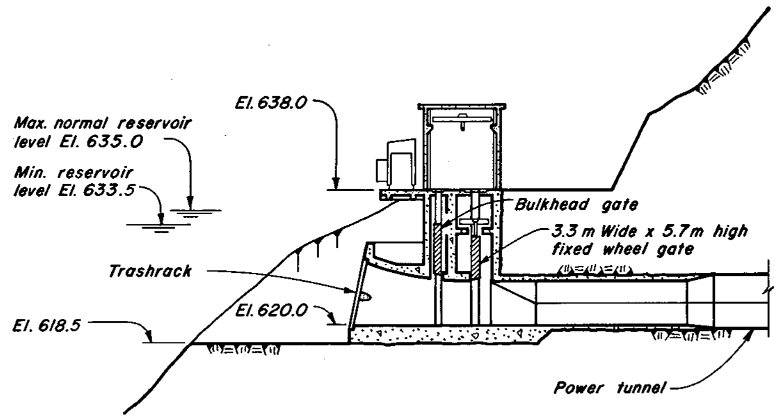
PROFILE
Scale A



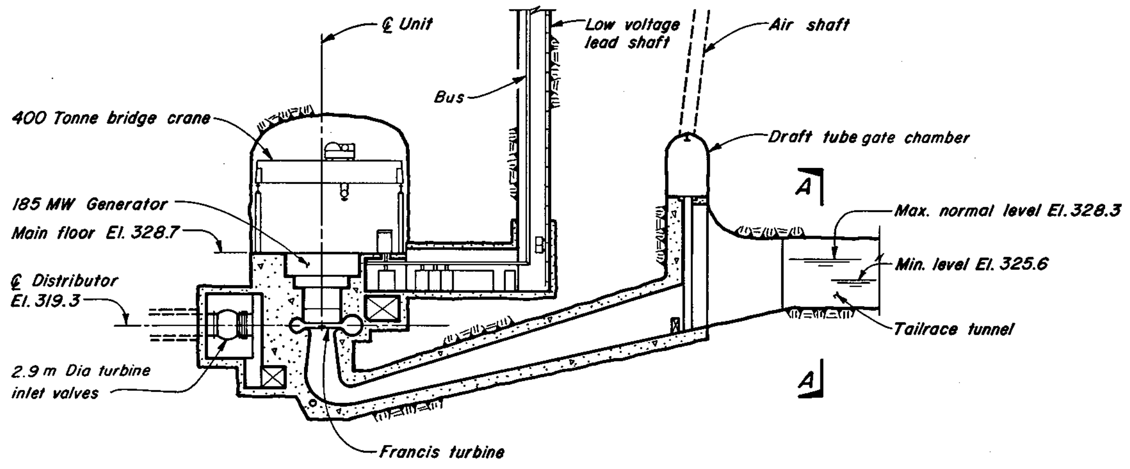
MAIN FLOOR PLAN
Scale B



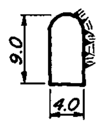
LONGITUDINAL SECTION
Scale B



SECTION THROUGH POWER INTAKE
Scale B



TRANSVERSE SECTION
Scale B



SECTION A-A
Scale B

Scale: B 10 0 10 20 30 40 Metres

Scale: A 50 0 50 100 150 200 Metres

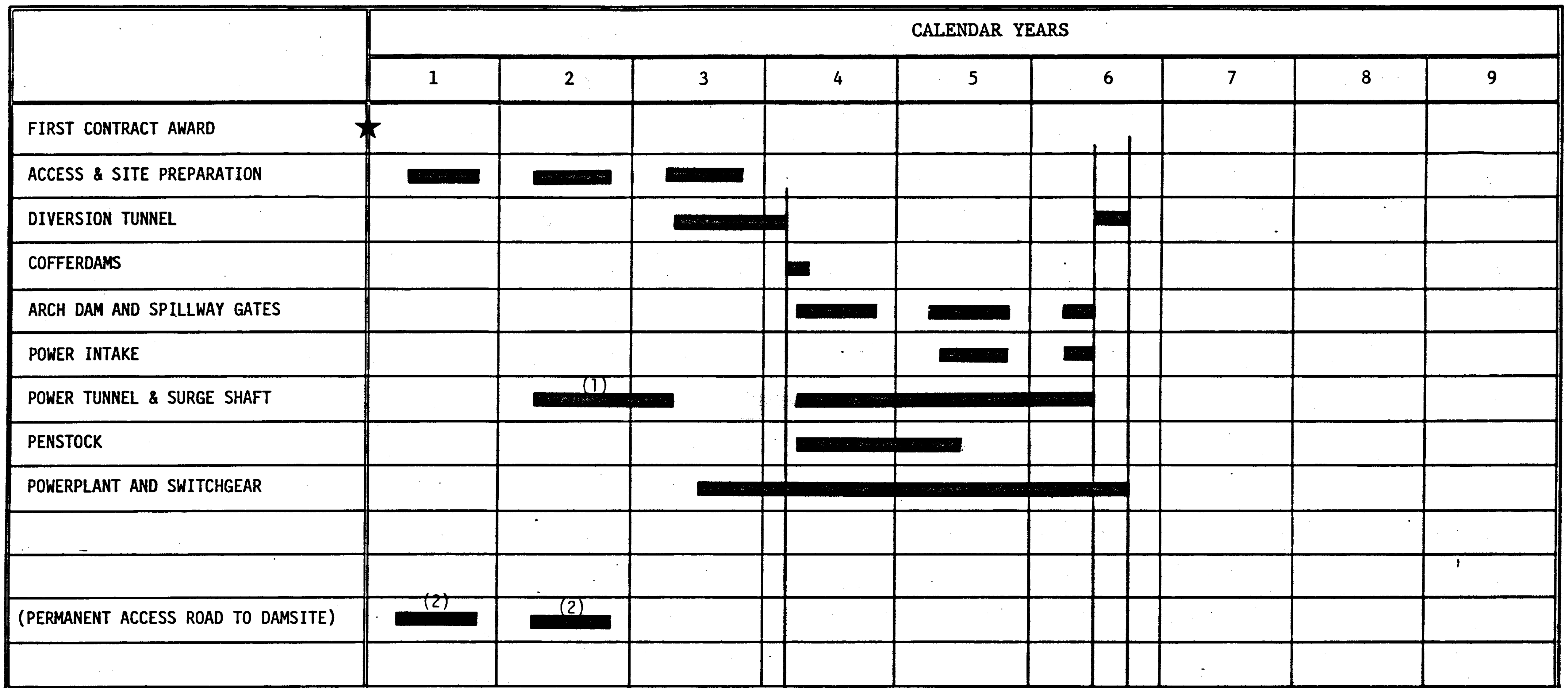
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

NUDE CANYON PROJECT
POWER FACILITIES
PLAN AND SECTIONS

DATE MAR 1984	FIG. 9-4
DWN GB	DWG No. 704-C14-U462

REPORT No H1688



LEGEND: (1) PROPOSED ENLARGED SECTION OF POWER TUNNEL USED AS MAIN CONSTRUCTION ACCESS TO DAMSITE.

(2) PERMANENT ACCESS ROAD TO DAMSITE IF REQUIRED, IN LIEU OF 1

NOTE: PROJECT APPROVAL REQUIRED SIX MONTHS PRIOR TO AWARD OF FIRST CONTRACT.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

NUDE CANYON PROJECT

CONSTRUCTION SCHEDULE

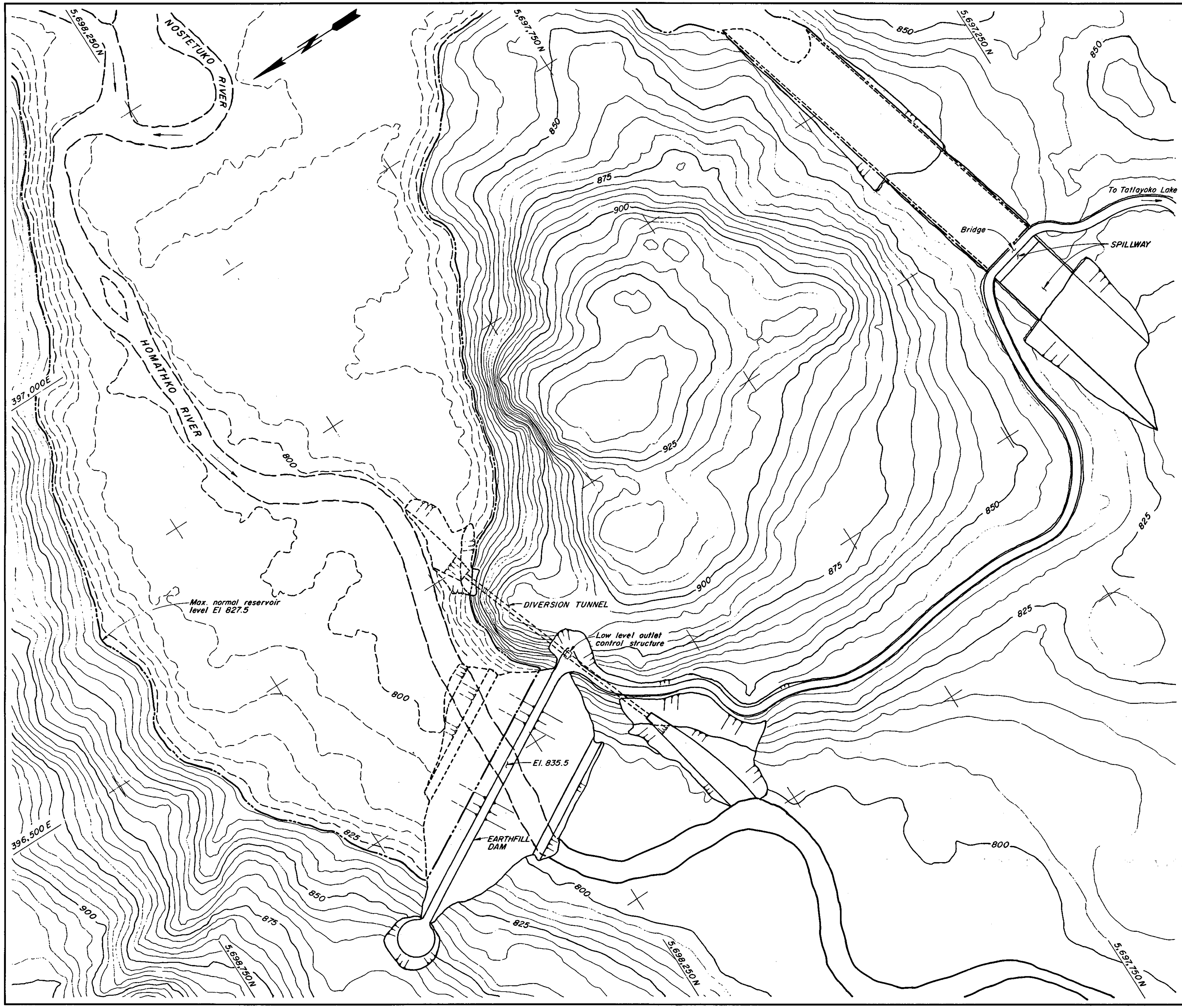
DATE MAR 1984

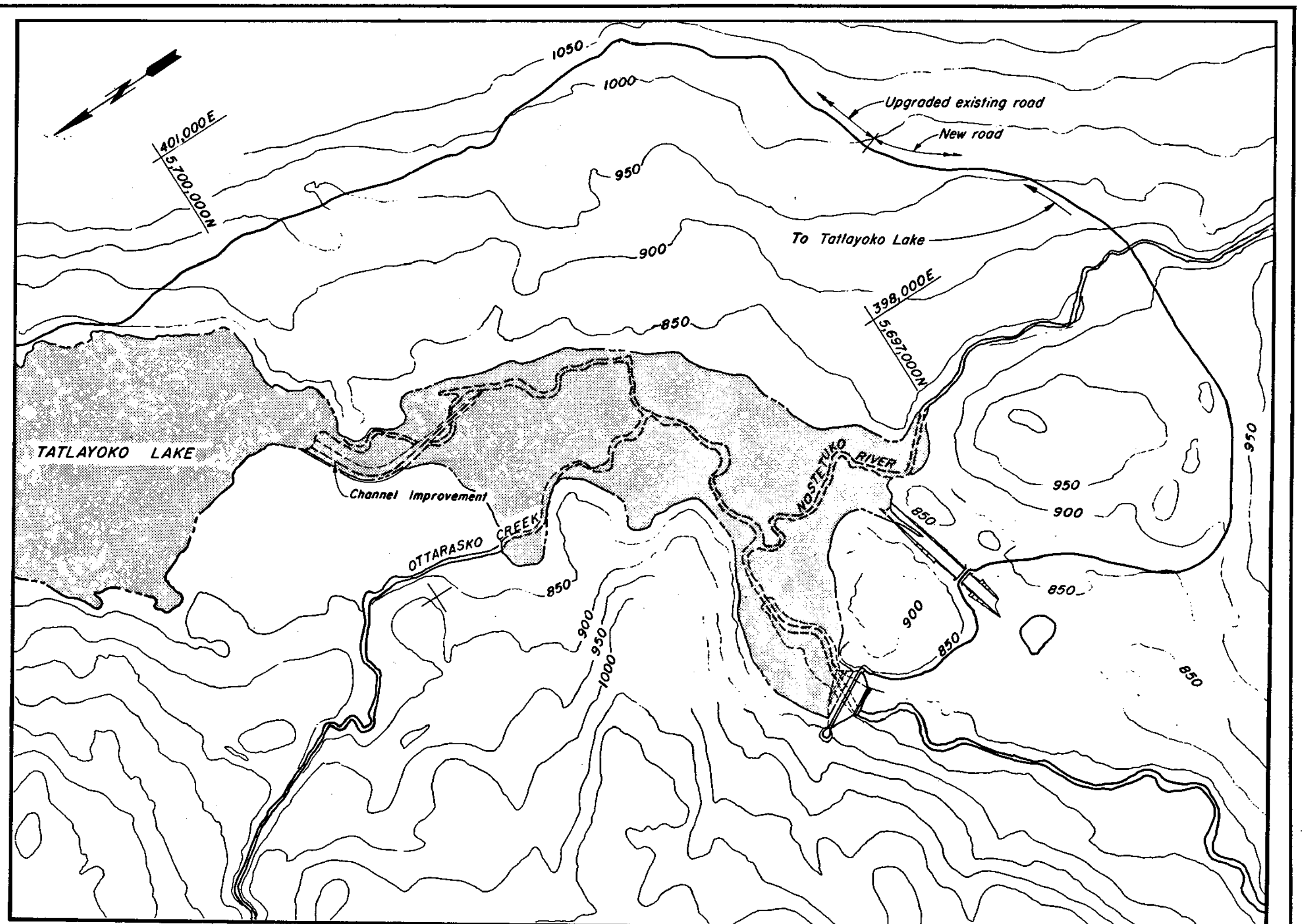
FIG. 9-5

DWN. A.SZ.

DWG. No 704-C14-B463

REPORT No H1688





LOCATION PLAN

Scale A

Scale: 50 0 50 100 150 200 Metres

Scale: A 0 500 1000 1500 2000 Metres

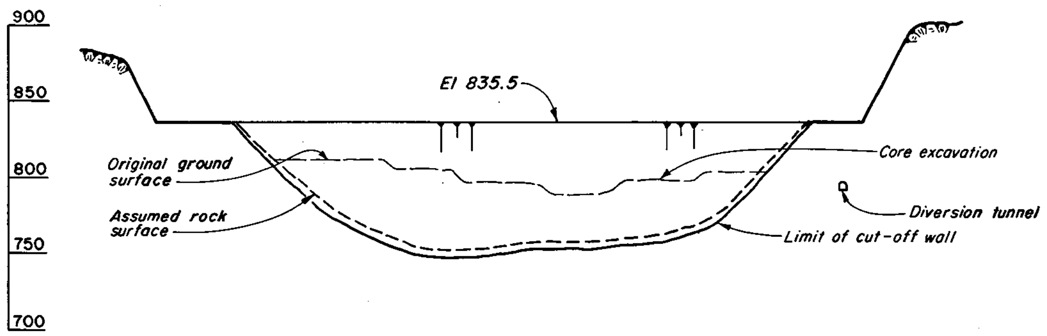
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

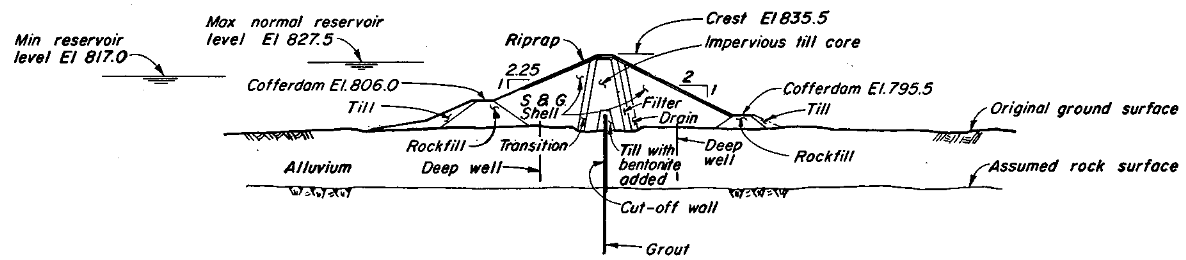
TATLAYOKO STORAGE PROJECT
NOSTETUKO AXIS
GENERAL ARRANGEMENT

DATE MAR 1984	FIG. 10-1
DWN FL	DWG No. 704-C14-U464

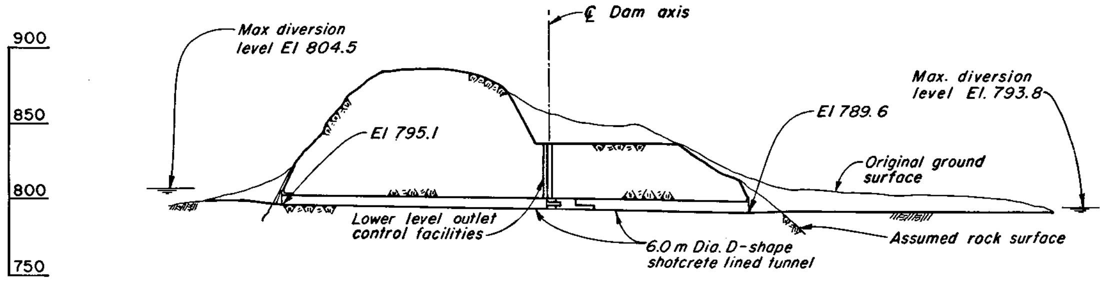
REPORT No H1688



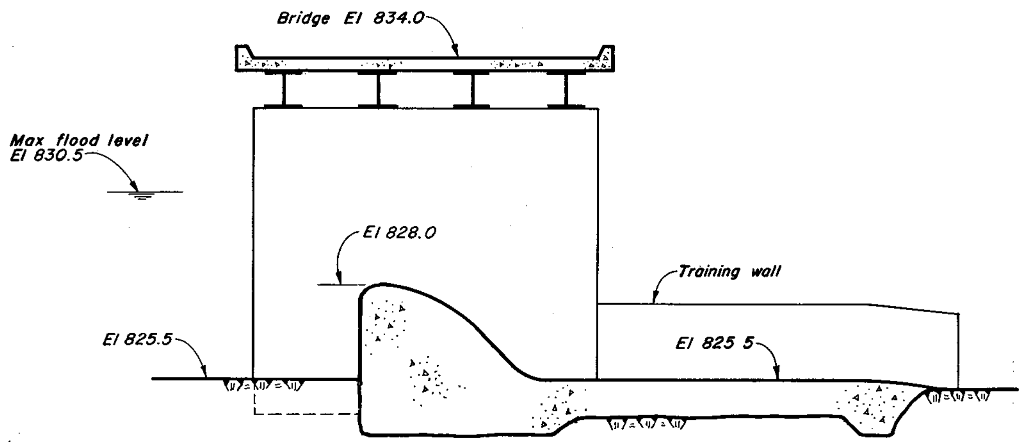
DOWNSTREAM ELEVATION EARTHFILL DAM
Scale A



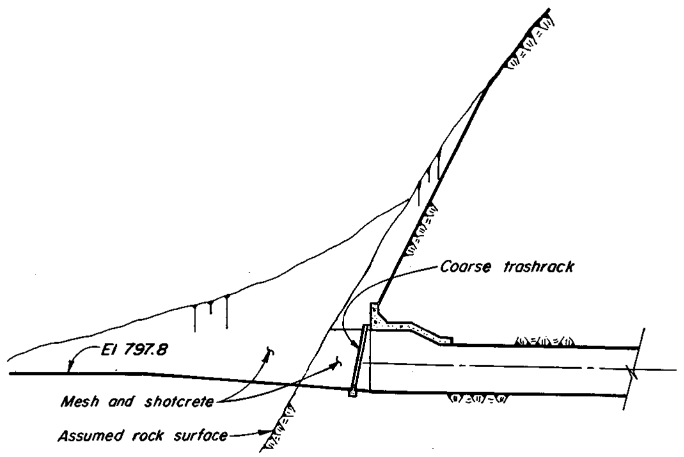
SECTION THROUGH EARTHFILL DAM
Scale A



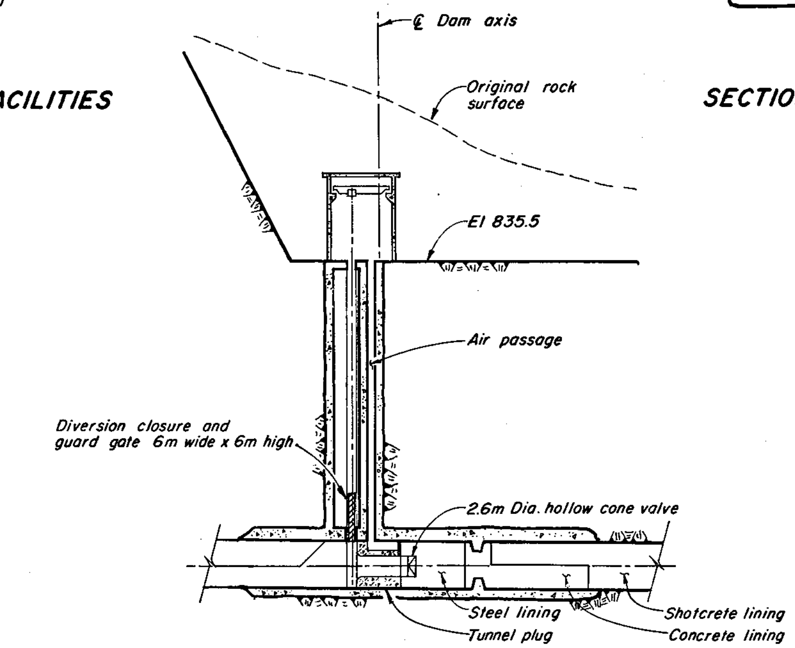
SECTION THROUGH DIVERSION TUNNEL & OUTLET FACILITIES
Scale A



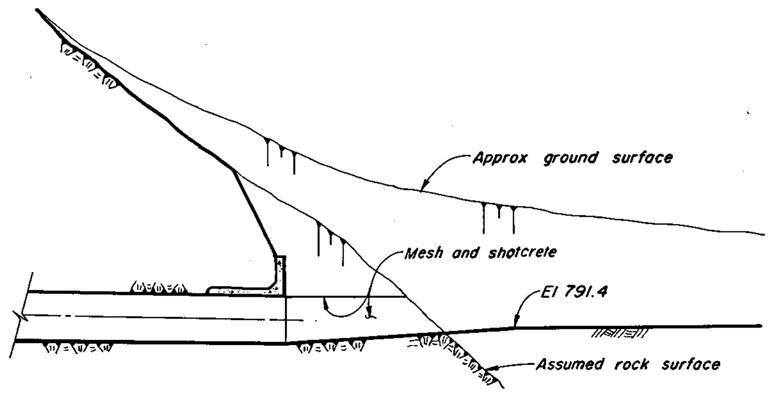
SECTION THROUGH SPILLWAY CREST
Scale B



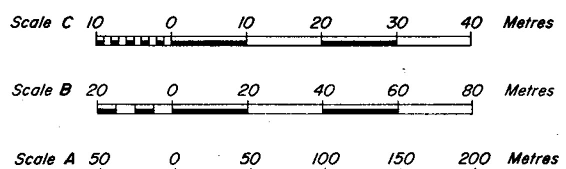
SECTION THROUGH INLET
OF DIVERSION TUNNEL
Scale C



SECTION THROUGH LOW LEVEL OUTLET FACILITIES
Scale C



SECTION THROUGH OUTLET
OF DIVERSION TUNNEL
Scale C

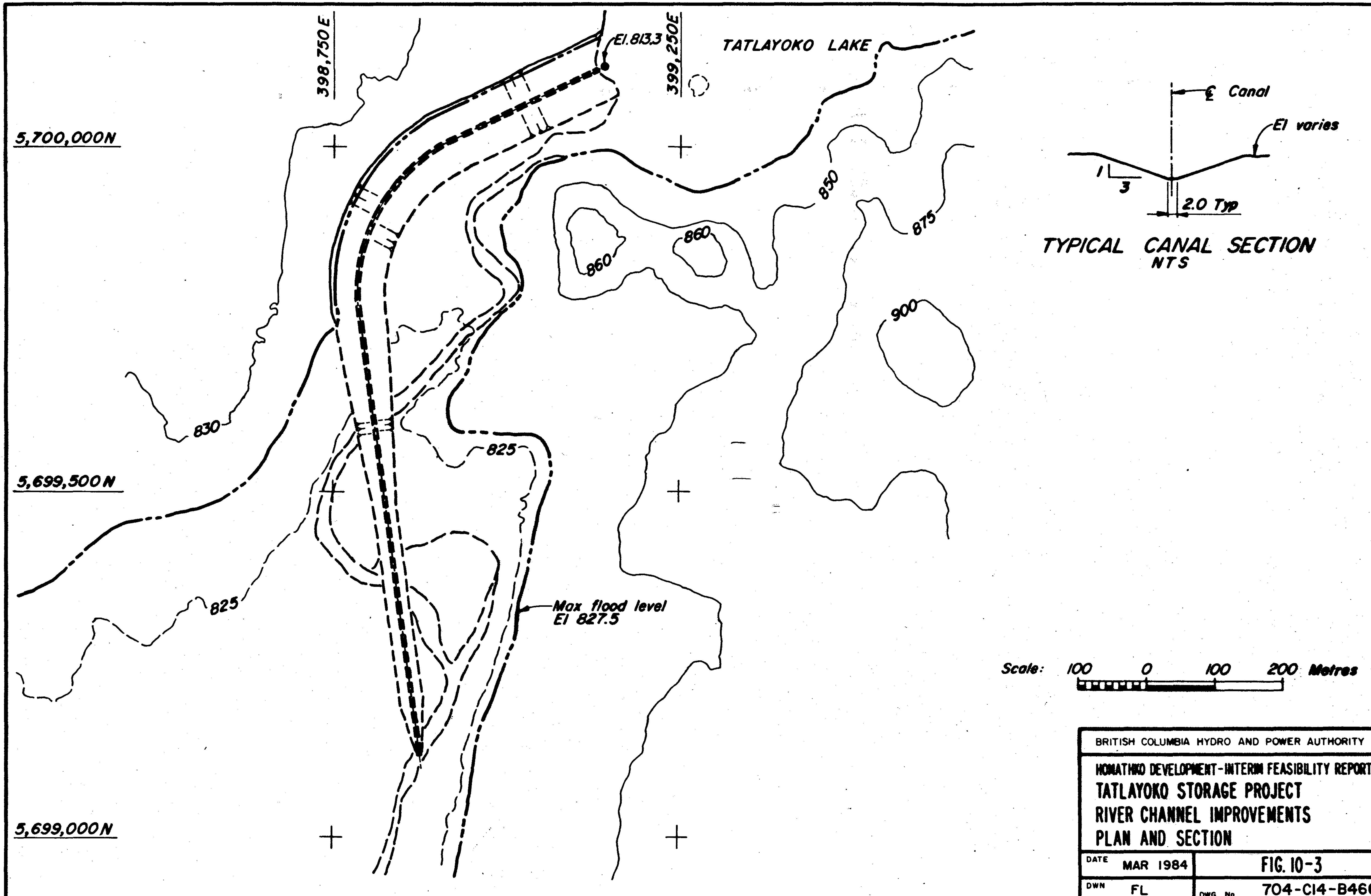


BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT
TATLAYOKO STORAGE PROJECT
NOSTETUKO AXIS
DAM ELEVATION AND SECTIONS

DATE MAR 19 4	FIG. 10-2
DWN FL	DWG No. 704-C14-D465

REPORT No. H1688



BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

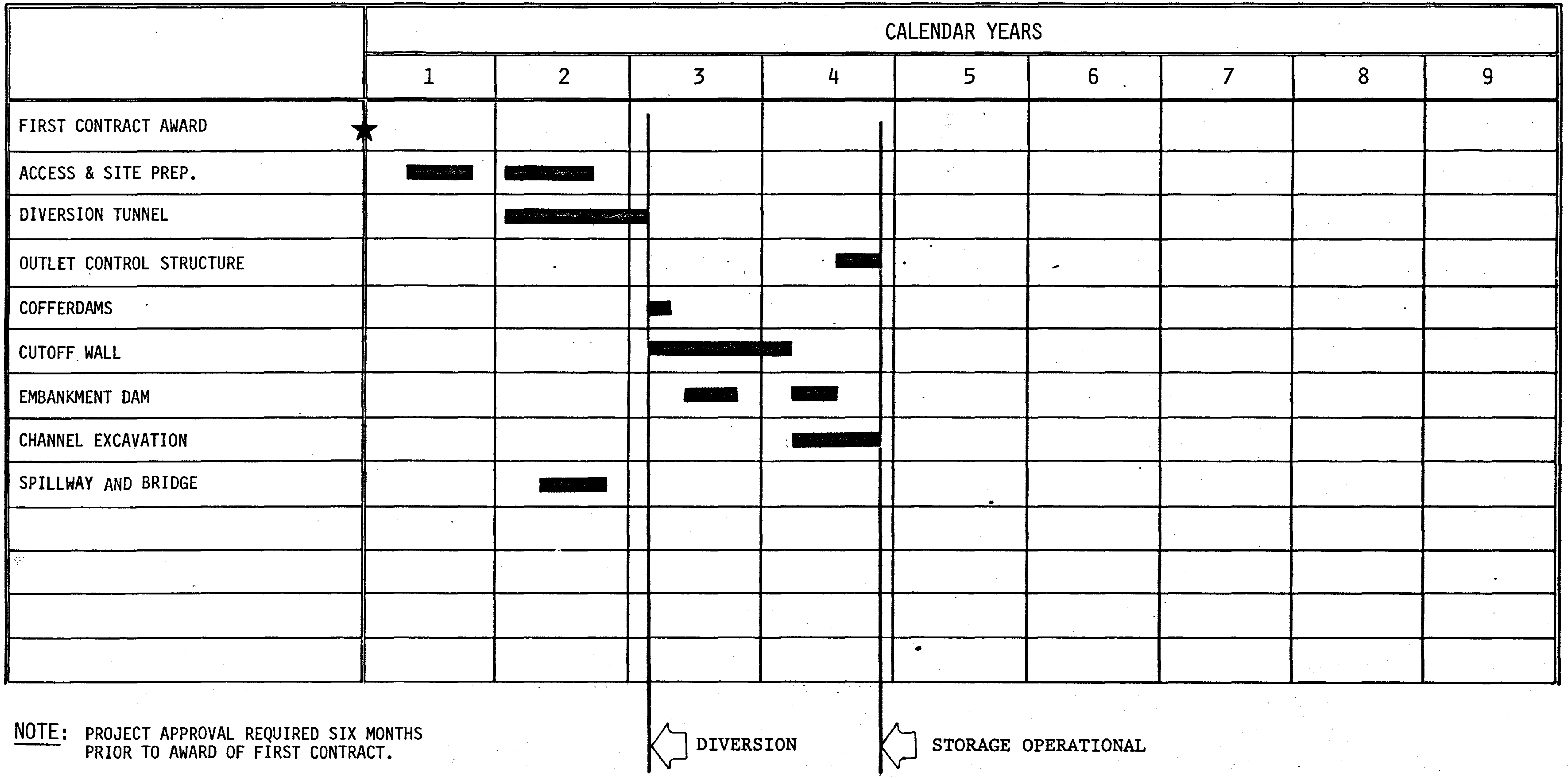
TATLAYOKO STORAGE PROJECT

RIVER CHANNEL IMPROVEMENTS

PLAN AND SECTION

DATE MAR 1984	FIG. 10-3
DWN FL	DWG. No. 704-C14-B466

REPORT No. H1688



NOTE: PROJECT APPROVAL REQUIRED SIX MONTHS PRIOR TO AWARD OF FIRST CONTRACT.

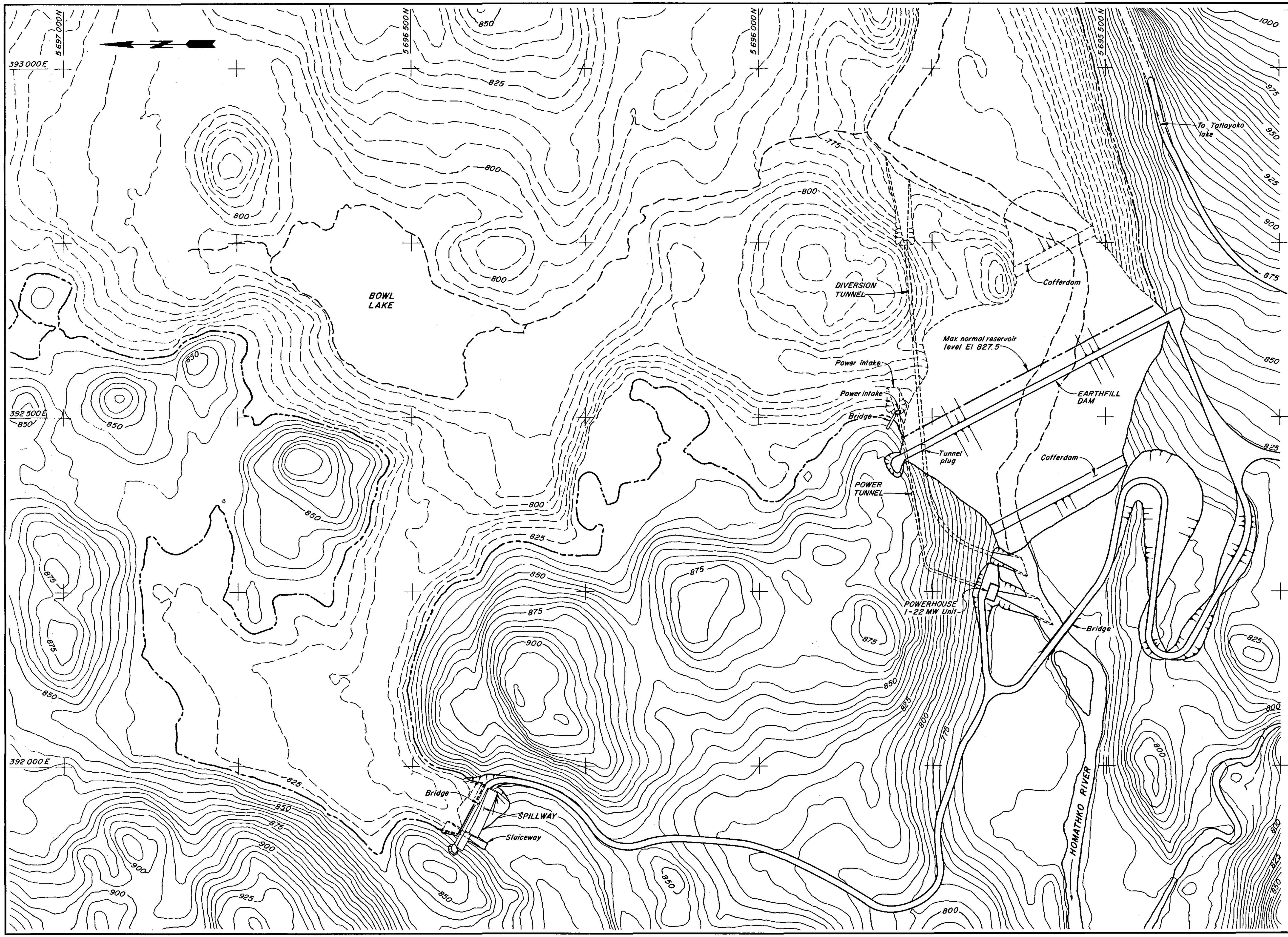
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

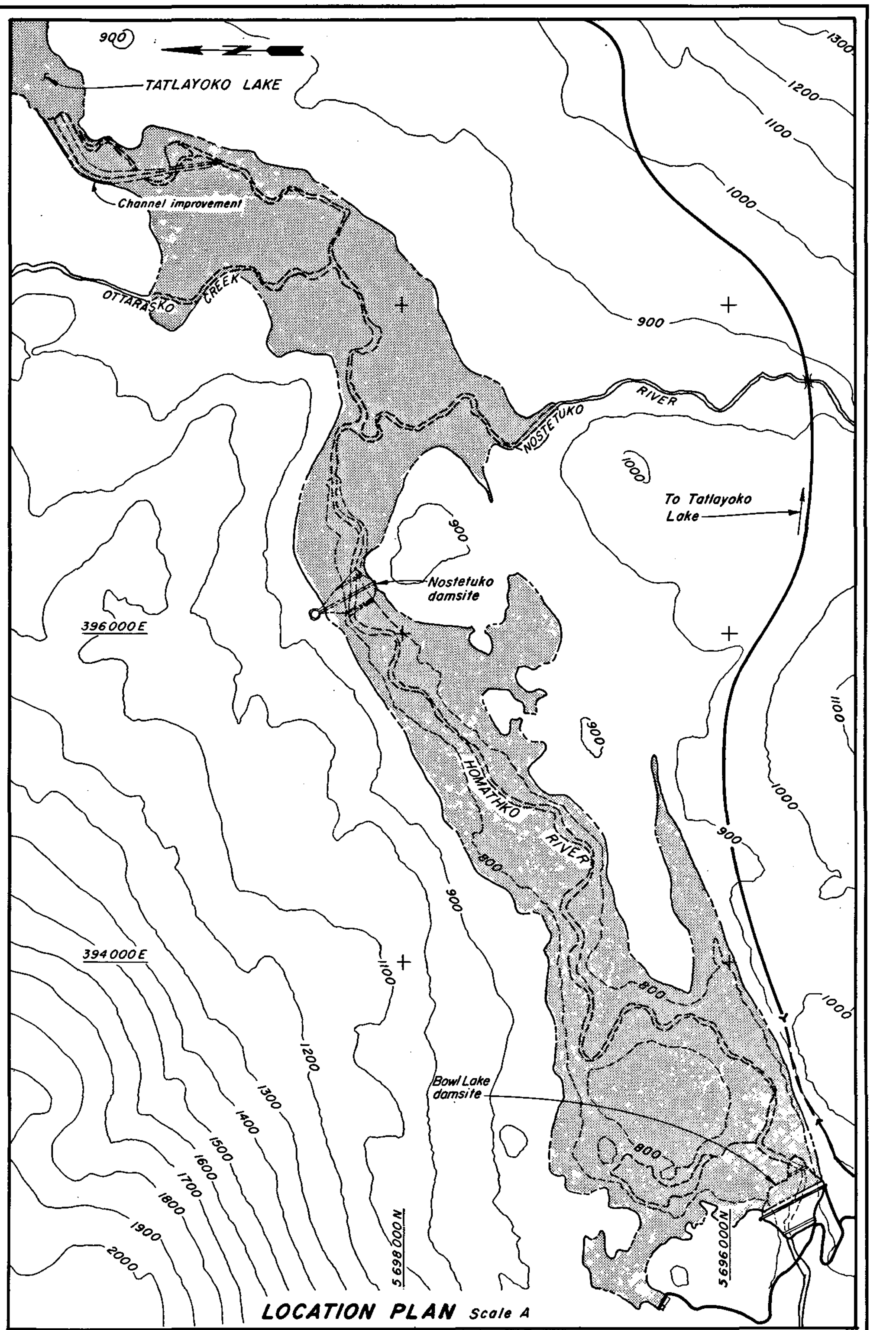
HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT
NOSTETUKO AXIS
CONSTRUCTION SCHEDULE

DATE MAR. 1984	FIG. 10-4
DWN A.Sz.	DWG. No. 704-C14-B467

REPORT No. H1688





LOCATION PLAN Scale A

NOTE:

THE BOWL LAKE AXIS WAS STUDIED AS AN ALTERNATIVE TO THE NOSTETUKO AXIS.

Scale: 50 0 50 100 150 200 Metres

Scale A 0 500 1000 1500 2000 Metres

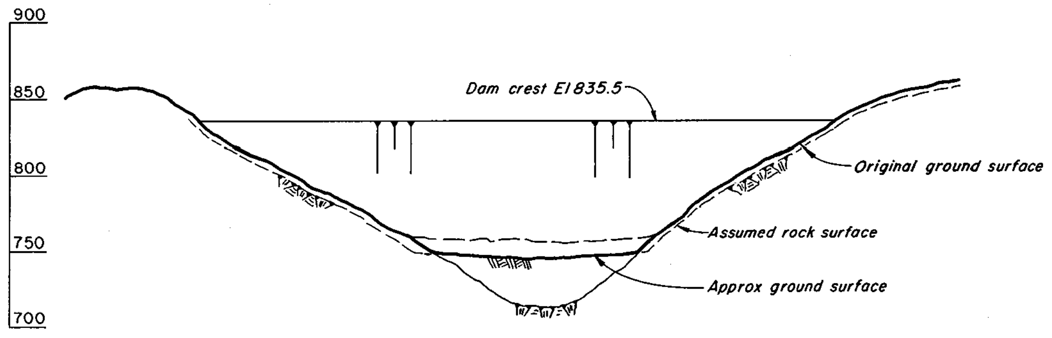
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT
BOWL LAKE AXIS
GENERAL ARRANGEMENT

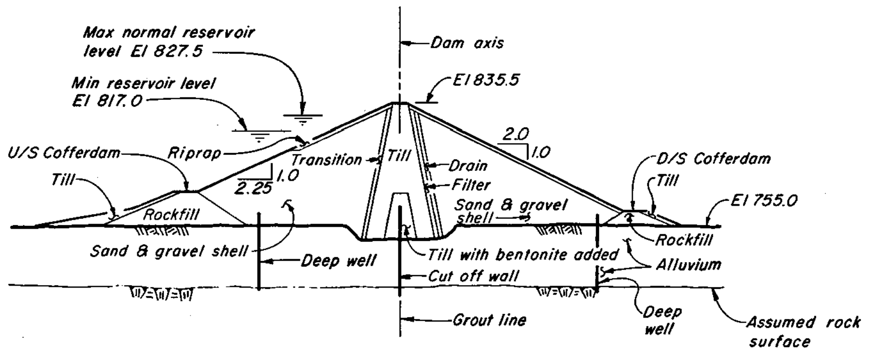
DATE MAR 1984	FIG. 11-1
DWN BE	DWG No. 704-C14-U468

REPORT No H1688



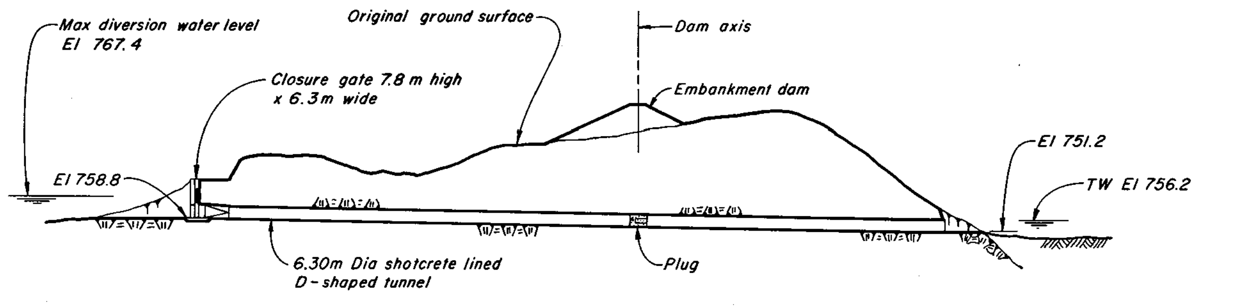
DOWNSTREAM ELEVATION

Scale A



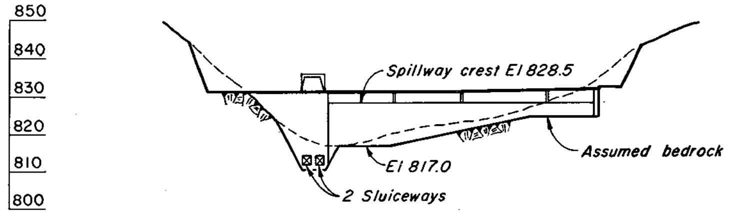
SECTION THROUGH EARTHFILL DAM

Scale : A



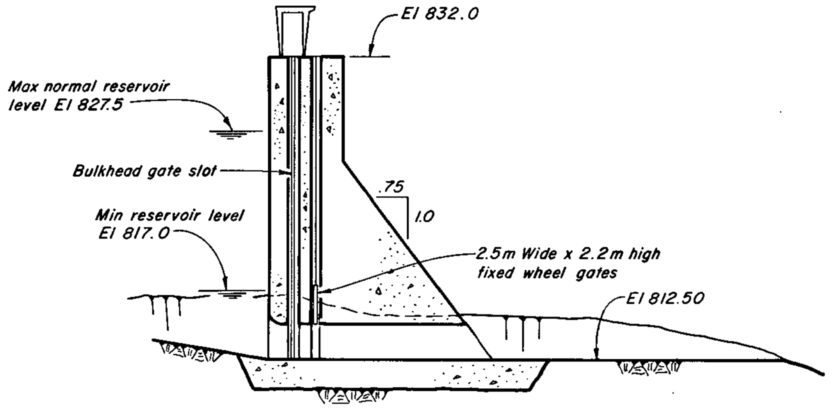
SECTION THROUGH DIVERSION TUNNEL

Scale A



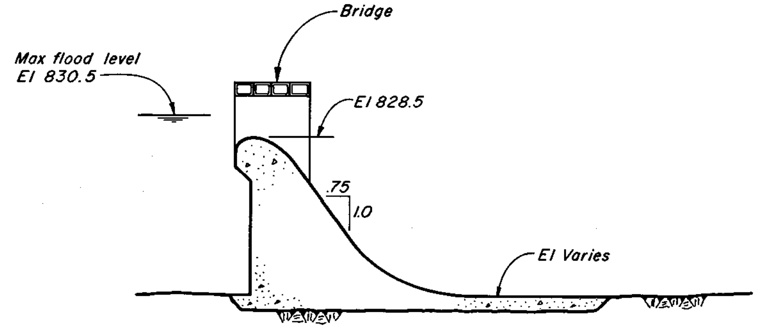
DOWNSTREAM ELEVATION OF SPILLWAY

Scale B



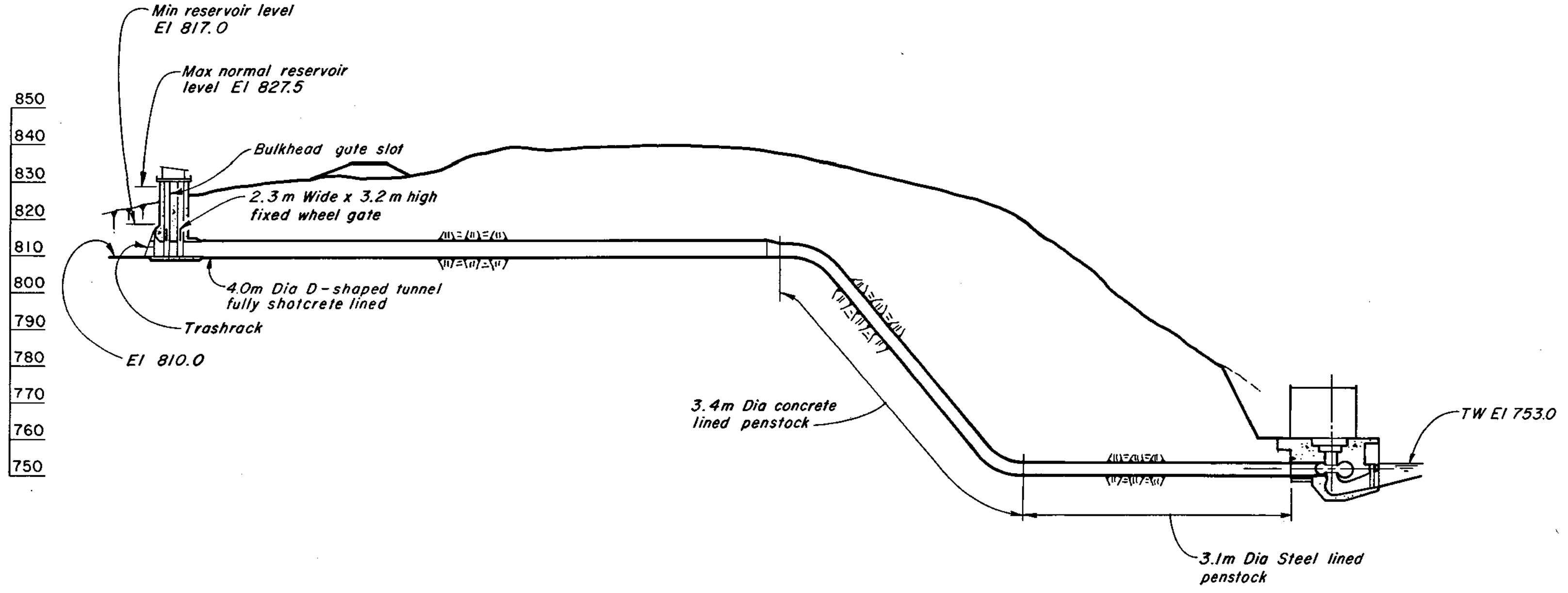
SECTION THROUGH SLUICEWAY

Scale C



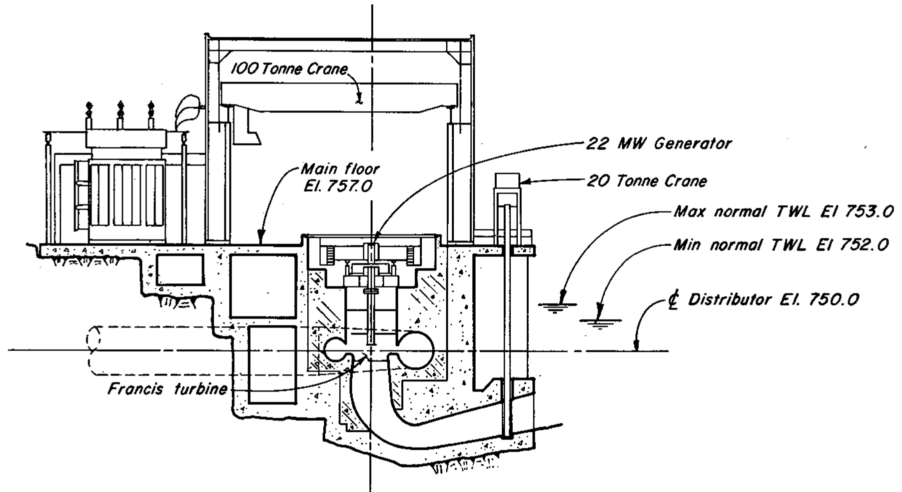
SECTION THROUGH SPILLWAY

Scale C



SECTION THROUGH POWER FACILITIES

Scale B

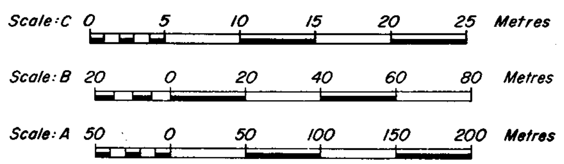


TRANSVERSE SECTION THROUGH POWERHOUSE

Scale C

NOTE:

THE BOWL LAKE AXIS WAS STUDIED AS AN ALTERNATIVE TO THE NOSTETUKO AXIS.



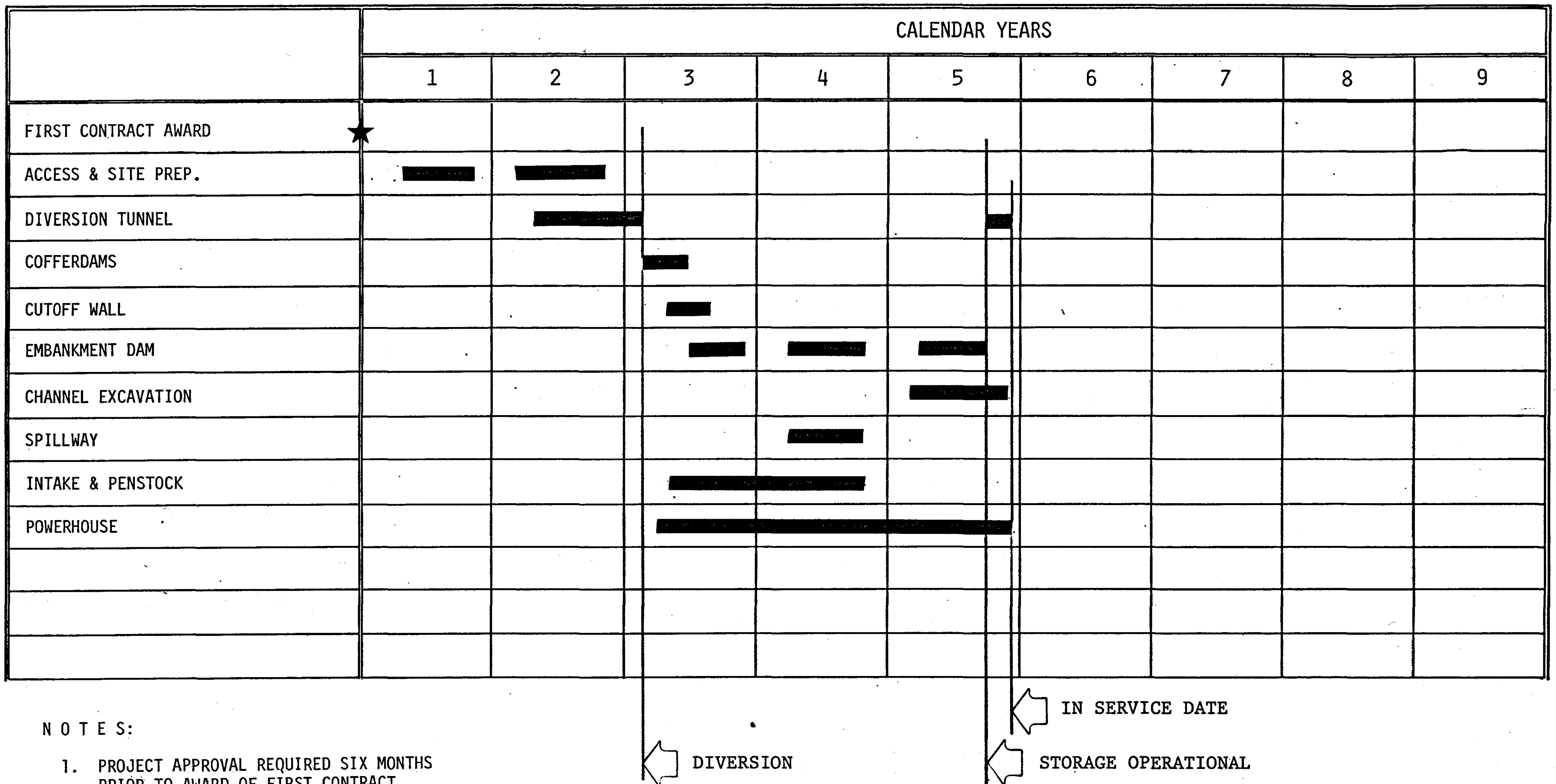
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT - INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT
BOWL LAKE AXIS
DAM ELEVATION & SECTIONS

DATE MAR 1984	FIG. II-2
DWN BE	DWG No. 704-C14-U469

REPORT No H1688



NOTES:

1. PROJECT APPROVAL REQUIRED SIX MONTHS PRIOR TO AWARD OF FIRST CONTRACT.
2. THE BOWL LAKE AXIS WAS STUDIED AS AN ALTERNATIVE TO THE NOSTETUKO AXIS.

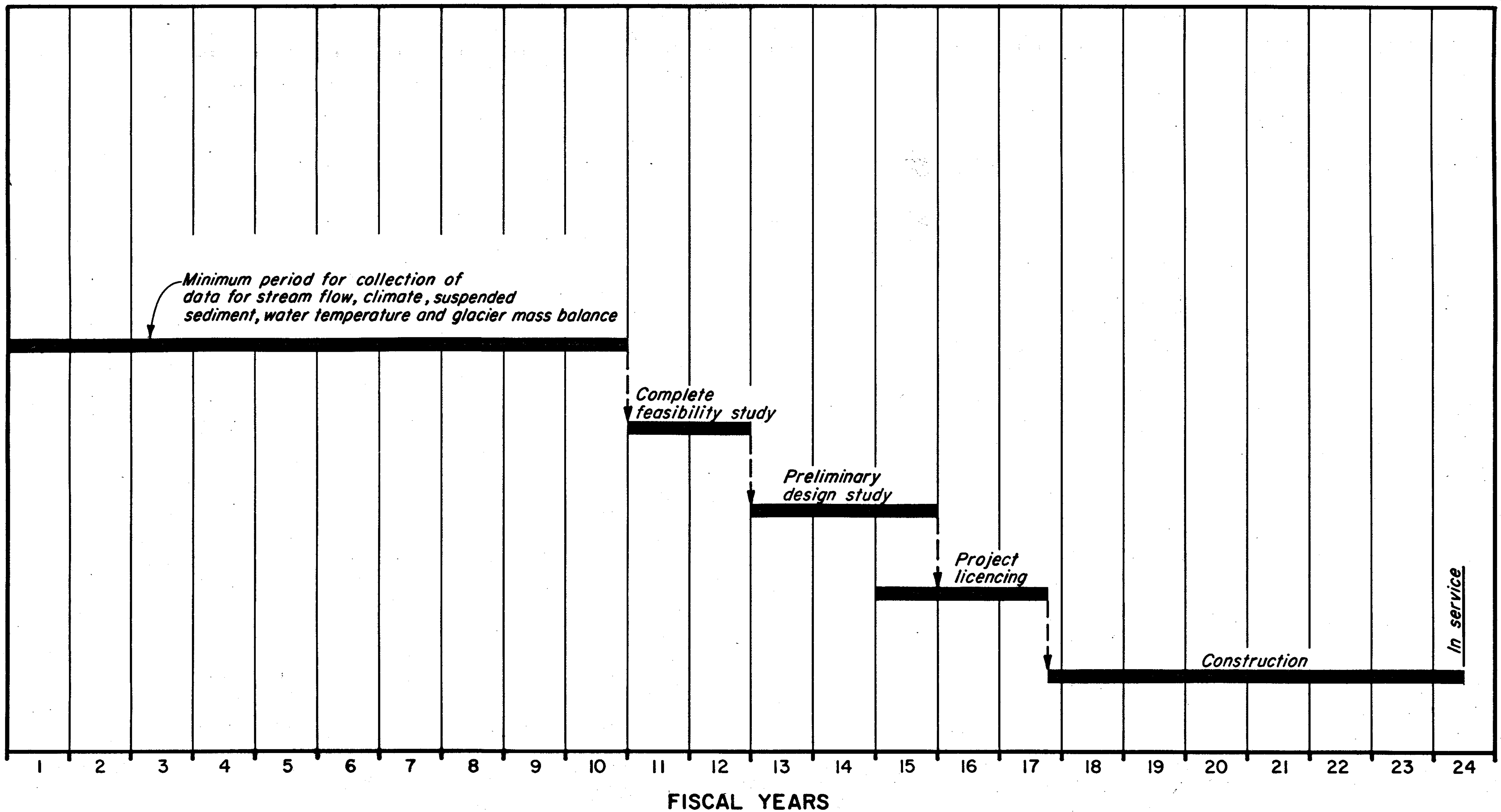
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

TATLAYOKO STORAGE PROJECT
BOWL LAKE AXIS
CONSTRUCTION SCHEDULE

DATE MAR. 1984	FIG. 11-3
DWN A. SZ.	DWG. No. 704-C14-B470

REPORT No. H1688



BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HOMATHKO DEVELOPMENT-INTERIM FEASIBILITY REPORT

RECOMMENDED SCHEDULE FOR FUTURE STUDIES

DATE MAR 1984	FIG. 13-1
DWN FM	DWG. No. 704-C14-B471

REPORT No. H1688