BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

Sandwell and Company Ltd. - Hat Creek Project - Cooling Water Supply Preliminary Design Study - Volume 1 - March 1978.

ENVIRONMENTAL IMPACT STATEMENT REFERENCE NUMBER: 8a

REPORT V4191/1
MARCH 1978

B.C. HYDRO AND POWER AUTHORITY

VANCOUVER B.C.

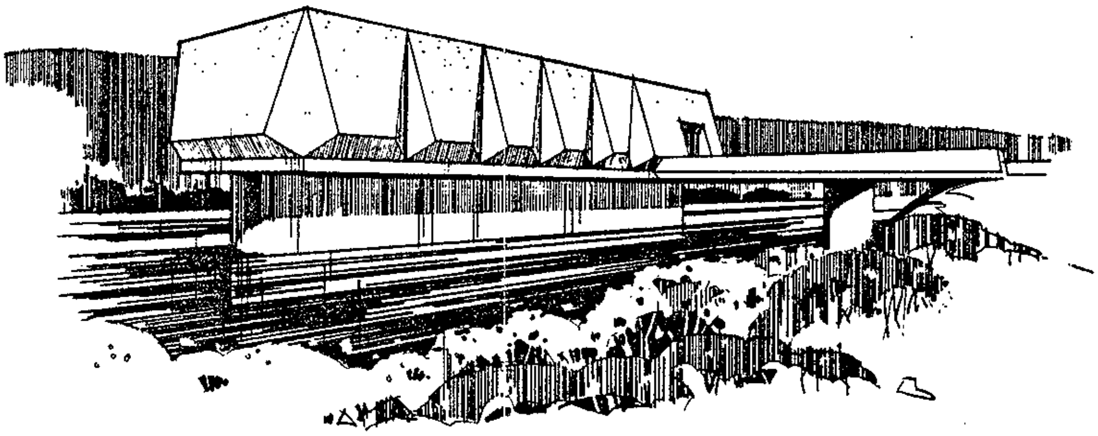
HAT CREEK PROJECT

COOLING WATER SUPPLY

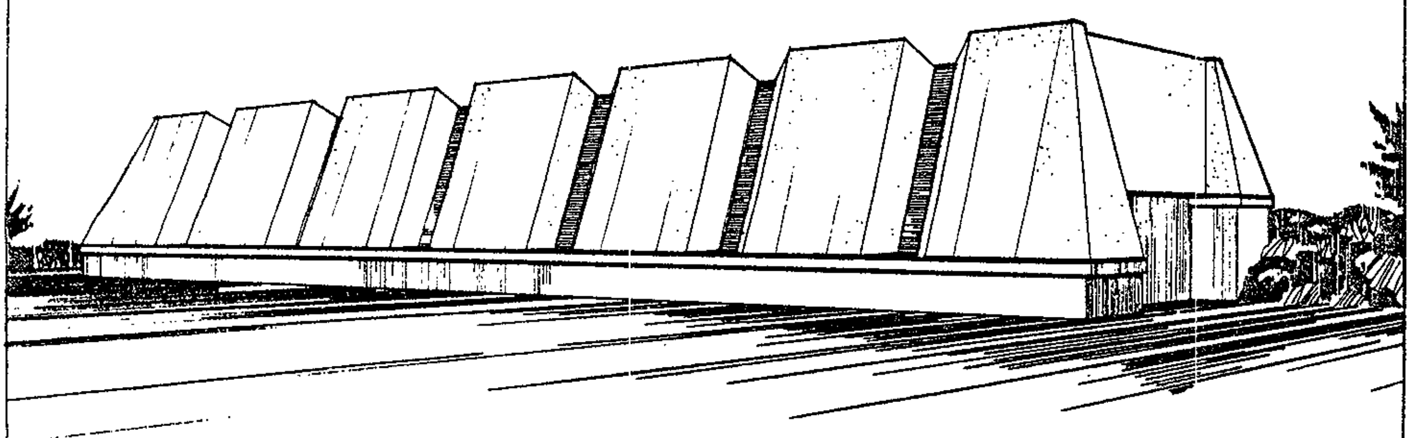
PRELIMINARY DESIGN STUDY

VOLUME ONE OF THREE

SANDWELL



INTAKE STRUCTURE



BOOSTER STATION

HAT CREEK PROJECT

COOLING WATER SUPPLY

SANDWELL AND COMPANY LIMITED
SUITE 601 — 1550 ALBERNI STREET, VANCOUVER
B.C., CANADA V6G 1A4
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10 March 1978

British Columbia Hydro and Power Authority
744 West Hastings Street, Ste. 500
Vancouver, British Columbia
V6C 1A5

Attention: C. K. Harman, P. Eng.
Project Manager, Off-Site Facilities

Reference: V4191 Hat Creek Project
Cooling Water Supply
021.50 Preliminary Design Study

Dear Sir:

We are pleased to present the attached copy of our Report V4191/1, Hat Creek Project, Cooling Water Supply, Preliminary Design Study, dated March 1978. One hundred copies have been sent to your attention under separate cover.

Yours truly

SANDWELL AND COMPANY LIMITED

BRMcConachy

B. R. McConachy, P. Eng.
Project Engineer

BRMcC/jc
Enclosure

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

DATE MARCH 1978

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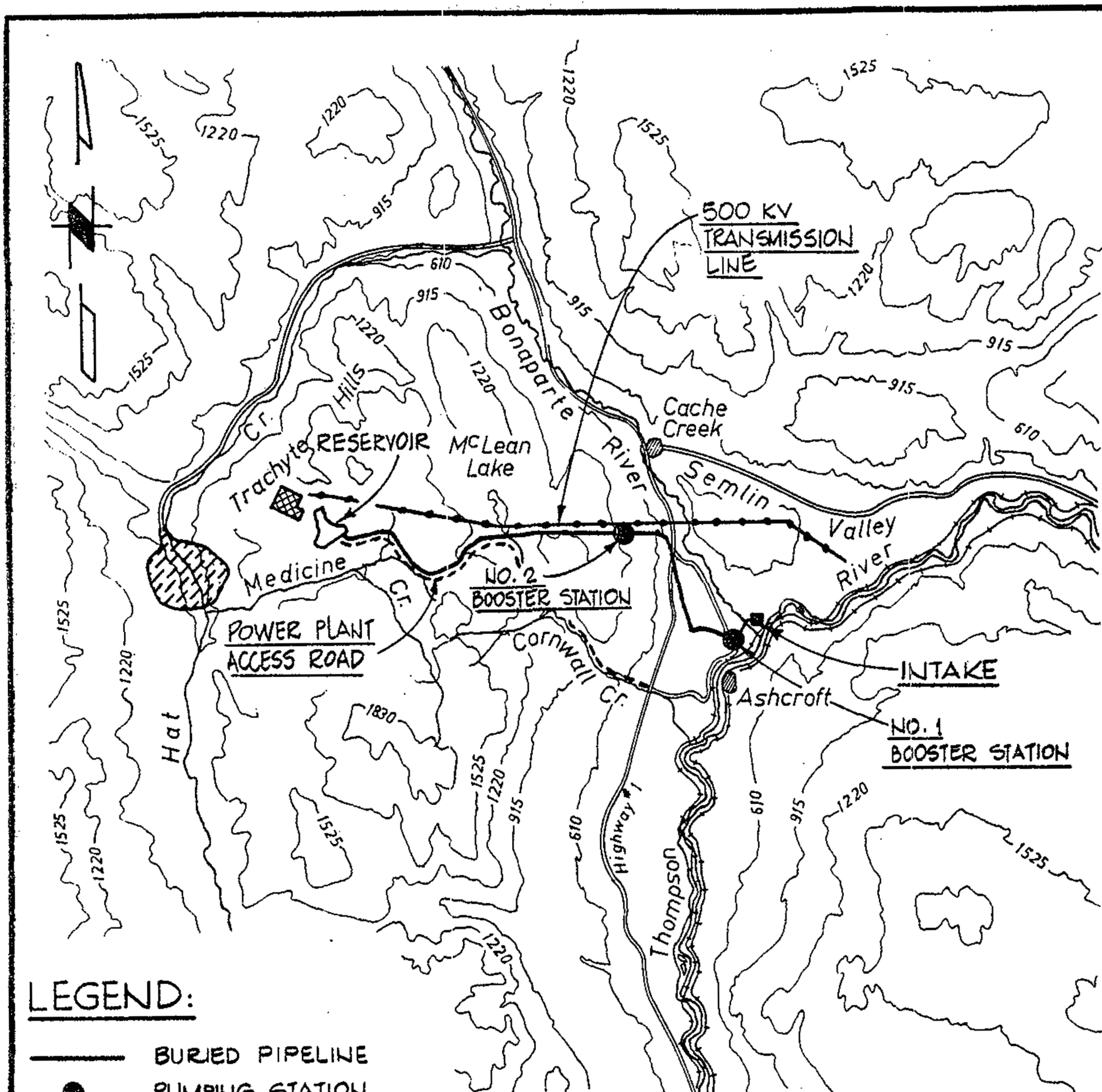

LEGEND:

BURIED PIPELINE

PUMPING STATION

2000 MW POWER PLANT SITE

OPEN PIT NO. 1

NOTE: CONTOUR ELEVATIONS IN METRES

SANDWELL DWG NO.	A 4191 - 1

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
KEY PLAN

DATE MARCH, 1978	SCALE 1:250 000	R
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REPORT No.

A

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

PRELIMINARY DESIGN STUDY

DATE MARCH 1978

SUMMARY

1. SCOPE AND PURPOSE OF DESIGN STUDY

B.C. Hydro and Power Authority is considering the development of a 2000 MW* thermal power plant fired by coal from the deposits in the Hat Creek Valley near Ashcroft, B.C. For location, see Key Plan on opposite page. In the Interim Report V4007/1 (Reference 1 **), Sandwell identified water sources and conduit routes for six potential power plant sites and prepared cost estimates. Subsequently, B.C. Hydro selected the Harry Lake Power Plant site for conceptual design. In January 1977, Sandwell presented their Conceptual Design Report V4007/2 (Reference 2) for a water supply system consisting of a direct intake in the Thompson River, and a buried steel pipeline to the water reservoir adjacent to the Harry Lake site.

In May 1977, Sandwell was awarded the preliminary design study for the water supply. The assignment was to review the conceptual design studies; study alternative pipeline routes, intake sites and pumping systems; and complete preliminary design of the recommended scheme including a detailed cost estimate and construction schedule.

This report presents the Preliminary Design and Cost Estimate for the Cooling Water Supply System. The power plant site is situated in the northeast corner of the Upper Hat Creek Valley, in the Trachyte Hills, and its water reservoir at elevation 1372 m (4500 ft) is the terminus for the water supply system. The water would be drawn from the Thompson River at about elevation 290 m (950 ft) through travelling screens, and pumped by low head intake pumps 0.7 km to a degritting clarifier. From there, it would be pumped in a buried pipeline 23 km (14 miles) to the plant reservoir by two booster stations with equal heads of 640 m (2100 ft) each.

Assessment of the environmental and social impact of the water supply project is under study by others.

2. PRELIMINARY DESIGN AND ROUTING

The pipeline route selected for Preliminary Design follows the Conceptual Design Route from the river to Boston Flats, the proposed 500 kV transmission line ***to McLean Lake and the project access road from McLean Lake to the plant reservoir. This route not only combines project service corridors but

* Mega-watt. For this and other abbreviations, see Appendix 1, Glossary of Terms.

** For references, see Appendix 2 of this report.

***In the late stages of preliminary design, the possibility of locating the proposed 500 kV transmission line several miles south of Ashcroft was introduced. However, this study assumes that the transmission line follows the route shown on Drawings D4191-13 and -14.

would encounter fewer steep rocky sections than the Conceptual Design route, enabling pipeline construction by more conventional techniques.

The system configuration of low-lift intake pumps followed by two equal lifts of booster pumping was adopted to avoid excessive pump wear by the provision of degritting prior to booster pumping; to avoid difficulties with the supply of pipe, fittings, valves and pumps; and to avoid difficulties with welding and construction. The pipeline diameter of 800 mm (32 inch) was selected to provide the least total of capital cost and present worth of operating costs for the average system discharge of 663* l/s (10,490 USGPM). Key system parameters are summarized below:

Summary of System Parameters

Item	Amount (S.I.)	Amount (Imperial)
Maximum Discharge	1580 l/s	25,000 USGPM
Elevation Difference	1083 m	3,550 ft
Pipeline Length (along slope)	23.5 km	14.6 miles
Pipe Diameter (nominal)	800 mm	32 in.
Booster Pump Motors, each station	4 @ 3600 kW	4 @ 4800 HP
Intake Pump Motors	5 @ 170 kW	5 @ 250 HP

The Thompson River Intake, developed with the help of hydraulic model studies, would incorporate a by-pass flow parallel to the travelling screen face to prevent entrapment of fish. The proposed location of the intake, on the right bank of the Thompson River about 2.5 km upstream of Ashcroft, was selected from five potential sites for reasons of fish protection, hydraulics and accessibility. A clarifier would be provided ahead of the booster pumps to remove grit thus minimizing pump wear.

Booster Station No. 1 which is near the intake and No. 2 which is about 9.3 km along the pipeline, have identical arrangement and equipment thereby providing cost and maintenance advantages. The booster pumps would be multi-stage horizontal type; rated for 395 l/s each against a 640 m delivery head; and driven by 3600 kW, 3600 rpm, squirrel-cage induction motors. An equalization tank at the No. 2 Booster Station would regulate discharges between the two stations and would provide a positive isolation of the two pipeline sections thus simplifying waterhammer control.

Power for the booster stations would be tapped from a new substation and new 69 kV transmission lines.

The pipeline would be buried, of welded steel, and with wall thickness of 8, 11 or 16 mm depending on pressure. The steel selected for preliminary Design is Grade 60 with reduced carbon to obtain good welding and impact characteristics. Corrosion protection would be provided by interior and exterior coatings and by cathodic protection. Waterhammer would be controlled by providing one-way surge tanks, by increasing pump-set inertia, and by controlling the rate of discharge valve movements. Freeze protection would be provided by deep burial of the pipe. Provisions would be made for inspection using "pigs", flow driven instrument packages which travel inside the pipe. Pipeline drainage facilities would include 1.6 km of 250 mm buried drain line at Boston Flats.

* Subsequent to Preliminary Design, average discharge was revised to 726 l/s (11,500 USGPM).

The system would be manually operated from the Power Plant and only be capable of operating at four discrete discharge rates determined by the capacities of the four pairs of booster pumps. Alarms and interlocks would ensure safe and simple operation.

The capacity of the reservoir adjacent to the plant is sufficient for approximately 70 days of powerplant operation at maximum predicted capacity factor during the most severe water demand period. Therefore, shutdown of the cooling water supply system for periods up to 70 days would not restrict energy production at the Hat Creek Thermal Plant.

3. COST ESTIMATES

The Capital Cost estimate has been based on the Preliminary Design as summarized above and upon fourth quarter 1977 prices. It includes the direct costs of land, structures and equipment, construction overhead, engineering, contingencies and corporate overhead but excludes interest during construction escalation, the 69 kV power supply, working capital, start-up expenses, Federal sales and municipal taxes, and premium time charges except for a limited allowance.

Structures

Dept.	Description	Material	Labour	Total
271.00	Thompson River Intake	$ 1,070,000	$ 1,570,000	$ 2,640,000
272.00	Water Pipeline	6,665,000	8,870,000	15,535,000
273.00	No. 1 Booster Station	550,000	400,000	950,000
274.00	No. 2 Booster Station	615,000	1,140,000	1,755,000
Total Structures		$ 8,900,000	$ 11,980,000	$20,880,000

Equipment

Dept.	Description	Material	Labour	Total
271.00	Thompson River Intake	$ 1,450,000	$ 330,000	$ 1,780,000
272.00	Water Pipeline	1,610,000	775,000	2,385,000
273.00	No. 1 Booster Station	2,500,000	930,000	3,430,000
274.00	No. 2 Booster Station	2,620,000	825,000	3,445,000
291.00	Power Supply & Distribution	1,750,000	595,000	2,345,000
Total Equipment		$ 9,930,000	$ 3,455,000	$13,385,000
Total Direct Cost		$18,830,000	$ 15,435,000	$34,265,000
Owner's Construction Overhead				2,740,000
Engineering				3,500,000
Contingencies				5,245,000
Total Construction Cost				$45,750,000
Corporate Overhead				2,250,000
Total Capital Cost				$48,000,000

SANDWELL

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

B. C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

DATE MARCH 1978

INTRODUCTION

Background

B. C. Hydro and Power Authority is considering the development of a 2000 MW* thermal power plant fired by coal from the deposits in the Hat Creek Valley near Ashcroft, B.C. For location, see Drawing A4191-1 **, Key Plan. In the Interim Report V4007/1 (Reference 1 ***), Sandwell identified water sources and conduit routes for six potential power plant sites and prepared cost estimates. Subsequently, B. C. Hydro selected the Harry Lake Power Plant site for conceptual design. In January 1977, Sandwell presented their Conceptual Design Report V4007/2 (Reference 2) for a water supply system consisting of a direct intake in the Thompson River and a buried steel pipeline to the Hat Creek Valley.

Terms of Reference

In May 1977, Sandwell was awarded the preliminary design study for the water supply. The assignment was to review the conceptual design studies; study alternative pipeline routes, intake sites and pumping systems; and complete preliminary design of the recommended scheme including a detailed cost estimate and construction schedule.

The detailed Terms of Reference for this assignment are attached to this Report as Appendix 3. The work was carried out generally in accordance with the Terms of Reference with the exception of the investigations into indirect water intakes which were more extensive than anticipated.

SYSTEM CONFIGURATION

General

The system configuration chosen for Preliminary Design has evolved from studies of intake locations, pipeline and tunnel routes, and pumping schemes. The current studies have determined the best location for a Thompson River intake and have selected the pipeline size, pipeline routing and the pumping arrangement. This section describes the alternatives studied and the reasons for selecting the Preliminary Design configuration.

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* Mega-watt. For this and other abbreviations, see Appendix 1 - Glossary of Terms.

** For drawings, see Appendix 6 - Illustrations.

*** For references, see Appendix 2.

Selection of Direct Intake

The merits of the various river intake designs that are available to reliably withdraw 1580 l/s (25,000 USGPM) from a major river with minimum adverse effects on fish life have been investigated and are reported in Project Memorandum V4007/1, Water Intake Design (Reference 3). Although this study concluded that "only the direct intake can provide an assured large water supply necessary for the successful operation of a thermal generating station", it was not specifically related to a particular reach of a specific river.

Since the Interim Report (Reference 1) had determined that the preferred location of a water intake for a power plant located at Harry Lake was in the Thompson River upstream of the confluence with the Bonaparte River, geotechnical studies were carried out to determine if this reach of the river was geologically suitable for an indirect intake (specifically, radial wells).

According to a report of 29 July 1977 by E. Livingstone Associates, Consulting Groundwater Geologists (Appendix 9), the most feasible location for radial wells in the vicinity of Ashcroft would be close to the confluence of the Thompson and Bonaparte Rivers. As recommended by Mr. Livingston, a borehole was drilled near the confluence of these rivers taking both strata and water samples. The results of this program are given in Golder Associates Report, Geotechnical Evaluation of Intake Site 10 (Appendix 16), and Beak Consultants letter of 28 November 1977 (Appendix 10). E. Livingston Associates were subsequently provided with copies of the Golder and Beak reports and their report of 21 December 1977 is attached as Appendix 9. Livingston's report states that "the chance of locating an aquifer capable of yielding 1580 l/sec...... anywhere in the Ashcroft area is extremely remote". As the geological conditions do not favour an indirect intake, the selection of a direct intake for this project was confirmed.

Site of Direct Intake

To facilitate a possible pipeline route within the proposed access road right-of-way along Cornwall Creek, the technical feasibility of an intake site (Site 16) downstream of Ashcroft between the 105 mile Post Indian Reserve 2 and Cheetsum's Farm Indian reserve was reviewed. Project Memorandum V4191/2* shows the location of Site 16 and evaluated it with respect to geophysical, geotechnical and fluvial considerations and compared Site 16 with Site 10 - the site selected during the Conceptual Design as the most feasible location for a water intake -- and concluded "That site 16 does not offer any intake location that could be considered viable for the Hat Creek Project". Therefore, a direct intake located at Site 10 was selected.

World Precedents for System Configuration

Only a limited number of projects in the world have pumping heads and discharge rates similar to the Hat Creek cooling water supply project. A list of some of these relevant projects is presented in Table A, Appendix 4. In these projects, the great variety of solutions reflects the complex nature of the problems faced. Some facilities, such as at Caracas, break down a high pumping head by using multiple pumping stations whereas others such as Lornex, Sar Cheshmeh and Edmonston, use a single pumping station for the entire lift. Considerations such as operational criteria, earthquake conditions, pipe sizes and friction losses have affected these solutions. These considerations make it impossible to

*For this and other project memoranda, see Appendix 8, (Volume 2).

formulate fixed rules for determining a system configuration based on other installations.

However, there is great value in knowing what has been done elsewhere and how that compares to the Hat Creek project. The total static head at Hat Creek, 1080 m (3550 feet), is well below that encountered on, for example, the Trans-Ecuadorian oil pipeline. The discharge rate of 1.58 m^3/s (25,000 USGPM) is low compared to many of the projects listed. Therefore, there are several good precedents from which a Hat Creek scheme may draw information and experience.

Optimal Route and Pumping Schemes

Project Memorandum V4191/4, System Design, provides a detailed description of the process of selecting the route and pumping configuration.

As discussed above, Intake Location 10, near the confluence of the Bonaparte and Thompson Rivers, was selected as the starting point for the pipeline. The power plant water reservoir in the Trachyte Hills above the Hat Creek Valley is the terminal point. The total static head for the proposed pipeline is about 1080 m (3550 feet) and the problem was how best to pump 1580 l/s (25,000 USGPM) of water over the 24 km (15 mile) distance.

The various routes studied, A through D, are shown on the Illustrations attached to Project Memorandum V4191/4. The potential pumping station locations, up to 5 in number, are shown marked on Route C by an "X". Schemes involving a range of pumping lifts, at any or all of these locations, were studied.

The comparison between schemes was made on the basis of total capital cost and present worth of operating costs over expected plant life. High costs for pumping stations tended to make schemes with more than three pumping installations (including the intake) uneconomic.

As discussed in the Project Memorandum, the most economic scheme is as follows:

1. Pipeline to follow Route C, the Conceptual Design Route.

2. A 365 m lift from the intake over Elephant Hill to the Boston Flats area.

3. A 945 m lift from the Boston Flats area to the power plant reservoir.

4. An 800 mm (32 inch) pipeline diameter throughout.

Selected Route and Pumping Scheme

Despite these results, factors other than economics entered into the selection process and altered the configuration from this optimal scheme. Consideration of pump wear, as detailed in Project Memorandum V4191/17, Pumps and Pump Wear, eliminated the use of high lift intake pumps and led to rejection of the most economic scheme. The next most economic scheme, which was to use low-lift intake pumps and a single high-lift booster station, was eliminated in favour of a scheme also using low-lift intake pumps but with two equal lifts of booster pumping. This scheme was selected for Preliminary Design because of the increased availability of a thinner wall pipe, the numerous precedents for pumping at a lower head for the required discharge rate and the ability to use conventional pipeline construction and welding methods.

The pipeline route selected for preliminary design is a combination of Route C (the conceptual design route) and Route A (the transmission line corridor). Although the selected route is based on the corridor-combination concept rather than the minimum cost concept of the shorter Conceptual Design Route, aerial reconnaissance of the selected Preliminary Design Route indicates this route does not contain as many steep rocky sections as the Conceptual Design Route. Thus, pipeline construction would be more conventional and could be scheduled with greater confidence. The equal lift pumping scheme is shown on the route profile, Drawing D4191-15. Table 1 summarizes the criteria of the selected system configuration.

Table 1 - Summary of System Configuration

Item	Amount (S.I.)	Amount (Imperial)
Design Discharge Rate	1580 l/s	25,000 USGPM
Average Discharge Rate*	663 l/s	10,490 USGPM
Maximum Elevation Difference		
- Intake to No. 1 Booster Station	36 m	118 feet
- No. 1 to No. 2 Booster Station	544 m	1785 feet
- No. 2 to Power Plant Reservoir	529 m	1736 feet
Maximum Difference	1083 m	3553 feet
Pipeline Length		
- Intake to No. 1 Booster Station	0.7 km	0.4 miles
- No. 1 to No. 2 Booster Station	9.3 km	5.8 miles
- No. 2 to Power Plant Reservoir	13.5 km	8.4 miles
Total	23.5 km	14.6 miles

Open System Versus Closed System

An open pumping system has tanks open to atmospheric pressure on the suction side of pumping stations to effectively isolate each stage of pumping. Water is pumped into the tank and flows by gravity from the tank to the inlet of the next set of pumps. Conversely, in a closed system, the first station would pump through the pipeline directly into the inlet header of the pumps at the next station.

Whereas the multiple pumping stations of oil and gas pipelines are often closed systems because the products are inflammable or volatile, water pipeline systems are usually open systems. A few examples of open water pipeline systems given in Table A, Appendix 4 are:

- Tijuana water supply, Mexico (Reference 9).
- Caracas water supply, Venezuela (Reference 10).
- Lake Huron water supply, Canada (Reference 11).
- Boulder City water supply, U.S.A. Reference 12).
- Maracaibo Water supply, Venezuela (Reference 13).

*Amounts given used in Preliminary Design Study. Average Discharge Rate since revised to 726 l/s (11,500 USGPM).

**In the late stages of preliminary design, the possibility of locating the proposed 500 kV transmission line several miles south of Ashcroft was introduced. However, this study assumes that the transmission line follows the route shown on Drawings D4191-13 and -14.

The reasons that water pumping facilities are usually open systems are:

1. Each section of pipeline is protected from the total system static pressure. In a closed system, the pipe would be subjected to the total static pressure if valves leaked.

2. The pipeline does not need to be designed for the full shutoff head of the pumps, as would be experienced if the downstream pump station valves closed while the upstream station continued to pump.

3. Self-regulation can be used to match pumping station discharges, as described in the following section. This avoids wasting energy by throttling or by-passing flow.

4. Waterhammer control problems are reduced.

One disadvantage of the open system is that a tank is required, which must be provided with overflow facilities and, for the Hat Creek project, protected from freezing.

Considering the above reasons and the recommendations of waterhammer specialists* Parmakian, Streeter, and Wylie; an open system has been selected for preliminary design.

To provide both the open system and to smooth flow variations between pumping stations, an open tank is provided between each set of pumps. The open tank at No. 1 Booster Station would be the clearwell following the degritting clarifier and would have a capacity of approximately 300,000 liters to provide a maximum rate of level change of 2.0 meters per minute with all intake pumps running and no booster pumps running.

The open tank at No. 2 Booster Station (called the equalization tank) also provides self-regulation of discharges from pumping stations No. 1 and No. 2 by variation of water level. For example, if the discharge from No. 1 station exceeds that from No. 2, the water level in the equalization tank rises. As a result of the level change, the pumps in No. 1 station work against an increased head which reduces their discharge. Simultaneously, the pumps in No. 2 station are subject to a greater inlet pressure which increases their discharge. With a suitable tank height at the right elevation, the discharges from the two stations will equalize under all operating conditions.

THOMPSON RIVER INTAKE

Selection of Intake Location at Site 10

In Sandwell's Conceptual design Report (Reference 2). Site 10 was identified as the prime site for a direct river intake to supply 1580 l/s (25,000 USGPM) of cooling water. This site is located on the right bank of the Thompson River just upstream of the confluence with the Bonaparte River, and is about 2.5 km upstream of Ashcroft.

*John Parmakian, previously with the U.S. Bureau of Reclamation, now a private consultant, Boulder Colorado; Victor Streeter, Professor Emeritus of Civil Engineering, University of Michigan; Benjamin Wylie, Professor of Civil Engineering, University of Michigan.

A helicopter survey of the site 10 area was conducted by Sandwell, Northwest Hydraulic Consultants Limited (the hydraulic consultant for this study) and Golder Associates Limited (the geotechnical consultant for this study) on 22 February 1977. On the basis of visual observations, five potential intake locations were identified in the section of the river from the vicinity of Ashcroft to about 3.5 km upstream of Ashcroft. These locations are referred to in this report as C, D, E, F, and G, and their positions are shown on Drawings D4191-5 and D4191-6. In assessing the merits of each location, the following characteristics were studied:

1. Hydraulics
2. Fish Protection
3. Construction Cost
4. Location

Each characteristic studied was rated on a scale of one to ten, with a score of ten being assigned to the most desirable location. Weighting factors from a scale of one to ten were then applied to account for the relative importance of the four characteristics.

The parameters included under the collective characteristic, Hydraulics, consisted of a study of river depth at the intake face, river velocity, and level of turbulence. The details of this study are contained in NHCL's report "Evaluation of Intake Sites" (Appendix 11). The Hydraulics characteristic was assigned a weighting factor of nine.

The characteristic, Fish Protection, was also given a weighting factor of nine. The sites were rated as part of a study carried out by Beak Consultants (Appendix 15) to evaluate the proposed design of the water intake with respect to the protection of the Thompson River fish resources. Since the five sites were evaluated during an upstream migration period, a further study should be carried out during the downstream migration period of 1978 to determine whether the scores need to be adjusted.

The various items studied under the characteristic, Construction Cost, are listed in Table 2 and the total costs were given a weighting factor of six.

Table 2 - Construction Costs of Intake Locations

No.	Item	C	D	E	F	G
1	Intake Access Bridge	$140,000	$175,000	$165,000	$165,000	$145,000
2	Bonaparte River Submerged Pipeline Crossing & Bridge	-	310,000	-	-	-
3	Thompson River Submerged Pipeline Crossing	440,000	-	395,000	440,000	-
4	Excavation (Includes Real Estate)	5,000	25,000	5,000	145,000	305,000
5	Pipeline-Low Pressure (Excludes River Crossing)	795,000	760,000	315,000	310,000	430,000
6	Pipeline-High Pressure	1,380,000	1,380,000	1,380,000	1,160,000	1,160,000
	Contingencies	130,000	190,000	340,000	460,000	500,000
	Total Costs	$2,890,000	$2,840,000	$2,600,000	$2,680,000	$2,540,000
	Scoring based on Total Costs	9	9	10	9	10

Note: Only items which vary with locations are listed in the above table.

The fourth characteristic studied, Location, consists of a combination of the following parameters:

1. Distance from Eroding Cliffs

   These cliffs commence just upstream of the confluence of the Thompson and Bonaparte Rivers and extend along the right bank about 3.5 km upstream to the second CNR bridge. Details of surficial investigation of these cliffs are contained in Golder's Report, Stability of Cliffs (Appendix 12).

2. Geotechnical Aspects

   Golder carried out a geotechnical evaluation of intake locations C through G and a copy of their letter report is included in Appendix 13.

3. Relation to the Bonaparte River

   An intake upstream of the Thompson/Bonaparte confluence would not be affected by the relatively high suspended solids load which the Bonaparte contributes to the Thompson. A detailed study of this topic was carried out by Beak in their report, Sediment Characteristics of the Thompson

River and the Effects of Algae Growth on the Hat Creek Water Supply Systems (Appendix 18).

4. Supervision

An intake location on the right bank of the Thompson River, being nearer to the No. 1 Booster Station than an intake on the left bank, is desirable as this simplifies the supervision of the two units.

Each of the above four items was then assessed and a weighting factor of three was applied.

The scores assigned to the four characteristics were then tabulated and computed as shown in Table 3.

Table 3 - Evaluation of Potential Intake Locations

Characteristic	Weighting Factor	Score for Intake Location C	D	E	F	G
Hydraulics	9	69	72	57	45	48
Fish Protection	9	72	90	27	45	54
Construction Costs	6	54	54	60	54	60
Location	3	12	22	15	19	27
Total Score		207	238	159	163	189

This numerical analysis technique assigned the highest score to Location D and this location was therefore chosen as the prime location for the intake. While location C is presently the most desirable back-up location, its position in the table may change when the fish protection aspects are reviewed during a downstream migration period. This review of the fish protection scoring would not alter the selection of location D.

Water Levels

To obtain data on the relationship between water surface elevations and river discharges at Site 10, recording of water levels commenced in December 1976 and terminated in July 1977. Details of this survey are included in Project Memorandum V4191/3, Thompson River - Water Level Data. A program to monitor low river water levels and peak levels during the freshet of 1978 was reinstated in December 1977. The results of this survey will be issued as an addendum to this report.

Type of Direct Intake

For location 10-D, there are two possible types of direct intakes - a bank intake or a pier intake. A bank intake is generally located on the outside edge of a river bend where depth and current are normally greatest, whereas a pier intake is located in a deep portion of the river not directly adjacent to the river bank thus requiring an access bridge. Except for the location and need for an access bridge, the pier intake is identical to the bank intake. The required distance from the shore to the face of a bank intake is determined by suitable river depths at minimum flow. For site 10-D, this distance would be 17 m (56 feet) at low water and 45 m (148 feet) at high water, or 23% and 35% of the respective total river widths. To construct a bank intake, the area behind the intake would be filled such that the shoreline is brought out to the face of the intake. Alternatively, the shoreline could be left in its original location and the structure connected to the shore by a bridge. Although this alternative is referred to in this report as a pier intake, it does not exactly fit the definition of a pier intake, as its location is not different from the bank intake.

Layouts and cost estimates for two bank intakes (Alternatives 1 and 2) and one pier intake (Alternative 3) at Location 10-D were prepared and are attached as Appendix 4E. In summary, the cost of the items which are not common to all alternatives are:

Alternative 1, Bank Intake:	$ 850,000
Alternative 2, Bank Intake:	1,000,000
Alternative 3, Pier Intake:	640,000

The pier intake alternative was selected for intake location 10-D for the following reasons:

1. Based on the cost comparisons, the pier intake would be more economical than the bank intake.

2. The hydraulic impact on the river from the pier intake would be minimal whereas the bank intake would remove an unacceptably large portion from the original river cross section.

3. In Beak report "Fish Protection Aspects" (Appendix 15), page 14 states that:

   "The offshore location of this pier-type intake is clearly superior to intakes along a river bank. This design acts to protect small steelhead, chinook, coho, and other fry which may occupy territories in river margin. This also allows passage of upstream migrants around the facility in the relatively shallow water which they commonly use".

<u>Geotechnical Investigations</u>

Once the intake had been established as Site 10-Location D, a geotechnical survey of the substrata in the vicinity of this site was carried out. This investigation consisted of drilling three boreholes in the positions shown on drawing D4191-5 to determine the bedrock level and the characteristics of the overburden. Water samples were also taken and permeability tests carried out to provide data for the indirect intake study discussed earlier in this report. The drilling was supplemented by a seismic survey along the four lines shown on the drawing to determine the bedrock surface between the drill holes. Both the drilling investigation and the seismic survey are detailed in Golder's report, Geotechnical Survey of Intake Site 10 Location (Appendix 16) which concluded that the shaley mudstone bedrock surface has a fairly consistent elevation of approximately 284 metres. At the location of the intake it is expected that the surface of the bedrock is about 3 m below the bed of the river. The material above the bedrock in the river bed is likely to be sand, gravel, cobbles and boulders.

To provide further information on the river bottom profile in this area and the type and size of material deposited there, an underwater survey was carried out by Swan Wooster Engineering, details of which are included in their report, Underwater Survey of Intake Site 10-Location D (Appendix 14). This survey provides useful data on the preparatory work which would be required on the river bed before construction of the intake.

The data obtained from these geotechnical and underwater surveys have confirmed the feasibility of constructing a pier intake at Site 10-D.

Biological Considerations

The major biological consideration of any water intake on a river is the fish life. In their report, Fish Protection Aspects of Water Intake Design for the Hat Creek Project (Appendix 15), Beak Consultants identified and described the life history of the key fish species of the Thompson River and investigated the behavioural aspects of these key fish species with respect to the proposed water intake. Although Beak concluded that both the design and the location of the proposed intake are good from a fish protection standpoint, they recommended a number of refinements to the design. These recommendations and Sandwell's response are shown on Table F, Appendix 4.

In their report, Suspended Sediment Characteristics of the Thompson River and Affects of Algae Growth on the Hat Creek Water Supply System (Appendix 18), Beak concluded that although certain types of algae will occur in great abundance in the Thompson River, they are not expected to foul the travelling screens or clog the trash racks.

Suspended Solids

In their report, Suspended Sediment Characteristics of the Thompson River and Effects of Algae Growth on Hat Creek Water Supply System (Appendix 18), Beak established that suspended sediment concentrations during the low flow period from November to April were in the order of less than 0.1 mg/l to 2.0 mg/l. However, during the rising freshet (May-June), suspended sediment concentrations may reach as high as 91.0 mg/l as reported by Northwest Hydraulic Consultants Limited in 1976.

A survey of other river intakes indicated that only where there was some type of settling area in front of the intake did the intake pumps have a reasonably long life. Those intakes which have been placed directly in the river (Cariboo Pulp and Paper at Quesnel and the Ashcroft Municipal Intake) have experienced a significant amount of sediment accumulation in the pump cells and wear on the intake pumps. The basic concept of the proposed water intake for the Hat Creek Project requires it to be in the stream of the river without benefit of a settling basin in front of the intake and, therefore, the intake would be designed to handle occasionally high concentrations of suspended solids.

Ice Conditions

In their report, Ice Conditions (Appendix 19), Northwest Hydraulic Consultants concluded that ice in the Thompson River does not present any major hazard in terms of ice forces or ice jamming although the intake structure must be designed for ice loads. The most significant operating problem would come from large quantities of frazil or slush ice which occasionally exists in the river. The travelling screens would require well-designed cleaning and backwashing systems to prevent plugging. A conveyor belt system rather than a simple sluice is recommended to dispose of large quantities of frazil ice taken up on the screens. Heating of critical gate slots, etc. will be essential.

Conceptual layout

The previous discussions of biological considerations, suspended solids and ice conditions have established some of the criteria which the layout of the proposed intake must accommodate.

The major component of a water intake is the screening system. The preferred location of the screening system is at the face of the intake. See Drawing D4191-9. With travelling screens, it is not feasible to mount the screen at the face of the intake because the above-water portion of the screens requires protection against ice, freezing temperatures, and debris. This protection is provided by a concrete curtain wall in front of the screens. However, fish may be trapped between this curtain wall and the screens. The proposed intake concept minimizes the possibility of entrapment of fish by providing a by-pass flow through the channel between curtain wall and screens, such that this channel becomes part of the river, permitting fish which have entered this channel to return to the river. The upstream end of the intake would be provided with an approach section having trash racks extending the full depth of the intake pier, allowing river flow to enter the channel over the full river depth.

Environment Canada regulations stipulate that approach velocity to the travelling screens shall not exceed 0.12 m/s (0.4 fps). For the minimum river water level at Site 10D, Sandwell selected the 1-in-100 year low water elevation. The stage discharge curve in Northwest Hydraulic Consultants report, Evaluation of Intake Sites (Appendix 11), estimates the 1-in-100 year low water level to be 289.20 m. Elevation 289.00 m, 0.20 m lower than the estimated value, was selected as the design minimum water level.

To prevent river solids from settling in the by-pass channel and restricting the flow, the bottom of the channel has been sloped away from the screens to direct settling solids back to the river as shown in Section D-D on Drawing D4191-10. This geometry places the top of the travelling screen boot at elevation 287.90 m, 1.70 m above the river bottom elevation of 286.20 m. This leaves a water depth on the screens of 1.10 m at design minimum river level.

The design flow rate through the travelling screens is 1738 l/s (27,500 USGPM) which is the sum of the system design capacity of 1580 l/s (25,000 USGPM) plus the following two 5 percent allowances:

- 79 l/s (1,250 USGPM) for intake pump wear and process losses such as degritting clarifier waste, pump seal water and travelling screen spray water. For details, see Project Memorandum V4191/17, Pumps and Pump Wear (Appendix 8).
- 79 l/s (1,250 USGPM) for increased flow through the screens due to reduced pipe friction when the minimum number of intake pumps are operating to feed one booster pump. See Project Memorandum V4191/14, Pumping System - Intake to Clarifier (Appendix 8).

The required travelling screen width W, is determined by the design flow through the screens Q, the minimum water depth at the screen d, the maximum approach velocity and a total screen efficiency coefficient of 0.883 in the following formula:

$$W = \frac{Q}{0.883 \times d \times v} = \frac{1.738 \text{ m}^3/\text{sec}}{0.883 \times 0.12192 \text{ m/sec} \times 1.10 \text{ m}} = 14.68 \text{ m } (48.2 \text{ ft}).$$

Leading manufacturers fabricate their travelling screens in incremental widths of 0.5 ft (0.15 m) up to a maximum width of 14 ft (4.27 m). There could be a minimum of four travelling screens, each 12.5 ft (3.81 m) wide. However, in this Preliminary Design Study, the screen width has been limited to approximately 10 ft (3.05 m) for the following reasons:

- To reduce the chance of non-uniform flow through the screens.

- To reduce potential vibration problems caused by by-pass flow.

- To allow for the installation of the approximately 10 ft (3.05 m) wide, flush face travelling screens presently being developed by a leading manufacturer.

The selected screen would be 10 ft (3.05 m) wide as this is the nearest size to the required width W of 48.2 ft (14.68 m) when utilizing five screens. The resulting intake cell width would be 3.40 m (11.2 ft), 0.35 m (1.2 ft) wider than the screen width to accommodate the screen side panels.

Hydraulic Model Studies

To confirm the concept of providing a by-pass flow through the intake, a hydraulic model of the previously-described conceptual layout was constructed and tested by Northwest Hydraulics Consultants Limited. The results of this investigation are contained in their report, Hydraulic Model of Intake (Appendix 17). The 1:40 hydraulic model resulted in an intake arrangement that provides acceptable sweeping velocities for all foreseeable operating conditions. Also investigated in this model study were sedimentation and trash accumulation aspects.

The original orientation of the intake approach section, full-depth inlet parallel to river flow, was found to be deficient in that by-pass flow was not obtained. When the full-depth inlet was oriented slightly into the river flow, satisfactory velocities were obtained even at minimum river flow. The most acceptable solution was obtained with all trash rack bars oriented into the flow at 45°. This intake configuration was adopted and successfully demonstrated to the various fisheries authorities.

Tests on sediment accumulation confirmed that the sloping sill in the by-pass channel would prevent accumulation that would be detrimental to the operation of the intake. The tests for trash accumulation showed that debris (normally surface-carried) is more likely to collect on the full-depth inlet of the approach section than on the low-level inlets which are submerged.

Site Layout

Once the location of the intake had been established and the configuration of the intake confirmed by the hydraulic model studies, the orientation of the intake in the river had to be established together with the inter-connection of the intake and other elements of the cooling water supply system.

The pier intake at site 10-D would be placed almost parallel to the river flow as shown on drawing D4191-7. This orientation would provide sufficient flow along the face of the screens to convey debris and guide fish past the intake.

Access to the intake would be provided as shown on Drawing D4191-22. The existing Bonaparte River road bridge, approximately 0.3 kilometres upstream of the CNR Bonaparte Bridge, could not carry the heavy construction traffic associated with this project and would therefore be replaced with a new bridge. The existing dirt road from the bridge to the CNR tracks would be upgraded and a new section of road would be constructed parallel to the CNR tracks from the existing right-of-way to the intake access bridge. A level track crossing would be provided at a location which would be sufficiently downstream of the existing overhead transmission lines to satisfy all regulatory requirements. Real estate negotiations would be required with the CNR and with the owners of lots adjacent to the CNR tracks.

Drawing D4191-22 also indicates the location of two new security fences, one of which would replace the existing fence bordering the CNR property.

Access to the intake from the river bank would be provided by a two span bridge which would also support the 900 mm (36 inch) pipeline from the intake to the clarifier. The selection of the diameter of this pipeline is discussed in Project Memorandum V4191/14, Pumping System - Intake to Clarifier. The section of pipeline on the bridge would be insulated for freeze protection while the balance of the pipeline would be buried. The pipeline route would generally follow the CNR tracks and cross underneath the Bonaparte River.

General Arrangement

The general arrangement of the intake is shown on Drawing D4191-9. The 45 m long pier includes 6 intake cells (one of which is a spare), a rounded upstream approach section and a rounded downstream end. The operating floor, placed 2.8 m above maximum river water levels, would be cantilevered 1.5 m over both sides of the pier and would accommodate travelling screen housing, pumps, electrical room and an equipment unloading bay. The entire floor would be serviced by a 20 tonne overhead travelling crane.

Trashracks and Trashrack Rakes

All inlets to the by-pass channel would be equipped with trashracks to exclude debris. To avoid blockage caused by any debris which might enter the by-pass, the outlet would not have racks. The racks in the approach section would extend full depth from sill to operating floor whereas the low-level racks in front of the intake cells would extend only to the underside of the curtain wall which is 28 cm below the 1-in-100 year minimum water level. Openings in the operating floor as shown on Drawing D4191-10 would be provided so that the travelling crane can be used to service the trash racks.

The low level racks would be parallel to the river flow and therefore not prone to collecting debris. The full-depth racks in the approach section would be more prone to collect trash as confirmed in the hydraulic model (Appendix 17). Accumulation of trash, however, is not likely to be a problem for the following reasons:

1. The Thompson River is basically a clean river.

2. Trash, if any, would mainly occur during the freshet when accumulation on the full-depth inlet trash rack would least interfere with the required by-pass flow. As most of the trash is floating, the low-level inlets would only be exposed to submerged debris.

3. The 23 cm spacing between trash rack bars would allow passage of all small debris.

4. The Lornex intake, located about 24 km downstream from the proposed Hat Creek intake, operates without problems of trash accumulation at the racks.

5. Based on Beak's report on the effects of algae growth (Appendix 18), algae are not expected to clog trash racks.

Although the accumulation of trash is not considered a problem, possible future addition of a mechanical rake at the full-depth inlet trash rack was investigated.

A standard mechanical rake lifts any accummulated trash up the racks and dumps it in a bin on the operating floor. Alternative 1 on Drawing D4191-8 shows the intake arrangement with a standard mechanical rake placed outside the intake housing with access bridge placed opposite the rake for ease of trash removal.

Alternative 2 on Drawing D4191-8 shows a custom-designed rake operated by means of a mechanism placed inside the pier. Rather than collect trash, the objective of this rake would be to dislodge trash such that it is swept downstream by the river flow. Table B, Appendix 4 compares the advantages and disadvantages of the two alternatives.

Although the custom-designed rake described as Alternative 2 cannot be considered proven technology, preference is given to this alternative because the guides and mechanism of a standard mechanical rake are designed to operate in an approach channel and are not suitable when exposed to a river flow parallel to the rake.

Upstream migrating fish could easily enter the unobstructed by-pass channel outlet on the downstream end and swim to the by-pass inlet. To ensure passage of the largest expected fish through the full depth trash rack at the approach section, the three bars located closest to the inside face of the by-pass would be spaced at 30 cm instead of the standard 23 cm. To minimize the chance of debris entering the by-pass through this increased bar spacing, an alternative arrangement could be made whereby the larger bar spacing would be provided in an extended rack placed parallel to the river flow. The arrangement is shown in Detail X on Drawing D4191-11. To ensure adequate downstream velocities through this extended by-pass, this refinement would have to be modelled during Final Design.

Travelling Screens

Five of the six intake cells would be equipped with travelling screens. The sixth cell, provided as a spare, would be isolated from the by-pass channel by stop logs. Ideally, the face of the travelling screens should form a continuous flush surface with the face of the walls dividing the intake cells. This could be accomplished by installing so called "flush face" travelling screens, presently being developed by leading screen manufacturers. However, as these screens have not yet been proven, standard vertical travelling screens are proposed in this preliminary design. To accommodate discontinuities caused by the sides of the trays on the standard screens, the screen face would be set back and the corners of the dividing walls would be rounded as shown on Section A-A, Drawing D4191-10.

The screen cloth would be fabricated from stainless steel wires with 2.5 mm (0.1 inch) clear openings to comply with Environment Canada requirements. As the travelling screens would operate behind fixed trash racks and be exposed to the sweeping action of the river, little debris is expected to collect on the screens. If necessary for trash removal, the screen baskets could be fitted with hooked lifting lips which would permit water to drain away from the screen to avoid entrapment of fish.

Cleaning of the screens would be by two rows of high-pressure water sprays at operating floor level. These sprays would also eliminate any build-up of algae on the screen cloth. Any debris carried up the screen would be returned to the river at the downstream end of the intake by means of a trough, a section of which would be equipped with a belt conveyor as shown in Section D-D on Drawing D4191-10 for fast removal of frazil ice which might be carried up on the screens.

Each screen requires approximately 30 l/s shower water at 600 kPa pressure. The demand is intermittent and water is only required when a screen becomes clogged. To avoid plugging and wear of the spray nozzles, clean water would be obtained by installing a 250 mm diameter gravity line from the clearwell following the degritting clarifier to a 60 l/s capacity booster pump located on the intake operating floor.

To ensure that the initial flow through the screen would be gradual and not adversely affect fish life, the intake pumps would be started against a closed discharge valve. When the pump has reached operating speed, the valve would be opened slowly. Furthermore, as pumps and screens are combined in integral units, the flow through any single screen could not exceed the design rate.

A travelling screen's effectiveness in blocking passage of salmon fry depends not only on the screen cloth, but to a great extent also on the condition of side and bottom seals. The screen cloth can easily be inspected in the screen housing at operating floor by rotating the screen. Special provision to inspect the seals would be provided as described in Project Memorandum V4191/10, Provisions for Inspecting Travelling Screens.

Intake Pumps

Since the Thompson River passes through areas susceptible to erosion and slides, it can have high concentrations of suspended solids. Solids entering the intake could pass through the intake pumps and cause severe wear if provisions were not made to minimize the effects of these solids. As pointed out in Project Memorandum V4191/17, Pumps and Pump Wear, the following design features must be incorporated into the intake pumps:

- Low rpm.
- Low head per stage.
- Abrasion-resistant materials.

For preliminary design, a speed of 900 rpm has been selected. During Final Design, suppliers should also be asked to quote on pumps operating at 720 rpm. Six suppliers have indicated that they could provide 900 rpm units while two indicated that 720 rpm units could be supplied.

The proposed intake pumps would be vertical, diffuser style, multistage units. Table 4 below gives typical specifications of these pumps.

Table 4 - Intake Pump Specifications

Item	Amount (S.I.)	Amount (Imperial)
Rated head	39 m	128 ft
Rated flow	330 l/s	5,250 USGPM
Speed	900 rpm	900 rpm
Number of Stages	1-3*	1-3*
Maximum Efficiency	80-86 %	80-86 %
Wear Ring Clearance	0.5 to 1.0 mm	0.02 to 0.04 in.
Bowl Diameter	600 to 700 mm	24 to 28 in.
Net Power	190 kW	250 hp

* Depends on supplier

Although larger clearances slightly reduce efficiency, they ensure longer pump life. Therefore, the radial and axial wear ring clearances should be as large as practical in order to pass most of the coarse particles expected in the water. Efficiency is not as critical for the intake pumps because power consumption is low compared to the booster pumps. The clearance specified for the intake pump wear rings should be at least 0.5 mm (0.020 in.). Both bowl and impeller wear rings would be included. The wear rings could also be fluted to pass larger particles with as little damage as possible.

The impellers would be the semi-open type which not only resists wear to a greater degree than the closed type but also can be adjusted axially for wear and can have replaceable bowl liners.

The metallurgy of the pump should be specified for its wear resistant properties. The wetted parts should be of very abrasion-resistant materials such as stainless steel for the impeller and bowl and stellite coated for the wear rings. Since every supplier specifies different materials, bids should be evaluated on the basis of material hardness as well as availability and cost.

A standard intake pump would usually have water lubricated bowl bearings of either bronze or rubber. These materials are adequate for pumps handling clean liquids but would not last where very abrasive solids are present in the pumped liquid. To prevent these solids from entering and destroying the bowl bearings, the intake pumps require a positive means of continuously purging the bearing surfaces with clean water. As pointed out in Project Memorandum V4191/17, Pumps and Pump Wear, this can be done by either using a rifle-drilled shaft or by using external piping to carry clean water to each bearing. Since the pressure required at the bowl bearings is higher than the intake pump discharge pressure, a small booster pump (rated at approximately 5 l/s and 600 kPa) is required to provide the flushing water to the bowl bearings. This water would be supplied by the 250 mm diameter line from the clearwell which also supplies the booster pump for the travelling screen shower water. During initial start-up, when the clearwell is empty, the bearing water booster pump would temporarily draw from the intake discharge header. The water for the bowl bearings would pass through filters, sized to trap any solids which could damage the bearings.

Each intake pump would exert thrust forces which would be taken up by a thrust bearing that would be built into either the motor or the pump. If built into the pump, a more expensive hollow-shaft vertical motor would not be required. Some pumps also have hydraulically balanced impellers which reduce pump thrust considerably. Thrust provisions vary with pump manufacturer, therefore, pump selection determines type of thrust bearing as well as type of motor. For the purpose of preliminary design, the thrust would be taken by the thrust bearing of the vertical hollow shaft electric motor.

The design capacity of the intake pumping system would be 5 percent greater than the system design discharge rate of 1580 l/s. The allowance has been added to cover miscellaneous services and wear as outlined in Project Memorandum V4191/9, Pump Design Allowance.

Drawing D4191-26 shows the variation in pump discharge flow with variation in river level and mode of operation. The pumps have been selected to meet the maximum flow demand at the most severe operating condition which would occur with minimum river level and all intake pumps operating. Therefore at river levels higher than the minimum, the intake pumps would deliver more than required by the booster pumps. This excess would overflow at the clearwell, back to the Thompson River as pointed out in Project Memorandum V4191/14, Pumping System - Intake to Clarifier.

To meet the suction head requirement, the intake pump inlet must be 1 to 1.5 m below the lowest water level. The intake pump discharge piping shown in Drawing D4191-10 would be carbon steel, ND 10 rating. The discharge header would be an extension of the 900 mm (36 in.) diameter line to the clarifier. Each pump would have a check valve and shut-off valve on its discharge. The valves would also be ND 10 rating with trim metallurgy chosen for wear resistance. The check valves would be the fully-opening type so as not to obstruct the flow. The shut-off valves would also be fully-opening gate or ball valves, sized to take the pressure drops encountered during start-up, and motorized to provide the controlled opening and closing rate. All fittings would be flanged for easy removal and each pump discharge would have a flexible connection to isolate it from the vibrations and movements of the other pumps and piping.

Other Mechanical Provisions

Slide gates are sometimes installed in the walls between the pump cells to by-pass a travelling screen when it is isolated by stop logs for maintenance. With one exception, these gates have been purposely left out to preclude the possibility that more than one pump could draw water through one screen, an operating condition which could lead to screen approach velocities in excess of the stipulated maximum of 0.12 m/s (0.4 fps). The exception here is a gate in the wall between the spare cell and the adjacent downstream cell, see Sections E-E and F-F on Drawing D4191-10. This gate would be provided to comply with Environment Canada's request to test a stationary screen in the spare cell as described in Project Memorandum V4191/11, Provisions for Testing a Stationary Screen.

To help provide frost protection to the inside of the structure, curtains would be provided at the upstream and downstream ends of the by-pass channel (see Drawing D4191-11) to prevent cold air from entering the intake. Each curtain would consist of two equal-length panels.

The lower panel would be electrically driven and would automatically follow the water surface. Both panels would be raised above operating floor level following the end of freezing weather until commencement of frost in the fall. Inspection covers upstream and downstream of each gate, would be provided at floor level.

Since solids less than 2.5 mm (0.1 in.) in size could pass through the mesh of the travelling screens, deposition may occur in the intake cell sumps. To facilitate solids removal, a trolley would be provided as shown in Section L-L on Drawing D4191-11. All sumps should be inspected annually after the freshet to determine the level of silt deposition. The solids depth, if any, could be determined with a rod. A portable solids discharge pump would be used to clean the sump.

Any items requiring storage in the intake would be kept at the downstream end of the structure adjacent to the wall opposite the main door. The travelling overhead crane would be used to raise the travelling screens for inspection of the lower section "in the dry". A 2 tonne capacity auxiliary hoist would assist in this inspection and other maintenance work..

Washroom facilities would not be provided in the intake as these would be located in the No. 1 Booster Station.

Heating and Ventilation Provisions

Space heating would be done by thermostatically controlled electric heaters. Components in the intake which would require heating are:

1. Screen cloth, only necessary when frazil ice present.

2. Screen seals, only when screen operation is necessary.

3. Trash racks, only necessary to protect against frazil or anchor ice.

4. Guides for curtains at the upstream and downstream ends of the by-pass channel, continuously for operation of lower panel.

5. Trash rack guides, only when rack removal required.

To prevent anchor ice from forming on the sloping concrete sill between the trash racks and screens, heating cables would be installed in ducts embedded in the concrete. The ducts and cables would be laid in duplicate to provide a factor of safety against cable failure.

With the exception of the screen cloth which is heated by radiation or hot air, all metal components requiring ice protection would be heated by thermostatically-controlled induction techniques to just above 0°C. Room ventilation would consist of package wall units and roof exhausters.

Electrical System

The power supply and distribution system to the intake is described later in this report. In addition to the major electrical equipment in the intake pumping station shown on Drawing D4191-9, the electrical room also would house transformers, panels, marshalling boxes, and control cabinets. A control panel containing all local controls and instrumentation would be located on the operating floor.

A 1.5 MVA 4160/600 V oil-filled power transformer would be located in a vault immediately adjacent to the electrical room and would be fed by a fully rated circuit breaker in the 4.16 kV MCC. This transformer has been sized at double the intake requirements so that only one stand-by unit would be required to serve all three pumping stations. In addition to the protective relaying shown on the single line diagram, Drawing D4191-28, there would be sensors to alarm low oil level and high-temperature. The secondary of this power transformer would be connected to a 600 V MCC.

All 600 V starters would be fused-contactor type for full voltage (across-the-line) starting. Each starter would have its own control power transformer.

All motors and feeders connected to the MCC would be provided with standard industrial type of electrical protection.

The five motors directly coupled to the intake pumps would be 190 kW (250 hp) 900 rpm, 4000 V, 60 Hz, 3 phase asynchronous (squirrel cage) induction type. The motors would be vertical, flange-mounted and would have an open drip-proof filter ventilated enclosure. The service factor would be 1.0. Depending on the manufacturer, the sound pressure level with the motor running unloaded would be in the range of 84 to 90 dBA, measured one meter from the enclosure. Motor windings would have embedded resistance temperature detectors (RTD) which would be used for alarm purposes. The motors would have anti-friction bearings. Vibration detectors for motor protection would be attached. During times when the pumps are shut down, heaters located inside the motor enclosures would prevent corrosion due to condensation. Inquiries have confirmed that suitable motors are available from both national and international manufacturers.

All 4160 V starters would be fused air or vacuum contactor type for full voltage starting and would have instantaneous short circuit protection, thermal overload protection and zero sequence ground fault protection. Sub feeders would be protected by fully rated circuit breakers to avoid single phasing problems. All 4160 V motor feeders would be interlocked armoured cable.

The main metering for the intake pumping station, consisting of a kilowatt, voltage and current metering would be located in the No. 1 Booster Station. In addition, voltage metering would be provided in the incoming section of the 4160 V intake station MCC.

General indoor illumination would be provided by mercury vapour fixtures. Outdoor floodlighting for security purposes would consist of photo-electric controlled mercury vapour fixtures. The lighting panel would also feed 120 V convenience outlets placed throughout the structure. Emergency lighting units would be installed in the electrical room and other strategic locations. One 4160 V and one 600 V starter of each size used would be installed in each respective motor control centre to serve as a stand-by unit. An empty spare cubicle or structure at each MCC would also be provided.

Structural and Architectural

The intake would be founded on bedrock and up to and including the structural floor level, would be constructed from reinforced concrete. The superstructure would consist of a steel frame with precast concrete wall and roof deck panels. Sound and thermal insulation would be provided by lining the inside face of the wall panels.

Drawing D4191-12 shows elevation views of the intake and access bridge prepared by the Project Architects - Toby, Russell, Buckwell and Partners. The architectural concept is directed towards matching the shape, texture and colour of the structure to the cliffs in the immediate background.

Several access bridge alternatives were studied and a prestressed, precast concrete double span bridge was chosen to preserve uniformity with the superstructure cladding. The concrete side upstands would also provide a means of screening the 900 mm diameter pipeline which would be supported by the bridge at deck level.

Operation and Control

The operation and control of the Thompson River Intake is shown schematically on Flow Diagram D4191-2.

The travelling screens would have differential level detectors to measure the degree of screen plugging. If the differential exceeded a preset limit, the screen wash pump would be started. After confirmation that wash water is available, the appropriate screen would be started and its associated wash water valve opened. Since the wash water requirement is approximately 30 l/s per screen and the pump capacity is 60 l/s, the logic would limit the screen wash cycle to a maximum of two screens at one time. In the unlikely event that the debris was not removed and the level differential exceeded a preset limit, the pump would be shut down.

Intake pump start signals, received through the telemetering system from the power plant, would energize the pump starters and, if all of the local interlocks were satisfied, the pump would start. When the pumps reach rated speed, the pneumatically-operated pump discharge valve would open at an adjustable, preset rate. In the event that more intake pumps were running than were required by the No. 1 Booster Station, a visual indication of this would be displayed at the central control console.

Operating data such as river level, water temperature, valve position, etc. would be telemetered to the central control console in the power plant.

The following additional auxiliary equipment is contained in the intake structure and would be controlled as noted:

Item	Control Mode
Compressor	Automatic/Local Manual Override
Deicing heaters	Remote Manual
Space heaters	Automatic/Local Manual Override
Seal Pump	Automatic/Local Manual Override
Curtains	Automatic/Local Manual Override

The following local interlocks would be provided in addition to those interlocks normally provided with the equipment and would inhibit the start command from the central station:

Item	Device
Pump discharge valve not closed	Limit switch
Drain valves not closed	Limit switch
Pump protection devices	Switches
Air pressure low	Pressure switch

WATER-TREATMENT

General

The extent of water treatment considered here is only that necessary to facilitate the pumping of Thompson river water to the reservoir adjacent to the power plant at Hat Creek. Therefore, any treatment required in the power generating process is excluded from these considerations.

Both dissolved solids which may result in corrosion and suspended solids which may result in erosion have been considered.

Dissolved Solids

The amount of dissolved solids in a river water can be determined by qualitative and quantitive analysis of water samples. Analyses of dissolved solids in Thompson River water, given in B.C. Hydro's Station Design Manual on Make-up Water Quality of 28 June 1977, indicate a soft water (hardness of 28 to 48 mg/l as $CaCO_3$), low in dissolved solids (40 to 109 mg/l). These analyses, together with the fact that the municipality of Ashcroft uses untreated Thompson River water, are indicative of the acceptable nature of this river water. Therefore, special provisions for corrosion protection are not required as standard methods such as the coating of ferrous metals or the use of corrosion-resistant metals in pumps, valve and fittings are adequate.

Suspended Solids and Pump Wear

The vulnerability of a water supply system to erosion depends on the amount of suspended abrasives in the water and the relative velocity between the suspended abrasives and the components of the system. Components in this system which function with high velocities are pumps and valves. As the valves are normally either fully open or fully closed, the amount of time that these components are in a throttled position and exposed to high velocities is limited and erosion is therefore not a concern. The pumps, on the other hand, are operating continuously with high water velocities and are, therefore, extremely vulnerable to erosion or wear. If solids are anticipated in the water, pump suppliers can compensate to a certain extent to limit wear by selecting designs with low rpm, low head per stage and extremely hard materials. In addition, booster pump suppliers recommend the size of suspended solids be less than 200 to 300 micron (0.008 to 0.012 inch).

As described in Project Memorandum V4191/17, Pumps and Pump Wear, the combination of high head with relatively low friction means that a minor degree of wear could drastically reduce the system's rated design capacity. Protection of the booster pumps against wear due to river solids was, therefore, a prime concern in the selection of the system configuration. Although available Thompson River data indicate a maximum suspended solids loading of only 91 mg/l* observed during the 1976 freshet and an average of less than 10 mg/l during the remainder of any year on record, the necessity for protection against wear by means of solids removal was deemed necessary in the conceptual design because the proposed river intake would be located 900 m (3,000 feet) downstream of a zone of eroding cliffs which are a source of river solids throughout the year.

During the preliminary design phase, further work was carried out to obtain solids samples and data on the size distribution and the abrasive character of these solids. Obtaining representative samples of suspended solids is a difficult procedure. Even if accurate data were available, laboratory methods to determine the effect of these solids on pump wear do not exist. Therefore, operating data on actual pump installations along the reach of the Thompson River between Kamloops Lake and Spences Bridge was collected. These investigations revealed the existence of some irrigation intakes near Walhachin, in addition to the Ashcroft municipal intake and the Lornex intake.

*See Beak's Report on "Suspended Sediment Characteristics of the Thompson River" contained in Appendix 18.

With the exception of Lornex, all installations with more than one year of operation suffered from pump wear and required impeller replacements.

Thompson River solids are discussed further in Project Memorandum V4191/5, Water Treatment, which recommends that a solids removal system be included to protect the booster pumps against wear.

Treatment Alternatives and Selection

For a solids removal system to work properly and efficiently, it should be capable of:

1. Removing solids larger than 200 micron (0.008 inch).

2. Absorbing shock loadings.

3. Avoiding blinding.

4. Disposing of removed solids.

The removal system should be capable of continuous operation even under freezing conditions. It should minimize land and supervision requirements, water wastage, and wear, and to avoid treatment of waste prior to discharge, should not require chemicals.

The following solids removal systems were considered:

- Settling ponds.

- Coagulation clarifiers.

- Degritting clarifiers.

- Micro strainers.

- Drum filters.

- Media filters.

- Cyclones.

Of these systems, the degritting clarifier best meets the conditions set forth above. This selection was made after reviewing data on size distribution of river solids and the particle sizes acceptable to the booster pumps. The majority of particle sizes anticipated in the raw water are between 2.5 mm, the gap between the wires of the travelling intake screens, and 0.1 mm (see Table 4 in Project Memorandum V4191/5, Water Treatment)whereas particle sizes acceptable to the booster pumps are less than 0.2 mm.

A media filter would collect the majority of these particles but since a media filter is cleaned by means of a reversed flow whereby only particles smaller than 0.1 to 0.5 mm (depending on media sizes) can be back washed, most of the river solids would be trapped permanently. These solids could be back washed by increasing the reversed flow but this would also remove filter media - an unacceptable condition. A media filter, therefore, is not suitable in this application as it would gradually fill up with solids.

The ability of a degritting clarifier to settle solids without the addition of chemicals depends not only on its design but also on the nature of the raw water. Based on operating experience at the Ashcroft and Lornex intakes, solids in the Thompson River water appear to settle very easily. However, operating experience on other degritting clarifiers operating without chemicals should be obtained during final design.

Treatment Scheme

Drawing D4191-2, Flow Diagram - Intake to Clarifier, shows the location of the degritting clarifier in the process flow. Located between the Intake and No. 1 Booster Station, it protects all booster pumps against solids. The clarifier overflows into an adjacent clearwell. Both vessels are located on a 20 m high plateau behind No.1 Booster Station as shown on Drawing D4191-22. This elevated location provides the necessary suction head for the booster pumps plus an additional head of 3 m (10 feet) to provide for future gravity filtering of the degritted water to further reduce pump wear. The need for this additional treatment may not be apparent until after many years of continuous operation.

The concrete clarifier vat would be 30 m in diameter, have 4.2 m high side walls, be covered by a domed roof and provided with space heating to prevent freezing. Solids would be concentrated into the centre well of the sloping clarifier bottom by means of a rotating rake mechanism and returned to the river by a gravity-flow pipeline. Discharge would be regulated by a timer operated valve.

Suspended Organic Solids

The suspended river solids referred to above are inorganic - mostly river sand and silt. However, the Thompson River also carries suspended organic solids in the form of logs, twigs, leaves and algae. Except for the algae, most of these solids would be prevented from reaching the intake pumps by the intake trashracks and travelling screens. Those organic solids which would pass through the screens may not settle in the clarifier as some are lighter than water. Although these solids would pass through the booster pumps, this is not a concern as these solids are not abrasive.

The type and extent of algae growth in the Thompson River is discussed in Beak report, Suspended Sediment Characteristics of the Thompson River and Effects of Algae Growth on Hat Creek Water Supply Systems (Appendix 18). This report concludes that chlorination is not required to control algae growth and that the Thompson River algae would not cause any operational problems in the water supply system provided the inside of the clarifier and tanks are not exposed to sunlight.

BOOSTER STATIONS

General

The Booster Stations, No. 1 near the intake and No. 2 about 9.3 km along the pipeline, would house the major mechanical equipment - the booster pumps and motors.

As the pumping head and discharge would be equal for each station for the reasons discussed under SYSTEM CONFIGURATION, the layout and major equipment for each station is identical. This duplication reduces equipment and construction costs and simplifies maintenance. The layout allows future expansion of both the pumphouse and substation. The pumphouse could be expanded lengthwise and the substation could be expanded sideways.

The substation is discussed separately under POWER SUPPLY AND DISTRIBUTION.

Location - No. 1 Booster Station

No. 1 Booster Station would be located on the right bank of the Bonaparte River adjacent to the CNR railway as shown on Drawing D4191-22. The booster station is approximately 450 m downstream of the Thompson River Intake.

The site, including clarifier, clearwell, substation and booster station, would occupy approximately 3 hectares of land, presently privately owned. Except for clarifier and clearwell, the site would be surrounded by a security fence with locked gates at the access points.

Access to No. 1 Booster Station would require upgrading of an existing dirt road from the highway. A new road to connect the clarifier to the booster station would be constructed.

The operating floor of No. 1 Booster Station would be at elevation 301.0 m which is approximately 3 m above the 1 in 100 year flood level. As discussed under WATER TREATMENT, the clarifier and clearwell would both be located on an elevated plateau approximately 20 m above the operating floor of No. 1 Booster Station. The decision to use an elevated clearwell to provide the NPSH for the booster pumps is explained in Project Memorandum V4191/7, Suction Pressure for Booster Pumps. The clearwell would be an insulated steel tank approximately 8 m in diameter and 6 m high and would have an overflow capable of returning the output from five intake pumps to the Thompson River. The buried pipeline from clearwell to booster station would be 1200 mm in diameter.

Location - No. 2 Booster Station

No. 2 Booster Station would be located, as shown on Drawing D4191-23, 9.3 km along the pipeline from No. 1 Booster Station. The site would be at an elevation selected to equally divide the pumping head between the two stations. To accomplish this, the upper water level in the equalization tank (869 m) has been chosen so that with all booster pumps running and the reservoir at maximum full pool level, the total dynamic head is equally divided between No. 1 and No. 2 Booster Stations. The lower water level in the equalization tank (843 m) would occur with one pump running in each booster station and the reservoir drawn down. The floor of the equalization tank would be set at elevation 840 m. To obtain the 20 m suction pressure required by the booster pumps, the operating floor of No. 2 Booster Station would be set at elevation 820 m.

For estimating purposes, the tank diameter has been chosen as 14 m to provide a maximum water level change rate of 0.61 m per minute (2 ft per minute). The tank height of 35 m is considered suitable for a 10 percent discrepancy in rated discharge between the No. 1 and No. 2 Booster Stations and for the various operating discharge rates.

The equalization tank would be of welded steel construction, on a concrete foundation, with a dome roof, mushroom vent, access hatches, ladders, over-flow and drain pipe. Although the equalization tank would be insulated, during long shutdowns the tank would be drained to prevent freezing.

Both the elevation and size of the equalization tank should be reviewed during Final Design. The factors which will determine the equalization tank sizes are:

- The reservoir inlet detail as this influences the static head at the No. 2 Booster Station.
- The variation of rated heads and discharges between the booster pumps.
- The final waterhammer control facilities.
- The difference in friction head between the two sections of pipeline.

Although the equalization tank could overflow in any of the following circumstances, overflow would be annunciated at the power plant control room in order that corrective measures can be taken:

- If a power outage occurs at No. 2 Booster Station, while No. 1 continues pumping.
- If a pump discharge valve at No. 2 Booster Station fails to close and the backflow from the second section of pipeline passes through the pumps and into the equalization tank.
- If a greater number of pumps is operating at No. 1 Booster Station than at No. 2 Booster Station.

Due to self-regulation by the tank, overflow would not occur when an equal number of pumps are operating in each booster station.

The overflow pipe from the equalization tank would discharge into a concrete lined trench. This trench would direct the overflow to a storage basin of approximately 60,000 m^3 capacity created by construction of an embankment structure, with a culvert through the bottom. The culvert would provide steady drainage of normal rainfall preventing a standing pool from forming in the basin. When the equalization tank overflows, the culvert would be of suitable size so that only a portion of the rated discharge flow would pass through it. The water would then collect behind the embankment, and eventually drain through the culver to natural water courses at an environmentally-acceptable rate. A spillway would be provided to prevent damage to the embankment in case the culvert became blocked. If the culvert were completely blocked, the storage basin would provide 10 hours of storage capacity at the rated discharge.

The site, including equalization tank, booster station and substation, would occupy 2.5 hectares. The storage reservoir would occupy 2 hectares. Access would be via an extension to an existing road which would require upgrading to transport heavy construction equipment. As with No. 1 Booster Station, the station will be surrounded by a security fence.

<u>Layout</u>

The general arrangement for both No. 1 and No. 2 Booster Stations is identical and is shown on Drawing D4191-24.

There would be four booster pumps and motors per station. They would be aligned along the building axis with the suction and discharge headers parallel to the line of pumps and motors. This arrangement is the most practical from maintenance, access, construction, and cost viewpoints because, with the large diameter suction and discharge headers, any other layout would require a complicated piping arrangement. The suction header would be above the floor to eliminate any low spots where solids could accumulate. The 800 mm discharge header would be in a trench approximately 2 m wide by 2.5 m deep which would be covered except at each pump discharge line.

Each pumping unit, consisting of pump and motor, would have adequate access space for maintenance. The pumping units would have approximately 1.5 m of their length allotted between the pump and motor for a flywheel required for waterhammer control and discussed under Waterhammer Control.

An access aisle, running the entire length of the booster station to the loading bay, would also serve as an equipment removal aisle.

The maintenance shop, stores, loading bay, washroom and electrical room would be at one end of the station. Washroom facilities would consist of a wash basin and toilet. Access to the station and loading bay would be through a truck door at the end of the station.

A 10 tonne service crane would be provided to serve the loading bay and all equipment in the station including the electrical room and maintenance shop but excluding the suction header. The electrical room would be enclosed but removable ceiling panels would be provided to facilitate equipment removal. The crane would be an electric, pendant operated, low-overhead type with single hook and would have such features as inching speeds and jogging to facilitate equipment assembly.

Room ventilation would consist of package wall units and roof exhausters. Heating would be provided by electric unit heaters.

General indoor pumping station illumination would be provided by mercury vapour fixtures, while electrical rooms and low ceiling areas would have fluorescent fixtures. Outdoor floodlighting for security purposes would consist of photo-electric controlled mercury vapour fixtures. Emergency lighting would be installed in the electrical room and other strategic locations.

Booster Pumps

The booster pumps and their driving motors are the main mechanical components of the water supply system. Each pump would be rated at approximately 395 l/s (6250 USGPM) and 640 m (2100 ft) head.

Table 5 below gives the major specifications of the booster pumps.

Table 5 - Booster Pump Specifications

Item	Amount (S.I.)		Amount (Imperial)	
Rated Head	640	m	2,100	ft
Rated Flow	395	l/s	6,250	USGPM
Speed	3,600	rpm	3,600	rpm
Number of Stages	2 to 3		2 to 3	
Efficiency (optimum)	80 to 85	%	80 to 85	%
Wear Ring Radial Clearance	0.3	mm	0.012	in.

Many manufacturers offer standard designs of pumps of the size and style required. These pumps are used mainly for boiler feed service but have been adapted to such services as:

- Hydraulic debarking.
- Steel mill descaling.
- Petroleum pipelines.
- Waterflooding for oil industry.
- Water supply.

The above services all utilize some form of treatment to remove solids prior to pumping as is intended for the Hat Creek cooling water supply system. The pumps, being designed mainly for boiler feed service, are of proven technology.

As pointed out in Project Memorandum V4191/6, Number of Booster Pumps, four pumping units were determined to be the optimum number from cost and availability viewpoints of both pumps and motors. As shown on Table 1 of this Project Memorandum, a pump size of 395 l/s is quite common and there are many units operating in various services and at heads equal to or greater than required for the water supply system. There are 8 potential suppliers of the booster pumps representing most major manufacturers in the world.

One supplier, experienced in manufacture of water supply pumps, recommends 2 pumps per booster station. It should be noted that a pump of this size (790 l/s) can only be supplied and manufactured by this one manufacturer. This alternative could be studied in Final Design from an economic and system flexibility standpoint.

As shown on the performance curves on Drawing D4191-26, the pumps were selected to deliver 395 l/s against the maximum head of 640 m which occurs with the maximum system flow of 1580 l/s.

The components of total head (friction and static head) are not equal for each station, and are as follows:

	Station No. 1	Station No. 2
Friction Head (at design discharge)	100 m	133 m
Static Head	540	507
Total Head	640 m	640 m

The friction values are calculated from the Darcy-Weisbach formula as explained in the section, PIPELINE DESIGN.

Each pump would be rated at exactly one quarter of the design flow requirement. As noted in Project Memorandum V4191/9, Pump Design Allowance, many tolerances and allowances are already built into the system and no further allowance should be included for wear. Although a spare rotating assembly would be provided and stored at the power plant warehouse, it is felt that a spare installed unit would not be required because the storage reservoir has a capacity of 70 days which should be adequate time to perform major pump and motor repairs.

The booster pumps would be either volute or diffuser style units depending on supplier since both types can meet the head and flow requirements. The decision on pump type would be based on cost and efficiency rather than style since there is little difference in their performance or reliability.

The pump would be of a single case construction and would have 2 or 3 stages, depending on supplier. It should be noted that from a wear standpoint, 3 stages would be preferable because of the lower head per stage.

To prevent cavitation, the NPSH (net positive suction head) requirement must be satisfied at the most critical operating condition which occurs when only one pump is operating. The maximum NPSH requirement varies from 18 m to 41 m according to pump manufacturer based on double suction first stage. Although No. 2 Booster Station could be located at a suitable elevation to permit the use of a single suction first stage, only 20 m of static head is available for No. 1 Booster Station. This corresponds to an available NPSH of 28 m which does not permit the use of a single-suction first stage but meets the requirement of all but two booster pump suppliers for double-suction first stage. To reduce spares and improve interchangeability, all booster pumps would be identical and would be double-suction first stage.

Booster pump efficiencies quoted by suppliers range from 80 to 85 percent. Since one percentage point in pump efficiency has a present worth of $300,000 at 20 mill/kWh, the system must be designed to pump most frequently at or near its B.E.P. (Best Efficiency Point). To obtain maximum efficiency, suppliers can, by choice of impeller design or trim, shift the location of the pump B.E.P. In evaluating tenders, the guaranteed efficiency and location of B.E.P. should be taken into account to make a fair comparison of all bidders.

For the pump efficiency curve shown on Drawing D4191-26, the efficiency drops from the B.E.P. with less than four pumps operating. This is acceptable provided that the desired mode of operation is with all four pumps operating. The selected pumps should have the B.E.P. specified after the mode of operation has been determined. If the mode of operation is not known at the time of ordering, then individual efficiency points should straddle the B.E.P.

Since the maximum efficiency greatly depends upon the wear ring clearances, care must be taken to ensure that quoted efficiencies are based upon clearances specified in the purchase inquiry.

Although the booster pumps will be directly coupled to nominally 3600 rpm motors, the actual pump speed is about 3575 rpm at full load. To minimize wear, a pump speed as low as practically possible is desired. Only one supplier could submit a proposal for a pump operating at 1800 rpm and pumps of the size required for 1800 rpm(larger and more stages) have not been manufactured to date.

Although 1800 rpm motors are more readily available than 3600 rpm motors, they would require speed increasers to bring the pump speed to 3600 rpm. The use of speed increasers is not favoured because of high capital cost, extra maintenance problems, and power loss. A suitably-sized gear unit would have an efficiency of approximately 98.5 percent and the cost of the power loss in gear units would have a present worth of $450,000.

Therefore, because of the high rpm required for the booster pumps and the resulting effect on wear, it is absolutely necessary that water treatment be provided prior to the pumps to ensure that reasonable pump life can be obtained. As explained earlier, a degritting clarifier would remove the suspended solids.

As the wear ring clearance increases, efficiency drops. Therefore, a wear ring radial clearance of 0.3 mm (.012 inch) should be specified to ensure that wear due to solids will not be excessive. Although this is slightly larger than the standard wear ring clearance and produces air efficiency decrease of about 1 percent, it is felt that excessive wear would result in more down time if a smaller clearance is specified.

The clarifier would be designed to remove particles larger than 200 microns which should be relatively easy. The pumps would be expected to pass particles smaller than 200 microns without excessive wear.

The metallurgy of the wear rings is as critical as the radial clearance. It must be sufficiently hard to resist wear and of suitable metallurgy to resist corrosion by river water yet still be readily available. As with intake pumps, the final metallurgy selection would be based on hardness, availability and cost.

Mechanical seals or throttle bushings would be specified for the booster pump stuffing boxes. They would be of a type utilizing clean flushing water for purging. The seal water, approximately 0.5 l/s per pump, would be tapped off the first stage of the booster pump. The water would be cleaned by in-line cyclonic separators before passing to the seals. Since the stuffing boxes are designed to operate at approximately 100 kPa over suction pressure, an orifice would be required to reduce the pump pressure.

The thrust bearing would be either a pivot-shoe or Michel-pad type, depending on supplier.

When the power supply to the pump motors is interrupted, the system is designed to allow reverse flow to the pipeline and subsequent rotation of the pumps and motors up to 120 percent of rated speed. Waterhammer pressures which are less than the pump shut-off head, would not cause concern. The pumps would have instrumentation to ensure that the bearings are still lubricated during reverse rotation. Pump bearings would be sleeve-type with forced oil lubrication. Couplings between motor and pump would be of gear type.

The sound pressure levels generated by the pumps within the station are estimated to be not more than 90 dBA at a distance of 1 metre. This level is adequate to meet the Workers' Compensation Board requirements for an 8 hour exposure as outlined in Section 13.21 of their Industrial Health and Safety Regulations.

The following items would be monitored to ensure that a pump would not operate during a malfunction:

- Oil pressure.

- Oil temperature.

- Seal water pressure.

- Casing temperature.

- Suction and discharge pressures.

- Vibration.

These items would be interlocked to stop the motor of the defective unit and could be annunciated at the central control station.

Motors and Electrical System

The motor rating for preliminary design, 3600 kW (4800 hp) is calculated for the maximum load condition which occurs when only one pump is operating. As shown on Drawing D4191-26, for average conditions of head at 560 m, flow capacity at 475 l/s, and efficiency at 76 percent, the average power requirement is 3400 kW. However, the higher rating motor has been selected to cover most of the values quoted by potential pump suppliers.

Each pump would be directly coupled to a 3600 kW (4800 hp) 3600 rpm, 4160 V, 60 Hz, 3-phase induction-type squirrel-cage motor which would be horizontal, footmounted and water cooled. The stator and rotor air cooling would be closed-circuit type. The service factor would be 1.0.

The sound pressure level of the motor at full load shall not exceed 90 dBA, measured one metre away from the motor. Sound absorbing hoods could be employed to obtain a lower sound pressure level if required for environmental reasons.

The motors would have sleeve bearings with forced oil lubrication. The bearings would be protected by resistance temperature detectors (RTD's) against overheating and could also have vibration detectors attached.

Motor windings would have embedded RTD's which would be incorporated into the protective relaying scheme for the motors.

During shutdowns, condensation would be prevented by heaters located inside the motor enclosure.

Water-cooled motors have been selected rather than air cooled. A forced-air cooling system on each motor consisting of a fan, filter, intake louvres and ducting would cost approximately $30,000 per motor while water cooling would cost approximately $15,000. The water-cooled motor would also take up much less space than an equivalent forced-air cooled motor. The water-cooled heat exchanger would dampen some sound, resulting in a quieter motor than the air-cooled type.

For preliminary design it is assumed that the additional inertia required for waterhammer protection would be provided by a flywheel situated between each pump and motor. During Final Design, motor suppliers should be asked if the additional inertia could be provided by increasing the rotor size thus eliminating the separate flywheel. The maximum allowable total inertia of the pump, motor and flywheel has been selected as the inertia the motor can accelerate from zero to full speed in 16 seconds. The flywheel does not affect the magnitude of the starting current and would consume a neglible amount of power.

Most well-known national and international manufacturers can supply motors of the required kilowatt and speed rating. Motors of the required power rating are not standard catalogue items but are custom designed and can be tailored closely to the requirements of the system and driven machine. Preliminary motor data received from several motor manufacturers showed the following typical specifications:

Items	Range
Efficiency at full load	96 - 96.8 %
Efficiency at 3/4 load	96 - 96.5 %
Power factor at full load	85 - 89.7
Power factor at 3/4 load	82 - 91.1
Locked rotor torque as a % of full load values	60 - 125 %
Pull-up torque	60 - 125 %
Break down torque	225 - 250 %
Locked rotor current	550 - 630 %

During Final Design when load requirements, energy costs and system voltage fluctuations are known, effects of variations in motor efficiency, power factor, torque-speed characteristics and locked rotor current would be evaluated.

For economic reasons, full-voltage starting (across-the-line starting) would be employed for the 3600 kW pump motors. In order to minimize voltage drop and other disturbances to the power supply system, the control logic would ensure that a pump cannot be started until the previously-started pump has accelerated to full speed. These large motors can only be started twice consecutively from a cold state and once only from a hot state after which the motor must be permitted to cool to the design ambient temperature of 40°C before attempting to restart.

A system study should be conducted by B.C. Hydro during final design to accurately determine the transient effects imposed on the B.C. Hydro power transmission system by the starting of these large pump motors in order to verify the feasibility of full-voltage starting. As it is not expected that the conclusions from this study will differ from this preliminary investigation, allowance for any form of reduced voltage starting has not been made in the cost estimate.

The 4160 V motor starters would be of the air circuit breaker type. Sub-feeders would be protected by circuit breakers to avoid single phasing problems.

Although not shown on the Single Line Diagram, the cost for a percentage differential protection scheme has been included in the estimate. The necessity of differential protection would be confirmed during final design.

All motor feeders would have running hour kilowatt and current metering.

An undervoltage relay connected to the switchboard would trip the four pump motors on loss of voltage to prevent a common automatic re-start of all motors upon restoration of the supply voltage. Sub-feeders would not be tripped.

Since the booster stations are electrically identical (with the exception of the additional intake feeder at No. 1 Booster Station), the following comments apply equally to both booster stations.

The electrical room layout for the booster stations is shown on Drawing D4191-24. The electrical room would be located adjacent to the power transformer end of the 69 kV incoming supply sub-station in order to keep the main 4160 V feeder cable lengths to a minimum. These feeder cables are terminated in the incoming section of the 4160 V switchboard, located in the centre of the switchboard in order to reduce the main bus bar size requirement

The electrical room would also house transformers, panels, marshalling boxes, and control cabinets.

A small battery room, accessible from the electrical room, would house the station battery to provide the 125 Volt DC-power required for tripping and closing the 69 kV main circuit breaker and the 4160 V starters.

Control panels containing all local controls and instrumentation would be located on the pump operating floors.

Motor protection surge arresters would be connected to the switchboard bus rather than to individual motors at their respective terminals. This is to avoid the complication of their inclusion in the terminal box. A review of this would be carried out during final design to ensure that the motors are adequately protected from switching surges.

600 Volt has been selected as the nominal voltage for the auxiliary pumping station services.

The 1.5 MVA 4160/600 V power transformer, which would be located in the incoming supply substation immediately outside the electrical room, would be fed from a fully-rated circuit breaker in the 4160 V MCC. In addition to the protective relaying for this oil-filled transformer shown on the Single Line Diagram (Drawing D4191-28), there would be alarms for low oil level and high temperature at the transformer. The secondary side of this power transformer would be connected to a 600 V circuit breaker switchboard which would feed two 600 V station service MCC's.

All 600 starters would be fused-contactor type for full-voltage (across-the-line) starting and each would have its own control power transformer. All motors and feeders connected to MCC's would be provided with standard industrial-type electrical protection.

One 1500 kVA 4160/600 V power transformer would be stored at the power plant warehouse to serve as a stand-by unit for both booster stations and the intake station. One 4160 V and one 600 V starter of each size used would be installed at each respective MCC to serve as a stand-by unit to facilitate fast restoration of service after a starter failure. An empty spare cubicle or structure at each MCC would also be provided. Costs for these stand-by components are included in the estimate.

Piping

All piping would be carbon steel with wall thickness designed for waterhammer pressures. The suction pipe and fittings would be ND 25 rating and the discharge pipe and fittings would be ND 40 rating.

The booster pump supply header, 1200 mm in diameter, is sized for a maximum velocity of 1.2 m/s.

The suction branch for each pump would be 600 mm and have an expansion-vibration coupling, a shut-off gate valve, and a magnetic flowmeter. This flowmeter would be installed for the following purposes:

- To assist in performance testing of booster pumps.

- To measure total flow to reservoir.

- To assist in leak detection.

- To assist in verification of waterhammer analyses.

- To determine wear rate of pumps for preventative maintenance purposes.

The discharge piping from each pump would be 350 mm diameter with an air-actuated ball valve for shutoff. The 800 mm diameter discharge header would be an extension of the main pipeline. It would lie in a trench below the floor at an elevation suitable to bring it below frost level where it emerges from the pumphouse.

Valves would be the fully-opening type because suspended solids in the water would erode any valve elements directly in the flow. Valves suited for this service would be either gate or ball types which, when fully open, do not obstruct the flow and therefore minimize head loss and wear.

All valves on the pump discharge lines would have either pneumatic, electric, or manual operators sized to open and close the valve against full differential pressure if necessary.

The discharge ball valve, not designed to be used for flow regulating purposes, must be sized to take the pressure drops encountered during opening and closing. The ball valve would have a pneumatic operator to open and close at a preset rate in accordance with waterhammer protection requirements.

Vibration and expansion joints would be provided in the pump suction lines but cannot be installed in the discharge lines because of the high pressure. The discharge piping would be designed to accommodate stress due to expansion and contraction of the line from temperature and pressure. Discharge pipeline drainage is discussed under section PIPELINE DESIGN - Drain Valves.

Auxiliary Equipment

Although the booster pumps could be used to fill the pipeline, they would require a control valve and pressure-reducing system to reduce the full pump discharge pressure to the line pressure which, during the initial filling, is near atmospheric. Control valves and pressure-reducing components are undesirable because of high noise levels, wear, cost, and maintenance. These valves and components would have to be installed on at least two booster pumps per station to ensure that pipeline filling would not be dependent on one booster pump. Variable speed drives for the booster pumps would be an alternative but the cost would be prohibitive. Also, the booster pumps would have to operate near their minimum flow capacity which is not advisable because of heating of the fluid in the pump and possible cavitation.

Therefore, to fill the pipeline, a separate filling system would be provided at each station as shown on Flow Diagrams D4191-3 and D4191-4 and as described on Drawing B4191-27. The filling system would have an average flow capacity of 64 l/s (1000 USGPM) which would fill the line in approximately 48 hours. The fill pumps would have the same size casings but the impeller diameters would be different to obtain the required discharge heads.

Motor-cooling water would be tapped off the first stage of the booster pumps, in a method similar to the seal water supply, with pressure-reduction orifices before the motor heat exchanger. The warmed water would be returned to the suction header. Water requirements would be less than 4 l/s per motor.

The potable water for both Booster Stations would be tapped from the suction header and chlorinated by a small package unit.

Each pump and motor set would have a common oil lubricating system with oil filter, reservoir, cooler and two gear-type oil pumps. One oil pump would be used for start-up and would have its own electric motor drive. The other pump would be driven from a gear on the pump shaft. Each pump would have a curb around its base to contain any oil spills. The discharge pipe trench would collect seal water as well as any oil or water spills. An oil skimming system would be installed in the trench to separate the oil from the water before the water is directed to natural drainage outside the pumphouse.

To provide air for the pneumatically-operated valves, each station would have 2 compressors supplying a 250 liter receiver. In the event of a power failure, there would be enough capacity in the receiver to close all necessary valves.

Since the booster pumping stations would be constructed of non-combustible concrete and steel and since all cable trays would be in covered trenches, fire insurance underwriters only require that fire extinguishers be provided.

Battery-operated emergency lighting units would be installed in the electrical room and other strategic locations.

Provisions would be made at each booster station for pneumatic operation of the drain valve during a power outage.

Structural and Architectural

The structure would consist of a steel frame with pre-cast concrete wall and roof deck panels. Sound and thermal insulation would be provided by lining the inside face of the pre-cast concrete panels.

The architectural concept shown on Drawing D4191-25 is directed towards blending the booster stations with their immediate surroundings through the use of precoloured concrete panels. The substation located at an intermediate elevation between the pumphouse and sloping terrain, is screened from view by the pumphouse.

At No. 1 Booster Station, the clarifier and clearwell are located on an upper bench, and are partially screened by landscaped berms moulded to blend with the surrounding terrain.

Maintenance

Ease of maintenance and repair are important considerations when choosing the booster station equipment and the equipment layout should be such that all areas are easily accessible for maintenance.

The booster station equipment has been arranged so that all pumps and motors are easily accessible. Each pumping unit has about 1.5 metres of clear space around it. A volute style pump, with the casing split horizontally, is relatively easy to repair because the top half of the casing can be easily removed, exposing the entire rotating assembly for inspection. The diffuser style pump requires more time to open for inspection or disassembly. This should be considered when comparing bids.

All mechanical spares would be stored at the power plant for safety and inventory purposes but lubricants would be stored at the booster stations. Two spare rotating assemblies complete with shaft, impellers and wear rings would be stocked rather than individual wearing parts. This is advantageous because an assembled rotating element could be installed in much less time than individual parts could be replaced. The worn or damaged rotating assembly could then be taken to the power plant to be repaired, without haste, under good working conditions. The repaired assembly would then become the spare.

The work shop provided at each booster station is only for day-to-day maintenance tasks, and only tools necessary for minor repairs would be provided. Major repair work would be performed at the power plant maintenance shop.

Since the booster stations would be monitored by instrumentation for data on operating conditions of equipment, maintenance crews need only visit the stations weekly for visual, vibration, and miscellaneous checks.

Operation and Control

The overall operation and control system is described under the section, OPERATION AND CONTROL OF THE SYSTEM. This description provides further definition of the part of the system within the booster stations.

The booster pumps would be controlled and operated remotely. Signals to start pumping, originating from the power plant, would be received by the telemetering system at the booster stations. If all interlocks provided to prevent improper operation of the pumps were satisfied, the pumps would start. When the pumps reach rated speed, the pneumatically-operated pump discharge valves would then open at adjustable preset rates to minimize pressure surges in the pipeline.

The self-regulation function of the equalization tank is discussed under SYSTEM CONFIGURATION. The booster pump discharge valves are only for isolation duty and stopping and starting pumps - not for flow control. The liquid level in both the clearwell and the equalization tank would be monitored continuously. Low clearwell level and both low and high equalization tank levels would be annunciated at the power plant control room. If corrective action were not taken, low level switches on both the clearwell and the equalization tank would immediately shut down all booster pumps to prevent cavitation damage. A high level switch on the equalization tank would shut down No. 1 Booster Station pumps to prevent unnecessary overflow. The normal shutdown procedure for the booster pumps would be to close the discharge valves at a preset rate, then stop the pumps.

Operating data such as suction and discharge pressure, valve position etc. would be telemetered to the central control console in the power plant.

The following auxiliary equipment would be contained in each booster station and would be controlled as noted:

Item	Control Mode
Two air compressors	Automatic/Local Manual Override
Two line filling pumps and valves	Local manual
Line drain valves	Remote manual
Space heaters	Automatic/Local Manual Override

The following local interlocks would be provided in addition to those interlocks normally provided with the equipment, and would inhibit the start command from the central station.

Pump discharge valve not closed	Limit switch
Local equipment isolation	Hand switch
Low clearwell level (No. 1 Booster)	Level switch
Low equalization tank level (No. 2 booster)	Level switch
Suction valve closed	Limit switch
Air pressure low	Pressure switch

The following interlocks would be provided through the central control logic and would inhibit the start command:

Item	Device
Pipeline drain valve open	Limit switch
High equalization tank level (No.1 Booster)	Level switch
Pipeline not full	Pressure switch

PIPELINE ROUTE

General

The selected pipeline route is shown in general on Drawing D4191-13 and D4191-14, and in detail on Drawings D4191-16 through D4191-21. A profile is shown on Drawing D4191-15. This route has been developed, as described under System Configuration, for the following purposes:

1. It combines with other service corridors for the project, specifically with the 500 kV transmission line right-of-way*, from Boston Flats to McLean Lake, and with the access road corridor from McLean Lake to the power plant.

2. It minimizes energy requirements and capital cost by following the McLaren Creek - Medicine Creek pass in the Trachyte Hills. This aspect is described in detail in Project Memorandum V4191/12, Pipeline Routing, McLean Lake to Power Plant Reservoir.

The route has been studied as follows:

1. Desk studies using maps and air photos.

2. Field study by helicopter over portions of the selected route on 15 September 1976, with representatives of Sandwell and Golder Associates.

3. Field study by helicopter on 8 November 1977 with representatives from Marine Pipeline, B.C. Hydro Gas Group and Sandwell.

4. Field studies by Golder Associates on foot and by four-wheel drive vehicle, 16 and 17 November 1977.

Based on map and aerial photograph interpretation as well as field studies, Golder Associates have concluded that the route appears to be free of geotechnical hazards. (Reference 7)

Therefore only construction conditions and a general description are discussed below. The pipeline route will be discussed in individual portions over which these characteristics are relatively uniform.

No. 1 Booster Station to Station 2 + 000**

See Drawing D4191-16 and Photograph 13-4A***

* In the late stages of preliminary design, the possibility of locating the proposed 500 kV transmission line several miles south of Ashcroft was introduced. However, this study assumes that the transmission line follows the route shown on drawings D4191-13 and 14.

** Station numbers represent the actual cumulative pipe length, with station 2 + 000 chosen at the Ashcroft highway intersection. Consistent with normal practice, hundreds of metres are separated from the kilometers by a + sign, so that 18 + 500 means 18,500 m.

***For photographs, see Appendix 7.

From No. 1 Booster Station, the first 2 km of pipeline route follows existing roads to minimize disturbance of the residential and agricultural development in the area.

The geotechnical assessment of this section is as follows (Reference 6): The ground from the No. 1 Booster station to Station 2 + 000 is flat, consisting of deltaic and alluvial deposits of layered sands and gravels, with an upper layer in places of cobbles and boulders. These deposits are unconsolidated and can be excavated by digging. In general, the maximum size range of the boulders is 20 to 60 cm, but an occasional large boulder up to 2 m size could be encountered. A thin capping layer of wind-blown silt and fine sand covers the flat ground in many areas.

Over this section, access is excellent and other construction conditions are favourable. During construction, safety provisions such as fencing along the trench would be made, since this area is developed.

The Inland Natural Gas pipeline feeding Ashcroft crosses the route at about Station 1 + 900. Special care would have to be employed during construction here. In addition to the gas pipeline, municipal services such as water, sewer, gas lines, and electrical services may be encountered in this area. These have not been investigated as yet, but an allowance for crossing them has been made in the cost estimate.

Station 2 + 000 to 3 + 500

See Drawing D4191-16 and Photograph 13-8A from Elephant Hill Summit. The reasons for selecting a surface crossing of Elephant Hill along this route are described in Project Memorandum V4191/4, System Design.

The gently sloping ground from Station 1 + 700 to the foot of the dissected terrace bluffs, Station 2 + 000, is a deposit of colluvium probably resulting from erosion of the terrace deposits. The material is a layered deposit of fine to medium grain sand, with some coarse sand and odd gravel. The material has already been borrowed to some extent, and it would provide a useful source of pipe bedding material. Approximately 75,000 m^3 of this material is available.

The gullied terrace, which rises steeply from the road (Station 2 + 000) consists of layered deposits of sands and gravels with a capping of loose silty gravel. A good exposure exists in the highway cut just to the north of the proposed pipeline route. Layers of cobbles and boulders (maximum size about 45 cm) are present. The material is unconsolidated and can be readily excavated by digging.

Although the face of the terrace displays deep erosion gullies, erosion should not be a problem if the pipeline is kept out of the gullies and precautions are taken to prevent the pipeline from becoming an erosion channel.

Ascending Elephant Hill, the route reaches a maximum gradient of about 45%. On this slope the pipeline would be constructed using the "yo-yo" technique, whereby equipment is held by winch and cable from the top of the hill. General access is available on the radio tower road.

In the section crossing Elephant Hill, from Station 2 + 500 to Station 3 + 500, the ground consists of a shallow veneer of sandy soil and gravel talus overlying bedrock. In general, the soil should be assumed to be not more than 30-60 cm thick, and in many places bedrock outcrops are visible. The flat top of the ridge above about elevation 625 m is covered in places with a veneer of silt or silty till. The thickness of cover is difficult to estimate but it is probably less than 1 m.

Station 3 + 500 to 7 + 000

See Drawing D4191-16, D4191-17 and Photograph 13-11A.

The section crossing Boston Flats, to Station 7 + 000 is comprised of deposits which appear to be a till-like mixture of silts, sands and gravels. In places the silty material predominates, and just below Station 3 + 500 the material seems to be easily erodible. For this reason, the pipeline might need to be buried deeper than elsewhere down the sloping east flank of the valley.

General access to the Boston Flats area is provided by Highway 1, and by the access roads for the hay fields. The original topsoil would be replaced after construction of the pipeline.

Station 7 + 000 to 8 + 500

See Drawing D4191-17 and Photograph 13-13A.

The route in this area follows the road to a garbage dump near Station 8 + 000, on a mild (20%) side hill. Near station 8 + 100 the pipeline route turns to follow the proposed 500 kV transmission line right-of-way to McLean Lake.

Station 8 + 500 to 11 + 000

See Drawings D4191-17, D4191-18 and Photographs 13-17A, 13-22A.

This section runs directly up the hillside at an average gradient of 23 percent, with a maximum gradient of 32 percent. The rock bluff at Station 8 + 750, seen on photo 13-17A, would require some blasting, as would the rock outcrop just east of the little lake at Station 11 + 000, shown on Photo 13-22A. The route is heavily forested above about Station 9 + 600.

Field surveys by Golder Associates (Reference 7) have indicated that assuming the maximum depth of excavation would not be greater than 3-4 m, the rock excavation should be less than 10 percent over the route from Station 8 + 500 to the reservoir. In most places, the soil consists of deposits of till and gravel, very thick in many places and generally greater than 2 m depth to bedrock. These deposits can be seen in the cuts along the numerous abandoned logging roads that cover the area.

Where rock is exposed it appears to be hard but closely jointed material that could be loosened and excavated by light blasting. Elsewhere, the till and gravel soil should present no difficulty in digging using conventional excavating equipment.

The location of No. 2 Booster station, at station 9 + 500, has excellent area drainage with a ground slope of about 18 percent, the appearance of favourable foundation conditions, and general access provided by old logging roads which would be improved.

Construction across the little lake at Station 11 + 000 has been assumed to be by temporarily draining the lake, although this aspect would be studied during Final Design.

Station 11 + 000 to 14 + 200

See Drawing D4191-18, D4191-19 and Photographs 13-23A, 13-25A.

This section runs through timber stands of variable density to the outlet of McLean Lake. Short segments requiring sidehill construction are encountered near Station 12 + 000. Except for a swampy section near Station 12 + 500, and a rock outcrop at about station 13 + 800, excavation should be in till and gravel. Access to this section would be on existing forest roads, improved as necessary.

Station 14 + 200

See Drawing D4191-19 and Photograph 13-25A.

At the outlet of McLean Lake, Cornwall Creek has formed a steep gully as indicated on the photo by a tongue of trees. This crossing must be designed to ensure that the creek will not cut into its bed and expose the pipeline. At present, it appears that Cornwall Creek is gradually cutting deeper into its course and threatening to drain McLean Lake. Moreover, the slopes of the gully consist of a silty, gravelly till of considerable thickness which appear unstable, but detailed studies would be needed to confirm this. For the purpose of Preliminary Design, this crossing is assumed to consist of an embankment bridging the gully. A culvert would carry the Cornwall Creek discharge and riprap downstream from the embankment would prevent the natural downcutting process from reaching the pipeline or McLean Lake. If found advantageous during Final Design, the pipeline and the power plant access road crossings of Cornwall Creek could be combined to utilize the same embankment.

Station 14 + 200 to Power Plant Reservoir

See Drawings D4191-19, D4191-20, D4191-21 and Photographs 13-28A, 13-33A.

This section of pipeline follows the power plant access road, and thus the design and construction of the two services must be coordinated.

The route up to Station 18 + 500 is heavily timbered, but beyond 18 + 500 passes through open meadow and lightly forested slopes. Throughout this section, the aerial photographs and the surficial geological information indicate that the ground is silty till.

Any deep excavations in till could encounter an underlying consolidated till deposit, which in the Hat Creek area, has been found to be very hard.

The final section of pipeline from Station 22 + 500, where the route departs from the road, would be reviewed during Final Design. As shown on Drawing D4191-21, it is assumed to pass under the dam at about elevation 1350 m. The design of this feature is discussed under Reservoir Inlet in the next section.

PIPELINE DESIGN

Steel Selection

Welded steel is the only appropriate pipe material for internal pressures over about 1.9 MPa (275 psi). Below this pressure, other materials such as ductile iron with O-ring connections, concrete pipe with O-ring connections, or prestressed-concrete steel-cylinder pipe with welded connections could be used. While some material and construction costs may be reduced in this manner, considerations such as the overall security of the line and the uniformity of inspection procedures suggest that welded steel construction should be employed throughout the line.

The selection of the exact steel composition required for the pipeline will be reviewed extensively during Final Design. For the purpose of this report and estimate, a preliminary specification has been made as described in Project Memorandum V4191/13, Pipeline Steel Selection, and is summarized below.

Standard:	CSA Z245.2 High Strength Steel Line Pipe 18 in. and larger in diameter.
Steel Grade:	60. Minimum yield strength - 414 MPa (60,000 psi) Minimum Tensile Strength - 518 MPa (75,000 psi)
Category:	II. For buried pipelines where toughness properties are important.
Deviation from Standard:	Maximum carbon equivalent - 0.40 Maximum carbon content - 0.12 Impact test - Charpy full size, 54 J. at -4°C (40 ft-lb. at +25°F)

Pipe Manufacturing and Supply

The pipe manufacturing processes most suited to the Hat Creek pipeline are:

- Spiral construction, submerged-arc welded, cold expanded.

- U-O* (longitudinal seam) construction, submerged-arc welded, cold expanded.

Seamless pipe is more expensive but not of better quality, and is not generally available in the sizes needed. The cold expansion process provides cold working which reduces grain size and improves the material toughness. The choice of U-O or spiral construction does not appear consequential, as both are in wide use.

Project Memorandum V4191/4, System Design, includes a summary of the availability of pipe from various sources. For the particular pipe specification and size as described here, 10 potential pipe suppliers were surveyed by letter. Of these, 4 replied they were willing to supply the entire order, and 2 were willing to supply a majority of the order (up to a limit of wall thickness). Two more would probably bid for the entire order, and one more only for an alternative diameter. During Final Design the question of whether the pipe should have a uniform internal or external diameter will be examined. The former is preferable hydraulically, but less commonly available.

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* U-O refers to the process by which a plate is first stamped into a U shape, then rounded to an O shape to become a pipe.

At the mill, inspection would consist of ladle tests to check the chemical composition of the steel and a variety of tests of the completed pipe to determine its mechanical properties. Such methods as pressure testing; Charpy and Drop-Weight Tear Tests; magnetic particle, x-ray and ultrasonic inspections would be considered.

Pipe Size

Project Memorandum V4191/4, System Design, describes how the choice of pipe diameter affects the total capital and operating cost of the system. A diameter of 800 mm (32 in.) was found to be the optimal size for the Conceptual Design route. During Final Design, this assessment will be reviewed using prevailing steel and power prices, and revised average discharge.*

One Canadian supplier (Steel Company of Canada), does not have tooling for the 800 mm size. Therefore, it may be advantageous to call tenders with a choice of pipe sizes from 800 to 900 mm diameter in order to obtain the most competative bids from domestic sources.

For the 800 mm diameter pipe size, hydraulic data are presented in Table 6.

Table 6 - Hydraulic Data for 800 mm diameter Pipeline

Outside Diameter:	813 mm
Average Inside Diameter:	About 790 mm (wall thickness varies from 8 to 16 mm)
Velocity:	3.3 m/s at system capacity of 1.58 m^3s 1.4 m/s at average discharge of 0.66 m^3s*
Friction Losses**	7.4 - 10.0 m/km at system capacity of 1.58 m^3s 1.7 - 1.9 m/km at average discharge of 0.66 m^3/s*

The concept of varying pipe diameter in order to save steel weight has been rejected owing to the requirement of using "pigs" for internal inspection. (See Inspection and Cleaning Facilities for explanation of "pigs"). However, a small variation such as would result from the use of pipe made to uniform outside diameter but with varying wall thickness, would not interfere with pigging. Where pipes of different wall thicknesses are joined, the thicker wall could be bevelled internally to match the thinner wall.

Design Codes for Pipe Thickness

Many design codes are used for different types of pressure piping so that judgement is required in deciding upon the appropriate design code for use on this project. Most codes relate allowable stresses to the minimum specified yield point of the steel although some consider a factor of safety based on the ultimate tensile strength, and thus pipe rupture as well. The provisions of various codes for internal and external pressures, are tabulated and attached as Table D in Appendix 4.

The USBR penstock design criteria has been selected for the Hat Creek Project for the following reasons:

* Subsequent to Preliminary Design Studies, average discharge has been revised from 0.66 m^3/s to 0.726 m^3/s.

** By Darcy-Weisbach formula with E = .015 mm for smooth condition and E = 0.06 mm for rough condition. This is consistent with use of coal tar epoxy coated steel pipe. See Reference 8.

- Suitably conservative.

- Has been used successfully for the California State Water Project (Reference 19).

- More detailed than the AWWA code.

- Recommended by waterhammer consultant John Parmakian.

Project Memorandum V4191/1, Design Criteria, provides more detail on the selected design code, and defines the normal, emergency, and exceptional conditions, which are referred to in Table D.

The ASME pressure vessel code was considered too conservative for this application as it applies to boilers in chemical plants, nuclear power plants and the like. Gas and oil pipeline codes, owing to the different weight, compressibility, and flow properties of oil and gas compared to water, were not considered applicable. The AWWA guidelines are perhaps more applicable to municipal water supply projects, but were not used mainly because of the better defined provisions of the USBR code.

Pipe Thickness Design

Table 7 gives the relationship between the pipe wall thicknesses used for Preliminary Design, and the corresponding internal pressure allowed for the normal design condition, using the USBR code for the selected steel (Grade 60).

Table 7 - Pipe Thickness and Pressure

Pipe Wall Thickness		Maximum Internal Pressure		
mm	(Inch)	MPa	m of Water	(psi)
8	(.315)	3.43	349	(497)
11	(.433)	4.73	481	(686)
16	(.630)	6.92	704	(1003)

Notes:
1. Formula used (Reference 8) accounts for the effect of wall thickness on maximum internal pressure. Outside diameter of 813 mm and allowable stress of 172.4 MPa used.
2. The 8 mm wall thickness was determined according to the Stewart formula, Reference 17.
3. The maximum pipeline internal pressure of 6.92 MPa, is the total dynamic head plus 10% waterhammer allowance.

The pipe stresses which govern minimum wall thickness are due to external soil and water pressures on the pipe when empty. The minimum thickness of 8 mm would be adequate both for collapse prevention and to prevent buckling during handling. Longitudinal stresses due to settling and temperature variation, stresses due to anchorages and bends, and other miscellaneous stresses such as earthquake stresses must be considered during Final Design. In general, buried pipelines are designed without an allowance for longitudinal thermal stresses as the usual construction practice is to make the last girth welds joining long segments of line when the pipe is cool, perhaps early in the morning, to reduce the variation of temperature from construction to operation. The thermal stresses which would result are not considered harmful. Pumps and special structures which may be damaged by thermally-induced stresses would be protected by anchor blocks.

Welding

The pipeline welds would be important components of the project requiring extensive investigations during Final Design and thorough care in construction and inspection. The requirements for the production of acceptable welds are proper choice of steel, use of qualified welders, and complete inspection.

The pipeline steel will have a low carbon equivalent value to reduce the chances of forming a brittle martensitic microstructure during welding. Other properties of the steel, such as alloy content and sulphur content, will be selected so that the pipe material is the best obtainable at reasonable cost.

Having ensured that a suitable material will be used, one can also ensure that the welders used on the project can produce an acceptable result by using qualification tests such as API specification API-1104.

Inspection and testing by an independent testing company is also required to achieve good welds. Test and inspection procedures are expected to include:

- Ultrasonic testing of the plate and the pipe ends at the mill to detect laminations.

- 100 percent x-ray inspection of field welds.

- Visual inspection of completed welds.

- Pressure testing of the pipeline after backfilling. Each of the three sections of pipeline would be held at a test pressure exceeding design pressure to locate any leaks.

X-ray inspection possibly augmented with ultrasonic and/or magnetic particle inspection, would give the best available definition and detection of the more serious welding defects such as lack of fusion between the weld metal and the pipe, lack of penetration of weld metal, and weld zone cracking (hydrogen cracking). Such defects may necessitate that a weld be cut out and replaced. These defects would not necessarily show up in a pressure test as the stresses at girth welds are mainly due to pipeline settling rather than internal pressure.

The shielded metal-arc or gas metal-arc welding processes, commonly used for pipelines, would be specified for this project. The pipe would probably require pre-heating to 50 °C and allowances for this procedure have been included in the cost estimates. The first pass could be with lower-strength electrodes such as E6010 with subsequent passes with higher-strength electrodes such as E8010. Such techniques are familiar to qualified pipeline contractors.

Waterhammer Control - General

Waterhammer is the name used to describe the waves of pressure which travel in a pipeline when changes in flow occur. These waterhammer waves often create the most severe conditions for the design of pipelines.

In the Hat Creek cooling water supply pipeline, waterhammer would occur when a pump is started or stopped, or when a valve position is adjusted. The various waterhammer conditions which must be considered in designing the pipeline are described in detail in Project Memorandum V4191/1. Design Criteria.

The following sections identify adequate measures for the control of the pressures generated by waterhammer. However, there are many factors to be considered during Final Design to fully define all aspects of a given waterhammer control system. Some of these factors are:

- Model tests of the selected pumps are required to define the hydro-mechanical characteristics which could influence the control facilities and pipe design.

- The waterhammer studies must be refined to suit the final pipeline profile.

The waterhammer control aspects are discussed below for the three pipeline sections: from the intake to the No. 1 Booster Station; from No. 1 to No. 2 Booster Station; and from No. 2 Booster Station to the plant reservoir.

Waterhammer in the pipeline from the intake to the No. 1 station was briefly examined by Sandwell. B.C. Hydro and Power Authority has provided preliminary waterhammer analysis for the portion of pipeline from the No. 1 to the No. 2 Booster Station and from the No. 2 Booster to the power plant reservoir. This analysis has subsequently been reviewed (excluding verification of computer studies) by waterhammer specialists Parmakian, Streeter and Wylie. They concur with the control measures recommended. During Final Design, the waterhammer analysis and proposed control measures would be reviewed using the pump characteristics determined by model testing, and using the final pipeline profile.

Waterhammer Control - Intake to No. 1 Booster Station

For this 900 mm (36 in.) diameter pipeline, waterhammer was sufficiently studied to devise a feasible control system for Cost Estimate purposes. A buried surge riser of 900 mm diameter pipe, extending to elevation 350 m as shown on Drawing D4191-22, would be provided. The intake pumps would be equipped with discharge valves and check valves.

Waterhammer Control - No. 1 to No. 2 Booster Station

In their report, Hydraulic Transient Analysis (Appendix 20), B. C. Hydro identify the following two alternatives for waterhammer control in this section:

1. Provide a one-way surge tank* (4 m diameter by 10 m high) on the summit of the Elephant Hill crossing, and provide at least 115 kg-m^2 of pump and motor inertia per unit.

2. Provide 400 kg-m^2 of pump and motor inertia per unit. European manufacturers could build this into the motor in the factory. However, in North America, this may have to be added by means of a flywheel.

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* A one-way surge tank is a tank filled with water, isolated from the pipeline by check valves, so that when a negative pressure wave passes down the pipeline, the check valves open and water from the tank flows into the pipeline to prevent water column separation.

Alternative 2, high-inertia, has been selected for Preliminary Design because:

- Waterhammer control by provision of inertia would not be subject to freezing or possible malfunction of check valves as would be the case with the one-way surge tank.

- The increased inertia is low enough to permit motor starting without overheating.

The pipeline would be designed for a maximum internal pressure 10 percent over the pressure at rated discharge at the pump end and decreasing linearly to zero at the downstream end. To keep pressures below this maximum, flow would be allowed to reverse through the pumps by closing the air-actuated pump discharge valves at a uniform rate over about 95 seconds following total power failure, resulting in the pumps and motors peaking at a reverse rotation of 105 percent of rated speed. Check valves have been deliberately omitted as they would create higher pressures.

Block valves are excluded from the pipeline to avoid increasing the design maximum pressure to the pump shutoff head which is about 16 percent above the pressure at full discharge.

Waterhammer Control - No. 2 Booster Station to Reservoir

The B.C. Hydro report (Appendix 20) also identifies the following control measures as being appropriate for the second section of pipeline:

1. Provide two one-way surge tanks (each 4 m diameter) the first with free surface elevation 1252 m, 10 m high, and the second at elevation 1345 m, 25 m high. (See Drawing D4191-18 and D4191-20 and, for location in profile, Drawing D4191-15). In addition to the tanks, provide pump and motor inertia of 400 kg-m^2 per unit. There are differences between the profile supplied to B.C. Hydro for analysis, and the current profile of the selected route, so that the one-way surge tanks have been taken as 30 m high for estimating purposes to allow for adjustments which may be made in future studies.

2. Allow reverse rotation of the pumps during total power failure. For the design pressure rise of 10 percent, the valve closure would be 95 seconds and the pump reverse speed 100 percent of rated speed. Block valves would be excluded from this section.

Emergency Provisions and Leak Detection

During Final Design, the pressures generated by any possible failure of the waterhammer protection system would be investigated as defined in Project Memorandum V4191/1, Design Criteria.

In addition to provision for these pressures, the pipe wall thickness and the steel impact toughness would be specified to prevent a small leak from growing in size. As discussed in Project Memorandum V4191/13, Pipeline Steel Selection, a crack as large as about 27 cm long would not tend to grow in size under full line pressure, with the impact requirements specified.

Should a break occur in the pipeline, the pumps could be stopped either manually or automatically, triggered by a leak detection system. Options for such a system are discussed in detail in Project Memorandum V4191/15, Pipeline Leak Detection. The proposed detection system would function continuously whether or not the pumps were operating. It would consist of permanent acoustic sensors attached to the pipeline at intervals of 300 m. These sensors would detect unusual vibrations caused by even a small leak. A scanner would sound an alarm and the system operators could pinpoint the location of the leak, stop pumping operations, and measure approximate leakage rate by changes in tank levels. Valves could be closed to isolate the reservoir and all tanks from the pipeline so that the loss of water would be restricted to the amount in the pipeline. If a further reduction of leakage volume is required, motorized block valves could be installed to divide the pipeline into shorter sections. However, this is not recommended as the pipeline is already divided in two and, as discussed under Waterhammer Control, the use of block valves would increase the pipeline wall thickness.

The design of the pipeline right-of-way would include trenches, dykes, or the line drainage facilities to channel any leaking water toward a natural water course to avoid erosion. The water course would either be of sufficient size to handle the flow without damage or would be provided with retention dykes. These dykes would not impede the natural maximum flow but would temporarily store water suddenly released by a pipe failure. The dykes, trenches, and associated works would be inconspicuous as they would be earth structures, small in scale, planted with grass, brush and trees to minimize erosion and visual impact.

These provisions are intended to provide for extreme circumstances. However, the chance of any of these facilities being required during the life of the project is extremely remote. The cost estimates include an allowance for these facilities, however their necessity should be confirmed during Final Design.

Pipe Fittings and Bends

Pipe fittings such as laterals, tees, and elbows are required to make sharp bends and to make connections to pig traps, air and vacuum valves, surge tanks and the like. Shop fabricated fittings would be used to keep field cutting, fitting and welding to a minimum.

As discussed below under Inspection and Cleaning Facilities, pigs will be used in the pipeline. Therefore, for the 800 mm diameter pipeline, a 2.4 m minimum radius of curvature is necessary so that the pig can pass through bends. However, long radius field bends would be used where possible.

Most pipeline bends would be made in the field using portable bending machines. For the topography of this area, the ideal pipe length for bending is 18 m (60 ft). However, considering the pipe weight 12 m (40 ft) has been selected for Preliminary Design. With this length, field bends are still practical. Trials conducted for Arctic pipeline construction have confirmed the feasibility of cold field bending of pipe of similar steel, weight, and wall thickness.

Corrosion Protection

The pipe would be protected from corrosion by interior and exterior coatings, and impressed current cathodic protection. The interior coating would consist of coal tar epoxy or phenolic epoxy such as that specified by the AWWA draft specification C210-76. Either single or double epoxy coating, with or without zinc-rich primer, would be suitable. Double coating without primer is included in the estimate. In addition to preventing corrosion, the interior lining would reduce friction. The coating would be shop-applied, and thoroughly inspected for holidays. The interior surfaces at field welded joints would be wire brushed and painted with a special epoxy coating.

Alternatives to epoxy interior coatings are coal tar enamel and cement mortar linings. Enamel, because of its relative softness, has been rejected due to concern it may be damaged by pigging. The cement mortar lining will be appraised during Final Design.

The exterior coating would be coal tar enamel according to AWWA Standard C-203, whereby the pipe is coated with layers of coal tar enamel and fibreglass mat and topped with Kraft paper. Portable coating machines could not negotiate the steeper sections of the pipeline route with the heavier wall thickness pipe required, so this coating would have to be shop-applied. Alternatively, epoxy exterior coatings have been developed which could be used. At field-welded joints, exterior surfaces would be ground smooth, primed with an epoxy-rich paint, and wrapped with special plastic tapes.

The impressed current cathodic protection system would require grounding beds and power supply at intervals along the pipeline.

Freeze Protection

The pipeline will be protected against freezing by burying it below the depth of frost penetration in the ground. In this way, the soil will provide heat to the pipeline so that even if the incoming water temperature is at 0°C, freezing will not occur.

The alternative methods of freeze protection which were studied are:

1. Heat tracing.
2. Pipeline drainage.
3. Continuous flow.
4. Insulation by pipe jacket.
5. Insulation in the trench.
6. Deep burial

These are discussed in detail in Project Memorandum V4191/8, Pipeline Freeze Protection. Deep burial was chosen as it reliably protects both standing and flowing water at minimum cost. For extra security, water temperature in the pipeline could be monitored.

Deep burial requires that the pipe centerline should be below the depth of frost penetration, calculated as 2.0 m. Backfill should consist of well-graded and well-compacted soils to provide suitable insulation.

Trench Design

Freeze protection criteria have determined the minimum depth of bury of the pipe, and the use of well-compacted backfill material.

The pipeline would be laid at specified grades and elevations to enable proper line drainage. Concrete bedding may be required on steep slopes to prevent washout of fill material. Tie-backs which firmly anchor the pipe to rock may be required to prevent long-term downhill creep.

Welded pipelines generally do not require anchor blocks at bends because the force counteracting the momentum change at the bend is transferred to the soil by friction along a length of pipeline. Therefore, anchorage at bends is not included in the cost estimate although this will be reviewed in Final Design.

In sections where construction is through rock, suitable borrowed backfill material would be needed to cushion the pipe and to ensure adequate insulation to prevent frost from reaching the pipe. Stockpiling and trucking costs for this material are included in the estimate.

Impervious plugs and surface drainage provisions are necessary to prevent erosion of bedding and backfill material. Impervious plugs could consist of clay, soil cement or concrete trench backfill at intervals of 50 m to 100 m. The size of the plug in the direction along the trench would be about 5 m for clay, 2 m for soil cement and 1 m for concrete. Clay construction has been assumed for Preliminary Design.

The surface drainage provisions would control the flow of surface run-off across or parallel to the right-of-way. Where surface flow runs across the pipeline, the crossing would be paved with cobble stones to prevent erosion of the right-of-way. Where parallel flow drainage might occur, shallow trenches or depressions would lead run-off away from the right-of-way and towards existing drainage courses. These trenches would commence at the plugs and would be paved where necessary with cobble stones.

Pipeline Right-of-Way

The pipeline right-of-way would be about 18 m (60 ft) wide and would contain a loose-surface access road, the buried pipeline, and a buried duct-bank carrying the control cables and power cables.

Immediately following completion of the pipeline, the right-of-way would be graded and re-vegetated, except that the road would be retained for maintenance access. Monthly inspection of the right-of-way would ensure that any erosion problems would be quickly controlled.

Photo 14-37 (Appendix 7) of the Afton mine water supply pipeline provides an example of how the lower reaches of the pipeline right-of-way would look immediately following construction but before re-seeding.

Where pipelines share rights-of-way with high-voltage transmission lines, the following factors must be considered (see Reference 20):

1. Electromagnetic induction from the load or fault currents can produce a current flow in the buried pipeline. Proper design of the pipeline should virtually eliminate any hazard to personnel and minimize danger of corrosion damage to pipeline equipment such as insulating joints and cathodic protection rectifiers.

2. Electrostatic induction or capacitive coupling can produce a charge on pipe above ground during construction. This hazard can be eliminated by grounding above-ground pipe.

3. Conductive energization which occurs when a fault current enters the earth in the vicinity of the pipeline, could cause breakdown of the pipe coating in spots and heating of the pipe steel which could eventually cause cracking. This aspect is currently under investigation by B.C. Hydro and a safe spacing between the pipeline and transmission line will be specified when the investigation has been completed.

Inspection and Cleaning Facilities

Visual inspection of the pipeline would not be practical. Therefore, the estimate includes provisions for inspection using so-called smart pigs, which are instrument packages propelled through the pipe by the flow. Outside contractors using their own pigs would provide a complete inspection report describing the condition of both internal and external pipe surfaces, and locating corrosion problems. During the construction of the line, pigs would be of great value in determining that the pipe is constructed to specifications.

The entry and retrieval of pigs would be facilitated by launching and receiving facilities (traps), provided at the ends of each pipeline section. Standard traps are available in the sizes and pressure ratings required.

Since the maximum pipe wall thickness for external surface inspection with smart pigs is currently limited to about 22 mm and since the maximum pipe wall thickness for the proposed pipeline is 16 mm, the entire exterior of the pipeline could be inspected. As provisions for pigging are expensive, their necessity should be confirmed during Final Design.

Provision has also been made in the cost estimate for man ways to the pipeline interior at four points.

Ball Valves

Full-opening ball valves are available for the 800 mm line size at the required pressure rating. A ball valve is preferred to other valve types as it would not interfere with pigging operations and does not cause a loss of head. Ball valves would be necessary at pig traps and could be used as block valves (as discussed under Emergency Provisions and Leak Detection) if desired.

In order to remove the valve from the pipeline for servicing, either a Dresser-style coupling for low pressure or a special sleeve-type coupling for high pressure would be installed on each valve. The valve and actuator would be enclosed in an underground concrete vault.

Air and Vent Valves

Air release valves, designed to release small amounts of air entrapped in the water, would be provided at all local summits of the pipeline route. These valves would be enclosed in underground concrete vaults.

Vent valves would be used to allow the entry and escape of large volumes of air during line filling and draining. It may be desirable to isolate these valves from the pipeline during normal operation so that they do not actuate during waterhammer conditions.

Drain Valves

If it were necessary to dewater the pipeline for inspection or maintenance, the drainage sequence would be as follows:

1. The drain valves in the Booster Stations would be opened allowing a controlled discharge of water from the upper pipeline into the equalization tank and from there into the lower pipeline, finally to be released into the Thompson River through the clearwell overflow. Note that there would not be any overflow from the equalization tank.

2. When the maximum amount has been released in this way, drain valves at local low points in the pipeline would be opened. These would include one at Cornwall Creek, one in the Boston Flats area, and others as shown on the Flow Diagrams D4191/3 and D4191/4, and on Detailed Pipeline Location Drawings D4191/16 to D4191/21.

The main environmental concerns with pipeline drainage are that erosion or siltation damage to streambeds may harm salmon spawning beds and that releases of Thompson River water into the Bonaparte River may disorient salmon migrating up the Thompson.

Therefore, to avoid erosion or siltation, every drain valve installation would be provided with a pressure-reducing fitting and with a discharge control valve. In this way, a predetermined maximum discharge corresponding to a portion of the natural discharge of the watercourse in question, can be obtained. During Final Design, hydrological and environmental studies would determine these discharges.

The drainage from the Boston Flats area could be accomplished by one of two methods to avoid disorienting salmon in the Bonaparte River.

1. Drain into the Bonaparte River at some suitably small discharge rate - certainly less than 100 l/s (1600 USGPM).

2. Drain into the Thompson River by pumping in a separate small diameter pipeline over Elephant Hill and re-introducing the flow back into the main pipeline.

These alternatives will be evaluated during Final Design but, for cost estimates, the former method is assumed.

For the current profile, eight drain points would be needed along the pipeline route, as shown on the drawings mentioned above. At Station 5 + 800 at Boston Flats, 1600 m of 250 mm diameter ductile iron buried pipeline would run to a stilling basin beside the Bonaparte River. All drains would operate by gravity except the drain at the little lake at Station 11 + 000, where a small pump may be required. However, the alternatives to a pump will be reviewed during Final Design.

Reservoir Inlet

The manner in which the pipeline enters the reservoir which is contained by an earthfill dam has hydraulic implications for the system design.

The two alternative arrangements of the reservoir inlet are:

1. Enter the reservoir over the dam. With this arrangement, the advantage of a lower equalization tank at No. 2 Booster Station would be gained as the system head would be independent of the reservoir water level. The disadvantages would be that more energy is required for pumping when the reservoir is drawn down than for the second alternative and that the relatively higher pipeline profile in this area may require additional waterhammer control measures.*

2. Enter the reservoir under the dam. The pipeline would be excavated into bedrock and backfilled with concrete. In this case, the equalization tank needs to be higher to allow for the reservoir water level variation. However, energy is not wasted as the water is pumped directly into the reservoir and the lower pipeline profile* in this area can be obtained if necessary.

These alternatives will be appraised during Final Design. For the purposes of the cost estimate, the second alternative has been assumed.

With either alternative, a concrete stilling basin would be provided at the end of the pipeline for use during initial reservoir filling. For the selected alternative, a motorized shut-off valve would be provided so that the reservoir could be isolated during line drainage. Also, a vent valve would be provided to admit air to the pipeline during draining. The pipeline would terminate at about Station 23 + 000, discharging into a concrete flume which would carry the water down to the stilling basin during initial reservoir filling. After initial filling, the open end of the pipe would simply discharge freely below the reservoir low water level.

Pipeline Power Supply

The Single Line Diagram, Drawing D4191-28, shows 3 unit substations between the No. 1 and No. 2 Booster Stations and 6 unit substations between No. 2 Booster Station and the plant water reservoir. These supply power for the surge tank heaters, motorized valves and cathodic protection along the pipeline.

The two unit substations at Elephant Hill would normally be fed from No. 1 Booster Station, the four at Cornwall Hill would normally be fed from No. 2 Booster Station and the three towards the end of the pipeline would normally be fed from the power plant.

* Waterhammer pressures in pipelines with high ground near the discharge end are more difficult to control than those with lower ground near the discharge end.

The buried subfeeders originating at the 4160 V switchboards in the booster stations and at the power plant are described in POWER SUPPLY AND DISTRIBUTION.

Since the major load is heating power, it is proposed that a continuous 4160 V supply cable be installed along the entire length of the pipeline. This cable and the two sectionalizing switches located as shown on the Single Line Diagram, would permit supply of any and all loads from any of the three feeding points.

The supply cable along the entire pipeline would be sized for the worst voltage drop condition. That is, when all loads are fed from either No. 1 Booster Station or from the Power Plant.

Interlocking of the feeding circuit breakers with the sectionalizing switches to prevent parallel feeding and the consequent overloads in the pipeline supply cable when a main booster pumping station service failed would not be feasible due to the long distances involved. Therefore, safe operation of the pipeline power supply system would have to be ensured by operating procedures.

An overhead pole line supply versus underground cable supply was evaluated. An overhead line, while having lower conductor costs, would require closer than standard pole spacing, since telecommunication cables to the power plant have to be carried along the same route.

Even with overhead design, large portions of the supply, e.g. in populated areas and at highway crossings, would have to be run underground for safety and environmental reasons. The frequent change from overhead to underground installation would make for a potentially-troublesome installation. Also, pole lines in remote areas could be targets for vandalism. The uninterrupted availability of power to the pipeline and especially the continuous functioning of the remote control and telemetering system would be of vital importance to the operation of the pumping system. Therefore, the installation of the entire length of power supply and telecommunication link in an underground duct bank has been selected. Ducts would be concrete encased at road and service line crossings and where these enter and leave pull pits and substations.

The duct bank would run parallel to the pipeline and would be installed at a suitable distance from the latter to prevent interference between the two during maintenance and construction. This distance would vary with topographical features and would have to be determined during Final Design.

The unit substations (purchased as a self-contained complete package) would be located immediately beside the pipeline valve pits or surge tanks they are serving. The transformers would be mounted on a concrete pad and would be housed in a kiosk together with their associated switchgear.

A transformer of each size used in the unit substations along the pipeline, complete with primary and secondary switchgear, would be kept in storage for immediate availability in the event of a transformer failure. Allowance for these stand-by units has been made in the cost estimate.

OPERATION AND CONTROL OF THE SYSTEM

General

The remote manual control system would enable unmanned operation of the intake, water treatment and booster stations. The power plant operator would select the desired pump combination and would send a command signal over a data transmission line to the individual pump stations. A feedback signal would confirm that the command has been carried out.

The data transmission link would be a phase-shift keyed, digitally-coded multiplex system transmitted at low speed over four individually shielded cable pairs with message security checks to detect and reject errors. Auxiliary equipment such as air compressors, space heaters, etc. would have automatic controls with local manual override. Line-filling pumps and associated valves would be local manual operated. No provision will be made for emergency local manual operation of the major equipment.

The control console for the system would be located at the power plant and would be of similar design to the other power plant control consoles. The panel would display, on command, the status of the equipment in the system and, in the event of a malfunction, would generate a visual and audible alarm identifying the malfunction. All of the control system logic would be located in the control console and would be capable of being programmed in a high-level data processing language.

The system would be operated at four discrete discharge rates determined by the capacities of the four pairs of booster pumps.

Instruments

Measuring instruments would generally be of the force balance type with signal levels of 4 to 20 ma DC for electronic devices and 20 to 100 KPa for pneumatic devices. Local indicators would be included at each measuring point.

Valves

Valves would be tight-seal butterfly type for low shut-off differentials and tight-seal full-opening ball valves for high shut-off differentials. Electrically-positioned valves would include open and closed limit switches.

Pneumatically-positioned valves would include hydraulic-damping devices to permit independent stem speed adjustment for rates of opening and closing. Drain valves with high differentials would include energy dissipation devices.

Emergency Provisions

Emergency provisions for the operation and control of the system are discussed individually under their appropriate sections. They include interlocks and air-actuated valves, drainage provisions, and leak detection.

POWER SUPPLY AND DISTRIBUTION

General

Electrical power for the pumping system would be supplied from the proposed Rattlesnake substation near Cache Creek. The electrical system would consist of transmission lines feeding two incoming supply substations located at No. 1 and No. 2 Booster Stations, power distribution and motor control centres, and nine unit substations located along the pipeline, all as shown on the Single Line Diagram, Drawing D4191-28, and as described in detail below.

Transmission Line

A 69 kV pole line would extend south from the Rattlesnake substation to a point where it would split into two 69 kV pole lines to the incoming supply substations located at each of the booster pumping stations. Since the routing and the detailed design of the 69 kV transmission line is the responsibility of B.C. Hydro, the capital cost estimate for the water supply system does not include the cost of the 69 kV transmission lines.

Incoming Supply Substations

A 69 kV incoming supply substation would be located immediately adjacent to each of the two booster pumping stations. To minimize visual impact, the substation would be a low-profile design and be situated behind the pumping stations. A layout of the substation, identical for both booster pumping stations, is shown on Drawing D4191-24, Booster Station - General Arrangement. The substation would occupy an area of approximately 40 by 18 metres.

The incoming transmission line would be terminated on a portal structure with surge arresters located below. Conductors would drop down to the main disconnect switch which would be connected to the circuit breaker with a rigid aluminum tubing bus structure. Similar bus work would connect the load side of the circuit breaker via transformer-mounted surge arresters to the primary bushings of the 20 MVA power transformer. The power transformer would be ONAN - cooled and would have a standard off-load tap changer. Space for a perpendicular bus tap to a future power transformer would be provided between the main disconnect switch and the circuit breaker.

For lightning protection a ground spire would be centrally located. All bus and supporting structures, the station grounding system, fencing and exterior lightning would be designed to B.C. Hydro standards.

Preliminary calculations by B.C. Hydro indicate that, due to the short length of the incoming transmission line and the exclusive nature of its use, the maximum voltage variation at the end of the line would be ± 5%. Therefore, on-load tap changers are not considered necessary for the main power transformers and have not been provided.

The main disconnect switch and circuit breaker would be interlocked such that the disconnect switch cannot be operated unless the circuit breaker is tripped.

In addition to the protective relaying for the power transformer shown on the single line diagram, there would be alarms for gas accumulation, low oil level and high temperature.

Power for tripping and closing the circuit breaker would be supplied from a station battery.

Revenue metering for the pumping stations is not required. Protection-class current and potential transformers connected to the secondary side of the 69 kV power transformers would be used to meter the power consumption of the pumping stations. This would consist of a kilowatt-hour and a kilowatt maximum-demand meter. Energy consumption and power demand readings of one booster station would be transmitted to the other booster station for summation purposes. Main voltmeter and ammeter with phase selector switches would also be provided.

Stand-by Power Transformer

A 20 MVA stand-by power transformer would be placed at No. 1 Booster Station. This transformer would be mounted on a concrete pad but would not be connected to the primary bus system as it may be necessary to transport it if the transformer at No. 2 Booster Station should fail. Electrical power would be connected to the stand-by transformer to operate space heaters to prevent condensation. The cost of this spare transformer is included in the cost estimate.

Selection of Medium Voltage

Various combinations of voltages for the intake and booster pump motors were investigated as detailed in Project Memorandum V4191/16, Selection of Medium Voltage. The 4160 V rating was found to be the most economical and has been used for Preliminary Design.

Main Distribution

A 4160 V switchboard, located in the electrical room of each booster pumping station, would be fed from the low-voltage side of the incoming supply transformer and would, in turn, feed the electrical equipment. Subfeeders, protected by circuit breakers to avoid single phasing, would feed the nine unit substations along the pipeline. Current-limiting reactors, located in the electrical rooms, would be connected in series with these subfeeders to avoid having to oversize the cables to withstand the available short circuit level at the 4160 V bus.

At No. 1 Booster Station, the 4160 V switchboard would also feed the motor control centre located in the electrical room of the intake structure by a cable routed in a direct buried bank which would be concrete encased at roadway and service line crossings. This duct bank would also contain the cable for the remote control telemetering and communications systems.

The 4160 V distribution systems would be low-resistance grounded and the 600 V distribution systems would be effectively grounded to conform to established B.C. Hydro practice.

Emergency Power Supply

In the event of a prolonged power outage, several components of the water supply system would be in danger of freezing and could require emergency power for heating. Not all components could be protected by draining the line and even this remedy would be difficult to control without power. Consideration has been given to providing emergency power to heat the critical components. However, the Preliminary Design cost estimate does not include emergency power supply as the chance of a failure of the 4160 V system at the Power Plant is considered extremely remote. However, this aspect should be reviewed during Final Design.

COST ESTIMATE

Table 8 summarizes the capital cost estimate for the water supply project. A detailed breakdown of the cost estimate and cash flow schedule is contained in Appendix 5.

Table 8 - Summary of Cost Estimate

Dept.	Description	Material	Labour	Total
Structures				
271.00	Thompson River Intake	$ 1,070,000	$ 1,570,000	$ 2,640,000
272.00	Water Pipeline	6,665,000	8,870,000	15,535,000
273.00	No. 1 Booster Station	550,000	400,000	950,000
274.00	No. 2 Booster Station	615,000	1,140,000	1,755,000
Total Structures		$ 8,900,000	$ 11,980,000	$ 20,880,000
Equipment				
271.00	Thompson River Intake	$ 1,450,000	$ 330,000	$ 1,780,000
272.00	Water Pipeline	1,610,000	775,000	2,385,000
273.00	No. 1 Booster Station	2,500,000	930,000	3,430,000
274.00	No. 2 Booster Station	2,620,000	825,000	3,445,000
291.00	Power Supply & Distribution	1,750,000	595,000	2,345,000
Total Equipment		$ 9,930,000	$ 3,455,000	$ 13,385,000
Total Direct Cost		$ 18,830,000	$ 15,435,000	$ 34,265,000
Owner's Construction Overhead				2,740,000
Engineering				3,500,000
Contingencies				5,245,000
Total Construction Cost				$ 45,750,000
Corporate Overhead				2,250,000
Total Capital Cost				$ 48,000,000

BASIS OF ESTIMATE

General

The capital cost estimate in Appendix 5 has been prepared based on prices in effect during the fourth quarter of 1977.

Labour estimates include payroll costs up to foreman level, board and lodgings, contractor's supervision and administration, equipment rentals, overhead and profit. An average charge-out rate of $35 per hour has been used for all construction trades.

Material costs include Provincial Sales Tax of 7 percent, packing, freight to site and insurance. Federal Sales Tax is not applicable and thus is not included. Cost of land purchase and right-of-way acquisition is included at $2,500 per hectare, except $15,000 per hectare at the No. 1 Booster Station, as estimated by B.C. Hydro.

Civil and Structural

The structural cost estimates have been based on preliminary designs developed for the intake and booster stations. The costs have been estimated using unit prices for the various items of work from Sandwell's records.

The estimates for booster station concrete foundations have been based only on visual observations of ground conditions. Intake foundation estimates are based on borehole data and a seismic survey.

Equipment

Cost estimates for the major pieces of equipment have been based on written budget quotations received from suppliers. Costs for the auxiliary equipment wiring and process controls have been estimated from Sandwell's records. An allowance has been made for spare parts. While pipe costs have been based on budget quotations provided by suppliers, these tend to be the most unpredictable prices as pipe availability is greatly influenced by other major pipeline projects. A bare pipe price of $710 per tonne, landed in Vancouver, has been used.

Pipeline Construction

In the pipeline construction estimates, variations in wall thickness and ground slope have been considered insofar as welding, handling and anchoring costs are concerned. Clearing, grading, stockpiling, ditching, hauling and stringing, padding, bedding, and backfill costs have been estimated according to the characteristics of each section of pipeline route. The assumed ditching and clearing requirements are given in detail in Table C of Appendix 4. Costs of concrete bedding and borrowed material are included.

Certain critical unit costs for pipeline construction have been developed in consultation with Marine Pipeline Contractors Limited.

Costs of sundry pipe, valves and fittings for the pipeline include such items as air valves, access manholes and drain installations. Automatically-actuated valves have been included in process control equipment. Pipe bends are assumed to be field bends except where sharper bends necessitate shop-fabricated bends. Coating costs are based on suppliers' quotations. The cost of field coating joints is included as is the cost for hydrostatic testing of the pipeline.

Railway and major highway crossings are assumed to be installed in sleeves bored under the roadbed. River and stream crossings would be buried under the bed and have been priced according to their individual requirements.

The cost of two one-way surge tanks together with associated valves and fittings, is included.

Owner's Construction Overhead

An allowance of 8 percent of the total direct cost has been included for owner's construction overhead. This allowance is made for those items not normally included under contractor's overhead which the owner must provide. The estimates allow for such items as suppliers' erection supervision, warehousing, temporary structures, temporary services and camp maintenance.

Engineering

The allowance for engineering includes design development, site surveys and drilling, hydraulic modelling, waterhammer computer programs, specialist consultants, calling of tenders and recommendations of equipment suppliers and contractors, the preparation of specifications and detailed drawings for construction, and resident engineering supervision of contract work.

The engineering allowance is a preliminary estimate only. Upon project authorization, an estimate of manhour requirements, specialized consulting and other purchased services and out-of-pocket disbursements would be made to arrive at the final engineering budget.

Contingencies

The allowance for contingencies is intended to provide for unforeseen items which may not become apparent until Final Design. This contingency allowance is not intended to provide for escalation or inflation during the planning, design and construction period. A contingency allowance of approximately 15 percent of the total direct cost has been included.

Corporate Overhead

An allowance of 5 percent of the total construction cost has been made for B.C. Hydro's corporate overhead.

Escalation

The time basis for the material and labour estimates in this budget is the fourth quarter of 1977. No allowance has been made for the effects of inflation on labour rates or material prices after this date.

Premium Time

Unit prices for pipeline labour include premium time as part of normal pipeline practice. Premium time is also included for construction of the intake structure. Otherwise, the labour rates used in compiling these estimates are based on a 37.5 hour work week. Should B.C. Hydro & Power Authority authorize an extended work week, aside from the pipeline and intake work, the costs for premium time payments may exceed the $500,000 included in this estimate.

Exclusions

In summary, it should be noted that the cost estimate does not include:

1. Transmission lines up to the connection to the booster station substations.
2. Working capital or the cost of operating materials not initially supplied under the terms of equipment purchase.
3. Interest accrued during the construction period or any other costs of financing the project.
4. Startup and pre-operating expenses such as recruitment and training of personnel.
5. Federal sales tax.
6. Municipal taxes.
7. Legal expenses.
8. The cost of any townsite or housing facilities.
9. The cost of any other preliminary development activities carried out by B. C. Hydro and Power Authority.
10. The cost of overtime premium except for the premium time for intake and pipeline noted above.
11. The cost of escalation in material, labour and construction costs after the fourth quarter of 1977.
12. The cost of the reservoir or piping from the reservoir to the thermal plant.
13. Operating costs.
14. Allowances for future expansion.

ENGINEERING AND CONSTRUCTION

General

The Engineering and Construction Schedule is shown on Drawing D4191-30. The following key dates provide the time framework within which the schedule has been prepared.

Item	Date
1. Commence Final Design	1 January 1979
2. Project Construction Authorization (No tenders to be issued prior to this date)	1 April 1980
3. Commence Filling Plant Reservoir	1 July 1984
4. First Unit In-Service	1 January 1986

Engineering

The Final Design for the project requires the following critical pieces of information be developed within the allotted time:

1. Confirmation of pipeline route.

2. Topographic surveys and soil tests for pipeline and booster stations.

3. Determination of booster pump characteristics. Pump model tests to be completed by manufacturer before pipeline waterhammer design is commenced.

4. Hydraulic models of intake cofferdam and of a single intake cell.

Tendering and Manufacture of Equipment

Tendering for the pumps, as discussed above, is necessary at the earliest possible date, 1 April 1980, to provide the input necessary for waterhammer analysis. A modern pipe mill could produce the entire pipe order in about one month. However, manufacture may be delayed due to other commitments of the mill. For example, one major Canadian pipe mill is already booked to capacity from late 1978 through 1981. Therefore, it is advisable to complete tenders early, based on preliminary specifications in order to ensure space in a mill's production schedule. The waterhammer studies could be completed and the pipe specification finalized well before the actual manufacturing period.

Approximate delivery times for major equipment are as follows:

Item	Delivery
Intake Pumps and Motor	8 - 10 months
Booster Pumps and Motor	12- 14 months
Pipe,(Depending on availability)	12 months
Valves and fittings	10 months
Transformers	12 months

Tendering and Award of Construction Contracts

The engineering schedule permits the completion of detailed design so that all construction tenders can be called on a lump-sum basis.

The following 3 construction contracts are anticipated:

1. Intake. To include Bonaparte River bridge, access road, cofferdam, intake structure, intake access bridge, travelling crane and placement of travelling screens.

2. General. To include structures for booster stations, clearwell, clarifier, equalization tank, surge tanks, valve pits and the like as well as the complete mechanical and electrical installation.

3. Pipeline.

Intake Construction

Intake construction is the most severely restricted item in the schedule - both by river flow and by the spawning of pink salmon in the fall of each odd-numbered year. Since Project Construction Authorization would be after the winter of 1978-1979, low water periods available for construction are the 1980-1981 winter and the 1982-1983 winter.

Discussions with B.C. Hydro, Golder and Associates, and Dillingham Corporation identified the three cofferdam construction alternatives for the intake shown on Drawing D4191-29. For literature reviewed during the development of the following alternatives see References 4 and 5.

1. Cellular Cofferdam/Berms.

2. Sheet Piling.

3. Earthfill Berm.

Alternative 2, Sheet Piling, was rejected because the dimensions of a sheet-piled caisson around this intake, with allowance for working space, would require extensive use of struts and a relatively long construction period. Alternative 3, Earth fill berm, was rejected because the wide cross section could cause unacceptably high water velocities and could also adversely affect upstream water levels.

Alternative 1, Cellular Cofferdam/Berms, has been selected as it would give more reliable protection to the construction works and should not significantly affect the river hydraulics. While it may be more costly than Alternatives 2 and 3, it would minimize the construction time required.

Completion of the intake structure up to structural floor level may not be possible in one winter period. Therefore, the river construction phases have been scheduled over two winter terms. The following two schemes are available to complete the intake structure.

1. After constructing the cellular cofferdam and berms during the relatively low period towards the end of 1980 (commencing work at approximately the beginning of October), as much of the intake structure as possible would be constructed before the start of the freshet in 1981. With this scheme, the berms would be removed before the freshet permitting the construction works to be flooded. The cellular cofferdam would be retained since it is approximately the same width as the intake and would not, therefore, sufficiently affect the river hydraulics. In the winter of 1982, the berms would be reconstructed, the intake completed, and all temporary works removed.

2. The second scheme depends on the survival of the berms through a freshet and on the expectation that the berms would not significantly affect the river hydraulics. A model study would be needed to confirm these factors. If the berms could be left in the river, the intake could be completed in the winter of 1981 - 1982. Although this winter period is a salmon spawning period, the construction work, being inside an existing dam, would not have any detrimental effects on fish spawning grounds in the Thompson River.

For the Preliminary Design Study the first scheme, requiring removal and reconstruction of the side berms between periods of construction, has been selected as it minimizes risks. During Final Design, the best scheme has to be developed based on a detailed construction schedule, model studies, risk considerations and economics.

All river construction would be carried out in such a way as to minimize siltation.

To facilitate construction of the intake, the access road and the bridge over the Bonaparte River are planned to be completed during the 1980 - 81 winter. The intake superstructure may be completed using a mobile crane on the river bank or on a falsework. The crane would be used to assemble the structural steel framework including the intake travelling crane and also to install the precast wall, roof, and bridge deck units. The screens, pumps, pipework, and ancillary equipment for the intake would then be positioned using the travelling crane.

Pipeline Construction

The Ashcroft climate would permit winter pipeline construction. However, the construction is scheduled for the summer of 1983. Construction methods typical to pipelines all over the world would be employed. After clearing, grading and trenching, the pipe lengths would be strung along the route, bent to fit the trench, and butt welded together on wooden blocks alongside the trench. After completion of coating at joints, and thorough inspection and approval of welds and coating, the pipe would be lowered into the trench in long sections. Backfilling, pressure testing and right-of-way clean-up would follow.

The rate of construction progress would be established by the rate of field welding. This rate has been estimated by Marine Pipeline Contractors at about 50 joints per day or 600 to 750 m per day. Including mobilization and clean-up, about three months would be needed to complete pipeline construction. For details of construction conditions such as soil, clearing and access along the pipeline route, see comments under PIPELINE ROUTE.

The section of the pipeline crossing the Bonaparte river would be constructed concurrently with the intake, in the 1982-1983 winter. High flows in the river would preclude construction during the summer of 1983, at the same time as the rest of the pipeline. Without diverting the river, the trench would be dug and the pipe weighted with a concrete jacket and placed.

Booster Station Construction

The booster stations would be constructed using standard techniques and materials and thus should not pose unusual problems. The installation of pumps and motors requires care but such installations are common.

Roads to the area of No. 1 Booster Station exist already and would be improved as necessary. Access to the No. 2 Booster Station would require upgrading and extension of existing forest roads to permit passage of large vehicles for construction and equipment delivery.

Prepared by

A. Copeland, P. Eng.

B. R. McConachy, P. Eng.

Approved by

Sandwell and Company Limited

SANOWELL

APPENDIX 1

GLOSSARY OF TERMS

(V4191/1)

APPENDIX 1 - GLOSSARY OF TERMS

Terms And Some Abbreviations

anchor ice	Ice which forms on the bottom and on channel obstructions such as gates, bars, screens and sills in swiftly flowing river water.
API	American Petroleum Institute
ASA	American Standards Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
AWWA	American Waterworks Association
carbon equivalent	As defined in the Canadian Standards Association pipe code, a number representing the fraction of carbon, plus other elements which behave like carbon, in steel.
cellulose electrodes	A type of electrode used for shielded metal-arc pipeline welding.
CSA	Canadian Standards Association
dBA	Decibels on the "A" scale - a measure of sound pressure level.
DC	Direct Current
frazil ice	Small flakes of ice formed in supercooled river water and transported in suspension.
fps	feet per second.
Gas Metal-Arc Welding	Welding using a continuous electrode and an externally-supplied inert gas for shielding.
holidays	Thin spots or holes in a coating.
hp	horsepower
Hz	hertz
J	joule

KPa	kilo-Pascal
kV	kilovolt
kVA	kilovolt-ampere
kW	kilowatt
l/s	liter per second
martensite	A brittle microstructure constituent of certain steels.
MCC	Motor control centre.
m/s	metre per second
m^3/s	cubic metre per second
ma	milli-ampere
mg/l	milligram per liter
micron	One thousandth of a mm.
mill	One thousandth of a dollar
MPa	Mega-Pascal
MVa	megavolt-ampere
ND 10,25,40	DIN Standard for nominal pressure rating of pipe fittings in kp/cm^2.
NPSH	Net positive suction head. The absolute pressure required on the suction side of a pump to ensure its operation without cavitation problems.
ONAN	Oil natural air natural
pigs	Instrument packages which travel through a pipeline propelled by the flow for inspection or cleaning purposes.
ppm	Parts per million
psi	pounds per square inch.
psig	pounds per square inch gauge.
rpm	revolution per minute.
RTD	Resistance temperature detector.

Submerged-Arc Welding	Welding in which the arc is shielded by a blanket of fusible granular material called flux.
Shielded metal-Arc Welding	Welding using a stick electrode, shielded by the electrode covering.
USAS	USA standard.
USBR	United States Bureau of Reclamation.
USGPM	U.S. gallons per minute.
V	Volt
Waterhammer	The waves of pressure which travel in a pipeline when changes in flow occur.

Metric Units

Quantity	SI* Unit	Abbreviation	Equivalent Imperial Unit	
Length	millimetre	mm	0.03937	inch
	centimetre	cm	0.3937	inch
	metre	m	3.28	feet
			39.37	inches
	kilometre	km	0.6214	mile
			3280	feet
Area	square metre	m^2	10.87	square feet
	hectare	ha	2.471	acres
Volume	cubic metre	m^3	35.314	cubic feet
			264.17	US gallons
	liter	l	.2642	US gallon
Discharge Rate	cubic metre per second	m^3/s	35.314	cubic feet per second
	liter per second	l/s	15.852	US gallons per minute
Force	newton	N	0.2248	pounds
Mass	tonne	t	2207	pounds
	kilogram	kg	2.207	pounds
Pressure	Pascal	Pa	0.000145	pounds per square inch
	Kilo-Pascal	KPa	0.145	pounds per square inch
	Mega-Pascal	MPa	145	pounds per square inch
Power	kilowatt	kW	1.34	horsepower
Velocity	metre per second	m/s	3.28	feet per second
Inertia	kilogram metre squared	$kg\text{-}m^2$	0.737	slug-feet squared

* International System of Units, as adopted by the Canadian Construction Industry.

APPENDIX 2

REFERENCES

APPENDIX 2 - REFERENCES

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19. California State Water Project, Bulletin No. 200, six volume set published by California State Department of Water Resources, Sacremento, California, 1974.

20. B.C. Hydro and Power Authority, "500 kV Line in Common Corridor With Water Pipeline", inter-office memo from R.M. Shier to D.M. McLeod, 11 July 1977.

APPENDIX 3

TERMS OF REFERENCE

HAT CREEK PROJECT

WATER SUPPLY STUDY - TERMS OF REFERENCE

A. GENERAL

Provide engineering services for the preliminary design of the water supply system for the Hat Creek Project. The work shall be based on a thermal plant located at Harry Lake Site B with the water delivered to the water storage reservoir located approximately 2 km to the southeast. The source of water shall be the Thompson River with an intake in the vicinity of Ashcroft. Design discharge for the water supply system shall be 1.58 m^3/s (25,000 USGPM).

The work shall include:

1. Review of the conceptual design studies.
2. Comparative engineering studies of alternative pipeline routes, intake sites and pumping systems.
3. Preliminary design of the recommended water supply scheme, including a detailed cost estimate and construction schedule.

The study shall be scheduled to meet the following completion dates:

1. Status memorandum to be prepared and completed by 1 September 1977.
2. Feasibility of the recommended scheme confirmed by 1 November 1977.
3. Description of the recommended scheme and information required to assess the associated environmental implications to be provided to the Environmental Consultants as required to meet the 1 November 1977 publication date for the Environmental Impact Assessment Report.
4. Draft of Final Report submitted to B.C. Hydro prior to 15 January 1978 and Final Report completed by 1 March 1978.

A. GENERAL - (Cont'd)

A detailed program and schedule, and an itemized budget estimate showing work item, the expected number of manhours to complete, and the estimated cost are required by 1 June 1977.

During the course of the work, monthly reports detailing study progress and costs will be required.

Progress meetings are to be held with B.C. Hydro as required to ensure coordination of all aspects of the study.

S.I. units are to be used exclusively for this study, including all design calculations, mapping and reports. Imperial equivalents of principal data and dimensions are to be shown in brackets immediately following the metric value on all drawings and reports.

All drawings are to become the property of B.C. Hydro.

B. SCOPE OF WORK

Intake

1. Study alternative intake sites along the Thompson River between Ashcroft and the C.N.R. railway bridge upstream of the Bonaparte River confluence (Site 10).

2. To facilitate a possible pipeline route within the proposed access road right-of-way along Cornwall Creek, the technical feasibility of an intake site downstream of Ashcroft between the 105 mile Post Indian Reserve 2 and Cheetsum's Farm Indian Reserve (Site 16) is to be reviewed.

3. Before proceeding with an assessment of the feasibility of an indirect river intake, establish through discussions with the Canada Department of Environment - Fisheries and Marine Service (discussions to include both engineers and biologists from the Fisheries Service) whether or not a direct intake, incorporating the latest design developments and thoroughly model tested to minimize environmental hazards, would be approved.

B. SCOPE OF WORK - (Cont'd)

Intake - (Cont'd)

4. Select the type and location of the Thompson River intake and confirm with subsurface exploration.

5. Provide information to the Environmental Consultants regarding fisheries aspects of the intake design, as required in Appendix B3 items 3, 4 and 5 of the Detailed Environmental Studies, Terms of Reference, Hat Creek Project. This information is required by 1 September 1977 for inclusion in the Environmental Impact Assessment Report.

Pipeline Route

1. Study alternative pipeline routes including the possibility of combining the pipeline route within the proposed transmission line corridor as defined in the 500,000 Volt Transmission Line, Kelly Lake Substation to Nicola Substation, Phase I Route Selection Study by Ian Hayward and Associates. The possibility of a tunnel route through "Unnamed Ridge" should also be considered, and the possiblity of a pipeline route around the east side of "Unnamed Ridge" should be reviewed.

2. A possible pipeline route within the proposed access road right-of-way along Cornwall Creek should be considered.

3. Select the pipeline route and conduct geotechnical investigations to determine overburden depth and slope stability to confirm pipeline location and provide data for cost estimates.

4. Review and confirm the selection of a buried pipeline, and review alternatives to the electrical heating provisions proposed in the Conceptual Design study.

5. Determine availability and delivery time for the proposed pipe.

Pumping System

1. Review the hydraulics, costs and construction of a single lift high pressure pumping scheme as envisaged in the Conceptual Design study, and confirm the feasibility of the use of extraordinary pipe thickness and welding techniques.

2. Study alternative pumping schemes and pump station locations and compare with the Conceptual Design scheme, including
   - possibility of multiple lift pumping stations to reduce pipeline pressures.
   - location of single lift pumping station at Boston Flats combined with higher lift vertical turbine pumps at the intake.

3. Select preferred pumping scheme/pipeline route combination and complete preliminary design, including
   - confirmation of most suitable method to obtain necessary NPSH requirements.
   - selection of the water treatment system to remove suspended solids for pump protection, if required.
   - determination, in cooperation with personnel of the Hydraulics Section of B.C. Hydro and using the computer program for analysis of hydraulic transients developed by B.C. Hydro, of required waterhammer protection provisions.
   - determination of the availability and delivery time of the main mechanical and electrical equipment items selected.

Drawings/Cost Estimate/Construction Schedule

1. Prepare the preliminary engineering drawings required, in sufficient detail to fully define all major components of the water supply system.

2. Prepare a detailed cost estimate based on 1977 dollars, and prepare a detailed construction schedule for the water supply system.

LJP:na

17 June 1977

APPENDIX B3
WATER INTAKE

1. Assemble information on and evaluate the Thompson River as a water supply source considering such fish factors as:
   - timing of fry/juvenile downstream migrations by species
   - size, behaviour of downstream migrants and also sufficiency of supply of water and water quality.

2. (a) Consider impact of reduced river flow on downstream ecology of the Thompson River.

   (b) Identify any probable options for future use of Thompson River water which are foreclosed by this development and advise.

3.* Discuss intake design, location and screening with reference to Canada Department of Environment, Fisheries and Marine Service recommendations and requirements. Explain the critical factors which protect against:
   - entrainment of migrating fish
   - creating a haven for predators
   - disorientation, and
   - clogging of intakes

4.* Evaluate the experience of Lornex Mines intake for reference in the Hat Creek intake design.

5.* Indicate the reasons for the selection of the Thompson River as preferred source of water for the Hat Creek Project.

---

* To be provided by consultants carrying out the preliminary engineering for the water supply.

APPENDIX 4

TABLES

APPENDIX 4 - TABLES

TABLE A - SOME RELEVANT WATER AND PIPELINE PROJECTS

Name of Facility	Country	Date	Type*	Head (m)	Discharge (m^3/s)	Comments	Reference**
Sar Cheshmeh Copper Mine	Iran	1977	W-S	1082 Total	0.75	1 pumping station 48 km Steel pipeline	1
Tijuana Water Supply	Mexico	1977	W-S	1090 Total	4.00	6 pumping stations in 35 km.	2
Trans Ecuadorian Oil Pipeline	Ecuador	1972	O-P	3800 Total	0.46	5 pumping stations in 200 km.	3
Lornex	Canada	1972	W-S	1614	0.50	1 lift; pipeline.	4
Edmonston (California Water Project)	USA	1970	W-S	820	100.	1 lift; tunnel.	5
Trans Andean Oil Pipeline	Colombia	1968	O-P	3200 Total	0.01	4 pumping stations 1220 m max. lift.	6
Caracas Water Supply	Venezuela	1968	W-S	1210 Total	7.20	3 pumping stations in 30 km.	7
Hongrin	Switzerland	1965	P-S	850	6.40		4
Bougainville Copper Project	Australia	?	W-S	790	1.4	Steel pipeline.	4
Nippon Coal Mining Co.	Japan	?	D	690	0.40	Mine Drainage.	4
Haas	USA	1958	H	795	21.5	Steel penstocks.	8
Balch Addition	USA	1958	H	730	15.9	Steel penstocks.	8
Gougra-Mottec	Switzerland	1955	P-S	628	3.26		4
Lünersee	Austria	1954	P-S	971	3.73	Tunnel.	4

APPENDIX 4 - TABLES

TABLE A - SOME RELEVANT WATER AND PIPELINE PROJECTS

Name of Facility	Country	Date	Type*	Head (m)	Discharge (m^3/s)	Comments	Reference**
Kemano	Canada	1954	H	870	57.0	Tunnel.	9
Aussois	France	1950	H	860	?	Banded steel penstock.	10
Dixence (Chandoline)	Switzerland	1952	H	1750	10.3	Banded steel penstocks.	10
Bear R.	USA	1952	H	640	5.7	Steel penstocks.	8
Serra No. 4,5,6	Brazil	1946 -1949	H	772	?	Multi-layer steel penstocks.	11
Alto Chiese	Italy	?	H	739	34.0	Banded steel penstocks.	10
Portillon	France	?	H	1413	?		10
Tremorgio	Italy	?	H	900	0.40	Tunnel.	10

* H = Hydroelectric Power Plant
I = Irrigation
O-P = Oil Pipeline
P-S = Pumped - Storage
W-S = Water Supply

** For References, see attached page 3.

APPENDIX 4 - TABLES
TABLE A - SOME RELEVANT WATER AND PIPELINE PROJECTS

REFERENCES

1. Scivier, J.B., and R.W. Riley, "Water Supply Scheme in Iran, with 1100 m Pumping Head", J. Inst. Water Engineers and Scientists, V. 31, No. 6, November 1977.

2. "Mexican Aqueduct Challenges High Sierras", Engineering News Record, 11 August 1977.

3. Franco, "Trans-Ecuadorian Pipeline Conquers the Andes", Oil and Gas Journal, 15 May 1972.

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6. Love, "Trans-Andean Pipeline - High and Rough", Pipeline Engineer, July 1968.

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APPENDIX 4 - TABLES

TABLE B - COMPARISON OF TRASH RACK RAKES

	Advantages			Disadvantages		
	Standard Rake		Custom Design Rake	Standard Rake		Custom Design Rake
Item	Case 1	Case 2		Case 1	Case 2	
Guides	-	Guides flush with Structure	Guides inside Structure	Guides protrude from Structure		-
Mechanical Rake	-	-	Mechanical Rake inside Structure	Mechanical Rake outside Structure	Mechanical Rake outside Structure	-
Trash Removal	-	-	Trash swept back into river by custom-designed rake profile	Trash disposal system required	Trash disposal system required	-
Design	Guides & Mechanical Rake are standard	-	-	-	Guides detail non-standard	Guides & Mechanical Rake non-standard
Access & Maintenance	-	-	Guides & Mechanical Rake accessible for maintenance	Guides & Mechanical Rake not readily accessible for maintenance	Guides & Mechanical Rake not readily accessible for maintenance	-
Aesthetics	-	More aesthetic than Case 1	More aesthetic than Cases 1 & 2	Less aesthetic than Case 3	Less aesthetic than Case 3	-
	-	-	Less headroom required than Custom Rakes	More headroom required than Custom Rakes	More headroom required than Custom Rakes	

Reference: Drawing D4191-8, Appendix 6.

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

DATE MARCH 1978

APPENDIX 4 - TABLES
TABLE C - ASSUMED DITCHING AND CLEARING REQUIREMENTS

From Station	To Station	Depth To Rock (m)	Clearing
0+300	2+000	very deep	sparse to open
2+000	3+500	0	sparse to open
3+500	8+000	very deep	sparse to open
8+000	8+500	1	sparse to open
8+500	8+800	0	heavy
8+800	9+600	1	sparse
9+600	10+500	1	very heavy
10+500	11+000	0	very heavy
11+000	12+000	1	very heavy
12+000	13+300	2	very heavy
13+300	13+700	2	open
13+700	13+800	0	open
13+800	14+200	2	open
14+200	18+500	2	heavy to very heavy
18+500	19+500	1	heavy to very heavy
19+500	20+500	1	open
20+500	22+000	2	heavy
22+000	22+500	2	open
22+500	23+090	2	heavy

Note: In the absence of test pits and boreholes, rock conditions are unknown at this time.

APPENDIX 4 - TABLES
TABLE D - PIPE DESIGN CODE AND PRACTICE COMPARISON

Authority	Ref.(g)	Design Condition	Internal Pressure: Based on Yield	Internal Pressure: Based on Ultimate	Type of Service	Collaprse Design
ASME	14	most severe condition of normal operation	-	F.S.(f)= 4(b,c)	Pressure Vessels	F.S. = 4
USAS	15	maximum design pressure	.6 fy(a,b)	-	Gas Pipelines	minimum thickness = 6 mm for 900 mm pipe
ASA	16	surge included	.72 fy(b)	-	Oil Pipelines	not stated
AWWA	17	surge included	.5 fy	-	Water Pipelines	1.5-2
		extreme conditions	.75 fy	-		
USBR	18	normal condition & surge	.67 fy	F.S. = 3(e)	Penstocks and water pipelines	
		intermittent condition	.8 fy	F.S. = 2.25		not stated (d)
		emergency condition	1.0 fy	F.S. = 1.5		1.5 used on embedded penstocks
		exceptional condition	-	F.S. = 1.0		

a. Value varies with population density from .4 - .72. The value of .6 corresponds to Class 2 population density. (Ashcroft suburb).

b. Joint efficiency 100% used, consistent with 100% x-ray examination and submerged arc welding.

c. Applies to welded alloy steel pressure vessel.

d. Implied that collapse and internal pressure factors of safety equal?

e. Whichever is more stringent, yield criteria or ultimate criteria, governs.

f. fy = yield stress of steel; F.S. = factor of safety.

g. For references, see Appendix 2 of this report.

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

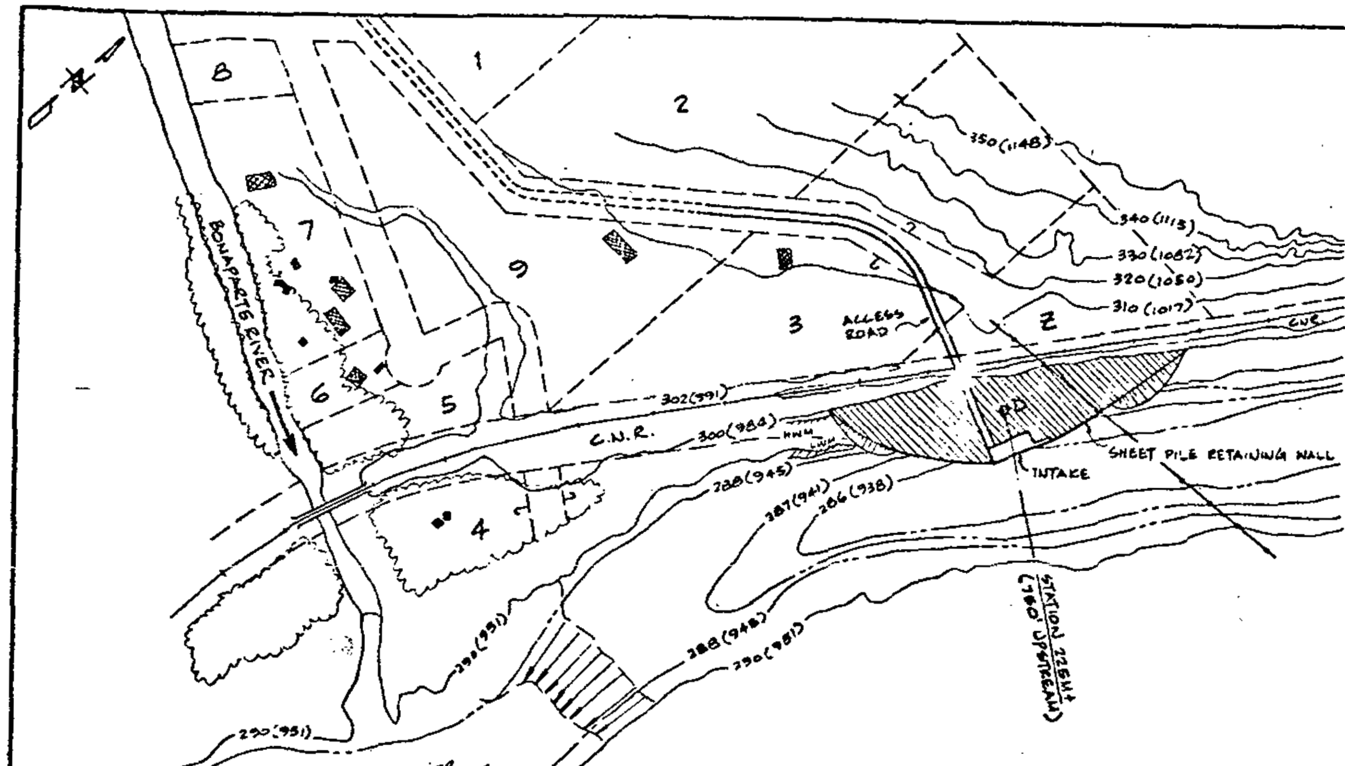
PRELIMINARY DESIGN STUDY

DATE MARCH 1978

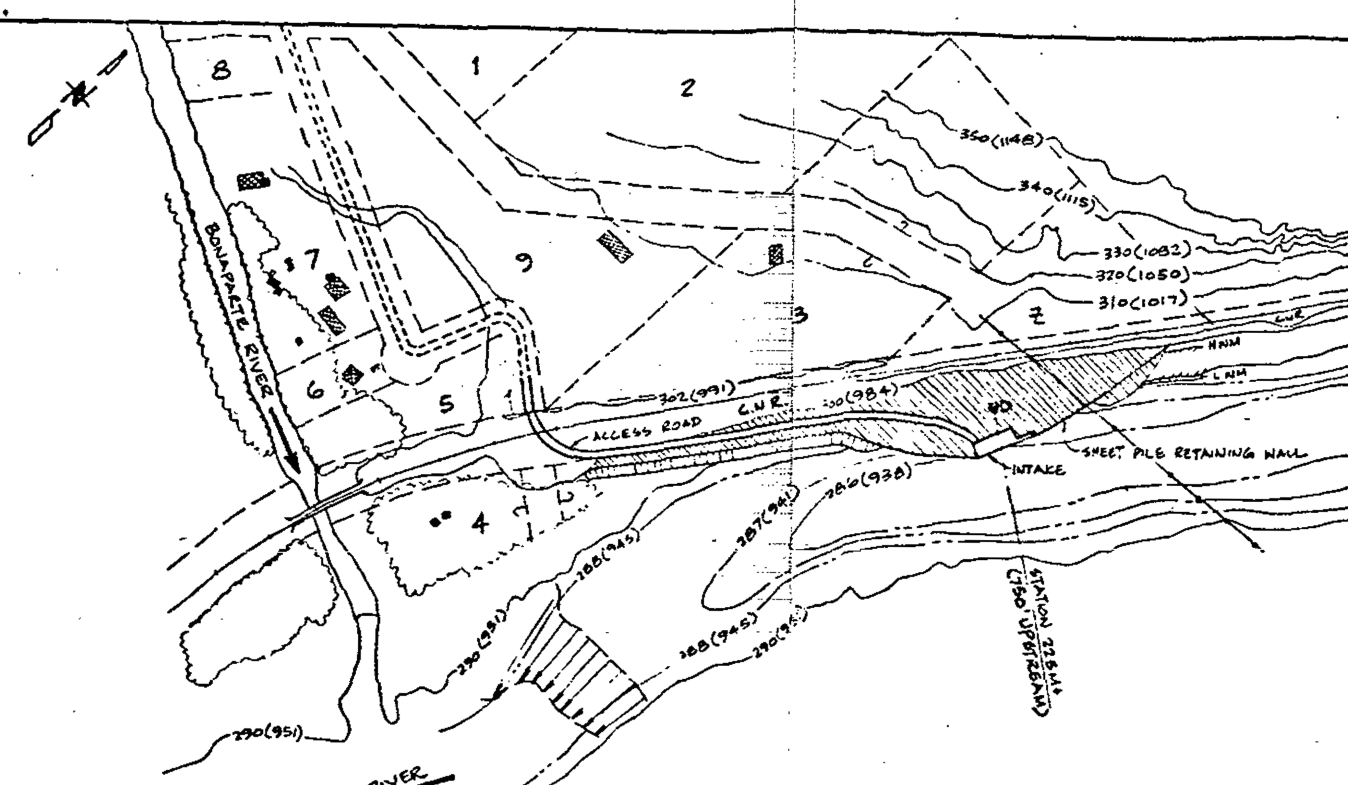
APPENDIX 4 - TABLES
TABLE E - COST COMPARISON OF ALTERNATIVE INTAKES

	Alternative 1	Alternative 2	Alternative 3
Item	Bank	Bank	Pier
Excavate Bank Material	$ 60,000	$ 75,000	$ -
Remove Material Off Site	110,000	125,000	50,000
Infill Material	460,000	574,000	20,000
Sheet Piling	188,000	188,000	167,000
Pile Bracing	—	—	101,000
Road Excavation	6,000	7,000	5,000
Road Laying (Includes Real Estate Allowance)	26,000	31,000	21,000
Access Bridge	—	—	176,000
River Work Contingencies	—	—	100,000
Total	$850,000	$1,000,000	$640,000

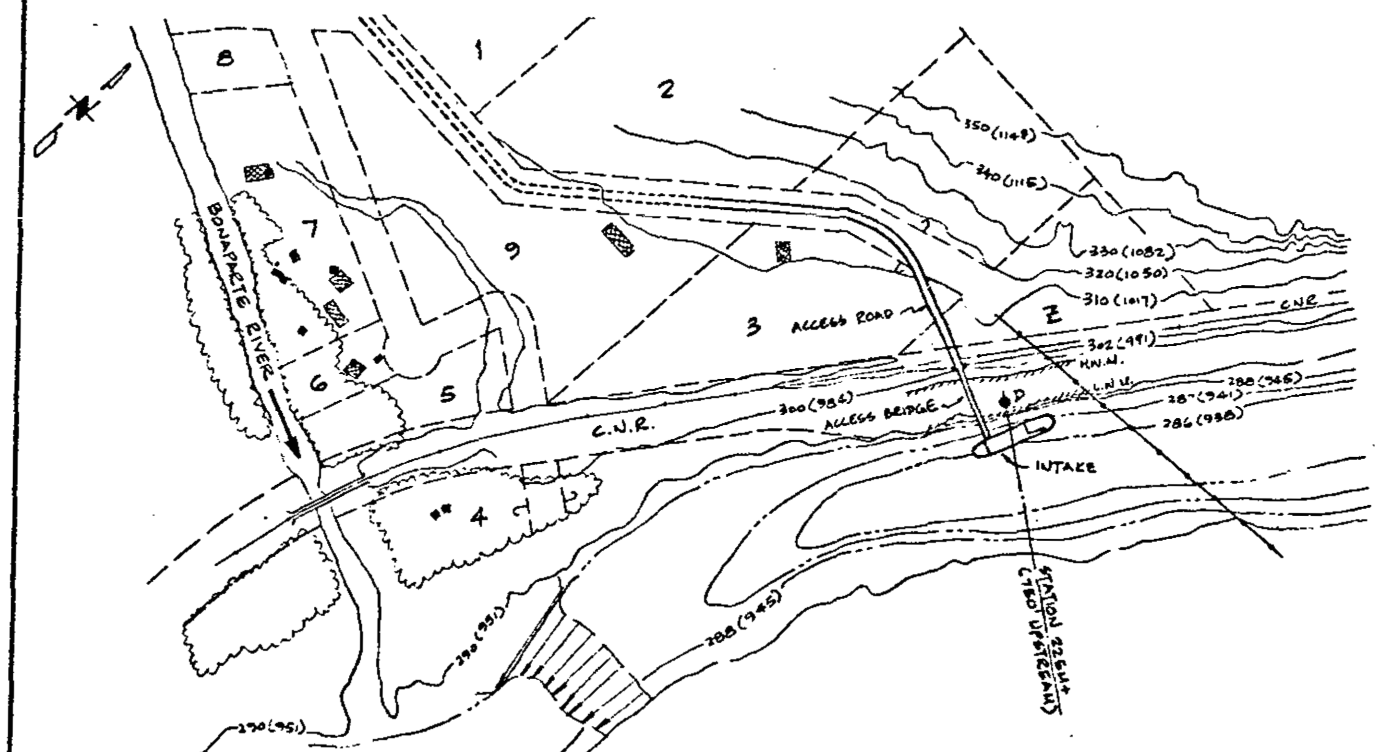
Note: 1. Only items which vary are listed in the above table.
2. For layouts of alternatives see attached Drawing D4191/4E-1



BANK INTAKE — ACCESS THROUGH LOTS 3 & Z
ALTERNATIVE 1 — NOT RECOMMENDED



BANK INTAKE — SHORELINE ACCESS THROUGH C.N.R.
ALTERNATIVE 2 — NOT RECOMMENDED



PIER INTAKE — ACCESS THROUGH LOTS 3 & Z
ALTERNATIVE 3 — RECOMMENDED

NOTE.
ALL CONTOURS ARE IN METRES AND (FEET).

LEGEND.

1. APPROXIMATE POSITION OF HIGHEST KNOWN WATER LEVEL.
2. APPROXIMATE POSITION OF LOWEST KNOWN WATER LEVEL.
3. RIVER BED CONTOUR POSITIONS OBTAINED FROM DATA FROM NORTHWEST HYDRAULIC CONSULTANTS LTD.
4. CONTOURS OBTAINED FROM DATA FROM MCELHANNEY SURVEYING & ENGINEERING LTD.
5. FILL BEHIND SHEETPILED WALL
6. DWELLINGS
7. EXISTING TRANSMISSION LINE
8. MEANS SAME OWNER EACH SIDE OF ACCESS ROAD CORRIDOR.
9. EXISTING ROAD
10. EMBANKMENT
11. PROPERTY BOUNDARIES 1 THROUGH 9 OBTAINED FROM DRAWING 92.1.11.F.51 (PRODUCED BY WESTERN MAPPING, KAMLOOPS).

50 0 50 100 150M.
100 0 100 200 300 400 500 FT.
1:2000

SANDWELL.	DWG. NO. D4191/4E-1		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
DES		RECD	
DWN	D TUNG	APPD (TECH)	HAT CREEK PROJECT
CHKD	JWCB	APPD	COOLING WATER SUPPLY
INSP		APPD	PRELIMINARY DESIGN STUDY
SUBD		DATE	MARCH 1978
MICROFILMED		JOB No. V4191	CONTRACT No.

INTAKE SITE 100 - INTAKE TYPE AND ACCESS ALTERNATIVES

SCALE 1:2000 | DWG No. D4191/4E-1 | R

REFERENCE DRAWINGS | REMARKS | DATE | MADE | CHKD | INSP | SUBD | RECD | APPD | APPD
REVISIONS

APPENDIX 4 - TABLES

TABLE F - INTAKE DESIGN REFINEMENTS

Refinements Recommended by Beak	Sandwell Response
1. To the extent possible, eliminate strong lights within the bypass channel and on the exterior structure.	1. There will not be any artificial lights in the bypass channel itself. Although access openings are provided at the top of the channel, these could be covered to prevent the artificial light in the intake superstructure from entering the channel. Only the access bridge would be provided with exterior lighting.
2. Within the bypass channel provide for a minimum turbulence, still water, or eddies by eliminating projection into the bypass channel; flush mounting travelling screens; and reducing, rearranging, and/or streamlining cross members.	2. Although the use of flush-mounted travelling screens is desirable, they are still in the development stage. Therefore, conventional screens have been used. All projections into the bypass channel have been streamlined.
3. Maintain a watered area between the intake and the shoreline of at least 3 m (10 ft) in horizontal distance.	3. Current design satisfies this proposal.
4. Provide for the use of stop logs, slats, or baffles behind the travelling screens which could be used to spread the approach velocity more evenly across the screen face.	4. Provision has been made for use of stop logs to isolate cell. The need to spread the approach velocity will be determined in Final Design with a hydraulic model of one intake cell.
5. Make provisions for travelling screens which have either (a) no horizontal trays or lips; (b) an inclined plane lip angled at approximately 45° to horizontal with no groove at the intersection of lip and screen face; (c) hooks rather than horizontal trays.	5. The proposed design incorporates conventional travelling screens. These have a horizontal tray at the joint between adjacent screen baskets. This tray has the dual function of carrying debris up the screen and forming a seal with the boot which is a curved plate at the bottom of the screen. a. Elimination of the horizontal tray would remove the screens ability to: - Carry up debris and frazil ice. - Effectively clean the boot seal. b. Installation of an inclined plane lip would remove the screens ability to: - Carry up debris and frazil ice. c. Hooks rather than horizontal trays would remove the screens ability to: - Carry up frazil ice. - Seal the bottom. The best solution would be a combination of b and c, whereby the inclined plane lip would be installed as a removable filler piece. This system would retain the screens ability to: - Carry up debris. - Clean the boot seal. - Carry up frazil ice by removing the filler pieces in winter when salmon fry are not present in the river.
6. Provide the downstream end of the structure with a more streamlined shape.	6. The present rounded downstream end of the intake pier accounts for 6 m (20 ft) of the total 45 m length. The cost-effectiveness of increasing the length of the downstream structure is questionable. No change made.

APPENDIX 4 - TABLES

TABLE F - INTAKE DESIGN REFINEMENTS (CONT'D)

Refinements Recommended by Beak	Sandwell Response
7. Use the reversed orientation of the upstream trash racks or portion of it to reduce bypass velocity only when it serves to reduce turbulence and eddies within the bypass channel or serves to exclude a high proportion of downstream migrants from entering the bypass channel by serving a louver system. This procedure should not be followed if it reduces bypass velocity below approximately 27 cm/sec (0.9 ft/sec).	7. Agreed.
8. A structure such as a curtain wall on the upstream approach section of trash racks should be used to exclude surface oriented downstream migrants only if it can be shown that this will not result in any back-flow of water in the bypass channel or reduction in bypass velocity to less than approximately 27 cm/sec (0.9 ft/sec).	8. The proposed curtain wall could create a pocket where migrating fish could become entrapped. No action taken.
9. An upstream approach section which places the trash rack section with 30 cm (12 in.) interbar spacing parallel to flow is recommended provided that a 1 m (3 ft) turning radius be allowed for large upstream migrants and that model studies indicate a downstream flow of water through this section.	9. This refinement, shown as Detail X on Drawing D4191-11, is described in the report (pg.14) as an alternative to increasing the spacing of the trash rack bars closest to the inside face of the bypass as a model study of the proposed refinement would be required during Final Design.

APPENDIX 5

DETAILS OF COST ESTIMATES

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

PRELIMINARY DESIGN STUDY

DATE MARCH 1978

APPENDIX 5 - DETAILS OF COST ESTIMATES

STRUCTURES

Department 271 - Thompson River Intake		Material	Labour	Total
271.11	Excavation and Fill 17/	$ 80,000	$ 390,000	$ 470,000
271.12	Piling 13/	40,000	275,000	315,000
271.13	Concrete 53/	600,000	525,000	1,125,000
271.14	Structural and Misc. Steel 100/	60,000	-	60,000
271.16	Precast Siding 46/	90,000	105,000	195,000
271.18	Roofing 33/	5,000	10,000	15,000
271.21	Interior Finishing 0/	-	5,000	5,000
271.22	Painting 100/	5,000	-	5,000
271.23	Interior Fire Protection 100/	5,000	-	5,000
271.24	Plumbing and Roof Drains 33/	5,000	10,000	15,000
271.25	Heating and Ventilating 50/	10,000	10,000	20,000
271.26	Lighting 33/	5,000	10,000	15,000
271.27	Apertures 50/	5,000	5,000	10,000
271.28	Insulation 33/	15,000	30,000	45,000
271.65	Seal Water Pipeline 36/	40,000	70,000	110,000
271.90	Land Costs	-	Included in	271.94 -
271.91	Fencing 50/	5,000	5,000	10,000
271.94	Access Roads 50/	10,000	10,000	20,000
	Sub-Total, Intake	$ 980,000	$ 1,460,000	$ 2,440,000
	Access Bridge			
271.11	Excavation and Fill	15,000	50,000	65,000
271.12	Piling	10,000	5,000	15,000
271.13	Concrete	65,000	55,000	120,000
	Sub-Total, Access Bridge	$ 90,000	$ 110,000	$ 200,000
Total, Department 271		$ 1,070,000	$ 1,570,000	$ 2,640,000

Department 272 - Water Pipeline		Material	Labour	Total
272.62	Clearing 0/	$ -	$ 115,000	$ 115,000
272.63	Grading 0/	-	295,000	295,000
272.64	Stockpile 0/	-	25,000	25,000
272.65	Pipe 100/	4,880,000	-	4,880,000
272.66	Haul and String 7/93	25,000	315,000	340,000
272.67	Trenching 0/100	-	3,400,000	3,400,000
272.68	Dewatering 16/84	40,000	210,000	250,000
272.69	Bending 0/100	-	510,000	510,000
272.70	Line-up 0/100	-	525,000	525,000
272.71	Welding 0/100	-	450,000	450,000
272.72	Patch Joints 21/79	60,000	220,000	280,000
272.73	Anchors 33/67	10,000	20,000	30,000
272.74	Lower-in and Tie-in 0/100	-	640,000	640,000
272.75	Bedding 48/52	175,000	190,000	365,000
272.76	X-Rays 100/0	75,000	-	75,000
272.77	Testing - Hydro and Pig 0/100	-	120,000	120,000
272.78	Backfill 17/83	40,000	195,000	235,000
272.79	Crossings - Road and Gaslines 26/74	30,000	85,000	115,000
272.80	Crossings - Railroad 33/67	10,000	20,000	30,000
272.81	Crossings - Stream 31/69	80,000	175,000	255,000
272.82	Clean-up and Hydro-Seeding 0/100	-	195,000	195,000
272.86	Drainage Pipelines 26/74	335,000	970,000	1,305,000
272.87	Access Manholes 80/20	40,000	10,000	50,000
272.88	Pig Traps 81/19	745,000	180,000	925,000
272.90	Land Cost 100/0	120,000	5,000	125,000
Total, Department 272		$ 6,665,000	$ 8,870,000	$ 15,535,000

Department 273 - No. 1 Booster Station		Material	Labour	Total
273.11	Excavation 0/100	$ -	$ 5,000	$ 5,000
273.13	Concrete 55/45	60,000	50,000	110,000
273.14	Structural and Misc. Steel 69/31	125,000	55,000	180,000
273.16	Precast Siding 45/55	75,000	90,000	165,000
273.18	Roofing 60/40	15,000	10,000	25,000
273.21	Interior Finishing 50/50	5,000	5,000	10,000
273.22	Painting 0/100	-	5,000	5,000
273.23	Interior Fire Protection 33/67	5,000	10,000	15,000
273.24	Plumbing and Roof Drains 50/50	10,000	10,000	20,000
273.25	Heating and Ventilating 75/25	30,000	10,000	40,000
273.26	Lighting 40/60	10,000	15,000	25,000
273.27	Apertures 50/50	5,000	5,000	10,000
273.28	Insulation 40/60	20,000	30,000	50,000
273.29	Furniture and Fixtures 100/0	5,000	-	5.000
273.86	Drainage Pipelines 36/64	50,000	90,000	140,000
273.90	Land Cost 100/0	105,000	-	105,000
273.91	Fencing 100/0	15,000	-	15,000
273.94	Access Roads 60/40	15,000	10,000	25,000
Total, Department 273		$ 550,000	$ 400,000	$ 950,000

Department 274 - No. 2 Booster Station		Material	Labour	Total
274.11	Excavation 0/100	$ -	$ 10,000	$ 10,000
274.13	Concrete 54/46	95,000	80,000	175,000
274.14	Structural and Misc. Steel 69/31	125,000	55,000	180,000
274.16	Precast Siding 45/55	75,000	90,000	165,000
274.18	Roofing 60/40	15,000	10,000	25,000
274.21	Interior Finishing 50/50	5,000	5,000	10,000
274.22	Painting 0/100	-	5,000	5,000
274.23	Interior Fire Protection 33/67	5,000	10,000	15,000
274.24	Plumbing and Roof Drains 50/50	10,000	10,000	20,000
274.25	Heating and Ventilating 75/25	30,000	10,000	40,000
274.26	Lighting 40/60	10,000	15,000	25,000
274.27	Apertures 50/50	5,000	5,000	10,000
274.28	Insulation 40/60	20,000	30,000	50,000
274.29	Furniture and Fixtures 100/0	5,000	-	5.000
274.86	Drainage Pipelines 44/56	35,000	45,000	80,000
274.90	Land Cost 100/0	5,000	-	5,000
274.91	Fencing 50/50	5,000	5,000	10,000
274.93	Overflow Reservoir 17/83	150,000	740,000	890,000
274.94	Access Roads 57/43	20,000	15,000	35,000
Total, Department 274		$ 615,000	$ 1,140,000	$ 1,755,000

EQUIPMENT

Department 271 - Thompson River Intake		Material	Labour	Total
271.31	Pumps 89/11	$ 245,000	$ 30,000	$ 275,000
271.34	Pipe, Valves and Fittings 90/10	220,000	25,000	245,000
271.36	Cranes, Hoists and Elevators 95/5	95,000	5,000	100,000
271.38	Process Controls 55/45	115,000	95,000	210,000
271.41	Motors 92/8	120,000	10,000	130,000
271.43	Starters and M.C.C. 87/13	65,000	10,000	75,000
271.44	Power Wiring 23/77	15,000	50,000	65,000
271.51	Travelling Screens 84/16	265,000	50,000	315,000
271.52	Vertical Lift Doors 73/27	40,000	15,000	55,000
271.53	Trash Racks 85/15	55,000	10,000	65,000
271.54	Ladders and Handrails 75/25	15,000	5,000	20,000
271.55	Gratings 80/20	20,000	5,000	25,000
271.56	Stop Logs 100/0	5,000	-	5,000
271.57	Trash and Frazil Ice Conveyor 57/43	20,000	15,000	35,000
271.58	Sluice Gate 100/0	5,000	-	5,000
271.59	Trash Rack Rake 86/14	30,000	5,000	35,000
271.60	Spare Parts 100/0	120,000	-	120,000
Total, Department 271		$ 1,450,000	$ 330,000	$ 1,780,000

Department 272 - Water Pipeline		Material	Labour	Total
272.38	Process Controls 53/47	$ 195,000	$ 175,000	$ 370,000
272.41	Motors	-	Included in 272.38	-
272.43	Starters and M.C.C. 75/25	30,000	10,000	40,000
272.44	Power Wiring 29/71	75,000	185,000	260,000
272.48	Telemetering System Wiring 74/26	325,000	115,000	440.000
272.83	Surge Tank Systems 79/21	595,000	155,000	750,000
272.84	Cathodic Protection 100/0	20,000	-	20,000
272.85	Leak Detection 50/50	70,000	70,000	140,000
272.89	Air/Vacuum Valves 82/18	300,000	65,000	365,000
Total, Department 272		$ 1,610,000	$ 775,000	2,385,000

Department 273 - No. 1 Booster Station		Material	Labour	Total
273.31	Pumps 87/13	$ 810,000	$ 125,000	$ 935,000
273.32	Tanks (Clearwell) 88/12	70,000	10,000	80,000
273.34	Pipe, Valves and Fittings 80/20	215,000	55,000	270,000
273.36	Cranes, Hoists and Elevators 95/5	100,000	5,000	105,000
273.38	Process Controls 53/47	180,000	160,000	340,000
273.41	Motors 98/2	590,000	15,000	605,000
273.43	Starters and M.C.C. 86/14	190,000	30,000	220,000
273.44	Power Wiring 26/74	45,000	130,000	175,000
273.60	Spare Parts 100/0	100,000	-	100,000
273.64	Clarifier 57/43	200,000	150,000	350,000
273.92	Pump Model and Testing 0/100	-	250,000	250,000
Total, Department 273		$ 2,500,000	$ 930,000	$ 3,430,000

Department 274 - No. 2 Booster Station		Material	Labour	Total
274.31	Pumps 87/13	$ 810,000	$ 125,000	$ 935,000
274.32	Tanks (Equalization) 56/44	390,000	305,000	695,000
274.34	Pipe, Valves and Fittings 80/20	215,000	55,000	270,000
274.36	Cranes, Hoists and Elevators 95/5	100,000	5,000	105,000
274.38	Process Controls 53/47	180,000	160,000	340,000
274.41	Motors 98/2	590,000	15,000	605,000
274.43	Starters and M.C.C. 86/14	190,000	30,000	220,000
274.44	Power Wiring 26/74	45,000	130,000	175,000
274.60	Spare Parts 100/0	100,000	-	100,000
Total, Department 274		$ 2,620,000	$ 825,000	$ 3,445,000

Department 291. Power Supply and Distribution		Material	Labour	Total
		82/18		
291.54	Incoming Supply Sub-stations(2)	$ 720,000	$ 160,000	$ 880,000
291.54	Intake Feeder 61/39	35,000	20,000	55,000
291.54	Intake Station Service 75/25	30,000	10,000	40,000
291.54	Pumping Station Services(2) 78/22	70,000	20,000	90,000
291.54	Pipeline Sub-stations complete with Feeders(9) 58/42	.67/33 520,000	375,000	895,000
291.54	125 V DC Sub-station Services(2)	20,000	10,000	30,000
291.54	Spare Transformers 100/0	275,000	-	275,000
291.54	Spare Starters 100/0	80,000	-	80,000
Total		$ 1,750,000	$ 595,000	$ 2,345,000

CASH FLOW SCHEDULE

34,265,000

1979	$ 1,000,000
1980	3,000,000
1981	4,000,000
1982	14,000,000
1983	22,000,000
1984	4,000,000
Total	$ 48,000,000

Notes for Cash Flow Schedule

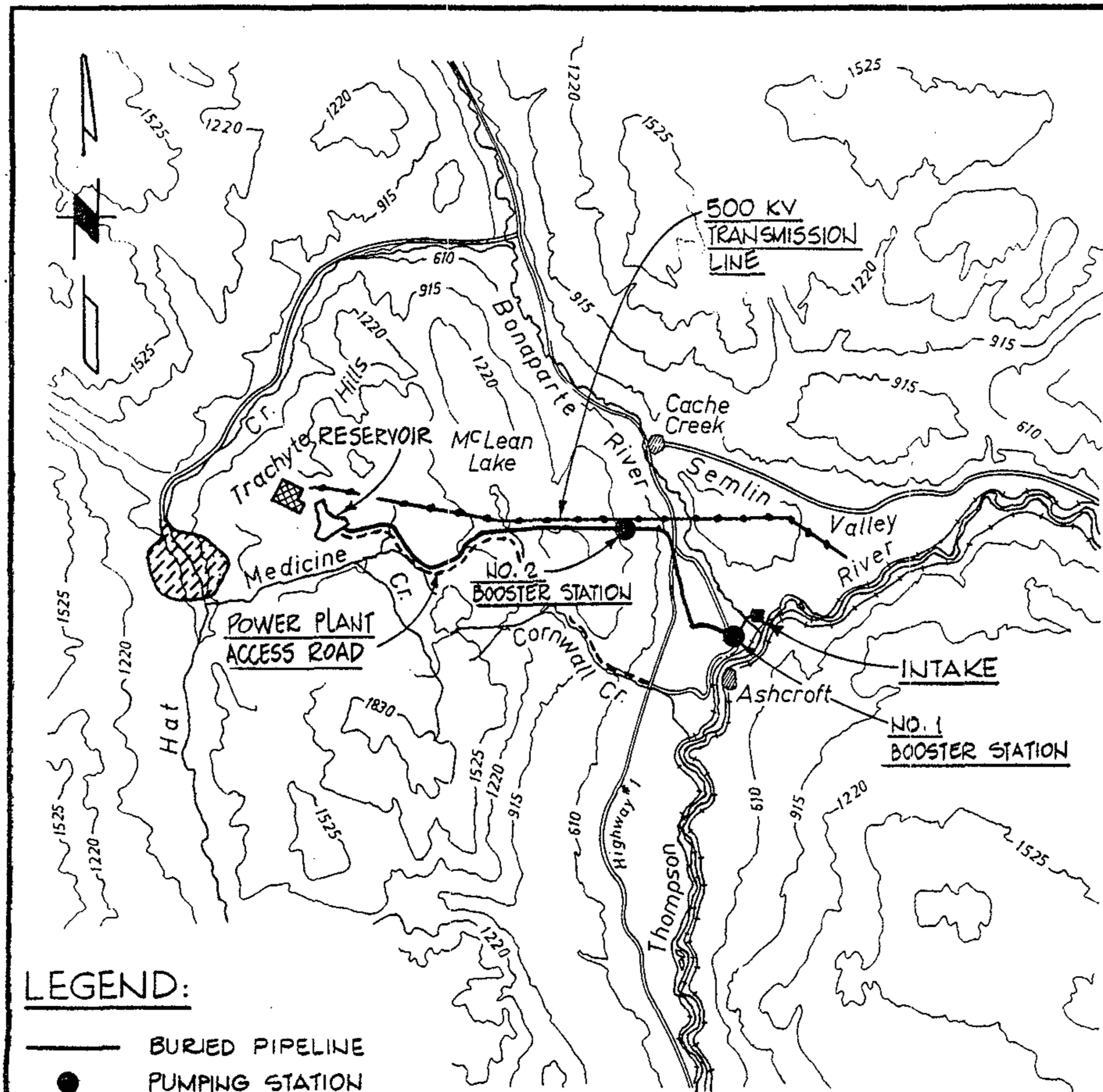
1. Payments for construction and equipment including construction overhead and contingency assumed to be uniform over the appropriate period shown on the Engineering and Construction Schedule, Drawing D4191-30.

2. Corporate overhead assumed to be uniform over the six years.

3. Years are fiscal years, 1 April to 31 March.

APPENDIX 6

ILLUSTRATIONS



LEGEND:

- BURIED PIPELINE
- PUMPING STATION
- 2000 MW POWER PLANT SITE
- OPEN PIT NO. 1

NOTE: CONTOUR ELEVATIONS IN METRES.

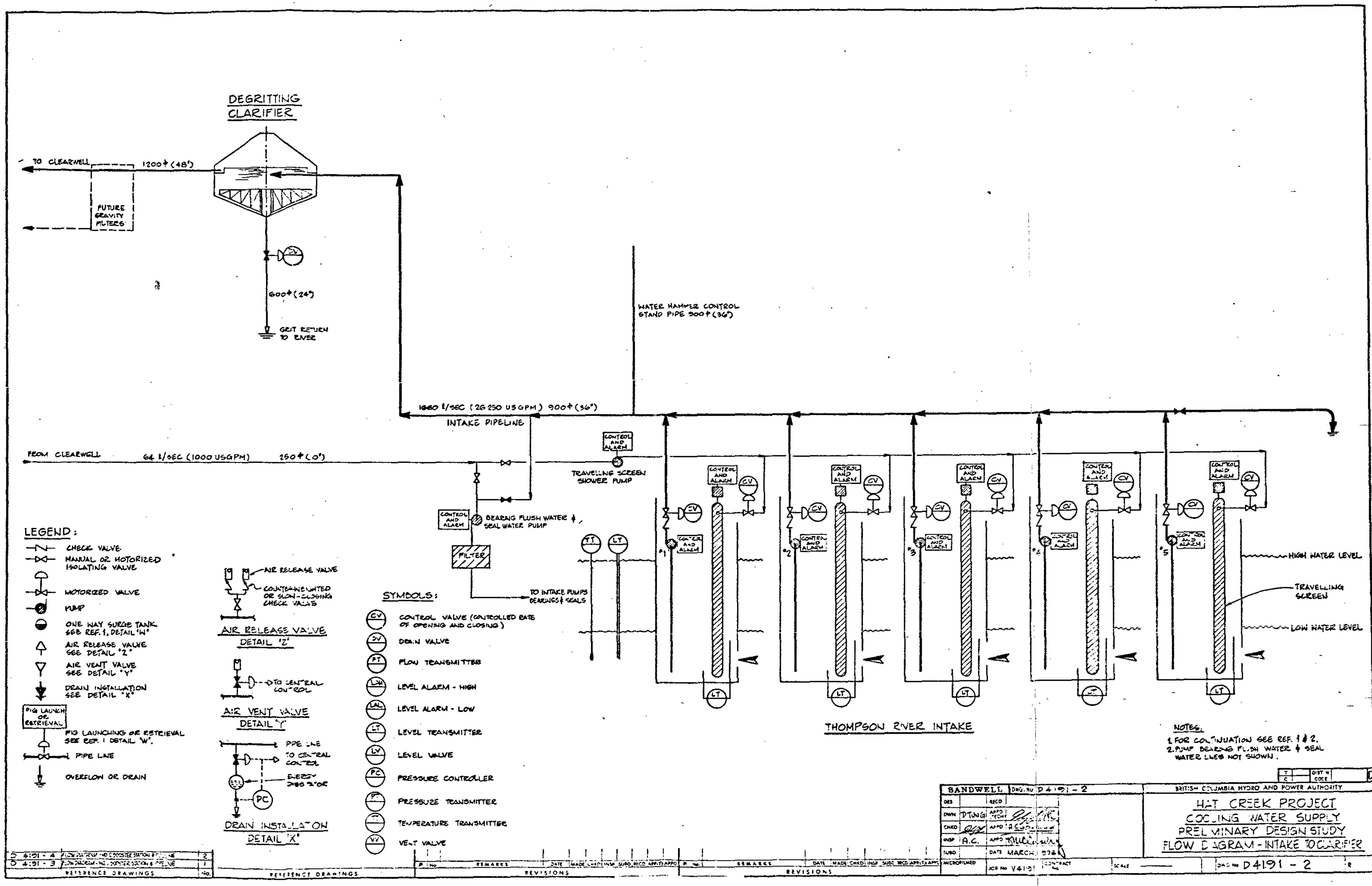
SANDWELL DWG NO.	A 4191 – 1
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
KEY PLAN

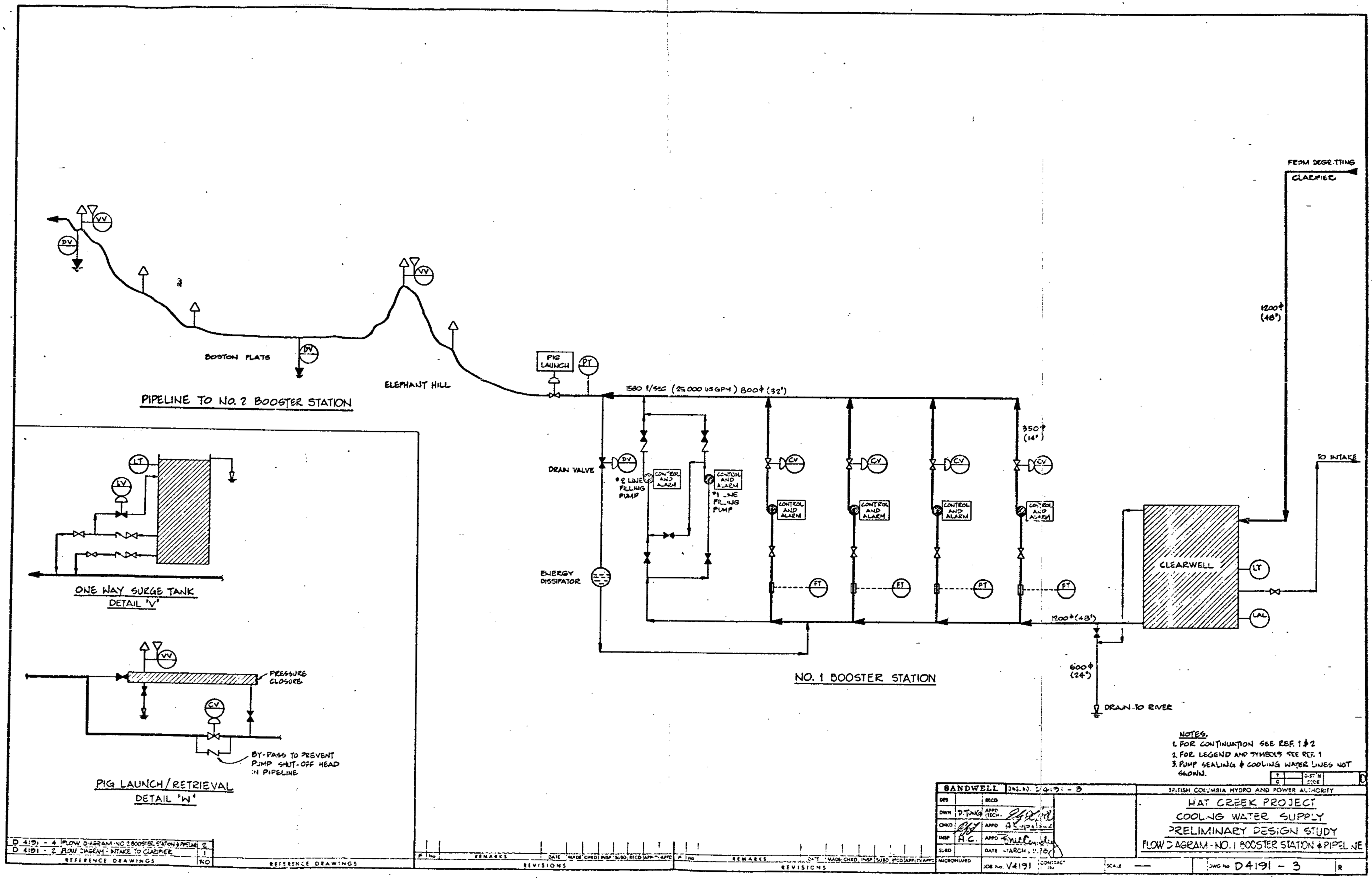
DATE MARCH , 1978	SCALE 1:250 000	R
DWN SH	DWG No. A 4191 – 1	

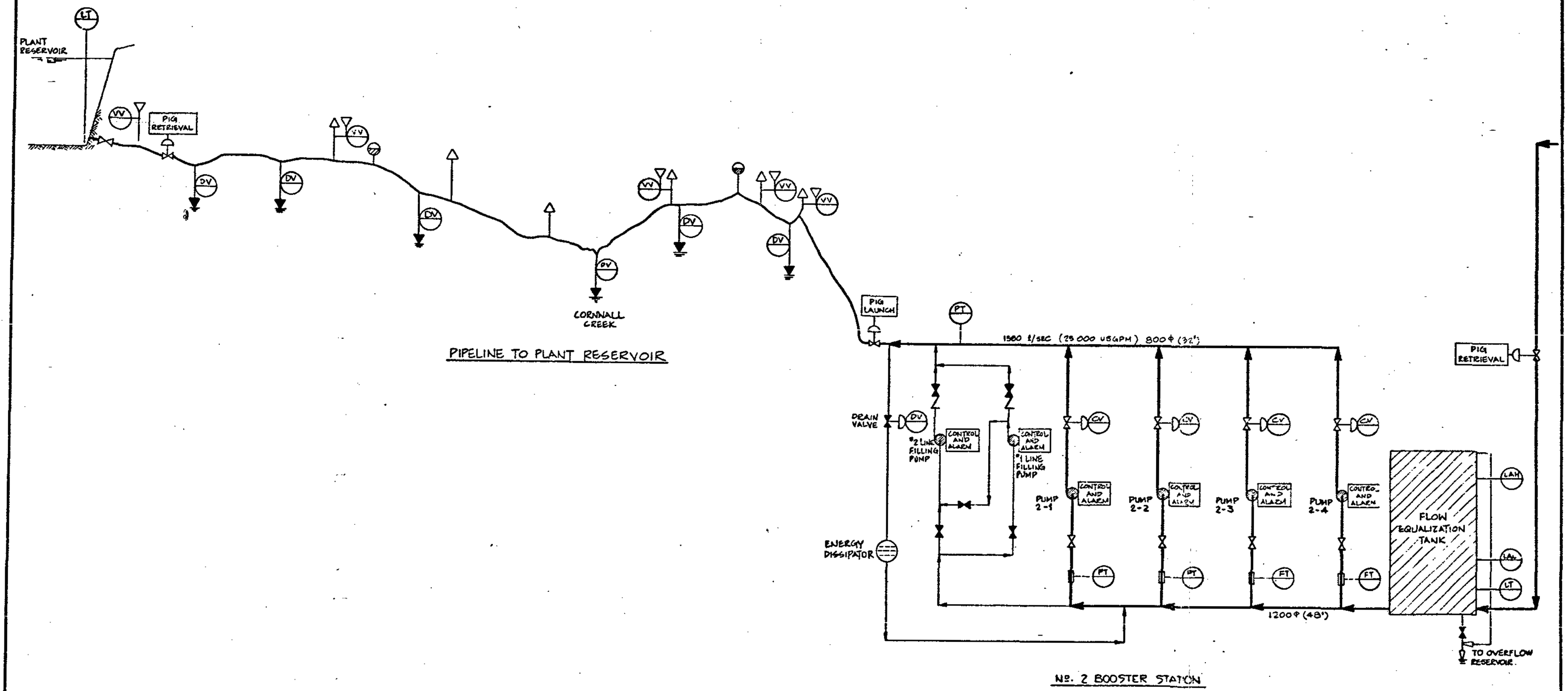
REPORT No.

A



DEGRITTING CLARIFIER
TO CLEARWELL
1200ϕ (48")
FUTURE GRAVITY FILTERS
SV
600ϕ (24")
GRIT RETURN TO RIVER
WATER HAMMER CONTROL STAND PIPE 900ϕ (36")
1660 l/SEC (26 250 USGPM) 900ϕ (36")
INTAKE PIPELINE
FROM CLEARWELL
64 l/SEC (1000 USGPM)
250ϕ (10")
CONTROL AND ALARM
TRAVELLING SCREEN SHOWER PUMP
BEARING FLUSH WATER & SEAL WATER PUMP
FILTER
TO INTAKE PUMPS BEARINGS & SEALS
HIGH WATER LEVEL
TRAVELLING SCREEN
LOW WATER LEVEL
THOMPSON RIVER INTAKE
LEGEND:
CHECK VALVE
MANUAL OR MOTORIZED ISOLATING VALVE
MOTORIZED VALVE
PUMP
ONE WAY SURGE TANK SEE REF. 1, DETAIL "W"
AIR RELEASE VALVE SEE DETAIL "Z"
AIR VENT VALVE SEE DETAIL "Y"
DRAIN INSTALLATION SEE DETAIL "X"
PIG LAUNCH OR RETRIEVAL
PIG LAUNCHING OR RETRIEVAL SEE REF. 1 DETAIL "W".
PIPE LINE
OVERFLOW OR DRAIN
AIR RELEASE VALVE
COUNTERWEIGHTED OR SLOW-CLOSING CHECK VALVE
AIR RELEASE VALVE DETAIL "Z"
TO CENTRAL CONTROL
AIR VENT VALVE DETAIL "Y"
PIPE LINE
TO CENTRAL CONTROL
ENERGY DISSIPATOR
DRAIN INSTALLATION DETAIL "X"
SYMBOLS:
CONTROL VALVE (CONTROLLED RATE OF OPENING AND CLOSING)
DRAIN VALVE
FLOW TRANSMITTER
LEVEL ALARM - HIGH
LEVEL ALARM - LOW
LEVEL TRANSMITTER
LEVEL VALVE
PRESSURE CONTROLLER
PRESSURE TRANSMITTER
TEMPERATURE TRANSMITTER
VENT VALVE
NOTES:
1. FOR CONTINUATION SEE REF. 1 & 2.
2. PUMP BEARING FLUSH WATER & SEAL WATER LINES NOT SHOWN.
SANDWELL
DWG. NO. D4191 - 2
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
FLOW DIAGRAM - INTAKE TO CLARIFIER
DWN D.TUNG
INSP A.C.
DATE MARCH 1976
JOB No V4191
DWG No D4191 - 2
D 4191 - 4 FLOW DIAGRAM - NO. 2 BOOSTER STATION & PIPELINE
D 4191 - 3 FLOW DIAGRAM - NO. 1 BOOSTER STATION & PIPELINE
REFERENCE DRAWINGS
REMARKS
DATE
REVISIONS


FROM DEGRITTING CLARIFIER
1200φ (48")
TO INTAKE
CLEARWELL
LT
LAL
1200φ (48")
600φ (24")
DRAIN TO RIVER
1580 l/sec (25 000 USGPM) 800φ (32")
350φ (14")
CV
CONTROL AND ALARM
FT
DRAIN VALVE
DV
#2 LINE FILLING PUMP
#1 LINE FILLING PUMP
ENERGY DISSIPATOR
NO. 1 BOOSTER STATION
PIG LAUNCH
PT
VV
BOSTON FLATS
ELEPHANT HILL
PIPELINE TO NO. 2 BOOSTER STATION
LT
LV
ONE WAY SURGE TANK
DETAIL "V"
PRESSURE CLOSURE
BY-PASS TO PREVENT PUMP SHUT-OFF HEAD IN PIPELINE
PIG LAUNCH/RETRIEVAL
DETAIL "W"
NOTES.
1. FOR CONTINUATION SEE REF. 1 & 2
2. FOR LEGEND AND SYMBOLS SEE REF. 1
3. PUMP SEALING & COOLING WATER LINES NOT SHOWN.
D 4191 - 4 FLOW DIAGRAM - NO. 2 BOOSTER STATION & PIPELINE 2
D 4191 - 2 FLOW DIAGRAM - INTAKE TO CLARIFIER 1
REFERENCE DRAWINGS NO
REFERENCE DRAWINGS
REVISIONS
REVISIONS
SANDWELL DWG. NO. D4191 - 3
DWN D.TUNG
CHKD
INSP A.C.
DATE MARCH, 1978
JOB No. V4191
SCALE —
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
FLOW DIAGRAM - NO. 1 BOOSTER STATION & PIPELINE
DWG No D4191 - 3



NOTES:
1. FOR CONTINUATION SEE REF. 2.
2. FOR LEGEND AND SYMBOLS SEE REF. 1
3. PUMP COOLING AND SEALING WATER LINES NOT SHOWN.

D 4191 - 3	FLOW DIAGRAM - NO. 1 BOOSTER STATION & PIPELINE	2
D 4191 - 2	FLOW DIAGRAM - INTAKE TO CLARIFIER	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS

REVISIONS

REVISIONS

DES		RECD	
DWN	D. TUNG	APPD	
CHKD		APPD	A. Copeland
INSP	A.C.	APPD	
SUBD		DATE	MARCH 1978
MICROFILMED		DIST'N CODE	

SCALE —

SANDWELL DWG. NO. D4191 - 4

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

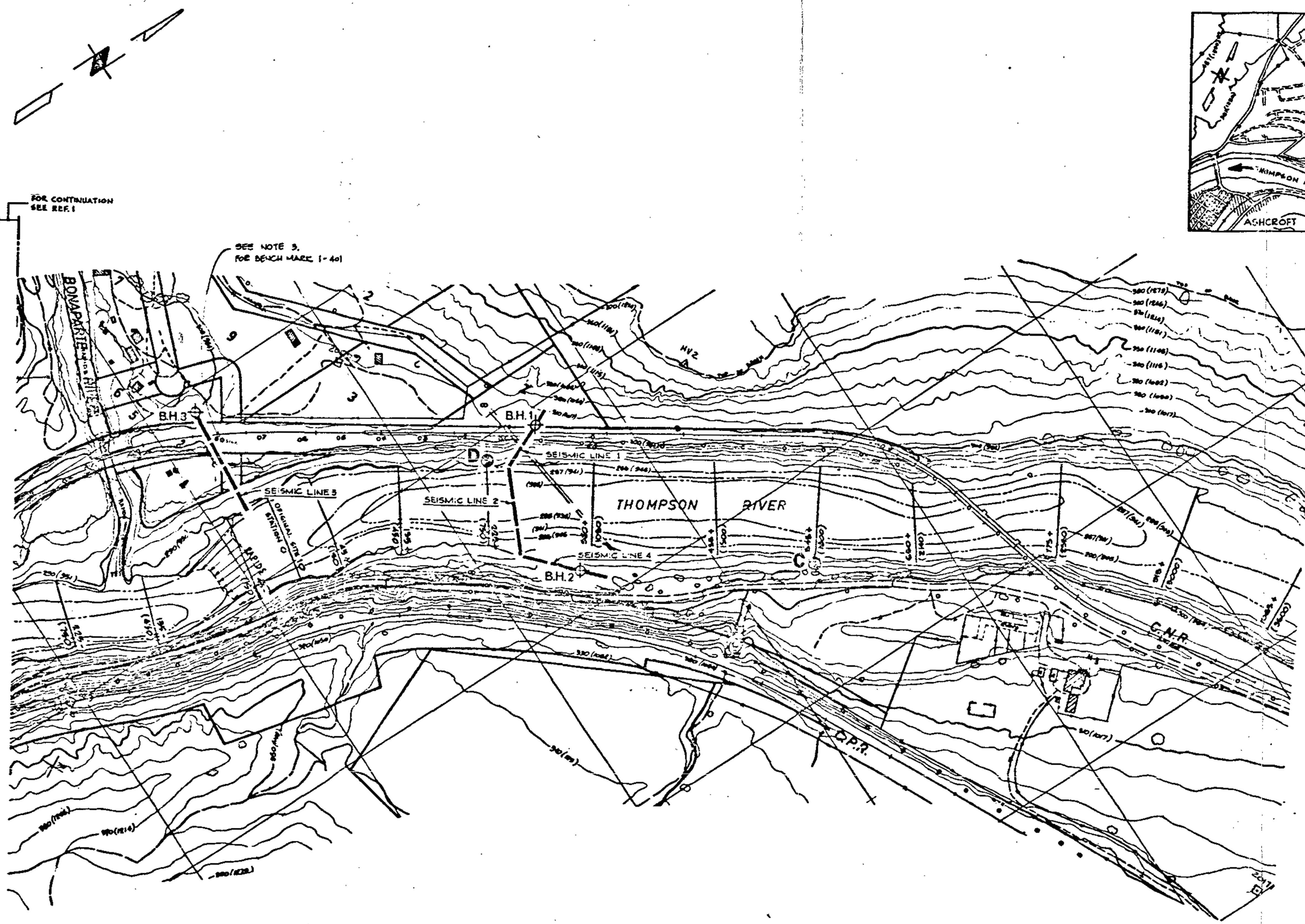
HAT CREEK PROJECT

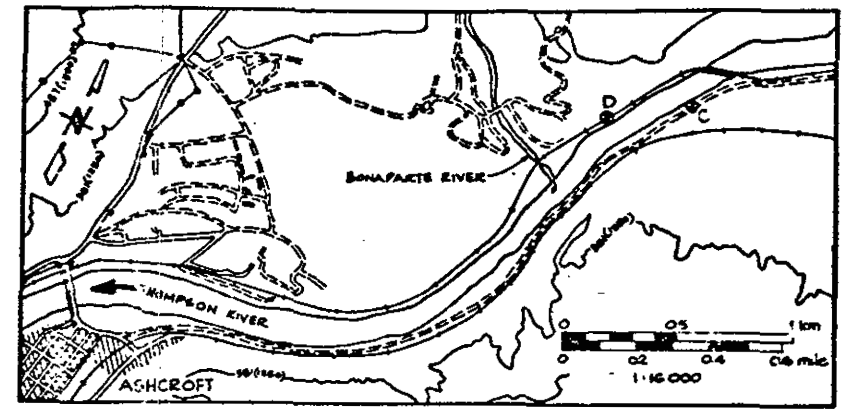
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

FLOW DIAGRAM - NO. 2 BOOSTER STATION & PIPELINE

DWG. No. D4191 - 4 R





KEY PLAN

LEGEND.

1. ——— CONTOUR DATA PREPARED BY McELHANNEY SURVEYING AND ENGINEERING LTD., FROM AERIAL PHOTOGRAPHS NUMBERS 43270 B AND 43271 B FLOWN BY PACIFIC SURVEY CORPORATION ON 16 APRIL 1970.
2. ---- RIVER BED CONTOUR DATA PREPARED BY NORTHWEST HYDRAULIC CONSULTANTS LTD.
3. LOT BOUNDARIES WITH LOT NUMBERS. THESE WERE OBTAINED FROM WESTERN MAPPING OF KAMLOOPS DRAWINGS 92-I-11-f-50 & 61.
4. MEANS SAME OWNER EACH SIDE OF ACCESS ROAD CORRIDOR.
   ACCESS ROAD (BETWEEN LOT BOUNDARIES).
5. ○ C.N.R. TELEGRAPH POLE (NUMBERED).
6. POTENTIAL INTAKE LOCATIONS.
7. △ TRIANGULATION SURVEY POINT.
8. ⊕ B.H DENOTES, BORE HOLE.

NOTES.

1. RIVER CROSS SECTION DISTANCES AND CONTOURS GIVEN IN METRES (FEET).
2. FOR DETAILS OF INTAKE SITES 10 E, F & G - TOPOGRAPHY SEE REF 1.
3. FOR DETAILED TOPOGRAPHY OF INTAKE SITE 10 D SEE REF 2.
4. BENCH MARK 1-401 IS A RAILWAY SPIKE IN TELEGRAPH POLE # 6 ELEVATION 301.880 m (990.42 ft.)

0 50 100 150 200 m
0 100 200 300 400 500 600 700 ft
1:2000

REFERENCE DRAWINGS		NO
D4191 - 7	INTAKE SITE 10 D - DETAILED TOPOGRAPHY	2
D4191 - 6	INTAKE SITES 10 E, F & G - TOPOGRAPHY	1

REFERENCE DRAWINGS | REVISIONS | REVISIONS

SANDWELL DWG. NO. D4191 - 5

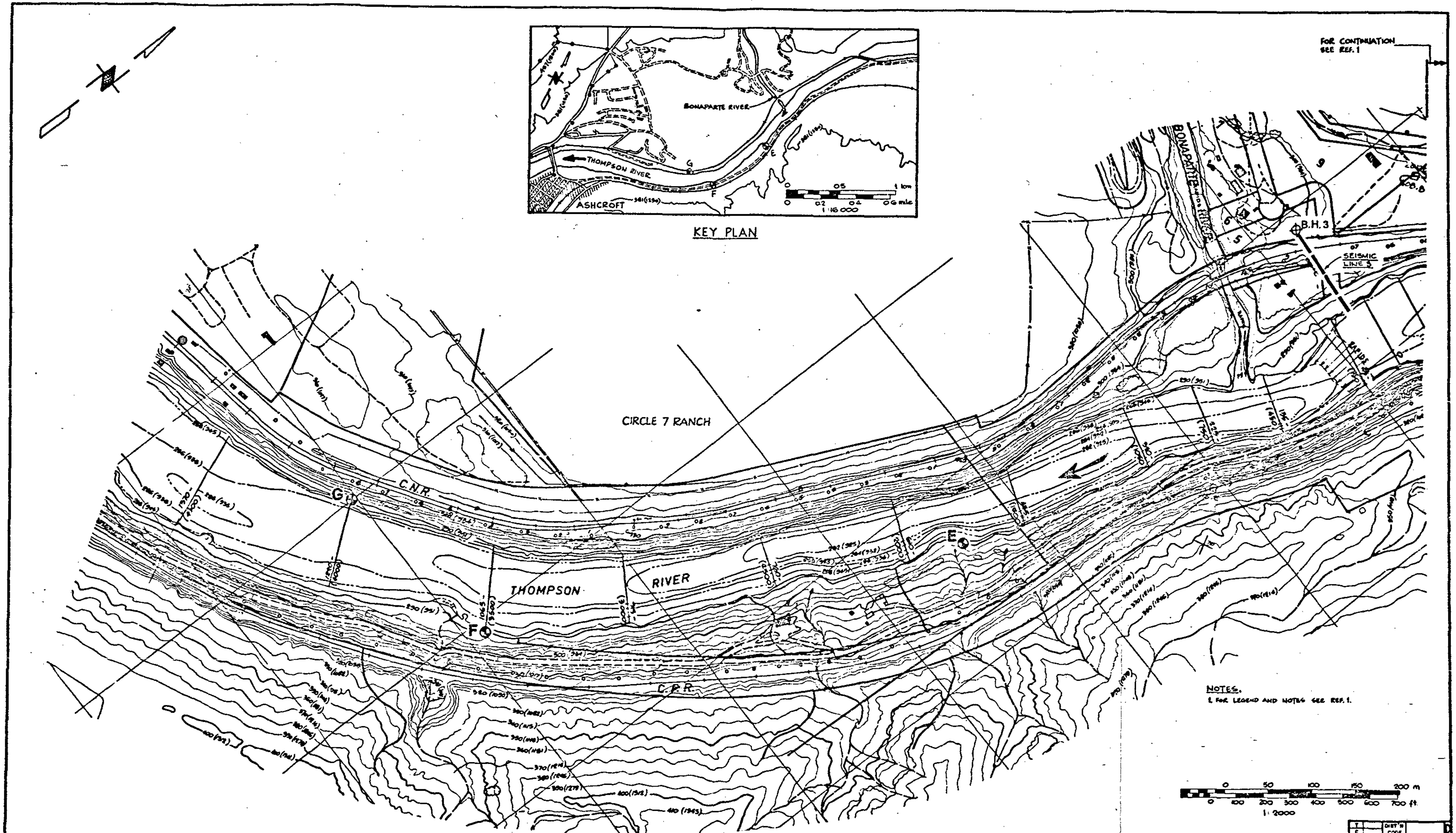
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE SITES 10 C & D - TOPOGRAPHY

DES		RECD	
DWN	GH	APPD (TECH)	[illegible]
CHKD	JWB	APPD	[illegible]
INSP	RC	APPD	[illegible]
SUBD		DATE	MARCH 1978
MICROFILMED		JOB No. V4191	CONTRACT No.

SCALE AS NOTED

DWG No. D4191 - 5



D 4191 - 5	INTAKE SITES 10 C & D - TOPOGRAPHY	1
REFERENCE DRAWINGS		NO

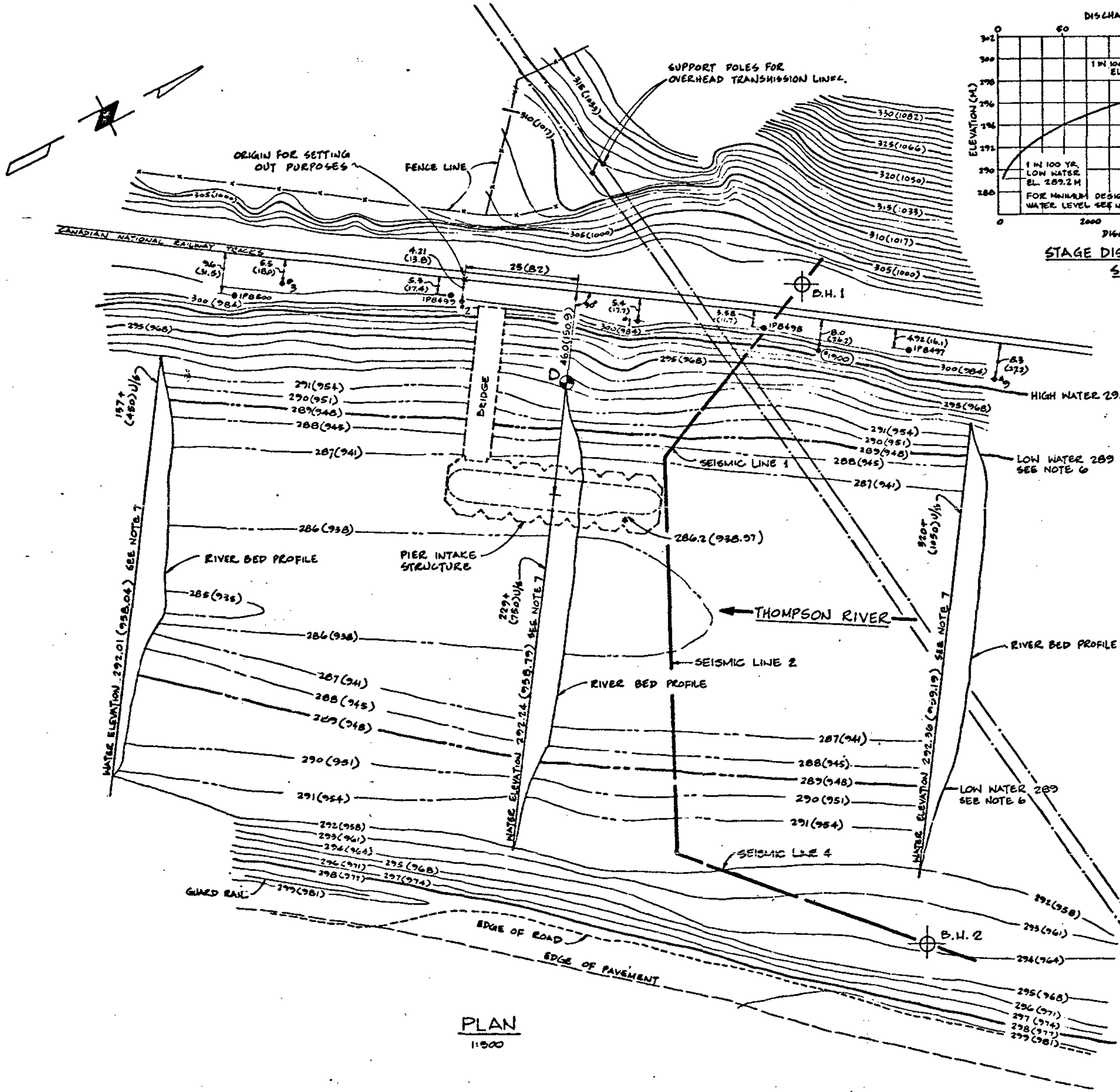
SANDWELL DWG. NO. D4191 - 6

DES		RECD	
DWN	SH	APPD (TECH)	
CHKD	JW/RB	APPD	
INSP	A.C	APPD	
SUBD		DATE	MARCH, 1978

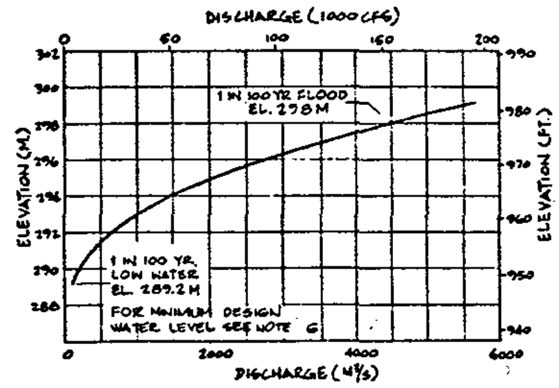
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE SITES 10 E, F & G - TOPOGRAPHY

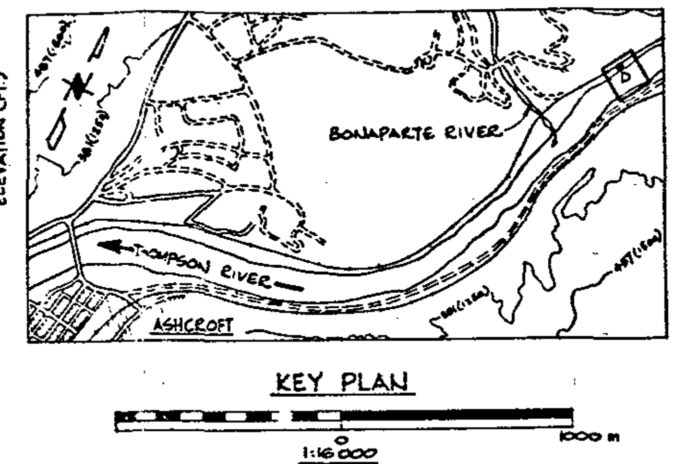
JOB No. V4191 | SCALE AS NOTED | DWG No. D4191 - 6



PLAN
1:500



STAGE DISCHARGE CURVE
SITE 'D'



KEY PLAN
1:16000

IRON PIN	ELEVATION M. (FT)
•8497	301.476 (989.09)
•8498	301.380 (988.78)
•8499	
•8500	300.960 (987.40)

LEGEND

1. RIVER BED CONTOUR DATA OBTAINED FROM NORTHWEST HYDRAULIC CONSULTANTS LTD
2. CONTOURS OBTAINED FROM DATA FROM McELHANNEY SURVEYING & ENGINEERING LTD., DURING SURVEY OF 10th–12th AUGUST 1977 WHEN WATER ELEVATION AT 'D' WAS 292.24 (958.79).
3. • IRON PIN WITH A McELHANNEY IDENTIFICATION TAG NUMBER ATTACHED.
4. • TELEGRAPH POLE WITH C.N.R. REFERENCE NUMBER.
5. OVERHEAD TRANSMISSION LINE
6. BORE HOLE AND SEISMIC DATA OBTAINED FROM GOLDER ASSOCIATES LTD.

NOTES

1. CONTOURS, ELEVATIONS AND CROSS SECTION DATA ARE GIVEN IN METRES AND (FEET).
2. THE INTAKE SILL WILL BE POSITIONED ADJACENT TO A RIVER BED LEVEL OF 286.2 (938.7) AS DETERMINED BY NORTHWEST HYDRAULIC CONSULTANTS LTD. DURING THE DEVELOPMENT OF THE INTAKE MODEL STUDY.
3. DRILL HOLES WILL BE LOCATED IN THE FIELD TO AN ACCURACY OF ±1.5M (5 FEET).
4. POLE #: IS THE TENTH POLE UPSTREAM OF THE THOMPSON/BONAPARTE CONFLUENCE.
5. THE STAGE DISCHARGE CURVE WAS OBTAINED FROM DATA FROM NORTHWEST HYDRAULIC CONSULTANTS LTD.
6. THE MINIMUM DESIGN WATER LEVEL WAS SELECTED AT 289.00 m, WHICH IS 0.20 m LOWER THAN THE ESTIMATED 1 IN 100 YEAR MINIMUM LOW WATER LEVEL.
7. RIVER BED PROFILES SHOW THE SURFACE WATER ELEVATIONS DURING McELHANNEY'S SURVEY OF 10-12 AUG. 1977, SEE LEGEND, ITEM 2.

10 0 10 20 30 40 50 m.
1:500

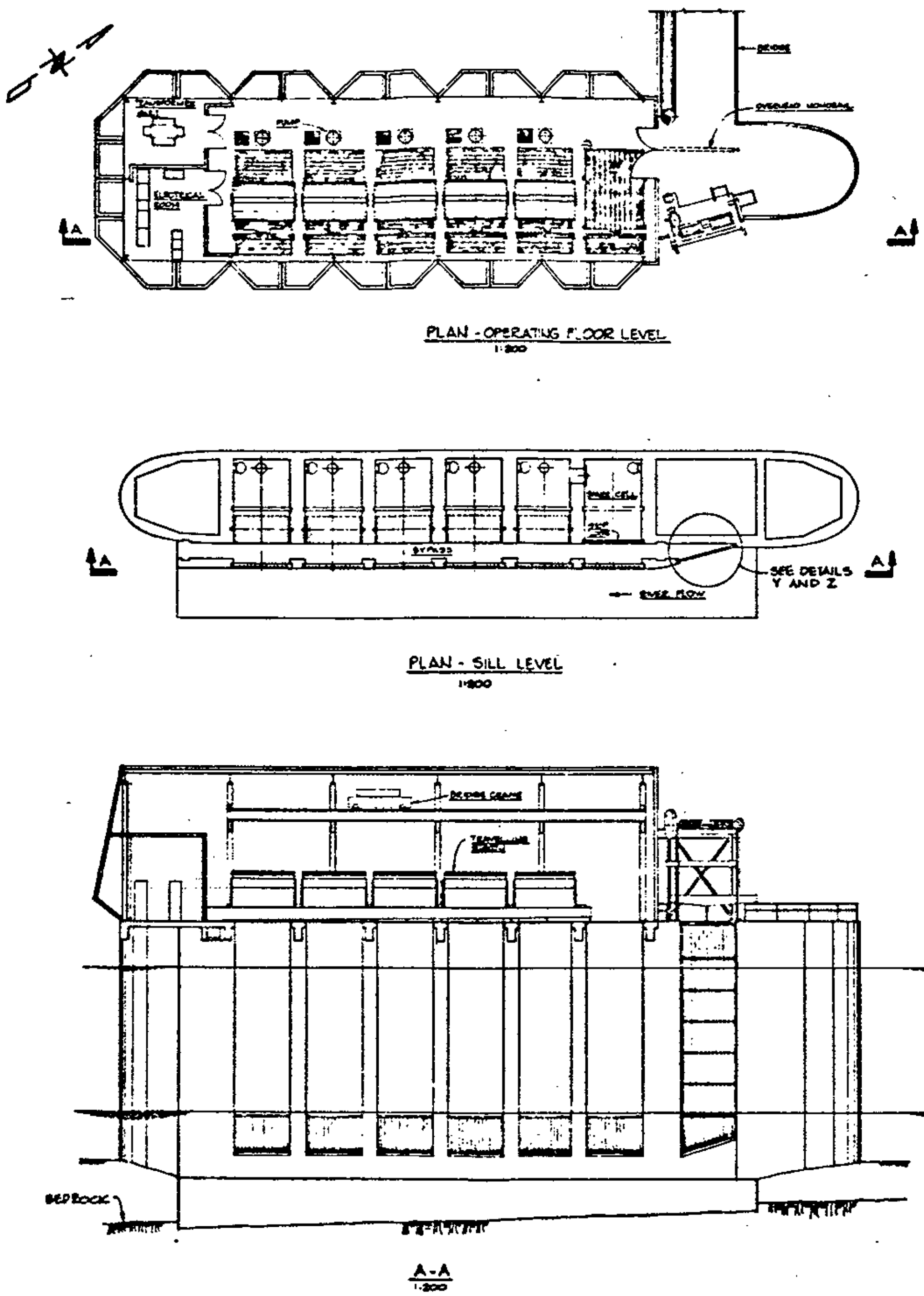
SANDWELL	DWG. NO. D4191 – 7
DES	RECD
DWN D. TING	APPD (TECH) J.C.
CHKD JWR	APPD R. Copeland
INSP A.C.	APPD
SUBD	DATE MARCH, 1979
MICROFILMED	JOB No. V4191

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE SITE 10D – DETAILED TOPOGRAPHY

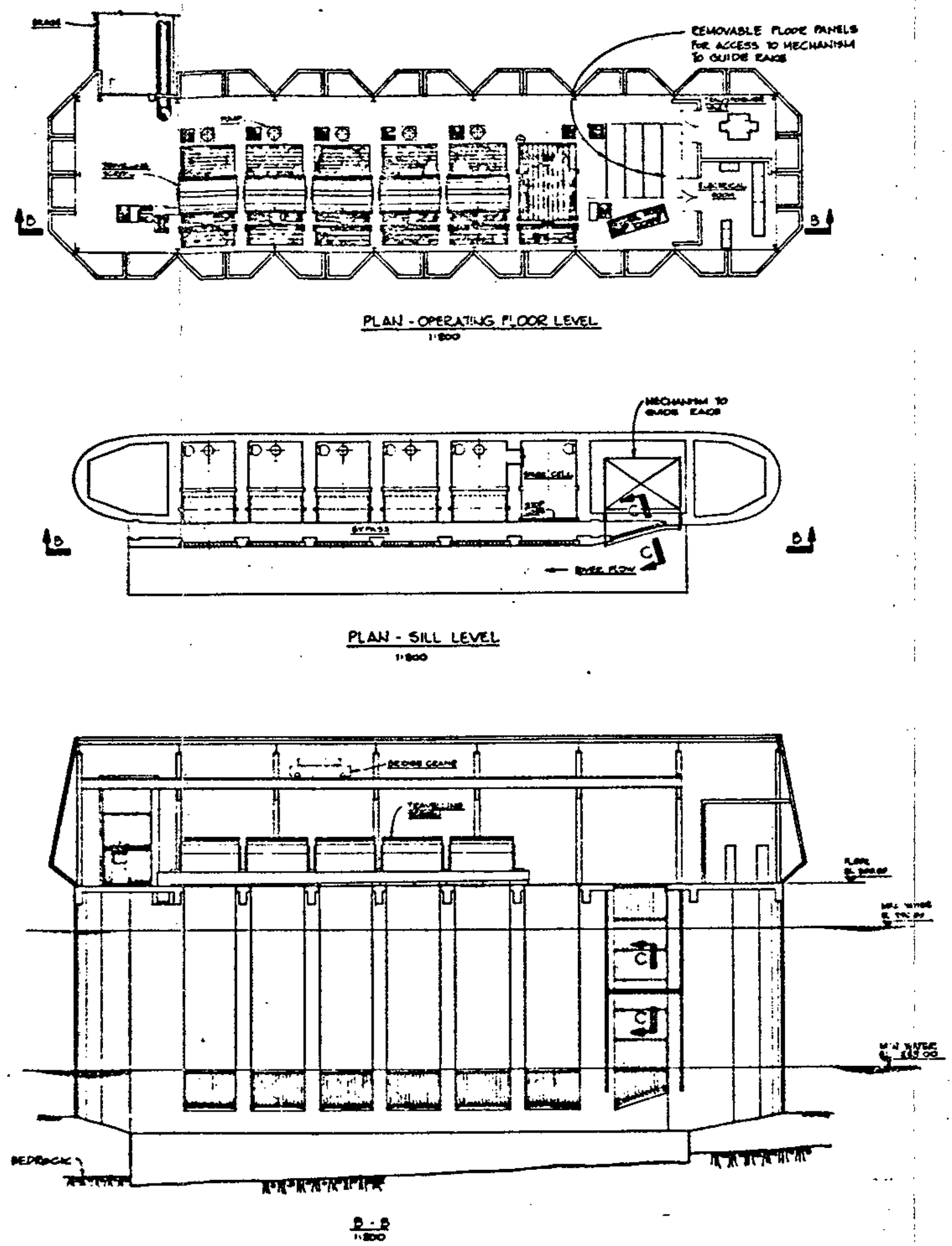
SCALE AS NOTED | DWG No. D4191 – 7

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS



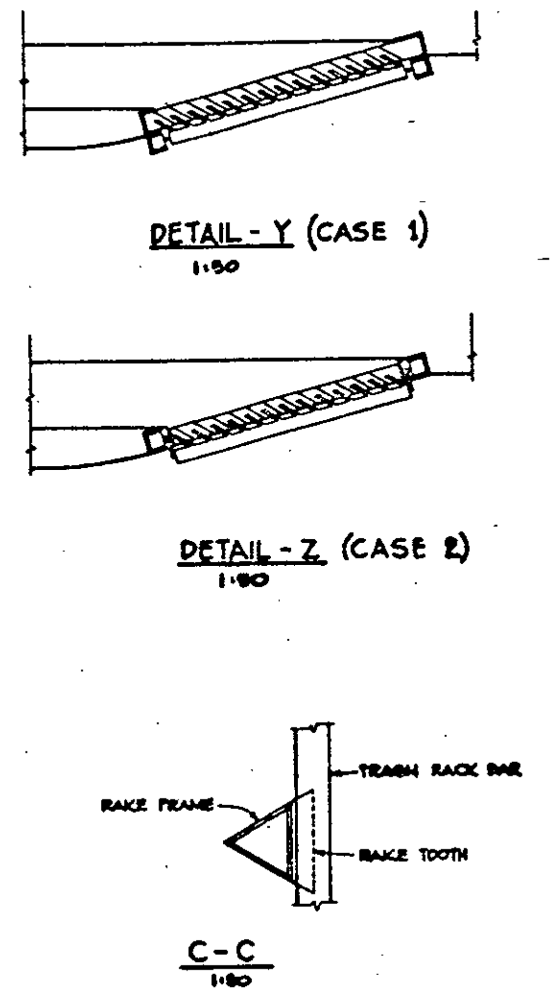
ALTERNATIVE 1

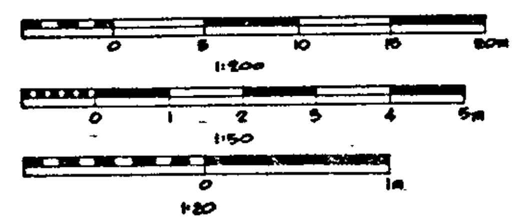
ARRANGEMENT WITH STANDARD MECHANICAL RAKE



ALTERNATIVE 2

ARRANGEMENT WITH CUSTOM DESIGNED RAKE





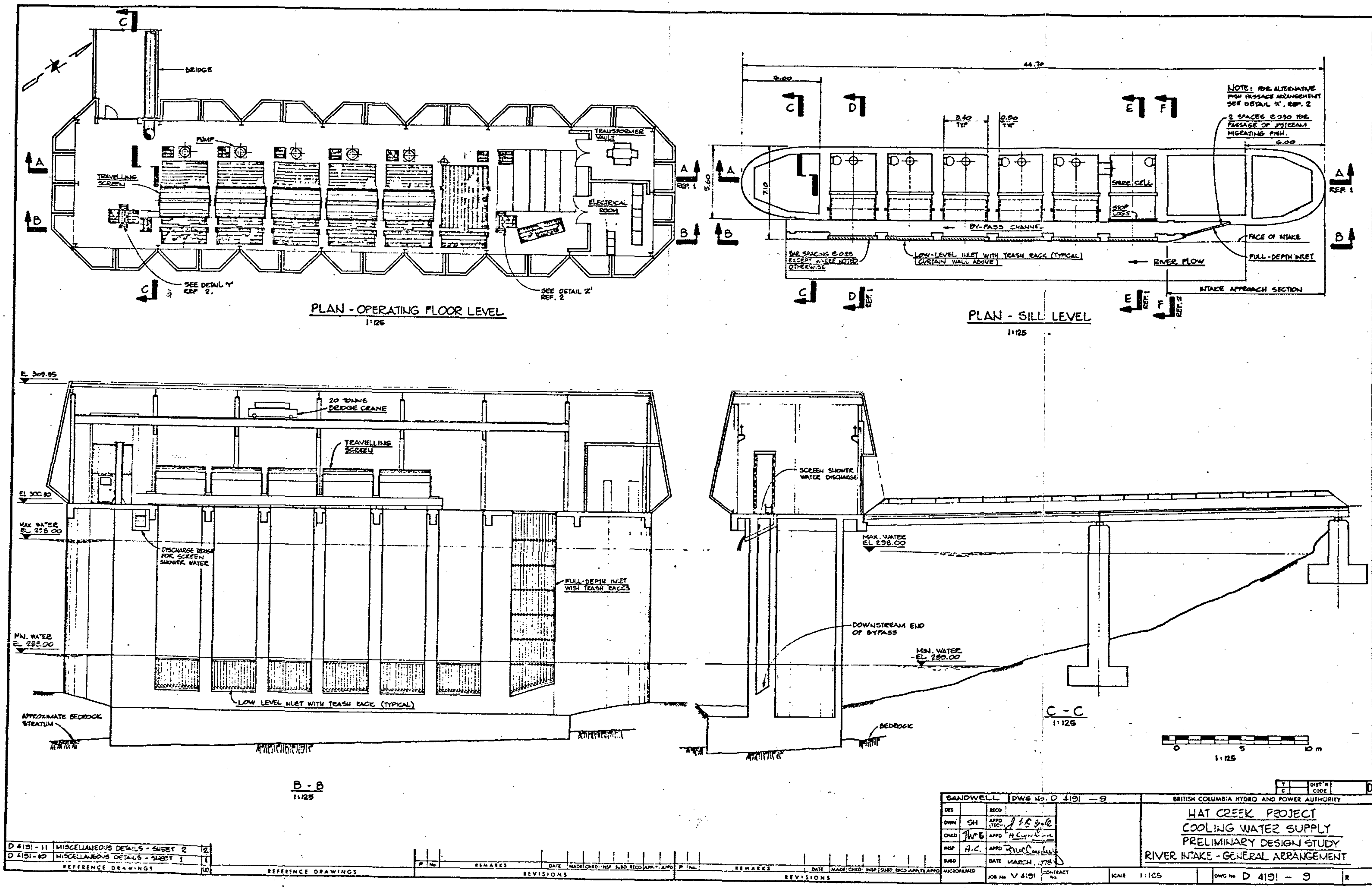
SANDWELL	DWG. NO. D 4191 - 8
DRN. 4H	APPD (TECH) J.C. Boyle
CHKD JWOB	APPD A Copeland
INSP AC	APPD R.McCaul
	DATE MARCH, 1977
JOB No. V 4191	CONTRACT No.

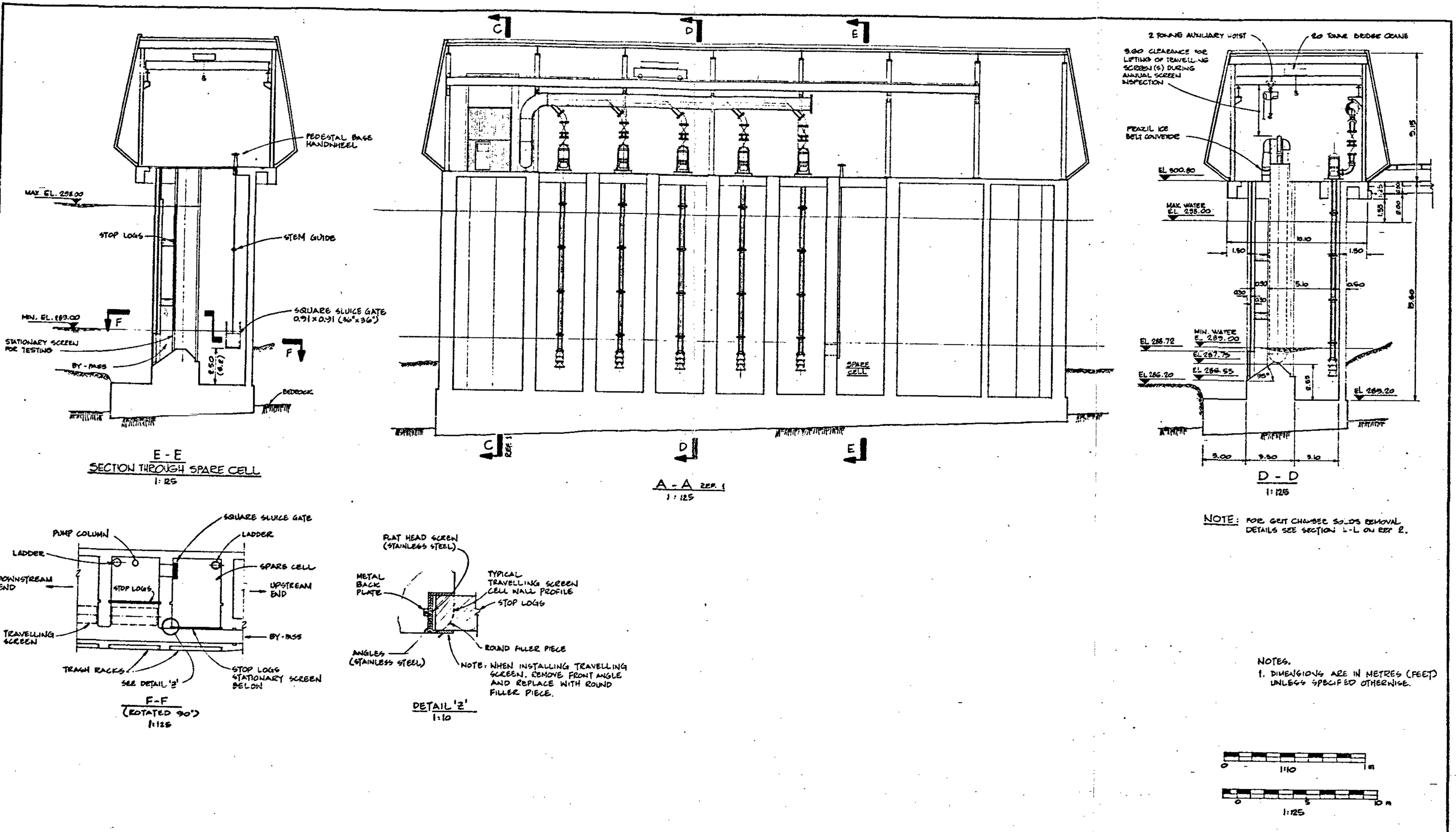
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
ALTERNATIVE INTAKE LAYOUTS

SCALE AS NOTED | DWG No. D 4191 - 8 | R

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REMARKS | REVISIONS | REMARKS | REVISIONS


BRIDGE
PUMP
TRAVELLING SCREEN
TRANSFORMER VAULT
ELECTRICAL ROOM
SEE DETAIL 'Y' REF. 2
SEE DETAIL 'Z' REF. 2
PLAN - OPERATING FLOOR LEVEL
1:125
44.70
6.00
3.40 TYP
0.50 TYP
NOTE: FOR ALTERNATIVE FISH PASSAGE ARRANGEMENT SEE DETAIL 'X', REF. 2
2 SPACES @ 0.30 FOR PASSAGE OF UPSTREAM MIGRATING FISH.
5.60
7.10
SPARE CELL
STOP LOGS
BY-PASS CHANNEL
FACE OF INTAKE
FULL-DEPTH INLET
BAR SPACING @ 0.025 EXCEPT WHERE NOTED OTHERWISE
LOW-LEVEL INLET WITH TRASH RACK (TYPICAL) (CURTAIN WALL ABOVE)
RIVER FLOW
INTAKE APPROACH SECTION
REF. 1
REF. 2
PLAN - SILL LEVEL
1:125
EL 309.85
20 TONNE BRIDGE CRANE
TRAVELLING SCREEN
EL 300.80
MAX. WATER EL 298.00
DISCHARGE TROUGH FOR SCREEN SHOWER WATER
FULL-DEPTH INLET WITH TRASH RACKS
MIN. WATER EL 283.00
LOW LEVEL INLET WITH TRASH RACK (TYPICAL)
APPROXIMATE BEDROCK STRATUM
B - B
1:125
SCREEN SHOWER WATER DISCHARGE
DOWNSTREAM END OF BYPASS
BEDROCK
C - C
1:125
0
5
10 m
1:125
D 4191-11 MISCELLANEOUS DETAILS - SHEET 2
D 4191-10 MISCELLANEOUS DETAILS - SHEET 1
REFERENCE DRAWINGS
REVISIONS
SANDWELL
DWG No. D 4191 - 9
DWN SH
CHKD
INSP A.C.
DATE MARCH 1978
JOB No V 4191
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
RIVER INTAKE - GENERAL ARRANGEMENT
SCALE 1:125
DWG No D 4191 - 9



D 4191 - 11	MISCELLANEOUS DETAILS - SHEET 1	2
D 4191 - 9	GENERAL ARRANGEMENT	1
REFERENCE DRAWINGS		NO.

REFERENCE DRAWINGS

REVISIONS

REVISIONS

SANDWELL	DWG. NO. D 4191 - 10		
DES		RECD	
DWN	D.TUNG	APPD (TECH)	JWC
CHKD	Theg	APPD	H. Coy
INSP	A.C.	APPD	
SUBD		DATE	MARCH, 1978
MICROFILMED	JOB No. V4191	CONTRACT No.	

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

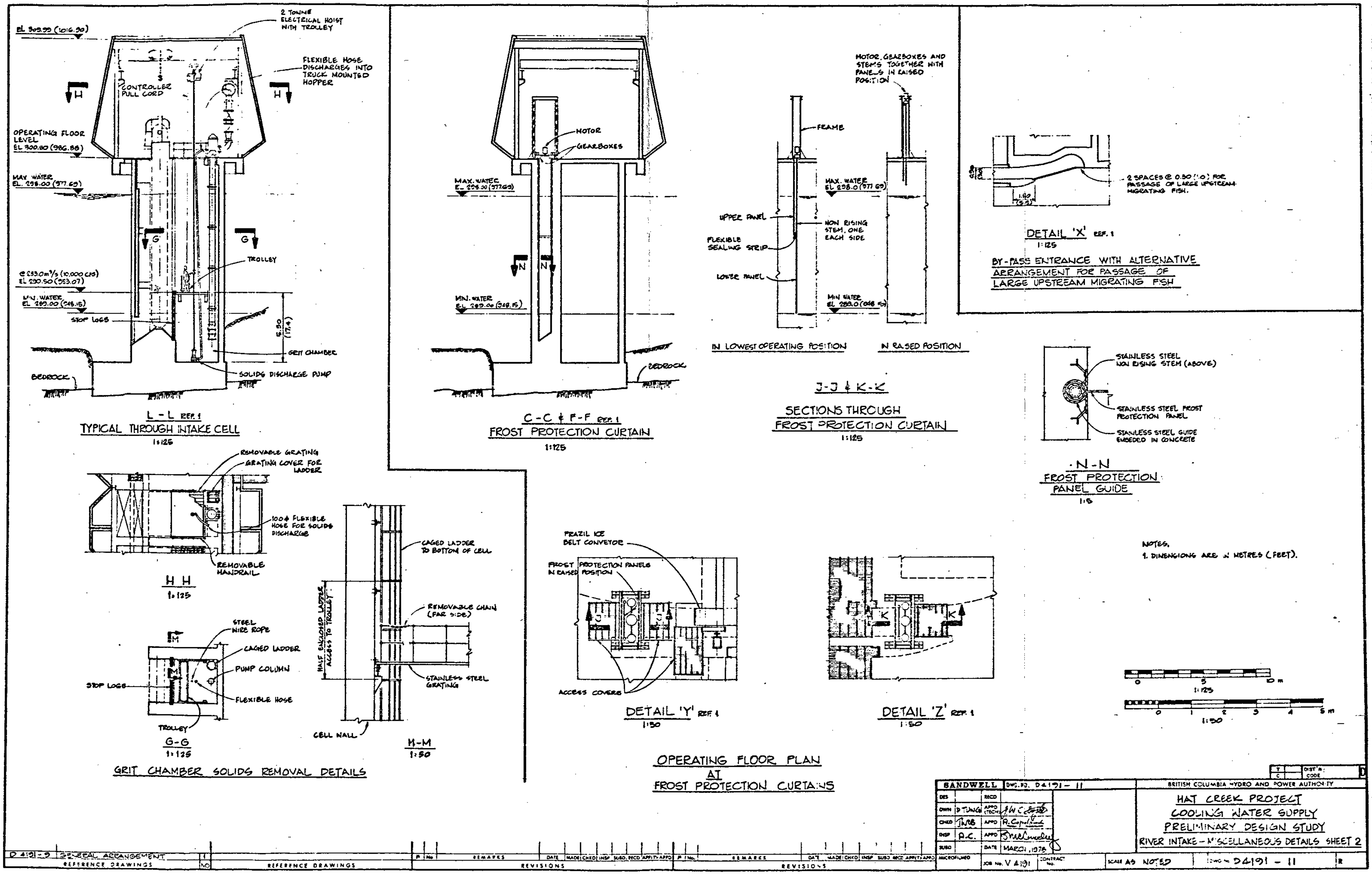
HAT CREEK PROJECT

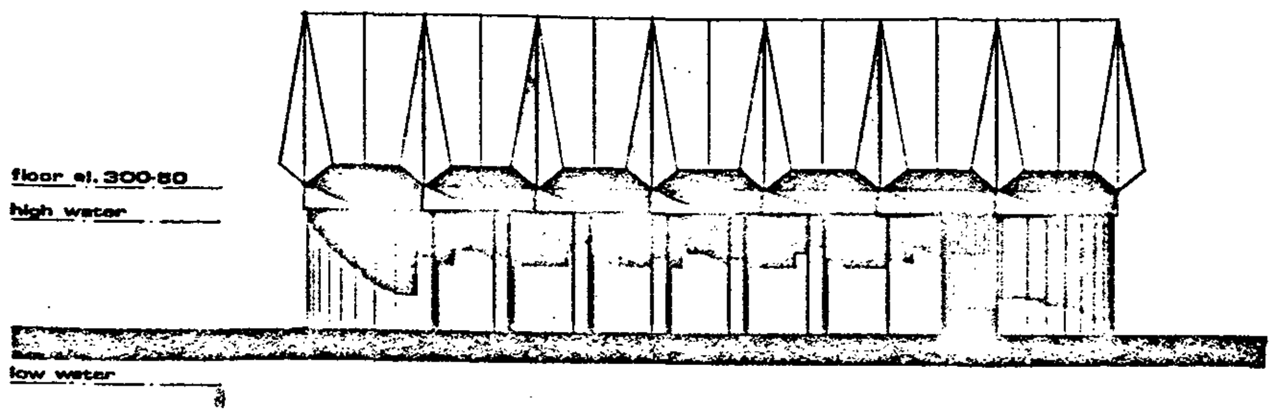
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

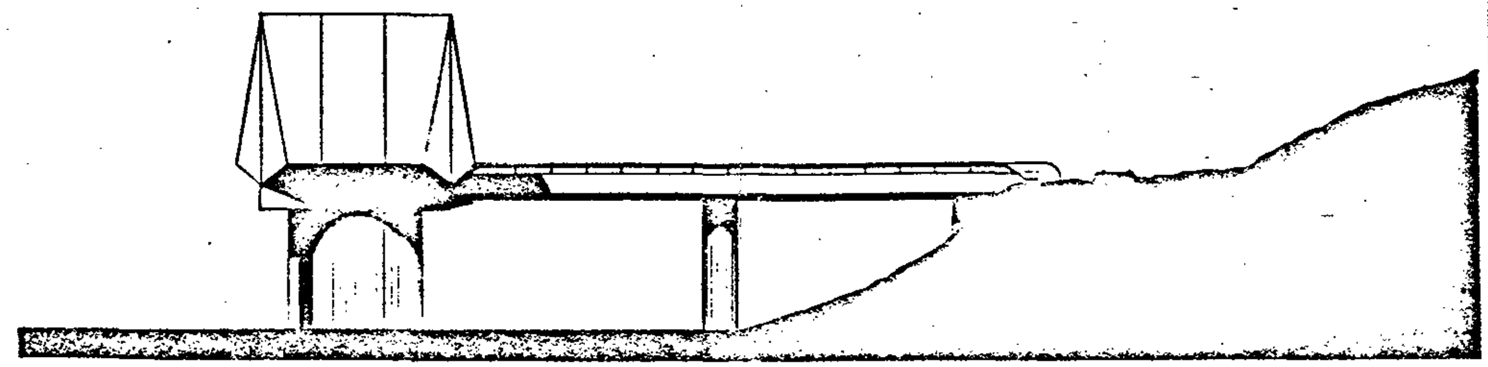
RIVER INTAKE-MISCELLANEOUS DETAILS SHEET 1

SCALE AS SHOWN | DWG No. D 4191 - 10 | R

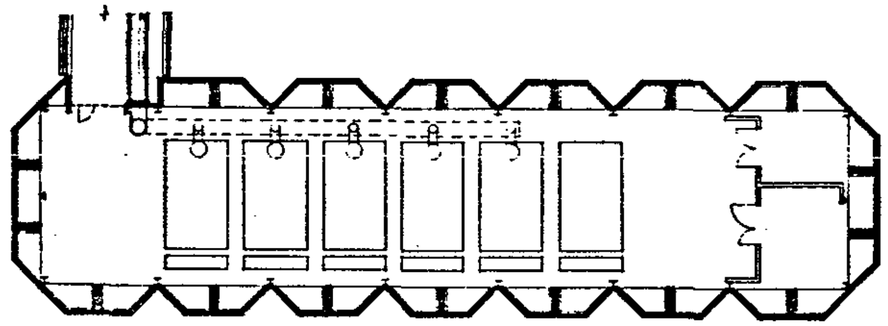

EL 309.99 (1016.90)
2 TONNE ELECTRICAL HOIST WITH TROLLEY
FLEXIBLE HOSE DISCHARGES INTO TRUCK MOUNTED HOPPER
CONTROLLER PULL CORD
OPERATING FLOOR LEVEL EL 300.80 (986.88)
MAX WATER EL 298.00 (977.69)
TROLLEY
@ 283.0m³/s (10,000 cfs) EL 290.50 (953.07)
MIN. WATER EL 289.00 (948.15)
STOP LOGS
5.30 (17.4)
GRIT CHAMBER
BEDROCK
SOLIDS DISCHARGE PUMP
L - L REF. 1
TYPICAL THROUGH INTAKE CELL
1:125
MOTOR
GEARBOXES
MAX. WATER EL 298.00 (977.69)
MIN. WATER EL 289.00 (948.15)
BEDROCK
C - C & F - F REF. 1
FROST PROTECTION CURTAIN
1:125
MOTOR, GEARBOXES AND STEMS TOGETHER WITH PANELS IN RAISED POSITION
FRAME
MAX. WATER EL 298.0 (977.69)
UPPER PANEL
NON RISING STEM, ONE EACH SIDE
FLEXIBLE SEALING STRIP
LOWER PANEL
MIN WATER EL 289.0 (948.15)
IN LOWEST OPERATING POSITION
IN RAISED POSITION
J-J & K-K
SECTIONS THROUGH FROST PROTECTION CURTAIN
1:125
2 SPACES @ 0.30 (1.0) FOR PASSAGE OF LARGE UPSTREAM MIGRATING FISH.
1.90 (5.5)
DETAIL 'X' REF. 1
1:125
BY-PASS ENTRANCE WITH ALTERNATIVE ARRANGEMENT FOR PASSAGE OF LARGE UPSTREAM MIGRATING FISH
STAINLESS STEEL NON RISING STEM (ABOVE)
STAINLESS STEEL FROST PROTECTION PANEL
STAINLESS STEEL GUIDE EMBEDED IN CONCRETE
N-N
FROST PROTECTION PANEL GUIDE
1:5
REMOVABLE GRATING
GRATING COVER FOR LADDER
100ø FLEXIBLE HOSE FOR SOLIDS DISCHARGE
REMOVABLE HANDRAIL
H H
1:125
CAGED LADDER TO BOTTOM OF CELL
HALF ENCLOSED LADDER ACCESS TO TROLLEY
REMOVABLE CHAIN (FAR SIDE)
STAINLESS STEEL GRATING
CELL WALL
M-M
1:50
STEEL WIRE ROPE
CAGED LADDER
PUMP COLUMN
STOP LOGS
FLEXIBLE HOSE
TROLLEY
G-G
1:125
GRIT CHAMBER SOLIDS REMOVAL DETAILS
FRAZIL ICE BELT CONVEYOR
FROST PROTECTION PANELS IN RAISED POSITION
ACCESS COVERS
DETAIL 'Y' REF. 1
1:50
DETAIL 'Z' REF. 1
1:50
OPERATING FLOOR PLAN AT FROST PROTECTION CURTAINS
NOTES.
1. DIMENSIONS ARE IN METRES (FEET).
0 5 10 m
1:125
0 1 2 3 4 5 m
1:50
SANDWELL
DWG. NO. D4191 - 11
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
RIVER INTAKE - MISCELLANEOUS DETAILS SHEET 2
DATE MARCH, 1978
JOB No. V 4191
SCALE AS NOTED
DWG No. D4191 - 11
D 4191 - 9 GENERAL ARRANGEMENT
REFERENCE DRAWINGS
REVISIONS



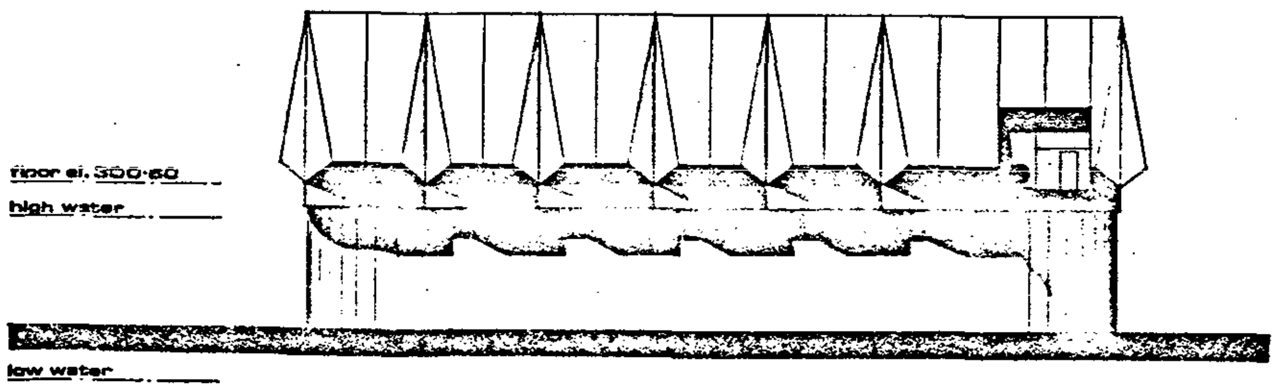
SOUTH ELEVATION



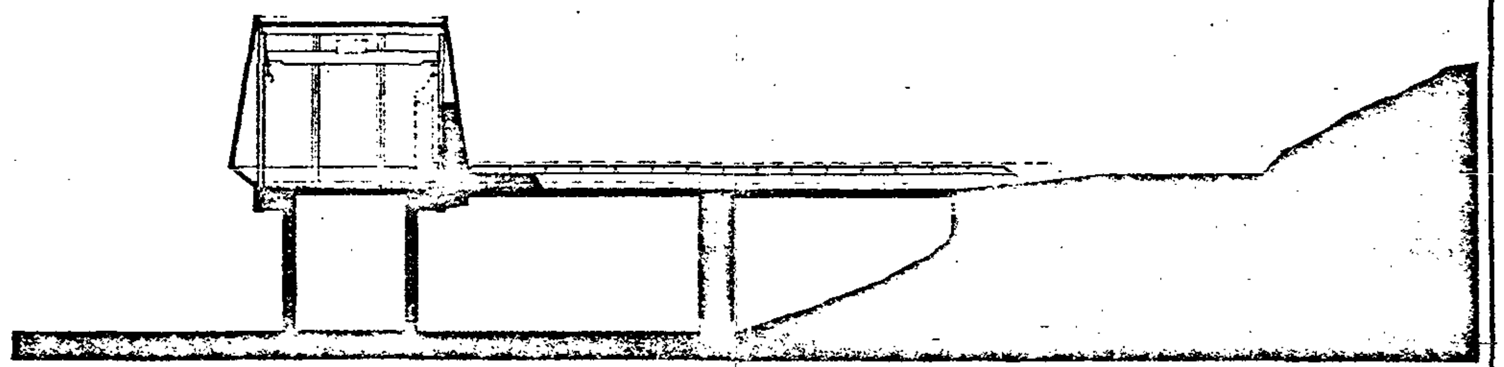
EAST ELEVATION



PLAN AT OPERATING FLOOR



NORTH ELEVATION



SECTION

SCALE 1:200 0 0 5 10 15 m

TOBY RUSSELL BUCKWELL & partners architects			
DES		RECD	
DWN		APPD (TECH)	
CHKD		APPD	
INSP		APPD	
SUBD		DATE	MARCH 1978
MICROFILMED		JOB No.	V 4191

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

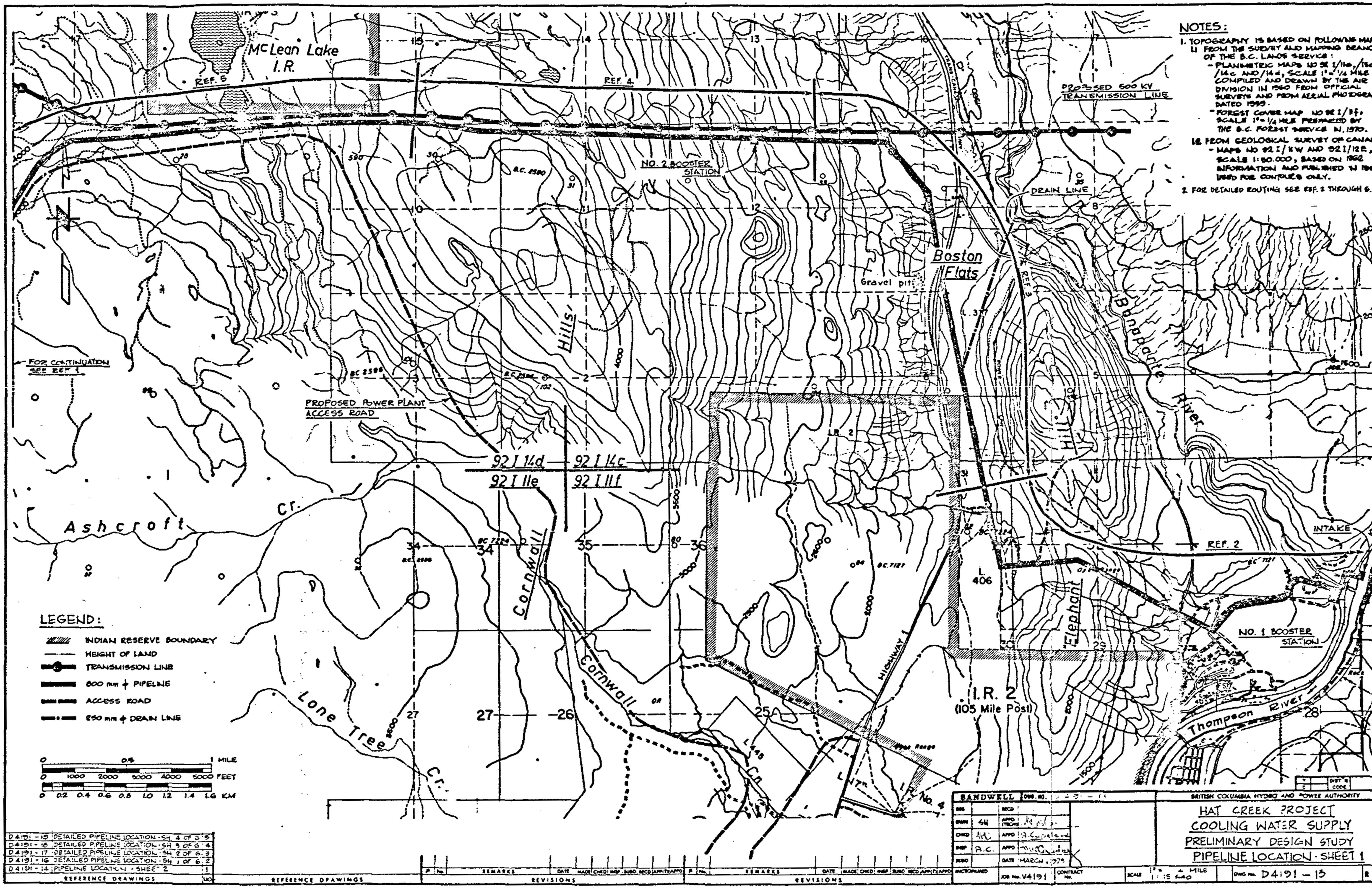
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE STRUCTURE ELEVATIONS

CONTRACT No. | SCALE 1:200 | DWG No. D 4191 - 12 | R

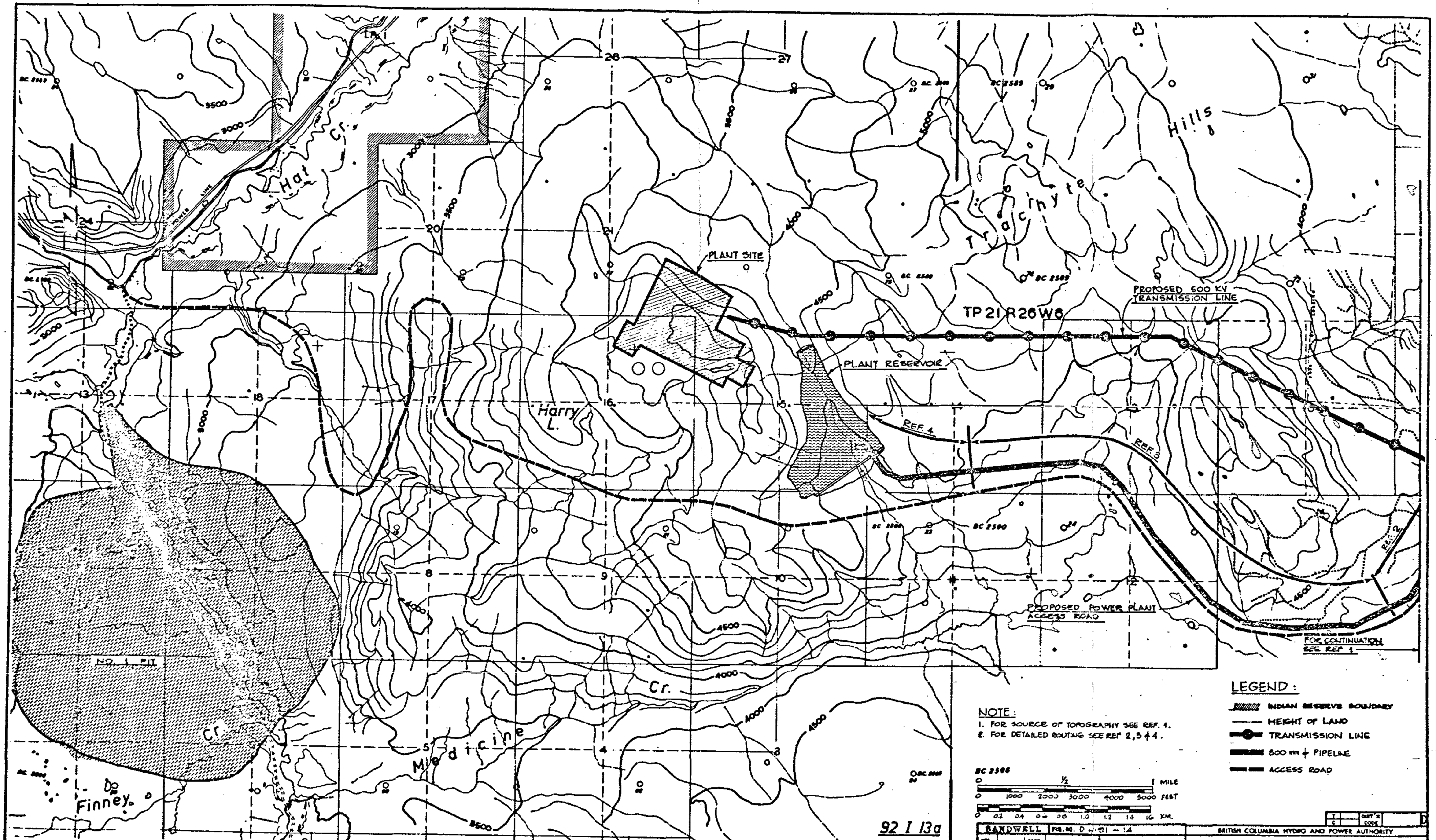
REFERENCE DRAWINGS | REFERENCE DRAWINGS

P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBD.	RECD	APP(T)	APPD	P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBD	RECD	APP(T)	APPD

REVISIONS | REVISIONS



McLean Lake
I.R.
REF. 5
REF. 4
PROPOSED 500 KV TRANSMISSION LINE
NO. 2 BOOSTER STATION
DRAIN LINE
Boston Flats
Gravel pit
REF. 3
Bonaparte River
Hills
Hill
FOR CONTINUATION SEE REF. 1
PROPOSED POWER PLANT ACCESS ROAD
92 I 14d
92 I 14c
92 I 11e
92 I 11f
Ashcroft
Cr.
Cornwall
Cornwall
Cornwall Cr.
Lone Tree
Cr.
I.R. 2
I.R. 2
(105 Mile Post)
L 406
Elephant
HIGHWAY 1
INTAKE
REF. 2
NO. 1 BOOSTER STATION
Thompson River
No. 4
NOTES:
1. TOPOGRAPHY IS BASED ON FOLLOWING MAPS
1.1 FROM THE SURVEY AND MAPPING BRANCH OF THE B.C. LANDS SERVICE:
- PLANIMETRIC MAPS NO 92 I/11e, /11f, /14c AND /14d, SCALE 1" = 1/4 MILE COMPILED AND DRAWN BY THE AIR DIVISION IN 1960 FROM OFFICIAL SURVEYS AND FROM AERIAL PHOTOGRAPHS DATED 1959.
- FOREST COVER MAP NO 92 I/11f, SCALE 1" = 1/4 MILE PREPARED BY THE B.C. FOREST SERVICE IN 1970.
1.2 FROM GEOLOGICAL SURVEY OF CANADA
- MAPS NO 92 I/11W AND 92 I/12E, SCALE 1:50.000, BASED ON 1962 INFORMATION AND PUBLISHED IN 1966. USED FOR CONTOURS ONLY.
2. FOR DETAILED ROUTING SEE REF. 2 THROUGH 5.
LEGEND:
INDIAN RESERVE BOUNDARY
HEIGHT OF LAND
TRANSMISSION LINE
800 mm ϕ PIPELINE
ACCESS ROAD
250 mm ϕ DRAIN LINE
0 0.5 1 MILE
0 1000 2000 3000 4000 5000 FEET
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 KM
D4191 - 19 DETAILED PIPELINE LOCATION - SH 4 OF 6 5
D4191 - 18 DETAILED PIPELINE LOCATION - SH 3 OF 6 4
D4191 - 17 DETAILED PIPELINE LOCATION - SH 2 OF 6 3
D4191 - 16 DETAILED PIPELINE LOCATION - SH 1 OF 6 2
D4191 - 14 PIPELINE LOCATION - SHEET 2 1
REFERENCE DRAWINGS
REFERENCE DRAWINGS
REVISIONS
REVISIONS
SANDWELL
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
PIPELINE LOCATION - SHEET 1
DATE MARCH, 1979
JOB No. V4191
SCALE 1" = 1/4 MILE 1:15 840
DWG No. D4191 - 13



D 4191 - 21	DETAILED PIPELINE LOCATION - Sh 6 OF 6	4
D 4191 - 20	DETAILED PIPELINE LOCATION - Sh 5 OF 6	3
D 4191 - 19	DETAILED PIPELINE LOCATION - Sh 4 OF 6	2
D 4191 - 13	PIPELINE LOCATION - SHEET 1	1
REFERENCE DRAWINGS		

REFERENCE DRAWINGS — REMARKS — DATE — MADE — CHKD — INSP — SUBD — RECD — APPR — APPD — REVISIONS

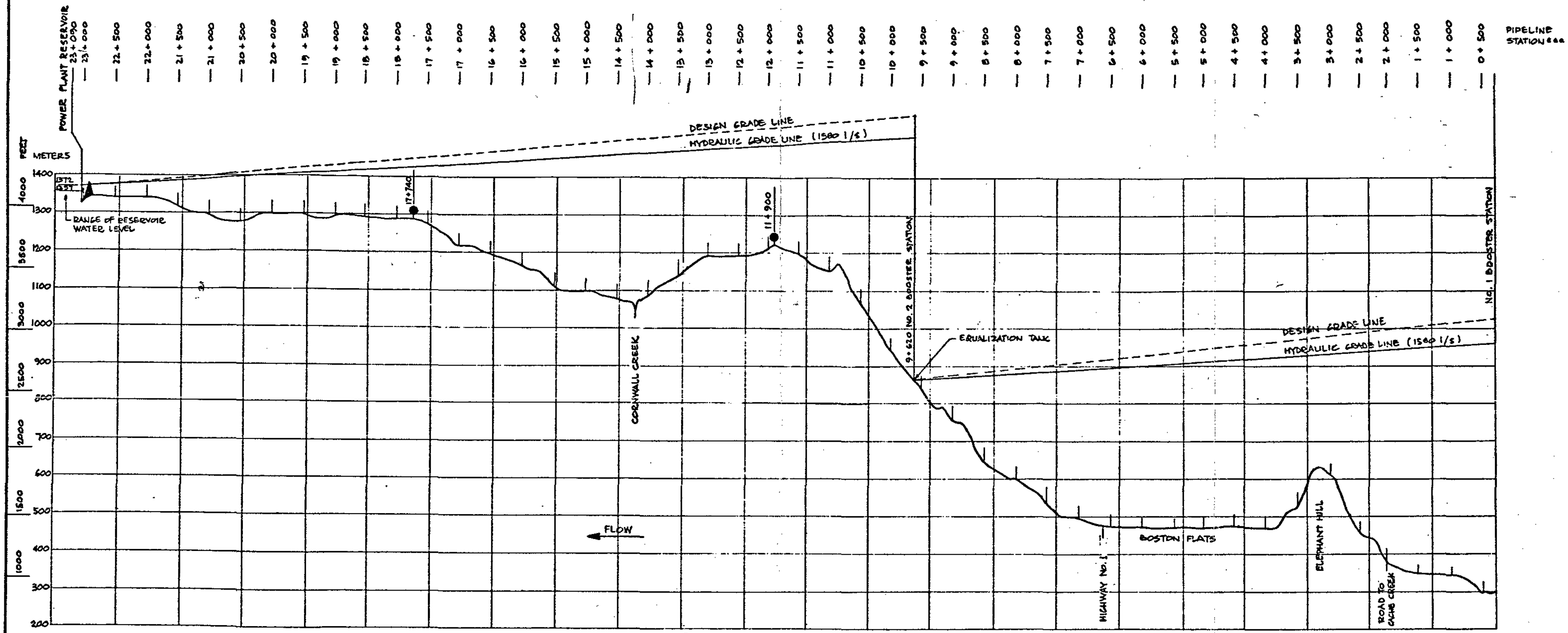
SANDWELL — File No. D 4191 - 14

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
PIPELINE LOCATION - SHEET 2

DRN	4H	APPD (TECH)	[signature]
CHKD	A.W.	APPD	[signature]
INSP	A.C.	APPD	[signature]
SUBD		DATE	MARCH, 1978

JOB No. V4191 — CONTRACT No. — SCALE 1" = 1/4 MILE 1 : 15 840 — DWG No. D4191 - 14 — R



TOTAL ACTUAL LENGTH	22,790	7480	2230	1980	880	870	2050	3150	680	1110	2360	ACTUAL LENGTHS (M) PER WALL THICKNESS
		8	11	8	11	16	8	11	16	11	16	WALL THICKNESS (MM)
		3.43	4.73	3.43	4.73	6.92	3.43	4.73	6.92	4.73	6.92	DESIGN PRESSURE (MPa) **

1910

9720

KILOMETERS *
MILES *

LEGEND & NOTES.

- ● ONE WAY SURGE TANK
- * HORIZONTAL LENGTH
- ** 1 MPa = 145 PSI
- *** CUMULATIVE PIPE LINE STATIONS ARE BASED ON ACTUAL LENGTH, WITH STATION 2+000 CHOSEN AT CROSSING WITH ROAD TO CACHE CREEK.

PROFILE

SCALES:

VERTICAL : 1:500 — 0 50 100 300 M

HORIZONTAL : 1:5000 — 0 1000 2000 3000 M

NOTE:

1. FOR DETAILED ROUTING SEE REF 1-6.

REFERENCE DRAWINGS		NO.
D 4191 - 21	DETAILED PIPELINE LOCATION - SH 6 OF 6	6
D 4191 - 20	DETAILED PIPELINE LOCATION - SH 5 OF 6	5
D 4191 - 19	DETAILED PIPELINE LOCATION - SH 4 OF 6	4
D 4191 - 18	DETAILED PIPELINE LOCATION - SH 3 OF 6	3
D 4191 - 17	DETAILED PIPELINE LOCATION - SH 2 OF 6	2
D 4191 - 16	DETAILED PIPELINE LOCATION - SH 1 OF 6	1

REFERENCE DRAWINGS — REVISIONS — REVISIONS

SANDWELL DWG. NO. D 4191 - 15

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
PIPELINE PROFILE

DES		RECD	
DWN	V.P.	APPD (TECH)	[illegible]
CHKD	[illegible]	APPD	A Copeland
INSP	A.C.	APPD	[illegible]
SUBD		DATE	MARCH, 978
MICROFILMED		JOB No. V 4191	CONTRACT No.

SCALE AS NOTED — DWG No. D 4191 - 15 — R

INDIAN RESERVE 2

(105 MILE POST)

NOTE:
1. FOR GENERAL NOTES SEE REF 3.

D4191 - 21	DETAILED PIPELINE LOCATION SHEET 6 OF 6	3
D4191 - 22	NO.1 BOOSTER STATION - GENERAL LAYOUT	2
D4191 - 17	DETAILED PIPELINE LOCATION SHEET 2 OF 6	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS

REVISIONS

REVISIONS

BANDWELL DWG. NO. D 4191 - 16

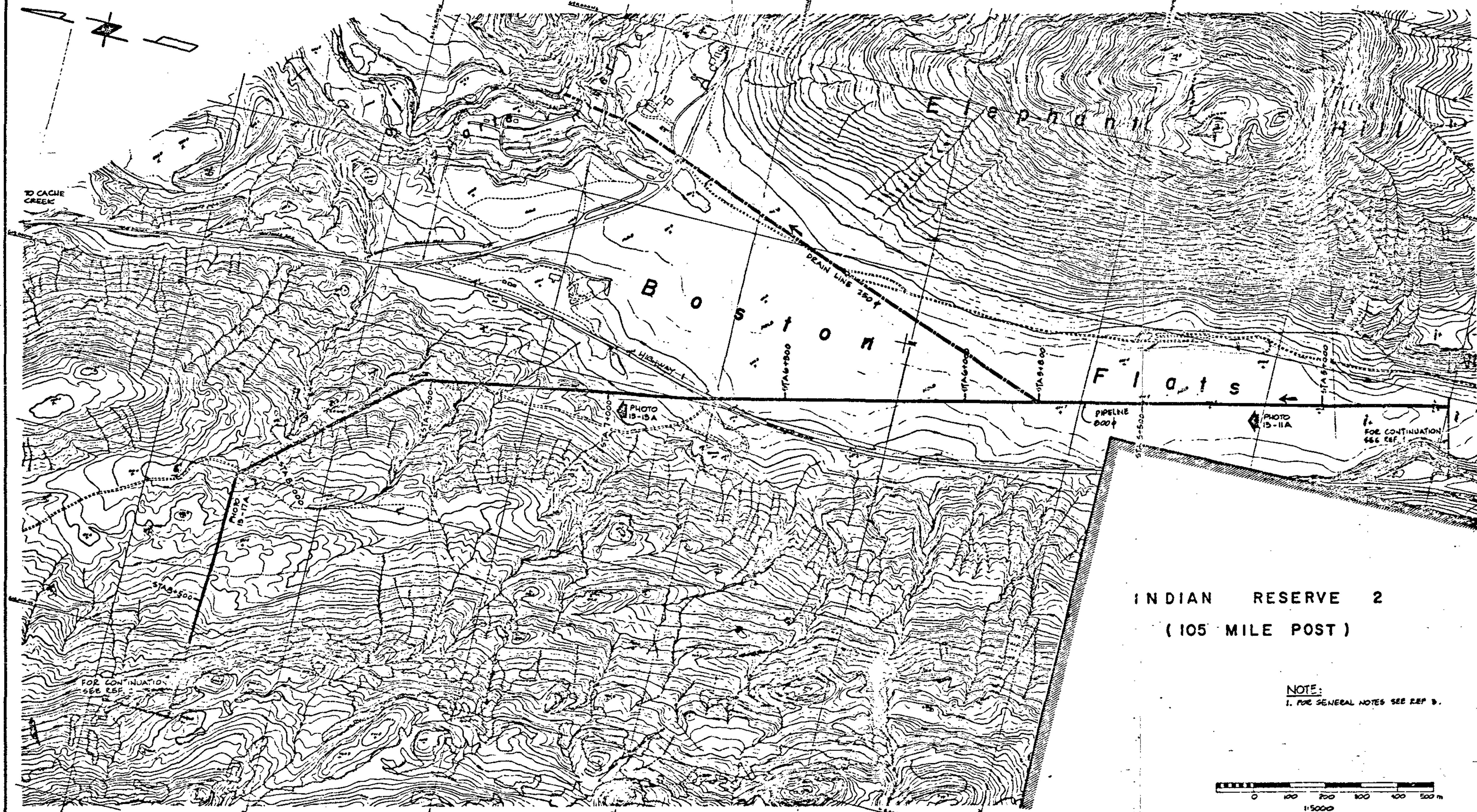
DES		RECO	
DWN	SH	APPD (TECH)	Ravi ...
CHKD	APB	APPD	A Copeland
INSP	A.C.	APPD	...
SUBD		DATE	MARCH 1976
MICROFILMED		JOB No. V419	CONTRACT No.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
DETAILED PIPELINE LOCATION SHEET 1 OF 6

SCALE 1:5,000

DWG No. D4191 – 16



24191-21	DETAILED PIPELINE LOCATION SHEET 6 OF 6	3
24191-18	DETAILED PIPELINE LOCATION SHEET 3 OF 6	2
24191-13	DETAILED PIPELINE LOCATION SHEET 1 OF 6	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS

REVISIONS

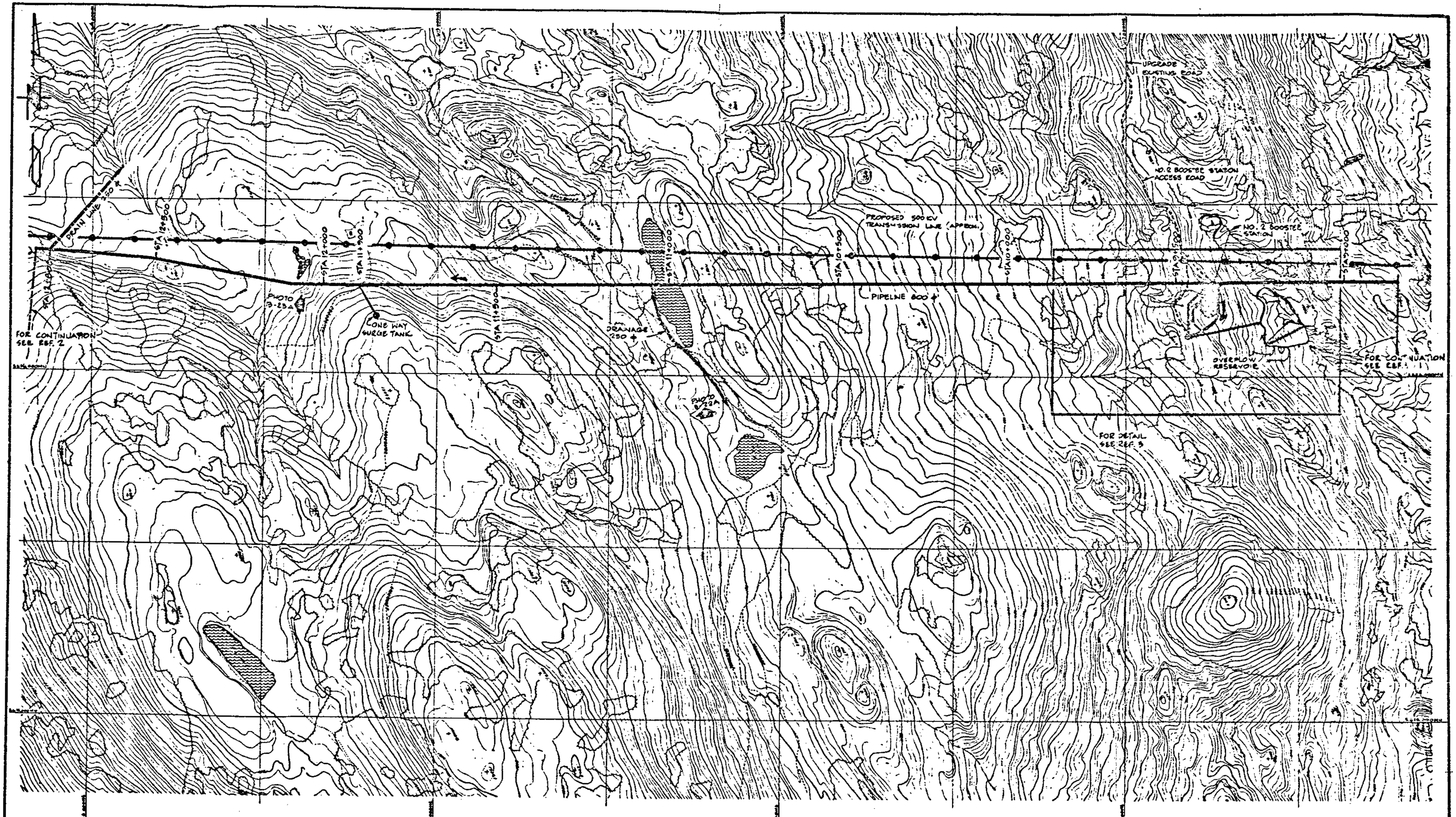
SANDWELL DWG. NO. 24191-17

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
DETAILED PIPELINE LOCATION SHEET 2 OF 6

DES		RECD	
DWN	4H	APPD (TECH)	[signature]
CHKD	HPB	APPD	[signature]
INSP	A.C	APPD	[signature]
SUBD		DATE	MARCH, 1975

MICROFILMED — JOB No. V4191 — CONTRACT No. — SCALE 1:5,000 — DWG No. 24191 — 17 — R



0 100 200 300 400 500m

1:5000

NOTE:

1. FOR GENERAL NOTES SEE REF 4.

D4191 - 21	DETAILED PIPELINE LOCATION SHEET 6 OF 6	4
D4191 - 23	NO 2 BOOSTER STATION - GENERAL LAYOUT	3
D4191 - 19	DETAILED PIPELINE LOCATION SHT 4 OF 6	2
D4191 - 17	DETAILED PIPELINE LOCATION SHT 2 OF 6	1
REFERENCE DRAWINGS		No

REFERENCE DRAWINGS

P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBD	RECD	APP(T)	APPD

REVISIONS

P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBD	RECD	APP(T)	APPD

REVISIONS

SANDWELL DWG. NO. D 4191 - 18

DSN		RECD	
DWN	SH	APPD (TECH)	K Dixon
CHKD	JPB	APPD	A Copeland
INSP	AC.	APPD	[signature]
SUBD		DATE	MARCH, 1978
MICROFILMED			

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

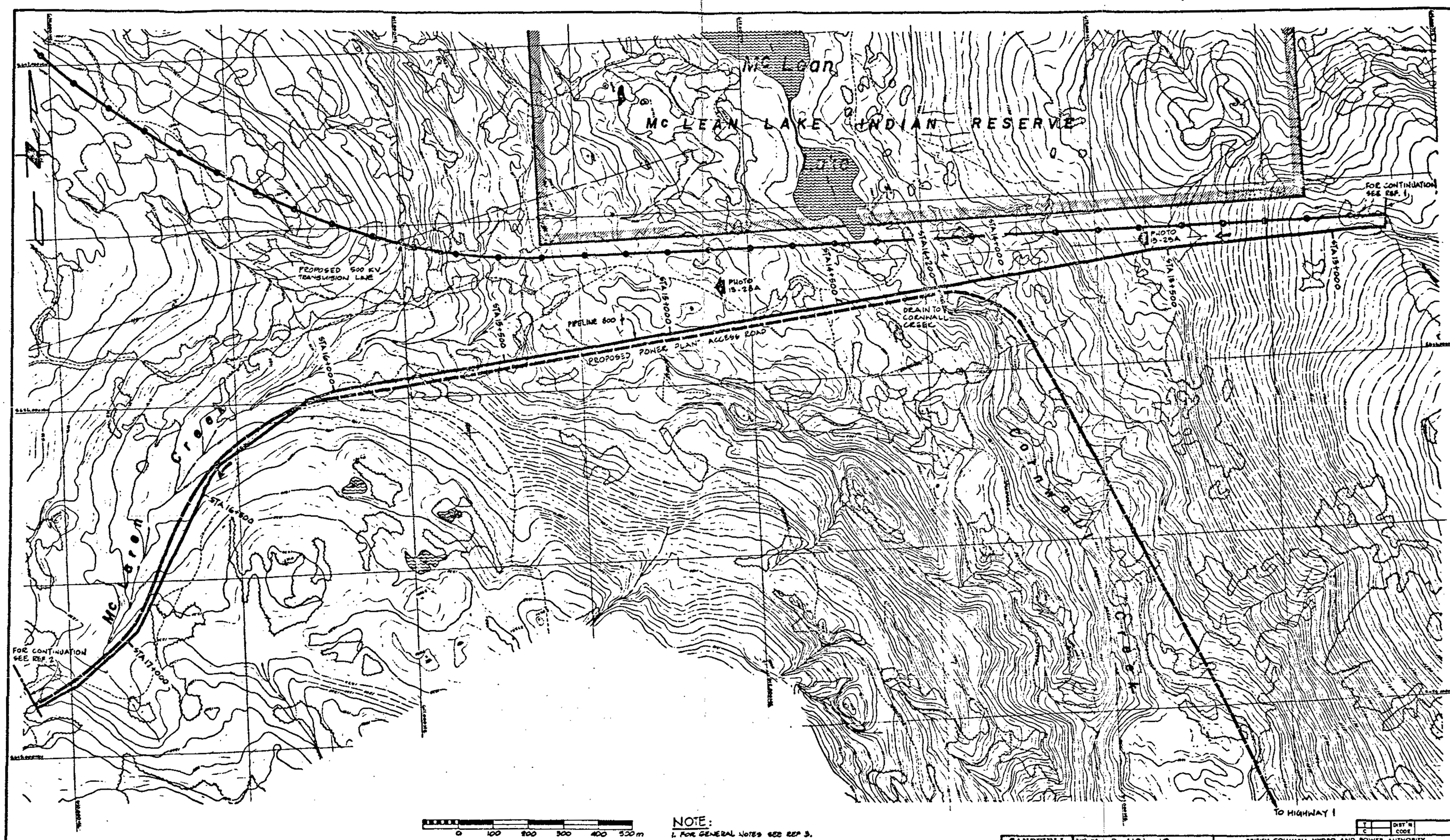
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

DETAILED PIPELINE LOCATION SHEET 3 OF 6

JOB No V4191 | CONTRACT No | SCALE 1:5,000 | DWG No D4191 - 18 | R

DIST'N CODE



NOTE:
1. FOR GENERAL NOTES SEE REF 3.

D4191 - 21	DETAILED PIPELINE LOCATION SHEET 6 OF 6	3
D4191 - 20	DETAILED PIPELINE LOCATION SHEET 5 OF 6	2
D4191 - 15	DETAILED PIPELINE LOCATION SHEET 3 OF 6	1
	REFERENCE DRAWINGS	NO

REFERENCE DRAWINGS | REVISIONS | REVISIONS

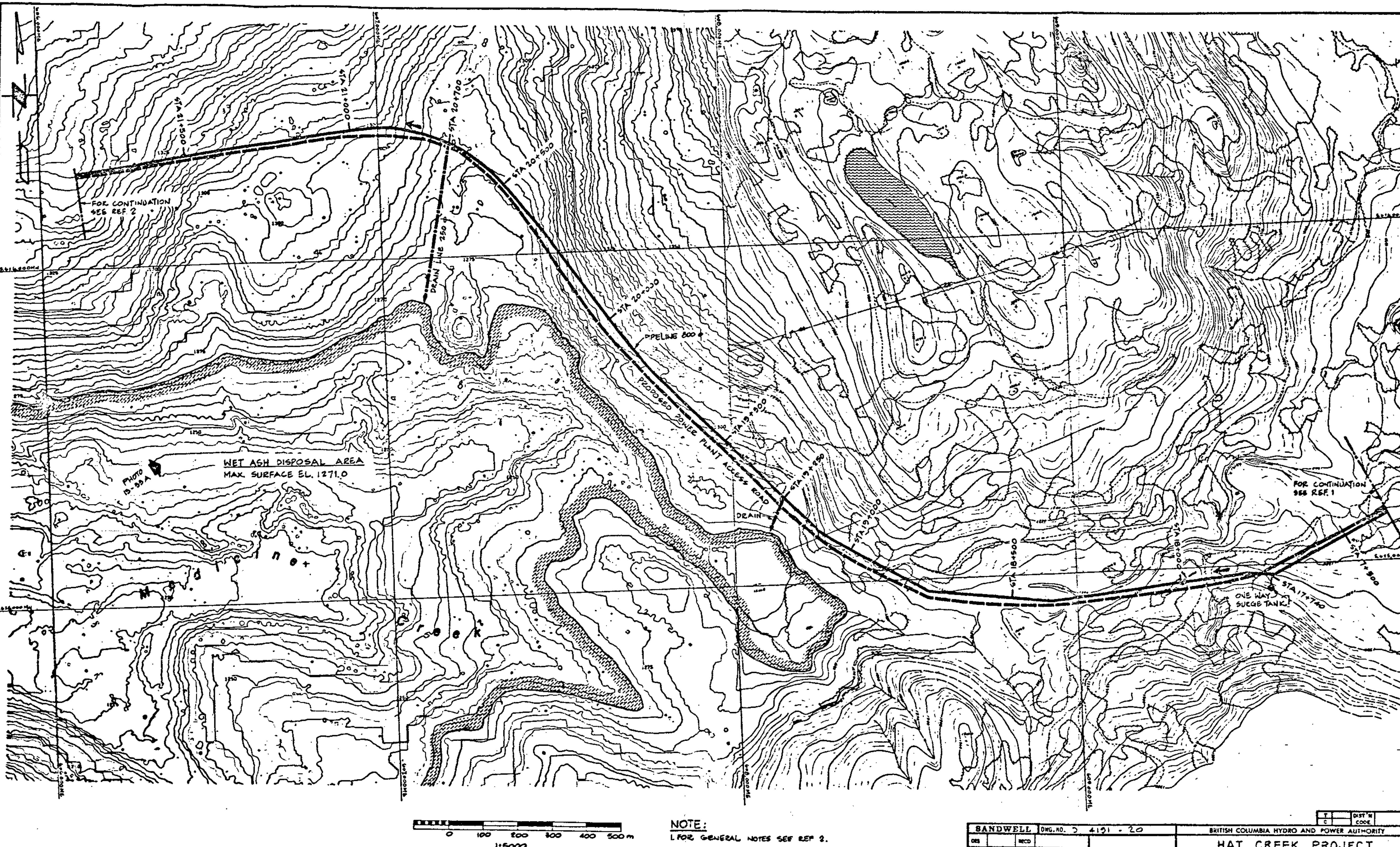
SANDWELL DWG. NO. D 4191 - 19

DES		RECD	
DWN	SH	APPD (TECH)	Barton
CHKD	APB	APPD	A Copeland
INSP	A.C.	APPD	[illegible]
SUBD		DATE	MARCH, 1978

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
DETAILED PIPELINE LOCATION SHEET 4 OF 6

MICROFILMED | JOB No. V419 | CONTRACT No. | SCALE 1:5,000 | DWG No. D4191 - 19 | R



NOTE:
1. FOR GENERAL NOTES SEE REF 2.

D4191 - 21	DETAILED PIPELINE LOCATION SHEET 6 OF 6	2
D4191 - 19	DETAILED PIPELINE LOCATION SHEET 4 OF 6	1
REFERENCE DRAWINGS		No.

REFERENCE DRAWINGS

P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBO	RECD	APP(T)	APPO

REVISIONS

P	No.	REMARKS	DATE	MADE	CHKD	INSP	SUBO	RECD	APP(T)	APPO

REVISIONS

SANDWELL DWG. NO. D 4191 - 20

DES		RECD	
DWN	SH	APPD (TECH)	[signature]
CHKD	JPB	APPD	A Copeland
INSP	A.C.	APPD	[signature]
SUBO		DATE	MARCH, 1978

MICROFILMED

JOB No. V 4191 CONTRACT No.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

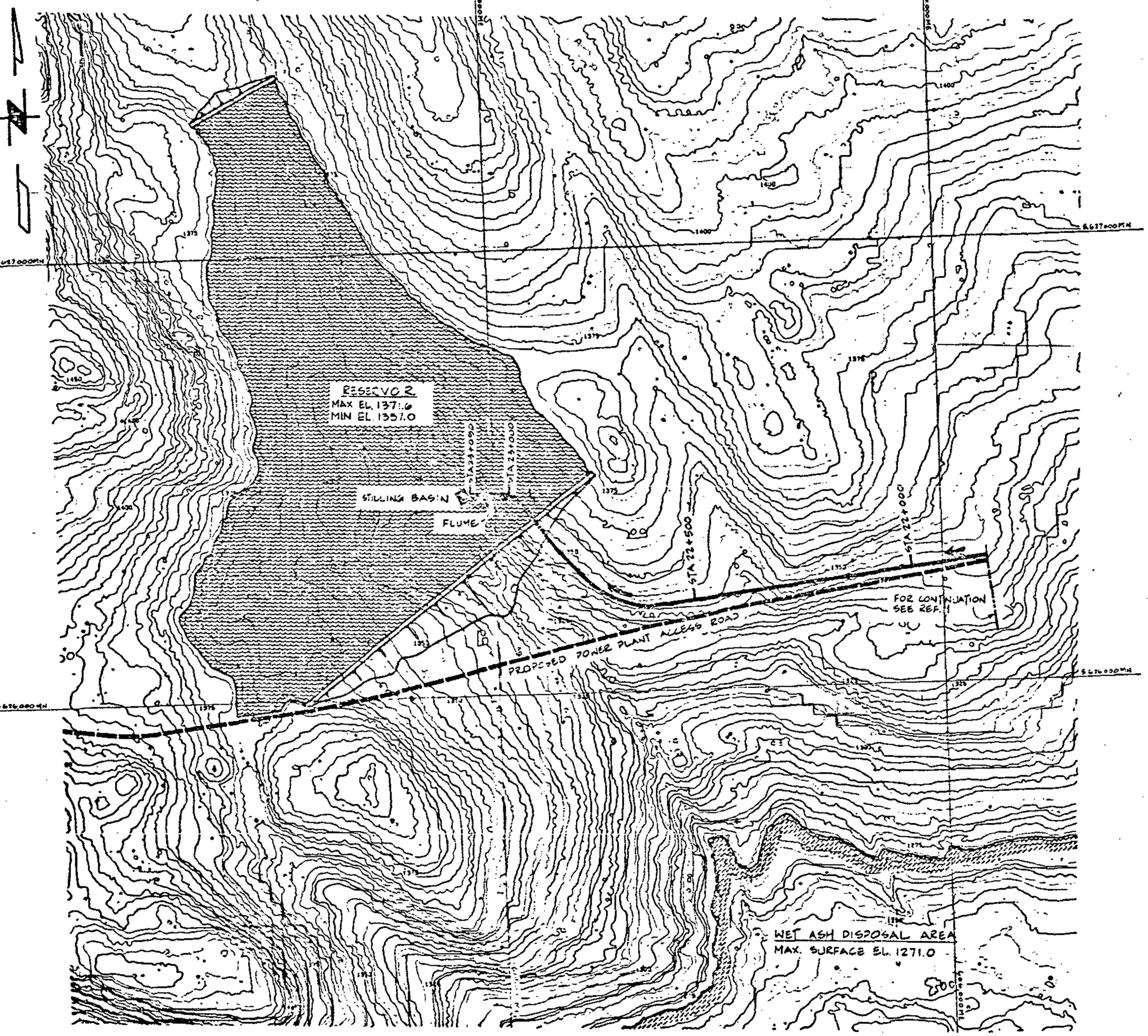
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

DETAILED PIPELINE LOCATION SHEET 5 OF 6

SCALE 1:5,000

DWG No. D4191 - 20 R



GENERAL NOTES:

1. TOPOGRAPHY FROM INTEGRATED RESOURSES PHOTOGRAPHY LTD., OCT 1977 AND FROM B.C. HYDRO AND POWER AUTHORITY.
2. FOR PHOTOGRAPHS SEE APPENDIX 7.
3. FOR GENERAL PIPELINE LOCATION SEE REF. 2 & 3.

LEGEND:

- BURIED 600 mm ϕ PIPELINE
- PROPOSED ACCESS ROAD
- DRAIN LINE
- PROPOSED 500KV TRANSMISSION LINE
- LOCATION OF PHOTOGRAPH

D 4191-14	PIPELINE LOCATION - SHEET 2	3
D 4191-18	PIPELINE LOCATION - SHEET 1	2
D 4191-20	DETAILED PIPELINE LOCATION - SH 5 OF 6	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS — REVISIONS — REMARKS — DATE — MADE — CHKD — INSP — SUBD — RECD — APPD — REVISIONS

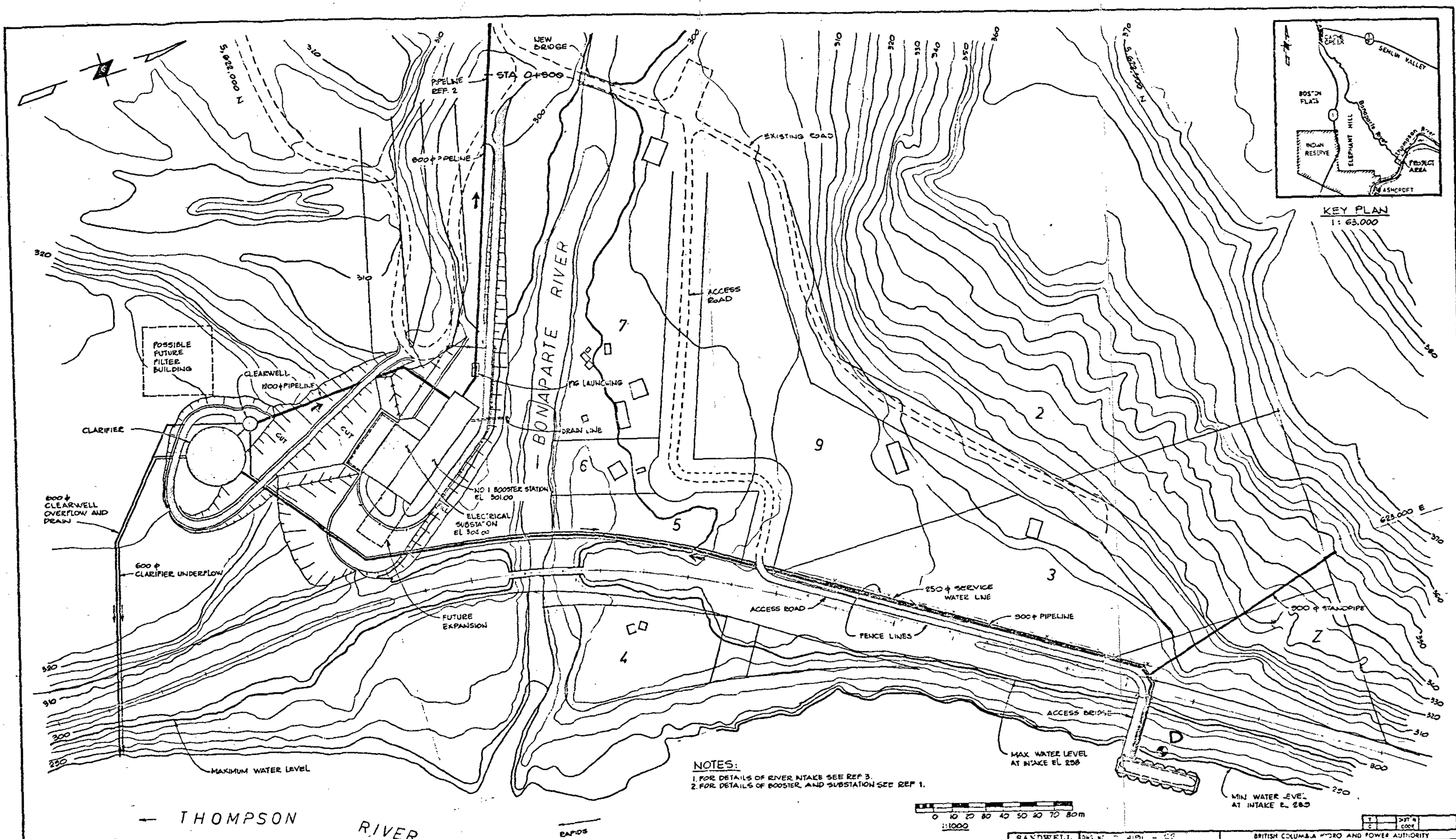
SANDWELL	DWG. NO. D 4191 - 21		
DES		RECD	
DWN	SH	APPD (TECH)	[signature]
CHKD	APC	APPD	A Copeland
INSP	A.C.	APPD	[signature]
SUBD		DATE	MARCH, 1978
MICROFILMED	JOB No. V4191	CONTRACT No.	

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
DETAILED PIPELINE LOCATION SHEET 6 OF 6

SCALE 1:5,000 — DWG No. D4191 - 21 — R

T C — DIST'N CODE



NOTES:
1. FOR DETAILS OF RIVER INTAKE SEE REF 3.
2. FOR DETAILS OF BOOSTER AND SUBSTATION SEE REF 1.

D 4191-5	RIVER INTAKE - GENERAL ARRANGEMENT	3
D 4191-16	DETAILED PIPELINE LOCATION - SHEET 1 OF 6	2
D 4191-24	BOOSTER STATION - GENERAL ARRG'T	1
REFERENCE DRAWINGS		No.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

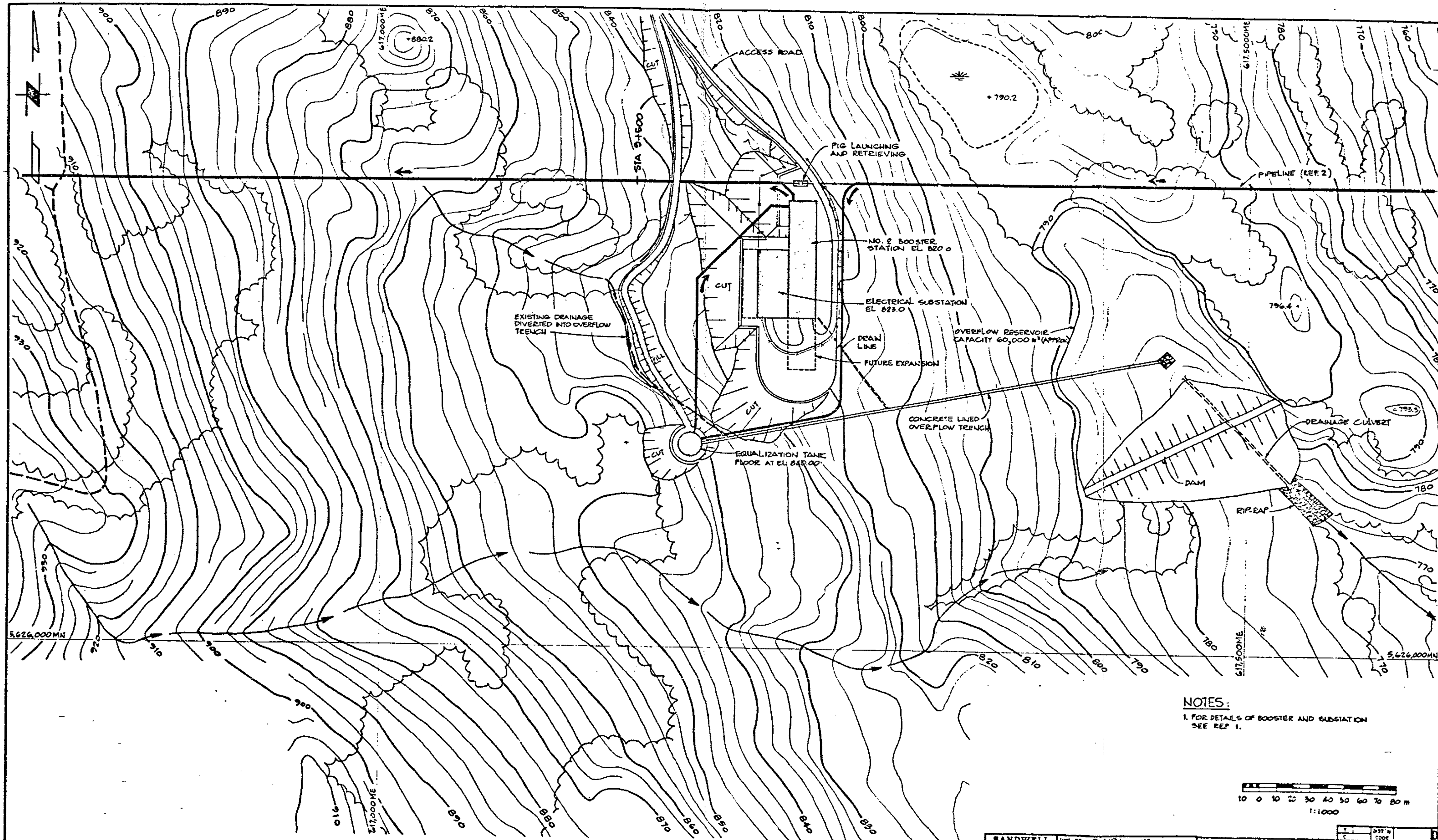
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
NO1. BOOSTER STATION - GENERAL LAYOUT

SANDWELL DWG. No D 4191 - 22

DES		RECD	
DWN	JH	APPD (TECH)	J C [signature]
CHKD	NRB	APPD	A Campbell
INSP	AC	APPD	[signature]
SUBD		DATE	MARCH '78
MICROFILMED		JOB No V 419.	CONTRACT No

SCALE AS NOTED | DWG. No D 419. - 22 | R

REFERENCE DRAWINGS — REVISIONS



D 4191-18	DETAILED PIPELINE LOCATION - SHEET 3 OF 6	2
D 4191-24	BOOSTER STATION - GENERAL ARR'G'T	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS

REVISIONS

SANDWELL DWG. NO. D 4191 - 23

DES		RECD	
DWN	SH	APPD (TECH)	[signature]
CHKD	[initials]	APPD	A Copeland
INSP	A.C	APPD	[signature]
SUBD		DATE	MARCH 1978
MICROFILMED		JOB No. V 4191	CONTRACT NO

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

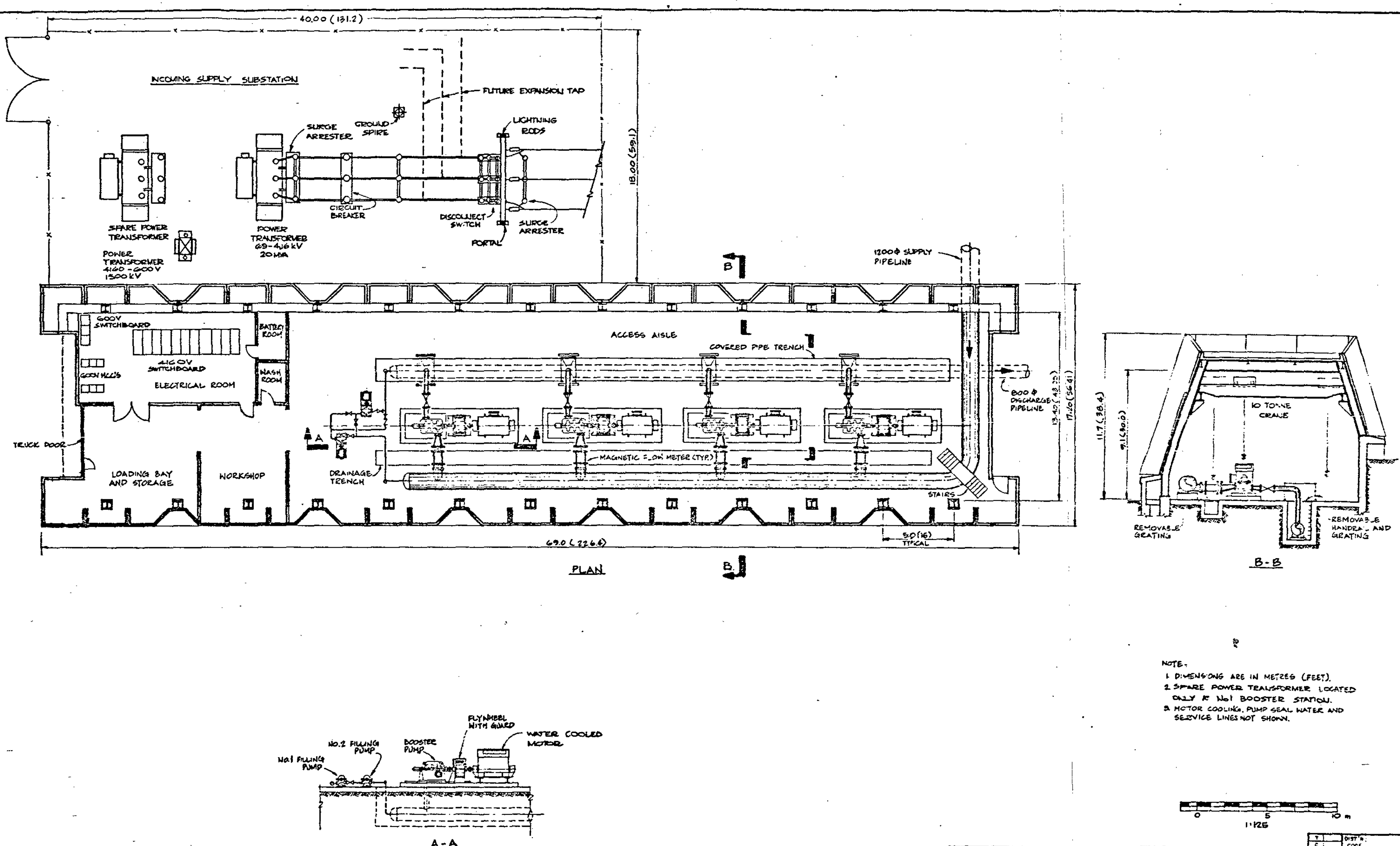
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

NO. 2 BOOSTER STATION - GENERAL LAYOUT

SCALE 1:1,000

DWG No. D 49 - 23

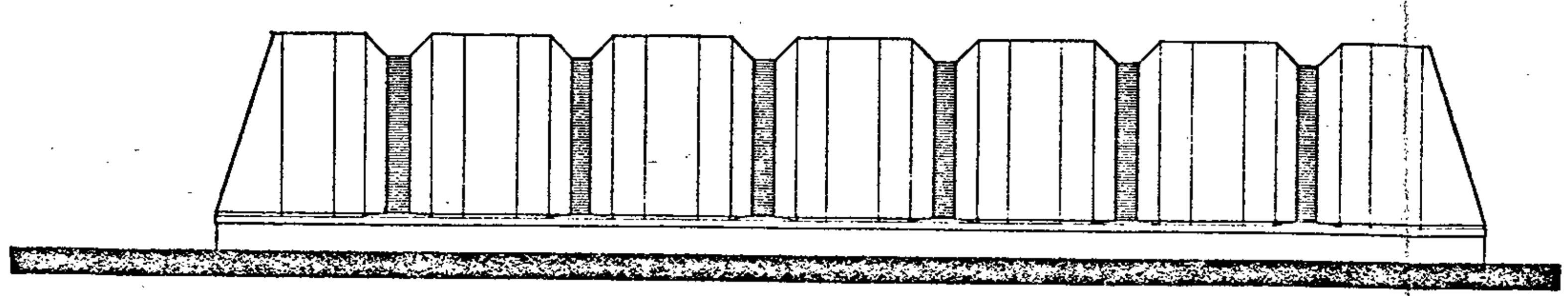


NOTE:
1. DIMENSIONS ARE IN METRES (FEET).
2. SPARE POWER TRANSFORMER LOCATED ONLY AT No.1 BOOSTER STATION.
3. MOTOR COOLING, PUMP SEAL WATER AND SERVICE LINES NOT SHOWN.

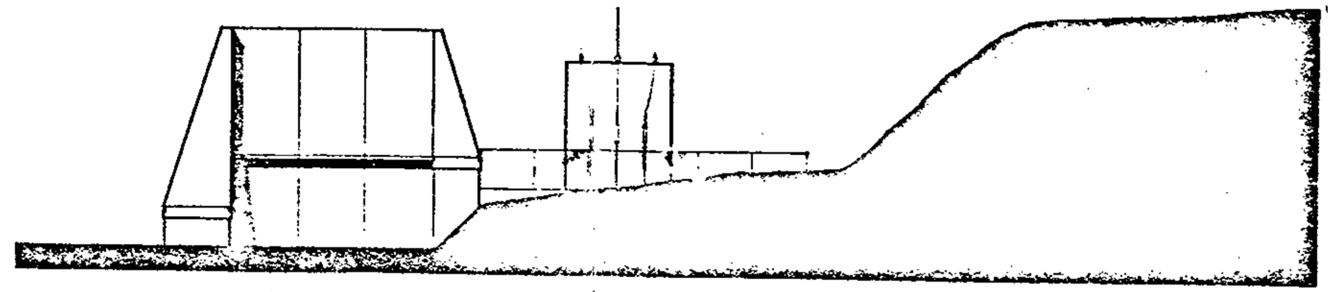
0 5 10 m
1:125

SANDWELL	DWG. NO. D 4191 - 26	BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
DES	RECD	HAT CREEK PROJECT
DWN D.TUNG	APPD (TECH) R Kallberg	COOLING WATER SUPPLY
CHKD R Kallberg	APPD A Lo	PRELIMINARY DESIGN STUDY
INSP A.C.	APPD McCammon	BOOSTER STATION GENERAL ARRANGEMENT
SUBD	DATE MARCH, 1978	
MICROFILMED	JOB No. V4191	CONTRACT No.	SCALE 1:125	DWG No. D4191 - 24	R

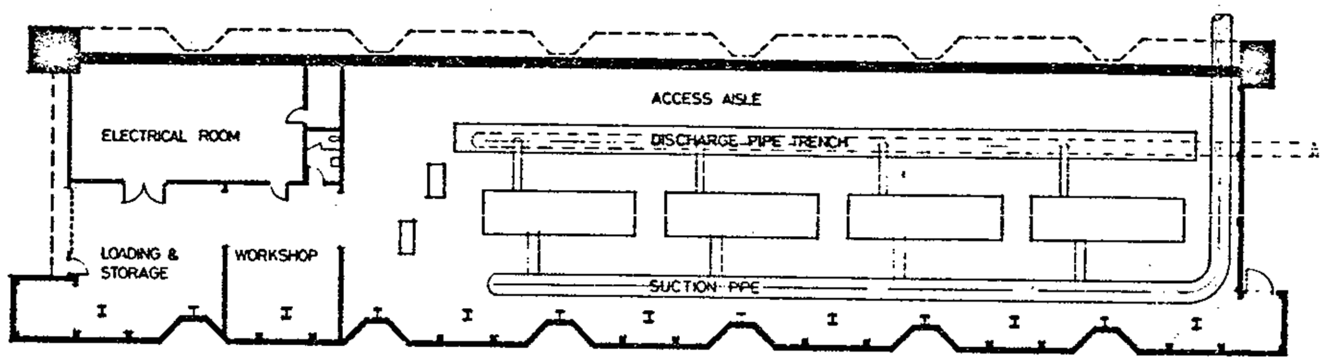
REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS



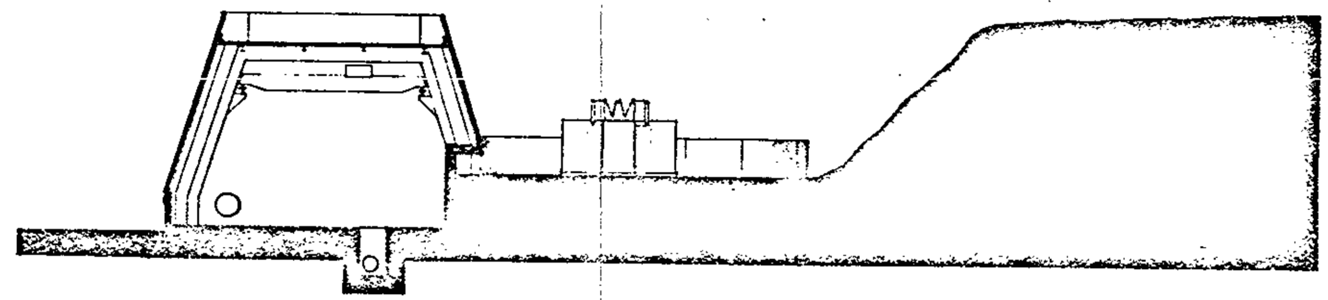
FRONT ELEVATION



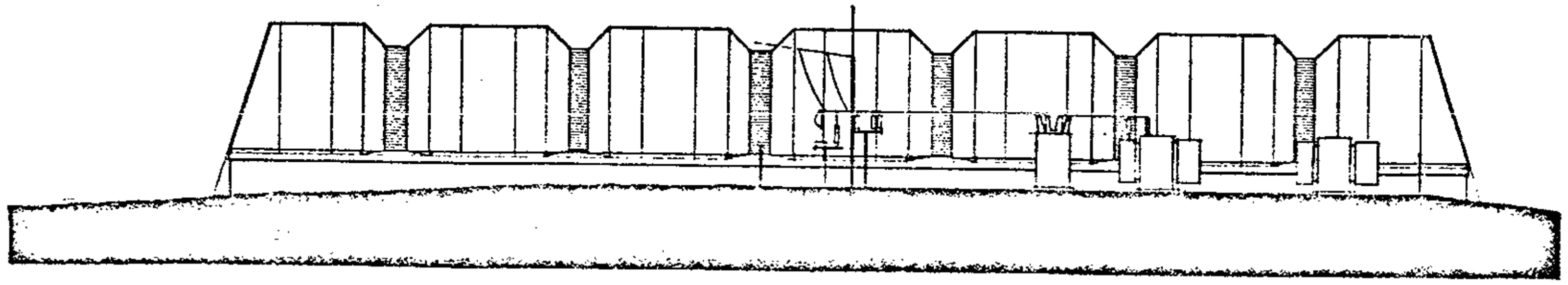
SIDE ELEVATION



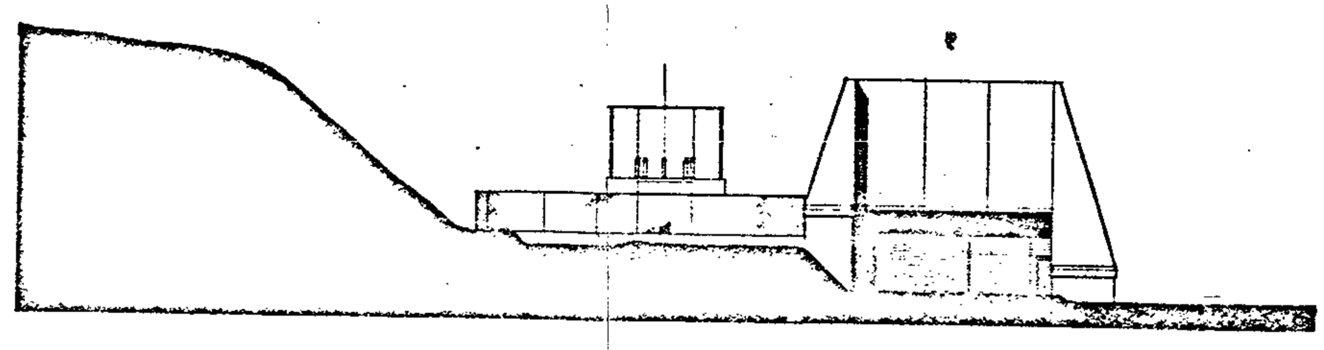
PLAN



SECTION



REAR ELEVATION



SIDE ELEVATION

SCALE 1:200 5 0 5 10 15m

TOBY RUSSELL BUCKWELL & partners architects			
DES		RECD	
DWN		APPD (TECH)	
CHKD		APPD	
INSP		APPD	
SUBD		DATE	MARCH, 1978
MICROFILMED	JOB No. V 4191	CONTRACT No	

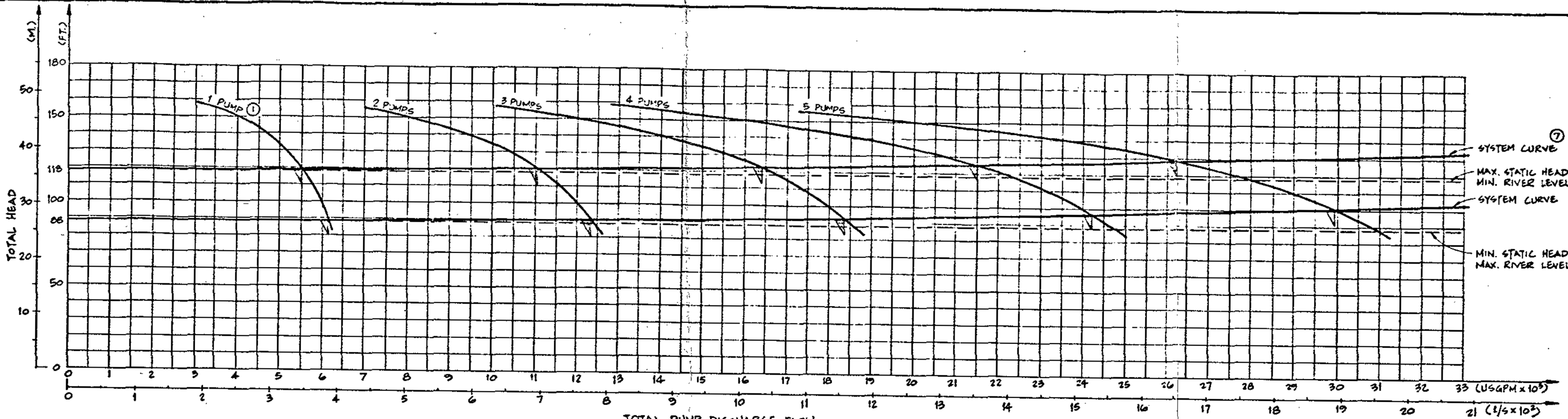
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
BOOSTER STATION ELEVATIONS

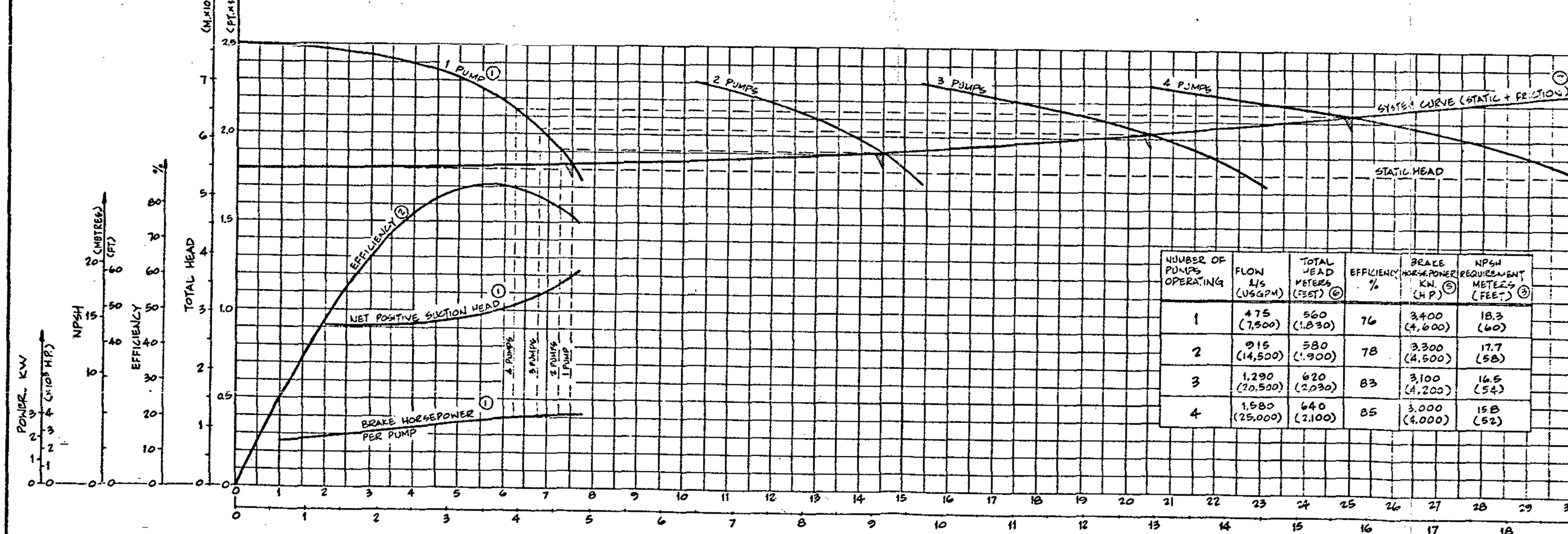
SCALE 1 : 200 DWG No. D 4191 - 25 R

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS (P No, REMARKS, DATE, MADE, CHKD, INSP, SUBD, RECD, APPIT, APPD) | REVISIONS (P No, REMARKS, DATE, MADE, CHKD, INSP, SUBD, RECD, APPIT, APPD)

DIST'N CODE



TOTAL PUMP DISCHARGE FLOW
INTAKE PUMP PERFORMANCE



NUMBER OF PUMPS OPERATING	FLOW ℓ/s (USGPM)	TOTAL HEAD METERS (FEET) ⑥	EFFICIENCY %	BRAKE HORSEPOWER KW. ⑤ (H.P.)	NPSH REQUIREMENT METERS (FEET) ③
1	475 (7,500)	560 (1,830)	76	3,400 (4,600)	18.3 (60)
2	915 (14,500)	580 (1,900)	78	3,300 (4,500)	17.7 (58)
3	1,290 (20,500)	620 (2,030)	83	3,100 (4,200)	16.5 (54)
4	1,580 (25,000)	640 (2,100)	85	3,000 (4,000)	15.8 (52)

TOTAL PUMP DISCHARGE FLOW
BOOSTER PUMP PERFORMANCE

NOTES.

① CURVES FOR EFFICIENCY, NPSH, HEAD-CAPACITY, AND BHP ARE APPROXIMATED FROM DATA OF SEVERAL MANUFACTURERS.

② EFFICIENCY CURVE CAN BE SHIFTED TO OBTAIN BETTER EFFICIENCIES WITH DIFFERENT MODES OF OPERATION.

③ NPSH VARIES WITH MANUFACTURER. TABLE ASSUMES DOUBLE SUCTION FIRST STAGE.

4. PERFORMANCE IS APPROXIMATELY EQUAL FOR BOTH BOOSTER STATIONS.

⑤ BRAKE HORSEPOWER REQUIRED PER PUMP.

⑥ TOTAL HEAD INCLUDING STATIC AND FRICTION.

⑦ THE SYSTEM CURVE SHOWN IS AN APPROXIMATE AVERAGE FOR THE TWO BOOSTER STATIONS. FOR ACCURATE DETERMINATION OF INTERMEDIATE FLOW RATES, THE VARIATION IN RESERVOIR AND EQUALIZATION TANK WATER LEVELS, DIFFERENCES IN STATIC HEAD AND EXACT PUMP CURVES NEED TO BE INCLUDED.

8. SYSTEM CURVE BASED ON 900 mm φ (36") PIPELINE FOR INTAKE PUMP AND 800 mm φ (32") PIPELINE FOR BOOSTER PUMPS.

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS

DES		RECD	
DWN	D TUNG	APPD	R VALLBERG
CHKD	[illegible]	APPD	R Copeland
INSP	A.C.	APPD	[signature]
SUBD		DATE	MARCH 197[illegible]
MICROFILMED		DIST'N CODE	

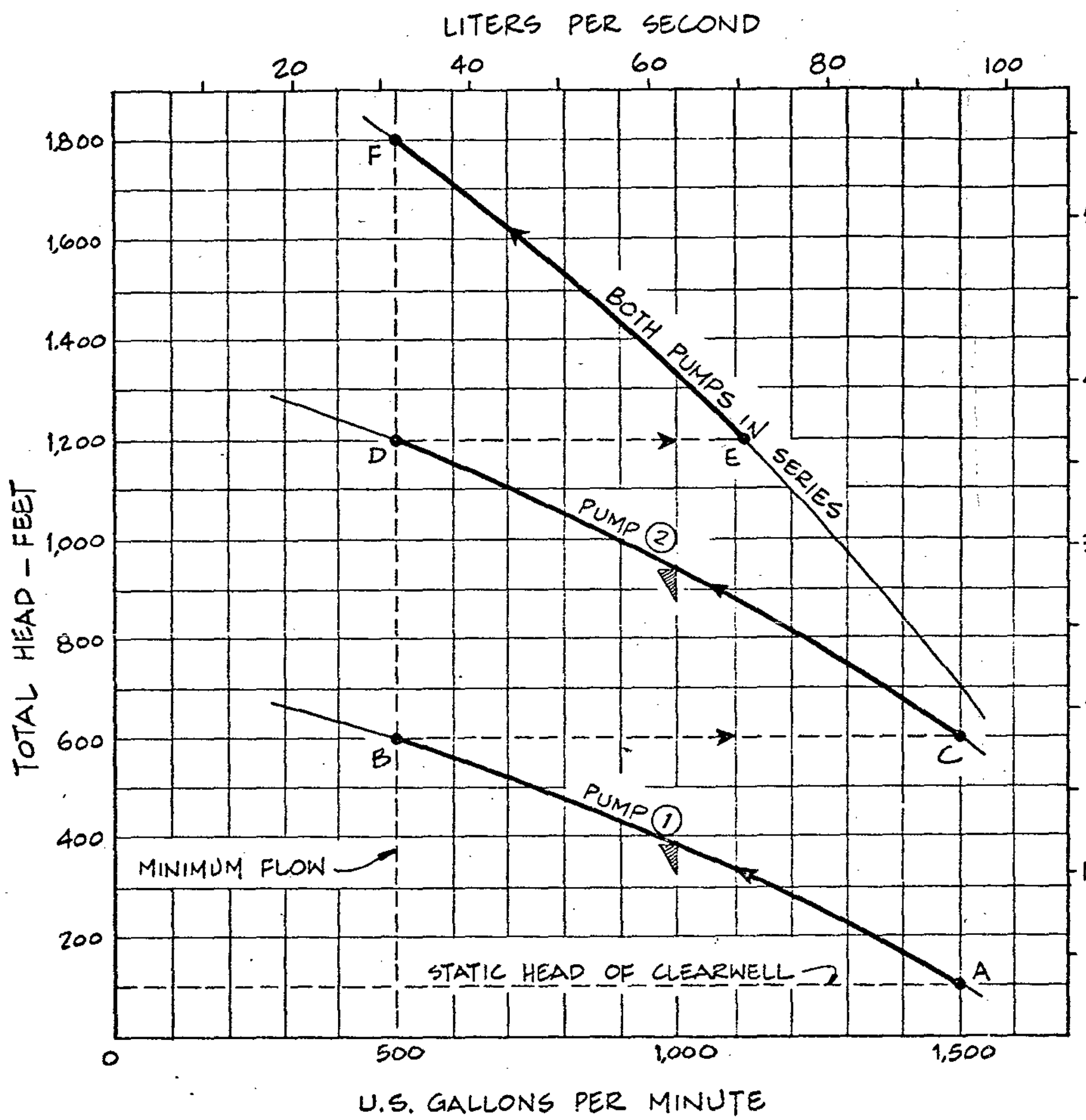
SANDWELL | DWG. NO. D 4191-26

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

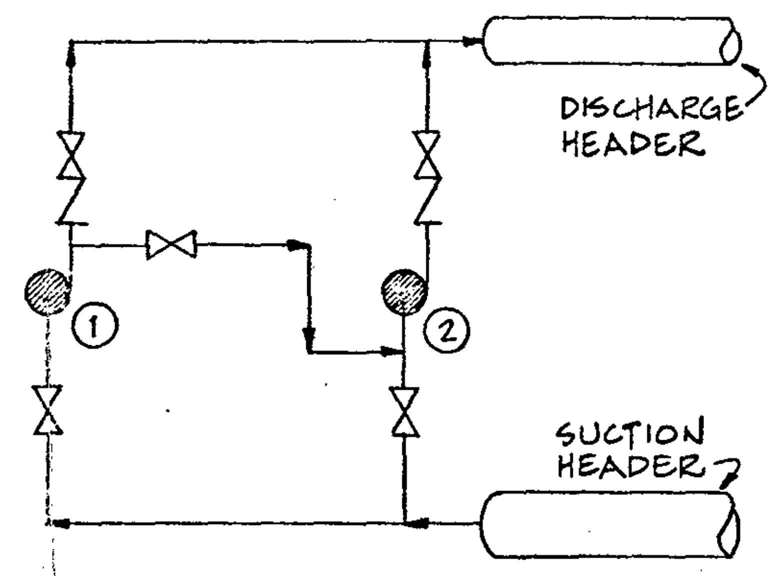
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
BOOSTER & INTAKE PUMPS - PERFORMANCE CURVES

SCALE

DWG No. D 4191 — 26 | R



LINE FILLING PUMPS-PERFORMANCE CURVES



LINE FILLING PUMPS
(AT EACH BOOSTER STATION)

FILLING SEQUENCE.

1, FILL LINE WITH STATIC HEAD OF CLEARWELL.
2, START PUMP ①, PUMP OPERATES FROM 'A' UNTIL HEAD REACHES 'B'.
3, SHUT PUMP ①, START PUMP ②, PUMP ② OPERATES FROM 'C' UNTILL HEAD REACHES 'D'.
4, RUN BOTH PUMPS ① AND ② IN SERIES FROM HEAD AT AT 'E' TO 'F'.
5. HEAD IS NOW SUFFICIENT TO START BOOSTER PUMPS.

NOTES.

1. BOTH PUMPS RATED FOR FULL DISCHARGE HEAD.
2. ALL DISCHARGE VALVES ND 40 RATING.

DWG. No.	REFERENCE	REF. No.	BY	DATE	PRELIMINARY ISSUE
REV.	BY	DATE	REVISION	APP'D.	

APPROVED

DATE MARCH, 1978

B 4191-27

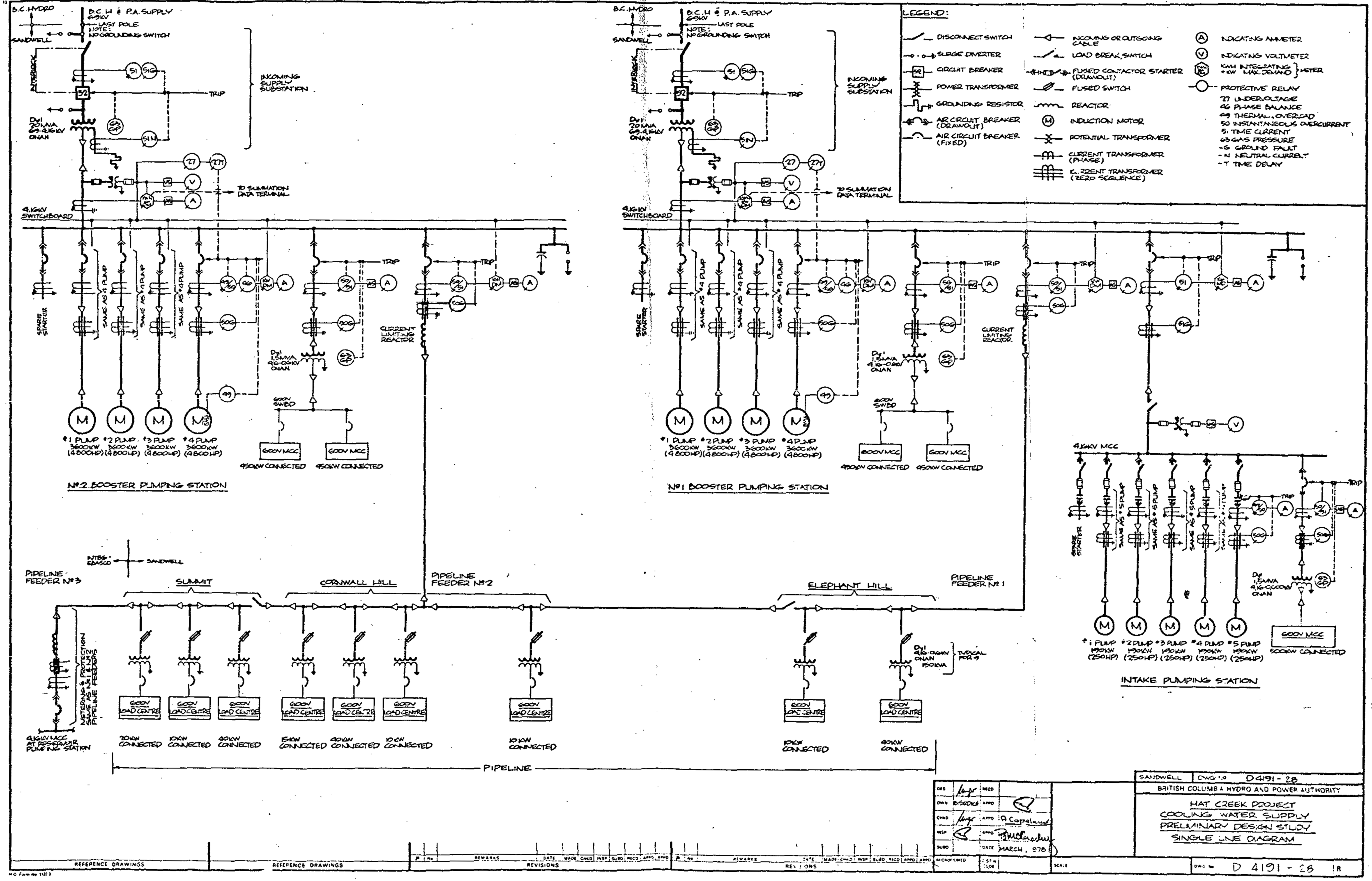
SCALE
DR'N. D. TUNG 22 12 77
CH'K'D. P. KALLBERG
APP'D. A. Sepolaw
APP'D.

SANDWELL

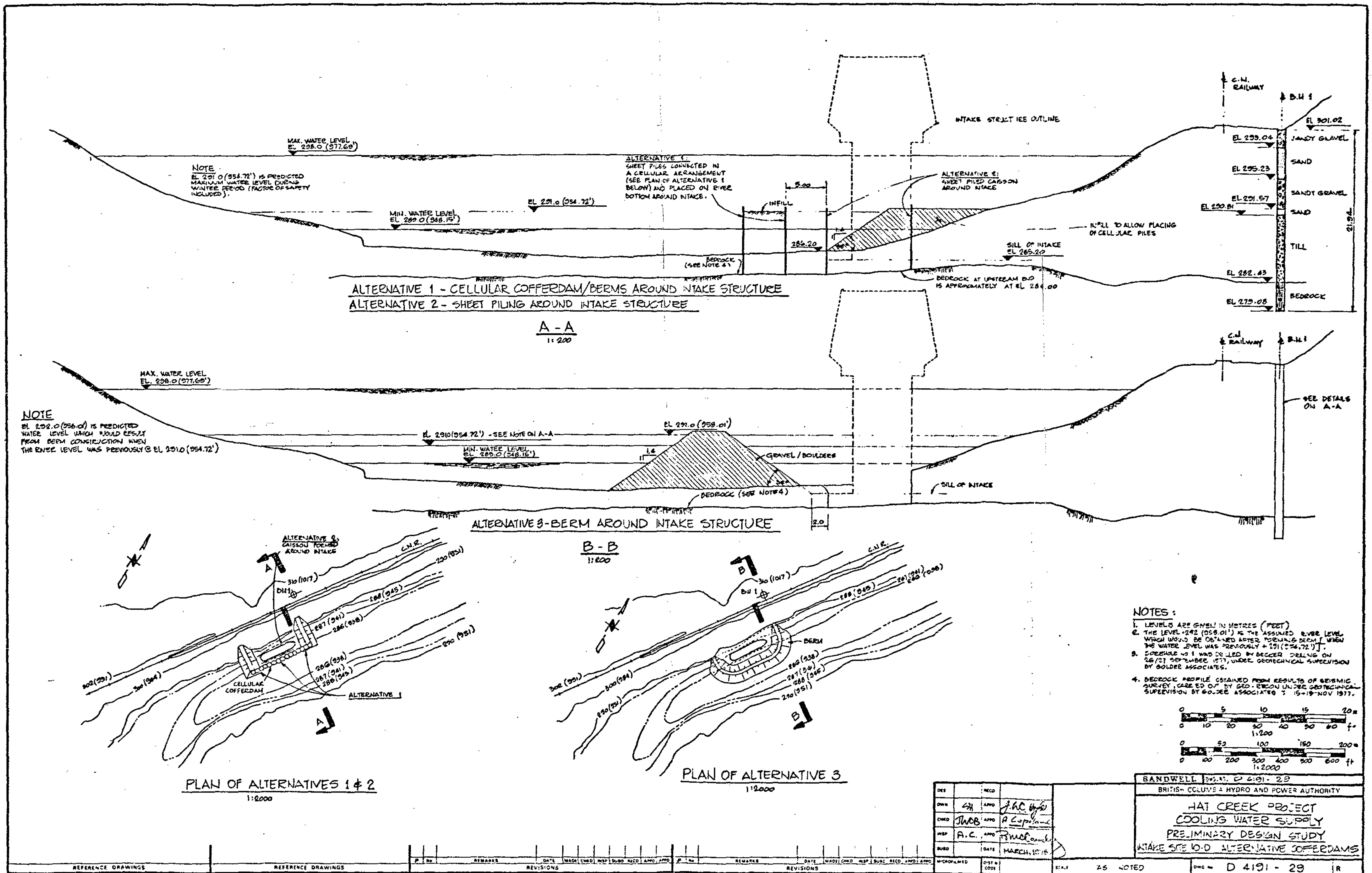
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
LINE FILLING SYSTEM

B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

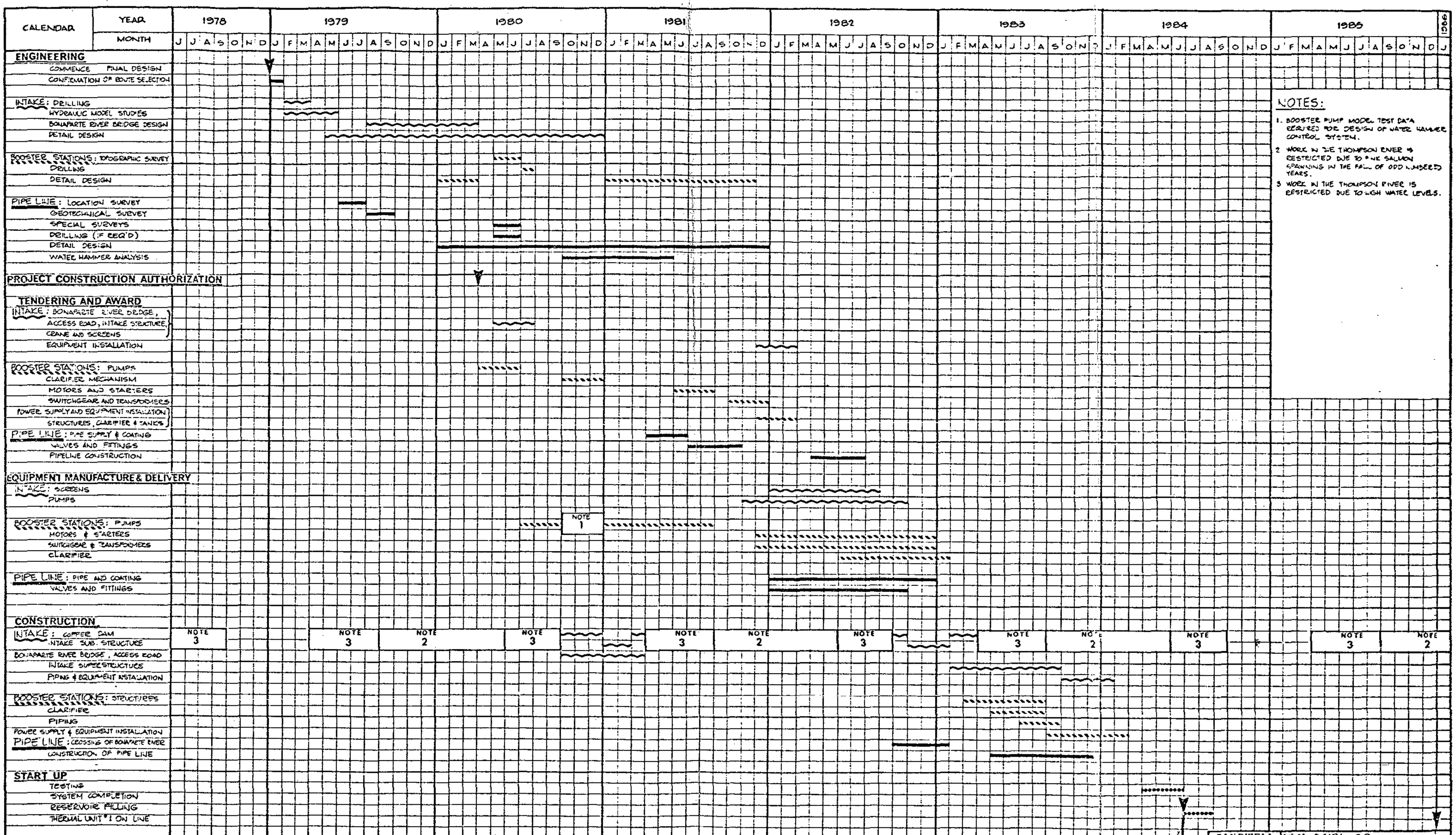
DWG. B 4191 - 27 REV



B.C. HYDRO
B.C.H & P.A. SUPPLY 69KV
LAST POLE
NOTE: NO GROUNDING SWITCH
SANDWELL
INCOMING SUPPLY SUBSTATION
TRIP
20 MVA 69-4.16KV ONAN
TO SUMMATION DATA TERMINAL
4.16KV SWITCHBOARD
SPARE STARTER
SAME AS #4 PUMP
CURRENT LIMITING REACTOR
1.5MVA 4.16-0.6KV ONAN
600V SWBD
600V MCC
450KW CONNECTED
#1 PUMP 3600KW (4800HP)
#2 PUMP 3600KW (4800HP)
#3 PUMP 3600KW (4800HP)
#4 PUMP 3600KW (4800HP)
Nº2 BOOSTER PUMPING STATION
Nº1 BOOSTER PUMPING STATION
LEGEND:
DISCONNECT SWITCH
SURGE DIVERTER
CIRCUIT BREAKER
POWER TRANSFORMER
GROUNDING RESISTOR
AIR CIRCUIT BREAKER (DRAWOUT)
AIR CIRCUIT BREAKER (FIXED)
INCOMING OR OUTGOING CABLE
LOAD BREAK SWITCH
FUSED CONTACTOR STARTER (DRAWOUT)
FUSED SWITCH
REACTOR
INDUCTION MOTOR
POTENTIAL TRANSFORMER
CURRENT TRANSFORMER (PHASE)
CURRENT TRANSFORMER (ZERO SEQUENCE)
INDICATING AMMETER
INDICATING VOLTMETER
KWH INTEGRATING + KW MAX DEMAND METER
PROTECTIVE RELAY
27 UNDERVOLTAGE
46 PHASE BALANCE
49 THERMAL, OVERLOAD
50 INSTANTANEOUS OVERCURRENT
51 TIME CURRENT
63 GAS PRESSURE
-G GROUND FAULT
-N NEUTRAL CURRENT
-T TIME DELAY
4.16KV MCC
SAME AS #5 PUMP
1.5MVA 4.16-0.6KV ONAN
600V MCC
500KW CONNECTED
#1 PUMP 190KW (250HP)
#2 PUMP 190KW (250HP)
#3 PUMP 190KW (250HP)
#4 PUMP 190KW (250HP)
#5 PUMP 190KW (250HP)
INTAKE PUMPING STATION
INTEG-EBASCO
SANDWELL
PIPELINE FEEDER Nº3
SUMMIT
CORNWALL HILL
PIPELINE FEEDER Nº2
ELEPHANT HILL
PIPELINE FEEDER Nº1
4.16-0.6KV ONAN 150KVA TYPICAL FOR 9
METERING & PROTECTION SAME AS Nº1 & Nº2 PIPELINE FEEDERS
4.16KV MCC AT RESERVOIR PUMPING STATION
600V LOAD CENTRE
20KW CONNECTED
10KW CONNECTED
40KW CONNECTED
15KW CONNECTED
40KW CONNECTED
10KW CONNECTED
10KW CONNECTED
10KW CONNECTED
40KW CONNECTED
PIPELINE
SANDWELL DWG No D4191-28
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
SINGLE LINE DIAGRAM
A. Copeland
MARCH, 1978
REFERENCE DRAWINGS
REVISIONS
SCALE
D 4191 - 28



ALTERNATIVE 1 - CELLULAR COFFERDAM/BERMS AROUND INTAKE STRUCTURE
ALTERNATIVE 2 - SHEET PILING AROUND INTAKE STRUCTURE
A - A
1:200
MAX. WATER LEVEL EL. 298.0 (977.69')
NOTE
EL 291.0 (954.72') IS PREDICTED MAXIMUM WATER LEVEL DURING WINTER PERIOD (FACTOR OF SAFETY INCLUDED).
MIN. WATER LEVEL EL 289.0 (948.15')
EL 291.0 (954.72')
ALTERNATIVE 1: SHEET PILES CONNECTED IN A CELLULAR ARRANGEMENT (SEE PLAN OF ALTERNATIVE 1 BELOW) AND PLACED ON RIVER BOTTOM AROUND INTAKE.
INFILL
ALTERNATIVE 2: SHEET PILED CAISSON AROUND INTAKE
INTAKE STRUCTURE OUTLINE
INFILL TO ALLOW PLACING OF CELLULAR PILES
SILL OF INTAKE EL 285.20
BEDROCK (SEE NOTE 4)
BEDROCK AT UPSTREAM END IS APPROXIMATELY AT EL 284.00
C.N. RAILWAY
B.H. 1
EL 301.02
EL 299.04
SANDY GRAVEL
SAND
EL 295.23
SANDY GRAVEL
EL 291.57
EL 290.81
SAND
TILL
EL 282.43
BEDROCK
EL 279.08
21.94
ALTERNATIVE 3 - BERM AROUND INTAKE STRUCTURE
B - B
1:200
MAX. WATER LEVEL EL. 298.0 (977.69')
NOTE
EL 292.0 (958.01') IS PREDICTED WATER LEVEL WHICH WOULD RESULT FROM BERM CONSTRUCTION WHEN THE RIVER LEVEL WAS PREVIOUSLY @ EL 291.0 (954.72')
EL 291.0 (954.72') - SEE NOTE ON A-A
MIN. WATER LEVEL EL 289.0 (948.15')
EL 292.0 (958.01')
GRAVEL / BOULDERS
BEDROCK (SEE NOTE 4)
SILL OF INTAKE
SEE DETAILS ON A-A
C.N. Railway
B.H. 1
PLAN OF ALTERNATIVES 1 & 2
1:2000
ALTERNATIVE 2: CAISSON FORMED AROUND INTAKE
CELLULAR COFFERDAM
ALTERNATIVE 1
C.N.R.
BH 1
PLAN OF ALTERNATIVE 3
1:2000
BERM
NOTES:
1. LEVELS ARE GIVEN IN METRES (FEET)
2. THE LEVEL +292 (958.01') IS THE ASSUMED RIVER LEVEL WHICH WOULD BE OBTAINED AFTER FORMING BERM WHEN THE WATER LEVEL WAS PREVIOUSLY + 291 (954.72').
3. BOREHOLE NO 1 WAS DRILLED BY BECKER DRILLING ON 26/27 SEPTEMBER 1977, UNDER GEOTECHNICAL SUPERVISION BY GOLDER ASSOCIATES.
4. BEDROCK PROFILE OBTAINED FROM RESULTS OF SEISMIC SURVEY, CARRIED OUT BY GEO-RECON UNDER GEOTECHNICAL SUPERVISION BY GOLDER ASSOCIATES 15-19 NOV 1977.
1:200
1:2000
SANDWELL
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE SITE 10-D ALTERNATIVE COFFERDAMS
MARCH, 1978
AS NOTED
D 4191 - 29
REFERENCE DRAWINGS
REVISIONS



REFERENCE DRAWINGS | REVISIONS | REMARKS | DATE | MADE | CHKD | WSP | SUBD | RECD | APPD | APPD

DES		RECD	
DWN	SH	APPD	
CHKD	[illegible]	APPD	A Copeland
WSP	A.C.	APPD	Rucknacker
SUBD		DATE	MARCH 1978
MICROFILMED		DIST'N CODE	

SANDWELL DWG. NO. D4191 - 30

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
ENGINEERING & CONSTRUCTION SCHEDULE

SCALE —

DWG No D4191 – 30 R

APPENDIX 7

PHOTOGRAPHS

Photo No: 13-4A
Sta. 0+300 to 2+000

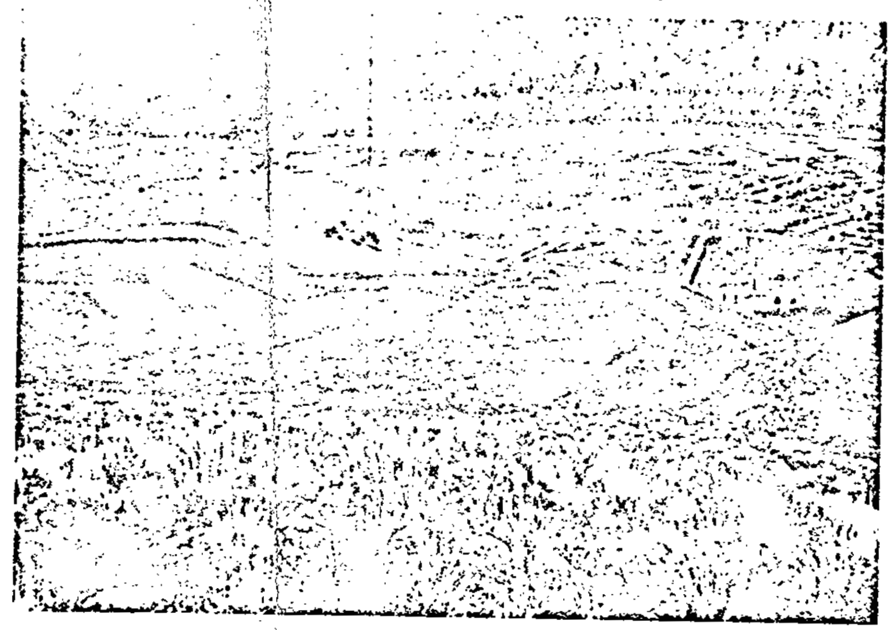


Photo No. 13-8A
From Sta. Elephant Hill Summit

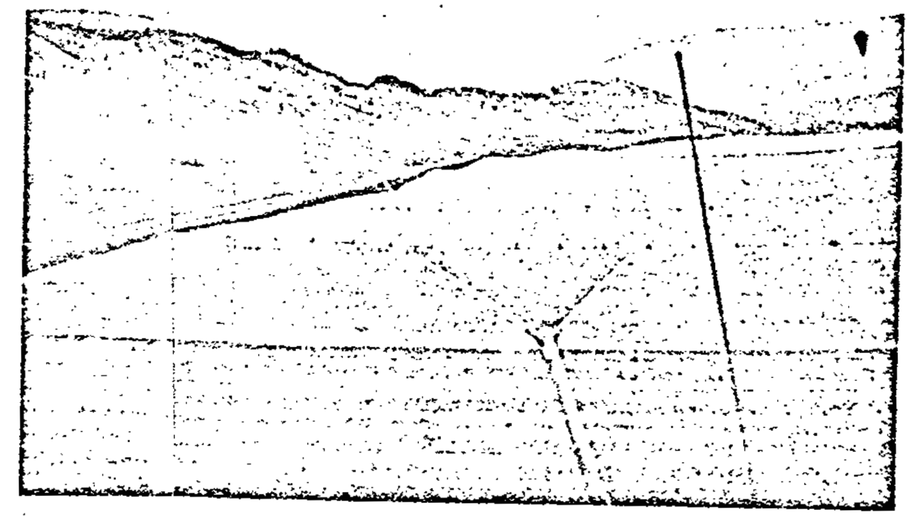


Photo No. 13-11A
Sta. 4+000 to 7+000

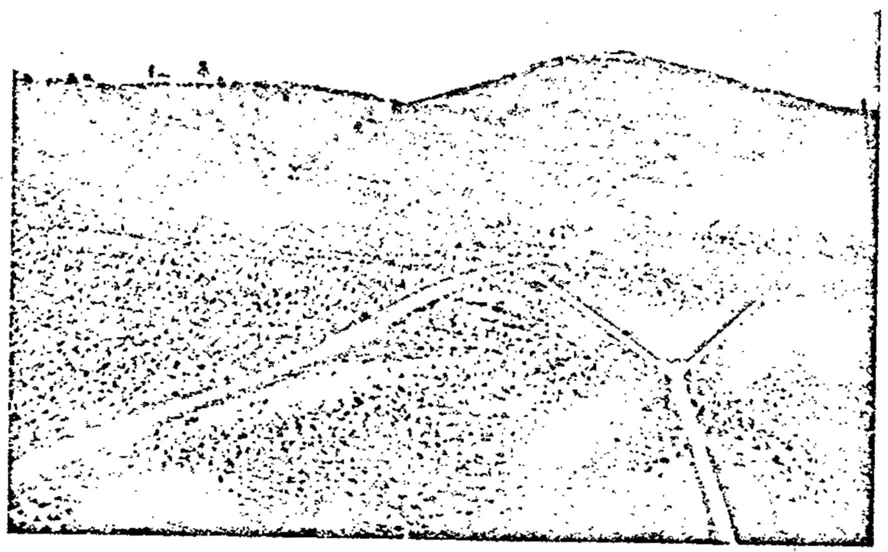


Photo No. 13-13A
Sta. 7+500

Photo No. 13-17A
Sta. 8+500

Photo No. 13-22A
Sta. 11+000

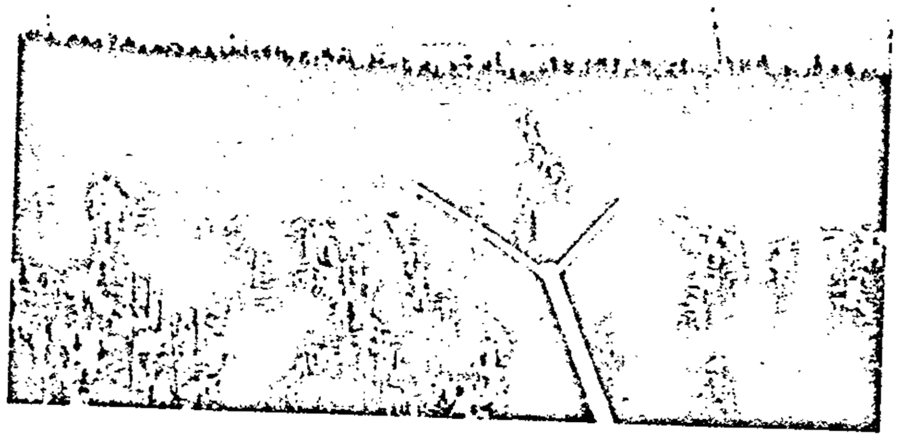


Photo No. 13-23A
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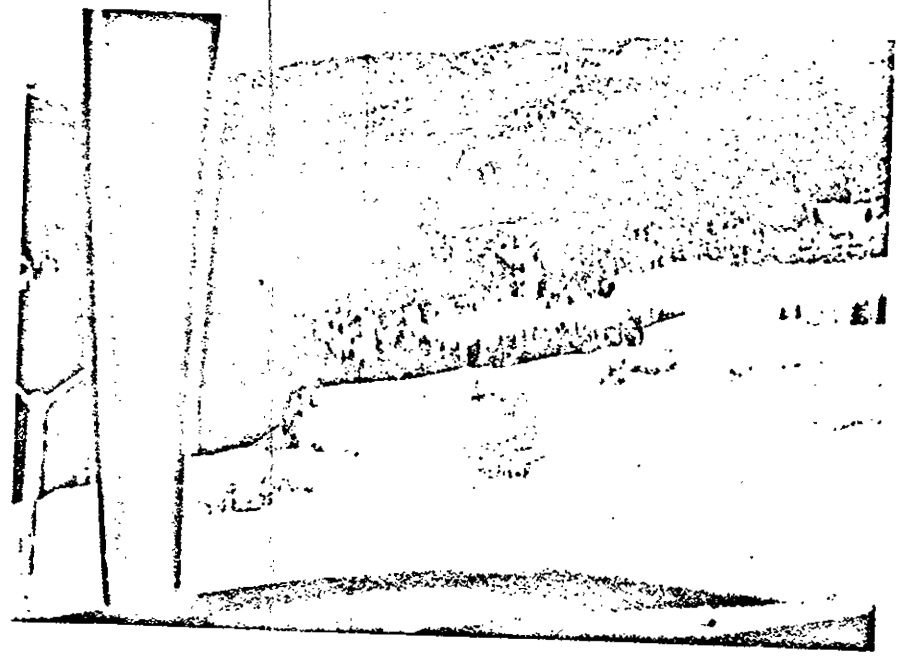

Photo No. 13-25A
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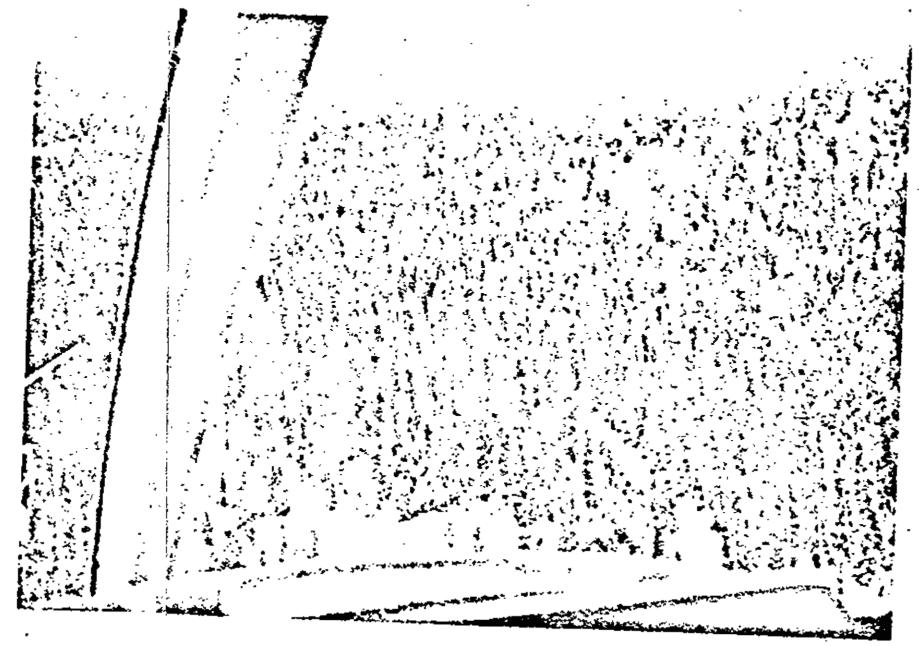

Photo No. 13-28A
Sta. 15+000

Photo No. 13-33A
Sta. 22+000

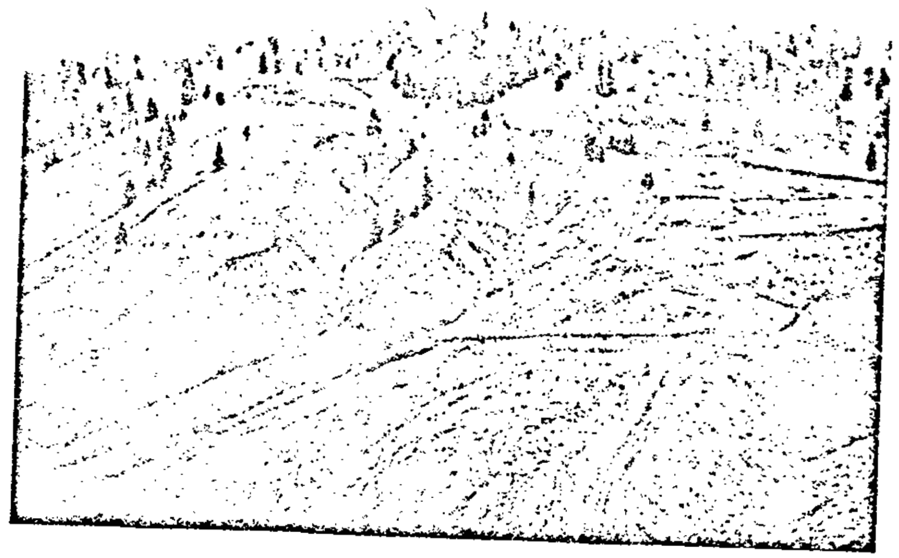

Photo No. 14-37
Afton Mines Water Supply Pipeline

(V4191, App. 7.)