BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

Sandwell and Company Ltd. - Hat Creek Project - Cooling Water Supply Preliminary Design Study - Volume 2 - March 1978.

ENVIRONMENTAL IMPACT STATEMENT REFERENCE NUMBER: 8b

REPORT V4191/1
MARCH 1978

B.C. HYDRO AND POWER AUTHORITY

VANCOUVER B.C.

HAT CREEK PROJECT

COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

VOLUME TWO OF THREE

SANDWELL

REPORT V4191/1
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PRELIMINARY DESIGN STUDY

DATE MARCH 1978

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PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 2 AUGUST 1977

REVISION 1 18 JANUARY 1978

PROJECT MEMORANDUM V4191/1
DESIGN CRITERIA

INTRODUCTION

This memorandum records the criteria used in the Preliminary Design Study, for the Hat Creek Cooling Water Supply System.

The water supply system is to provide cooling water make-up from the Thompson River to the Hat Creek Thermal Generating station in the Trachyte Hills west of Ashcroft, B.C. The station is to produce 2000 MW of power by burning coal from the deposiits of the Hat Creek Valley nearby.

In the design of the cooling water supply system, water treatment shall only be to the extent necessary to protect the pumping equipment, and no consideration shall be given to the water treatment requirements of the power plant.

BASIC REQUIREMENTS

The basic requirements of the water supply system are outlined in Table 1.

Table 1 - System Requirements

Item	Amount			
	S.I. Units		Imperial Units	
Maximum Discharge	1577	l/s	20,815	Igpm
Average Discharge	663	l/s	8,750	Igpm
Maximum Static Lift	1082.8	m	3,558	ft.
Normal Full Reservoir Elevation	1372	m	4,500	ft.
Minimum Reservoir Elevation	1357	m	4,450	ft.

METEOROLOGICAL CONDITIONS

The water supply line encounters a range of climatic conditions, see Table 2.

Table 2 - Meteorological Conditions

Item	Unit	Amount: Ashcroft	Amount: Hat Creek Station
Elevation	m	300	899
	ft	1000	2950
Mean daily temperature	°C	7.4	3.2
Extreme maximum temperature	°C	38.9	34.4
Extreme minimum temperature	°C	-38.3	-42.8
Mean annual precipitation	mm	235	317
15 minute rainfall (25 yr. return frequency)	mm	14* See [illegible]	14*
1 hour rainfall	mm	25*	25*
Greatest 24 hour snowfall	mm	318**	424
Maximum hourly wind speed	m/s	22.4** see NBC	
Probable maximum gust	m/s	31.3** See NBC	
Prevailing wind direction		SW**	

Note: Temperature and precipitation data taken from "Temperature and Precipitation 1941-70 for B.C.", prepared by Atmospheric Environment Service, Environment Canada, for Ashcroft and Hat Creek Stations.

RIVER WATER

The discharge and quality characteristics of the Thompson River near the intake location are described in Table 3.

* Values extrapolated from data obtained from "Rainfall Intensity - Duration - Frequency Maps for B.C.", D.O.T. Meteorological Branch report by W.A. Murray, April 1964.

** For Ashcroft Manor weather station, courtesy Environment Canada.

Table 3 - River Characteristics

Item	Amount			
	SI Units		Imperial Units	
Mean annual discharge*	790	m^3/s	27,900	cfs
Maximum recorded discharge*	4140	m^3/s	146,000	cfs
Minimum recorded discharge*	125	m^3/s	4,410	cfs
Design flood return period	100	years		
Design flood	4534	m^3/s	160,000	cfs
Design low flow return period	100	years		
Design low flow	113	m^3/s	4,000	cfs
Mean hardness as $CaCO_3$	35	mg/l	35	ppm
Maximum hardness	42	mg/l	42	ppm
Water temperature - maximum	19.5	°C	67	°F
- minimum	0	°C	32	°F

INTAKE

Criteria for intake design are given in Table 4.

Table 4 - Intake Design

Item	Amount	
	SI Units	Imperial Unit
Approach velocity for gross screen area	0.12 m/s	0.4 ft/s
Low water level	289.2 m	948.8 ft
High water level	298.0 m	977.7 ft
Maximum screen opening size	2.5 mm	0.1 in

Allowance for pump wear and process losses: 10% of maximum discharge.

BOOSTER STATIONS

Booster stations are to be designed according to the criteria of Tables 5 and 6.

Table 5 - Booster Station Design

Allowance for pump wear: 0 %
Permissible noise levels: According to Workers Comepnsation Board - Accident Prevention Regulations.

* Thompson River near Spences Bridge - Station No. 08LF051. Taken from Historical Streamflow Summary, British Columbia to 1973, Inland Water Directorate Water Resources Branch, Ottawa, 1974.

Table 6 - Motor and Power Supply Requirements

Item	Unit	Amount		
		Intake	No. 1 Booster Station	No. 2 Booster Station
Transmission voltage		-	69,000	69,000

STRUCTURES

Structures are to be designed according to the National Building Code of Canada, ~~1975~~ edition.

PIPELINE

Waterhammer Design Criteria*

The design of the pipeline, pumps, valves and appurtenances subjected to effects of water in the pipeline shall be in accordance with the following conditions:

1. Normal Conditions of Pump Operation

   a. Manual or automatic starting and stopping of pumps.

   b. Shutoff head develops upstream of any shutoff valves on pump manifold or on pipeline.

   c. Power failure at all pump motors and valves.

   d. Control devices, such as surge tanks, air chambers, check valves, surge suppressors, and pressure control devices function as designed.

   e. Check valves close immediately upon flow reversal.

   f. Air chambers are at minimum air volume which would start a compressor.

   During these conditions, water column separation shall not occur. Collapse of the pipe shall be prevented at minimum head.

2. Emergency Conditions of Pump Operation

   Some malfunction of the control equipment occurs as follows:

   a. Either of surge suppressor or pressure relief valve does not operate.

   b. One check valve closes at time of maximum reverse flow.

   c. Air inlet valves inoperative.

   d. Air chamber at emergency low air level which would automatically trip out the pumps.

   e. A two speed control valve closes at the faster rate.

* "Waterhammer Design Criteria", by John Parmakian, Proc. ASCE, PO2, April 1957.

3. Exceptional Conditions of Pump Operation

The most unfavourable malfunctioning of equipment is assumed to determine the worst possible result.

Waterhammer Design Criteria for the pipeline are contained in Table 7.

Table 7 - Pipeline Design

Item	Unit	Amount
Impact - energy	J	54
absorved at -4°C	ft-lbs	40
in Charpy Test (1)		
Allowable stresses		
Normal Condition	MPa(psi)	177 (25,600)
Emergency Condition	MPa(psi)	354 (51,200)
Exceptional Condition	- -	Not Used
Hydraulic roughness E		
(coal tar epoxy lined)		
maximum	mm	0.06
minimum	mm	0.015

(1) The allowable stress is based on requirements for a minimum factor of safety on ultimate bursting strength of the member, and limits of stress relative to the yield strength of the material as follows:

Condition	Minimum* Factor of Safety	Maximum* Proportion of Yield Stress
Normal	3	0.8
Emergency	1.5	1.0
Exceptional	- -	- -

The values relate to CSA Z245.2, Grade 60 pipe.

* "New Criteria for USBR Penstocks" by Arthur & Walker, Proc. ASCE, January 1970.

FINANCIAL AND COST CRITERIA

Criteria for cost analyses are shown in Table 8. The base period for cost estimates was Fourth Quarter, 1977.

Table 8 - Financial and Cost Criteria

Item	Unit	Amount
Cost of power	mils/kwh	20
Long-term interest rate	%	8
Short-term interest rate	%	8
Federal Sales Tax	%	12
Provincial Sales Tax	%	7
Land Cost	$/hectare	2,500
Corporate overhead	%	5
Amortization period	year	35

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Approved by
A. Copeland, P. Eng.

B.R. McConachy
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Revision 1: General

PROJECT MEMORANDUM V4191/2

WATER INTAKE SITE 16

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO & POWER AUTHORITY
VANCOUVER B.C.

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DATE 27 JUNE 1977

INTRODUCTION

This Project Memorandum presents a feasibility study for an intake at site 16* on the Thompson River, to provide 1,577 l/sec. (25,000 USGPM) of cooling water makeup to a 2,000 MW Thermal Power Station, fired by coal from deposits of the Hat Creek Valley, being considered by B.C. Hydro and Power Authority.

As can be seen from drawing A4191/2-1 (Appendix 3), there are two alternative corridors from the Thompson River to the Hat Creek Power Plant Site, through which the cooling water supply pipeline could be routed. These corridors are for the access road and the 500 kV transmission line. The objective of this Project Memorandum is to consider the feasibility of site 16 for an intake to serve a pipeline along the road corridor. Various photographs of site 16 are included in Appendix 2.

The main section of the Memorandum presents potential intake locations and intake types at site 16, together with the geophysical, geotechnical, and fluvial characteristics of the area.

Because site 10 was previously selected as the most feasible location for an intake on the Thompson River (Reference 1),** a comparison between sites 16 and 10 has been included.

SITE 16

Intake Types and Locations

The portion of the Thompson River identified as site 16 is the bend located between 105 Mile Post Indian Reserve 2 and Cheetsum's Farm Indian Reserve 1. (Drawing A4191/2-2, Appendix 3).

With reference to Sandwell's Memorandum V4007/1 (Reference 2) on Water Intake Design, two intake types were considered at site 16 i.e. the bank and pier intake. The bypass intake has not been considered as the topography of site 16 does not lend itself to such a design. Drawing A4191/2-2 (Appendix 3) illustrates potential intake locations, and the characteristics of each are given in Table 1.

* For this and other locations see drawing A4191/2-1 contained in Appendix 3.
** For this and other references see Appendix 1

Table 1 - Characteristics of Various Intake Locations

Intake Location	Type	Adjacent To Relatively Deep Water	Relationship to Cornwall Creek	River Crossing Required	Comments
A	Bank	N	U	Y	
B	Bank	Y	D	N	
C	Pier	Y	D	Y	Additional hazards due to river construction work.
D	Bank	N	D	N	
E	Bank	Y	D	Y	

Note:

All the above sites are influenced by effects from rapids and potential slides.

Abbreviations:

Y: Yes, N: No, U: Upstream, D: Downstream

Geophysical Considerations

For bank intakes located along the right bank of the Thompson River in the area under study, the relative steepness of the topography would make access a major concern. This would be further complicated by extensive restrictions which would be imposed by the Canadian National Railway regarding protection, maintenance and operation of their track during the construction phase. The gradient of the terrain is such that road access, both temporary and permanent, would require a route with steep inclines. (Photographs 2 & 3, Appendix 2, illustrate these points.)

Pier intakes in the area of site 16 would possess similar engineering drawbacks to those mentioned above together with additional construction difficulties associated with off-shore river installations.

While intakes located at positions A and E at the left bank of the Thompson River (Drawing A4191/2-2, Appendix 3) should not pose any significant access difficulties along the River bank and/or the Canadian Pacific Railway property, their main drawback would be the requirement of a river crossing.

Geotechnical Considerations

The geological background of the Thompson River Valley around Ashcroft was given in Sandwell's Interim Report V4007/1 (Reference 3). The following statement is a quote from this report:

"The geology of the Thompson Valley in the reach of interest has strong implications on the selection of an intake location. Jurassic rocks of conglomerates and arkosic sandstones interbedded with black shales outcrop along the river between Ashcroft and Basque. The shales are disturbed and crushed. The surficial soils are dominated by the glacio-lacustrine silt deposits, up to 300 feet thick, from Ashcroft to south of Spences Bridge".

If intake 16 were located on the right bank of the river, it could likely be founded on bedrock close to the bank. However, as the CN railway is relatively close to the edge of the right bank of the Thompson River, the bank would have to be extended locally into the river to house the intake pumphouse and ancillary works.

The ground above the site is a steep bedrock slope that resulted from erosion caused by the river. While the surface stability of the rock slope is probably only marginal, there is no visible evidence of major deep seated rock instability in the concave bend of the river. The rock is a sedimentary type, dipping into the hillside.

The major geotechnical concern with site 16 would be the instability of the previously identified glacio-lacustrine silt deposits downstream and upstream of the area, particularly on the left bank of the river, (Photograph 1, Appendix 2). Two very large slides in the silt deposits, No. 1 and No. 5, exist about half a mile downstream and upstream of site 16. Downstream slide No. 1 is currently active and is a continuing problem to CPR. Attempts at stabilization have so far been rather unsuccessful. In addition a smaller slide in the surficial deposits exists on the left bank of the river immediately downstream where Cornwall Creek joins the Thompson River (No. 3). This slide was active in 1969.

Taking the above points into consideration, it is clear that a major reactivation of these slides, for whatever reason, could result in an intake at site 16 being completely inoperative, particularly if the slide occurred upstream and the subsequent washout buried the mouth of the intake with sediment.

Fluvial Considerations

Examination of Photograph 1 (Appendix 2) indicates the following significant characteristics of the Thompson River in the vicinity of site 16.

1. The average width of the river upstream and downstream of the bend is 100 - 125 m (330-410 ft), whereas the average width within the bend is about 200 m (660 ft). Consequently, the wider reach within the bend is expected to be shallower than adjacent reaches.

2. Examination of Photograph 1, taken at relatively low water conditions when the discharge at Spences Bridge was 243 m^3/s (or 8570 cfs,*) indicates numerous rapids, the most severe of which are at the entrance to the bend. In addition, a large rock outcrop is visible on the left bank opposite Cornwall Creek. As such, this reach of river would be considerably steeper than adjacent reaches. The result of this is that substantially higher

*) Maximum recorded = 4,134 m^3/s or 146,000 cfs:
Minimum recorded = 116 m^3/s or 4,100 cfs.

velocities and turbulence are anticipated, especially at higher flows. Consequently, a river intake would have to be designed to protect the system from damage from large particles (probably up to gravel or small cobble size) which could be thrown into suspension at high flows.

In addition, it may be very difficult to obtain uniform velocities through the screen for a direct intake, both because of suspected occurrence of very strong large-scale turbulence and because of possible large-amplitude water-level surging which often is associated with rapids.

Further examination of the aerial photograph indicates a poorly defined low-water channel. Quite different than what is normally expected, it appears that the low-water channel near the site is on the inside (left bank, facing downstream) of the bend, extending from the mouth of Cornwall Creek to the downstream end of the left bank point bar. It is thus surmised that considerable surface bedrock underlies the channel adjacent to the site. If so, bedrock outcrops may have to be excavated to ensure that a low-water channel of adequate depth is located adjacent to any specific potential intake site.

In addition to the above, results from a sampling analysis program have indicated that the suspended sediment concentration increases downstream of the confluence with the Bonaparte River (Appendix 1, Reference 4, Page 4).

The existence of rapids at site 16 produces an increased chance of frazil ice conditions during winter.

All the above comments are in agreement with the classification of the hydraulic suitability "of site 16 as being poor" as detailed in Table 3 page 8 of Sandwell's Interim Report V4007/1.

COMPARISON OF SITE 16 WITH SITE 10

As site 10 was previously selected as the most feasible location for the water intake (Reference 1), a comparison between sites 10 and 16 follows to determine the relative merits of these sites.

Site 10

Advantages

1. Based on river surveys carried out in January and March 1977 (Reference 5) the area around site 10 gives ample choice of hydraulically sound intakes.

2. Based on visual observations, the area is considered geotechnically sound.

3. An intake selected upstream of the confluence with the Bonaparte River would avoid taking in water originating from this river which is higher in turbidity than that of the Thompson River.

4. Generally, both access and room for construction are good.

5. The site is in the vicinity of a town, namely Ashcroft.

Disadvantages

1. Compared with site 16, site 10 is further away from the access road corridor and therefore requires a 2 mile longer pipeline.

Site 16

Advantages

1. Convenient location for routing the pipeline along the access road corridor.

2. Nearer to the power plant site at Harry Lake than site 10, and therefore requiring a shorter pipeline length.

3. Based on visual observations, the right bank of the river at the site is considered geotechnically sound.

Disadvantages

1. Based on visual observations, the site is considered hydraulically unsuitable.

2. Site has poor accessibility.

3. There is insufficient room between the CNR track and the existing shore profile to construct the intake pumping station.

4. The site is further away from Ashcroft than site 10.

5. There is a greater possibility of the intake becoming blocked by slide deposits than for site 10.

6. There is a greater possibility of the intake being affected by frazil ice due to the rapids in this area.

CONCLUSIONS

On the basis of intake types and locations, geophysical, geotechnical and fluvial considerations, it is found that:

1. None of the intakes listed in Table 1, page 2 provides an acceptable combination of location characteristics.

2. Access would be very costly, because of the relative steepness and the limited space along the right bank.

3. The operability of an intake would be threatened by the danger of slides in the silt deposits.

4. The river reach is hydraulically unsuitable, because the deep water channel is along the left bank of the river and the presence of rapids contribute to suspended solids and the formation of frazil ice.

Therefore, it is concluded that site 16 does not offer any intake location that could be considered viable for the Hat Creek Project.

Of the two intake locations discussed in this memorandum, sites 10 and 16, each has certain advantages over the other. However, since site 16 is on most counts inferior to site 10, the recommended choice is overwhelmingly in favour of site 10 as being the most feasible location for a river intake to supply the proposed thermal power station at Harry Lake.

Prepared By John W.C. Boyle
J.W.C. Boyle

A. Copeland, P. Eng.

Approved By
B.R. McConachy, Project Engineer

APPENDIX 1

REFERENCES

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO & POWER AUTHORITY
VANCOUVER B.C.

PROJECT MEMORANDUM V4191/2
WATER INTAKE SITE 16

DATE 27 JUNE 1977

APPENDIX 1 - REFERENCES

1. Letter from B.C. Hydro and Power Authority to Sandwell and Company Limited dated 2 September 1976.

2. Sandwell and Company Limited "Project Memorandum V4007/1 - Hat Creek Project - Water Supply Study - Water Intake Design". Memorandum to B.C. Hydro and Power Authority, January 1977.

3. Sandwell and Company Limited "Interim Report V4007/1 - Hat Creek Project - Water Supply Study". Report to B.C. Hydro and Power Authority, October 1976.

4. Northwest Hydraulic Consultants Limited, "Hat Creek Project - Water Supply Hydrology (Interim Report)", November 1976.

5. Northwest Hydraulic Consultants Limited, "Hat Creek Water Supply Study Site 4A and Site 10 - January 1977" and "Hat Creek Water Supply Study - CN Bridge to Ashcroft - March 1977".

APPENDIX 2

PHOTOGRAPHS OF WATER INTAKE SITE 16

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

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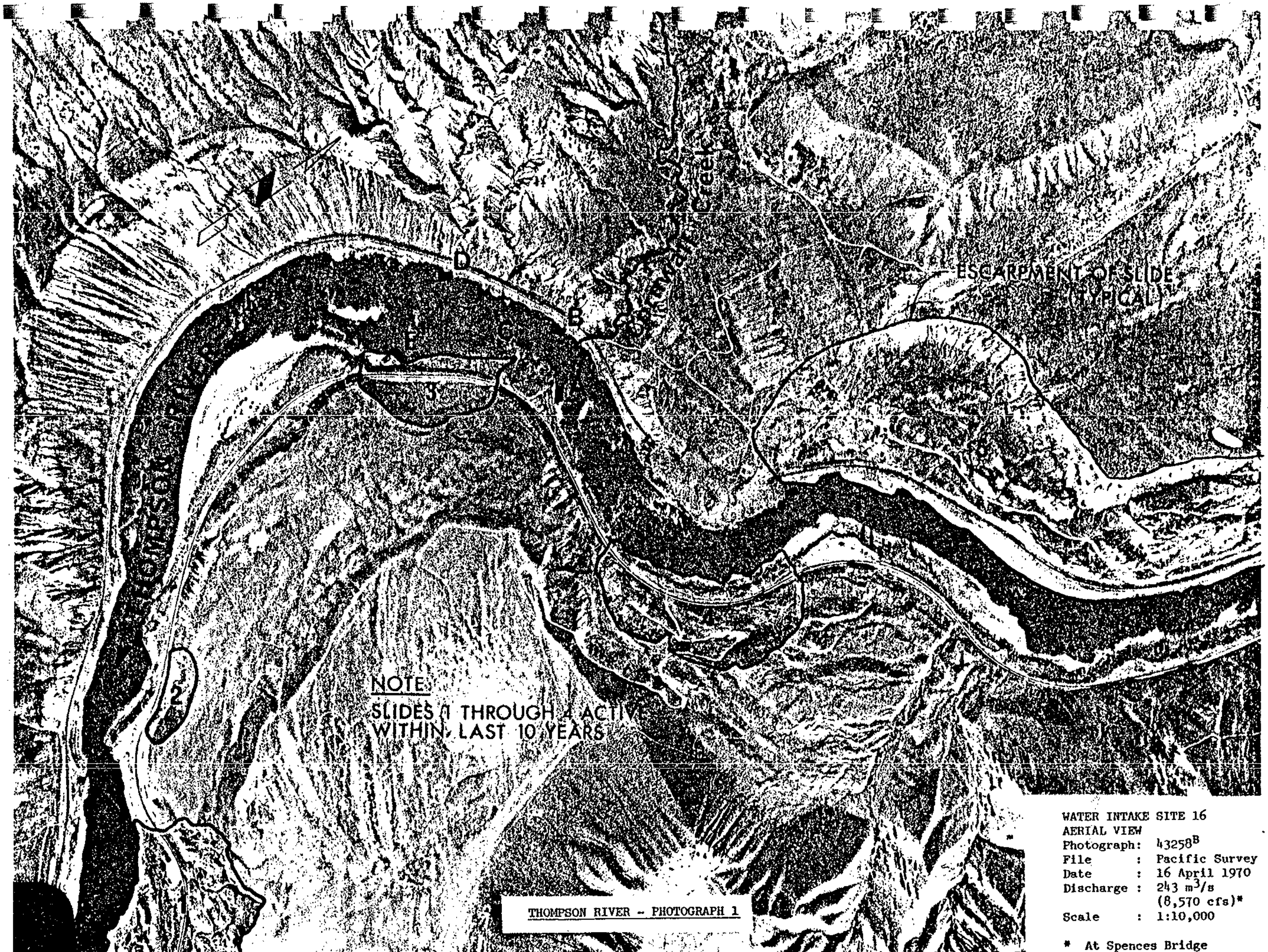
WATER INTAKE SITE 16

APPENDIX 2 - PHOTOGRAPHS OF WATER INTAKE SITE 16

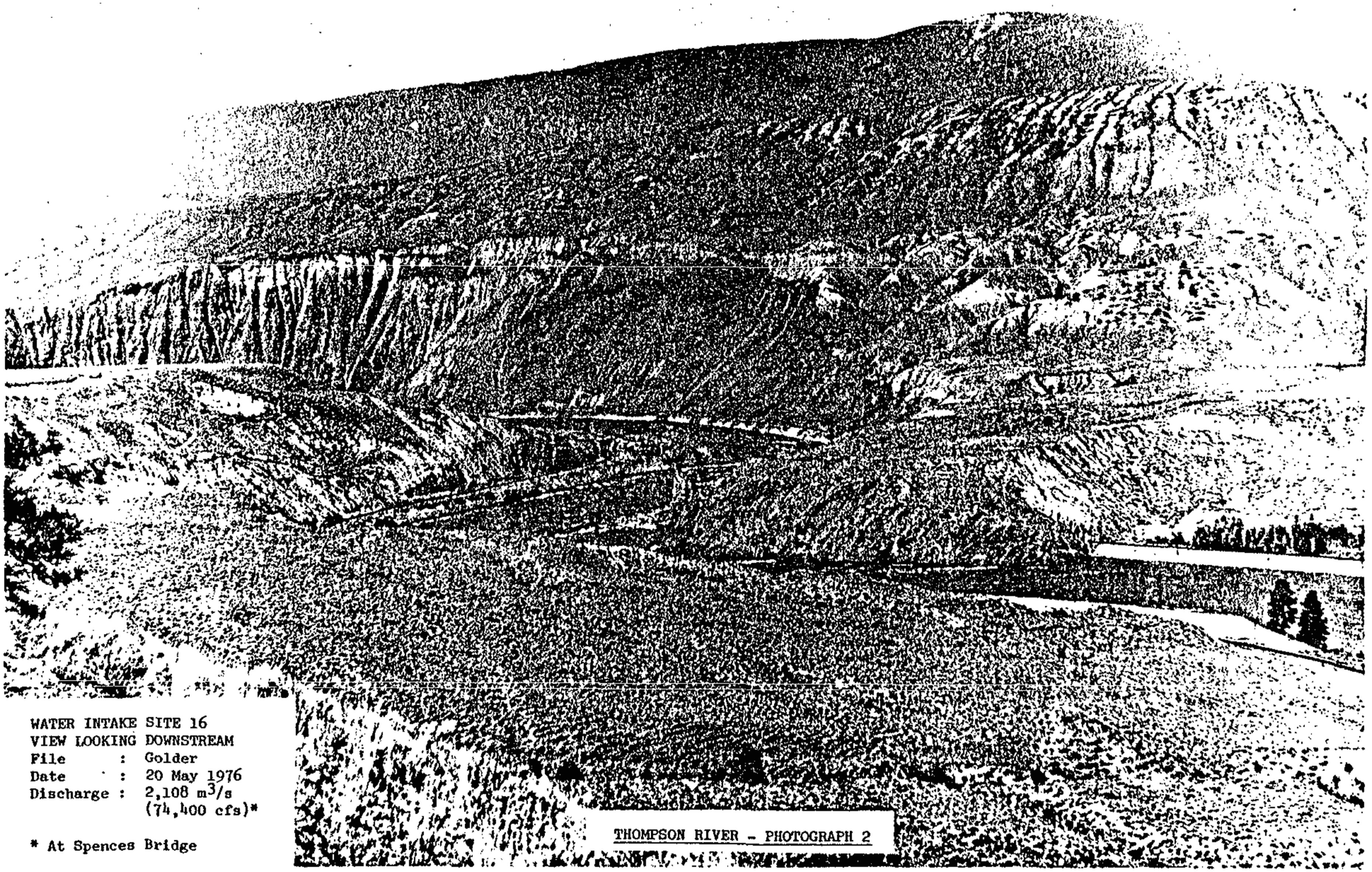
Photograph	File	Description
43258B	Pacific Survey	Aerial view of Site 16 with potential slide zones indicated on overlay.
-	Golder	View of Site 16 looking downstream.
2 - 15	Sandwell	View of Site 16 looking downstream.



THOMPSON RIVER - PHOTOGRAPH 1

WATER INTAKE SITE 16
AERIAL VIEW
Photograph: 43258^B
File : Pacific Survey
Date : 16 April 1970
Discharge : 243 m^3/s
(8,570 cfs)*
Scale : 1:10,000

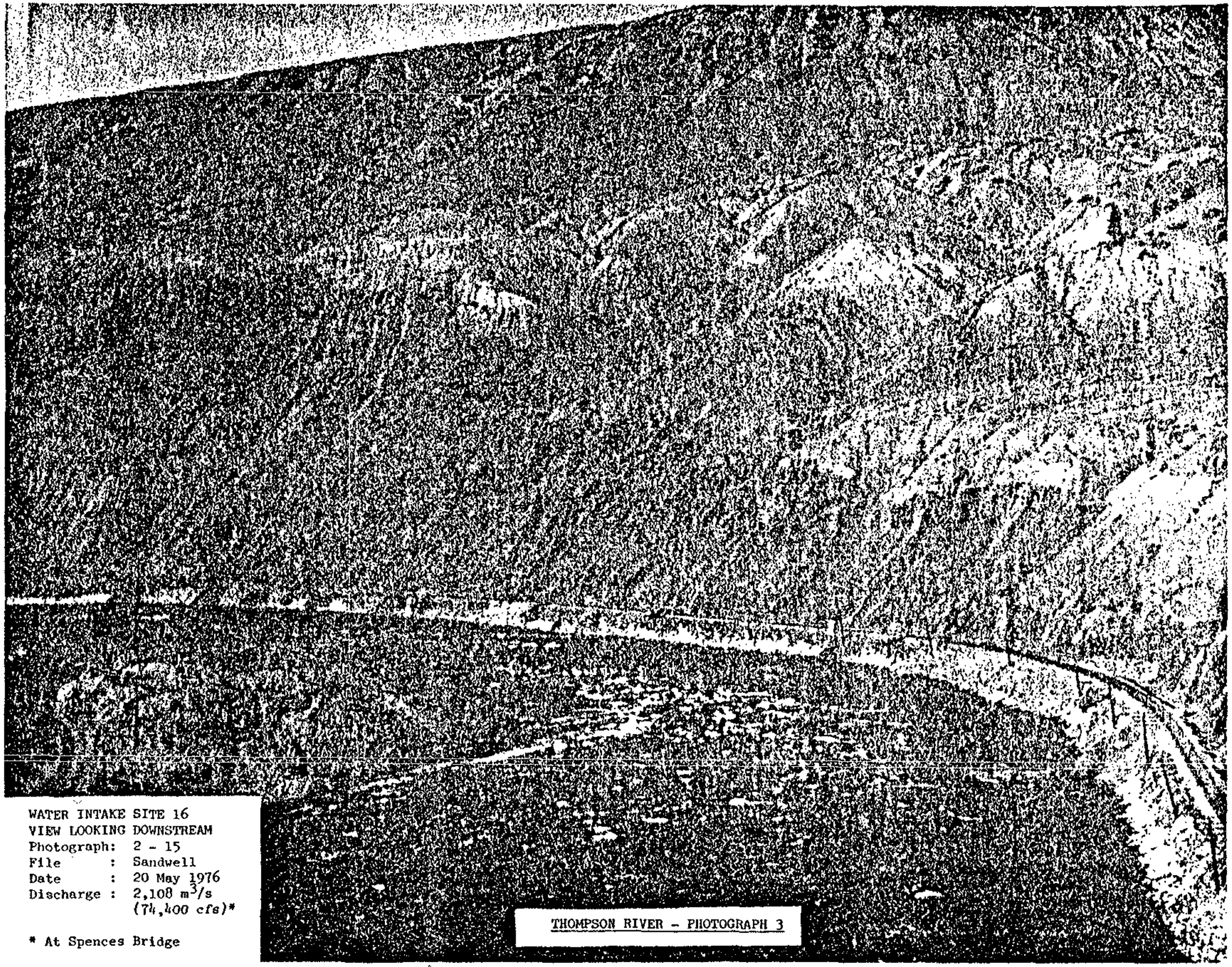
* At Spences Bridge



WATER INTAKE SITE 16
VIEW LOOKING DOWNSTREAM
File : Golder
Date : 20 May 1976
Discharge : 2,108 m^3/s
(74,400 cfs)*

* At Spences Bridge

THOMPSON RIVER - PHOTOGRAPH 2



WATER INTAKE SITE 16
VIEW LOOKING DOWNSTREAM
Photograph: 2 - 15
File : Sandwell
Date : 20 May 1976
Discharge : 2,108 m^3/s (74,400 cfs)*

* At Spences Bridge

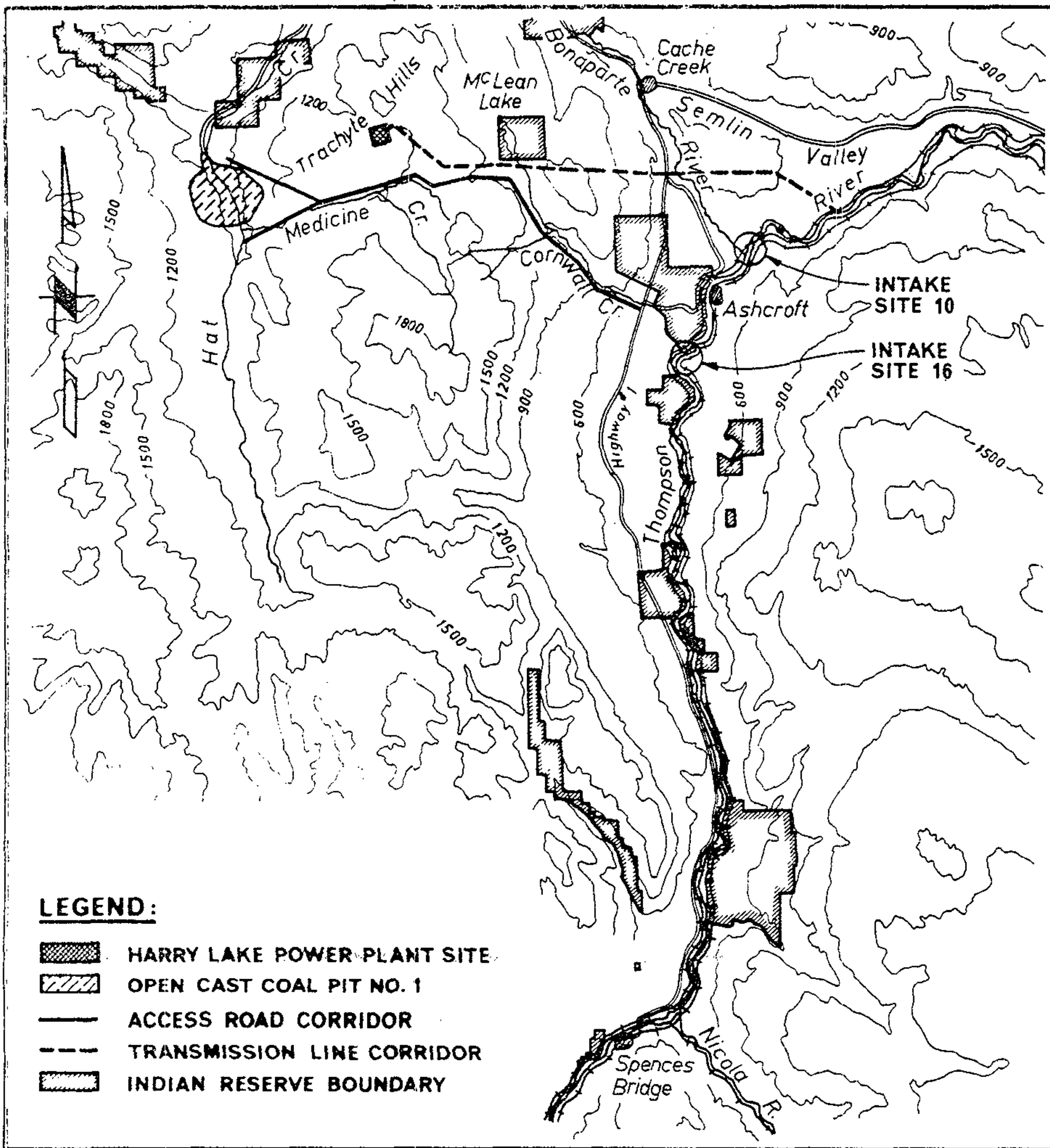
THOMPSON RIVER - PHOTOGRAPH 3

SANDWELL

APPENDIX 3

ILLUSTRATIONS

(PM V4191/2)



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APPROVED

DATE

		DAY	MO.	YR.
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DR'N.	SH	2	6	77
CH'K'D.	J.W.G.B.			
APP'D.				
APP'D.				

A 4191/2-1

HAT CREEK PROJECT

COOLING WATER SUPPLY

WATER INTAKE SITE 16

KEY PLAN

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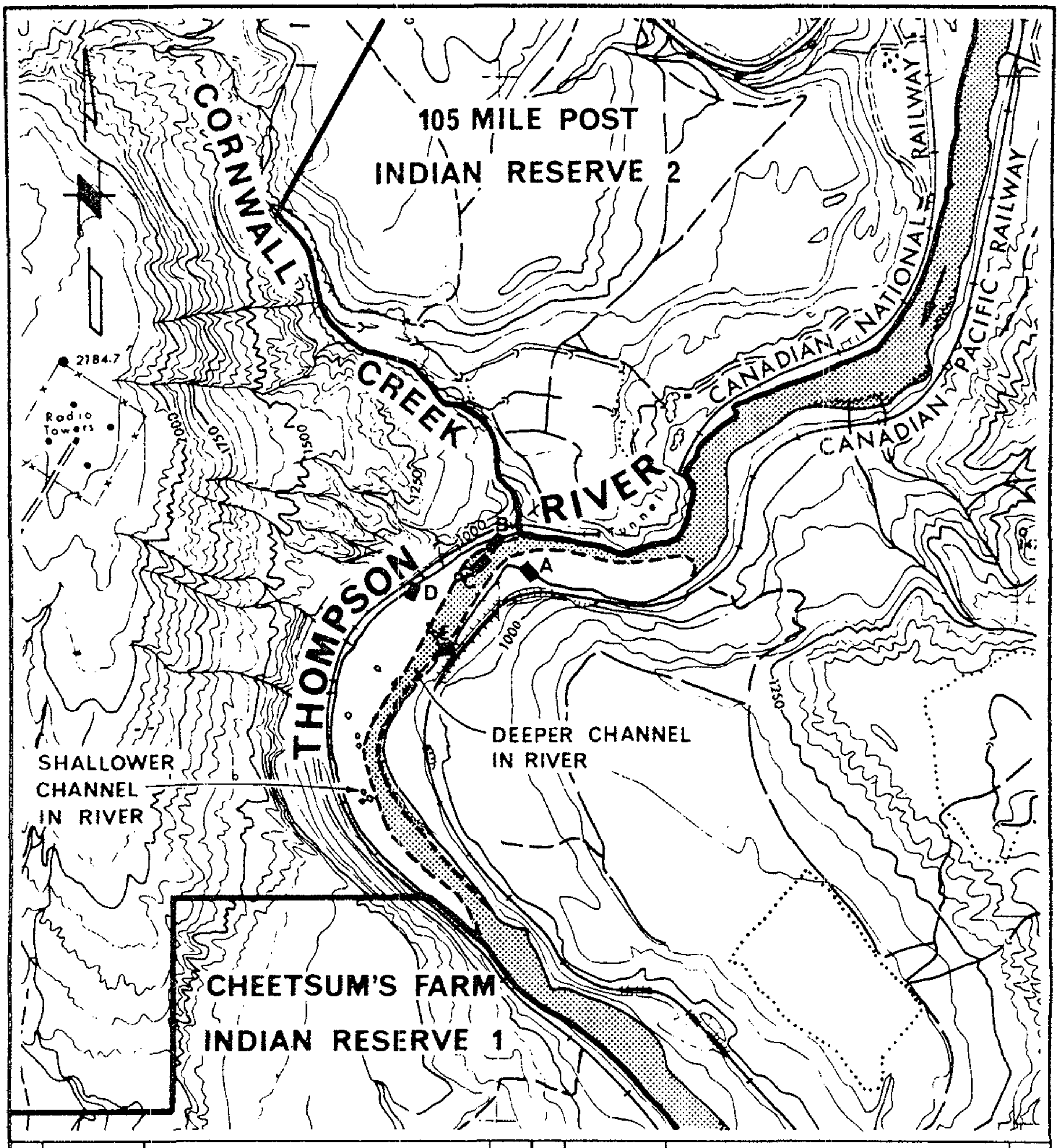
DWG. A 4191/2 - 1 REV



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APPROVED

DATE

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DR'N.	SH	2	6	77
CH'K'D.	J.M.B.			
APP'D.				
APP'D.				

A 4191/2-2

HAT CREEK PROJECT
COOLING WATER SUPPLY
WATER INTAKE SITE 16
POTENTIAL LOCATIONS

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DWG. A 4191/2-2 | REV

PROJECT MEMORANDUM V4191/3

THOMPSON RIVER - WATER LEVEL DATA

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

PROJECT MEMORANDUM V4191/3
THOMPSON RIVER - WATER LEVEL DATA

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

DATE 21 OCTOBER 1977

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PROJECT V4191
HAT CREEK PROJECT
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B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

DATE 21 OCTOBER 1977

PROJECT MEMORANDUM V4191/3
THOMPSON RIVER - WATER LEVEL DATA

PURPOSE

This Project Memorandum presents data on Thompson River water levels taken at potential intake sites 4A and 10, see Drawing A4191/3 - 1*.

With reference to Sandwell's Conceptual Design Report V4007/2, dated January 1977, site 10 was recommended as the prime site for the intake. This site is located on the right bank of the Thompson River just upstream of the confluence with the Bonaparte River, about 2.5 kilometers (1.5 miles) upstream of Ashcroft. Site 4A was the recommended back-up site, also on the right bank, about 11 kilometers (7 miles) upstream of site 10.

To obtain data on the relationship between water surface elevations and river discharges, water level readings at these sites commenced on 6 December 1976. Site 4A was subsequently rejected because of its longer distance to the Hat Creek Plant Reservoir compared with site 10, and because further field surveys produced several potential back-up intake locations in the vicinity of site 10.

DATA

McElhanney Surveying and Engineering Ltd. were retained by Sandwell on 2 December 1976 to establish two temporary bench marks at each of the sites 4A and 10 (Appendix 1) and to carry out water level readings. These bench marks, located on Drawings A4191/3 - 2 and - 3, respectively, provide reference elevations for water levels at both sites 4A and 10. Water level readings are recorded in Tables 1, 2 and 3, Appendix 2. The locations of the main stations where these readings were taken are indicated on Drawings A4191/3 - 2 and - 3.

Readings at Site 4A were discontinued after 15 May 1977, when it was rejected as the potential back-up site. Instead, readings in the site 10 zone were expanded with a third station, 1939 m (6,360 ft) downstream of Site 10.

The relationship between water surface elevations and river discharges computed for Site 10-D, the stage discharge curve, is shown on Drawing B4191/3 - 4.

Drawing D4191/3 - 5 shows hydrographs of the Thompson River near Spences Bridge for the years 1918 through 1977. These graphs serve as historic background for the current 1976 and 1977 hydrographs, the details of which are shown on Drawing D4191/3 - 6. This drawing also illustrates how the water level readings cover the 1976-1977 low water period and the 1977 freshet, and how these relate to the minimum and maximum flows on record.

* For this and other drawings see Appendix 4.

The photographs listed in Table 1 below and contained in Appendix 3, illustrate the river shoreline(s) in the vicinity of Site 10 at various discharges.

Table 1 - Photographs

Photograph No.	File	River Discharge at Site 10 m^3/s	cfs	Date	Computed Water Surface-Elev. at Intake Site 10-D m	ft	Scale
43270[B]*	Pacific Survey	230	8,000	16 April 1970	290.2	952.1	1:2500
1-30	NHCL	200	7,000	22 Feb 1977	289.8	950.8	-
1-18	NHCL	2000	70,600	20 May 1976	295.1	968.2	-
1-19	NHCL	2000	70,600	20 May 1976	295.1	968.2	-

* The overlay on Photograph 43270[B] indicates the proposed location and plan of the intake structure.

Prepared by John W.C. Boyle

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A. Copeland, P. Eng.

Approved by B.R. McConachy

B.R. McConachy, Project Engineer

APPENDIX 1

GROUND SURVEY DATA

McElhanney Surveying
& Engineering Ltd

200 - 1200 W. Pender St., Vancouver, B.C.
Canada V6E 2T3 (604) 683-8521
Telex 04-51474 Cable SURVENG

McElhanney

December 15, 1976

Our File: 07375-0

Mr. A. Copeland
Sandwell and Company Limited
Suite 601 - 1550 Alberni Street
Vancouver, B.C.

Re: Hat Creek Water Supply

As per your instructions of December 2, 1976, we sent a two man survey team to the project area on the morning of December 6th, where they met with Mr. J. White of Northwest Hydraulic Consultants Ltd. He in turn showed our men the T.B.M. (temporary bench mark) at site 4-A and site 10.

At site 10, bench mark 1-401, elevation 990.42, was set on a pole within the Canadian National Railway, right-of-way. This bench mark was based on the existing geodetic survey bench mark (75-J), elevation 988.98. Level loops were double run with a misclose of 0.02 of a foot. Two additional temporary bench marks, numbers TBM1 and TBM2 were set near the waters edge.

At site 4-A, bench mark 1-402, elevation 1034.82, was set on a pole within the Canadian National Railway, right-of-way. This bench mark was based on the existing geodetic survey bench mark (75-J), elevation 1006.32. Level loops were double run with a misclose of 0.04 of a foot. Two additional temporary bench marks, numbers TBM1 and TBM2 were set near the waters edge.

This survey was complete on December 10th and the survey crew returned to Vancouver the same day.

Enclosed find a list of the bench marks descriptions and elevations.

If any further information or clarification of this data is required, please let us know. It has been a pleasure to be of service to you with respect to this work.

Yours truly,

McELHANNEY SURVEYING & ENGINEERING LTD.

Rupert Seel

Rupert R. Seel
Manager, Engineering Surveys Dept.

RRS:ms

VERTICAL CONTROL DATA
FOR
HAT CREEK WATER SUPPLY

Submitted by
McElhanney Surveying & Engineering Limited
1200 West Pender Street, Vancouver, B.C.

December 15, 1976

McElhanney

BENCH MARKS AT SITE #10

Access to BM-1-401 is 3/4 of a mile from Ashcroft on the highway to Cache Creek Junction, turn right and follow the dirt road to Bonaparte Creek to the third pole north from the railway bridge over Bonaparte Creek. Bench mark, 1-401, is identified by a plastic tag, number 1-401. This bench mark is a railway spike driven into a telephone pole #8, on the CNR right-of-way, which is situated 15 feet south of a sign saying,

"High water mark
--October 1880--
when big slide 5 miles west
blocked river for 44 hours"

30 feet east of the tracks on the west side of the river.

Bench Mark 1-401 Elevation 990.42

*TBM1

The railway spike is in a eight inch diameter poplar tree. This spike is about eight feet above the base of the tree which is situated 135 feet east of the CNR tracks and 145 feet northeast of bench mark, 1-401. This bench mark is the previous one used by Northwest Hydraulic Consultants Ltd.

TBM1 Elevation 975.44

TBM2

A railway spike in a poplar tree 150 feet east of the CNR tracks, that is near the edge of the high water mark. It is also 160 feet northeast of the third telephone pole from Bonaparte Creek bridge which has a bench mark, 1-401 on it.

TBM2 Elevation 969.17

* Revision of TBM1 elevation from elevation 969.17 to elevation 975.44 -
February 16, 1977

McElhanney

BENCH MARKS AT SITE 4-A

The entrance to McAbee is 8.6 miles east from Cache Creek Junction on Highway #1.

For permission to enter the property, check at the house located on the south side of Highway #1, one mile farther east.

Bench mark, 1-402, is identified by a plastic tag, number 1-402. This bench mark is a railway spike in a telephone pole, which is the fifteenth one west on a control box and 27 feet east of the CNR tracks, and 15 feet northeast of a sign south of McAbee, saying "McAbee 1 mile".

Bench Mark 1-402	Elevation 1034.82

TBM1

There is a railway spike in a stu-p at the toe of the river bank, approximately 100 feet southeast of bench mark 1-402 and 70 feet east of the railway tracks.

TBM1	Elevation 1013.39

TBM2

A railway spike in a 2" by 4" wooden stake, is located at the base of the river bank, approximately 200 feet south of bench mark 1-402 and approximately 100 feet east of the CNR tracks. This is the bench mark used by Northwest Hydraulic Consultants Ltd.

TBM2	Elevation 1011.21

APPENDIX 2

WATER SURFACE ELEVATIONS

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 21 OCTOBER 1977

PROJECT MEMORANDUM V4191/3
THOMPSON RIVER - WATER LEVEL DATA

APPENDIX 2 - WATER SURFACE ELEVATIONS

Table 1 - Water Surface Elevations in Metric Units

Date	Intake Site: 4A, 600 Ft Down Stream, Elevation (M)	Change (M)	10, 150 Ft Upstream, Elevation (M)	Change (M)	10, 1050 Ft Down Stream, Elevation (M)	Change (M)	Thompson River (See Note 2): Discharge (M3/s)	Change in Discharge (M3/s)
76 Dec 6	303.74		290.21		289.23		298	
		- 0.32		- 0.49		- 0.54		- 97
77 Feb 1	303.41		289.72		288.69		201	
		- 0.01		- 0.02		- 0.01		- 6
Feb 14	303.40		289.70		288.68		195	
		0		- 0.02		- 0.02		0
Mar 1	303.40		289.68		288.66		195	
		- 0.04		- 0.03		- 0.04		- 6
Mar 15	303.36		289.65		288.62		189	
		- 0.02		- 0.01		0		- 3
Mar 31	303.34		289.64		288.62		186	
		+ 0.43		+ 0.57		+ 0.63		+ 102
Apr 15	303.77		290.21		289.25		288	
		+ 1.32		+ 1.68		+ 1.76		+ 566
May 2	305.09		291.89		291.01		854	
		+ 0.89		+ 1.32		+ 1.33		+ 425
May 15	305.98		293.21		292.34		1,279	
	10, 6300 Ft Down Stream							
				- 0.09		- 0.07		- 25
May 29	289.96		293.12		292.27		1,254	
		- 0.19		- 0.26		- 0.25		- 116
June 2	289.77		292.86		292.02		1,138	
		+ 0.54		+ 0.84		+ 0.76		+ 377
June 15	290.31		293.70		292.78		1,515	
		- 0.16		- 0.24		- 0.23		- 167
July 2	290.15		293.46		292.55		1,348	
		- 0.36		- 0.63		- 0.53		- 193
77 July 15	289.79		292.83		292.02		1,155	

Note: 1. This program of recording river water levels was terminated on 15 July 1977 as the peak of the freshet had passed the intake site.

2. Discharges are for Sites 4A and 10 (150 ft upstream). Discharges were obtained by subtracting Bonaparte and Nicola River flows (Stations 8LF02 and 8LG06) from Thompson River flow at Spences Bridge (Station 8LF51).

Table 2 - Water Surface Elevations in Imperial Units

	Intake Site						Thompson River (See Note 2)	
	4A		10					
	600 Ft Down Stream		150 Ft Upstream		1050 Ft Down Stream			
Date	Elevation (Ft)	Change (Ft)	Elevation (Ft)	Change (Ft)	Elevation (Ft)	Change (Ft)	Discharge (CFS)	Change in Discharge (CFS)
76 Dec 6	996.53		952.14		948.94		10,500	
		- 1.06		- 1.60		- 1.77		- 3,400
77 Feb 1	995.47		950.54		947.17		7,100	
		- 0.04		- 0.07		- 0.04		- 200
Feb 14	995.43		950.47		947.13		6,900	
		- 0.03		- 0.05		- 0.07		0
Mar 1	995.40		950.42		947.06		6,900	
		- 0.11		- 0.10		- 0.13		- 200
Mar 15	995.29		950.32		946.93		6,700	
		- 0.05		- 0.05		- 0.01		- 100
Mar 31	995.24		950.27		946.92		6,600	
		+ 1.40		+ 1.89		+ 2.07		+ 3,600
Apr 15	996.64		952.16		948.99		10,200	
		+ 4.32		+ 5.49		+ 5.78		+ 20,000
May 2	1000.96		957.65		954.77		30,200	
		+ 2.93		+ 4.35		+ 4.37		+ 15,000
May 15	1003.89		962.00		959.14		45,200	
	10 6360 Ft Down Stream							
				-0.31		-0.22		- 900
May 29	951.34		961.69		958.92		44,300	
		-0.64		-0.83		-0.82		- 4,100
June 2	950.70		960.86		958.10		40,200	
		+1.78		+2.75		+2.49		+ 13,300
June 15	952.48		963.61		960.59		53,500	
		-0.53		-0.81		-0.75		- 5,900
July 2	951.95		962.80		959.84		47,600	
		-1.19		-2.02		-1.74		- 6,800
77 July 15	950.76		960.75		958.10		40,800	

Note: 1. This program of recording river water levels was terminated on 15 July 1977 as the peak of the freshet had passed the intake site.

2. Discharges are for Sites 4A and 10 (150 ft upstream). Discharges were obtained by subtracting Bonaparte and Nicola River flows (Stations 8LF02 and 8LG06) from Thompson River flow at Spences Bridge (Station 8LF51).

Table 3 - Water Surface Profile

Intake Site 10

Location (See Note 3) m	ft	Elevation m	ft
+1065	3500 u/s	290.40	952.50
+ 915	3000	290.09	951.50
+ 780	2550	290.03	951.30
+ 670	2192	289.98	951.14
+ 365	1192	289.94	950.99
+ 140	450	289.83	950.65
+ 45	150 u/s	289.79	950.54
0	centerline	289.64	950.02
- 35	108 d/s	289.61	949.93
- 65	208	289.56	949.77
- 125	408	288.75	947.13
- 245	808	288.78	947.20
- 320	1050	288.78	947.20
- 550	1808	288.73	947.06
- 850	2808	288.70	946.96
-1010	3308	288.66	946.81
-1160	3808	288.50	946.30
-1315	4308	288.42	946.03
-1465	4808	288.16	945.18
-1730	5808	288.00	944.64
-1925	6308	287.81	944.03
-2075	6808 d/s	287.40	942.68

Notes

1. All stations upstream of and including +780 m (2550 ft u/s) were surveyed by NHCL on 2 and 3 March 1977 when river discharge was approximately 200 m^3/s (7,000 cfs).

2. All stations downstream of and including +670 m (2192 ft u/s) were surveyed by McElhanney on 31 January through 2 February 1977 when river discharge was approximately 205 m^3/s (7,300 cfs).

3. For locations of river cross sections see Drawing A4191/3 - 3

APPENDIX 3

PHOTOGRAPHS OF INTAKE SITE 10-D

(PM V4391/3)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

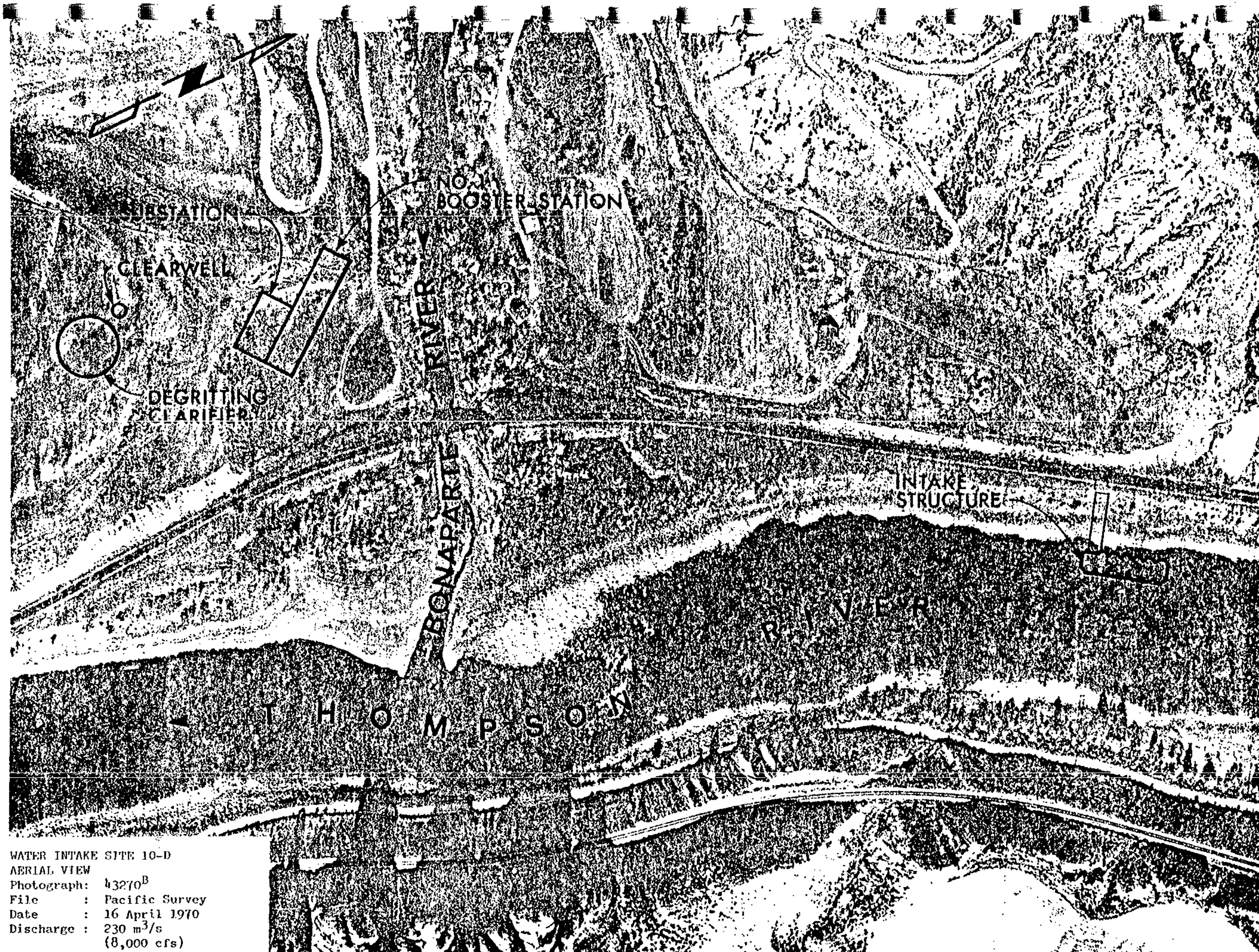
DATE 21 OCTOBER 1977

PROJECT MEMORANDUM V4191/3
THOMPSON RIVER - WATER LEVEL DATA

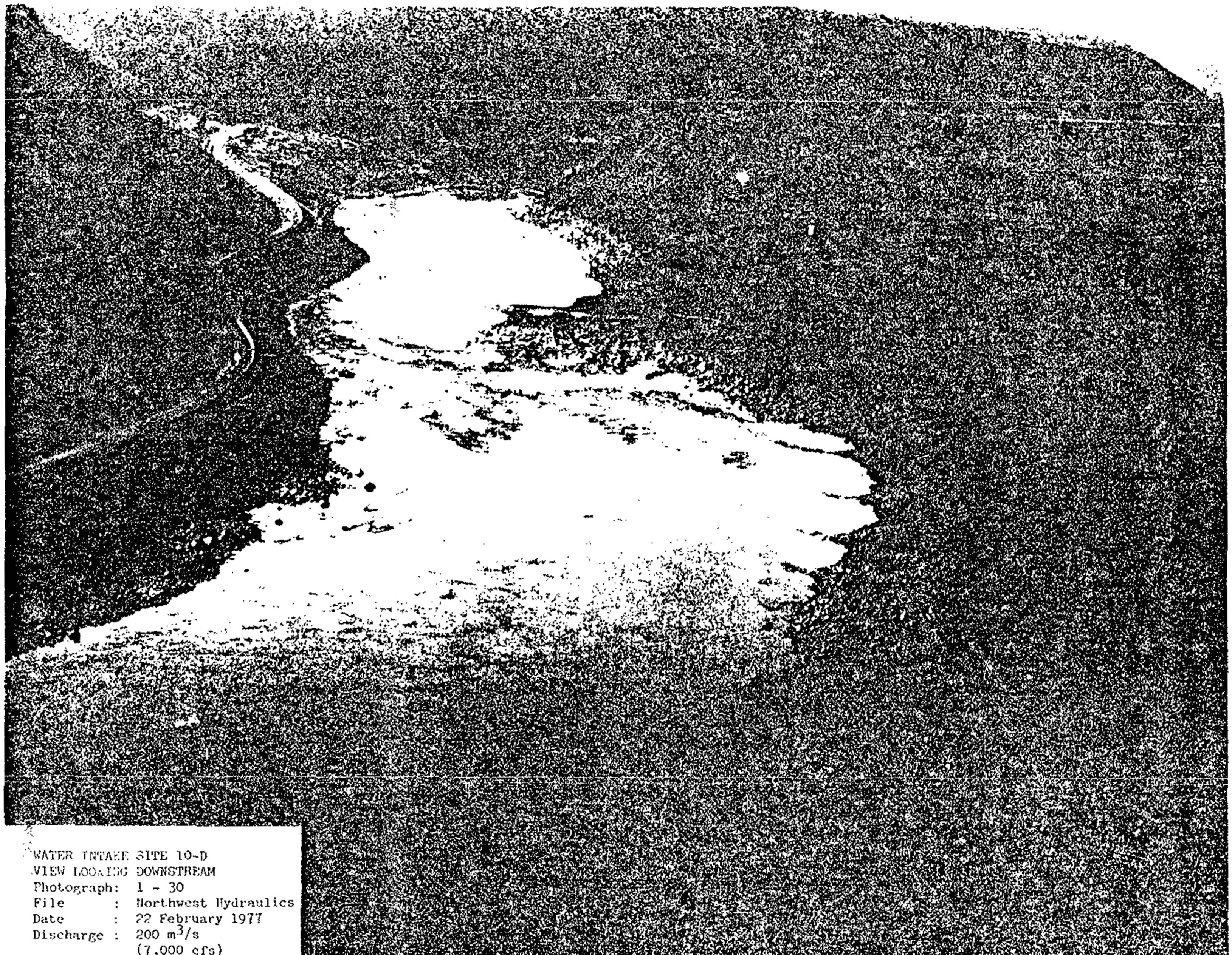
APPENDIX 3 - PHOTOGRAPHS OF INTAKE SITE 10-D

Table 1 - Photograph Descriptions

Photograph	Description
43270^{B}	Aerial view of Site 10-D with proposed intake position indicated on overlay.
1-30	Thompson River in vicinity of Site 10-D during typical low water period, (looking downstream). Rapids are at Thompson/Bonaparte confluence.
1-18	Thompson River looking downstream from CNR bridge. Site 10-D in middle of photograph and Ashcroft in distance.
1-19	Close up of Site 10-D and Thompson River looking downstream with Bonaparte confluence in top left of picture.

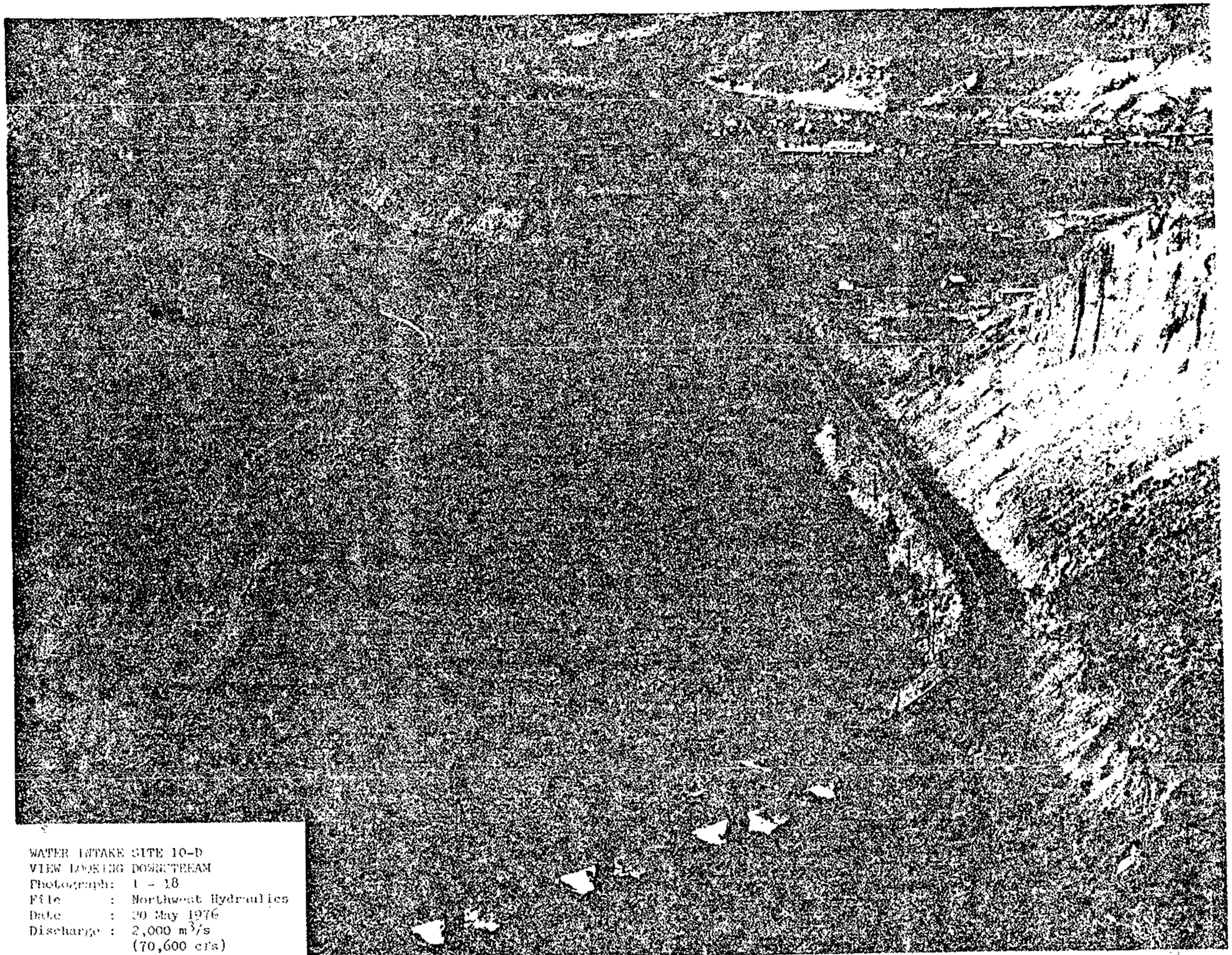


WATER INTAKE SITE 10-D
AERIAL VIEW
Photograph: 43270B
File : Pacific Survey
Date : 16 April 1970
Discharge : 230 m^3/s (8,000 cfs)
Scale : 1:2,500

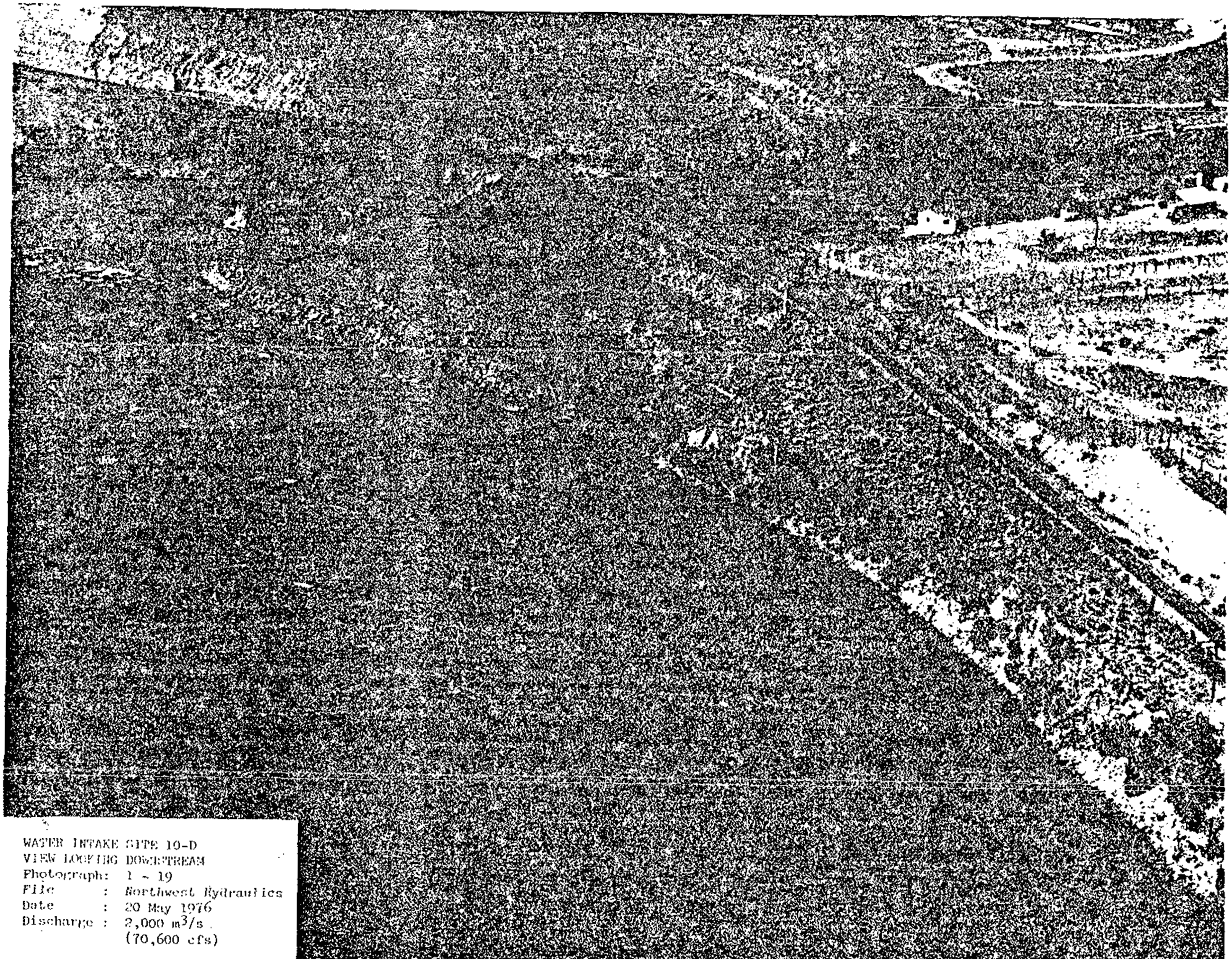


WATER INTAKE SITE 10-D
VIEW LOOKING DOWNSTREAM
Photograph: 1 - 30
File : Northwest Hydraulics
Date : 22 February 1977
Discharge : 200 m^3/s
(7,000 cfs)

SANDWELL



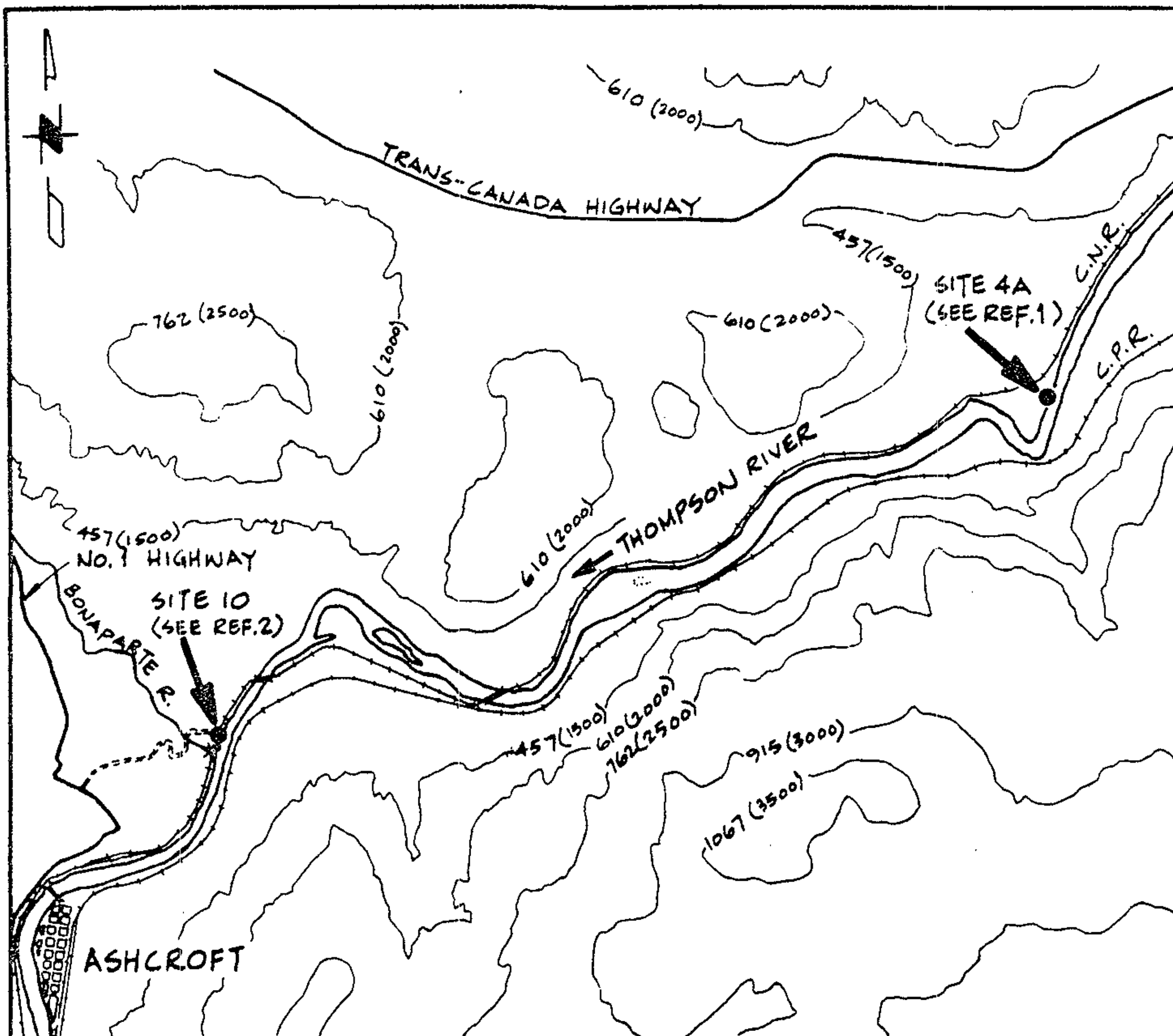
WATER INTAKE SITE 10-D
VIEW LOOKING DOWNSTREAM
Photograph: 1 - 18
File : Northwest Hydraulics
Date : 30 May 1976
Discharge : 2,000 m^3/s
(70,600 cfs)



WATER INTAKE SITE 10-D
VIEW LOOKING DOWNSTREAM
Photograph: 1 - 19
File : Northwest Hydraulics
Date : 20 May 1976
Discharge : 2,000 m^3/s
(70,600 cfs)

APPENDIX 4

ILLUSTRATIONS



NOTE:
CONTOURS ARE GIVEN IN METERS (FEET)

1 0 1 2
SCALE IN KMS.

1 0 1
SCALE IN MILES

DWG. No.	REFERENCE	REF. No.	REV.	BY	DATE	REVISION	APP'D.
A 4191/3-3	SITE 10 – WATER LEVEL STATIONS	2					
A 4191/3-2	SITE 4A – WATER LEVEL STATIONS	1					

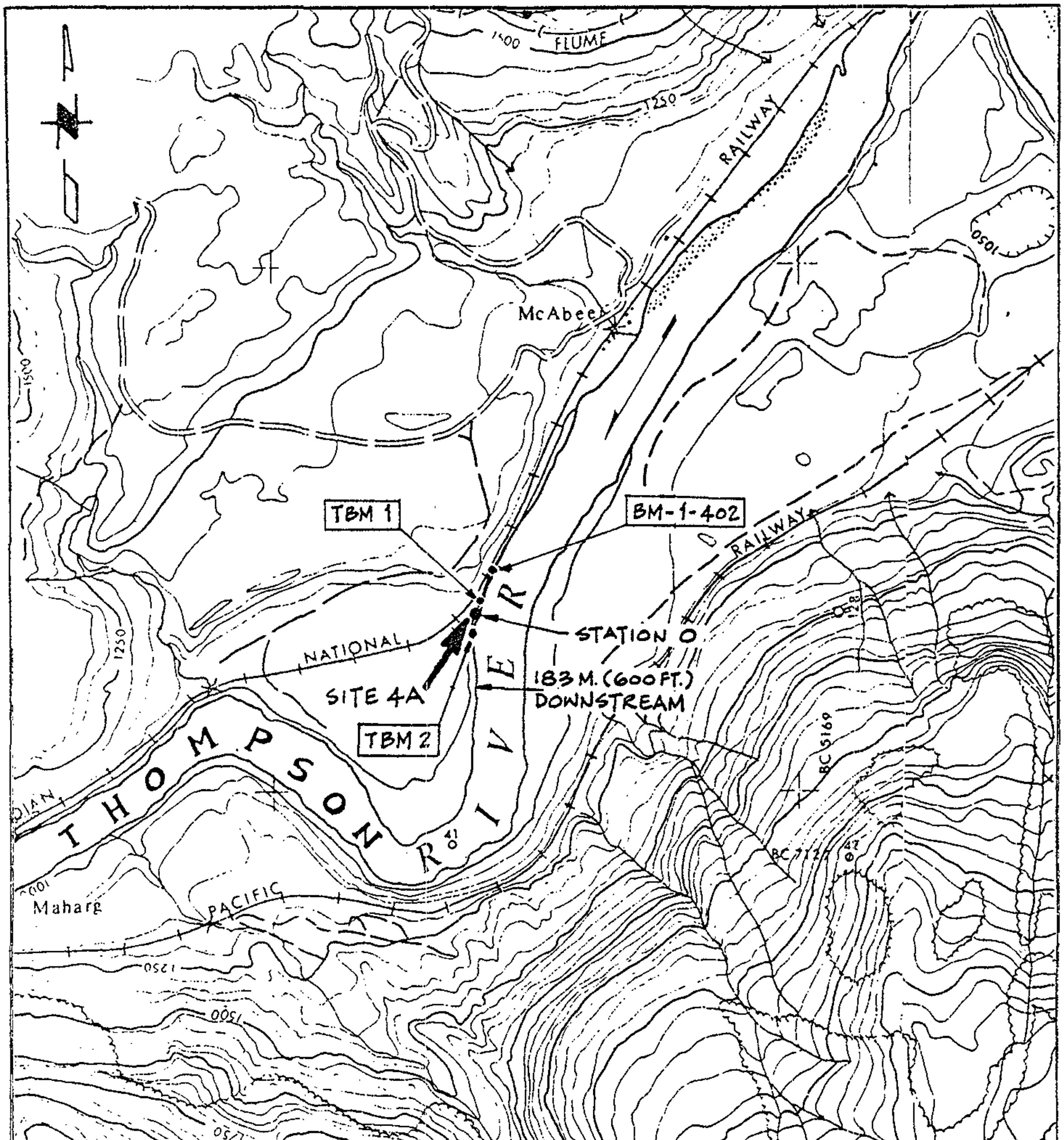
			DAY	MO.	YR.
APPROVED	SCALE	1:50,000			
DATE	DR'N.	D. TUNG	9	9	77
	CH'K'D.	JWCB.			
A 4191/3-1	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
THOMPSON RIVER SURVEY
SITE LOCATION MAP

SANDWELL

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

DWG. A4191/3-1 REV



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

APPROVED

DATE

		DAY	NO.	YR.
SCALE	1:16,000			
DR'N.	D. TUNG	8	9	77
CH'K'D	JWCB			
APP'D.				
APP'D.				

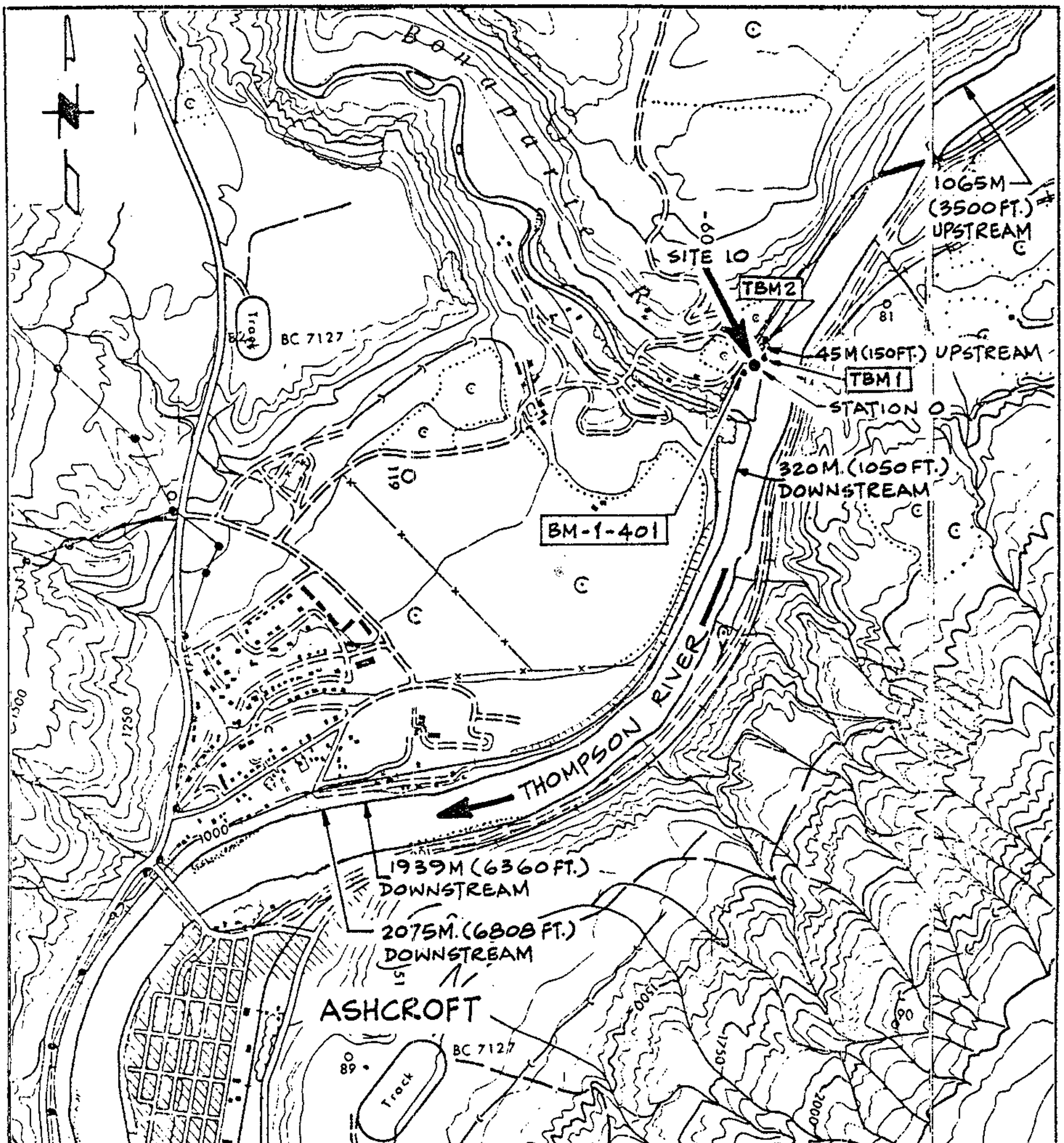
A 4191/3-2

HAT CREEK PROJECT
COOLING WATER SUPPLY
THOMPSON RIVER SURVEY
SITE 4A — WATER LEVEL STATIONS

SANDWELL

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

DWG. A 4191/3-2 REV



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

APPROVED

DATE

		DAY	MO.	YR.
SCALE	1: 16,000			
DR'N.	D. TUNG	8	9	77
CH'K'D.	JWCB			
APP'D.				
APP'D.				

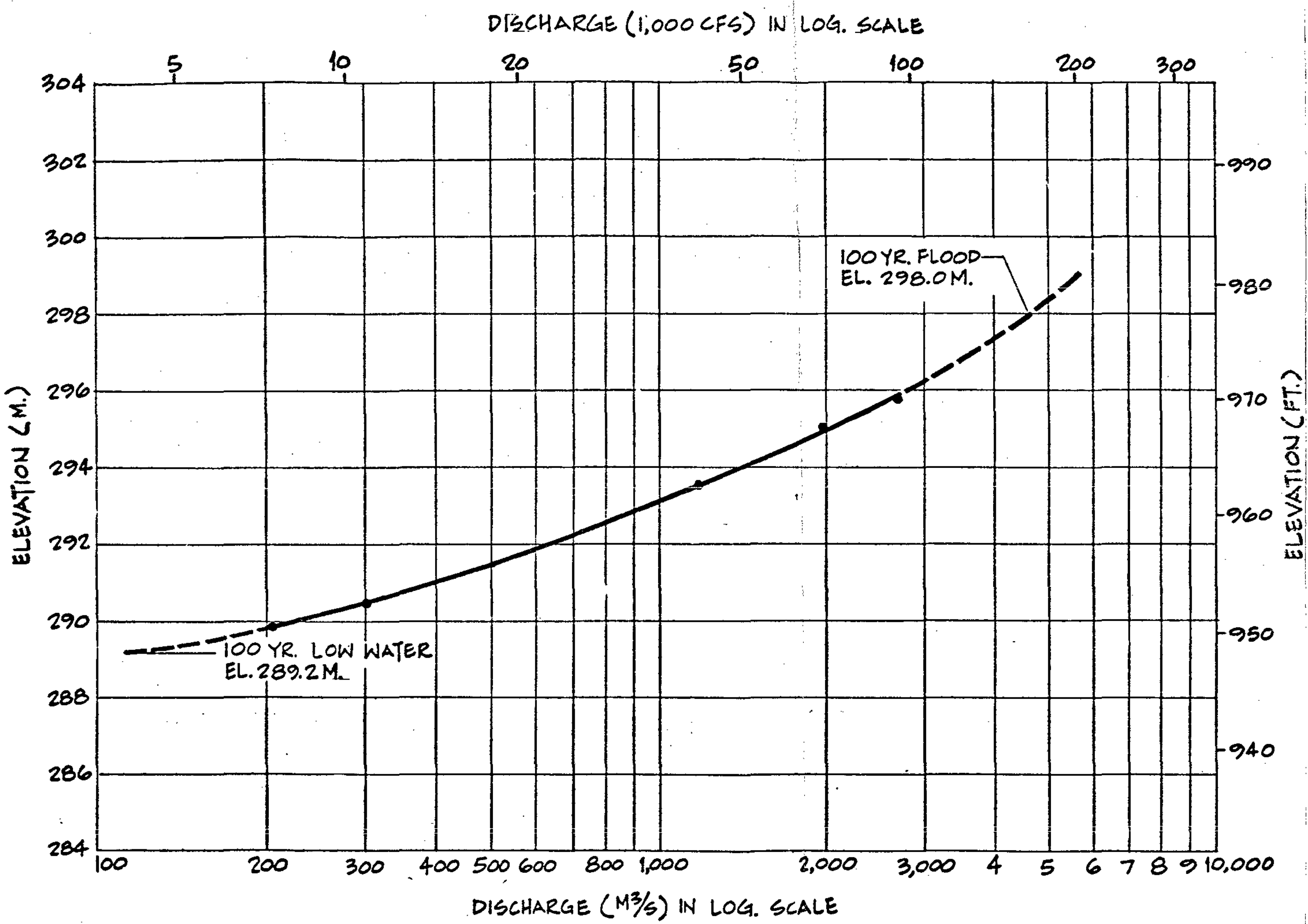
A 4191/3-3

HAT CREEK PROJECT
COOLING WATER SUPPLY
THOMPSON RIVER SURVEY
SITE 10 — WATER LEVEL STATIONS

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

SANDWELL

DWG. A 4191/3-3 REV



LEGEND:

——— CURVE BASED ON AVAILABLE FIELD DATA

– – – ESTIMATED CURVE

NOTES:

1. THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL. THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN 200 AND 2500 M^3/S (7,000 AND 88,000 CFS). PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.
2. THE INFORMATION ON THIS DRAWING WAS OBTAINED FROM NORTHWEST HYDRAULIC CONSULTANTS LTD.

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

APPROVED	SCALE	———	DAY	MO.	YR.
DATE	DR'N.	D. TUNG	29	9	77
	CH'K'D.	JWCB			
B 4191/3-4	APP'D.				
	APP'D.				

SANDWELL

HAT CREEK PROJECT
COOLING WATER SUPPLY
STAGE – DISCHARGE CURVE
SITE 10-D

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

DWG. B 4191/3-4 REV

NOTES:

1. HYDROGRAPHS ON THIS DRAWING WERE PREPARED BY NHCL FROM WATER SURVEY OF CANADA DATA.
2. FOR DETAILED 1976 & 1977 HYDROGRAPHS NEAR SPENCES BRIDGE SEE REF 1.
3. THE MINIMUM FLOW ON RECORD NEAR SPENCES BRIDGE IS 116 M³/S (4.00 CFS) – 8 JAN. 1937, AND THE MAXIMUM RECORDED FLOW IS 4,134 M³/S (146,000 CFS) – 15 JUNE 1972.

LEGEND:

* INDICATES PERIODS WHEN DATA WERE NOT RECORDED

SANDWELL	DWG. NO. D4191/3-5
DES	RECD
DWN D TUNG	APPD (TECH)
CHKD JWB	APPD
INSP	APPD
SUBD	DATE OCT 1977
MICROFILMED	JOB No. V4191
CONTRACT No.	SCALE —

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

THOMPSON RIVER HYDROGRAPHS, 1918 – 1977

DWG. No. D4191/3-5

D4191-SK78 HYDROGRAPHS NEAR SPENCES BRIDGE, 1976-1977

REFERENCE DRAWINGS

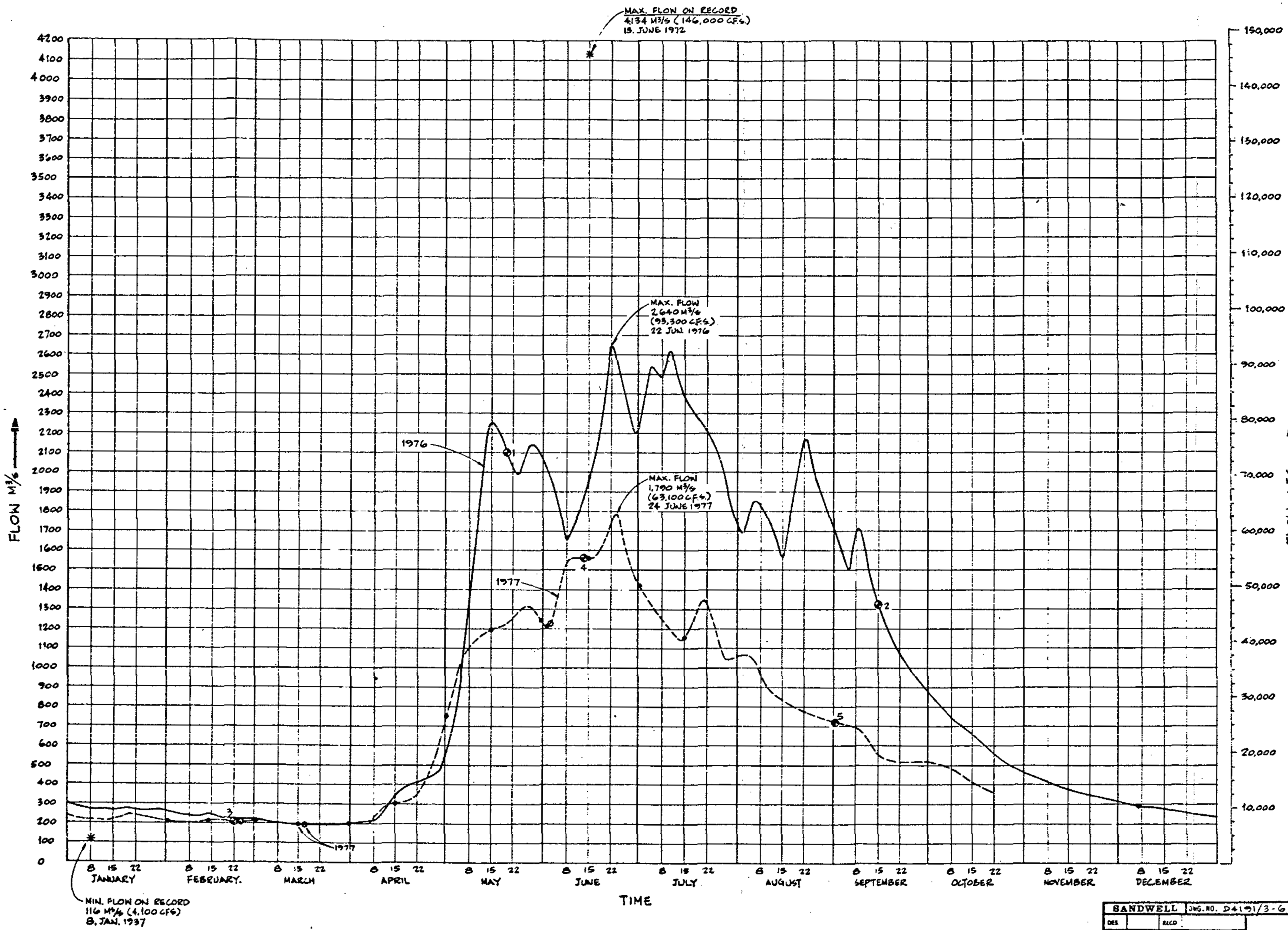
REFERENCE DRAWINGS

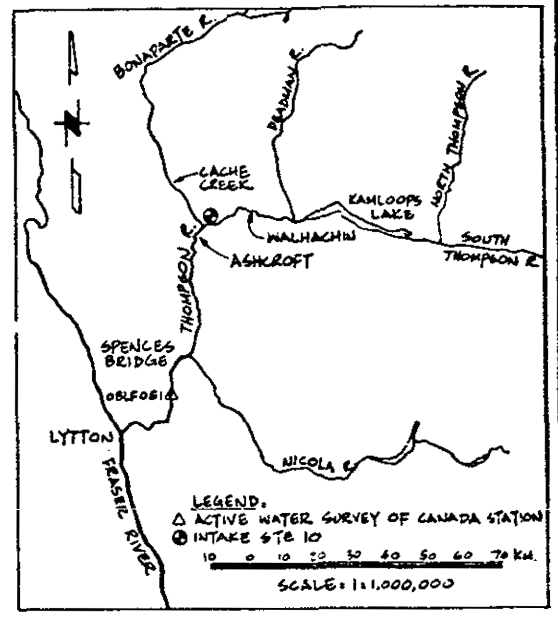
No. REMARKS DATE MADE CHKD INSP SUBD RECD APP(T) APPD

REVISIONS

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

REVISIONS





KEY PLAN

LEGEND:

- ● WATER LEVELS TAKEN AT INTAKE SITE 10.
- ⊗ FIELD VISITS MADE AT INTAKE SITE 10.
- O WATER SAMPLES TAKEN AT WALHACHIN BRIDGE.

NOTES.

1. THE DATA USED TO PLOT THESE HYDROGRAPHS FOR THE THOMPSON RIVER NEAR SPENCES BRIDGE WAS SUPPLIED BY THE WATER SURVEY OF CANADA.
2. THE 1977 CURVE IS BASED ON INCOMPLETE DATA AND, THEREFORE, THE PEAKS MAY NOT BE EXACTLY IN THE CORRECT POSITIONS.

⊗ RECORD OF FIELD VISITS AT INTAKE SITE 10

NO.	DATE	REMARK
1	20 MAY 1976	FIRST HELICOPTER VISIT
2	16 SEPT. 1976	SECOND HELICOPTER VISIT
3	22 FEB. 1977	THIRD HELICOPTER VISIT
4	13 & 14 JUNE 1977	GROUND VISIT
5	1 SEPT. 1977	GROUND VISIT

SANDWELL	DWG. NO. D4191/3-6
DES	RECD
DWN D. TONG	APPD (TECH)
CHKD JWB	APPD
INSP	APPD
SUBD	DATE OCT. 1977
MICROFILMED	JOB No. V4191
CONTRACT No	SCALE AS NOTED

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
THOMPSON RIVER HYDROGRAPHS, 1976 & 1977

DWG No D4191/3-6 | R

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS

PROJECT MEMORANDUM V4191/4

SYSTEM DESIGN

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/4
SYSTEM DESIGN

DATE 10 FEBRUARY 1978

CONTENTS

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/4
SYSTEM DESIGN

DATE 10 FEBRUARY 1978

SCOPE AND METHODOLOGY OF STUDY

The system design studies comprised two main tasks. The first task was to select the least costly schemes based on quantifiable costs and assign dollar values to each scheme. The second task was to study these results using engineering judgement and experience, to weigh the differences in dollar values against practical considerations, the costs of which are not easily expressed in dollar terms.

In the discussion following, details of the cost analysis methodology are discussed under "Optimization Study" and details of judgement factors under "Practical Considerations".

OPTIMIZATION STUDY

General

Many possible combinations of pumping schemes, pipeline routes and pipe sizes have been investigated to determine the optimal scheme. The pumping schemes studied range from the single high-life booster station of the Conceptual Design, to multiple booster stations. Four main routes were studied:

A - Following the proposed 500 kV transmission line to the proposed power plant access road corridor.

B - Following an alternate transmission line location to the access road corridor.

C - The Conceptual Design route.

D - Following Highway 1 to Cornwall Creek, and then following the access road corridor.

Pipe diameters from 600 to 1,000 mm were examined. Comparisons were based on total capital and operating cost estimates.

Costs have been derived from those developed during the Conceptual Design Study except that additional equipment prices were obtained where necessary. Capital costs include owner's construction overhead, engineering and contingencies. Operating costs have been expressed as equivalent present worth and were added to capital costs. Pumping stations in series, currently assumed to operate with open tanks, were assumed to be closed systems. The cost of water treatment is not included.

Routing

The main effort in the optimization procedure was expended on the Conceptual Design route, C, shown on drawing A4191/4-1[1]. Prior to optimizing the pumping arrangement along this route, the alternative routings in the area of Elephant Hill were studied. A detailed report on the findings is contained in Appendix 3 of this Project Memorandum, and can be summarized as follows:

1. A pipeline route along the Bonaparte River skirting Elephant Hill to the northeast, is not feasible due to geotechnical hazards.

2. A tunnel through Elephant Hill would require further investigation by drilling to confirm, but is expected to be feasible. However, it is more costly than a surface crossing, and could present a scheduling difficulty.

3. A buried pipeline crossing along the Conceptual Design route is practical from a construction viewpoint, and is the least costly alternative for Elephant Hill routing.

Therefore, the routes studied in this document utilize a buried pipeline crossing Elephant Hill along the Conceptual Design route.

As shown on drawing A4191/4-1, the route alternatives studied here are:

A - Transmission line corridor. This route was developed to combine the pipeline with the proposed 500 kV transmission line corridor for the power plant.

B - Alternate transmission line corridor. This route would require relocation of the proposed transmission line location to follow the Conceptual Design pipeline route.

C - Conceptual Design route, as described in Report V4007/2.

D - Cornwall Creek route, developed to follow the proposed power plant access road corridor along Cornwall Creek.

Note that routes A, B and D all follow the access road corridor through the Medicine Creek pass to the power plant reservoir.

Optimization Procedure

Optimization was performed manually by the method of "dynamic programming"[2]. The method is best explained with the aid of drawing A4191/4-2. A network is superimposed onto the profile, with vertical lines ("stages") representing potential pumping station locations, and sloping near-horizontal lines ("states"), at a slope corresponding to friction losses in the pipeline, representing potential hydraulic grade lines. Each segment of each of these horizontal lines has a cost associated with it, which is the cost of that length of pipeline at that pressure and of the diameter under study. Further, at each potential pumping station location, a cost is calculated for a pump station suitable for lifting the water from each lower state to each higher state.

---

1. For this and other drawings, see Appendix 4.
2. See references, Appendix 1.

One works through the diagram from right to left, and at each "Current node" the calculation would proceed as follows:

Cost to reach current node from previous stage:

From state 1: C_1 + Cost of Pipeline a + Cost of Pump Station a

From state 2: C_2 + Cost of Pipeline b + Cost of Pump Station b

From state 3: C_3 + Cost of Pipeline c

From state 4: Not feasible as head would be wasted.

In these calculations, C_1, C_2 and C_3 are the minimum cumulative costs to reach the previous stage from the starting node. The appropriate minimum cumulative cost from those above, would be recorded at the current node. Other nodes in the current stage would be completed, then the procedure would be repeated until the network was completed. Finally, the energy cost would be added according to the assumed pipe diameter.

A similar diagram was made for each pipeline diameter under study for route C. After the optimization for route C was completed, and it was determined that 800 mm was the optimal diameter, diagrams were made for the other routes using only that diameter.

After completion of the diagrams, the sensitivity of the solution was assessed by determining second and third ranking solutions.

Pipeline Cost Estimate

Pipeline cost can be broken down into two categories, namely those costs which depend on the pressure but not on route, and those which depend on route but not on pressure. By dividing the cost this way, a pressure-dependent cost per unit length for a given route, such as that shown on drawing A4191/4-3 for route C (Conceptual Design), can be determined. For simplicity, the same unit costs have been used to evaluate all routes, although strictly they apply only to route C.

These unit costs were estimated using the Conceptual Design cost estimate (Report V4191/2, January 1977, Conceptual Design), as follows:

1. Items in the estimate which depend only on the route, and not on pipe diameter or pipeline pressure, were added together and a price per unit length was calculated. These items include: all "pipeline structures" - such as valve pits, manholes, etc.; and from "pipeline equipment" - process control, motors, starters and MCC, power and control wiring, communications, access roads, clearing, grading, stockpile, haul and string, dewatering, anchors, heat tracing, repair coating, padding and rock shield, drain lines, testing, road, railroad and gas line crossings, steam crossings, cathodic protection, drainage control, seeding, surge chambers, access manholes, and pig traps. The total of these is $6,235,000 or $285 per metre.*

* After completion of the optimization studies, it was discovered that an item from the Conceptual Design Cost Estimate, "Substations along Pipeline and Feeder Cable", totalling $1.33 million or $60/m, was omitted from the above list. This would add a uniform cost, regardless of pumping scheme or pipe diameter, to each route, and thus does not affect the comparative costs of various schemes.

2. The items excluded from the list above, were divided into pressure-dependent and pressure-independent components. The latter were determined for the range of pipe diameters of interest as follows:

Table 1 - Pressure - Independent Pipeline Costs

Item	Total Cost for Route C ($1000s) for Nominal Pipe Diameter (mm) 700	800	900	1,000
Sundry Pipe, Valves and Fittings[1]	680	778	825	972
Coating and Sect Carrier[2]	1,758	1,854	1,950	2,046
Ditching[3]	821	882	985	1,075
Bedding[4]	1,480	1,480	1,695	1,810
Placing	1,240	1,240	1,240	1,240
Bends[1]	622	711	800	890
Backfill and Compact[3]	191	207	230	251
Total	6,792	7,152	7,725	8,284
Unit Cost ($/m)	310	326	352	377
Unit Cost ($/m) - Diameter-independent	285	285	285	285
Total - Unit Cost ($/m) - Pressure-independent	595	611	637	662

1. Assumed cost is proportional to diameter.
2. Coating cost proportional to surface area.
3. Excavation quantities calculated with 1:1 side slopes, 1.5 m cover on pipe.
4. Assumed proportional to trench width.

3. The pressure-dependent costs are for pipe and welding only. Pipe cost was calculated using material prices from the Conceptual Design. Federal and provincial sales taxes, and freight were added. Welding cost was estimated using the Conceptual Design man-hour labour rates.

These pressure-dependent costs, and total costs, are shown on Table 2.

Table 2 - Total Pipeline Cost

Nominal Pressure (MPa)[1]	Item	Cost ($/m) for Nominal Diameter[2] 700 mm	800 mm	900 mm	1,000 mm
2.9	Pressure-independent	595	611	637	662
	Pipe	110	135	160	190
	Welding	25	34	53	68
	Total	730	780	850	920
4.9	Pressure-independent	595	611	637	662
	Pipe	160	200	235	290
	Welding	30	39	53	68
	Total	785	850	925	1,020
7.4	Pressure-independent	595	611	637	662
	Pipe	225	285	345	415
	Welding	40	49	58	73
	Total	925	945	1,040	1,150
9.8	Pressure-independent	595	611	637	662
	Pipe	290	370	445	545
	Welding	40	59	68	133
	Total	925	1,040	1,150	1,340
12.8	Pressure-independent	595	611	637	662
	Pipe	375	480	580	675
	Welding	50	84	123	153
	Total	1,020	1,175	1,340	1,490

1. 1 MPa (mega Pascal) = 145 psi.
2. Cost rounded. These results are plotted on drawing A4191-4-3.

Pumping Station Cost Estimate

Pumping station cost can be divided into location-dependent and location-independent costs. Further, for the intake and for a given booster station, certain costs, such as structural costs, would be largely independent of the delivery pressure, whereas the cost of pumping equipment and piping are pressure-dependent. Therefore, the portion of the total pumping station cost which does not depend on location, is as given in Table 3.

Table 3 - Location-Independent Pump Station Costs

Type of Pumping Station	Discharge Head (m)	Cost ($ 1,000's) Structures	Equipment	Substation	Total	Total[1] Including Contingency & Misc.
Intake	0-100	3,600	1,850	120	5,570	7,510
	400	3,780	2,800[2]	420	7,000	9,510
Booster Stations	0-400	720	3,500	470	4,690	6,380
	520	720	3,600	470	4,790	6,530
	670	720	4,000	560	5,280	7,180
	890	720	4,500	680	5,900	8,020
	1,300	820[3]	6,000	790	7,610	10,340

(1) Following the Conceptual Design Study cost estimate, a total addition of 36 percent is allotted for Owners' construction overhead, engineering costs, and contingency. This factor is also applied to pipeline costs, but was added at a later stage of computation.
(2) Includes an extra maintenance cost allowance in view of pump wear.
(3) Structure size increased as five rather than four pumps required.

These costs have been plotted on drawing A4191/4-4. Maintenance cost and the location-dependent costs, such as for transmission lines, land, excavation and access roads, have been determined for each potential site and are shown on Table 4. Again, for simplicity, the location-dependent costs assessed for route C were used for the other routes as well.

Table 4 - Location-Dependent and Operating Costs
Cost ($1,000's)

Site[1]	Road and Excavation[2]	Transmission Line[3]	Land and Right-of-Way	Operating Cost[5]	Total	Total[6] Including Overhead, Misc
Thompson River Intake	Included[7]	540	80[4]	670	1,290	1,660
Thompson-Bonaparte Confluence	Included[7]	20	25	670	715	970
Boston Flats at Highway	Included[7]	100	25	670	795	1,080
Above Highway at Elevation 610 m	110	140	10	670	930	1,270
Above Highway at Elevation 730 m	110	160	10	670	950	1,300
Above Highway at Elevation 1,220 m	60	260	10	670	1,000	1,360

(1) For route C. See drawing A4191/4-1 for approximate locations of pumping stations.
(2) Includes access roads at \$20/m, site excavation at \$15/m^3.
(3) For substation to provide power from the existing 230 kV transmission line in the Semlin Valley, and feeders to the various sites. (Cost data from B.C. Hydro).
(4) An allowance of \$75,000 for the entire pipeline right-of-way is included in the intake location cost only.
(5) Includes maintenance and maintenance materials, plus an annual overhaul, for the project life of 35 years capitalized at eight percent interest.
(6) See note 1, Table 3.
(7) Included in location-independent cost.

Waterhammer Control

During the optimization study, it was not practical to make detailed calculations for the effects of waterhammer in all possible combinations of routes and pumping configurations. Therefore, the following allowances[1] in pipe design were made uniformly to all pipelines:

1. Pipeline maximum pressure was calculated as the operating pressure plus 10 percent on the discharge side of pumping station, and as operating pressure plus 300 m on the suction side of pumping stations in series. The 300 m pressure rise was to allow for the effect of a closed system.

2. The above figures of 10 percent and 300 m apply to 900 mm diameter pipelines. For other diameters, these figures were adjusted in proportion to the velocity at rated discharge.

3. The pipe was designed for a full vacuum. Protection against water column separation was included as a lump sum in the unit pipeline cost.

4. All pump stations were assumed to have the same control equipment installed.

Following the completion of optimization studies, it was determined that air chambers may be required to limit pressure rise to 10 percent. As it was not practical to compute air chambers requirements for each possible scheme, the cost of these was added afterwards. At the date of writing, air chambers are no longer thought necessary.

Power Cost

Power cost was calculated as the present worth of the value of the power required to achieve the average flow, 640 l/s (10,130 USGPM), over 35 years. This sum, for each different pipe diameter, is given in Table 5 below:

Table 5 - Present Value of Pumping Energy
(At 8% interest, 20 mills per kWh, over 35 years)

Pipe Diameter (O.D.) mm	Minimum Cost (Continuous 640 l/s Discharge)*	Maximum Cost (Discharge 1580 l/s 40% of the time)
610	$ 23.06 (Million)	$ 37.35 (Million)
711	21.45	27.41
813	20.75	23.79
914	20.43	22.15
1,016	$ 20.28	$ 21.30

*Assumed for Optimization Study

The power expended against friction decreases as the pipe diameter increases, however, because of the relatively high static lift, the savings that can be made in this way are limited. As shown in drawing A4191/4-5, the power cost for static lift alone is about $20.08 million.

1. Based on a letter dated 4 July 1977, to Sandwell from Mr. I. C. Dirom, Hydroelectric Design Division, B.C. Hydro.

Results of Optimization

During the optimization procedure three basic schemes, shown on drawing A4191/4-6, emerged with lower costs than all others:

Scheme 1: High lift intake pumps, with one booster pumping station at Boston Flats or vicinity.

Scheme 2: Low lift intake pumps, with two booster pumping stations, one at the Thompson River and one at Boston Flats or vicinity.

Scheme 3: Low lift intake pumps, with one booster pumping station at the Thompson River. (Conceptual Design).

The optimization study concentrated first on route C, for which an 800 mm pipe size offered the least cost, see drawing A4191/4-7. Thereafter, this pipe size was used throughout in the appraisal of other routes.

Table 6 summarizes the approximate total cost index, including power and operation, for the routes and schemes studied.

Table 6 - Summary of System Design Study, 800 mm (32 inch) Pipeline

		Capital and Operating Cost Index[1]		
Route (See A4191/4-1)	Route Description	Scheme 1 High Lift Intake	Scheme 2 Two Booster Stations	Scheme 3 Conceptual Design
FOR CLOSED SYSTEM, EXCLUDING AIR CHAMBERS				
A	Transmission Line	102	110	106
B	Alternate Transmission Line	101	-	105
C	Conceptual Design	100	108	104
D	Cornwall Creek Access Road	109	114	114
FOR CLOSED SYSTEM, INCLUDING AIR CHAMBERS[2]				
C	Conceptual Design	104	112	106

1. One index point out of 100 constitutes $660,000 present value.
2. Air chambers were considered necessary to achieve suitable waterhammer control for all routes and schemes, however, air chamber calculations and cost estimates have only been made for the three route C schemes. The extra cost of air chambers raises the base index cost by $2.9 million for all three route C schemes.

Table 6 shows that:

- For all routes, Scheme 1 has the lowest cost, followed by Scheme 3 and Scheme 2.

- When air chamber costs are included in the analysis for route C, Scheme 3 is about 1.3 percent ($890,000) more than Scheme 1.

- Routes C and B have the lowest cost compared to other routes. Route D would cost an additional $5.7 million, but route A only an additional $1.9 million. Such comparisons may be misleading, however, as the same unit costs which were developed for route C were used for all other routes.

One can conclude from this:

- The cost for following either the access road (route D) or transmission line (route A) is higher than for route C or B.

- An 800 mm diameter pipeline should be selected for Scheme 1.

- Scheme 1 has the least cost.

Practical considerations which will be discussed later, and pipe supply in particular, led to the development of a variation of Scheme 2 using two equal-lift booster pumping stations. When the cost of air chambers and tanks for an open system is included, the total cost for route C for this and the other two schemes is as follows:

- Scheme 1	$70.0 million
- Scheme 3, (Conceptual Design)	$70.4 million
- Scheme 2, with equal lifts	$73.9 million

Further detail is provided in Appendix 2, Details of Cost Estimates.

These cost figures must be weighed against practical considerations, as described in the section following.

PRACTICAL CONSIDERATIONS

Pump Wear

Pump wear is a serious concern with the installation of high lift pumps of the type required for this project. For reasons described in detail in Project Memorandum V4191/17, Pumps and Pump Wear, pump wear considerations led to the following recommendations.

1. Use low lift intake pumps.

2. Provide water treatment to remove grit from the water prior to booster pumping.

Scheme 1, with its high lift intake pumps, is not compatible with these criteria, whereas either of Scheme 2 or 3 is.

Pipe Supply

In the course of this study, major pipe suppliers were contacted to determine available pipe wall thickness and material for the pipe diameters of interest. The results are shown on drawing A4191/4-8, together with the maximum wall thickness required for Schemes 1, 2 and 3.

The drawing shows that few suppliers are capable of supplying the thickest wall pipe for Scheme 3. Schemes 1 and 2 have a more widely available requirement, still outside the range of Canadian mills. It is evident that using the variation of Scheme 2 with equal-lift booster pumping stations, the maximum wall thickness pipe required is reduced, and thus is available from more suppliers, including Canadian mills.

Welding

Welding cost was included in the pipeline cost, however, there is no doubt that field welding difficulties compound as maximum wall thickness increases. For example, suppliers have provided information on heat treatment requirements for welding various pipe wall thicknesses and grades, and some of this information is presented on drawing A4191/4-9.

This drawing is intended merely to indicate that as wall thickness and steel grade increase, the difficulties of performing successful welds increase. For example, most pipeline welding is done with cellulose electrodes, which are limited to about 20 mm wall thickness. Otherwise, low hydrogen electrodes must be used with the result that cost and difficulty increase. Further, since heat treatment requirements increase with wall thickness, one may conclude that some aspects of welding cost, such as repair of defects, may not be adequately covered by the costs included.

Contractor's Familiarity

There are few precedents that Sandwell is aware of in the world, with heads and discharges resembling Scheme 3. These precedents will be described in detail with references in the Preliminary Engineering Report, but to summarize here, it suffices to say that:

1. Systems with comparable heads (The Trans-Andean Oil Pipeline, Colombia, the Lornex water supply system, B.C.) are of smaller diameter pipe. Therefore the wall thicknesses are less than those for Scheme 3 and suitable pipe is available from more mills.

2. Systems with comparable heads and with similar or larger discharges (Lunersee, Austria; Tremorgio, Switzerland; Edmonston, California, U.S.A.) use tunnels rather than pipelines, thereby transferring some of the water pressure to the rock.

3. European hydroelectric power stations with high heads built about 1950 (Aussois, France; Dixence, Switzerland) utilized banded steel penstocks to reduce the wall thickness requiring welding. Such conduits are not in common use in North America.

4. Where similar and higher lifts have been encountered elsewhere, the procedure has generally been to break the head down by providing intermediate booster pumping stations (water supply to Caracas, Venezuela).

Therefore, a scheme such as Scheme 3 would have many unique features. Contractors may need to apply large contingencies to their bids to account for uncertain conditions they may encounter in construction.

It should be noted that in the case of Lornex, the use of two lifts rather than one was recommended in one study, and that in the case of the Edmonston Plant, earthquake and tunnelling considerations favoured the use of a single lift, although a two lift scheme was seriously contemplated.

Steel Quantity

For route C, with an 800 mm diameter pipeline, the total pipe tonnage for Scheme 1 or Scheme 2 would be about 5,000 metric tons, and for Scheme 3 7,000 metric tons. Historically, steel prices have been subject to large fluctuations. Therefore, there is an advantage in reducing the amount of steel required, as cost uncertainties are reduced somewhat.

Logistics

All schemes except Scheme 2 with equal lift booster stations may require some materials from foreign suppliers, thereby making communications and tendering more complex and deliveries longer than would be the case for Scheme 2. Further, shipping involves greater distances and heavier weights, pipe coating is more difficult due to the pipe weight, and more pipeline construction equipment, rated for heavier loads, is necessary. Therefore, Scheme 2 with equal lifts is preferable to the others from a logistics viewpoint.

Recommendations

Sandwell has received the following recommendations from various qualified parties:

1. Steel mills recommended reducing the pipe wall thickness in order to reduce welding problems.

2. Specialist consultants in waterhammer recommended:

   - Avoid extending into unknown technology. Quoting from the letter from Professor V.L. Streeter and E.B. Wylie of the University of Michigan, U.S.A., to Sandwell, 5 August 1977:

     "Basically we have some reservation and concern with the high-lift installation. It appears as if the single-lift proposal would be pressing current technology on many fronts, i.e. pump design, motor design, pipeline design, etc. Should problems arise, for example, at the acceptance testing stage the severity of the situation becomes extremely critical. These may be vibration problems, material defects, cavitation problems, or many conditions that cannot be predicted. Corrective measures should such a condition develop, are not likely to be obvious and would surely be costly. Consequently our attitude, although conservative, favours multi-station lower lift installations."

- Pump with two equal lifts. Quoting from the letter from John Parmakian, Consulting Engineer, Colorado, U.S.A., to Sandwell, 5 September 1977:

  "I suggest that the main high lift be broken down into two lifts with equal dynamic heads at the pumps. The reasons for this are:

  a. Greater safety in the design of the entire system.
  b. Lower pipe shell thicknesses and hence much more dependable welds in the pipeline.
  c. Lesser waterhammer controls required.
  d. Same pump design at both pumping stations."

3. Consultants in pipeline construction (Williams Brothers of Calgary) recommended:

   - Try to keep pump stations identical (Minutes of Meeting No. 6).

   - High pressure pipe, valves, flanges and fittings suitable for Scheme 3 are not common for the diameter contemplated.

4. B.C. Hydro's Gas Division recommended reducing the pipe wall thickness to make the pipeline construction closer to the usual assignments undertaken by pipeline contractors.

CONCLUSIONS

Table 7 summarizes scheme selection criteria:

Table 7 - Summary of Scheme Selection

Item	Scheme 1 High Lift Intake	Scheme 2 Equal Lift Boosters	Scheme 3 High Lift Booster
Cost	First	Third	First
Pump Wear	Third	First	First
Pipe Supply	Second	First	Third
Welding	Second	First	Third
Contractor Familiarity	Second	First	Third
Steel Quantity	First	First	Third
Logistics	Second	First	Third
Recommendations	Second	First	Third

Therefore, the only drawback to Scheme 2 is the 5% additional cost, whereas in every other respect it is superior to Schemes 1 and 3. Sandwell recommends that this additional cost is worthwhile in view of the advantages of Scheme 2.

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B.R. McConachy
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Project Engineer

(PM V4191/4)

APPENDIX 1

REFERENCES

APPENDIX 1 - REFERENCES

1. David Stephenson Pipeline Design for Water Engineers, No. 6 in series: "Developments in Water Science", advisory editor, V.T. Chow, Elsevier Scientific Publishing Company, Amsterdam, 1976, Pages 34 to 36.

2. Kally, Elisha "Pipeline Planning by Dynamic Computer Programming", J. AWWA, March 1969, Pages 114 to 118.

3. Stark, R.M. and Nicholl, R.L. Mathematical Foundation for Design of Civil Engineering Systems, McGraw-Hill, Pages 213 to 217.

APPENDIX 2

DETAILS OF COST ESTIMATES

(PM V4191/4)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/4
SYSTEM DESIGN

DATE 10 FEBRUARY 1978

APPENDIX 2 - DETAILS OF COST ESTIMATES

Item	Description	Cost ($1,000's) Scheme 1	Scheme 2	Scheme 3
	Structures			
1	River Intake	3,780	3,600	3,600
2	Power Supply and Distribution	*	*	*
3	Booster Stations	720	1,440	820
4	Pipeline	1,080	1,080	1,080
	Total	5,580	6,120	5,500
	Equipment			
1	River Intake	5,470	2,550	2,550
2	Power Supply and Distribution	1,860	2,050	1,470
3	Booster Stations			
	a. River	-	5,860	7,725
	b. Second	5,880	5,430	-
4	Pipeline	17,360	17,070	19,280
	Total	30,570	32,960	31,025
	Total Direct Cost	36,150	39,080	36,525
	Owner's Construction Overhead	3,000	3,300	3,025
	Engineering	3,600	3,870	3,600
	Contingencies	6,500	6,900	6,500
	Total	49,250	53,150	49,650
	Capitalized Power Cost	20,750	20,750	20,750
	Total Comparative Cost	70,000	73,900	70,400

Note: These estimates were made for route C, and include the cost of air chambers and for an open system configuration.

* Included in Booster Station(s).

APPENDIX 3

ROUTING AT ELEPHANT HILL

(PM V4191/4)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/4
SYSTEM DESIGN

DATE 10 FEBRUARY 1978

APPENDIX 3 - ROUTING AT ELEPHANT HILL

Introduction

This document records the background and findings of routing studies in the vicinity of Elephant Hill*. The findings were based on field investigations and office studies. Geotechnical and construction considerations, rather than detailed cost analysis, are foremost in the decision process used.

Three alternatives, as shown on drawing A4191/4-10 attached, were considered for routing in this area.

X - Surface pipeline route along Bonaparte River.

Y - Tunnel.

C - Surface pipeline route over Elephant Hill (Conceptual Design).

Letters X and Y were chosen to avoid confusion with route designations elsewhere in this Project Memorandum. C is a portion of route C as discussed in the main memorandum. A route was not developed which would pass to the south of Elephant Hill, in order to avoid crossing Indian Reservation land.

Bases of Comparison

The criteria for selection of a route are:

- The route must be free of geotechnical hazards.

- The route must be suitable for standard construction procedures.

- Capital and operating costs should be minimized.

X - Surface Route Along Bonaparte River

This route is threatened by rock slides in the northeast slopes of Elephant Hill A dramatic example of an old slide is shown on the accompanying photograph. In addition, there is little room for construction along the steep rock slope. Cutting into this slope would endanger the highway, workers, road traffic, and the pipeline once installed.

Therefore, geotechnical instabilities alone completely eliminate this route from further consideration.

* Elephant Hill is the proper name for the hill north of Ashcroft, identified as "unnamed ridge" in the Conceptual Design Report V4007/2.

Y - Tunnel

The tunnel as aligned in drawing A4191/4-10 is suitable for connecting the routes A, B and C with the intake location.

According to the 10 January 1975 report to B.C. Hydro by Dolmage, Campbell and Associates, and to Geological Survey of Canada mapping, Elephant Hill consists of Jurassic age sediments: conglomerate, sandstone and shale. These rocks are described in that report (p. 8) as "competent rocks of moderate hardness and should be quite suitable for boring". Due to the short length of tunnel to be driven, it is unlikely to be bored, however the suitability for tunnelling is assumed to hold true for the entire hill. However, variable and complex geology is expected to be encountered.

The tunnel would be about 2,150 m long, and constructed by conventional drill and blast techniques. As it would cost much less to carry the water within a pipe than to line the tunnel suitably to withstand internal pressure, a concrete or gunnite tunnel liner would be used and the pipeline would continue through the tunnel.

Prior to deciding to use a tunnel, a detailed drilling program would be essential to confirm its feasibility. Even so, there are always uncertainties with tunnelling, as poor rock can be encountered which would make progress slow.

In favourable conditions, a tunnelling rate of about 6 m per shift could be expected, so that at two working shifts per day, about a year would be needed to set up portals and complete the tunnel. Depending on the limitations of the construction schedule, this may present an unacceptable time constraint.

C - Surface Route Over Elephant Hill (Conceptual Design)

Route C passes well to the south of the low point in the saddle of the hill, as that point denotes a fault. Although rock excavation would be required for trenching along this route, and although special construction[1] methods would be required on the steep slopes of the hillside (in sections to 40 percent), this would provide a feasible crossing of the hill. Special measures to control erosion of the backfill material would be necessary, possibly even to the extent of using concrete bedding in sections.

Following this route does not add to the total static lift of the system, and thus does not require extra power.

Evaluation of Alternatives

In the main body of this Project Memorandum, the reasons for the choice of the selected pumping scheme and route are given. The cost difference of Y over C for this particular combination is given on Table 1.

1. The feasibility of pipeline construction over this route has been confirmed by consultation with a contractor, see Field Visit Report of 8 November 1977.

Table 1 - Extra Cost of Tunnel (Y) Over Surface Crossing (C) of Elephant Hill

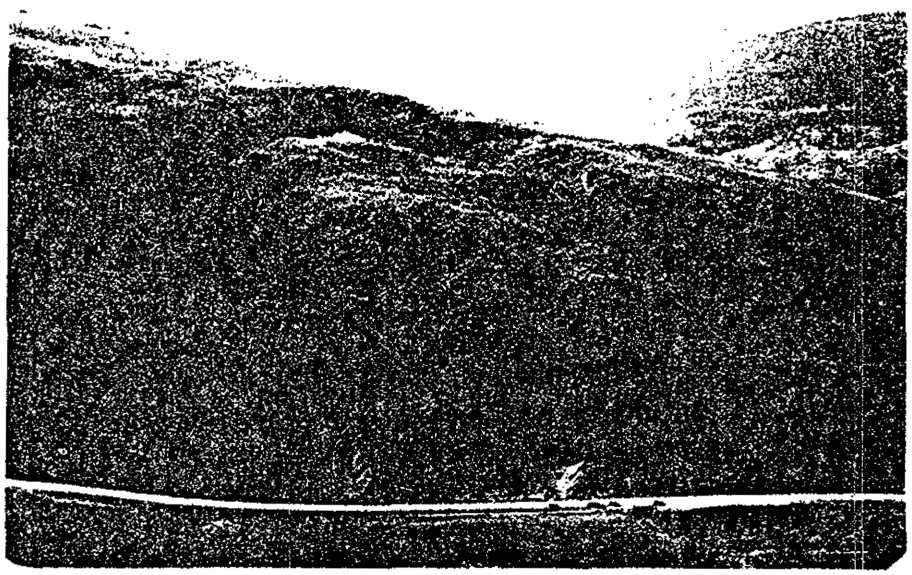
Item	Unit Cost	Units	Cost ($1,000's)
Drive and Line Tunnel	$ 2,000/m	2,150 m	$4,300
Install Pipe in Tunnel	325/m	2,150 m	700
Difference in Pipeline Length	850/m	-2,600 m	($2,250)
Elephant Hill Surge Tank [1]	350,000 ea.	1	(350)
Net Extra Cost of Y over C			$2,400

1. A one-way surge tank may be required to prevent water column separation at the summit of Elephant Hill.

Conclusions

Route C, the surface crossing over Elephant Hill, is feasible and is the least costly route for this section of pipeline.

A tunnel may be feasible, but involves uncertainties, scheduling difficulties, and extra cost of about $2.4 million for the selected system configuration.



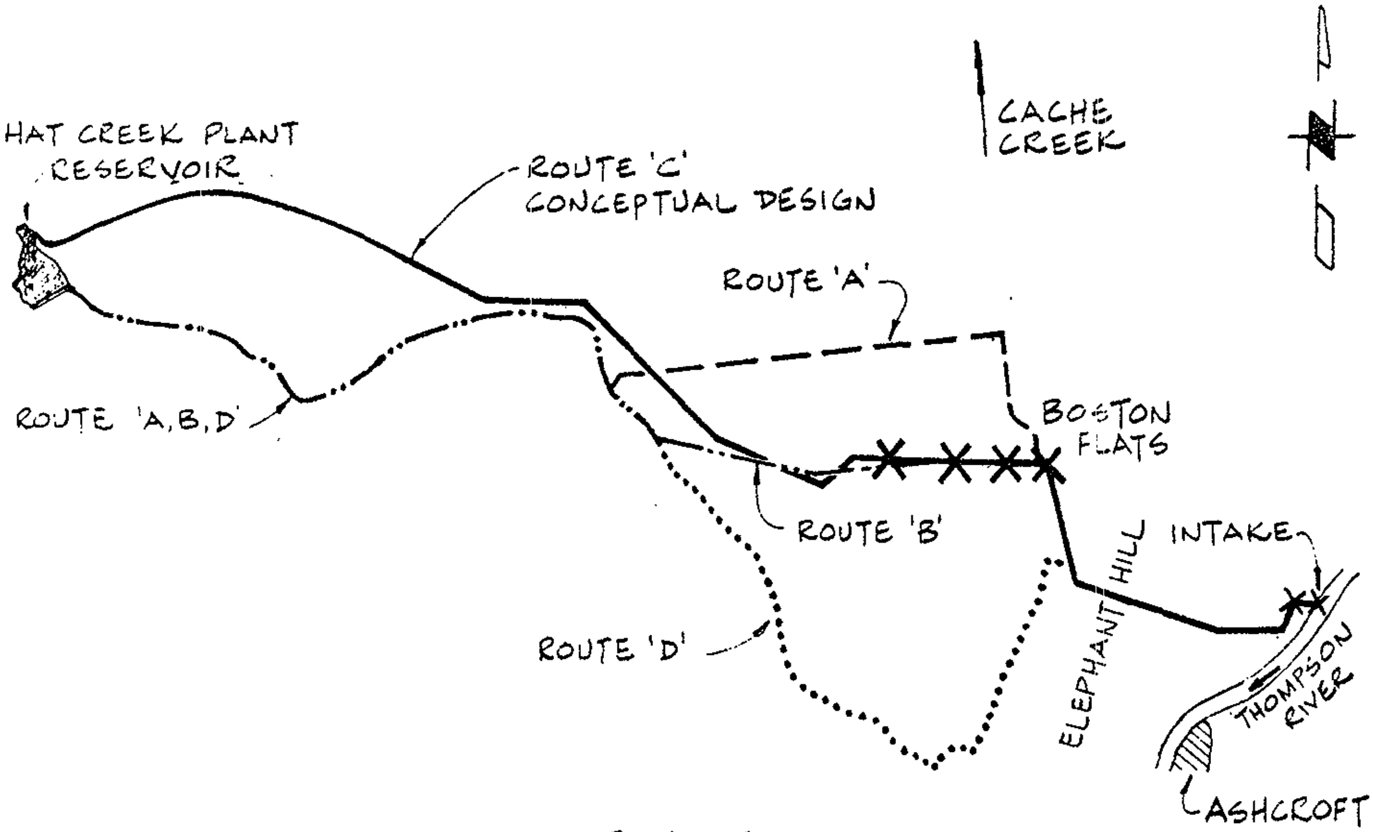
ANCIENT ROCKSLIDE IN ELEPHANT HILL

(North end)

PROJECT MEMORANDUM V4191/5

WATER TREATMENT

(V4191/1)



ROUTES

A — ALONG 500 KV TRANSMISSION LINE

B — (ALTERNATE)

C — CONCEPTUAL DESIGN

D — CORNWALL CREEK ACCESS ROAD

X — APPROXIMATE LOCATIONS STUDIED FOR POTENTIAL PUMPING STATIONS (FOR ROUTE 'C')

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

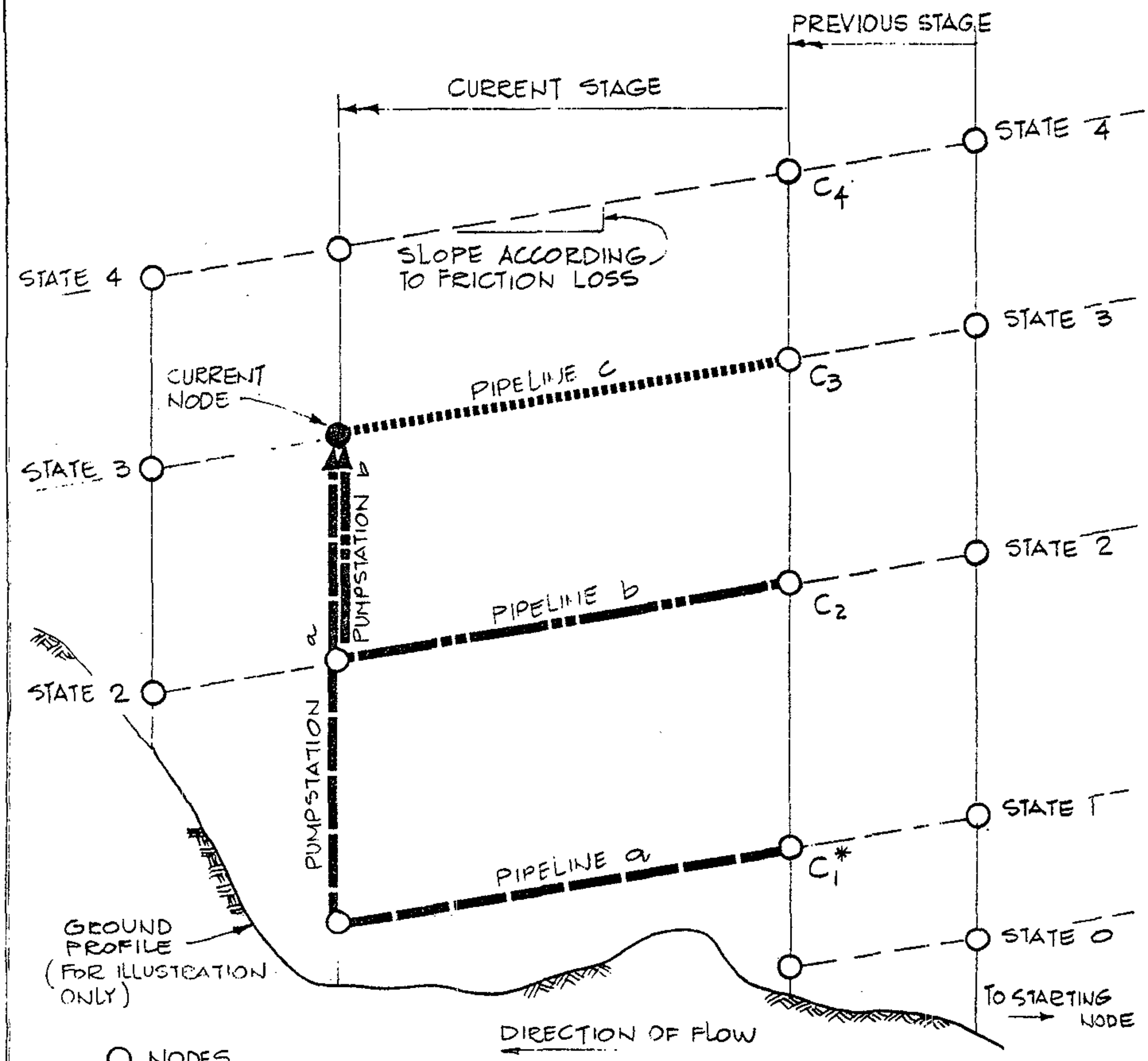
			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	30	11	77
	CH'K'D	APB			
A 4191/4-1	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
SYSTEM DESIGN — ROUTES

SANDWELL

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

DWG. A4191/4-1 REV



○ NODES

● CURRENT NODE

* NOTE: C_1 IS THE CUMULATIVE COST TO REACH THIS NODE FROM STARTING NODE.

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

			DAY	MO.	YR.
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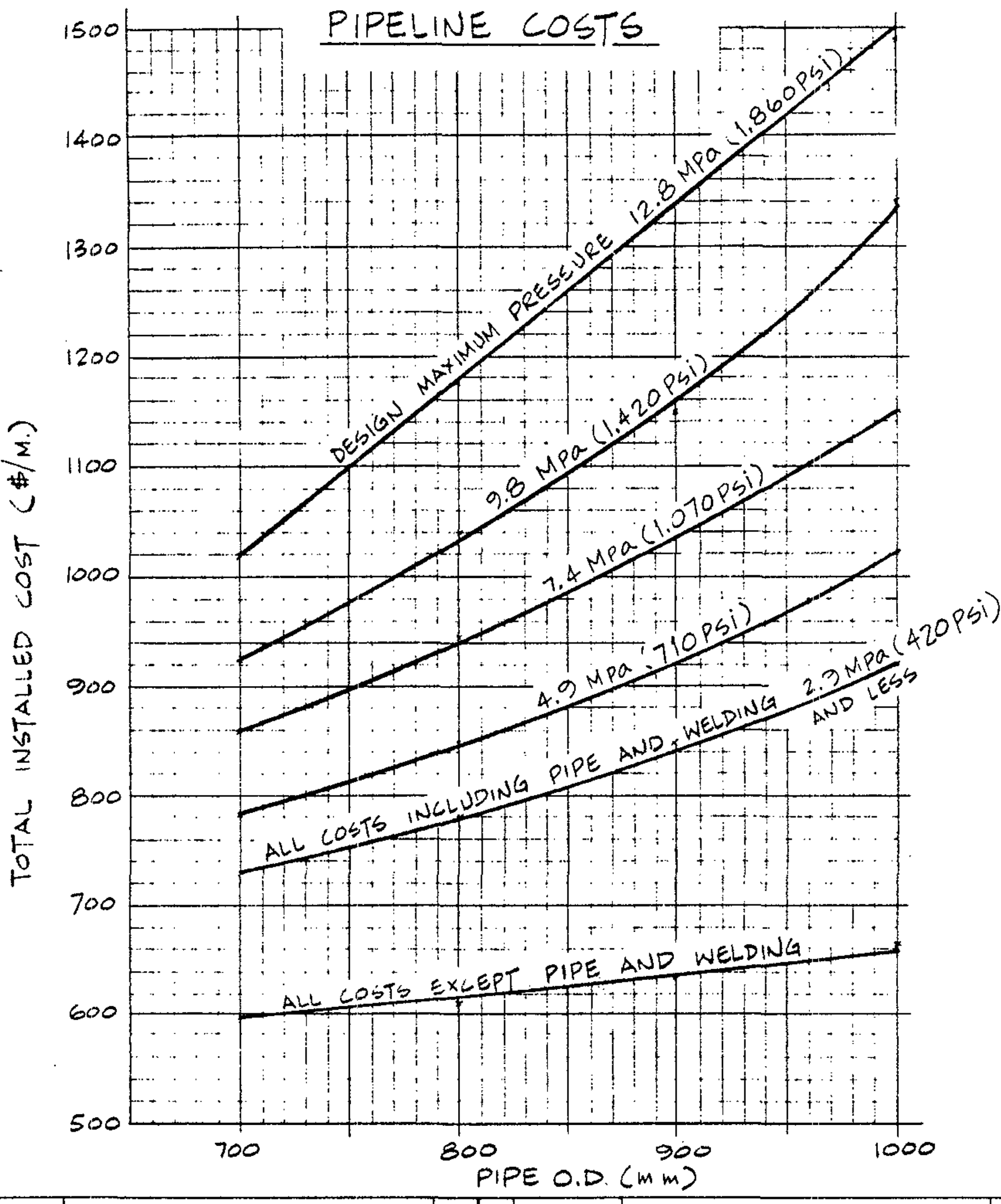
A 4191/4-2	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
OPTIMIZATION DIAGRAM

SANDWELL

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

DWG. A 4191/4-2 REV



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

APPROVED	SCALE		DAY	MO.	YR.
DATE	DR'N.	D. TUNG	30	11	77
	CH'K'D.	APB	10	2	78
A 4191/4-3	APP'D.				
	APP'D.				

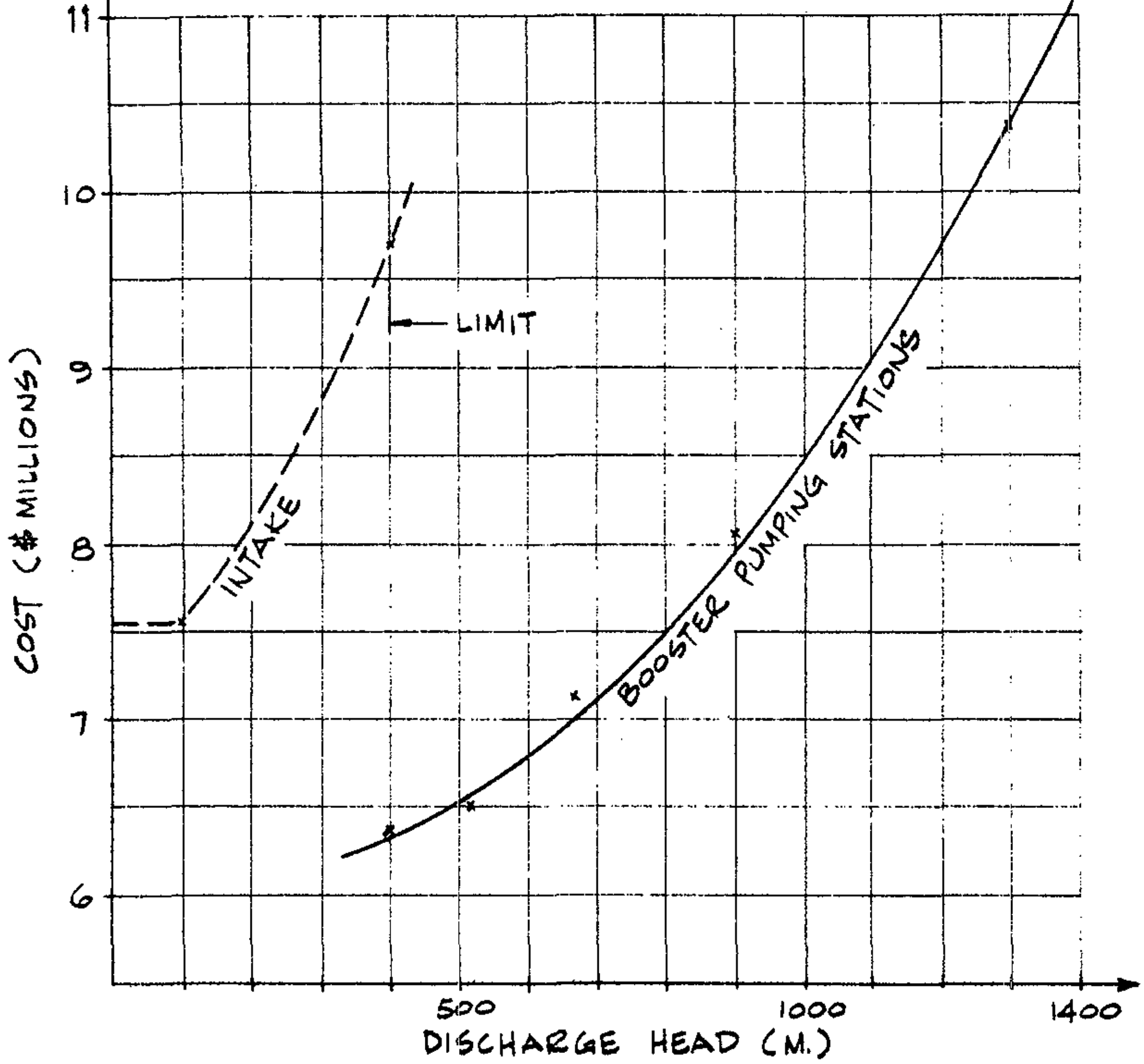
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
PIPELINE COSTS

SANDWELL

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

DWG. A4191/4-3 REV

PUMPING STATION COSTS
(LOCATION - INDEPENDENT PORTION)



NOTE: CLOSED SYSTEM

INCLUDES: STRUCTURES, PUMPS, PIPING, VALVES + FITTINGS, SUBSTATION, OPERATING COSTS, ENGINEERING, OWNER'S OVERHEAD, CONTINGENCY.

EXCLUDES: SITE-DEPENDENT COSTS, TRANSMISSION LINE, LAND, DRAIN LINES, SUCTION RESERVOIRS.

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

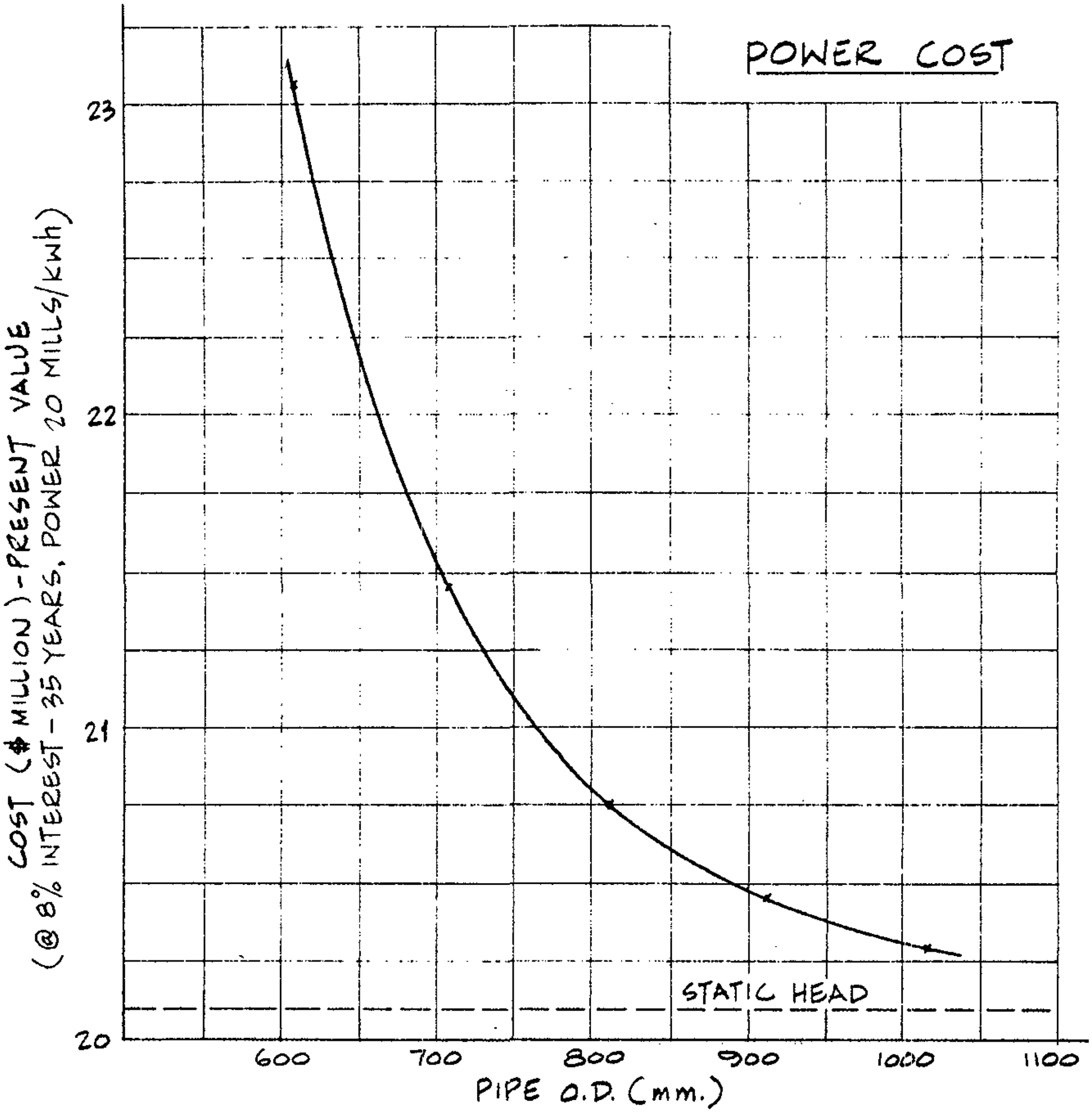
			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	30	11	77
	CH'K'D.	APB	10	2	78
A 4191/4-4	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
INTAKE AND PUMPING STATION COST

SANDWELL

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

DWG. A 4191/4-4 REV



NOTE:
CONTINUOUS DISCHARGE 640 l/s (10,130 USGPM) AGAINST TOTAL STATIC AND FRICTION HEAD.

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

			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	30	11	77
	CH'K'D.	APB	10	2	78
A 4191/4-5	APP'D.				
	APP'D.				

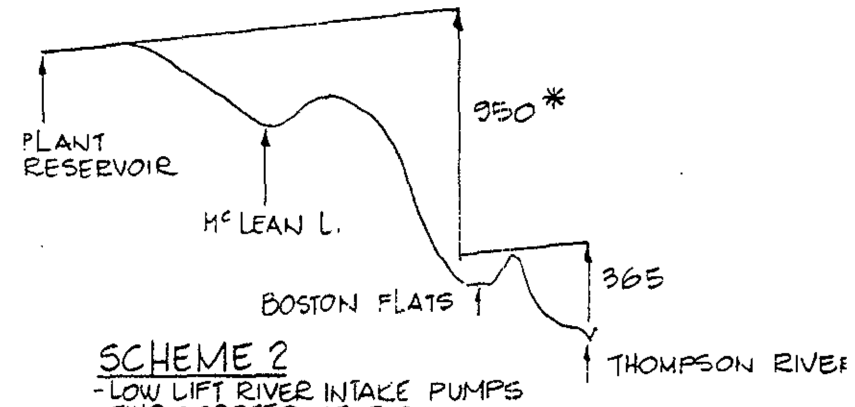
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
POWER COST

SANDWELL

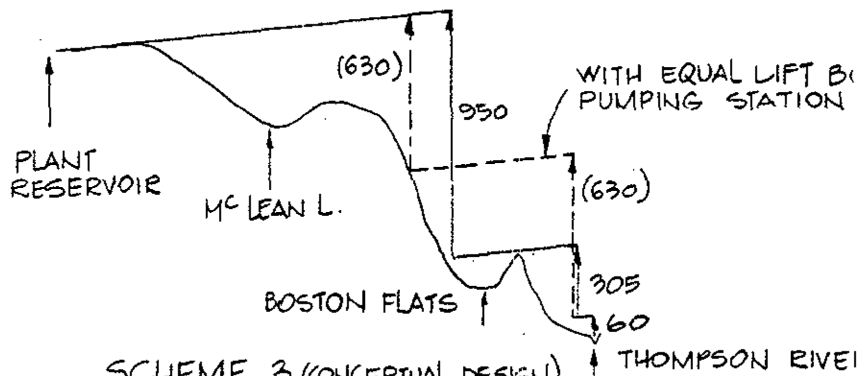
B.C. HYDRO AND POWER AUTHORITY
VANCOUVER B.C.

DWG. A4191/4-5 REV

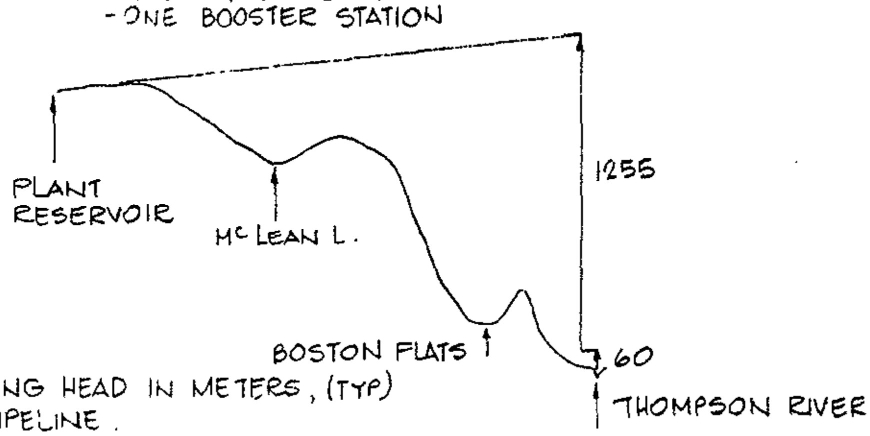
SCHEME 1
- HIGH LIFT RIVER INTAKE PUMPS
- ONE BOOSTER STATION



SCHEME 2
- LOW LIFT RIVER INTAKE PUMPS
- TWO BOOSTER STATIONS



SCHEME 3 (CONCEPTUAL DESIGN)
- LOW LIFT RIVER INTAKE PUMPS
- ONE BOOSTER STATION



* INDICATES PUMPING HEAD IN METERS, (TYP) FOR 800 mm D. PIPELINE.

NOTE: PROFILE IS FOR ROUTE C.

DWG. No.	REFERENCE	REF. No.	REV.	BY	DATE	REVISION

APPROVED

DATE

A 4191/4-6

		DAY	MO.	YR.
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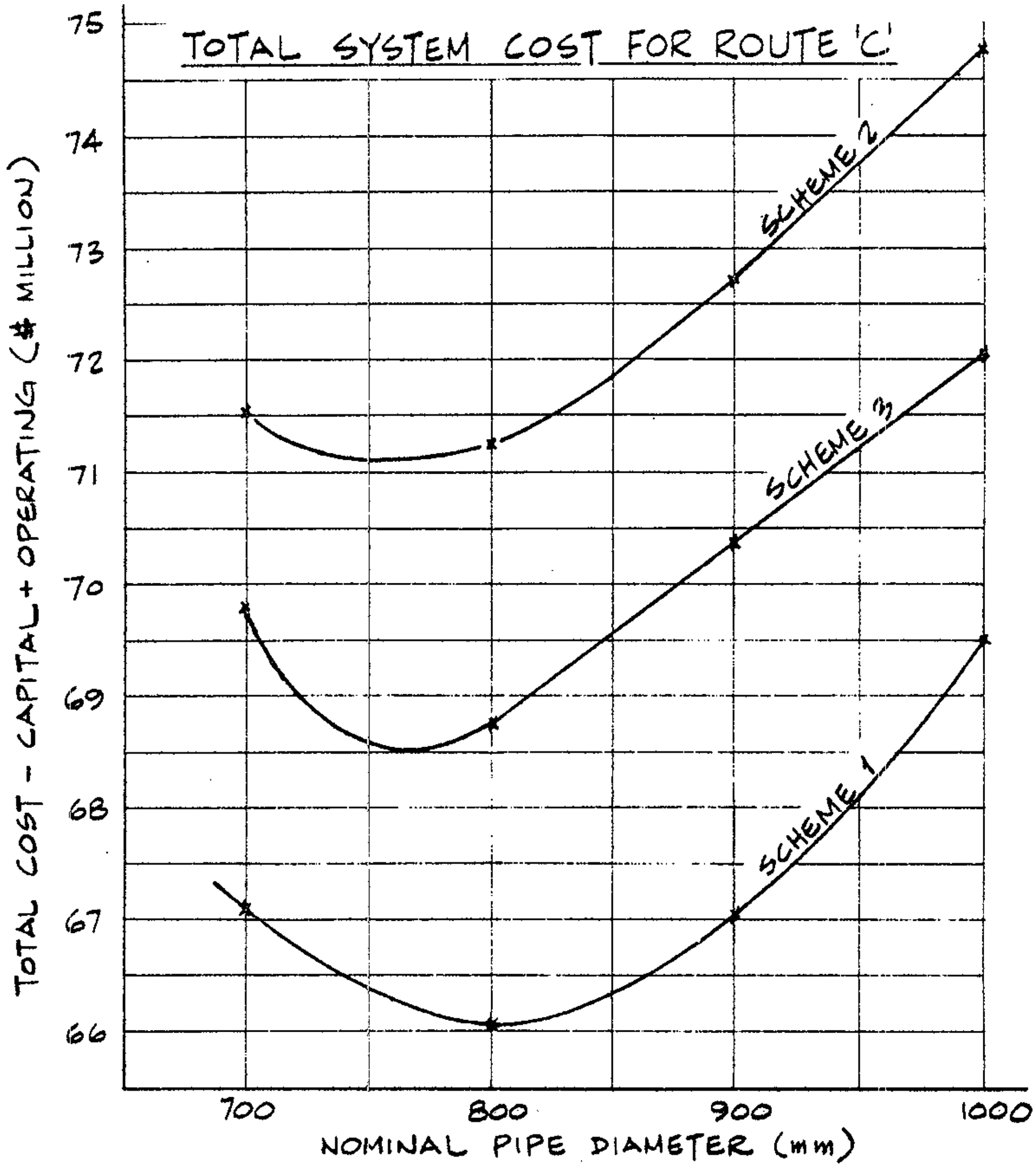
APP'D.				
APP'D.				

HAT CREEK PRO
COOLING WATER S
PRELIMINARY DE
PUMPING SCHEM

B.C. HYDRO AND POWER AUT
VANCOUVER

SANDWELL

DWG. A 4191/4-6



NOTES:
1. AIR CHAMBERS NOT INCLUDED.
2. CLOSED SYSTEM.

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

			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	30	11	77
	CH'K'D.	APB	10	2	78
A 4191/4-7	APP'D.				
	APP'D.				

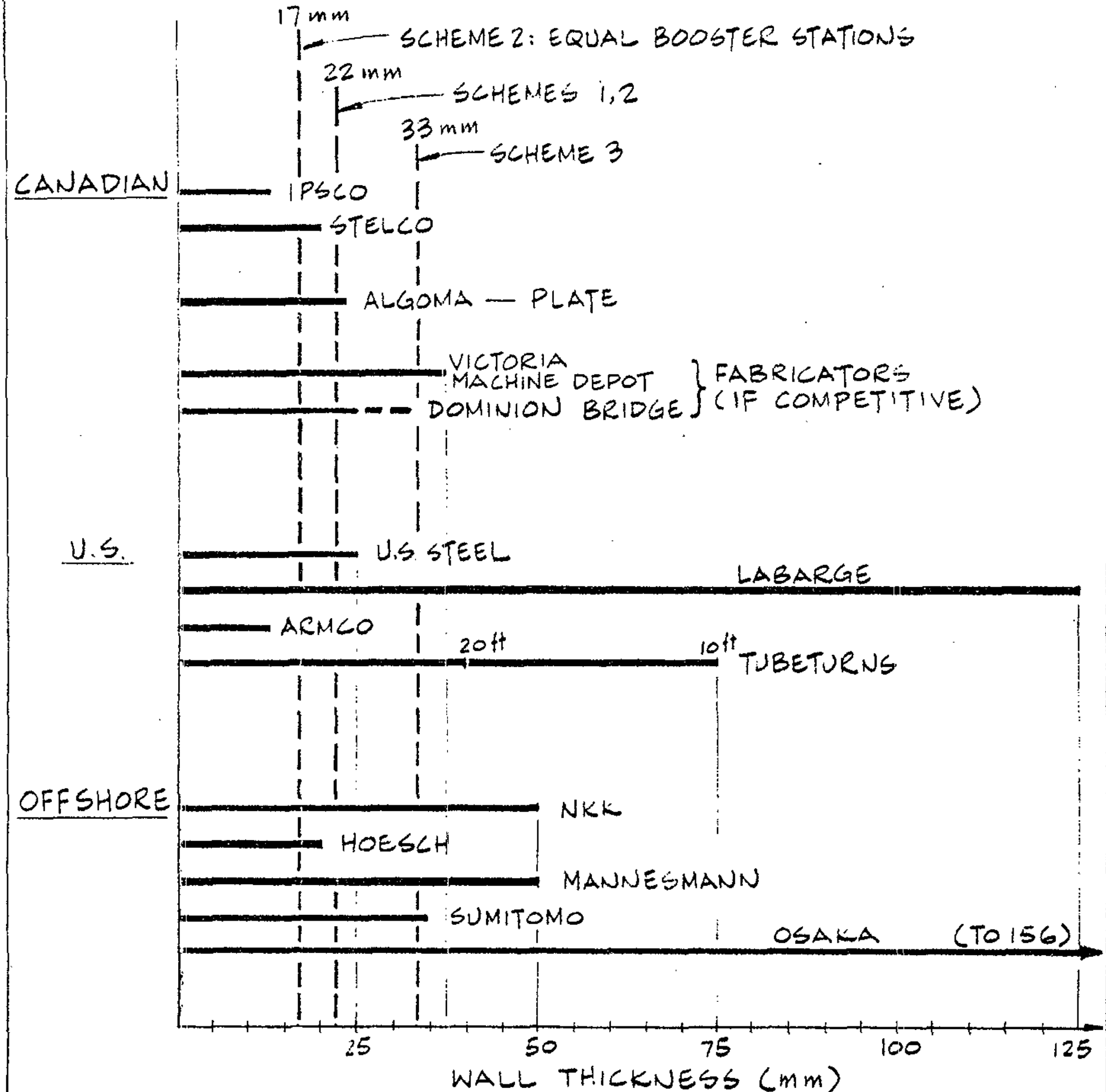
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
TOTAL COST VS. PIPE DIAMETER

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

SANDWELL

DWG. A4191/4-7 | REV

PIPE AVAILABILITY
FOR 800 MM DIA. X-56 STEEL



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

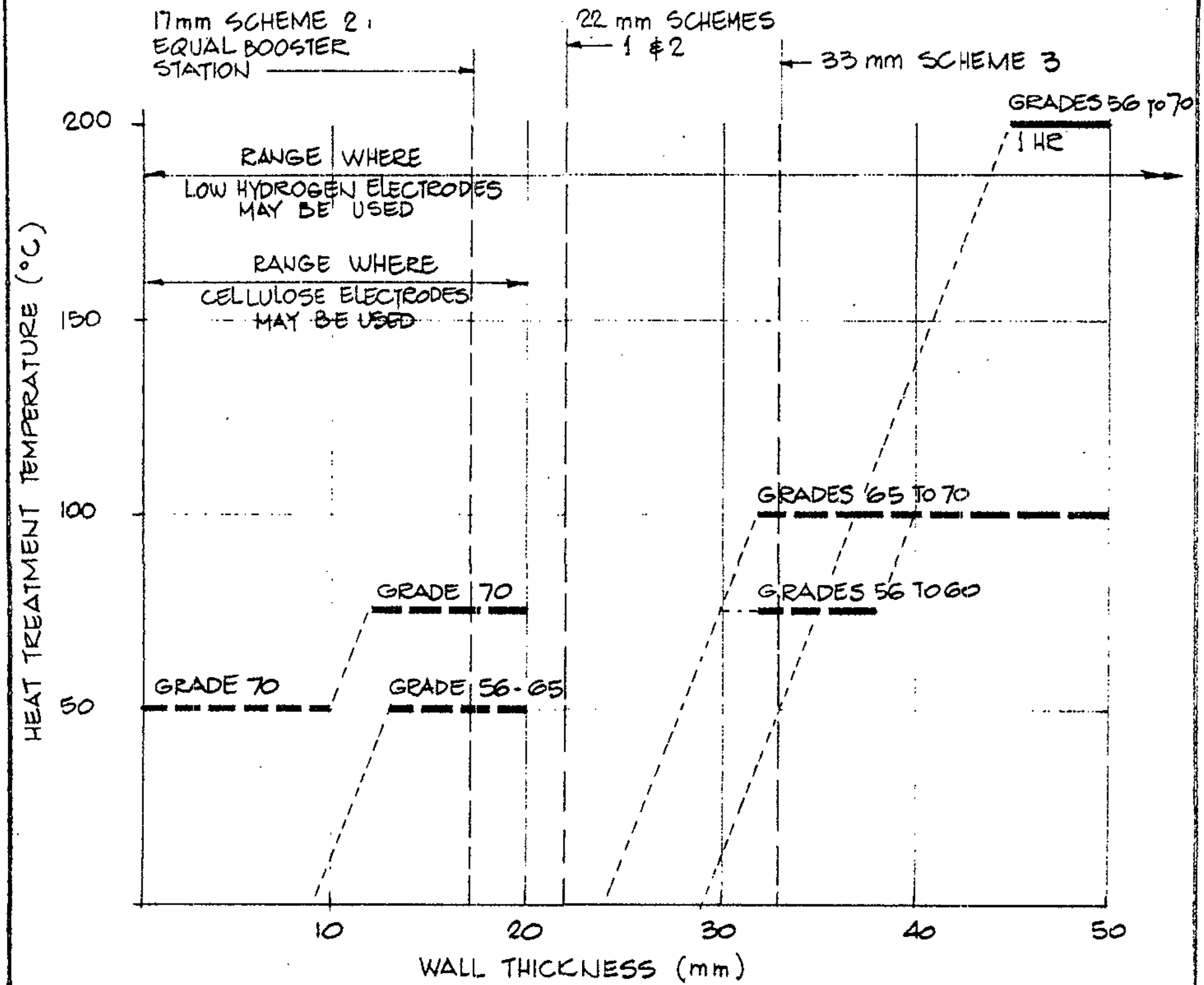
			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N	D. TUNG	30	11	77
	CH'K'D.	APB	10	2	78
A 4191/4-8	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
PIPE AVAILABILITY

SANDWELL

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

DWG. A4191/4-8 REV



* NOTE: THESE LINES ARE SIMPLY INTENDED TO INDICATE THAT HEAT TREATMENT REQUIREMENTS DECREASE AS WALL THICKNESS DECREASES. THEIR SLOPES HAVE NO SIGNIFICANCE

** INFORMATION FROM SUMITOMO METAL INDUSTRIES LTD.

LEGEND:

— — —	PRE-HEAT **
———	POST-HEAT **
- - - - - -	* EXTRAPOLATED

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

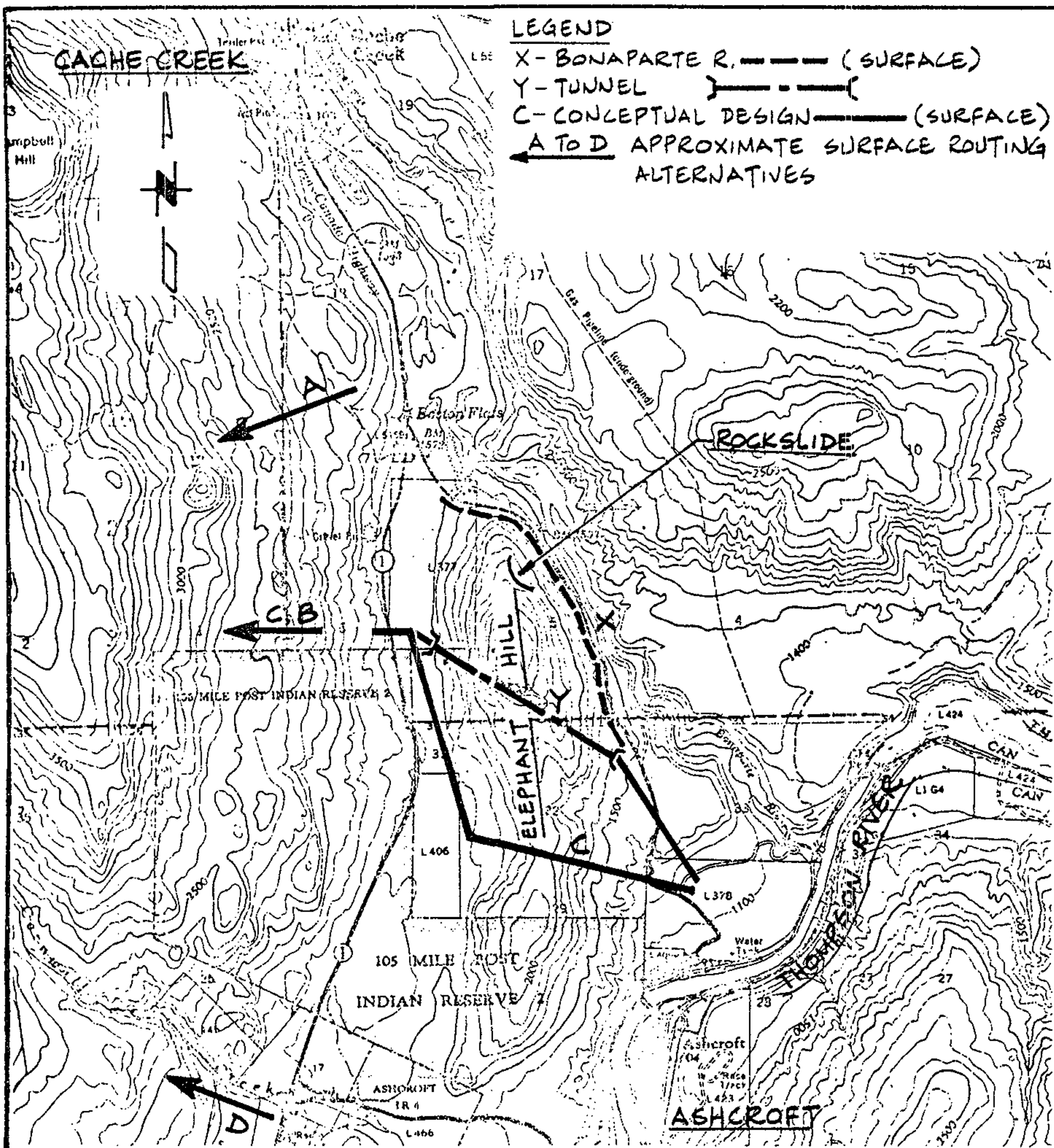
			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	SH			
	CH'K'D.	APB	10	2	78
A 4191/4-9	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
WELDING HEAT TREATMENT REQUIREMENTS

SANDWELL

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

DWG. A 4191/4-9 REV



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

APPROVED	SCALE	1:50,000		
		DAY	MO.	YR.
DATE	DR'N. D. TUNG	12	8	77
	CH'K'D. APP	10	2	78
A4191/4-10	APP'D.			
	APP'D.			

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
ALTERNATIVE ROUTINGS AT "ELEPHANT HILL"

SANDWELL

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

DWG. A4191/4-10 REV

PROJECT MEMORANDUM V4191/5

WATER TREATMENT

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 22 DECEMBER 1977

PROJECT MEMORANDUM V4191/5
WATER TREATMENT

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 5 October 1977 which responds to B.C. Hydro's letter of 12 September 1977 and answered the following questions.

Question 1 Whether or not conclusive information on sediment concentration could be obtained?

Question 2 Whether or not Sandwell would recommend a program to obtain data on sediment concentration?

Question 3 Sandwell's opinion on the possibility of delaying the decision to install the proposed grit removal system, until after the first period of operation?

RIVER SOLIDS

General

The amount and size distribution of solids which may be in suspension in a river is a complex function of many variables, some of which are river bottom roughness, turbulence, velocity, river bank erosion and input from slides and rainstorms. Under steady state conditions, equilibrium exists between the rate at which particles tend to fall under their own weight and the rate at which they are lifted through the mixing process of fluid turbulence. In a river, however, conditions are far from stable. Velocity and turbulence change due to variations of river cross section, river bottom gradient, discharge and river bottom roughness. Consequently, solids could be picked up at point A in a river and be deposited at point B further downstream. Under these conditions, the amount of solids in suspension upstream of A and downstream of B would be lower than those in suspension between A and B. Other factors which alter the amount of solids in suspension in a river are slides and eroding river banks.

From the foregoing it is evident that the amount of solids which may be in suspension in a river can vary a great deal at any one time.

Thompson River - Suspended Solids

The following tables with data on suspended solids are attached:

Table 1 Data from Beak Consultants Limited Report on Suspended Sediment Characteristics of the Thompson River, Appendix 18.

Table 2 Data from B. C. Hydro and Power Authority.

Table 3 Data from Northwest Hydraulic Consultants Limited, Hydrology Report of November 1976.

Of interest are the maximum suspended solid concentrations near Savona and Walhachin. These all are in the order of 10 mg/l. However, a much higher solids concentration of 91 mg/l was found on the right bank near the Ashcroft Bridge, see table 3. This higher figure can most likely be attributed to solids originating from the Bonaparte River and/or the eroding Ashcroft cliffs, see Drawing A4191/5-1.

The proposed intake location for the Hat Creek Project would be on the right bank of the Thompson River, 360 m (1,200 feet) upstream of the confluence with the Bonaparte River, see Drawing A4191/5-1. The suspended solids load at this point in the river is not known. It is expected, however, that the amount of suspended solids at the proposed intake site is much higher than those found at Walhachin and Savona. The reason for this is the presence of the Ashcroft Cliffs commencing only 900 m (3,000 feet) upstream of the intake site. Although the rate of erosion of these cliffs does not pose a threat to the operation of the intake, the erosion intrudes solids all year round. This introduction is expected to be at its highest during the freshet when rising water levels erode recent shore deposits from slides. Further introduction of solids takes place all year round when minor slides fall into the river, as illustrated on the enclosed photograph 5 - 7 taken from a helicopter on 15 September 1976. For location of slide, see Drawing A4191/5-1.

Whether the material from these slides would reach the river bottom upstream of the intake is not known as this would depend on many factors, such as distance of the slide to the intake, river turbulence, velocity, and grain size.

Based on the foregoing discussion on river solids, the answer to Question 1 is that in Sandwell's opinion, conclusive information on sediment concentration could indeed be obtained provided sampling of the Thompson River for suspended solids is carried out in the vicinity of the proposed intake location (that is, in the reach between the CNR bridge and the confluence with the Bonaparte River)

Thompson River - Settled Solids

Qualitative information was obtained on solids which had settled at the following locations:

- Thompson River Bank Opposite Ashcroft Cliffs
- Ashcroft Municipal Intake
- Lornex Intake

Sieve analyses on samples taken at the above locations were carried out only on particles passing No. 8 sieve, 2.36 mm (.093 inch). This sieve approximates most closely the maximum particle size passing through the intake travelling screens with stipulated maximum mesh opening of 2.54 mm (0.10 inch).

Solids from Thompson River Bank

On 15 June 1977, a solids sample was taken from a bar on the left river bank opposite the Ashcroft Cliffs (see Drawing A4191/5-1). The sieve analysis on this sample is shown in the attached Figure 1.

Solids from Ashcroft Municipal Intake

The municipality of Ashcroft operates an intake on the left bank of the Thompson River just downstream of the road bridge (see Drawing A4191/5-1). The intake consists of a pump well which is connected to the river by means of a 375 mm (15 inch) diameter buried pipe which protrudes approximately 0.50 m (1.6 feet) above the river bottom (see Drawing A4191/5-2). During the freshet, river solids settle out in the bottom of the pump well. These solids caused severe wear in the vertical turbine pumps until a program of periodic cleaning of the pump well during the freshet season was instituted. For this purpose, the pump well is equipped with an air lift. During a cleaning operation on 13 June 1977, Sandwell obtained 12 samples as described in Sandwell's Field Visit Report of that date. The maximum particle size encountered in the samples is in the order of 30 mm (1.2 inch). The sieve analyses are shown in the attached Figures 2 and 3.

Solids from Lornex Intake

Lornex Mining Corporation operates an intake on the left bank of the Thompson River approximately 21 km (13 miles) downstream of Ashcroft. The orientation of this intake in relation to the river shore is shown on Drawing A4191/5-3. On 4 August 1977, a sample was taken of solids carried up by the lifting lips on one of the travelling screens. The sieve analysis is shown in attached Figure 4.

Discussion of Sieve Analyses

Comparison of sieve analyses, see Table 4, reveals a striking resemblance in size distribution between the sample taken at the river bar opposite the Ashcroft Cliffs and those of the Ashcroft intake. The Lornex sample, on the other hand, indicates a much lower content in particles between 2.36 mm (0.093 inch) and 0.50 mm (0.020 inch).

Table 4 - Thompson River Solids
Particle Size Distribution in % of Dry Weight

Particle Size		River Bar	Ashcroft Intake		Lornex Intake
mm	inch		Sample #2	Sample #10	
2.36 - 1.00	.093 - .039	20	27	20	2
1.00 - 0.50	.039 - .020	34	31	46	5
0.50 - 0.30	.020 - .012	29	26	24	17
0.30 - 0.10	.012 - .004	14	14	9	56
< 0.10	< .004	3	2	1	20
2.36 - 0.30	.093 - .012	83	84	90	24
0.30 - 0.10	.093 - .004	14	14	9	56
< 0.10	< .004	3	2	1	20

The reason why the particles in the Lornex intake sample are much smaller than those in the Ashcroft intake sample is most likely due to the Lornex intake being set back from the river (see Drawing A4191/5-3) so that larger particles settle out before reaching the intake. The Ashcroft intake, on the other hand, most likely withdraws water from a zone high in suspended solids which have not had a chance to settle out before being drawn into the intake.

Solids Anticipated in Proposed Hat Creek Intake

The intake arrangement of the proposed Hat Creek intake is shown on Drawing D4191/5-4. This intake would withdraw water directly from the river before solids have had a chance to settle out and from a zone most likely high in suspended solids. Although this zone of water withdrawal would, on the average, be further above the bottom than that of the Ashcroft intake, it is considered very unlikely that the size distribution of particles (smaller than 2.54 mm (0.10 inch) anticipated in the Hat Creek intake, would be much different than those found in the Ashcroft intake.

WATER TREATMENT

Based on the foregoing, it is concluded that suspended solids will be taken in with the Thompson River water and in all likelihood in sufficient quantities to pose a threat to the reliability of operation of the cooling water supply system. The extent of this threat, however, would not be known until some time after commencement of pumping.

Since the principle of water withdrawal from the Thompson River by means of low-head intake pumps has been accepted by B. C. Hydro, water treatment to protect the high pressure pumps against wear can be added at some later date provided that adequate real estate has been set aside. The question is when and to what extent facilities for water treatment should be included. Alternative 1 on attached Drawing A4191/5-5 shows how a complete water treatment scheme is envisioned at this time. Water from the intake would flow through a degritting clarifier and a filter into the clearwell. Head loss through the clarifier and filter is in the order of three meters (10 feet). As this system limits the clearwell height, a special set of low-head pumps would be required at the clearwell to provide the necessary NPSH for the high pressure pumps. The aspect of high clearwell versus low clearwell with special NPSH pumps is discussed in Project Memorandum V4191/7, Appendix 8. The topography adjacent to intake site 10-D would accommodate a gravity flow system from clarifier to clearwell.

In British Columbia, water treatment clarifiers have to operate in winter. Ice forms on the walls and surface, but this does not interfere with the operation. The perimeter weir does not freeze provided the clarifier operates continuously. For the Hat Creek Project, allowance will have to be made for intermittent pumping to suit electrical load requirements. Therefore, to prevent unacceptable freezing of the clarifier during periods of shutdown, a dome has been added as shown on Drawing A4191/5-5.

The effluent from the degritting clarifier would contain particles in the range of 100 micron (0.004 inch) and smaller. These particles down to a range of 10 micron (0.004 inch) would be removed by the filters. In this process, both the degritting clarifier and filters would operate without the addition of chemicals.

As serious concern for pump wear due to grit commences with particles larger than 300 micron (0.01 inch), the degritting clarifier should provide adequate protection against wear. Therefore, filters could be left out as shown in Alternative 2, see Drawing A4191/5-5. However, the arrangement of this scheme would allow for a future inclusion of filters if found desirable based on actual operating experience.

A further simplification of the treatment process is shown in Alternative 3, which provides for the degritting clarifier vat only. The mechanism for removal of solids would be left for future installation if found desirable. As the vat would operate as a clearwell, the water level would fluctuate and the vat would, therefore, not operate as a degritting clarifier.

However, the vat would be located as in Alternative 2, so that a clearwell could be added if found desirable.

Alternative 4 does not provide for water treatment. However, the clearwell would be of the low head type equipped with NPSH pumps, so that this scheme could easily be converted to any one of the previous alternatives.

Of the four alternatives, the first one can be discarded as it is considered unlikely that this scheme, namely filters, would be required. To assist in choosing from the remaining three alternatives, capital cost differences for these were prepared, as shown in Table 5.

Table 5 - Water Treatment

Capital Cost Differences for Alternatives 2 through 4
Drawing A4191/5-5

Alternative 2	Installed	Spare Parts	Total
Degritting clarifier vat	$150,000		
Piping	105,000		
Domed cover	125,000		
Clarifier mechanism	115,000		
Clear well	75,000		
Total	$570,000	-	$570,000
Alternative 3			
Degritting clarifier vat	$150,000		
Piping	105,000		
Domed cover	125,000		
Four spare pump rotating assemblies		400,000	
Total	$380,000	$400,000	$780,000
Alternative 4			
Clear well	$ 75,000		
Four spare pump rotating assemblies		400,000	
Total	$ 75,000	$400,000	$475,000

The delivery time of pump rotating assemblies to replace worn units is in the order of 18 months. Because of this long delivery and because Alternatives 3 and 4 do not provide adequate protection against pump wear, these two alternatives would require the stocking of pump rotating assemblies in order to safeguard the thermal power plant against a shutdown of long duration. The present concept envisages eight identical pumps, four per pumping station. As the average cooling water requirement is in the order of 50 percent of the installed capacity, Table 5 contains an allowance for four spare pump rotating assemblies to guarantee the average requirement.

The pros and cons of Alternatives 2 through 4 are given below:

Alternative 2: Degritting Clarifier and Clearwell

Advantages

1. Offers full protection against pump wear due to grit.

Disadvantages

1. Higher in capital costs than Alternative 4, however, only by $95,000.

Alternative 3: Degritting Clarifier Vat Only

Advantages

1. Some protection against pump wear.
2. Offers some provision for removal of solids.
3. Can easily be converted to Alternative 2.

Disadvantages

1. Reliability of protection against pump wear is still in doubt.
2. Highest in capital cost.

Alternative 4: Clearwell Only

Advantages

1. Lowest in capital cost. This initial advantage, however, would be lost if eventually impeller maintenance due to pump wear would be required.

Disadvantages

1. No protection against pump wear.
2. No provisions for removal of solids from the clearwell.

Of these three alternatives, Alternative 3 can be discarded as it is the most expensive of these three schemes. Of the remaining two alternatives, Sandwell recommends Alternative 2 as it is more reliable to supply cooling water on a continuous basis and as it is most likely more economical in the long run than Alternative 4, owing to the absence of possible maintenance cost due to grit.

As mentioned before, sampling for suspended solids could be carried out in the reach of the Thompson River between the CNR bridge and the confluence with the Bonaparte River. In reply to Question 2, Sandwell does not recommend such a program for the following reasons:

1. A suspended solids sampling program would be very costly. Because of the high forces involved during sampling, a boat is not considered suitable. The only available access to the river would be from the CNR bridge. Sampling from this bridge, however, could only be carried out from special platforms which would have to be attached to the bridge, subject to CNR's approval.

   For a sampling program to be reliable, staff would have to be available on short notice in order to sample immediately after rainstorms when solids loading, reportedly, are very high.

   The collection of conclusive data on the influence of the erosion from the Ashcroft Cliffs would be a study in itself - very time consuming and costly.

2. In order to obtain adequate data, sampling would have to take place during various freshets. This, however, would still not assure that sampling would be carried out during freshets with high river discharges.

3. Even if data were obtained during various freshet seasons and during very high river discharges, the possibility exists, although remote, that changes in the Thompson River and developments in the catchment area between the proposed intake site and Kamloops Lake (mining, logging, etc.) would increase the suspended solids loading.

4. Even if reliable data were available, the question as to whether the known solids concentrations would actually pose a threat to the pumps might remain unanswered.

5. As the inclusion of a degritting system would be simple from a system configuration point of view and provide good insurance against pump wear at a relatively low premium of $95,000 it does not appear justified to carry out an expensive sampling program, the success of which would not be guaranteed and the long term value of which would remain doubtful.

In answer to Question 3, Sandwell does not recommend delaying the decision to install a grit removal system until after the first period of operation for the following reasons:

1. The first period of operation could be during a year with low river water discharges and therefore would be inconclusive.

2. Whether or not a grit removal system would be required after the first period of operation could only be ascertained by taking one or more pumps apart to inspect for wear - an expensive operation in itself.

3. The absence of a grit removal system would necessitate adding pump rotating assemblies to the spare parts inventory because of long delivery in the order of eighteeen months. On the other hand, the immediate installation of a grit removal system would afford more reliability and would in the long run, most likely, be more economical.

Prepared by ______________________
A. Copeland, P. Eng.

Approved by ______________________
B. McConachy, P. Eng.
Project Engineer

TABLE 1 Suspended Sediment Concentrations Observed in the Thompson River in the Vicinity of Ashcroft, British Columbia

From Beak Consultants Ltd., report on Suspended Sediment Characteristics of the Thompson River.

	Date	Source	Location	Concentration (mg/l)	Discharge* m^3/s	Discharge* CFS
Jan.						
Feb.	23/02/77	4	Walhachin Bridge	< 1.0-2.0	No Data	
March	17/03/77	3	1.0 km above Bonaparte	3.0		
	17/03/77	4	Walhachin Bridge	1.0-2.0	190	6800
April						
May	1973	2	Near Ashcroft	< 1.0	No Data	
	23/05/77	3	1.0 km above Bonaparte	4.0	1130	40,000
	15/05/76	5	Walhachin Bridge	9.0-13.0	2250	79,500 *
	20/05/76	5	Walhachin Bridge	7.0-17.0	2115	74,700 *
	15/05/76	5	Ashcroft Bridge	16.0-91.0	2250	79,500 *
June	19/06/72	1	Near Savona	9.0	No Data	
	02/06/77	4	Walhachin Bridge	2.0	1030	36,400
July						
Aug.						
Sept.	18/09/76	3	1.0 km above Bonaparte	2.0	1210	42,600
Oct.						
Nov.	18/11/71	1	Near Savona	< 1.0	No Data	
Dec.	06/12/76	3	1.0 km above Bonaparte	1.0	1540	54,300

SOURCES

1. Pollution Control Branch, B.C. Water Resources Service as Cited in B.C. Research and Dolmage and Campbell (1975)
2. Beak Consultants Limited (1973)
3. Beak Consultants Limited (Quarterly Report (1977b)
4. 1977 BEAK Survey (Range Observed at Three Transect Points)

5. Northwest Hydraulic Consultants Ltd. (1976)

* Flows estimated for above the Bonaparte except those referenced by NHCL which are given at Spences Bridge.

Note: This table does not represent the entire data base on suspended sediments in the Thompson River

TABLE 2

B.C. HYDRO & POWER AUTHORITY

HAT CREEK PROJECT

SUMMARY OF WATER QUALITY ANALYSES - THOMPSON RIVER

LOCATION WATER SOURCE	Position in Thompson River Not Stated			SAVONA, B.C. Station No. 0600004		
PERIOD OF RECORD	Dec. 1974-Oct. 1975			Jan. 1971-June 1976		
AGENCY	CALGON CORPORATION			POLLUTION CONTROL BOARD		
PARAMETER(1)	Minimum	Average	Maximum	Minimum	Average	Maximum
Total Dissolved Solids	73		109		57.4(2)	72.0(2)
Total Solids	75		115		60.4(2)	74.0(2)
Suspended Solids	2		6		3.1(2)	7.6(2)
Turbidity (JTU)	3		8		1.8(2)	8.5(2)
Specific Conductance (μmhos/cm)	78		117		98(2)	224 (2)
Oil & Grease					<1.0(2)	2.0(2)
pH (units)	7.1(3)		7.6(3)		7.5(2)	8.6(2)
Alkalinity ($CaCO_3$)					35.1(2)	44.8(2)
Hardness ($CaCO_3$)	28		42		38.2(2)	47.6(2)
Calcium (dissolved)	8		13		12.1(2)	14.6(2)
Magnesium (dissolved)	1.3		2		1.9(2)	2.6(2)
Chloride	1		7		1.5(2)	3.1(2)
Sulphate	7		14		7.2(2)	10.0(2)
Silica (SiO_2)	3		6		4.8(2)	6.5(2)
Colloidal Silica	0.016		2.1		-	-
Nitrate - Nitrogen	0.1(4)		0.2(4)		0.09(2)	0.22(2)

(1) All parameters expressed in mg/1 unless otherwise noted
(2) Average values represent monthly annual averages, all other parameters represent total sample averages

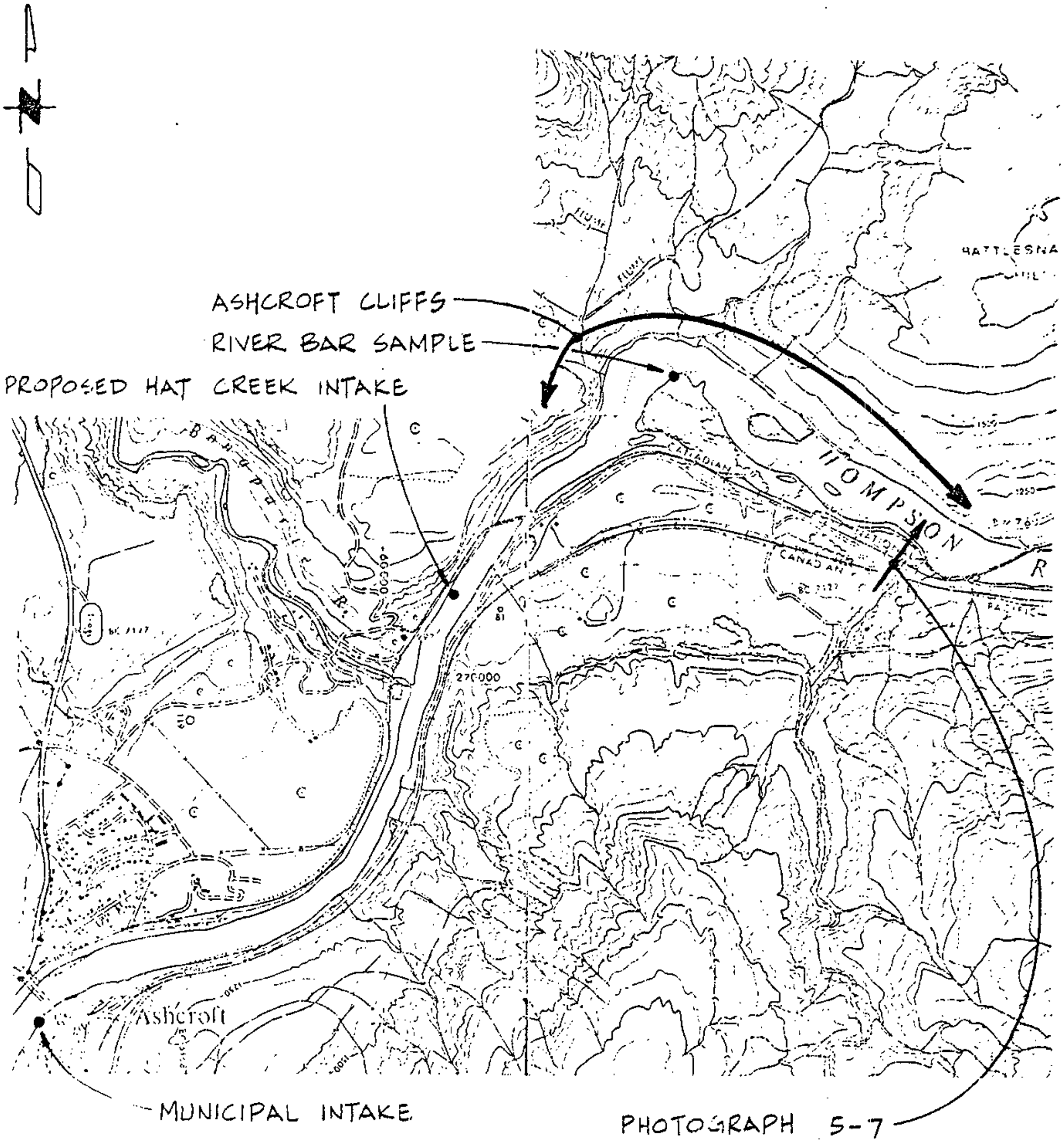
TABLE 3. THOMPSON BASIN SEDIMENT CONCENTRATIONS

FROM

NORTHWEST HYDRAULIC CONSULTANTS LTD., HYDROLOGY REPORT OF NOVEMBER 1976

Site	Location	Sample	14 May 1976		15 May 1976		20 May 1976		2 June 1976	
			Conc. (mg/l)	Discharge (cfs)	Conc. (mg/l)	Discharge (cfs)	Conc. (mg/l)	Discharge (cfs)	Conc. (mg/l)	Discharge (cfs)
Hat Creek Below Gauge	Mid Channel	1	64	42	12	41				
		2	18		10					
Bonaparte River below Cache Creek	Left ¼ Channel	1		1140*		975*	123	850*		
		2								
	Mid Channel	1	151		165		135			
		2	100		147					
	Right ¼ Channel	1					119			
		2								
Thompson River at Walhachin Bridge	Vertical 1	1			10	79,500**	8	74,700**		
		2			10		7			
	Vertical 2	1			9		17			
		2			10		10			
	Vertical 3	1			13		9			
		2			10		10			
	Vertical 4	1			13		8			
		2			9		9			
Thompson River at Ashcroft Bridge	Left Bank	1			24	79,500**	22	74,700**		
		2			16		16			71,000**
	Vertical 1	1							5	
		2								
	Vertical 2	1							8	
		2								
	Vertical 3	1							11	
		2								
	Vertical 4	1							6	
		2								
	Right Bank	1			91				8	
		2			26					

*WSC Unverified Estimate
**Discharge at Spences Bridge



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

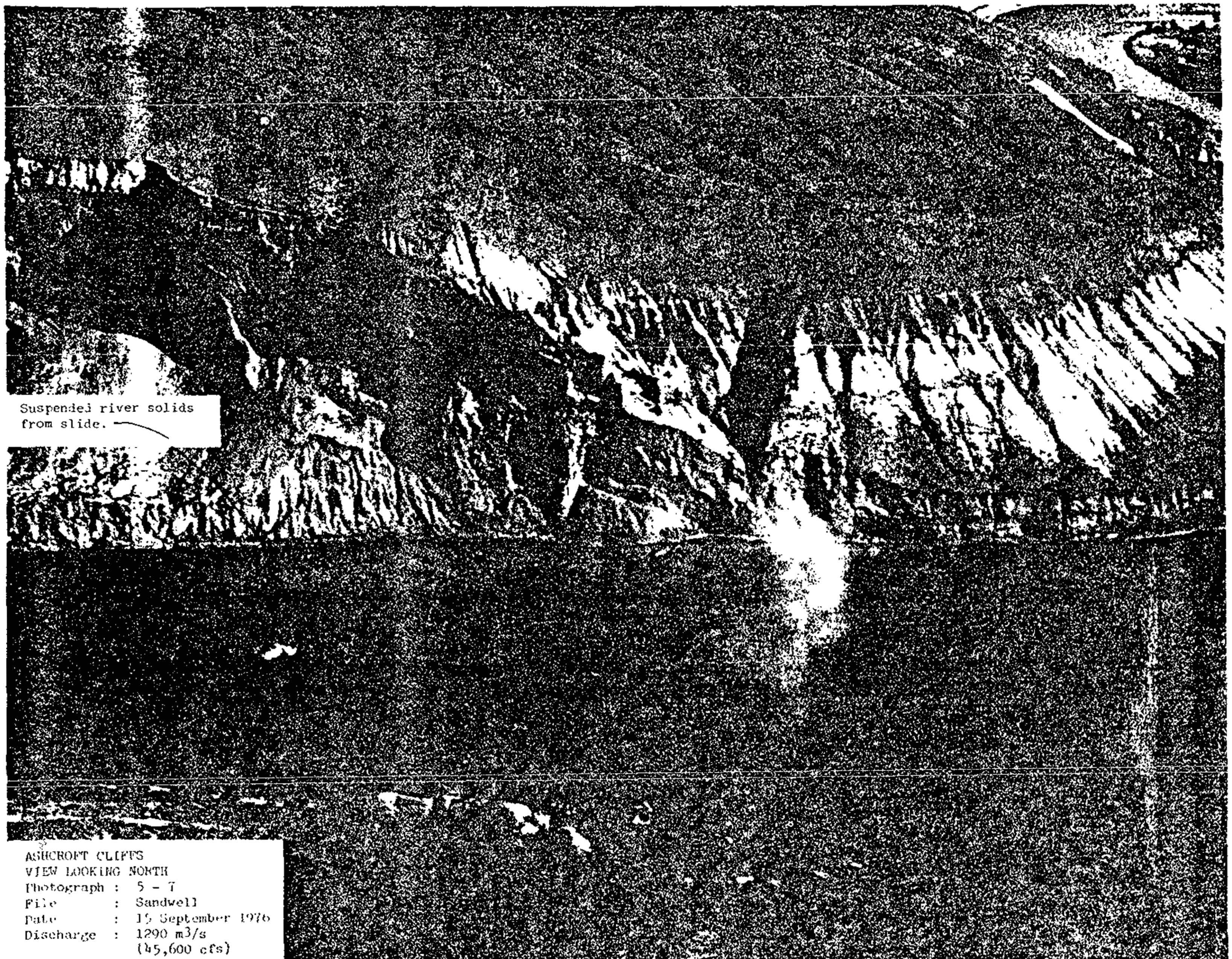
			DAY	NO.	YR.
APPROVED	SCALE	1:25,000			
DATE	DR'N.	D. TUNG	22	9	77
	CH'K'D.	A.C.			
A 4191/5-1	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
THOMPSON RIVER
ASHCROFT INTAKE & ASHCROFT CLIFFS

SANDWELL

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

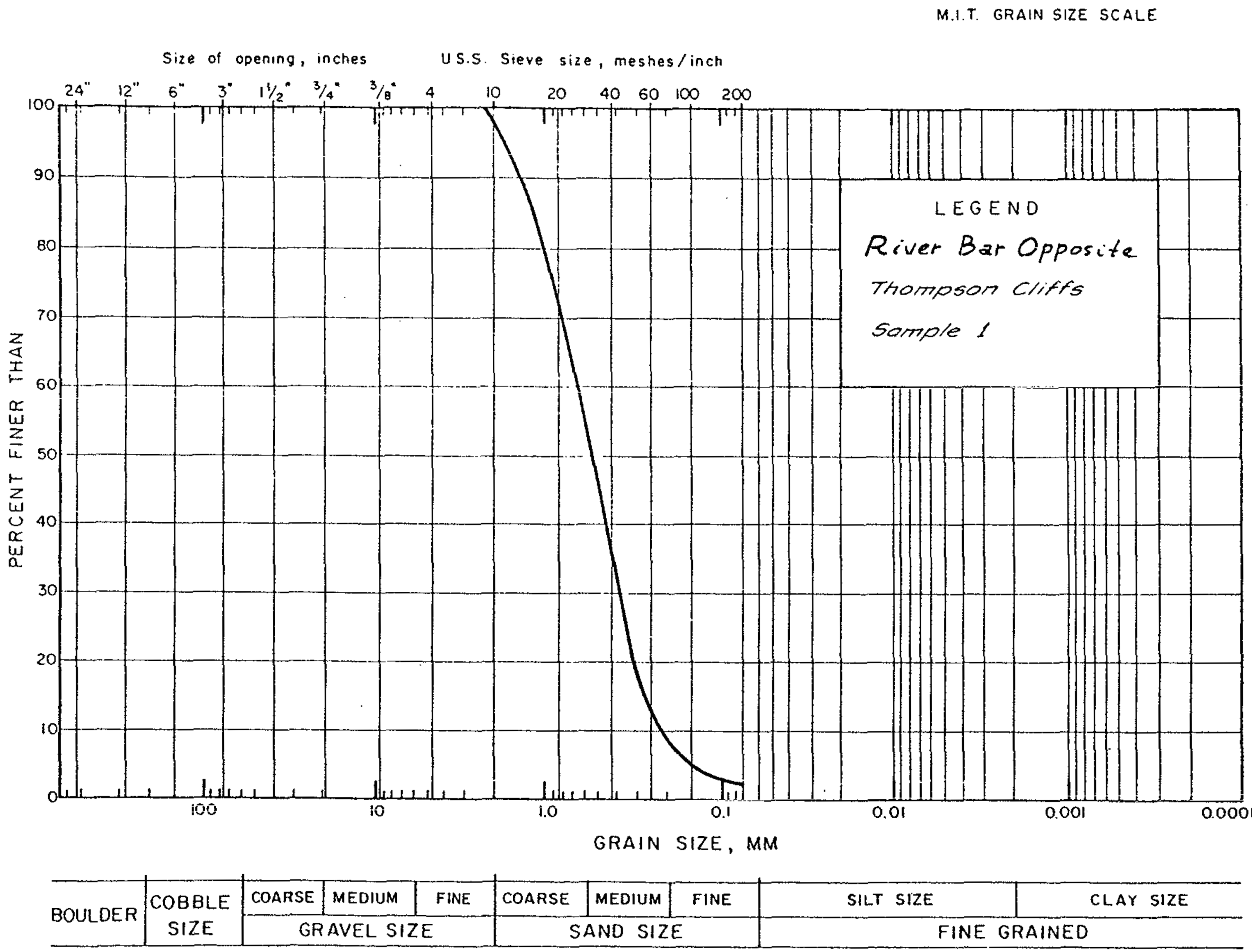
DWG. A 4191/5-1 REV



ASHCROFT CLIFFS
VIEW LOOKING NORTH
Photograph : 5 - 7
File : Sandwell
Date : 15 September 1976
Discharge : 1290 m3/s
(45,600 cfs)

SANDWELL

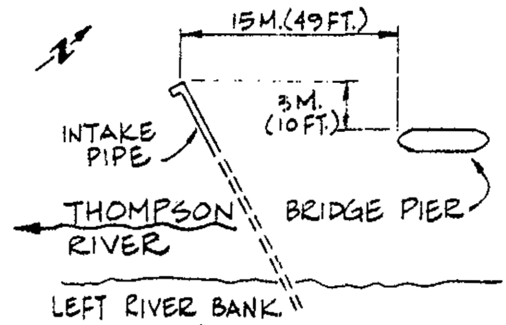
V77047 A.T. NAS Sept '77



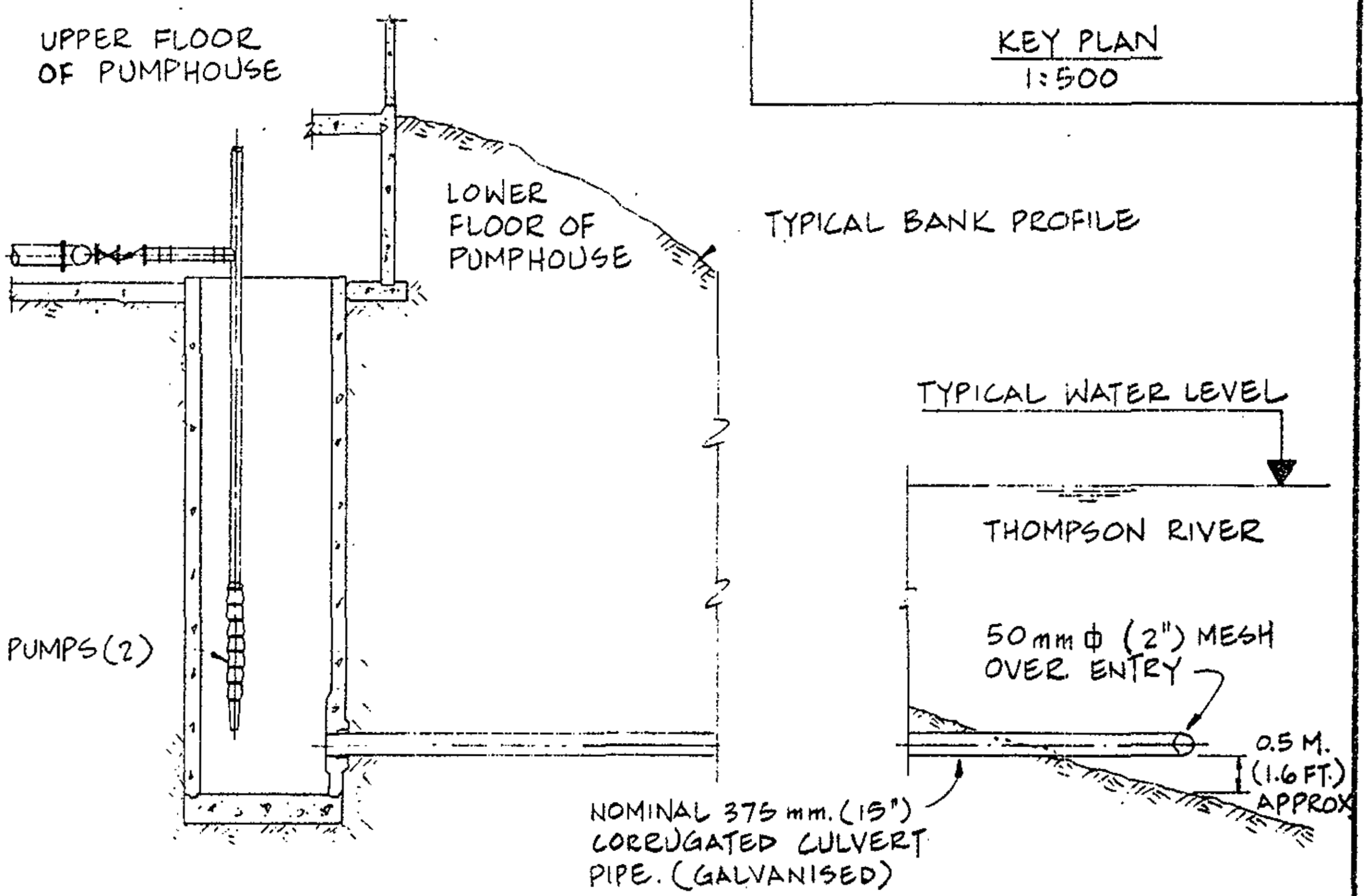
Golder Associates

GRAIN SIZE DISTRIBUTION
HAT CREEK WATER SUPPLY
THOMPSON RIVER

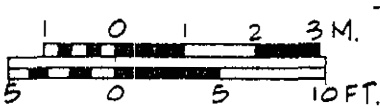
Figure 1



KEY PLAN
1:500



SECTION THROUGH ASHCROFT MUNICIPAL INTAKE
LOOKING DOWNSTREAM



1:70

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

APPROVED

DATE

		DAY	MO.	YR.
SCALE	AS NOTED			
DR'N.	D.TUNG			
CH'K'D.	A.C.			
APP'D.				
APP'D.				

A 4191/5-2

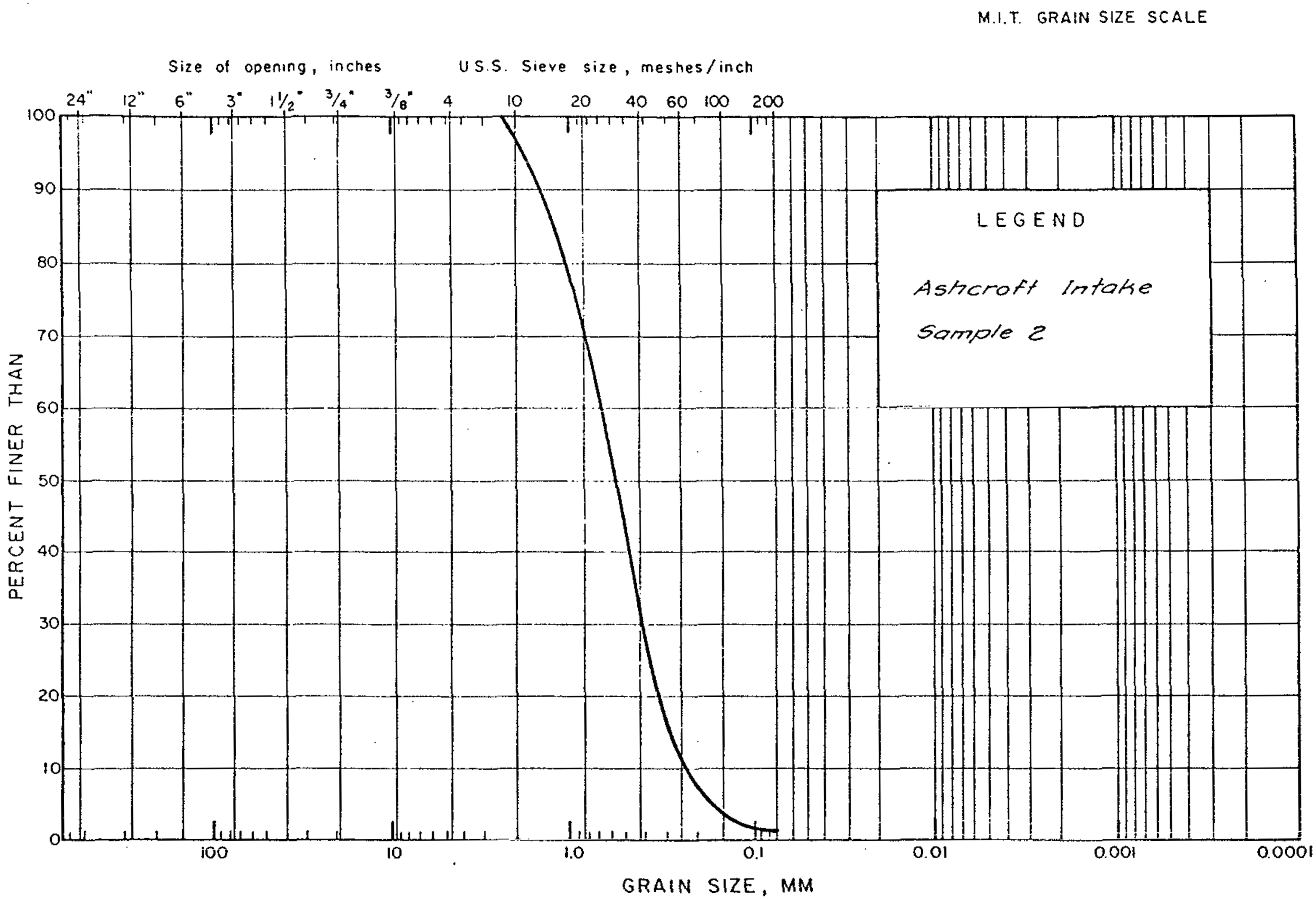
HAT CREEK PROJECT
COOLING WATER SUPPLY
EXISTING THOMPSON RIVER INTAKES
ASHCROFT MUNICIPAL INTAKE

SANDWELL

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

DWG. A4191/5-2 REV

Y77047 A.t HAS Sept '77



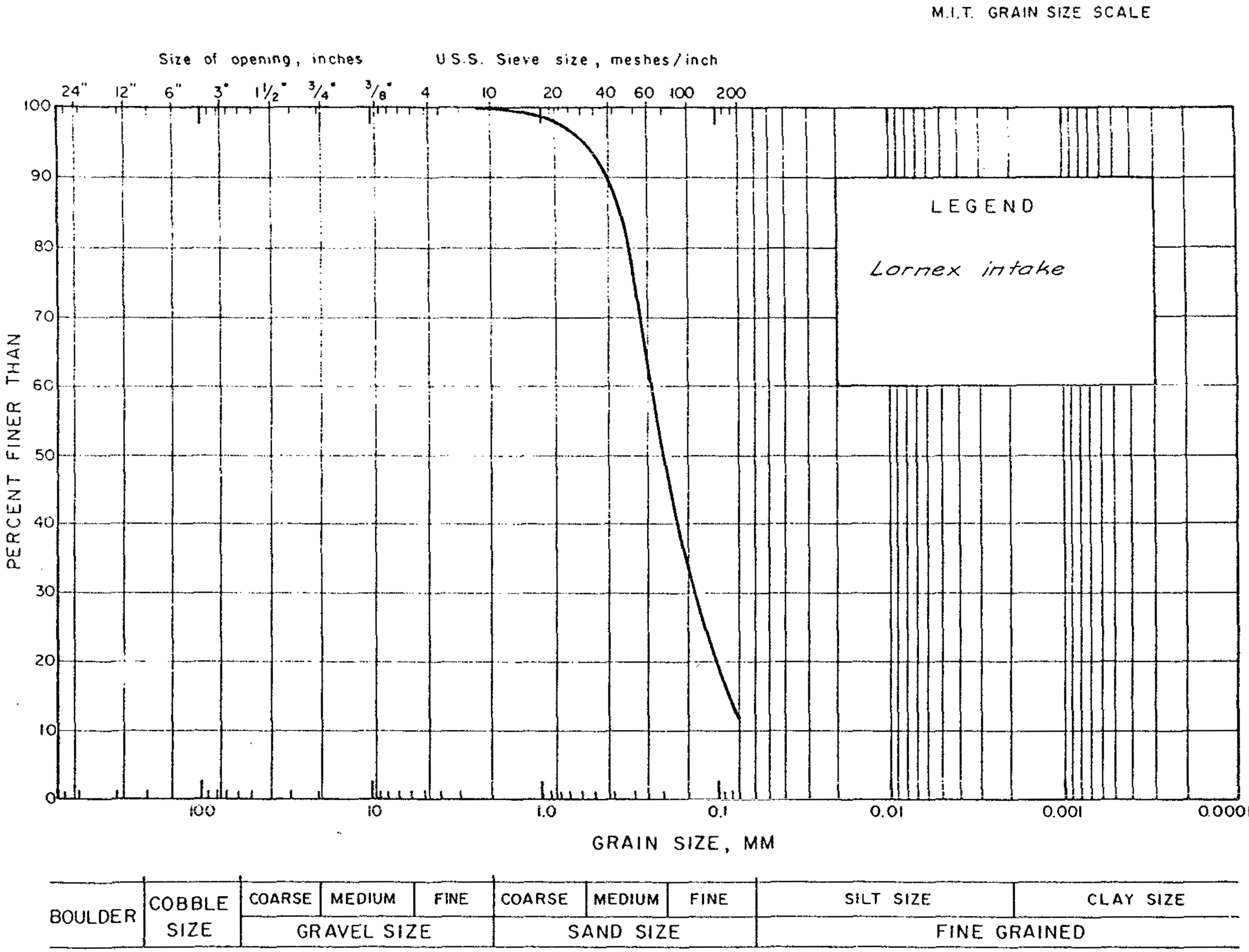
BOULDER	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

Golder Associates

GRAIN SIZE DISTRIBUTION
HAT CREEK WATER SUPPLY
THOMPSON RIVER

Figure 2

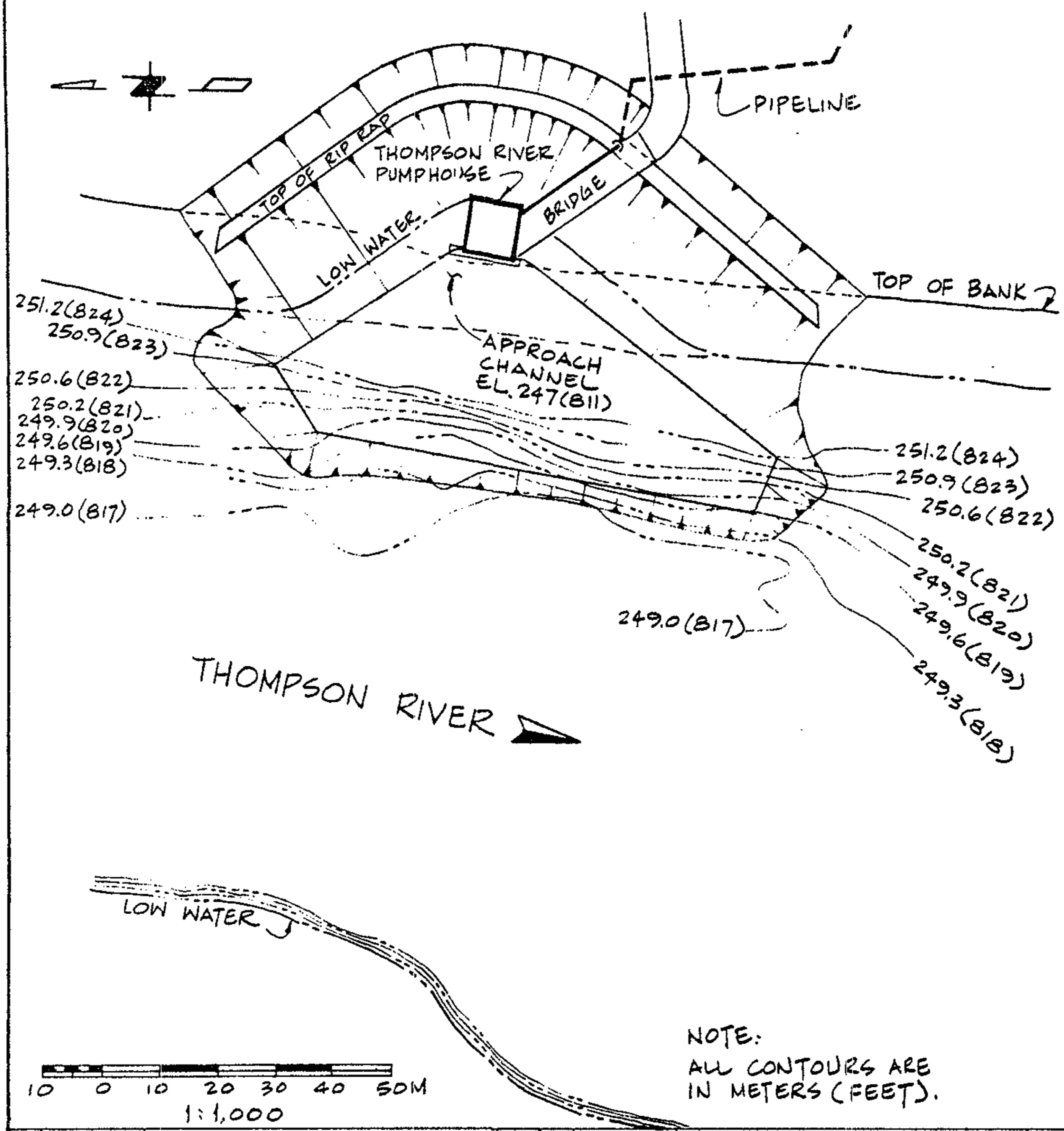
Y77047 N.J. NAS Sept '77



Golder Associates

GRAIN SIZE DISTRIBUTION
HAT CREEK WATER SUPPLY
THOMPSON RIVER

Figure 4



NOTE:
ALL CONTOURS ARE IN METERS (FEET).

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

			DAY	MO.	YR.
APPROVED	SCALE	1:1,000			
DATE	DR'N.	D. TUNG	27	9	77
	CH'K'D.	A.C.			
A 4191/5-3	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
EXISTING THOMPSON RIVER INTAKES
PLAN OF LORNEX INTAKE

SANDWELL

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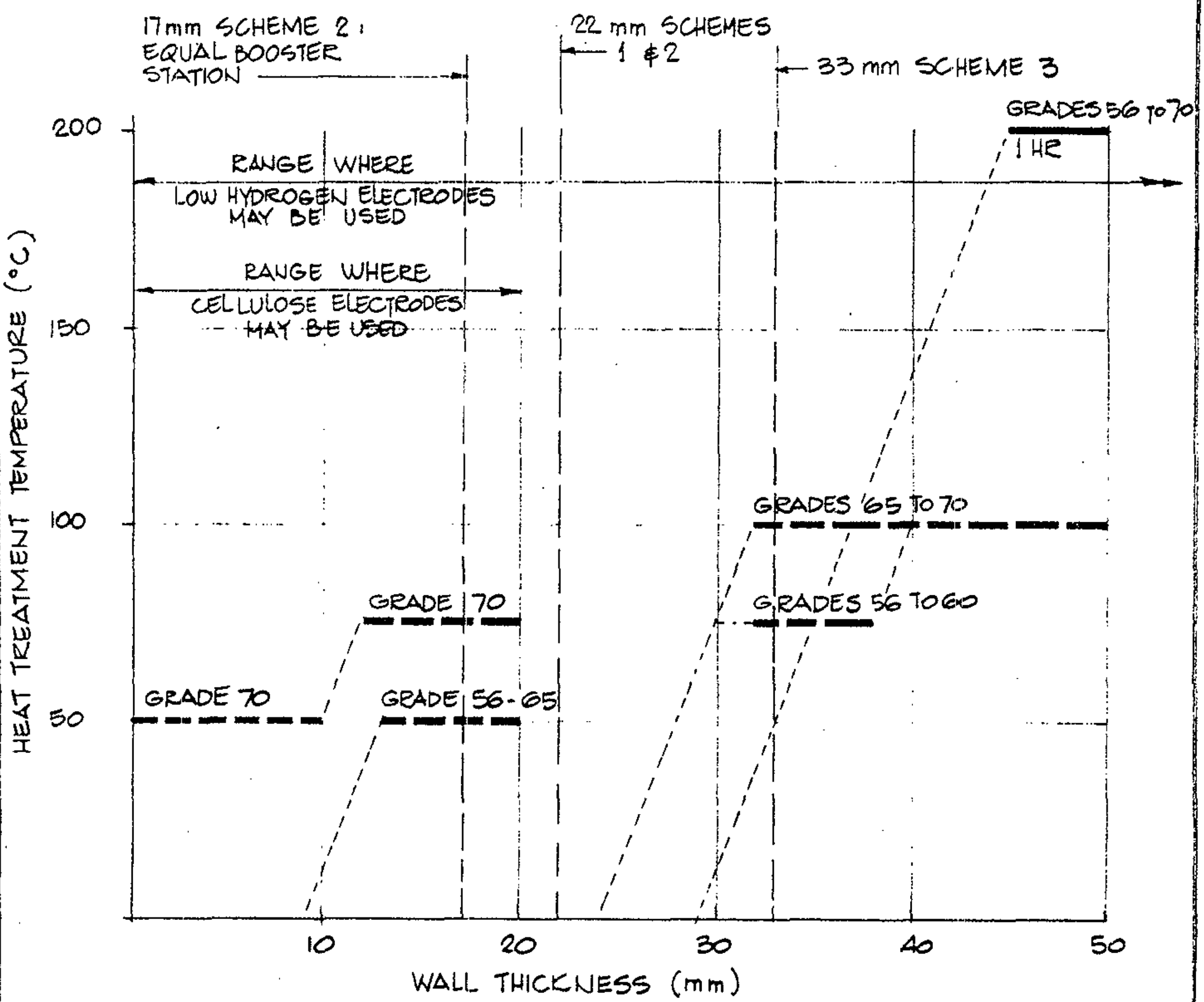
VANCOUVER B.C.

DWG. A4191/5-3 | REV



* NOTE: THESE LINES ARE SIMPLY INTENDED TO INDICATE THAT HEAT TREATMENT REQUIREMENTS DECREASE AS WALL THICKNESS DECREASES. THEIR SLOPES HAVE NO SIGNIFICANCE

** INFORMATION FROM SUMITOMO METAL INDUSTRIES LTD.

LEGEND:

PRE- HEAT }**
POST- HEAT }**
* EXTRAPOLATED

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

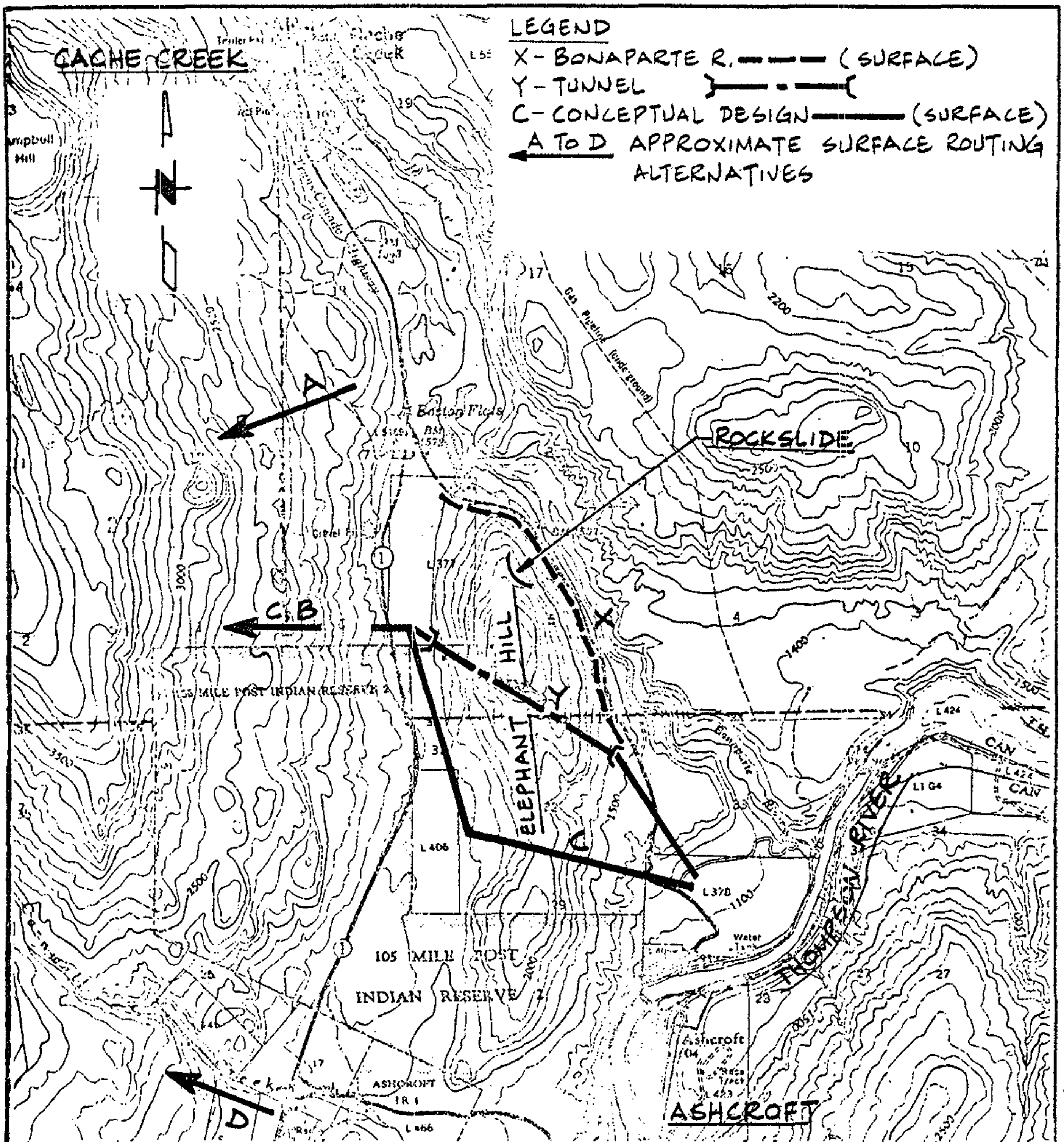
APPROVED	SCALE		DAY	MO.	YR.
DATE	DR'N.	SH			
	CH'K'D.	APB	10	2	78
A 4191/4-9	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
WELDING HEAT TREATMENT REQUIREMENTS

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

SANDWELL

DWG. A 4191/4-9 REV



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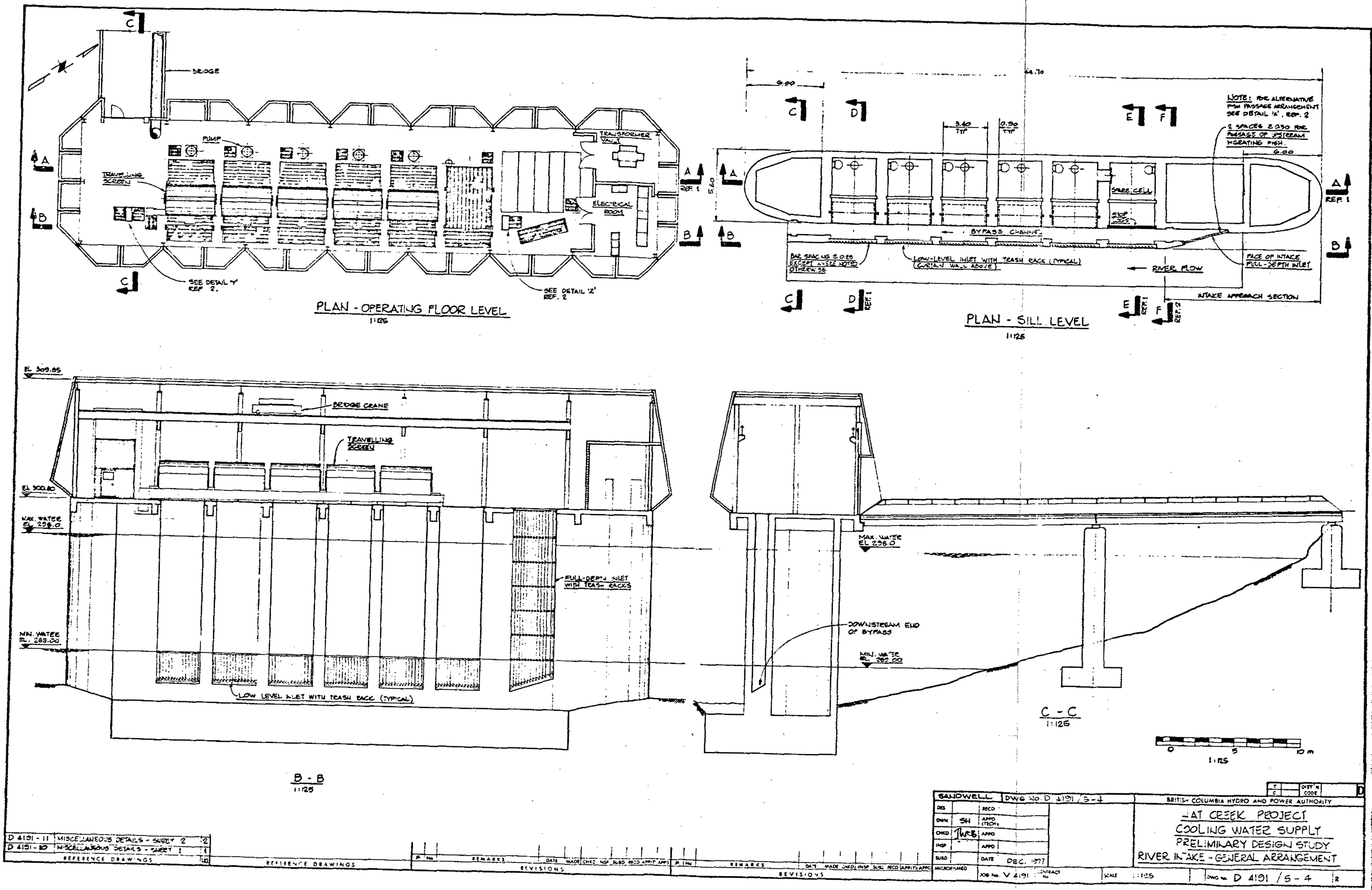
APPROVED	SCALE	1:50,000		
DATE	DR'N.	D. TUNG	12 / 8 / 77	
	CH'K'D.	APP.	10 / 2 / 78	
A4191/4-10	APP'D.			
	APP'D.			

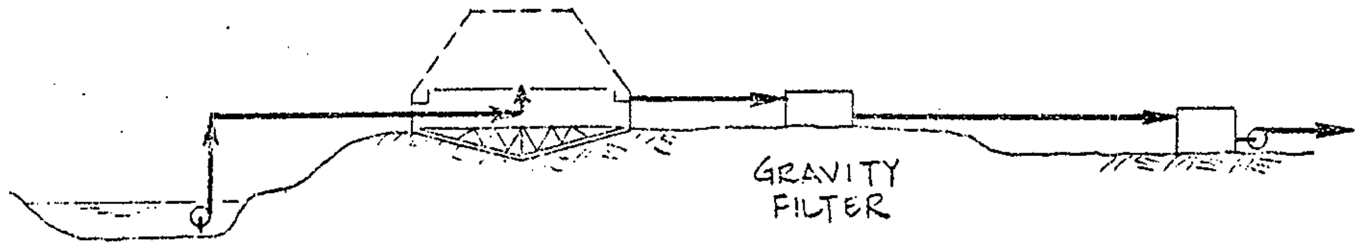
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
ALTERNATIVE ROUTINGS AT "ELEPHANT HILL"

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

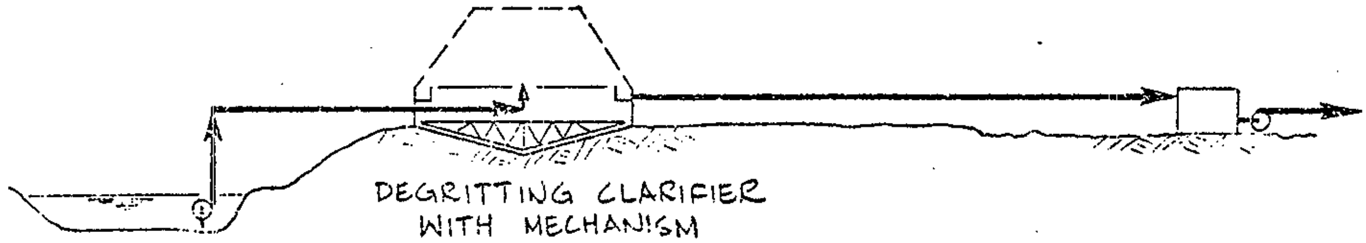
SANDWELL

DWG. A4191/4-10 REV

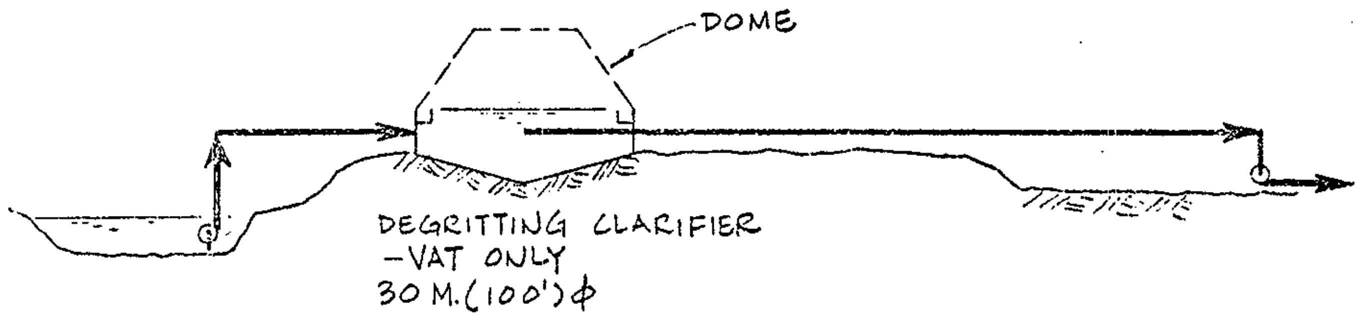

BRIDGE
PUMP
TRAVELLING SCREEN
TRANSFORMER VAULT
ELECTRICAL ROOM
SEE DETAIL 'Y' REF. 2
SEE DETAIL 'Z' REF. 2
PLAN - OPERATING FLOOR LEVEL
1:125
NOTE: FOR ALTERNATIVE FISH PASSAGE ARRANGEMENT SEE DETAIL 'A', REF. 2
2 SPACES 2.030 FOR PASSAGE OF UPSTREAM MIGRATING FISH.
SPARE CELL
BYPASS CHANNEL
LOW-LEVEL INLET WITH TRASH RACK (TYPICAL)
CURTAIN WALL ABOVE
RIVER FLOW
FACE OF INTAKE FULL-DEPTH INLET
INTAKE APPROACH SECTION
PLAN - SILL LEVEL
1:125
EL 305.85
BRIDGE CRANE
TRAVELLING SCREEN
EL 300.80
MAX. WATER EL 298.0
FULL-DEPTH INLET WITH TRASH RACKS
MIN. WATER EL 288.00
LOW LEVEL INLET WITH TRASH RACK (TYPICAL)
B - B
1:125
DOWNSTREAM END OF BYPASS
C - C
1:125
1:125
D 4191 - 11 MISCELLANEOUS DETAILS - SHEET 2
D 4191 - 10 MISCELLANEOUS DETAILS - SHEET 1
REFERENCE DRAWINGS
REVISIONS
SANDWELL DWG No. D 4191 /5-4
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
RIVER INTAKE - GENERAL ARRANGEMENT
DATE DEC. 1977
JOB No. V 4191
SCALE 1:125
DWG No. D 4191 /5-4



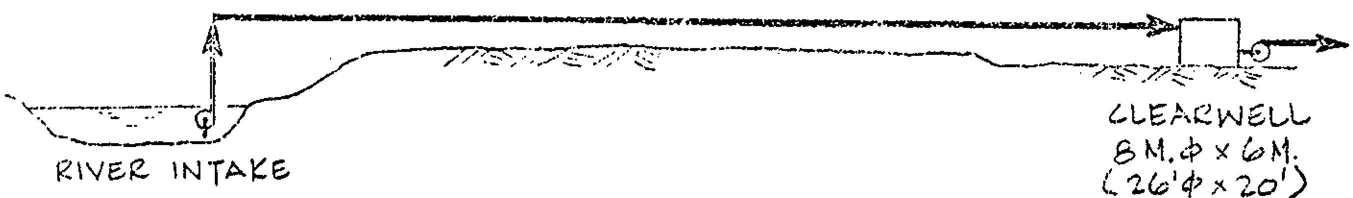
ALTERNATIVE 1



ALTERNATIVE 2



ALTERNATIVE 3



ALTERNATIVE 4

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

			DAY	NO.	YR.
APPROVED	SCALE	NONE			
DATE	DR'N.	D. TUNG	30	9	77
	CH'K'D.	A.C.			
A 4191/5-5	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
WATER TREATMENT
ALTERNATIVES

SANDWELL

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

DWG. A 4191/5-5 REV

PROJECT MEMORANDUM V4191/6

NUMBER OF BOOSTER PUMPS

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/6
NUMBER OF BOOSTER PUMPS

DATE 22 DECEMBER 1977

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 6 October 1977 on the optimum number of pumps per booster station.

NUMBER OF BOOSTER PUMPS

Configurations using from 2 to 5 pumps per station to meet the total flow requirements of 1580 l/s (25,000 USGPM) were studied.

A search of suppliers' installation lists was conducted for both pumps and motors of the required ratings and these findings are summarized in Tables 1 and 2 on the attached sheet. The tables show that the number of installations decreases with increased ratings for both pumps and motors.

The two-pump alternative was eliminated from further consideration because there are very few 6700 kW (9000 HP) motors in service and there is a limited number of suppliers for both pumps and motors. The following table gives the ratings and costs of the remaining alternatives.

Item	Unit	3 Units	4 Units	5 Units
Pump Capacity	l/s (USGPM)	530 (8350)	400 (6250)	320 (5000)
Motor Rating	kW (HP)	5100 (6800)	3700 (5000)	2700 (3600)
Direct Cost*	$	4,900,000	5,200,000	5,600,000
Cost Index	-	1.00	1.06	1.14

*Not including engineering, contingencies, and construction overhead.

Increasing the number of pumping units has the following effects:

a. Advantages

- Sound pressure level (SPL) decreases with smaller units. For example, a 2700 kW motor has a SPL approximately 5 dB(A) lower than a 5100 kW motor.

- NPSH requirements decrease, which means a lower clearwell or NPSH pumps of lower rating.

- Flexibility increases and capacity loss per unit out of service decreases.

- Total system inertia increases, therefore reducing the effects of waterhammer.

b. Disadvantages

- Total station cost increases.

- Station size increases by 110 m^2 (1200 ft^2) per unit.

- Spare parts inventory increases.

- Maintenance requirements increase.

- Amount of control information to transmit and monitor at the power plant increases.

- Motor efficiency decreases slightly.

Wear of pumps will not affect the selection, since rpm and head per stage are the same for all alternatives.

Although pumps and motors for the three-unit alternative are available, Tables 1 and 2 show that there are relatively few installations from which operating experience could be obtained. It could thus be considered that the three-unit system would be less reliable than a four or five-unit system. It is Sandwell's opinion that adequate flexibility can be provided with a four-unit pumping system as opposed to a five-unit system. It is therefore, recommended that for greater reliability, the premium cost be allocated for a four-unit system as opposed to a three-unit system.

Preliminary design of the booster pumping stations is proceeding based on the following specifications:

Item	Amount
Number of booster pumps per station	4
Flow rate per pump	400 l/s (6250 USGPM)
Discharge head per pump	645 meters (2120 feet)
Motor rating	3700 kW (5000 HP)

Prepared by R. T. Kallberg

R. T. Kallberg, P. Eng.

[redacted]opeland, P. Eng.

Approved by B. R. McConachy

B. R. McConachy, P. Eng.

SANDWELL

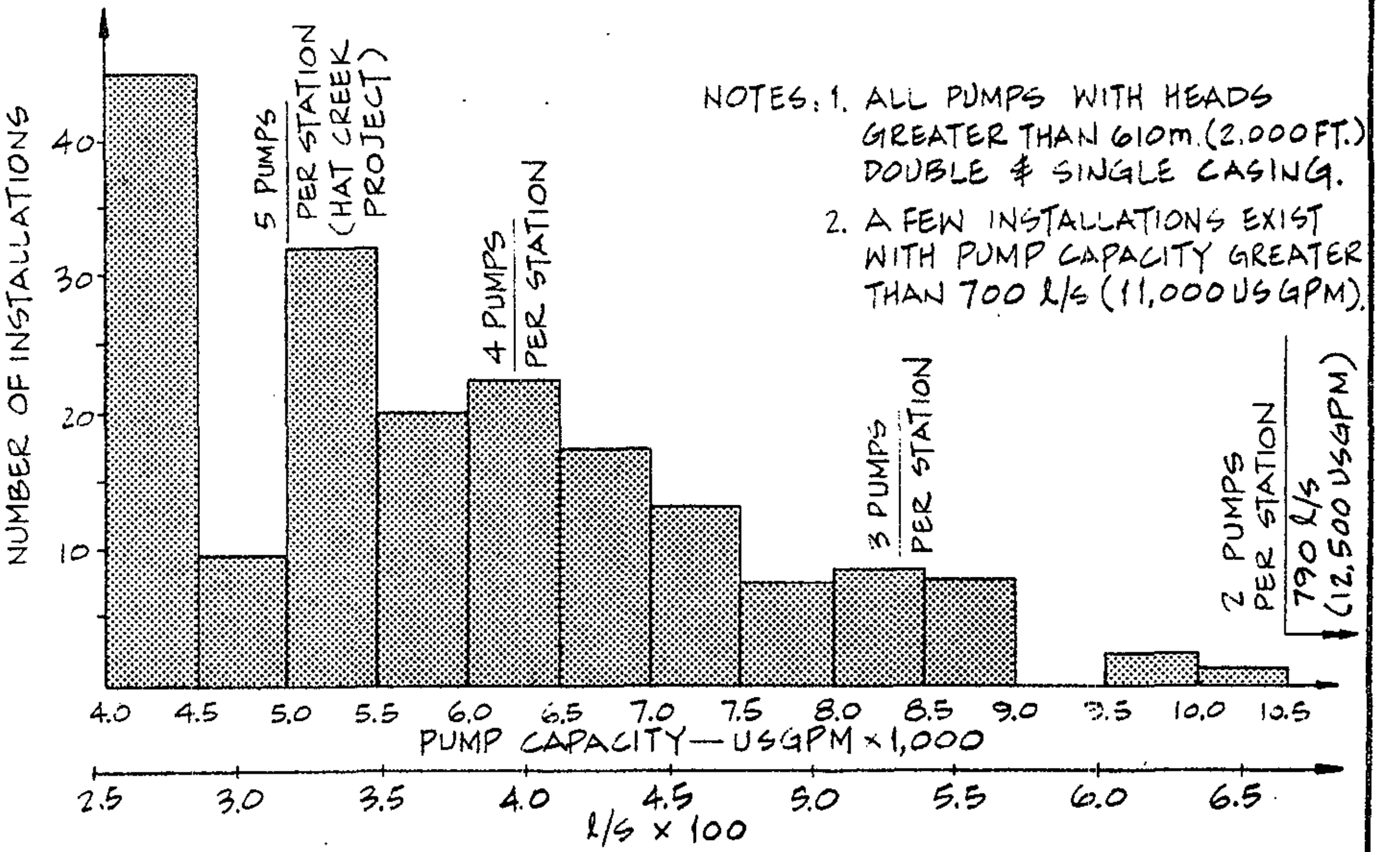


TABLE 1: NUMBER OF PUMP INSTALLATIONS VS PUMP CAPACITY

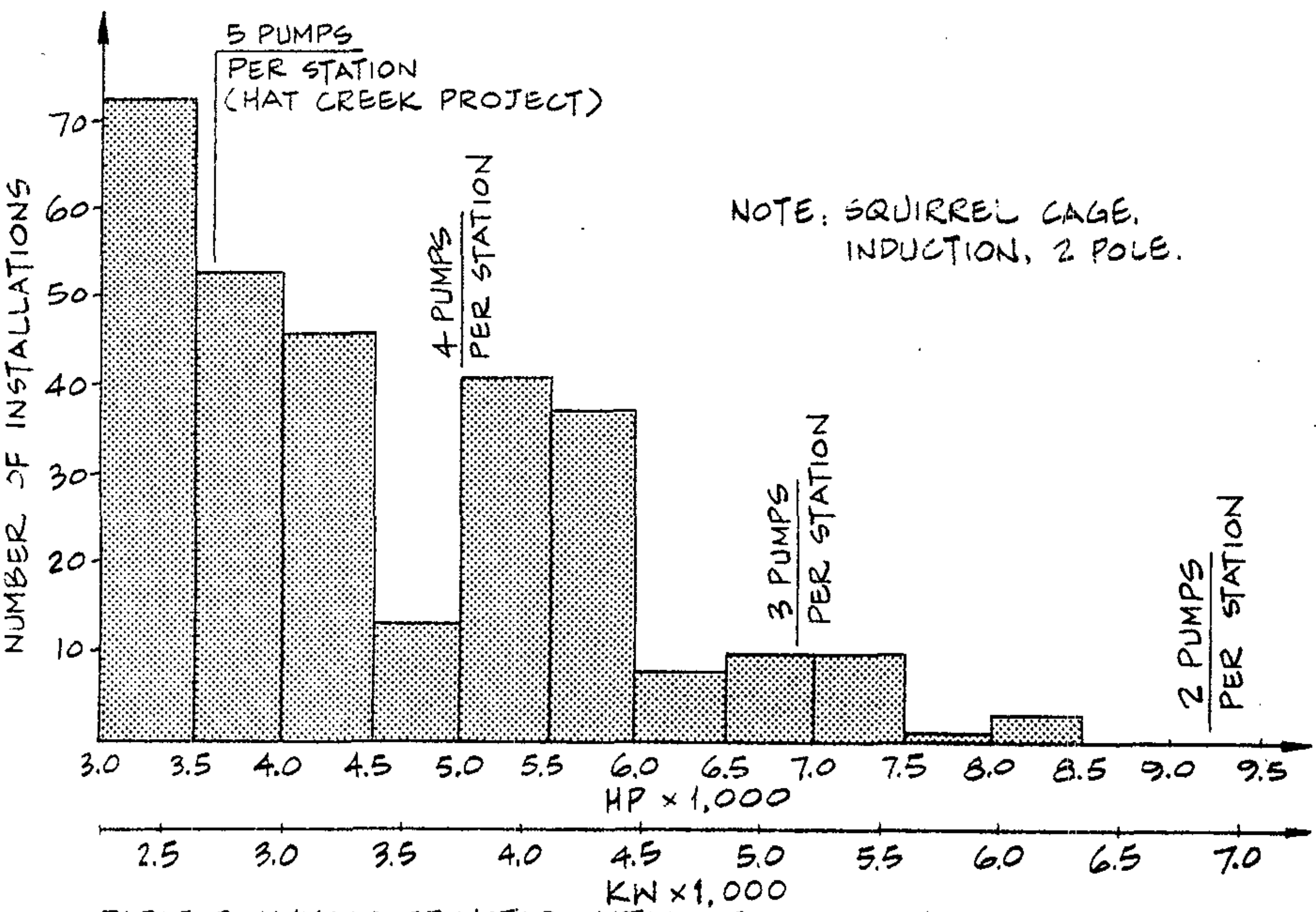


TABLE 2: NUMBER OF MOTOR INSTALLATIONS VS MOTOR HORSE POWER

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 22 DECEMBER 1977

PROJECT MEMORANDUM V4191/7
SUCTION PRESSURE FOR BOOSTER PUMPS

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 27 October 1977 on the means to satisfy the NPSH* requirements of the booster pumps.

NPSH REQUIREMENTS OF BOOSTER PUMPS

The study was based on locating the first booster station on the right bank of the Bonaparte River. Preliminary indications from suppliers show that to meet the NPSH requirements at the maximum flow condition, the booster pumps require a suction water level ranging from a minimum of 11 m (35 ft) to a maximum of 20 m (65 ft) above the pump centerline, depending on manufacturer.

These NPSH requirements can be met by any of the schemes shown diagrammatically on the attached Drawing A4191/7-1.

Schemes 1 and 2, which utilize separate pumps to provide the NPSH, would be necessary in the absence of high land adjacent to the booster station. These schemes are essentially the same except in Scheme 2, the NPSH pump has a separate drive and associated electrics rather than being driven in tandem with the booster pump as in Scheme 1.

Scheme 3 utilizes a plateau for the clearwell, 11 m (35 ft) to 20 m (65 ft) higher than the booster station (depending on NPSH requirements). Scheme 4 is essentially the same as Scheme 3 except a much taller clearwell would be required situated adjacent to the booster station.

Any of the four schemes could be accommodated at the proposed location on the right bank of the Bonaparte River. The total direct cost associated with each scheme is as follows:

Scheme	Description	Total Direct Cost
1.	Pumps in tandem with booster pumps	$400,000
2.	Separate pumps	$440,000
3.	Low clearwell on plateau	$140,000
4.	High clearwell adjacent to booster station	$400,000

* Net Positive Suction Head

The cost includes only equipment required to provide the necessary NPSH.

The separate pumps of Scheme 2 add approximately $40,000 to the direct cost and offer little advantage over Scheme 1. Also, because of other disadvantages of Scheme 2 such as, added motor and controls and greater space requirements, it has been eliminated from further consideration.

Since Scheme 4 costs approximately $260,000 more than Scheme 3 and offers no real advantage, it also has been eliminated from further consideration.

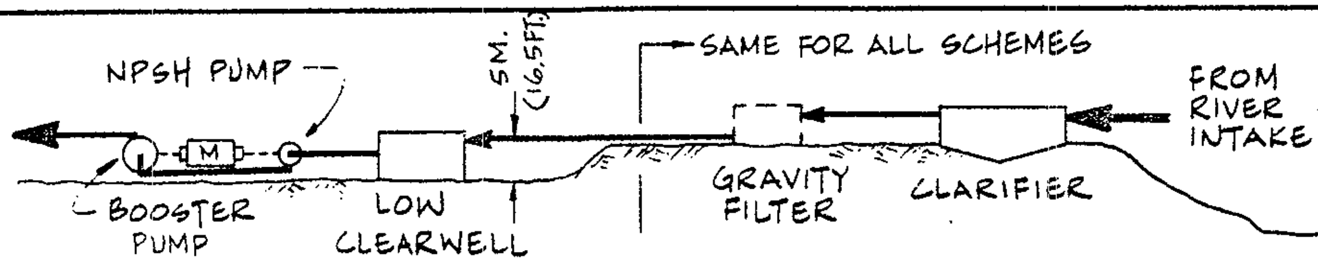
This narrows the selection to Schemes 1 and 3. Scheme 3 is not only economically more advantageous but also has maintenance and space advantages. Therefore, provision of NPSH by means of an elevated clearwell located on a plateau adjacent to the booster station is recommended. The layout is shown on attached Drawing A4191/7-2.

At the second booster pumping station, the necessary NPSH will be obtained by means similar to Scheme 3, with an equalization tank located above the pumping station. Preliminary engineering will proceed on the basis of Scheme 3.

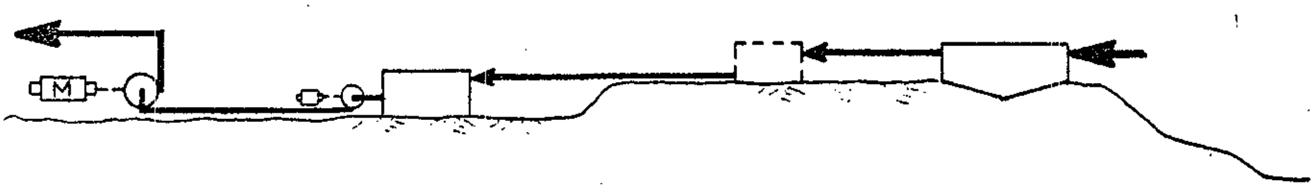
Prepared by R.T. Kallberg
R.T. Kallberg, P. Eng.

A. Copeland, P. Eng.

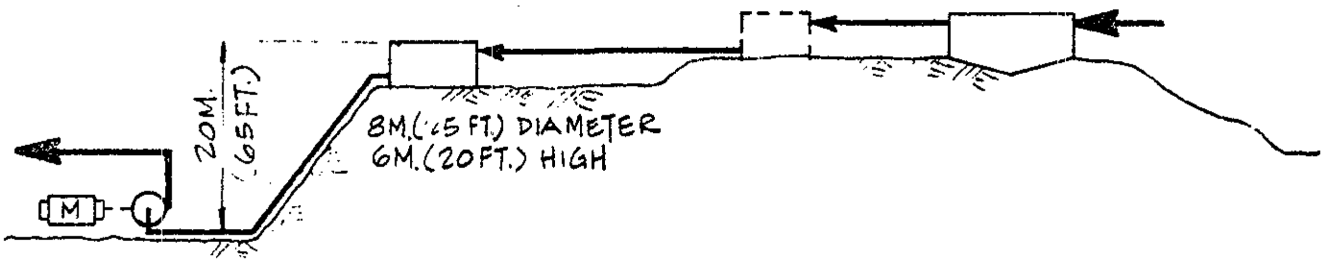
Approved by B.R. McConachy
B.R. McConachy, P. Eng.
Project Engineer



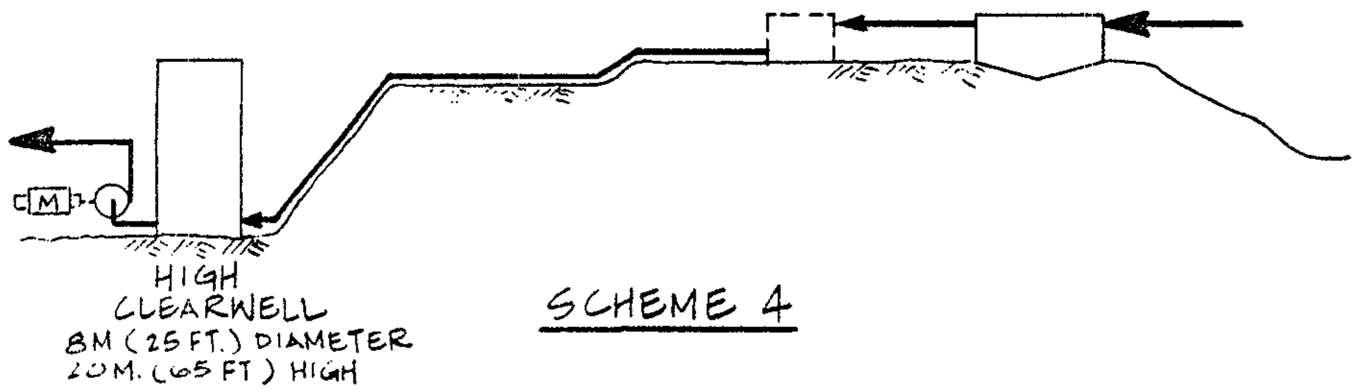
SCHEME 1



SCHEME 2



SCHEME 3



SCHEME 4

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

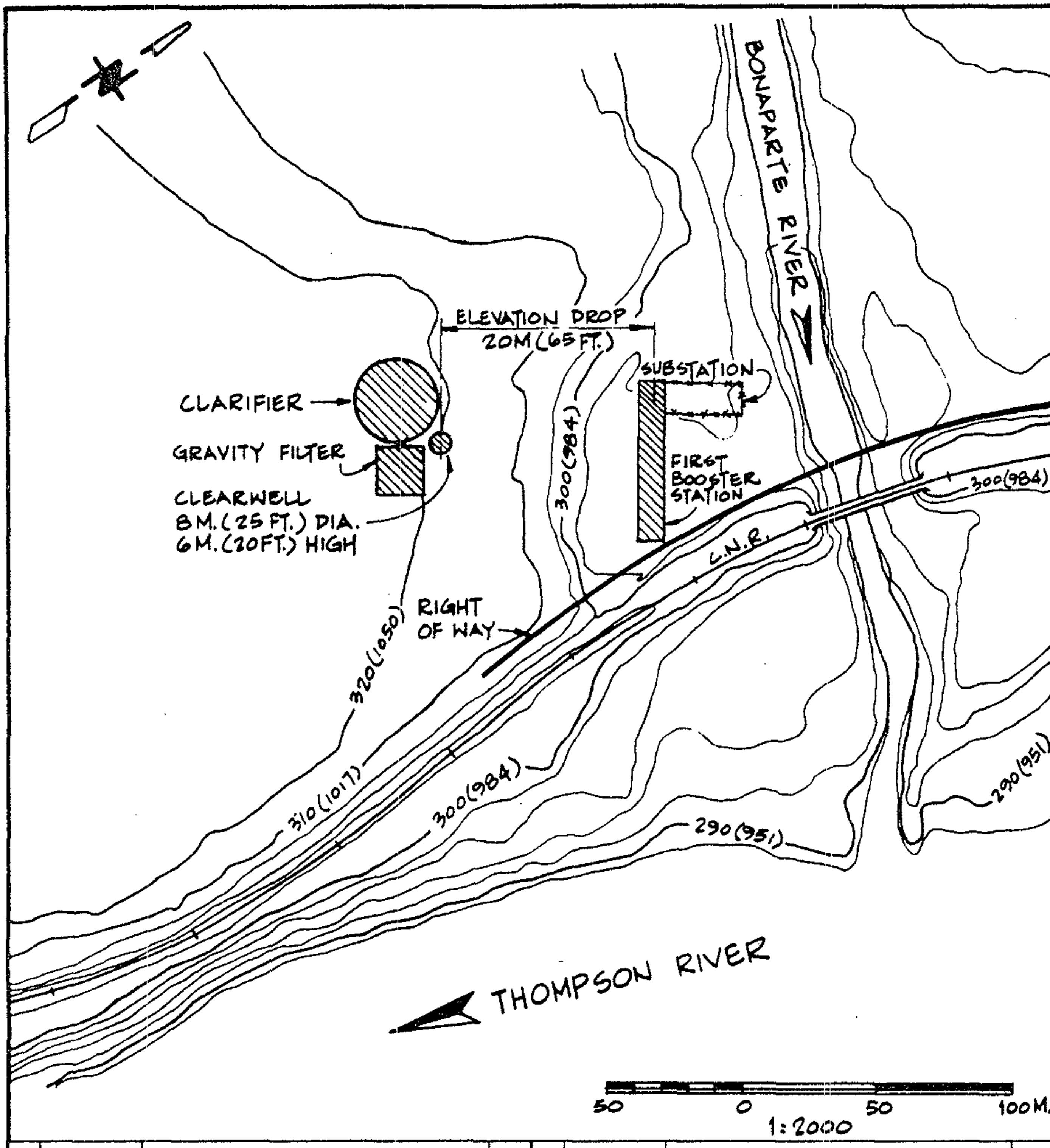
APPROVED	SCALE	NONE	DAY	MO.	YR.
DATE	DR'N.	D. TUNG	12	10	77
	CH'K'D.	R. KALLBERG			
A 4191/7-1	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
FIRST BOOSTER STATION
PROVSION OF NPSH

SANDWELL

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

DWG. A4191/7-1 REV



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

APPROVED

DATE

A 4191/7-2

		DAY	NO.	YR.
SCALE	1:2000			
DR'N.	D.TUNG	20	10	77
CH'K'D.	R KALLBERG			
APP'D.				
APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
FIRST BOOSTER STATION
LOCATION

SANDWELL

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

DWG. A 4191/7-2 REV

PROJECT MEMORANDUM V4191/8

PIPELINE FREEZE PROTECTION

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 22 DECEMBER 1977

PROJECT MEMORANDUM V4191/8
PIPELINE FREEZE PROTECTION

PURPOSE

The purpose of this Project Memorandum is to record the infromation originally presented in a letter dated 25 November 1977.

The Terms of Reference, Page 3, states that Sandwell shall:

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

BURIED PIPELINE

The selection of a buried pipeline has been confirmed based on cost, constructibility, and security considerations; therefore only freeze protection for the buried pipeline remains to be reviewed.

FREEZE PROTECTION - GENERAL

The available literature on the subject has been reviewed (See attached Literature Survey), and calculations made relating the depth of bury of the pipeline to the rate of heat loss and to elapsed time before freezing. The effectiveness of various methods of protection such as insulation, heat tracing and depth of cover has been evaluated.

While these calculations provide some insight into the mechanisms of freezing, assumptions are required - the effects of which have great impact on the result. For example, the degree of saturation of the soil, the makeup of the backfill material, the nature of vegetative cover, and the depth of snow cover are very sensitive factors. Therefore, Sandwell has limited confidence in the accuracy of computations in this area.

The objectives of freeze protection facilities are as follows:

1. Prevent freezing of any of the valves or tanks which are required for operation of the pipeline or water hammer protection. These are to be separately heated and not discussed further here.

2. Prevent ice formation in the pipeline which would impair the delivery of the design discharge.

These objectives are to be met with inlet water temperature as low as 0°C, during the most severe winter on record or expected during the life of the facility, and with a coincident extended power outage at the Booster Pumping Stations.

There are two operating modes which must be considered:

1. Pumps not running, water standing.

2. Pumps running, water flowing.

Therefore, in the discussion that follows, protection for standing water and for flowing water will be discussed separately. Two definitions are of help in discussing freeze protection systems:

Active systems are those that add heat to the water in the pipe.

Passive systems are those that reduce the rate of heat loss from the water in the pipe.

PROTECTION FOR STANDING WATER

a. Protection by Depth of Bury

In order to simplify computations, the US Corps of Engineers "Design Curve" as presented in AWWA Manual M-11, page 217 was used to calculate maximum frost penetration. Temperature records for the 1968-69 winter[1] from Ashcroft and Hat Creek weather stations, extrapolated to elevation 1370 m (4500 feet) have been used and thus the degree-days below freezing have been computed as 3100. For this, the US Corps "Design Curve" gives a design frost depth of less than 2.0 m (6.7 feet).

If this design curve is valid, and provided the pipe centerline is at or below 2.0 m, regardless of whether the water in the pipe is standing or flowing, then as the soil temperature around the pipe is above freezing, at least as much heat flows into the pipe as out when the water temperature is at 0°C. This is because the ground at depth acts as a heat reservoir from season to season and this method of protection is thus an active system.

---

1. Northwest Hydraulic Consultants Limited, in their April 1977 report to Sandwell entitled "Thompson River Ice Condition"*, presented air temperature records from Kamloops and Ashcroft over the period 1941-1975. These records show that the winters of 1949-50 and 1968-69 were the most severe on record. Of these, the former is more severe but temperature records for the Hat Creek weather station exist only from November 1960 on. Therefore, for the purpose of Preliminary Engineering, the data for 1968-69 was used for these calculations.

   * See Report Appendix 19.

Backfill should be fine granular or clay material to provide reasonable insulation. Due to the amount of rock excavation expected, such backfill may not be available in certain areas unless trucked in. Also, after the soil is disturbed it can take many years to settle sufficiently to provide good insulation. Therefore, backfill should be well compacted.

b. Protection by Insulation

Insulation can be exchanged for soil cover, but as this is a passive protection system, it serves to delay rather than prevent freezing. If a 900 mm (36 inches) pipeline were buried with 75 cm (30 inches) of soil cover, and with a 5 cm (2 inches) polyurethane foam insulating jacket, it would theoretically take about 23 days to freeze the pipe solid, with an ambient air temperature of $-20^{\circ}C$. As this duration and intensity of cold is highly improbable, insulation provides a high level of freeze protection for standing water but at high cost. An insulated jacket on the pipe costs about \$160/m compared to the total trenching cost of about \$40/m, plus there would be significant extra costs for handling and installing the pipe. Thus the total extra cost to the project would be at least \$2.5 to 3.0 million.

Alternatives to an insulated jacket are:

- In-situ insulation using styrofoam sheets: may be of value where rock excavation is encountered, to reduce the requirement for borrowed backfill materials, but has not been considered for protection of the entire route.

- In-situ insulation: using sulphur foam, asphaltic materials, thermal concrete, hydrocarbon granules and the like are either experimental or expensive, and have not been further considered.

c. Protection by Heat Tracing

The pipeline could be protected by providing thermal energy to balance the heat loss to the environment. An efficient method of heat tracing is described by Carson (See attached Literature Survey). This would require a 30 mm (1-¼ inch) steel tube tack-welded to the exterior of the pipeline. The tube would act as an electrical conductor and heating element.

Heat tracing would prevent freezing by applying a uniform heat to the pipe along its length, thus is an active system.

The disadvantages to this system are:

a. Requires more difficult construction due to the heat tube attached to the pipe.

b. Requires enough time to drain the pipeline before it freezes, if the tracing system fails. Therefore, a reasonably deep trench is still required in addition to the tracing system.

d. Protection by Line Drainage

Line drainage alone is an operational difficulty, as about 15 to 20 hours would be required to drain the pipeline, and 30 to 35 hours to refill it. Numerous local drains and vents would have to be actuated in order to drain the pipeline, and without power, this would have to be a manual operation. (This is neither an active nor passive system, as rather than influencing the heat loss characteristics of the pipe, one merely removes the potential for ice formation).

e. Continuous Flow

Continuous flow in the uphill direction cannot be maintained if there is a power failure at a pumping station. Alternatively, maintaining continuous flow by leaking water from the reservoir back into the Thompson River would be difficult to initiate and control if power is out. (This would be an active system if the water in the pipeline were colder than the water coming in).

PROTECTION FOR FLOWING WATER

Thompson River water temperatures as low as 0.5°C have been recorded at Walhachin, about 20 km (12 miles) upstream from the intake location. Ice conditions, including frazil ice, are expected to occur during severe winters at the intake. The intake is to be designed to operate without blockage due to these conditions.

The main booster pumps and line friction provide some heating to the water. Assuming that 5 percent of the power output of the pump is converted to heat in the water, each of the two main booster stations would raise the water temperature about 0.09°C at any discharge rate. Line friction would raise the water temperature about 0.06°C at average discharge rate. The total temperature rise from these sources would be about .25°C, a rather insignificant amount.

Ice could form in the line while flowing if the heat loss into frozen soil along the pipeline is more than that sufficient to cool the water temperature to 0°C. In this case, assuming that the intake water is at freezing, there should be a net gain of heat, or at least no loss, along the pipeline. As frictional heating is small, ice can only be prevented by ensuring that the ambient temperature surrounding the pipe is above 0°C, i.e. by active methods.

Therefore, the methods of freeze protection discussed under "Protection for Standing Water" would protect running water as follows:

a. Depth of Bury

As this is an active system, sufficient depth of bury would prevent ice formation in flowing water.

b. Insulation

As this is a passive system, the amount of insulation required must be sufficient to prevent a temperature fall to 0°C. As calculated above, the water temperature could be assumed at 0.25°C, including pumping heat and line friction. This margin is too close to freezing for security against ice, as it is below the recommended minimum temperature of 0.3°C (See References 1 and 4 in the attached literature survey). Therefore, insulation does not provide a satisfactory solution to the problem of ice prevention in flowing water.

c. Heat Tracing

Heat tracing, being an active system, can prevent ice formation in flowing water.

d. Line Drainage

The concept of line drainage is inconsistent with the protection of flowing water from ice formation.

3. Continuous Flow

As determined in the calculations above, the heat supplied by pumping is negligible. Therefore, continuous flow is an insufficient active method. In order to make this method sufficient, the water at the inlet would need to be heated enough that, over the length of the pipeline, it cools to a temperature above 0°C. Water heaters are not considered an economical solution.

SUMMARY

Table 1 summarizes the various protection measures.

Table 1 - Freeze Protection Methods

	Method	Protects Standing Water	Protects Flowing Water	Comments
a.	Depth of bury (active)	Yes	Yes	Requires compacted select backfill.
b.	Insulation (passive)	Yes	No	Costs $2.5 - 3.0 Million more than a.
c.	Heat Tracing (active)	Yes	Yes	System could malfunction. Use d. as backup.
d.	Line Drainage	Yes	No	Pipeline must be out of service about 2 days at minimum.
e.	Continuous Flow (active)	No	No	Not practical during power failure.

Sandwell recommends option a., depth of bury. The only drawbacks to this option are uncertainty due to the assumptions outlined earlier, and uncertainty in the forecast of the most severe winter expected. These uncertainties can be eliminated by:

a. Use well-compacted select backfill, and a well-drained trench with at least 2.0 m depth to the pipe centerline.

b. Continuously monitor water temperatures at the inlet and outlet of the pipeline, to know when freezing conditions are imminent. Then use line drainage as a last resort.

c. During the first and second winter of operation, measure the rate of heat loss from the water, at a few selected points along the pipeline. This would provide operating experience useful in predicting exceptional conditions which would require line drainage.

This procedure should provide a high degree of safety from freezing, while minimizing capital cost, operational difficulties, and design uncertainties.

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Project Engineer

LITERATURE SURVEY

1. Steel Pipe - Design and Operation
AWWA Manual M-11, American Water Works Association, New York, 1964.

The soil surrounding the pipe must fall below 0°C in order for the water in pipe to freeze. A freezing index based on cumulative degree-days below freezing can be related to depth of frost penetration, by U. S. Army Corps of Engineers data presented.

The warning is given that water temperature should be maintained above 0.3°C to avoid blockage due to frazil ice formation.

2. Stephenson, David, Pipeline Design for Water Engineers
No. 6 in Developments in Water Science, ed. Ven Te Chow, Elsevier, Amsterdam, 1976.

Formulae are given for calculating the rate of heat loss and cooling from a wrapped pipeline, given the ambient soil temperature and water temperature.

3. Creager, William P., and Justin, Joel D., Hydroelectric Handbook
2nd Edition, Wiley, New York, 1950, Page 657.

Creager and Justin states that "there is practically no danger from freezing in a pipe covered with .... earth, ... provided the centre line of the pipe is below the frost line". That is, provided the mean soil temperature surrounding the pipe is above 0°C.

4. Cameron, James J. "Buried Utilities in Permafrost Regions", proceedings of Symposium on Utilities Delivery in Arctic Regions held March 16, 17 and 18, 1976, Edmonton, Alberta, Pages 151 to 200.
Economic and Technical Review Report EPS3-WP-71-1
Environment Canada, January, 1977

The water temperature should not be allowed to drop below 0.3°C. The time for the pipe to freeze should be designed to be long enough to undertake remedial measures or repair, or to drain the pipeline, in the event of a failure of pumping, or of heat tracing, whichever is preventing freezing.

5. Gilpin, R. R. "A Study of Pipe Freezing Mechanisms"
Seminar as 4, Pages 207 to 220.

The mechanism of ice formation is described: water cools to -0.1°C (river water) before nucleation, then a sudden formation of dendritic ice crystals occurs, and temperature returns to 0°C. Thereafter, annular ice grows in layers on the pipe walls until the pipe is frozen solid.

In small pipes, the dendritic stage creates a slush ice which requires increased pressure to drive along. Data on increased pressure is not presented for pipes of the sizes of interest here, implying that for sizes larger than a few inches, this is not a problem.

6. Thornton, D. E. "Calculation of Heat Loss From Pipes"
Seminar as 4, Pages 131 to 150.

Methods are given for calculation of cooling and freezing rates of pipelines buried in various soils, given water and air temperatures.

7. Carson, N. B. "A New Method for Heat Tracing Pipelines"
ASME Paper No. 74-Pet-35 for meeting September 15 to 18, 1974.

A discussion of the skin-effect method of heat tracing.

8. Legget, R. F., and Crawford, C. B. "Soil Temperatures in Water Works Practice" J. AWWA, October 1952, pp 923-939.

Comprehensive discussion of preliminary results of a frost penetration study conducted in the Ottawa area. Tests basically confirmed the applicability of the U. S. Corps of Engineers Design Curve (Reference 1), however, the warning of uncertainties involved is given. Further research is necessary to get a better definition of the problem and solutions.

PROJECT MEMORANDUM V4191/9

PUMP DESIGN ALLOWANCE

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 3 FEBRUARY 1978

PROJECT MEMORANDUM V4191/9
PUMP DESIGN ALLOWANCE

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 30 November 1977.

INTRODUCTION

The use of a degritting system to protect the booster pumps from excessive wear has been accepted. Therefore the matter of additional pumping capacity remains only to be viewed with respect to a general pump design allowance to compensate for normal wear.

BOOSTER PUMPS

Tolerances and Allowances

There are many tolerances and allowances which are built into the equipment and pumping system. The most important are:

1. Manufacturing Design Tolerance

   Pumps built according to the Hydraulic Institute-Centrifugal Pump Test Code, must ensure that:

   ... no minus tolerance or margin shall be allowed with respect to capacity, total head or efficiency at the rated or specified conditions.

   Pumps shall be within the following tolerance:

   At rated head: plus 10 percent of rated capacity, or
   at rated capacity: plus 3 percent of rated head.

   It follows that a flow allowance of up to 10 percent may be built into a pump by the manufacturer to ensure that it meets specified conditions.

2. Pipe Friction Tolerance

   The estimate of friction for the cooling water supply pipe line is conservative. A value of 10 m/1000m is used, which is the estimated pipeline friction after extended service of coal-tar epoxy lined steel pipe. Actually, the friction does not increase appreciably over the lower, new pipe value of 8 m/1000 m. This indicates that the actual flow, because of lower friction head, could be approximately 7 percent greater than design, throughout the life of the project.

3. Reduced Friction at Lower Flows

The above tolerances are for the maximum flow condition. Depending on the mode of operation, there may be additional capacity in the system resulting from reduced friction head at lower flows. By choosing a pumping mode with less than 4 booster pumps, operating over a longer time span, there would be additional capacity as shown on attached Drawing D4191/9-1 and summarized as follows:

Number of Pumps Operating	Actual Flow l/sec (USGPM)	Rated Flow/Unit X No. of Pumps l/sec (USGPM)	Additional Flow Capacity*
4	1580 (25,000)	1580 (25,000)	0
3	1290 (20,500)	1185 (18,750)	9%
2	915 (14,500)	790 (12,500)	16%
1	475 (7,500)	395 (6,250)	20%

Note that at any flow except maximum, the pumps will deliver considerably more than their rated individual capacity.

There are two other areas which could have built-in allowances although these are difficult to quantify at this time. One is the maximum design capacity of 1580 l/sec (25,000 USGPM) being 44 percent higher than the average demand of 1099 l/sec (17,423 USGPM) at 100 percent capacity factor. The other area is the allowance in cooling water requirements which the power plant designers have already included.

Consequences of Flow Allowance in Design

Many adverse effects result when an allowance is added to the head or flow capacity of a high pressure pump.

The attached Drawing A4191/9-2 shows what effects a normal flow allowance of 10 percent has on the head of, and power for the pump. For our system, an additional 10 percent flow increases the total head by 3.3 percent and increases the power by 14 percent. The velocity through the pumps and piping increases by 10 percent.

Specifically, the consequences of a flow allowance are:

1. Power Requirements

A 10 percent flow allowance necessitates motors of 14 percent higher horsepower rating and larger cables to accommodate the resulting increased current.

$$* \quad \frac{\text{Actual flow - Rated flow}}{\text{Rated flow}} \times 100 \text{ percent}$$

2. Flow Velocity

For valves and some pipe fittings, a 10 percent increase in velocity would result in an estimated 33 percent increase in wear, as wear is approximately proportional to the cube of the velocity. Pressure changes due to waterhammer would also increase in proportion to the velocity.

For the pump, a flow allowance would increase the NPSH requirements and wear within the pump.

INTAKE PUMPS

The 5 intake pumps have each been sized for 330 l/sec (5,250 USGPM) providing a total capacity of 1660 l/sec (26,250 USGPM) at minimum river level. This is 5 percent greater than required because a flow allowance is added to cover miscellaneous additional requirements such as booster pump seal water, intake pump bearing flush water, travelling screen showers, clarifier or filter backwash, washroom facilities, clean-up hoses, etc. Also, since effective degritting cannot be provided prior to the intake pumps, some allowance has been included for wear. The intake pumps are designed to deliver 330 l/sec (5,250 USGPM) at minimum river level but because of the varying river level and the varying pipe line friction depending on number of pumps in operation, the allowance will actually vary from 5 percent at minimum river level with five pumps operating to 20 percent at maximum river level with two pumps operating. The 5 percent allowance would be adequate to cover wear and miscellaneous services.

RECOMMENDATIONS

Based on the above, Sandwell recommend that:

1. For the booster pumps, additional capacity not be added to compensate for normal wear and tear because of many tolerances and allowances already included in the system and equipment and because of adverse effects resulting from added capacity.

2. For the intake pumps, a 5 percent additional capacity allowance be added to compensate for pump wear, including process losses.

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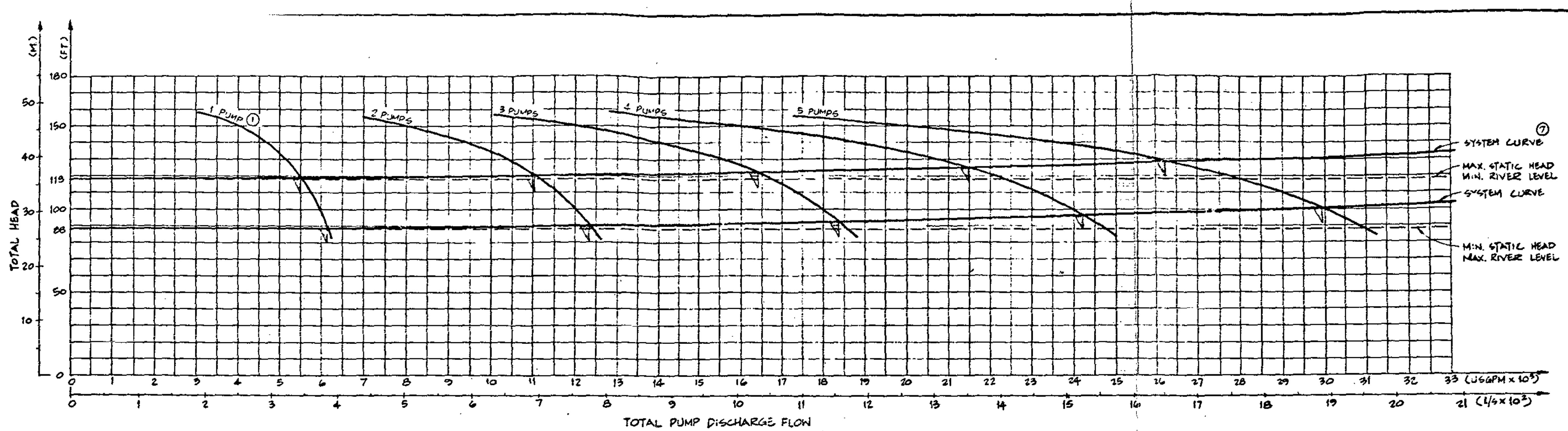
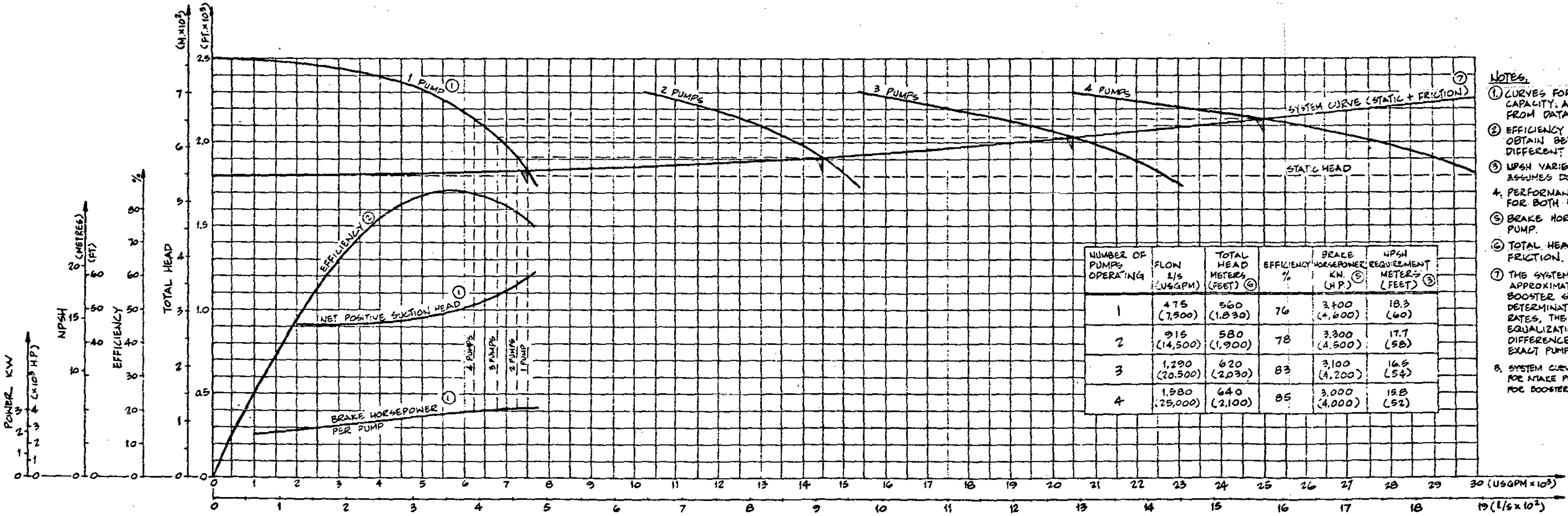
Project Engineer

(PM V4191/9)



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 900mm⌀ (36") PIPELINE FOR INTAKE PUMP AND 800mm⌀ (32") PIPELINE FOR BOOSTER PUMPS.

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS

DRWN D.TANG
CHKD [illegible]
DATE FEB. 1978
MICROFILMED
DIST'N CODE
SCALE

SANDWELL DWG. NO. D 4191 / 9-1

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

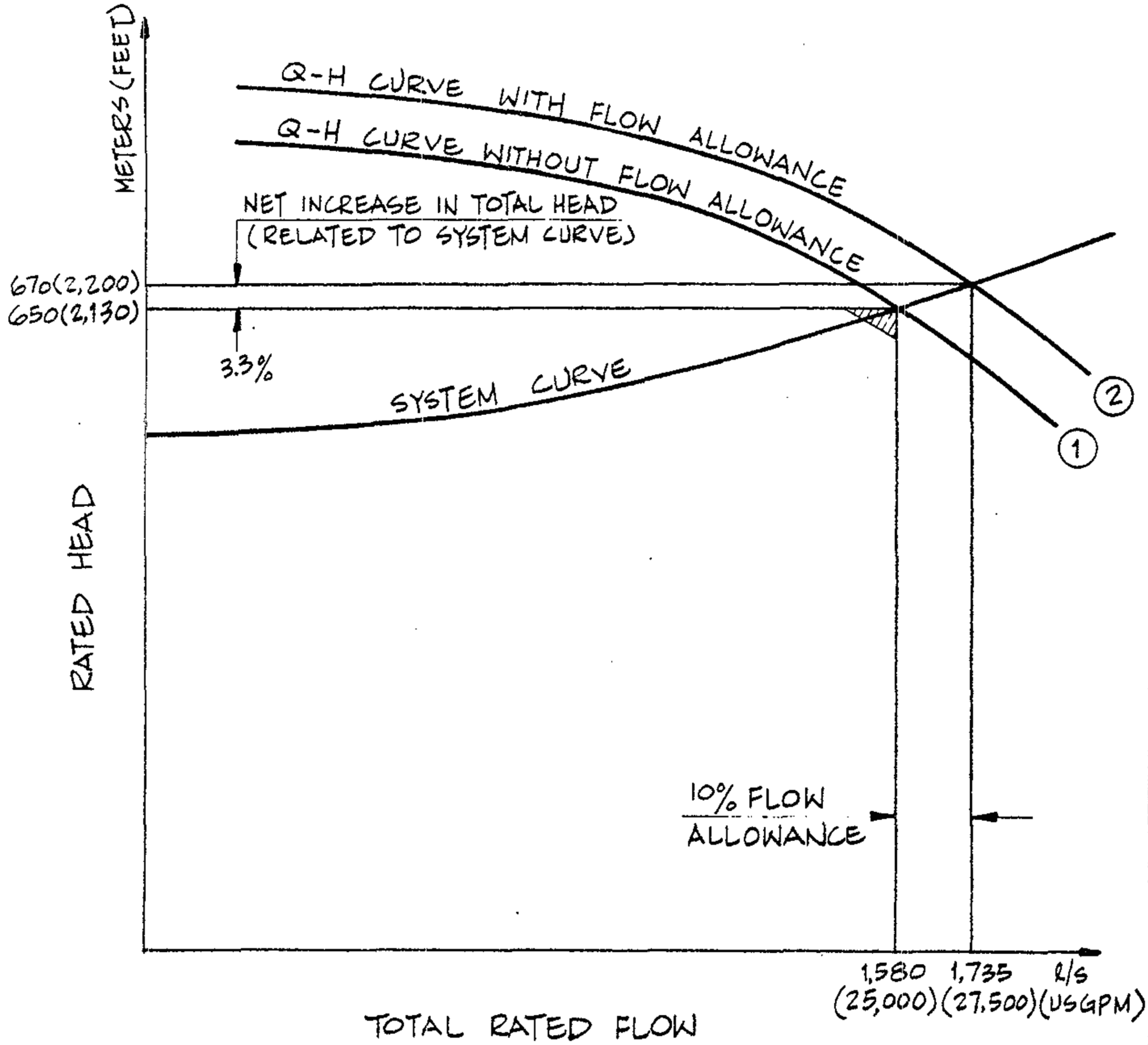
COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

BOOSTER & INTAKE PUMPS-PERFORMANCE CURVES

DWG No. D 4191 / 9 - 1 R

$$\frac{BHP_2}{BHP_1} = \frac{27{,}500\,(2{,}200)}{25{,}000\,(2{,}130)} = 1.14^{*}$$



* ASSUMING EQUAL EFFICIENCY

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

			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	1	11	77
	CH'K'D.	R. KALLBERG			
A 4191/9-2	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
BOOSTER PUMPS — FLOW ALLOWANCE

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

SANDWELL

DWG. A4191/9-2 REV

SANDWELL

PROJECT MEMORANDUM V4191/10

PROVISIONS FOR INSPECTING TRAVELLING SCREENS

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/10
PROVISIONS FOR INSPECTING TRAVELLING SCREENS

DATE 22 DECEMBER 1977

PURPOSE

The purpose of this Project Memorandum is to describe the provisions made for inspecting the travelling screens in order to merit the requirements of the Fisheries and Marine Services of Environment Canada.

PROVISIONS FOR INSPECTING TRAVELLING SCREENS

The general arrangement of the water intake and the proposed provisions for inspection of the travelling screens are illustrated on Drawing D4191/10-1 of December 1977.

The following components of a conventional travelling screen require periodic inspection to ensure fish tightness:

- Screen cloth
- Side seals
- Bottom seals

Inspection of the screen cloth can be carried out in the screen housing at operating floor at any time, simply, by rotating the screen until each panel has had its turn.

The inspection of the side and bottom seals requires special timing and provisions, both of which are described below:

Bottom and side seals of the travelling screens could be inspected annually during low water. To enable this work to be carried out 'in the dry', a flow of 283 m^3/s (10,000 cfs) has been adopted as the maximum winter river flow (see Drawing D4191/10-2 of December 1977). The level of 290.50 m (953 feet), shown on Section A-A of Drawing D4191/10-1 is the calculated water surface for this flow. The pumphouse headroom, crane and screen frame would be designed to enable lifting of the entire screen unit above this water level to allow inspection of the bottom and side seals 'in the dry'.

To prevent, if necessary, fish from entering the pump sump while the screen unit is in the raised position for inspection, stop logs would be positioned as shown in Section A-A.

Quick release of spray water and electrical connections would be provided at operating floor to enable inspecting with the minimum of down-time.

A removable platform would be provided for the actual inspection phase and maintenance, if necessary, of the bottom and side seals (Section A-A). This platform would be folded (Detail 'Z') when passing through the narrow opening at operating floor between the curtain wall and travelling screen housing. Movement of the platform in the vertical direction would be achieved by an auxiliary electrical hoist as shown in Section A-A. This hoist would be manually operated by a 'controller pull cord', which would extend to the lowest position of the inspection platform.

This arrangement would require only one inspection platform for all six cells.

Before the entire screen unit would be returned to its operating position, the sill would have to be cleaned of river solids that might have been deposited during the inspection period. This cleaning would be done by means of a high pressure water lance operated from the inspection platform.

A hatch has been incorporated as shown in the Plan of the inspection platform to provide access for a diver by means of a ladder to the concrete sill and trash rack. This access hatch would also provide a means of closely inspecting fish behaviour and obtaining fish counts, etc. in the by-pass channel. Removable inspection lamps would be clipped to the platform handrail to assist in the inspection and fish observations.

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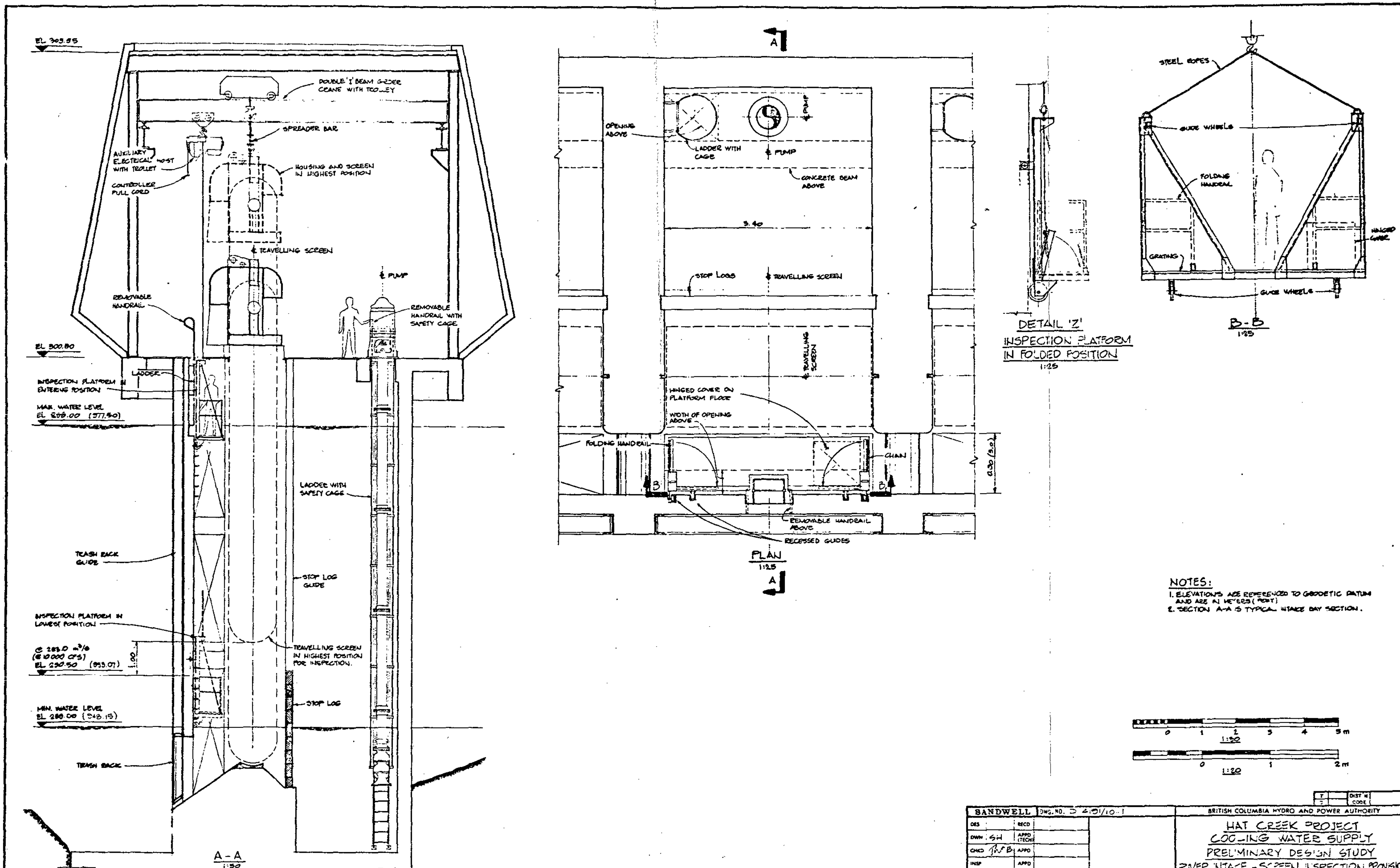
Approved by B McConachy
B.R. McConachy, P. Eng.
Project Engineer



NOTES:
1. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM AND ARE IN METERS (FEET)
2. SECTION A-A IS TYPICAL INTAKE BAY SECTION.

0 1 2 3 4 5 m
1:50

0 1 2 m
1:20

SANDWELL	DWG. NO. D 4191/10-1
DES	RECD
DWN. SH	APPD (TECH)
CHKD	APPD
INSP	APPD
SUBD	DATE DEC. 1977
MICROFILMED	JOB NO. V 4191

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
RIVER INTAKE - SCREEN INSPECTION PROVISIONS

SCALE AS SHOWN
DWG No. D 4191/10-1

D 4191-SK117 RIVER INTAKE - GENERAL ARRANGEMENT
REFERENCE DRAWINGS
REVISIONS

NOTES:

1. HYDROGRAPHS ON THIS DRAWING WERE PREPARED BY NHCL FROM WATER SURVEY OF CANADA DATA.
2. FOR DETAILED 1976 & 1977 HYDROGRAPHS NEAR SPENCES BRIDGE SEE REF. 1.
3. THE MINIMUM FLOW ON RECORD NEAR SPENCES BRIDGE IS 116 M³/S (4,100 CFS) – 8 JAN. 1937, AND THE MAXIMUM RECORED FLOW IS 4,134 M³/S (146,000 CFS) – 15 JUNE 1972.

LEGEND:

* INDICATES PERIODS WHEN DATA WERE NOT RECORDED

SANDWELL	DWG. NO. D419_/_-2	BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
DES	RECD	HAT CREEK PROJECT	
DRWN D.TUNG	APPD (TECH)	COOLING WATER SUPPLY	
CHKD [illegible]	APPD	PRELIMINARY DESIGN STUDY	
INSP	APPD	THOMPSON RIVER HYDROGRAPHS, 1918 – 1977	
SUBD	DATE DEC. 1977		
MICROFILMED	JOB No. V4191	CONTRACT No.	SCALE — DWG. No. D4191/10-2 2

REFERENCE DRAWINGS

D4191-SK-7-8 HYDROGRAPHS NEAR SPENCES BRIDGE 1976–1977

REVISIONS

SANDWELL

PROJECT MEMORANDUM V4191/11

PROVISIONS FOR TESTING A STATIONARY SCREEN

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/11
PROVISIONS FOR TESTING A STATIONARY SCREEN

DATE 22 DECEMBER 1977

PURPOSE

The purpose of this Project Memorandum is to describe the provisions made for testing a stationary screen in order to accommodate the request of the Fisheries and Marine Services of Environment Canada.

PROVISIONS FOR TESTING A STATIONARY SCREEN

The concept of an intake with stationary screens was presented as Type IB on Drawing D4007/1/1 in Sandwell's Project Memorandum V4007/1 on Water Intake Design, dated January 1977. In Section F-F, the stationary screen is basically in line with the curtain wall and exposed directly to the river flow. A similar placement of a stationary screen for testing purposes in the proposed Thompson River Intake would physically be possible by interchanging a trash rack for a stationary screen. Hydraulically, however, the stationary screen would not be connected to the pump cell behind it because of the by-pass between trash rack-curtain wall and travelling screens. Extending the cell walls, by means of stop logs, to the trash rack-curtain wall would provide this hydraulic connection but it would interfere with the by-pass flow - an unacceptable condition for the operation of the remaining intake cells. Therefore, the only place suitable for testing a stationary screen would be at the face of an intake cell, where the screen would be subject to the by-pass flow (See Sections E-E and F-F, Drawing D4191/11-1 of December 1977).

As the test screen would be placed in the spare cell which would not have a pump, flow through this cell would have to be obtained from the adjacent pump cell. To achieve this, a sluice gate would be provided. During testing of the stationary screen, stop logs would isolate the adjacent cell from the by-pass and the sluice gate would be in the open position, so that all the pumped flow would go through the stationary screen.

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Approved by B.R. McConachy
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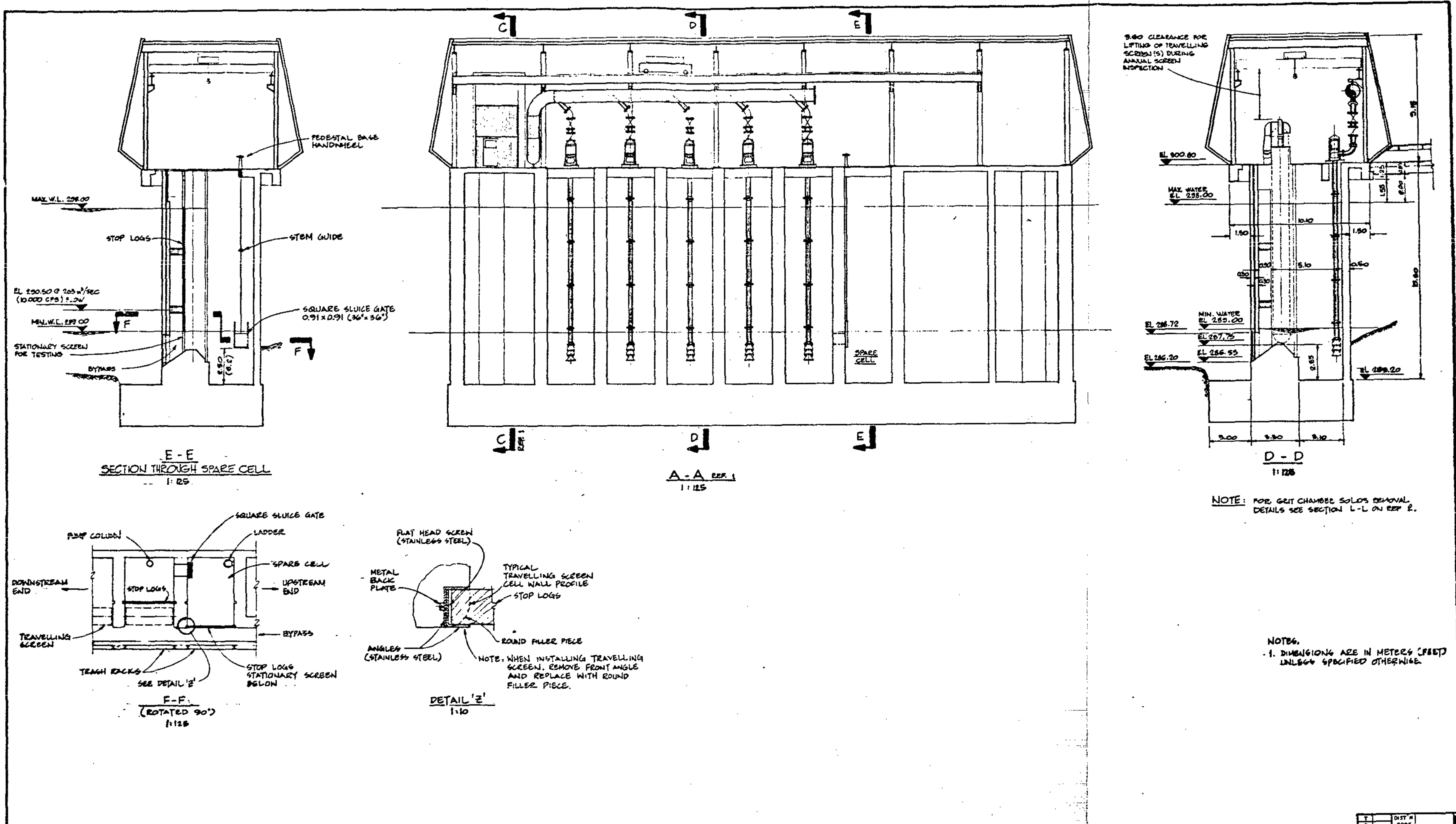
Project Engineer



REFERENCE DRAWINGS	
D 4191 - 11	MISCELLANEOUS DETAILS - SHEET 1
D 4191 - 8	GENERAL ARRANGEMENT

SANDWELL DWG. NO. D4191/11-1

DRN: D. TUNG · CHKD · DATE: DEC. 1977 · JOB No. V4191

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. D4191/11-1

SANDWELL

PROJECT MEMORANDUM V4191/12

PIPELINE ROUTING - McLEAN LAKE TO PLANT RESERVOIR

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/12
PIPELINE ROUTING - McLEAN LAKE TO PLANT RESERVOIR

DATE 22 DECEMBER 1977

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 12 December 1977.

INTRODUCTION

The topography between McLean Lake and the Power Plant reservoir presents various pipeline routing alternatives which are shown on the enclosed drawings D4191-12-1 and D4191-12-2.

These pipeline routing alternatives are:

A	Summit Route	- Gravity Flow, Full Pipe (solid line)
B	Summit Route	- Gravity Flow, Partly Full Pipe (dotted line)
C	Medicine Creek Route	- Pumped Flow, Full Pipe

The objective is to select the most economical alternative. While considerations of combining service corridors in this area may play a part in selecting the route, these considerations are not included in the analysis.

DESCRIPTION AND MERITS OF ROUTES

A. Summit Route - Gravity Flow, Full Pipe

This alternative, shown as a solid line on drawing D4191-12-2, would be a gravity pipeline, starting at the summit tank and kept full by a discharge valve on the outlet. The summit route was chosen during Conceptual Design to gain an advantage in waterhammer control by shortening the length of the pump discharge line. At that time, the discharge flow rate was to be continuous at the system capacity of 1600 l/s. By setting the water level in the summit tank to coincide with the pipeline friction between this tank and the power plant reservoir, energy was not wasted.

However, during this Preliminary Engineering stage of studies, it has developed that the average discharge is only 663 l/s. At this lesser discharge rate, the friction between the tank and reservoir would be much less, so that to follow the summit route would involve pumping over a hump and wasting energy. Energy would be wasted even if periodic pumping at system capacity, for shorter durations of time, was carried out to provide the make-up water requirements, as pipeline friction is much higher at this capacity than at the average rate.

B. Summit Route - Gravity flow, Partly Full Pipe

This alternative, shown on drawing D4191-12-2 as a dashed line is a partially filled pipeline starting at the summit tank. In order to be partially filled throughout its length, it is laid at a steady gradient of 0.75 percent which is why it must follow a circuitous route. The route length has been minimized by using earth berms for low spots, and cuts for high spots.

Compared to A, the advantages are:

- self draining.
- air valves unnecessary.
- discharge control valve and control apparatus unnecessary. Summit tank level would be controlled by an overflow weir.

However, compared to A, the disadvantages are:

- an 1150 mm diameter pipeline, rather than 800 mm diameter, is required.
- the route is longer.
- earth fill berms and deep rock cuts are required.

C. Medicine Creek Alternative - Pumped Flow, Full Pipe

This alternative was developed specifically to save energy when pumping at discharge rates less than the system capacity. When the route was first developed, the waterhammer control implications were unknown. Subsequently, as discussed in the following section, it has been determined that there is only a small difference in waterhammer control requirements between A and C.

Compared to A, the advantages of this route are:

- the total route is slightly shorter.
- energy is saved when the pumping rate is less than 1600 l/s, the system capacity.
- a downstream discharge control valve is not required, as the pipeline would enter the reservoir below the low water level.

Waterhammer Protection

Waterhammer protection requirements for A, B and C are shown on Table 1.

Table 1 - Waterhammer Protection Requirements

Route	Pump Flywheels[1]	Downstream Discharge Control Valve	One Way Surge Tank[2]	Summit Tank or Overflow Structure[3]
A Summit Route - Full Pipe	Yes	Yes	1 Required	Yes
B Summit Route - Partly Full Pipe	Yes	No	1 Required	Yes
C Medicine Creek	Yes	No	2 Required	No

COST COMPARISON

To assist in the selection of the most attractive alternative, cost differences relative to Alternative A are given on Table 2.

1. Flywheels on pumps increase the inertia at the pump and thus reduce the severity of flow and pressure change caused by power failure.
2. One-way surge tanks are tanks isolated from the pipeline by check valves. When a negative pressure wave passes down the pipeline, the check valves open and water flows into the pipeline from the tank, thereby preventing water column separation and cavitation.
3. The summit overflow structure would consist of a tank with internal overflow weir, designed to keep the pipeline upstream submerged, and to separate it from the section downstream.

Table 2 - Cost Summary

Item	Unit Cost	Summit Full Pipe A	Summit Partly Full Pipe B	Medicine Creek C	Extra Cost ($1000s) A	Extra Cost ($1000s) B	Extra Cost ($1000s) C
Pipeline							
3.3 MPa, 800 mm D.	$ 780/m	Base	- 2,480 m	-510 m	0	-2,730	-400
4.9 MPa, 800 mm D.	$ 850/m	Base	0	-150 m	0	0	-130
7.4 MPa, 800 mm D.	$ 940/m	Base	0	- 10 m	0	0	- 10
3.3 MPa, 1150 mm D.	$ 1,100/m	Base	4,270 m	0	0	4,700	0
Air and drain valves	$ 17,000ea	Base	- 4	0	0	- 70	0
Rock excavation	$ 13/m^3	Base	16,000 m^3	0		210	0
Earth berm construction	$ 2.50/m^3	Base	185,000 m^3	0	0	460	
Total					0	+2,570	-540
Waterhammer Protection							
Flywheels	$ -	Base	0	0	0	0	0
1-way surge tank	$ 350,000 ea.	Base	0	+1	0	0	350
Discharge control valve	$ 100,000 ea.	Base	-1	-1	0	- 100	-100
Summit Overflow Structure	$ 150,000 ea.	Base	0	-1	0	0	-150
Total					0	- 100	+200
Energy Cost							
Extra Pumping Head at Average Discharge	$ 17,500/m	Base	0	-33.30 m	0	0	-580
TOTAL EXTRA COST					0	+2,470	-920

SELECTION OF ROUTE

As shown on Table 2, the Medicine Creek route, Alternative C, is the least costly scheme due to the shorter length of pipeline and reduced power cost. The summit route, A, using a full pipe costs almost $1 million more, or using a partly full pipe $3.4 million more.

Therefore, Sandwell recommends the Medicine Creek route be adopted. The comparative cost data provided will enable B.C. Hydro to evaluate any combination of corridors.

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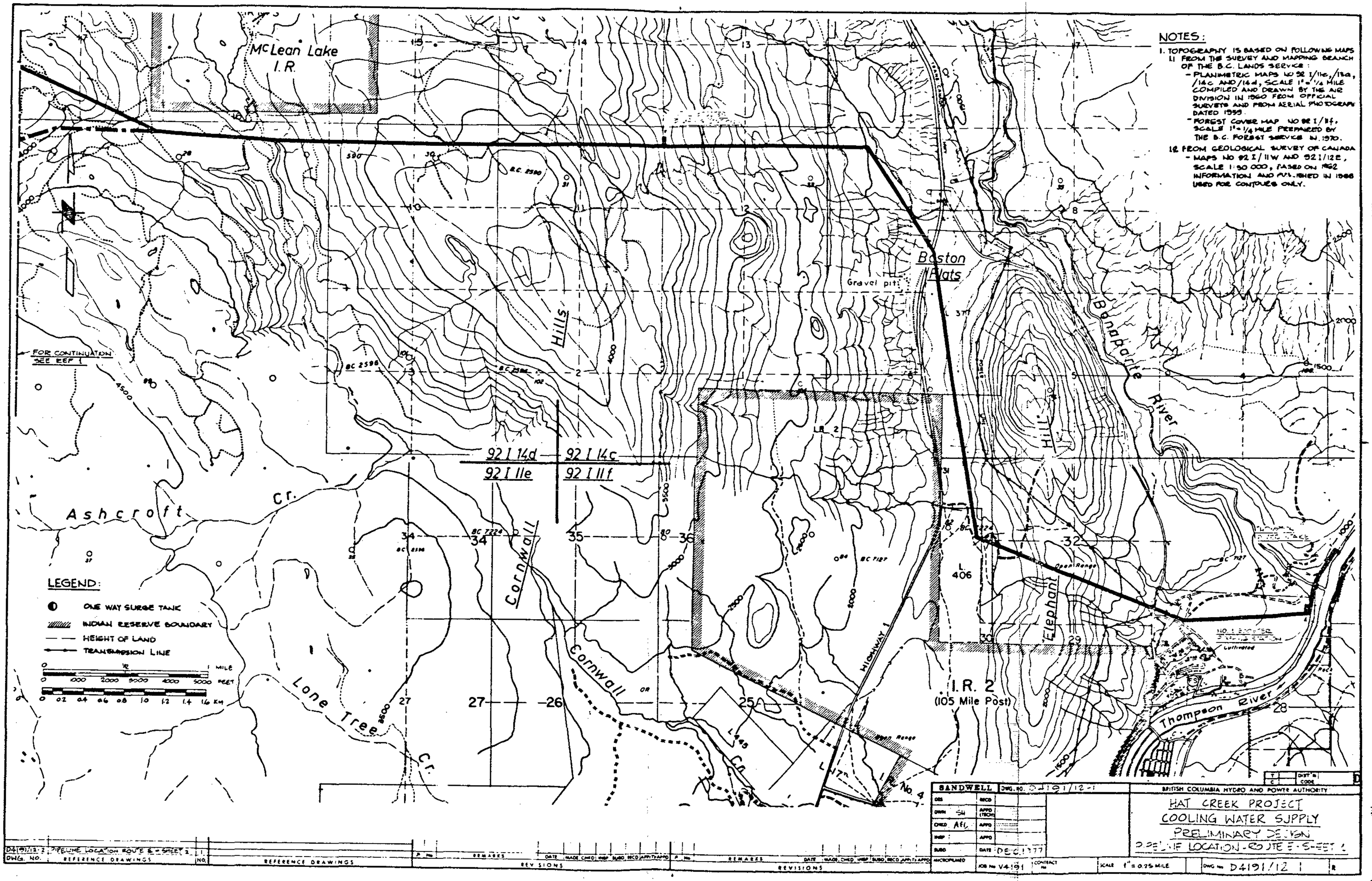
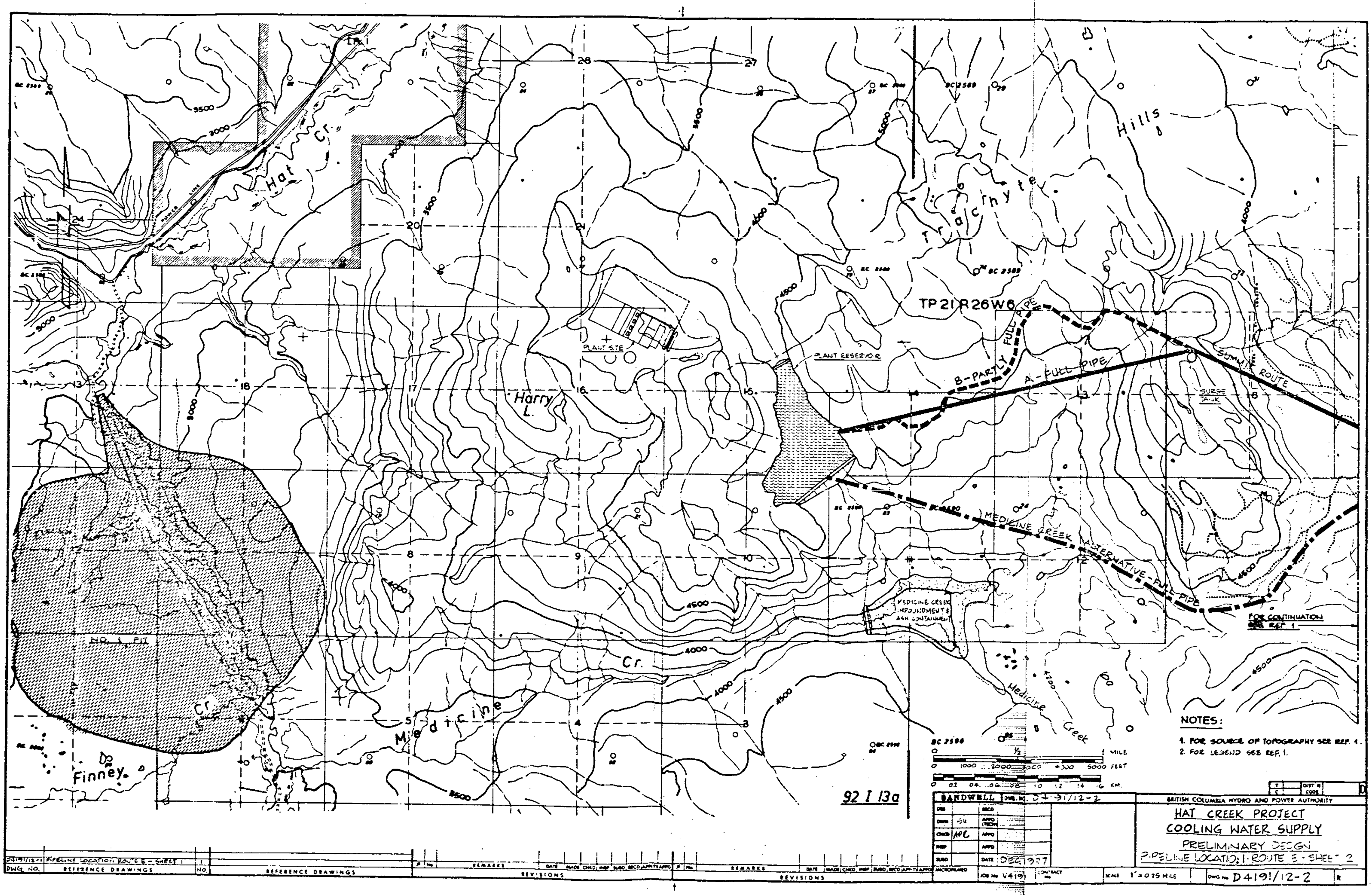

McLean Lake
I.R.
Boston
Flats
Gravel pit
Hills
Hill
Elephant
Bonaparte River
Ashcroft
Cr.
Cornwall
Cornwall
Lone Tree
Cr.
Thompson River
I.R. 2
(105 Mile Post)
HIGHWAY 1
I.R. No. 4
FOR CONTINUATION
SEE REF 1
92 I 14d
92 I 14c
92 I 11e
92 I 11f
NOTES:
1. TOPOGRAPHY IS BASED ON FOLLOWING MAPS
1.1 FROM THE SURVEY AND MAPPING BRANCH OF THE B.C. LANDS SERVICE:
- PLANIMETRIC MAPS NO 92 I/11e, /13a, /14c AND /14d, SCALE 1" = 1/4 MILE COMPILED AND DRAWN BY THE AIR DIVISION IN 1960 FROM OFFICIAL SURVEYS AND FROM AERIAL PHOTOGRAPHY 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.
LEGEND:
ONE WAY SURGE TANK
INDIAN RESERVE BOUNDARY
HEIGHT OF LAND
TRANSMISSION LINE
1 MILE
1000 2000 3000 4000 5000 FEET
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 KM
SANDWELL
DWG. NO. D4191/12-1
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
PIPELINE LOCATION - ROUTE E - SHEET 1
DWN SH
CHKD AFL
DATE DEC 1977
JOB No V4191
SCALE 1" = 0.25 MILE
DWG No D4191/12 1
D4191/12-2 PIPELINE LOCATION ROUTE E - SHEET 2
DWG. NO. REFERENCE DRAWINGS
REVISIONS



Hat Cr.
POWER LINE
Harry L.
PLANT SITE
PLANT RESERVOIR
Trachyte Hills
TP 21 R 26 W6
B-PARTLY FULL PIPE
A-FULL PIPE
SUMMIT ROUTE
SURGE TANK
MEDICINE CREEK ALTERNATIVE-FULL PIPE
MEDICINE CREEK IMPOUNDMENT & ASH CONTAINMENT
FOR CONTINUATION SEE REF. 1
NO. 1 PIT
Finney Cr.
Medicine Cr.
Medicine Creek
92 I 13a
NOTES:
1. FOR SOURCE OF TOPOGRAPHY SEE REF. 1.
2. FOR LEGEND SEE REF. 1.
MILE
FEET
KM.
SANDWELL
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN
PIPELINE LOCATION ROUTE E - SHEET 2
DATE DEC. 1977
JOB No V419
SCALE 1" = 0.25 MILE
DWG No D419/12-2
D419/12-1 PIPELINE LOCATION ROUTE E - SHEET 1
DWG. NO.
REFERENCE DRAWINGS
REVISIONS

SANDWELL

PROJECT MEMORANDUM V4191/13

PIPELINE STEEL SELECTION

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

B.C. HYDRO & POWER AUTHORITY
VANCOUVER B.C.

PROJECT MEMORANDUM V4191/13
PIPELINE STEEL SELECTION

DATE 15 DECEMBER 1977

SUMMARY

Only detailed engineering studies can make the final pipeline steel selection, however, as this item comprises some $5-6 million in cost for pipe supply, delivery, and welding, a preliminary specification is needed for cost estimate purposes.

Using information from various pipe mills, a reasonable preliminary specification can be made. In addition, a total direct cost analysis for pipe and welding cost components was made, and as a result the following steel selected:

Standard

CSA Z245.2 High Strength Steel Line Pipe 18 in. and Larger in Diameter.

Grade 60 Category II
Minimum Yield Strength 414 MPa (60,000 psi)
Minimum Tensile Strength 518 MPa (75,000 psi)

Deviations: Maximum carbon equivalent 0.40
Maximum carbon content 0.12
Impact test - Charpy v-notch, full size, 54 J at -4°C (40 ft lb at +25°F).

CODE SELECTION

The following organizations write codes which were considered:

AWWA - American Water Works Association

C201- Fabricated Electrically Welded Steel Water Pipe
- covers pipe and steel, similar to API for steel.

C201- AWWA Standard for Mill Type Steel Water Pipe - covers pipe and steel, similar to API for steel.

ASTM - American Society for Testing and Materials

A516-74a Pressure Vessel Plates, Carbon Steel, for Moderate and Lower Temperature Service.
- for welded pressure vessels where impact is important.

A537-74a Pressure Vessel Plates, Heat Treated, Carbon-Manganese-Silicon.
- normalized or quenched and tempered.

API - American Petroleum Institute

API-5LX - Specification for High Test Line Pipe
- materials and pipe fabrication.

ASME - American Society of Mechanical Engineers

ASME Boiler and Pressure Vessel Code Section VIII
- covers materials, design, fabrication and erection of unfired pressure vessels.

CSA - Canadian Standards Association

Z245.1 General Requirements for Plain End Welded and Seamless Steel Line Pipe.
- covers general pipe specification and testing.

Z245.2-1974 High Strength Steel Line Pipe 18 inches and larger in diameter.
- covers material and pipe fabrication.

The CSA standard is very similar to the API-5LX standard, however for Canadian conditions, especially for cold temperature services, the Canadian standard is superior. Either CSA or API standards should be used, as they are more commonly specified than AWWA, ASME or ASTM, and mills are thus more familiar with their requirements.

GRADE SELECTION

The choice of steel grade from within the range included in CSA Z245.2 is as shown on Table 1.

Table 1 - Steel Grades

Grade	Minimum Yield Strength		Minimum Tensile Strength		Allowable Stress*	
	psi	MPa	psi	MPa	psi	Mpa
52	52,000	359	66,000	455	22,000	152
56	56,000	386	71,000	490	23,600	163
60	60,000	414	75,000	518	25,000	173
65	65,000	448	77,000	531	25,600	177
70	70,000	483	82,000	574	27,300	188

* For normal conditions, as defined in Design Criteria, equals lesser of Yield Strength x 0.8 or Tensile Strength x 0.33.

The following recommendations were received during the course of studies to date:

Steel Co. of Canada (Meeting 5 October 1977, letter 18 October 1977, File 272.34)

Recommend grades 60 to 70, to have the most range for adjustment of steel chemistry to tailor it to the specific purposes. Recommend pearlite-reduced, or especially, acicular ferrite microstructure, such as, in particular, the letter recommends steel types as follows:

1. A 0.12% carbon maximum, ferrite - pearlite type steel, which was used for 1067 mm (42") O.D. x 9.7 mm(.380") wall Gr. 65 for B.C. Hydro.

2. A 0.10% carbon maximum pearlite - reduced molybdenum bearing steel, currently being used for Gr. 70 line pipe.

Table 2 shows the available wall thickness for 920 mm O.D. pipe from this supplier.

Table 2 - Steel Co. of Canada Pipe Supply

Welland - Stelform Pipe Mill 920 mm. O.D. (1)

Grade	Thickness (mm) at Limit of Supply	Maximum Required Thickness (mm) for 7.91 MPa
42, 46 (2)	38.1	24, 23
52	31.5	22
56	29.0	20
60	26.4	19
65	24.9	19
70	23.1	18
80 (2)	20.3	16

(1) Minimum I.D. from Stelform Mill 882.7 mm. UO Mill cannot meet wall thickness requirements.

(2) Not covered by CSA Z245.2

Therefore, the Stelco supply limits have no influence on the selection of steel grade, at this diameter of 920 mm O.D.

Algoma Steel (Letter, 8 August 1977)

Limits of plate supply may influence Grade selection, as shown on Table 3:

Table 3 - Algoma Steel Plate Supply

Grade	Thickness (mm) at Limit of Supply	Maximum (1) Required Thickness (mm)
52	36.3	19
56	27.1	18
60	19.2	17
65	12.5	17*
70	11.7	16*

* Required thickness exceeds available supply
(1)For nominal 800mm diameter pipe, for 7.91 MPa.

Therefore, if the pipe were made from plate from this supplier, Grade 60 would be the maximum possible to supply the maximum plate thickness requirement.

Interprovincial Steel and Pipe Co. (See V4007 - Minutes of Meeting 3).

Recommend use of Grade 70 pipe. Limit of supply is at 14.3 mm for 863 and 914 mm O.D., and 12.7 mm for 813 mm O.D. size, regardless of grade selected.

Mannesmann Pipe and Steel Corp.

Suggest use of higher grade, such as 70, to reduce total tonnage of steel. Limits of supply do not affect grade selection.

B.C. Hydro Gas Division - (See V4007 - Minutes of Meeting 2)

Recommend extensive study to select steel, current practice: have used Grade 65 successfully.

Nippon Kokan K.K. (Minutes of Meeting 11)

Have supplied Grades 42 to 75, but for this application recommend Grade 60 or lower. Limits of supply from UOE Mill - 25.4 mm wall thickness regardless of grade, for 813-914 mm O.D.

Sumitomo Metal Industries, Ltd. (Letter - undated)

Grade 70 and heavy wall thicknesses of Grade 65 could have some bainitic* microstructure, therefore use Grade 60 or lower. Limits of supply - 38.1 mm wall thickness regardless of grade, for 813-914 mm O.D.

Williams Brothers Canada Ltd. (See V4007 - Minutes of Meeting No. 1)

Recommend Grade 52 or 56 to avoid problems of welding and field bending, particularly prevalent with thin-walled pipelines.

These recommendations, although not completely consistent, suggest -

1. Avoid grades that are too low (less than 56) due to wall thickness becoming excessive. Also, the necessity for controlled chemistry for weldability and impact requirements, automatically gives higher yield strengths.

2. Avoid grades that are too high, (i.e. 70), to avoid thin-wall bending and welding problems, also to avoid bainitic microstructure.

3. Recommended range is generally Grade 56, 60 and 65. Therefore, a brief cost analysis was made, including the cost of pipe supply, delivery, and welding, for these three grades. The results are shown on Table 4.

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* Bainite is an undesirable microstructure constituent which lowers the impact toughness of steel.

Table 4 - Steel Grade Cost Analysis

For Pipeline Profile as shown on Sketch D4191-SK64 dated October 77.

Grade	Pressure MPa	Wall Thickness (1) mm	Length km	Unit Cost (2) $/m	Total Cost ($1000's)	Total Difference ($1,000's)
56	3.0	8	8.83	202.5	1790	
	4.9	12	10.77	273.4	2940	
	7.3	18	3.03	380.5	1150	+500
					5880	
60	3:0	8	8.83	205.4	1810	
	4.9	12	10.77	277.7	2450	
	7.3	17	3.03	368.8	1120	
					5380	o Least Costly
65	3.0	8	8.83	205.6	1820	
	4.9	11	10.77	273.4	2940	
	7.3	17	3.03	380.4	1150	
					5910	+530

(1) To nearest mm. - Mill tolerance

(2) Unit cost based on 900 D. pipe, for which comparative data are available. According to Sumitomo, F.O.B. Vancouver, import duty paid, includes F.S.T., P.S.T., freight to Ashcroft. Welding included (at $68-$82/m) depending on wall thickness but not steel grade.

Therefore, Grade 60 apparently saves about $500,000 over Grades 56 and 65, and as it also falls within the range of most recommendations, has been selected for Preliminary Engineering purposes.

Impact Requirements

The pipeline will be buried, and normally will only be stressed when filled with water. Therefore, when installed, the pipe steel is unlikely to fall below 0 °C, both due to the water it carries and the depth of burial generally below frost line.

The primary purpose of impact toughness requirements is to prevent crack growth, so that if a small crack forms, either due to a construction oversight, or accidental impact (for example, accidental strike by a backhoe), it would not be able to grow into a major leak.

Crack growth criteria are the subject of intensive study by a CSA Code Sub-Committee. When a crack is less than a certain size, it will not grow when the pipe is stressed to the indicated level, according to the following criteria, as shown on Table 5.

Table 5 - Crack Growth Criteria

Courtesy: F. Christensen, Steel Co. of Canada.

Pipe Size: 914 mm. O.D.
Pipe Stress: 0.4 x yield strength
Impact Toughness: Charpy 2/3 size test: 27 J at - 4 °C (20 ft. lbs at 25 °F)

Wall Thickness (mm)	Through-thickness* Critical Flaw size (mm)
19	315
6.4	274

* The length of a flaw which passes completely through the thickness of the plate, such that if the length were any greater, the crack would increase in size at the stated stress level.

Therefore, for example, if the pipeline at its maximum wall thickness of say 19 mm, were punctured while operating at the design stress level, the puncture would not grow if less than 315 mm long.

It is feasible to specify impact toughness requirements exceeding those of Table 5, and thus increase the critical flaw size. Toughness up to about 54 J (40 ft. lbs) should be attainable for the required wall thickness and temperature. Therefore this higher attainable level is assumed for cost estimate purposes.

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SANDWELL

PROJECT MEMORANDUM V4191/14

PUMPING SYSTEM - INTAKE TO CLARIFIER

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 24 JANUARY 1978

PROJECT MEMORANDUM V4191/14
PUMPING SYSTEM - INTAKE TO CLARIFIER

PURPOSE

The purpose of this Project Memorandum is to investigate the pumping system between the intake and the degritting clarifier.

GENERAL

The design of the delivery system from intake to clarifier, as shown on the attached Drawing A4191/14-1, is governed primarily by the maximum permissible approach velocity to the intake travelling screen of 0.12 m/s (0.4 ft/s) at minimum river level* as stipulated by Environment Canada. The intake cells, intake pump and pipeline to clarifier - the major components of the pumping system - are discussed below to determine how Environment Canada's stipulation can be met.

The flow imbalances caused by the variation in the river level and by intake pumps feeding booster pumps are also discussed.

INTAKE CELLS AND INTAKE PUMPS

If the intake cell dimensions used in the model study are retained, the flow at minimum river level must be limited to 350 l/s (5,500 USGPM) for which each cell is designed. The five intake pumps are selected to deliver 330 l/s (5,250 USGPM) each at the condition of minimum river level combined with total discharge flow from all five pumps. If at minimum river level less than five intake pumps discharge into a single pipeline, the pipeline friction would be reduced, thereby increasing each remaining pump's discharge flow. The increase in flow due to reduced friction must not exceed 350 l/s (5,500 USGPM) per cell to satisfy Environment Canada's stipulations.

To ensure that the pump discharge flow does not exceed 350 l/s (5,500 USGPM) with only two** pumps operating, the decrease in pipeline friction shall be limited to 2.7 m (9 feet) as shown on the pump curve of Drawing A4191/14-2. In other words, the pipeline diameter (from intake to clarifier) will have to be sized such that the friction head increase when all five pumps are running is not more than 2.7 m (9 feet) over the friction head when only two pumps are running.

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* Sandwell selected the one in 100 year minimum water level as the minimum design river level.

** A minimum of two intake pumps would be used since one intake pump operating alone would not be able to match the flow of a single booster pump. See Drawing A4191/14-3.

Since the correct choice of pipeline diameter will ensure that stipulated velocities through the travelling screens are not exceeded, control valves to limit flow would not be required.

PIPELINE FROM INTAKE TO CLARIFIER

The optimum pipeline diameter between the intake and clarifier must satisfy the following criteria:

a. Minimum velocity in the pipeline shall be 0.9 m/s (3 ft/s) to prevent settling of solids.

b. Maximum velocity in the pipeline shall be 3 m/s (10 ft/s) to keep wear of pipe and valves to a minimum.

c. Friction between maximum and minimum flow in the pipeline at minimum river level shall be less than approximately 2.7 m (9 feet) as outlined earlier.

The attached Table 1 gives the velocities and friction head for various pipeline diameters considered for the pipeline from the intake to the clarifier. From this table it is apparent that the 900 m (36 inch) diameter line best meets the above criteria. It is also a more standard diameter for low pressure pipeline than either 850 mm (34 inch) or 950 mm (38 inch).

Therefore, it is recommended that the pipeline from the intake to the clarifier be a single 900 mm (36 inch) diameter line.

MATCHING INTAKE AND BOOSTER STATIONS

As shown on Drawing A4191/14-1, the intake pumps feed the clarifier which overflows into the clearwell located before the booster pumps. Any excess not required by the booster pumps and other services would overflow from the clearwell to the river.

With intake pumps feeding booster pumps, there is an excess flow delivered by the intake pumps to the clearwell. The amount of excess varies with mode of operation and river level. The amount of excess capacity of the intake pumps is shown on Drawing A4191/14-3 with flows extracted from attached Drawing D4191/14-4.

Drawing A4191/14-3 indicates that the excess capacity is minimal during low water levels occurring in winter but quite substantial during the high water levels of summer. This excess could be either overflowed from the clearwell back to the Thompson River or eliminated by control valves on each pump. The capital cost is approximately equal for each method of handling the excess.

Control valves are undesirable because of extra maintenance requirements, reduced reliability, wear of control components and dependence on controls.

With overflowing of the excess, there is some wasted power but it is estimated to have a present worth of only $150,000.

Therefore, it is recommended that the excess be overflowed.

SUMMARY

It is recommended that:

1. During Final Design, care be taken to ensure that velocity through the travelling screens does not exceed the maximum as stipulated by Environment Canada. Specifically, that preference be given to the selection of a pump with a steep head-capacity curve and that the permissible differential friction head be confirmed on the basis of the pump curve of the selected equipment.

2. The pipeline from intake to clarifier be a single 900 mm (36 inch) diameter line.

3. The excess flow from the intake pumps be overflowed to the river.

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PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/14

DATE 24 JANUARY 1978

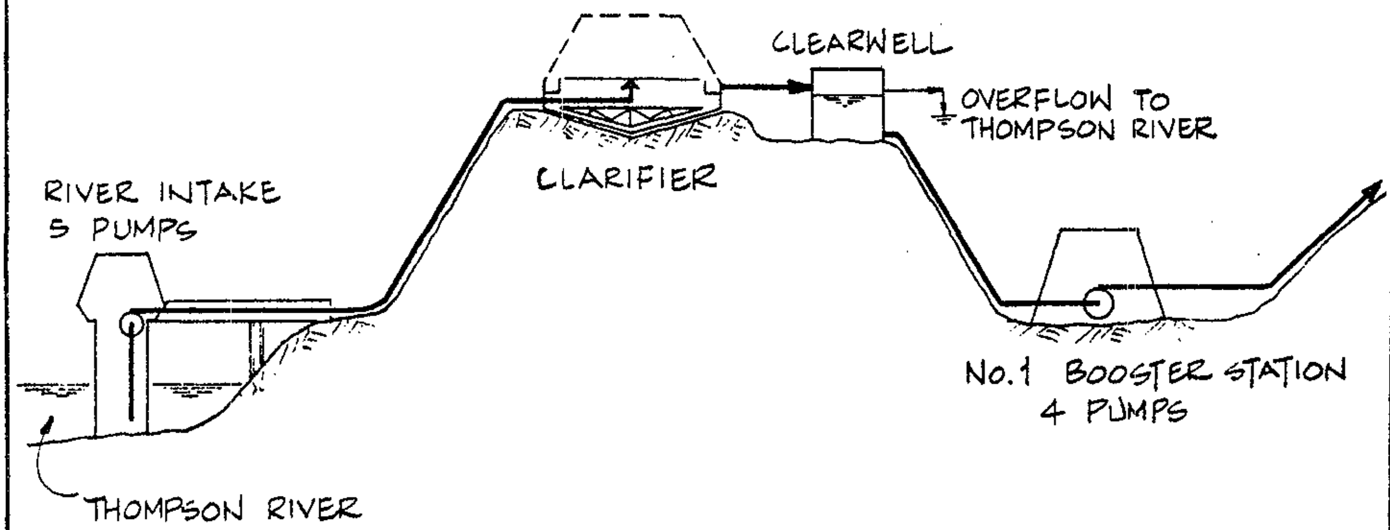
TABLE 1 - PIPELINE SIZING - INTAKE TO CLARIFIER

Pipeline Diameter mm (inches)	Minimum* Velocity m/s (ft/s)	Maximum** Velocity m/s (ft/s)	Maximum Differential Friction*** Head m (ft)
800 (32)	1.4 (4.4)	3.6 (11.9)	4.5 (14.9)
850 (34)	(1.2) (3.9)	3.2 (10.6)	3.4 (11.1)
900 (36)	1.1 (3.5)	2.9 (9.4)	2.5 (8.3)
950 (38)	0.96 (3.1)	2.6 (8.5)	1.9 (6.3)
1000 (40)	0.86 (2.8)	2.3 (7.6)	1.5 (4.9)
Preferred Values			
	more than 0.9 (3.0)	less than 3.0 (10.0)	less than 2.7 (9.0)

* Minimum river level, two pumps operating.
** Maximum river level, five pumps operating.
*** Difference in total friction head at minimum river level between maximum (five) and minimum (two) number of pumps operating.

$$H_f = 0.015 \frac{L}{D} \frac{V^2}{2g}$$

and : Not acceptable.



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

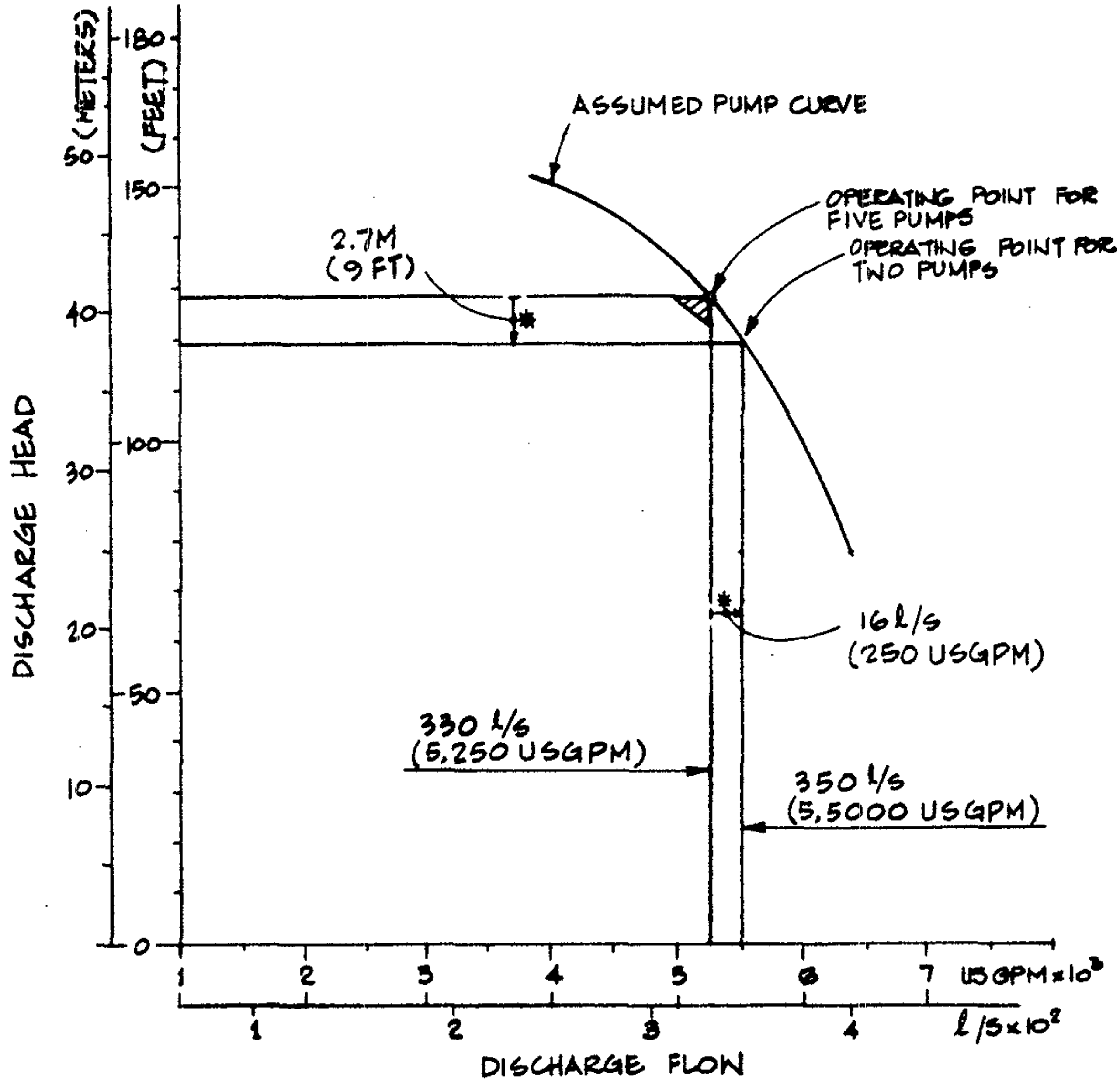
			DAY	MO.	YR.
APPROVED	SCALE				
DATE	DR'N.	D. TUNG	5	12	77
	CH'K'D.	R. KALLBERG			
A 4191/14-1	APP'D.				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
SYSTEM — INTAKE TO No.1 BOOSTER STATION

SANDWELL

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

DWG. A 4191/14-1 REV



* AT MAXIMUM STATIC HEAD (MINIMUM RIVER LEVEL) AND FOR A 900 mm (36") DIAMETER PIPELINE.

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CH'K'D. R. KALLBERG

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A 4191/14-2

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE PUMP - TYPICAL CURVE

B.C. HYDRO AND POWER AUTHORITY
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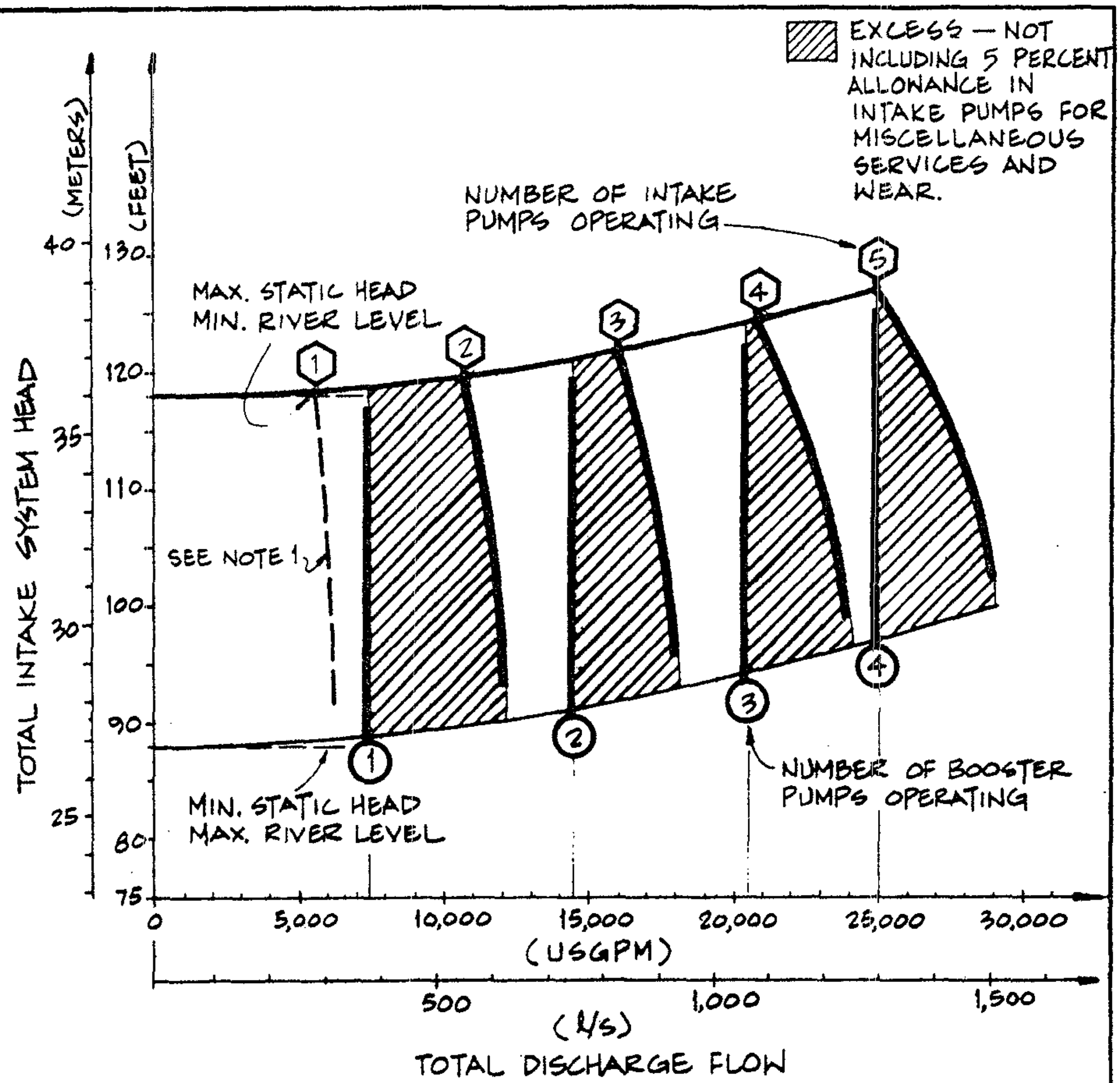

NOTES.
1. TWO INTAKE PUMPS MUST BE RUNNING TO MEET DEMAND OF ONE BOOSTER PUMP OPERATING ALONE.
2. PUMP DATA TAKEN FROM D4191/14-4

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HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE & BOOSTER PUMPS — MATCHING

SANDWELL

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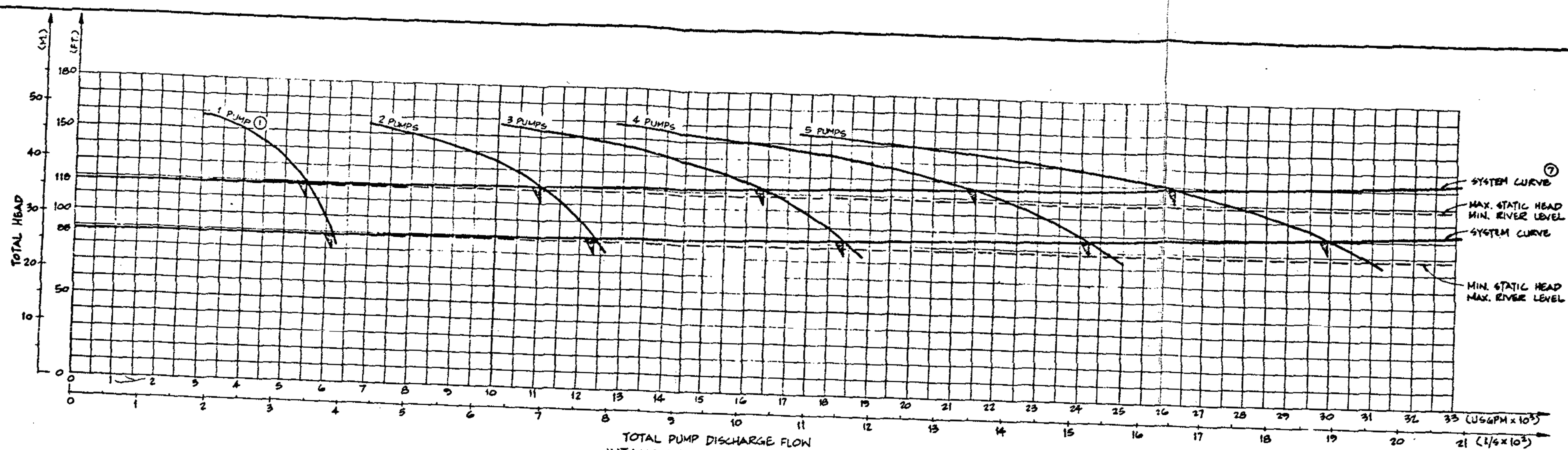
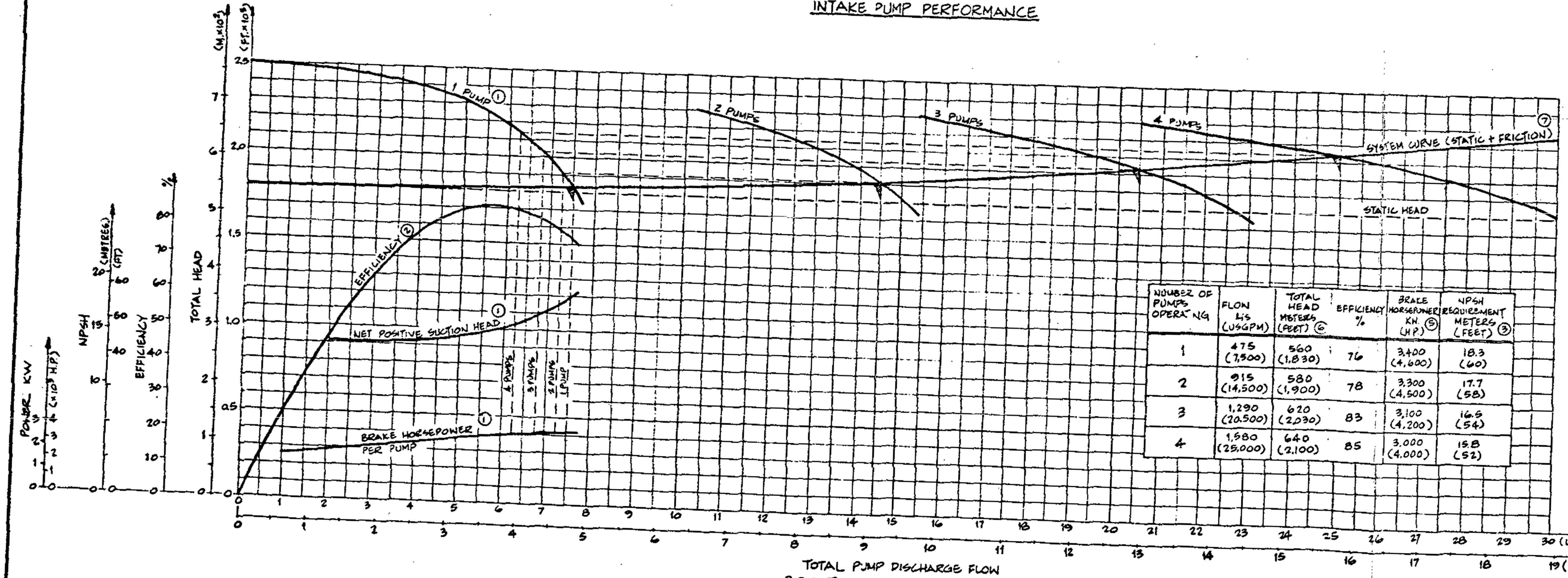


INTAKE PUMP PERFORMANCE



NUMBER OF PUMPS OPERATING	FLOW L/S (USGPM)	TOTAL HEAD METERS (FEET) ⑥	EFFICIENCY %	BRAKE HORSEPOWER KW. (H.P.) ⑤	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)

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 900mm⌀ (36") PIPELINE FOR INTAKE PUMP AND 800mm⌀ (32") PIPELINE FOR BOOSTER PUMPS.

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REMARKS | DATE | MADE | CHKD | REVISIONS

DES | DWN D.TANG | CHKD | DATE DEC. 1977

SANDWELL | DWG. NO. D 4191/14-4

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

COOLING WATER SUPPLY

PRELIMINARY DESIGN STUDY

BOOSTER & INTAKE PUMPS - PERFORMANCE CURVES

DWG NO. D 4191/14-4 | R

PROJECT MEMORANDUM V4191/15

PIPELINE - LEAK DETECTION

(V4191/1)

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

DATE 22 DECEMBER 1977

PROJECT MEMORANDUM V4191/15
PIPELINE - LEAK DETECTION

PURPOSE

The purpose of this Project Memorandum is to record the information originally presented in a letter dated 10 December 1977.

In a letter of 20 September 1977, from Mr. Waite of B.C. Hydro's Generation Planning to Mr. Y.I. Fellman of the Department of Environment, it was stated:

"Controls will be designed to detect leaks, to immediately shut down the system in the event of a small leak, and to limit the length of line that would drain in the unlikely occurrence of pipe rupture."

To meet the above criteria, Sandwell has surveyed leak detection technology in order to determine a suitable system for the Hat Creek cooling water supply pipeline.

Sandwell feels that the probability of pipe leakage which could have an adverse effect on the environment is very low, given the extent of geotechnical examinations and pipeline inspection procedures envisioned, and given the appropriately conservative design code to be used. This Project Memorandum reports on what can be done and the approximate cost.

GENERAL

Leak detection systems can be classified according to whether they operate periodically or continuously, and these classifications are discussed separately. Further distinctions are made between methods that require the pumping operation be stopped, between those that detect large or small leaks, and those that locate or do not locate leaks.

PERIODIC LEAK DETECTION METHODS

Methods for periodic leak detection are divided according to whether they work while the pipeline is in or out of operation.

Pipeline Watered, but not in Operation

1. Acoustic Pig

Acoustic surveying with an internal probe (pig) is based on recording leak noise as a function of pig position. This method is successful for small leaks, as such leaks are known to produce high frequency vibration. The cost of one survey, excluding any cost associated with the pig launching facility (which would be installed anyway), is expected to be between $10,000 and $20,000, on a contracting basis. This cost would include a complete graphical representation of leak position.

Cost references were not available for purchasing such a system.

2. Pressure Testing

Pressure testing requires a long testing period in order to detect the accumulated effect of fluid loss. Prior to the testing period it is necessary to allow for thermal stabilization as the ground and water temperatures are normally different. Due to expected difficulties with establishing a typical thermal stabilization period this method is not suitable for small leaks.

3. Line Volume Balance

The same comments regarding thermal stabilization apply to this method.

Pipeline in Operation

1. Acoustic Survey

Acoustic surveying is accomplished by "listening" to the pipeline, from the surface, with appropriate vibration sensors and amplifiers. Leaks produce a characteristic noise, and thus can be located as well as detected. This method is currently successfully employed for locating leaks in municipal water distribution systems (Vancouver, Penticton). Even very small leaks at depth can be found.

A drawback of this method is that close to pumping stations, high frequency bearing noise may interfere, so that it may be necessary to test these sections when the pumps are not operating. The high static pressure would make it practical to detect leaks while not operating.

The total expected cost of this equipment is about $7,000. Regular maintenance personnel could be trained to operate it. Alternatively, this testing may be accomplished by outside consultants for about $2,000 per survey.

2. Line Volume Balance

Line volume balancing may be accomplished by integrating flow measurements and correlating them with tank level changes. Leak detection limits are set by instrument accuracy. A computer is required to make the balance calculations. This method is more sensitive to medium to large leaks.

The advantage of this system is that the testing frequency can be increased as desired. With increasing frequency, periodic testing would virtually become continuous.

A major disadvantage is that leak position cannot be determined by this method.

Most of the equipment necessary for this method would be already provided for process control functions.

CONTINUOUS LEAK DETECTION METHODS

1. Rate-of-Pressure Drop

This method consists of measuring the rate of pressure drop at various points along the pipeline. Leaks show up as rapid or slow pressure losses, depending on their magnitude. The sensors themselves do not restrict the resolution of the system, rather the problem is that any transient condition could set off alarms. Therefore this method is de-activated during transient conditions, such as adjustment of valves or stopping and starting of pumps. Moreover, the pressure drop which would cause an alarm would be set at a fairly high level. Thus, the rate of pressure drop method is not suitable for leaks smaller than about 60 l/s (1000 USGPM).

Medium sized leaks, up to and including pipe ruptures, can easily be detected by this method. Depending upon the spacing of the sensors, some degree of leak location is also achieved.

The cost of this system with 10 rate-of-pressure drop units would be approximately $100,000 and the resulting leak location definition would be about 2.4 km (1.5 miles).*

2. Flow Metering

This method would compare the readings from flow meters on each end of a reach of pipeline to detect leaks. The system must be de-activated during transients.

Overall accuracy of flow loss detection is in the order of 0.5 percent, and thus leaks of 8 l/s (125 USGPM) can be detected without much difficulty.

The disadvantages of this system are:

- Leak location is not possible between metering points.
- Extra meters are needed each side of surge tanks, as metered sections should not contain surge tanks with fluctuating levels.

Approximate equipment cost for each metered section is about $12,000, with three surge tanks for waterhammer protection, a minimum cost of this method would be $200,000, based on five metered sections.

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* Cost estimates include installation, which was omitted from those costs quoted in the 10 December 1977 letter.

3. Mass Balancing - Computerized

The flexibility of this system is limited only by computing capacity and the number of sensing devices used.

The computer would process inputs such as flow at each line section, temperature and pressure, and thus would calculate leakage even during transient conditions, by taking account of thermal expansion and pipe size variation with pressure.

Small leaks could be detected by mass balancing during steady state conditions whereas medium to large leaks would be detected during steady or transient conditions by sudden pressure drops. Some measure of leak location would be possible for large leaks, depending on the number of pressure sensors.

The equipment cost components of this system, excluding the computer, are as follows:

- For each line section metered	$12,000
- For each pressure sensor required	$ 2,000
- For each temperature sensor required	$ 2,000
- Program for computer	$10,000

A simple system based on 5 flow sections, 10 pressure sensors, and 10 temperature sensors would cost approximately $300,000 including installation but excluding the computer.

4. Permanent Acoustic Sensors

This method would be a more sophisticated version of the method described under Periodic Acoustic Survey. Instead of "listening to" the line from the surface, permanent vibration sensors would be attached to the pipe and the signals are processed by a central unit.

Sensors would be spaced according to the acoustic attenuation in a pipeline. Suggested spacing for an 800 mm diameter (32 inch) water line is about 300 m (1,000 feet) or less.

The central processing unit would scan the sensors and display the position being monitored on a visual readout unit. Unless unusual vibrations are encountered the scan proceeds to the next sensor.

When a leak occurs the vibration intensity triggers the scanner to stop and to sound an alarm. An operator then could scan the sensors manually to locate and confirm the leak by comparing vibration intensity levels from several sensors. The operator could also listen to the amplified sound, and thus could check the validity of the alarm.

A computer could be used to advantage, but is not necessary.

The advantages of this system are:

- High resolution for location of leaks.
- High sensitivity, as small leaks at high pressure produce distinctive high frequency vibrations.

- High versatility, as the system may be used when pumps are not operating or during transient flow conditions.
- High cost effectiveness as acoustic sensor cost is about one tenth that of flow measuring devices.

A disadvantage is that false alarms may be initiated by other acoustic sources. These alarms, however, are not likely to persist unless the source is a permanent feature of some new installation on the pipeline site. Permanent acoustic sources, such as bearing noise, can be filtered out.

The approximate cost of a fully automatic acoustic leak detection system consisting of 100 sensing points is $140,000 including the telemetry to the control room.

CONCLUSIONS

The various systems discussed above are summarized on Table 1, attached.

Sandwell recommends that for Preliminary Design purposes, a system which can locate and continuously detect even small leaks be included in the Cost Estimate. However, the necessity of this system should be reviewed during Final Design.

Among the continuous detection methods, the Permanent Acoustic System provides a very high resolution of leaks and their location, is simple to understand and operate, and has a modest cost. Therefore Sandwell recommends it for use in the Hat Creek pipeline.

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A. Copeland, P. Eng.

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

Table 1 - Summary of Leak Detection Methods

Category	Method	Leak Size Resolution	Leak Location Resolution	Approximate Cost ($1000)	Comments
Periodic					
Pipeline not operating	1. Acoustic pig	Any	Excellent	10-20 per survey	
	2. Pressure testing	Medium to large	Nil	Not determined	*
	3. Line volume balance	Medium to large	Nil	Not determined	*
Pipeline in Operation	1. Portable acoustic detector	Any	Excellent	2 per survey or 13 for equipment	
	2. Line volume balance	Medium to large	Nil	Not determined	
Continuous **					
	1. Rate-of-pressure drop	60 l/s (1000 USGPM)	2.4 km (1.5 miles)	100	
	2. Flow metering	8 l/s (125 USGPM)	About 4-6 km with 5 sections	200 with 5 sections	
	3. Mass balance-computerized	Any	2.4 km	300	Computer not included
	4. Permanent acoustic sensors	Any	300 m (1000 feet)	140	

* Thermal stabilization problem.

** Cost for these methods estimated within about $50,000, and includes installation cost omitted from letter of 10 December 1977.

PROJECT MEMORANDUM V4191/16

SELECTION OF MEDIUM VOLTAGE

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/16
SELECTION OF MEDIUM VOLTAGE

DATE 22 DECEMBER 1977

PURPOSE

The purpose of this Project Memorandum is to select the medium voltage supplying the intake and booster pump motors.

INTRODUCTION

The selection of the medium system voltage was based on the requirements at No. 1 Booster Station and the Intake where widely varying motor ratings must be accomodated. The six intake pump motors (five installed, one future) are rated at 225 kW (300 HP) while the four booster pump motors are rated at 3700 kW (5000 HP).

ALTERNATIVES

The following three alternative combinations of voltages were identified for investigation:

Alternative No.	Booster Pump Motor Voltage	Intake Pump Motor Voltage
1	6600 or 4000	575
2	6600	6600
3	4000	4000

COST COMPARISON

The following table identifies those costs for the intake only which vary with the selected voltage:

Item/Alternative	1	2	3
Intake Supply Voltage	6600 or 4000	6600	4000
Intake Pump Motor Voltage	575	6600	4000
Intake Pump Motors (6-300HP/1200 RPM)			
Cable including Ducts & Tray	$ 90,000	$ 132,000	$ 105,000
Starters	18,000	108,000	39,000
	40,500	3,500	4,500
2.5 MVA Power Transformer	26,000	-	-
750 kVA Power Transformer (on intake)	-	13,000	13,000
Capitalized Transformer Losses	13,500	-	-
Relative Cost	$188,000	$ 256,500	$161,500
Cost Index	1.16	1.59	1.00

DISCUSSION AND RECOMMENDATIONS

The 4000 V rating was selected for the following reasons:

1. The Thompson River Intake would be subfed on the selected medium voltage level from the No. 1 Booster Station. If 6600 V were chosen, there would be a limited availability and a high price premium for the intake pump motors because of their low kW-rating. The use of 575 V pump motors would require a fully-rated transformer which would be difficult to accommodate on the intake structure. Locating this power transformer away from the intake on top of the river bank would make this scheme uneconomical due to high 600 V feeder cable costs.

2. A maximum fault level of 1000 MVA (B.C. Hydro advised a preliminary minimum fault level of 475 MVA) at the primary bushings of the 20 MVA power transformer with standard impedance will yield a secondary fault level of less than 350 MVA. This is a standard fault level for 4160 V switchgear of both the magnetic air breaker or minimum oil breaker type. Therefore, from an interruption capacity point of view, there is no reason to use a voltage higher than 4160 V.

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

(PM V4191/16)

PROJECT MEMORANDUM V4191/17

PUMPS AND PUMP WEAR

PROJECT V4191
HAT CREEK PROJECT
COOLING WATER SUPPLY

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

PROJECT MEMORANDUM V4191/17
PUMPS AND PUMP WEAR

DATE 24 JANUARY 1978

PURPOSE

The purpose of this memorandum is to discuss how suspended solids affect pump wear, why preference is given to the selection of low lift intake pumps rather than high lift intake pumps, and why a degritting system is required before the booster pumps.

INTRODUCTION

The intake pumps would withdraw water from the Thompson River near Ashcroft. As discussed in Project Memorandum V4191/5, Water Treatment, the Thompson River has at times a very high suspended solids load. To ensure that the water supply system functions under all river conditions it must be designed to operate throughout the year regardless of the amount of solids present in the water. This memorandum outlines the provisions which should be made for the Hat Creek Cooling Water Supply System to ensure acceptable reliability and performance.

EFFECT OF SUSPENDED SOLIDS ON PUMPS

Wear occurs on all internal pump parts when solids are present in the pumped liquid. Normally, wear occurs at the wear rings but if the particles are abrasive and large enough, they could also cause wear of the pump's impellers, shaft and casing.

Since the wear rate is proportional to the size of the particles, care must be taken to remove particles as large as economically possible.

Drawing A4191/17-1 shows, in an enlarged scale, the size of particles that could be expected in the intake cell. The largest particle, 2500 microns in diameter, is the maximum size which could pass through the travelling screen cloth. Although most particles of this size would settle in the intake cell, some could get into the intake pumps and damage the wear rings.

All pumps are built with wear rings which are designed to keep leakage between high and low pressure regions to a minimum. Replaceable wear rings are installed in critical positions and are intended to wear instead of the pump's casing and shaft.

Particles which are of a size close to the wear ring clearance may pass between the rotating surfaces and abrade the surface metal, eventually widening the clearances and lowering the pump's efficiency and flow capacity. Since standard wear ring clearances range from 300 to 500 microns, particles close to this size and larger should be removed from the pumped liquid. For the intake pumps, since there are no means to remove the suspended solids, the pump design must ensure that they are not adversely affected by the solids.

The booster pumps, on the other hand, could be protected from solids by some means of degritting. If water treatment is not provided before the booster pumps, the suspended solids could damage the pump's wear rings and internal elements.

The decrease in a pump's head and flow capacity due to wear of the wear rings is shown on Drawing A4191/17-2. It can be seen that wear has a more serious effect on a system such as Hat Creek. This is because friction is a relatively small portion of the total dynamic head resulting in a flatter system curve. Therefore, more precautions must be taken to protect the Hat Creek pumps from wear because of their sensitivity to wear.

INFORMATION ON WEAR

In the course of the study on pump wear, extensive information was gathered, including supplier's information and published articles. Assistance was also received from Sandwell's affiliate, Electrowatt Engineering Services of Switzerland.

The information included actual operating data, as well as many articles on pumps and wear. Many articles discussed proper pump design to minimize the effects of wear.

One of the most useful articles found on pumps and wear was "Storage Pumps and Glacial Waters" by A. Bezinge and F. Schafer*. This article describes various pumped storage systems in Switzerland with a total of 15 multistage pumps rated from 1265 to 2530 l/s at 300 m total dynamic head. The water was pumped from storage reservoirs and large lakes after passing through sand removal equipment. Despite the protective measures taken, severe wear occurred on all wetted parts of all units, necessitating major maintenance after only 80 to 240 days of operation. One pump was so badly worn that the rotating assembly could not be repaired. The efficiency of one pump dropped 22 percentage points because of wear. This rate of wear necessitated stocking of spare rotating assemblies for each pump.

Drawing A4191/17-3 attached (taken from the article) shows how widening of the wear ring clearances due to wear affected the efficiency of the pumps.

The authors concluded that in order to minimize wear, proper metallurgy must be used and the suspended solids must be removed prior to pumping.

A serious drop in efficiency for the Hat Creek Cooling Water Supply System could be expected if some means of solids removal were not provided before the booster pumps.

Several articles noted that the following relationships give pump wear rate fairly accurately:

a. Wear is proportional to V^3 or $(rpm)^3$ where V = Velocity
b. Wear is proportional to $H^{3/2}$ where H = Head per stage

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* Published in English by the British Hydromechanics Research Association, under Number T1019 and published originally in French in Bull. Tech. Suisse Romande Number 49 of October 1968.

These relationships, when applied to the Hat Creek Cooling Water Supply System yield the following wear indices:

a. rpm	rpm	Wear Index
Low lift intake Pump	900	1.0
	1,200	2.4
High Lift Intake Pump	1,800	8.0
Booster Pump	3,600	64.0

b. Head/Stage	Head (Meters per Stage)	Wear Index
Low Lift Intake Pump	25	1.0
High Lift Intake Pump	60	3.7
Booster Pump	300	42.0

From the above it can be seen how pump wear increases as the rpm and head per stage of the pump increase.

INTAKE PUMPS

The river intake pumps would be vertical, diffuser style, multistage units. Since the proposed river intake would be designed for fish protection and not wear prevention, little settling of solids would be expected to occur before the intake pumps. The pumps must therefore be selected to minimize the effects of wear from solids by:

- keeping rpm as low as possible
- keeping head per stage as low as possible
- use of abrasion-resistant materials for wetted parts

These are the most important design considerations which minimize wear and are best achieved with low lift intake pumps.

A low lift intake pump, on a service similar to the proposed would run several years before requiring maintenance. Based on the above wear relationships, a high lift pump which would operate at 1800 rpm, would be expected to last one-eighth as long as the low lift pump operating at 900 rpm. This is an unacceptable lifetime for any pump and dictates the use of low rpm and low lift intake pumps.

A standard intake pump would usually have water lubricated bowl bearings of either bronze or rubber. This arrangement is used on pumps handling clean liquids but would not last if very abrasive solids are present in the pumped liquid. To prevent rapid wear of the bowl bearings, the intake pumps require continuous purging of the bearings with clean water. The purge water could be provided externally by lines to each bowl bearing or internally by a rifle drilled shaft. As the number of stages increases, so does the complexity of the purge water system because a different pressure is required at each stage. A high lift intake pump with 6 to 15 stages would be more difficult to lubricate in this manner than a low lift pump with one or two stages.

The researched articles listed several metals which reduce overall pump wear rate, but the final choice would be dependant on availability from the chosen pump supplier and must be selected on the basis of hardness and cost. Inquiries were sent to pump suppliers to determine the availability of low and high lift vertical intake pumps. Of the 42 suppliers contacted, only eight could supply the high lift intake pump, while 20 could supply the low lift pump.

The users lists showed many installations of low lift vertical intake pumps on services similar to the proposed. On the other hand, there were very few high lift intake pump installations. Only 11 installations were found with a pumping head greater than 180 m (600 feet) but these were on relatively clean water.

Therefore, because of the problems associated with pumping suspended solids, high lift intake pumps were not considered suitable and were eliminated from further consideration.

BOOSTER PUMPS

The booster pumps would be horizontal, multistage units with single casing, and of volute or diffuser design. As pointed out earlier, because of the high rpm and high head per stage, wear would have a more severe effect on the booster pumps than on the vertical intake pumps.

Ten suppliers of high pressure pumps could manufacture pumps that met the head and flow requirements of the proposed cooling water supply system. From their installation lists, many pumps of a comparable size were found on services such as:

- boiler feed
- hydraulic debarking
- steel mill descaling
- pipelines
- waterflooding for oil fields

All these services pumped liquids that were either very clean or had been treated to remove solids.

All booster pump suppliers expressed concern about solids and recommended that some means of solids removal be employed.

Although the booster pumps could have good design features such as proper metallurgy of impellers and wear rings as with the vertical intake pumps, the high rpm and high head per stage remain and are the most detrimental features with respect to wear. Therefore, it is imperative that these pumps be protected by some means of grit removal.

Benefits of a grit removal system are that maintenance, repair and replacement costs would be greatly reduced. Provision of a grit removal system would also eliminate the need to stock costly spare rotating assemblies otherwise required because of the 18 month delivery period.

Power costs for booster pumps have been calculated to be $25,000 per year per percentage point of change in pump efficiency yielding a present worth of $300,000. It can be seen that the efficiency would not have to fall much in order to recover the costs of a degritting system.

The cost of a degritting system could also be justified by reduced maintenance and reduced spare parts.

RECOMMENDATIONS

Based on the above, Sandwell recommends that the water supply system be designed with low lift intake pumps followed by a degritting system before the booster pumps.

Prepared by R. T. Kallberg
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Approved by
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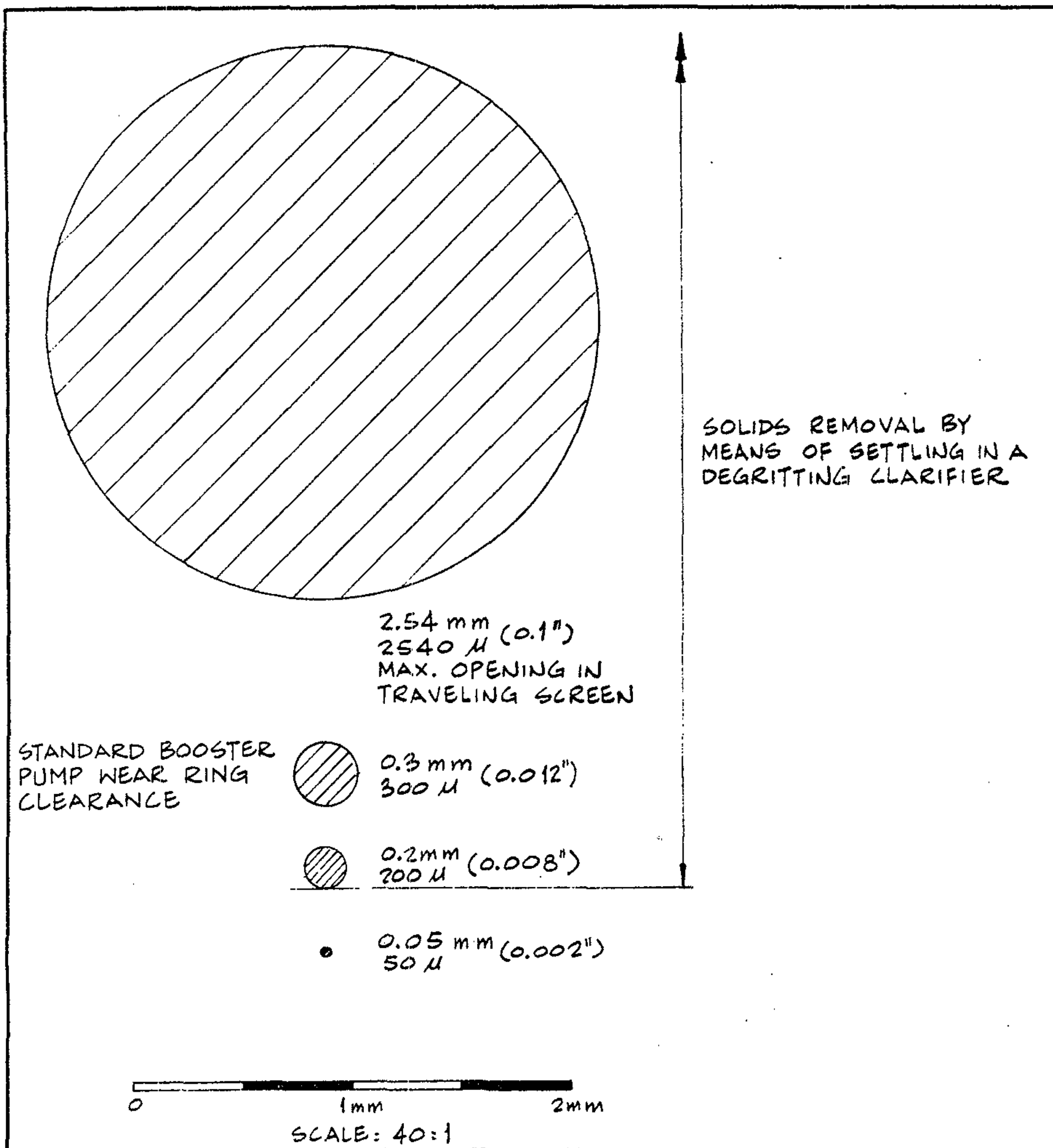

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A 4191/17-1

HAT CREEK PROJECT
COOLING WATER SUPPLY
THOMPSON RIVER INTAKE
RELATIVE SIZE OF RAW WATER SOLIDS

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

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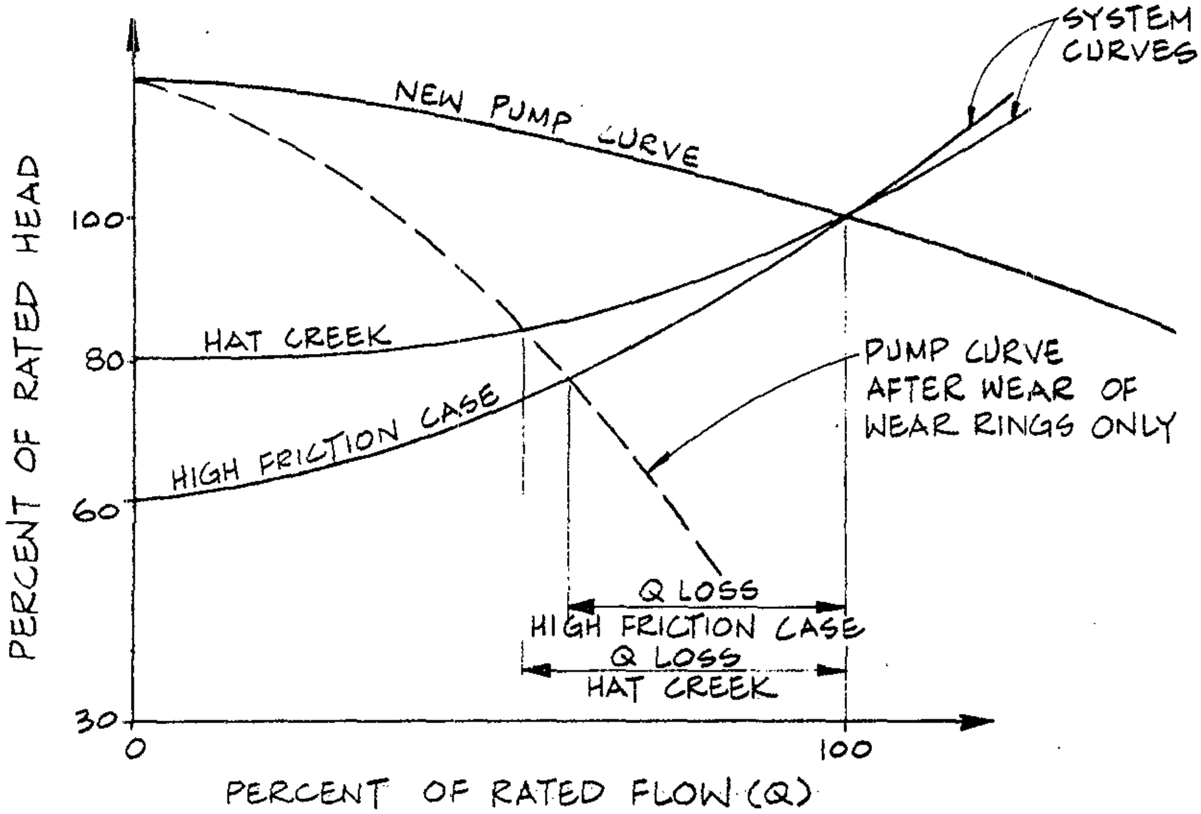
DWG. A 4191/17-1 REV

	HIGH FRICTION CASE (SEE NOTE 1)	HAT CREEK (AVERAGE VALUE)
TOTAL DYNAMIC HEAD (M.)	640	640
STATIC HEAD (M.)	385	520
FRICTION (M.)	255	120
FRICTION AS % OF TOTAL DYNAMIC HEAD	40	19



NOTE:

1. FOR THE HIGH FRICTION CASE, FLOW RATE AND DIAMETER ARE THE SAME AS HAT CREEK BUT FRICTION IS MUCH HIGHER BECAUSE OF MUCH LONGER LINE. THE COLSTRIP COOLING WATER SUPPLY SYSTEM IN MONTANA, USA., IS SIMILAR TO THIS HIGH FRICTION CASE.

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HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
PUMP WEAR

SANDWELL

B.C. HYDRO AND POWER AUTHORITY
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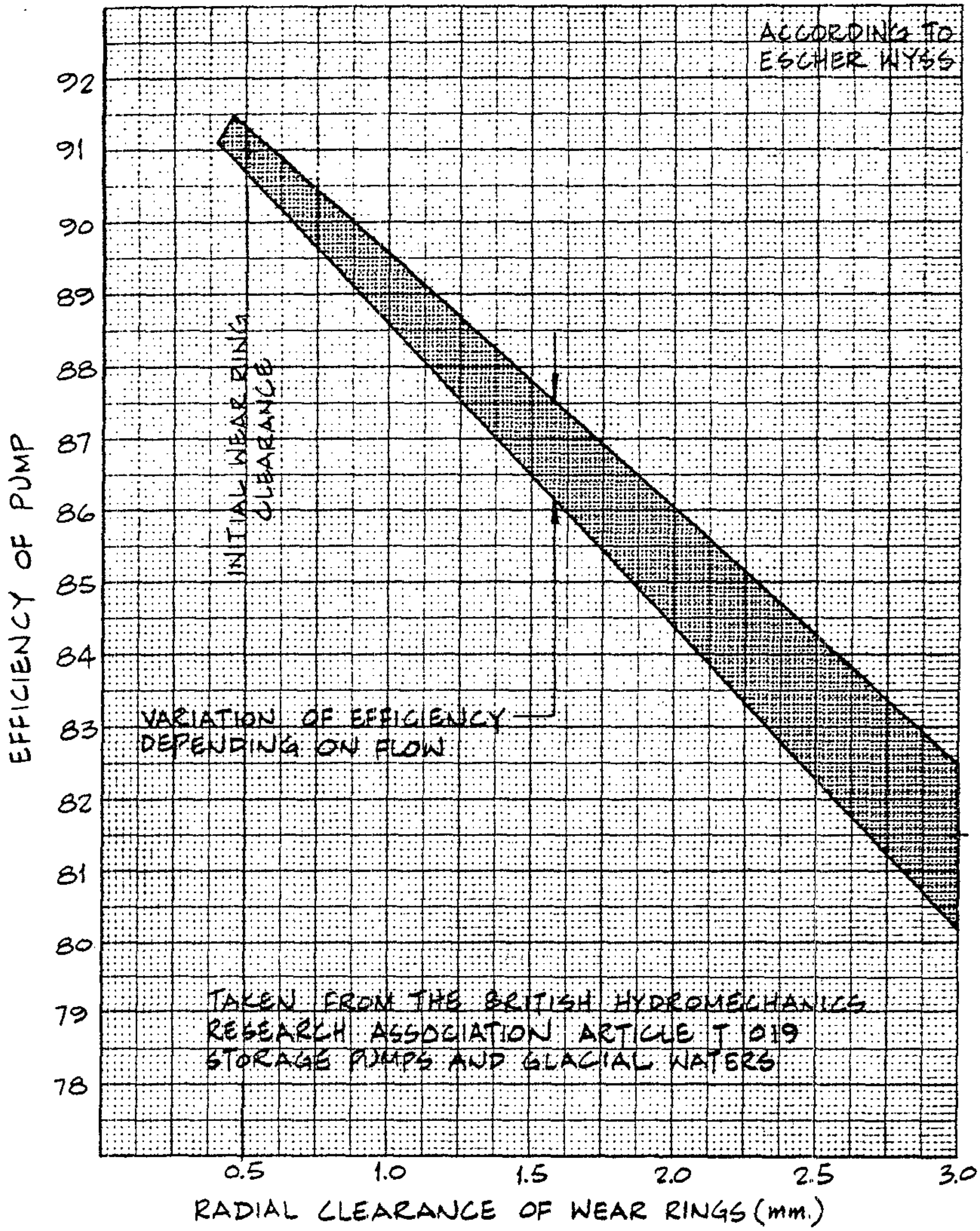

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DATE	DR'N	D. TUNG	18	8	77
	CH'K'D	R. KALLBERG			
A 4191/17-3	APP'D				
	APP'D.				

HAT CREEK PROJECT
COOLING WATER SUPPLY
EFFECT OF WEAR
ON PUMP EFFICIENCY

SANDWELL

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

DWG. A 4191/17-3 | REV

E. LIVINGSTON, P. Eng.
A. BADRY

E. LIVINGSTON ASSOCIATES

CONSULTING GROUNDWATER GEOLOGISTS

1401 WEST BROADWAY,
VANCOUVER 9, B.C.
TELEPHONE: 738-9232

July 29, 1977.

Mr. Bryan McConachy, P. Eng.,
Sandwell and Co. Ltd.,
1550 Alberni Street,
Vancouver, B.C. V6G 1A4

Dear Sir:

This letter is further to my discussion with you and Mr. Boyle of Sandwell & Co. about the possibility of obtaining cooling water from wells near the Thompson River, in the vicinity of Ashcroft, for the proposed Hat Creek Project. It also discusses the result of field work which I carried out in that area in late July.

In order to get the required amount of water, 25,000 igpm, from wells we must locate an exceptionally good aquifer. Such an aquifer must be composed of gravel or sand and gravel with a transmissivity in the order of one million gallons per day per foot width. It would need to have adequate recharge, probably from a surface water body such as the Thompson River.

Such aquifers have been found in various locations in the Province; several examples are at Prince George, near Castlegar, in Similkameen Valley, near Chilliwack and at Fort St. James. None of this caliber have been found to date in the Thompson River Valley. Such aquifers originate as water deposited sediments or occasionally as ice contact deposits. In the Thompson Valley near Ashcroft the only geologic setting in which we believe such an aquifer can be found is a buried channel of the Thompson River or of the Bonaparte River at its confluence with the Thompson.

With this in mind, I spent a couple of days in the field around Ashcroft to attempt to work out the surficial geology as it relates to an aquifer with the characteristics mentioned above. This meant trying to determine the following:

1. The location of the old buried channel of the Thompson River in the Ashcroft area.
2. Its width and depth.
3. The type of material it contains.

These questions are closely related to the glacial history of the area, particularly what happened at the time of the most recent glacial episode locally and what has happened since.

At the time of the last regional glaciation the whole area was buried to a depth of several thousand feet by ice. This ice sheet tended to scour out the main north-south valleys depositing till along the deepest parts. At the time of ice melting the main valleys served as meltwater channels to carry sediment-laden meltwater to the sea. In some places gravel and sand were deposited in the bottom of such valleys by meltwater.

Following the last regional glaciation there was a short glacial episode during which ice advanced from upland areas into the large valleys. These local advances often blocked drainage to form huge lakes in which were deposited silty, sandy lake beds during the time when ice was melting on the uplands. Such a lake occupied the Thompson Valley in the Ashcroft area. Its surface elevation, shown by extensive raised deltas at the mouth, of tributary creeks, was about 1450 ft. Small relict terraces probably indicate a brief period at higher elevations perhaps 1600 ft. and higher.

The valley was partially filled with lake beds by this process. Near the mouths of tributary creeks these lacustrine deposits are thick; where no tributaries were present they are thinner. These lake beds are well exposed upstream and downstream from Ashcroft as light coloured bluffs.

When the River cut down to its present level in these deposits, it cut a new valley which in some places corresponded to its old valley. In other places, where it cut down outside its old valley, it cut into the rocks of the old valley wall. This seems to have been the case in several places downstream from Ashcroft where the River runs through modern rock canyon.

There is little subsurface information in this area. I was able to obtain information on several wells near the mouth of the Bonaparte R. and one across the Thompson River and upstream on the old farm now part

continued... 3

of the DuPont explosives facility. The well owners report that the wells are between 70 and 90 ft. deep at sites about 35 to 45 ft. elevation above the river. One well (Muir) is reported to have bottomed in rock at about 70 ft. The owners report that they pass through clay with water-bearing gravel near the bottom. We think that in general the reports are correct but we feel that the report of bedrock may not be correct mainly because we find that many drilling contractors do not check to make sure that the hole is in rock particularly if they have already encountered enough water-bearing gravel in which to construct a well.

I mapped the location of most rock outcrop in the area in an attempt to establish the course of the old rock valley. Outcrop is rather sparse over much of the area so it is difficult to define the limits of the valley. In the area North of Ashcroft the River is within the old valley but south of Ashcroft the River is cutting a new canyon in rocks on one side or the other of the old valley. The site being considered, namely at the confluence of the Bonaparte and Thompson Rivers, certainly appears to be within the old rock valley.

The depth of the old rock valley is unknown. The subsurface data show that it is at least 35 ft. below the River level. We can speculate further about this on the basis of the materials overlying bedrock where rock is exposed near River level south of Ashcroft. To the south the gently sloping rock surfaces near River level are overlain by till or compact till-like outwash. Only at one place, just south of the first CN tunnel, is there a few feet of very coarse gravel on the rock. At the second CN bridge upstream from Ashcroft there are exposures of till and related peculiar till-like sediments at River level. Although rock is not exposed we think that these may be the materials in contact with rock so the distance to rock may be small.

In contrast to this evidence for shallow bedrock there is conflicting evidence that the depth to rock may be great. On the west side of the River at the first bend upstream from the first CN bridge north of Ashcroft there is a large collapse structure in the silty, sandy valley fill. This collapse feature is filled with younger fan deposits washed in from the northwest. The beds in the middle have slumped about 100 ft. and the collapse structure extends

continued... 4

below River level. This implies that there was over 100 ft. of ice buried below River level and indicates that the buried valley is over 100 ft. in depth below the River at this point.

The field work shows that there is evidence that the buried rock valley is deep and also conflicting evidence that it is relatively shallow. The materials on bedrock along the River are generally not favourable as far as permeability is concerned but it is possible that highly permeable gravel occurs in the old channel. The subsurface data indicate that conditions are not particularly favourable for at least 35 ft. below the River.

We believe the presence or absence of a high capacity aquifer can be confirmed by drilling one hole in the vicinity of the confluence of the Bonaparte and Thompson Rivers. Ordinarily we do not favour a one hole exploration program in situations of this kind. In this case, however, the target must have a large size to be of any value (i.e. to produce 25,000 gpm). For this reason we believe that a single testhole is justified.

It should be drilled 8" diameter using a cable tool rig or an air rotary rig equipped with a casing hammer (not a Becker Drill). An 8" diameter testhole is large enough to permit a pump test if favourable conditions are found. If data on foundation conditions are of any value, foundation testing equipment (split spoon, Shelvy tube samplers etc) can be used in the same hole.

We estimate the cost of a test well to 200 ft. as follows:

1.	Move equipment to and from Ashcroft	$ 800.
2.	Drill 8" to 200 ft. @ $25/ft.	5000.
3.	Hourly work pulling casing, taking bailer samples etc. 25 hr @ $50/hr.	1250
		$7050.

If an aquifer is encountered, a long screen is set and a pump test carried out, the additional cost might be as much as $6,000.00.

In summary:

1. An exceptionally good aquifer could yield the required amount of water for the Hat Creek Project. The only aquifer of this type

continued... 5

in the Ashcroft area would be a buried channel of the Thompson River.

2. A study of surficial geology shows that there is an old buried Thompson River channel.

3. The confluence of the Bonaparte and Thompson Rivers seems to be in the old channel.

4. Subsurface information is sparse but it shows that the old channel extends at least 35 ft. below River level.

5. There is conflicting evidence as to the depth of the old buried channel.

6. A single testhole near the confluence of the Thompson and Bonaparte Rivers is probably justified at a cost as high as \$7,100 if unsuccessful, or if successful, as high as \$13,000.

Yours truly,

E. LIVINGSTON ASSOCIATES

E Livingston

E. Livingston, P. Eng.

EL/lw

E. LIVINGSTON, P. Eng.
A. BADRY

E. LIVINGSTON ASSOCIATES

CONSULTING GROUNDWATER GEOLOGISTS

1401 WEST BROADWAY,
VANCOUVER, B.C. V6H 1H6
TELEPHONE: 738-9232

December 21, 1977

Sandwell and Company Ltd.
#601, 1550 Alberni Street
Vancouver, B. C. V6G 1A4

Attention: Mr. Arno Copeland, P. Eng.

Your reference: V4191 B. C. Hydro and Power Authority
271.1 Indirect Water Intake - General

Dear Sir:

This is in reply to your letter of December 13 about the water intake on the Thompson River north of Ashcroft.

We have carefully examined the data included with your letter. The results of the drilling and the seismic survey certainly fit our concept of the geology of the area.

We note, in the Geo-Recon Explorations Ltd. report on page 2, mention of a high velocity boulder pavement in the area but no mention of the low velocity layer shown on the sections just below surface. From our experience, it is material with this low velocity (when dry) that has the high permeability required for high capacity wells or a radial collector. The sections show that this material is very thin everywhere and that it is missing entirely over part of the area.

The next layer, with a velocity of 975 to 1524 m/sec is probably too compact to have high permeability. It is quite thin except near borehole 3. Most of the valley fill is till which rests directly on rock. The surface of the rock, according to the seismic data, has little relief within the area of investigation. The greatest depth to rock is about 15 m. below the water table (surface of the river).

It is interesting to contrast the subsurface at the intake site with that at the CNR bridge at mile 45.8*. The situation is entirely different with no till, unless some of the bouldery sediments are till. The part that most resembles till in the old drill logs for the CNR bridge is at the top of BH no. 5, where there is 1.8 m of "boulders, sand and gravel". There is an exposure on the river bank about 30 m from the test hole. This is a contorted mixture of fine silt, sand and stone which is not till. The old logs do not show whether the holes reached rock. In any case, the lower part of the section in all but one hole is fine sand. One hole, BH no. 3, reached an elevation of 274.2 m, approximately, the same as the elevation of the bottom of the rock valley at the intake site according to the seismic survey. There is nothing on the logs to indicate whether the testholes at the bridge reached rock.

* See attached Drawing D4191-SK79

cont'd...2

The fact that the subsurface section from the bridge and the proposed intake are so dissimilar, reflects the complexity of the geology in the river valley. The till in the vicinity of the intake site may be associated with the Bonaparte River. The configuration of the layers, as shown by the seismic sections, may be the result of torrential flow in the Thompson River. We have seen other inconclusive evidence for such flow upstream as far as Savona. A short period of flow at an extremely high rate would erode the compact valley fill producing an irregular smooth profile with an irregular layer of gravel at surface, capped by a boulder pavement. This is of no particular interest in the present investigation, other than to explain the shallow sub-surface geology.

The recent work and the subsurface data from the bridges upstream are evidence that an aquifer of very high capacity does not exist in the vicinity of the mouth of the Bonaparte River or along the Thompson River up or downstream. For a very high capacity aquifer we must have:

(a) a deep rock valley below the present river level,
(b) clean sand and gravel filling such a valley,
(c) an hydraulic connection between the sand and gravel and the river.

The subsurface data near the mouth of the Bonaparte River show that the maximum depth to rock below river level is only about 15 m. The test holes at the bridge several miles upstream do not define the depth to bedrock unless we assume that the deeper ones stopped at the rock surface. The minimum depth to rock is also about 15 m below the river.

The fill in the rock valley at the Bonaparte is mostly till or compact gravel (outwash) both with low permeability. At the bridge upstream most of the fill is sand which we believe is not likely to have extremely high permeability.

The only evidence for a <u>possible</u> deep valley is the collapse structure in the valley fill which may be observed about a mile upstream from the proposed intake site. A photograph of this structure is included with this report. (This photo was not available to accompany our letter of July 29). This is certainly inconclusive evidence and it is quite possible that the collapse occurred with only 20 m of ice in the valley below river level. This feature is probably associated with the till found at the proposed intake site.

In view of the additional negative evidence from the recent investigation, we conclude that the chance of locating an aquifer capable of yielding 1580 l/sec. in the vicinity of the mouth of the Bonaparte River, or anywhere in the Ashcroft area, is extremely remote. No further work is required to substantiate this conclusion.

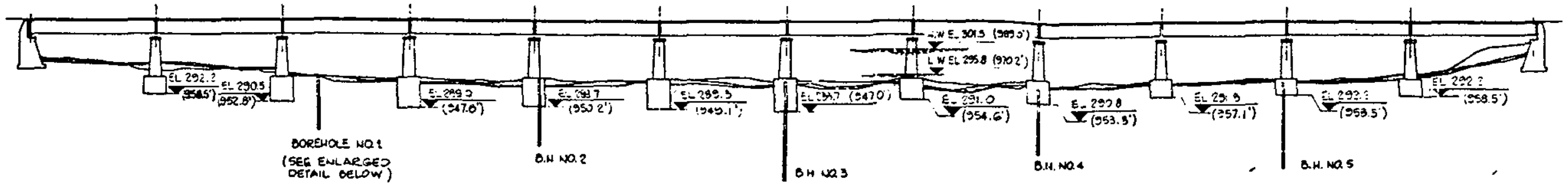
Yours truly,

E. LIVINGSTON ASSOCIATES

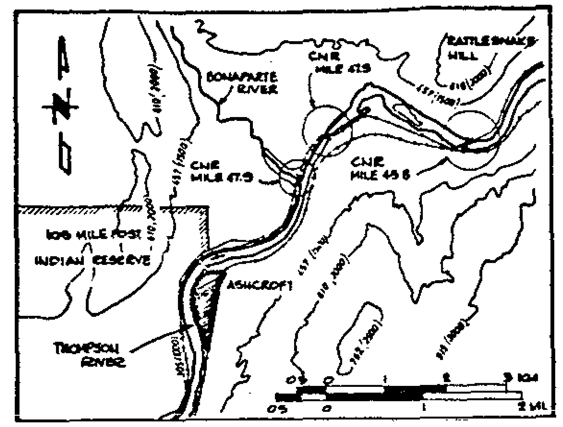
E Livingston

E. Livingston, P. Eng.

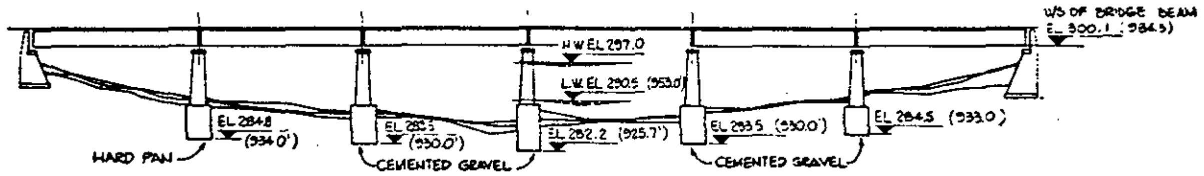
encl.



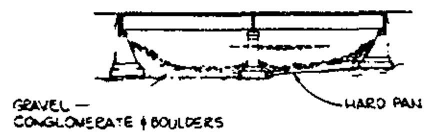
C.N.R. BRIDGE AT MILE 45.8
LOOKING DOWNSTREAM
SCALE : 1:600
1" = 50'



KEY PLAN



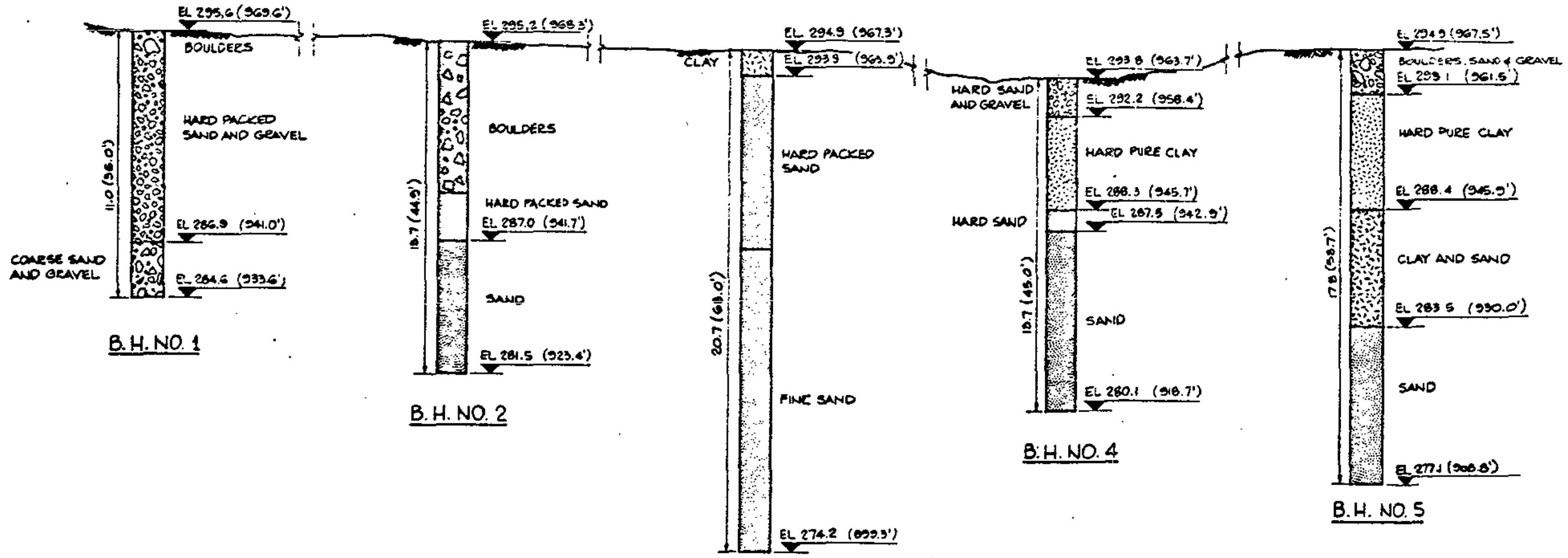
C.N.R. BRIDGE AT MILE 47.3
LOOKING DOWNSTREAM
SCALE : 1:600
1" = 50'



C.N.R. BRIDGE AT MILE 47.9
LOOKING DOWNSTREAM
SCALE : 1:600
1" = 50'

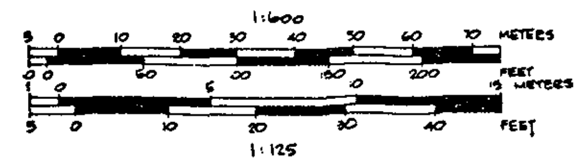
NOTES:

1. THE SUBFACE DATA OF C.N.R.'S BRIDGE CROSSINGS AT MILES 45.8 AND 47.3 WERE OBTAINED DURING THE ORIGINAL SURVEY OF 1914. THE DETAILS AS PRESENTED ON THIS DRAWING WERE ABSTRACTED FROM A REPORT PREPARED BY NORTHWEST HYDRAULICS CONSULTANTS LTD CALLED "HYDRAULIC INVESTIGATIONS OF THOMPSON RIVER BRIDGES - APRIL 1977."
2. THE DETAILS OF THE C.N.R. BRIDGE OVER THE BONAPARTE RIVER WERE OBTAINED FROM C.N.R., AS WERE THE HIGH AND LOW WATER ELEVATIONS SHOWN.
3. LEVELS ARE GIVEN IN METERS (FEET).



B.H. NO. 3

BOREHOLE DETAILS - C.N.R. BRIDGE AT MILE 45.8
VERTICAL SCALE : 1:125
1" = 10.4'



SANDWELL	DWG. NO. D 4191 - SK 79		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
DES		RECD	HAT CREEK PROJECT
DWN	SH	APPD (TECH)	COOLING WATER SUPPLY
CHKD		APPD	PRELIMINARY DESIGN STUDY
INSP		APPD	WATER INTAKE - CNR SUBSURFACE DATA
SUBD		DATE OCT. 27, 1977	
MICROFILMED	JOB No. V 4191	CONTRACT No.	SCALE AS SHOWN — DWG No. D 4191-SK 79

REFERENCE DRAWINGS | REFERENCE DRAWINGS | REVISIONS | REVISIONS

Collapse Structure Showing Contorted Beds

Beak Consultants Limited

Montreal
Toronto
Calgary
Vancouver

Laboratory/3851 Shell Road
Richmond/British Columbia
Canada/V6X 2W2
Telephone (604) 273-1601
Telex 04-508736

beak

Environmental Specialists

28 November 1977

Sandwell & Co. Ltd.
Suite 601 - 1550 Alberni St.
Vancouver, B.C.
V6G 1A4

Attention: A. Copeland

Reference: J5012/85

Dear Sir:

Enclosed please find the results of the analyses you requested. I have also included 2 copies of our booklet "Field Preparation of Water for Laboratory Analysis" which I hope you will find useful.

As we discussed on the phone, nonfiltrable (suspended) residue was not analyzed because the heavy layer of sediment in the bottles appeared to be a result of sampling rather than actually part of the water.

We have used our usual methods of analysis for all the parameters. They are either the same as or give results comparable to those methods listed on the table sent with your letter. If you would like any specific details on our methods I will be pleased to discuss them with you.

According to "Standard Methods for the Examination of Water and Wastewater", 14th edition (APHA, AWWA, WPCF), page 487, "terms such as "colloidal", "crystalloidal" and "ionic" have been used to distinguish between various forms of silica in waters. Such terminology cannot be substantiated". For this reason we use the term "molybdate-reactive silica", although the 1971 Annual Book of ASTM Standards, Part 23 (ASTM) refers to it as crystalloidal (non-colloidal).

If you have any further questions do not hesitate to contact me. We look forward to continuing our services to you in the future.

Yours truly

BEAK CONSULTANTS

Sheila McMeekin

Sheila McMeekin
Laboratory Supervisor

A MEMBER OF THE SANDWELL GROUP

Beak Consultants Limited

385 Shell Road
Richmond/British Columbia
Canada/V6X 2W2
Telephone (604) 273 1601
Telex 04 507893

beak

Client: Sandwell & Co. Ltd.
Suite 601 - 1550 Alberni St.
Vancouver, B.C.
V6G 1A4

Attention: A. Copeland

Report of analysis:

Project: J5012/85
Date received: 7 November 1977
Date analyzed:
Number of samples: 3

Sample reference	B.H.1	B.H.2	B.H.3			
Total Alkalinity	339	348	197			
Chloride	4.5	19	6.2			
Conductivity (μmhos/cm)	1120	3560	1330			
Hardness (by titration)	226	1388	522			
pH	8.5	8.2	8.1			
Filtrable (Dissolved) Residue	771	3400	1090			
Sulfate	320	2000	630			
Total Organic Carbon*	2290	1850	2310			
Molybdate-Reactive Silica	17.9	12.3	12.8			
Total Silica	17.9	12.3	12.8			
Total Sodium	150	380	96			

Ref:

* Sample was not preserved.

All results expressed as ppm except
pH unless otherwise stated
1 ppm = 1 mg/l = 1 lb/100,000 Imp. gal
Alkalinity and Hardness are
expressed as mg/l $CaCO_3$

Shiela McMeekin

Prepared for:

SANDWELL AND COMPANY, LTD.

Prepared by:

NORTHWEST HYDRAULIC CONSULTANTS LTD.

Edmonton - Vancouver - Calgary

December 1977

HAT CREEK PROJECT
COOLING WATER SUPPLY STUDY
EVALUATION OF INTAKE SITES

TABLE OF CONTENTS

LIST OF FIGURES

1. INTRODUCTION AND BACKGROUND

Sandwell and Company's (Sandwell) Interim Report V4007/1, dated October 1976 for the cooling water supply for British Columbia Hydro and Power Authority's proposed Hat Creek Project included the evaluation of numerous potential river water intake sites on the Fraser and Thompson Rivers. As the Consultant for the study, Sandwell retained Northwest Hydraulic Consultants Ltd. (NHCL) to assess the river engineering and hydrologic aspects of these evaluations.

In their Conceptual Design Report V4007/2, dated January 1977, Sandwell recommended Site 10 as the primary site for the intake. This site is located, as shown on Figure 1, on the right bank of the Thompson just upstream of the mouth of the Bonaparte River, about 2.5 kilometers (1.5 miles) upstream of Ashcroft; a recommended alternative was Site 4A, also on the right bank, about 11 kilometers (7 miles) upstream of Site 10.

Subsequent to the submission of Sandwell's report V4007/2, NHCL conducted river bed and bank surveys and mid-winter low-flow observations at both sites (References 4, 5, 8). A preliminary evaluation of this additional data raised questions as to the suitability of both Site 10 and Site 4A as specific intake locations because of shallows that were found to exist near the right bank in their immediate vicinities. However, at the same time the field data indicated that a fresh look at a larger reach of river from about 3.5 kilometers (2.25 miles) upstream of Ashcroft to the vicinity of Ashcroft would be worthwhile. Subsequently, additional river survey data were obtained for this larger reach (Reference 9).

A proposal to evaluate Sites 10 and 4A was submitted to Sandwell by NHCL on 16 February, 1977. In the course of a low-flow reconnaissance conducted on 22 February, 1977 by representatives of Sandwell, NHCL and Golder Associates, Ltd., the geotechnical consultant for the study, two regions (designated A and B) along the right bank near Site 4A and five regions (C,D,E,F, and G, shown on Figures 2 and 3) on either bank in the reach upstream of Ashcroft were identified for specific attention during this proposed evaluation.

Before the evaluation commenced, it was decided to delete Site 4A from further consideration. Thus it was the purpose of this study to evaluate the hydraulic and river engineering aspects of five more-or-less specific sites in the reach immediately upstream of Ashcroft.

Data and results presented in this report are expressed in metric units, with imperial units shown in brackets. However, since

(1) Existing streamflow data are currently published in rounded cfs (imperial units), and

(2) NHCL's previous hydrology study (Reference 6) and field surveys were conducted using imperial units,

the numbers shown have generally been selected in round imperial units and translated directly to approximate metric units.

2. HYDROLOGIC/HYDRAULIC DATA AND ANALYSES

2.1 Hydrology

In order to define design flows for preliminary engineering purposes, a brief hydrology review was undertaken. In a previous Interim Report dated November 1976, "Hat Creek Project, Water Supply Hydrology", NHCL established high and low flow frequency curves from Water Survey of Canada (WSC) data for the Thompson River at Spences Bridge, located about 35 kilometers (22 miles) downstream of the current study reach. Subsequently, available WSC flow data were examined to establish what difference in extreme high/low flows exists between Spences Bridge and the Aschroft area. During periods of peak flows on the Thompson River, it was found that peak flows on the Nicola River (the only significant stream downstream of the Bonaparte) occurred well before peak Thompson flows. Similarly, the Bonaparte flows were also found to occur well before peak Thompson flows. In fact, during Thompson River peaks, the total Nicola plus Bonaparte River flows represents from 2 to 5 percent of the Thompson flow at Spences Bridge. By including the Nicola and Bonaparte flows, a conservative flood value is obtained in the study reach. Consequently the flood frequency curve for the Thompson River at Spences Bridge was assumed to apply to the sites under investigation in this study. This assumption ignores about 1040 square kilometers (400 square miles) of local inflow between the Bonaparte River and Spences Bridge.

During extreme low-flow events on the Thompson River, it can safely be assumed that a drought would be widespread enough to encompass the Nicola and Bonaparte River basins. Thus, for example during rare low-flow events on the Thompson, it is expected that similar rare (or at least very) low-flow events would be occurring simultaneously on the Nicola and Bonaparte

Rivers. The estimated 100-year low-flow events for the Thompson at Spences Bridge (Figure 5), Nicola[1] and Bonaparte Rivers[1] are respectively 115, 1.7 and 0.3 m^3/s (4000, 60, and 10 cfs). Hence, for this condition, the total Nicola plus Bonaparte flow is about 1.7% of the Thompson flow at Spences Bridge, which is considerably less than the accuracy (±5%) of flows gauged by WSC. As such, the low-flow frequency data for the Thompson River at Spences Bridge were also assumed to apply to the sites under investigation for this study.

Both the high-and low-flow frequency curves are reproduced in Figures 4 and 5. Note that at extreme low flows, a second, flatter-sloping curve is indicated. The trend in data for extreme low-flow events indicates a leveling off to about 115 m^3/s (4000 cfs). This is assumed to be a result of the natural control provided by both Kamloops and Shuswap Lakes, located respectively about 30 and 130 kilometres (19 and 80 miles) upstream of Ashcroft. Frequency analyses by WSC, outlined in Reference (1), show similar slope changes for extreme low-flow events.

From Figures 4 and 5, the following return periods were adopted for preliminary engineering purposes:

Return Period	Flow in m^3/s (cfs)
>1000 year flood	5665 (200,000)
100 year flood	4535 (160,000)
10 year flood	3685 (130,000)
2 year flood	2720 (96,000)
100 year low-flow	115 (4,000)

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[1] See Reference (1), which shows approximate low-flow analyses for these rivers.

2.2 Hydraulics

Figure 1 shows the location of both the study reach and the five sites. Figures 2 and 3 show the plan form (ie. the river geometry as viewed from above) and variation in cross section of the sub-reaches that contain the specific sites above and below the Bonaparte. (Each of the sites can be regarded as referring to the length of bank extending perhaps 100 metres (330 feet) upstream and downstream of the exact locations marked. Precise siting will be determined during the detailed design stage of the project.)

The initial step of the site evaluation consisted of computing water surface profiles, depths, and mean velocities for the range of discharges that can be expected in the Thompson. For this purpose a standard backwater analysis computer program (U.S. Army Corp of Engineer's HEC-2) was used. Calibration of the program was relatively straight forward as neither flood channel nor flood plain flows were involved, even at the highest-discharge runs. For the calibration, determination of appropriate values for Manning's 'n' was based on available water surface profiles discharges and spot elevations obtained from various sources, as outlined below:

Item	Source	Dates	Approximate Discharge Upstream of Bonaparte River in m^3/s (cfs)
Spot Elevation (at Site 10, Station 0)	NHCL	May 20, 1976	2,000 (70,600)
Spot Elevation (at CN bridge, just upstream of Site C)	NHCL	June 24, 1976	2,490 (88,000)
Water Surface Profile (as recorded during NHCL's field survey of December 4-6, 1976 Reference 4)	NHCL	December 5, 1976	300 (10,650)
Water Surface Elevations (as recorded during Sandwell's water level monitoring program, Appendix A, Table 1)	McElhanney Surveying and Engineering Company	December 6, 1976 until July 15, 1977	Ranging from 186 (6,600) to 1,515 (53,500)
Water surface profile (as recorded during Sandwell's water level monitoring program, Appendix A, Table 3)	McElhanney Surveying and Engineering Company	January 31 - February 2,1977	205 (7,200)

The above values of discharge used on the given dates may vary slightly from figures now available from WSC, and as shown in Appendix A. However, these values reflect our best estimate of Thompson flows at the time the backwater study was undertaken. Any difference between the above flows and current WSC data would have a negligible effect on results.

Site Discharges were determined by using available preliminary data for Nicola River at Spences Bridge (WSC gauge 8LG6), Bonaparte River at Cache Creek (8LF2), and Thompson River at Spences Bridge (8LF51). Flows downstream of the Bonaparte were estimated by subtracting Nicola (8LG6) flows from Thompson (8LF51) flows. Upstream of the Bonaparte, flows for the Bonaparte (8LF2) and the Nicola (8LG6) were subtracted from Thompson (8LF51) flows. As Nicola and Bonaparte River flows were not available for all calibration points at the time of the study, they were estimated (where necessary) on the basis of drainage area.

The highest discharge for which a corresponding water level was available was 2490 m^3/s (88,000 cfs). Values for Manning's 'n' for higher flows were estimated on the basis of river engineering experience, after considering the trend of the n values established for the flows up to 2490 m^3/s (88,000 cfs). The values of n used were as follows:

Discharge m^3/s	Discharge cfs	n	
205	7,200	.052	From calibration runs
300	10,650	.052	
1190	42,000	.052	
2000	70,600	.047	
2490	88,000	.040	
2720	96,000	.039	Estimated
3685	130,000	.037	
4535	160,000	.035	
5665	200,000	.034	

Using the above-listed values for n and the river channel cross sections obtained in the river surveys, water surface profiles were calculated for discharges of various return periods. The results are given on Figure 6, along with a plot of computed mean velocities and a plot of representative river width.

In general, for higher discharges, Figure 6 shows relatively low velocities and large depths in the relatively wide upstream end of the reach, high velocities and large depths in the central, narrow, portion of the reach, and intermediate velocities and shallower depths in the wide downstream end of the reach. At minimum discharges, there is adequate depth for an intake throughout most of the study reach; velocities are low, 0.6 m/s (2 ft/sec) or less everywhere except at the Bonaparte rapids and at shallows at Station 915+ (3000 u/s)[2].

Figures 7 through 11 inclusive are stage discharge curves for all five sites. Considerable judgement has been applied to estimate water levels at higher discharges. Stage measurements at the selected site for discharges over, say, 3000 m^3/s (106,000 cfs) will be required to confirm the accuracy of these estimates before detailed design.

Figure 12 presents the river channel cross sections at (or near) the sites evaluated and at the original Site 10 centerline. The shallow depths near the right bank at Station 0 - mentioned in the Introduction - are apparent. The thalweg at this location has apparently been forced to the left by the deposition of coarse material transported into the Thompson by the Bonaparte during the development of the Bonaparte's gorge.

[2] 915+ indicates approximate distance in metres upstream of station 0 (Figure 1); 3000 u/s indicates approximate distance in feet upstream of station 0.

3. EVALUATION CRITERIA

It was the objective of this study to evaluate only the hydraulic and river engineering aspects of the various sites. Other factors such as access, suitability of adjacent terrain, additional pipeline length, whether or not a river crossing would be required, etc., were not considered.

The basic criterion was the existence of adequate depth near the bank at design low water. Two other main criteria concerned minimizing anticipated design problems related to both fish protection aspects and the intake of suspended sediment.

The intake concept considered for this project involves creating a relatively gentle low-velocity downstream flow between the trashrack/curtain wall, and face of the travelling screens*, while drawing water at a maximum screen approach velocity of 0.12 m/s (0.4 ft/sec). Therefore, high velocities and high turbulence levels in the river should be avoided as much as possible.

High velocities and high turbulence would also increase both the concentration and size of suspended sediment. The net screen opening is 2.5 mm (0.1 inches) square; hence any such increases in concentration of sediments up to this size would aggravate maintenance conditions for the wet wells and pumps, particularly if high-lift pumps were to be used in the intake.

Another consideration was the proximity of eroding cliffs along the right valley wall, extending from about Site C to about 5 kilometers (3.5 miles) upstream of Ashcroft. These cliffs can supply a major continuing source of both suspended and bed load sediment in the study reach. There has also been

*This flow is referred to herein after as the "by-pass flow".

some concern that a slide large enough to result in a significant bed-sediment wave passing through the study reach could occur. Golder Associates have subsequently undertaken (Reference 2) a preliminary investigation of the stability of upstream cliffs. Based on their draft report and a brief discussion between Golder and NHCL, two considerations relating to hydraulics of intake siting are warranted:

(1) The large bluffs beginning in the large river bend upstream of Site C (and extending some distance upstream of this bend) are composed of two distinct materials/areas:

i) Fine sandy silt, containing material mostly less than about 0.1 mm is evident upstream of the aforementioned river bend.

ii) Layered sands and gravels, with cobbles up to about half-way up the cliffs and silts on the top half of the cliffs, exist within the large river bend. This material is probably well-graded overall, and is an active erosion area.

These cliffs are capable of supplying long-term sediments to the river through erosion processes. Large scale sliding of the order of 1/4 million cubic metres is possible, but is not likely to dam off the river. As such, it is unlikely that bed forms large enough to reach the intake sill (ie. higher than 1-1½ metres or 3-5 feet) would occur at any of the sites being investigated.

(2) The near-vertical silt buffs between Site D and the CN railway bridge are in a marginal state of stability. Sloughing of these bluffs could cause

short-term slugs of silts to be carried by the river at any of the sites under investigation, but would not supply enough material to raise adjacent bed levels to the intake sill.

Based on current knowledge of the upstream cliffs, some weight was given in this evaluation to the distance of a particular intake site from the slide area, on the basis that the height of a possible sediment wave would decrease with distance, and that with increasing distance, additional warning time would be available before the effects reached the intake.

The location of the intake sites with respect to the Bonaparte rapids was also a consideration. During periods of low flow the rapids control the minimum water level at Sites C and D. Complete scouring away of the rapids would result in minimum depths being about 1 metre (3 feet) less than at present. This would considerably reduce the degree of suitability of the upstream sites by reducing the available depth at low flow.

In summary, three major criteria were applied:

1. Available depth;
2. Mean velocity at high flow;
3. Estimated levels of turbulence;

and two minor factors were considered:

1. Distance from the upstream erosion area;
2. Location with respect to the Bonaparte rapids.

4. DISCUSSION

4.1 Available Depth

The face of either a bank-type or pier-type intake would be located as close to the bank as possible (in order to minimize the degree of constriction resulting from a bank intake, or the required length of bridge for a pier intake) yet in as deep water as possible. After considering the above factors, the locations shown on Figure 12 were chosen for the face of the intake at each site. Modification (that is, excavation) of the natural bed of the river to improve depths was not considered at this time. More will be said of this possibility further on in this report.

The minimum and maximum depth-at-face-of-intake data are summarized in Table 1 below:

Site	Minimum Depth Metres	Minimum Depth Feet	Maximum Depth Metres	Maximum Depth Feet	Distance from 1:100 Yr Flood Water's Edge Metres	Distance from 1:100 Yr Flood Water's Edge Feet
C	3.9	13	13.7	45.0	45.5	149*
D	3.1	10	11.8	38.7	36.5	120
E	5.2	19	13.2	43.3	36.5	120
F	2.7	9	9.8	32.2	45.5	149
G	2.7	9	9.6	31.5	36.5	120

TABLE 1

Site E is the best site according to the depth criterion as the great depth would permit a large vertical dimension for the intake port and hence reduce the required total length of the structure. Site C then Site D follow in preference.

*Assuming fill as shown on Figure 12.

The maximum water surface elevation is of interest to the designer as it determines the required elevation of the working floor of the pumphouse. The height of the working floor above the intake sill is a factor in determining the cost of some elements of the intake. Table 2 presents water surface elevations at the various sites for the 1:100 year flood and for what would be an extremely rare flood. Data for the latter is included to allow the designer to become aware of the sensitivity of stage to flood return period, which is a factor to consider in determining freeboard.

Site	Water Surface Elevation 4535 m^3/s (160,000 cfs) 1:100 yr		5665 m^3/s (200,000 cfs) >1:1000 yr	
	Metres	Feet	Metres	Feet
C	298.4	978.8	299.6	982.8
D	297.9	977.2	299.1	981.0
E	296.2	971.7	297.3	975.0
F	294.9	967.3	295.7	970.0
G	294.5	965.9	295.2	968.4

TABLE 2

4.2 Mean Velocity

In designing the intake under consideration, the difficulty in maintaining suitable by-pass flow conditions during flood periods will increase with velocity. Thus, lower mean velocities were judged to be a benefit. The velocities at the various sites are presented in Table 3, below:

Site	Mean Velocity 3685 m^3/s (130,000 cfs) m/s	ft/sec	4535 m^3/s (160,000 cfs) m/s	ft/sec
C	3.17	10.4	3.54	11.6
D	3.57	11.7	3.96	13.0
E	3.96	13.0	4.48	14.7
F	4.15	13.6	4.70	15.4
G	4.30	14.1	4.89	16.0

TABLE 3

By this criterion, the sites rank in order C,D,E,F,G, from most favourable to least favourable. The sites above the Bonaparte (C and D) are to be preferred.

The degree of scour that could occur around an intake structure is a hazard that could be controlled at any site through proper hydraulic design. Thus, although the scouring tendency would increase with increasing velocity and level of turbulence, the difference in scour potential did not enter into the evaluation.

4.3 Level of Turbulence

Large scale turbulence created by rapids, bridge piers, bank irregularities, large boulders on the bed, etc., would both complicate the design of the by-pass flow system and increase the size and concentration of suspended material. Thus preference was given to low-turbulence sites. The quantification of a turbulence factor was not possible, but qualitative estimates of turbulence levels were made on the basis of knowledge of the reach.

The sub-reach above the Bonaparte is generally regular in both plan form and cross section, although some turbulence is created by the constriction and large piers at the CN bridge.

The Bonaparte Rapids are the major source of turbulence in the study reach. The standing waves and large scale eddies generated by such a constriction would be severe at high flood flows, and would persist for some distance downstream. The plan form of the river between the Bonaparte and cross section 1220- (4000 d/s) is considerably more irregular than the sub-reach above the Bonaparte. Several bank promontories along the left side would generate considerable turbulence, a fact that was observed at 2000 m^3/s (70,600 cfs) during the reconnaissance of May 20, 1976 (Reference 3).

The roughness of the bed in the sub-reach below the Bonaparte is also relatively great as indicated by the low flow n value of 0.052, which is very high, especially when the relatively large depth of flow is considered.

The qualitative estimates for the anticipated levels of turbulence are summarized in Table 4 below:

Site	Level of Turbulence
C	Relatively low
D	Lowest
E	Highest (unacceptably high)
F	High
G	Relatively high

TABLE 4

4.4 Other Considerations

The eroding areas upstream of Site C have been mentioned as a source of sediment and as a possible source of a bed material wave. However, the relative seriousness of this as a hazard to any particular site is not possible to quantify. It is nonetheless judged that the risk at Site C is greater than at the remaining sites. This is based on two factors: first, Site C is closest to the eroding area and hence the magnitude of sedimentation effects may be greatest and the period of warning shortest; and second, Site C is on the inside of a mild bend and hence may be in a zone of deposition if a large sediment wave were to be initiated upstream.

Another consideration previously mentioned was the possibility of degradation of the Bonaparte rapids, which would result in decreased low-water depths at Sites C and D. This concern has been discounted for two reasons, first, the rapids appear to be highly stable - floods from 1928 to date, including the 1:50 year flood of 1972, did not have a significant effect, as judged from pre-1972 air photos; and second, if partial degradation did occur during a flood of rare magnitude, the rapids could be re-established artificially.

A further minor consideration is the input of suspended sediment from the Bonaparte River. In relative terms, this consideration simply means that at certain times of the year, Sites E, F, and G (particularly G) could have more intake of suspended sediments (than Sites C and D) due to the Bonaparte contribution. It is not possible to define the amount of extra suspended sediment supply to these sites, but it is expected to be small in comparison to supply from possible upstream cliff erosion.

5. SITE SELECTION BASED ON HYDRAULIC CONSIDERATIONS

After consideration of all the above-discussed factors, Site D is recommended as the preferred site. Site C, then Site G, follow in preference. Site E is not favoured because of its proximity to the rapids, and Site F because of a combination of high velocities, high turbulence and relatively shallow depth near the left bank at that section.

Site D has nominally been located at Cross Section 225+ (750 u/s). If access to this location were made by the construction of a berm along the right bank, a question can be raised as to how far downstream from 225+ could the specific site be. Inspection of the data in NHCL's river survey report of January 1977 (Reference 4) indicates that at Cross Section 135+ (450 u/s) the depth near the right bank is 0.9 metres (3 feet) less. Thus, unless bed excavation is considered, it would appear that the intake location could not be moved very far downstream. (At present cross section data are available only at 90 metre (300-foot) intervals near Site D. Partial cross sections at chosen intervals would be required to refine the intake location any further.)

The practicability of bed excavation cannot be assessed until river bed borehole information is available. If, for example, fine-grained material exists under a 2 or 3 foot armor layer, modifying the bed would entail over-excavation into the fine material followed by the replacement of the armor layer. Otherwise uncontrolled scour could develop. A cost-benefit analysis would determine if bed modification was advisable.

If non-hydraulic considerations strongly favour a site other than D (G, for example) a reasonable question would be: How much better, hydraulically, is Site D? The degree of preference cannot be quantified at this time. However, the magnitude of the design problems associated with higher velocities and higher turbulence (at Sites C and G) would have to be investigated in a conceptual hydraulic model study. (In addition, the higher maintenance costs associated with intake of suspended sediments at Site G, for example, would have to be weighed.) If the problems associated with high velocities and high turbulence are, in fact, not insurmountable, and if pump and wet well maintenance problems are minimized, then Site G could well be as acceptable hydraulically as Site D.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. Site D is the preferred site from the river and hydraulic engineering point of view.
2. Site C is second preference and Site G is third preference. Both appear to be acceptable hydraulically.
3. Because of high velocities and/or high levels of turbulence, Sites E and F appear to be relatively poor alternatives.
4. The exact location of the intake within Site D can only be determined after sub-bed data are available and additional partial cross sections obtained at closer intervals.

6.2 Recommendations

Stage measurements at the selected site will be required for flows over about 3000 m^3/s (106,000 cfs) to confirm the accuracy of estimates made in this study.

REFERENCES

(1) "Low Flows in British Columbia, Annual Seven Day Averages", Inland Waters Directorate, Pacific Region, Water Survey of Canada, Vancouver, January 1974.

(2) Draft of Report to Sandwell by Golder Associates Ltd., "Hat Creek Water Supply, Thompson River Intake, Stability of Cliffs, Ashcroft, B.C.", dated August 1977 (V77047).

(3) "Hat Creek Water Supply, Photo Log, May 20 Reconnaissance, Potential Intake Sites", Northwest Hydraulic Consultants Ltd., May 1976.

(4) "Hat Creek Project Water Supply Study, Thompson River Surveys, Data Report, Site 4A and 10", Northwest Hydraulic Consultants Ltd., January 1977.

(5) "Hat Creek Water Supply, Photo Log, February 22, 1977, Reconnaissance, Thompson River Potential Intake Sites", Northwest Hydraulic Consultants Ltd.

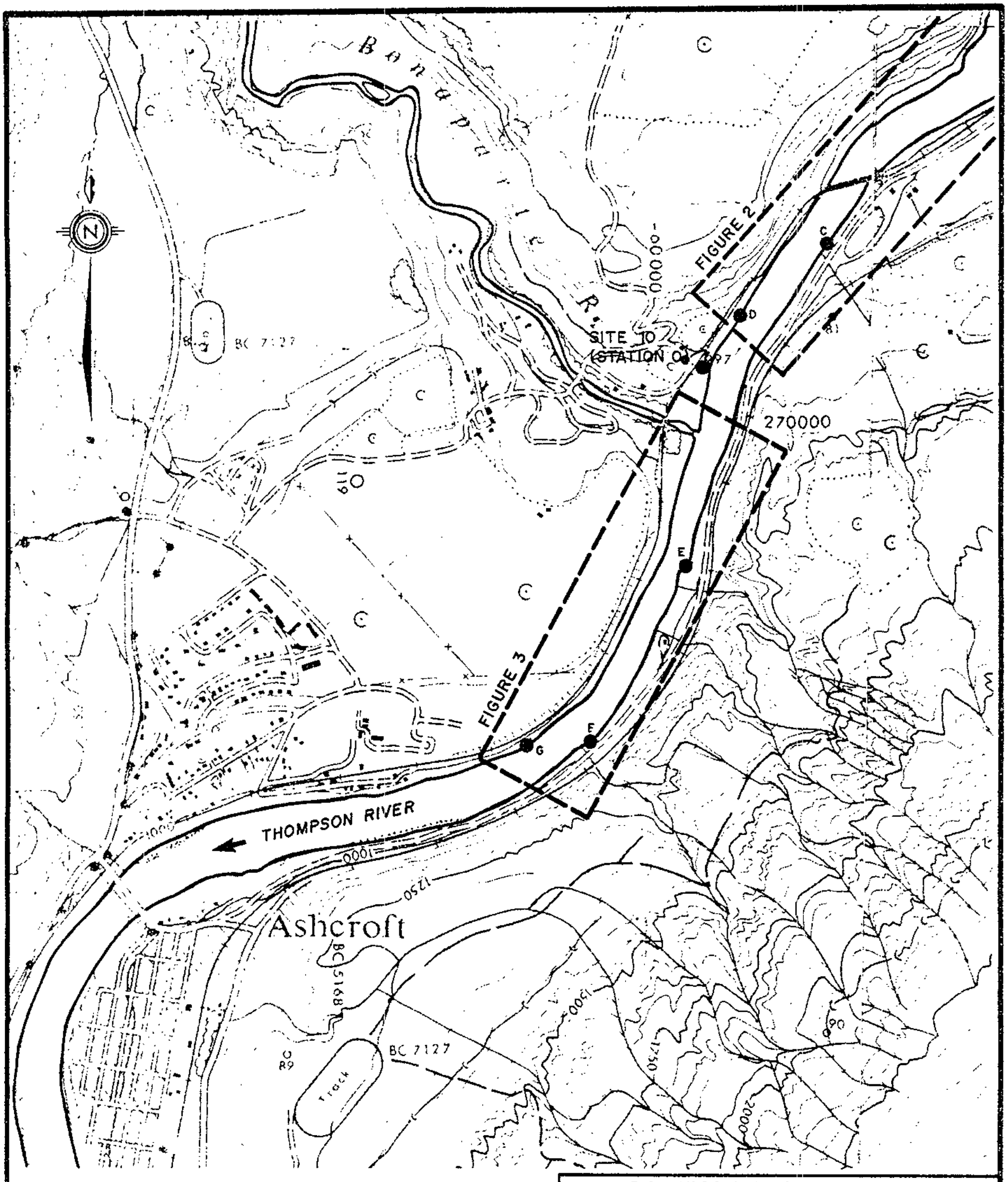
(6) "Hat Creek Water Supply, Hydrology (Interim Report)", Northwest Hydraulic Consultants Ltd., November 1976.

(7) "Hat Creek Water Supply, River Hydraulic Site Assessments and Photo Log, September 15, 1976. Reconnaissance, Selected Intake Sites", Northwest Hydraulic Consultants Ltd.

(8) "Hat Creek Project, Photo Log for Thompson River Surveys, Sites 4A and 10", Northwest Hydraulic Consultants Ltd., December 1976.

(9) "Hat Creek Water Supply Study, Thompson River Survey, CN Bridge to Ashcroft", Northwest Hydraulic Consultants Ltd., March 1977.

FIGURES

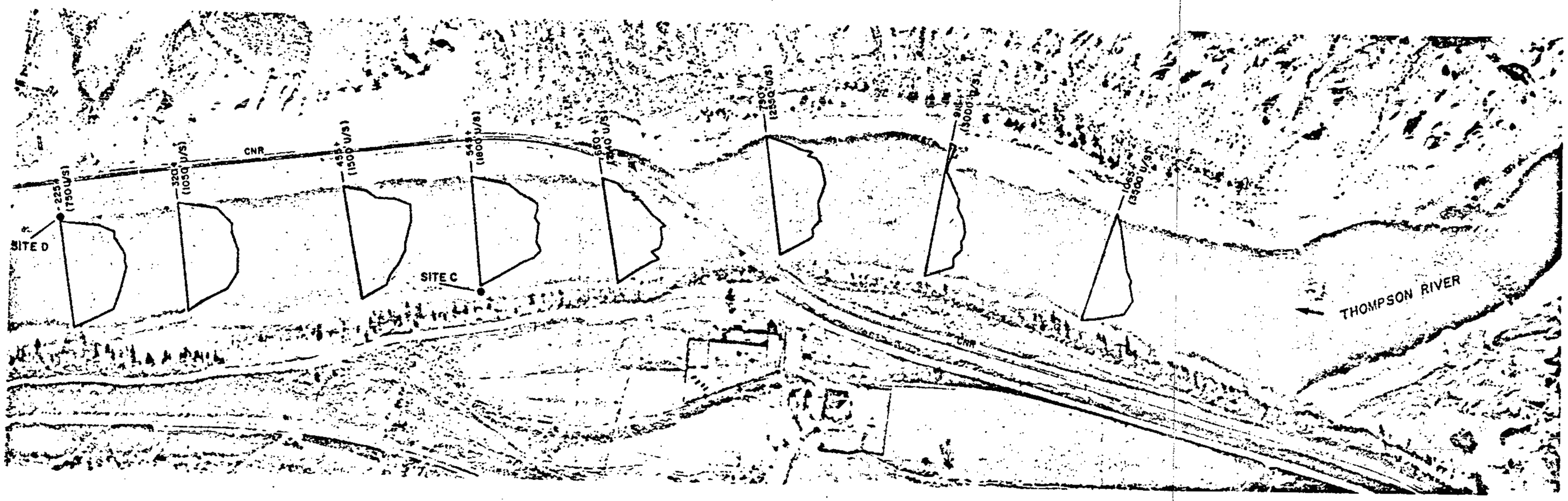


200 0 200
SCALE IN METRES

LEGEND:

● — POTENTIAL INTAKE SITES.

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
LOCATION PLAN		
northwest hydraulic consultants ltd.		
Drawn: SD.	Date: JULY, 1977	FIGURE 1



0 20 40 60 80 100
SCALE (metres)

NOTES:

1. DATE OF PHOTOGRAPH – APRIL 16, 1970.
2. DISCHARGE ON DATE OF PHOTOGRAPH – 225 m^3/s (7950 cfs)
3. DATE OF SURVEY – MARCH 2-3, 1977.
4. DISCHARGE ON DATE OF SURVEY – 200 m^3/s (7000 cfs)
5. SCALE OF CROSS-SECTIONS
   HORIZONTAL – 1:2400
   VERTICAL – 1:240
6. CROSS-SECTION LABELS ARE APPROXIMATE DISTANCE IN METRES (FEET) EITHER UPSTREAM (+ OR U/S) OR DOWNSTREAM (– OR D/S) FROM STATION 0, LOCATED AT SITE 10.

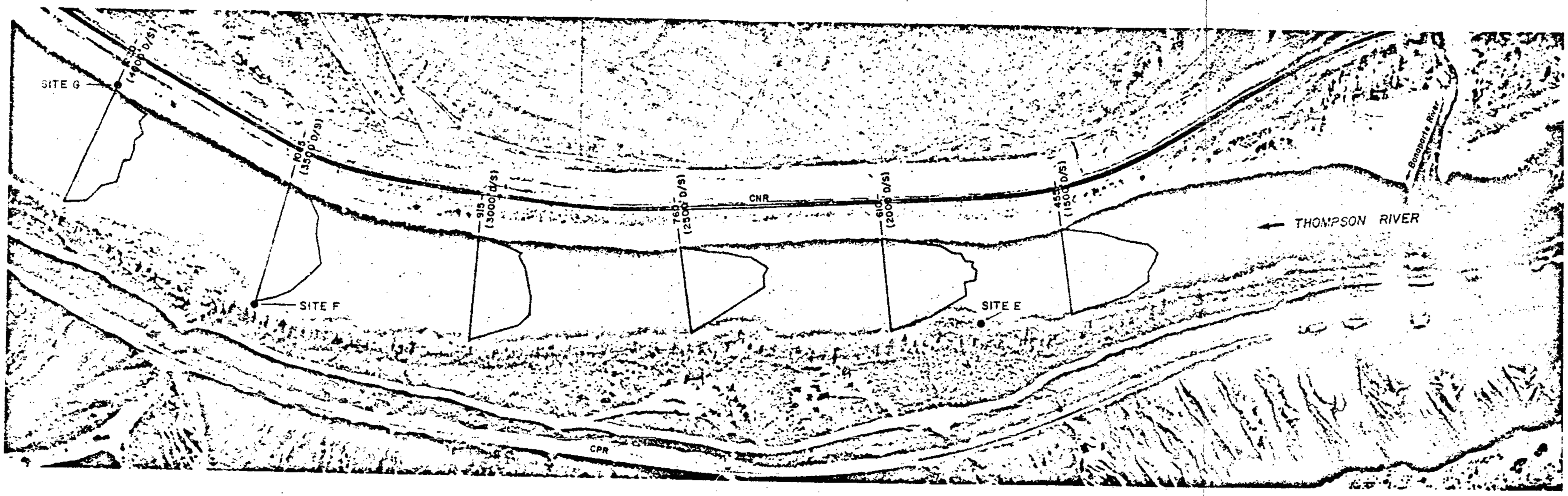
B.C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY

SITE LOCATIONS AND CHANNEL MORPHOLOGY
SITES C AND D

northwest hydraulic consultants ltd.
edmonton vancouver

Drawn SD.	Date JULY, 1977	FIGURE 2

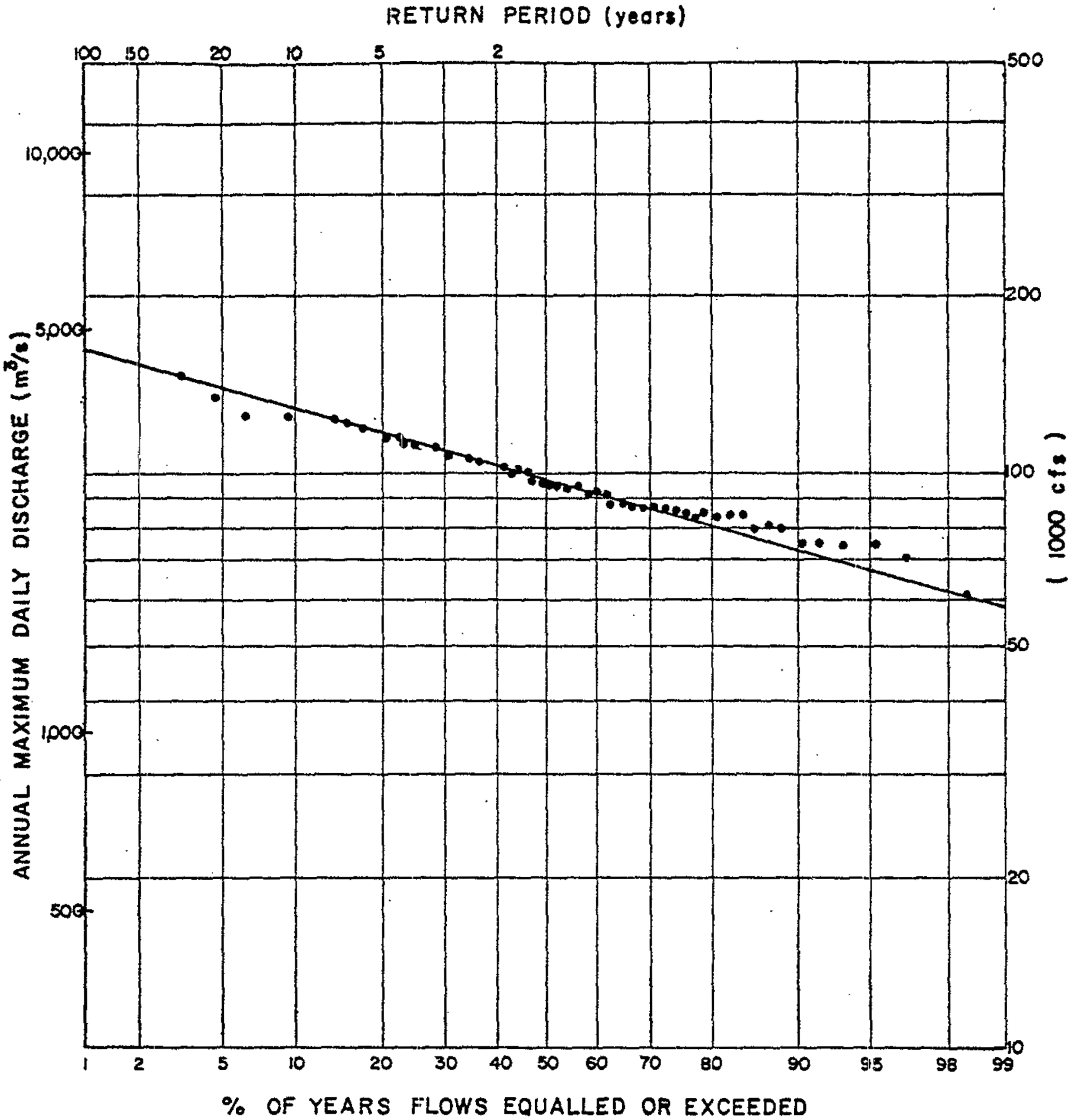


0 20 40 60 80 100
SCALE (metres)

NOTES:

1. DATE OF PHOTOGRAPH – APRIL 16, 1970.
2. DISCHARGE ON DATE OF PHOTOGRAPH – 225 m^3/s (7950 cfs)
3. DATE OF SURVEY – MAR. 2-3, 1977.
4. DISCHARGE ON DATE OF SURVEY – 200 m^3/s (7000 cfs)
5. SCALE OF CROSS-SECTIONS
   HORIZONTAL – 1:2400
   VERTICAL – 1:240
6. CROSS-SECTION LABELS ARE APPROXIMATE DISTANCE IN METRES (FEET) EITHER UPSTREAM (+ OR U/S) OR DOWNSTREAM (– OR D/S) FROM STATION 0, LOCATED AT SITE 10.

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY **SITE LOCATIONS AND CHANNEL MORPHOLOGY** SITES E, F AND G		
northwest hydraulic consultants ltd. edmonton vancouver		
Drawn SD.	Date JULY, 1977	FIGURE 3



NOTES:

1. CURVE SHOWN DEVELOPED FROM DATA OF WSC STATIONS 08 LF 022 AND 08 LF 051
2. PERIOD OF RECORD: 1912 – 1931, 1934 – 1975

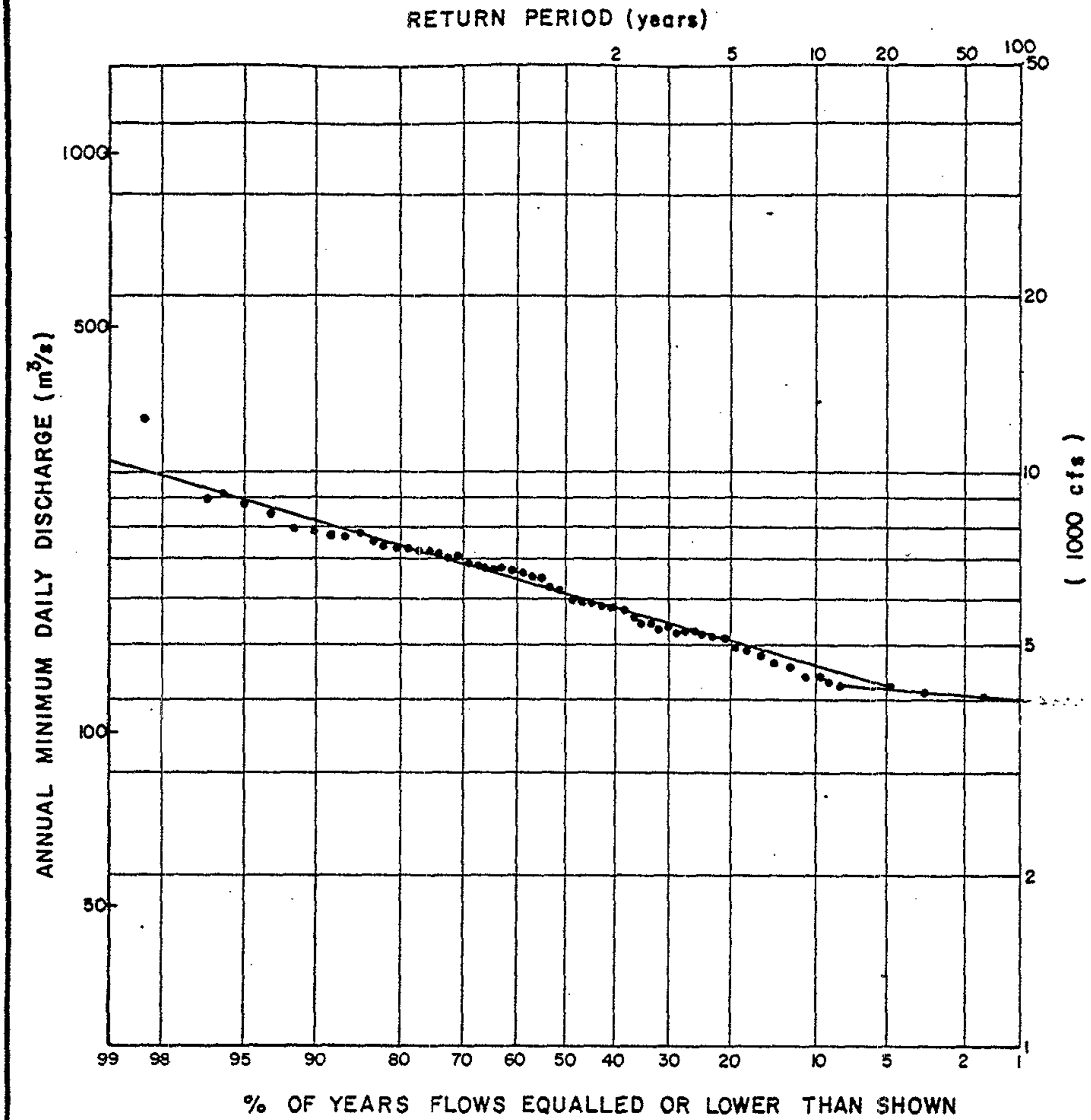
B.C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY

MAXIMUM FLOW FREQUENCY CURVE
THOMPSON RIVER
NEAR SPENCES BRIDGES

northwest hydraulic consultants ltd.

Drawn: V.U.	Date: AUG. 1977	FIGURE 4



NOTES:

1. CURVE SHOWN DEVELOPED FROM DATA OF WSC STATIONS 08 LF 022 AND 08 LF 051
2. PERIOD OF RECORD: 1912 – 1931, 1934 – 1975
3. AVAILABLE DATA INDICATE A LEVELLING OFF OF MINIMUM FLOWS FOR EXTREME LOW-FLOW EVENTS. IT IS ASSUMED THAT THIS PATTERN IS DUE TO THE NATURAL CONTROL OF KAMLOOPS AND SHUSWAP LAKES.

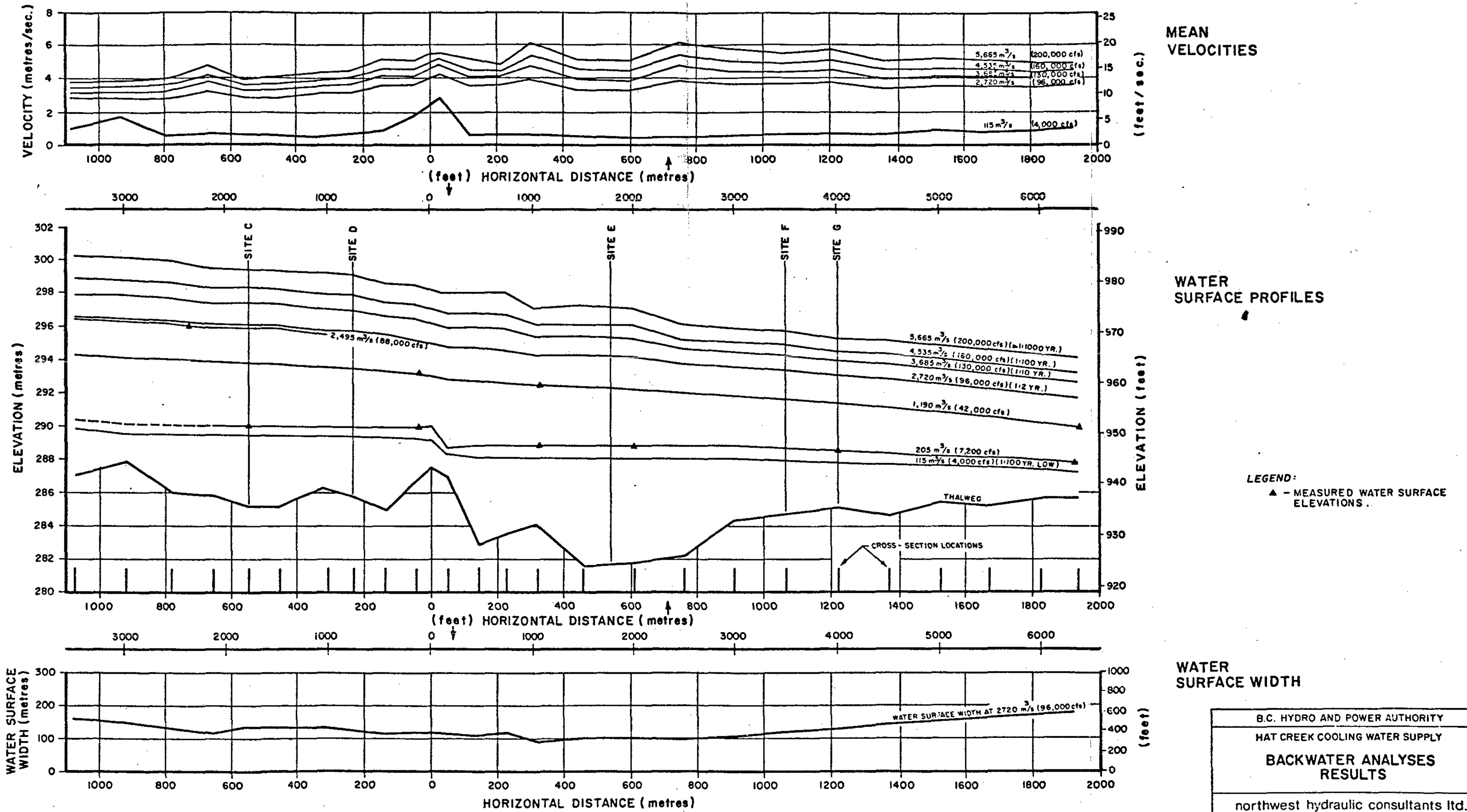
B.C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY

MINIMUM FLOW FREQUENCY CURVE
THOMPSON RIVER
NEAR SPENCES BRIDGES

northwest hydraulic consultants ltd.

Drawn: V.U.	Date: AUG. 1977	FIGURE 5



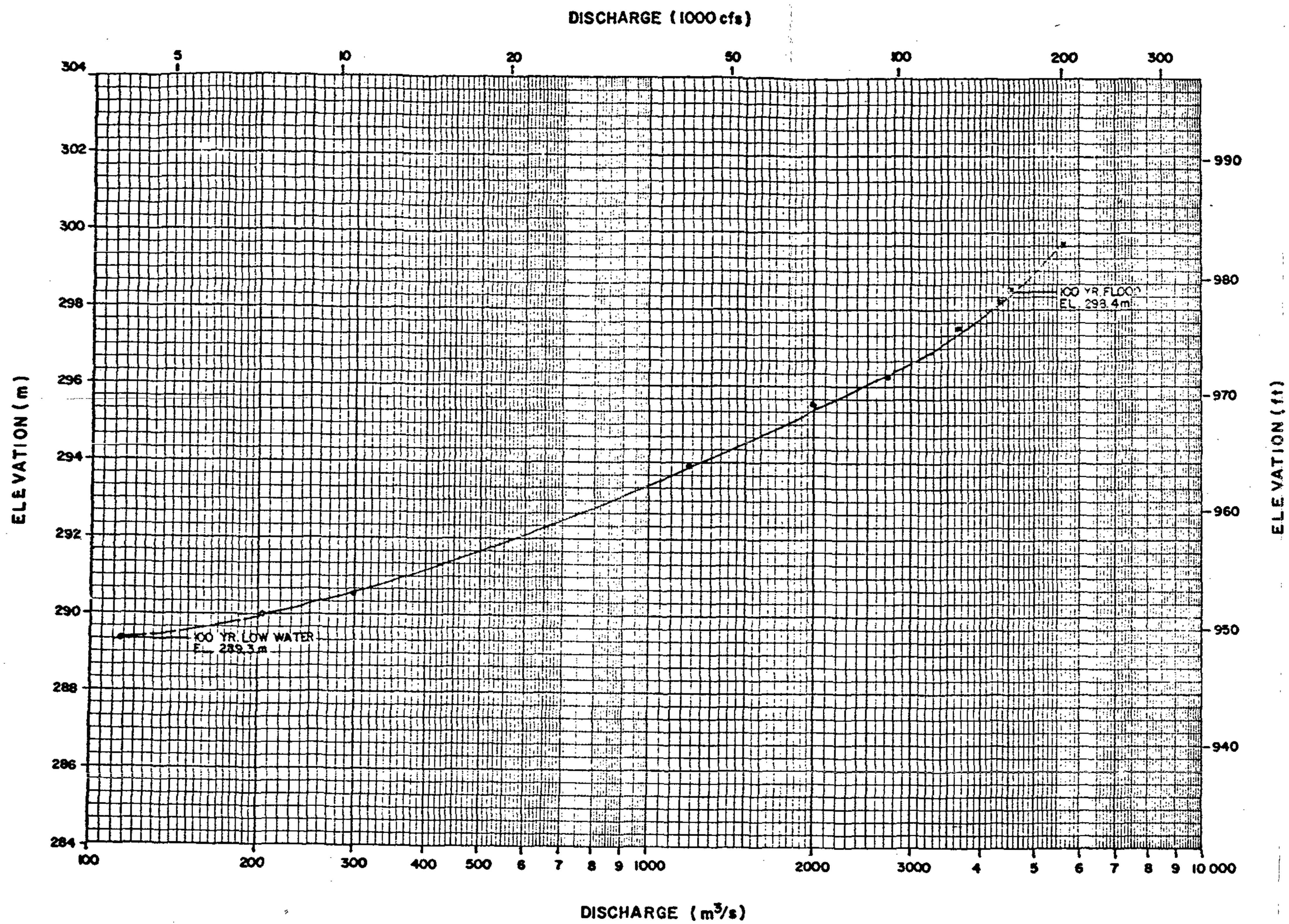
B.C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY

BACKWATER ANALYSES RESULTS

northwest hydraulic consultants ltd.

Drawn SD.	Date JULY, 1977 REV. AUG. 1977	FIGURE 6 1073



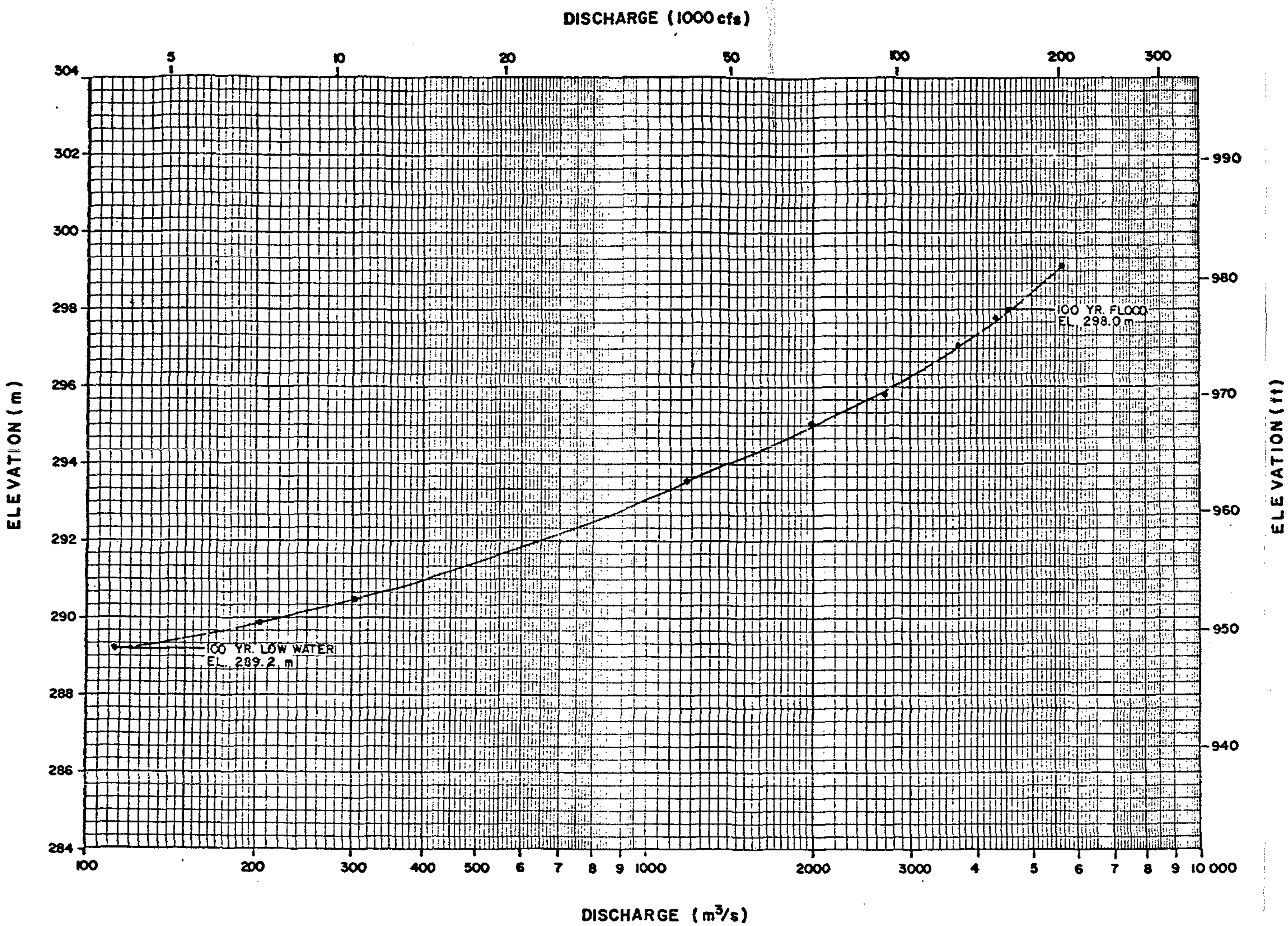
LEGEND :

- ——— CURVE BASED ON AVAILABLE FIELD DATA
- - - - - ESTIMATED CURVE
- • COMPUTER MODEL POINTS

NOTE :

THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL
THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN
200 AND 2 500 m^3/s (7000 AND 68,000 cfs).
PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
STAGE-DISCHARGE CURVE SITE C		
northwest hydraulic consultants ltd.		
Drawn. V. U.	Date. AUG. 1977	FIGURE 7



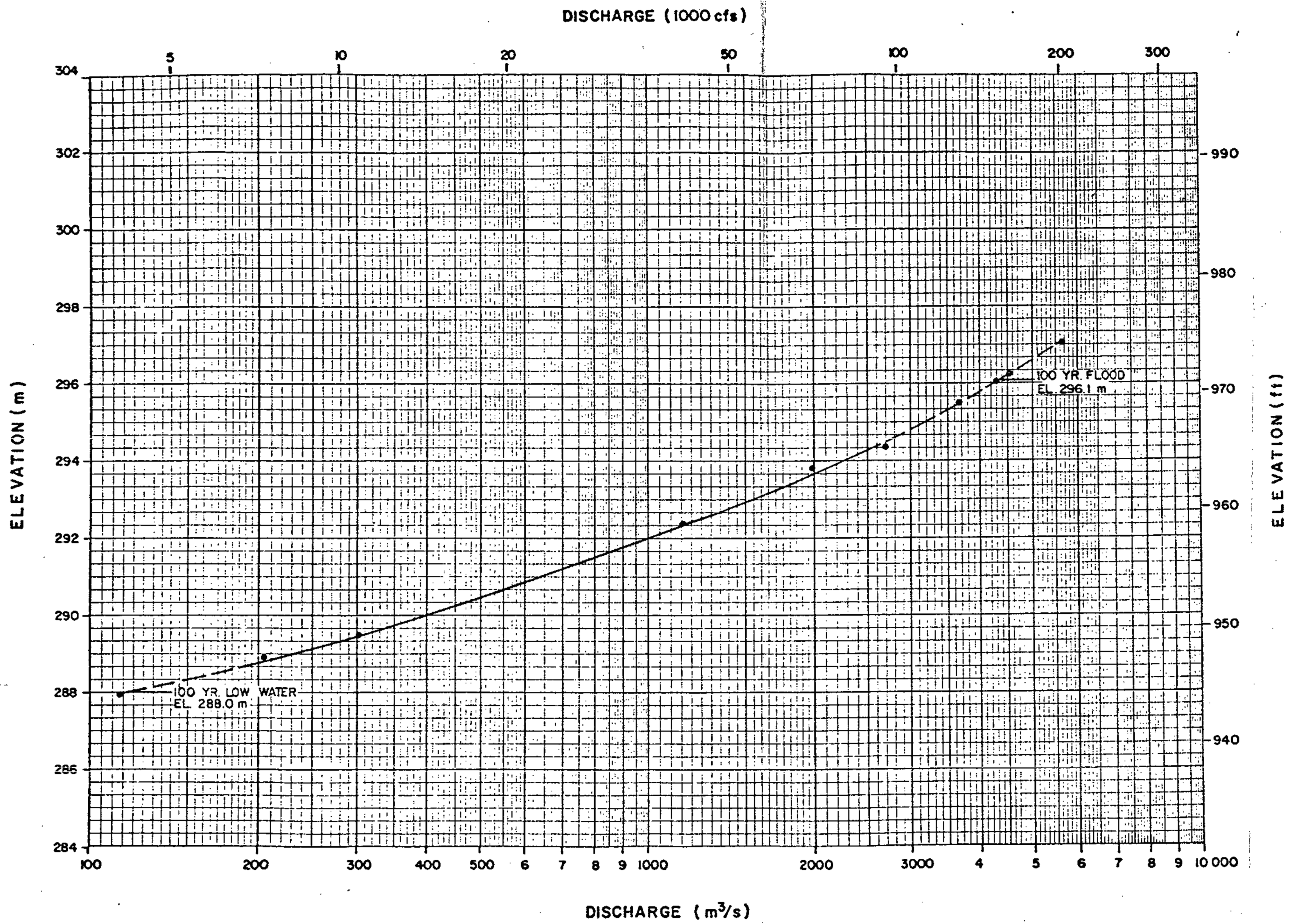
NOTE :

THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL.
THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN 200 AND 2 500 m^3/s (7000 AND 88,000cfs).
PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

LEGEND :

- ——— CURVE BASED ON AVAILABLE FIELD DATA
- - - - - ESTIMATED CURVE
- • COMPUTER MODEL POINTS

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
STAGE-DISCHARGE CURVE SITE D		
northwest hydraulic consultants ltd.		
Drawn. V. U.	Date. AUG. 1977	FIGURE 8



NOTE :

THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL.
THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN
200 AND 2 500 m^3/s (7000 AND 88,000 cfs).
PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

LEGEND :

- ——— CURVE BASED ON AVAILABLE FIELD DATA
- – – – ESTIMATED CURVE
- • COMPUTER MODEL POINTS

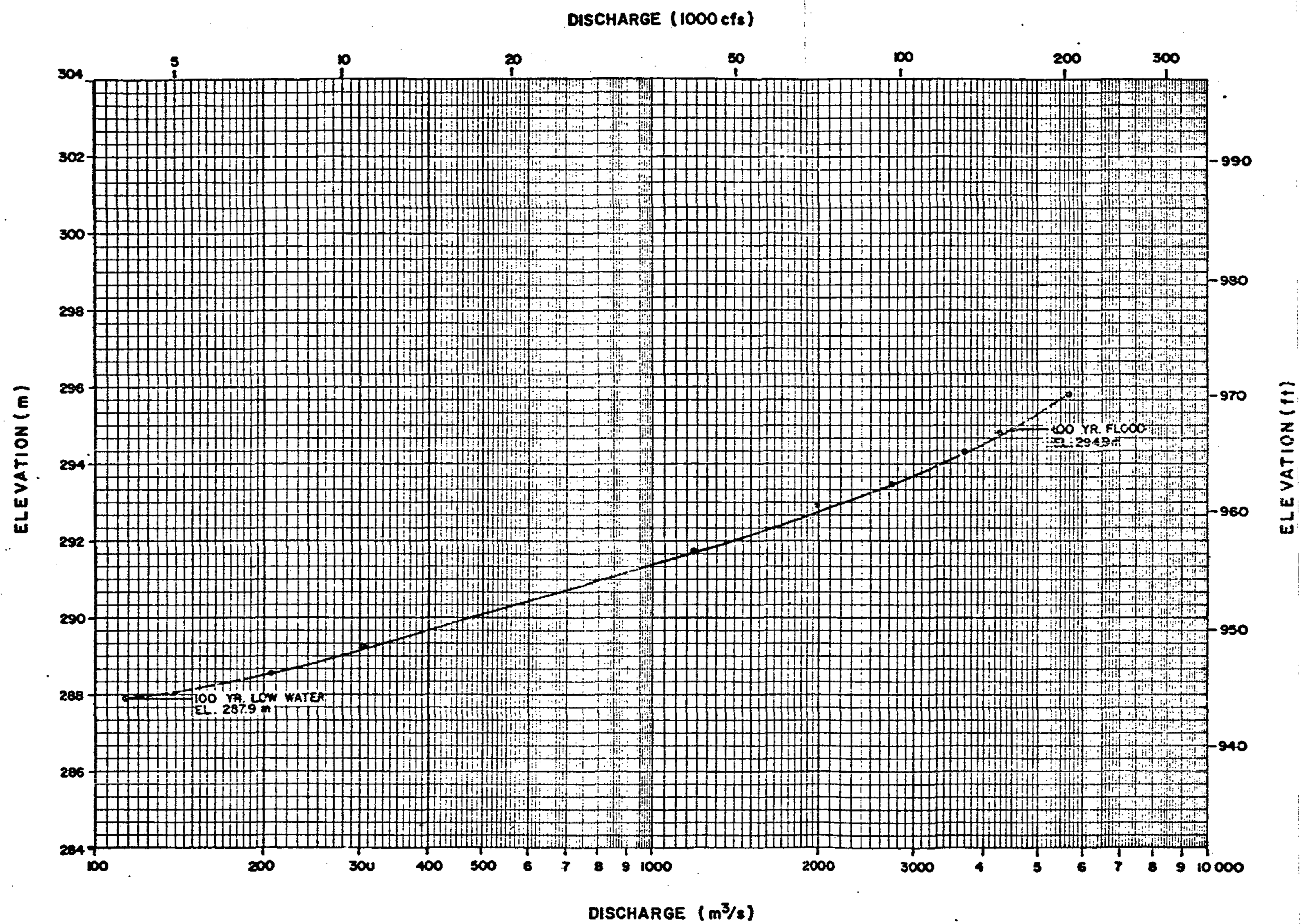
B. C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY

STAGE - DISCHARGE CURVE
SITE E

northwest hydraulic consultants ltd.

Drawn: V. U.	Date: AUG. 1977	FIGURE 9



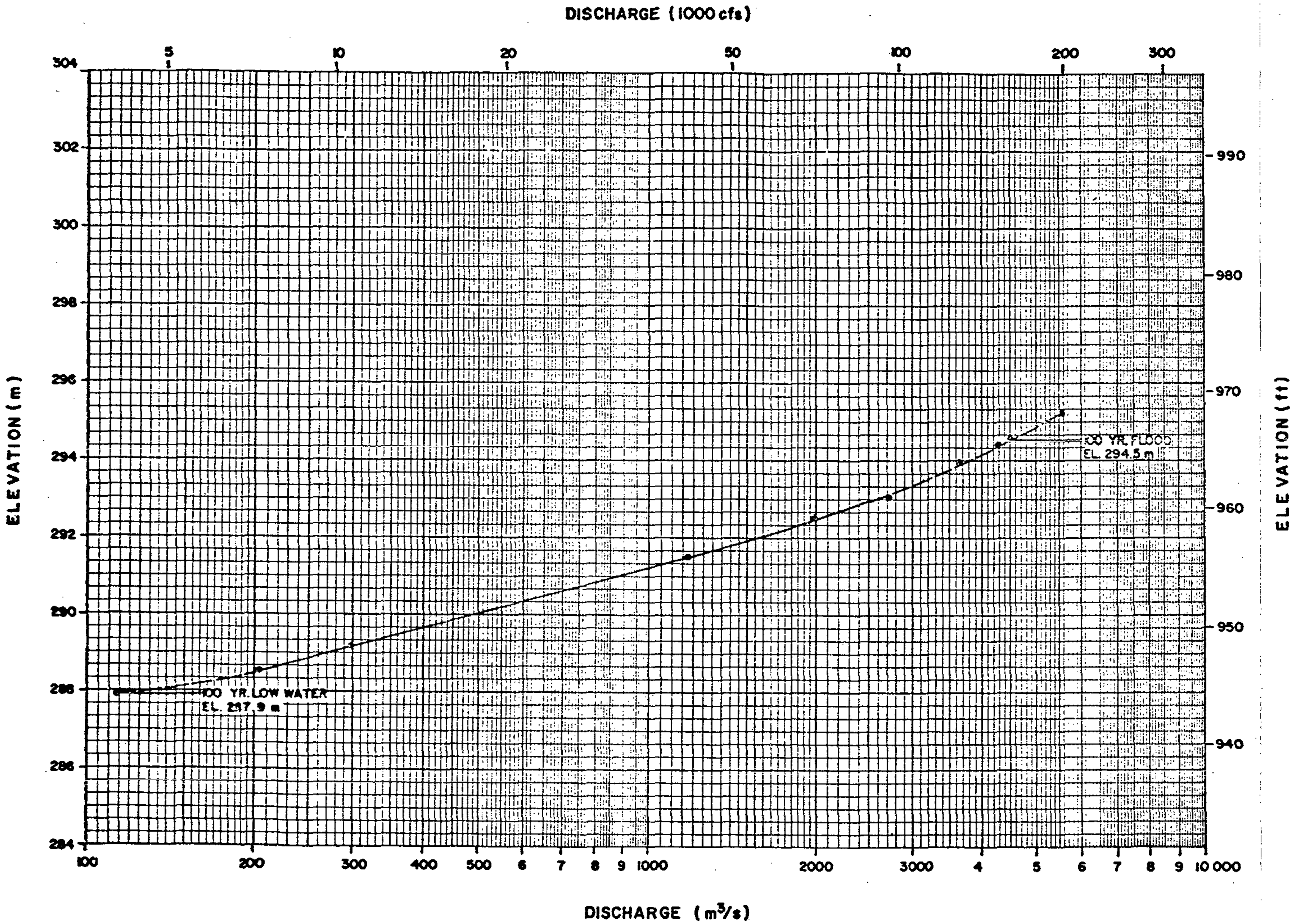
LEGEND:

- ——— CURVE BASED ON AVAILABLE FIELD DATA
- - - - - ESTIMATED CURVE
- • COMPUTER MODEL POINTS

NOTE:

THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL.
THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN 200 AND 2 500 m^3/s (7000 AND 88,000 cfs).
PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
STAGE-DISCHARGE CURVE SITE F		
northwest hydraulic consultants ltd.		
Drawn: V. U.	Date: AUG. 1977	FIGURE 10



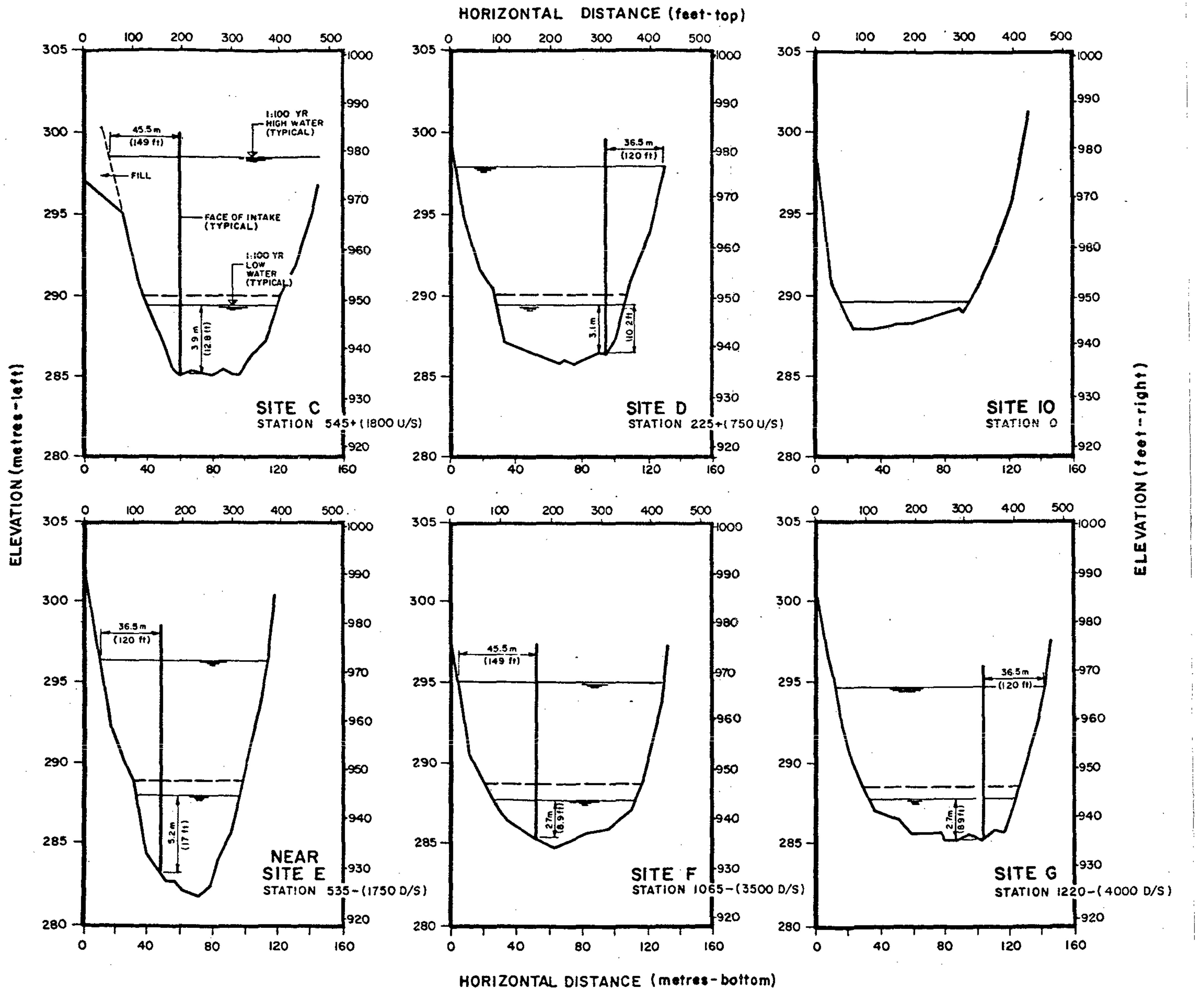
NOTE :

THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL.
THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN
200 AND 2 500 m^3/s (7000 AND 88,000 cfs).
PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

LEGEND :

- ——— CURVE BASED ON AVAILABLE FIELD DATA
- - - - - ESTIMATED CURVE
- • COMPUTER MODEL POINTS

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
STAGE - DISCHARGE CURVE SITE G		
northwest hydraulic consultants ltd.		
Drawn. V. U.	Date. AUG. 1977	FIGURE 11



NOTES:

1. CROSS-SECTION SCALES:
   HORIZONTAL - 1:2500
   VERTICAL - 1:250
2. ALL SECTIONS VIEWING DOWNSTREAM.
3. HORIZONTAL DOTTED LINE IS WATER LEVEL DURING NHCL SURVEYS.

B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY		
CROSS-SECTIONS		
northwest hydraulic consultants ltd.		
Drawn SD.	Date JULY, 1977 REV: SEPT. 1977	FIGURE 12 1073

APPENDIX "A"

"THOMPSON RIVER NEAR ASHCROFT
WATER LEVEL MONITORING PROGRAM,
DECEMBER 1976 - JULY 1977"

Table 1 - Water Surface Elevations in Metric Units

Date	Intake Site: 4A, 600 Ft Down Stream, Elevation (M)	Change (M)	Intake Site: 10, 150 Ft Upstream, Elevation (M)	Change (M)	Intake Site: 10, 1050 Ft Down Stream, Elevation (M)	Change (M)	Thompson River (See Note 2): Discharge (M³/s)	Change in Discharge (M³/s)
76 Dec 6	303.74		290.21		289.23		298	
		- 0.32		- 0.49		- 0.54		- 97
77 Feb 1	303.41		289.72		288.69		201	
		- 0.01		- 0.02		- 0.01		- 6
Feb 14	303.40		289.70		288.68		195	
		0		- 0.02		- 0.02		0
Mar 1	303.40		289.68		288.66		195	
		- 0.04		- 0.03		- 0.04		- 6
Mar 15	303.36		289.65		288.62		189	
		- 0.02		- 0.01		0		- 3
Mar 31	303.34		289.64		288.62		186	
		+ 0.43		+ 0.57		+ 0.63		+ 102
Apr 15	303.77		290.21		289.25		288	
		+ 1.32		+ 1.68		+ 1.76		+ 566
May 2	305.09		291.89		291.01		854	
		+ 0.89		+ 1.32		+ 1.33		+ 425
May 15	305.98		293.21		292.34		1,279	
	10, 6300 Ft Down Stream							
				- 0.09		- 0.07		- 25
May 29	289.96		293.12		292.27		1,254	
		- 0.19		- 0.26		- 0.25		- 116
June 2	289.77		292.86		292.02		1,138	
		+ 0.54		+ 0.84		+ 0.76		+ 377
June 15	290.31		293.70		292.78		1,515	
		- 0.16		- 0.24		- 0.23		- 167
July 2	290.15		293.46		292.55		1,348	
		- 0.36		- 0.63		- 0.53		- 193
77 July 15	289.79		292.83		292.02		1,155	

Note: 1. This program of recording river water levels was terminated on 15 July 1977 as the peak of the freshet had passed the intake site.

2. Discharges are for Sites 4A and 10 (150 ft upstream). Discharges were obtained by subtracting Bonaparte and Nicola River flows (Stations 8LF02 and 8LG06) from Thompson River flow at Spences Bridge (Station 8LF51).

Table 2 - Water Surface Elevations in Imperial Units

Date	Intake Site 4A 600 Ft Down Stream Elevation (Ft)	4A Change (Ft)	Intake Site 10 150 Ft Upstream Elevation (Ft)	Change (Ft)	10 1050 Ft Down Stream Elevation (Ft)	Change (Ft)	Thompson River (See Note 2) Discharge (CFS)	Change in Discharge (CFS)
76 Dec 6	996.53		952.14		948.94		10,500	
		- 1.06		- 1.60		- 1.77		- 3,400
77 Feb 1	995.47		950.54		947.17		7,100	
		- 0.04		- 0.07		- 0.04		- 200
Feb 14	995.43		950.47		947.13		6,900	
		- 0.03		- 0.05		- 0.07		0
Mar 1	995.40		950.42		947.06		6,900	
		- 0.11		- 0.10		- 0.13		- 200
Mar 15	995.29		950.32		946.93		6,700	
		- 0.05		- 0.05		- 0.01		- 100
Mar 31	995.24		950.27		946.92		6,600	
		+ 1.40		+ 1.89		+ 2.07		+ 3,600
Apr 15	996.64		952.16		948.99		10,200	
		+ 4.32		+ 5.49		+ 5.78		+ 20,000
May 2	1000.96		957.65		954.77		30,200	
		+ 2.93		+ 4.35		+ 4.37		+ 15,000
May 15	1003.89		962.00		959.14		45,200	
	10 6360 Ft Down Stream							
				-0.31		-0.22		- 900
May 29	951.34		961.69		958.92		44,300	
		-0.64		-0.83		-0.82		- 4,100
June 2	950.70		960.86		958.10		40,200	
		+1.78		+2.75		+2.49		+ 13,300
June 15	952.48		963.61		960.59		53,500	
		-0.53		-0.81		-0.75		- 5,900
July 2	951.95		962.80		959.84		47,600	
		-1.19		-2.02		-1.74		- 6,800
77 July 15	950.76		960.75		958.10		40,800	

Note: 1. This program of recording river water levels was terminated on 15 July 1977 as the peak of the freshet had passed the intake site.

2. Discharges are for Sites 4A and 10 (150 ft upstream). Discharges were obtained by subtracting Bonaparte and Nicola River flows (Stations 8LF02 and 8LG06) from Thompson River flow at Spences Bridge (Station 8LF51).

Table 3 - Water Surface Profile

Intake Site 10

Location (See Note 3)		Elevation	
m	ft	m	ft
+1065	3500 u/s	290.40	952.50
+ 915	3000	290.09	951.50
+ 780	2550	290.03	951.30
+ 670	2192	289.98	951.14
+ 365	1192	289.94	950.99
+ 140	450	289.83	950.65
+ 45	150 u/s	289.79	950.54
0	centerline	289.64	950.02
- 35	108 d/s	289.61	949.93
- 65	208	289.56	949.77
- 125	408	288.75	947.13
- 245	808	288.78	947.20
- 320	1050	288.78	947.20
- 550	1808	288.73	947.06
- 850	2808	288.70	946.96
-1010	3308	288.66	946.81
-1160	3808	288.50	946.30
-1315	4308	288.42	946.03
-1465	4808	288.16	945.18
-1730	5808	288.00	944.64
-1925	6308	287.81	944.03
-2075	6808 d/s	287.40	942.68

Notes

1. All stations upstream of and including +780 m (2550 ft u/s) were surveyed by NHCL on 2 and 3 March 1977 when river discharge was approximately 200 m^3/s (7,000 cfs).
2. All stations downstream of and including +670 m (2192 ft u/s) were surveyed by McElhanney on 31 January through 2 February 1977 when river discharge was approximately 205 m^3/s (7,300 cfs).
3. For locations of river cross sections see Drawing A4191/3 - 3