

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT

Sandwell and Company Ltd. - Hat Creek Project - Cooling Water
Supply Preliminary Design Study - Volume 3 - March 1978

ENVIRONMENTAL IMPACT STATEMENT REFERENCE NUMBER: 8c

REPORT V4191/1
MARCH 1978

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

HAT CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
VOLUME THREE OF THREE

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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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REPORT TO
SANDWELL AND COMPANY
ON
HAT CREEK WATER SUPPLY
THOMPSON RIVER INTAKE
STABILITY OF CLIFFS

ASHCROFT BRITISH COLUMBIA

December 1977

V77047

Golder Associates

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1.0 SUMMARY AND CONCLUSIONS

Rev. 1

a) The general mode of degradation of the cliffs on the right bank of the Thompson River upstream of Ashcroft seems to be one of continuing small scale collapse rather than major sliding. Continuing observations and photographic recordings of the degradation of the cliffs should be carried out to confirm this. A suitable time interval would be twice yearly, after the winter run off and in the late fall.

However, large scale sliding involving up to about 200-300,000 m³ (260-400,000 cu. yds.) of material can occur in the cliffs. Significantly larger volume slides seem unlikely unless hydrostatic water pressures occur in strata behind the cliffs or unless the cliff materials are underlain by weak strata such as clay shale seams. Our assessment of stability is based on the assumption that such detrimental conditions do not exist. To confirm this, drilling and sampling could be carried out from the top of the cliffs, at 2 or 3 locations together within the installation of piezometers to measure pore water pressures.

b) In the unlikely event that large scale sliding of the order of 1/4 million m³ (1/3 million cu. yds.) occurs, the material is unlikely to dam off the river because it would form a shallow angle cone at the base of the cliffs. The river would subsequently erode the toe of the cone and transport the material downstream.

c) The steep silt cliffs just upstream of the Bonaparte River and downstream of the CN Rail bridge appear to be in a marginal state of stability being heavily incised and oversteepened. If an intake were located directly below or opposite these cliffs, consideration should be given to flattening the cliffs. This would offer protection not only to the water supply intake but also to disruption of the CN Rail at the base of the cliffs. Much improvement could be achieved by cutting back the cliffs from El. 330 m (1,080 ft.) to about El. 370 m (1,210 ft.) to a 40° slope. This would be necessary over a length of about 150 m (500 ft.).

Rev. 2

d) A collapse large enough to cause blockage of the river could occur if intensive irrigation of the land above cliffs alongside the river is carried out. For this reason it is recommended that B.C. Hydro and Power Authority should take precautions to ensure that irrigation is not allowed on land above cliffs alongside the river at any location where blockage of the river resulting from a landslide could seriously impede operation of the water supply intake.

2.0 INTRODUCTION

Based on instructions from Sandwell, Golder Brawner & Associates (GBA) carried out detailed surface soils investigations of the high bluffs on the right bank (west and north) of the Thompson River from just upstream of the confluence with the Bonaparte River to the second CNR bridge about 3-1/2 km (2 miles) upstream. The location is shown in Figure 1 and a panoramic view of the section of the river under study is shown on Plate 1 view a. At the request of B.C. Hydro, drilling was not included in this study. Stability of the bluffs was assessed on the basis of examination of air photography, mapping of the soil exposures on the cliff face and slope stability analyses.

An initial investigation of the cliffs was made on February 22, 1977 during the course of a helicopter reconnaissance of the Thompson River in the company of Sandwell and representatives of Northwest Hydraulic Consultants. Helicopter landings were made at various locations on the right bank of the river, which at that time was at a low flow of 200 m³/s (7,000 cfs.). The materials were examined and their properties assessed both at river level, and at various levels above the river attained by climbing the bluffs. This was carried out by N.A. Skermer of

GBA, who in addition carried out detailed geotechnical mapping from the left bank of the river on June 13-15, 1977. This latter information was plotted on topography prepared from 1970 air photographs that had been flown by Pacific Survey and were available at a scale of 1 to 12,000. This was the best available recent air photography from which to prepare contour drawings. Topographic mapping was produced by McElhanney Surveying & Engineering Ltd. at a scale of 1:2000 and at 10 m contour intervals. They also prepared topography from 1 to 33,480 air photographs flown in 1948 by the B.C. provincial government, and from a further set of 1 to 24,000 photographs flown by McElhanney in 1976. This was done to enable a comparison to be made of the rate of erosion from the cliffs over a reasonable time span. Examination of the topography indicated areas for plotting of cross-sections, and this information was prepared by McElhanney.

3.0 DESCRIPTION OF CLIFF DEPOSITS

The full extent of the exposures mapped on the right bank is shown on Drawing 1 and details of the soil deposits that were found at each location are shown on Drawings 2 and 3.

The materials exposed on the cliff faces are entirely fine to coarse sized granular materials ranging in particles from silt, through sand and gravel to occasional cobble and boulder seams. Clay and till strata may be present at greater depths below the base of the cliffs. The exposed materials are predominantly glacial outwash with minor amounts of slopewash material - colluvium. Many of the exposures on the lower slopes towards river level are covered by talus derived from erosion of material

above. The erosion has resulted from rain, frost and wind action, and under-cutting by the river, resulting in collapse of the slabs.

Bedrock is exposed at one location in the back of the gully at approximately station 22,300N and 31,900E, see Drawing 3. Elsewhere east of this location the rock can be seen exposed above and at the back of the cliffs on the flank of Rattlesnake Hill. The cover of glacial deposits in this section of the cliffs is relatively thin compared to downstream of the bend.

Major bodies of glacial lake deposited silt are present in this section of the Thompson River. The lakes are believed to have been formed by ice damming in the valley further downstream at the close of the glacial period. The silt is clearly visible in these cliffs as a creamy white near vertical standing slope often heavily incised by water erosion. The incisions are particularly noticeable in the cliffs just upstream of the Bonaparte River and immediately downstream of the CN bridge. Inspection of the top of the cliffs showed that the incisions have cut back 10-15 m (30-50 ft.) so that the silt in the cliffs tends to become isolated into individual blocks. These blocks rise very steeply above the CN Railway and, as discussed later, if a large collapse occurred the debris material would very likely over-run the rail into the river.

Alluvial fans, gravelly deltaic deposits and sandy gravelly colluvial materials cap the silt at the top of the cliffs in most locations. In general this capping layer is thin, but at the bend between 22,000N and 31,600E the colluvial material is much thicker and overlies a collapse or slump structure in the underlying glacial lake sediments. The slump may have been caused by melting of a large block of stagnant glacial

ice, and the result is dipping and faulting in the soil beds. This can be seen in Plate 1 view b. The colluvial material in this section is a well graded silty sandy gravel bedded near horizontally, which would have been derived from slope wash material from the hillside to the north.

A general description of the glacial geology of this section of the Thompson River can be found in the following publication:

"Terrain Inventory and Quaternary Geology Ashcroft, British Columbia", J.M. Ryder, Page 74-49, Geological Survey Canada 1976.

4.0 CLIFF STABILITY

Many sections of the cliffs are standing near to vertical for heights up to about 50 m (160 ft.). This applies to slopes in both the thick silt deposits and the more sandy gravelly colluvium at the bend in the river. In a thoroughly broken down condition these materials stand at an angle of repose of about 35° as is demonstrated by the accumulations of talus materials. Where slopes are standing at steep angles, the material in its undisturbed state possesses a small amount of cohesion. This may have arisen due to the ground water flowing through the material in the past and weakly cementing the particles with carbonate or silicate compounds. In the case of the massive silt beds, the cementing agent probably includes clay, which becomes even harder on desiccation as occurs near to the face of the slopes. Visual examination of the soils in the slopes indicates that to varying degrees, cohesion is present in all the materials. Some of the sandy material is so strongly cemented that it is classified as a weak sandstone.

The topography of the right bank of the river is shown on Drawing No. 1, the full set of contours shown faint in the background being from

the 1970 topography. The heavy lines at the top of the bank are from the 1948, 1970 and 1976 mapping and are shown to indicate the recession of the cliffs. Sections 1 to 7 indicated on the plan are shown on Figures 2 to 8. These illustrations show that since 1948 the major areas of cliff erosion have been confined to: (a) slide in the silt bluffs around section 7, which would appear to have been a single slide of about $200,000 \text{ m}^3$, (260,000 cu. yds.), and (b) the large, but probably discontinuous, erosion of the cliffs in the bend between 22,100N and 31,600E. The continuing process of erosion in the bend shows up by examining the earliest aerial photography of the river taken in 1928, which unfortunately was not good enough for topography mapping in the detail required in this study.

During the field mapping on 13-15 June 1977, small scale erosion of the cliffs in the bend was seen to be occurring more or less continuously. Sand and silt poured down the face of the cliffs in various chutes, and on one occasion prior to a storm when a strong wind blew, the cliffs were practically obscured in a cloud of whirling dust. This process of small scale erosion is clearly apparent by comparing photographs taken on 22 June and 15 November 1977, respectively Plates 2 and 3. The volume of material that appears to have slipped from the top of the cliffs is about $3-4000 \text{ m}^3$ and most of this material has finished up in the river directly below. Another interesting feature that appears on the November picture is the cone-fan at river level on the left of the view. This seems to have formed from material breaking loose from about the mid height of the cliffs just above the cone.

In addition to small scale erosion, however, the slide scarps apparent on section 2 and on Plate 1 view b show that larger unit slides can also occur in this area.

It is possible to make an estimate of the average value of the cohesion of the soil in any section of the slope using Taylor's method of slope stability analysis and by making the assumptions that:

- a) the overall slope material is homogeneous,
- b) water seepage pressures do not exist within the potential sliding mass of soil.

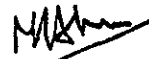
Such analyses are rather simplistic, but give a valuable indication of the condition of stability of the bluffs and of the volume of material that could be involved in sliding. The failure arc is assumed to be circular and this is reasonably valid as evidenced by the scarps of the existing slides in the cliffs. The critical failure arc for steep slopes passes through the toe of the slope with the lowest point on the failure surface being at the toe of the slope, see Figure 8. This analysis was applied to the failure of the silt bluffs at section 7 where the slope height was 115 m (337 ft.) and the average slope angle prior to failure was 62 degrees. Assuming that the angle of internal friction for the silt is 30° , the calculated value of the cohesion is 130 kPa (19 psi) which roughly agrees with the consistency of the material in hand specimens. A similar analysis of the failure in the sands and gravels at sections 2 and 3 where the slope height is 110 m (360 ft.), the slope angle prior to failure was 40 degrees and assuming angle of internal friction of 35° yields a cohesion of 20 kPa (3 psi). This is less than the value calculated for the silt but it is consistent with the coarser particle size of the materials at this location.

Yours very truly

GOLDER BRAUNER & ASSOCIATES LTD.



Per: N.R. McCammon, P. Eng.



N.A. Skermer, P. Eng.

Rev. 1 p. 1 G.A., Dec., 1977 - Description of general mode of collapse of
cliffs.

Rev. 2 p. 2 G.A., Dec., 1977 - Recommendation on control of irrigation
above cliffs.

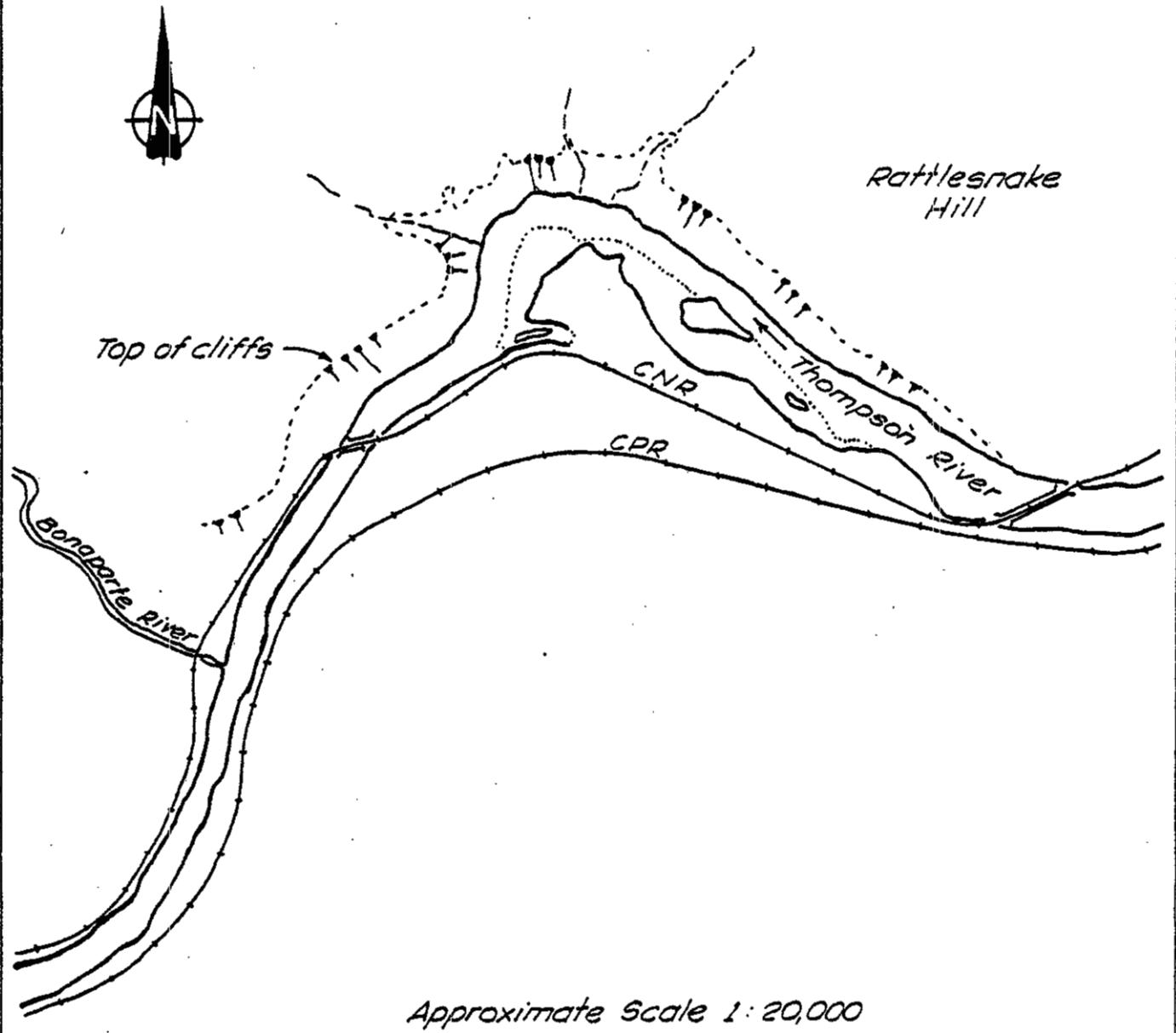
Rev. 3 p. 6 G.A., Dec., 1977 - Plates 2 and 3 added showing 1977 slip from
cliffs and cone-fan below.

NRM/NAS:rme

V77047

LOCATION PLAN

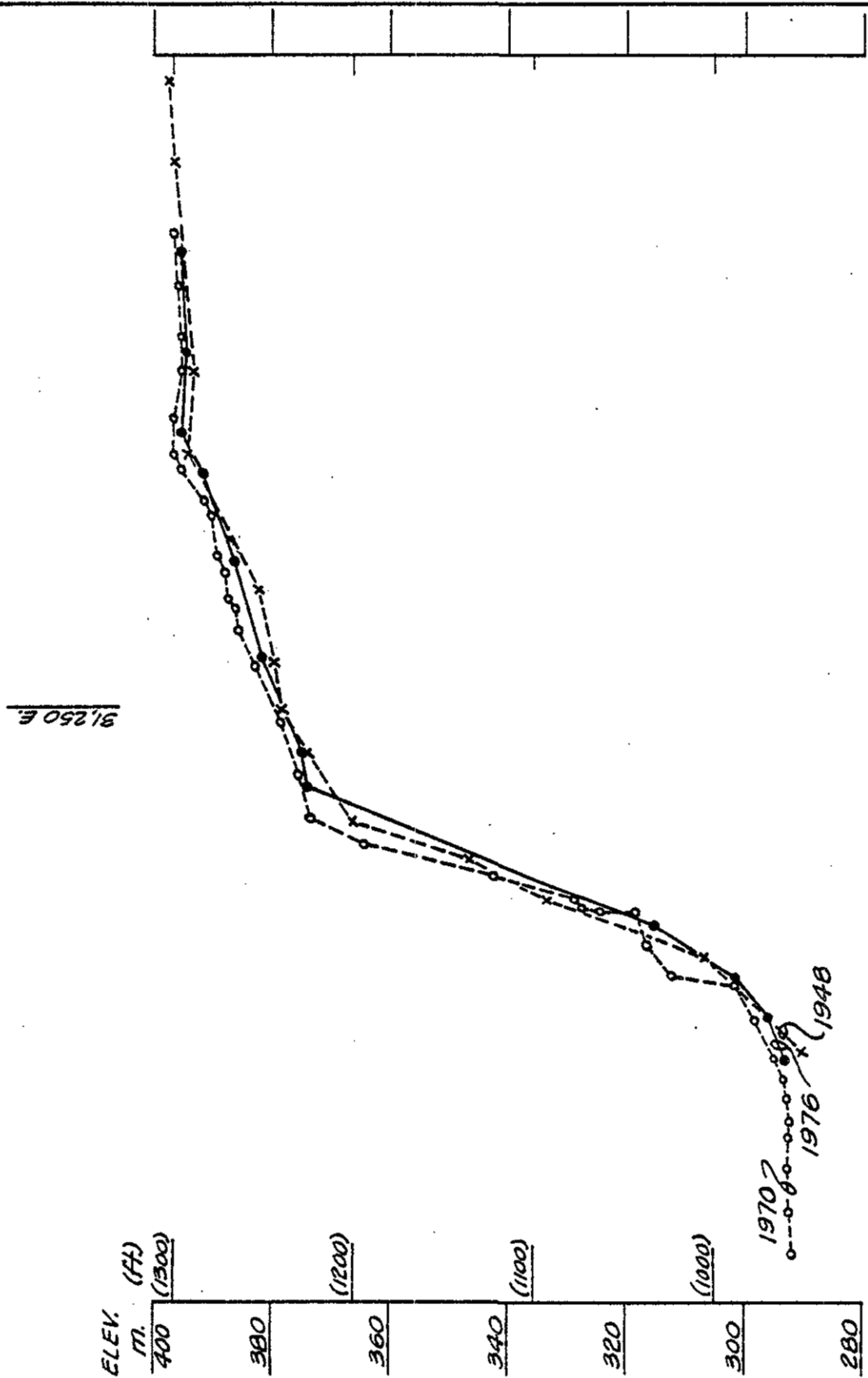
Figure 1



Project No. Y77ca7 Draw MAS Reviewed Date 21.3.77

SECTION 1-1

Figure 2



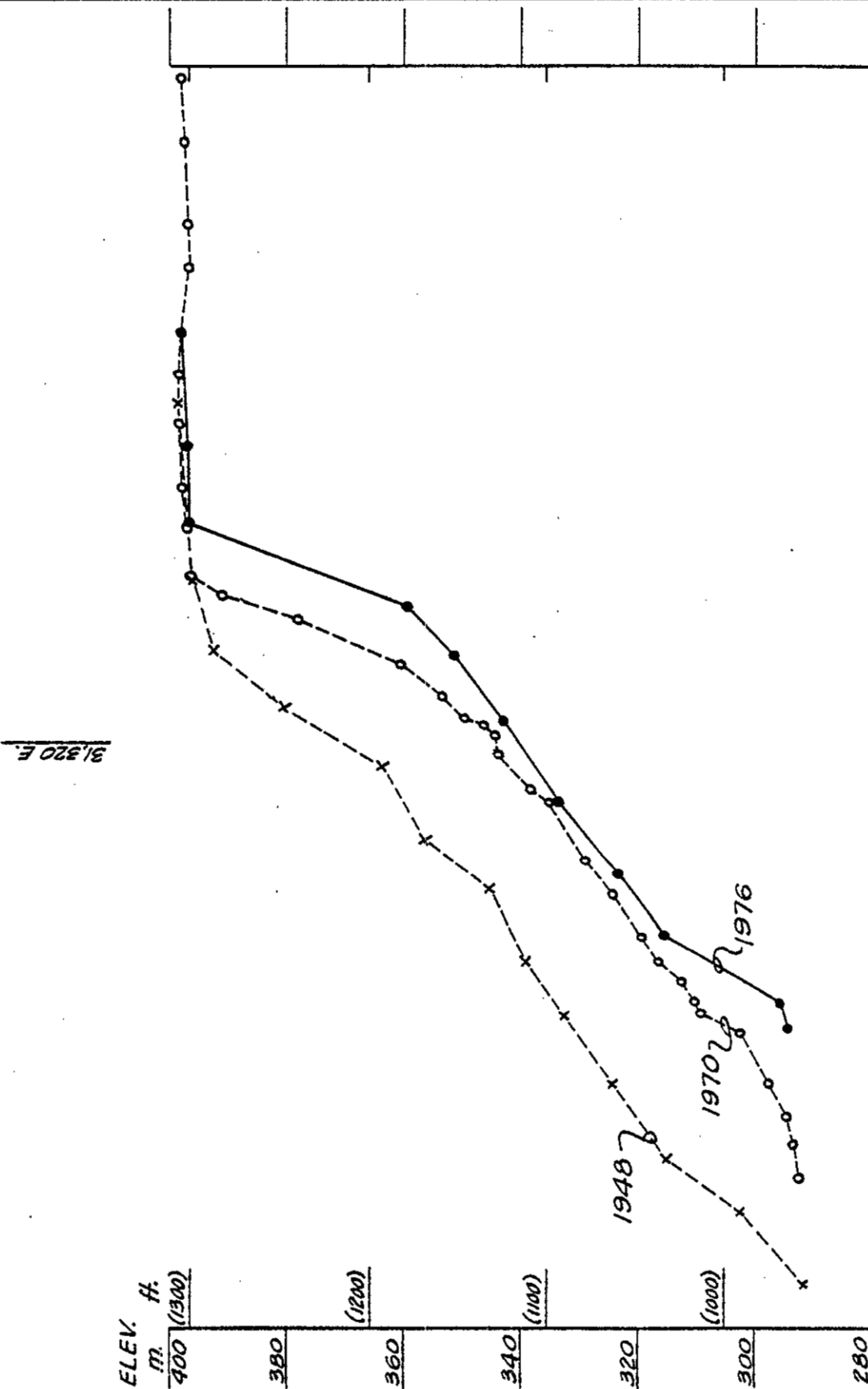
Scale: 1:1000

For location of section, see Drawing No. 2

Project No. V77047. Drawn by [signature] Reviewed M.A.S. Date July 77.

SECTION 2-2

Figure 3



Scale: 1:1000

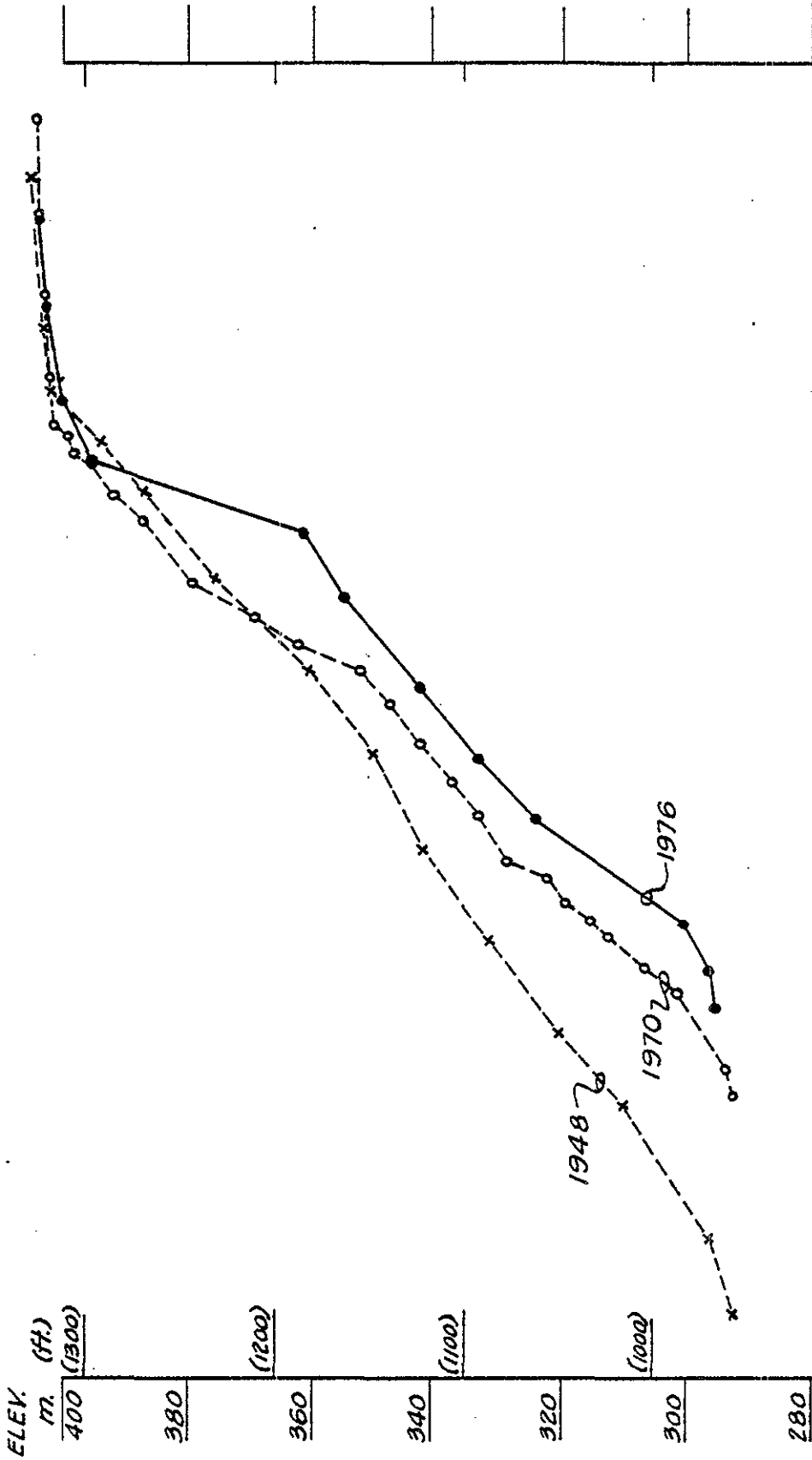
For location of section, see Drawing No. 2

Project No. K77047 Drawn by J. Reviewed MAS Date July '77

SECTION 3-3

Figure 4

31,430 E



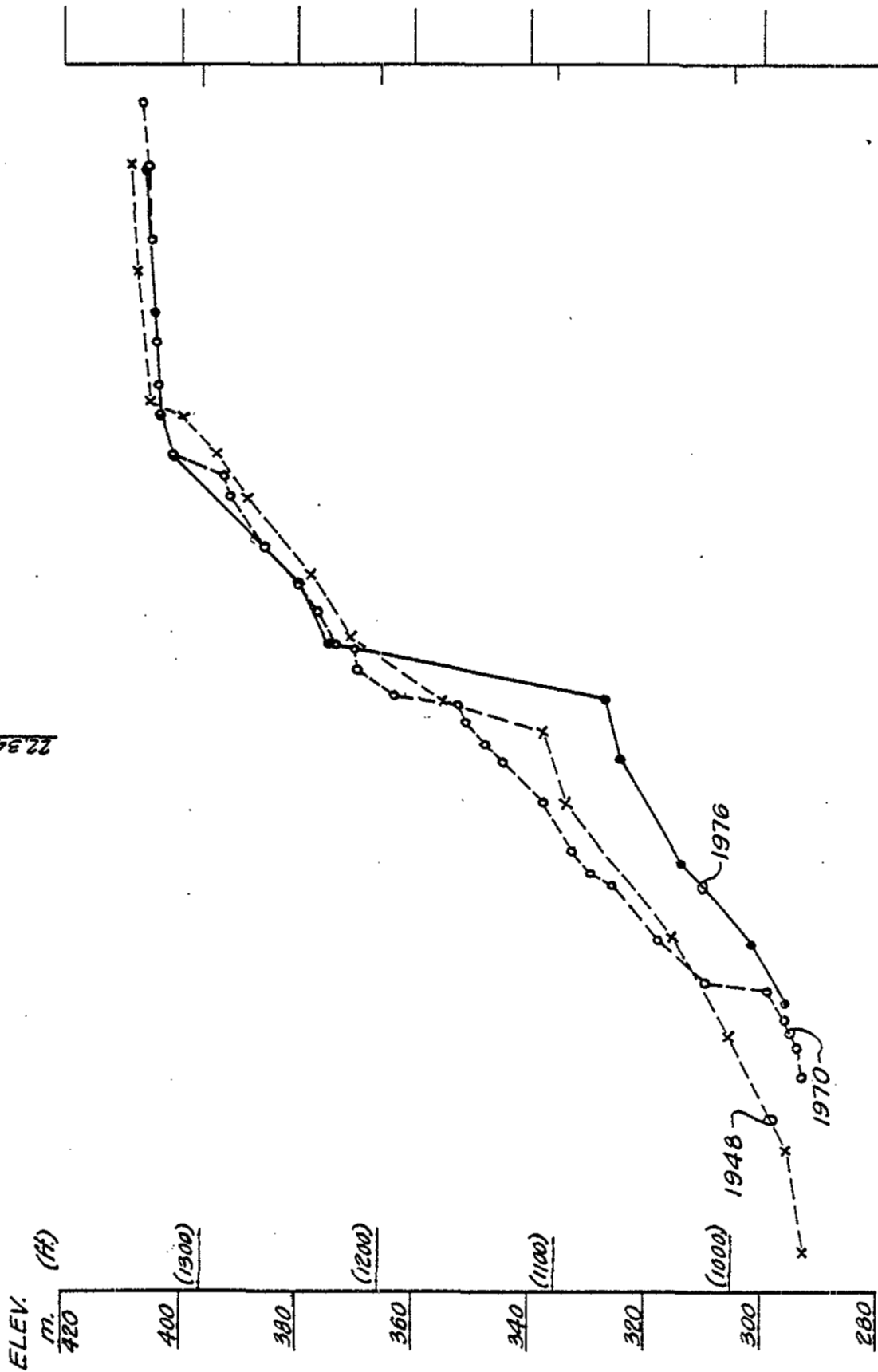
Scale: 1:1000

For location of section, see Drawing No. 2

Project No V77042 Drawn by [signature] Reviewed N.A.S. Date July 77

SECTION 4-4

Figure 5



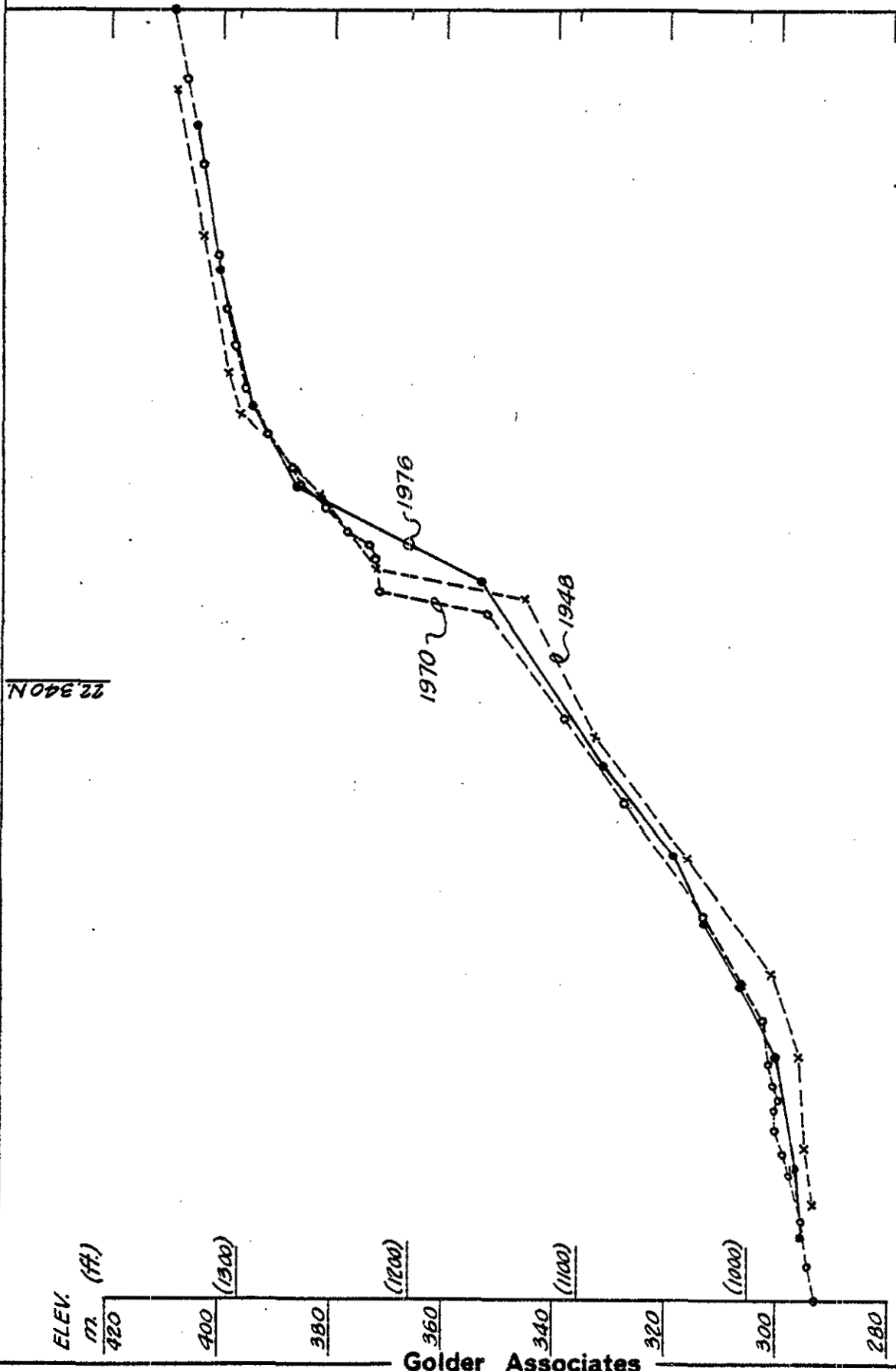
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For location of section, see Drawing No. 2

Project No. VT1047. Drawn by [unclear] Revised N.A.S. Date July '77

SECTION 5-5

Figure 6



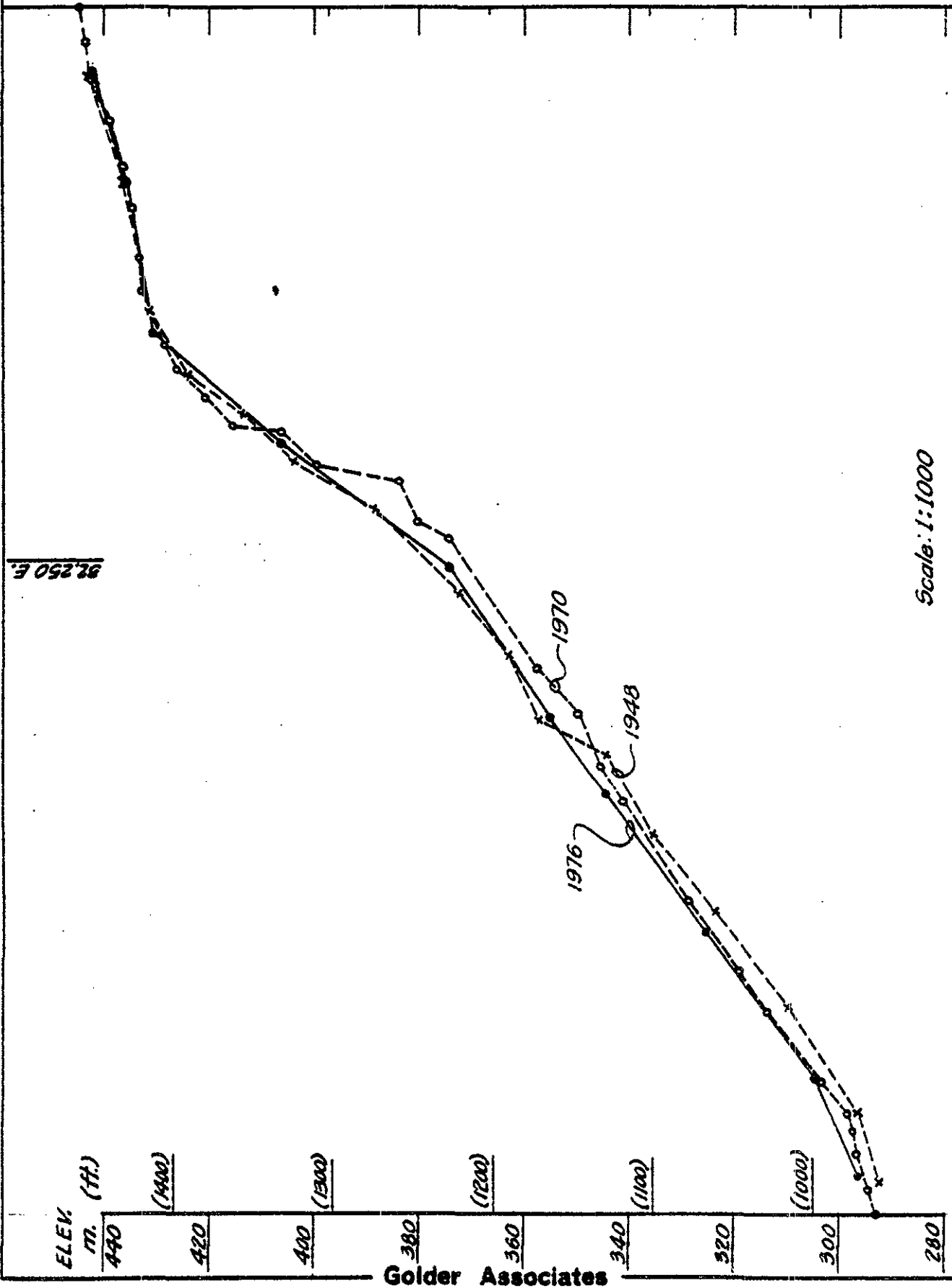
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For location of section, see Drawing No. 2

Project No. K71047 Drawn *d* Reviewed MAS Date July 77

SECTION 6-6

Figure 7



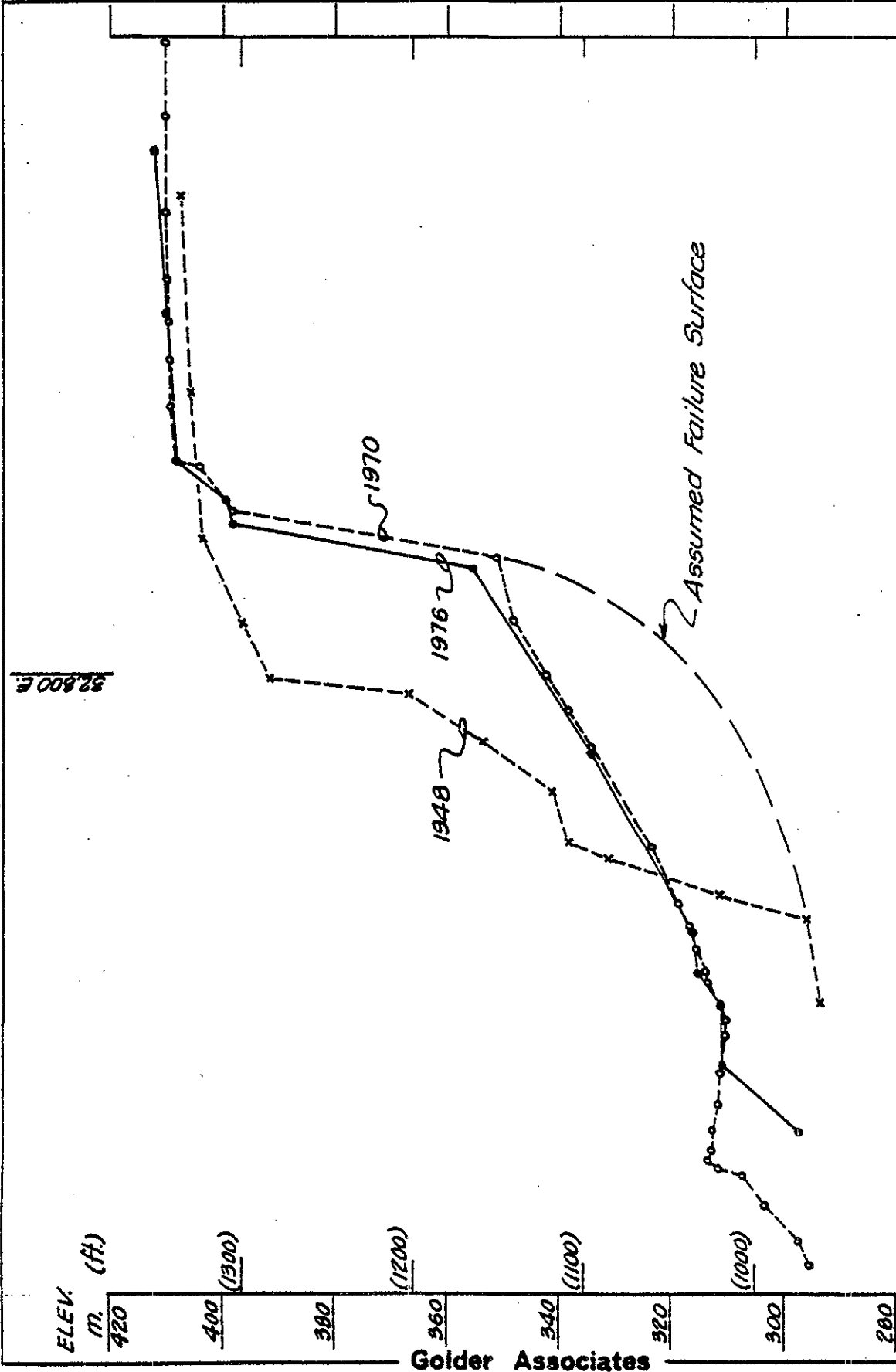
Scale: 1:1000

For location of section, see Drawing No. 2.

V77047 1400 ft. 1976 July 76

SECTION 7-7

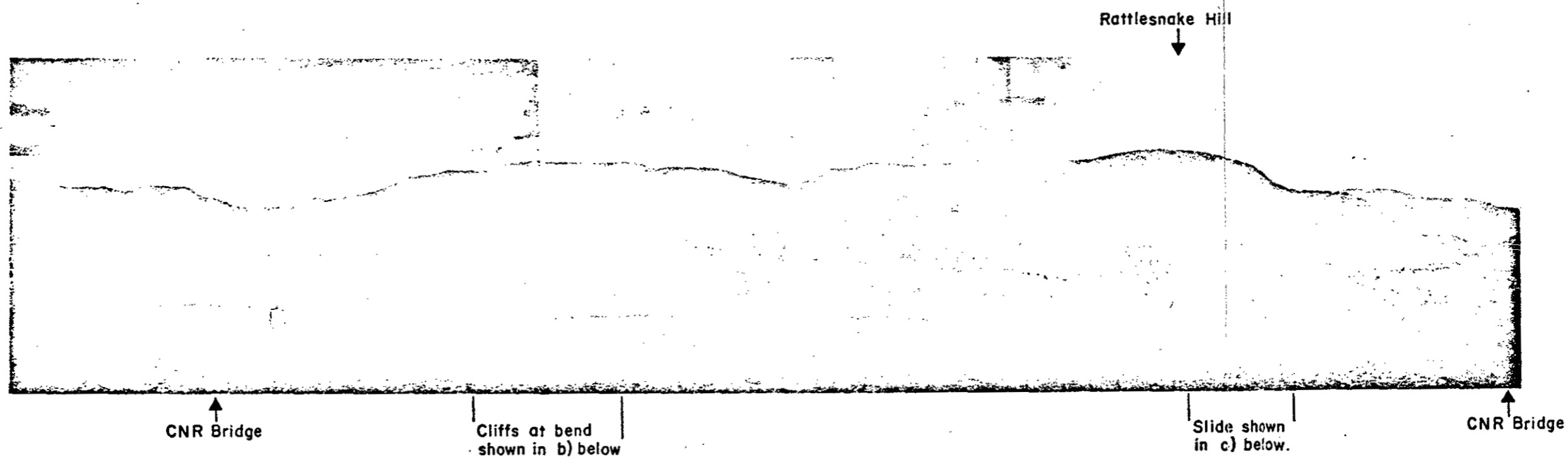
Figure 8



Scale: 1:1000

For location of section, see Drawing No. 2.

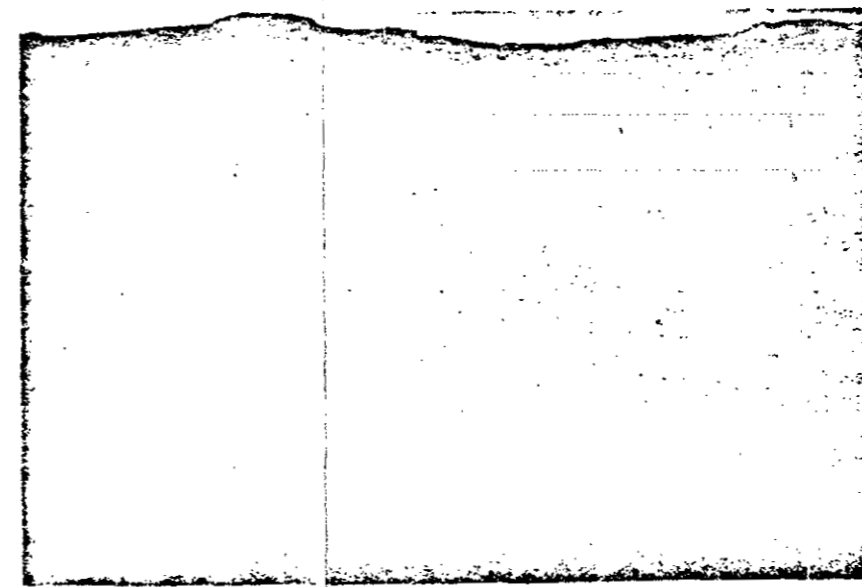
Project No. V77047... Drawn by... Reviewed NAS Date July '77.



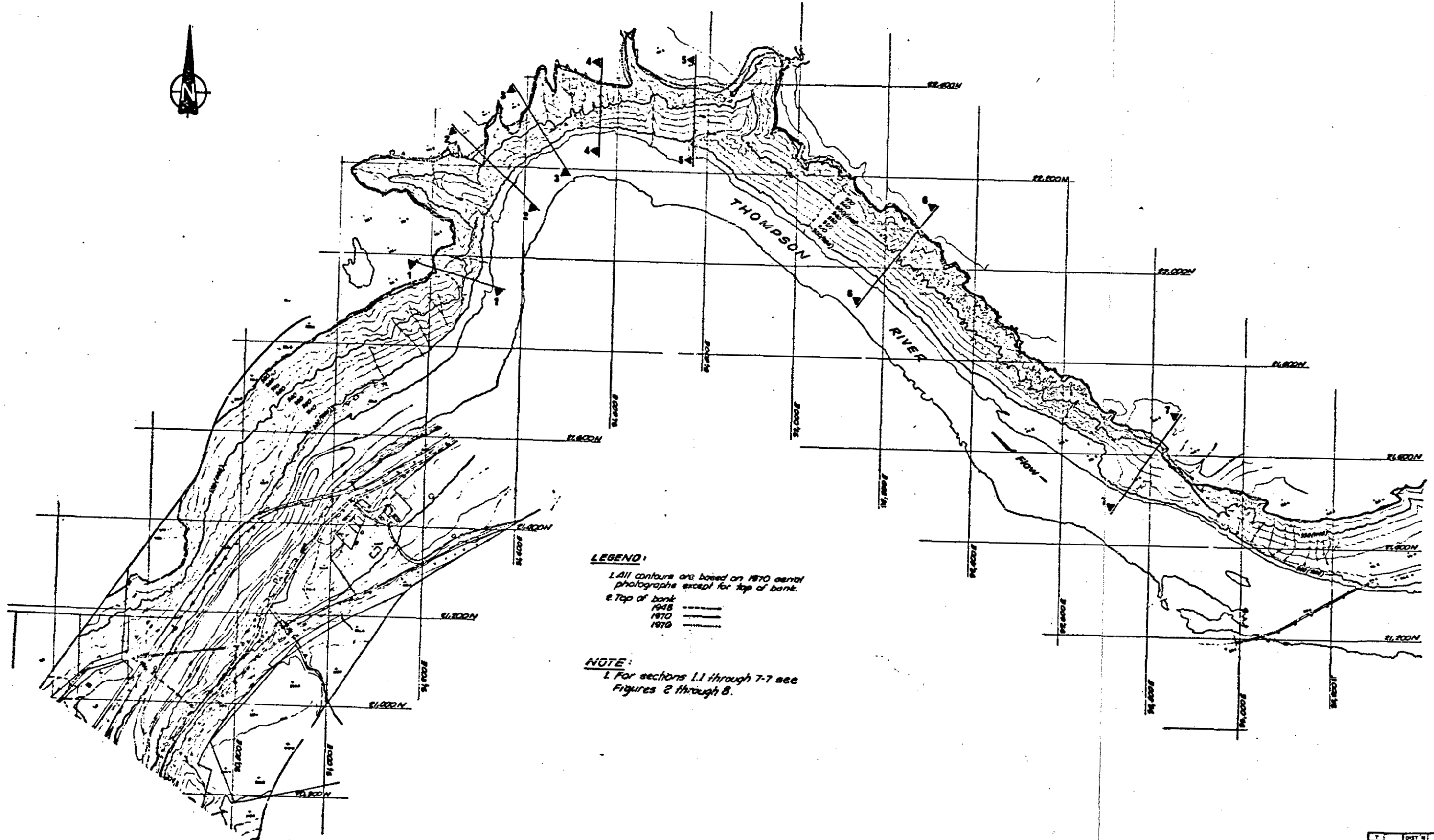
a) Panoramic view of right bank of the Thompson River looking (from left to right) west to north.
 Note CP Railway with a train passing through in the middle distance on the left bank of the river. The valley of the Bonaparte River is on the extreme left in the middle to far distance.



b) Photographs taken from left bank looking across the river to cliffs at the bend between 22,000 N (left) to 31,600 E (right).



c) Photograph taken from left bank looking across the river to the recent slide in the silt bluffs at 21,600 N and 32,800 E.



LEGEND:
 1. All contours are based on 1970 aerial photographs except for top of bank.
 2. Top of bank
 1948
 1970 ———
 1978 ———




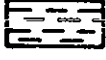

NOTE:
 1. For sections 1.1 through 7-7 see Figures 2 through 8.

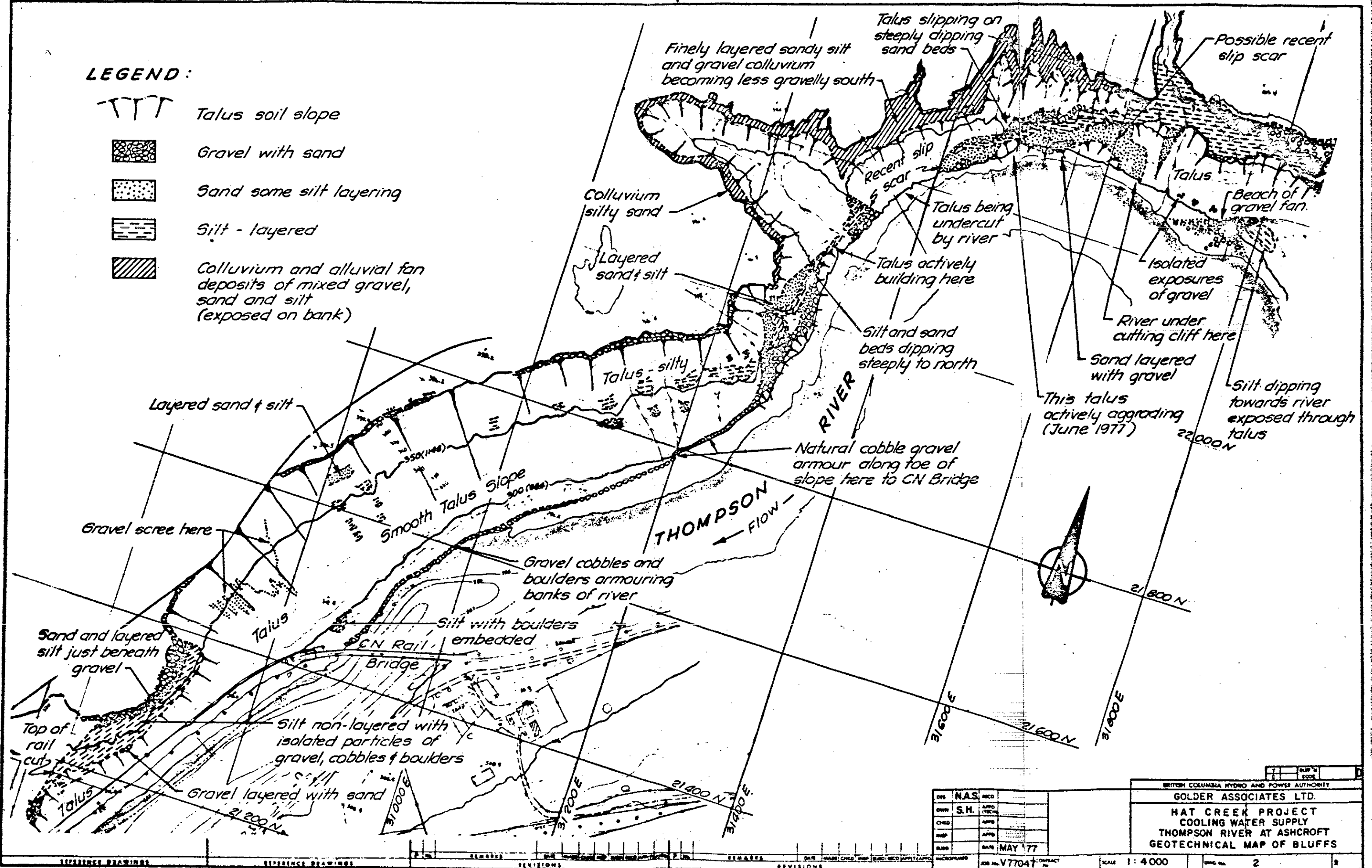
DES	N.A.S.	REC'D
DWN	S.N.	APP'D (TECH)
CHKD		APP'D
IMP		APP'D
SURD		DATE

T		DIST N	
C		CODE	
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY			
GOLDER ASSOCIATES LTD.			
HAT CREEK PROJECT COOLING WATER SUPPLY THOMPSON RIVER AT ASHCROFT TOPOGRAPHY OF BLUFFS			
CONTRACT NO.	SCALE	CWG No.	
V77047	1:4000	I	

REFERENCE DRAWINGS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS

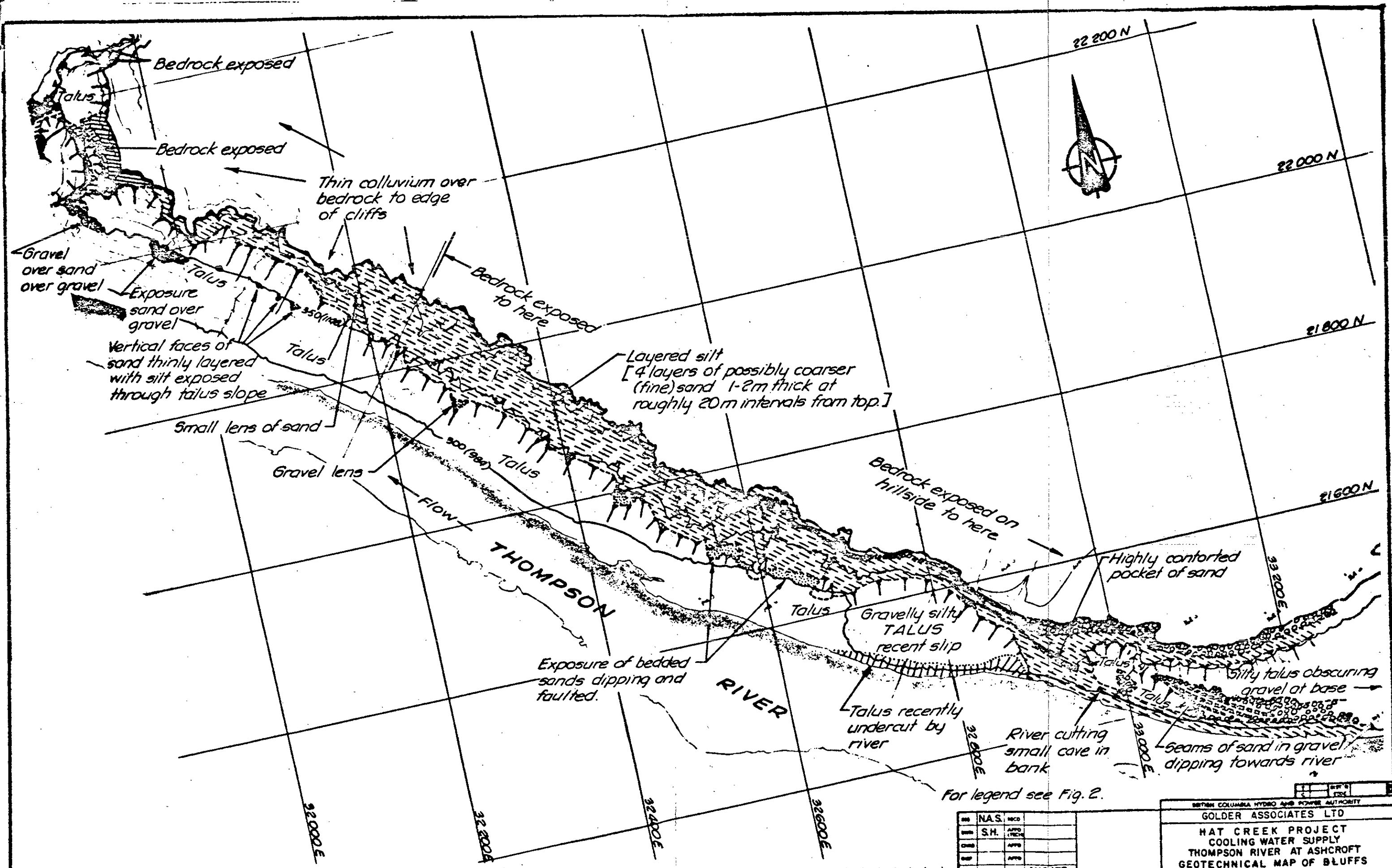
LEGEND:

-  Talus soil slope
-  Gravel with sand
-  Sand some silt layering
-  Silt - layered
-  Colluvium and alluvial fan deposits of mixed gravel, sand and silt (exposed on bank)



DWG	N.A.S.	REC'D	
CHKD	S.H.	APP'D	
DATE		DATE	

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
GOLDER ASSOCIATES LTD.
 HAT CREEK PROJECT
 COOLING WATER SUPPLY
 THOMPSON RIVER AT ASHCROFT
 GEOTECHNICAL MAP OF BLUFFS



For legend see Fig. 2.

DES	N.A.S.	REC'D	
APP	S.H.	APPRO	
CHK		APPRO	
APP		APPRO	
DATE	AUG '77		
JOB NO.	V77047	CONTRACT NO.	

B.C. HYDRO AND POWER AUTHORITY
GOLDER ASSOCIATES LTD
 HAT CREEK PROJECT
 COOLING WATER SUPPLY
 THOMPSON RIVER AT ASHCROFT
 GEOTECHNICAL MAP OF BLUFFS

SCALE: 1:4000
 SHEET NO.: 3

Goldier Associates

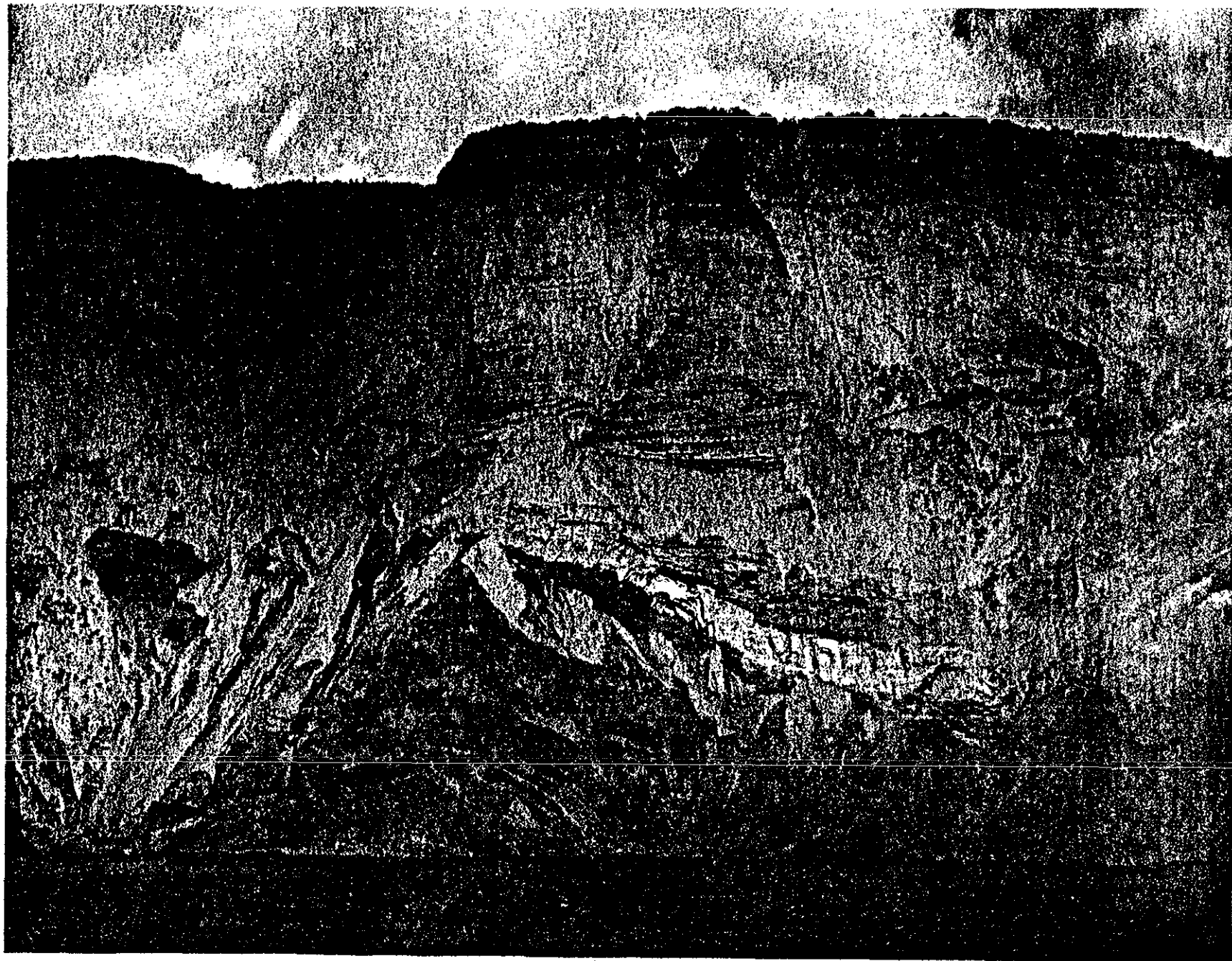
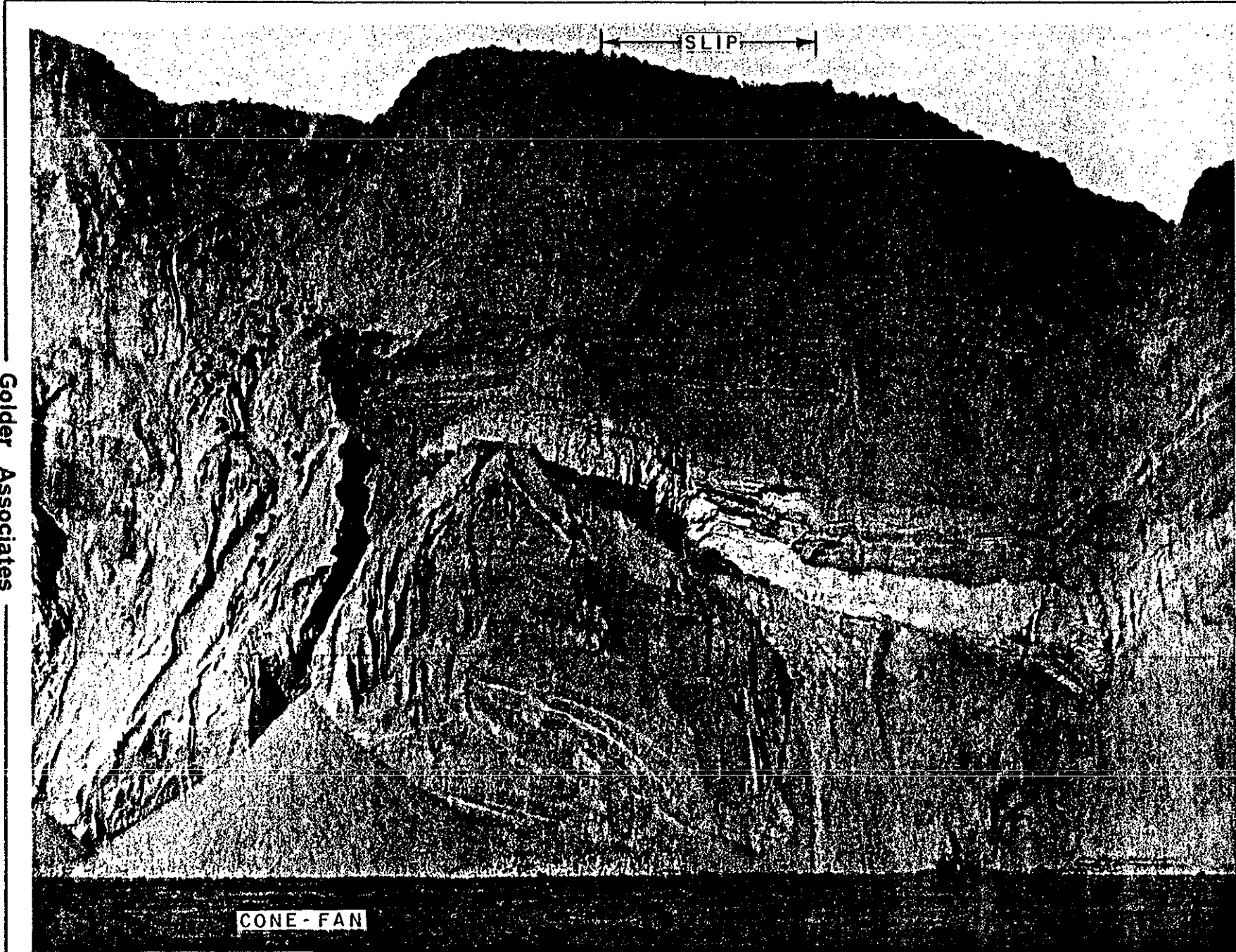


PLATE 2, 22 June 1977



Golder Associates

CONE - FAN

PLATE 3, 15 November 1977



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

E/77/1174
August 17, 1977

Sandwell and Company Limited
Suite 601 - 1550 Alberni Street
Vancouver, B.C.
V6G 1A4

ATTENTION: Mr. Arno Copeland, P. Eng.

Re: Hat Creek Water Supply
Intake Evaluation

Dear Sir:

In response to your letter of August, 3 1977, our geotechnical evaluation of intake sites C to G is given herein.

Geotechnically all five sites, C to G, are acceptable but we would have a preference for sites D and G on the right bank of the river. Sites E and F on the left bank are located beneath fairly steep high bluffs and although protected to some extent by C.P. Rail's track, the history of the rail in this general region has been one of instability. Sites F and G are beneath deep erosion gullies which do not appear to have suffered any recent major collapse, but continuing small scale removal of material can be expected to continue. Site C is rather near to the CN bridge and reworking of the bed of the river can occur due to the influence of the bridge piers in the river.

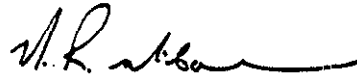
An intake at site C could be affected structurally by possible collapse of the steep silt bluffs above the CN Rail track on the right bank of the river downstream of the bridge. The white silt is deeply incised by water erosion. Inspection of the top of the cliffs showed that the incisions have cut back many meters so that the silt in the cliffs tends to become isolated into individual blocks. These blocks rise very steeply above the CN Rail track and they are in a state of marginal stability. If collapse occurred the debris material would very likely over-run the rail into the river. This might pose a serious threat to the operation of the water intake.

It is most unlikely however, that a collapse large enough to cause blockage of the river would occur, but intensive irrigation of the land at the back of the cliffs could lead to such a collapse. This occurred in the last century downstream of Ashcroft where massive silt slides into the river were attributed to farm irrigation. For this reason we would recommend that control of irrigation should be exercised by B.C. Hydro and Power Authority.

Consideration might also be given to flattening the cliffs. This would offer protection not only to B.C. Hydro's water supply intake but also to disruption of the CN Rail track at the base of the cliffs. Much improvement could be achieved by cutting back the cliffs from elevation 330 to about elevation 370 to a 40 degree slope. This would be necessary over a length of slope of about 150 meters.

Yours very truly

GOLDER BRAWNER & ASSOCIATES LTD.



Per: N.R. McCammon, P. Eng.



N.A. Skermer, P. Eng.

NRM/NAS:rme

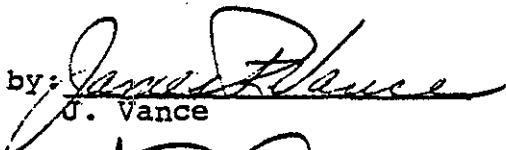
V77047

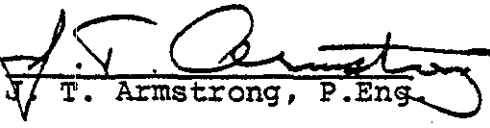
SANDWELL AND COMPANY LTD.

HAT CREEK PROJECT

COOLING WATER SUPPLY

UNDERWATER SURVEY OF INTAKE SITE 10, LOCATION D

Prepared by: 
J. Vance

Approved by: 
J. T. Armstrong, P.Eng.

File: 9214
Date: 1978 02 10

SWAN WOOSTER ENGINEERING CO. LTD.
Consulting Engineers
1525 Robson Street
Vancouver, B.C. V6G 1C5
Canada

SANDWELL AND COMPANY LTD.
HAT CREEK PROJECT
UNDERWATER SURVEY OF INTAKE SITE 10, LOCATION D

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1.0 INTRODUCTION

As requested by Sandwell and Company Ltd., in a meeting with our Mr. G. Zonailo on January 18, 1978, Swan Wooster Engineering Co. Ltd., has completed an underwater survey of the Thompson River at site 10, location D, near Ashcroft, B.C.

The purpose of this survey was to acquire sufficient information to produce a map of bottom features relative to Sandwell's grid layout and to assess the feasibility of driving sheet piles.

Photographs of typical bottom formations were to be taken, and soundings made to confirm bottom contours.

In addition, Swan Wooster was to inspect and photograph the Village of Ashcroft's water intake and report on its condition.

The survey was done between January 31 and February 4, 1978 by Swan Wooster personnel - J. Armstrong, P.Eng., diver; J. Vance, diver; A. Cadenhead, diver; and B. Van Dijk, surveyor. The results of these inspections are described in the following report.

2.0 DESCRIPTION OF SURVEY

Using the point on the rail (N100, E100), a baseline was established at E127 as shown on Drawing No. A1-9214-1.

Three lead filled lines were placed in the river parallel to this baseline running from N190 to N60 at locations E177, E167 and E154. In addition, a floating polyethylene line was placed at location E145. All these lines were marked at 5m intervals and were anchored upstream. The three lead filled lines were used for the underwater inspection in the relatively deeper water, while the fourth line was used for inspecting the bottom from the surface in the shallower water.

Levels were taken from McElhaney BM 1-401, which is located on telephone pole #8 on the CNR right-of-way, and water levels were measured at the river's edge at locations N60, N125 and N190.

An echo sounder was utilized to get depth readings of the river bottom, and contours were produced using these soundings, see Drawing No. A1-9214-1.

3.0 OBSERVATIONS

3.1 Site 10, Location D

Prior to any diving being done, the river current was roughly measured along lines E167, E177, and E192 starting at N190. The method used was to measure the time required for a group of floating objects to travel 90 m downstream. Times measured were 70 seconds, 65 seconds, and 85 seconds indicating different water velocities at different locations. These times averaged to yield an approximate river current at surface level of 1.25 m/sec. The weather on the days the diving was done was sunny with some cloudy periods. The air temperature was about -12°C and the water temperature about $+0.7^{\circ}\text{C}$. There was very little wind.

Observations from the underwater inspection were as follows:

Line 1: N190 E177 to N60 E177 (Dwg.A1-9214-2)

Visibility was good, in the range of 6 to 8 m. The divers initially crossed the river from south to north upstream from the survey area. The bottom was uniform coarse gravel and cobbles with most material being in the range from coarse sand to 0.3 m diameter. The first 60 m from N190 to N130 showed a similar gravel bottom with some medium sized rounded boulders of about 0.3 m in diameter (photograph 3). There were a few larger boulders in the 1 m diameter range randomly located in this area.

Line 1: N190 E177 to N60 E177 (cont'd)

From N130 to N100, the bottom remains gravelly with the number and size of boulders increasing, some being about 1.3 m diameter.

From N90 to N60, smooth black bedrock was encountered (photograph 4). Outcrops were widely spaced at N90 and increased to cover three-quarters of the river bottom at N60.

Line 2: N190 E167 to N60 E167 (Dwg. A1-9214-2)

For the first 50 m, the bottom is sand and gravel with some 0.3 m boulders. At approximately N150 there is a 1.5 m x 3 m boulder with a slightly smaller one beside it (photograph 5).

At N140 there are several 0.7 m x 1.1 m boulders. From N140 to N115 there is just the odd 1 m diameter boulder with the predominance being small 0.3 m to 0.5 m boulders.

At N110 there is a small section of bare rock outcrop. From N100 to N60 again bare rock outcrops are evident (photograph 6). There are several sharp ridges in this area from 1 m to 1.7 m high (photograph 7).

Line 3: N190 E154 to N60 E154 (Dwg. A1-9214-2)

For the first 40 m a gravelly bottom with some small boulders to 0.6 m in diameter was again encountered.

At N150 there is a very large boulder about 3.5m in diameter (photograph 8). About 60mm of the boulder protruded above the water surface. Beside this boulder are a couple of smaller 2 m diameter boulders.

Line 3: N190 E154 to N60 E154 (cont'd)

From N150 to N100 the bottom is covered mostly in smaller boulders up to 0.5 m in diameter, with some up to 1 m to 1.2 m in diameter (photographs 9&10).

From N100 to N60, till was encountered generally on the shoreward side of the line (photographs 12, 13, 14 and 15). This till was quite hard insitu, but became crumbly when brought to the surface. The till consists of mainly 6 inch minus gravel set in a matrix of hard sandy clay. Bedrock outcrops were observed on the river side of the line, downstream from N90. (photograph 11).

Line 4: N190 E145 to N60 E145 (Dwg. A1-9214-2)

The inspection was done on the surface as a snorkel dive since the water was quite shallow. At N190 there is a large 2.5 m diameter boulder.

From N190 to N145, the bottom was mostly gravel and sand with some 1 m diameter boulders evident.

At N140 there is a 3 m diameter boulder and at N130 there is a 2.5 m x 1.2 m boulder and three 1.2 m diameter boulders.

From N120 to N80, the bottom is composed mostly of boulders with maximum size being about 1 m to 1.2 m. This section was similar to the rocky sections above water beside the river. From N70 on, till was evident.

3.2 Ashcroft Water Intake

On Friday, February 3rd, the water intake for the Village of Ashcroft was inspected by divers J. Vance and A. Cadenhead. The summary of the results are noted on Drawing No. B-9214-3. Photograph 16 shows the 50mm screen on the end of the intake.

The intake pipe is located in a large eddy downstream of the southeast highway bridge pier. Current velocity was very low at the intake as a result.

4.0 FEASIBILITY OF DRIVING SHEET PILES

Alternate methods of utilizing sheet piles for the construction of the cooling water intake are being considered by Sandwell as shown on Sandwell Drawing D-4191-29.

Alternative 1 consists of a cellular sheet pile cofferdam parallel to shore, connected back to shore by fill dams at each end.

Alternative 2 consists of a sheet pile caisson immediately surrounding the intake structure.

Our comments on pile driving for these schemes are as follows:

4.1 Alternative 1

Driving of sheet piles into the river bottom will be difficult or impossible because of the occurrence of boulders in the overburden. At the downstream end of the site, bedrock outcrops of irregular profile will be encountered, as well as layers of till. We anticipate considerable difficulty in driving piles through the till. Because of these factors design of the cellular cofferdam should allow for only minimal pile penetration.

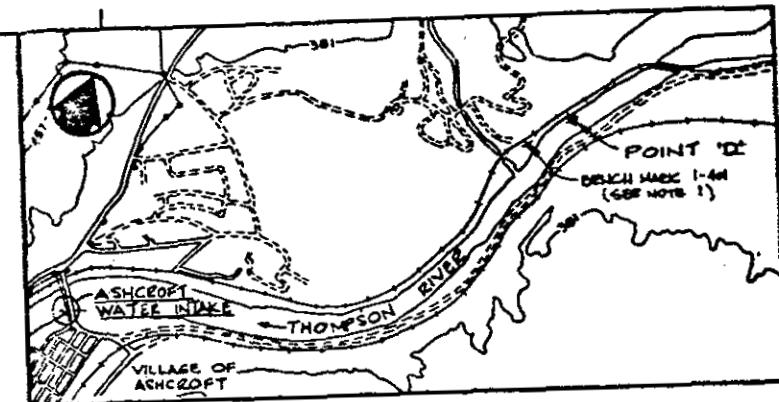
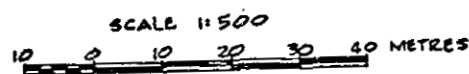
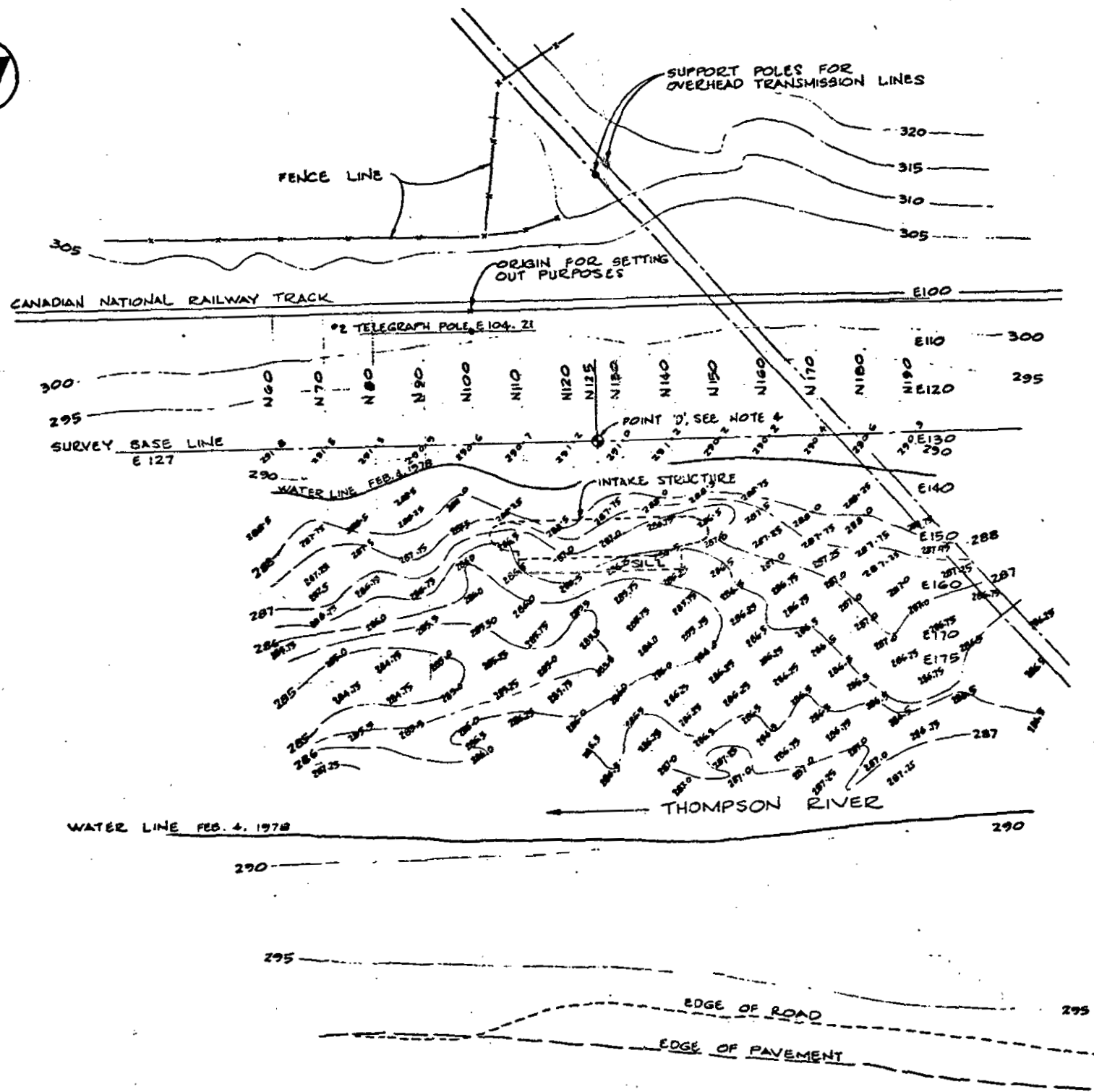
4.2 Alternative 2

If this scheme requires penetration of sheet piling to bedrock or till, the boulders in the overburden will have to be removed prior to pile driving.

APPENDIX A

DRAWINGS

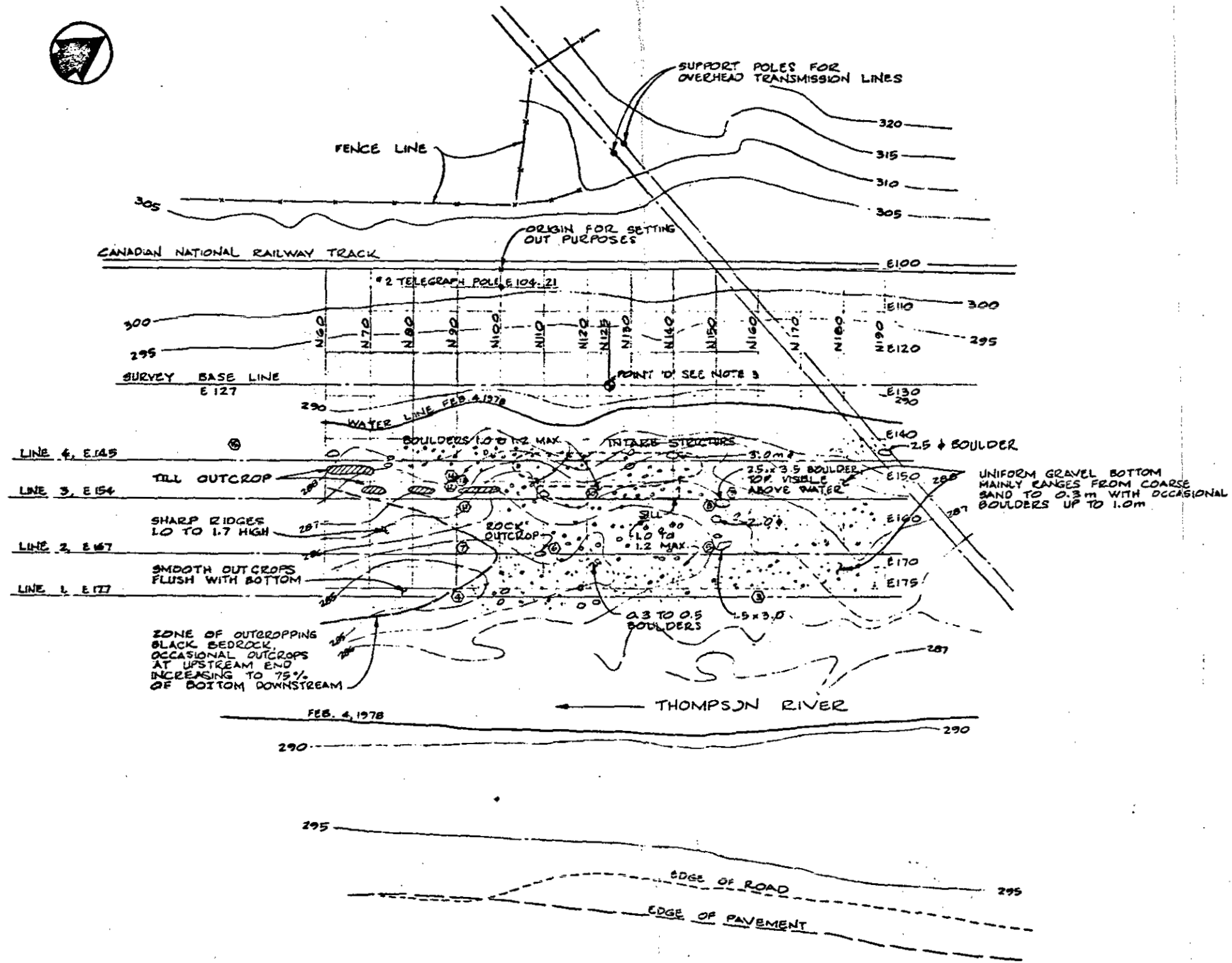
- A1-9214-1 - SOUNDINGS & CONTOUR PLAN
- A1-9214-2 - RIVER BOTTOM DESCRIPTION
- A1-9214-3 - ASHCROFT WATER INTAKE
- D-4191-29 - ALTERNATIVE COFFERDAMS



KEY PLAN
0 0.5 1 KM
SCALE: 1:16,000

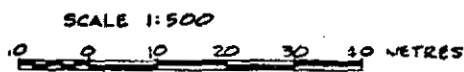
- NOTES:
1. CONTOUR LEVELS REFERENCED TO MCELHANNNEY BENCH MARK 1-401, ELEV. 301.88
 2. CONTOURS ABOVE WATER LINE ARE FROM SANDWELL DWG. D4191-7
 3. SOUNDINGS WERE PERFORMED BY SWAN WOOSTER ENGINEERING ON FEB. 4, 1978
 4. WATER LEVEL AT POINT 'O' ON FEB. 4, 1978 WAS 289.53

SWAN WOOSTER ENGINEERING CO. LTD.		SANDWELL & CO. LTD.	
VANCOUVER B.C. 67 GARDEN DRIVE		HAT CREEK PROJECT	
PORTLAND ORE. 1000 1/2 ST. ATLANTA, GA.		COOLING WATER INTAKE SITE 100	
		SOUNDINGS & CONTOUR PLAN	
		A1-9214-1	
		REV. I	



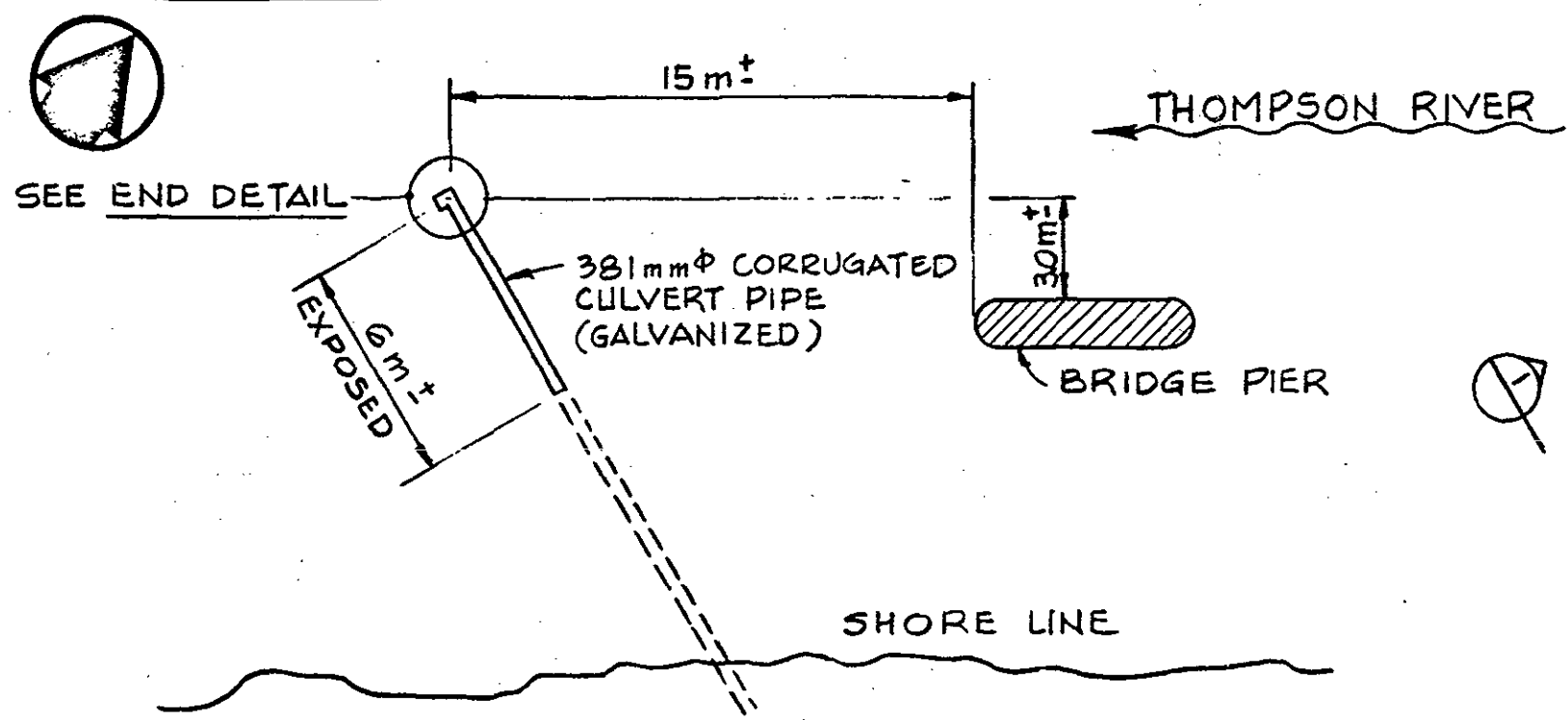
- NOTES:**
1. CONTOUR LEVELS REFERENCED TO McELHANNAY BENCH MARK 1-401, ELEV. = 301.88
 2. CONTOURS ABOVE WATER LINE ARE FROM SANDWELL DWG. D4191-7.
 3. WATER LEVEL AT POINT 'D' ON FEB. 4, 1978 WAS 289.53

- LEGEND:**
- ⊙ UNDERWATER PHOTOGRAPH NUMBER (PHOTOS 1 & 2 TAKEN ABOVE WATER)
 - ▨ TILL (GRAVEL, HARD SANDY CLAY)
 - ⊙-20 GRAVEL: COARSE SAND TO 0.3 m WITH SOME BOULDERS UP TO 1.0 m
 - ⊙-20 INDIVIDUAL LARGE BOULDER. (LOCATION 25)

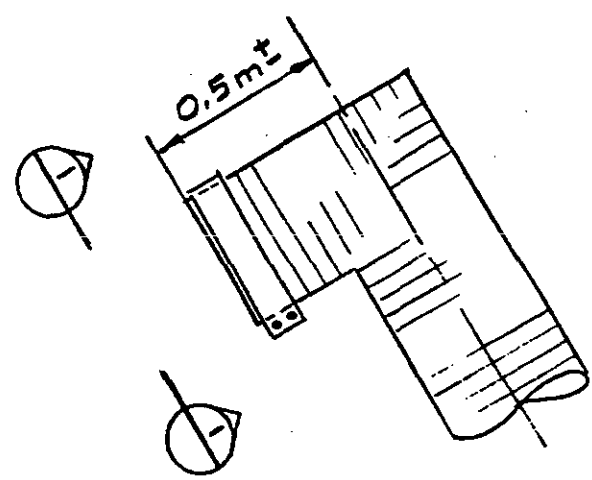


UNDERWATER CONTOUR INTERVAL 0.5 METRES

SANDWELL & CO. LTD.	
HAT CREEK PROJECT	
COOLING WATER INTAKE SITE 100	
RIVER BOTTOM DESCRIPTION	
SWAN WOOSTER ENGINEERING CO. LTD.	A1-9214-2
REVISIONS	REV.

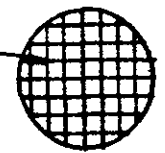


PLAN SHOWING APPROXIMATE INTAKE LOCATION
SCALE 1:200

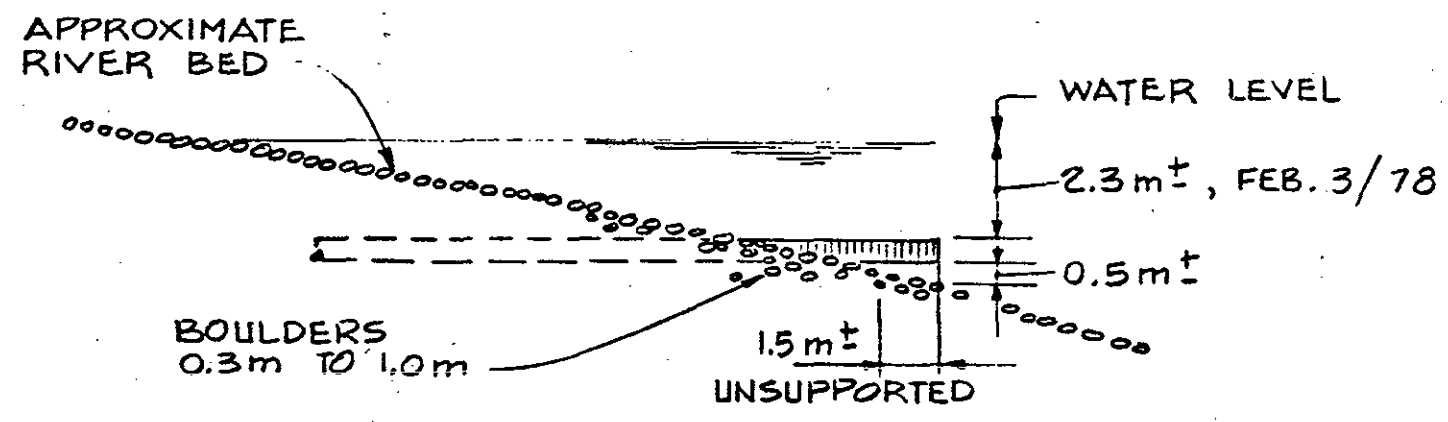


END DETAIL
SCALE 1:20

φ12mm STEEL BARS FORMING 50mm SQUARE OPENINGS.



ELEVATION 1
SCALE 1:20



SECTION LOOKING DOWNSTREAM
SCALE 1:200

NOTES:

1. RIVER BOTTOM IS COMPOSED MOSTLY OF ROUNDED BOULDERS 0.3m TO 1.0m IN DIAMETER.
2. THE INTAKE IS LOCATED IN A BACK EDDY DOWNSTREAM OF THE BRIDGE PIER, CONSEQUENTLY THERE WAS NO NOTICEABLE CURRENT IN THIS AREA.
3. GALVANIZED PIPE IS IN GOOD CONDITION. GRATING AT END IS RUSTED BUT NOT SEVERELY.

No.	By	Date	Revision	App'd	No.	By	Date	Revision	App'd

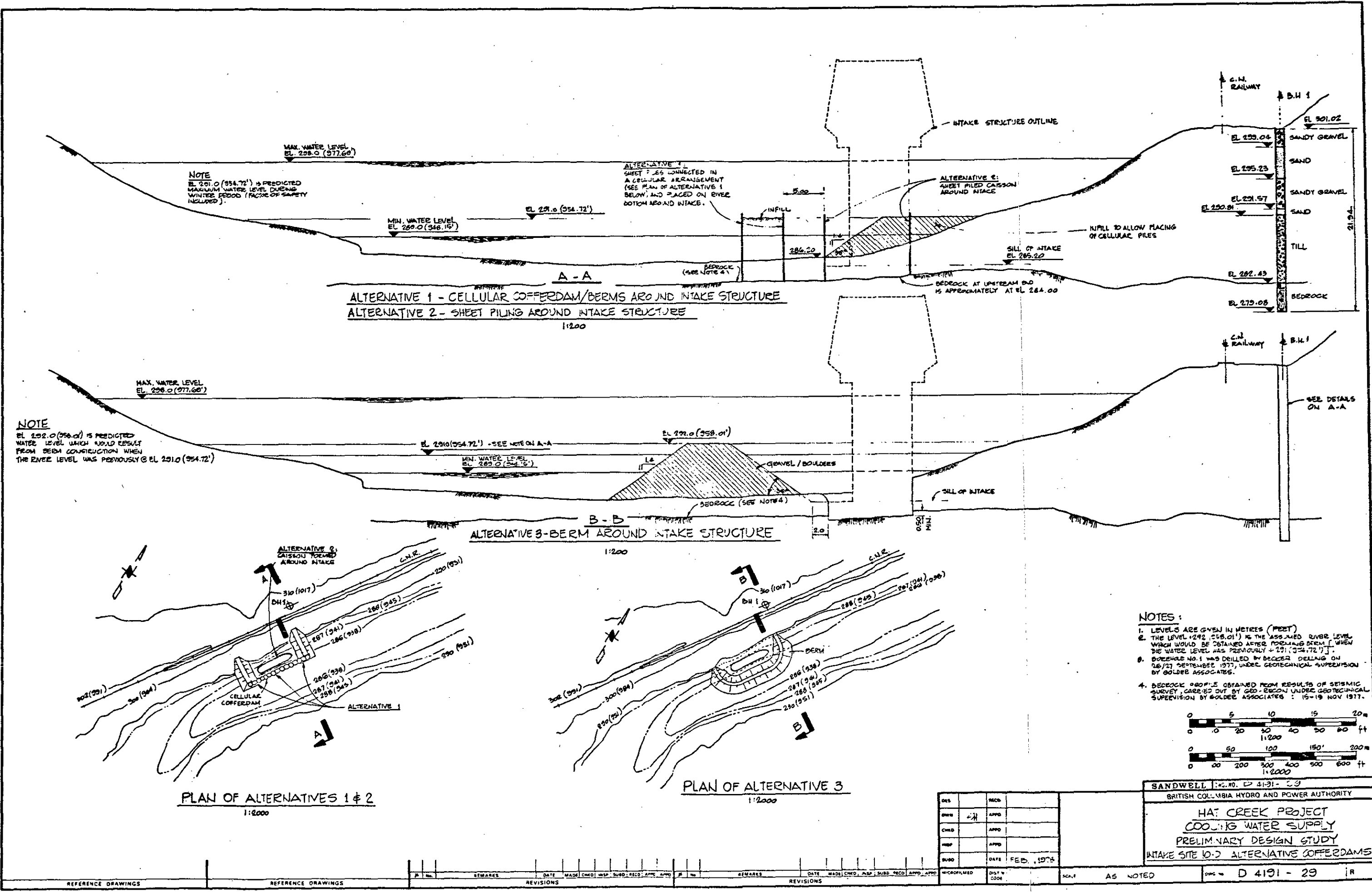
APPROVED	DATE	SCALE	AS SHOWN	DAY	MO	YR
APPROVED	DATE	DESIGN				
		DWG. CKD.				
		DSN. CKD.				
		APP'D.				

SWAN WOOSTER ENGINEERING CO. LTD.
 VANCOUVER, B.C. ST. CATHARINES, ONT. MONTREAL, QUE.
 PORTLAND, ORE. ATLANTA, GA. JAKARTA

SANDWELL & CO. LTD.

VILLAGE OF ASHCROFT
THOMPSON RIVER
WATER INTAKE

B-9214-3 REV



DES	REC'D	DATE
OWN	APP'D	
CH'D	APP'D	
IMP	APP'D	
SUBD	APP'D	
MICROFILMED	DIST. CODE	SCALE

SANDWELL FIG. NO. D 4191-29
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAI CREEK PROJECT
COOLING WATER SUPPLY
PRELIMINARY DESIGN STUDY
INTAKE SITE 10-2 ALTERNATIVE COFFERDAMS

REFERENCE DRAWINGS	REFERENCE DRAWINGS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS	REVISIONS
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APPENDIX B

PHOTOGRAPHS



PHOTO. 1 **VIEW OF SOUTHEAST BANK**
NOTE: SIZE OF BOULDERS ALONG BANK



PHOTO. 2 **DOWNSTREAM VIEW SHOWING RIVER**
NARROWING AT RAPIDS

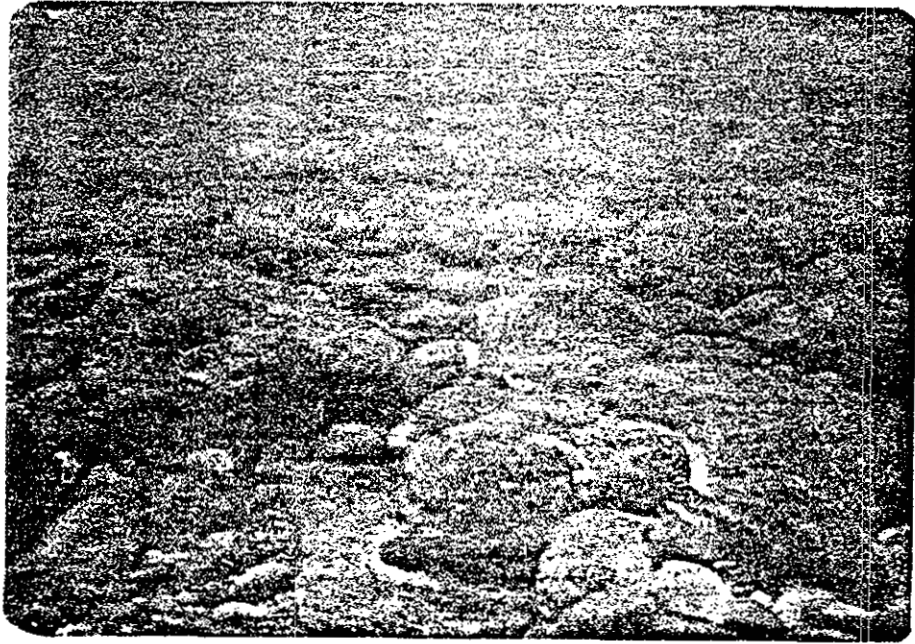


PHOTO. 3 TYPICAL GRAVEL BOTTOM MATERIALS

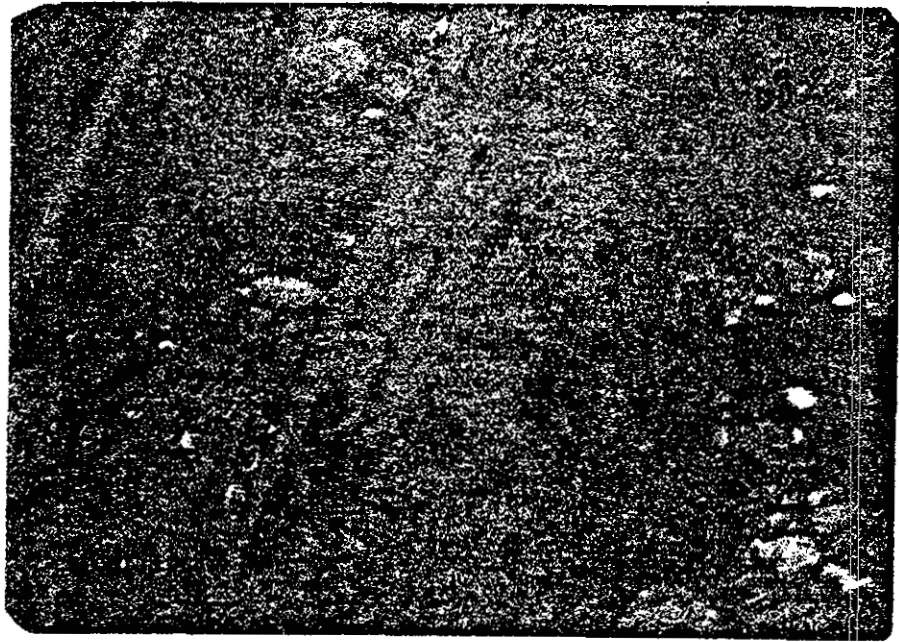


PHOTO. 4 BLACK BEDROCK - FLUSH WITH BOTTOM

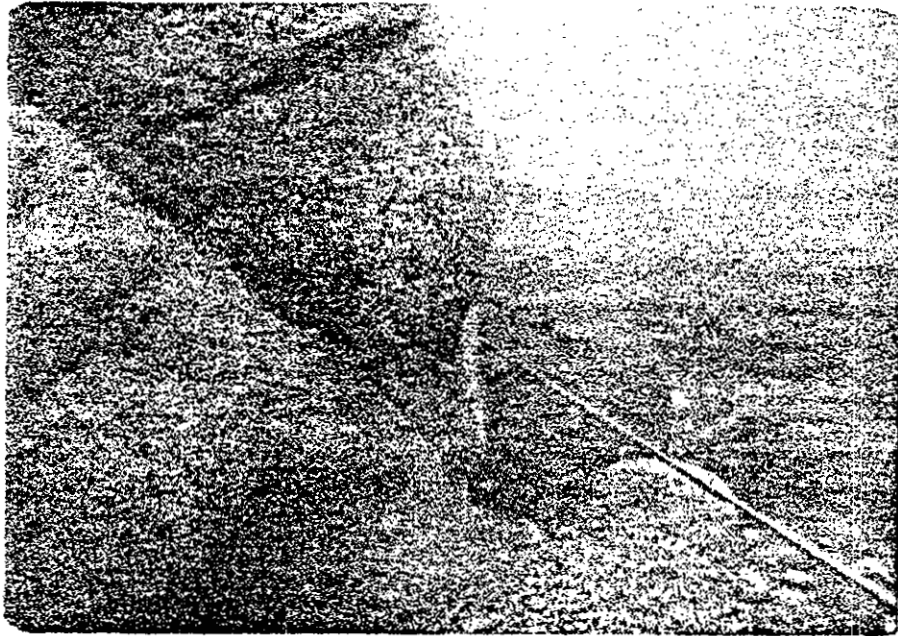


PHOTO. 5

LARGE BOULDER
NOTE: GRAVEL BOTTOM IN
SURROUNDING AREA



PHOTO. 6

BEDROCK -
NOTE: IRREGULAR SURFACE

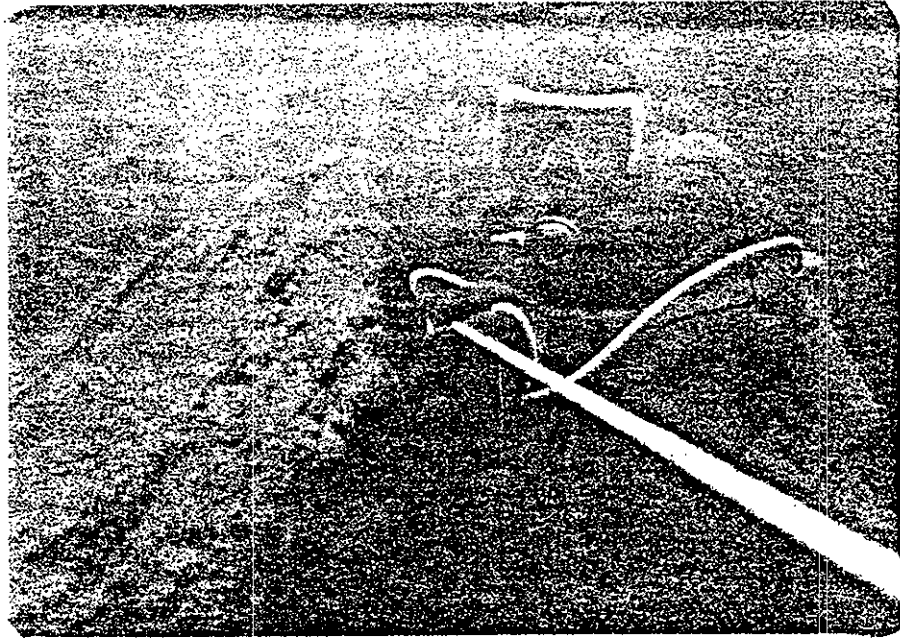


PHOTO. 7

BEDROCK OUTCROP

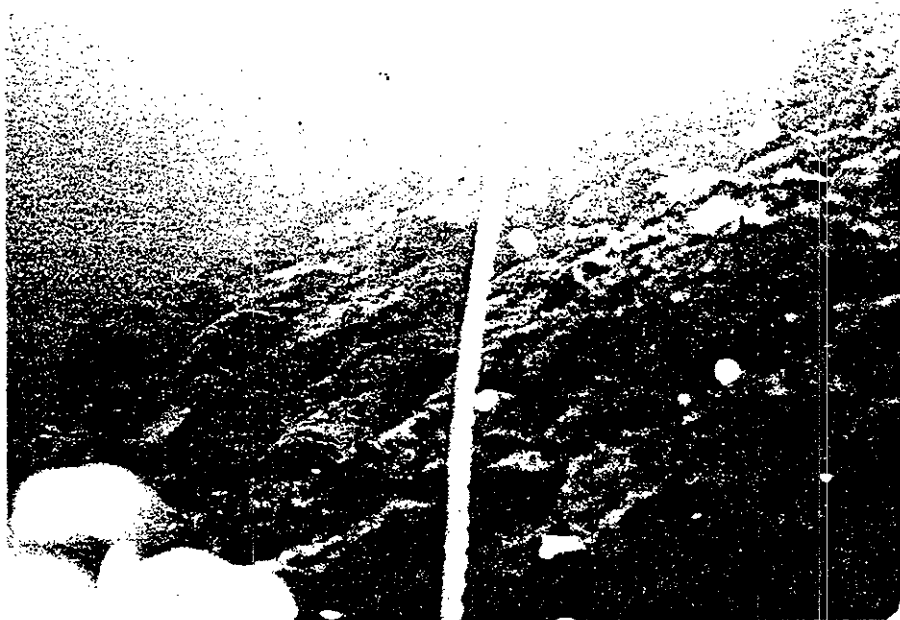


PHOTO. 8

LARGE BOULDER -
PHOTO TAKEN LOOKING UP



PHOTO.9

GRAVEL BOTTOM -
NOTE: PRESENCE OF SMALL BOULDERS



PHOTO.10

BOULDERS -
NOTE VARYING SIZES

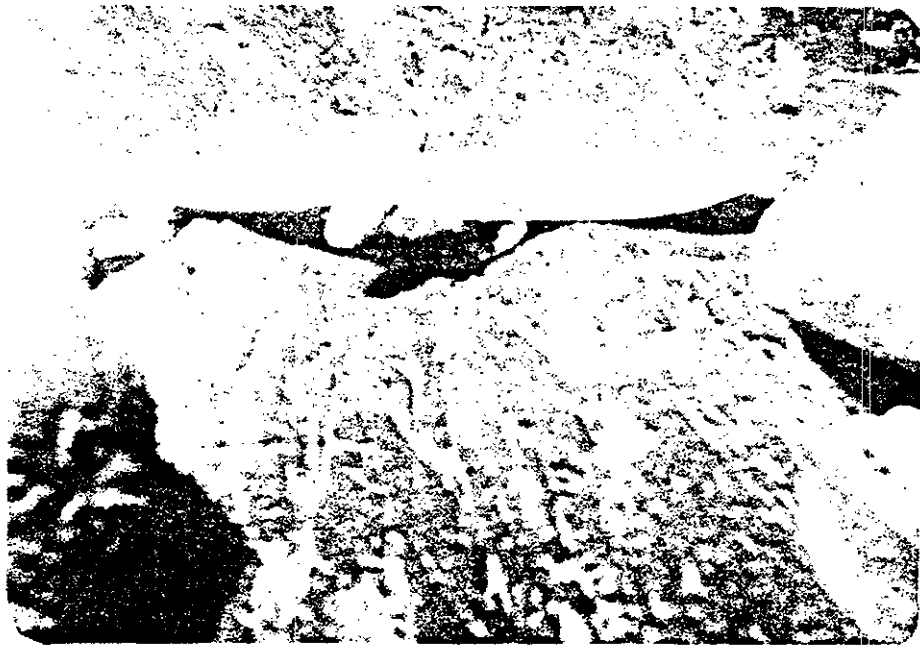


PHOTO.11

BEDROCK

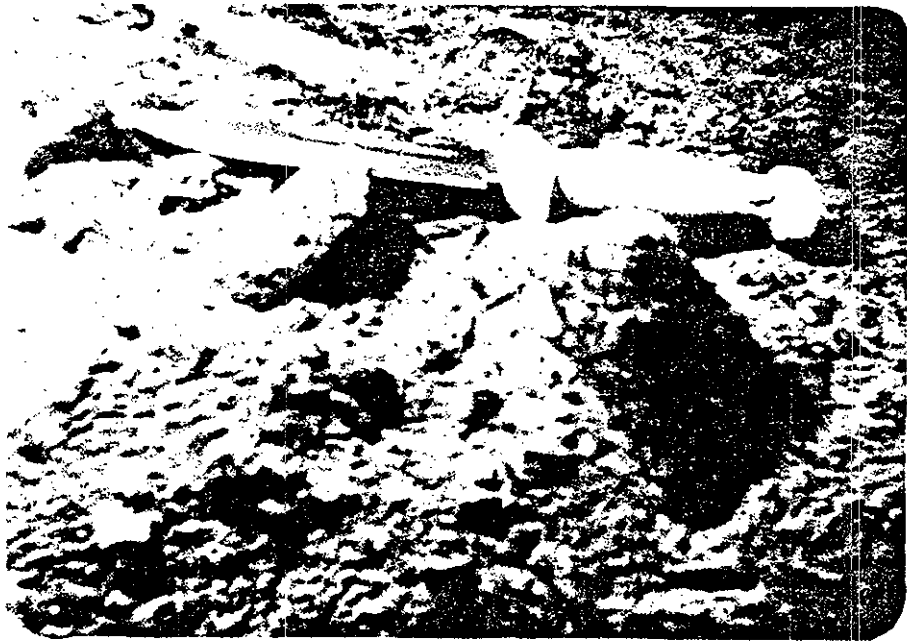


PHOTO.12

TILL

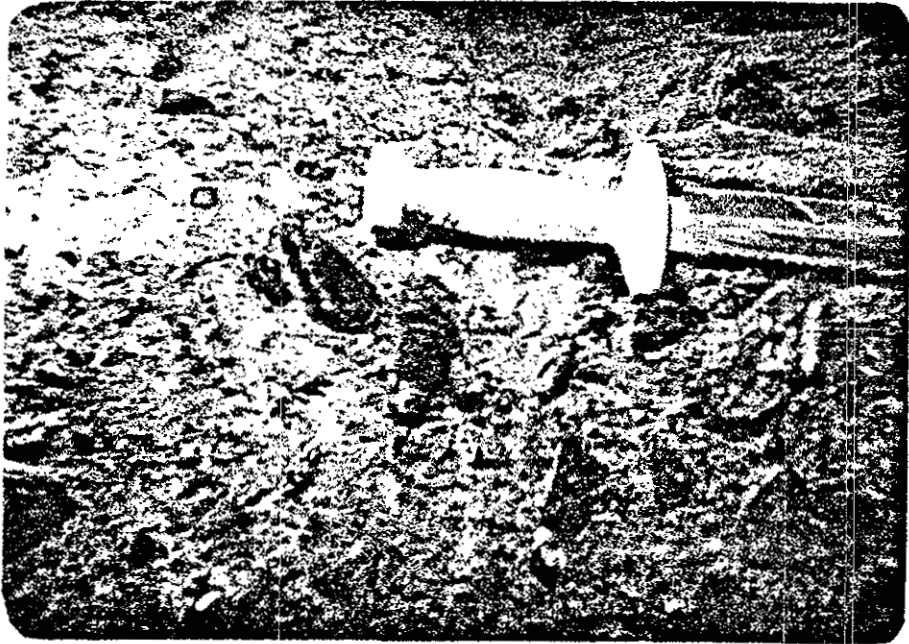


PHOTO. 13

TILL



PHOTO. 14

TILL

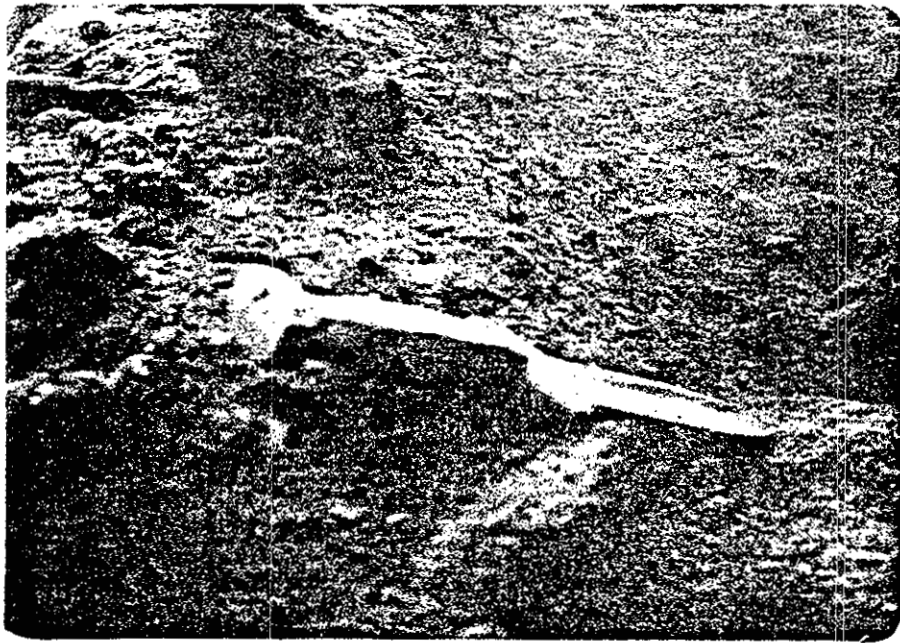


PHOTO. 15

TILL



PHOTO. 16

ASHCROFT WATER INTAKE --
NOTE SIZE OF BOULDERS IN
IMMEDIATE AREA

FISH PROTECTION ASPECTS
OF THE
SANDWELL INTAKE DESIGN
FOR THE
HAT CREEK PROJECT

prepared for

Sandwell and Company Limited
1550 Alberni Street
Vancouver, B.C., Canada V6G 1A5

by

E. L. Mulvihill and K. P. Campbell

Beak Consultants Limited
Suite 602 - 1550 Alberni Street
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BEAK Project No. K4318

February, 1978

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INTRODUCTION

BEAK was retained by Sandwell to evaluate the design of the makeup cooling water intake for the Hat Creek project relative to protection of the fishery resources of the Thompson River. Appendix B contains Sandwell drawings V4007/46, D4191-5, 6 and SK63 which indicate major features of the presently envisioned intake design and areas of potential intake location.

Consultation was supplied to Sandwell by BEAK and interested regulatory agencies during the development of the preliminary design. This report represents an evaluation of the structure as presently envisioned. The terms of reference for design were that the structure could operate at 100% capacity during all anticipated conditions of river flow and fish distribution and that the design would assure operation without significant impact upon the fishery resource of the Thompson River. The design was to assure fish guidance before impingement, not fish handling after impingement.

SECTION I. PRESENT ENVIRONMENTAL SETTING - FISHERIES

A. Key Fish Species

The Thompson River fish fauna is known to consist of 13 species including anadromous steelhead trout, and chinook, coho, pink and sockeye salmon. The resident fauna includes rough fish such as northern squawfish and largescale sucker as well as more desirable game fish such as rainbow trout and brook trout. The Bonaparte River is known to contain an additional five species. A list of species occurring in the Thompson River and Bonaparte River in the area of the intake is given in Table 1 (Tables are found in Appendix A).

Certain of the species in Table 1 may be more susceptible than others to injury at a water intake and some have higher commercial and recreational

importance. Based on these factors certain species can be used as key species for the evaluation of an intake design. In general, we may say that anadromous fish are more vulnerable than resident fish. While resident fish may occupy territories or home ranges in the vicinity of an intake, only a small portion of a resident fish population can conceivably be exposed to such a structure. However, large fractions of the anadromous fish populations can pass through a given river section on upstream and downstream migrations. Thus, the anadromous salmon and steelhead trout will be considered key species for the evaluation of the fish protection features of this intake.

B. Life History of Key Fish Species

Among the anadromous salmonids, certain life history aspects and ecological interactions may be identified which are pertinent to intake location and design. For example, primary spawning sites should be avoided and an intake should not be constructed in areas where fish may congregate or prefer to migrate. The timing of migrations is of concern in evaluating fish intake interactions. The timing of salmon and steelhead life stages in the Thompson River is outlined in Table 2 along with comments on spawning location, predators, prey, etc. A more detailed account of the distribution of pink salmon is contained in Table 3. In numerical terms the pink salmon and sockeye salmon are most important (Table 1).

Spawning

Pink salmon spawn in the Thompson River upstream of the proposed intake and to a small extent in the Bonaparte River. The closest known Thompson River spawning location is along the right bank^{*)} immediately upstream of the Canadian National Railroad Bridge which crosses the river upstream of the proposed intake. Scuba surveys were conducted by the International Pacific Salmon Fisheries Commission during the 1977 pink salmon run. No spawning pairs or redds were sited in the 8,000 foot stretch of river in which the intake is to be located. It was

*Looking downstream.

concluded that the area had poor spawning habitat conditions including low water velocity and unsuitable gravel (Personal Communication, P. Andrew). Appendix E to this study contains the written report on the International Pacific Salmon Fisheries Commission's 1977 survey. Their report was received too late for incorporation into this report. Sockeye salmon are not known to spawn at any locations close to the proposed intake location. Some chinook, coho and steelhead could possibly spawn in the area of the intake.

Rearing and Downstream Migrations

Aspects of the life history of salmonid fry determine the types of intake interactions which will occur during downriver migration or instream rearing. Pink salmon actively migrate downriver immediately after emergence, at which time they are approximately 30 mm in total length. Steelhead, coho and chinook fry establish residence in shallow stream areas where they feed and grow prior to migrating downriver. Sockeye fry generally enter lakes to take up residence for an extended period prior to migrating downriver. Migrating sockeye smolts will move actively downstream, while coho, chinook and steelhead can be expected to make a slower "feeding migration" (Personal Communication, E. Brannon). The juvenile pink salmon which encounter the facility will all be quite small, while most of the migrating sockeye, chinook, coho and steelhead will have attained a much greater size. It is possible that some small coho, chinook or steelhead fry may rear in the vicinity of the intake and these fish may encounter the intake at sizes as small as the pink fry. The relative fraction of these species populations which encounter the facility at a small size will be quite low for all species except pink salmon. Swimming ability improves with increasing size. Pink salmon fry will be least able to avoid interaction with the intake structure. Therefore, designing the intake to protect pink salmon fry is of high priority.

Upstream Migrants

Upstream migrating adult salmonids may encounter the intake facility and provisions must be incorporated in the design to allow free passage through or

around the facility. Observations of upstream migrating pink and sockeye salmon in 1977 by E. Mulvihill, K. P. Campbell and A. Auld indicated a movement confined to an area within approximately 10 meters from the shoreline. These observations also indicated a substantially greater fraction of the migrants passing upstream along the left shoreline than along the right shoreline. Some upstream migrating salmonid fishes will hold in desirable areas between periods of active upstream migration. Scuba surveys conducted during 1977 indicated that the intake location was not an area utilized by pink salmon for holding (See International Pacific Salmon Fisheries Commission Survey in App. E).

Haven for Predators

Among the potential problems which one must consider in design of water-handling and fish-handling structures is the development of areas which will concentrate fish predators. A number of fish which are known to prey on young salmonids are present in the Thompson River (Table 1). These include northern squawfish, brook trout, rainbow trout, and Dolly Varden. The design features of the proposed Thompson River intake should avoid providing additional habitat for these species. We see no difference in the intake structure and other large obstructions in the river (bridge piers, boulders, etc.) as areas which will concentrate predators.

SECTION II. FISH PROTECTION DESIGN CONSIDERATIONS

A. Behavioral Aspects of Key Fish Species

Behavioral aspects of the key fish species which may relate to refinements of the proposed intake design have been considered. The behavioral categories which are considered relevant are swimming ability, downstream migration and potential for guidance. These subjects are discussed below.

Swimming Ability

A number of studies of swimming speed, metabolic rate, etc., have been conducted. Greenland and Thomas (1972) tested chinook fry in a stamina tunnel at various velocities ranging from 18-27 cm/sec (0.6-0.9 ft/sec), for durations of 3, 6 and 9 minutes and determined the number still swimming (not impinged). At 18 cm/sec (0.6 ft/sec) 98% of 33-35 mm fry were still swimming after 3 minutes. At 21 cm/sec (0.7 ft/sec) 88% were still swimming and at 24 cm/sec (0.8 ft/sec) 72% were still swimming. Larger fry performed better but a slump in performance was noted at 37.0-38.9 mm, the approximate size at yolk-sac absorption. They recommended designing screening facilities to accommodate fish with a swimming speed of 18 cm/sec (0.6 ft/sec). At high velocity (27 cm/sec (0.9 ft/sec)), approximately 50% were still swimming after 9 minutes. Considering the length of the intake a sweeping velocity in excess of 27 cm/sec should serve to remove chinook fry from the vicinity of the screen face rather rapidly.

Brett and Glass (1973) studies swimming speeds of sockeye salmon as related to size and temperature. In this study there were no small fry (<50 mm), however, mathematical relationships were derived which have been applied to smaller sizes of other species. Among the equations derived is the following for sustained 60 min swimming speed in cm/sec (Y) as related to total length in cm (X) at 2°C:

$$\log Y = 0.9053 + 0.6294 \log X$$

This equation indicates a sustained swimming speed of 16 cm/sec (0.53 ft/sec) for 30 mm fry at 2°C. Higher temperatures (within limits) yield greater swimming speeds. Brett (1973), in a further study of sockeye, observed bursts at the onset of fatigue of 2-3 times the maximum sustained speed. These bursts could only be maintained for a period of approximately 20 seconds.

Davis et al. (1963) studied the swimming performance of chinook and coho salmon at various temperatures. The chinook underyearlings ranged from 51-76 mm fork length and the coho underyearlings ranged from 67-90 mm fork length. They determined velocity of first failure at different temperatures and dissolved

oxygen levels. At 8-9 ppm dissolved oxygen first failures of chinook under-yearlings occurred at 49-55 cm/sec (1.6-1.8 ft/sec). Under similar conditions coho first failures were also between 49-55 cm/sec (1.6-1.8 ft/sec). Certain problems in the experimental animals and techniques were noted, possibly resulting in failures at lower swimming speeds than would occur in nature.

Fisher et al. (1976) studied the swimming performance of chinook salmon ranging in size from 30-50 mm at flows ranging from 6 to 30 cm/sec (0.2 to 1.0 ft/sec) and durations up to 6 hours. Results were expressed as mean % success during the last 3 hours. A dip in performance was noted at 40 mm which was associated with yolk-sac absorption. Performance was inversely related to velocity and directly related to size. At velocities less than 18 cm/sec (0.6 ft/sec) young chinook were observed to lie on the screens even when not impinged.

Pyper (1976) observed pink and chinook fry in the Thompson River which gathered in the relatively quiet water in front of an intake screen where approach velocities ranged from 5 to 6 cm/sec (0.17 to 0.21 ft/sec) (or less). No impingement was observed at these velocities. Small schools of 20-30 fish were observed in the surface water in front of the screen mesh, facing into the current, and holding a stationary position with their tails within one inch of the screen.

Andrew (1977) cited a study by Chapman (1935) that showed less than 4% of chum fry were swept over a revolving drum screen after 5 hrs at 12 cm/sec (0.4 ft/sec), but 70.5% were swept over after 1 hr at 18 cm/sec (0.59 ft/sec). Andrew (1977) also cited Edgeworth that chinook, coho and steelhead were not impinged on irrigation ditch screens at 12 cm/sec (0.4 ft/sec) but suffered total mortality at 30 cm/sec (1.0 ft/sec).

Based on all of the above observations it would appear that small salmonids (about 30 mm) can withstand intake approach velocities below 15 cm/sec (0.5 ft/sec) for relatively long periods of time. This conclusion is consistent with recommendations by the U.S. Environmental Protection Agency (1976), the U.S. Nuclear Regulatory Commission (1975) and the U.S. Fish and Wildlife Service (1975, 1977)

that the maximum velocity protecting most small fish is 15 cm/sec (0.5 ft/sec). It should be noted that this conclusion is in lieu of a bypass velocity which can be used to escort fish from the face of a screen before they become fatigued.

Downstream Migration

Pink salmon fry actively migrate downstream to sea, often covering up to 10 miles in a night. Neave (1955) reported an active migration at a velocity greater than the downstream current, creating a "bow-wave" in front of fry on the surface. Some reports indicate nighttime migration with fry seeking cover in gravel during daylight while other reports indicate migrations during both day and night. (Hart 1973, Scott and Crossman 1973). In the Thompson River catches of pink fry have been similar during day and night (Pyper 1976). Apparently a preference for light develops shortly after exposure to light (Hoar 1956) after which the fry school up and migrate actively in daylight. In the turbid Fraser River fry generally move downstream while maintaining a more or less random depth distribution except for a tendency to congregate nearer the surface in daylight (Hoos and Packman 1974, Vernon 1966). Vernon (1966) found that in the Thompson River pink fry tended to be concentrated near the surface except when turbidity was high. Neave (1955) reported catching 62% of pink fry in the upper third of a 46 cm (18-inch) column of water. Neave (1955) also noted an ability of migrating fry to avoid solid objects: "The fish traveled singly at a speed much greater than the current, and often appeared to avoid expertly the solid objects which sometimes made a straight course impossible." McDonald (1960) found that above 40 cm/sec (1.3 ft/sec), lateral distribution was directly related to current velocity in Williams Creek with greater fish densities in mid stream where velocity was higher.

Sockeye salmon smolts migrate to sea after having spent approximately one year in fresh water. The downstream migrants swim actively downstream in quiet reaches of rivers, but proceed tail first through rapids (Hart 1973). Travel speed ranges from 3-40 km (2-25 miles) per day (Hart 1973). McDonald (1960) found that lateral distribution was closely and positively related to currents

greater than 40 cm/sec (1.3 ft/sec). Hoar (1954) found an increased activity of smolts leading to downstream displacement at night.

The downstream migration of the territorial coho salmon occurs when the fish are approximately 100 mm long. The smolts form schools prior to migration and the schools move mainly at night (Scott and Crossman 1973, Hart 1973). Sudden elevations in water level increased the rate of downstream displacement (Hoar 1951).

Chinook salmon migrate downstream at varying sizes dependent on the stock involved. Some migrate almost immediately after emergence while others remain in the streams for as long as two years (Scott and Crossman 1973). Their behavior after emergence is school formation at first but later individual territories are established in the stream margins (Scott and Crossman 1973).

Steelhead generally remain in fresh water for 2 or 3 years prior to migrating to sea. At this age they are relatively large compared with the pink, chum or chinook smolts.

Farr (1976) noted that in reservoirs juvenile salmonids are normally in the top 9-15m (30-50 ft) of water during migration and when entering a dam intake 75-80% are concentrated in the top 5m (15 ft) of the intake. Fields (1964) found that 90% of fish used a bypass 5m (15 ft) below the surface at McNary Dam, but an entrance 15m (50 ft) below the surface was not as successful. Rees (1956) found few fish below 14m (45 ft) and the area from 0-5m (0-15 ft) (18.6% of cross-sectional area) contained 88.8% of the fish.

In general, salmonid smolts migrate in the upper layers of water. This statement is, however, subject to numerous exceptions, especially in the case of pink salmon fry in the Frazer River which show varying behavioral attributes depending on conditions.

Potential for Guidance

Bates and Vinsonhaler (1957) investigated the use of louvers for guiding

fish. Tests indicated that chinook salmon (>70 mm) could be successfully diverted with a louver bypass system. With a louver frame at 16° to the main flow, louvers 2.5 cm (1 inch) apart were 84-100% efficient. Louvers 8.9 cm (3.5 inches) apart were 86-98% efficient. It was observed that if velocity did not exceed the maximum swimming speed of the fish, fish would drift downstream tail first. At higher velocities the fish were turned sideways.

Bates and Jewett (1961) tested louver efficiency in deflecting downstream migrant steelhead. The louver frame in the $2.83 \text{ m}^3/\text{sec}$ (100 cfs) canal was at 20° to the flow and louver spacing varied from 5.1 to 25.4 cm (2 to 10 inches). The bypass efficiency was 98% for 5.1 cm (2 in) spacing, 90-95% for 10.8 cm (4.25 in) spacing and 62.6% for 25.4 cm (10 in) spacing. Variations in approach velocity did not appear to affect efficiency.

Fisher et al. (1976) noted that chinook salmon actively attempted to pass through screen openings and that a maximum clear opening of 4 mm (diagonal) was a positive barrier to 30-50 mm chinook salmon. Their studies indicated that a conventional louver concept was not efficient for small fish. However, Schuler (1974) found that louvers with 2.5 cm (1 in) gaps between vanes were better than screens for bypass of medium sized saltwater fishes. Schuler's bypass velocity exceeded the approach velocity.

The literature indicates that almost all fish will avoid being impinged for the period of time before fatigue sets in. They will generally do this by maintaining position at some distance in front of the test screen. Most studies, especially on young salmonids, have been conducted with flows perpendicular to the screen and with no lateral component to the flow. If a lateral flow component (bypass flow) is added to the intake component fish will be transported laterally until fatigue sets in due to fighting the intake flow. If the fish is transported past the six screen faces before fatigue sets in, it will be removed from the danger of impingement. Experiments conducted with louvers and angled screens, and experience at operational installations document the effectiveness of bypass flows.

Based on the above principal, Andrew (1977) calculated the passage time that should be designed into the Hat Creek intake. He recommended a passage time of not more than ten minutes in the bypass channel assuming a screen approach velocity of about 12 cm/sec (0.4 ft/sec).

B. Interviews of Knowledgeable Individuals

The anticipated specific behavioral interactions of the key fish species with the intake design illustrated in Sandwell drawing A4191-SK63 (Appendix B) were discussed with a number of experts in the Pacific Northwest. Interviews were conducted with Mr. George Eicher and Dr. Stanley Katkansky of Portland General Electric Company; Mr. Richard Duncan of the Portland Office of the U.S. Army Corps of Engineers, Project Operations Division; Dr. Roy Hamilton of Pacific Power and Light Company; Dr. Ernest Brannon of the University of Washington; Mr. Clifford Long of the National Marine Fisheries Service; and Mr. Andrew Auld of EBASCO. With each of these individuals we discussed the problems of downstream migrants, upstream migrants, predators and carcasses as they relate to the proposed design. A copy of the question sheet used for these interviews is contained in Appendix C. to this report. Consensus opinions are reflected below.

In general most individuals felt that fry would tend to be passed through the bypass channel given the approach velocity of 12 cm/sec (0.4 ft/sec) and minimum bypass channel velocity of 21 cm/sec (0.7 ft/sec) determined in the physical model studies. However, there was some concern that fry may hold position in the bypass channel particularly in areas where eddies may develop such as in the vicinity of the piers between the traveling screens. Mr. Long felt that water in the near vicinity of the screen face would be approaching the screen face at a right angle and, as such, would interfere somewhat with downstream movement of fry within this zone of influence. It was generally agreed that a bypass velocity in excess of the approach velocity is desirable. The need for an upper limit to velocities in the bypass channel was generally discounted except that very high velocities

(6 m/sec (20 fps)) may induce excessive turbulence. A potential reason for limiting velocity in this channel would be to avoid turbulence and provide a more laminar flow. Downstream migrants are expected to be found along shore as well as in midriver with variation among species. A preference for upper water levels was noted, and diel differences in migration rates were expected for pink salmon based on their emergence at night.

Darkness in the bypass channel was not generally considered a problem. One expert felt that darkness may affect guidance and saw no advantage to darkening the area. Two experts felt that guidance would be enhanced in the dark because the fish would have no visual keys causing them to hold position.

The presence of lights on the external structure was projected to have potentially different effects depending upon species and light intensity. Dr. Brannon noted that sockeye are attracted to strong beams of light. Consequently strong lights should be avoided. Pink salmon fry were noted as being photonegative. However, exposure to light apparently has differing effects on their behavior depending on conditions (Hoar 1956). It is not clear whether strong lights would serve to increase or decrease interaction of pink salmon fry with the intake.

The potential for creating a haven for predators was discussed. This potential was not considered a serious drawback of the design. The structure will not exclude predators, but it was not expected to produce concentrations of prey or predators substantially above those found elsewhere in the river.

It was generally agreed that some upstream migrants would enter the bypass channel while others would pass around the outside of the facility. Opinions were divided as to which route would be preferred. A 30 cm (12 in.) opening in the trash racks at the upstream approach section was considered satisfactory for passage of even the largest chinook salmon if adequate room (e.g., 1 m) was provided to allow larger fish to turn.

Impingement of carcasses on the upstream trash racks was expected, but no one felt that this would be a severe problem. Carcasses which enter the bypass channel are expected to be passed downstream. - 11 -

The location of the proposed intake at site D was considered satisfactory and alternate locations below the Bonaparte River were considered undesirable because of the added number of downstream and upstream migrants passing either to or from the Bonaparte. The pier-type location was considered superior to a bank-type intake, and a relatively straight location in a deep section of river away from riffles or rapids was considered desirable.

C. Suggested Design Refinements

During discussions with the above mentioned individuals, a number of potential design refinements were suggested. From an engineering standpoint not all of these refinements may prove feasible and they do not all reflect the opinions of the authors of this report. In the interest of completeness they are all presented here.

The general comments of all experts contacted indicated that the design presented to them was excellent considering the condition encountered in the Thompson River and reflected the state of the art in fish protection at intake structures.

With respect to protection of migrating fry and smolts, a generally recommended change was removal of projections into the bypass channel by utilizing flush-mounted traveling screens, preferably without lips. The use of louvers as a guidance device at the upstream approach section was also mentioned several times. Replacement of the upper portion (near water surface) of the upstream approach section of trash racks with a curtain wall, stop logs, etc., was suggested to take advantage of the surface orientation of most migrating salmonids. It was also suggested to make provisions to place stop logs at intervals behind the traveling screens to more evenly distribute intake flow across the screen face if uneven flows cause impingement problems during operation. Use of flow directing vanes within the bypass channel close to the traveling screens and within the traveling screens was suggested to assure that the flow would strike the screen at a shallow angle (approximately 20°) and thus assist bypassing of downstream migrants within the near vicinity of the screen.

If flush mounting of traveling screens cannot be achieved, it was suggested that an air pump be used at the leading edge of each pier to lift fry in this vicinity up and out of the screens. Also it was suggested that several mechanical means, e.g., water jets and paddlewheels, could be used to induce a more rapid flow in the bypass channel. Removal of supports between the curtain wall and piers was recommended to provide more laminar flow.

Most individuals did not see how predator use of the facility could be avoided; however, a few suggestions were obtained. Since predators and prey may concentrate in any eddies formed at the downstream end of the structure, it was suggested that if the downstream end of the structure was tear-drop shaped it would be an advantageous modification. An induced bypass velocity of 0.9-1.2 m/sec (3-4 ft/sec) was recommended to eliminate predators from the bypass channel.

The passage of upstream migrants was not considered a problem, and most individuals felt that a 20 cm (8 in) opening between trash rack bars would be sufficient. Others felt that a 30 cm (12 in) opening would be necessary. An alternative which places the trash rack section with 30 cm (12 in) openings parallel to the river flow was considered viable so long as water flowed through these openings and there was ample room (1 m) to allow larger fish to turn. Some individuals recommended excluding upstream migrants by placing racks at the downstream end of the bypass channel.

SECTION III. FISH PROTECTION AT CHOSEN INTAKE

A. Intake Location

Potential intake locations C, D, E, F and G are indicated in Sandwell drawings D4191-5 and 6 (Appendix B). In choosing an intake location, areas which might be selected as being desirable fishing locations should be avoided. In this way one can avoid areas which are more likely to contain fish as well as avoid placement of an intake in someone's favorite fishing location. Restrictions in river width may lead to forcing most of a migrating population through a narrow

area. From a biological viewpoint, the ideal location for an intake in the Thompson River would be a relatively deep, wide, straight channel, without riffles or other turbulence. Among the alternative sites, site D is clearly the best with respect to all of these factors. Site C on the opposite bank is second best. Use of this site would, however, probably require a submerged pipe river crossing which would not be desirable. Aesthetics would be a consideration if a bridge crossing were constructed. Sites E and F are not desirable locations due to riffles and/or flow restrictions. Site G is on the depositional side of a curve in the river. Sites G, E and F are all downstream of the confluence of the Bonaparte River and will thus have additional input of migrant fish and suspended solids with which to deal. The relative ranking of acceptability (from most to least) of the possible intake sites from a fish protection viewpoint is D, C, G, F, E.

The offshore location of this pier-type intake is clearly superior to intakes located along a river bank. This design acts to protect small steelhead, chinook, coho, and other fry which may occupy territories in the river margin. This also allows passage of upstream migrants around the facility in the relatively shallow water which they commonly use. It is suggested that a minimum distance of 3 m (10 ft) of watered area be maintained between the bank side of the intake structure and the river bank. This will protect the shallow water rearing habitat which is used prior to and during the downstream migration of many species.

B. Intake Design

The Thompson River intake design incorporates numerous fish protection features. The primary feature of the design is a bypass channel which sweeps the screen face at approximately two or more times the approach velocity. This feature coupled with approach velocities of 12 cm/sec (0.4 ft/sec) or less will serve to transport migrating fry which enter the channel past the traveling screens and return them to the river. The screens themselves will be constructed of a fine (2.5 mm (0.1 in)) mesh which will serve to exclude small fry from possible

entrainment. The pier-type structure will isolate the intake from the many young fish which utilize the shoreline areas of the river, and may also isolate the structure from upstream migrants since at least pink and sockeye salmon adults seem to migrate along the shoreline. The generally streamlined bullet-shaped configuration of the structure should serve to minimize eddies and turbulence resulting in fewer fish holding position in the immediate vicinity of the intake. The normal spacing between trash rack bars will allow passage of many upstream migrants and the area of 304 mm (12 in) spacing will allow passage of the largest chinook salmon.

Individuals with experience in the operation and design of Northwest intake structures saw this as being a considerable improvement over most bank-type intakes and comments indicated that this design was considerably advanced with respect to the state of the art in designing intake structures to protect fish. This is not to say that some of the detailed design aspects cannot be refined to provide an even greater degree of fish protection.

Comparing this structure with the Lornex Mining Corporation intake on the Thompson River will provide some perspective on those features which may be modified to provide further refinement. Pyper (1976) performed a series of detailed observations at the Lornex intake which indicated the type of behavior which pink and chinook salmon fry exhibited with respect to this structure and the mode of entrapment and impingement of fish by the intake. The Lornex intake is set back in a U-shaped indentation in the river bank 12 m (40 ft) from the original shoreline. Such a configuration leads to eddies and still water in front of the intake, with a rather complex water circulation. Migrating fish which encounter such an area may experience difficulty in maintaining downstream movement. Furthermore, as pink fry begin to congregate in the area they may begin to school, especially in daylight, and switch from negative rheotaxis to positive rheotaxis (swimming against a current rather than with it). The complicated behavioral changes associated with exposure to light, experience with schooling and initial

direction of swimming have been studied by Hoar (1956). Neave (1955) reported on field observations of migrating pink and chum salmon. Based on the results of these two authors, we feel that intake designs which create eddies and still water can cause a change in behavior which halts downstream movement. For this reason an intake structure should provide little or no inhibition to the downstream movement of fry. The location of the Lornex intake along the shoreline also leads to interaction with species which reside along the shoreline, thus Pyper (1976) observed chinook fry as well as pink fry holding position in front of the intake screens facing into the current. The chinook fry were probably shoreline residents exhibiting a normal positive rheotaxis and the pink fry were probably downstream migrants which had become disoriented and were exhibiting abnormal positive rheotaxis.

Pyper (1976) also made observations on the manner in which fry were captured by the screens at Lornex. Impingement was not observed, nor would it have been expected considering the low approach velocities. Fry were captured during operation of the traveling screens with fish being carried up on the screen trays and then washed off into the wash trough. The horizontal bottom lip of the screen tray intercepted fish which were holding position in front of the screen face. Those fish which subsequently flipped to the front edge of the lip would fall back into the water, while those that flipped in the opposite direction were caught in a groove at the base of the screen. These fish were alive after having been carried up the screen face and washed into a trough. They then fell 12 m (40 ft) to the water surface.

The comparison with the Lornex intake is made for the sake of improving the state of the art during the present design. The number of fish captured at Lornex was small and capture did not necessarily mean mortality.

The Sandwell design should attempt to avoid the situations which led to the capture of fry at the screen face. A number of design features are involved. A primary factor should be minimization of any tendency to create eddies or still

water within the bypass channel. To this end, flush mounting of traveling screens, elimination and/or reduction of all cross members and other structural components within the bypass, and design of traveling screens without horizontal trays or lips should be considered. Pyper (1976) suggested an inclined lip on the traveling screen and elimination of the groove between the back of the lip and the screen mesh. This type of modification is clearly desirable. Pyper also suggested elimination of lights associated with the intake facility and, considering the attraction to strong light which some species exhibit and the behavioral changes in pink salmon associated with exposure to light (Hoar 1956), elimination of lights would seem to be a desirable feature.

Considering the experience at the Lornex intake and at other Pacific Northwest intakes we feel that certain features can be refined in the Sandwell design to provide a greater measure of fish protection. These recommended refinements are listed below:

1. To the extent possible, eliminate strong lights within the bypass channel and on the exterior structure.
2. Within the bypass channel provide for a minimum of turbulence, still water, or eddies by eliminating projection into the bypass channel; flush mounting traveling screens; and reducing, rearranging, and/or streamlining cross members.
3. Maintain a watered area between the intake and the shoreline of at least 3 m (10 ft) in horizontal distance.
4. Provide for the use of stop logs, slats, or baffles behind the traveling screens which could be used to spread the approach velocity more evenly across the screen face.

5. Make provisions for traveling screens which have either (a) no horizontal trays on lips; (b) an inclined plane lip angled at approximately 45° to horizontal with no groove at the intersection of lip and screen face; (c) hooks rather than horizontal trays.
6. Provide the downstream end of the structure with a more streamlined shape.
7. Use the reversed orientation of the upstream trash racks (or portion of it) to reduce bypass velocity only when it serves to reduce turbulence and eddies within the bypass channel or serves to exclude a high proportion of downstream migrants from entering the bypass channel by serving as a louver system. This procedure should not be followed if it reduces bypass velocity below approximately 27 cm/sec (0.9 ft/sec).
8. A structure such as a curtain wall on the upstream approach section of trash racks should be used to exclude surface oriented downstream migrants only if it can be shown that this will not result in any back-flow of water in the bypass channel or reduction in bypass velocity to less than approximately 27 cm/sec (0.9 ft/sec).
9. An upstream approach section which places the trash rack section with 30 cm (12 in) interbar spacing parallel to flow is recommended providing that a 1 m (3 ft) turning radius be allowed for large upstream migrants and that model studies indicate a downstream flow of water through this section.

It is realized that all of these refinements must be tempered by engineering feasibility and cost-effectiveness.

SUMMARY

The design and location of the Thompson River intake have been evaluated and both the design and location are good from a fish protection standpoint. The design represents the state of the art in fish protection and the primary location (D) is the most desirable of the alternatives. A number of design refinements and specifications have been recommended based on a review of fish behavior and interaction with other intake facilities. Incorporation of these refinements will enhance the fish protection effectiveness of the facility.

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APPENDIX A

TABLES

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TABLE 1: List of Fish Species in the Thompson and Bonaparte Rivers in the Vicinity of the Proposed Intake Location.

Common Name	Scientific Name	Thompson River	Bonaparte River	Comments
Pink salmon	<u>Oncorhynchus gorbuscha</u>	+++	+	Odd year only run of pinks
Coho salmon	<u>Oncorhynchus kisutch</u>	++	+	Spawning near intake
Sockeye salmon	<u>Oncorhynchus nerka</u>	+++		Migrating to N & S Thompson
Chinook salmon	<u>Oncorhynchus tshawytscha</u>	++	+	Spawning near intake
Mountain whitefish	<u>Prosopium williamsoni</u>		P	Food and game fish
Rainbow trout	<u>Salmo gairdneri</u>	P	P	Sport fish
Steelhead trout	<u>Salmo gairdneri</u>	P	P	Large-size steelhead, popular sport fish especially at Spences Bridge
Brook trout	<u>Salvelinus fontinalis</u>		P	Food and game fish
Dolly Varden	<u>Salvelinus malma</u>	P		
Peamouth chub	<u>Mylocheilus caurinus</u>	P		
Northern squawfish	<u>Ptychocheilus oregonensis</u>	P		
Longnose dace	<u>Rhinichthys cataractae</u>		P	
Leopard dace	<u>Rhinichthys falcatus</u>		P	
Redside shiner	<u>Richardsonius balteatus</u>	P	P	Forage fish
Finescale sucker	<u>Catostomus catostomus</u>	P		
Bridgelip sucker	<u>Catostomus columbianus</u>		P	Forage fish
Largescale sucker	<u>Catostomus macrocheilus</u>	P		
Aleutian sculpin	<u>Cottus aleuticus</u>	P		
Slimy sculpin	<u>Cottus cognatus</u>	P		

From: Environment Canada, Fisheries and Marine Service (1974), B.C. Ministry of Recreation and Conservation, Fish and Wildlife Branch (1976, 1977), B.C. Research and Dolmage Campbell & Associates (1975), Beak Consultants Limited (1977)

- + Uncommon - 0-1000 spawning and migrating fish
- ++ Moderately abundant - 1,000-100,000 spawning and migrating fish
- +++ Abundant - 100,000-500,000 spawning and migrating fish
- P Present

TABLE 2: Timing of Life History Stages of Salmon and Steelhead in the Thompson River.

Species	Life Stage	Time	Comments
Pink (odd years only)	incubate to fry stage	Aug. 28 ¹ - May ²	-
	rear	-	none (estuary rearing)
	downstream migration	Mar. 1 - May 15 (Peak Apr. 15 - May 9) ³	at Spatsum, 1976 12 miles d/s Ashcroft, fry make direct migration to ocean
	adult migration	Aug. 22 - Oct. 29 (Peak Sept. 29 - Oct. 12) ⁴	Thompson River, general
	spawning	Aug. 28 - Nov. 5 (Peak Oct. 4-18) ^{1, 5}	Thompson River, general
	predators (adult)	-	man - some bear
	predators (fry, smolts)	-	rainbow trout, Dolly Varden, coho smolts ²
	prey	-	-
Sockeye	incubate	-	none
	rear	-	none (lake rearing)
	downstream migration	Apr. 15 - June 20 ⁶	at Spences Bridge
	adult migration	July 20 - Oct. 28 ⁶	Thompson River, general
	predators	-	man - some bear ⁶
	prey	-	zooplankton, insects ⁶
Coho	incubate to fry stage	late Sept. - late Jan. ²	coho, general, 11 wks. incubation ²
	rear	Feb. - April ^{7, 8}	at intake and elsewhere
	downstream migration	Oct. ⁷	through proposed intake area
	adult migration	mid-Sept. - early Nov. ⁴	Thompson River, general
	spawning	late Sept. - early Nov. ⁹	Thompson River, general
	predators	-	man
	prey	-	insects

TABLE 2 Continued: Timing of Life History Stages of Salmon and Steelhead in the Thompson River.

Species	Life Stage	Time	Comments
Chinook	incubate	mid-Oct. - Spring ²	chinook, general
	rear	Feb., Mar. ⁷ , April ^{3*}	near intake and elsewhere
	downstream migration	May - Sept. ^{7, 8}	through proposed intake area
	adult migration	-	-
	spawning	early Sept. ⁹ - mid-Oct. (Peak late Sept.)	Thompson River, general
	predators (adult)	-	man
	predators (fry, smolts)	-	-
	prey	-	insects and crustaceans ²
Steelhead	incubate	Mar. - July ¹⁰	steelhead, general
	rear	-	-
	downstream migration	not known	-
	adult migration	late Sept. - April ¹¹ Oct. - Feb. (Peak Sept. - Oct.) ¹²	Thompson River, general Thompson River, angler data, 1976 ¹³
	spawning	March - June ¹¹	Thompson River, general
	predators (adult, fry, sm.)	-	man, trout, coho smolts ²
	prey	-	plankton, fish ²

* In the Shuswap area fry rearing an extra year in freshwater were common.

TABLE 2 Continued: Timing of Life History Stages of Salmon and Steelhead in the Thompson River.

Footnotes

- 1 International Pacific Salmon Fisheries Commission (1967-76)
- 2 Scott and Crossman (1973)
- 3 Pyper (1976)
- 4 Adult migration was calculated by subtracting 6 days from the spawning period - sockeye and pinks are known to spend 6 days on the grounds before spawning.
- 5 Spawning period was calculated on the basis of 5 years of data: 20 days was added on to either side of the mid-point of the peak spawning period over 5 years for pinks, 15 days for sockeye, (Fred Andrew IPSFC pers. comm.)
- 6 International Pacific Salmon Fisheries Commission, pers. comm., Fred Andrews (Oct., 1977)
- 7 Fisheries and Environment Canada, Fisheries and Marine Service (1977), pers. comm. L. Goodman, Kamloops
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- 10 Carlisle (1975)
- 11 B.C. Ministry Recreation & Conservation, Fish & Wildlife Branch (1977), pers. comm. John Cartwright, Kamloops
- 12 B.C. Ministry Recreation and Conservation, Fish and Wildlife Branch (1977), pers. comm. Sandy MacDonald, Kamloops
- 13 B.C. Ministry Recreation and Conservation, Fish and Wildlife Branch (1977), Thompson River steelhead survey 1976/77. Ted Antifeau, Kamloops 30 pp + fig. unpublished manuscript.

TABLE 3: Distribution of Spawning Pink Salmon in the Thompson River from the Confluence with the Fraser River to Kamloops Lake.

Portion of River	Number of Spawners ¹	% of Thompson River ¹
Downstream of Bonaparte (River miles 0 to 48) ²	58,278	22%
Upstream of Bonaparte (River miles 48 to 72)	206,623	<u>78%</u>
		100% ³
Confluence of Nicola to mile 35 (River miles 24 to 35)	18,543	7%
River miles 35 to 42	31,788	12%
River mile 42 to confluence of Bonaparte (River mile 48)	7,947	3%
River mile 48 to Kamloops Lake (River mile 72)	206,623	<u>78%</u>
		100%
Mile 0 to Walhachin Bridge (River mile 63)	98,013	37%
Walhachin Br. to Kamloops Lake (River miles 63 to 72)	166,888	<u>63%</u>
		100%

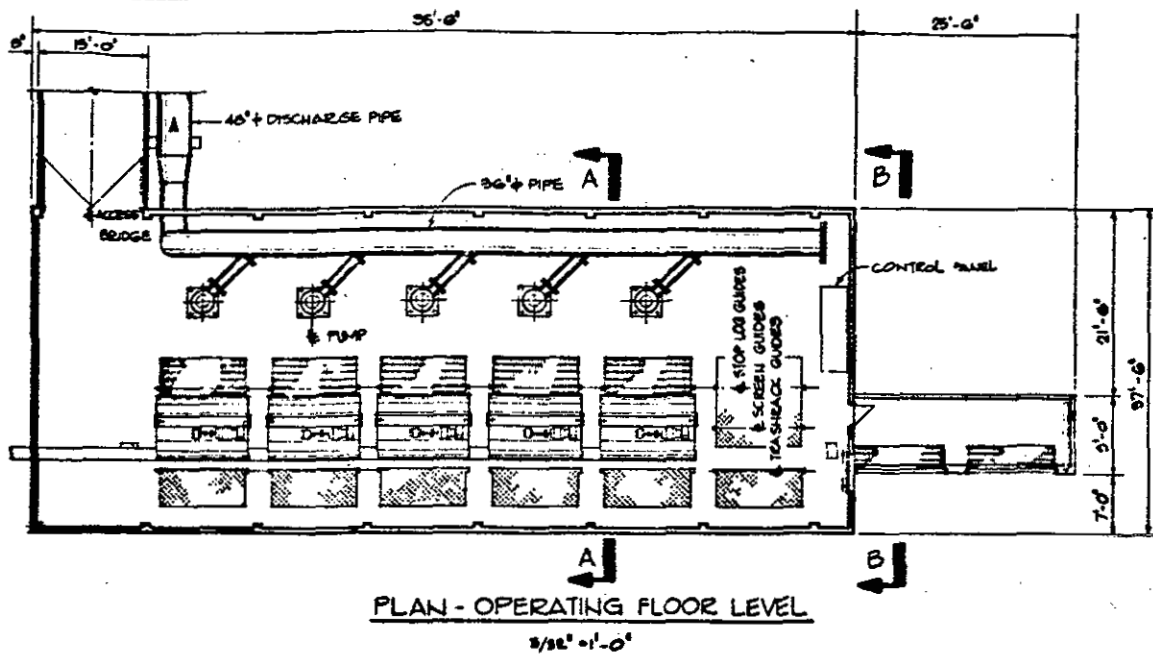
Source: International Pacific Salmon Fisheries Commission
Annual Reports
B.C. Research & Dolmage Campbell & Assoc., 1975
Federal-Provincial Thompson River Task Force, 1976

Footnotes

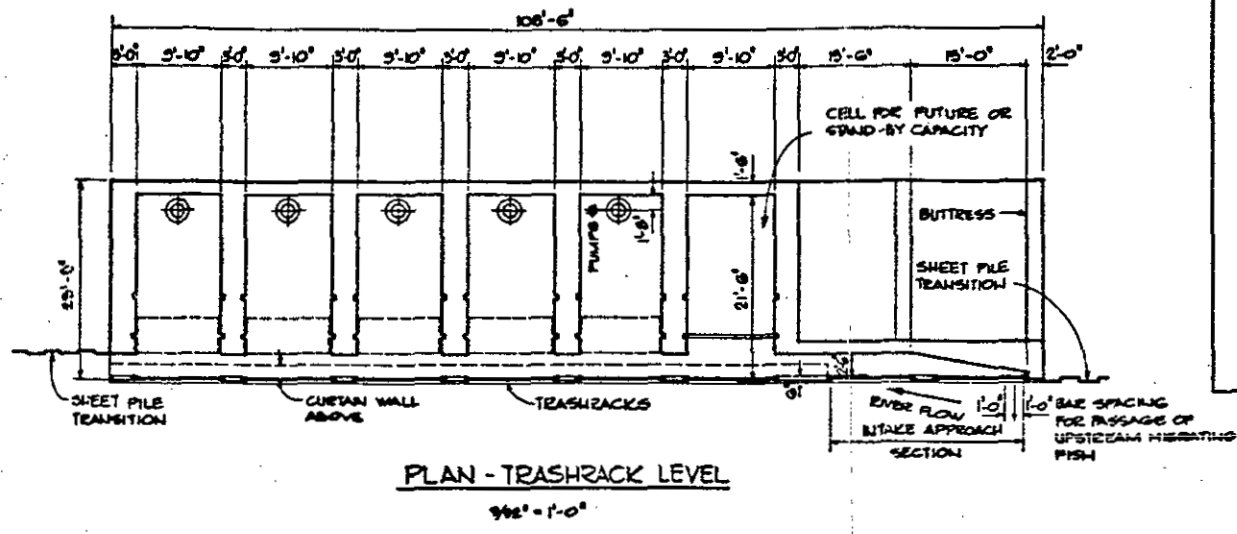
- 1 Five-year average 1975, 73, 71, 69, 67 - 264,901 spawners
- 2 Mile 0 is confluence of Thompson and Fraser River
- 3 In 1977 82% of the 973,000 pink salmon in the run spawned above the intake area (Personal Communication, F. Andrew)

APPENDIX B

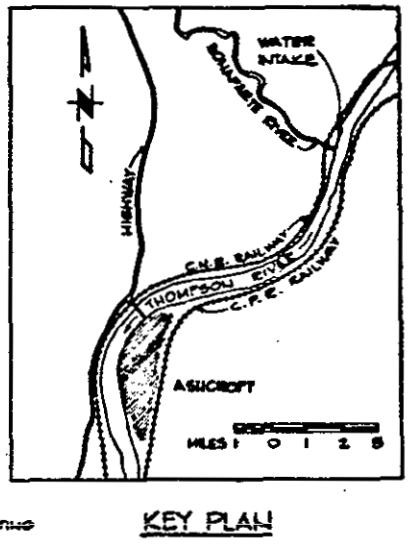
DRAWINGS



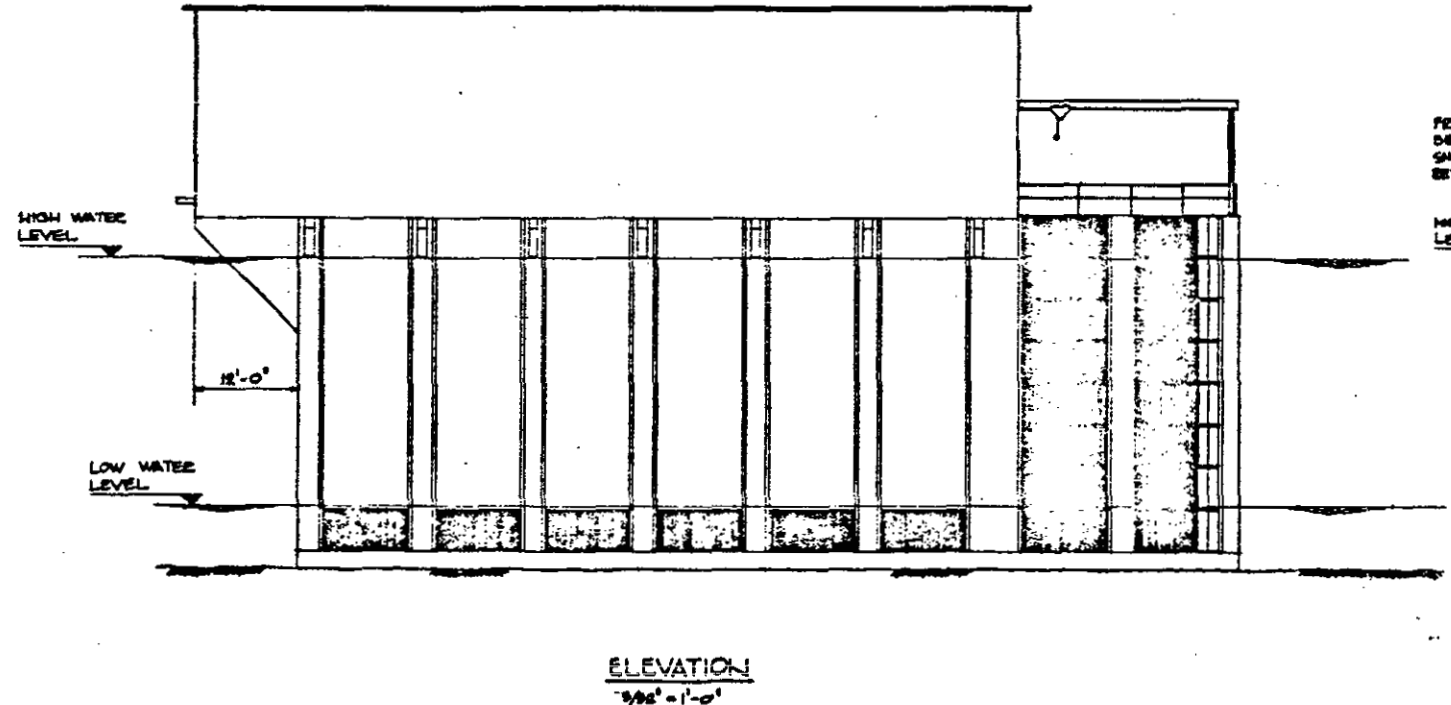
PLAN - OPERATING FLOOR LEVEL
97'-6"



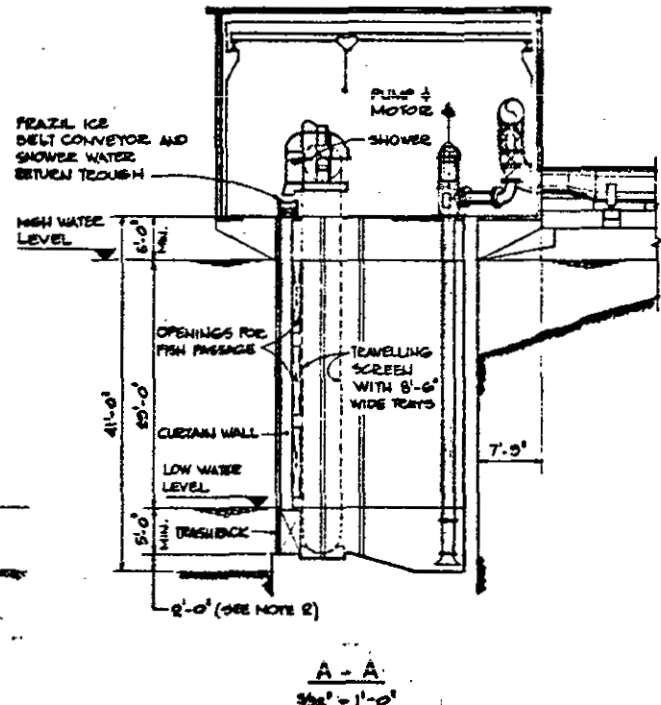
PLAN - TRASHRACK LEVEL
96'-1-0"



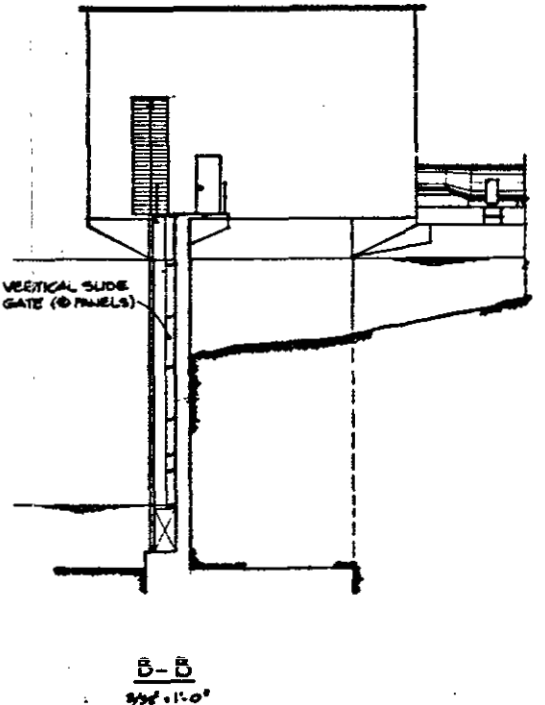
KEY PLAN



ELEVATION
96'-1-0"



A-A
96'-1-0"

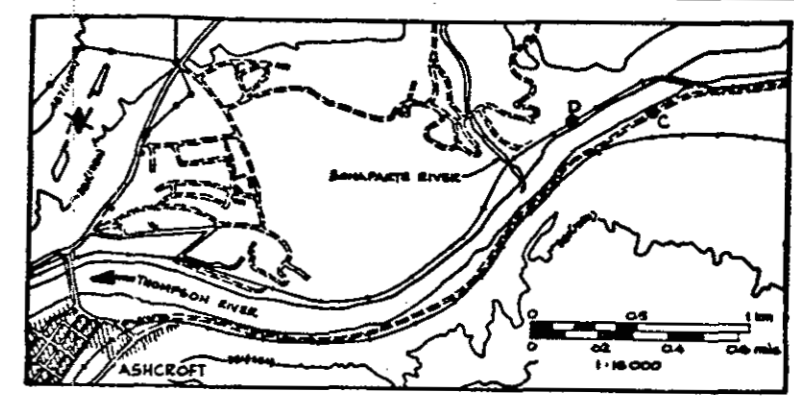
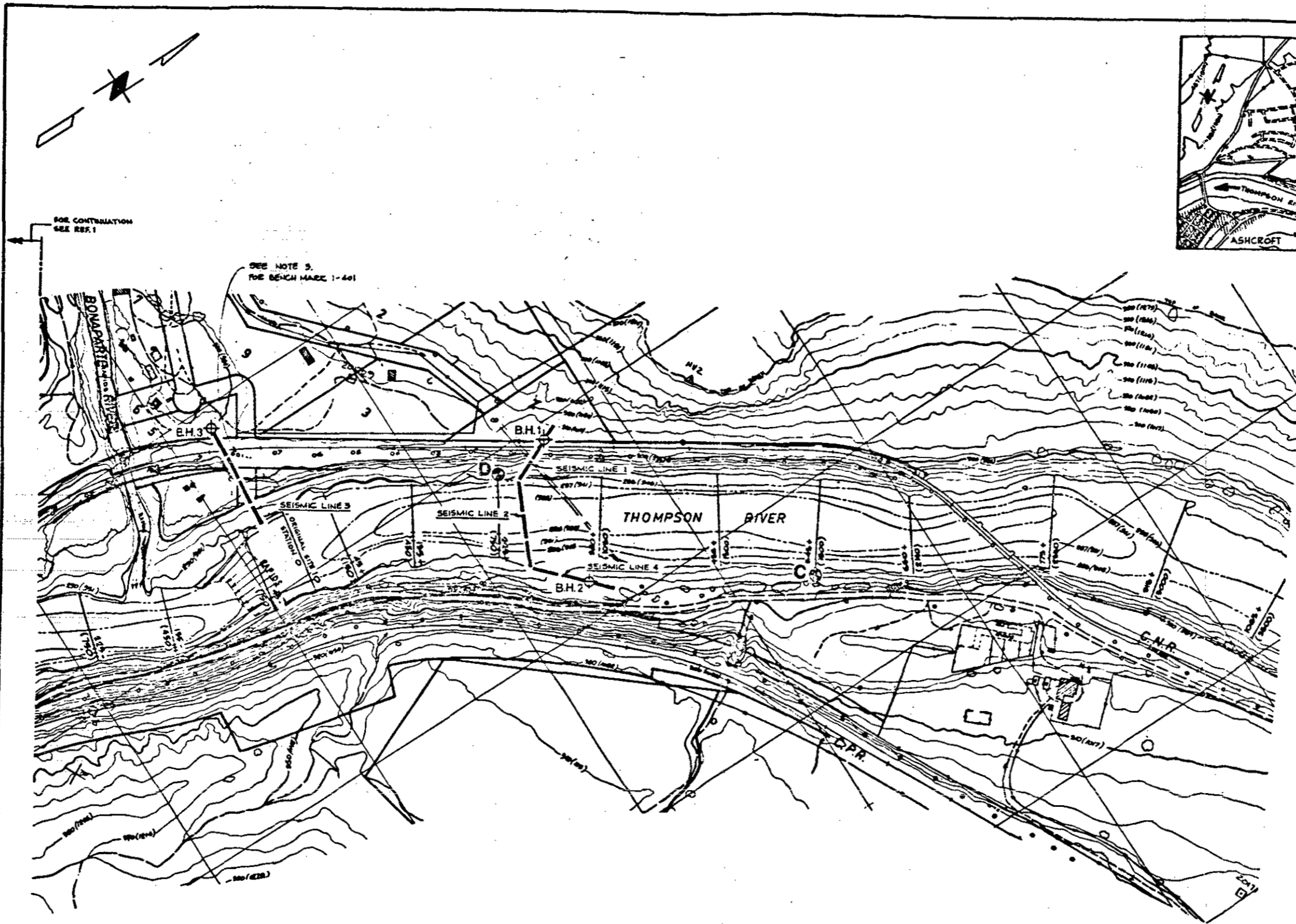


B-B
96'-1-0"

NOTES:
1. FOR DETAILED ORIENTATION SEE REF 1.
2. DISTANCE OF INTAKE SILL TO ENDS BOTTOM HAS TO BE CONFIRMED BY FIELD SURVEYS AND HYDRAULIC MODEL STUDIES.



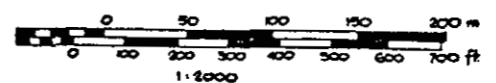
D 4007-ASSEMBLED TOGETHER - STRUCTURE PART 1										DATE: 7 JAN 77 FOR REPORT V 4007 12 7 JAN 77 FOR REPORT V 4007 12		APPROVED: D 4007 / 46 SANDWELL		HAT CREEK PROJECT WATER SUPPLY STUDY CONCEPTUAL DESIGN RIVER INTAKE - LAYOUT B.C. HYDRO AND POWER AUTHORITY VANCOUVER B.C. DWG. D 4007 / 46 REV	
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KEY PLAN

- LEGEND:**
- 1. CONTOUR DATA PREPARED BY McELMURNEY SURVEYING AND ENGINEERING LTD. FROM AERIAL PHOTOGRAPHS NUMBERS 49270 B AND 49271 B RUNN BY PACIFIC SURVEY CORPORATION ON 16 APRIL 1970.
 - 2. RIVER BED CONTOUR DATA PREPARED BY NORTHWEST HYDRAULIC CONSULTANTS LTD.
 - 3. LOT BOUNDARIES WITH LOT NUMBERS. THESE WERE OBTAINED FROM WESTERN MAPPING OF KAMLOOPS, DRAWINGS 92-1-11-1, 50 & 51.
 - 4. MEANS SAME ON BOTH SIDES OF ACCESS ROAD CORRIDOR. ACCESS ROAD (BETWEEN LOT BOUNDARIES).
 - 5. C.N.R. TELEGRAPH POLE (NUMBERED).
 - 6. POTENTIAL INTAKE LOCATIONS.
 - 7. TRIANGULATION SURVEY POINT.
 - 8. B.H. DENOTES BENCH MARK.

- NOTES:**
- 1. RIVER CROSS SECTION DISTANCES AND CONTOURS GIVEN IN METERS (FEET).
 - 2. FOR DETAILS OF INTAKE SITES 10 E, F & G - TOPOGRAPHY SEE REF. 1.
 - 3. FOR DETAILED TOPOGRAPHY OF INTAKE SITE 10 D SEE REF. 2.
 - 4. BENCH MARK 1-401 IS A RAILWAY SPIKE IN TELEGRAPH POLE # 6 ELEVATION 301.060 m (987.42 ft).

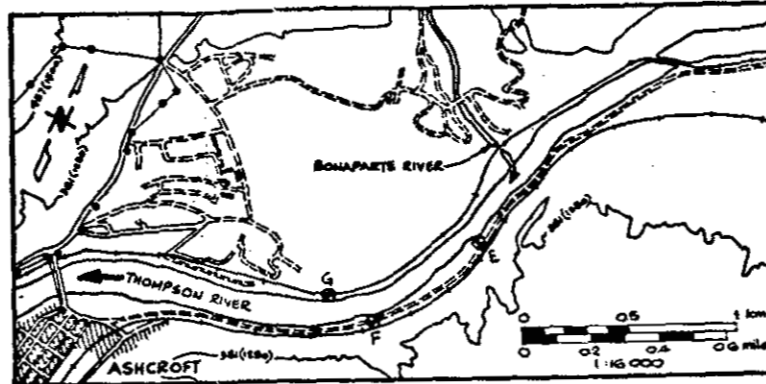
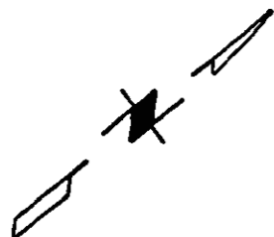


D 4191 - 7	INTAKE SITE 10 D - DETAILED TOPOGRAPHY	2
D 4191 - 6	INTAKE SITES 10 E, F & G - TOPOGRAPHY	1
REFERENCE DRAWINGS		NO

REFERENCE DRAWINGS

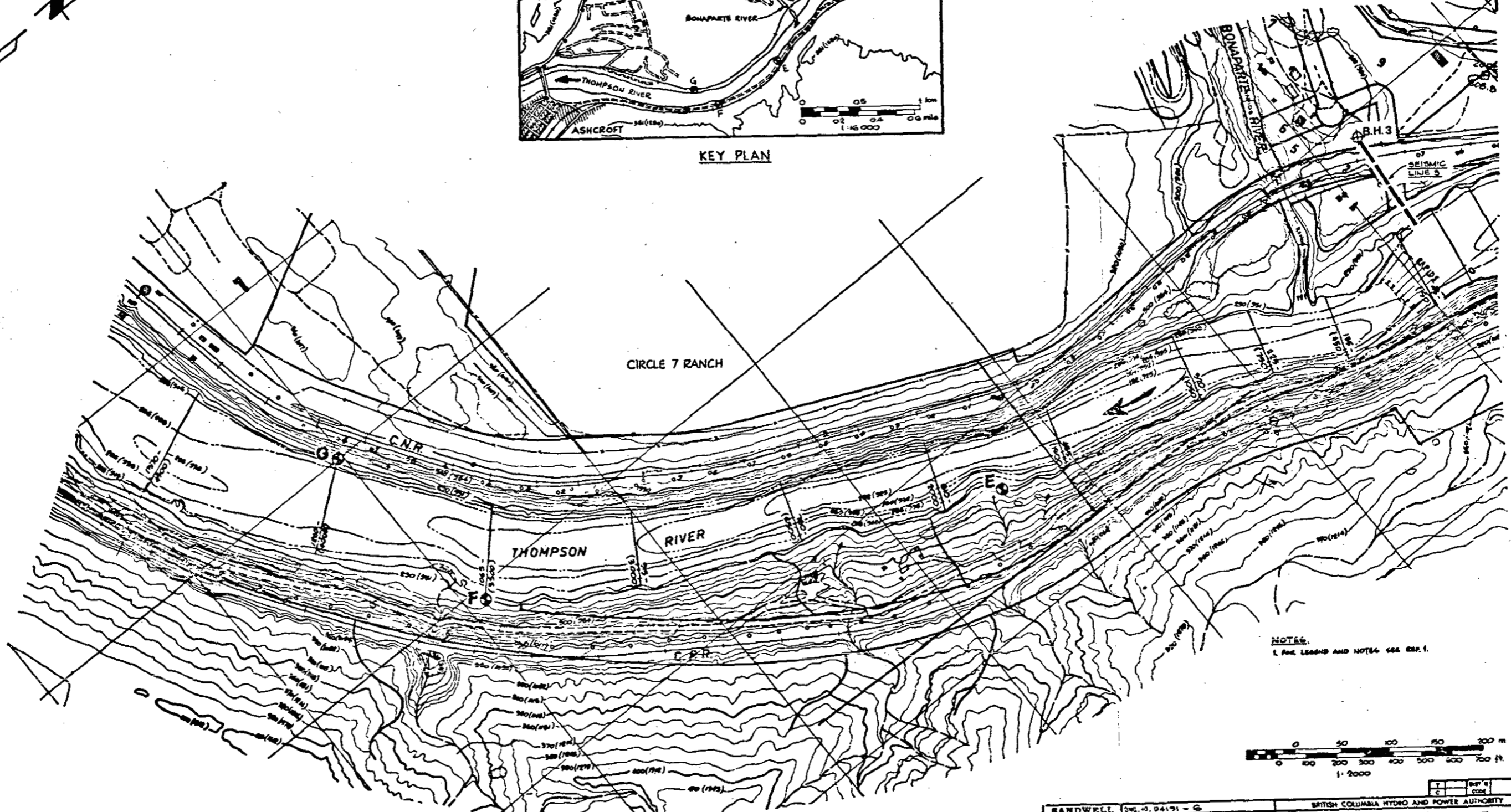
NO.	REVISIONS	DATE	BY	CHKD	APPD	REMARKS

SANDWELL Dwg. No. D4191 - 5		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
DESIGNED BY G.H.	CHECKED BY J.H.B.	DATE SEPT. 1977	SCALE AS NOTED
HAT CREEK PROJECT COOLING WATER SUPPLY PRELIMINARY DESIGN STUDY INTAKE SITES 10 C & D - TOPOGRAPHY		Dwg. No. D4191 - 5	

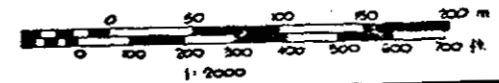


KEY PLAN

FOR CONTINUATION
SEE REF. 1

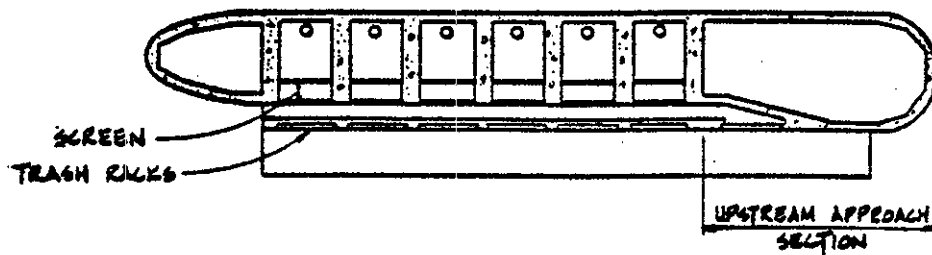


NOTES:
1. FOR LEGEND AND NOTES SEE REF. 1.

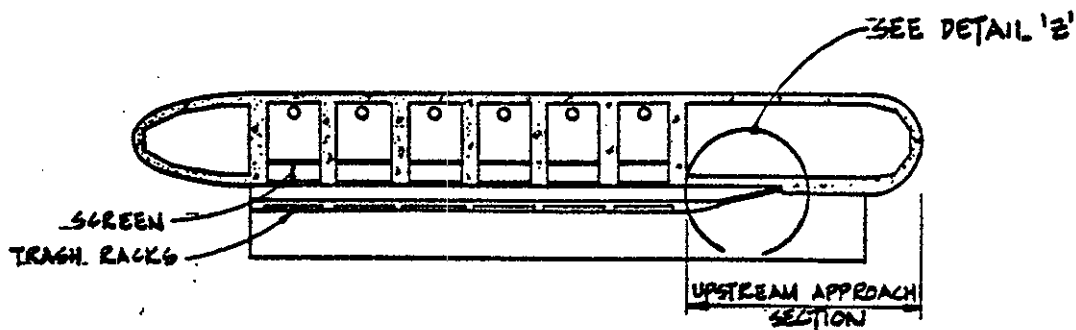


SANDWELL INC. NO. D4191 - G		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
DESIGNER	APPD. (T/C)	HAT CREEK PROJECT COOLING WATER SUPPLY PRELIMINARY DESIGN STUDY INTAKE SITES TO E, F & G - TOPOGRAPHY	
CHKD.	APPD.		
ENGR.	APPD.		
DATE	SEPT., 1977		
PROJECT NO.	V4191	CONTRACT NO.	SCALE AS NOTED
DRAWING NO. D4191 - G		SHEET NO. 1	

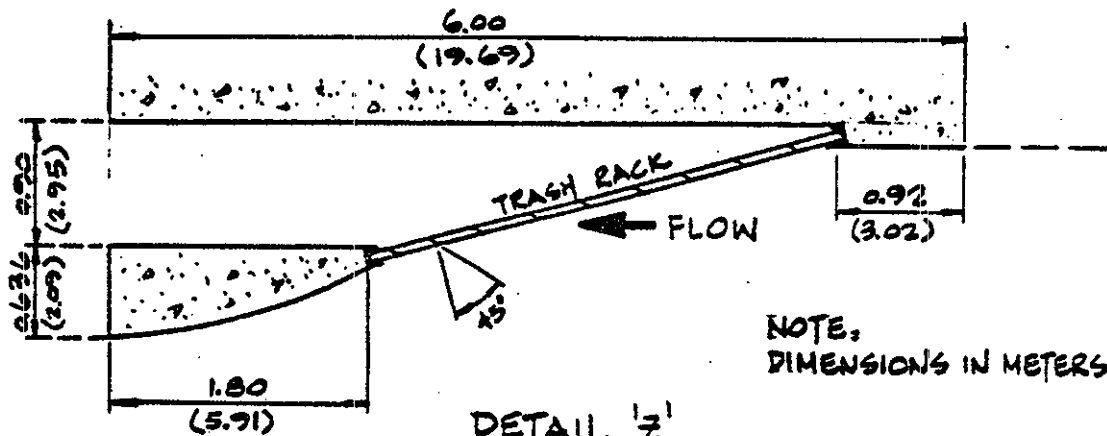
D4191 - G	INTAKE SITES TO C&D - TOPOGRAPHY	1	REFERENCE DRAWINGS	1	REVISED	DATE	BY	CHKD.	APPD.	DATE	BY	CHKD.	APPD.
REVISIONS													



PLAN OF INTAKE WITH ORIGINAL APPROACH SECTION.



PLAN OF INTAKE WITH MODIFIED APPROACH SECTION



NOTE:
DIMENSIONS IN METERS (FEET)

DETAIL 1/2"
SCALE: 1:50

DWG. No.	REFERENCE	REV. No.	REV. BY	DATE	REVISION	APP'D.
APPROVED	SCALE AS NOTED	DAY	NO.	YE.	HAT CREEK PROJECT COOLING WATER SUPPLY RIVER INTAKE MODEL MODIFIED APPROACH SECTION	
DATE	DR'M.	D.TUNG	21	977		
A	CHK'D.					
	APP'D.					
		<small>THIS DRAWING IS THE PROPERTY OF SANDWELL AND MUST BE RETURNED TO SANDWELL OR DESTROYED IF CONTAINED ARE SECRET AND CONFIDENTIAL. ANY INFORMATION OBTAINED BY INSPECTION OF THIS DRAWING SHALL NOT BE USED FOR ANY OTHER THAN THE SPECIFIC PURPOSE FOR WHICH ITS INSPECTION IS AUTHORIZED BY SANDWELL.</small>			B.C. HYDRO AND POWER AUTHORITY VANCOUVER B.C.	
SANDWELL		DWG. A 4191-SK63			REV	

APPENDIX CFISH PROTECTION QUESTIONSFry

1. At the low flow bypass velocity, as determined in the model, of 22 cm/sec (0.72 ft/sec) (minimum) or 27 cm/sec (0.89 ft/sec) (average) and screen approach velocity of 12 cm/sec (0.4 ft/sec), will migrating fry tend to bypass the screens or hold position in front of the screens?
2. What bypass/approach ratio is necessary for successful bypass of migrating fry?
3. How low a ratio would be detrimental? 1:1?
4. Should there be any upper limit to bypass velocities at high flow?
5. Would you expect migrating fry to be found alongshore, in mid-river or both?
6. Will migrating fry show any depth preference?
7. Will rates of migration or position in water show a diel variation?
8. Will darkness in the bypass channel affect guidance at the screens?
9. Would lights serve to increase guidance or attract more fry?
10. Are there any species specific factors which would make this design more or less effective in guiding certain salmonid species?

Predation

1. Will this structure provide additional habitat for predators? What species - Dolly Varden, squawfish?
2. Will this structure serve to concentrate fry leading to concentrations of predators?
3. Would predator concentration be a serious drawback of this design?

Upstream Migrants

1. Will upstream migrants enter the bypass channel at the downstream end with bypass velocity \sim 22 cm/sec (0.72 ft/sec) or will they go around the intake structure on either side?
2. If upstream migrants enter the bypass channel will the trash rack area with 30 cm (1 ft) between bars allow passage?

3. What would be minimum spacing of bars to allow passage of upstream migrants, and how wide an area of such spacing would be recommended?

Carcasses

1. Considering the approximate 15 cm (6 in) spacing of bars in the trash rack, will carcasses pass through the rack, become impinged upon it or slough off into the river?
2. If some carcasses do pass into the bypass channel will they continue on through the 30 m (100 ft) long channel or will they settle out at a low-flow bypass velocity?

Any General Comments on Design

1. Trash racks (vertical bars 30 cm (1 ft) apart)
2. Screen design (standard vertical traveling)
3. Distance from shore
4. Depth of intake (1.5-10.4 m (5-34 ft))
5. Screen bypass channel (0.9 m (3 ft) width)
6. Screen recesses (~15 cm (6 in))

Intake Location Comments

1. Depth of intake
2. Meanders vs straight runs
 - o Accretion side
 - o Erosion side
3. Bank vs pier intake
4. Riffle areas vs pool areas

Potential Design Refinements - Comments

1. Racks at downstream end of bypass
2. Altered width between trash rack bars
3. Intermittent vs continuous operation of traveling screens
4. Inclined plane lips on traveling screens with hooks for frazil ice and debris
5. Others

INTAKE FACT SHEET

1. Pier intake ~30.5 m (100 ft) from shore
2. 1/10 inch mesh vertical traveling screens
3. Vertical trash racks top to bottom at upstream approach
4. 10.4 m (34 ft) curtain wall and 5 foot trash racks along remaining intake
5. Intake flow = $1.58 \text{ m}^3/\text{sec}$ (55.7 cfs)
6. River flow = $115 \text{ m}^3/\text{sec}$ (4,060 cfs) 100 year minimum, $4134 \text{ m}^3/\text{sec}$ (146,000 cfs) 100 year maximum
7. Screen approach velocity = $12 \text{ cm}/\text{sec}$ (0.4 ft/sec)
8. Bypass velocity = $22 \text{ cm}/\text{sec}$ (0.72 ft/sec) minimum, $27 \text{ cm}/\text{sec}$ (0.89 ft/sec) average
9. Bypass channel width = 0.9 m (3 ft)
10. Bypass channel length ~30.5 m (100 ft) 23.8 m (78 ft) screened
11. Low flow depth ~1.5 m (5 ft)
12. High flow depth ~10.4 m (34 ft)
13. Traveling screen recesses ~15 cm (6 in)

APPENDIX D.GLOSSARY

Anadromous - A life history pattern in which adult fish return from the sea to spawn in fresh water.

Diel - Referring to a 24 hour day, e.g., feeding patterns in some species may exhibit a diel periodicity with heavy feeding at sunrise and/or sunset.

Forage fish - Fish which serve as prey for larger game fish.

Fork Length - Length of a fish measured from the tip of the snout to the fork of the tail fin.

Metabolic Rate - The rate at which life processes proceed; may vary with temperature and other factors.

Predator - Within the context of this report, predators are fish which consume other fish.

Rough fish - Fish which may compete with game fish for food, but are not considered valuable game fish or commercial fish.

Spawning - Egg deposition and fertilization.

INTERNATIONAL PACIFIC SALMON FISHERIES COMMISSION

DISTRIBUTION OF PINK SALMON SPAWNERS IN THE THOMPSON RIVER SYSTEM IN 1977 WITH SPECIAL REFERENCE TO THE PROPOSED HAT CREEK THERMAL PLANT WATER SUPPLY FACILITIES

INTRODUCTION

The object of this project was to record the numbers and distribution of adult pink salmon in the Thompson River system in 1977 with detailed observations in areas that might be affected by the proposed Hat Creek thermal plant water supply facilities in Thompson and Bonaparte Rivers. The proposed initial stage of the Hat Creek thermal power plant proposed by B.C. Hydro would involve a 65 cfs water supply intake in Thompson River, with some water from the supply pipeline extending between this intake and the power plant being discharged into Bonaparte River when required for drainage.

Observations were made of spawning pink salmon in the Thompson River system between Kamloops Lake and Spences Bridge, including the Nicola, Bonaparte, and Deadman Rivers. Approximately 0.58% of the migrating adult pink salmon were captured, tagged and released in the Thompson Canyon approximately 10 miles downstream of Spences Bridge. The carcasses of the spawned-out fish that were swept onto the river banks were counted including the number of tagged fish recovered, so that the total spawning population could be determined.

Detailed observations were made in an 8000-ft reach of Thompson River near the proposed water intake, as well as in a 1.5-mile reach of Bonaparte River between the mouth and an impassable falls.

METHODS

Before reaching the spawning areas upstream of Lytton, 5676 pink salmon were tagged and released. After the salmon had spawned and had begun dying, the dead were recovered from both banks of the river and a record was kept by sex and by area of recovery of both the numbers of dead and the numbers of tags recovered. An estimate of the pink salmon

population in the Thompson River system was then calculated based on the proportion of tags recovered and the dead recovery data. The spawning populations in the Nicola, Bonaparte and Deadman Rivers were estimated by applying the usual index factor of 2.6 to the maximum live count plus the cumulative dead recovery up to the date of the maximum live count. The sex ratio of pink salmon in Nicola, Bonaparte and Deadman Rivers was assumed to be the same as that in Thompson River.

Observations of the numbers and distribution of spawning pink salmon were made from both banks of the Thompson River over the full length of the spawning area. Where visibility was poor, observations were recorded from a boat. Observations in the 8000-ft reach in which the proposed water intake would be located were made from a boat as well as from both banks. Detailed live counts in the proposed water intake area and in Bonaparte River between the mouth and falls were completed every four days during the pink salmon spawning period.

An underwater survey was carried out in the proposed water intake area by two divers employed by B. C. Hydro, and at the same time a live count was completed in the standard manner. The divers had to make three drifts of the 8000-ft reach in order to cover the full width of the river.

As well as completing a live count, the divers photographed the bottom features in a section of the river bottom 60 ft from shore and 150 ft long at proposed water intake Site D. Detailed underwater observations were also made in a spawning area approximately 0.5 miles upstream of Site D. The divers reported directly to B. C. Hydro concerning composition of materials on the river bottom but their fish observations are given in this report.

TIMING OF THE PINK SALMON RUN

Migration of pink salmon into the Thompson River system began about September 18 and continued until approximately October 15. Arrival times in 1977 were 5 to 7 days earlier than usual. Spawning was first observed on September 25 and continued until October 22. The peak of spawning in the Thompson and Bonaparte Rivers occurred October 3-6, while

the peak of spawning in the Nicola and Deadman Rivers occurred October 3-10. The pinks began dying on September 27 and dying continued until October 27, the peak of dying occurring October 8-18.

Tagging procedures were carried out September 20-29 during the peak of the pink migration through Thompson Canyon, and the live count observations began on October 3 at the start of the peak of spawning and were continued until October 26.

NUMBER OF PINK SALMON SPAWNERS

The pink salmon population in the Thompson River system was estimated to be 972,941, of which 971,157 spawned in Thompson River (TABLE 1). This population estimate was based on a dead recovery of 99,667 and a tag return of 576 of the 5676 tags put out. Males and females were enumerated separately and the sex ratio was determined to be 44.98% male and 55.02% female. The females were examined for egg retention and the success of spawning was determined to be 91.92%.

TABLE 1. Numbers of pink salmon spawners in the Thompson River system.

River	Maximum Live Count	Dead Recovery			Population Estimate		
		♂	♀	Total	♂	♀	Total
Thompson		36,253	63,075	99,328	436,859	534,298	971,157
Nicola	355	86	137	223	444*	544*	988**
Bonaparte	202	22	82	104	275*	336*	611**
Deadman	32	3	9	12	83*	102*	185**
Total spawning population							972,941

* Calculated using Thompson River sex ratio

** Calculated from maximum live count

Population estimates for the Nicola, Bonaparte, and Deadman Rivers were based on maximum live count and dead recovery data. The maximum

live count on Nicola River was 355, which occurred on October 7 and up to that date 25 dead had been recovered, giving a total population estimate of 988. In Bonaparte River, a maximum live count of 202 on October 5 plus 33 accumulated dead gave a population estimate of 611. A population estimate of 185 pink salmon in Deadman River was based on a dead recovery of 12, a live count of 32, plus 71 pinks known to have passed through a weir located half a mile upstream from the creek mouth. The sex ratio in all three rivers was assumed to be the same as in the Thompson. By counting the number of eggs retained in the carcasses of spawned females the success of spawning in Bonaparte, Nicola and Deadman Rivers was determined to be 96.34, 99.27 and 100.00%, respectively.

DISTRIBUTION OF SPAWNERS BASED ON DEAD RECOVERY

The numbers of dead recovered by area in the main Thompson River are shown in TABLE 2. Because female salmon do not move far from their redds after spawning, the distribution of spawners can be estimated from the numbers of dead females recovered in each area. The seven dead recovery areas in Thompson River and the three tributary spawning areas are shown in FIGURE 1. Approximately 80% of the female carcasses were recovered in Areas 1A, 1B and 2, which are upstream from the proposed water intake.

TABLE 2. Distribution of pink salmon carcasses recovered from both banks of Thompson River.

Area	Number of Dead Recovered			Distribution of females %
	♂	♀	Total	
1A	5,837	16,777	22,614	26.82
1B	9,122	14,834	23,956	23.71
2	10,469	18,156	28,625	29.02
3	3,945	5,858	9,803	9.37
4	5,583	5,742	11,325	9.18
5	725	984	1,709	1.57
6	21	208	419	0.33
Total	35,892	62,559	98,451	100.00

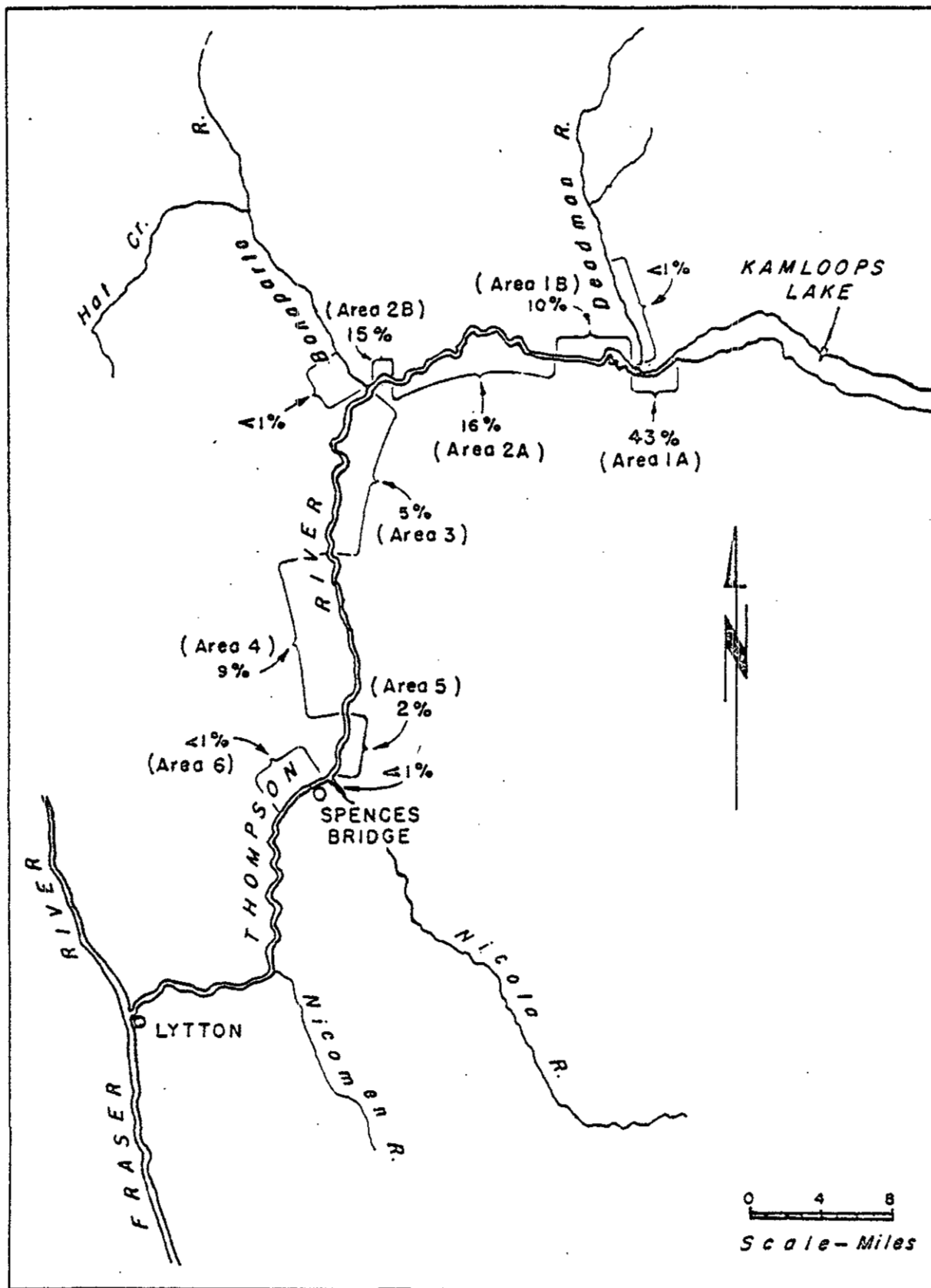


FIGURE 1. - The Thompson River system showing the distribution of spawning pink salmon, estimated on the basis of live counts.

DISTRIBUTION OF SPAWNERS BASED ON LIVE COUNTS

The distribution of spawners was also determined on the basis of the numbers of live fish seen in each of the areas shown in FIGURE 1. The live count method of determining spawner distribution may have been more accurate than the dead recovery method since the water was clear enough to see fish spawning throughout the length of the river, and therefore the error introduced by downstream drift of spawned-out fish was avoided. The proportion of the total number of spawners utilizing each of the designated areas and the tributary streams is shown in FIGURE 1 and the numbers of spawners seen in each of the areas in Thompson River are given in TABLE 3.

Heaviest spawning density occurred between Kamloops Lake and Deadman River. A heavy concentration of spawning was observed 0.5 mile upstream of the proposed water intake site (FIGURE 2).



FIGURE 2. Photograph of part of the pink salmon spawning area approximately 2000 ft upstream from the proposed Hat Creek water intake site.

TABLE 3. Maximum live counts of pink salmon spawning in designated areas of Thompson River.

Area	Live Count	% of Total Live Count
1A	9,738	43.2
1B	2,300	10.2
2	6,984	31.0
3	1,100	4.9
4	2,060	9.1
5	350	1.6
6	-	-

Light spawning was recorded in the mouth of Nicola and Deadman Rivers as well as in Bonaparte River upstream to the falls. Together these three rivers accounted for less than 1% of the total number of spawning pink salmon observed in the Thompson River system.

The live counts indicated that approximately 84% of the spawning occurred between Kamloops Lake and 0.5 mile upstream from Bonaparte River. As previously indicated, 80% of the female carcasses were recovered in this area. It is therefore concluded that approximately 82% of the spawning occurred upstream from the proposed water intake sites.

OBSERVATIONS IN THE WATER INTAKE AREA

Five alternate sites for the water intake structure were being considered by B. C. Hydro at the time of this study and these are shown in FIGURE 3, in the locations given in B. C. Hydro drawings D4191-SK54 and SK55. Each pink salmon observed in the 8000-ft reach of the Thompson River that might be affected by the water intake facilities is shown as a dot on FIGURE 3.

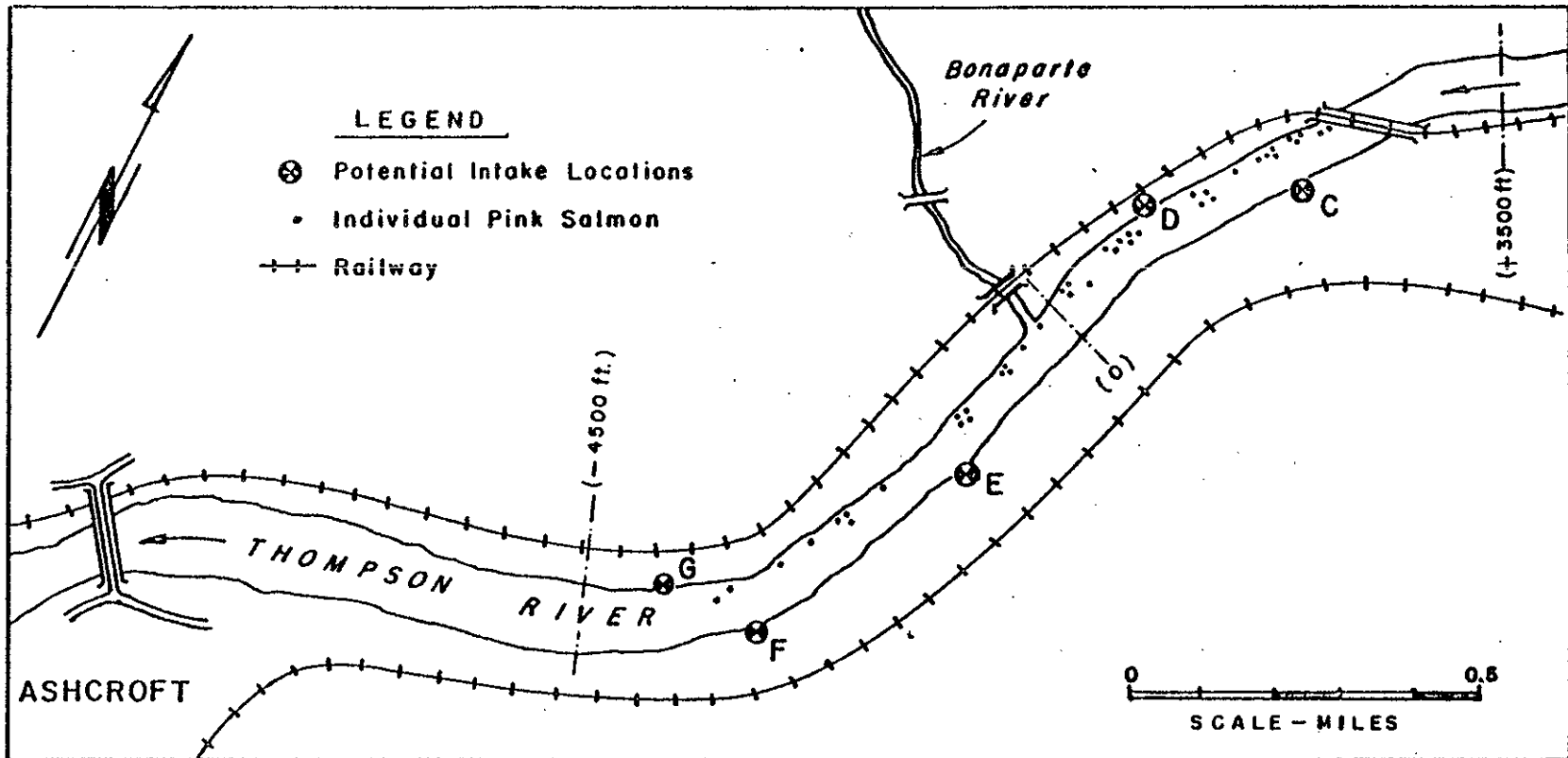


FIGURE 3 - The distribution of pink salmon observed from October 3 - 26 near the proposed Hat Creek water supply intake. Each dot represents a single pink salmon observed.

Only 41 pink salmon were recorded in this 8000-ft reach, including those seen by the divers. Of these 41 fish, 34 were spawned males, 4 were spawned females and 3 were migrating fish. There was no evidence that pink salmon spawned in this reach of the river or that migrating fish accumulated in schools or rested in the region of the water intake sites under consideration.

Spawning conditions in the study area were poor. As indicated in FIGURES 4 and 5, the water was deep and slow flowing and in many areas the river bottom was covered with boulders 1-2 ft in diameter.

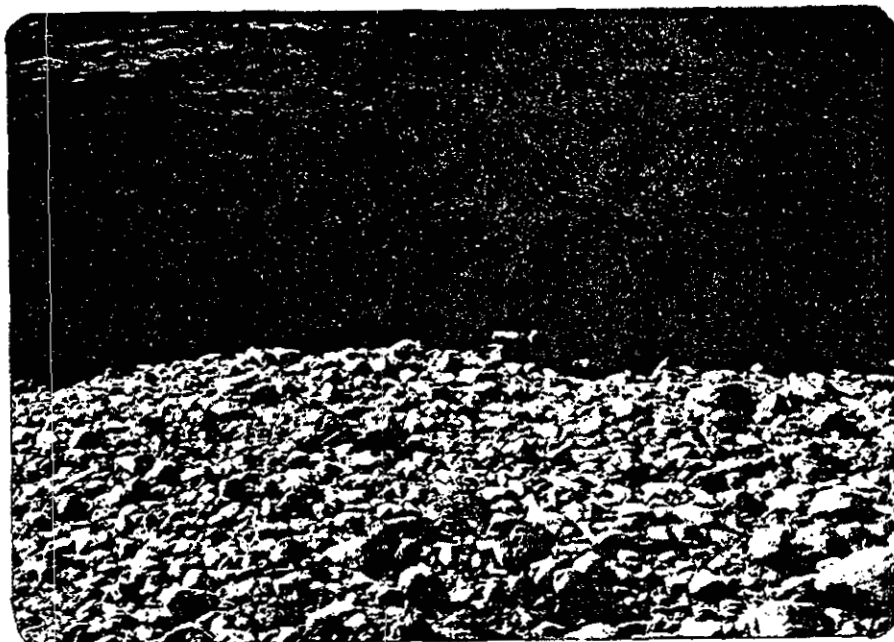


FIGURE 4. River conditions at proposed water intake Site D.

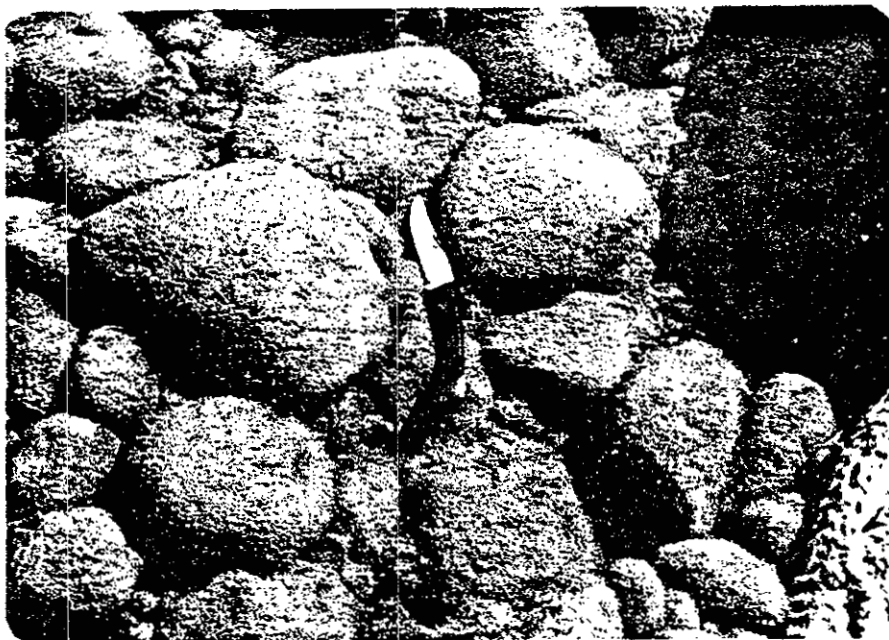


FIGURE 5. Riverbed material at proposed water intake Site D.

PINK SALMON DISTRIBUTION IN BONAPARTE RIVER

The distribution of migrating, holding, and spawning pink salmon in Bonaparte River is shown in FIGURE 6 . Values shown on the drawing are numbers of fish recorded during a live count made at the peak of spawning.

Only about 0.1% of the pink salmon spawners in the Thompson River system utilized Bonaparte River. Spawning was observed from the Canadian National Railway bridge at the mouth upstream to the falls, a distance of 1.5 miles. No fish were observed upstream of the falls but the water velocities, depths and riverbed materials appeared excellent for pink salmon spawning. Most of the area of Bonaparte River downstream

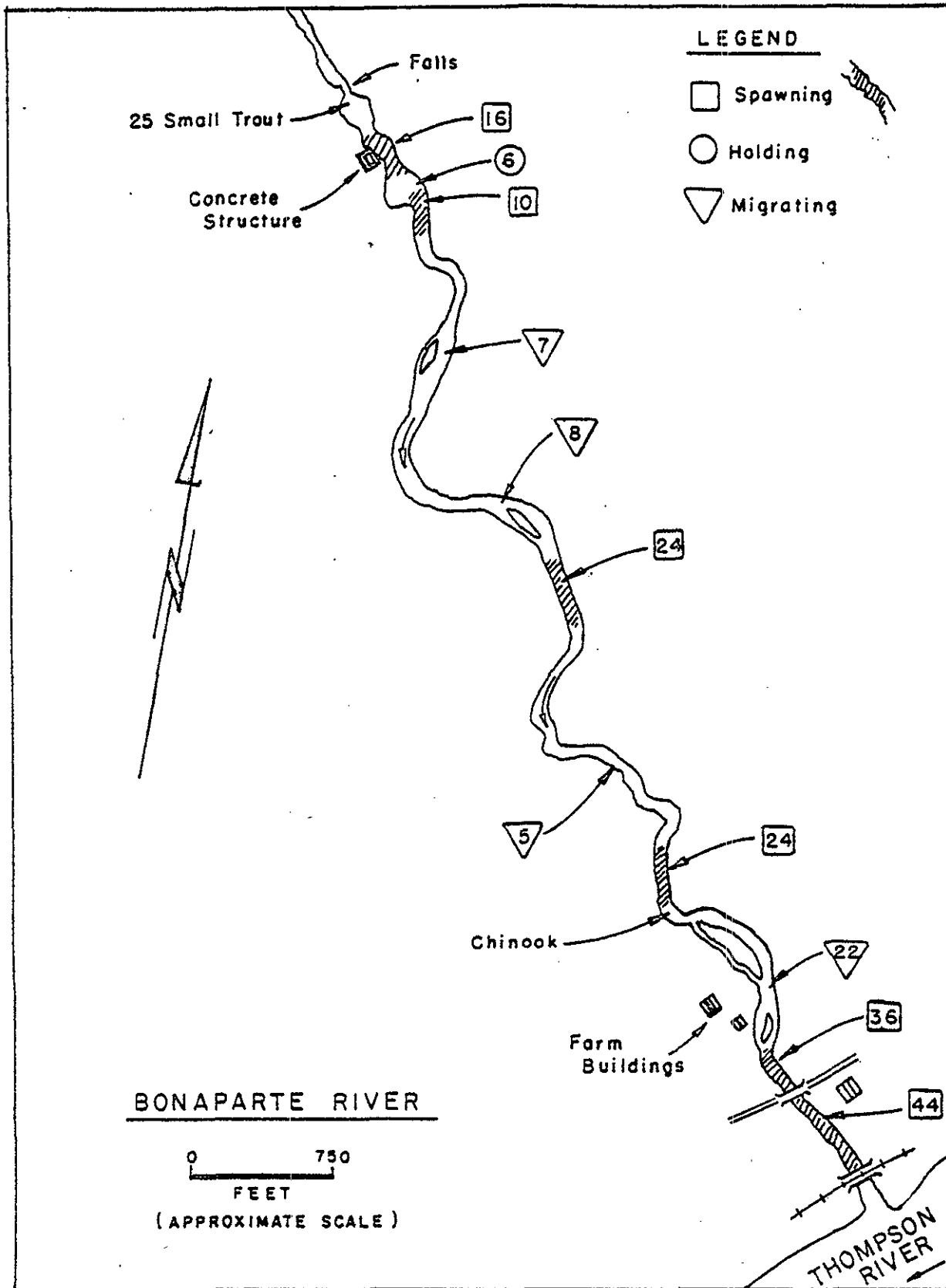


FIGURE 6 - The distribution of migrating, holding, and spawning pink salmon in Bonaparte River. Values given are numbers of live pink salmon observed during the peak of spawning.

of the falls was composed of 8-10 in. boulders with silt collecting between the boulders. Approximately 20% of the stream below the falls exhibited good spawning conditions with 2-3 in. gravel (FIGURE 7) and heavy spawning occurred in these areas. The area upstream from the falls appeared capable of supporting very large numbers of spawners.



FIGURE 7. Typical gravel conditions in Bonaparte River in areas utilized by pink salmon spawners below the falls.

OBSERVATIONS BY DIVERS

The divers observed 23 pink salmon, 8 suckers and 1 Rocky Mountain Whitefish in the region of the proposed water intake sites. All the pink salmon observed were spawned males. The divers did not see any evidence of spawning, holding or migrating salmon in the 8000-ft reach observed. Spawning conditions in this reach were reported by the divers as being very poor. The river bottom was covered with large boulders and silt was collecting on the downstream

side of these boulders. The water velocity along the bottom was estimated to be only 0.3 fps.

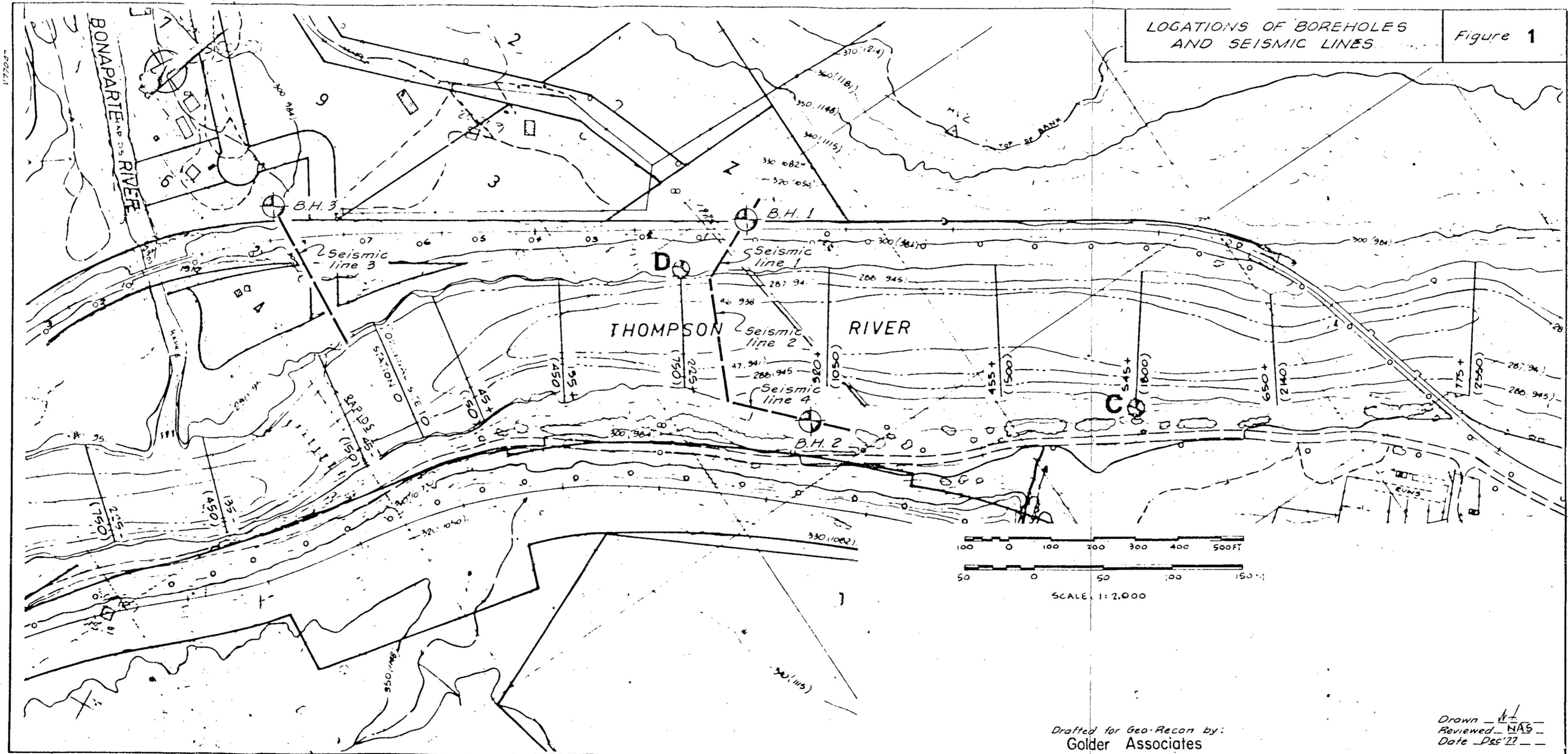
CONCLUSIONS

1. Approximately 82% of the 972,941 pink salmon spawning in the Thompson River system in 1977 utilized areas upstream from the proposed water intake site.
2. No pink salmon spawning occurred in the region of the proposed water intake sites. Conditions for spawning in this area were poor owing to low velocities and scarcity of suitable spawning gravel.
3. There was no evidence that the area which might be affected by the proposed water intake facilities had any biological significance as a holding area for migrating adult pink salmon.
4. Approximately 611 pink salmon spawned in Bonaparte River. The spawning occurred in concentrated areas between the mouth and falls. The falls prevented the fish from migrating further upstream, where velocities and gravel conditions appeared to be excellent for pink salmon spawning.

December, 1977.

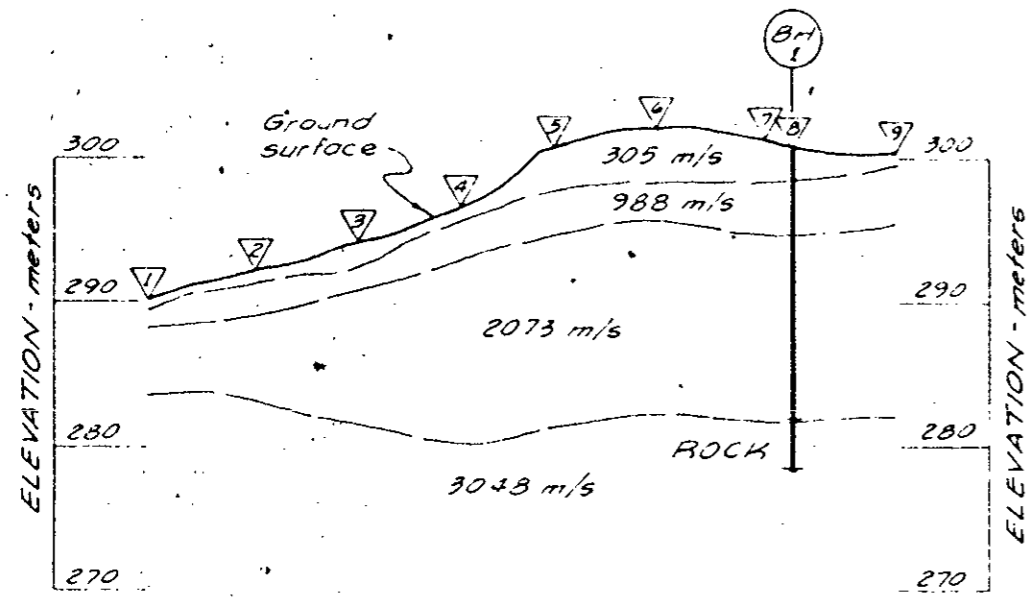
Ken Morton

LOCATIONS OF BOREHOLES AND SEISMIC LINES Figure 1

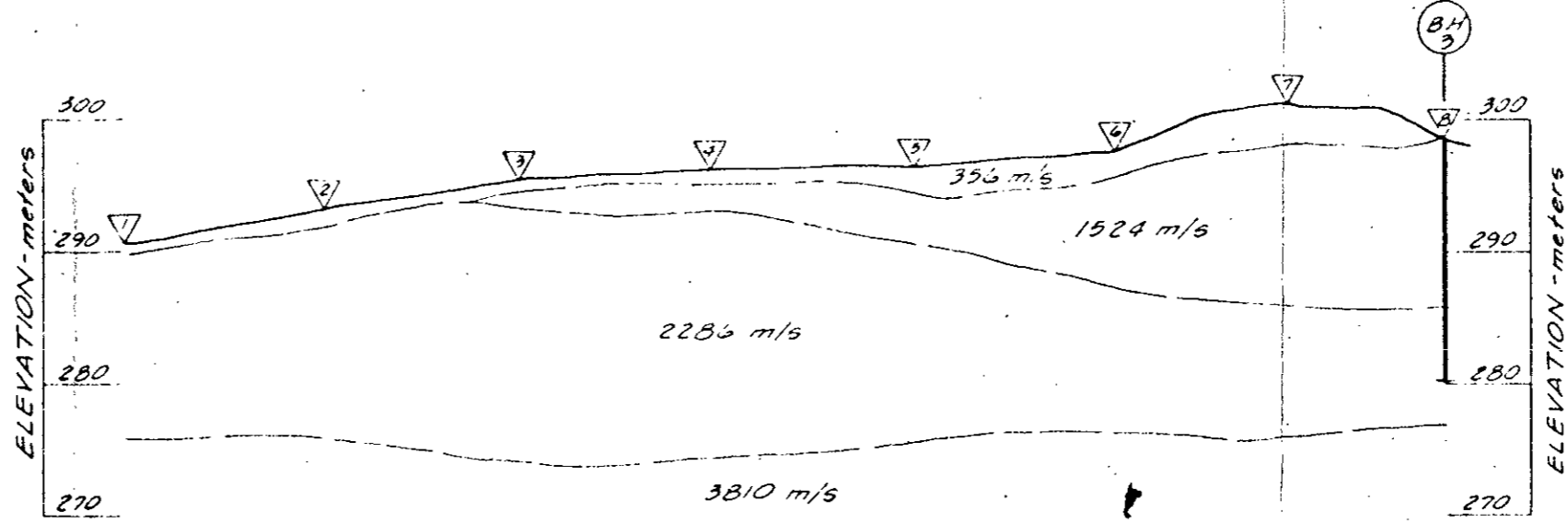


Drafted for Geo-Recon by:
Golder Associates

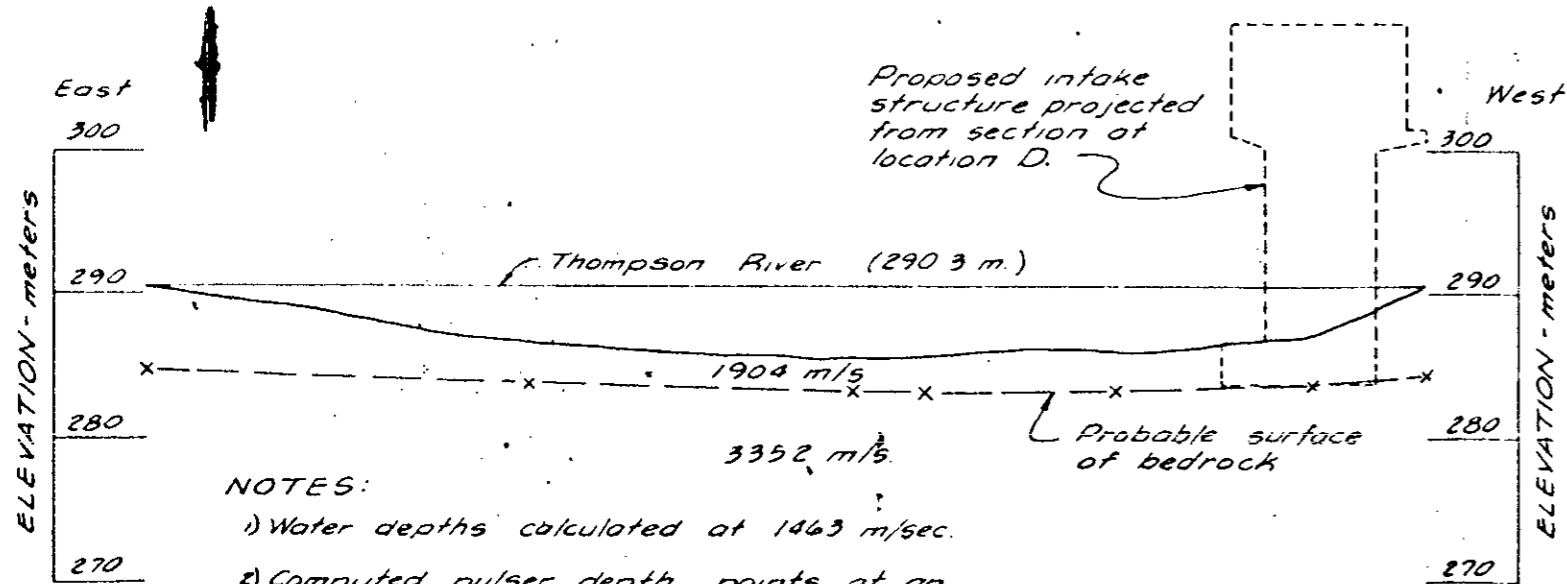
Drawn MA
Reviewed NAS
Date Dec '77



SEISMIC LINE 1
(Looking Downstream)

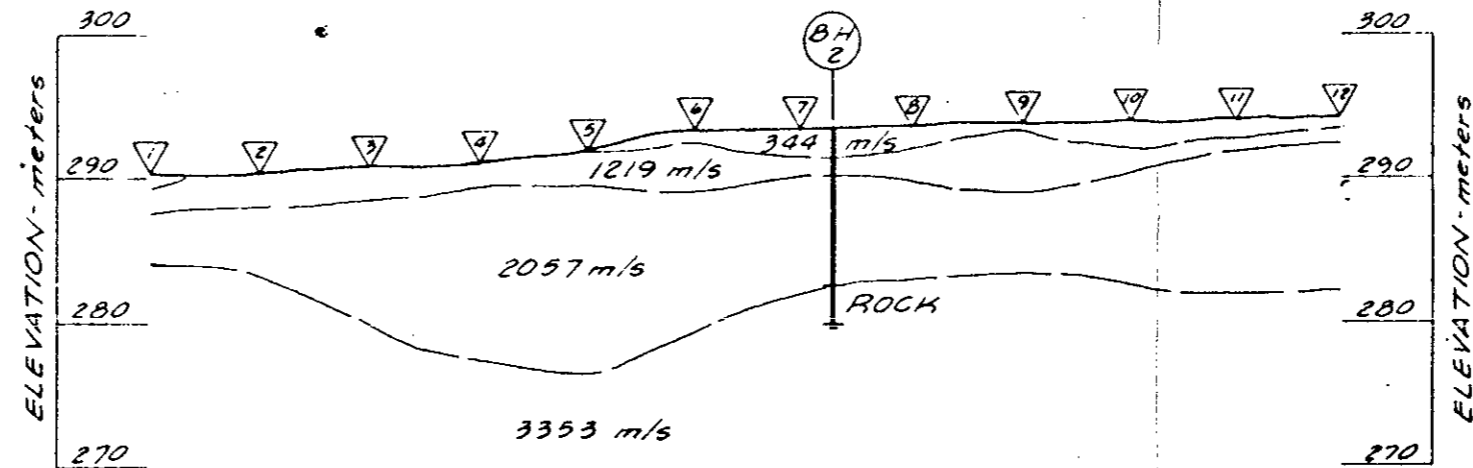


SEISMIC LINE 3
(Looking Downstream)



NOTES:
 1) Water depths calculated at 1463 m/sec.
 2) Computed pulser depth points at an overburden velocity of 1900 m/sec

SEISMIC LINE 2
(Looking Downstream)



SEISMIC LINE 4
(Looking towards River)

LEGEND

- Denotes borehole in section
- Denotes geophone locations in section.

NOTES

- a) For locations of boreholes and seismic lines see Figure 1
- b) Seismic line 2 is a river cross-section located approx. 20m upstream of intake location 'D' - see text.

Scale - 1:500

Drafted for Geo-Recon by:
Golder Associates

Drawn AL
 Reviewed NAS
 Date Dec 77

177087

REPORT TO
SANDWELL AND COMPANY
ON
HAT CREEK WATER SUPPLY
THOMPSON RIVER INTAKE
GEOTECHNICAL SURVEY OF
INTAKE SITE 10 - LOCATION D

ASHCROFT BRITISH COLUMBIA

December, 1977

V77047

Golder Associates

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3.0 SEISMIC INVESTIGATION	3
4.0 BEDROCK PROFILE	4

Figure 1 - Locations of Boreholes and Seismic Lines

Figure 2 - Seismic Velocity Profiles

Appendix A - Records of Boreholes

Appendix B - Report by Geo-Recon Explorations Ltd.

1.0 INTRODUCTION

This report summarizes the results of drilling and surface exploration carried out for preliminary engineering purposes in the vicinity of intake site 10- location 'D' on the Thompson River just upstream of the confluence with the Bonaparte River. Proposals had been invited for drilling directly over the site of the intake from a barge moored in the river. This proved to be too costly for present needs however, and it was decided to drill on both banks of the river on the line of the intake, and to supplement the findings by subbottom seismic profiling techniques.

2.0 DRILLING INVESTIGATION

Three boreholes, numbers 1 to 3, were sunk on shore using a Becker truck mounted hammer drill employing 143 mm (5-5/8 inch) OD casing. Basically the drill operates by advancing casing into the ground with a pile driving hammer mounted on the rig boom, and forcing the sample return up the inside of the casing by air under pressure to the bit via an annulus in the wall of the casing. Despite the apparent crudity of the soil sampling, the rig is well suited to exploration of granular deposits; in general, the grading of the samples returned to the surface are surprisingly representative of the deposits in situ. The particular machine used in this work was fitted in addition with a rotary rock coring attachment, and it was used to drill bedrock and hard glacial till.

Drilling was carried out on September 26-29, 1977 and it was fully supervised by a field technician from Golder Brawner & Associates Ltd. Boreholes 1 and 2 were sunk for the purpose of discovering the location of

bedrock on the line of the intake site, while Borehole 3 was sunk for the evaluation by others of a possible radial well. The location of the borings is shown on the attached Figure 1 and the detailed borehole records are presented in Appendix A. Borehole Nos. 1 and 2 were moved somewhat upstream of intake location 'D' in order to find a level piece of ground on which to set up the drill rig. Similarly, the seismic line 2 discussed in Section 3 below was moved a short distance upstream. In reading the borehole records the following should be noted:-

- a) The sample descriptions are detailed in lower case letters, but the essential constituent of each deposit is written in upper case letters.
- b) The casing driving resistance is a useful indication of the compactness of the deposit through which the drilling bit is advancing. The resistance is measured as the number of hammer blows per 0.305 m (1.0 ft.) and the values are recorded continuously.
- c) Occasional Standard Penetration Tests were carried out and the results - 'N' values - are shown as the conventional Imperial units of blows/foot. The sample recovered by this means is identified by the letter 'S' in the sample type column on the log. Samples identified as 'C' were disturbed samples taken from a cyclone, which was attached to the head of the casing to catch the sample return.
- d) Samples of water were taken where encountered in the boreholes. These were dispatched to Sandwell and Company for analysis.

- e) A rough check on ground permeability was carried out by performing simple rising head tests in the drillhole casing. Basically, the driller flushes the water out of the casing using air pressure, the drill is then stopped and the technician records the rise in the water level with time. The coefficient of permeability is then calculated from this data, together with the diameter of the inside of the casing and the height to standing water table, which is found on completion of drilling. The coefficient of permeability thus deduced, is a mean value between the horizontal and the vertical values. More granular deposits such as were encountered in parts of this drilling usually have permeabilities in the horizontal and vertical directions differing by at least one order of magnitude.

3.0 SEISMIC INVESTIGATION

Seismic investigations were undertaken during the period November 15 to November 19, 1977, by Geo-Recon Explorations Ltd., Seattle, Washington. Russell Hillman of Golder Brawner & Associates Ltd. worked in conjunction with Geo-Recon field personnel as an integral part of the field crew. Three land seismic refraction lines and one overwater reflection line were undertaken in this period. The detailed results of the survey are presented in Appendix B of this report.


4.0 BEDROCK PROFILE

The shaley mudstone bedrock surface within the area is relatively flat lying. The three seismic traverses near to intake Location D indicate that the bedrock has a fairly consistent elevation of approximately 284 metres. At the location of the intake at Point D we would therefore expect the surface of the bedrock to be about 3 m (10 ft.) below the bed of the river. The material above the rock in the river bed is likely to be sand, gravel, cobbles and boulders. On the left bank roughly 150 metres downstream from Borehole No. 2, there is a bedrock outcrop, which was visible at low water level in November at the time of the seismic survey. This indicates that the river bed at this location is rock controlled. The bedrock surface along Seismic Line 3 appears to be slightly deeper which may indicate a shallow dip to the west.

Based on the rock encountered in the drillings and on the measured seismic velocities it is our opinion that the bedrock could not be ripped, and would require light blasting for removal. Furthermore, the rock is too hard for penetration by sheet piling during cofferdam construction, and it would not therefore be possible to seal the excavation from water by this means.

Yours very truly

GOLDER BRAWNER & ASSOCIATES LTD.



Per: N.R. McCammon, P. Eng.



N.A. Skermer, P. Eng.

NRM/NAS:rme

V77047

Golder Associates

APPENDIX A

Records of Boreholes

RECORD OF BOREHOLE /

LOCATION (See Figure 1)

BORING DATE *Sept. 26, 27, 1977*

BOREHOLE TYPE *Becker hammer drill with coring facility.*

BOREHOLE DIAMETER *139.7 mm.*

Elevation top of borehole - *301.02 meters, 987.6 feet.*

Project No. *177047*

SOIL PROFILE		STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT 'N' value	CORE RECOVERY	CASING DRIVING RESISTANCE blows/foot for 139.7 mm drill casing			REMARKS AND ADDITIONAL TESTING
DEPTH	DESCRIPTION						100	200	300	
m. ft.										
Ground surface										
0.0'			1	C						
1.90 m 6.5'			2	S	132 12"					
			3	S	138 12"					
5.79 m 19.0'			4	S	82 12"					
8.23 m 27.0'			5	C						
9.45 m 31.0'			6	C	40 2"					
9.80 m 32.5'			7	S	111 11"					
10.5 m 34.5'			8	S						
			9	S						
			10	S	102 8"					
			11	C						
			12	C						
			13	S	102 9"					
18.59 m 61.0'			1	C						
			2	C						
			3	C						
			4	C						
21.94 m 72.0'			5	C						
End of borehole.										

Water level 5.97 m. (19.6') below ground surface Sept 28/77.

Water sample taken at 16.30 m (53.5')

Permeability
Rising head test at 16.76 m (55') -
k = 3.85 x 10⁻⁶ m/s
(7.6 x 10⁻⁴ ft./min.)

VERTICAL SCALE 1 inch to 10 feet 1:120 metric

Golder Associates

Logged by W.R.

DRAWN *W.R.*
CHECKED *NAS*

RECORD OF BOREHOLE 2

LOCATION (See Figure 1)

BORING DATE *Sept. 26, 1977*

BOREHOLE TYPE *Becker hammer drill with coring facility.*

BOREHOLE DIAMETER *139.7 mm.*

Elevation top of borehole - *293.16 meters, 961.8 feet.*

Project No. *V77047*

DEPTH		DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT 'N' value	CORE RECOVERY	CASING DRIVING RESISTANCE blows/foot for 139.7 mm drill casing			REMARKS AND ADDITIONAL TESTING
m.	ft.							100	200	300	
Ground surface											
<i>0.0'</i>	<i>0.91 m</i>	Compact grey cobbles, small boulders and SANDY GRAVEL									
<i>3.0'</i>	<i>2.44 m</i>	Very dense brown silty fine sand and gravel, trace clay, TILL.		<i>1</i>	<i>C</i>						Water level 1.52 m (5') below ground surface Sept 28/77.
<i>8.0'</i>	<i>4.27 m</i>	Very dense grey silty fine sand and gravel, trace clay, occasional small cobbles, TILL.		<i>2</i>	<i>C</i>						
<i>14.0'</i>	<i>6.70 m</i>	Very dense grey GRAVELLY CLAY AND SILT, some fine sand, occasional small boulders.		<i>3</i>	<i>S</i>	<i>75</i> <i>78"</i>					<i>644</i>
				<i>4</i>	<i>C</i>						
<i>22.0'</i>	<i>10.82 m</i>	Very dense grey silty fine sand and gravel, some med. and coarse sand trace of clay, occasional cobbles, occasional layer of gravelly clay and silt, some fine sand, TILL		<i>5</i>	<i>S</i>	<i>120</i> <i>10"</i>					
				<i>6</i>	<i>C</i>						Water at till bedrock contact. <i>373 1/4</i>
<i>35.5'</i>	<i>13.87 m</i>	SHALE BEDROCK		<i>7</i>	<i>S</i>	<i>116</i> <i>12"</i>					
				<i>8</i>	<i>C</i>						Water sample taken at 10.82 m (35.5')
<i>45.5'</i>	<i>13.87 m</i>	End of borehole									

VERTICAL SCALE
1 inch to 10 feet

1:120 metric

Golder Associates

Logged by W.R.

DRAWN *W.R.*
CHECKED *NAS*

RECORD OF BOREHOLE 3

LOCATION (See Figure 1)

BORING DATE *Sept. 28/29, 1977*

BOREHOLE TYPE *Becker hammer drill with coring facility.*

BOREHOLE DIAMETER *139.7 mm.*

Elevation top of borehole - *297.64 meters, 976.5 feet.*

Project No. *Y770-97*

SOIL PROFILE		STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT 'N' value	CORE RECOVERY	CASING DRIVING RESISTANCE blows/foot for 139.7 mm drill casing			REMARKS AND ADDITIONAL TESTING
DEPTH	DESCRIPTION						100	200	300	
0.0'	Ground surface									
8.53 m 28.0'	Compact becoming very dense grey fine, medium & coarse SANDY GRAVEL, trace of silt occasional small boulder, lens of medium and coarse sand.		1	C						
			2	S	85 6'					
			3	C						
			4	S	81 6'					
			5	C						
			6	S	72 1'					
9.49 m 31.15'	Very dense grey silty fine SAND & GRAVEL, some medium and coarse sand, trace clay.		7	C					Stratum from 9.42 - 9.49 m (30.9 - 31.15') water bearing.	
			8	S	84 12'					
11.95 m 39.2'	Very dense grey TILL, silty fine sand and gravel some medium and coarse sand, trace of clay, occasional cobble.		9	C					Water level 9.94 m (32.6') below ground surface 8:00 am, Sept 29/77. Water sample taken at 9.45 m. (31').	
			10	S	30 6'					
18.29 m 60.0'	More cobbles and boulders below 11.89 m. (39 ft.) Cored NX 14.02 - 18.29 m. (46 - 60 ft.)		11	C					410 457 790 490 416 424 Permeability No value calculated since final water level undetermined.	
			NX							
			12	R.C.						
			NX		Cored					
			13	R.C.						
					Poor					
18.29 m 60.0'	End of borehole.									

VERTICAL SCALE 1:120 metric
1 inch to 10 feet

Golder Associates

Logged by W.R.

DRAWN *H.J.*
CHECKED NAS

APPENDIX B

Report by Geo-Recon Explorations Ltd.

GEO RECON

Explorations Ltd.

G-132

December 9, 1977

Golder Brawner & Associates Ltd.
224 West 8th Ave.
Vancouver, B.C. V5Y 1N5

GEOPHYSICAL SURVEY - THOMPSON RIVER, COOLING WATER SUPPLY, HAT CREEK PROJECT

This letter and Golder Associates Figure 2 presents the results of a land seismic refraction and the overwater acoustic reflection survey, conducted by your authorization, at the proposed location of the cooling water supply intake for the Hat Creek project, on the Thompson River near Ashcroft, B.C. The purpose of the survey was to delineate the bedrock surface between the land borings and the edge of the river and across the river along pre-designated line locations. The field phase of the survey was conducted between November 17 and 20, 1977. Mr. Nigel Skermer of your office supervised the field phase of the survey and Mr. Russell Hillman from your office assisted in field operations.

The survey consisted of four seismic lines. The seismic velocity profiles, as computed from the field-gathered data, are shown on Figure 2. The seismic velocity profile interpretation for Seismic Lines 1, 3 and 4 was from data obtained with a conventional SIE RS49, 24-channel seismograph. The seismic velocity profile for Seismic Line 2 was developed from data gathered by an overwater acoustic reflection profiling device (Pulser).

For the velocity profile of Seismic Line 2, the computed pulser depth points were obtained as the pulser crossed the axis of the seismic line. The acoustic profiler was operated parallel to the flow of the river current, from downstream of the seismic line, up to and across the axis of the seismic line. At the moment of crossing the seismic line, a location was determined by a transit and the record was marked. These marked points on the

1661 West 8th

1608 X18P01100X2W4M0KX1N02TX1AN03M0KX1B1K1H X06L10K
X18P100X1H0KX095-12XX1E1KX04-50X1S0KX1W1X01H-023-0058

(604) 736-7361

acoustic records were plotted to produce the river profile.

The geophysical information in this report is based upon geophysical measurements made by generally accepted methods and field procedures and our interpretation of these data. These results are interpretive in nature and are considered to be a reasonably accurate presentation of existing conditions within the limitations of the methods employed.

The land seismic refraction lines exhibited four-layer velocity profiles. These layers, with their velocity ranges are listed below:

<u>Layer</u>	<u>Velocity Range</u>
Top layer	305 - 365 m/s (1000 - 1200 fps)
Second layer	975 - 1524 m/s (3200 - 5000 fps)
Third layer	1900 - 2286 m/s (6250 - 7500 fps)
Fourth layer	3048 - 3810 m/s (10,000 - 12,500 fps)

A thin layer of boulders not included in the above list or shown on Figure 2 is present on the surface on some of the seismic lines. This boulder pavement appears to be shallow (0.3 to 0.6 meters) and has an apparent velocity of 3261 m/s (10,700 fps). This is essentially a horizontal velocity and is markedly different than the vertical velocity through boulder pavement.

The first layer shown above is typical of surficial materials. Correlation of remaining layers to the materials found in the boreholes located along the seismic lines indicate that the second layer is either weathered till and/or sand and gravel, the third layer is till and the fourth layer is bedrock.

Figure 2, Seismic Line 2, shows the derived seismic velocity profile across the Thompson River. Water depths were plotted utilizing a water velocity of 1463 m/s (4800 fps). A two-layer subbottom was noted on the records. Correlation to the land seismic velocity profiles indicates that the top of the second

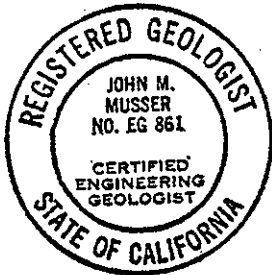
layer on Seismic Line 2 is equivalent to the top of the fourth layer (bedrock) on Seismic Lines 1 and 4. Subbottom depths were computed at 1904 m/s (6250 fps).

The accuracy of the seismic velocity profiles is considered to be within reasonable limits for a preliminary siting of the intake structure. However, it is recommended that an overwater seismic refraction profile be developed between the shore and the proposed intake structure location to verify the accuracy of the acoustic profiling technique with a more precise definition of subbottom material velocity. Also, a shore line upstream and downstream at the intake location could be run to define the bedrock configuration in a direction normal to the proposed center line of the intake structure.

Please advise us if you have any questions regarding this report or if we may be of further service.

Sincerely,

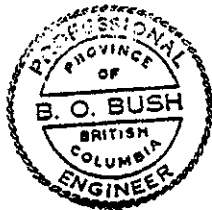
GEO-RECON EXPLORATIONS



BY

John M. Musser

John M. Musser, CEG
Senior Geologist



BY

Boyd O. Bush

Boyd O. Bush, P. Eng.
Principal Geologist

JMM:lr

by Rpm

Prepared for:

SANDWELL AND COMPANY LTD.

Prepared by:

NORTHWEST HYDRAULIC CONSULTANTS LTD.

Edmonton - Vancouver - Calgary

December 1977

HAT CREEK PROJECT
COOLING WATER SUPPLY INTAKE
HYDRAULIC MODEL OF INTAKE

SUMMARY

The cooling water supply intake for the proposed Hat Creek Project has been designed as a pier intake located in the Thompson River. Concern has been expressed regarding the effect of the intake on the survival of salmon fry hatching in upstream spawning areas. The fry could be entrapped between the travelling screens protecting the pumps from small trash and the curtain wall and trashracks that protect against larger trash and ice. A concept was developed to create a bypass channel that would accept river water and create a sweeping velocity between the screens and curtain wall/trashracks to carry the fry back to the river. This report represents the results of a study using a 1:40 scale hydraulic model to develop this concept and demonstrate it to be a technically feasible solution to the problem. The study resulted in an intake arrangement that provided an acceptable sweeping flow for all foreseeable operating situations. Also investigated were sedimentation and trash accumulation problems and specific recommendations were made relative to these problems.

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northwest hydraulic consultants ltd.

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1. INTRODUCTION

In Western Canada, water supply intakes which take water directly from rivers have unique operational problems. They must perform satisfactorily during periods of sustained sub-zero temperatures and during periods when considerable debris or ice are being transported. Consequently, the trend has been to utilize a curtain wall and trashracks located in front of the pump bays, while the pumps are further protected from intake of foreign solids by a fine screen. The curtain wall extends below the design low water surface so that, together with the pump housing superstructure, it creates an environment which is protected from the damaging action of extreme cold temperatures. However, these intakes do not provide for a river flow along the screen face and fish can, therefore, become trapped between the curtain wall/trashracks and screens.

Recognizing this potential fish problem in their design of an intake for water supply to the proposed Hat Creek Project for the British Columbia Hydro and Power Authority (B.C. Hydro), the engineering consultant, Sandwell and Company (Sandwell) explored methods of preventing fish entrapment. A conceptual design to create a downstream flushing flow between the curtain wall/trashracks and the finer screens was subsequently recommended. As this concept was basically unproven, Sandwell requested that NHCL design, construct and test a hydraulic model to verify the concept.

In a letter to Sandwell, dated 27 May 1977, NHCL proposed to assess the hydraulic performance of flow between the curtain wall/trashracks and the (finer) travelling screens by testing a 1:40 scale model. The model, constructed in a flume, would be conceptual in nature, but would be designed using parameters (eg. elevations, water depths, etc.) from the preferred Site D⁽⁴⁾*; in this way, results regarding flow between the curtain wall/trashracks and travelling screens could generally be transferred to other sites.

Sandwell, in a letter dated 1 June 1977, authorized NHCL to proceed with the model study as proposed. Prior to model construction, NHCL provided advice to Sandwell on the hydraulic design of the intake, and the resulting prototype design provided a basis for the model design.

As the preliminary engineering for the water intake would parallel the hydraulic model study, and because design changes might be required to assure an acceptable flow between curtain wall and travelling screens, two key items were agreed upon at the outset of the study:

- (1) The model would be constructed in sections so as to facilitate quick design changes by interchanging smaller components.
- (2) Sandwell would regularly review model progress to ensure compatibility with the overall preliminary engineering phase.

NHCL personnel included Albert G. Mercer, P. Eng., who directed the study, Brent A. Berry, E.I.T., who conducted the testing program and Michael H. Okun, P. Eng. who provided

*Superscript numerals in brackets refer to references listed at the end of this report.

co-ordination with other phases of the water supply study.

2. PROJECT DESCRIPTION AND BACKGROUND

2.1 General

B.C. Hydro has been considering development of a thermal plant in the Hat Creek area of south-central British Columbia. Specifically, the site is located in the northeast corner of the Upper Hat Creek Valley, in the Trachyte Hills. An open pit coal mine constructed just west of the powerplant would provide fuel for the project. Cooling water for the powerplant will be provided from a reservoir with make-up cooling water supplied to the reservoir from outside the Hat Creek area.

Sandwell was commissioned to undertake feasibility studies to conceptually design the required 1600 l/s (25,000 U.S.G.P.M.) make-up cooling water supply system for the powerplant.⁽¹⁾⁽²⁾ On the basis of their studies, Sandwell determined that water could best be supplied by constructing a water intake on the Thompson River, and pumping through a buried pipeline to the reservoir. In addition, Sandwell determined⁽³⁾ that only a bank or pier-type intake would meet the technical requirements of the project. Accordingly, Sandwell adopted⁽¹⁾ a bank-type intake utilizing travelling screens, protected by a cutrain wall and trashracks.

2.2 Intake Site Characteristics

On the basis of Sandwell's engineering evaluation of the water supply system* it became apparent that the

*The evaluation included Sandwell's Conceptual Design,⁽¹⁾ subsequent collection of river engineering and geotechnical data, and ensuing discussions with B.C. Hydro, Golder Associates (the geotechnical consultants), and NHCL.

optimum area for a water intake was in a 2 km (1.25 mile) reach just upstream of Ashcroft. Subsequently, five sites were examined, two upstream of the Bonaparte River, and three downstream. Based on economic, geotechnical and river engineering considerations⁽⁴⁾⁽⁵⁾, Site D, located on the right bank of the Thompson River, about 380 metres (1250 feet) upstream of the Bonaparte River, was selected as the prime site. ✓ The site is about 500 metres (1600 feet) downstream of a Canadian National Railway bridge crossing and lies in a straight reach of river where both the bed and banks of the river are armored with cobbles and boulders. About 275 metres (900 feet) downstream, the river is controlled at lower flows by the presence of the Bonaparte Rapids.

Based on a review⁽⁴⁾ of available data, the following range and frequency of streamflows was found to be applicable:

Return Period	Flow in m ³ /s (cfs)	Comments
>1000 year flood	5665 (200,000)	
100 year flood	4530 (160,000)	Design high flow
10 year flood	3685 (130,000)	
2 year flood	2720 (96,000)	Approximate average annual flood.
100 year low-flow	115 (4,000)	Design low flow

At Site D, water levels and velocities for various discharges have previously been determined⁽⁴⁾ and are shown on Dwg. 2.2.1 herein.

Historically, the Thompson River near Ashcroft has not had sufficient quantities of ice cover (and subsequent ice jamming) to affect the intake design.⁽⁶⁾ There can be, however,

large quantities of frazil or slush ice passing the site, and the formation of anchor ice is also probable, depending on the severity of the winter.⁽⁶⁾ Trash, either floating on the surface or in the form of sunken water-logged wood, can be expected during the spring, summer and fall.

Since the Thompson River flows out of Kamloops Lake, located about 30 kilometers (19 miles) upstream of Site D, the suspended sediment load of the river is normally very low. In addition, the bank and bed armoring of cobbles and boulders, which extends all the way down from Kamloops Lake, leaves little source for sediment supply. (It is surmised that the bed of the Thompson River becomes mobile only during extreme flood events, if at all.) However, there are numerous steep bluffs of fine sands and silts bordering the river banks upstream of the site, which can contribute slugs of fine material to the stream, and this possibility has been considered in the intake design.

2.3 Conceptual Intake Design

2.3.1 Background

Sandwell's conceptual design, as recommended in the January 1977 Report,⁽¹⁾ is shown on Dwg. 2.3.1, which is a copy of their Dwg. D 4007/46. After considering the location of the face of the structure required to obtain sufficient depth of flow, and by considering the engineering economics of a bank versus pier type intake, Sandwell and NHCL selected a pier-type intake for Site D. It was agreed that except for site-specific elevations, this concept could be transferred to other adjacent sites, should Site D be dropped from consideration for any reason.

2.3.2 Design for Hydraulic Model

Dwg. 2.3.2, which is a copy of Sandwell's Dwg. D 4191-SK14*, August 1977, shows the prototype design concept from which the hydraulic model was designed. The intake consists of a pier containing 6 pump bays. Across the front of each bay is a conventional travelling screen and in front of each travelling screen is the curtain wall containing (for each screen) a low level entrance protected by a trashrack. Upstream of the pumping bays is a full-depth entrance, also protected by trashracks, that supplies the extra flow needed to maintain the sweeping velocity between the curtain wall/trashracks and travelling screens.

The specific dimensions of the structure were established having in mind criteria set by regulatory agencies. For instance, the approach velocity to the travelling screens was not to exceed 0.12 m/s (0.4 feet/sec) based on gross area below the water surface at low river flow. Also the sweeping velocity between the trashrack/curtain wall and the travelling screens was to be at least as great as 0.12 m/s (0.4 feet/sec) under all conditions of operation.

3. DESCRIPTION OF MODEL

3.1. Modelling Criteria

The primary objective of the model was to demonstrate that the concept of providing a sweeping velocity past the travelling screens under all conditions of flow was technically feasible and that any foreseeable problems could be resolved in the final design.

*Note: This drawing gives a minimum design water level of 289.00 m, which is 0.20 m lower than the 1 in 100 year minimum on Dwg. 2.2.1. This lower level was used in the model as the 1 in 100 year minimum water level is in the estimated portion of the stage discharge curve.

The model scale was selected considering laboratory flow and flume capabilities, Reynolds criteria for transition from laminar to turbulent flow, and conventional laboratory capabilities for velocity measurement. The scales chosen were:

Linear scale - 1:40

Velocity scale - $1:(40)^{\frac{1}{2}} = 1:6.32$

Time scale - 1:6.32

At low river flows, the velocities in the model were doubled to facilitate velocity measurement so that the velocity scale became 1:3.16 and the time scale 1:12.64.

3.2 Model Geometries

A flume measuring 0.61 m (24 inches) deep, 0.47 m (18.5 inches) wide and 5 m (16 feet) long was available for the test program. This width was not sufficient to permit an adequate flow past a model pier located in the middle of the flume, so it was effectively widened by constructing an embayment into the side of the flume. The model was inserted in the embayment as shown on Dwg. 3.2.1 so that only the flow around the front face of the pier was reproduced. The model was free to be moved around within the embayment so that observations could be made with the pier orientation slightly oblique to the flow. The bed of the river was reproduced with well rounded gravel that was stable under the highest flow tested. Recirculating discharge through the flume was provided by a pump with a capacity of 0.15 m³/s (5 cfs). A calibrated orifice in the delivery pipe was used for flow measurement. The intake flows were withdrawn from the model by copper pipes simulating the pumps. These operated under syphon action and delivered their flows back to the laboratory sump through small individual orifice tanks.

The two geometries modelled are shown in Dwg. 3.2.2. They are labeled Geometry A and B and differ only in the arrangement of the full depth bypass entrance. The concrete

features of the intake were reproduced using clear acrylic sheets as shown in Photo 3.2.1. The model intake was built in sections to facilitate testing of the different arrangements. These sections included the streamlined head-form, the block containing the full depth bypass entrance, the upstream three pumping bays, the downstream three pumping bays, and the streamlined tailform. The pumping bays were constructed extra long in case this length was needed to obtain proper approach flow to the pumps. However, the tests showed that lengths indicated in Dwg. 2.3.2 were adequate. Therefore, bulkhead walls were used for all tests to simulate the proper size of bay.

No attempt was made to model the individual components of the travelling screens. Instead, the travelling screens were modelled using a belt of plastic fly-screening having a mesh spacing of 6.4 mm (0.25 inches) prototype and a net area equal to 62 percent of the gross area. The screen face was recessed 0.2 m (8 inches) back from the front of the bays. From the point of view of maintaining a smooth sweeping flow it would be preferable to have this screen flush with the front face but present prototype screen designs are not capable of producing this flush facing.

The four arrangements of the model trashracks are shown in Dwg. 3.2.3 and they differ mainly in the orientation of the trashrack bars. Trashracks were constructed with the rectangular bars orientated 90° , 75° , 60° and 45° to the front face of the trashrack. One of the 45° trashracks is shown in Photo 3.2.3. The bars measured 32 mm (1.25 inches) by 160 mm (6.5 inches) and their centre-to-centre spacing varied from 160 mm (6.5 inches) for 90° orientation to 230 mm (9 inches) for 45° orientation. A fifth arrangement consisted of round bars rather than rectangular ones. These were 32 mm (1.25 inches) in diameter and spaced 160 mm (6.5 inches) apart. In the model, the lower level entrances were protected by single trashracks and the full depth entrance was protected by 5 trashracks stacked on top of each other. Geometry A, the initial geometry tested, had a full depth entrance aligned

with the lower entrances. Difficulties with this arrangement which will be detailed in the section on model results, led to Geometry B which aligns the full depth entrance obliquely to the flow and to the lower entrances (see Photo Photo 3.2.2).

3.3 Instrumentation and Data Collection Methods

In the course of the study, data were collected on water velocities, flow passage times, flow directions, sediment behaviour and trash behaviour. Water velocities were measured using a miniature propellor meter with a propellor diameter of 1.0 cm, capable of operating at velocities down to 0.2 m/s. However, for most reliable results model velocities were kept above 0.3 m/s by augmenting the flow when required as discussed in Section 3.1. Flow passage times were measured by timing the travel of small dye clouds. These clouds also permitted the observation and recording of flow directions. Assessment of sediment accumulation and trash accumulation behaviour was mostly a subjective process but some quantity measurements were made as described later in the report.

4. MODEL RESULTS

4.1 Geometry A

In general the magnitude of the bypass velocity depended on the magnitude of the river velocity, which is minimum at low flows. As a result, the emphasis of the testing on Geometry A was on attempting to achieve an adequate bypass velocity at the minimum design flow.

At minimum design flow of $115 \text{ m}^3/\text{s}$ (4000 cfs), the average river velocity is 0.52 m/s (1.7 fps) at the face of the intake where the depth then is 2.8 m (9.2 ft). When Geometry A was tested at this flow with 90° trashracks a continuous flow down the bypass was not achieved. The full depth bypass entrance failed to attract any of the river flow. The water entering the first low

level entrance would flow upstream and exit through the full depth bypass entrance. Photo 4.1.1 shows dye being injected in the river flow adjacent to the full depth bypass entrance. The dye stained water was carried by the river to the first low level entrance where it entered the bypass and flowed back upstream. Dye injected in front of Bay 3 also moved back up the bypass channel as shown in Photo 4.1.3.

Even though there was no flow down the bypass at the upstream end, the original geometry showed promising features. There was a flow out of the bypass at the downstream exit indicating a favorable pressure gradient at the tail. Photos 4.1.2 and 4.1.4 depict the downstream movement of dye introduced in the river at Bay 5. The flow through the travelling screens into the pump bays was also observed to be uniform in its overall distribution.

In an attempt to make use of some of the dynamic head of the water flowing past the pier to help attract flow into the entrances, the trashracks with the bars angled upstream were installed. Angles of 75° , 60° and 45° were tried, but none induced the flow downstream in the bypass. Because of the accelerated flow around the head form it appears that there was a larger pressure head at the first low level entrance than at the full depth bypass entrance. This caused the water entering the first low level entrance to flow upstream and exit through the full depth bypass entrance.

4.2 Geometry B

Once the problem of insufficient flow down the bypass channel had been identified as the result of an adverse pressure gradient imposed through the full depth bypass entrance, it was realized that proper flow could be induced by creating a high positive pressure head at the bypass entrance. This was achieved by orienting the full depth

entrance so that the river flow was deflected away increasing the pressure at the entrance. This principle was accomplished in Geometry B.

Geometry B was tested at the minimum design flow with several trashrack configurations. Table 4.2.1 summarizes the sweeping velocities in the by-pass channel for 5 tests. Appendix A provides the complete set of all velocity measurements.

TABLE 4.2.1.

VELOCITIES IN THE BYPASS CHANNEL FOR GEOMETRY B
AT LOW DESIGN RIVER DISCHARGE OF $115 \text{ m}^3/\text{s}$ (4000 cfs) WITH
WATER SURFACE AT EL. 289.0

Trashrack Arrangement	Bypass Velocity in m/s (fps)		
	Maximum Value*	Minimum Value*	Average Value*
90° Trashracks	0.35 (1.2)	0	-
60° Trashracks	0.35 (1.2)	0.15 (0.5)	0.20 (0.7)
45° Trashracks	0.45 (1.5)	0.30 (1.0)	0.35 (1.1)
No Trashracks	0.50 (1.6)	0.35 (1.2)	0.45 (1.4)
Round Bars	0.45 (1.5)	0.20 (0.7)	0.30 (1.0)

*Maximum, minimum and average of a set of seven velocities measured at El. 288 upstream and downstream of each bay.

The change in the full-depth entrance geometry produced satisfactory entrance velocities as shown in Photos 4.2.1 and 4.2.2. However, with increased flow through the full-depth entrance less flow entered the lower entrances and the bypass velocity decreased with distance down the channel. With 90° trashracks the velocities became zero near the third and fourth bay. They increased again at the fifth and sixth bay as the flow was attracted to the downstream bypass exit. Directing the trashrack bars into the river flow (decreasing

the trashrack angle) increased the flow through the entrances and increased the bypass velocities. It appears that the flow through the entrances can be increased in this manner if there exists a favorable pressure gradient across the entrance but a flow cannot be created this way against an unfavorable gradient.

The greatest bypass velocities were generated when the trashracks were removed completely. This discovery prompted the testing of trashracks with round bars which have little directional bias but the resulting velocities were appreciably lower than the best achieved with directional trashracks.

The most acceptable solution was obtained with the use of 45° trashracks in all openings. Photo 4.2.3 shows dye being injected outside of the bypass and flowing through the trashracks and down the channel. Photo 4.2.4 depicts the flow out the exit of the bypass channel.

Geometry B was further tested at four other discharges with the resulting velocities summarized in Table 4.2.2.

TABLE 4.2.2.

VELOCITIES IN THE BYPASS CHANNEL FOR GEOMETRY B
WITH 45° TRASHRACKS AT VARYING RIVER DISCHARGES

River Discharge m ³ /s (cfs)	River Velocity m/s (fps)	Velocity Measurement Elevation M	Bypass Velocity in m/s (fps)		
			Maximum Value*	Minimum Value*	Average Value*
115 (4,000)	0.50 (1.7)	288	0.45 (1.5)	0.30 (1.0)	0.35 (1.1)
200 (7,000)	0.76 (2.5)	288	0.65 (2.2)	0.30 (0.9)	0.40 (1.3)
285 (10,000)	1.00 (3.3)	288	0.95 (3.1)	0.55 (1.8)	0.65 (2.2)
1980 (70,000)	2.50 (8.2)	294	1.30 (4.2)	1.20 (3.9)	1.20 (4.0)
		291	1.30 (4.2)	0.80 (2.6)	0.95 (3.2)
		288	1.65 (5.4)	1.05 (3.5)	1.25 (4.0)
4530 (160,000)	3.95 (13.0)	297	2.05 (6.7)	1.85 (6.1)	1.95 (6.3)
		294	1.65 (5.5)	1.60 (5.2)	1.55 (5.1)
		291	2.20 (7.3)	1.65 (5.5)	1.75 (5.8)
		288	2.40 (7.9)	1.85 (6.1)	2.00 (6.5)

*Maximum, minimum and average of a set of seven velocities measured at that Elevation upstream and downstream of each bay.

These measurements show that the design low discharge definitely produces the lowest bypass velocities and that velocities increase with increasing river discharge.

Photos 4.2.5 and 4.2.6 show the model operating with the river discharge at 1980 m³/s (70,000 cfs) while Photos 4.2.7 and 4.2.8 show sub-surface views with the upper design discharge of 4530 m³/s (160,000) cfs. At 1980 m³/s (70,000) the velocity in the bypass channel averages slightly more than 1 m/s (3.5 fps) and was fairly quiet but at the higher discharge the velocity in the bypass channel approached 2 m/s (7 fps) and became noticeably more turbulent. Later tests were directed towards reducing the bypass flow at this condition. Also, it was observed at this condition that the full depth entrance accepted so much flow that water flowed out of the low-level entrances as shown in Photo 4.2.8.

4.3 Effects of Restricted Pump Operation

In order to determine the variation in the bypass velocity as it is influenced by having different pumps in operation, five additional tests were conducted at the minimum design flow. Table 4.3.1 summarizes the results along with a comparison to the full configuration with all six pumps operating.

As evident from Table 4.3.1 the bypass velocity was reduced slightly when the pumping capacity was reduced. The greatest reduction in velocity occurred when the two downstream pumps were off. This minimum velocity occurred in front of the two bays which were not operating and, therefore, in a zone where there was no velocity through the travelling screens. In front of the operating pumps the velocities in each case exceeded 0.15 m/s (0.5 fps).

TABLE 4.3.1.

VELOCITIES IN THE BYPASS CHANNEL FOR GEOMETRY B
 WITH 45° TRASHRACK AT 115 m³/s (4,000 cfs)
 RIVER DISCHARGE WITH RESTRICTED PUMP OPERATION

Pumps Operating	Bypass Velocity in m/s (fps)		
	Maximum Value*	Minimum Value*	Average Value*
All pumps on	0.45 (1.5)	0.30 (1.0)	0.35 (1.1)
No pumps on	0.35 (1.1)	0.25 (0.9)	0.30 (1.0)
Four downstream pumps on	0.40 (1.2)	0.25 (0.9)	0.30 (1.0)
Two downstream and two upstream pumps on	0.45 (1.5)	0.20 (0.6)	0.30 (1.0)
Four upstream pumps on	0.45 (1.5)	0.15 (0.5)	0.25 (0.8)
All pumps at 1/3 capacity	0.35 (1.2)	0.20 (0.6)	0.30 (0.9)

*Maximum, minimum and average of a set of seven velocity measurements at El. 288 upstream and downstream of each bay.

4.4 Flow Restrictions to Limit High Bypass Velocity at Higher Flows

The possibility exists that the larger velocities and increased turbulence in the bypass at the higher river discharges may be undesirable. In order to assess the possibility of reducing the higher bypass velocities several tests were conducted with different flow restricting devices. Table 4.4.1 summarizes the results for the two higher flows tested.

The first series of tests consisted of reversing the 45° trashrack angle in the upstream full depth bypass intake so that the entrance flow was obstructed rather than aided. The effect was to reduce the velocities by approximately 40 percent.

TABLE 4.4.1

BYPASS VELOCITIES FOR GEOMETRY B
WITH RESTRICTIONS TO REDUCE THE BYPASS FLOW

Test Conditions	Velocity Measurement Elevation M	Bypass Velocity in m/s (fps)		
		Maximum Value*	Minimum Value*	Average Value*
<u>River Discharge of 1985 m³/s (70,000 cfs), water surface at El. 295</u>				
All trashracks 45° upstream	294	1.30 (4.2)	1.20 (3.9)	1.20 (4.0)
	291	1.30 (4.2)	0.80 (2.6)	0.95 (3.2)
	288	1.65 (5.4)	1.05 (3.5)	1.25 (4.0)
Full depth trashracks pointing 45° downstream	294	0.90 (3.0)	0.45 (1.5)	0.65 (2.1)
	288	1.00 (3.2)	0.65 (2.1)	0.80 (2.6)
Full depth trashracks 90°	294	1.25 (4.1)	0.90 (3.0)	1.05 (3.4)
	288	1.30 (4.3)	0.85 (2.9)	1.00 (3.3)
Full depth trashracks part blocked off on downstream side	294	1.55 (5.1)	1.10 (3.6)	1.30 (4.2)
	288	1.85 (6.0)	1.20 (4.0)	1.40 (4.6)
Full depth trashrack part blocked off on upstream side	294	1.35 (4.4)	1.10 (3.6)	1.20 (3.9)
	288	1.65 (5.4)	1.10 (3.6)	1.30 (4.2)
<u>River Discharge of 4530 m³/s (160,000 cfs), water surface at El. 298</u>				
All trashracks 45° upstream	297	2.05 (6.7)	1.85 (6.1)	1.95 (6.3)
	294	1.65 (5.5)	1.60 (5.2)	1.55 (5.1)
	291	2.20 (7.2)	1.65 (5.5)	1.75 (5.8)
	288	2.40 (7.9)	1.85 (6.1)	2.00 (6.5)
4530 m ³ /s (160,000 cfs) Full depth trashracks pointing 45° downstream	297	1.15 (3.7)	0.50 (1.6)	0.75 (2.4)
	294	1.30 (4.2)	0.85 (2.9)	1.00 (3.4)
	288	1.50 (5.0)	0.90 (3.0)	1.20 (3.9)
4530 m ³ /s (160,000 cfs) Full depth trashracks 90°	297	1.85 (6.1)	1.15 (3.7)	1.45 (4.8)
	294	1.50 (5.0)	1.15 (3.7)	1.30 (4.3)
	288	1.85 (6.1)	1.50 (5.0)	1.55 (5.1)

*Maximum, minimum and average of a set of seven velocity measurements at the Elevation indicated upstream and downstream of each bay.

Also tested was the use of 90° trashracks in the full depth bypass intake. As expected this was less effective, reducing velocities about 20 to 30 percent.

A wall behind the full depth bypass intake partially blocking the entrance flow was also tried. The obstruction consisted of a solid wall extending halfway across the trashrack at the top half of the entrance and tapering to zero obstruction at the level of the top of the low level trashracks. This baffle wall could be mounted to obstruct either the upstream half or the downstream half of the trashrack. The baffle wall configuration produced a jetting effect in the bypass as the flow through the unobstructed trashrack area expanded to fill the full width of the bypass. This condition produced zones of local eddy currents. In any case, the velocity reduction was minimal. It is evident from Table 4.4.1 that the greatest reduction in bypass velocities occurred when the 45° trashracks were reversed in their orientation.

4.5 Sediment Flushing Characteristics of the Intake

The Thompson River normally has a low sediment load and sediment inflow to the intake is not expected to adversely affect the intake operation. Some of the fine suspended sediments will pass through the mesh of the travelling screen and into the pumps and some will be deposited in the pumping bays and will require periodic removal. These sediments were not the immediate concern of the model study. Of concern were coarser sediments that could deposit themselves on the floor of the bypass channel and accumulate there to the detriment of the bypass flow. These could be deposited during higher river stages and remain to interfere with the bypass flow at lower stages.

To prevent these sediments from depositing, the bottom of the bypass channel was sloped 35° towards the outside of the intake structure. This angle is a conservative estimate of the submerged angle of repose for any sediments that could

be expected to be transported by the Thompson River. Several tests were conducted to determine the effectiveness of the sloping floor of the bypass and any areas that might collect sediment. Both suspended material and bed transported material were investigated. To test suspended material, sediment was continually released into the flow upstream of the model and the pattern of sand accumulation was examined. Two sediments were tested - a natural silica sand and a light weight aggregate composed of crushed walnut shells.

The sand had a mean diameter of 0.6 mm and a sediment fall velocity of approximately 0.1 m/s. This is equivalent in the prototype to a fall velocity of 0.6 m/s, which corresponds to a prototype gravel with a diameter of 10 mm. The crushed walnut shell had a mean diameter of 0.3 mm and a specific gravity of 1.3. Based on a similar analysis, the walnut shell was equivalent to a prototype sand with a diameter of 0.5 mm.

At a discharge of $285 \text{ m}^3/\text{s}$ (10,000 cfs) with velocities of 1.0 m/s (3.3 fps) the prototype "gravel" did not remain in suspension and did not enter the intake. At $1980 \text{ m}^3/\text{s}$ (70,000 cfs) with velocities of 2.5 m/s (8.2 fps) the particles remained suspended until they entered the bypass where, due to the reduced velocity, they dropped out. These particles slid down the sloping floor and out of the intake except for a small amount of gravel which accumulated on the floor of the bypass between the full depth bypass entrance and the first low level intake where the sloping floor was intercepted by the curtain wall. This material cleaned itself out gradually when the introduction of material ceased or the concentration of suspended sediments was reduced. Photo 4.5.1 shows the accumulation when the flow was stopped before it had time to clean itself. At $4530 \text{ m}^3/\text{s}$ (160,000 cfs) the bypass kept itself clear of any sediment accumulation.

The walnut shell representing prototype "sand" accumulated in the bypass at $285 \text{ m}^3/\text{s}$ (10,000 cfs) in a manner similar to the gravel tests $1980 \text{ m}^3/\text{s}$ (70,000 cfs), but at higher discharges the walnut shell washed through the bypass.

To investigate the intake under an extreme but unlikely condition, a sediment dune was allowed to move down the bed of the flume with the discharge at $1980 \text{ m}^3/\text{s}$ (70,000 cfs). This dune was higher than the bottom of the bypass channel and filled in the channel as it migrated downstream. The bypass channel cleaned itself as the dune continued downstream. Photo 4.5.2 shows the dune in front of Bay 3 and it is evident that no sand exists upstream in the channel where the dune had previously passed.

4.6 Trash Accumulation

The trash load in the Thompson river is not known to be high but tests were conducted to demonstrate the effect trash would have on the structure.

The first type of tests involved floating and submerged logs approaching the intake (submerged logs may be dislodged from the bed and shore area of Kamloops Lake upstream). Logs 4 m (13 ft) long and 0.5 m (20 inches) in diameter were individually released immediately upstream of and in direct line with the pier and a count was made of those that hit the head form, the full depth bypass trashracks or any portion of the curtain wall downstream. Table 4.6.1 contains the percentage of both floating and submerged logs that impacted at three different discharges. Photo 4.6.1 shows logs passing the structure when the flow was $1980 \text{ m}^3/\text{s}$ (70,000 cfs).

TABLE 4.6.1

PERCENTAGE OF LOGS APPROACHING GEOMETRY B
THAT TOUCHED ON THE STRUCTURE

Test Conditions	Head Form	Percent Logs Hitting	
		Bypass Trashracks	Downstream Curtain wall or Trashracks
River Discharge of 285 m ³ /s (10,000 cfs)			
Floating Logs	20	20	12
Submerged Logs	24	44	16
River Discharge of 1980 m ³ /s (70,000 cfs)			
Floating Logs	16	20	4
Submerged Logs	8	28	4
River Discharge of 4535 m ³ /s (160,000 cfs)			
Floating Logs	24	8	8
Submerged Logs	4	20	4

These results show that the bypass trashracks are more exposed to impact by floating and submerged logs. The submerged logs do not have as much momentum and are not as serious as far as impact is concerned, but they could accumulate at the bypass entrance.

Water soaked confetti was used to investigate the effect of such trash as leaves or algae. When this confetti was introduced under water upstream of the structure it spread out and became distributed throughout the entire depth of flow as shown in Photo 4.6.2. It was observed that approximately 5 times as much confetti was caught on each of the full depth bypass trashracks as on the low level trashracks emphasizing the exposure to trash at the full depth bypass entrance.

A conventional trashrack raking scheme did not appear suitable to the vertical trashracks exposed to cross flows as high as 4 m/s (13 fps). Experiments on the model indicated that what would be needed would be a device to dislodge the entrapped particles so that they would be swept away with the flow.

5. DISCUSSION OF RESULTS

5.1 External River Flows

The model was set in the embayment so that the flow in the laboratory flume would behave the same as the flow that would pass the front side of the prototype. This was necessary so that the pressures at the entrances would be as nearly as possible representative of those in the actual structure. For the purposes of this study, however, it was not necessary to produce other aspects of the external flow hydraulics. These include:

- i the flow around the back of the intake adjacent to the right river bank and around any piers supporting the bridge to the intake
- ii the wake downstream of the intake
- iii local scour which could occur around the base of the intake
- iv the affect of the alignment of the intake with the river flow streamlines.

The river is sufficiently regular in its character and the bed and banks are sufficiently stable that all foreseeable design questions related to these aspects can be resolved reasonably without recourse to further river modelling. There is, however, the question of satisfying regulatory agencies of the adequateness of the solutions and hydraulic models are powerful tools for demonstrating the completeness of

a hydraulic design. For this purpose, a hydraulic model of approximately one kilometer of the river, to an undistorted scale of 1:80 would be completely adequate.

5.2 Bypass Flows

The bypass channel with seven points of influx and efflux represented a complex hydraulic configuration. The flow down the bypass depended on fairly small pressure gradients across the different openings. Geometry A created a lower pressure at the full depth bypass entrance than at the low-level entrances and this had to result in flow exiting from the bypass entrance. This basic shortcoming was corrected by the configuration of Geometry B.

At high river discharges the amount of water moving down the bypass channel was large compared to the amount being pumped so that the flow entering (or exiting) the lower level entrances was rather insignificant and had little bearing on the bypass velocities. At the low design flow the balance of flows entering or leaving the different openings was, however, delicate and each entrance had to contribute its share or an adequate bypass flow would not be maintained down the channel. Angling the bars of the trashracks provided the necessary control. As mentioned earlier, water cannot be forced into an entrance against an adverse pressure gradient by adjusting the bar angle but, given a favourable gradient, the bar angle will control the amount of flow. The angled trashrack bars are important to maintaining an adequate bypass flow.

At the low design flows especially, water did not flow down the bypass as a continuous stream. Generally, the flow entering the full depth entrance entered the upststream pumping bay and the flow entering the upstream low-level entrance fed the next bay downstream, etc., while the downstream low-level

entrance supplied water to the bypass exit. From the point of view of the fry, however, there did exist a continuous downstream component that would sweep the fry out the exit. Some turbulence did exist in the bypass, but there were no fixed eddies that could hold the fry entrapped. The passage time for the full length of the bypass was approximately one minute and did not exceed two minutes for any condition tested on the recommended design.

Maximum permissible velocities in the bypass channel have not been established. The velocities at high flows reach 2 m/s (7 fps) and in the event that these prove excessive, either for the fry or for the equipment such as the screens, the velocities can be reduced by constricting the flow at the full-depth intake as indicated by the tests. Reversing the orientation of the trashrack bars is a preferred method because it does not create the dead flow areas that are caused by more solid constrictions.

High bypass velocities also tend to disturb the uniformity of flow through the travelling screens. At high velocities the flow through the screens was highest over the downstream half of the screens. However, it is not necessary to have uniform flow if the velocity approaching any part of the screen does not exceed the value specified for fisheries.

At low river flows the submerged screen area is smallest and the average velocity approaching the screen is highest. Fortunately the bypass velocity is also low so that the flow can be expected to be approximately uniform through the screen. At high river flows the submerged area is larger so that some non-uniformity can be tolerated without exceeding the limiting approach velocity.

The uniformity of velocity through the screen depends

on head loss through the screen as well as the bypass velocity. A denser screen will result in a more uniform flow. The 1:40 scale model is not adequate for measuring this uniformity because the velocities through the screens are too low to measure and because the details of the screens are not adequately represented. To verify uniformity, a model of the bypass channel, trashrack and screen in the vicinity of one bay to a scale of about 1:10 would be adequate for representing the flow distribution.

5.3 Trash Accumulation

The amount of trash in a river is very site specific. In the absence of a field study one can only speculate on the severity for any particular intake. The tests performed for this study provide only indicators of how the structure responds to trash. It is reported that the Thompson River is relatively free of trash of any kind, particularly during low flow periods when the capacity of the trashracks to pass flows is most critical. The possibility of masses of algae being caught on the racks in the summer has been suggested. A trash problem is expected to be an unusual occurrence associated with a specific event such as a major flood or a bank subsidence upstream.

The tests show that the full depth trashracks are more exposed to trash accumulation than are the low level trashracks. However, the actual need for rakes has not been established and a rake arrangement that would operate at the high flow velocities could be expensive to design and construct. A field program might help assess the seriousness of the problem and the need for rakes.

5.4 Sediment Accumulation

Sediment transport rates are low in the Thompson River so that sediment accumulation in the bypass is not expected to be a serious problem. The tests conducted in the model confirm that the 35 degree slope provided in the bypass channel floor is sufficient to prevent accumulation of any amount of sediment detrimental to the operation of the bypass.

6. CONCLUSIONS AND RECOMMENDATIONS

As a result of this study it is possible to conclude:

1. that, on the basis of Geometry B in this report, a satisfactory bypass channel can be designed which will produce over the entire range of expected river flows a sweeping velocity in front of the travelling screens in excess of the minimum velocity of 0.12 m/s (0.4 feet/sec) stipulated by regulatory authorities.
2. that all foreseeable design problems related to the flow external to the intake pier can be resolved reasonably without recourse to further modelling.
3. that the flow distribution through the screens appears uniform on the basis of dye observations in the 1:40 model, but that a larger model will be required to verify this by measuring the exact velocity distribution at various flow conditions. This model could be limited to the area around one bay.
4. that the accumulation of sediment on the floor of the bypass channel, to the extent that it interferes with the flow in the bypass channel or with any other operational aspect of the bypass, will not be a problem if the sloping floor of the conceptual design is incorporated.
5. that the full depth entrance is more exposed to trash accumulation than are the low-level entrances.

As a result of these conclusions it is recommended that:

1. the design of the bypass channel be based on Geometry B of this study.
2. if a specific inflow distribution is to be verified a 1:10 scale hydraulic model of one intake bay and the associated bypass channel and low-level entrance be constructed and tested.
3. a 1:80 scale river model be constructed and tested only if the need arises to satisfy regulatory agencies of the adequacy of certain aspects of the design as it affects external flow and bed scour.

7. REFERENCES

- (1) "Hat Creek Project, Water Supply Study, Conceptual Design", Report V4007/2 Sandwell & Company Ltd., January 1977.
- (2) "Hat Creek Project, Water Supply Study", Interim Report V4007/1, Sandwell & Company Ltd., October 1976.
- (3) "Hat Creek Project, Water Supply Study, Water Intake Design", Project Memorandum V4007/1, Sandwell and Company Ltd., January 1977.
- (4) "Hat Creek Project, Cooling Water Supply Study, Evaluation of Intake Sites", Northwest Hydraulic Consultants Ltd., December 1977.
- (5) 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)
- (6) "Hat Creek Water Supply, Thompson River Ice Conditions", Northwest Hydraulic Consultants Ltd., April 1977.
- (7) Letter from Environment Canada, Fisheries and Marine Service, to International Pacific Salmon Commission, "Design of Fish Screens for Hat Creek Intake", dated September 8th 1977.

APPENDIX A
PHOTOGRAPHS

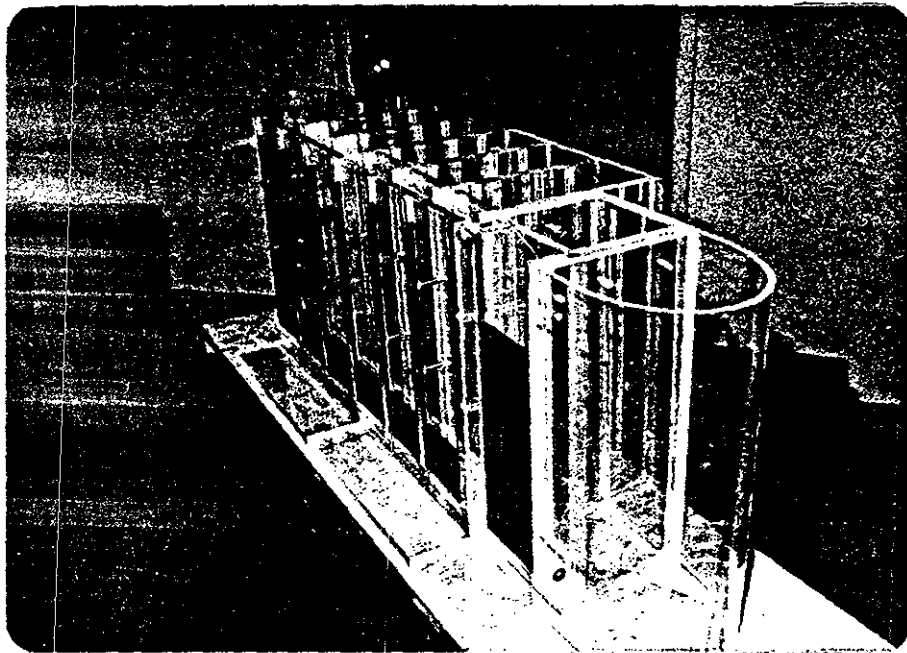


Photo 3.2.1. Geometry A before installation in flume.

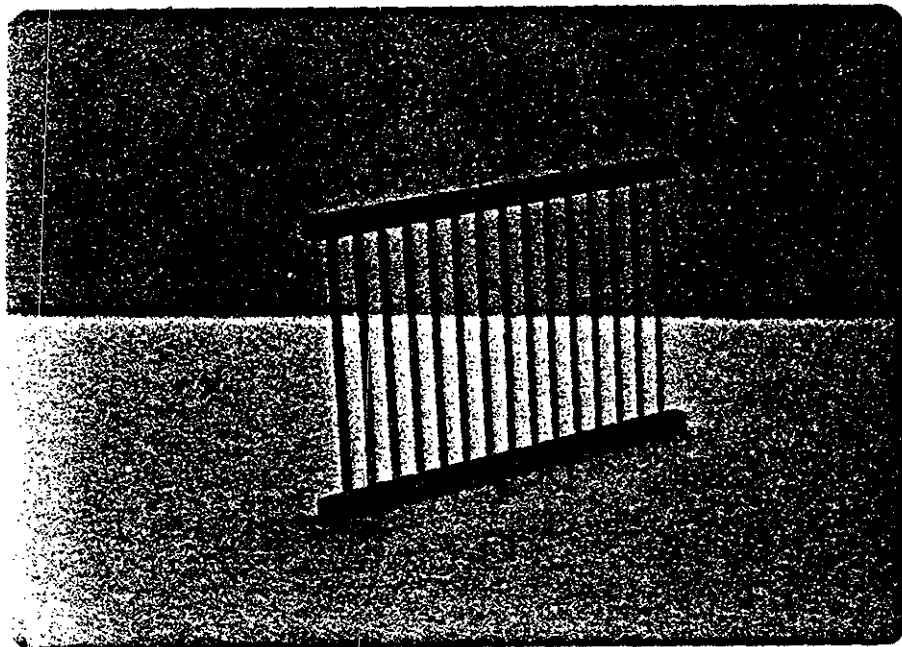


Photo 3.2.3. Model trashrack with 45° bar orientation.

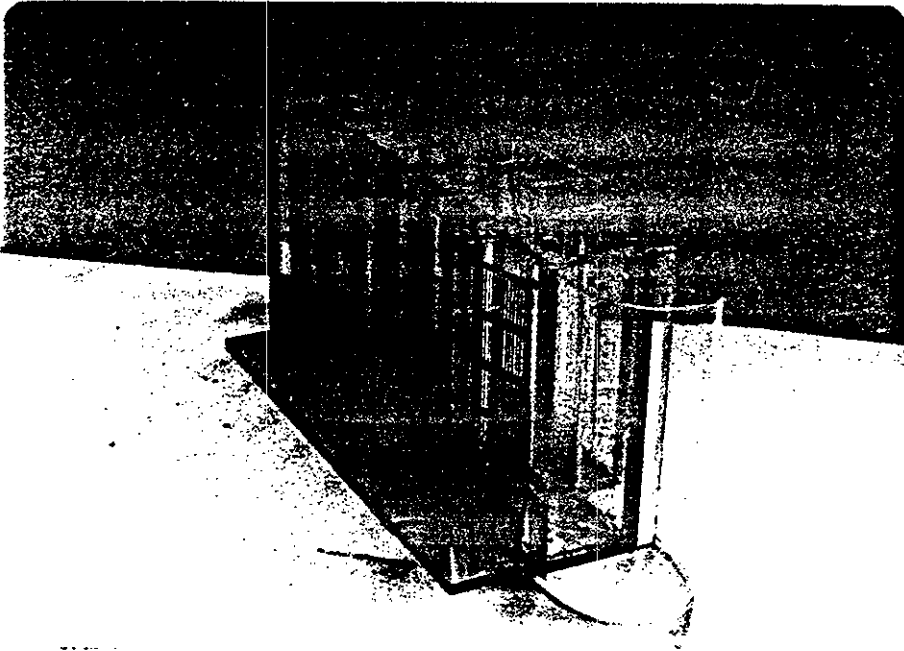


Photo 3.2.2. Geometry B before installation in the flume.

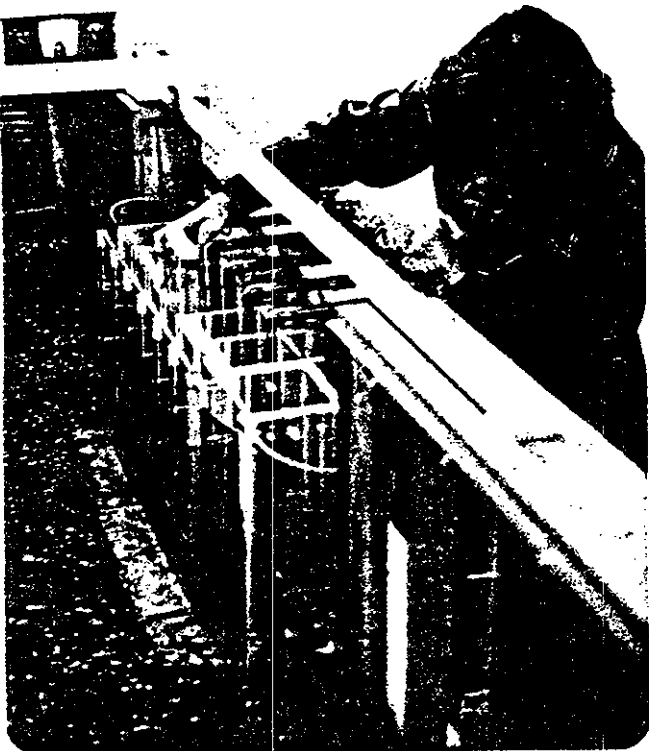


Photo 3.2.4.
Geometry B
installed in
the laboratory
flume.

Model Configurations

- Photo 3.2.1.
- Photo 3.2.2.
- Photo 3.2.3.
- Photo 3.2.4.



Photo 4.1.1. Dye being released at the entrance for the bypass flow.



Photo 4.1.3. Dye released in the bypass channel in front of Bay 3 flows upstream.

The flow is from right to left. The trashracks have



Photo 4.1.2. Dye released outside of bypass channel in front of Bay 5 flows downstream through the trashracks.

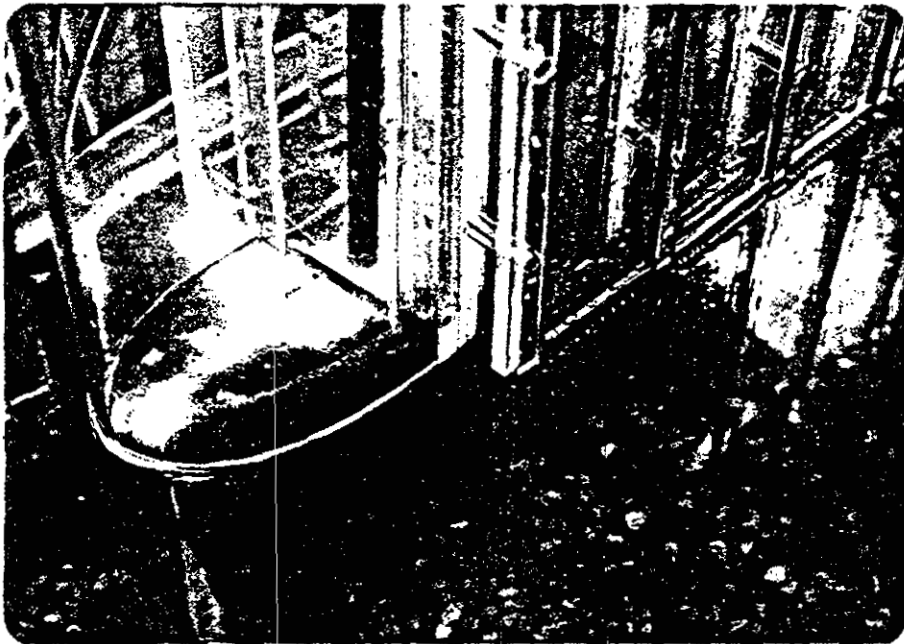


Photo 4.1.4. Dye released in the bypass channel in front of Bay 5 flows downstream.

Geometry A with prototype discharge of $115 \text{ m}^3/\text{s}$ (4000 cfs).

Photo 4.1.1.

Photo 4.1.2.

Photo 4.1.3.

Photo 4.1.4.

° bar orientation.



Photo 4.2.1. Dye released upstream of the bypass entrance flows into the bypass channel.

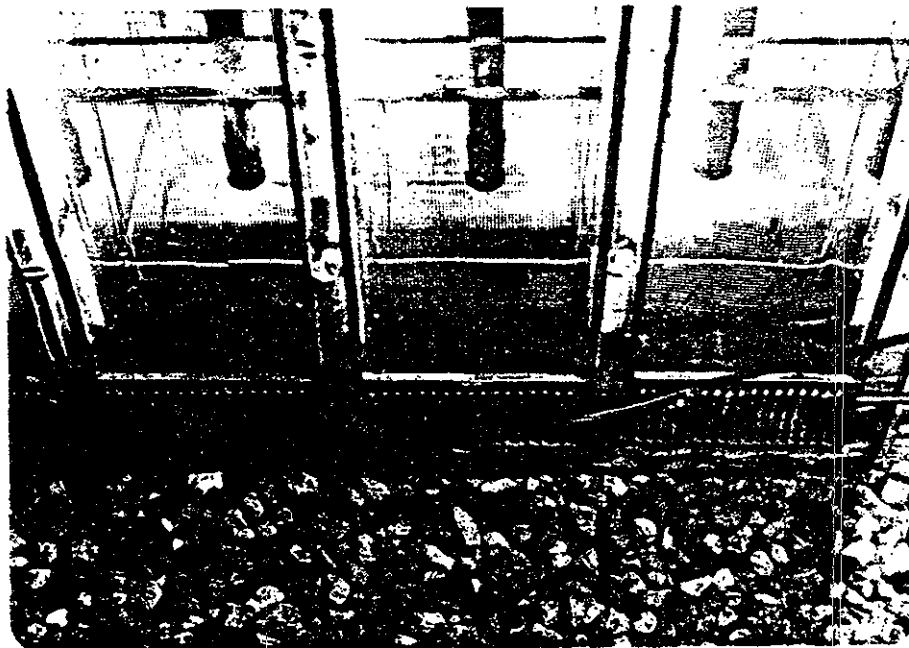


Photo 4.2.3. Dye released outside the bypass channel flows through the trashracks.

The flow is from right to left. The trashracks hay



Photo 4.2.2. The dye wand is inside the bypass entrance.

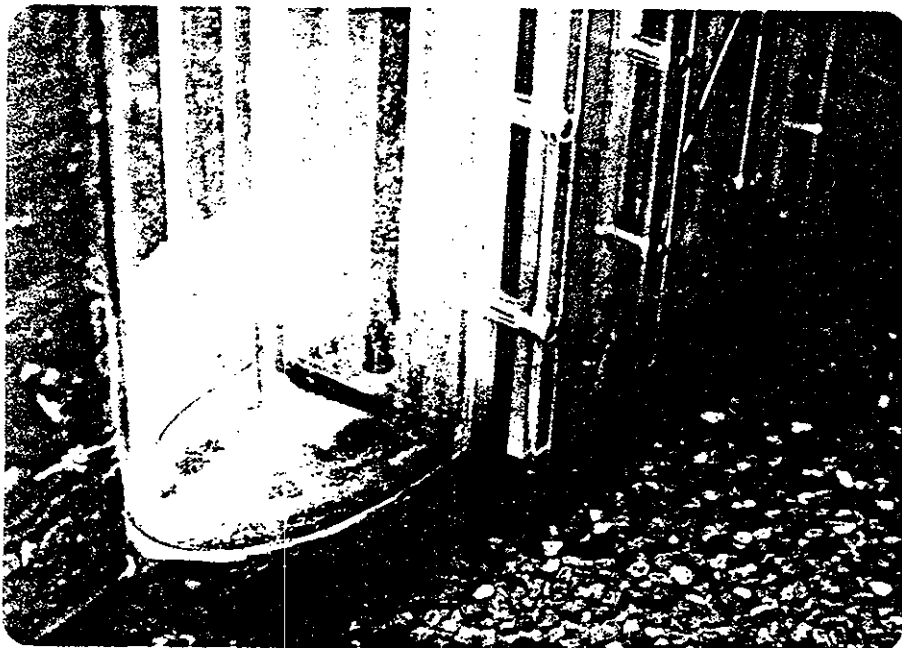


Photo 4.2.4. Dye released in the bypass channel flows out the exit.

Geometry B with prototype discharge of $115 \text{ m}^3/\text{s}$ (4000 cfs).

Photo 4.2.1.

Photo 4.2.2.

Photo 4.2.3.

Photo 4.2.4.

45° bar orientation.

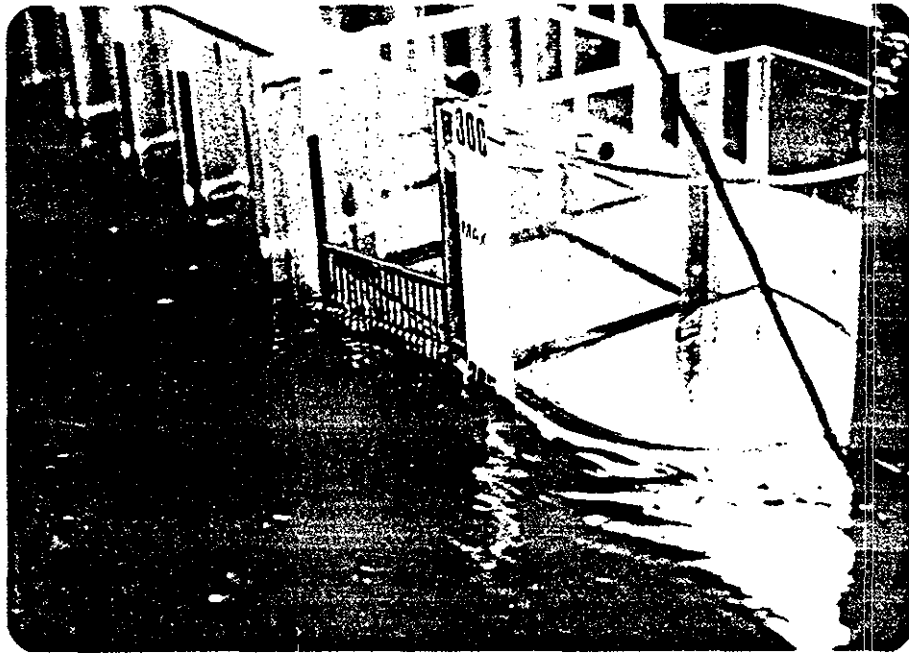


Photo 4.2.5. The dye is released upstream of the pier. The prototype discharge is $1980 \text{ m}^3/\text{s}$ (70,000).



Photo 4.2.7. Underwater shot of dye being released upstream of the bypass entrance. The prototype discharge is $4535 \text{ m}^3/\text{s}$ (160,000 cfs).

The flow is from right to left.

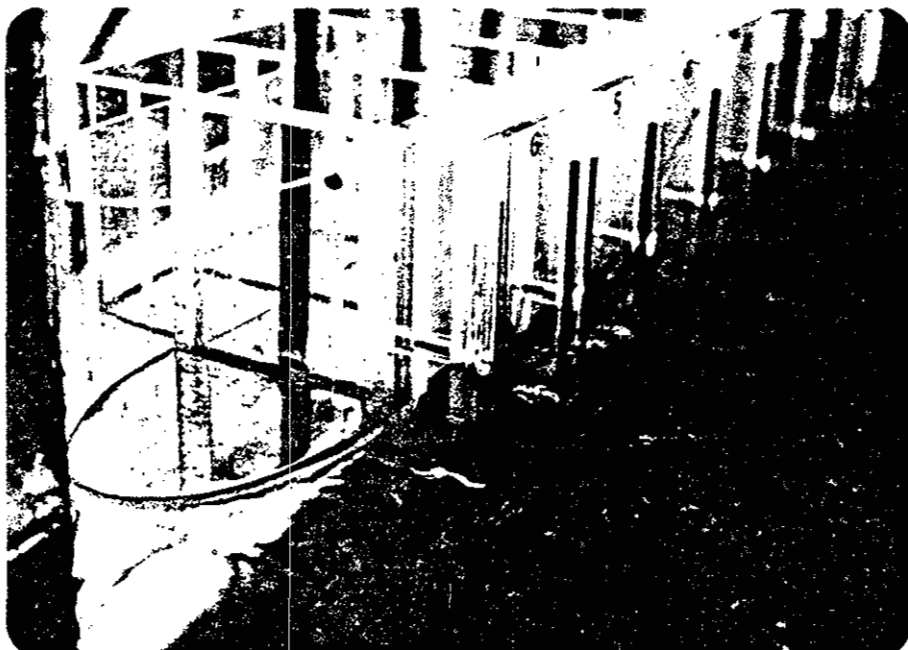
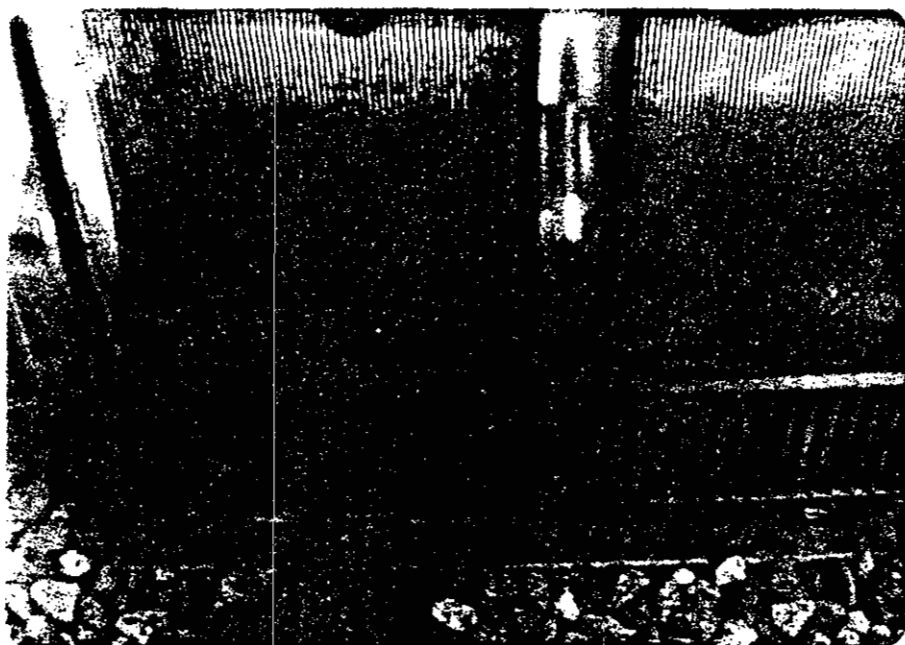


Photo 4.2.6. The dye is released in the bypass channel. The prototype discharge is $1980 \text{ m}^3/\text{s}$ (70,000 cfs).



Geometry B with 45° trashracks
at two high discharges.

Photo 4.2.8. Underwater shot of dye being released in the bypass channel in front of Bay 5. The prototype discharge is $4535 \text{ m}^3/\text{s}$ (160,000 cfs).

Photo 4.2.5.

Photo 4.2.6.

Photo 4.2.7.

Photo 4.2.8.



Photo 4.5.1. Sediment has accumulated in the bypass channel between the bypass entrance and the first low level entrance.

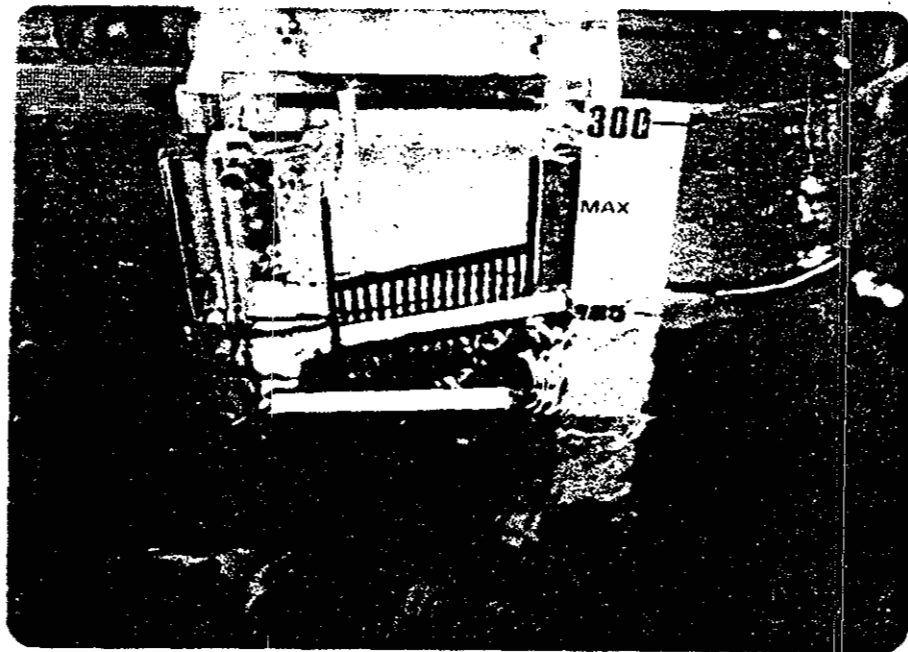


Photo 4.6.1. Model logs floating past the bypass entrance.

The flow is from right to left.

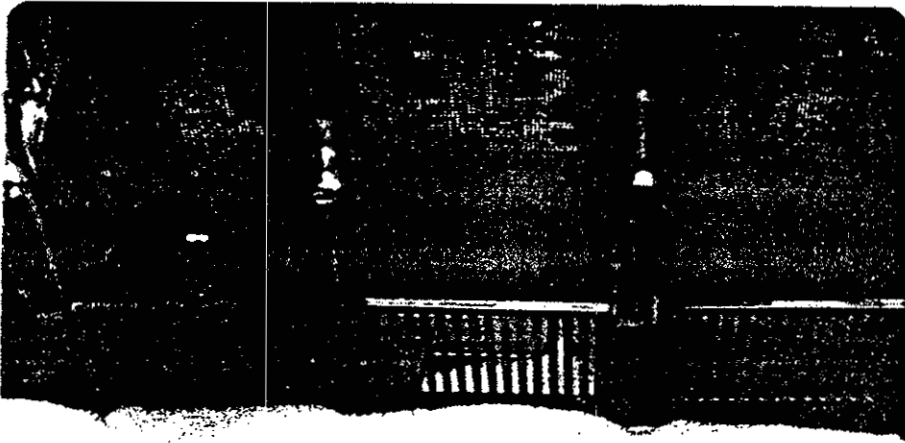


Photo 4.5.2. The sediment dune has migrated downstream and is covering the bottom of the bypass channel in front of Bay 3.

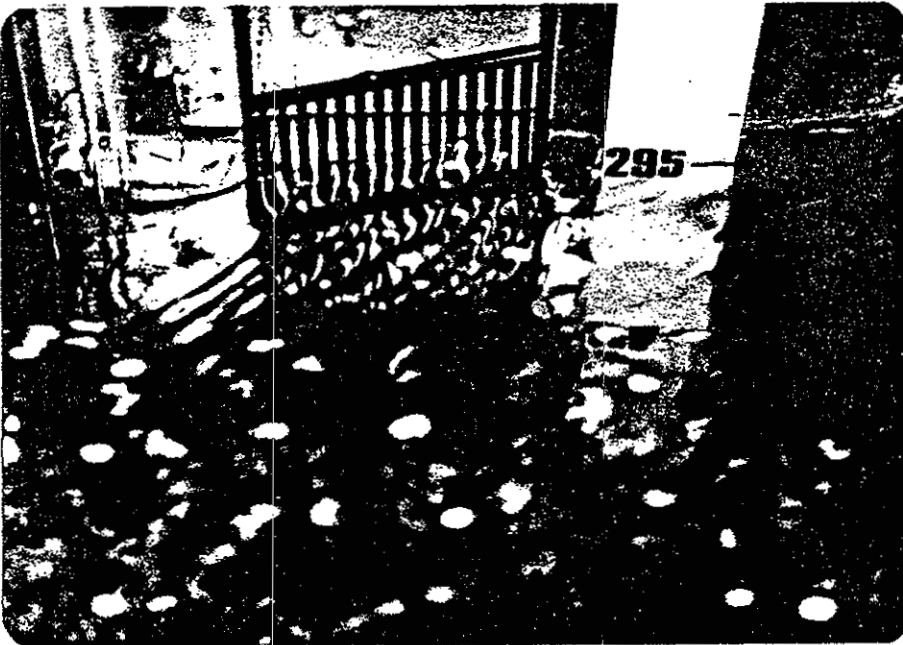


Photo 4.6.2 Wet confetti flowing past the pier.

Geometry B with prototype discharge of $1980 \text{ m}^3/\text{s}$ (70,000 cfs)

Photo 4.5.1.

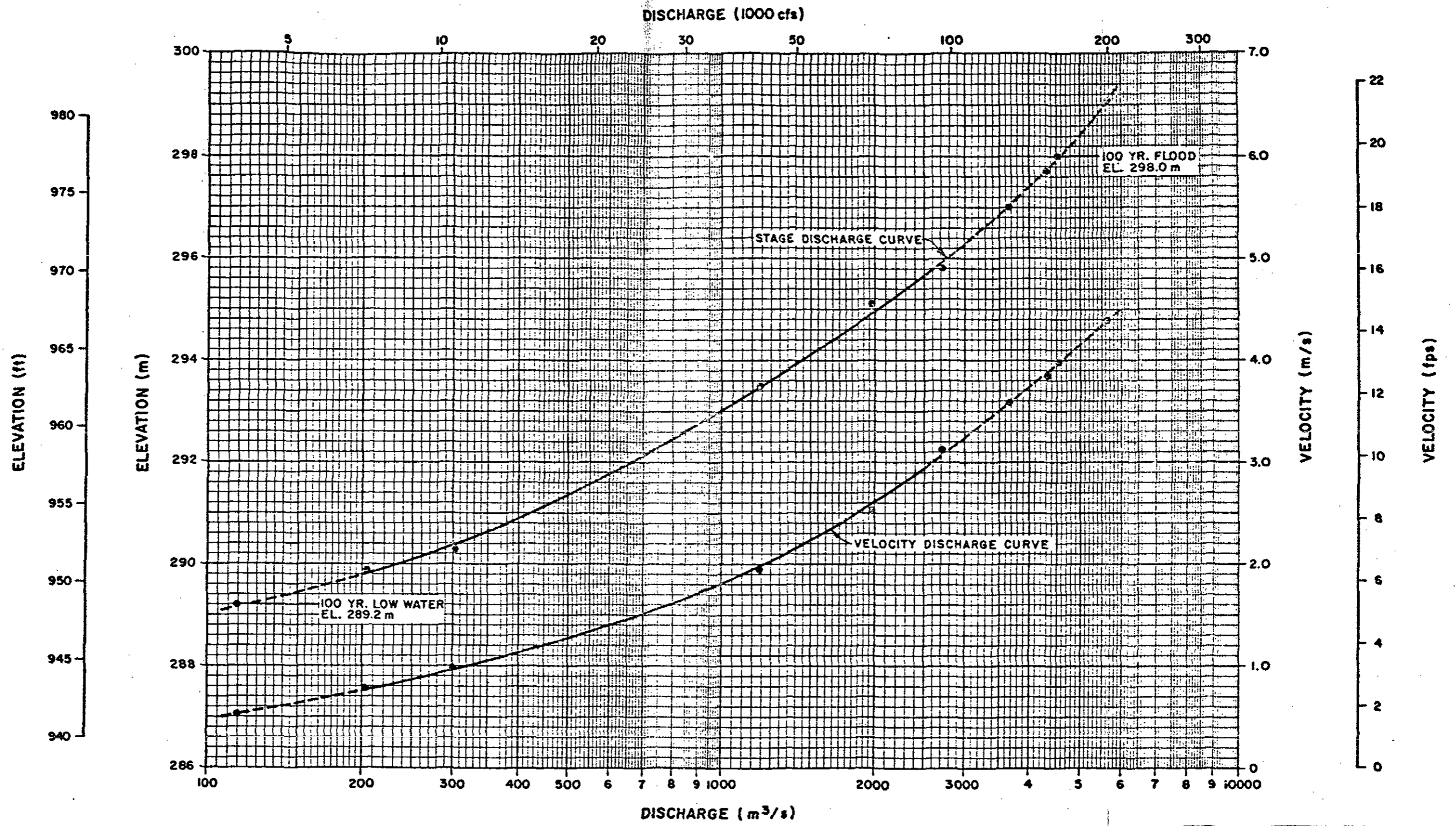
Photo 4.5.2.

Photo 4.6.1.

Photo 4.6.2.

APPENDIX B

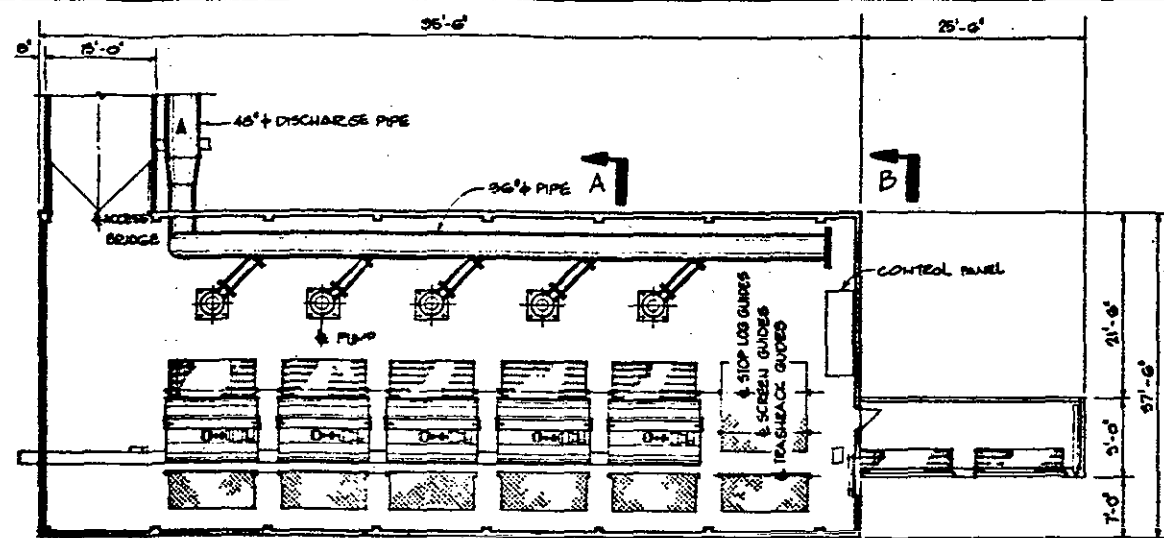
DRAWINGS



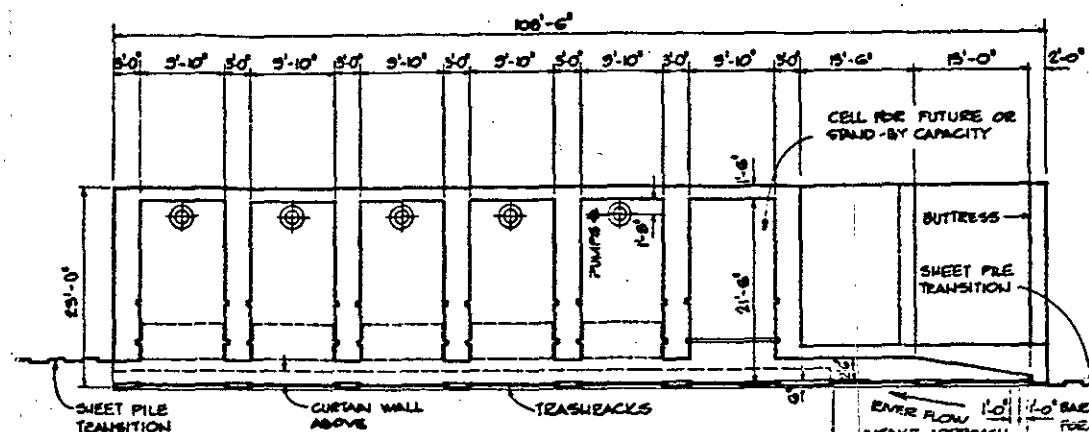
NOTE :
 THIS CURVE WAS DERIVED FROM RESULTS OF A BACKWATER COMPUTER MODEL.
 THE MODEL WAS CALIBRATED WITH AVAILABLE DATA RANGING BETWEEN
 200 AND 2500 m³/s (7000 AND 8800 cfs).
 PORTIONS OF THE CURVE OUTSIDE THIS RANGE ARE ESTIMATES ONLY.

LEGEND :
 ——— CURVE BASED ON AVAILABLE FIELD DATA
 - - - ESTIMATED CURVE
 ● COMPUTER MODEL POINTS

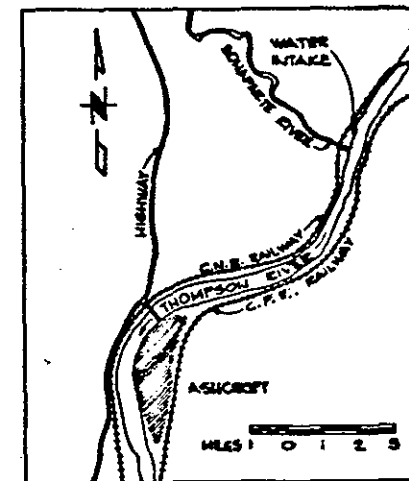
B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY HYDRAULIC MODEL OF INTAKE		
INTAKE SITE 10-D STAGE & VELOCITY RATING CURVES		
northwest hydraulic consultants ltd.		
Drawn: I.M.	Date: NOV. 1977	Dwg: 2.2.1



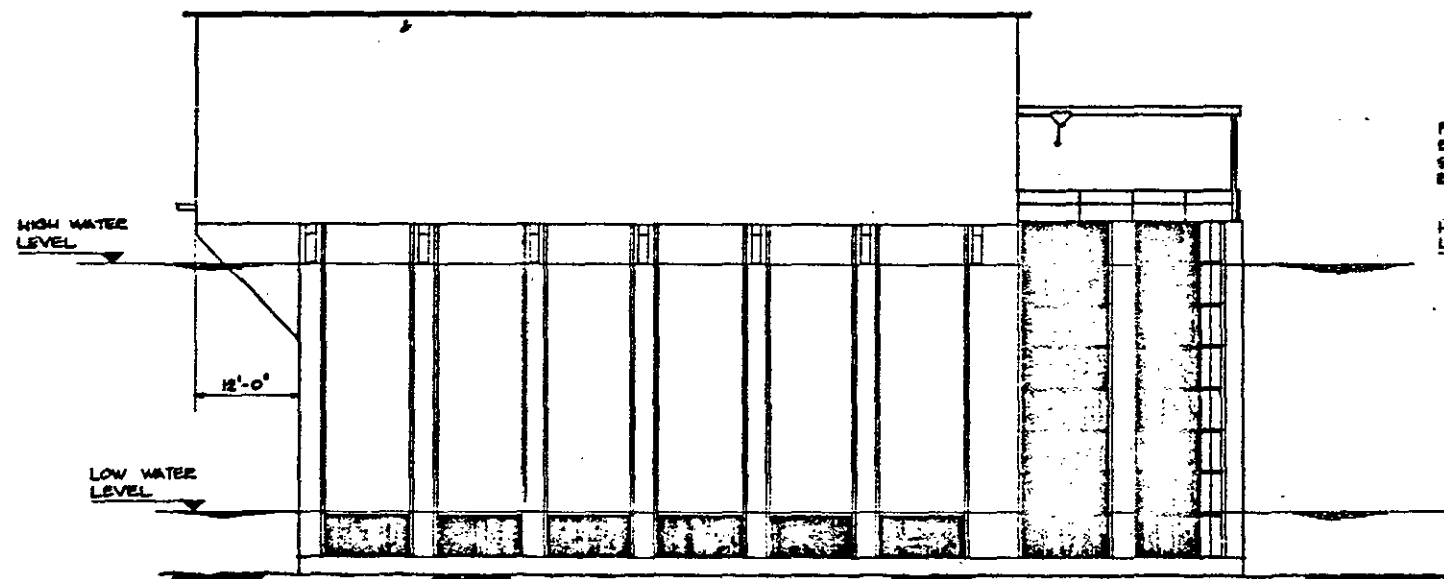
PLAN - OPERATING FLOOR LEVEL
392'-11-0"



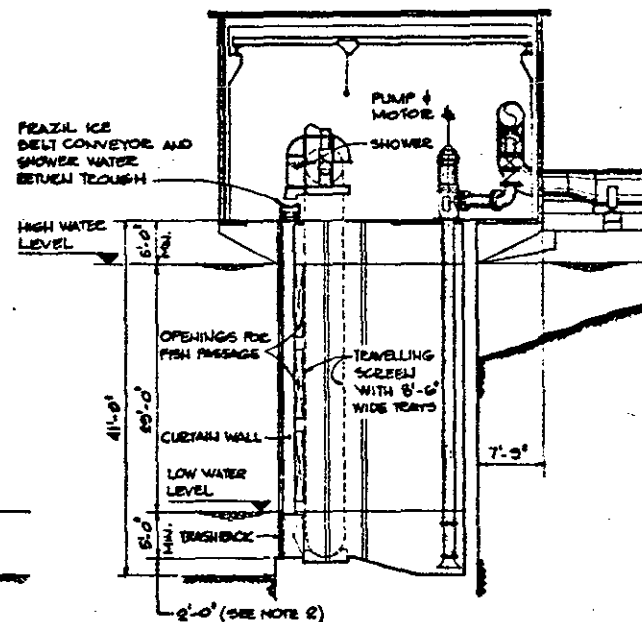
PLAN - TRASHRACK LEVEL
392'-11-0"



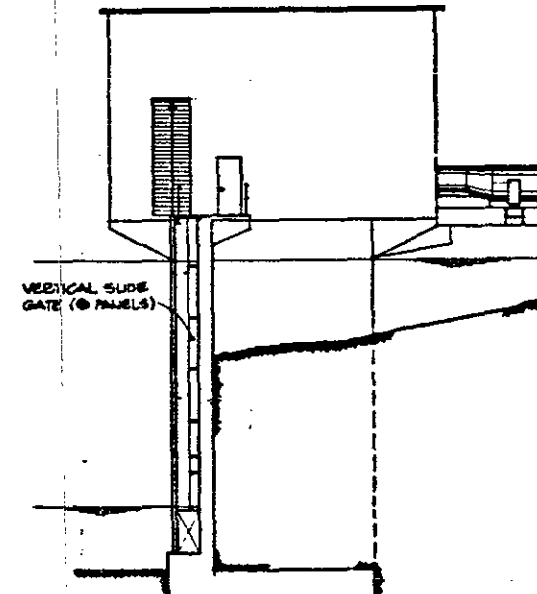
KEY PLAN



ELEVATION
392'-11-0"



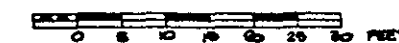
A - A
392'-11-0"



B - B
392'-11-0"

NOTES:

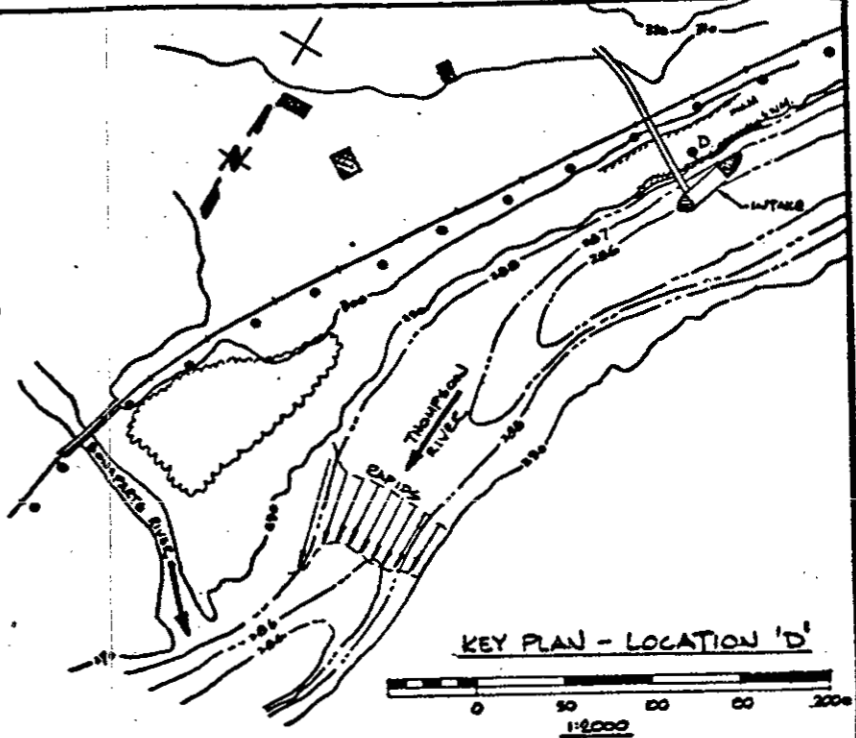
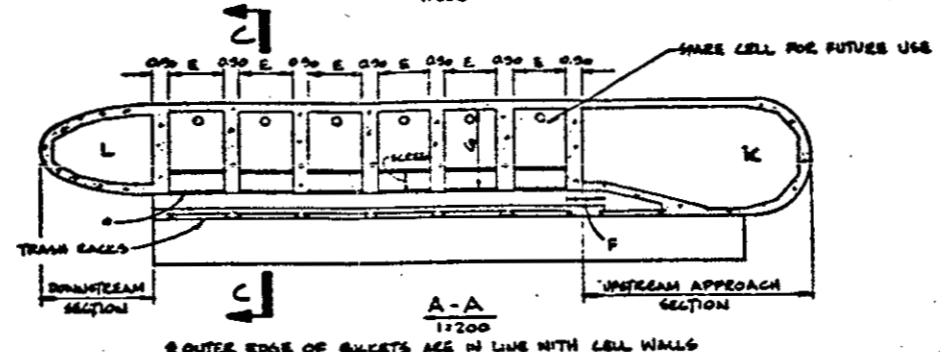
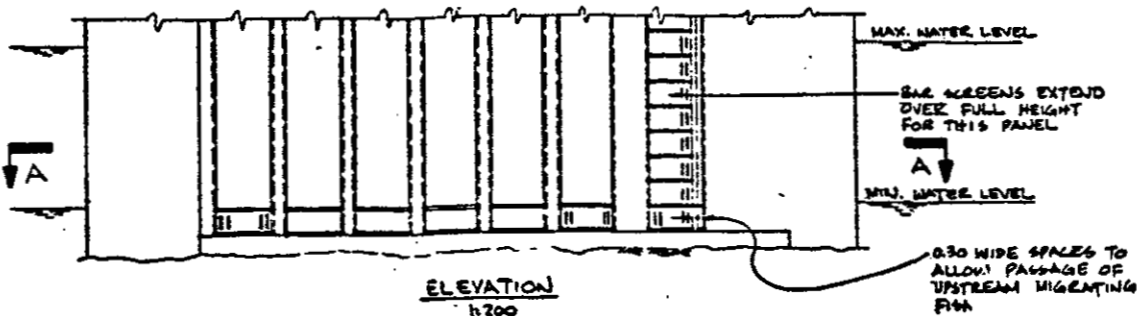
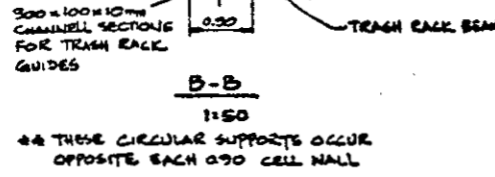
- FOR DETAILED ORIENTATION SEE REF 1.
- DISTANCE OF INTAKE SILL TO RIVER BOTTOM HAS TO BE CONFIRMED BY FIELD SURVEYS AND HYDRAULIC MODEL STUDIES.



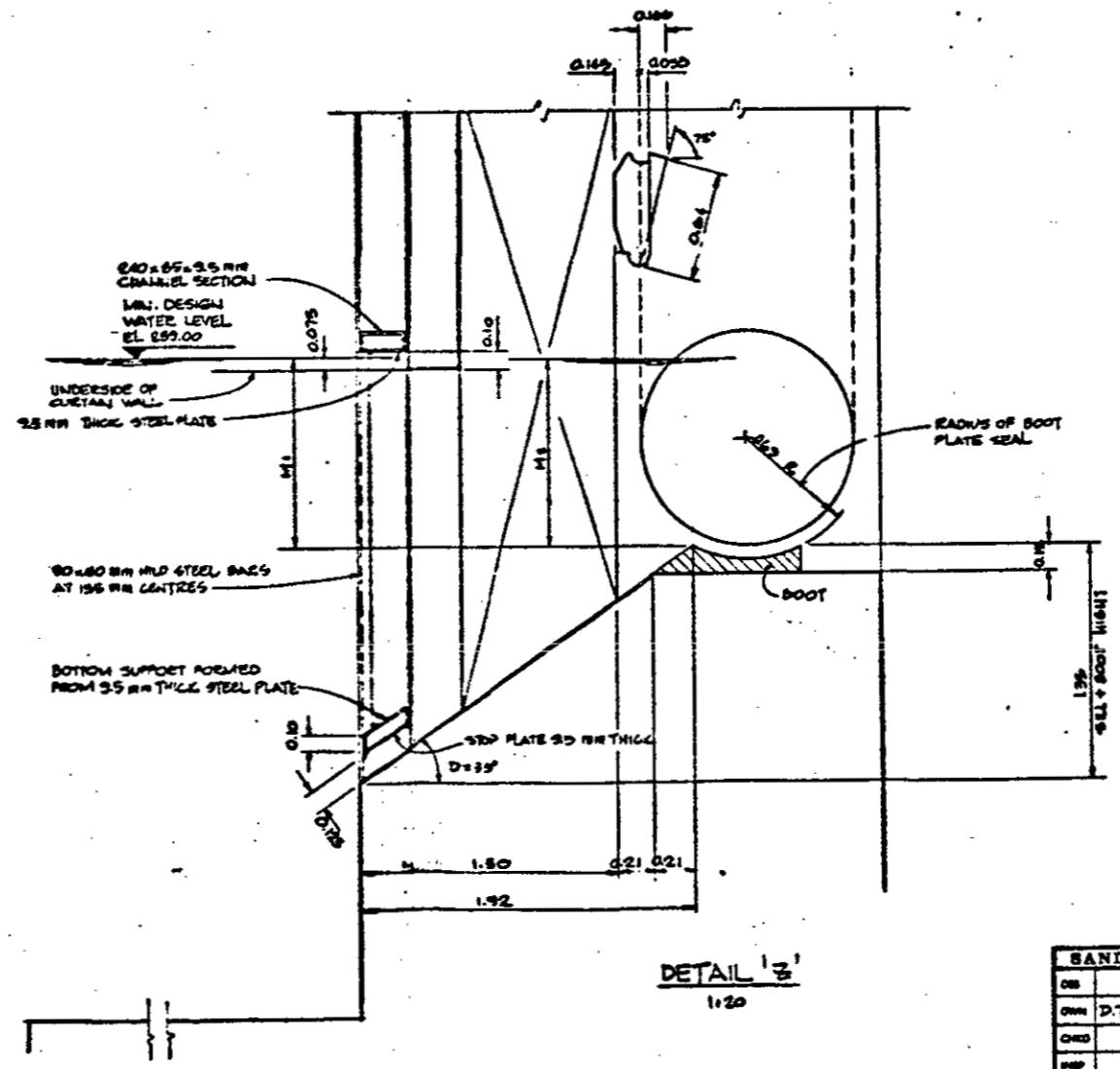
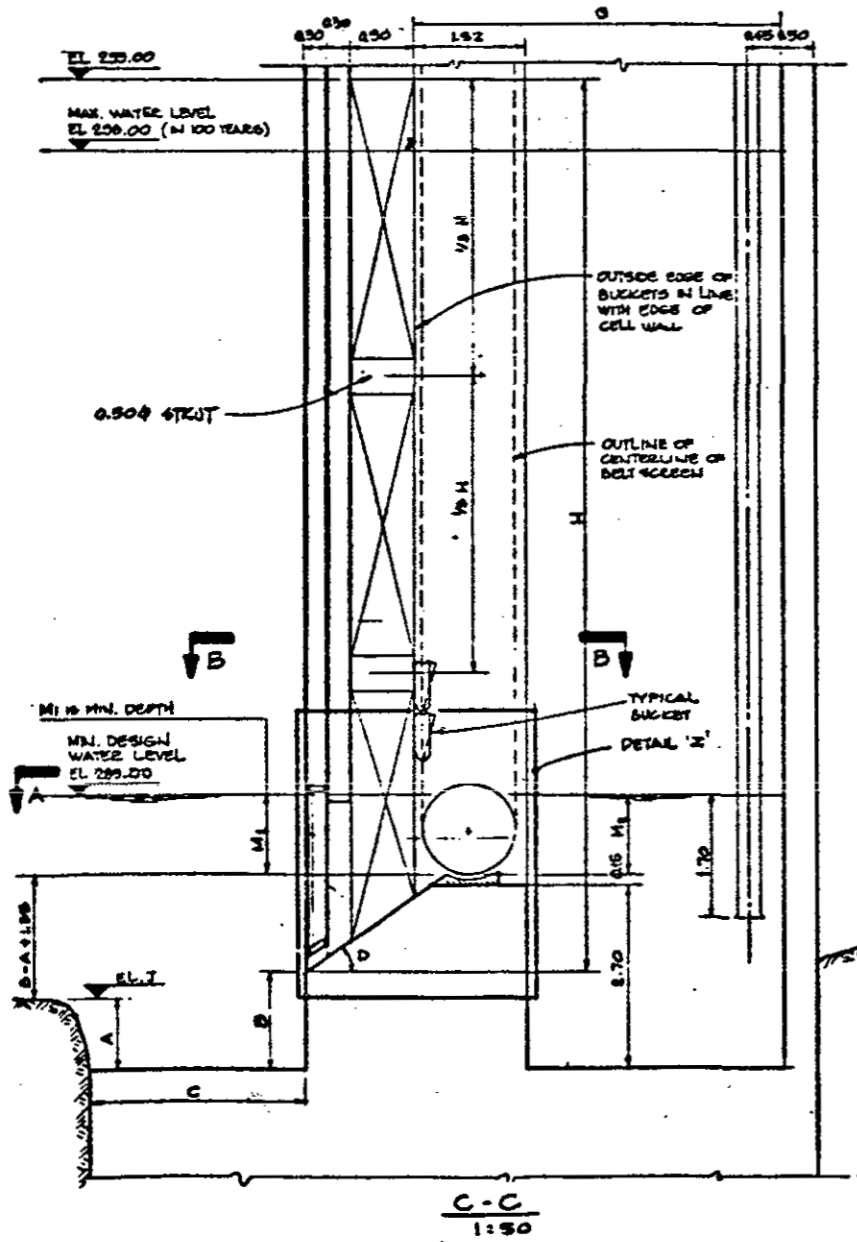
DWG. 2.3.1

PROJECT: HAT CREEK PROJECT WATER SUPPLY STUDY CONCEPTUAL DESIGN RIVER INTAKE LAYOUT		SCALE: AS SHOWN DATE: 1/12/76 DRAWN: SH CHECKED: [] APPROVED: [] D 4007 / 46	B.C. HYDRO AND POWER AUTHORITY VANCOUVER B.C. DWG. D 4007 / 46 [REV]
SANDWELL			

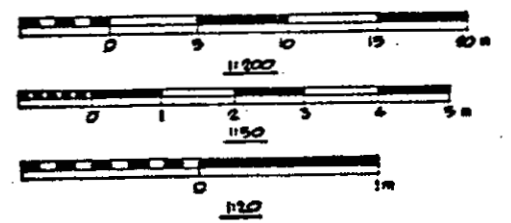
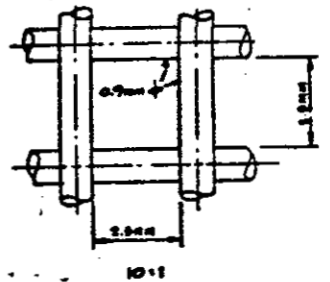
TABLE OF DIMENSIONS AND SHAPES	
A	1.00M
B	1.35M
C	3.00M
D	35°
E	340M
F	2.10M MINIMUM
G	7.5 X 8 = 8.50M MAXIMUM
H	17.43M
EL. F	292.2M
K	LENGTH OF SECTION 17.00M
L	LENGTH OF SECTION 6.00M
M	1.10M
N	1.10M



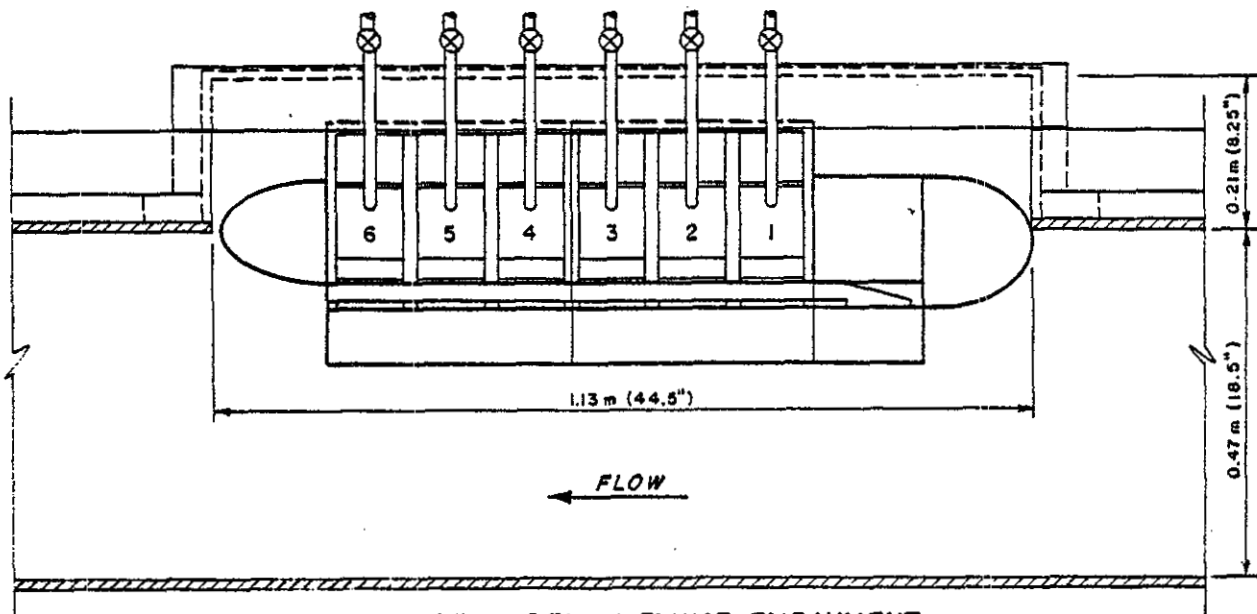
THESE VALUES WERE RECOMMENDED BY NHCL ON 9 AUG. 1977



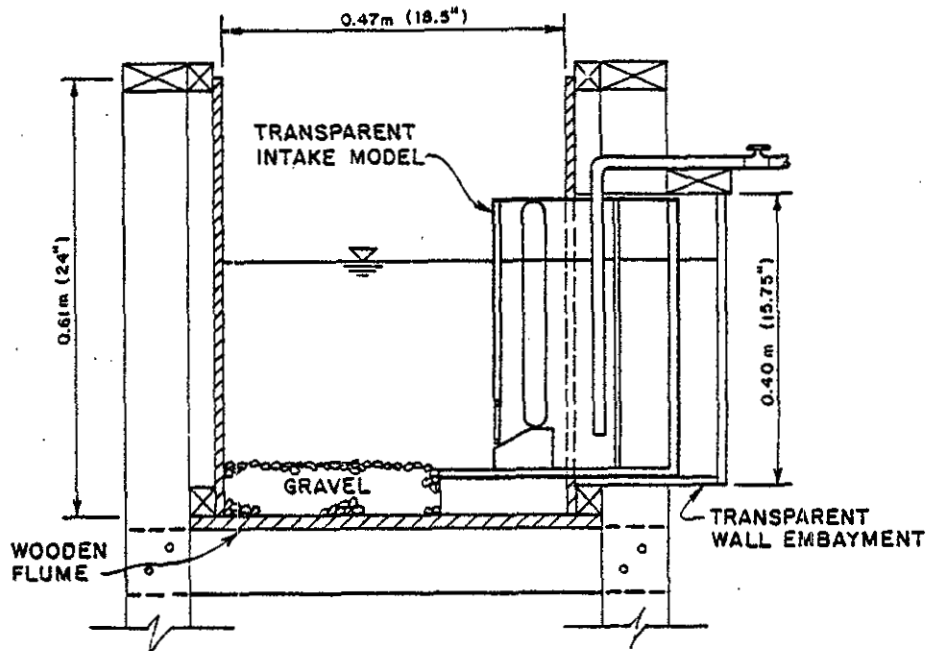
- NOTES**
- MODEL SCALE TO BE 1:40
 - THE MODEL WILL ONLY BE USED TO STUDY FLOW PATTERNS THROUGH THE MODEL - (NOT THROUGH THE RIVER, THIS STUDY WILL BE CARRIED OUT DURING THE DETAILED DESIGN STAGE.)
 - AT COMMENCEMENT OF STUDY MAKE 5°-35°
 - ALL DIMENSIONS AND ELEVATIONS SHOWN ARE IN METERS, EXCEPT WHERE SHOWN OTHERWISE.
 - THE SCREENS WILL BE FABRICATED FROM 0.9mm x 1mm WITH A CLEAR OPENING OF 25mm GIVING A CLEAR OPENING OF 54%



SANDWELL		DWG. NO. D4191-SK 14		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
DESIGNED BY	APPROVED BY	DATE	11 AUG 1977	HAT CREEK PROJECT	
OWNED BY	APPROVED BY	DATE		COOLING WATER SUPPLY	
CONTRACT NO.	APPROVED BY	DATE		PRELIMINARY DESIGN STUDY	
SCALE	AS NOTED	DWG. NO.	D 4191-SK 14	INTAKE MODEL-SITE/D-BASIC OUTLINE DIMENSIONS	

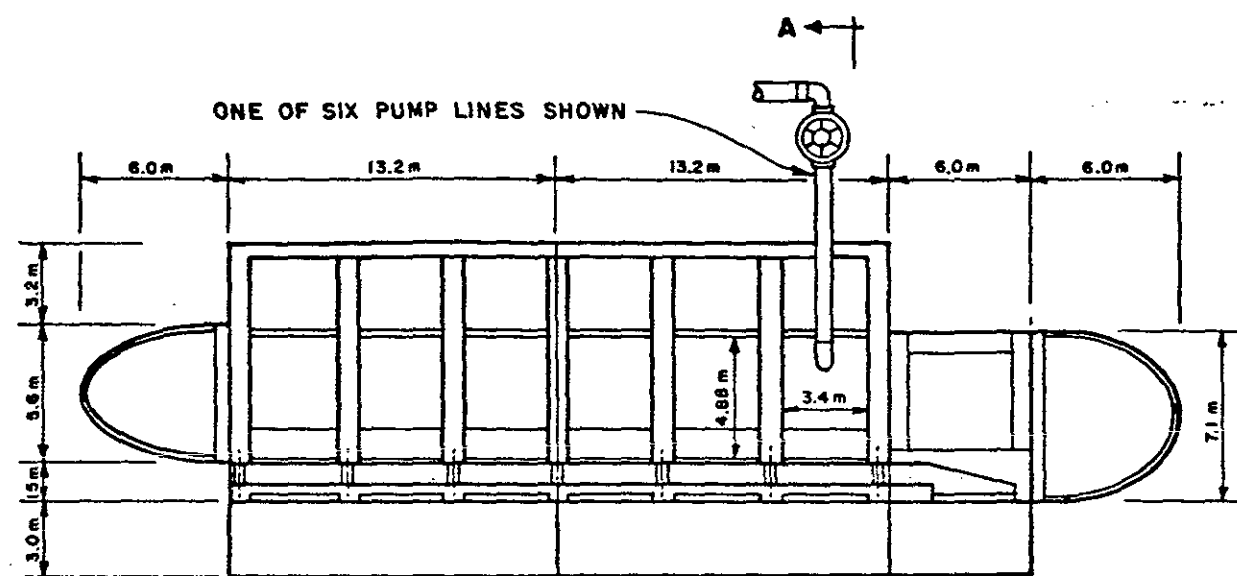


TOP VIEW OF MODEL IN FLUME EMBAYMENT

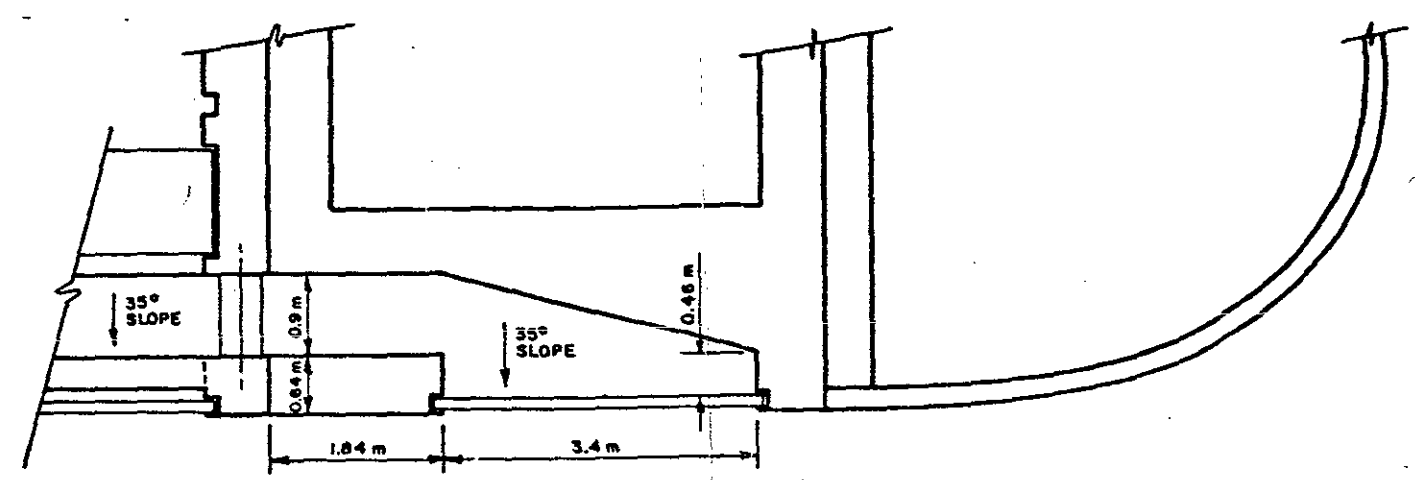


VERTICAL SECTION THROUGH FLUME EMBAYMENT

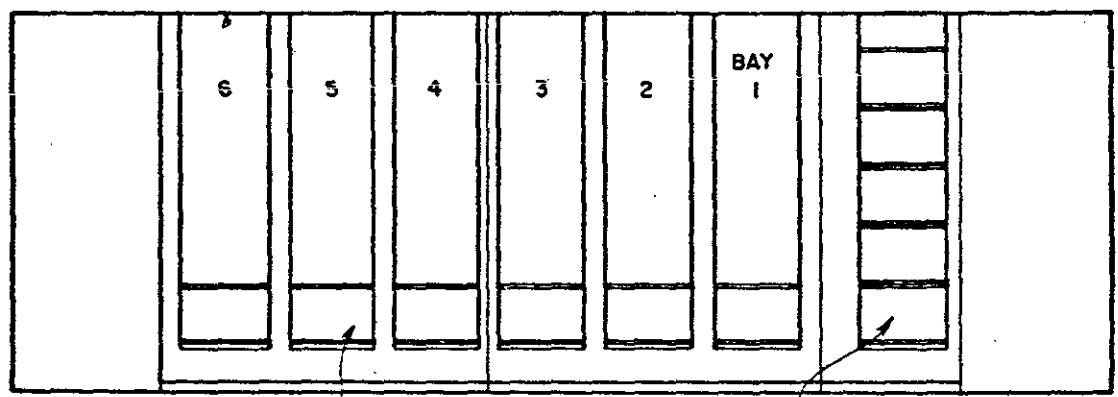
B.C. HYDRO AND POWER AUTHORITY		
HAT CREEK COOLING WATER SUPPLY HYDROLOGIC MODEL OF INTAKE		
INTAKE MODEL LOCATION IN LABORATORY FLUME		
northwest hydraulic consultants ltd.		
Drawn: I. M.	Date: NOV. 1977	Dwg: 3.2.1



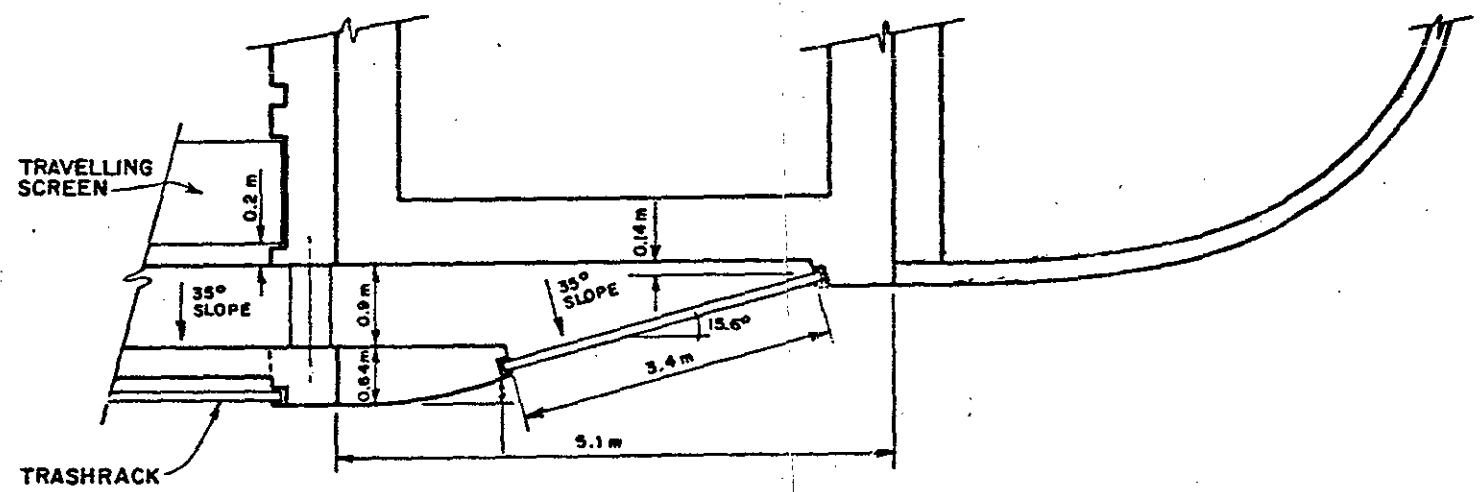
PLAN



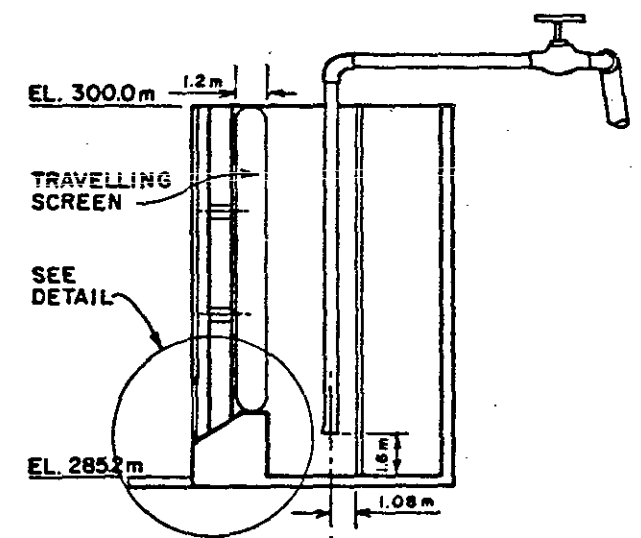
BYPASS ENTRANCE GEOMETRY A



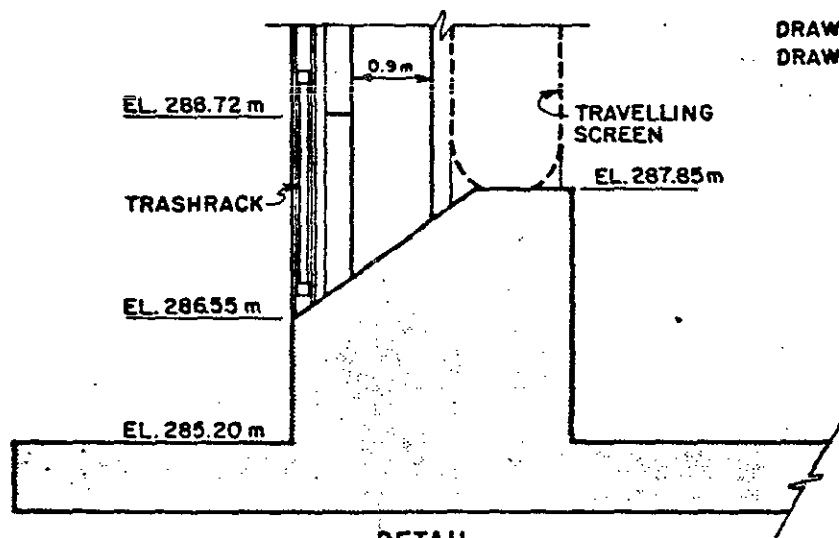
FRONT VIEW



BYPASS ENTRANCE GEOMETRY B



SECTION A-A



DETAIL

DRAWING TO MODEL SCALE 1:2
DRAWING TO PROTOTYPE SCALE 1:60

B.C. HYDRO AND POWER AUTHORITY		
MAT CREEK COOLING WATER SUPPLY		
HYDRAULIC MODEL OF INTAKE		
INTAKE MODEL CONFIGURATION		
northwest hydraulic consultants ltd.		
Drawn I. M.	Date NOV. 1977	Dwg: 3.2.2

0.176 m x 0.032 m BARS



15 BARS AT 45° TO FRONT FACE 0.23 m SPACING

0.16 m x 0.032 m BARS



18 BARS AT 60° TO FRONT FACE 0.19 m SPACING

0.144 m x 0.032 m BARS

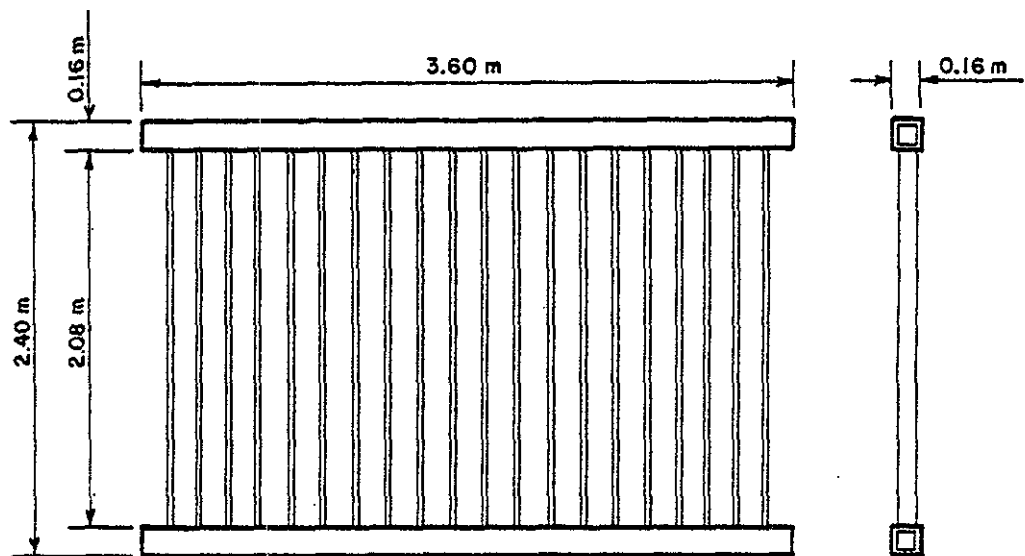


20 BARS AT 75° TO FRONT FACE 0.17 m SPACING

0.128 m x 0.032 m BARS



20 BARS AT 90° TO FRONT FACE 0.16 m SPACING



B.C. HYDRO AND POWER AUTHORITY

HAT CREEK COOLING WATER SUPPLY
HYDRAULIC MODEL OF INTAKE

TRASHRACK ARRANGEMENTS

northwest hydraulic consultants ltd.

Drawn: I. M.

Date: NOV. 1977

Dwg: 3.2.3

APPENDIX C
VELOCITY DATA

TABLE C1
 VELOCITY DATA FOR GEOMETRY B WITH DIFFERENT TRASHRACK CONFIGURATIONS AND DIFFERENT FLOWS
 TESTS A1 - A9

Run Number	Configuration	River Discharge m ³ /s (cfs)	River Depth* m (ft)	River Velocity m/s (fps)	Elevation of Velocity Measurement M	Velocity in m/s (fps)						
						Upstream of Bay 1	Upstream of Bay 2	Upstream of Bay 3	Upstream of Bay 4	Upstream of Bay 5	Upstream of Bay 6	Downstream of Bay 7
A1	90° Trashracks	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.35(1.5)	-	-	-	-	-	-
A2	60° Trashracks	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.37(1.21)	0.23(0.75)	0.22(0.72)	0.20(0.66)	0.19(0.62)	0.17(0.56)	0.16(0.52)
A3	45° Trashracks	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.47(1.54)	0.30(0.98)	0.30(0.98)	0.29(0.95)	0.33(1.08)	0.30(0.98)	0.29(0.95)
A4	No Trashracks	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.49(1.61)	0.34(1.12)	0.42(1.38)	0.42(1.38)	0.49(1.61)	0.49(1.61)	0.45(1.48)
A5	Round Bars	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.47(1.54)	0.27(0.89)	0.27(0.89)	0.27(0.89)	0.25(0.82)	0.27(0.89)	0.21(0.69)
A6	45° Trashracks	200(7,000)	3.6(11.8)	0.76(2.5)	288	0.65(2.13)	0.45(1.48)	0.37(1.21)	0.31(1.02)	0.30(0.98)	0.28(0.92)	0.29(0.95)
A7	45° Trashracks	285(10,000)	4.3(14.1)	1.0 (3.3)	288	0.95(3.12)	0.76(2.49)	0.57(1.87)	0.57(1.87)	0.57(1.87)	0.57(1.87)	0.66(2.16)
A8	45° Trashracks	1980(70,000)	8.9(29.2)	2.5 (8.2)	294	1.28(4.20)	1.21(3.97)	1.21(3.97)	1.18(3.87)	1.18(3.87)	1.18(3.87)	1.25(4.10)
					291	1.28(4.20)	1.06(3.48)	0.87(2.85)	0.83(2.72)	0.80(2.62)	0.91(2.98)	1.02(5.31)
					288	1.65(5.41)	1.21(3.97)	1.21(3.97)	1.18(3.87)	1.18(3.87)	1.10(3.61)	1.06(3.48)
A9	45° Trashracks	4530(160,000)	11.8(38.7)	4.0(13.0)	297	2.05(8.20)	1.86(6.10)	1.86(6.10)	1.86(6.10)	1.86(6.10)	1.86(6.10)	2.05(6.72)
					294	1.52(4.99)	1.67(5.48)	1.52(4.99)	1.52(4.99)	1.52(4.99)	1.52(4.99)	1.67(5.48)
					291	2.21(7.25)	1.86(6.10)	1.67(5.48)	1.67(5.48)	1.67(5.48)	1.67(5.48)	1.67(5.48)
					288	2.40(7.87)	2.21(7.25)	1.86(6.10)	1.86(6.10)	1.86(6.10)	1.86(6.10)	

*Depth above bed at El. 286.2

TABLE C2
 VELOCITY DATA FOR GEOMETRY B WITH 45° TRASHRACKS AT 115 m³/s (4000 cfs)
 THE VARIOUS PUMP CONFIGURATION TESTS B1 - B5

Run Number	Configuration	River Discharge m ³ /s (cfs)	River Depth* m (ft)	River Velocity m/s (fps)	Elevation of Velocity Measurement M	Velocity in m/s (fps)						
						Upstream of Bay 1	Upstream of Bay 2	Upstream of Bay 3	Upstream of Bay 4	Upstream of Bay 5	Upstream of Bay 6	Downstream of Bay 6
B1	No pumps on	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.34(1.12)	0.29(0.95)	0.29(0.95)	0.27(0.89)	0.27(0.89)	0.29(0.95)	0.33(1.08)
B2	Four downstream pumps on	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.33(1.25)	0.31(1.02)	0.31(1.02)	0.34(1.12)	0.25(0.82)	0.25(0.82)	0.25(0.82)
B3	Two downstream and two upstream pumps on	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.46(1.51)	0.23(0.75)	0.23(0.75)	0.19(0.62)	0.25(0.82)	0.25(0.82)	0.25(0.82)
B4	Four upstream pumps on	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.46(1.51)	0.25(0.82)	0.21(0.69)	0.19(0.62)	0.17(0.56)	0.19(0.62)	0.29(0.95)
B5	All pumps at 1/3 capacity	115(4,000)	2.8(9.2)	0.52(1.7)	288	0.35(1.15)	0.27(0.89)	0.19(0.62)	0.20(0.66)	0.25(0.82)	0.28(0.92)	0.34(1.12)

*Depth above bed at El. 286.2

TABLE C3
VELOCITY DATA FOR GEOMETRY B
THE VELOCITY RESTRICTION TESTS C1 - C6

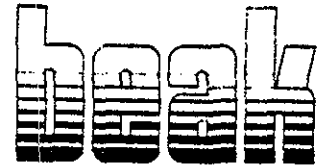
Run Number	Configuration	River Discharge m ³ /s (cfs)	River Depth* m (ft)	River Velocity m/s (fps)	Elevation of Velocity Measurement M	Velocity in m/s (fps)						
						Upstream of Bay 1	Upstream of Bay 2	Upstream of Bay 3	Upstream of Bay 4	Upstream of Bay 5	Upstream of Bay 6	Downstream of Bay 6
C1	Full depth trashracks pointing 45° downstream	1980(70,000)	8.9(29.2)	2.5(8.2)	294	0.73(2.39)	0.50(1.64)	0.46(1.51)	0.58(1.90)	0.65(2.13)	0.76(2.49)	0.91(2.98)
					288	0.95(3.12)	0.65(2.13)	0.65(2.13)	0.69(2.26)	0.73(2.39)	0.87(2.85)	0.99(3.25)
C2	Full depth trashracks 90°	1980(70,000)	8.9(29.2)	2.5(8.2)	294	1.25(4.10)	0.99(3.25)	0.99(3.25)	0.91(2.98)	0.91(2.98)	0.99(3.25)	1.21(3.97)
					288	1.31(4.30)	0.87(2.85)	0.87(2.85)	0.99(3.25)	0.99(3.25)	0.99(3.25)	0.99(3.25)
C3	Full depth trashracks part blocked off on downstream side	1980(70,000)	8.9(29.2)	2.5(8.2)	294	1.54(5.05)	1.25(4.10)	1.18(3.87)	1.10(3.61)	1.28(4.20)	1.28(4.20)	1.31(4.30)
					288	1.84(6.04)	1.31(4.30)	1.43(4.69)	1.31(4.30)	1.31(4.30)	1.28(4.20)	1.21(3.97)
C4	Full depth trashracks part blocked off on upstream side	1980(70,000)	8.9(29.2)	2.5(8.2)	294	1.28(4.20)	1.10(3.61)	1.10(3.61)	1.18(3.87)	1.18(3.87)	1.21(3.97)	1.35(4.43)
					288	1.66(5.44)	1.25(4.10)	1.25(4.10)	1.28(4.20)	1.28(4.20)	1.21(3.97)	1.10(3.61)
C5	45° trashracks in bypass entrance reversed	4530(160,000)	11.8(38.7)	4.0(13.0)	297	0.87(2.85)	0.50(1.64)	0.46(1.51)	0.54(1.77)	0.73(2.39)	0.95(3.12)	1.14(3.74)
					294	1.06(3.48)	1.06(3.48)	1.02(3.35)	0.87(2.85)	0.87(2.85)	1.06(3.48)	1.28(4.20)
					288	1.06(3.48)	0.91(2.98)	1.14(3.74)	1.14(3.74)	1.18(3.87)	1.28(4.20)	1.52(4.99)
C6	90° trashracks in bypass entrance	4530(160,000)	11.8(38.7)	4.0(13.0)	297	1.86(6.10)	1.52(4.99)	1.33(4.36)	1.14(3.74)	1.33(4.36)	1.33(4.36)	1.52(4.99)
					294	1.52(4.99)	1.33(4.36)	1.33(4.36)	1.14(3.74)	1.14(3.74)	1.33(4.36)	1.25(4.10)
					288	1.86(6.10)	1.52(4.99)	1.52(4.99)	1.52(4.99)	1.52(4.99)	1.25(4.10)	1.67(5.48)

*Depth above bed at El. 286.2

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SUSPENDED SEDIMENT CHARACTERISTICS
OF THE THOMPSON RIVER AND EFFECTS
OF ALGAE GROWTH ON HAT CREEK WATER
SUPPLY SYSTEMS

Prepared for:

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Submitted by:

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Vancouver B.C.



A MEMBER OF THE SANDWELL GROUP

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SUMMARY

On February 23, March 17, and June 2, 1977, Beak Consultants Limited collected water samples from the Thompson River at Walhachin Bridge in order to provide supplementary information describing seasonal fluctuations in sediment loads and to evaluate the effects of algae growth on proposed Hat Creek water intake systems. Analyses included suspended sediment (mg/l), particle size, and phytoplankton composition and relative abundance.

Suspended sediment concentrations monitored during the sampling period (less than 0.1 to 2.0 mg/l) are indicative of suspended sediment loads which prevail in the Thompson River during the low flow period from November to April. During rising freshet (May-June) suspended sediment concentrations may reach as high as 91.0 mg/l (Northwest Hydraulic Consultants Limited, 1976). Particle size analyses of sediments obtained during the program indicate that the majority of particles were less than 50 microns in diameters.

During the low flow period from November to March reduced solar radiation and presence of ice cover in winter reduce phytoplankton densities but favor periphyton growth in late fall and early spring (Langer and Nassichuk, 1975; Bothwell, 1977). Periphyton (colonizing algae which attaches to submerged substrates) occur in great abundance in the Thompson River and would appear at the intake structure during low flow. This growth, however, is not expected to be of concern in fouling travelling screens or clogging trash racks. Although some growth can be expected on travelling screens, this will be kept to a minimum by pressurized water spray and low light levels. Phytoplankton of the kinds observed during the survey would pass through the screens and pumped without difficulty. If exposed to light, growth of periphyton on the walls of the degritting clarifier will be rapid, but would not be a problem if light were excluded from the clarifier.

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The absence of light will control nuisance growth in the intake pipe, however, bacterial growth could occur. These growths are not anticipated to reach nuisance levels. It is our opinion that there will be no need for filters to control algae prior to pumping. If media filters are necessary to eliminate suspended sediments, some clogging by phytoplankton and bacteria can be expected during peak periods of production.

Chlorination of filtering and pipeline systems is not considered necessary to control algae growth. If algae grows in the clarifier it will be pumped without problem.

INTRODUCTION

As part of the Hat Creek water supply study, Beak Consultants Limited was requested by Sandwell and Company Limited (Sandwell) to conduct water sampling of the Thompson River near Ashcroft, British Columbia, between February and June, 1977, and to evaluate the effect of algae growth in the Thompson River on the Hat Creek water supply systems. This report presents the results of the water analyses and discusses briefly the implications of potential algae fouling and entrainment problems associated with screen, filter, and pump systems of the proposed water intake system on the Thompson River.

Water sampling was conducted on three occasions, February 23, March 17, and June 2, 1977 to supplement existing information describing seasonal fluctuations in suspended sediment levels and species composition and relative abundance of phytoplankton (usually single celled or short chained algae that drift in the current). Periphyton (algae which grow attached to submerged surfaces) are taken into consideration but were not sampled during the field program. Early in the field program it became apparent that although suspended sediment concentrations were low throughout much of the year, it was the abrasiveness of particles constituting normal sediment loads which were of concern in designing booster pumping systems. Suspended sediment analyses were therefore expanded to include particle size analysis.

MATERIALS AND METHODS

Suspended Sediment

Due to the size of the Thompson River and the need for midstream sampling, field sampling was conducted from Walhachin Bridge located approximately 21.5 km (13.4 miles) upstream of the confluence of the Bonaparte and Thompson Rivers near Ashcroft. Water samples were obtained at three points across the river which approximated 1/4, 1/2 and 3/4 of the total width. Samples were collected using a CAE model D49 depth integrating suspended sediment sampler. A detailed description of the sampler is provided in Appendix A. Approximately 12 subsamples of 350 ml were obtained at each transect point and combined to achieve a total sample volume of 4 litres. Samples were collected over a continuous depth range from surface to bottom. Water samples were stored unpreserved in clean polyvinyl chloride bottles and returned to BEAK laboratories for analyses of either suspended solids and/or particle size.

Laboratory analyses of suspended solids were determined by gravimetric methods as outlined in APHA Standard Methods (1975). Particle size analyses including sediment density and size were determined using a Coulter R Counter Model B equipped with a Model M Volume Converter as described by Sheldon and Parsons (1967).

Two determinations were made per sample using 560 and 100 micron diameter apertures. The former was used to measure the total spectrum of particle size and the latter to measure smaller size ranges which dominated the Thompson River samples. The 560 micron aperture was calibrated using corn pollen (mean diameter 89.8 μ , volume 379,000 μ^3), the 100 micron aperture using ragweed pollen (mean diameters 19.5 μ , volume 3,880 μ^3).

The operation of a Coulter Counter requires that the medium be an electrolyte, therefore sample water was diluted 1:1 with filtered sea water (15 psi vacuum through a 0.35 Ultipor RDFA filter) to increase its conductivity. A "blank" sample was run which consisted of a 1:1 dilution of filtered sea water and tap water.

Algae

On each sampling occasion a single phytoplankton sample was collected at midstream (1/2 width transect point). Samples were collected from surface to bottom using the depth integrated suspended sediment sampler. Samples were placed in 1 litre polyvinyl chloride bottles and preserved with Lugol's solution. Water samples were prepared for plankton counts and identification by filtration through a 25mm 0.45 um MF-Millipore filter with subsequent clarification of the filter using immersion oil (Rand et al. 1976). Algal units (300) within a diameter transect of the effective circular filter surface were identified and counted at a magnification of 400 x (10x oculars) with an A0 phase contrast microscope. subsequent examination of difficult taxa was made with oil immersion at 1000 x and in settling chambers with Wild M40 inverted microscope. Computer programs, developed by BEAK were used to summarize and format data derived from this study.

RESULTS AND DISCUSSION

Suspended Sediment Loading in the Thompson River

Suspended sediment concentrations observed in the Thompson River near Ashcroft are summarized in Table 1. These data include those obtained during the field survey as well as those described in recent reports. In order to place sediment loadings characteristics in perspective with river flow, corresponding discharge information for the Thompson River above the Bonaparte River is also provided. In addition, a hydrograph (Drawing No. D4191-SK78) prepared by Sandwell Company for 1976 and 1977 Thompson River flows at Spences Bridge is shown in Figure 1.

Maximum suspended sediment concentrations appear to occur in the Thompson River as velocities and discharge peak during rising freshet. Highest recorded sediment concentrations (91.0 mg/l) were measured by Northwest Hydraulic Consultants Limited (NHCL) at Ashcroft Bridge on 15 May 1976 at a corresponding discharge (measured at Spences Bridge) of $2260 \text{ m}^3/\text{s}$ (79800 CFS). On the same day water samples taken at Walhachin Bridge indicated less sediment loading (9.0 to 13.0 mg/l) further upstream. Suspended sediment levels measured by NHCL at Walhachin Bridge, although considerably less than those monitored at Ashcroft still represent high levels for the Thompson River. In a summary of the water quality analyses for the Thompson River (B.C. Hydro & Power Authority) average and maximum suspended sediments monitored at station No. 0600004, Savona B.C., by the Pollution Control Branch (PCB) are recorded as 3.1 mg/l and 7.6 mg/l, respectively. This period includes the high flow year of 1972 when peak Thompson River discharges reached $4,134 \text{ m}^3/\text{s}$ (146,000 CFS) on 15 June 1972 (Figure 1). B.C. Research and Dolmage & Campbell & Associates Ltd. (1975) report maximum recorded suspended concentrations of 9.0 mg/l were recorded by the PCB on 19 June 1972, four days following maximum recorded peak discharge.

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

	Date	Source	Location	Concentration (mg/l)	Discharge*	
					m ³ /s	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	190	6800
	17/03/77	4	Walhachin Bridge	1.0-2.0		
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

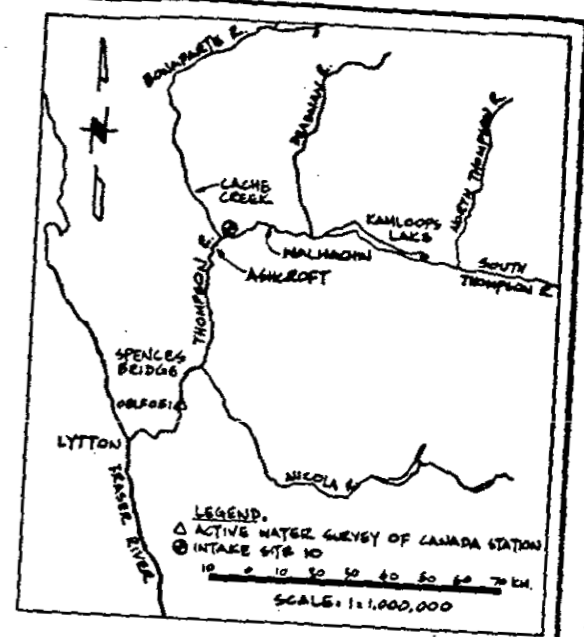
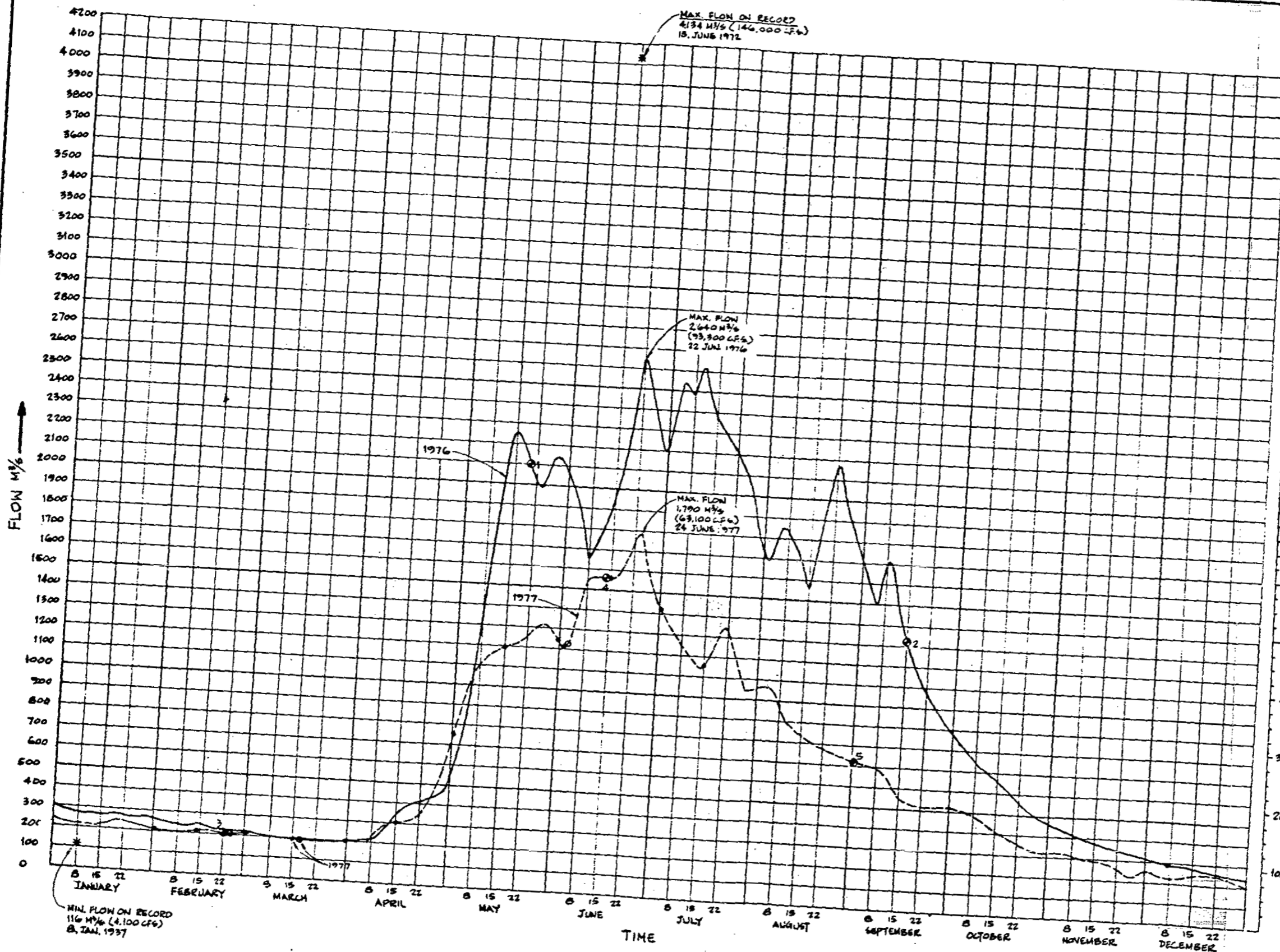
SOURCES

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

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- LEGEND:**
- WATER LEVELS TAKEN AT INTAKE SITE 10.
 - ⊙ FIELD VISITS MADE AT INTAKE SITE 10.
 - WATER SAMPLES TAKEN AT MALHACHIN 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

NR	DATE	REMARKS
1	20 MAY 1976	FIRST HELICOPTER VISIT
2	18 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

FIGURE 1

SANDWELL DWG. NO. D4191-SK 78		BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
DES	REC'D	HAT CREEK PROJECT	
DWN	APPROV	COOLING WATER SUPPLY	
CHKD	APPROV	PRELIMINARY DESIGN STUDY	
INSP	APPROV	THOMPSON RIVER HYDROGRAPHS, 1976 & 1977	
SURD	DATE	DEC. 1977	
MICROFILMED	JOB NO. V. 4. 71	SCALE AS NOTED	DWG. NO. D4191-SK 78

During the 1977 BEAK water survey, suspended solids were measured at each of three transect points across the Thompson River at Walhachin Bridge on three occasions. The results of the survey are presented by transect in Table 2. Concentrations of suspended solids ranged from 1.0 mg/l in late February to 2.0 mg/l in early June. Little variation in suspended sediment concentrations was observed across the river transect on any of the sampling occasions. The overall low values obtained on the survey are considered representative of suspended sediment concentrations generally present in the river during low flow periods from November through April. The June 2 sampling results suggest that in low flow years river waters can be relatively solids free as late as June. Discharge at Walhachin Bridge at the June 2 sampling was estimated at 1030 m³/s (36,400 CFS). Thompson River discharge at Spences Bridge peaked on 24 June at 1,790 m³/s (63,100 CFS) (Figure 1).

Particle Size Analysis

As indicated by the available suspended sediment data for the Thompson River, suspended sediment loading under normal conditions is particularly low. Results of the particle size analyses further indicated that those particles present under normal conditions are in the silt/clay category.

The particle size distribution and frequency of occurrence of suspended sediments and samples obtained at Walhachin Bridge are given in Tables 3 and 4. In the March 18 samples no particles were detected with the 560 micron aperture (capable of measuring particles up to 560 microns). A more sensitive 100 micron aperture indicated the greatest portion (71%) of detectable particles ranged in size from 20 microns to 41 microns and less than 1% of the observed particles were greater than 52 microns.

Particle size analyses of samples obtained during the June 2 sampling period indicated similar distributions of detectable particles. Approximately 75% of the particles observed in the transect samples were contained in the 12.0 to 17.3 micron size range. No data are available for particles smaller than

TABLE 2 Suspended Sediment Concentrations Measured at Transect Points Across the Thompson River at Walhachin Bridge During the 1977 Survey.

Date	Transect Point			Discharge*	
	1/4	1/2	3/4	m ³ /s	CFS
23/02/77	2.0	1.0	1.0	No Data	
17/03/77	1.0	2.0	1.0	190	(700)
02/06/77	2.0	2.0	2.0	1030	(364000)

* Estimated Thompson River discharge above the Bonaparte River, Inland Waters Branch, Water Survey of Canada, Vancouver, 1977.

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Table 3. Size Distribution of Suspended Sediment from the Thompson River Sampled at Walhachin Bridge, March 17, 1977.

<u>APERTURE</u>	<u>DIAMETER (μ)</u>	<u>Particles/ml</u>		
		<u>$\frac{1}{2}$ width</u>	<u>$\frac{1}{2}$ width</u>	<u>$\frac{3}{4}$ width</u>
100 μ	<20	beyond detection range		
	20-28	0.0	6.0	26
	28-33	1.2	7.5	9.0
	33-37	0.0	5.5	2.8
	37-41	0.3	1.2	2.8
	41-43	2.0	1.2	1.8
	43-46	2.5	2.2	2.0
	46-48	1.2	2.0	1.2
	48-50	1.8	1.6	1.0
50-52	1.2	2.3	1.8	
560 μ	>52	0.1	<0.1	<0.1

Table 4 Size Distribution of Suspended Sediment from the Thompson River Sampled at Walhachin Bridge, June 2, 1977

Aperture	Diameter (μ)	Particles/ml		
		993	994	995
	<12.0	Beyond detection range		
100 μ	12.0 - 17.3	154 \pm 18.2	204 \pm 20.7	167 \pm 21.5
	17.3 - 20.5	22.5 \pm 7.1	30.5 \pm 7.5	23.0 \pm 7.2
	20.5 - 22.9	13.8 \pm 3.2	11.5 \pm 2.3	10.0 \pm 2.4
	22.9 - 24.9	4.7 \pm 1.4	4.4 \pm 2.5	3.7 \pm 2.9
	24.9 - 26.6	2.1 \pm 1.9	3.5 \pm 1.4	1.8 \pm 2.2
	26.6 - 28.2	2.1 \pm 1.4	2.5 \pm 3.0	1.8 \pm 1.9
	28.2 - 29.5	2.0 \pm 1.1	2.8 \pm 1.3	0.7 \pm 0.8
	29.5 - 30.8	2.1 \pm 1.9	2.5 \pm 3.5	0.5 \pm 1.2
	30.8 - 32.0	2.7 \pm 2.0	2.0 \pm 0.9	1.3 \pm 1.8
560 μ	26.4 - 38.1	0.6 \pm 1.8	0.0 \pm 1.2	0.5 \pm 1.4
	38.1 - 45.2	0.3 \pm 0.4	0.2 \pm 0.3	0.0 \pm 0.0
	45.2 - 50.5	0.2 \pm 0.3	<0.1 \pm 0.1	0.0 \pm 0.0
	50.5 - 55.0	0.0 \pm 0.0	0.0 \pm 0.0	0.0 \pm 0.0
	55.0 - 58.8	0.0 \pm 0.0	0.1 \pm 0.3	0.1 \pm 0.2
	58.8 - 62.1	0.1 \pm 0.3	0.0 \pm 0.0	0.0 \pm 0.0
	62.1 - 65.2	0.0 \pm 0.0	0.0 \pm 0.0	0.0 \pm 0.0
	65.2 - 67.9	0.0 \pm 0.0	0.1 \pm 0.2	0.0 \pm 0.0
	67.9 - 70.5	<0.1 \pm 0.1	0.0 \pm 0.0	0.0 \pm 0.0
	70.5 - 90.3	Beyond detection range		
	90.3 - 101	0.0 \pm 0.7	0.0 \pm 1.1	1.0 \pm 1.3
	101 - 110	0.1 \pm 0.6	0.4 \pm 0.9	0.0 \pm 0.7
	110 - 118	0.2 \pm 0.6	0.0 \pm 0.4	0.2 \pm 0.4
	118 - 124	<0.1 \pm 0.5	0.0 \pm 0.4	0.1 \pm 0.4
	124 - 130	0.2 \pm 0.6	0.1 \pm 0.4	0.2 \pm 0.6
	130 - 136	0.0 \pm 0.0	0.0 \pm 0.0	0.2 \pm 0.3
	136 - 141	0.0 \pm 0.0	0.0 \pm 0.0	0.0 \pm 0.0
>141	Beyond detection range			

12 microns or larger than 41 microns and 560 micron apertures, respectively. Similarly, no data are available for particles 70.5 to 90.3 microns in diameter. It is believed, however, that the great majority of particles were included in the size ranges counted.

Algae Growth in the Thompson River

A schematic representation of algae growth and of the hydrological regime above the Bonaparte River and below Kamloops Lake is provided in Figure 2. Sources of this information were the short-term studies of Servizi (1976), Langer and Nassichuk, (1975), the present study, and a study of the lower Fraser by Northcote, Ennis and Anderson (1975). Algae growth is primarily controlled by nutrient, light and hydrologic regimes. Ice cover during winter and decreased solar radiation reduce phytoplankton densities during the period of low water from November to March. These conditions are, however, favorable to periphyton growth which peaks in early spring and late fall (Langer and Nassichuk, 1975; Bothwell, 1977). Results of the BEAK phytoplankton study on the Thompson River in 1977 show phytoplankton densities increased markedly from February through June (Appendix B). These data and information describing phytoplankton abundance in the upper Fraser River (Northcote, Ennis and Anderson, 1975), suggest phytoplankton densities may be at their highest in early summer, with low densities occurring through fall.

Nutrient enrichment from human activity in the upper Thompson River has produced an increasing longitudinal periphyton density gradient downstream of Kamloops Lake (Langer and Nassichuk, 1975; Bothwell, 1977). During the early spring and late fall the periphyton is dominated by a few species of diatoms of the genera *Gomphonema*, *Cymbella* and *Fragilaria*. In spite of their small individual cell sizes (15-50 um), their growth habit of attachment by gelatinous stalks and ribbon-like proliferation produced fluffy or stringy mats on rock substrates. Trapped air bubbles and water turbulence remove clumps or portions of the growth which are subsequently transported downstream.

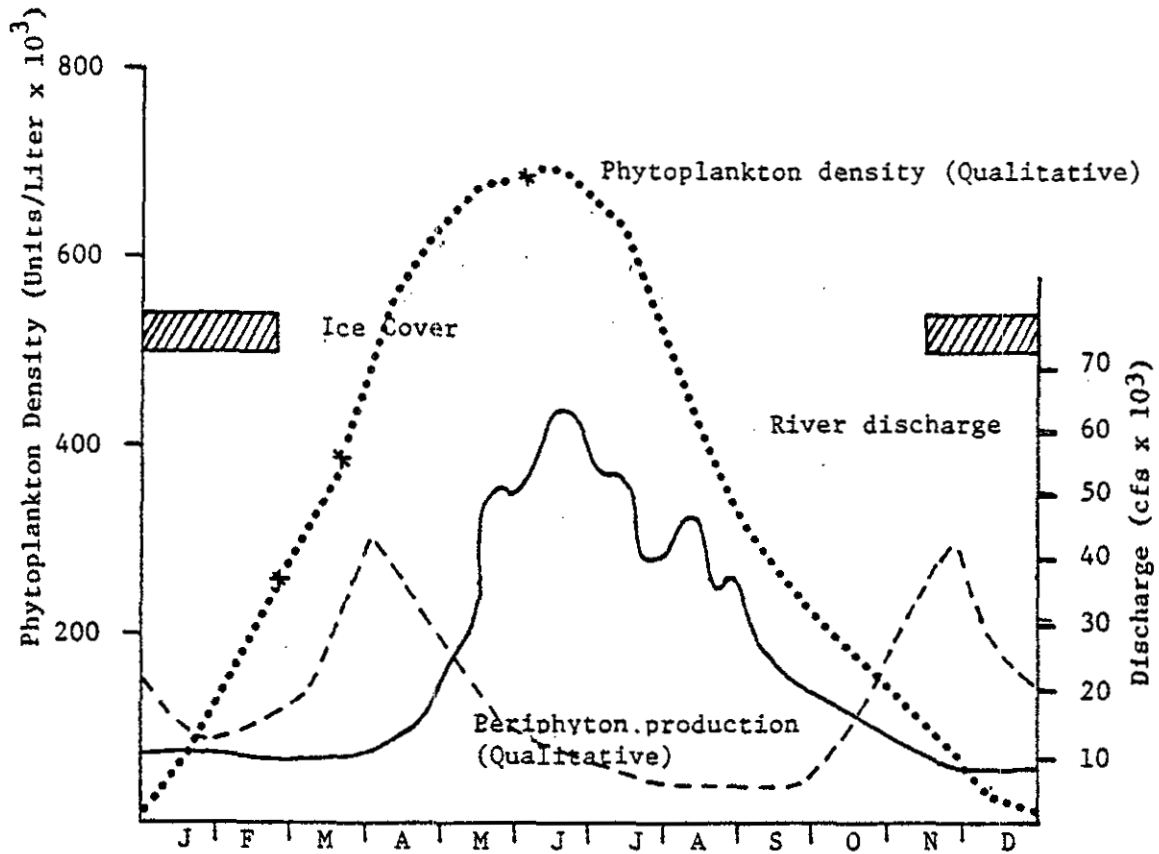


Figure 2 Qualitative periphytic algae production and a typical hydrologic regime in the Thompson River near Waltham, B.C. based on Langer and Nassichuk (1975), and Sandwell, Vancouver, B.C. drawing No. D4191-SK47. Phytoplankton densities from data in Appendix 2 of this report (*) and from Northcote, Ennis and Anderson (1975).

Entrapped air may enable pieces to drift near the surface of the river. The total amount of periphyton material sloughed off during the highest growth period may be considerable. The size of the fragments which would be present in the river cannot be determined from previous work, however, from our experience clumps less than 5 to 8 cm in diameter and mat-like sections 24 x 1 cm may be expected. The composition of these materials (small cells attached by gelatinous material) allows rapid disassociation in turbulent water.

During summer growth periods the composition and density of the periphyton differs from the low-flow cold weather period. Inferring from Northcote, *et.al.* (1975) green algae (e.g. *Ulthrix*) are prominent along with diatoms. Phytoplankton densities increased from 262,871 units/liter February 23 to 383,332 units/liter on March 17 to a high of 692,265 units/liter on June 2, 1977 (Appendix B). Dominant algae in February were the pyrrophyte flagellate (*Rhodomonas minuta*), a green *Chlorella*-like cell, and two diatoms *Acnantes minutissima* and *Synedra vaucheriae*. *Rhodomonas* and the *Chlorella*-like algae were probably derived from the plankton of Kamloops Lake. The two diatoms were probably derived through periphyton washing into suspension. A total of 54 different taxa of algae were observed. In March the phytoplankton were dominated by diatoms and the *Chlorella*-like algae. Again, the diatoms were typical periphyton forms. A total of 55 different taxa were observed in March. In June the composition of the phytoplankton was typical of a lake assemblage with relatively few periphyton forms. Dominant were pyrrophytes (*Rhodomonas* spp.), chrysophytes (e.g., *Rhizolenia*) and various planktonic diatoms (*Asterionella*, *Cyclotella* spp.). The periphytic diatom *Acnantes minutissima* was, however, still present. Total taxa observed in June were 46.

In summary, the composition of the pytoplankton reflects both the influence of the river periphyton (especially in March when periphyton growth has previously been greatest) and the phytoplankton of Kamloops Lake. The forms of planktonic algae observed were primarily discrete single-cell units rather

than long, filamentous diatoms or blue green algae. It can be assumed that blue-green algae which are filamentous (*Oscillatoria*, *Arthrospira*, *Aphanizomenon* and *Anabaena*) are present in late summer and fall.

The literature did not contain reports of excessive growths of macrophytic flowering plants and evidence of such growth was not noted during a site visit on September 21, 1977.

Hat Creek Water Supply Systems and Algae Growth

Intake Structure The potential for clogging the openings of the traveling screen (.254 cm) and trash racks (15.25 cm) were two primary concerns expressed by Sandwell. Floating pieces of periphyton material which will appear at the structure in greatest abundance during low water, will be easily removed from the traveling screens by the pressurized water screen wash. Most of these pieces will be reduced to small clusters of cells by the water pressure and passed through the screens or out the screen wash sluice. Growth of periphyton on the screens will be kept to a minimum by the pressurized water spray and by the very low level of light at the screens. Phytoplankton of the kind BEAK observed February-June, 1977, and that are expected from growth in Kamloops Lake should easily pass through the screens. Due to the nature of the nutrient enrichment of the Thompson River and Kamloops Lake the growth of the slime bacteria *Sphaerotilus* has been expected for many years. This heterotrophic organism could grow on surfaces in the dark and has caused fouling of aquatic structures in rivers where it has been abundant. Investigations on the Thompson River by Servizi and Brukhalter (1970) and Servizi (1976) on the presence of *Sphaerotilus* growth in 1964, 1965 and 1973 showed an absence of the bacterial slime on submerged unglazed tiles. This would suggest that growth of slime bacteria in pipelines and the intake structure will be minimal if occurring at all. No effect upon the operation or maintenance of the structure is expected.

It is likely that periphyton will grow on the lower trash racks especially during low flow periods. Turbulence of the water through the racks should prevent flocculant accumulation of algae on the rack surfaces and no problem is anticipated at the intake. Algal or bacterial problems have not been encountered at the Lornex intake downstream on the Thompson River (A. Copeland, personal communication).

Degritting Clarifier Most of the algae in the Thompson River will pass through the traveling screens. Much of this drift algae will consist of periphytic diatoms able to colonize stable substrates. If the walls of the degritting clarifier are exposed to sunlight they will be rapidly covered with periphyton. Periphytic material will slough off periodically and be carried through the pumps and piping to the power plant. Excluding light from the degritting clarifier will greatly reduce this. Even if growth and sloughing occurs it is not expected that it will cause any operational or maintenance problems. The high head pumps will easily fragment any algal material that is pumped.

Media Filters Given the relative abundance and species composition of phytoplankton in the Thompson River, there appears to be no need to filter algae from makeup water prior to pumping. Should media filters be required to remove suspended sediments, phytoplankton would be filtered routinely as other particulate matter at no disproportionate loss in efficiency to the filtering media. Most algae present in the Thompson River would pass through the media bed, however, larger species such as *Asterionella formosa*, *Cyclotella* spp., *Synedra* spp. and *Fragilaria* spp. and dislodged periphyton would be collected. During peak periods of phytoplankton production, greater numbers would be filtered, however, normal backwashing and filter maintenance is considered sufficient to avoid any problems attributed to clogging by algae. Since most filters are positioned in enclosed containers, environmental conditions are unsuitable for algae growth in or on the surface of the filtering media.

Some bacteria growth may occur in the absence of light, however, no slime producing bacterial have been found in the Thompson River, hence are not anticipated to occur in the system.

Chlorination Based on conclusions drawn by the Boeing Company (Boardman Environmental Analysis, 1976) regarding treatment of makeup water supplied to a proposed coal fired power plant station on the Columbia River, it appears that chlorination of makeup water from the Thompson River would not be required to prevent nuisance algae and bacterial growths in the intake and pipeline systems. At the proposed Boardman station a review of nearby irrigation systems using nutrient rich Columbia River waters indicated chlorination was not necessary to maintain flow in large diameter pipelines. In comparison to the Thompson River, phytoplankton densities in the Columbia River reach some 8 to 29 times the densities (Beak, 1975-1977a) observed in the Thompson River.

Lack of light in the intake pipe will be sufficient to control nuisance algae, however, some bacterial growth could occur. Since the primary slime producing bacteria *Sphaerotilus* is not anticipated in the Thompson River, nuisance bacterial growths would not likely be encountered. Should some buildups occur the scouring action of water flow within the intake pipe is considered sufficient to control accumulation before clogging or significantly reduced flows occur.

Similarly there appears to be no need to chlorinate media filters to control algae prior to pumping. Phytoplankton and dislodged filamentous algae present in the makeup water would be collected in the filtering media regardless of being killed prior to entering the media bed. Chlorination would reduce potential bacterial growths in the filtering media. However, since no nuisance bacterial growths are anticipated, and conditions are unfavourable for algae growth, there appears to be no need for chlorination pretreatment. Backwashing of the filtering media is considered sufficient to assure continued filtering efficiency.

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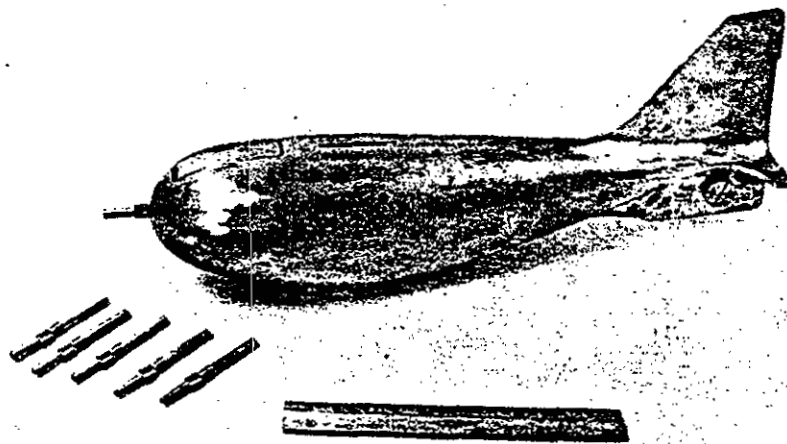
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APPENDIX A

D 49 - CAE Depth Integrating
Suspended Sediment Sampler



**D-49 SUSPENDED SEDIMENT SAMPLER
CAE MODEL 314 c/w CASE**

This 62 pound bronze sampler is intended for use in streams up to 18 feet in depth. It is raised or lowered by means of a steel cable attached to a hand reel. A glass pint bottle is used to collect the sample.

The head of the unit is hinged to permit access to the sample container.

Nozzles of 1/4", 3/16" and 1/8" are supplied with sampler.

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APPENDIX B

Computer Print Out of Phytoplankton
Data From the Thompson River Study.

TABLE 5: PHYTOPLANKTON DATA
THOMPSON RIVER STUDY

DATE: 23 FEBRUARY 1977

TAXA	UNITS/LITER	%
RHODOMONAS MINUTA	53519	20.4
CHLORELLA-LIKE #1	41686	15.9
ACNANTHES MINUTISSIMA	35032	13.3
SYNEDRA VAUCHERIAE	20987	8.0
ACNANTHES LINEARIS	13569	5.2
GOMPHONEMA OLIVACEOIDES	12701	4.8
RHODOMONAS LACUSTRIS	12224	4.7
TABELLARIA FENESTRATA	10558	4.0
FRAGILARIA CROTONENSIS	6542	2.5
CYCLOTELLA STELLIGERA	5249	2.0
CYMBELLA MINUTA	4798	1.8
ASTERIONELLA FORMOSA	3973	1.5
CRYPTOMONAS OVATA	3514	1.3
TETRASELMIS #1	2203	.8
RHIZOLENIA ERIENSIS	2195	.8
GOMPHONEMA DICHOTOMUM	2195	.8
STEPHANODISCUS ASTRAEA	2186	.8
ARTHROSPIRA JENNERI	2178	.8
FRAGILARIA CONSTRUENS	1761	.7
AMPHORA PERPUSILLA	1752	.7
CHRYSOPHYTE STATOSPORE #11	1752	.7
NAVICULA CRYPTOCEPHALA V. VENET	1744	.7
MELOSIRA DISTANS V. ALPIGENA	1735	.7
GOMPHONEMA SUBCLAVATUM	1319	.5
NITZSCHIA RECTA	885	.3
ACNANTHES LANCEOLATA	885	.3
NITZSCHIA SILICA	885	.3
TREUBARIA TRIAPPENDICULATA	885	.3
CYCLOTELLA KUTZINGIANA	876	.3
SCENEDEDMUS DENTICULATUS	876	.3
MELOSIRA ITALICA	868	.3
OCHROMONAS-LIKE	868	.3
DIATOMA TENUE	868	.3
NITZSCHIA FRUSTULUM	868	.3
STEPHANODISCUS ASTRAEA V. MIN	442	.2
NITZSCHIA LINEARIS	442	.2
NITZSCHIA GRACILIS	442	.2
HANNAEA ARCUS	442	.2
CHROMULINA-LIKE	442	.2

TABLE 5: PHYTOPLANKTON DATA (CONTINUED)

TAXA	STATION 346	
	UNITS/LITER	%
ULOTHRIX ZONATA	442	.2
CALONEIS HYALINA	442	.2
COSCINODISCUS ROTHII	434	.2
NITZSCHIA ACICULARIS	434	.2
NAVICULA #13	434	.2
CYMBELLA CISTULA	434	.2
NAVICULA MINIMA	434	.2
ACNANTHES HAUCKIANA	434	.2
ACNANTHES PERGALLI	434	.2
CYMBELLA SINUATA	434	.2
NITZSCHIA DISSIPATA	434	.2
SYNEDRA DELICATISSIMA	434	.2
NITZSCHIA PALEA	434	.2
CYMBELLA AFFINIS	434	.2
CHLAMYDOMONAS-LIKE	434	.2
TOTAL MEAN DENSITY (UNITS/LITER)	262871	
STANDARD ERROR OF MEAN DENSITY	2576	
COEFF. OF VARIATION OF REPLICATES(%)	1.39	
TOTAL TAXA/STATION	54	
MEAN UNITS COUNTED/REPLICATE	300.00	
NUMBER OF REPLICATES	2	

**** = LESS THAN .1%

TABLE 6: PHYTOPLANKTON DATA
THOMPSON RIVER STUDY

DATE: 17 MARCH

1977

TAXA	UNITS/LITER	%
ACNANTHES MINUTISSIMA	98039	25.6
CHLORELLA-LIKE #1	46023	12.0
SYNEDRA VAUCHERIAE	45567	11.9
GOMPHONEMA OLIVACEOIDES	30818	8.0
RHODOMONAS MINUTA	27375	7.1
CYMBELLA MINUTA	17304	4.5
HANNAEA ARCUS	13666	3.6
RHODOMONAS LACUSTRIS	11153	2.9
ACNANTHES LINEARIS	10612	2.8
NITZSCHIA RECTA	5458	1.4
ASTERIONELLA FORMOSA	5068	1.3
CHLAMYDOMONAS-LIKE	4721	1.2
CYCLOTELLA KUTZINGIANA	4028	1.1
SYNEDRA RUMPENS	4028	1.1
NITZSCHIA PALEA	3682	1.0
GOMPHONEMA OLIVACEUM	3444	.9
CYCLOTELLA STELLIGERA	3292	.9
TABELLARIA FENESTRATA	3097	.8
FRAGILARIA CROTONENSIS	2945	.8
TETRASELMIS #1	2902	.8
GOMPHONEMA SUBCLAVATUM	2902	.8
CYMBELLA AFFINIS	2556	.7
OCHROMONAS-LIKE	2209	.6
CYMBELLA CISTULA	2209	.6
NITZSCHIA FRUSTULUM	2014	.5
NITZSCHIA ACICULARIS	2014	.5
GOMPHONEMA HEDINII	1819	.5
NITZSCHIA SILICA	1819	.5
FRAGILARIA LEPTOSTAURON	1624	.4
MELOSIRA ITALICA	1473	.4
CHROOMONAS NORDSTEDII	1473	.4
CRYPTOMONAS OVATA	1278	.3
SYNEDRA RADIANS	1278	.3
ARTHROSPIRA JENNERI	1278	.3
AULOMONAS PURDYI	1083	.3
RHIZOLENIA ERIENSIS	1083	.3
DIATOMA VULGARE	1083	.3
NITZSCHIA BREVIROSTRIS	736	.2
ACNANTHES FLEXELLA	736	.2

TABLE 6: PHYTOPLANKTON DATA (CONTINUED)

TAXA	STATION 401	
	UNITS/LITER	%
SCENEDESMUS DENTICULATUS	736	.2
AMPHIPLEURA PELLUCIDA	736	.2
OSCILLATORIA LIMNETICA	736	.2
FRAGILARIA CONSTRUENS	736	.2
ANKISTRODESMUS FALCATUS	541	.1
NITZSCHIA SUBACICULARIS	541	.1
CHROMULINA-LIKE	541	.1
STAURONEIS ANCEPS	541	.1
NITZSCHIA FONTICOLA	541	.1
NITZSCHIA DISSIPATA	541	.1
TREUBARIA TRIAPPENDICULATA	541	.1
NITZSCHIA LINEARIS	541	.1
SYNEDRA MAZAMAENSIS	541	.1
ACNANTHES LANCEOLATA	541	.1
SYNEDRA ULNA	541	.1
GOMPHONEMA PARVULUM	541	.1
TOTAL MEAN DENSITY (UNITS/LITER)	383332	
STANDARD ERROR OF MEAN DENSITY	58449	
COEFF. OF VARIATION OF REPLICATES (%)	21.56	
TOTAL TAXA/STATION	55	
MEAN UNITS COUNTED/REPLICATE	300.00	
NUMBER OF REPLICATES	2	

*** = LESS THAN .1%

TABLE 7: PHYTOPLANKTON DATA
THOMPSON RIVER STUDY

DATE: 2 JUNE

1977

TAXA	UNITS/LITER	%
RHODOMONAS MINUTA	232468	33.4
RHIZOLENIA ERIENSIS	111991	16.1
ACNANTHES MINUTISSIMA	39684	5.7
ASTERIONELLA FORMOSA	35138	5.1
CYCLOTELLA KUTZINGIANA	34104	4.9
CYCLOTELLA STELLIGERA	29843	4.3
RHODOMONAS LACUSTRIS	29754	4.3
OCHROMONAS-LIKE	18968	2.7
CRYPTOMONAS OVATA	17310	2.5
SYNEDRA VAUCHERIAE	16508	2.4
SYNEDRA PUMPENS	14191	2.0
CHROMULINA-LIKE	10875	1.6
NITZSCHIA ACICULARIS	10500	1.5
CHLORELLA-LIKE #1	9413	1.4
SYNEDRA RADIANS	9270	1.3
CYMBELLA MINUTA	6953	1.0
SCENEDESMUS DENTICULATUS	5865	.8
DINOBRYON SERTULARIA	5723	.8
ACNANTHES LINEARIS	5723	.8
FRAGILARIA CONSTRUENS	4492	.6
NITZSCHIA PALEA	3690	.5
MELOSIRA ITALICA	3548	.5
STEPHANODISCUS ASTRAEA	3405	.5
NITZSCHIA RECTA	3262	.5
CRUCIGENIA QUADRATA	2460	.4
DIATOMA TENUE	2460	.4
ARTHROSPIRA JENNERI	2318	.3
NITZSCHIA FRUSTULUM	2175	.3
GOMPHONEMA OLIVACEOIDES	2175	.3
FRAGILARIA CAPUCINA	2175	.3
NITZSCHIA SILICA	1230	.2
DINOBRYON BAVARICUM	1230	.2
TETRASELMIS #1	1230	.2
CLADOPHORA	1230	.2
OSCILLATORIA LIMNETICA	1230	.2
NITZSCHIA GRACILIS	1230	.2
OOCYSTIS PUSILLA	1230	.2
CHLAMYDOMONAS-LIKE	1230	.2
RHIZOCHRISIS #1	1230	.2

TABLE 7: PHYTOPLANKTON DATA (CONTINUED)

TAXA	STATION 450	
	UNITS/LITER	%
SYNURA UVELLA	1230	.2
NAVICULA PUPULA	1087	.2
HANNAEA ARCUS	1087	.2
MALLOMONAS PSEUDOCORONATA	1087	.2
NAVICULA #8	1087	.2
NAVICULA CRYPTOCEPHALA V. VENET	1087	.2
ACNANTHES LANCEOLATA	1087	.2
TOTAL MEAN DENSITY (UNITS/LITER)	695265	
STANDARD ERROR OF MEAN DENSITY	42786	
COEFF. OF VARIATION OF REPLICATES(%)	8.70	
TOTAL TAXA/STATION	46	
MEAN UNITS COUNTED/REPLICATE	300.00	
NUMBER OF REPLICATES	2	

**** = LESS THAN .1%

Prepared for:
SANDWELL AND COMPANY LTD.

Prepared by:
NORTHWEST HYDRAULIC CONSULTANTS LTD.

April 1977

HAT CREEK WATER SUPPLY STUDY

THOMPSON RIVER
ICE CONDITIONS

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northwest hydraulic consultants ltd.

ACKNOWLEDGMENTS

The assistance of Mr. P. Sexvik of the International Pacific Salmon Fisheries Commission, of Mr. Bob Maxwell of B.C. Hydro and Power Authority and of local residents of the Ashcroft-Kamloops area was appreciated. The generous assistance and cooperation of Water Survey of Canada's Kamloops office-- particularly Mr. J. McFarland--is gratefully acknowledged.

1. INTRODUCTION

B.C. Hydro and Power Authority's proposed Hat Creek Project involves the construction of a major water intake on the Thompson River. As part of the ongoing studies for the intake, Sandwell and Company Ltd., the consultants for the water supply, retained Northwest Hydraulic Consultants Ltd. (NHCL) to investigate the river ice regime of the Thompson in order to identify potential design and/or operations and maintenance problems related to river ice. Figure 1 shows the study area.

The terms of reference for the study are included herein as Appendix A. Authorization to proceed with the study was received by Sandwell on 20 January, 1977.

2. GENERAL

It is standard practice in carrying out the design of a water intake structure to allow for the various possible effects of ice:

1. Ice forces, both impact of floes and expansion of a solid ice cover;
2. Freeze-up jams, and their effects in terms of both low water and flow conditions.
3. Break-up jams, and their effects in terms of both high water levels and ice forces;
4. Screen clogging or jamming, or maintenance problems related to the intake of either broken ice or frazil ice; and
5. Intake port blockage or screen jamming due to development of anchor ice on structural elements partly submerged and partly exposed to the atmosphere.

General knowledge of the climatic conditions of the Thompson River region and consideration of the effect on river temperatures of Kamloops Lake led to a general impression at the outset of the study that problems associated with Points 1, 2 or 3 would be unlikely. The collection of some formal data, was, however, necessary to confirm this impression. The possibility of problems related to Points 4 and 5 appeared to be more serious however, in view of the fact that periods of intense cold do occur in the region.

In order to determine the magnitude of possible ice-related problems, various sources of relevant information were pursued: a program of field observations was established, existing data records were searched, and interviews with knowledgeable local residents were carried out.

3. DATA COMPILATION

3.1 Field Program

A field program of river and lake ice observations was initiated in mid-January, 1977. B.C. Hydro and Power Authority provided an observer in the person of Mr. Bob Maxwell, Landsman, of B.C. Hydro's Kamloops office. The objectives and methodology of the program were discussed with Mr. Maxwell in a series of phone conversations and letters, and joint site visits were carried out on January 31. Informal instruction was given regarding river ice phenomena and the effect of those phenomena on the operation of an intake in order that Mr. Maxwell would be better prepared to make observations and record comments.

Appendix B contains a sample of instructions and a copy of the trial report sheet for Mr. Maxwell's observations.

As it turned out, no river or lake ice observations were recorded this season due to the mild winter that the region experienced. However, we are in a position to re-activate the program at short notice and at little cost next winter should it be desirable to do so.

3.2 River Temperature Data

The International Salmon Commission carries out river water temperature measurements with a continuous-recording thermograph at Walhachin, about 6 miles downstream of Kamloops Lake and about 20 miles

upstream of Ashcroft, during the winter months, usually every second year. Table 1 presents a summary of the data for the years of record.

As can be seen, the lowest temperature recorded in the 8 winters for which data exists was 33° F. However, these particular 8 winters were relatively mild, as will be seen after considering the data discussed below. It seems likely that the water temperature of Walhachin would have been down to 32° during colder winters.

3.3 Meteorological Data

Meteorological data were inspected to determine the degree of severity of winters that can occur along the Thompson, to confirm reports of ice conditions by either Water Survey of Canada or local residents, and to see if an approximate correlation between air temperatures and occurrence of ice could be determined. Relevant meteorological data are presented on Figure 2 and Table 2.

Figure 2 shows mean monthly and minimum recorded temperatures at Ashcroft for the month of January for the period 1941 through 1975. Also shown are the years that ice conditions have either been reported or are strongly suspected to have occurred. Table 2 gives the maximum and minimum daily temperatures at Ashcroft for the specific periods for which occurrence of ice is known or suspected.

It can be seen from both the figure and the table that--as one would expect--there is a reasonable

correlation between reports of ice and cold periods, but some discrepancies are apparent. The met data confirmed the report by a local resident that 1949-50 was the worst winter for river ice in his memory, and most of the specific dates of ice conditions (Table 2) correspond to severe cold spells. However, some of the reported periods of ice are probably incorrect, such as January 1953, most of December and the early part of January 1956-57, and February 1962. Apparently, the occurrence of a brief very cold period (up to 3 or 4 days) in an otherwise warm month does not trigger significant ice formation. An example of this is January 1947 (Figure 2).

Generally, it appears that a significant ice cover has formed if the mean monthly temperature was below about plus 10°F and if the month included a period when temperatures fell below about minus 20°F.

3.4 Water Survey of Canada Ice Data

The records from four Water Survey of Canada gauging stations were searched for comments indicating ice-affected conditions. ('Ice affected', in WSC records, indicates that there was sufficient ice to affect the stage discharge relationship of the station. Thus the notation does not necessarily indicate that river was completely frozen over, but it is likely that the percent-ice-cover was substantial.)

<u>Station Number</u>	<u>Station Name</u>	<u>Period of Record</u>
08LF033	Thompson near Savona	1925-32 1950-55
08LF043	Thompson near Walhachin	1932-48
08LF022	Thompson at Spences Bridge	1911-32 1934-51
08LF051	Thompson near Spences Bridge	1952-Present

The first two records included no comments whatsoever regarding ice. Table 3 presents a summary of the information obtained from the last two stations.

As noted on the table, in some instances the length of time shown for periods of ice varies from the Water Survey of Canada record. No criticism of Water Survey is intended in this analysis and discussion; it must be kept in mind that the gauging stations are visited usually only once a month, and hence periods of ice must be estimated from inspection of the gauge recording. On occasion, gauge malfunction or icing prevents any firm conclusions regarding ice from being drawn and entered in the formal record.

The approximate river discharge for the period involved is presented for general information, and also to indicate a possible explanation for some of the apparent discrepancies in ice reports. For example, the discharge in January 1950 was very high, which might have minimized ice formation at Water Survey's gauge; on the other hand, ice conditions were reported in January 1935, when the discharge was even higher. The freeze reported in January 1953 might have occurred because of abnormally low flow.

In conclusion, on at least 9 occasions, substantial ice conditions have apparently existed for relatively prolonged periods of time. It is virtually certain, however, that even during the coldest periods, open water conditions existed in the rapids sections and in those reaches where the mean flow velocity was high.

3.5 Interviews

A number of long-term local residents who were familiar with the river were sought out and questioned regarding their knowledge of the history of river ice conditions. Brief notes from each of the interviews are included herein as Appendix C.

Of particular interest are the comments of Mr. Lyons and Mr. Christianson. Many of their comments were borne out in subsequent inspection of WSC and meteorological files.

4. DISCUSSION

4.1 Ashcroft Area

The investigation confirmed the initial impression that river ice conditions on the Thompson will not present any significant design problems in terms of ice forces, freeze-up jams that could significantly affect low-flow flow conditions, or break-up jams that could produce either high water levels or unusually large forces.

Indications are that about one year in five at Spences Bridge, an ice cover of significant extent forms. On a somewhat less frequent basis, a complete ice cover forms, but probably only in the deeper, more slow-moving reaches. Rapids likely never freeze over, but would produce large volumes of frazil during intense cold.

About one year in ten, the ice covers that do form persist for approximately a month during the January/February period.

When such ice covers develop downstream of rapids substantial depths of frazil probably accumulate under the ice. The combination of the existence of the ice cover and the deposition of frazil may produce a temporary rise in water levels of several feet.

Mr. Walch's observation that no portion of the Ashcroft 'window' has frozen over completely in the last 18 years is noteworthy. (See Appendix C.) However, those 18 winters have been relatively mild with the exception of 1968-69 (see Figure 2). If the reach near Ashcroft did not freeze over that winter, it might have been because the discharge was relatively high (see Table 3). We do know from WSC data and from Mr. McFarland

directly that the river did freeze over on two occasions that winter at Spences Bridge.

Although no ice thickness measurements exist, local residents interviewed had the firm impression that the ice away from the shore would never be safe, being probably a foot thick at most. It is considered, however, that in prolonged cold periods such as January of 1949-50, the ice cover probably develops to a somewhat greater thickness than one foot. It would be prudent in designing the curtain wall to use thermal ice pressures corresponding to, say, a two-foot ice thickness.

Break-up appears to occur quietly, with the ice thinning, weakening, and unravelling from the downstream edge. Local residents did not recall ever seeing a heavy run of ice. There is, however, no 'hard' data on how break-up actually proceeds after a prolonged period of intense cold. If the ice thins, weakens, and unravels from the various ice-covered reaches more-or-less simultaneously, then the break-up would indeed be uneventful. On the other hand, if because of the influence of the lake, the ice covers break-up sequentially from upstream to downstream, then in the downstream reaches substantial jamming could occur, particularly below major rapids. The jams would likely not persist for long, and the water level fluctuations would likely not be great - perhaps in the order of 10 feet at most. Thus the water levels produced should not be a design factor.

Ice forces can be generated during break-up either through direct impact or through an ice floe being wedged against a structure. In the case of a bank intake with its face at a mild angle to the flow, it is thought that the forces due to break-up would be less than those due to expansion forces of the pre-break-up ice cover.

If a pier-type structure is located in a reach where a solid ice cover can form, it will have to be designed to withstand the impact force generated by the ice cover just as it starts to break-up. The design values for depth of ice and strength of ice cannot be stated at this time.

There remains the possible design and/or maintenance problems related to frazil ice and anchor ice. It is apparent that large quantities of frazil would on occasion be generated in the river, as temperatures of -30° and -40° F can occur with open water conditions. In addition to frazil being produced during periods of intense cold, heavy snowstorms could also contribute large quantities of slush ice.

If the intake were to be located in a reach that developed an ice cover and that was downstream of a lengthy open reach with numerous rapids, it is possible that a substantial depth of frazil and broken pan ice could develop under the solid ice cover at the intake site, as mentioned above. Thus the intake system would have to be designed to cope with severe frazil conditions.

There is also a potential for severe anchor ice problems as air temperatures of minus 30° to minus 40° F do occur. Thus troublesome ice build-ups could occur around intake ports and on equipment unless design steps are taken to prevent it.

4.2 Site 4A

The above comments, though partially based on data for Spences Bridge, are likely a reasonably conservative description of river ice cover conditions

in the vicinity of Site 10. However, at Site 4A, river ice conditions may be somewhat milder. Indeed, Mr. Christianson (see Appendix C) stated emphatically that he had never seen the river frozen over anywhere between Kamloops Lake and his ranch (Site 4A) in the last 16 or 18 years, and ice conditions have never been reported at Walhachin. Again, the last 16 or 18 winters, with the exception of 1968-69, have been relatively mild, and the river discharge in that winter was high.

Anchor ice conditions at Site 4A would be comparable to those at Site 10 inasmuch as air temperature conditions would be similar.

As for ice coming out of Kamloops Lake, it is reported that the lake seldom freezes over (1 year in 7), and if strong winds occur that break up the ice cover, they are usually west winds that push the lake ice away from the river. Thus the lake ice usually rots in place, and if any floes enter the river they quickly break up and melt.

CONCLUSIONS AND RECOMMENDATIONS

1. The incidence and degree of ice cover formation is such that it does not present any major hazards in terms of ice forces or ice jamming, but the design of the intake structure will have to take into account ice loading.
2. Large quantities of frazil, slush ice and possibly broken pan ice will on occasion exist in the river and will tend to be drawn into the intake system. The travelling screens will require well-designed heating and back-washing systems to prevent plugging. The backwash water sluice should incorporate a conveyor belt system as the quantity of ice taken up by the screens could overwhelm a simple sluicing system.
3. The river and climatic conditions are such that severe anchor ice plugging and jamming problems could occur, and thus the capability for heating of ports, of equipment, of critical gate slots, etc., will be essential.
4. The program of field observations should be re-activated in the coming winters only if severe cold weather is experienced.

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TABLES

TABLE 1
SUMMARY OF RIVER TEMPERATURE DATA
THOMPSON AT WALHACHIN
INTERNATIONAL PACIFIC SALMON
FISHERIES COMMISSION

	Degrees F							
	1957	1959	1960	1961	1965	1967	1969	1973
	-58	-60	-61	-62	-66	-68	-70	-74
Dec 1	41.5	41.5		41.0	42.0	42.0	42.0	40.5
10	41.0	40.0	40.0	39.0	42.5	41.5	41.0	40.0
20	40.0	39.5	40.0	38.0	40.5	38.0	40.5	40.0
Jan 1	38.5	37.0	39.0	37.5	36.5	38.0	37.0	37.0
10	39.5	35.0	39.0	36.5	35.0	35.5	36.0	34.0
11								33.5
16							33.0	
20	38.5	33.5	38.0	34.5	34.5	37.0	35.0	34.5
21				33.0				
23					33.5			
Feb 1	38.0	35.0	36.5	35.0	34.0	35.0	35.0	34.5
10	37.0	35.0	35.5	35.0	34.0	34.0	35.0	35.5
20	37.5	35.0	36.0	34.0	34.5	36.0	35.5	34.5
Mar 1	36.5	35.0	36.5	35.0	34.0	35.0	35.0	35.0
9		33.0						
10	36.5	33.5	37.0	33.5	35.0	35.5	35.5	36.5
20	38.0	34.0	38.5	35.5	35.5	36.0	38.0	36.0
30	38.5	37.0	38.0	36.0	38.0	37.0	37.5	37.0

TABLE 2
AIR TEMPERATURES AT ASHCROFT (°F) FOR SELECTED
COLDER WINTERS SINCE 1940

	1942-43**	1949-50	1951-52	1952-53	1953-54	1956-57	1961-62	1968-69	1971-72	
	Max Min	Max Min	Max Min	Max Min	Max Min	Max Min	Max Min	Max Min	Max Min	
Dec 1						32 23				Dec 1
2						34 21				2
3						31 15				3
4						4 1				4
5						2 -11				5
6						2 -23				6
7						1 -22				7
8						23 10				8
9						* 56 17				9
10						* 50 20				10
11						* 31 19				11
12						* 24 12				12
13						* 29 18				13
14						* 18 20				14
15		37 25	23 13			* 51 27				15
16		30 19	27 16			* 34 30				16
17		28 16	26 16			* 46 28				17
18		10 7	29 9			* 51 33				18
19		4 -15	2 -8			* 52 34				19
20		13 -8	2 -17			* 46 32				20
21		14 -1	8 -4			* 39 22			23 12	21
22		25 5	11 1			* 37 19			22 10	22
23		31 24	7 -8			* 32 20			19 3	23
24		21 8	5 -2			* 35 23			15 4	24
25		25 8	10 -5			* 38 24		20 17	4 1	25
26		24 17	13 -4			* 40 24		12 3	6 -4	26
27		21 13	13 2			* 37 24		-6 -12	* -13 -20	27
28		7 4	14 -6			* 30 23		* -15 -21	* 15 -4	28
29		7 -5	20 7			* 31 27		* -21 -36	* 12 0	29
30		0 -18	0 -6			* 31 26		* -14 -37	* 14 0	30
31		10 -8	-11 -29			* 32 25		* 5 -30	* 27 8	31
Jan 1		5 -10	-4 -22			* 37 29		10 -6	28 17	Jan 1
2		-12 -22	2 -14			* 33 29		12 -3	35 24	2
3		-9 -29	12 -6			* 26 22		27 2	10 -16	3
4		-2 -19	15 -5			* 29 23		40 18	23 5	4
5		4 -14	26 -6			* 30 15		40 24	28 16	5
6		13 -8	23 19			* 34 23		28 21		6
7		19 1	26 16			* 32 23		27 16		7
8		14 -1	31 11			* 7 1		16 0		8
9		8 -5	33 16			* 7 -4	20 10	22 9		9
10		9 0	23 12		30 22	* 11 0	26 12	12 8		10
11		7 -1	16 6		34 22	* 8 4	22 11	4 -8		11
12		-3 -7	18 2	39 8	21 9	* 11 3	24 15	5 -14		12
13	34 20	-10 -27	27 6	4 -2	24 18	* 0 -13	18 6	6 -10		13
14	41 31	-13 -30	* 26 11	6 -6	20 3	* 17 -12	23 6	6 -9		14
15	33 16	-8 -25	1 -4	* 20 -7	-5 -11	* 14 -4	17 -9	5 -11		15
16	1 -16	-11 -33	0 -14	* 28 8	-1 -27	* 5 -13	17 -1	10 -8		16
17	-1 -15	-15 -35	23 -5	26 14	4 -13	* 4 -11	16 -4	* 15 1		17
18	* -3 -19	-13 -35	29 0	30 20	-8 -22	* 4 9	0 -19	* 3 -14		18
19	* 6 -10	-2 -26	32 14		-4 -21	* 16 -4	5 -20	* 7 -7		19
20	* -9 -17	11 -7	36 19		3 -18	* 22 8	14 -14	* 8 -5		20
21	* -11 -27	17 5	2 0		4 -9	* 7 -12	11 -8	* 4 -8		21
22	* -3 -20	18 13	-6 -16		0 -12	* 5 -15	6 -9	* -7 -27		22
23	* -2 -25	5 -6	-2 -15		-2 -11	* 2 -18	16 -2	* -10 -28		23
24	* -1 -24	-4 -23	7 -5		-2 -10	* -3 -26	* 22 11	* -1 -23		24
25	* 3 -17	-15 -33	15 -1		0 -19	* -5 -28	* 32 12	* 5 -19		25
26	* 16 2	0 -19	22 -7		8 -7	* -3 -22	44 22	* -3 -7		26
27	* 22 7	-4 -29	* 30 18		6 -16	* 3 -15	48 33	* -3 -11		27
28	* 27 14	-6 -25	30 17		20 -3	* 12 -7	49 39	* -8 -25		28
29	* 30 -4	-3 -24	38 20		16 -1	* 16 1	46 30	* -14 -35		29
30	* 22 -4	-6 -23	43 28		32 10	* 17 8	46 31	* -9 -27		30
31	* 27 0	0 -20	42 28		35 15	* 26 9	39 28	* 1 -18		31
Feb 1	* 31 17	3 -17			29 21	* 16 5	39 31	* 11 -9		Feb 1
2	* 36 24	6 -18			36 20	* 8 -13	40 29	* 11 -10		2
3	* 34 19	8 -15			42 24	* 12 -1	50 28	* 42 0		3
4	* 33 7	22 -5				* 18 1	* 34 19	* 42 30		4
5	* 43 10	36 10				* 20 9	* 20 8	* 34 17		5
6	* 47 8	40 20				* 20 -1	25 14	* 22 1		6
7	* 30 3	41 20				* 21 9	38 25	* 28 2		7
8	* 27 6	40 28				* 21 3	37 25	39 15		8
9	* 27 8	30 17				* 38 5		34 20		9
10	* 41 23	31 19				* 41 24		35 12		10
11	41 21					* 39 19				11
12	43 20					* 51 25				12
13						* 35 28				13
14						* 46 18				14
15						* 33 10				15
16						* 33 10				16
17						* 32 10				17
18						* 44 16				18
19						* 21 13				19
20						* 23 -1				20
21						* 20 3				21
22						4 -10				22
23						22 3				23
24						40 11				24

* Dates of ice conditions at Spences Bridge, as reported by Water Survey of Canada.
** Temperatures for 1943 only are from Kamloops station.

TABLE 3

SUMMARY OF WSC RECORDS REGARDING RIVER ICE
For The Period 1911-76

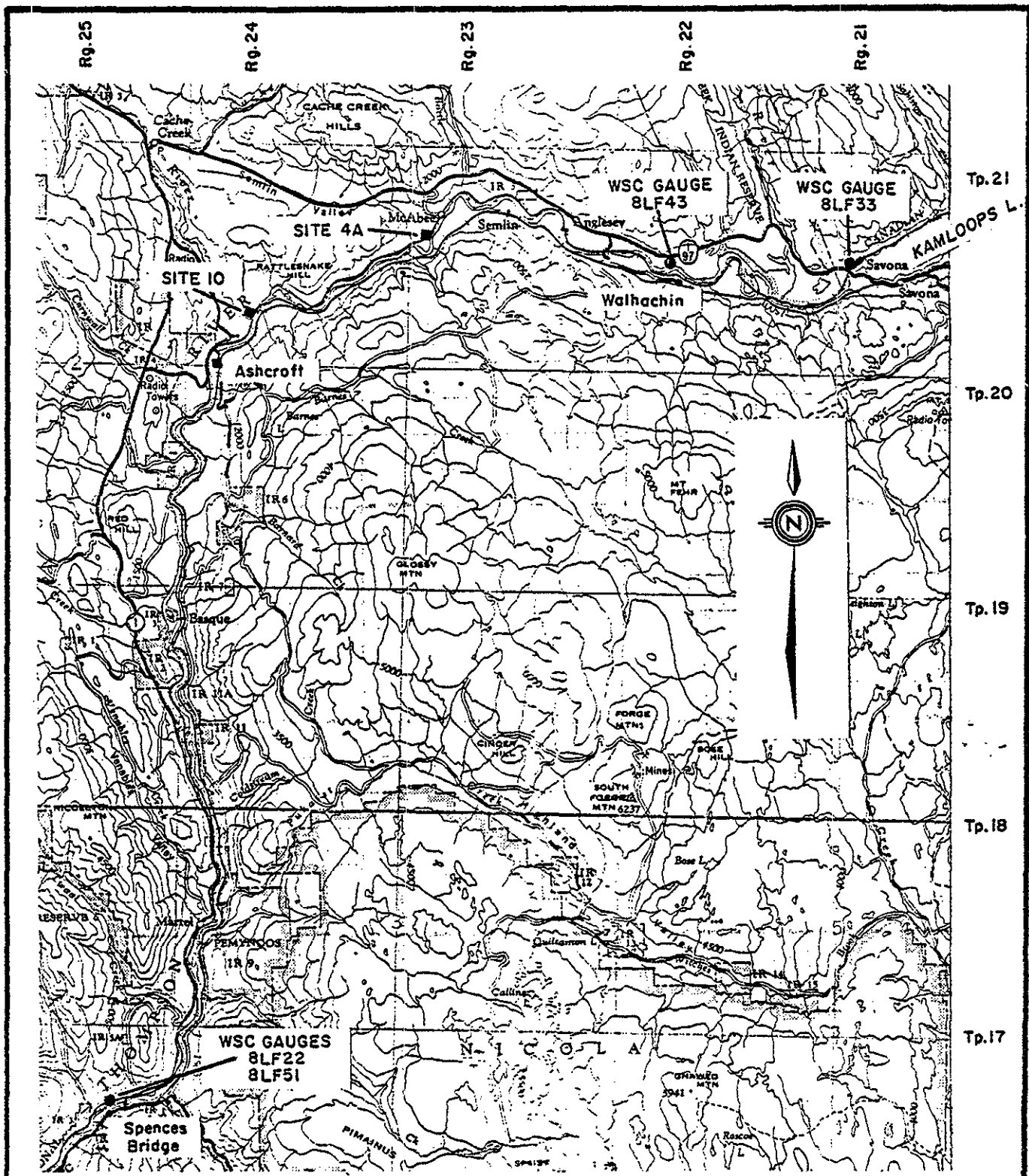
Winter of	Ice Affected Conditions Reported at Spences Bridge	Approx. Discharge cfs	Length of Time Days	Comments
Station 08LF022 (@ Spences Br.)				
1915-16	Jan 26 - Mar 4	5,000	38	
1928-29	Jan 23 - Mar 4	4,500	41	
1934-35	Jan 14 - Jan 25	11,900	12	
1935-36	Feb 6 - Mar 7	6,800	30	
1942-43	Jan 18 - Feb 8	7,500	22	
1949-50*	Ice not reported	10,300	Possibly 50	No WSC data, but reported to have been worst ice in memory.
Station 08LF051 (near Spences Br.)				
1951-52	Jan 1, 7, 14, 27	5,700	28**	
1952-53	Jan 15, 16	4,900	2	
1953-54*	Ice not reported	9,000	Possibly 10	
1956-57	Dec 9 - Feb 21	6,000	40**	
1961-62	Jan 24, 25	8,000	9**	
	Feb 4, 5		0**	
1968-69	Dec 28 - 31	10,000	4	
	Jan 17 - Feb 7	9,000	22	
1971-72	Dec 26 - 31	8,000	6	
	Years of record:		65	
	Number of years ice conditions reported:		12	(14, if 1949-50 and 1953-54 are included)
	Longest period of ice conditions; days:		41	(possibly 50 days, if 1949-50 is included)

* 1949-50 and 1953-54 are included in this table as it was judged likely that ice conditions in fact occurred during these winters.

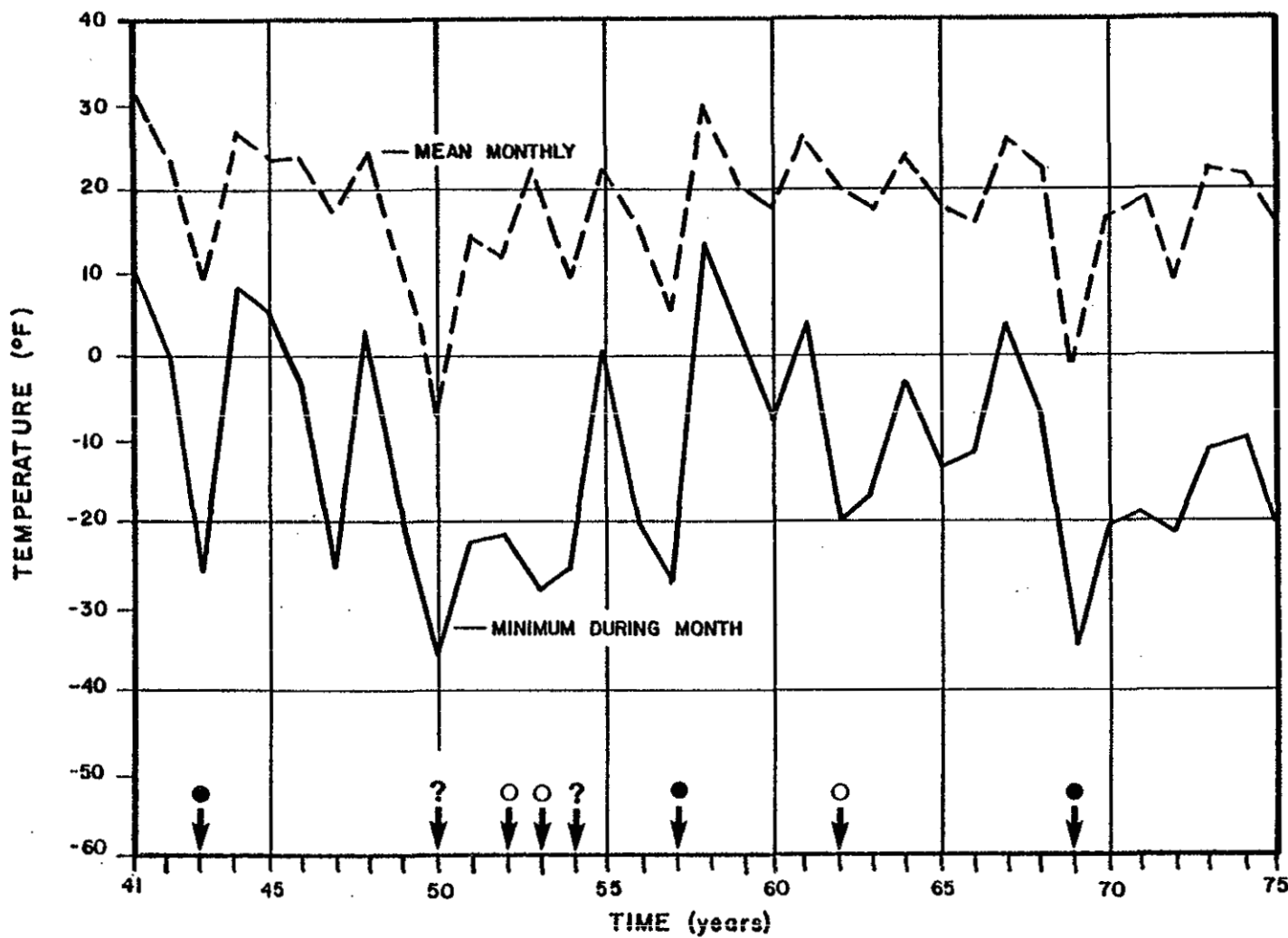
** The actual length of time that the river likely had a substantial percent-ice-cover has been estimated by NHCL after consideration of both the WSC data and the air temperature records for the corresponding period. Air temperature records pre-dating 1943 were not inspected. See Table 3.

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FIGURES



B.C. HYDRO. AND POWER AUTHORITY		
HAT CREEK WATER SUPPLY		
LOCATION PLAN		
STUDY REACH OF THOMPSON RIVER		
northwest hydraulic consultants ltd.		
Drawn: SD.	Date: MAR., 1977	FIGURE 1



LEGEND :

- - MINOR PERIODS OF ICE IN JANUARY
- - MAJOR PERIODS OF ICE IN JANUARY
- ? - NO REPORT OF ICE BY WSC, BUT OCCURRENCE OF ICE KNOWN OR SUSPECTED

NOTES :

- 1- ICE CONDITIONS AT SPENCES BRIDGE AS REPORTED BY WATER SURVEY OF CANADA
- 2- 1941-44 & 1973: KAMLOOPS TEMP DATA
- 1945-72: ASHCROFT (R or M) TEMP DATA
- 1974-75: ASHCROFT TEMP. DATA

B.C. HYDRO AND POWER AUTHORITY
 HAT CREEK WATER SUPPLY
AIR TEMPERATURES FOR THE MONTH OF JANUARY
 northwest hydraulic consultants ltd.
 Drawn: SS Date: MAR, 1977 FIGURE 2

northwest hydraulic consultants ltd.

4823 - 99 st., edmonton, alberta T6E 4Y1
tel. (403) 436-5868

edmonton
vancouver

Our File Number

January 14, 1977

Sandwell and Company Limited,
601 1550 Alberni Street,
Vancouver, B.C. V6G 1A4.

Attention: A. Copeland, P.Eng.

Dear Sir:

Re: Revised Suggested Terms of Reference for Thompson
River Ice Study; Supercedes Item 5 of NHCL Letter
to Sandwell of September 22, 1976/

This letter confirms the revised terms of reference and revised cost estimate as discussed with you by telephone on January 14, 1977.

Revised Item 5 is as follows:

5. River Ice

Design of the intake will be affected by the presence of bank ice, frazil ice or ice jams in the Thompson River. However, only limited information is available on historic ice conditions in the Thompson near the sites.

It is proposed that during the 1976-77 winter season, data be collected which, when analysed with other available data, would aid in establishing bounds of the potential ice problems. Work proposed is as follows:

- a) Site visit by an experienced river engineer to interview local residents and obtain from them first hand information on historic ice conditions. People to be contacted would include Ashcroft village employees, Lornex staff, railway maintenance staff and Water Survey of Canada staff.

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northwest hydraulic consultants ltd.

Page 2
January 14, 1977
Sandwell and Company Limited

- b) Instruct an observer (to be provided by B.C. Hydro) to obtain and record visual observations regarding ice conditions at or near Sites 4A and 10 at intervals throughout the cold weather season. Particular attention will be given to making observations during periods of lowest temperatures. Photos will be taken.
- c) The local observer would contact the NHCL office when extreme or unusual conditions were encountered and a NHCL engineer familiar with ice problems would visit the site when extremes were noted. We anticipate that one such trip would be required, at which time the tasks outlined in Item 5(a) could also be carried out.
- d) Review existing data on river ice conditions, river water temperatures, and extreme minimum air temperatures.
- e) Analysing and reporting of the potential for ice related problems at the intake including recommendations for additional data collection if required.

The time estimates for the above work are:

Principal/Specialist	10 days
Engineer	5 days
Technician/drafting	3 days

Estimated total costs are \$5,500, including \$400 for disbursements. This cost estimate includes one trip to the site.

If you require further information on this revised proposal, please do not hesitate to contact me.

Yours truly,

NORTHWEST HYDRAULIC CONSULTANTS LTD.


A. L. Charbonneau, P.Eng.

ALC:jk

northwest hydraulic consultants ltd.

4823 - 99 st., edmonton, alberta T6E 4Y1
tel. (403) 436 - 5868

edmonton
vancouver

Our File Number

January 14, 1977

B.C. Hydro and Power Authority,
155 Victoria St.,
Kamloops, B.C.

Attn: Mr. Bob Maxwell

Dear Mr. Maxwell:

Enclosed herewith please find four maps:

1. 1" = 4 miles map showing the portion of the Thompson River we are interested in.
2. 1" = 0.8 miles map showing the two sites.
3. 1" = 0.25 miles map of Site 4A.
4. 1" = 0.25 miles map of Site 10.

Map 1 also shows possible points for making observations. I did not mention it during our phone conversation, but I would like you to note the extent of the ice cover in Kamloops Lake as well. Just a rough estimate of the percent cover will do.

Also enclosed are a number of copies of a trial check-off report sheet. Try them out and let me know if they are useful.

Let me emphasize a few things:

1. Try to make observations during colder periods, particularly the coldest period.
2. Frequent observations are not necessary.
3. It is not necessary to observe conditions at many points along the river each time. Observations at Site 4A and Site 10 only will probably be enough.

..... 2

4. Detailed comments are not necessary.
5. Fill out a report sheet even if the river is ice free, but don't bother with a photograph.
6. Don't be surprised if you don't see much worth reporting!

If you wish to discuss your observations or the trial report sheet, or if you have any questions, please do not hesitate to call.

When my next site visit is finalized, I will contact you to arrange a meeting either in Kamloops or in the field.

Yours truly,

NORTHWEST HYDRAULIC CONSULTANTS LTD.



A.L. Charbonneau, P.Eng.

ALC:lh
encl.

HAT CREEK PROJECT WATER SUPPLY STUDY
THOMPSON RIVER ICE OBSERVATIONS REPORT SHEET

Date: _____

Time: _____

Location: _____

Photo taken _____

Roll number _____

Frame number(s) _____

River Ice Conditions:

- _____ No ice
- _____ Shore ice in slack areas only
- _____ Continuous shore ice
- _____ Extends _____ feet from shore
- _____ Estimated thickness: _____ inches
- _____ Complete ice cover over river
- _____ Ice floes moving down river
- _____ Size: _____-foot diameter
- _____ Slush ice (frazil ice) in river
- _____ Along shore
- _____ Moving down river

Comments: _____

Lake Ice Conditions:

- Extent of ice cover:
- _____ None
 - _____ Some shore ice
 - _____ 1/4
 - _____ 1/2
 - _____ 3/4
 - _____ Complete cover

THOMPSON RIVER ICE CONDITIONS
Interviews with Residents

Date: Feb 1/77
Name: Doug Lyons, Kamloops
Position: CP Rail Employee
Experience with River: Worked on Kamloops - Ashcroft section for 35 years.
Comments: - River has had portions frozen over 3 or 4 times.
- Worst freeze (winter of 48-49 or 49-50?)
- at most, 50% or river length frozen over (guess)
- ice near shore maybe 8-10" thick
- lasted 2-3 wks (guess)
- Never jams
- Some ice floes occasionally
- Lake freezes over on occasion
- winds usually from west; do not drive lake ice into Savona (ie. into Thompson)
- ice usually melts in lake

THOMPSON RIVER ICE CONDITIONS

Interviews with Residents

Date: Jan 31/77

Name: Jack Christianson

Position: Rancher

Experience with River: Owns property at Site 4A; has lived in present house 16 or 18 years, can see river from house; lived all his life in area.

Comments:

- Has never seen river frozen over between lake and Site 4A.
- Shore ice forms; maybe 1 ft thick.
- Never many ice floes; few cakes, not big.
- Never jams.
- Kamloops Lake
 - freezes over 1 year out of 7
 - last time probably 1968 (Dec.)
 - froze over two years in a row in 1920's
 - ice usually rots out does not break up until 'honeycombed' (ie. candled)
 - not much comes down river

THOMPSON RIVER ICE CONDITIONS
Interviews with Residents

Date: Jan 31/77
Name: Jim McFarland
Position: WSC Kamloops
Experience with River: Operated Spences Bridge gauge
(among others) for 10 years.
Comments: - Total ice cover observed in
gauge reach on Dec. 30, 1968
- probably only few inches
thick in center
- several miles in extent
- at, at least two locations
- Anchor ice observed a few times
- Lake froze over completely in '68
- Ice floes out of lake
- move right through to Fraser
- small percent area coverage
- No jams that he is aware of
- Solid cover in reach above CN
bridge just above Site 10
observed once

THOMPSON RIVER ICE CONDITIONS
Interviews with Residents

Date: Sept 15/76; Mar 17/77
Name: Ed Walsh
Position: Town Superintendent, Ashcroft
Experience with River: He lived in Aschroft since 1958; worked for town since 1964; has inspected town water pumphouse nearly daily for 12 years, except winter of 1968-69.

Comments:

- No ice problems with intake in 12 years
 - Intake consists of 12 inch diameter pipe with screened end; end of pipe submerged about 10 feet at low water. Unheated wet well.
- River ice:
 - Shore ice forms
 - Ice around piers
 - Ice in backwater areas (thick enough to skate on)
 - Has never seen the river completely frozen over between the CN bridge and Ashcroft; always an open channel left flowing. Another resident since 1956 confirmed this observation.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HYDROELECTRIC DESIGN DIVISION

DEVELOPMENT DEPARTMENT

HAT CREEK PROJECT

REPORT ON HYDRAULIC

TRANSIENT ANALYSIS OF MAKE-UP COOLING

WATER SUPPLY SYSTEM

FEBRUARY 1978

Prepared by

M. H. Chubbey D. E. Cassell

Hydraulic Section

Approved by

[Signature]

Manager, Development Department

REPORT NO. DD108

I. INTRODUCTION

Under Assignment No. 477-036, the Hydroelectric Design Division (HEDD) was assigned to: develop and verify a computer program to analyze hydraulic transients in pumping systems and run the computer program and interpret the results as required by the Water Supply Consultant* for the design of the make-up cooling water supply system for the Hat Creek Project.

The water supply system for pumping water from the Thompson River to the plant reservoir as presently planned would be comprised of an 800 mm diameter buried pipeline approximately 23 km long; a pumping station having five pumping-units at the river intake; two booster stations each having four pumping-units and a free-surface suction tank; and a reservoir near the powerplant. The average and maximum discharges would be 0.725 and 1.60 m³/s respectively, and the maximum total static lift from the river intake to the plant reservoir would be 1083 m. The river intake would be located on the right bank of the Thompson River, 2.4 km northeast of Ashcroft, B.C.

This report presents a brief description of the computer program and its verification, results of hydraulic transient studies, and recommendations for control devices.

II. COMPUTER PROGRAM

The computer program uses the method of characteristics (1,2)** to compute the transient conditions in the pipeline and includes the following boundary conditions:

* Sandwell and Company Limited (Sandwell) Vancouver, B.C.

** Numbers in parenthesis refer to references listed at the end of the report.

1. Constant-level reservoir at the upstream or at the downstream ends;
2. Series junctions;
3. Check valves;
4. Valves located at the downstream end or at an intermediate location;
5. Simple or orifice surge tanks;
6. One-way surge tanks;
7. Air chambers;
8. Centrifugal pumps.

The program was written to analyze transient conditions in a series piping system following pump failure and/or valve operation. Provision was made for the addition of branch pipes at a later date. There can be up to fifty pipe-segments in the system and each pumping station can have up to five parallel pumping-units which are manifolded just near the pumps into single suction and discharge lines. Similarly, there can be up to five each of the following control devices: surge tanks, one-way surge tanks, valves, check valves and air chambers. The number of pipes and control devices can be increased simply by modifying the DIMENSION statement of the program.

The following cases for instantaneous, simultaneous power failure on all pumping-units can be analyzed:

1. Pump allowed to go to run-away speed in the reverse direction;
2. Check valve closes instantly upon flow reversal through the pump;
3. Discharge valve closes slowly, reverse pump rotation allowed;
4. Check valve closes instantly upon flow reversal, pressure regulating valve opens quickly and then closes slowly.

The pump characteristics and valve-opening versus time curve are specified by storing in the computer discrete points on the curves; intermediate points are parabolically interpolated.

The surge tanks and the one-way surge tanks may include standpipes between the tank and the pipeline. The head losses and the inertia of the water inside the standpipe are taken into consideration in the analysis. When there is no outflow from a one-way surge tank, it is replaced by a series junction.

The boundary conditions and solution procedures presented in Ref. 2 are used in the program.

A first order finite-difference method is used to solve the characteristic form of the dynamic and continuity equations. To avoid errors introduced by interpolations, wave velocities are adjusted slightly, if necessary, so that the characteristics pass through the grid points.

The program was verified by comparing computed transient pressures with either those measured on a prototype; those computed by using the graphical method, or those obtained by using other simpler, problem-oriented computer programs.

Table I summarizes the cases used for verification and Figs. 1 and 2 show a comparison of computed and measured results. In all cases, the agreement was satisfactory to very good.

TABLE I

VERIFICATION OF COMPUTER PROGRAM

TEST NO.	DESCRIPTION	DATA USED FOR COMPARISON	MAX. PRESSURE*		MIN. PRESSURE*		REMARKS
			Standard**	Computer Program	Standard**	Computer Program	
1.	Power failure						
	(a) Discharge valve remains open.	Prototype test data obtained by the Dept. of Water Resources, Sacramento California at the Wind Gap Pumping Plant	1.10	1.09	0.65	0.58	Action of siphon outlet valve not modelled.
	(b) Discharge valve closes	"	1.09	1.07	0.61	0.58	"
	(c) Pressure regulating valve opens rapidly, then closes gradually	Graphical analysis (3,4)***	1.18	1.18	0.08	0.06	
2.	Valve Closure	Computer program (2)***	2.76	2.76	0.01	0.01	
3.	Air chamber	Graphical analysis (5)***	2.33	2.26	0.30	0.30	

* Relative pressures with respect to rated or static head.

** Measured on prototype or computed by the graphical method.

*** References listed at the end of report.

III. DATA

The following information on the water supply system was provided to HEDD by Sandwell.

(a) Pipeline

Figs. 3 and 4 show the profile and wall thickness of the 800 mm diameter pipeline. There are three pumping stations. Two potential routes - the Summit Route and the Medicine Creek Route - for the pipeline downstream from Booster Station No. 2 were considered.

(b) Pumping Stations

1. Station at River intake

Number of pumps = 5

2. Booster Station No. 1

Number of pumps = 4

Elevation of pump centerline : El. 305

3. Booster Station No. 2

Number of pumps = 4

Elevation of pump centerline :

Medicine Creek Route : El. 843

Summit Route: El. 858

There is a free-surface tank on the suction side of each booster station.

(c) Pumps and Motors

For the pumps and motors at each booster station, the data for each pump and motor as used in the transient analyses are as follows:

Rated flow = $0.4 \text{ m}^3/\text{s}$

Rated head = 670 m *

Rated speed = 3580 rpm

Number of stages = 3

Specific speed = 39.2 (SI units)

Moment of inertia of pump, motor, shaft, and entrained water in the impeller = 62 kg m^2 (If required, the total inertia for each unit can be increased to 420 kg m^2 without exceeding the limits of the pump start-up time. The inertia could be increased by custom design of the motor or by adding a flywheel).

Only characteristics for the normal zone of pump operation were supplied by Sandwell and no data were provided for the zones of energy dissipation and turbine operation. Therefore, pump characteristics (See Fig. 5) for these two zones were taken from Ref. 2. As there was close agreement between the characteristics for the normal zone, as provided by Sandwell, and those presented in Ref. 2 for $N_s = 25$ (SI units), the characteristics of Ref. 2 were used for all zones of operation.

* 670 m is used as the rated head throughout this report. It should be noted that the design operating conditions for each pumping station were different from the listed rated conditions for the pumps.

(d) Downstream Reservoir

Medicine Creek Route

Maximum water level = El. 1372

Minimum water level = El. 1357

Summit Route

Water level in the control structure = El. 1427

The control structure at the summit was taken as the downstream reservoir and the free-flowing conduit from the summit control structure to the plant reservoir was not included in the analysis.

IV. DESIGN CRITERIA

Sandwell defined the following design criteria in their letter of 8 September 1977 (6):

(a) Normal Conditions

1. Manual or automatic starting and stopping of pumps;*
2. Shutoff head develops on the pump manifold or on the pipeline upstream of any shutoff valve;
3. Power failure occurs simultaneously on all pump-motors;
4. Check valve, if present in the system, closes immediately upon flow reversal; and
5. Air chambers, if present in the system, are at minimum air volume.

* Not critical since the pumps will be started and stopped with the discharge valve closed. The opening and closing times for the discharge valve will be set so that the maximum and minimum pressures are within the design limits.

During the above operations, control devices, such as surge tanks, air chambers, check valves, surge suppressors, and pressure control devices function as designed. The water column does not separate although the pressure may become sub-atmospheric. The pipe design is such that it does not collapse at minimum head.

(b) Emergency Conditions

One of the following control device malfunction occurs:

1. One of the surge suppressors, pressure relief valves, or pressure regulating valves does not operate;
2. One check valve closes at the time of maximum reverse flow;
3. Air inlet valves are inoperative;
4. An air chamber is at emergency low air volume;
5. A two-speed control valve closes at the faster rate; or
- 6.* A discharge valve fails to close following power failure.

(c) Exceptional Conditions

The equipment malfunctions in the most unfavourable manner.

V. ANALYSIS

Because there is a free-surface tank close to the pumps on the suction side of each booster station, transients in the discharge line were analyzed neglecting the effects of transients in the suction line. For the Summit Route, the free-flowing pipeline from the control structure at the summit to the plant water supply reservoir (see Fig. 3) was not included in the analysis.

* Added by HEDD

Transients caused by the pumps at the river intake were not considered to be important enough to be analyzed during the preliminary design stages, because of the short length of the pipeline. This should, however, be done during final design.

(a) Normal Conditions

The following procedure was used to select appropriate waterhammer control devices:

1. Column Separation

The system was analyzed for the case of simultaneous power failure to all pumps assuming there were no control devices. Water column separation occurred in the pipeline between Booster Stations Nos. 1 and 2, and in the pipeline downstream of Booster Station No. 2 for both the Summit and Medicine Creek Routes. Provision of additional inertia at the pumps and one-way surge tanks prevented column separation. The data for these devices are listed in the next section

2. Maximum Pressure

It was assumed during the initial design of the pipeline that with appropriate control devices, pressure rise* could be limited to 10 percent of the rated head of the pumps. With check valves located downstream of the pumps, the pressure rise following power failure exceeded 10 percent in a number of cases (see Table II). However, the pressure rise could be reduced to less than 5 percent by slowly closing the pump

* Pressure rise = Maximum transient state pressure - steady state pressure

discharge valves. A pressure regulating valve - check valve combination could also be used but was not considered because of the availability of the above less expensive alternative

(b) Emergency Conditions

The only applicable case was that of failure of the discharge valve following power failure since a single-rate closure was assumed for the discharge valve and no check valves or air chambers were used as control devices.

(c) Exceptional Conditions*

In a one-way surge tank, simultaneous failure to open of both check valves should be considered as an exceptional operating condition. In such a case, the one-way surge tanks would be inoperative and, as discussed in item (a)-1 above, column separation would occur. The minimum pressures would then be equal to the vacuum pressures.

The computation of the maximum pressures when the separated columns rejoin was not considered important during the preliminary design for such an exceptional condition. This should however, be done during the final design.

VI. RESULTS

The maximum and minimum hydraulic grade lines following power failure are shown on Figs. 3 and 4 for the system containing suitable control equipment.

* Not analyzed.

A. Column Separation

The following control devices would successfully prevent column separation in the various segments of the pipeline.

(a) Pipeline from Booster Station No. 1 to 2

The following two alternatives are available:

1. Increase the WR^2 of each pump-motor to 115 kg m^2 for the Medicine Creek Route and 130 kg m^2 for the Summit Route and provide a 4 m diameter one-way surge tank at the top of Elephant Ridge with the steady state water level in tank at El. 627 (10 m above the ground surface).
2. Increase the WR^2 of each pump-motor to 390 kg m^2 for the Medicine Creek Route and to 370 kg m^2 for the Summit Route.

With the above controls, the minimum pressures in the pipeline remain above atmospheric pressure.

(b) Pipeline Downstream of Booster Station No. 2

1. Summit Route

The following combination of control measures will prevent column separation:

- (i) Increase the WR^2 of each pump-motor to 400 kg m^2 ;
- (ii) Provide a 4 m dia. one-way surge tank at Sta. 11 + 175 with the steady state water level in the tank at El. 1252 (10 m above ground level); and
- (iii) Provide a control structure at the summit with water surface elevation at El. 1427 to provide a minimum submergence of 5 m.

With these measures, the minimum pressures do not become sub-atmospheric except at Sta. 18 + 00 where minimum pressure is -4 m.

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- (iii) Provide a control structure at the summit with water surface elevation at El. 1427 to provide a minimum submergence of 5 m.

With these measures, the minimum pressures do not become sub-atmospheric except at Sta. 18 + 00 where minimum pressure is -4 m.

2. Medicine Creek Route

The following combination of control measures will prevent column separation:

- (i) Increase the WR^2 of each pump-motor to 370 kg m^2 ;
- (ii) Provide a 4 m dia. one-way surge tank at Sta. 11 + 175 with steady state water level in the tank at El. 1252 (10 m above ground level); and
- (iii) Provide a 4 m dia. one-way surge tank at Sta 17 + 480 with steady state water level in the tank at El. 1345 (25 m above ground level).

With these measures, the minimum pressures along the pipeline are above atmospheric pressures.

B. Maximum Pressure

Table II presents the maximum pressures following power failure when check valves are located downstream of the pumps. The check valves are assumed to close instantly upon flow reversal.

TABLE II: MAXIMUM PRESSURES
AT PUMP FOLLOWING POWER FAILURE

<u>Pipeline</u>	Maximum Pressure Elevation, in m	Pressure Rise (Percent of Rated Head)
<u>Booster Station No.1 to 2</u>		
Medicine Creek route* ($WR^2 = 115 \text{ kg m}^2$ and one-way surge tank)	1119	24.2
Summit Route ($WR^2 = 370 \text{ kg m}^2$)	1118	22.4
Medicine Creek Route ($WR^2 = 390 \text{ kg m}^2$)	1103	21.8
<u>Downstream of Booster Station No. 2</u>		
Summit Route ($WR^2 = 400 \text{ kg m}^2$ and one-way surge tank)	1587	9.2
Medicine Creek Route ($WR^2 = 370 \text{ kg m}^2$ and two one-way surge tanks)	1485	0.0

* Conditions with $WR^2 = 130 \text{ kg m}^2$ for Summit Route are about the same.

The pressure rise following power failure could be reduced by slowly closing the pump discharge valves. With the closing times listed in Table III, the pressure rise at the pump following power failure would be less than 5 percent of the rated head. A single rate closure was assumed in these computations.

TABLE III
DISCHARGE VALVE CLOSING TIMES AND PRESSURE RISE

Pipeline	Closing Time (in. sec.)	Pressure Rise at the Pump (Percent of Rated Head)
<u>Booster Station No. 1 to 2</u>		
WR ² = 115 kg m ² and one-way surge tank	90	4.0
WR ² = 390 kg m ² (Medicine Creek Route)	95	3.2
WR ² = 370 kg m ² (Summit Route)	90	4.6
<u>Downstream of Booster Station No. 2</u>		
Summit Route (WR ² = 400 kg m ² and one one-way surge tank)	100	2.7
Medicine Creek Route (WR ² = 370 kg m ² and two one-way surge tanks)	95	4.7

Due to the slow closure of the discharge valve, the pump will reverse for a short time (The pumps reach their maximum reverse speed within 75 seconds and then begin to slow down). The maximum reverse pump speed following power failure for the cases when the discharge valve remains open and when the discharge valve is closed are listed in Table IV. However, as shown in this table, the maximum negative speed in all cases was less than the following maximum permissible limits specified by the pump manufacturer: 130% of rated speed for less than 30 seconds, and 120% of rated speed for a longer period.

C. Emergency Conditions

As an emergency condition, the discharge valves were assumed to remain open following power failure. Because of the higher than normal inertia, the maximum pressure at the pump in all cases remained less than the steady state pressure. The maximum reverse pump speeds and the time at which they occur are listed in Table IV. Since only a single-rate closure was assumed, no computations were done for closure of the valve at the faster rate.

TABLE IV
MAXIMUM REVERSE PUMP SPEED FOLLOWING POWER FAILURE

Pipeline	Maximum Reverse Pump Speed (% of Rated)	
	Discharge valve remains open	Discharge Valve Closed*
<u>Booster Station No. 1 to 2</u>		
WR ² = 115 kg m and one-way surge tank	113% at approx. 37 sec.	112% at approx. 35 sec.
WR ² = 370 kg m ² (Summit Route)	105% at approx. 76 sec.	89% at approx. 65 sec.
WR ² = 390 kg m ² (Medicine Creek Rte)	105% at approx. 80 sec.	90% at approx. 70 sec.
<u>Downstream of Booster Station No. 2</u>		
Summit Route (WR ² = 400 kg m ² and one one-way surge tank)	104% at approx. 85 sec.	88% at approx. 75 sec.
Medicine Creek Route (WR ² = 370 kg m ² and two one-way surge tanks)	100% at approx. 80 sec.	84% at approx. 70 sec.

* Discharge valve closed at a rate that would keep the maximum pressure rise at the pump less than 5% of the rated head.

VII. DISCUSSION

The above results were obtained using assumed friction factors, and assumed pump characteristics. In addition, both the topographic information and the data for the discharge valves were not precisely known. As the design operating conditions for each pumping station were different from the rated conditions of pumps for which data were provided, the water level in the suction tank had to be artificially lowered in the analysis to obtain the correct downstream head for the given pump speed. Artificial lowering of the suction reservoir should have a negligible effect on the computed pressures in the discharge line. However, test runs using different pump characteristics showed that variation in the pump characteristics and/or in the data for the discharge valve could substantially change the computed maximum and minimum pressures. Similarly, significant changes in the ground topography would change the hydraulic grade line relative to the pipeline, thus possibly resulting in situations where column separation could occur. A difference in the friction losses could also affect the maximum and minimum pressures.

No cushioning stroke was assumed during the discharge valve closure. This should be taken into consideration during the final design as it could reduce the maximum pressures slightly.

With the presently available data, the maximum pressure rise at the pump can be kept below 5% of the rated head. However, as discussed above, the pressure rise may be higher due to significant variations in the data for the system. If necessary, the pressure rise can be decreased

by increasing the discharge valve closing time which would result in an increase in the time period for which the pump runs in the reverse direction. Although the maximum reverse pump speed is within the limits specified by the pump manufacturers, reverse flow through the pumps for an extended period may partially drain the pipeline at high points in the case of Summit Route and the plant reservoir in the case of the Medicine Creek Route. Therefore, it is recommended that until better data is obtained and a sensitivity analysis of the effects of changes in the variables affecting pressure rise is made, the maximum pressure rise at the pump-end should be taken equal to 10% of the rated head and the elevation of the maximum hydraulic grade line shown in Figs. 3 and 4 be adjusted proportionately. Since by increasing the design pressures the uncertainty in the maximum pressures is allowed for, this should be taken into consideration while selecting the factor of safety for the pipeline design.

With the specified control measures, the minimum hydraulic grade line is always above the pipeline except near the control structure in the Summit Route where it drops 4 m below the pipeline. At Elephant Ridge and at the summits downstream of Booster Station No. 2, the minimum hydraulic grade line is less than five meters above the pipeline. During the final design, however, when better data will be available, this should be investigated in detail and, if necessary, the safety margin could be increased.

The free-standing height of the downstream one-way surge tank on the Medicine Creek Route may be reduced by locating the tank on an adjacent high ground and connecting the tank to the pipeline with a pipe. If high ground is not available nearby, then the pipeline route may be changed slightly. In addition, the possibility of reducing the tank height by rerouting that segment of the pipeline in which water column separation may occur should be investigated.

Air valves should be provided at high points along the pipeline. These will be helpful for filling and emptying the line and would prevent collapse of a long length of the pipeline should a break occur in the pipeline at a lower elevation. The procedure outlined in Ref. 7 may be used for sizing these valves. In addition, valves could be provided along the line to isolate and drain segments of the line for inspection, repair etc. Transients caused by the operation of these valves, if provided, should be studied during the final design. In our opinion, no check valves should be provided at the interior locations along the pipeline since they have been reported to cause resonance (8). However, at the downstream end of the Medicine Creek Route, a check valve should be installed to prevent draining of the plant reservoir and a vent pipe should be provided for emptying the pipeline.

The one-way surge tanks should have two pipes for water outflow from the tanks into the pipeline. This should considerably reduce the possibility of a tank becoming inoperative due to the failure of a check valve to open.

Two alternatives are available to prevent column separation in the pipeline between Booster Station Nos. 1 and 2. The alternative with increased inertia only is recommended from an operational point of view as the one-way surge tank is not as foolproof and in addition requires constant maintenance. In addition, for operational flexibility and for ease in exchanging the spare parts, it would be prudent that all units have equal inertia; a $WR^2 = 400 \text{ kg m}^2$ is recommended. Because of this increase in inertia, the pressure rise and drop will be slightly less than that shown in Figs. 3 and 4.

In the Summit Route, the free-flow segment of the pipeline downstream of the control structure at the Summit would require a very careful analysis since air entrainment in the lower part, where a hydraulic jump could form, may result in blowbacks and pipe vibrations. Pressurizing this length of the pipeline would require provision and appropriate operation of a control valve located at the downstream end and would result in a higher pumping head.

As far as the waterhammer control of the Summit and Medicine Creek Routes is concerned, one route does not have any significant advantage over the other; for the Summit Route, a control structure and a one-way surge tank and for the Medicine Creek Route, two one-way surge tanks are required.

During the final design, the possibility of operational instability between various pumping stations should be investigated.

VIII. RECOMMENDATIONS

1. The following controls are recommended to prevent water-column separation:

(a) Pipeline Between Booster Station No. 1 and 2

(i) Summit Route

At Booster Station No. 1, provide pump-motor WR^2 equal to 400 kg m^2 .

(ii) Medicine Creek Route

At Booster Station No. 1, provide pump-motor WR^2 equal to 400 kg m^2 .

(b) Pipeline Downstream of Booster Station No. 2

(i) Summit Route

Provide a 4 m diameter, one-way surge tank at Station 11 + 175 with the steady state water level in the tank at El. 1252; a control structure at the summit with the steady state-water level at El. 1427, and pump-motor WR^2 of 400 kg m^2 for each unit.

(ii) Medicine Creek Route

Provide a pump-motor WR^2 equal to 400 kg m^2 for each unit; one-way surge tanks at Sta. 11 + 175 (initial steady state water level = El. 1252) and at Sta. 17 + 480 (initial state water level = 1345).

2. The following discharge valve closing times are recommended to keep the maximum pressure rise at the pump-end to less than 5% of the rated head:

<u>Pipeline</u>	<u>Closing time (in sec.)</u>
(a) <u>Between Booster Station No. 1 and 2</u>	
Medicine Creek Route	95
Summit Route	90
(b) <u>Pipeline Downstream of Booster Station No. 2</u>	
(i) Summit Route	100
(ii) Medicine Creek Route	95

As discussed in Section VII, until more definite system data are available and a sensitivity analysis can be made, the design pressure rise at the pump-end should be 10% of the rated head and the elevation of the maximum hydraulic grade line as shown in Figs. 3 and 4 should be adjusted proportionately.

3. Air valves should be provided at high points of the pipeline.
4. No check valves should be provided at intermediate locations along the pipeline. However, a check valve and a vent pipe should be installed at the downstream end of the Medicine Creek Route.

5. The following additional investigations should be carried out during final design:

(i) Maximum pressures caused by column separation due to one-way surge tank being inoperative because of failure of both check valves to open.

(ii) Operational instability between various pumping stations.

(iii) Maximum pressures following power failure taking into consideration the cushioning stroke of the discharge valve.

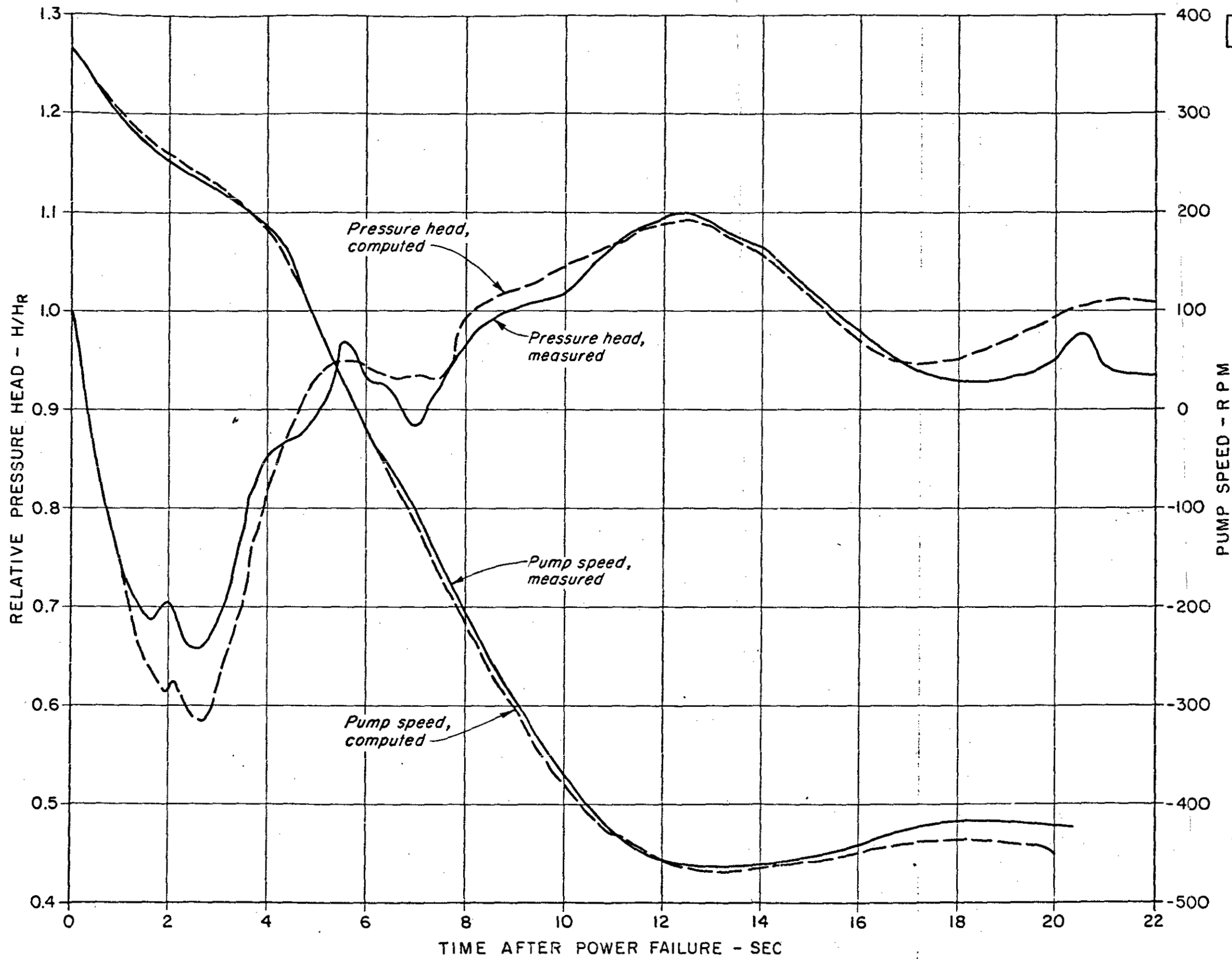
(iv) Maximum and minimum pressures for simultaneous power failure to three pumps, fourth pump initially not operating.

(v) A sensitivity analysis assuming possible variations in the pump characteristics, friction factors and discharge valve characteristics.

6. To avoid compounding of safety factors a decision should be made as to how safety factors should be divided between hydraulics and structural components of design.

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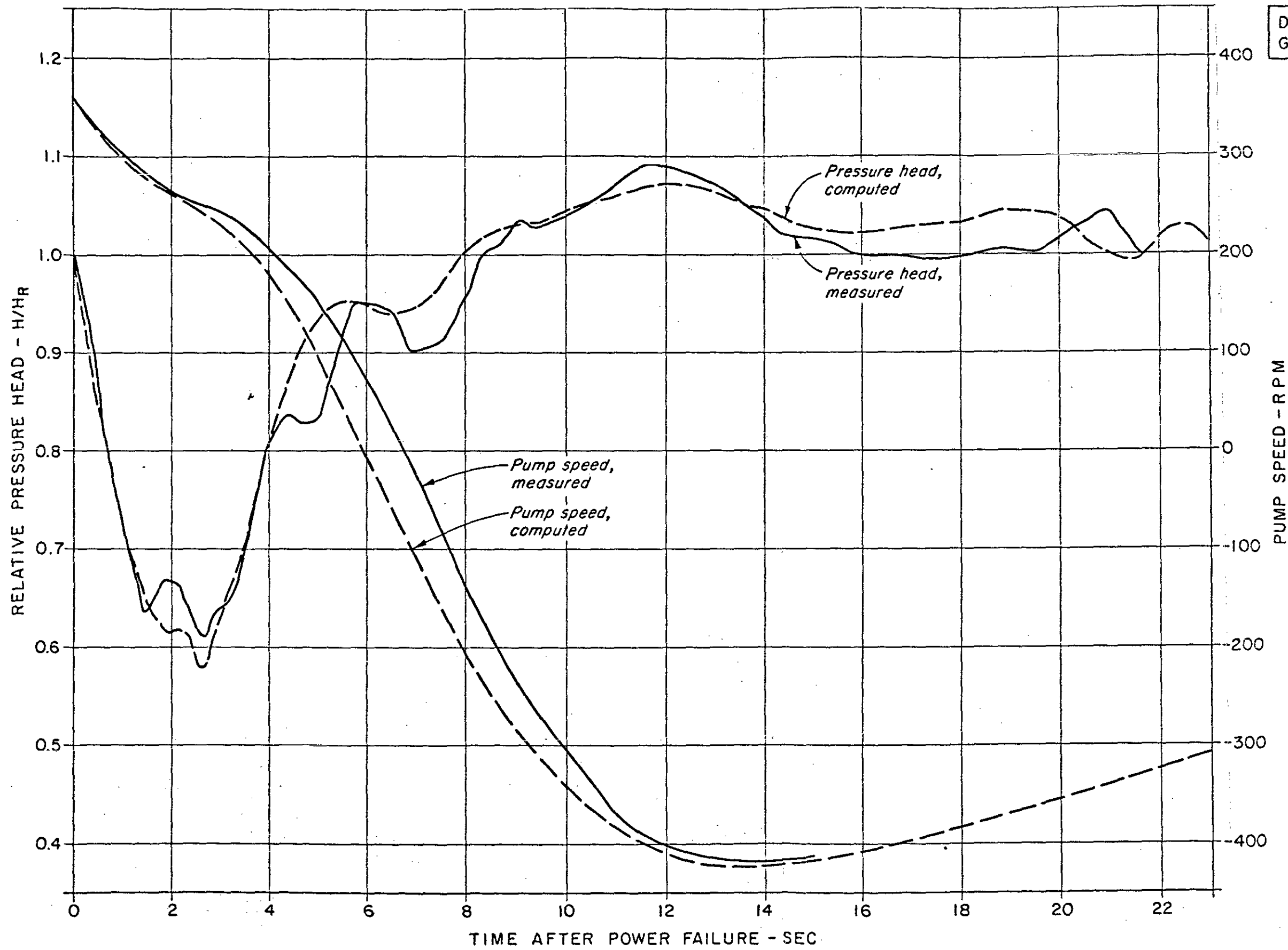
NOTE:

- MEASURED RESULTS WERE RECORDED DURING PROTOTYPE TESTS CONDUCTED BY THE DEPARTMENT OF WATER RESOURCES, STATE OF CALIFORNIA, SACRAMENTO, CALIFORNIA.
- H = TRANSIENT STATE PRESSURE AT PUMP-END
H_R = RATED HEAD OF THE PUMP

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY		
HAT CREEK PROJECT		
WIND GAP PUMPING PLANT (UNIT No. 4)		
COMPARISON OF COMPUTED & MEASURED RESULTS FOLLOWING POWER FAILURE		
DATE	DEC 1977	FIG. 1 R
DWN	MJS	DWG No. 604H-C14-B31

REPORT No. 900

DISCHARGE VALVE CLOSED
GRADUALLY IN 22 SECONDS



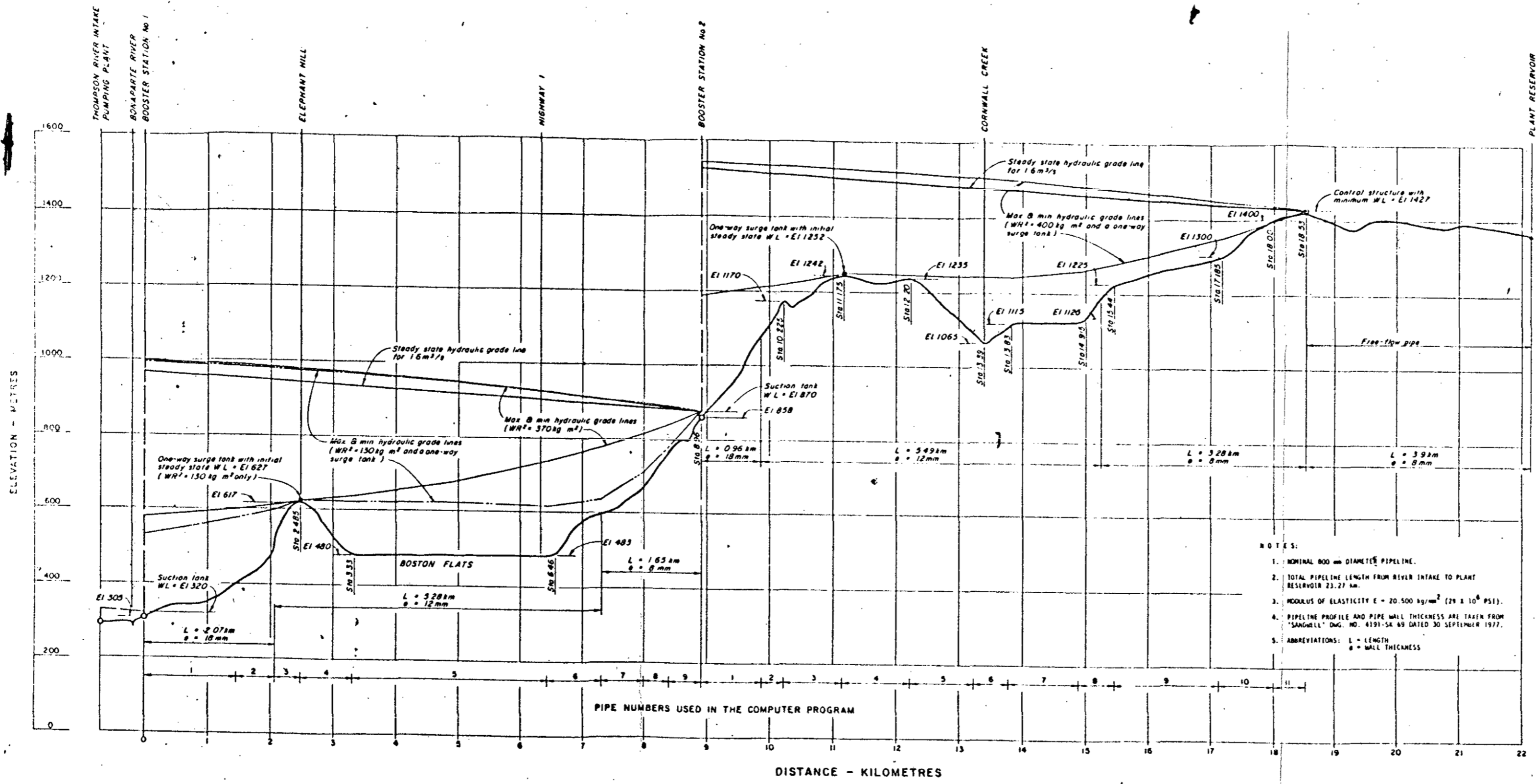
NOTE:

1. MEASURED RESULTS WERE RECORDED DURING PROTOTYPE TESTS CONDUCTED BY THE DEPARTMENT OF WATER RESOURCES, STATE OF CALIFORNIA, SACRAMENTO, CALIFORNIA.
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BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
HAT CREEK PROJECT
WIND GAP PUMPING PLANT (UNIT No. 4)
COMPARISON OF COMPUTED & MEASURED
RESULTS FOLLOWING POWER FAILURE

DATE	DEC 1977	FIG. 2	R
DWN	MJS	DWS No. 604H-C14-B 32	

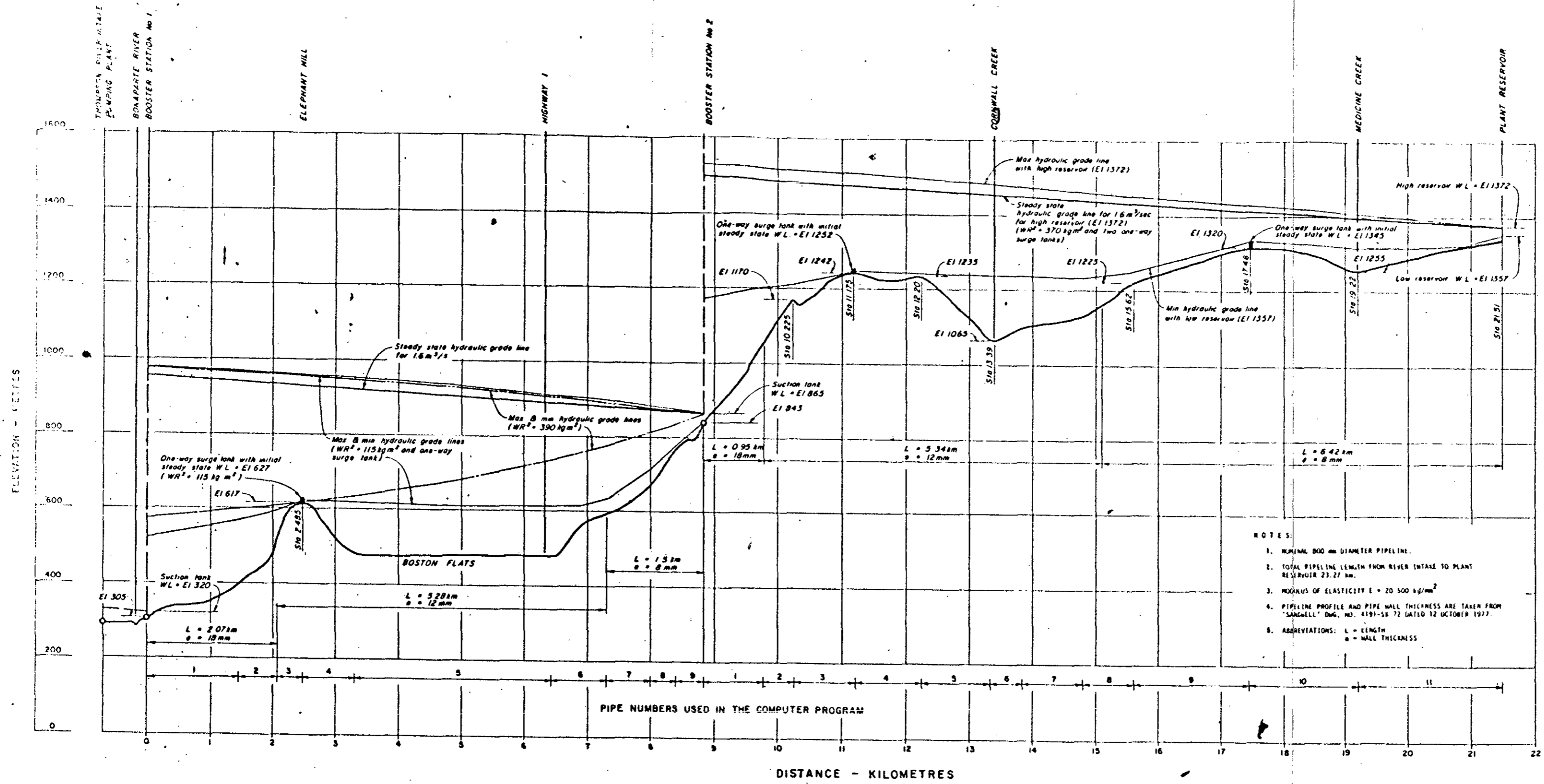


- NOTES:
1. NOMINAL 800 mm DIAMETER PIPELINE.
 2. TOTAL PIPELINE LENGTH FROM RIVER INTAKE TO PLANT RESERVOIR 23.27 km.
 3. MODULUS OF ELASTICITY $E = 20,500 \text{ kg/cm}^2$ ($29 \times 10^6 \text{ PSI}$).
 4. PIPELINE PROFILE AND PIPE WALL THICKNESS ARE TAKEN FROM "SANGWELL" DWG. NO. 4191-SK 69 DATED 30 SEPTEMBER 1977.
 5. ABBREVIATIONS: L = LENGTH, t = WALL THICKNESS

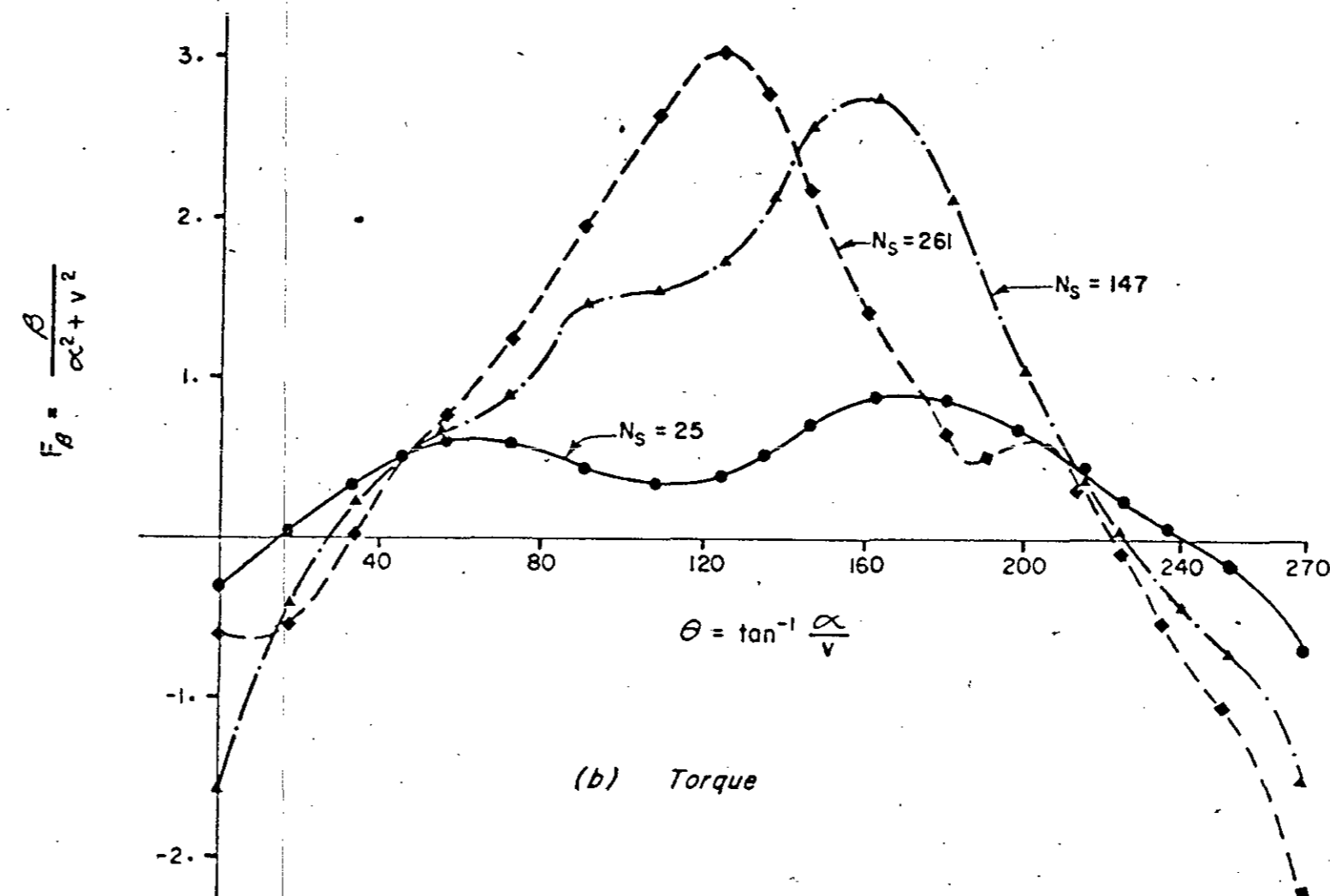
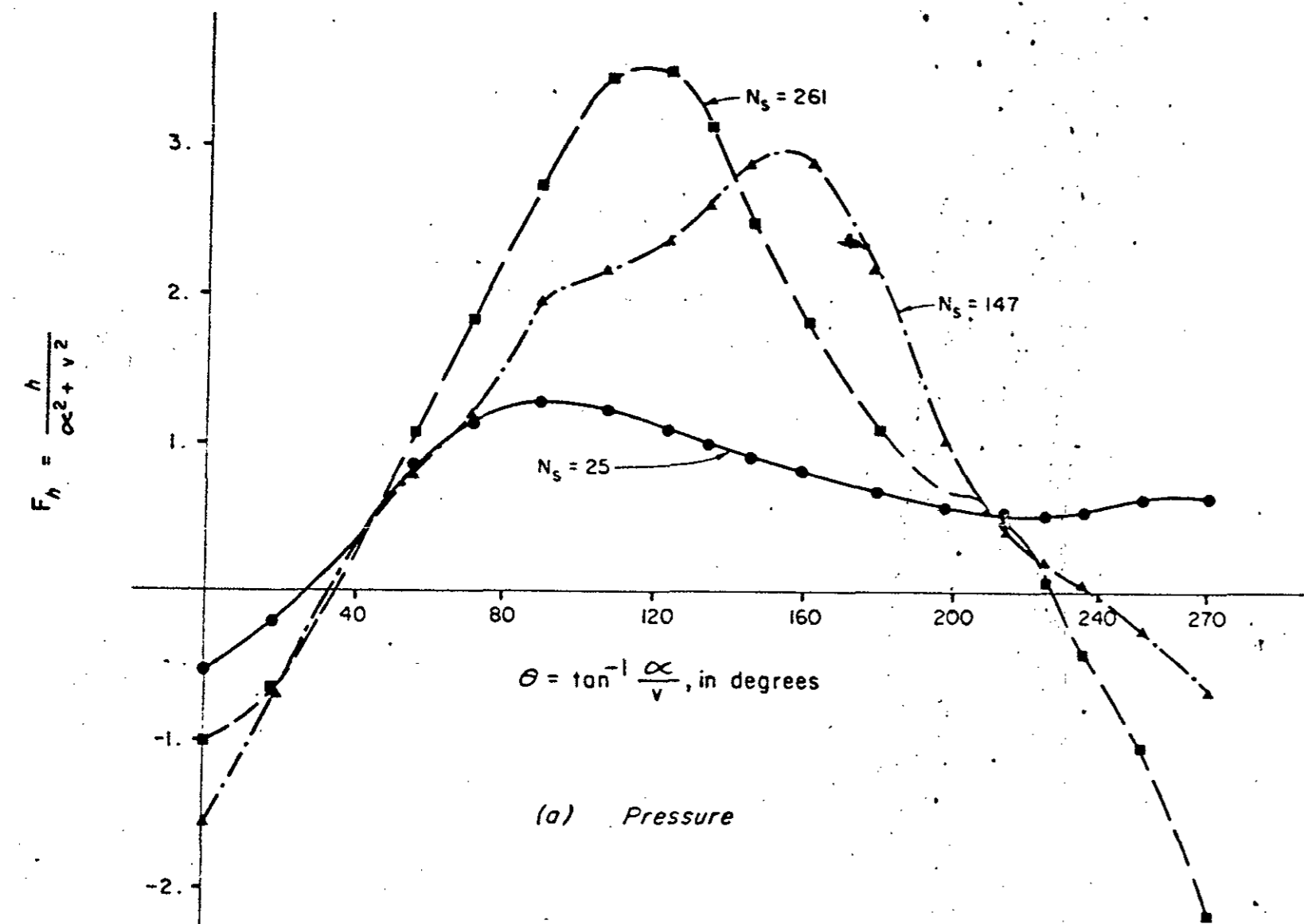
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

HAT CREEK PROJECT
 COOLING WATER SUPPLY
 PIPELINE PROFILE
 SUMMIT ROUTE

DATE	DEC 1977	FIG. 3
OWN	PSW	(HWG NO) 60-4H-C14-U33



BRITISH COLUMBIA HYDRO AND POWER AUTHORITY		
HAT CREEK PROJECT COOLING WATER SUPPLY PIPELINE PROFILE MEDICINE CREEK ROUTE		
DATE	DEC 1977	FIG. 4
DWN	DBG	DWG No 60411-C14-U34



NOTES:

1. This figure is taken from Applied Hydraulic Transients by M. H. Chaudhry, Ref. 2.

2. $\alpha = N/N_R$; $v = Q/Q_R$

$\beta = T/T_R$; $h = H/H_R$

$$N_s = \frac{N_R \sqrt{Q_R}}{H_R^{3/4}}$$

In these expressions, N = speed, T = torque, Q = flow, H = pumping head and subscript "R" refers to rated conditions.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
HAT CREEK PROJECT COOLING WATER SUPPLY CHARACTERISTICS OF PUMPS OF VARIOUS SPECIFIC SPEEDS	
DATE DEC 1977	FIG. 5
DWN DBG	DWS No 604H-C14-D35