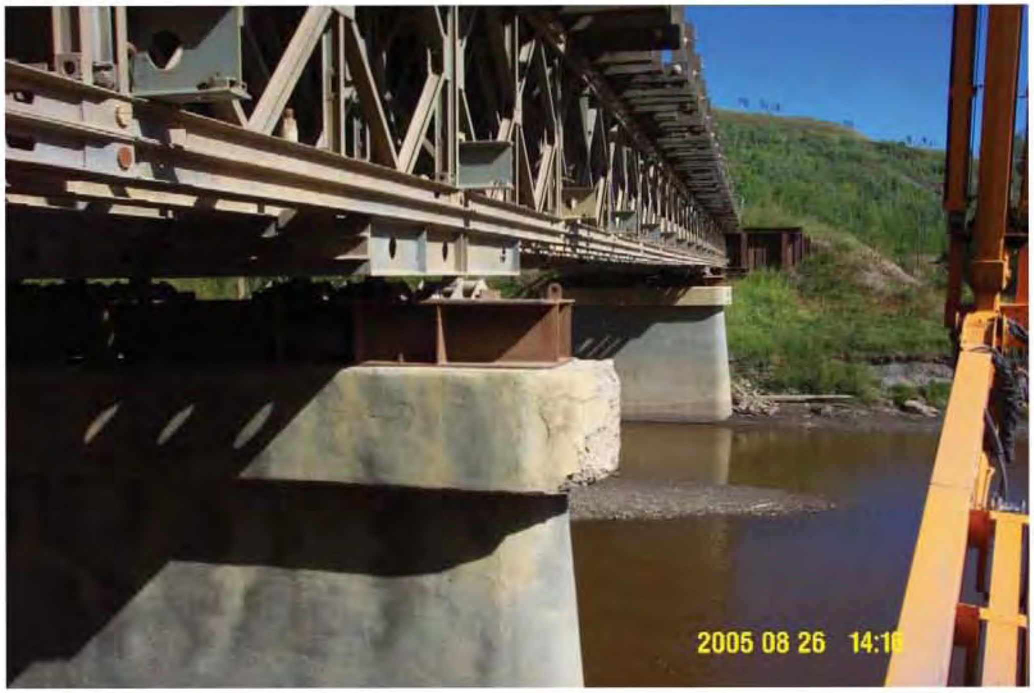

10/23/2008


10/23/2008


2005 08 26 14:16


2005 08 26 14:36


2005 08 26 15:03


7/30/02 09:25


09/25/2009 15:17


15/06/2010


15/06/2010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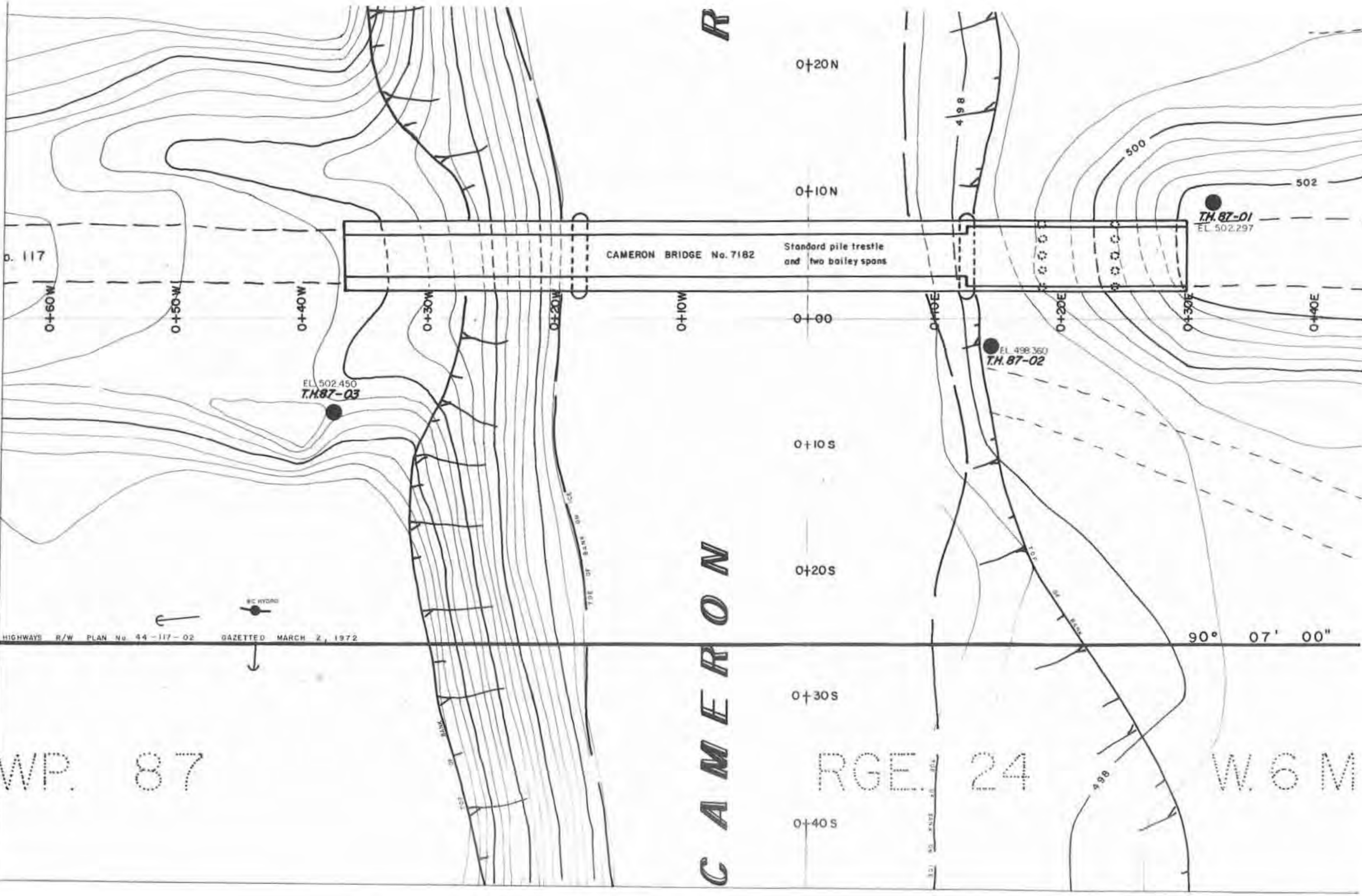
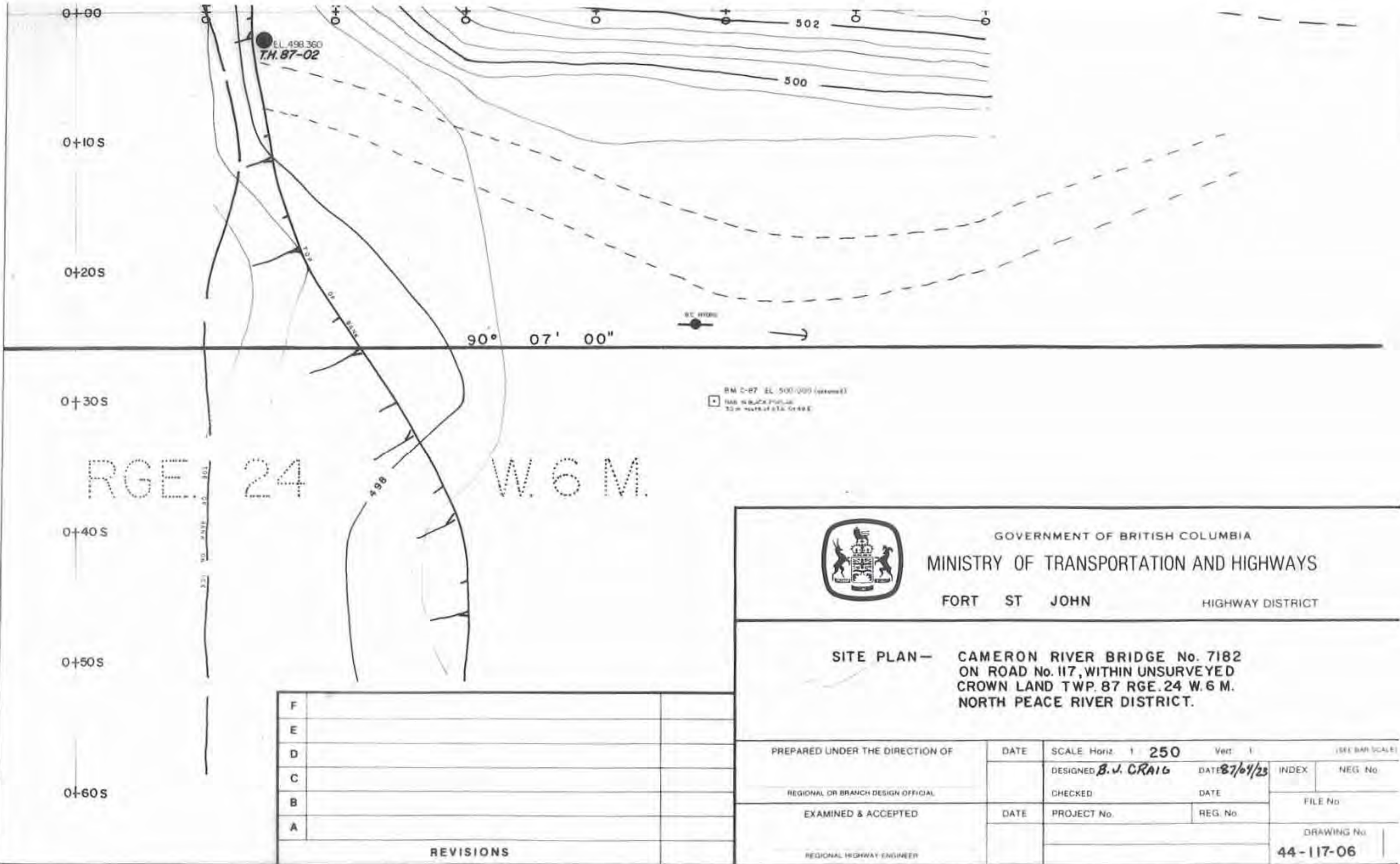
15/06/2010

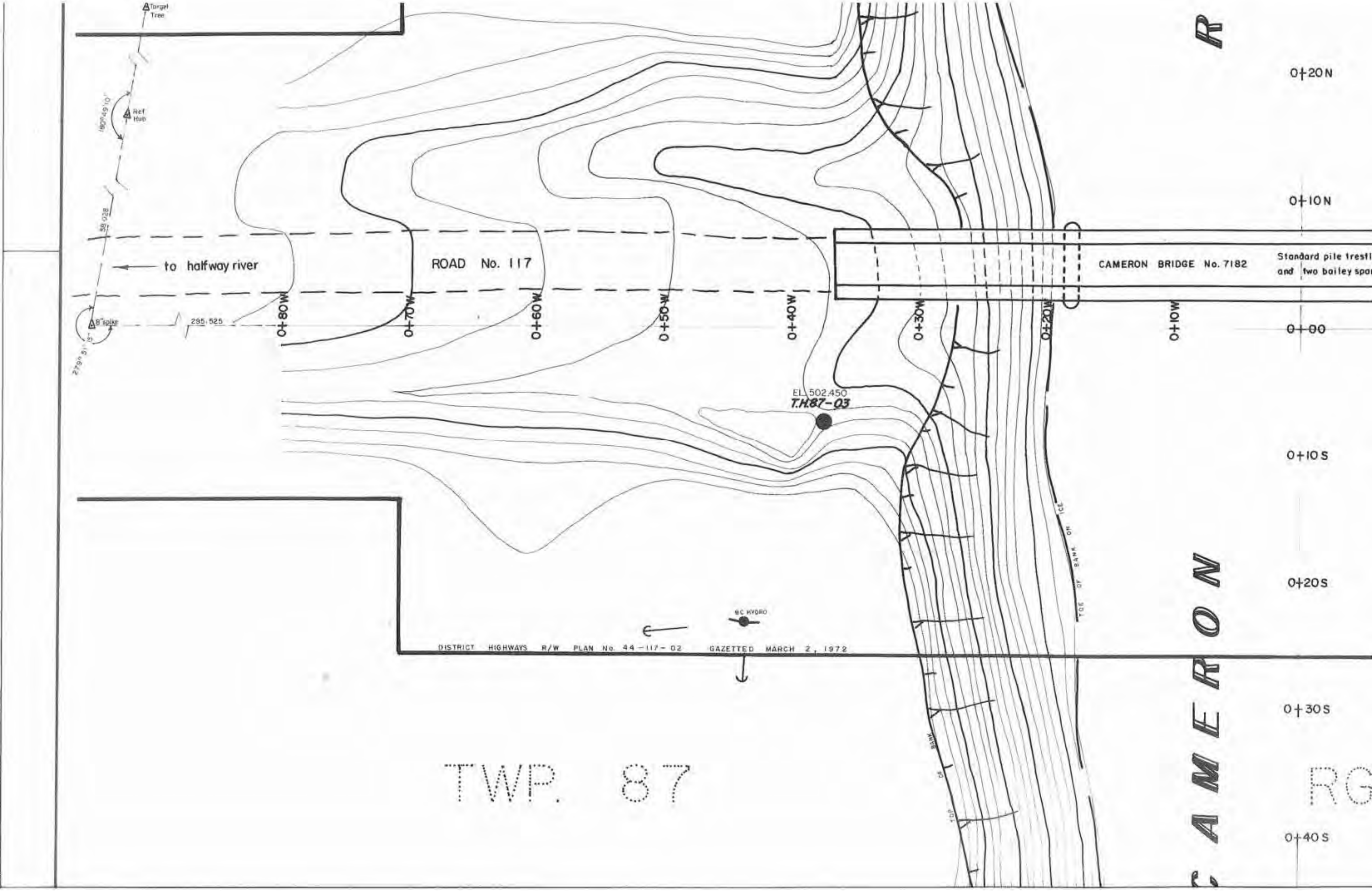

15/06/2010

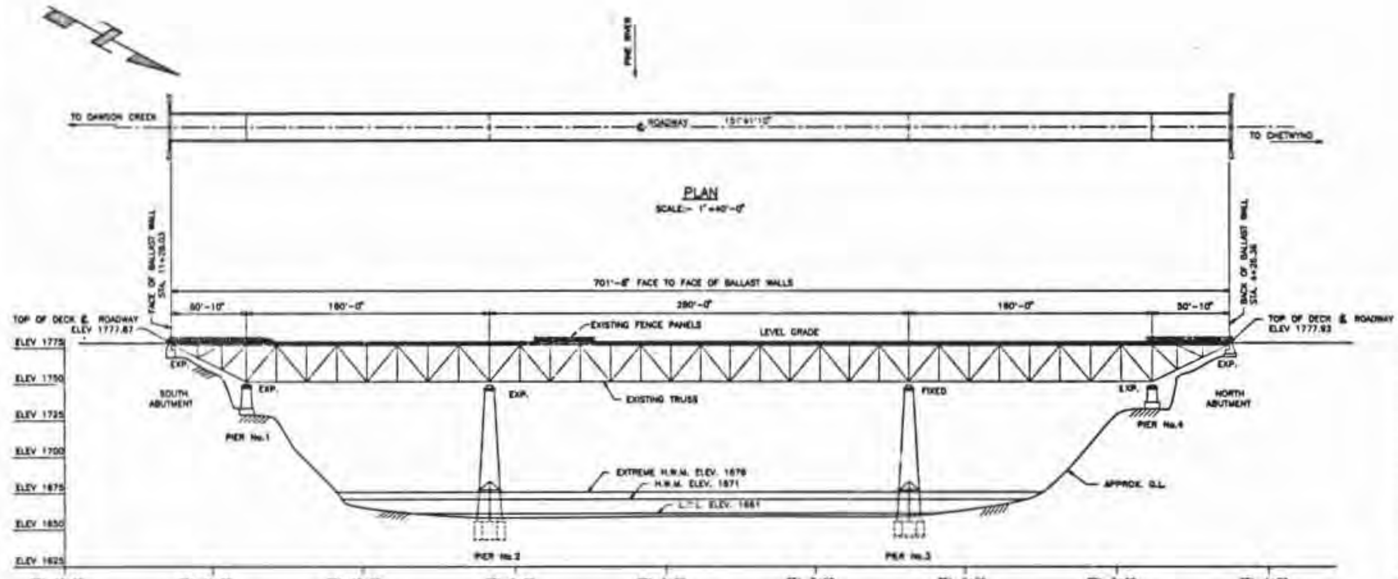


CAMERON BRIDGE No. 7182
Standard pile trestle and two bailey spans
T.H. 87-01
EL. 502.297
T.H. 87-02
EL. 498.360
T.H.87-03
EL. 502.450
0+20N
0+10N
0+00
0+10S
0+20S
0+30S
0+40S
0+60W
0+50W
0+40W
0+30W
0+20W
0+10W
0+10E
0+20E
0+30E
0+40E
500
502
498
HIGHWAYS R/W PLAN No. 44-117-02 GAZETTED MARCH 2, 1972
BC HYDRO
90° 07' 00"
TOE OF BANK
TOP OF BANK
CAMERON R
WP. 87
RGE. 24
W. 6 M



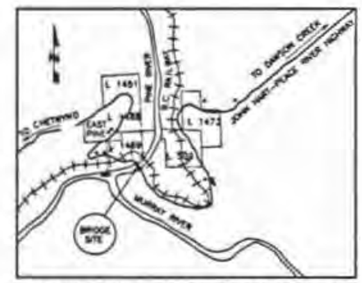
0+00
0+10 S
0+20 S
0+30 S
0+40 S
0+50 S
0+60 S
EL. 498.360
T.H. 87-02
502
500
498
TOP OF BANK
B.C. HYDRO
90° 07' 00"
RGE. 24 W. 6 M.
GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
FORT ST JOHN HIGHWAY DISTRICT
SITE PLAN— CAMERON RIVER BRIDGE No. 7182
ON ROAD No. 117, WITHIN UNSURVEYED
CROWN LAND TWP. 87 RGE. 24 W. 6 M.
NORTH PEACE RIVER DISTRICT.
PREPARED UNDER THE DIRECTION OF
DATE
SCALE Horiz. 1 : 250
Vert. 1 :
(SEE BAR SCALE)
DESIGNED B. J. CRAIG
DATE 87/04/23
INDEX
NEG. No.
REGIONAL OR BRANCH DESIGN OFFICIAL
CHECKED
DATE
FILE No.
EXAMINED & ACCEPTED
DATE
PROJECT No.
REG. No.
DRAWING No.
44-117-06
REGIONAL HIGHWAY ENGINEER
F
E
D
C
B
A
REVISIONS


Target Tree
Ref Hub
8" spike
295.525
to halfway river
ROAD No. 117
CAMERON BRIDGE No. 7182
Standard pile trestl
and two bailey spa
0+80W
0+70W
0+60W
0+50W
0+40W
0+30W
0+20W
0+10W
0+00
0+10N
0+20N
0+10S
0+20S
0+30S
0+40S
EL. 502.450
T.H.87-03
BC HYDRO
DISTRICT HIGHWAYS R/W PLAN No. 44-117-02 GAZETTED MARCH 2, 1972
TOE OF BANK ON ICE
TOP OF BANK
TWP. 87
RG
CAMERON R



PLAN
SCALE:- 1" =40'-0"

ELEVATION
SCALE:- 1" =40'-0"



THE BRIDGE IS LOCATED ON THE JOHN HART-PEACE RIVER HIGHWAY OVER THE PINE RIVER, APPROXIMATELY 30 Km EAST OF CHETWYND.

KEY MAP
SCALE:- 1:50 000

LIST OF DRAWINGS	
DWG. No.	TITLE
1174 - 40	SITE PLAN
" - 41	GENERAL ARRANGEMENT
" - 42	ABUTMENTS
" - 43	PIERS No. 1 AND 4
" - 44	PIERS No. 2 AND 3
" - 45	DETAILS AT BEARINGS
" - 46	BEARINGS
" - 47	TRUSS LAYOUT SHEET No. 1
" - 48	TRUSS LAYOUT SHEET No. 2
" - 49	CROSSFRAME ELEVATIONS
" - 50	TRUSS DETAILS SHEET No. 1
" - 51	TRUSS DETAILS SHEET No. 2
" - 52	TRUSS DETAILS SHEET No. 3
" - 53	TRUSS DETAILS SHEET No. 4
" - 54	CONNECTIONS TO EXISTING TRUSS
" - 55	FLOORBEAMS AND STRINGERS
" - 56	STRENGTHENING OF EXISTING TRUSSES
" - 57	DECK SHEET No. 1
" - 58	DECK SHEET No. 2
" - 59	DECK JOINTS
" - 60	PARAPET EXPANSION JOINTS
" - 61	DRAINS AND DOWNPIPES
" - 62	MODIFICATIONS TO EXISTING BEARINGS
" - 63	STRENGTHENING OF SUBSTRUCTURES
TE-89033-1	SITE PLAN - NOTES
TE-89033-2	WIRING DIAGRAM - ELEVATIONS - DETAILS
R4-86 - 02	PLAN
R4-86 - 03	PROFILE AND TYPICAL SECTIONS

STANDARD DRAWINGS	
DWG. No.	TITLE
2784 - 1	STANDARD BRIDGE PARAPET
" - 2	STANDARD BRIDGE PARAPET TRANSITION
2785 - 1	STANDARD BRIDGE PARAPET STEEL RAILING

NOTES
1. SURVEY BY:- DESIGN AND SURVEYS BRANCH
2. DATUM:- GEODETIC
3. GBM No. 470-E TABLET IN CONCRETE OF SOUTH ABUTMENT RIGHT OF STA. 11+15.48 AND 4'-7 1/4" BELOW BRIDGE DECK, ELEV. 1773.38.
4. GBM No. 471-E TABLET IN CONCRETE OF NORTH ABUTMENT RIGHT OF STA. 4+39.63 AND 4'-7 1/4" BELOW BRIDGE DECK, ELEV. 1773.41.

364297

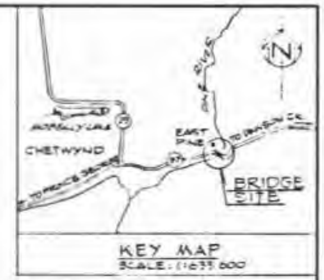
REFERENCE DRAWINGS	
DWG. No.	TITLE
1174-7	NORTH AND SOUTH ABUTMENTS
" -8	PIERS No. 1 AND No. 4
" -9	PIERS No. 2 AND No. 3
" -10	PROTECTION ANGLES AND NOSE PLATES
" -11	DETAIL OF CONCRETE DECK
V1804-9	VERTICALS AND JACKING GIRDERS
" -10	VERTICALS-JACKING GIRDERS AND BRACING
" -11 TO 18	TRUSS SECTIONS
" -19	FLOOR BEAMS, STRINGERS AND CURB CHANNELS
" -20 TO 24	BRACINGS
" -25 TO 27	TRUSS SECTIONS
" -28	BRACINGS
" -29	STRUTS AND CURB CHANNELS
" -45 TO 48	PIERS MEMBERS - BEARINGS
" -50	HANDRAILING AND POSTS
V1804-D2	ERECTION PROCEDURE
" -D3	ERECTION DEFORMATIONS
" -D4	CAMBER DIAGRAM
" -D5	ERECTION STRESSES
" -D6	ERECTION ANCHORS AT ABUTMENTS
" -D7	FALSE BENT
" -D8	30' TRUSS SPAN DESIGN
V1804-E3 TO E8	ERECTION DIAGRAMS

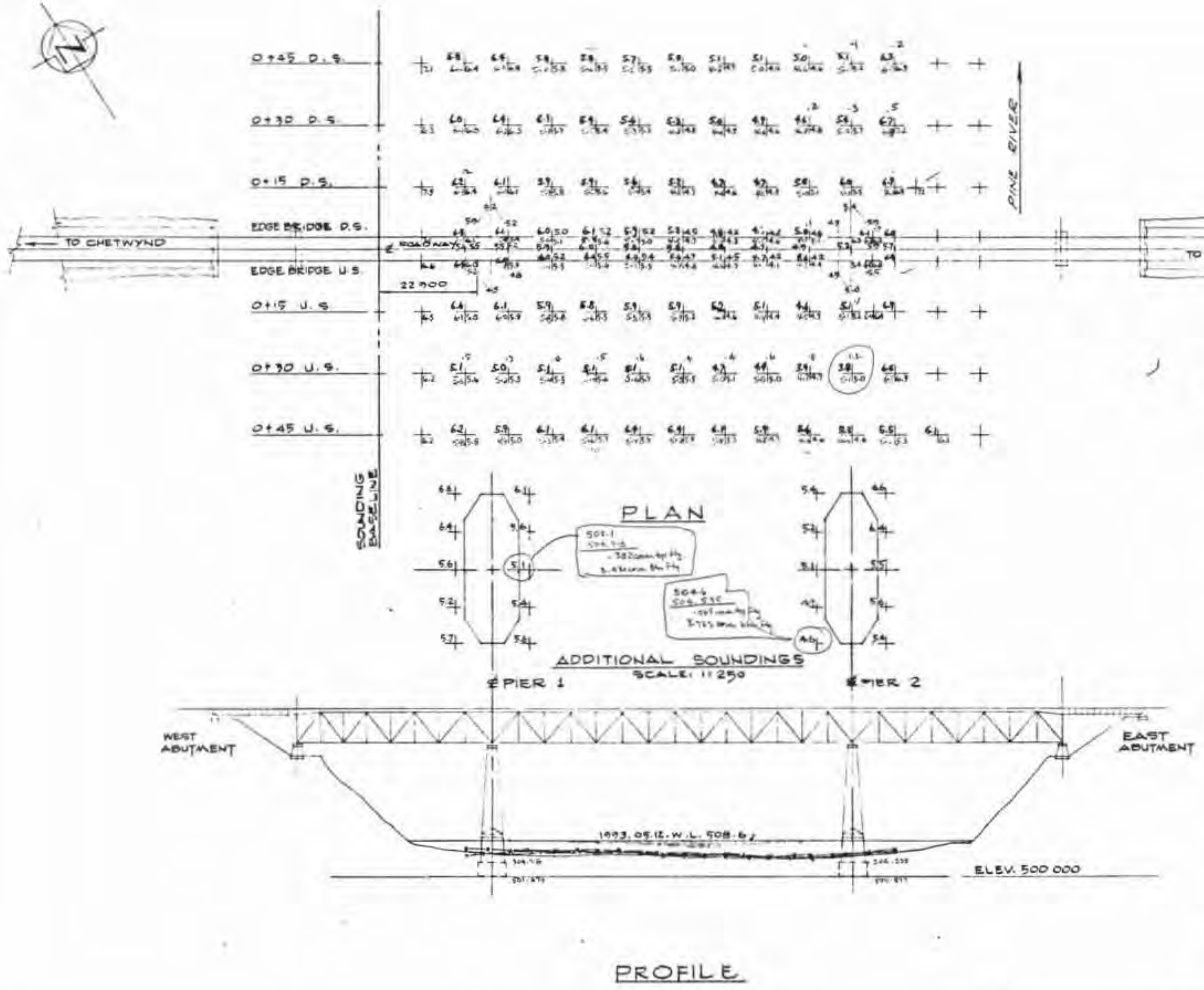
D		
C		
B	AS BUILT	
A	LIST OF DRAWINGS	
	REVISIONS	

GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH

DAWSON CREEK DISTRICT
JOHN HART-PEACE RIVER HIGHWAY
EAST PINE BRIDGE WIDENING
SITE PLAN

PREPARED UNDER THE DIRECTION OF	DATE	SCALE AS NOTED			NEG. No. 364297	
ORIGINAL SIGNED BY		DRAWN	G.W.	88/6/13	CHECKED G.D.W.	
RECOMMENDED	DATE	ACCEPTED FOR CONSTRUCTION		DATE	DRAWING No.	
ORIGINAL SIGNED BY		ORIGINAL SIGNED BY E.A. LUND			1174-40B	





LEGEND

1993.05.12. | 1988.10.07. (HAND DROP SOUNDING)
1990.06.19

PROFILES.

1978.05. PROFILE
1993.05.12. PROFILE AT EDGE BRIDGE D.S.
1993.05.12. PROFILE AT ℄
1993.05.12. PROFILE AT EDGE BRIDGE U.S.

[illegible]

NOTES:

1. ALL SOUNDINGS ARE IN METRES ABOVE 500 000 GEODETIC.
2. BENCHMARK, SET AT 20TH HANDRAIL POST WEST OF EAST ABUTMENT ON SOUTH SIDE OF BRIDGE TOP OF RAIL (KICK RAIL) ELEV. 542 110.
3. PROFILES ARE ON CENTRE LINE OF ROADWAY OR AS NOTED.
4. FOR PREVIOUS SOUNDINGS SEE DWG. Nº 1174-52.
5. ADDITIONAL SOUNDINGS AROUND PIERS ARE ABOUT 3 000 SPACING, OR AS NOTED.

MAY93-1

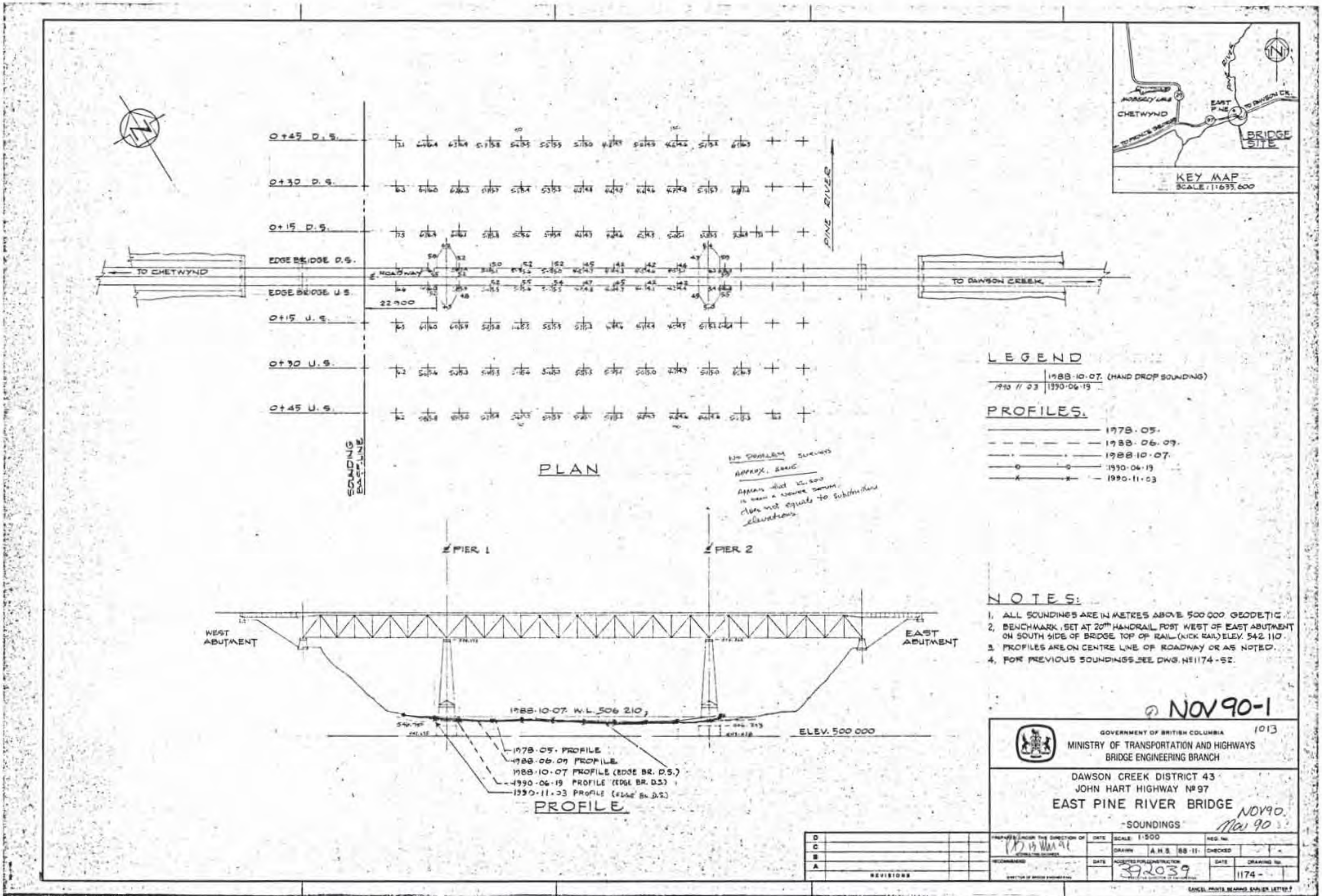
GOVERNMENT OF BRITISH COLUMBIA 1018
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH

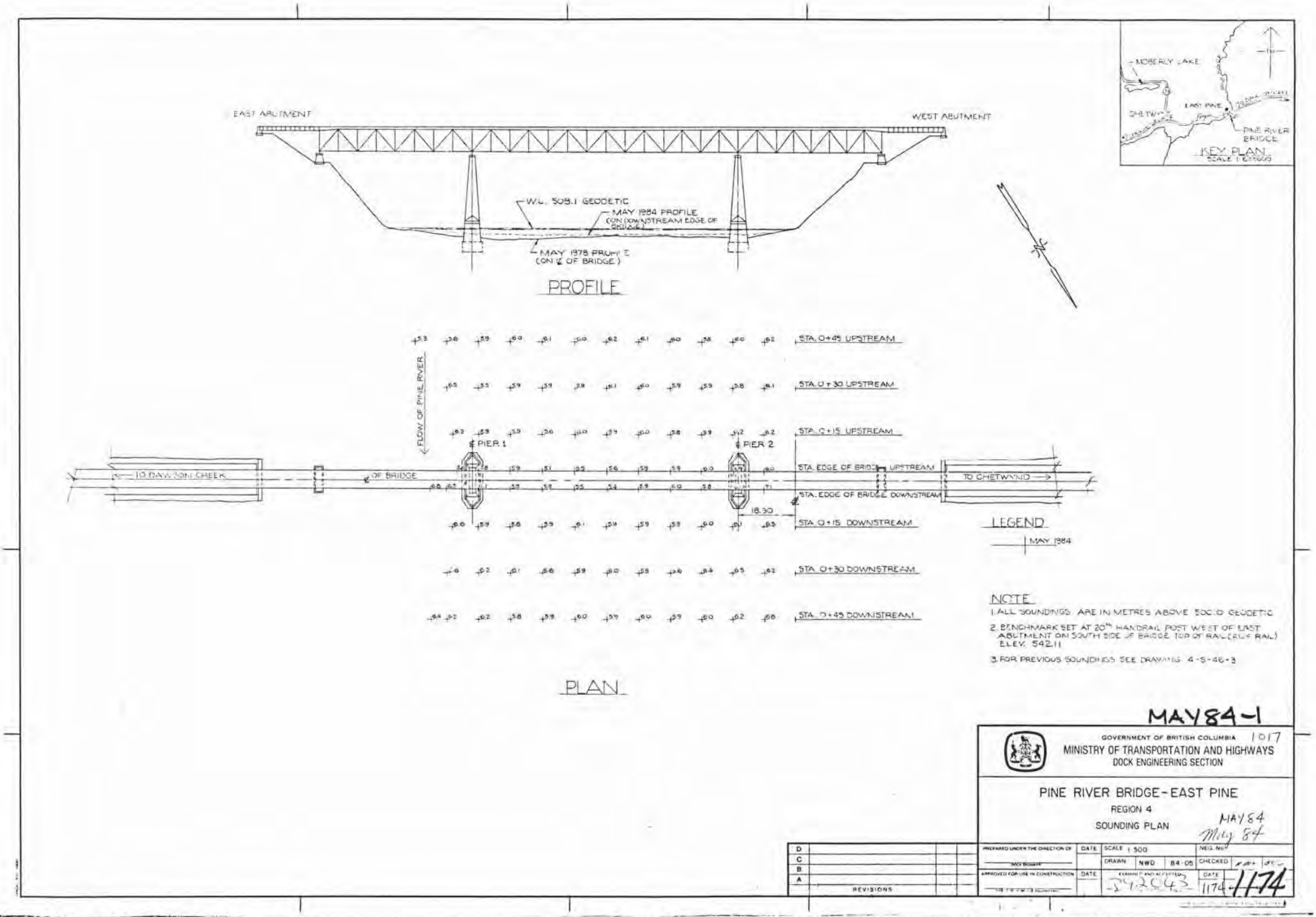
DAWSON CREEK DISTRICT 43
JOHN HART HIGHWAY Nº 97
EAST PINE RIVER BRIDGE
SOUNDINGS MAY 93

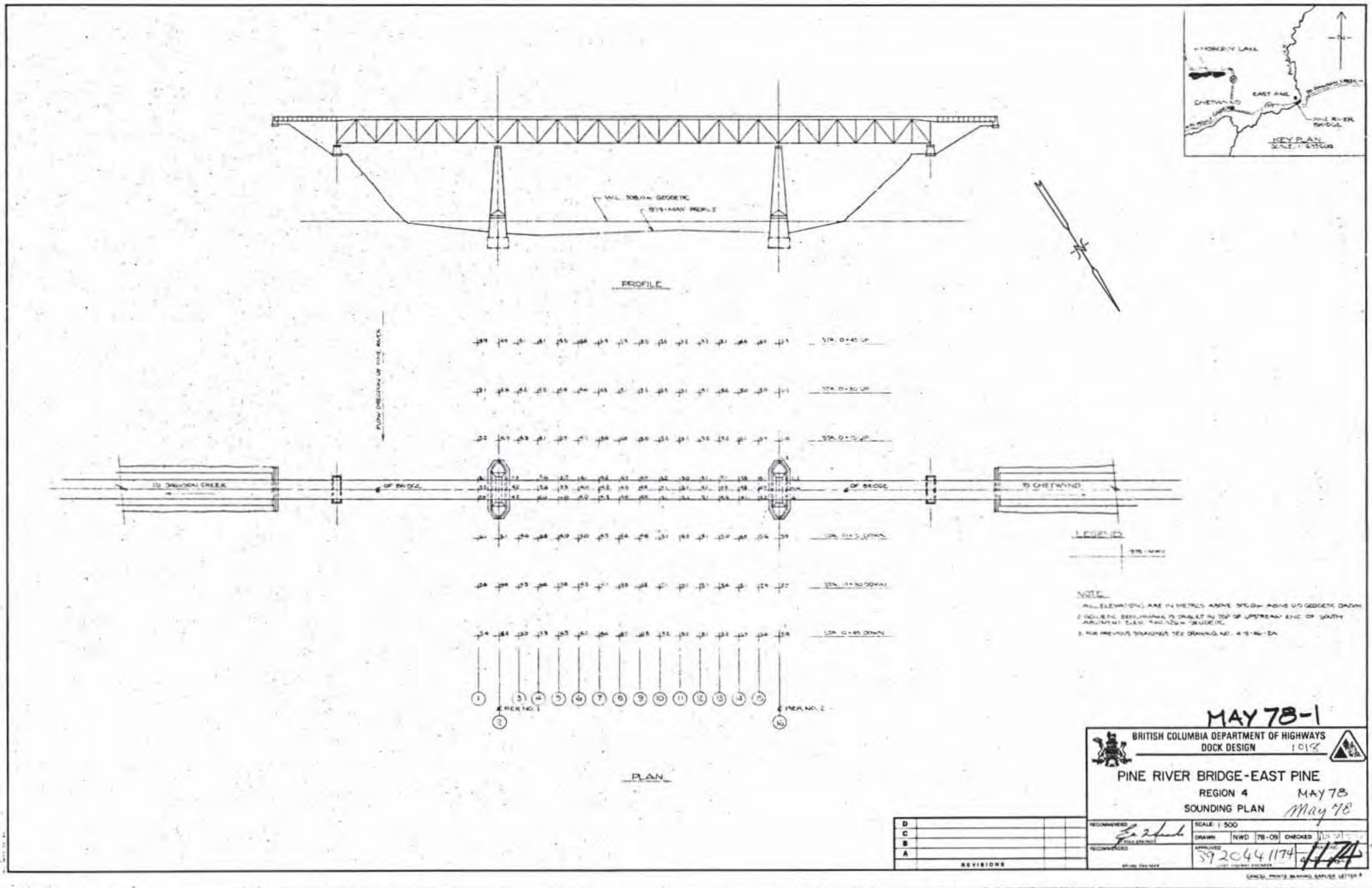
D			
C	1993.05.12. SOUNDINGS ADDED	AHS	93.06
B			
A			
REVISIONS			

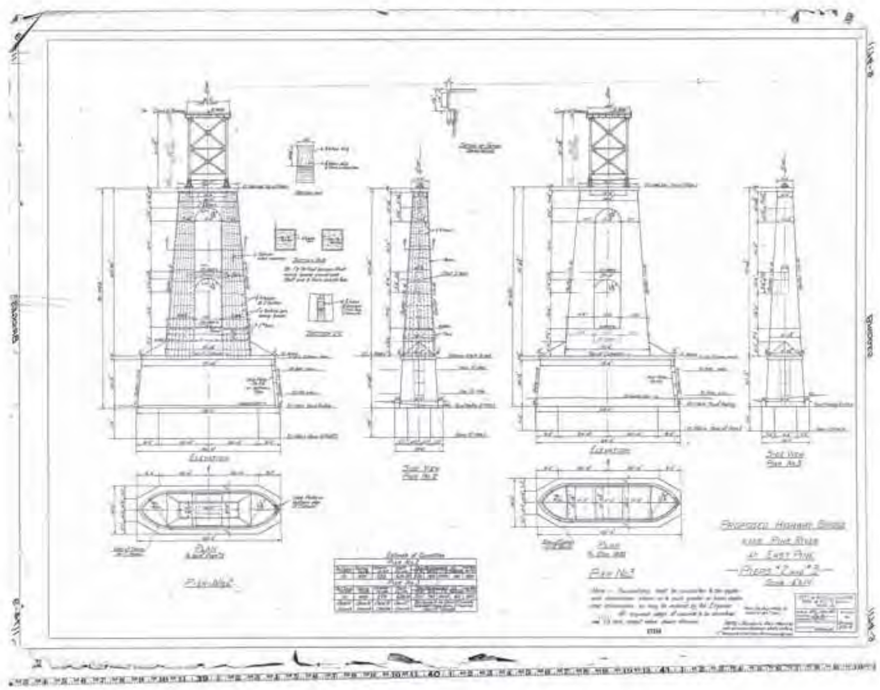
PREPARED UNDER THE DIRECTION OF	DATE	SCALE 1:500			NEG. No.	
		DRAWN	A.H.S.	88.11	CHECKED	
RECOMMENDED	DATE	ACCEPTED FOR CONSTRUCTION			DATE	DRAWING No.
		392038				1174 - C

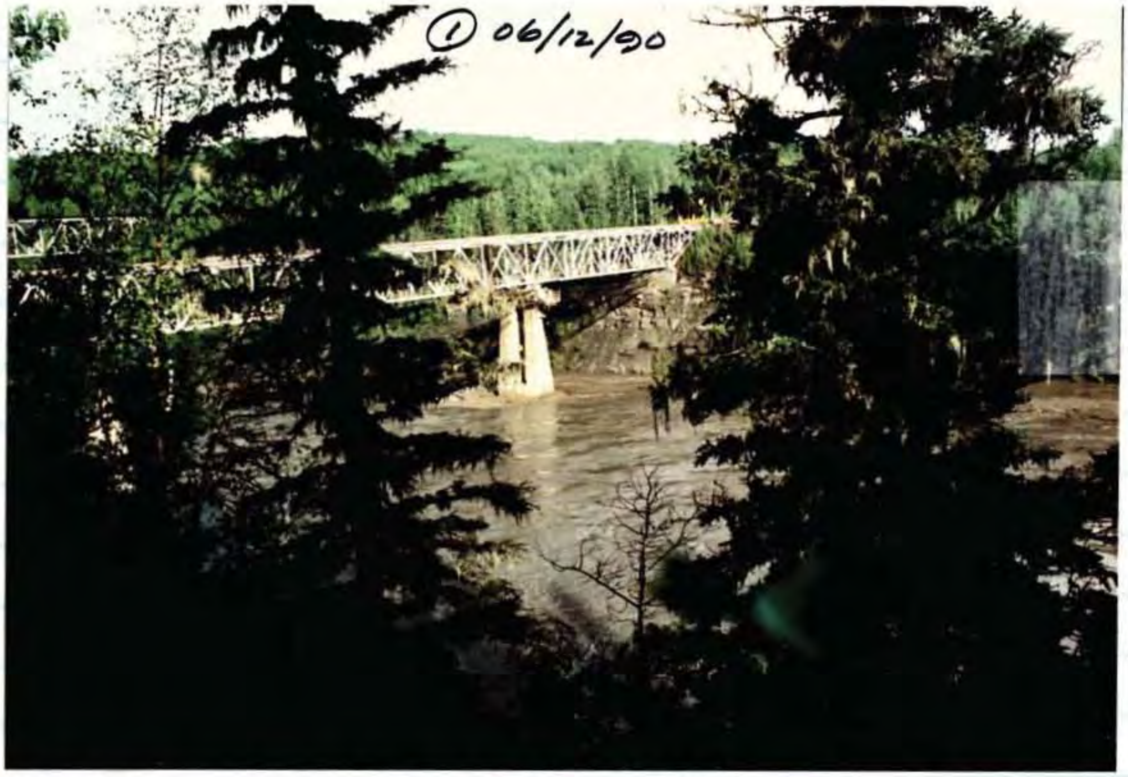
CANCEL PRINTS BEARING EARLIER LETTER


0+45 D.S.
0+30 D.S.
0+15 D.S.
EDGE BRIDGE D.S.
TO CHETWYND
EDGE BRIDGE U.S.
22 900
0+15 U.S.
0+30 U.S.
0+45 U.S.
PINE RIVER
TO DAWSON CREEK
SOUNDING BASE LINE
PLAN
No problem surveys
approx. same
Appears that EL. 500
is now a newer datum.
does not equate to substructure
elevations
℄ PIER 1
℄ PIER 2
WEST ABUTMENT
EAST ABUTMENT
1988-10-07 W.L. 506 210
ELEV. 500 000
1978-05- PROFILE
1988-06-09 PROFILE
1988-10-07 PROFILE (EDGE BR. D.S.)
1990-06-19 PROFILE (EDGE BR. D.S.)
1990-11-03 PROFILE (EDGE BR. D.S.)
PROFILE
KEY MAP
SCALE: 1:633,600
CHETWYND
EAST PINE
TO DAWSON CR.
BRIDGE SITE
LEGEND
1988-10-07. (HAND DROP SOUNDING)
1990 11 03 | 1990-06-19
PROFILES.
1978-05-
1988-06-09.
1988-10-07.
1990-06-19
1990-11-03
NOTES:
1. ALL SOUNDINGS ARE IN METRES ABOVE 500 000 GEODETIC.
2. BENCHMARK. SET AT 20TH HANDRAIL POST WEST OF EAST ABUTMENT ON SOUTH SIDE OF BRIDGE TOP OF RAIL (NICK RAIL) ELEV. 542 110.
3. PROFILES ARE ON CENTRE LINE OF ROADWAY OR AS NOTED.
4. FOR PREVIOUS SOUNDINGS SEE DWG. Nº 1174-52.
NOV90-1
1013
GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH
DAWSON CREEK DISTRICT 43
JOHN HART HIGHWAY Nº 97
EAST PINE RIVER BRIDGE
SOUNDINGS
NOV90.
Nov 90
SCALE 1:500
DRAWN A.H.S. 88-11-
REVISIONS
392039
1174-


EAST ABUTMENT
WEST ABUTMENT
W.L. 508.1 GEODETIC
MAY 1984 PROFILE (ON DOWNSTREAM EDGE OF BRIDGE)
MAY 1978 PROFILE (ON ℄ OF BRIDGE)
PROFILE
MOBERLY LAKE
EAST PINE
PINE RIVER BRIDGE
KEY PLAN
N
FLOW OF PINE RIVER
STA. 0+45 UPSTREAM
STA. 0+30 UPSTREAM
STA. 0+15 UPSTREAM
℄ PIER 1
℄ PIER 2
TO DAWSON CREEK
℄ OF BRIDGE
STA. EDGE OF BRIDGE UPSTREAM
TO CHETWYND
STA. EDGE OF BRIDGE DOWNSTREAM
18.30
STA. 0+15 DOWNSTREAM
STA. 0+30 DOWNSTREAM
STA. 0+45 DOWNSTREAM
LEGEND
MAY 1984
NOTE
1. ALL SOUNDINGS ARE IN METRES ABOVE 500.0 GEODETIC
2. BENCHMARK SET AT 20TH HANDRAIL POST WEST OF EAST ABUTMENT ON SOUTH SIDE OF BRIDGE TOP OF RAIL (RIGHT RAIL) ELEV. 542.11
3. FOR PREVIOUS SOUNDINGS SEE DRAWING 4-5-46-3
PLAN
MAY84-1
GOVERNMENT OF BRITISH COLUMBIA
1017
MINISTRY OF TRANSPORTATION AND HIGHWAYS
DOCK ENGINEERING SECTION
PINE RIVER BRIDGE - EAST PINE
REGION 4
SOUNDING PLAN
MAY84
May 84
SCALE 1:500
DRAWN NWD 84-05
CHECKED
REVISIONS
D
C
B
A
1174-1174


PROFILE
PLAN
KEY PLAN
LEGEND
NOTE
MAY 78-1
BRITISH COLUMBIA DEPARTMENT OF HIGHWAYS
DOCK DESIGN
1018
PINE RIVER BRIDGE-EAST PINE
REGION 4
MAY 78
SOUNDING PLAN
May 78
SCALE: 1:500
DRAWN
NWD
78-09
CHECKED
APPROVED
592044 1174
1174
D
C
B
A
REVISIONS





① 06/12/90 East Pine Bridge - No. 2 Pier
High Water

② 06/12/90 East Pine Bridge - No. 2 Pier
High Water

③ 06/12/90 Pier 2 High water
East Pine Br.

④ 9/89 Pier 2 Contrast with ③ above
East Pine Bridge

Province of British Columbia

Ministry of Transportation and Highways

MEMORANDUM

To: S.D. Gladysz
District Highways Manager,
Dawson Creek, B.C.

Date: February 26, 1985
Branch/Dist: Region 4
Address: #213-1011 Fourth Ave,
Prince George, B.C.
V2L 3H9
Phone: 565-6508
Hdqtrs. File:
Region File: R4-40-0
District File: 40 0

RE: Bridge Soundings: Pine River Bridge (East Pine)

We have received 1984 soundings for the above bridge's channel(s). No unusual scouring or infilling has occurred.

If you require any information concerning these soundings, please contact the undersigned.

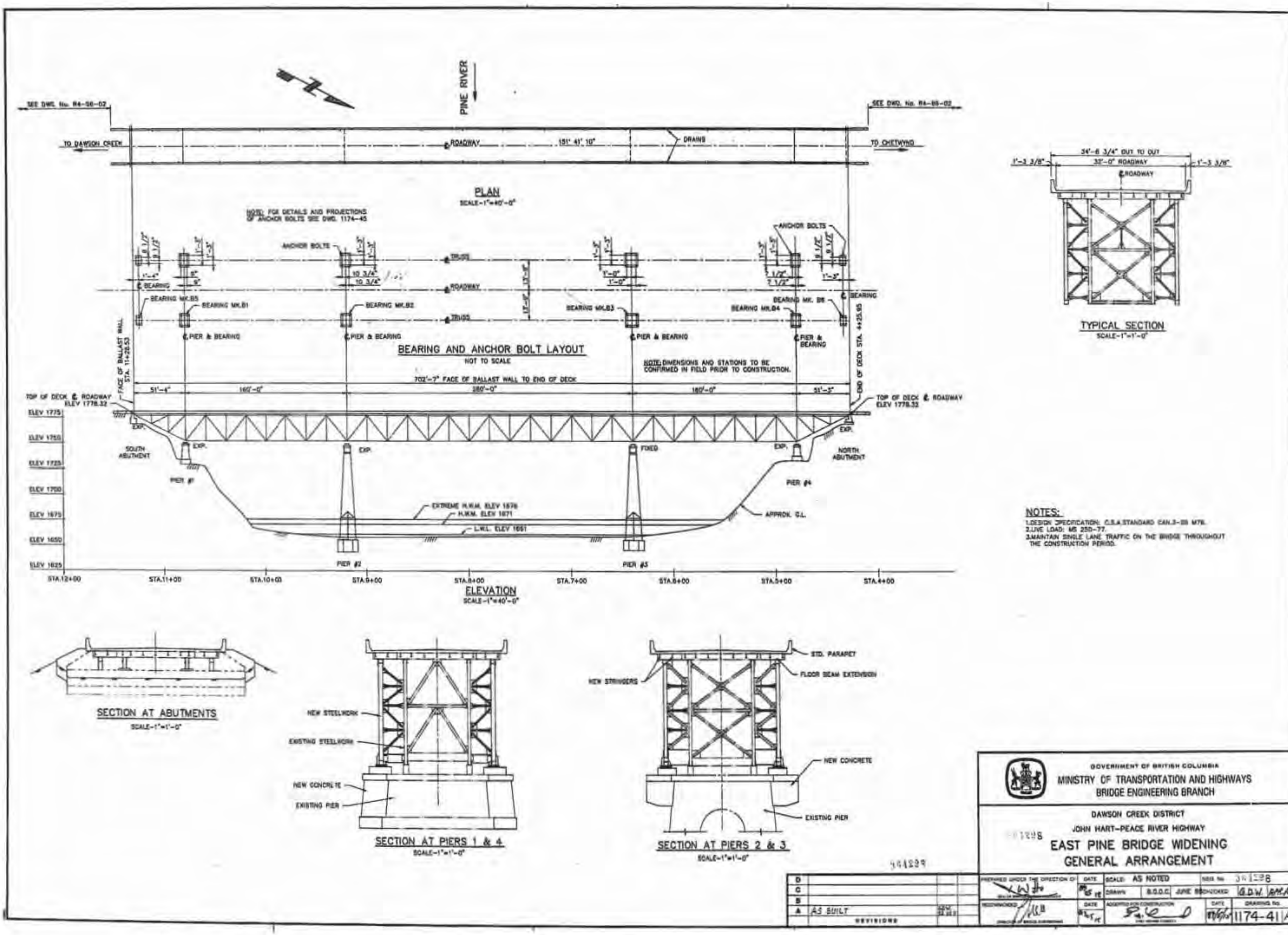
J.N. Ryan
Acting Regional Bridge Technician

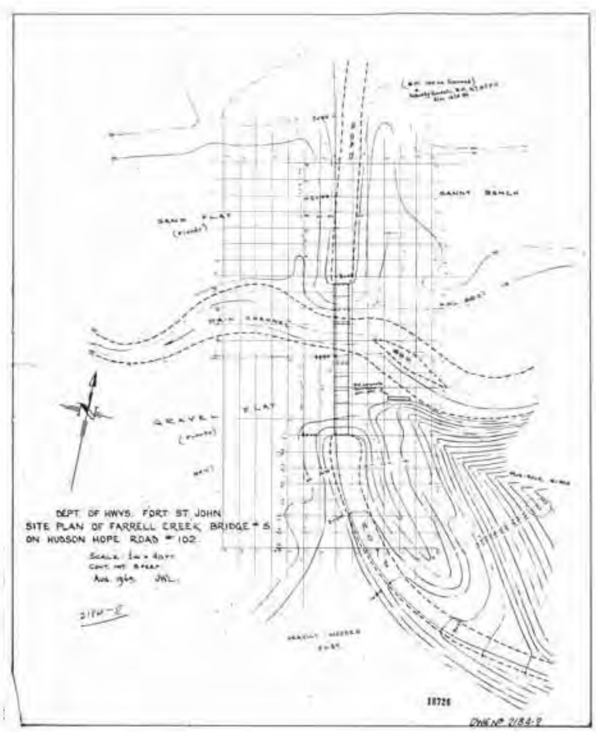
JNR/rs

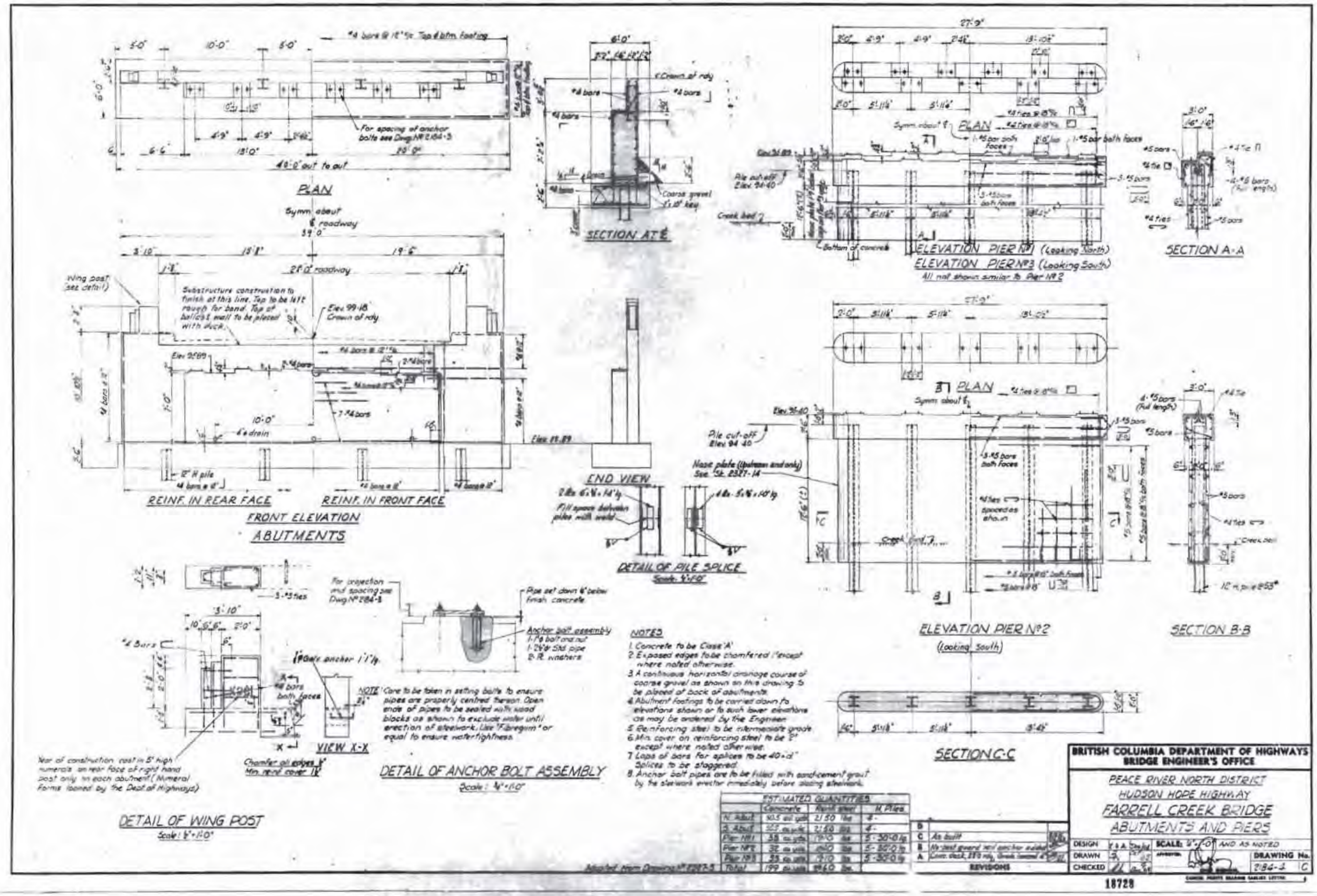
Att: Peter Tali – your info
R. Grant – for the Bridge record.

SM 85/02/28

H.118 (REV. 84 05) W 440


PINE RIVER
TO DAWSON CREEK
TO CHETWYND
PLAN
BEARING AND ANCHOR BOLT LAYOUT
NOT TO SCALE
NOTE: DIMENSIONS AND STATIONS TO BE CONFIRMED IN FIELD PRIOR TO CONSTRUCTION.
702'-7" FACE OF BALLAST WALL TO END OF DECK
SOUTH ABUTMENT
NORTH ABUTMENT
PIER #1
PIER #2
PIER #3
PIER #4
APPROX. G.L.
ELEVATION
TYPICAL SECTION
SCALE-1"=1'-0"
NOTES:
1.DESIGN SPECIFICATION: C.S.A.STANDARD CAN.3-S6 M78.
2.LIVE LOAD: MS 250-77.
3.MAINTAIN SINGLE LANE TRAFFIC ON THE BRIDGE THROUGHOUT THE CONSTRUCTION PERIOD.
SECTION AT ABUTMENTS
SCALE-1"=1'-0"
NEW STEELWORK
EXISTING STEELWORK
NEW CONCRETE
EXISTING PIER
SECTION AT PIERS 1 & 4
SCALE-1"=1'-0"
STD. PARAPET
FLOOR BEAM EXTENSION
NEW STRINGERS
SECTION AT PIERS 2 & 3
SCALE-1"=1'-0"
GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH
DAWSON CREEK DISTRICT
JOHN HART-PEACE RIVER HIGHWAY
EAST PINE BRIDGE WIDENING
GENERAL ARRANGEMENT
SCALE: AS NOTED
A AS BUILT
REVISIONS
1174-41 A


SANDY BENCH
MAIN CHANNEL
GRAVEL FLAT
DEPT. OF HWYS. FORT ST. JOHN
SITE PLAN OF FARRELL CREEK BRIDGE # 5
ON HUDSON HOPE ROAD # 102
10726


PLAN
SECTION AT ℄
FRONT ELEVATION
REINF. IN REAR FACE
REINF. IN FRONT FACE
ABUTMENTS
END VIEW
DETAIL OF PILE SPLICE
ELEVATION PIER Nº1 (Looking North)
ELEVATION PIER Nº3 (Looking South)
All not shown similar to Pier Nº2
SECTION A-A
ELEVATION PIER Nº2
(Looking South)
SECTION B-B
SECTION C-C
VIEW X-X
DETAIL OF ANCHOR BOLT ASSEMBLY
DETAIL OF WING POST
NOTES
1. Concrete to be Class 'A'
2. Exposed edges to be chamfered 1" except where noted otherwise.
3. A continuous horizontal drainage course of coarse gravel as shown on this drawing to be placed at back of abutments.
4. Abutment footings to be carried down to elevations shown or to such lower elevations as may be ordered by the Engineer.
5. Reinforcing steel to be intermediate grade.
6. Min. cover on reinforcing steel to be 2" except where noted otherwise.
7. Laps of bars for splices to be 40×d. Splices to be staggered.
8. Anchor bolt pipes are to be filled with sand-cement grout by the steelwork erector immediately before placing steelwork.
ESTIMATED QUANTITIES
BRITISH COLUMBIA DEPARTMENT OF HIGHWAYS
BRIDGE ENGINEER'S OFFICE
PEACE RIVER NORTH DISTRICT
HUDSON HOPE HIGHWAY
FARRELL CREEK BRIDGE
ABUTMENTS AND PIERS
REVISIONS
C As built
DESIGN
DRAWN
CHECKED
SCALE: ¼"=1'-0" AND AS NOTED
DRAWING No.
2184-2
C
18728

4423

PROVINCE OF BRITISH COLUMBIA

DEPARTMENT OF HIGHWAYS

TO: Bridge Engineer,
VICTORIA, B.C.

ATTENTION:

SUBJECT:
Lynx Creek Bridge and Farrell Creek Bridge.
Hudson Hope to Charlie Lake Road.

SENDER'S ADDRESS: Prince George, B.C.

DATE: September 15, 1965

ELECTORAL DISTRICT: North Peace River

HEADQUARTERS FILE: 4423

REGIONAL FILE: R4-N27-40-0

DISTRICT FILE:

REFERENCE: DATED:

I refer you to your letter of August 18th and your Plan No. 2327-3 accompanying it, referring to Lynx Creek.

Since this plan was prepared we have experienced a disastrous flood on this creek on June 28th, and I enclose a photograph of the old bridge, washed out, and the flow at peak flood. This we estimate to be at least 130 feet wide. At present we have 120 feet of bailey bridge in position.

Although this may be considered a 100-year flood, I believe we invite criticism and maybe disaster if we do less than design for it.

With this in mind I enclose a cross-section showing an additional 50 foot span to make the bridge consist of 2 - 50 foot main spans and 2 - 30 foot approach spans. I am enquiring if you are in agreement.

In the same flood we lost the Farrell Creek Bridge which consisted of a 50 foot steel beam span with trestle approaches. We have a temporary bridge at Farrell Creek comprising 190 feet of double single bailey. I enclose a photograph of it and I also enclose a cross-section we have prepared showing an identical bridge to Lynx Creek which I believe would suit here very well. I also enclose a plan of the Farrell Creek Crossing showing the present Bailey bridge.

I believe it is essential we replace both these bridges this winter as I do not feel the Bailey bridges are reliable for break-up and spring run-off and floods.

I suggest the following course. That you design Farrell Creek similar to Lynx as shown on Drawing No. 2327-3 with the addition of a new centre pier on each. Drilling of both sites by diamond drill has already been requested and your assistance in having this expedited is asked. Farrell Creek bottom is suspected to be mud rock.

1550136

BR. ENGR.

FOR DEPARTMENTAL CORRESPONDENCE ONLY.

FORM H. 183

PROVINCE OF BRITISH COLUMBIA

DEPARTMENT OF HIGHWAYS

TO:	SENDER'S ADDRESS: Prince George, B.C.
	DATE: September 15, 1965
	ELECTORAL DISTRICT: North Peace River
	HEADQUARTERS FILE: 4423
ATTENTION:	REGIONAL FILE: R4-N27-40-0
	DISTRICT FILE:
SUBJECT: Lynx Creek Bridge and Farrell Creek Bridge. Hudson Hope to Charlie Lake Road.	REFERENCE: DATED:

FOR DEPARTMENTAL CORRESPONDENCE ONLY.

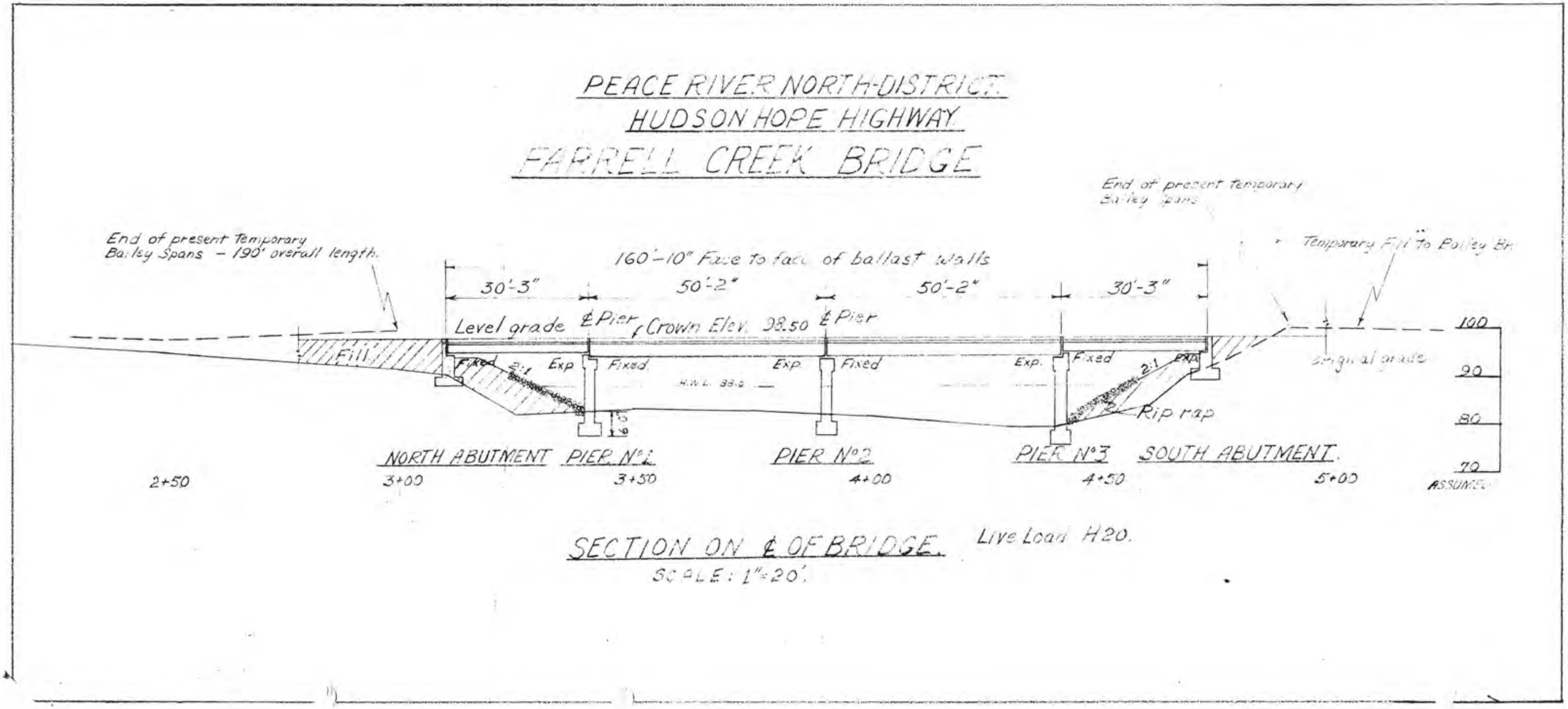
Then I believe the Department should call two contracts, one for the substructure on both bridges and the second for the hauling and placing of the steel beams and diaphragms (assuming beams are available in Vancouver). Then the district bridge crew can place the laminated sub deck on both bridges and erect the fence and put down a timber running deck to be replaced by asphalt mix next summer.

The district bridge crew will be fully engaged to do this and the Upper Halfway and Cameron Bridges which also were washed out. The district could not handle the concrete work.

Please advise what the decision is for these two bridges.

R.G. Harvey,
Regional Highway Engineer.

RGH/1
Encl.


PEACE RIVER NORTH-DISTRICT
HUDSON HOPE HIGHWAY
FARRELL CREEK BRIDGE
End of present temporary Bailey Spans
End of present Temporary Bailey Spans - 190' overall length.
Temporary Fill to Bailey Br.
160'-10" Face to face of ballast walls
30'-3"
50'-2"
50'-2"
30'-3"
Level grade
℄ Pier
℄ Crown Elev. 98.50
℄ Pier
Fill
Fixed
2:1
Exp
Fixed
Exp
Fixed
Exp.
Fixed
2:1
Exp
H.W.L. 88.0
Rip rap
original grade
100
90
80
70
ASSUMED
NORTH ABUTMENT
PIER Nº1
PIER Nº2
PIER Nº3
SOUTH ABUTMENT.
2+50
3+00
3+50
4+00
4+50
5+00
SECTION ON ℄ OF BRIDGE.
Live Load H20.
SCALE: 1"=20'.

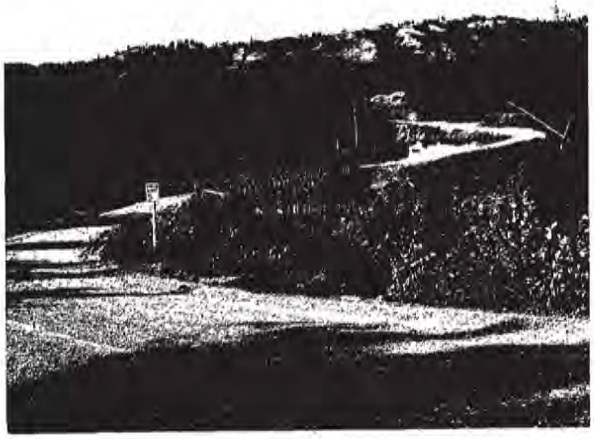
FORT ST JOHN DISTRICT

Farrell Creek Bridge #5

Sept ' 1965.

proposed location of concrete piers and abutments for 2 - 30 ft + 2 - 50 foot steel beam spans shown in yellow).

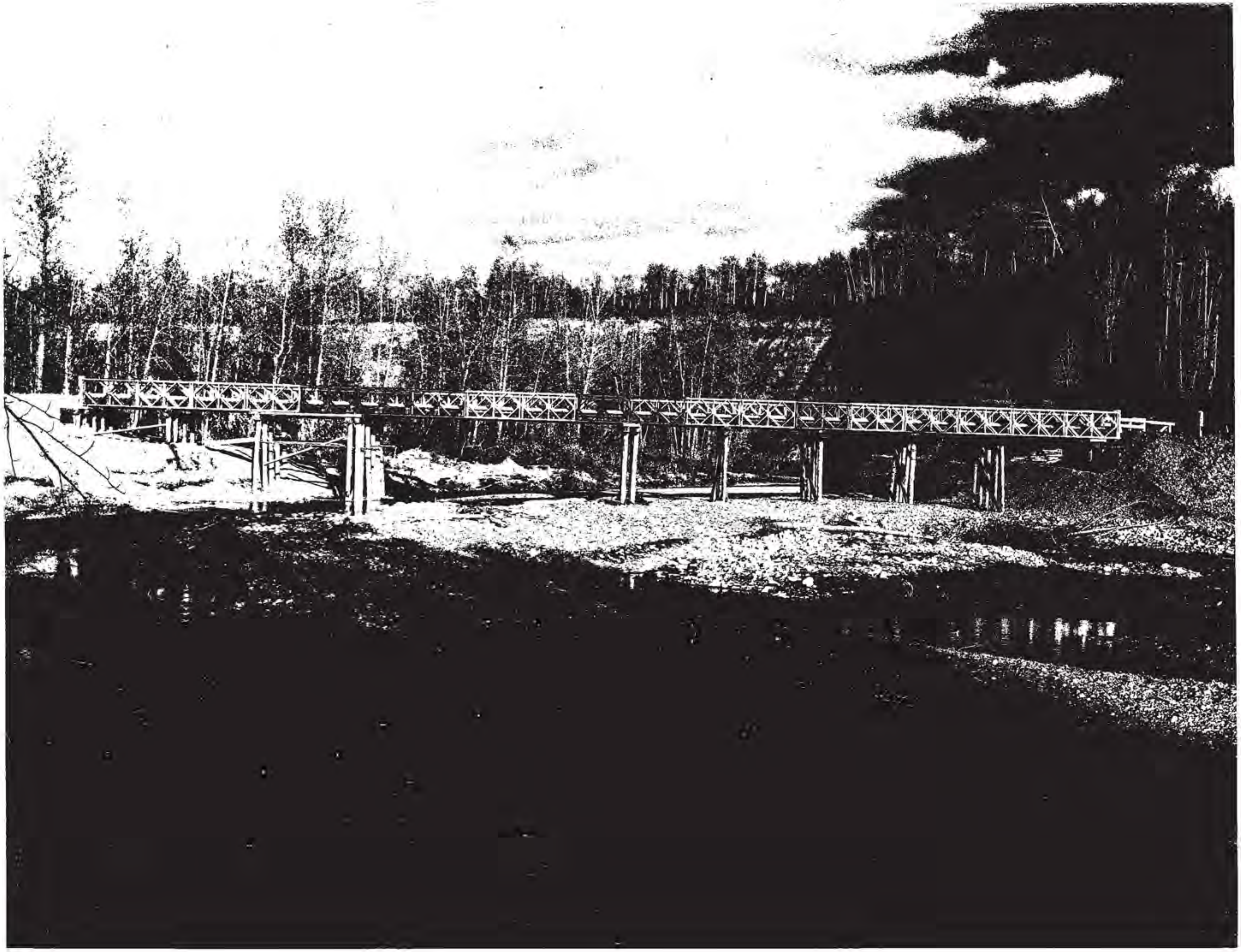
FORT ST JOHN DISTRICT



Farrell Creek Bridge #5

Sept / 1965.

proposed location of concrete piers and abutments for 2 - 30 ft & 2 - 50 feet steel beam spans shown in yellow).



PETER W. PARSONS
MOTION PICTURE & STILL PHOTOGRAPHY
125 E 26th ST., North Vancouver, B.C.
Phone 985-1468

No. A 102 B

Location

Taken

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Phone 985-[illegible]

No. A - 102 A

Location CREEK BR - EAST HUDSON HOPE

Taken 9/28/65

PLEASE QUOTE NUMBER WHEN REORDERING

H. 292 O

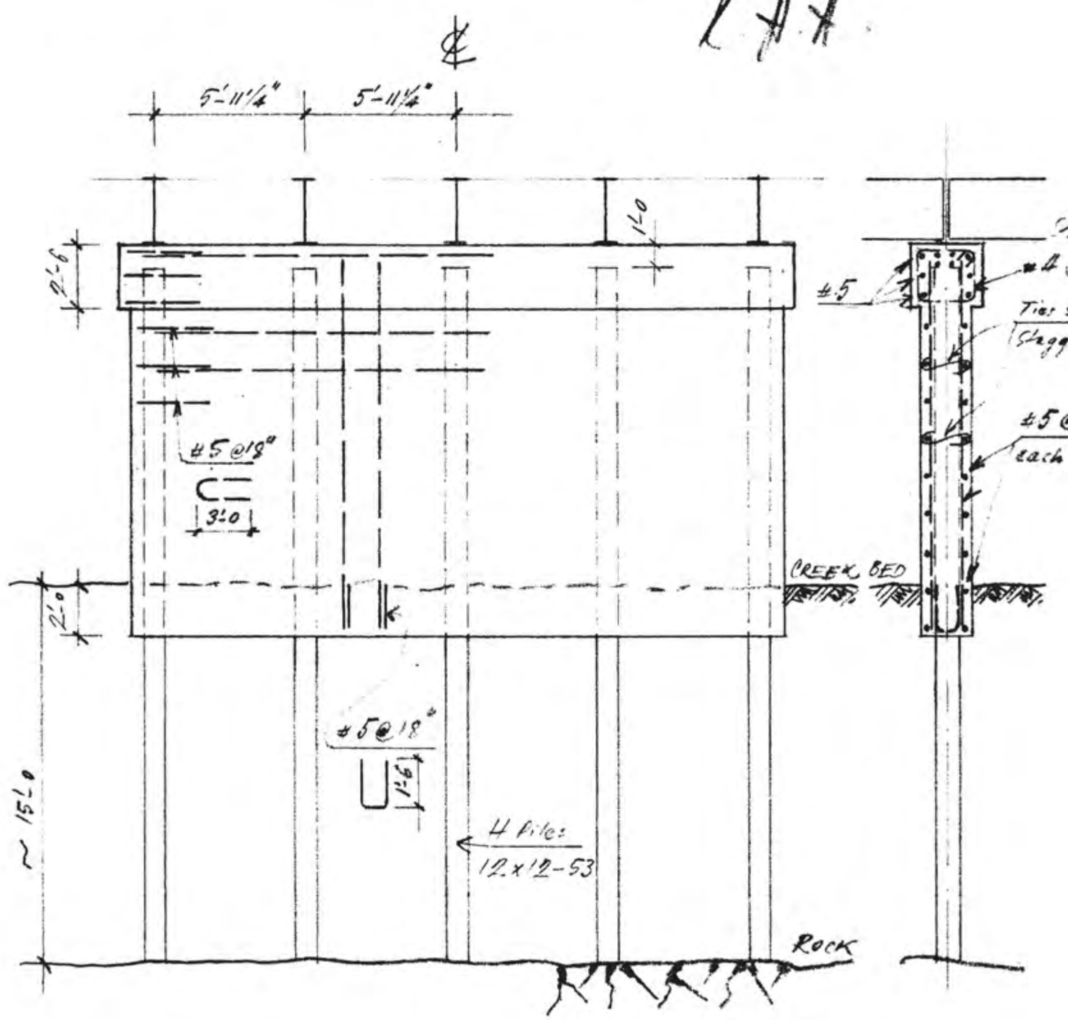
CALCULATIONS

Sheet No. SK 1

Name of work FARRELL CK, PIER

Made M. H. Date Nov 25, 65

Checked L.H.H. Date



H. 282 O

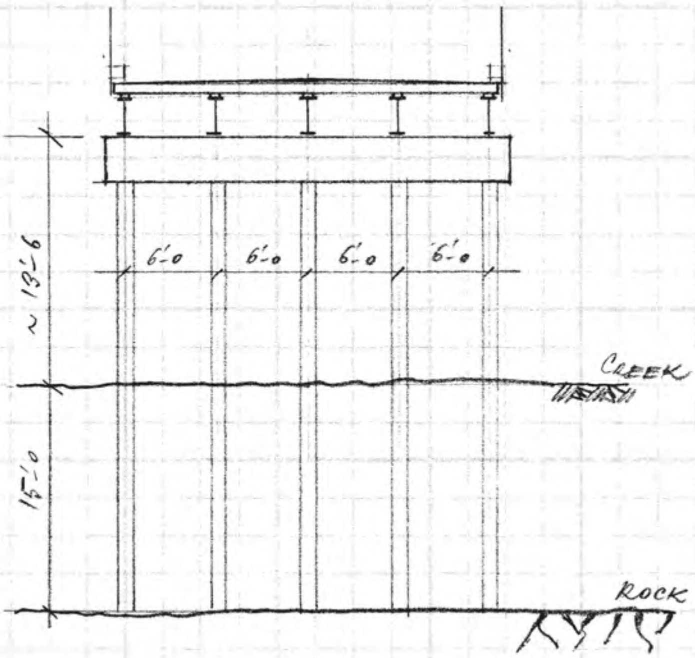
CALCULATIONS

Sheet No. 1

Name of work FARRELL CK. Made MH Date Nov 65

Checked Date

PIER



K.H.A. Nov. 24, 65: Use Steel H-Piles encased in concrete above ground level.

Loading:

D.L.

Deck: $\frac{6.0}{12} \times 25.0 \times 50.0 \times 0.050$ = 31.2 K

W.S.: $\frac{2.56}{12} \times 25.0 \times 50.0 \times 0.15$ = 40.0

Fence: $0.20 \times 2 \times 50.0$ = 20.0

Stringers & diaphr.: $0.18 \times 5 \times 50.0$ = 45.0

(136.2)

Pier cap: $3.0 \times 3.0 \times 26.0 \times 0.15$ = 35.1

171.3 K

Concrete casing $1.167 \times 12.0 \times 25.0 \times 0.15$ = 52.5 K

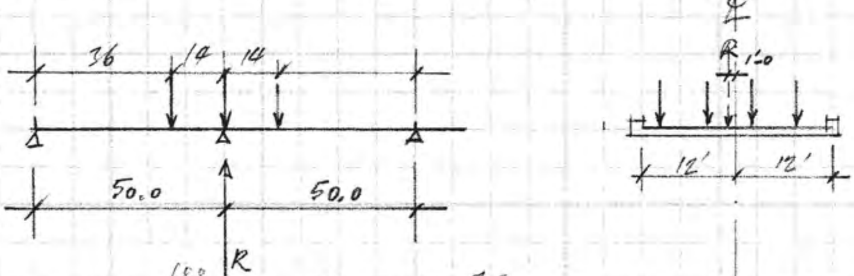
CALCULATIONS

Sheet No. 2

Name of work FARRELL CK. Made MH Date Nov. 65

Checked Date

L.L. Use H25 - S20



$R = \left[40.0\left(1+\frac{36.0}{50}\right)+10.0\times\frac{36}{50}\right]\times 2 = 152\ k$

Transverse moment due to lateral eccentricity:

$M = 152 \times 1.0 = 152$ ft-k

Force of stream flow:

$P = K \times v^2$ — assume $v = 20$ ft/sec.
Square ends

$P = 1.33 \times 20 = 26.6$ #/ft² ≈ 30 #/lin ft of pile

Ice pressure:

Wind

Area = $5.5 \times 50.0 \times 1.5 = 413$ ft²

a) 50# Wind

$P = 413 \times 0.05 = 20.7$ k

b) 15# Wind

$P = \frac{15}{50} \times 20.7 = 6.3$ k

c) $\frac{1}{2}$ Wind on L.L.: $P = 0.10 \times 50 = 5.0$ k

BR. F
Lynx & Farrell

Flood Report by

L.A. Broddy, Regional Highway Engineer, Prince George

11:00 A.M., June 5, 1967

Fraser down 0'.2 at Prince George last evening expect it to rise again today.

Isle Cache - quite a few have evacuated
- holding roads - raising them - 2 1/2' head on road
- built on garbage

H.P. slough in South Fort George - our road is not in city
- Department of Highways have put in pre-fabricated gate and controlled the water. There is a 3 1/2' differential. OK unless it rains which will fill slough side. Has a big drainage basin.

Quesnel - minor flooding
- anticipate flooding on west side south of one-lane bridge - arterial will be under water. Will raise road with gravel to carry traffic if necessary. In £964 1'-18" water over road.

Hudson Hope Road - Halfway Creek few inches from bottom of Bailey. Same at Lynx and Farrell.

cc: H.T. Miard, Deputy Minister.
cc: Sr. Maintenance Engineer
cc: Sr. Bridge Engineer

R.

RECEIVED
DEPT. OF HIGHWAYS
VICTORIA, B. C.

JUL 3- 1968

ANSW'D.
NOTED
DATE

SENIOR BRIDGE ENGINEER
VICTORIA B C

SUBJECT: FARREL AND LYNX CREEK BRIDGES

THE RECENT FLOODS HAVE EXPOSED THE FARREL CREEK ABUTMENT AND 5 FEET OF PILING. THERE IS ALSO EROSION ON LYNX ABUTMENT. IT WOULD APPEAR THAT THE BEDS OF THE STREAMS ARE GRADING DOWN AS THE PEACE RIVER FLOW HAS NOW BEEN CUT OFF AND THERE IS NOTHING TO HOLD BACK THE FLOOD WATER IN THESE CREEKS. PLEASE ADVISE WHAT REMEDIAL ACTION SHOULD BE TAKEN AND IF FUNDS WILL BE AVAILABLE. OTHERWOSE WE WILL HAVE TO REQUEST ADDITIONL STORM DAMAGE FUNDS FOR PEACE RIVER NORTH.

R4-44-40-0
R4-0-84
R4-21-30

L A BRODDYR
REGIONAL HIGHWAY ENGINEER
PRINCE GEORGE B C
JUNE 3/68
JULY 3/68
4:08 P M
MSGE RCD VIC

99988

~~LCJ~~ BF

LYNX CREEK

Jly 9/68

FARRELL CREEK

Jly 9/68

T.E.L.E.X.

1978-11-28

File: 2184

A.L. Freebairn
Regional Maintenance Operations Manager
Prince George

Re: Farrell Cr. Bridge #2184

B.C. Hydro personnel report severe scour at this bridge.

W.A. Bowman
Director of Bridge Engineering.

WAB/sh

HWY R4 PGEO

HWY HQ VIC

A L FREEBAIRN
REG MAINT OP MGR
PRINCE GEORGE

RE FARRELL CREEK BRIDGE 2184

B C HYDRO PERSONNEL REPORT SEVERE SCOUR AT THIS BRIDGE

W A BOWMAN
DIR OF BRIDGE ENGINEERING

VICTORIA
NOV 28/78

BRIDGE
NOV 28 1978
BRANCH

WPB

This is centre pier, and was fixed several years ago. Pier is founded on steel piles which extend to bedrock, and scour took place when the Peace was lowered by the dam. They excavated to bedrock and encased the piles in concrete, extending the pier to bedrock. Bridge register indicates this was done during winter of 1969

Vic


Love

s22


19/04/2012


19/04/2012


19/04/2012


AFTER
New Love?


Love


2012/01/24


2012/01/24


2012/01/24


2012/01/24


2012/12/18


2012/12/18


2012/12/18


2012/12/18


2012/12/18

Province of British Columbia

Ministry of Transportation

MEMORANDUM

By Fax: (250) 787-3279
(Original letter to follow)

Rick Blixrud
Project Manager, Recovery
North Peace District Office
FORT ST JOHN BC

Bridge Engineering
4D-940 Blanshard Street
PO BOX 9850 STN PROV GOVT
VICTORIA BC V8W 9T5
Fax: 387-7735
Phone: 387-7763
July 4, 2001

Re: North Peace Flood Damage – Assessment & Preliminary Hydraulic Design Constraints for Unnamed Creek Replacement Structure – Farrell Creek Road, Hwy #29, North Peace Highway District (File 14600-65-R4)

With reference to the June 28, 2001 flood damage assessment by Rick/Ljubomir/Mike/Gordon/Heather/Percy, please note the following comments and preliminary hydraulic design constraints for the replacement structure. The hydraulic design constraints will be refined on receipt of survey information on the site plan, gradient of creek, profile of existing water surface elevations and cross-sections of creek at 10 metre intervals, 50 metres upstream and downstream of the crossing. The cross-sections shall be plotted to the same horizontal and vertical scales.

A. Background Information

The 1.5 metre diameter culvert is a helical pipe and was installed under a fill of 10 metres. This culvert accommodates 3.5 m^3/s under inlet control. During the recent flood of June 12, 2001, the upstream section of the culvert was ponded to a depth of 8.5 metres above the invert level. The resulting hydrostatic pressure was of sufficient magnitude to blow out the outlet end of the conduit including couplers as well as unraveling the lock seams of the helical pipe over length of 10 metres.

The local maintenance crew has confirmed that the culvert had been cleaned of debris prior to the storm event and they believe that the large number of beaver dams on the upstream side may have exacerbated the flood scenario.

B. Geology

It originates from the Lower Cretaceous Period and belongs to the Fort St. John Group. It is classified as the Upper Shale and consists of gray shale and silty shale; minor amounts of sandstone and siltstone; and thin bands and scattered concretions of ironstone (marine). The Upper Shale ranges from 430 metres to 550 metres in thickness and weathers readily into clay.

. . . /2

C. Hydraulics

The drainage basin of Unnamed Creek covers an area of 15 km^2 and a gradient of 1.5% has been estimated during the site inspection. A Q_{100} design flow has been used on the assumption that the crossing is located on a low volume road.

1. Bridge Option

• Q_{100} design flow	12 m^3/s
• Minimum width of channel bed	3 m
• Depth of Q_{100} design flow	1.1 m
• Minimum clearance above Q_{100} design flow	1.0 m
• Average velocity during Q_{100} design flow	3.7 m/s
• Riprap size (0.7 metre thick and placed on slope of 1.5:1 over a layer and non-woven geotextile fabric)	100 kg Class
• Depth of local scour	1.0 m

2. Culvert Option

A 2.59 metre diameter Structural Plate Corrugated Steel Pipe is suitable for a Q_{100} design flow of 12 m^3/s under inlet control. Both ends of the culvert shall be anchored to reinforced concrete cut-off walls. 250-kg Class riprap shall be used for armouring both ends of the culvert.

D. Comments & Recommendations

s13

. . . /3

Please contact this office if you require additional information or clarification.

Percy A Thambirajah, P. Eng.
Senior Hydraulics Engineer

PAT/dh

cc: Bruce Mackay, District Highways Manager, Dawson Creek. Fax: (250) 784-2222
Miles Webster, P.Eng., Regional Manager Planning & Professional Services, Northern Region - By Fax (250) 565-6065
Ljubomir Stevanovic, P.Eng., Bridge Rehab Engineer, Northern Region - By Fax: (250) 565-6524
Mike Odowichuk, Area Manager Bridges, North Peace District Office, Fort St John
Bill Eisbrenner, P.Eng., Regional Geotech & Materials Engineer, Northern Region - By Fax (565) 565-6928
Gordon Hunter, P.Eng., Geotechnical Engineer, Northern Region, Prince George
Peter H. Brett, P.Eng., Chief Bridge Engineer, Engineering Branch, HQ
Turgut Ersoy, P.Eng., Manager Geotechnical Engineering, HQ


2005 09 29 15:16


2011/03/09


2005 08 26 10:58


2005 08 26 11:36


2005 08 26 15:04


5/16/02 14:35


5/16/02 14:38

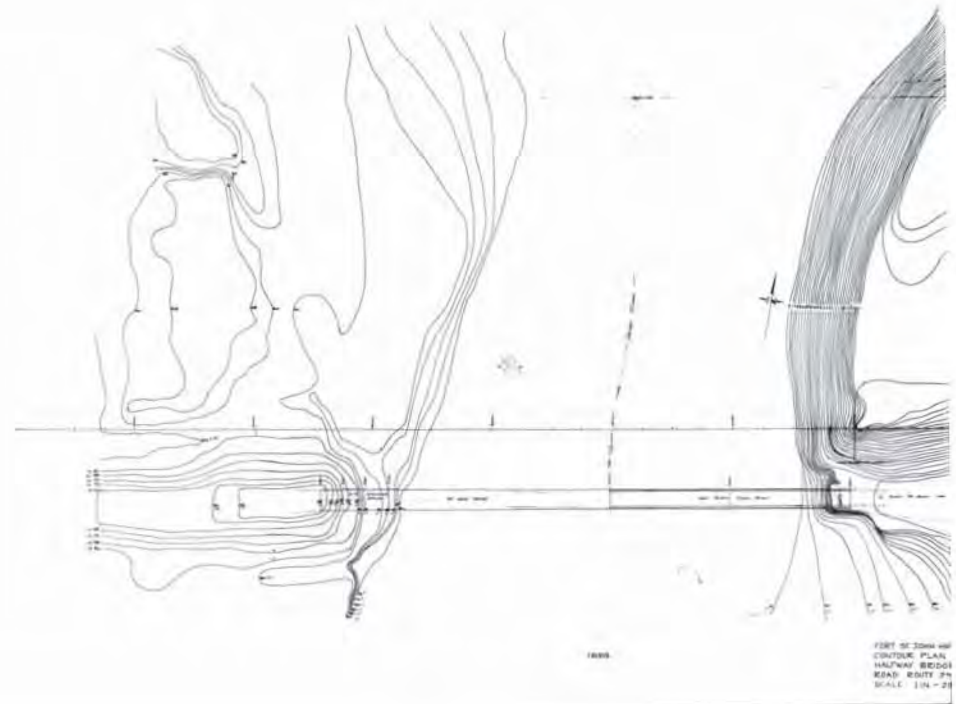

5/16/02 14:39

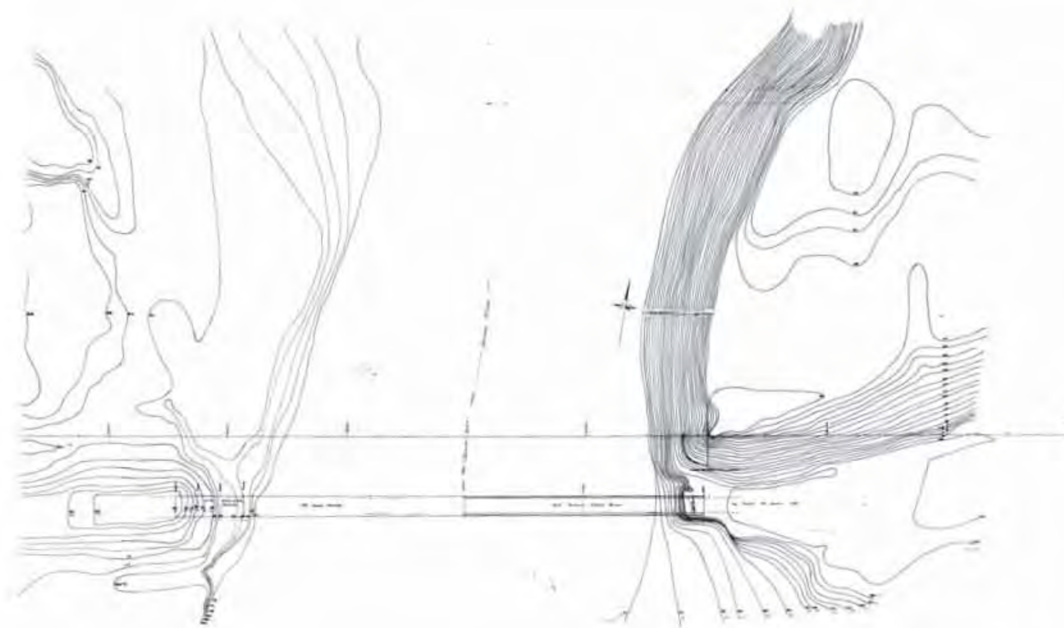

14/06/2010


14/06/2010

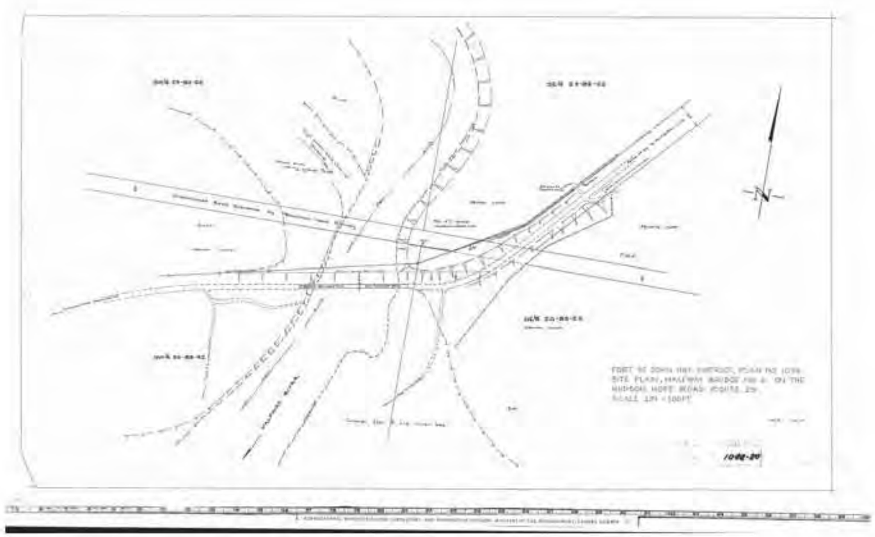

14/06/2010

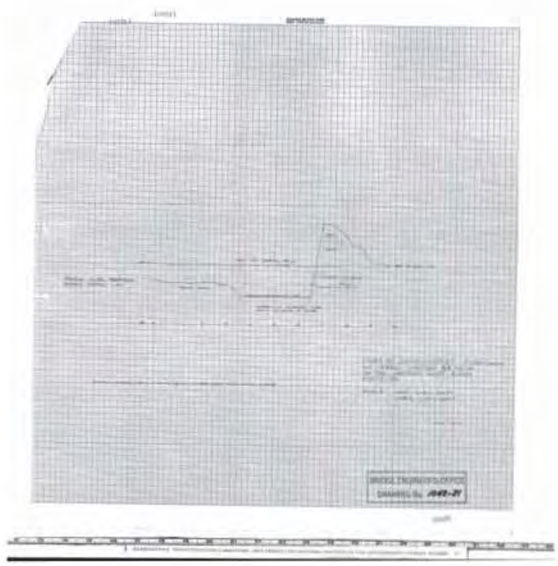

14/06/2010

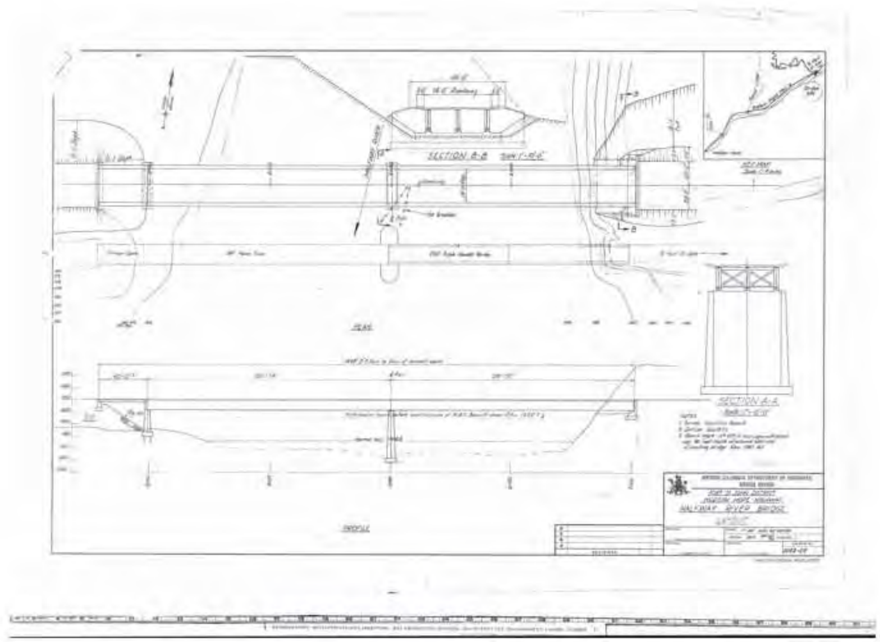

CONTOUR PLAN
HALFWAY BRIDG
ROAD ROUTE
SCALE 1 IN

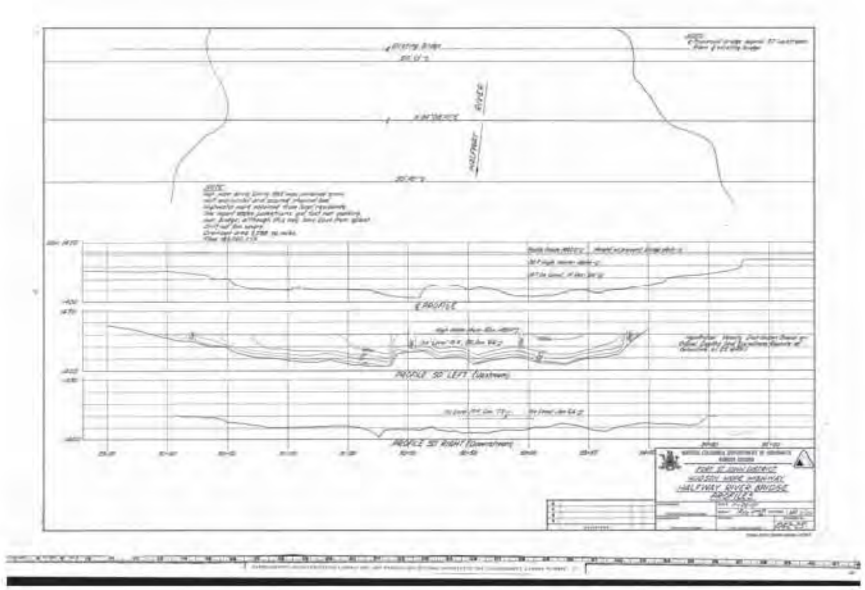


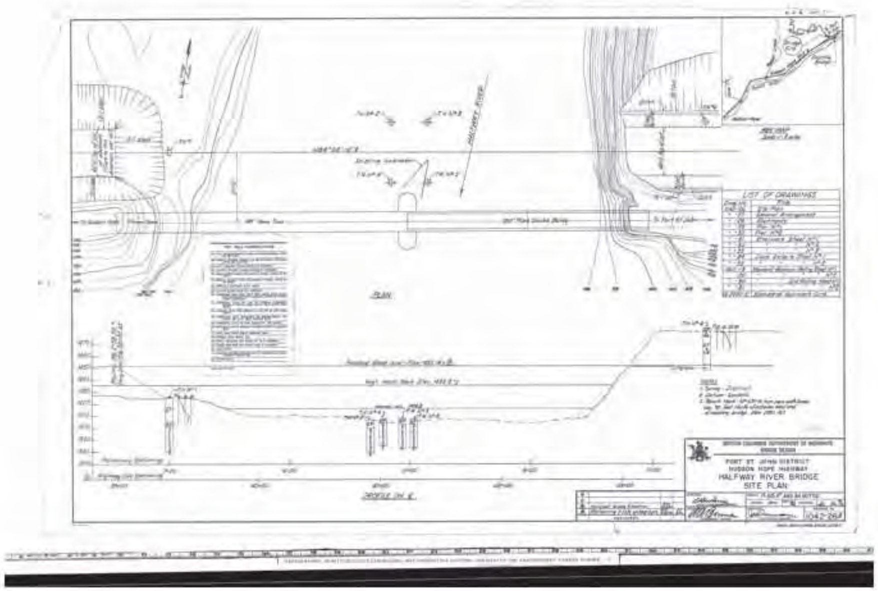
FORT ST JOHN HWY DISTRICT. PLAN NO [illegible]
CONTOUR PLAN OF AREA AROUND THE
HALFWAY BRIDGE NO.8, HUDSON HOPE
ROAD ROUTE 29
SCALE [illegible]



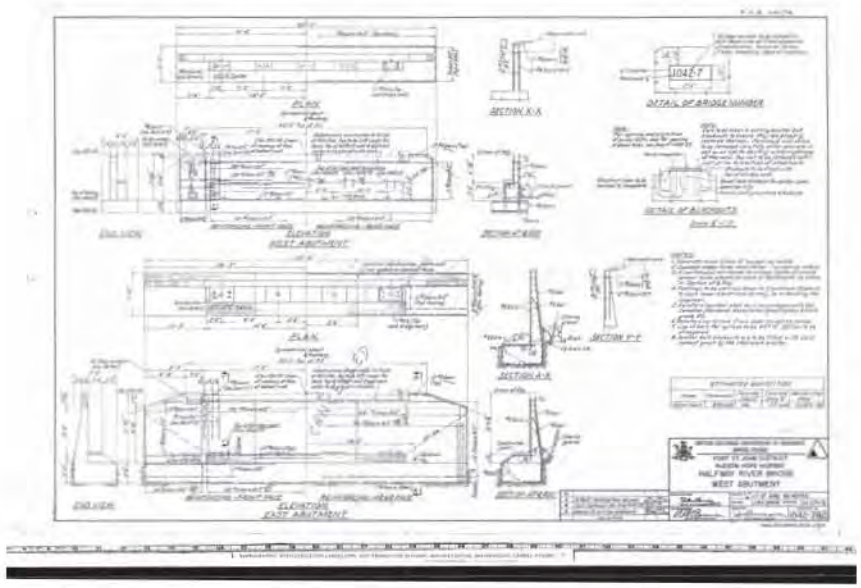


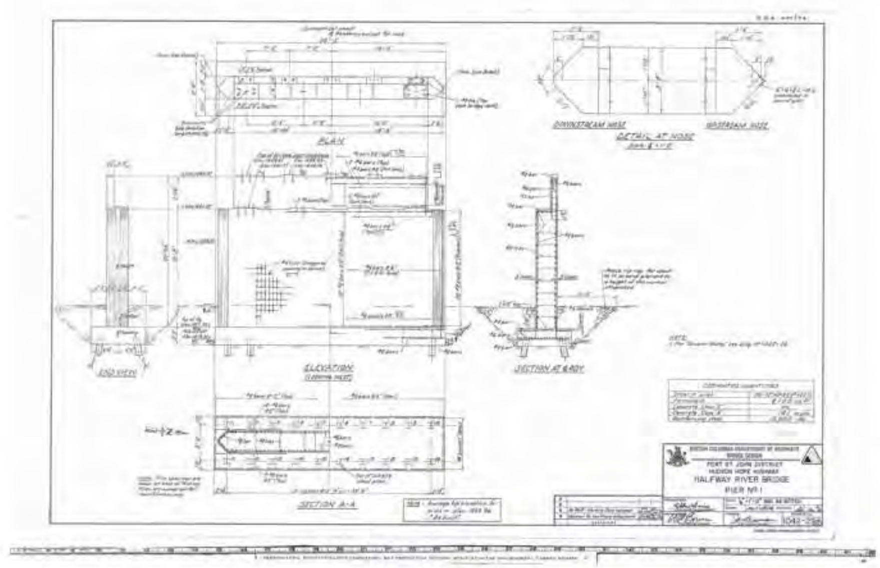

SECTION A-B
PLAN
SECTION A-A
PROFILE
FORT ST. JOHN DISTRICT
HUDSON HOPE HIGHWAY
HALFWAY RIVER BRIDGE

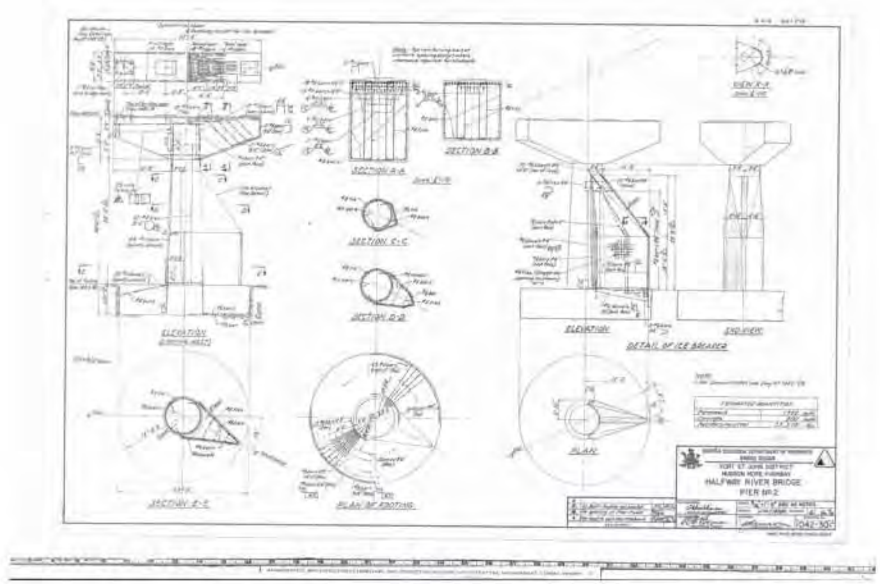

HALFWAY RIVER BRIDGE

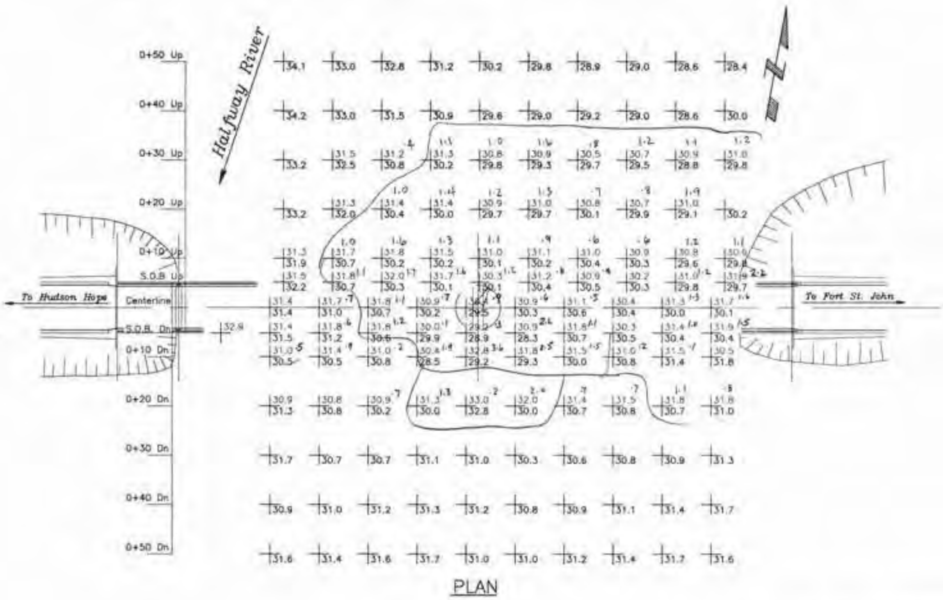


FORT ST. JOHN DISTRICT
HUDSON HOPE HIGHWAY
HALFWAY RIVER BRIDGE
SITE PLAN


HALFWAY RIVER BRIDGE
WEST ABUTMENT

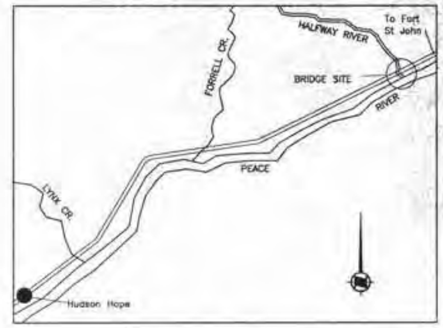

PLAN
DETAIL AT NOSE
ELEVATION
END VIEW
SECTION AT ⊄ PIER
SECTION A-A
FORT ST. JOHN DISTRICT
HUDSON HOPE HIGHWAY
HALFWAY RIVER BRIDGE
PIER Nº I


ELEVATION
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
PLAN OF FOOTING
VIEW X-X
ELEVATION
END VIEW
DETAIL OF ICE BREAKER
PLAN
FORT ST. JOHN DISTRICT
HUDSON HOPE HIGHWAY
HALFWAY RIVER BRIDGE
PIER No 2

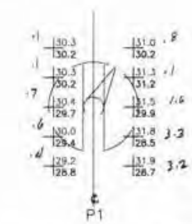


PLAN

GENERAL SCOUR - removal of infill indicated on last survey.



KEY PLAN



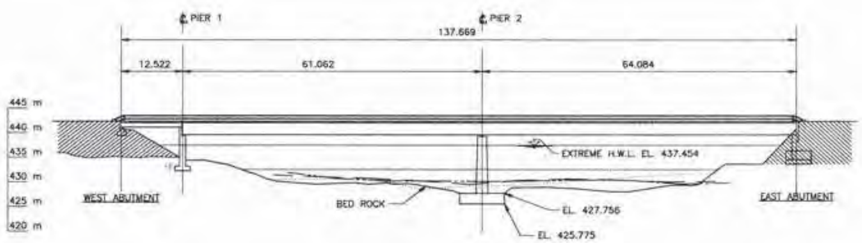
Lowest elev. 428.5
Top of Ftg 427.8
0.7 cover

SET IN BED ROCK

last survey was 1.4 cover

ADDITIONAL SOUNDINGS

SCALE 1:250



PROFILE

LEGEND

June 30/94
AUG. 02/96

PROFILES

10.0 m Up Stream
Centerline of Bridge
10.0 m Down Stream

DATE	WATER LEVEL ELEVATION	SURVEY BY
June 30/94	432.5 m	ALPHA
Aug. 02/96	435.0 m	ALPHA

NOTES:

1. ALL SOUNDINGS ARE IN METERS ABOVE 400.000 GEODETIC.
2. PROFILES ARE AT C/L BRIDGE AND AS NOTED.
3. ALL ELEVATIONS ARE IN METERS ABOVE 0.000 GEODETIC

17 OCT 96 received

Province of British Columbia
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH 983

PB 17 Oct 96

SCALE 1 : 500 AND AS NOTED

DESIGNED ____ DATE ____
CHECKED ____ DATE ____
DRAWN ____ DATE ____

Rev	Date	Description	Init

REVISIONS

REGION 4, DISTRICT 22
HUDSON HOPE HIGHWAY
HALFWAY RIVER BRIDGE
SOUNDING DRAWING

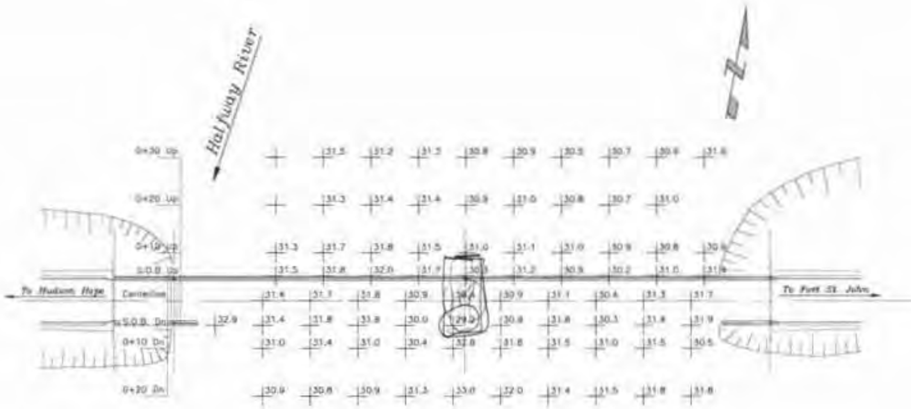
PREPARED BY | RECOMMENDED | ACCEPTED FOR CONSTRUCTION

BRIDGE REHAB./MAINT. ENGINEER | DIRECTOR OF BRIDGE ENGINEERING | CHIEF HIGHWAY ENGINEER

DATE | DATE | DATE

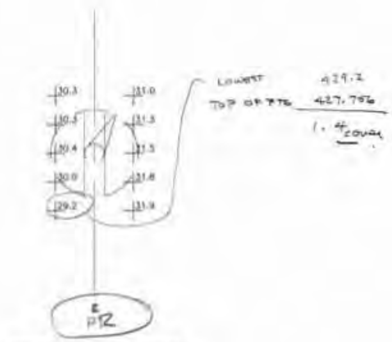
FILE No. | NEGATIVE No. 392009 | REEL | DRAWING No. 1042SD1

CANCEL PRINTS BEARING PREVIOUS LETTER



PLAN

KEY PLAN

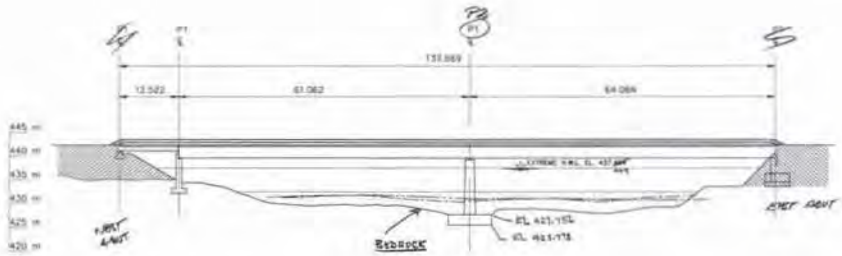


ADDITIONAL SOUNDINGS

SCALE 1:250

First Survey

Possible scour @ 3/4 and of P2



PROFILE

LEGEND

June 30/94

PROFILES

10.0 m Up Stream
Centerline of Bridge
10.0 m Down Stream

DATE	WATER LEVEL ELEVATION
June 30/94	432.5 m

NOTES:

1. ALL SOUNDINGS ARE IN METERS ABOVE 400.000 GEODETIC
2. PROFILES ARE AT C/L BRIDGE AND AS NOTED.
3. ALL ELEVATIONS ARE IN METERS ABOVE 0.000 GEODETIC

JUN 94-1

1042

SCALE 1:500 AND NOTED

DESIGNED ______ DATE ______
CHECKED ______ DATE ______
DRAWN ______ DATE ______

Rev	Date	Description	Init

REVISIONS

Province of British Columbia
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH 984

FORT ST. JOHN DISTRICT

HUDSON HOPE HIGHWAY

HALFWAY RIVER BRIDGE
SOUNDING PLAN

PREPARED BY | RECOMMENDED | ACCEPTED FOR CONSTRUCTION

DIRECTOR OF BRIDGE ENGINEERING | CHIEF HIGHWAY ENGINEER

DATE | DATE | DATE

FILE No. | NEGATIVE No. 392010 | REG. | DRAWING No. 1042SD1

CANCEL PRINTS BEARING PREVIOUS LETTER

HIGHWAY 29

ATTACHIE LANDSLIDE

HYDRAULIC MODEL

DRAFT

prepared by:

NORTHWEST HYDRAULIC CONSULTANTS LTD.

prepared for:

BRITISH COLUMBIA

MINISTRY OF TRANSPORTATION AND HIGHWAYS

April, 1984

1318-1423A

TABLE OF CONTENTS

		PAGE
1.0	INTRODUCTION	1
	1.1 General	1
	1.2 Background	2
	1.3 Study Objective	3
2.0	TEST CONDITIONS	5
	2.1 General	5
	2.2 Slide Parameters	5
	2.3 Surface Water Elevation	7
	2.4 Model Scales	10
3.0	TEST RESULTS AND DISCUSSION	11
	3.1 Flume Tests	11
	3.2 Main Model Tests	12
	3.2.1 Baseline Tests	13
	3.2.2 Phase I Tests	15
	3.2.3 Phase II Tests	19
	3.2.3.1 General	19
	3.2.3.2 Raised Highway	20
	3.2.3.3 Design Concepts	21
	3.2.3.4 Final Design	23
4.0	SUMMARY DISCUSSION OF FINAL DESIGN	26
5.0	MISCELLANEOUS CONSIDERATIONS	28
6.0	CONCLUSIONS	30
	REFERENCES	
	FIGURES	
	PHOTOGRAPHS	

1.0 INTRODUCTION

1.1 General

Under an agreement between Northwest Hydraulic Consultants Ltd. (NHC) and the British Columbia Ministry of Transportation and Highways (MOTH), a model study was conducted to aid in the design of works to protect the Highway 29 Halfway River bridge from inundation by a landslide-generated wave.

Authorization to proceed with the study was received on August 9, 1983 and the test program was initially completed by September 1983. The model was dismantled and the model topography placed in storage. The protective scheme developed during the initial phase of the testing was subsequently proved not feasible to construct for geotechnical reasons; therefore, a second phase of testing was initiated in February 1984. The model was reassembled and the second phase of testing was completed by March 1984.

During the course of the study, interim test results were transmitted to MOTH. This report combines the key findings of the model study for both phases of the test program.

1.2 Background

The Peace River emerges from the Rocky Mountains, where its headwaters are impounded in Williston Lake behind the Portage Mountain Development (PMD), and flows east through a 200 metre deep valley across northeastern British Columbia towards the Alberta border. The slopes of the Peace River Valley have a long history of natural slope movements. Debris from previous landslides is visible in many areas along the river.

Most of these slides have been small and slow; however, a rapid slide at Attachie in 1973, with a volume of 14 x 10^6 m^3, showed that large, rapid slides can occur under certain adverse conditions. Evidence visible after the 1973 landslide indicated that during the event trees located up to elevation 457 m, or 25 m above the river, were downed by the force of the water wave generated by the slide.

A potential landslide has been identified adjacent to the 1973 Attachie slide, directly opposite the confluence of the Halfway and Peace Rivers, where the Highway 29 Halfway River Bridge is located. The extent of wave runup during the 1973 Attachie landslide suggests that a similar wave generated from the potential landslide would inundate a section of the highway and the bridge with several metres of water. MOTH has decided that the highway and the bridge could best be protected by constructing an earthfill berm riverward of the highway, which would prevent the wave from reaching the highway and bridge.

1.3 Study Objective

The exact magnitude of the wave that would be generated during a rapid slide movement into the Peace River is unknown. Furthermore, the design of the most cost effective means of protection cannot be arrived at analytically. Therefore a hydraulic model test program was conducted to assess the potential wave heights, and to develop a design for the protective berm. The program involved rests with two separate models: a section model was tested in a flume, and then three-domensional tests were carried out in a tray.

A series of flume tests was conducted in which a two-dimensional representation of the 1973 landslide was simulated. These tests determined the magnitude of the 1973 slide displacement velocity, which was assumed to be representative for the potential landslide. In addition, the flume tests were used to develop a proper model mechanism for the generation of landslide waves. Verification of the slide displacement-time modelling procedure was accomplished by the duplication of the observed 1973 wave runup. During the second stage of the study, tests were conducted on the main three-dimensional model to develop alternative berm geometries to protect the highway and bridge.

During the study, the scope was expanded to encompass a brief hydrologic investigation to determine a design water level in the Peace River for the potential landslide event.

2.0 TEST CONDITIONS

2.1 General

The rapid displacement of water by a landslide results in the formation of a large wave which moves away from the immediate vicinity of the landslide. The magnitude of the landslide-generated wave is influenced by both slide parameters and terrain parameters. Of the various slide parameters, such as slide material composition, density, porosity, and slide shape, two major characteristics, the total slide volume of the slide displacing water and the velocity at which the slide travels through the water, dominate in the generation of waves. Of the various terrain parameters, such as the topographic relief and ground cover roughness, it is the depth of water which the slide displaces that governs the extent of wave generation. Thus the slide volume, slide velocity, and the water depth (i.e., water surface elevation) must be all correctly simulated in a model in order to properly represent prototype landslide-generated waves.

2.2 Slide Parameters

The total volume of the slide displacing water is a critical slide parameter in determining the generated wave height. The physical slide dimensions can be estimated from the extent of potential failure planes and observation of tension cracks, or in the case of an actual slide, the dimensions can be measured after the event.

For the purpose of this study, the mechanics of the potential landslide were assumed to be similar to those of the 1973 slide; that is, the slide material would move across the Peace River valley bottom in a thin layer, and stop when the topographic rise of the far valley wall was encountered. In both models (the flume tests of the 1973 slide and the main model tests of the potential slide) the landslide was simulated by a constant-thickness articulated block towed along the surface topography. Both model slides simulated only the valley-bottom movement; that is, there was no simulation of the initial slide movement on the benchland. The leading edge of the model slide was a 7.7 m* vertical face towed at right angles with respect to the direction of travel. The flume was constructed to represent a two-dimensional section of the 1973 slide with a slide path perpendicular to the Peace River. The potential slide in the main model was contructed with a full width of 550 m and with the direction of slide travel rotated slightly towards the Halfway River (about seven degrees east of perpendicular to the Peace River). This alignment corresponds to the general alignment of the south valley wall where the potential slide would originate.

* prototype units are used throughout the report

The slide displacement-time relationship (velocity) for either a potential slide or a historical slide is considerably more difficult to estimate than the slide dimensions. The velocity of the slide travel during the 1973 event is unknown. However, the magnitude of the resulting water wave is directly documented by evidence of tree knock down on the far valley wall. Therefore, flume tests of the 1973 slide were conducted to determine the slide velocity that would have resulted in comparable wave heights. This velocity was in turn assumed to be applicable to the potential landslide. The results of these tests are discussed in a later section.

2.3 Surface Water Elevation

The surface water elevation of the Peace River will be an important factor in regard to the magnitude of the landslide-generated wave not only because the height of the wave will be a function of the depth of water pushed by the slide, but also because the elevation of the crest of the wave will be a function of the initial water elevation in front of the wave. Thus potential runup of the wave, and hence the required elevation of the berm, is dependent on the water level at the time of slide as well as on slide characteristics.

The surface water elevation at a given site is dependent on both the magnitude of the river discharge and the stage-discharge relationship at the site. The level of risk associated with the design of a protective berm for a given surface water elevation must consider the combined probabilities of both the slide occurring and a given river discharge occurring simultaneously. It is necessary to assume a design water level higher than average since the chances of the slide occurring during a higher than average water level would be about 50 percent. On the other hand, designing for an extreme flood water level (say 1:100 years) would be very conservative since the combined probabilities of the slide occurring during such a flood event are remote. It was concluded that reasonable design water level would correspond to about the mean annual flood (the discharge exceeded about one day every two years); a conservative level would be the five year flood (the discharge exceeded about one day every 5 years).

River levels at the landside site are determined by the combined flows in the Halfway and Peace Rivers. The peak flood event on the Peace River is the result of melting snow, whereas the peak flood event on the Halfway River is the result of summer rain storms.

Normally, an estimate of future flood flows can be made on the basis of a statistical analysis of the historical streamflow records. However, the Peace River flows are currently regulated upstream by B.C. Hydro, so a standard flood frequency analysis based on the historical streamflow record would be invalid. An alternative method for estimating flood frequencies involves combining the anticipated Peace River flow-duration curves (based on the existing flood regulation and power operating procedures) with the unregulated historical flood frequency curves from the Halfway River. An analysis/ simulation can be used to develop a synthetic flood frequency relationship which combines the flow charateristics from the two rivers. This method of analysis was previously employed in a study [1*] for B.C. Hydro's proposed Site C Damsite (to be located 38 km downstream from Attachie). The flood frequency analysis for the Site C damsite is for the most part applicable to the Attachie landslide site. Based on that analysis, the mean annual flood is estimated as 2100 m^3/s, and the five year flood 3000 m^3/s.

Flood levels on the Peace River near the landslide site are difficult to determine due to lack of hydraulic and stage-discharge data. The data available for estimating the stage-discharge relationship are limited to the following:

i) The water elevation during field surveys in August 1983 averaged about 432.8 m, with a corresponding river discharge of approximately 1400 m^3/s .

ii) The maximum water elevation in the spring/summer of 1983 was reported to have overtopped the gravel bar

* Superscript numerals refer to reference listed at the end of the report.

located at the mouth of the Halfway River. This was reportedly the highest water level in "several" years. The river stage at the site peaked on two separate occasions in 1983, first on July 8 (3373 m^3/s; comprised of 2820 m^3/s on the Peace River and 553 m^3/s on the Halfway River) and again on September 8 (instantaneous peak discharge of 3350 m^3/s, daily mean of 2050 m^3/s).

iii) A preliminary stage-discharge relationship determined for B.C. Hydro for a section located about 2 km downstream from the Attachie site was available.[2]

After review and analysis of this limited data, it was estimated that the water level elevations corresponding to mean annual and five-year return period floods would be about 432.4 m and 433.4 m, respectively. These two water levels were adopted for the subsequent model testing.

2.4 Model Scales

The models were constructed to an undistorted scale of 1:200 and operated in accordance with Froude Criterion for dynamic similarity. The use of the Froude Criterion led to the following model/prototype scaling ratios:

Length Ratio	1:200
Area Ratio	1:40,000
Velocity Ratio	1:14.1
Time Ratio	1:14.1

3.0 TEST RESULTS AND DISCUSSION

3.1 Flume Tests

The flume model was constructed and tested to simulate conditions of the 1973 Attachie Slide, for which it was known that the landslide-generated wave reached about elevation 457 m on the opposite bank. The flume topography was constructed to represent a 160 m section of the pre-1973 Peace River valley bottom. As pre-1973 riverbed survey data do not exist for the site, and the topography had to be estimated based on post-slide data and pre-1973 air photographs.

The slide displacement-time relationship was determined during calibration tests in the flume model. The Peace River water level just prior to the 1973 slide was estimated to have been about 432.0 m. The flume model tests showed that model slide velocities of 10 m/s and 15 m/s resulted in wave runups to about elevations 455 m and 460 m, respectively. The tests indicated that the wave runup generated by the slide was not sensitive to variations in the initial velocity or the terminal velocity, and a constant slide velocity was used in all subsequent tests.

The aforementioned test results were determined in the absence of any roughness to represent bank and island vegetation; other sensitivity tests showed that such roughness might decrease the maximum wave runup by as much as five metres. However, subsequent to the sensitivity tests, all tests in the main three dimensional model were performed without simulation of vegetation roughness as the future existence of such vegetation could not be guaranteed.

3.2 Main Model Tests

The main model was constructed to represent a 1600 m reach of the Peace River extending 1200 m across the river and encompassing the mouth of the Halfway River (Figure 1). The topography was constructed in accordance with the 1983 field survey conducted by MOTH.

The flume tests had indicated that a constant slide velocity of between 10 and 15 m/s would have resulted in the wave heights reported with the 1973 slide event. For the purpose of the main model tests the following test characteristics were used: the model slide was constructed to simulate a slide 7.7 m thick by 550 m wide; the slide displacement-time curve consisted of a uniform velocity of 15 m/s; rapid acceleration and short deceleration periods were imposed, with the toe of the slide coming to rest at the north bank of the Peace River; tests were conducted with the Peace River stage at two water levels (elevations 432.4 m and 433.4 m.

Assessment of performance of berm alternatives was based on whether or not the highway was overtopped and on whether or not the Halfway River bridge was threatened. Visual observations made during tests were augmented in some tests by photographs and wave crest indicators which recorded maximum wave heights. The extent the highway overtopping was measured by referencing distances to the west abutment of the Halfway River bridge. A video record of the Phase II tests was used to examine the details of the wave behavior and interaction with the berm.

3.2.1 Baseline Tests

A series of initial baseline tests was performed to assess the height of wave runup and the extent of highway overtopping which would occur in the absence of a protective berm. The wave generated by a slide (15 m/s) at the five year water level (433.4 m) resulted in an inundation of approximately 750 m of highway to the west of the bridge, and 30 m of overtopping along the west end of the bridge deck. The height of the initial wave just before it encountered the highway was determined to be approximately 6 m; this height was consistent with the wave height over the highway and was found to be more or less constant along the full width of the slide path. The general height of wave overtopping was insensitive to variations in either the highway elevation or riverbank ground elevation. However, variations in the wave height overtopping of the highway, at specific points between consecutive tests ranged from a low of 4 m to a high of 8 m.

The height of the wave over the highway was also insensitive to the terminal position of the slide. Separate trials were conducted in which the terminal position of the slide was varied from a short slide stopping at the river's edge to a long slide stopping at the toe of the highway embankment. During these tests it was observed that velocity of the wave was faster than the velocity of the slide, and hence the initial wave left the slide front before the wave encountered the river bank. Therefore the wave no longer "felt" the slide front and as a result height of the wave overtopping the highway was independent of the terminal position of the slide.

The overtopping of the bridge deck was caused by a wave which formed from the initial wave reflecting off the highway embankment and which travelled eastward along the highway embankment, rather than by a wave which approached directly from the face of the slide. The crest of the wave which approached the bridge directly from the slide was about 2 m below the bridge deck.

A series of tests wsd conducted with the river stage lowered to the mean annual level, 432.4 m. The wave heightd generated with the mean annual water level imposed on the model were generally about 2 m lower than the wave heights generated with the five year water levels. The lateral extent of the highway inundation was similar to the inundation observed with the higher water level, but no overtopping of the bridge deck was observed with the lower water level. Photos 1 through 7 are a sequence showing the wave runup for the 15 m/s slide at the mean annual flood water level of 432.4 m.

3.2.2 Phase I Tests

At the outset of testing of the main model it was decided (on the basis of required fill volumes) that a desirable berm arrangement would consist of two relatively low berms rather than a single higher berm. The alignment of berms was chosen so the first, or riverward, berm ran roughly parallel to the north bank of the Peace River. The second berm, located roughly halfway between the first berm and the highway, ran parallel to the first. A number of informal tests were conducted to refine and optimize the lengths and heights of the berms.

Most configurations tested in the model prior to development of the final Phase I geometry resulted in inadequate highway protection, and are not discussed herein. One exception to this is an alternative in which a deep borrow trench was excavated between the berms; the bottom elevation of the trench was set at 425 m, which was over seven metres below the tested low water level. The wave energy dissipation in the water-filled trench was found to be beneficial to highway protection; however, the alternative was dropped when it was realized that excavation of a trench to this depth would not be practical. In subsequent berm geometries a shallower trench was tested.

The benefit of the two-berm-plus-trench design was derived partially from the volume of water initially ponded in the trench and partially from the volume subsequently 'trapped' between the berms. Both components were important in effecting energy dissipation: a substantial volume of ponded water to absorb and dissipate energy, and a second berm to reflect and trap a sizeable portion of the wave.

Photo 8 shows the Phase I berm geometry. The top elevation of the first (riverward) berm was 446.0 m; the top elevation of the second berm varied from 446.0 m to 444.0 m, as shown in

Figure 2. The trench between the two dikes had a bed elevation of 430.0 m, which was 2.4 m below the tested low water level. The approximate above-ground heights of the first and second berms were 11 m and 9 m, respectively; the lengths of the two berms were about 750 m and 600 m, respectively. The total material volume required for both berms was about 265,000 m^3.

The berm geometry provided sufficient highway protection at the tested low water level (432.4 m) and marginal protection at the tested high water level (433.4 m). Photo 9 shows the maximum extent of wave runup for the tested low water level; Photos 10 through 15 show the progression of the wave for the tested high water level.

It is believed that the berm design would provide sufficient protection at the higher water level if the following supplementary measures were taken:

i) The trench between the two berms should be constructed as deep as field conditions will permit. If the trench is deeper than the tested trench (invert elevation 430.0 m) the additional wave energy dissipation will retard the advance of the wave.

ii) The terrain between the second berm and the highway should be made as rough as possible. Either bulldozing discrete mounds of material at least 3 m high or grading irregular trenches would aid in the dissipation of wave energy.

The tested berm geometry was developed for an assumed slide path alignment corresponding to the general alignment of the south valley wall where the potential slide would originate. To provide protection for possible alternative slide path alignment, extensions to the ends of tested berms would be required as follows.

berm extension west

The west end of the berm should be extended, as shown in Figure 2, to provide additional wave protection if the assumed slide path direction is rotated to the west of the tested direction. The west end of the berm should be constructed so as to tie the crest elevation of the berm to the existing 446.0 m contour. Westward of this point, the highway grade rises sufficiently to prevent overtopping by a wave.

berm extension

If the assumed slide path is rotated further to the east, an additional length of berm would be required on the east end of the riverward berm. The maximum extent of the berm would intersect a line defined between the east end of the bridge and the east end of the potential slide area. The projection of this line is shown in Figure 1. The maximum eastern extension would require a dog-legged alignment, as shown in Figure 2, to avoid the complete blockage of the Halfway River, and a channel through the gravel bar at the Halfway River mouth to allow the Halfway River flows to exit toward the Peace River. Protection from erosion by the flood velocities in the Halfway River would be required on the eastern end of the first berm. The limits of such riprap is shown in Figure 2.

3.2.3 Phase II Tests

3.2.3.1 General

The motivation behind the Phase II testing was twofold. The berm design developed during the Phase I tests was subsequently proved to be geotechnically unsound and the costs of re-channelization of the Halfway River expensive. The Phase II testing was devised to explore several alternative designs for protection of the highway and bridge.

The model had been dismantled after the completion of the Phase I tests and therefore required re-assembling and model verification testing before proceeding to the final Phase II tests. The model verification was accomplished in two stages.

First, the model was configured to represent the baseline conditions (i.e. no protective berms) and the height of the wave overtopping the unprotected highway was documented. The berm design developed during the Phase I tests was subsequently installed on the model and the protection provided by the berms was documented. The similarity in the baseline conditions and in the performance of the protection works with the performance observed in the earlier Phase I tests demonstrated that the rebuilt model was adequately verified.

3.2.3.2 Raised Highway

The observation during the Phase I tests, that the wave which overtopped the bridge deck was not a wave approaching directly from the slide, but rather, a wave which formed from the initial wave reflecting off the highway embankment and which travelled eastward along the embankment towards the bridge, led to the idea of an alternative wave protection scheme. The scheme consisted of raising the highway to prevent overtopping and constructing a berm perpendicular to the highway to protect the bridge from the reflected wave traveling along the embankment.

Tests were conducted to establish the highway grade required to prevent wave overtopping by the 15 m/s slide at the five year water level 433.2 m. The tests indicated that in the absence of any additional protective measures riverward of the highway, the highway grade would have to be raised to an elevation of 461 m to prevent overtopping. This corresponded to a wave runup along the highway embankment exceeding 3 times the height of the approaching wave.

Additional tests were conducted in which one or more low berms (4 m or less) were installed riverward of the highway to reduce the magnitude of the required height of the highway grade. However, it was determined that a significant lowering of the highway grade could be realized only if a sizeable portion of the volume of the approaching wave was trapped between berms before reaching the vicinity of the highway.

3.2.3.3 Design Concepts

During the course of the test program several design concepts contributing to the reduction in the magnitude of the wave reaching the highway were developed. Among these, the existence of a water-filled trench between two berms was one of the more effective measures. A comparison of Photo sequence 16 to 18 (trench at elevation 430 m) and the Photo sequence 19 to 22 (no trench) illustrates the impact of a trench. The effectiveness of the energy dissipation in the water-filled trench is clearly demonstrated by the reduction of the volume of water overtopping the second berm (Photo 18 vs Photo 22). However, if the invert elevation of the trench was higher than the river level, and hence the trench was dry on the day of the slide, the trench would lose the energy dissipation effect resulting from the transfer of knematic energy from the wave to the still water. Without water in the bottom of the trench, the trench provides only a marginal increase in the relative height of the second berm.

With berms of practicable heights combined with a trench of practicable depth, a substantial portion of the wave continued on to attach the highway. Testing indicated that either the highway embankment would still have to be raised in excess of 5 m, or a third and perhaps fourth berm would be required.

"attack"?

Testing also indicated that a third berm with a vertical face resulted in significantly more energy dissipation than a sloping faced berm of similar height: the vertical face was apparently more effective in destroying the forward progression of the high velocity flow which resulted when the wave overtopped the second berm. However, the effectiveness of the vertical face diminished if large depth of flow escaped over the berm. Once the leading edge of the wave had impacted the berm and the depth of flow exceeded the berm height, the vertical face and sloping face berms performed similarly.

3.2.3.4 Final Design

Upon the development of the aforementioned general design concepts, MOTH provided the following construction constraints before the final design was established:

- the maximum berm elevation could not exceed 445 m
- the berm could be constructed at a 1.5 H to 1 V side slope
- the minimum trench invert could not fall below 433 m
- the trench could be excavated at a 2 H to 1 V side slope

In addition it was determined that the use of 4 m high vertical face berms were feasible.

Based on these criteria, the final design developed using the model consisted of two large berms at elevation 445 m on either side of a trench at elevation 433 m. A second trench was excavated to elevation 433 m behind the second berm, followed by two 4 m vertical face berms. In the vicinity of the west bridge abutment, a third vertical face berm was constructed to prevent overtopping of the bridge deck. Details of the berm geometry are given in Figure 3.

Photo sequence 23 to 29 show the progression of the wave generated by the 15 m/s slide at the five year flood (water surface elevation 433.4 m) past a configuration similar to the final design. (The eastern extent of the first berm in the final design is discussed below). The design prevented the wave from overtopping the highway and provided sufficient protection at the west end of the bridge for the five year event.

During the course of the study it was decided (in conjunction with MOTH) that the slide path alignment assumed for the model tests was realistic and that the ℄ of the potential slide would not ~~hinge and rotate~~ was not likely to be oriented further eastward during failure. The berm design shown in the Photo Sequence 23 to 29 was constructed entirely on the riverbank, eliminating the need for any in-river work. However, with this design the east abutment of the bridge was overtopped by the wave approaching directly from the slide as shown in Photo 28.

To prevent the overtopping of the east abutment, the eastern end of the first (riverward) berm was extended into the Halfway River approximately 110 m. The berm extension intercepted and reflected a portion of the wave travelling directly toward the bridge and prevented overtopping of the east abutment. Photo Sequence 30 to 32 and 33 to 36 show a comparison of the magnitude of the wave approaching the bridge with and without the eastern extension of the first berm into the Halfway River. The exposed end of the first berm extension should be protected from erosion caused by the flood velocities in the Halfway River.

4.0 SUMMARY DISCUSSION OF FINAL DESIGN

The wave protection scheme relies on the following design concepts:

1) The two riverward berms must trap a sufficient portion of the wave volume so the supplemental design measures can stop the remaining runup.
2) The trench between the two riverward berms is an effective energy dissipator if the depth of the trench invert permits the trench to fill with water. A shallow trench would provide only a marginal increase in the relative height of the second berm. (The energy dissipation effect of the trench is demonstrated by the design differences between the Phase I and the final Phase II layouts. Although the two riverward berms are similar in height, without the deep trench, two additional 4 m vertical-faced berms are required with the Phase II design to stop the wave overtopping the highway).
3) The vertical face berms are effective in destroying the wave energy and stopping the forward progression of the wave provided the depth of flow is relatively small (less than the berm height).

4) The first berm must extend into the Halfway River 110 m in order to prevent the overtopping of the west end of the bridge deck for the assumed slide path alignment. The berm extension should be protected from erosion by the Halfway River. However, the extension should not require re-channelization of the Halfway River.

5.0 MISCELLANEOUS CONSIDERATIONS

The final design was developed to provide protection against a wave generated by a 15 m/s slide during the five year flood water levels in the Peace River. The risk or probability of this design event is comprised of several factors:

- The probability associated with a given water level on a particular day when the slide might occur.
- The uncertainty of the slide velocity. The flume test results indicated the slide velocity of the 1973 event could have been in the order of 10 to 15 m/s; the main tests were conducted at the higher end of the velocity range (with a corresponding lower probability of occurrence.
- The probability of the slide occurring during the project life. Although the date for the highway relocation required is B.C. Hydro's proposed site C project is uncertain, the remaining life of the bridge may dictate a project life.

The combined probability of the slide occurring with a velocity of 15 m/s simultaneously during the occurrence of a five year flood event is rare and may be excessively conservative.

It may be possible to adopt a less severe design event for the protection of the highway and bridge. Of the various parameters that affect the magnitude of the wave approaching the highway and bridge, the water level is the only one which can be reasonably monitored. Therefore, a possible protection scheme adopting a lower design event could be developed with the use of automatic water level monitoring equipment. At the onset of a high water level event, the automatic equipment would alert the authorities to establish a manned station for the duration of high water thus allowing the road to be monitored and/or closed for the duration of the flood event.

An alternative would be to accept one level of risk for the bridge and a higher level of risk for the highway. The overtopping of the highway is a short term event, whereas, the destruction of the bridge is lasting. The chance of traffic travelling the road during the short period in which the highway is overtopped is considerably lower than the chance of traffic encountering the failed bridge.

6.0 CONCLUSIONS

1) The flume test results indicate the slide velocity of the 1973 event could have been in the order of 10 to 15 m/s.

2) The adopted design slide consisted of the following parameters:
   - In-river dimensions of 550 m wide by 7.7 m thick
   - Slide path alignment perpendicular to the valley wall where the slide would originate
   - Constant slide velocity of 15 m/s
   - Terminal position of the leading edge of the slide at the north edge of the Peace River.

3) The hydrology analysis indicated the water levels at the site for the mean annual flood and the five year flood would be 432.4 m and 433.4 m, respectively.

4) In the absence of other protective works, the highway would have to be raised by 20 m to prevent overtopping. In addition, works would be required to protect both ends of the bridge.

5) The Phase I design was hydraulically acceptable, but not feasible due to geotechnical constraints.

6) The design shown on Figure 3 will protect the highway and the bridge at the five year flood level.

7) The riverward berms must be high enough to trap a sizeable volume of the wave.

8) A shallow or empty trench is not effective in dissipating the wave energy.

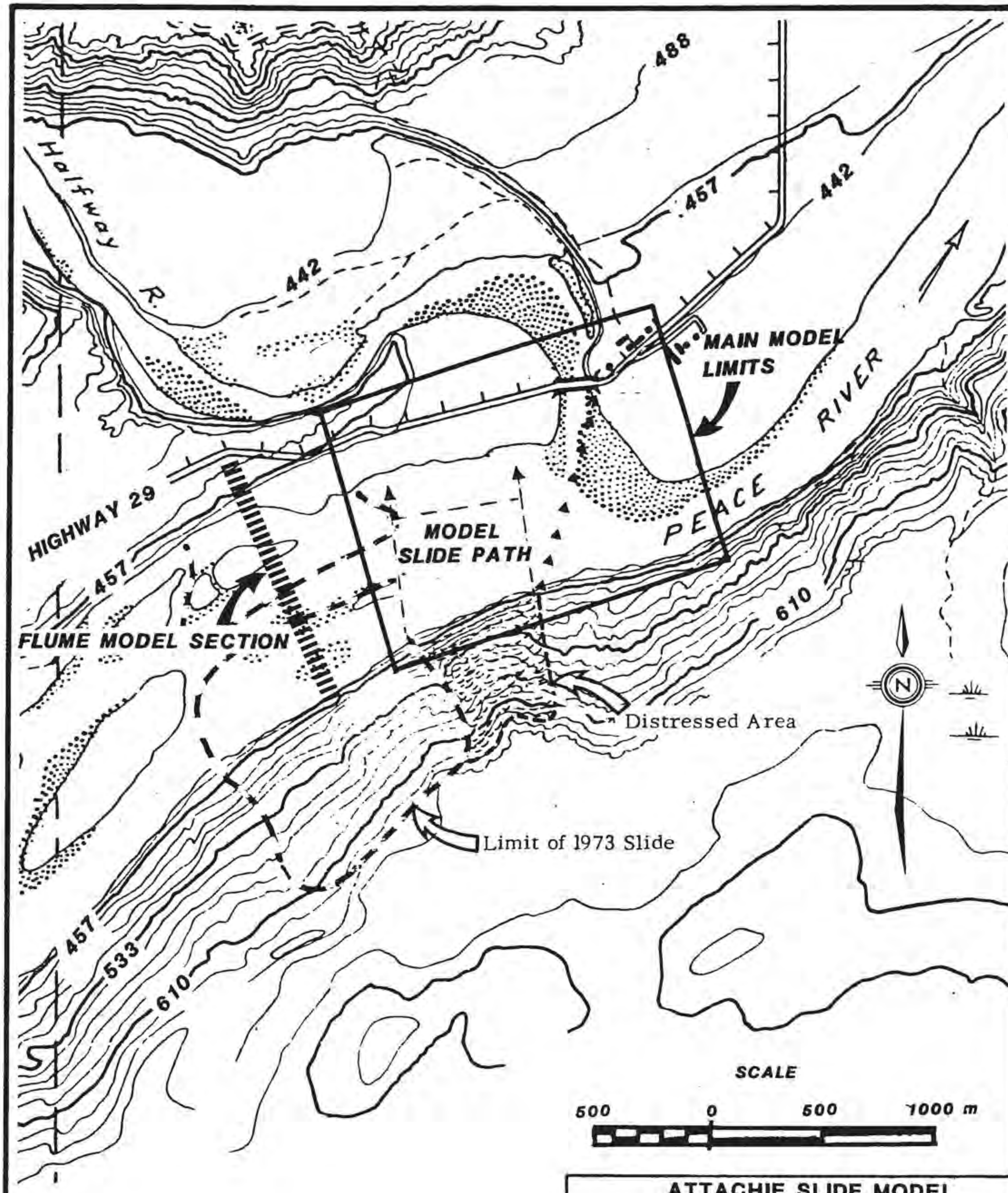
9) Vertical berms are effective only if the first two berms are high enough to limit the depth of the wave approaching the vertical berms.

10) The first berm should extend 110 m into the Halfway River to prevent overtopping of the east abutment of the bridge. This should not require re-channelization of the Halfway River.

11) The in-river portion of the berm should be protected from erosion attach by the Halfway River flow velocities.

REFERENCES

(1) Northwest Hydraulic Consultants Ltd., "Interim Report - Peace River Site C Project River Control and Flood Studies / Diversion Design Flood", April, 1979.



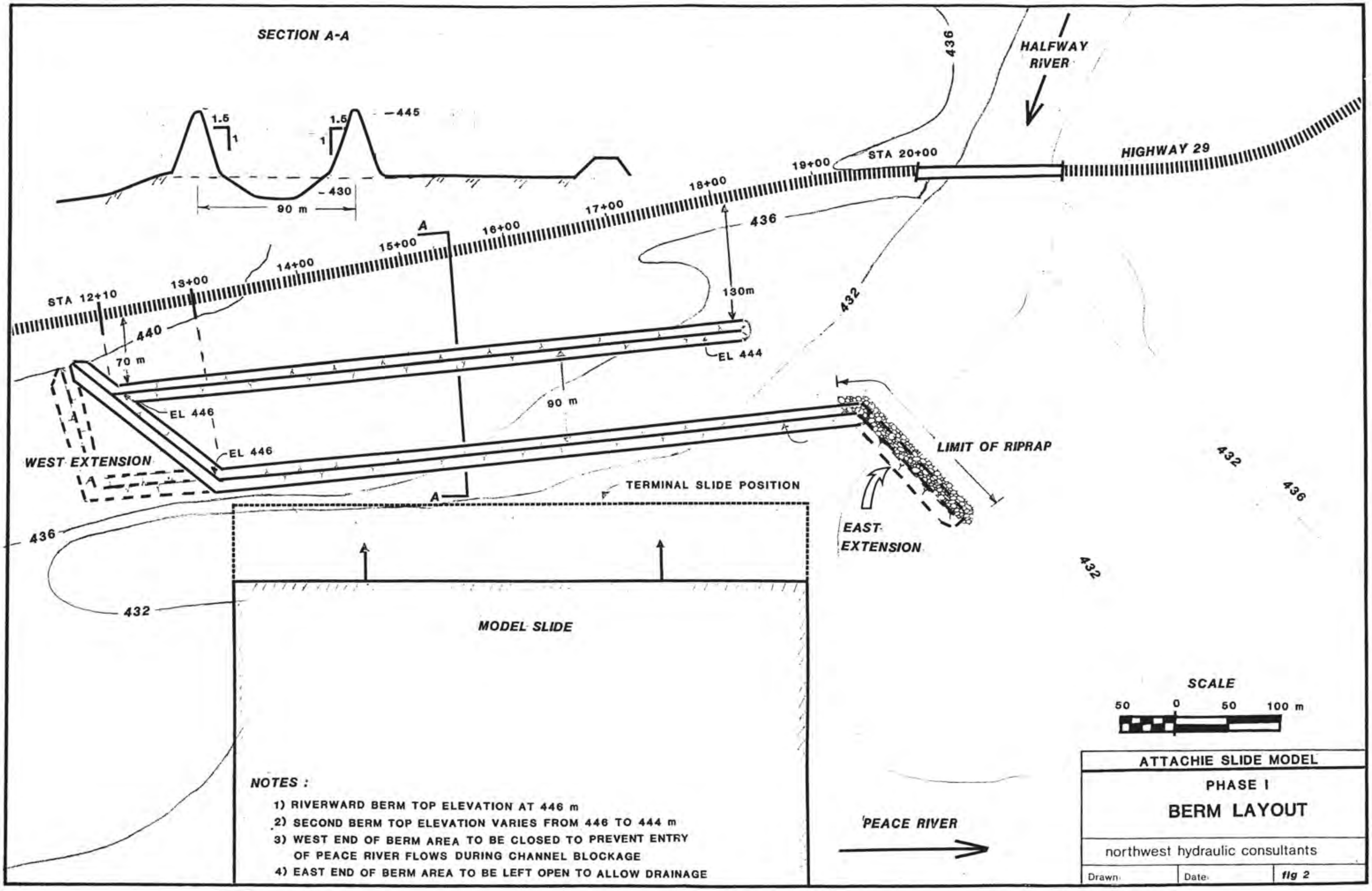
ATTACHIE SLIDE MODEL

LOCATION MAP

northwest hydraulic consultants

Drawn: | Date: | fig 1

Reference: National Topographic Series
1:50,000 Sheet 94 A/3 $W\frac{1}{2}$



NOTES :

1) RIVERWARD BERM TOP ELEVATION AT 446 m
2) SECOND BERM TOP ELEVATION VARIES FROM 446 TO 444 m
3) WEST END OF BERM AREA TO BE CLOSED TO PREVENT ENTRY OF PEACE RIVER FLOWS DURING CHANNEL BLOCKAGE
4) EAST END OF BERM AREA TO BE LEFT OPEN TO ALLOW DRAINAGE

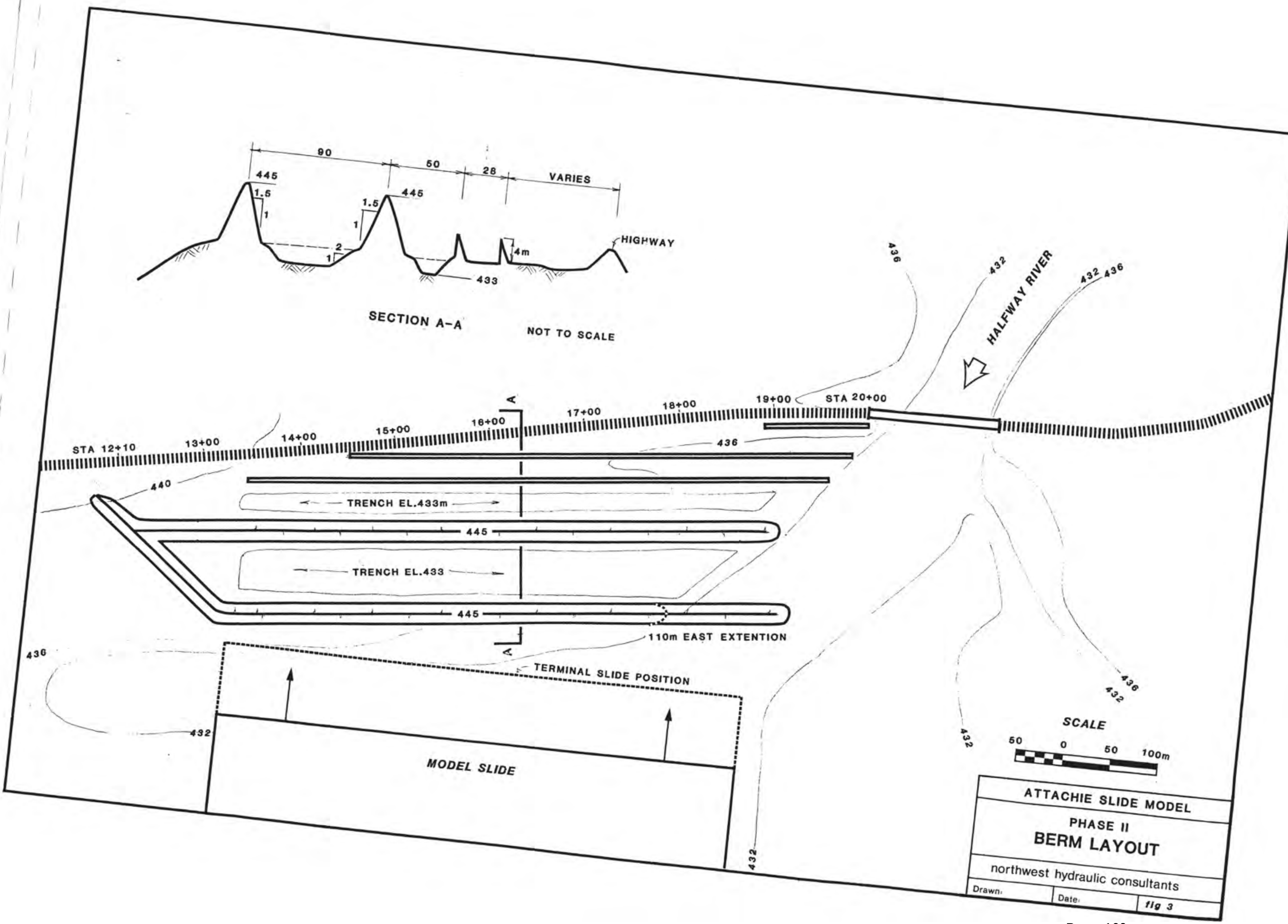
ATTACHIE SLIDE MODEL

PHASE I

BERM LAYOUT

northwest hydraulic consultants

Drawn: Date: fig 2



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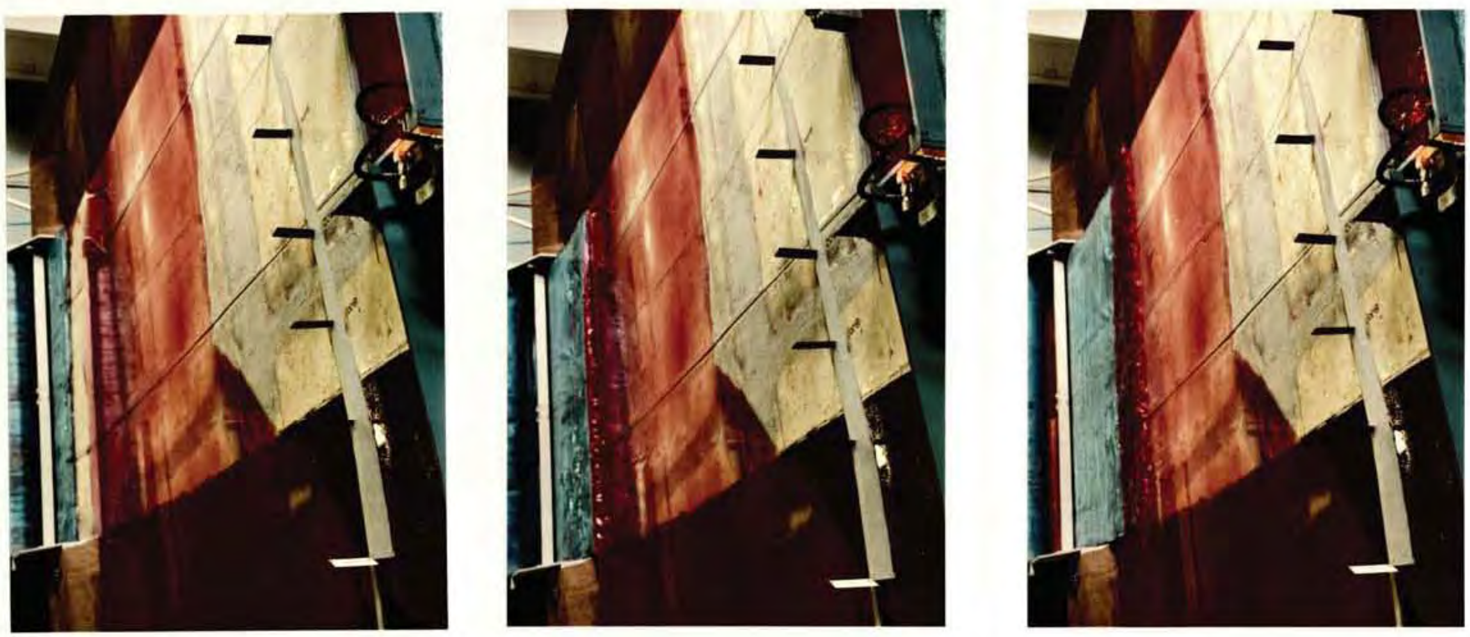
50
28
VARIES
445
1.5
1
445
1.5
1
2
1
4m
HIGHWAY
433
SECTION A–A
NOT TO SCALE
436
432
HALFWAY RIVER
432
436
A
STA 12+10
13+00
14+00
15+00
16+00
17+00
18+00
19+00
STA 20+00
436
440
TRENCH EL.433m
445
TRENCH EL.433
445
110m EAST EXTENTION
A
436
TERMINAL SLIDE POSITION
432
MODEL SLIDE
432
436
432
432
SCALE
50
0
50
100m
ATTACHIE SLIDE MODEL
PHASE II
BERM LAYOUT
northwest hydraulic consultants
Drawn:
Date:
fig 3

ATTACHIE SLIDE NEGATIVES

PHOTO NO.	NEG NO.
1	1349-2-11
2	1349-2-12
3	1349-2-13
4	1349-2-15
5	1349-2-17
6	1349-2-18
7	1349-2-19
8	1318-6-20A
9	1318-6-8A
10	1318-6-11A
11	1318-6-12A
12	1318-6-13A
13	1318-6-14A
14	1318-6-15A
15	1318-6-16A
16	1349-4-9
17	1349-4-10
18	1349-4-11
19	1349-4-12
20	1349-4-15
21	1349-4-16
22	1349-4-17
23	1349-5-1
24	1349-5-2
25	1349-5-3
26	1349-5-4
27	1349-5-5
28	1349-5-6
29	1349-5-7
30	1349-5-16
31	1349-5-17
32	1349-5-18
33	1349-5-22
34	1349-6-2A
35	1349-6-3A
36	1349-6-4A

PHOTOS 4-7



PHOTOS 1-3

PHOTOS 1-7
Baseline Tests

Sequence showing the wave runup for 15 m/s slide at the mean annual flood water level of 432.4. Photos 2, 3, & 4 show that the wave velocity exceeds the slide velocity.

PHOTO 8 Phase I Tests. 1318-6-20A
View looking towards the Phase I berm geometry consisting of two berms with an excavated trench between them. The water is dyed red to aid visibility. The slide path is indicated in white.

PHOTO 9. Phase I Tests 1318-6-8A
Maximum wave runup for Phase I berm geometry when tested at the mean annual flood water level (Elev. 432.2 m). The slide (blue) is at rest at the toe of the first (riverward) berm. Slight overtopping of highway is visible in the background at a low spot in highway.



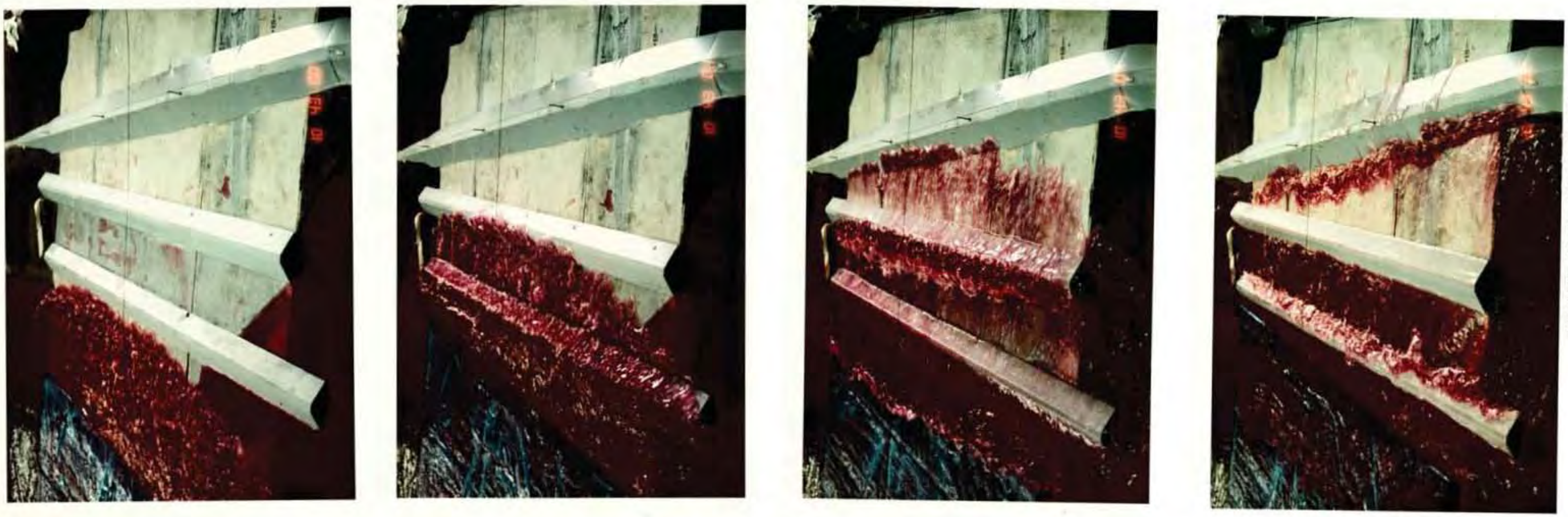
PHOTOS 10-12



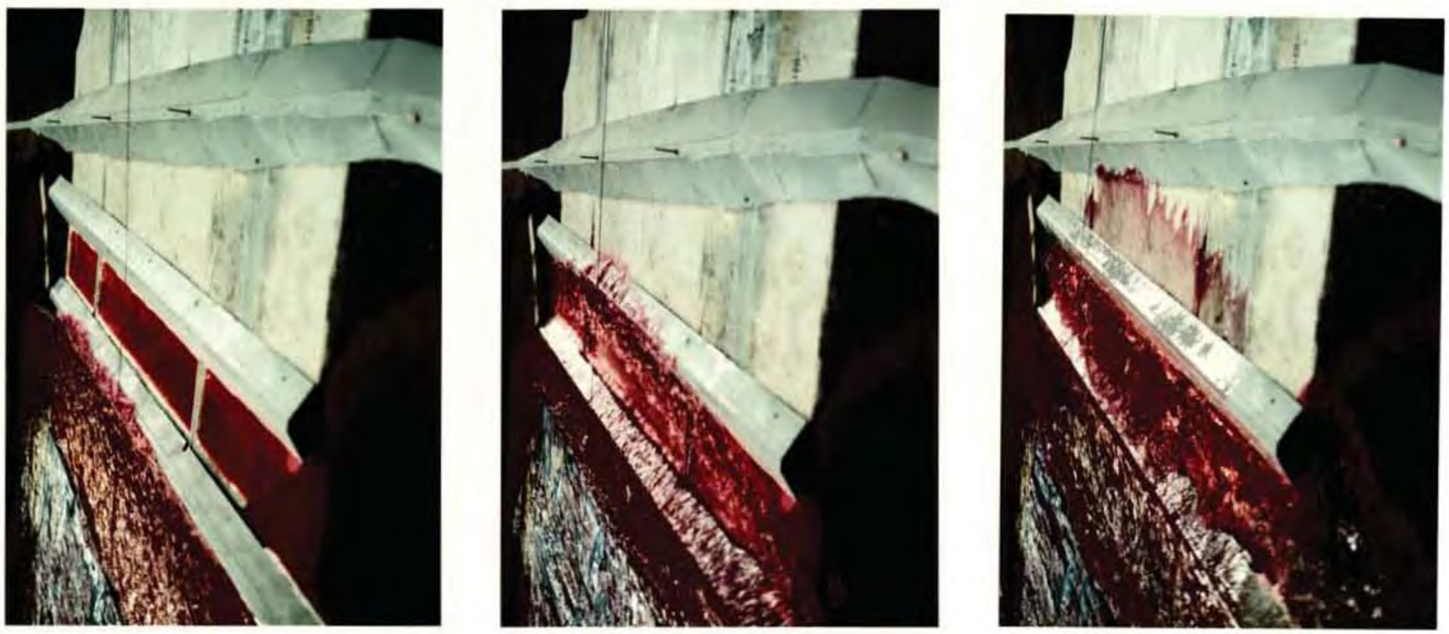
PHOTOS 13-15

PHOTOS 10-15
Phase I Tests

Sequence showing the wave progression for the Phase I berm when tested at the five year flood water level of 433.4 m.



PHOTOS 19-22 (No trench)



PHOTOS 16-18 (With trench)

PHOTOS 16-22
Phase II Tests

Photos 16-18 (trench at Elev. 430.0 m) and Photos 19-22 (No trench) illustrate the effect of a water-filled trench. Compare the wave volumes overtopping the second berm.



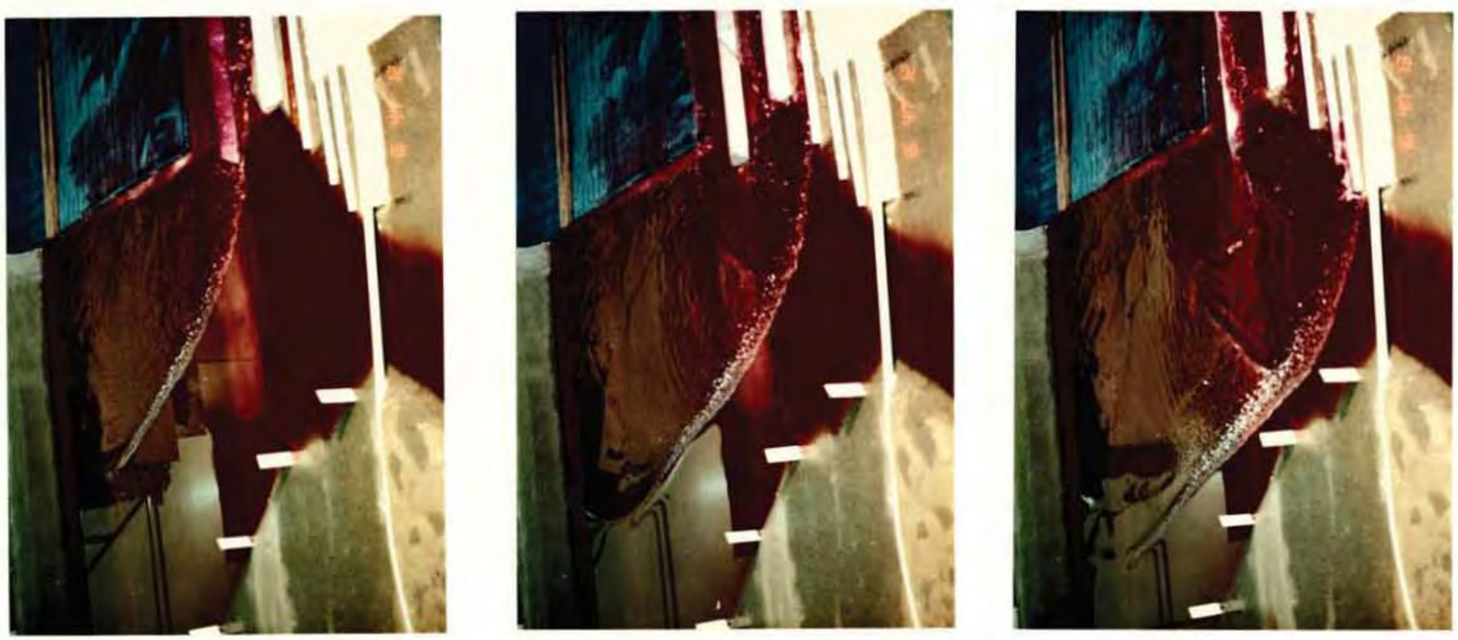




PHOTOS 23-29
Phase II Tests

Sequence showing the progression of the wave generated by the 15 m/s slide at the five year flood water level of 433.4 m. Without an extension of the first riverward berm the wave overtops the east bridge abutment (foreground of Photo 29).

PHOTOS 33-36 (With Extension)



PHOTOS 30-32 (Without Extension)

PHOTOS 30-36
Phase II Tests

Photos 30-32 (with 110 m extension) and Photos 33-36 (Without extension) illustrate the impact of the extension of first riverward berm on the magnitude of the wave directly approaching the bridge. The wave reflecting off the extension is shown in Photo 35.

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Halfway River: Hydrological Analysis for June 2001 Flood

160000 - HYDROTECHNICAL ENGINEERING - Hydrotechnical structure case files

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Halfway River: Hydrological Analysis for June 2001 Flood

Final
March 2002

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March 22, 2002

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Provincial Emergency Program
Ministry of Public Safety and Solicitor General
P.O. Box 9201 Stn. Prov. Govt.
455 Boleskine Road
Victoria, B.C. V8W 9J1

Dear Mr. Whyte:

RE: BC PROVINCIAL EMERGENCY PROGRAM
Halfway River: Hydrological Analysis for June 2001 Flood
Our File 254.018

We are pleased to submit four copies of the Halfway River: Hydrological Analysis for June 2001 Flood report, in final form.

The report provides a comprehensive review of the available hydrological data, and analysis of the factors contributing to the June 2001 flood. In addition, a frequency analysis is provided to rank the June 2001 flood event against others in the available records.

We thank you for the opportunity to work on this interesting assignment. We look forward to any further assistance we can provide.

Yours truly,

KERR WOOD LEIDAL ASSOCIATES LTD.

Dave N. Murray, P.Eng., A.Sc.T.
Project Manager

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Encl.

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Halfway River: Hydrological Analysis for June 2001 Flood

Final
March 2002

KWL File No. 254.018

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CONTENTS

1.	INTRODUCTION	1-1
1.1	BACKGROUND	1-1
1.2	STUDY TEAM	1-1
1.3	ACKNOWLEDGEMENTS	1-1
2.	DESCRIPTION OF STUDY AREA	2-1
2.1	HALFWAY RIVER WATERSHED	2-1
2.2	BIOGEOCLIMATIC ZONES	2-2
3.	HYDROLOGICAL DATA	3-1
3.1	CLIMATE DATA	3-1
	RELEVANT CLIMATE STATIONS	3-1
	PRECIPITATION DATA	3-2
	ANALYSIS OF 2001 PRECIPITATION DATA	3-3
3.2	SNOW SURVEY DATA	3-4
	RELEVANT SNOW SURVEY STATIONS	3-4
	ANALYSIS OF SNOW SURVEY DATA	3-4
3.3	HYDROMETRIC DATA	3-4
	RELEVANT HYDROMETRIC STATIONS	3-4
	ANALYSIS OF HYDROMETRIC DATA	3-5
4.	ANALYSIS OF 2001 HYDROMETRIC DATA	4-1
4.1	FLOW COMPARISONS	4-1
4.2	METHODOLOGY FOR FREQUENCY ANALYSIS	4-2
4.3	HALFWAY RIVER FREQUENCY ANALYSIS	4-2
4.4	GRAHAM RIVER FREQUENCY ANALYSIS	4-3
4.5	MOBERLY RIVER FREQUENCY ANALYSIS	4-4
4.6	SIKANNI CHIEF RIVER FREQUENCY ANALYSIS	4-4
4.7	SUMMARY OF FREQUENCY ANALYSIS RESULTS	4-5
5.	JUNE 2001 FLOOD ANALYSIS	5-1
5.1	WEATHER CONDITIONS	5-1
5.2	CONTRIBUTING FACTORS	5-1
5.3	SUMMARY OF ANALYSIS	5-2
6.	CONCLUSIONS	6-1
6.1	REPORT SUBMISSION	6-2

FIGURES

1-1	Location Map	after 1-1
3-1	Hydrological Stations	after 3-1
3-2	Fort St. John Airport Monthly Precipitation Totals	3-2
3-3	Regional Precipitation Variation	opposite 3-3
3-4	Pine Pass Snow Water Equivalent Comparison	opposite 3-4
4-1	Hydrograph Comparison for 2001	4-1
5-1	Halfway River Flood Event Data Summary	5-1

TABLES

2-1	Halfway River Watershed Characteristics	2-1
2-2	Cameron River Watershed Characteristics	2-1
2-3	Graham River Watershed Characteristics	2-1
2-4	Biogeoclimatic Zones	2-2
3-1	Climate Stations	3-1
3-2	Fort St. John Airport Monthly Precipitation Summary	3-2
3-3	Regional Precipitation in June	3-3
3-4	Hydrometric Stations	3-5
4-1	Halfway River Peak Flow Record	opposite 4-2
4-2	Halfway River Frequency Analysis Results	4-3
4-3	Graham River Peak Flow Record	opposite 4-3
4-4	Graham River Frequency Analysis Results	4-3
4-5	Moberly River Peak Flow Record	opposite 4-4
4-6	Moberly River Frequency Analysis Results	4-4
4-7	Sikanni Chief River Peak Flow Record	opposite 4-5
4-8	Sikanni Chief River Frequency Analysis Results	4-5

APPENDICES

A Environment Canada June 2001 Weather Analysis

Section 1

Introduction

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1. INTRODUCTION

1.1 BACKGROUND

The Halfway River is located in northeastern British Columbia and is a tributary to the Peace River downstream of BC Hydro's W.A.C. Bennett Dam and upstream of Fort St. John as shown on Figure 1-1.

Widespread flooding occurred on the Halfway River in June 2001. First reports of flooded land and road wash-outs came on June 2 and 3 from the upper Halfway River. The peak of the flooding occurred from June 10 to 12 and caused damage throughout the basin. Most of the damage from flooding occurred to farms, roads and bridges along the lower reaches of the tributary creeks and along the main stem of the Halfway River. Numerous bridges and many kilometres of roads were lost as a result of the flooding. Compensable damage in the Halfway River basin is estimated to be 23 million dollars to critical government infrastructure and private property. Non-compensable damage in the form of private business losses and economic losses may be as much as 20 million dollars.

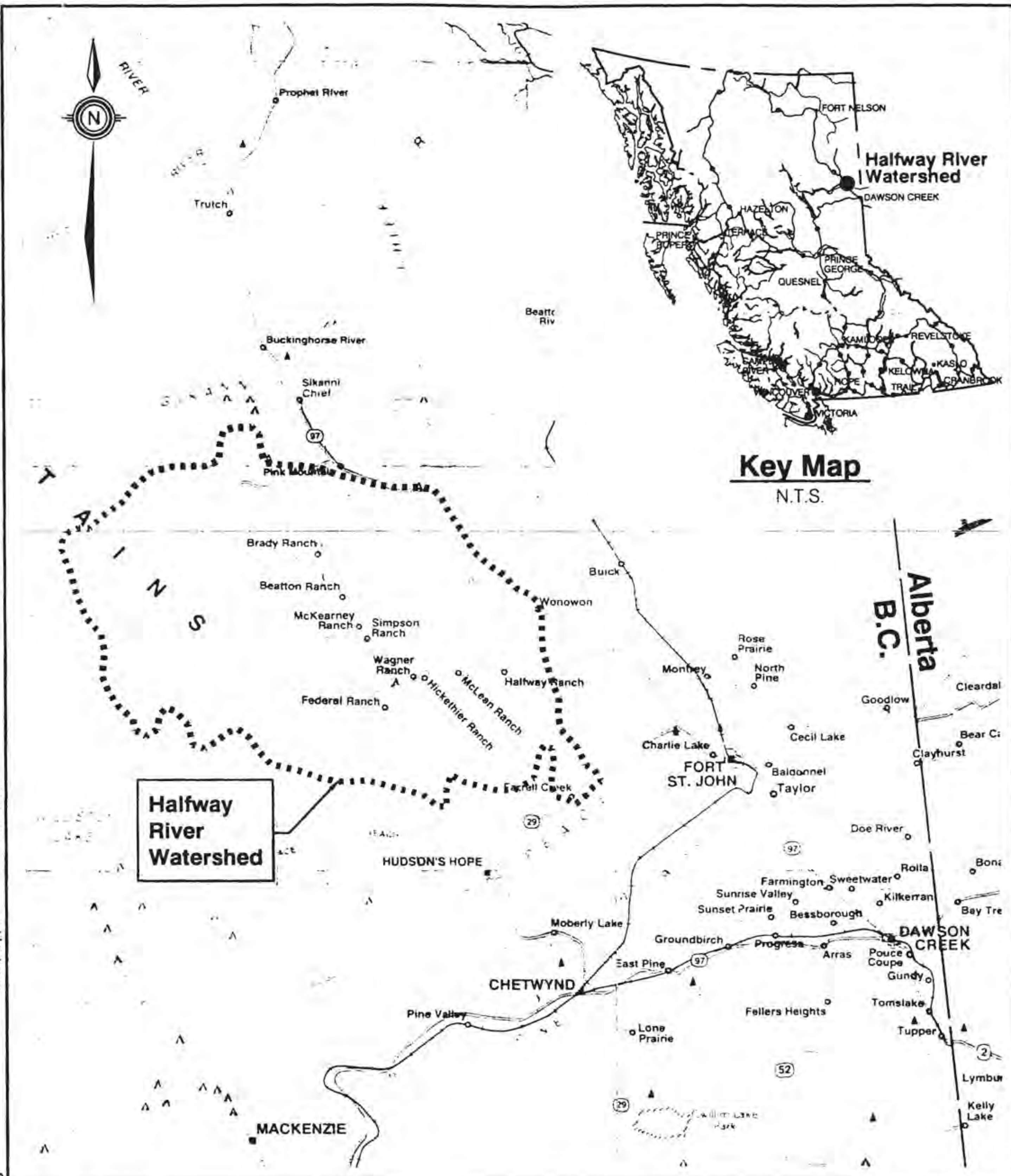
This study was undertaken to determine the type and cause (or causes) of the flood and provide a return period rating of the flood event.

1.2 STUDY TEAM

Troy Jones, P.Eng., completed the hydrologic analyses and wrote this report. Review was provided by Mike Currie, M.Eng., P.Eng., and Dave Murray, P.Eng.

1.3 ACKNOWLEDGEMENTS

The study team wishes to acknowledge the assistance of Jim Whyte of BC Provincial Emergency Program for assistance in completing this study, and for providing background information and data to facilitate the hydrologic analyses. Lynne Campo of Water Survey of Canada also provided helpful assistance in hydrometric data collection.



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Scale in Kilometres

Halfway River Hydrological Analysis

Location Map

Figure 1-1

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Section 2

Description of Study Area

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2. DESCRIPTION OF STUDY AREA

2.1 HALFWAY RIVER WATERSHED

The Halfway River watershed has a total drainage area of 9,350 km^2 (to the location of the Water Survey of Canada station near Farrell Creek, upstream of the confluence with the Peace River). The watershed ranges in elevation from approximately 450 m to 2,700 m, and has moderately steep slopes, particularly in the headwaters of the basin. There are no significant lakes within the watershed.

The majority of the watershed is forested, with sparse growth in the alpine areas. The valley areas are typically ranches and farmland. The Halfway River watershed has the following physical attributes:

Table 2-1
Halfway River Watershed Characteristics

Drainage area	9,350 km^2
Elevation range	450 – 2,700 m
Approximate total river length	235 km
Average river gradient	1%

Halfway River has two significant tributaries, Cameron River and Graham River. The physical attributes of each river are as follows:

Table 2-2
Cameron River Watershed Characteristics

Drainage area	2,020 km^2
Elevation range	550 – 1,200 m
Approximate total river length	135 km
Average river gradient	0.5%

Table 2-3
Graham River Watershed Characteristics

Drainage area	2,200 km^2
Elevation range	750 – 2,400 m
Approximate total river length	165 km
Average river gradient	1%

2.2 BIOGEOCLIMATIC ZONES

The Halfway River watershed area is comprised of three primary Biogeoclimatic Zones as defined by the British Columbia Ministry of Forests (1988). The three zones and their descriptions are provided in the following table.

Table 2-4
Biogeoclimatic Zones

Zone	Description
Alpine Tundra	This is a treeless zone characterized by a very harsh climate typically found on high mountains. This zone has long, cold winters and short, cool growing seasons that results in little or no woody plants.
Engelmann Spruce – Subalpine Fir	This zone has a harsh climate with long, cold winters and short, cool growing seasons. Trees species that can tolerate extended frozen ground conditions, such as Engelmann spruce and subalpine fir, occur in clumps and are interspersed with meadow and grasslands.
Boreal White and Black Spruce	This zone is part of the extensive boreal coniferous forest that exists across Canada. The winters are long and cold and the growing season is short, resulting in low productivity forests. This zone can also contain valuable agricultural land as is found in the Peace River valley.

The mountain peak areas (approximately 20% of the Halfway River watershed area) are within the Alpine Tundra and Engelmann Spruce – Subalpine Fir Biogeoclimatic Zones. The remainder of the Halfway River watershed is composed of the Boreal White and Black Spruce Biogeoclimatic Zone.

Section 3

Hydrological Data

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3. HYDROLOGICAL DATA

Hydrological data is available from a number of sources including climate stations, snow survey stations, and hydrometric stations. This sections reviews hydrological data sources, summarizes relevant hydrologic data, and analyses the available data in the context of the June 2001 periods. Figure 3-1 shows the type and location of all hydrological data stations that are referred to in this section. In general, the availability of data for the purposes of this study was limited, with the exception of the hydrometric data. Snow survey and rainfall stations are not present within the Halfway River basin and data from the nearest available stations was used for analysis.

3.1 CLIMATE DATA

RELEVANT CLIMATE STATIONS

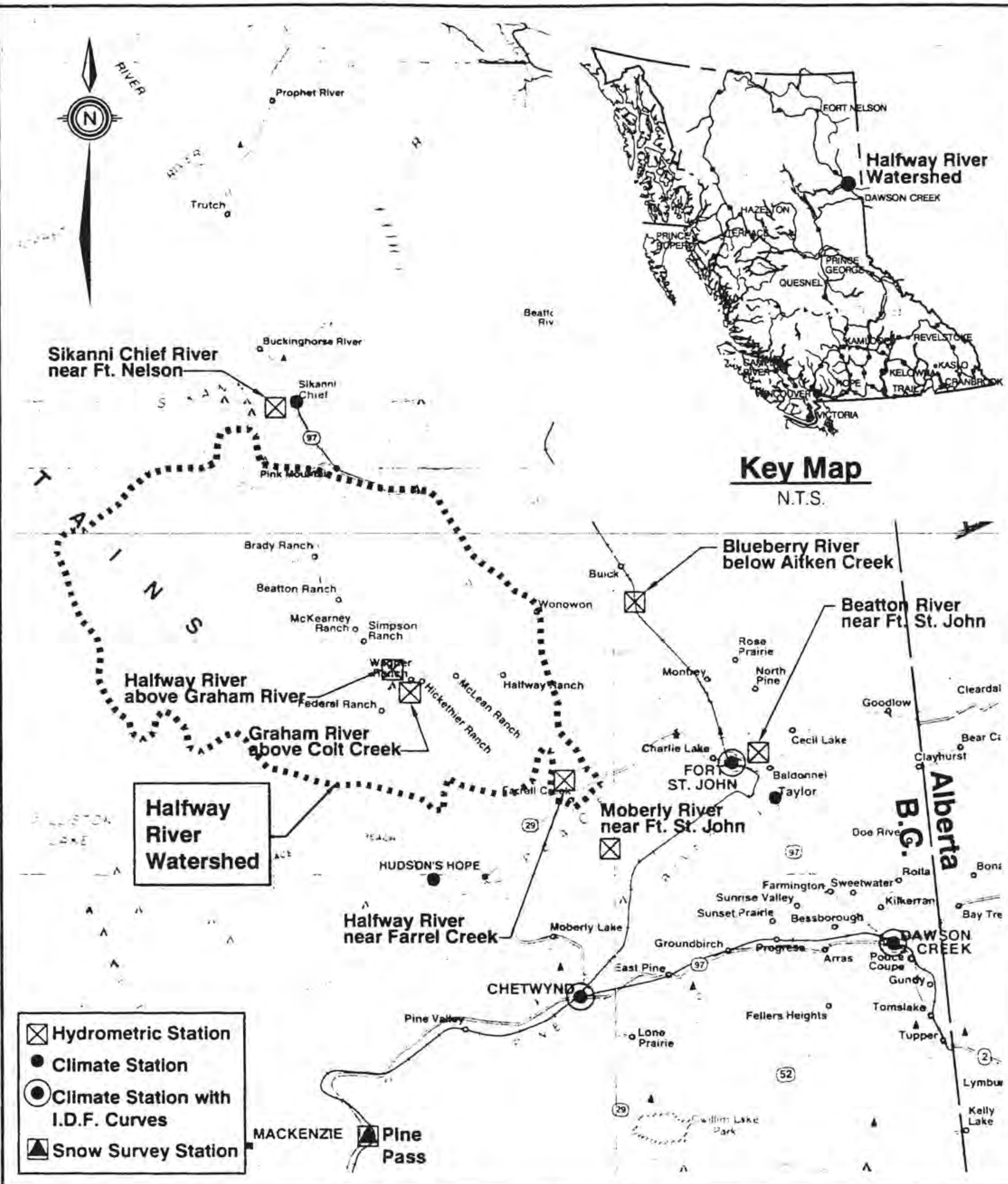
The Atmospheric Environment Service (AES) of Environment Canada co-ordinates a network of climate stations throughout British Columbia. Various types of data are collected including precipitation, temperature, relative humidity, wind speed and wind direction.

Precipitation data is available from several nearby climate stations. The data includes both rainfall and snowfall. It should be noted that precipitation intensity can vary significantly over a region during any given storm event, therefore there is a need to exercise caution when comparing data from different climate stations. Precipitation is an important factor in river hydrology; however antecedent hydrologic conditions such as river flows, snowpack, and ground saturation are typically the most significant factors.

The AES stations listed in Table 3-1 are selected as being the most representative active stations for Halfway River that also have existing Intensity Duration Frequency (IDF) curves.

Table 3-1
Climate Stations

Station Name	Station Elevation	Period of Record	Data Available
Fort St. John Airport	694 m	1942 – 2002	Daily Totals IDF Curves based on 16 years
Chetwynd Airport	610 m	1970 – 2002	Daily Totals IDF Curves based on 28 years
Dawson Creek Airport	652 m	1968 – 2002	Daily Totals IDF Curves based on 15 years
Note: IDF is the Intensity Duration Frequency			



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Halfway River Hydrological Analysis
Hydrological Stations
Figure 3-1

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All three of the climate stations listed have a relatively long period of record, and IDF curves that include the 2 to 100-year return period rainfall amounts are available. Precipitation data from Fort St. John Airport was used as the primary data source for this study because the station is the closest to the Halfway River watershed. However, it must be noted that hourly precipitation data recorded within the Halfway River basin would have been preferable for the analysis because of the large spatial variation that is typical of precipitation events, especially considering the large size of the Halfway River basin.

PRECIPITATION DATA

Data from the Fort St. John Airport station was analyzed and summaries of total monthly precipitation are presented in the Table 3-2 and Figure 3-2. Precipitation amounts include both rainfall and snowfall. The following table and figure provide a comparison between 2001 and the typical precipitation amounts. Monthly precipitation is relatively constant throughout the year, with the exception of the summer months when more significant precipitation is typical.

Table 3-2
Fort St. John Airport Monthly Precipitation Summary

	Monthly Precipitation Totals (mm)											
	Jan.	**Feb.**	**Mar.**	**Apr.**	**May**	**Jun.**	**Jul.**	**Aug.**	**Sep.**	**Oct.**	**Nov.**	**Dec.**
Mean	29	24	24	21	37	67	73	55	39	28	30	30
Minimum	4	3	2	1	0	10	8	4	3	4	2	3
Maximum	80	75	74	91	110	155	183	157	118	125	97	86
2001	8	5	12	**27**	**66**	**111**	71	22	23	N/A	N/A	N/A

Note: N/A indicates that data is not available

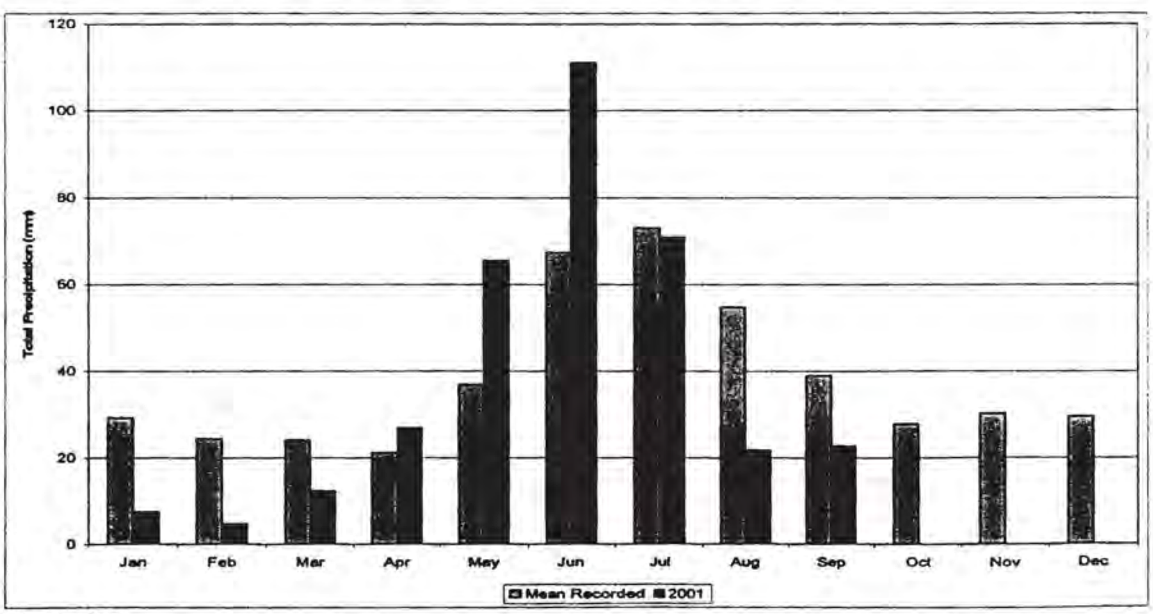

Figure 3-2
Fort St. John Airport Monthly Precipitation Totals

ANALYSIS OF 2001 PRECIPITATION DATA

In 2001, the months of April, May, and June had above-average total precipitation amounts; however, no daily rainfall totals exceed the 2-year return period event during these three months. The monthly totals and the daily precipitation values, however, indicate that above-average precipitation amounts and long storm durations characterize the precipitation pattern in the spring of 2001.

Table 3-3 lists the precipitation recorded throughout the region in June 2001. The climate stations listed are those closest to the Halfway River watershed that were active in June 2001.

Table 3-3
Regional Precipitation in June

Station Name	Years of Record Available	Average June Precipitation	Maximum June Precipitation	June 2001 Precipitation	Historical Monthly Rank
Hudson Hope - Brenot Creek	10	80	132	144	highest recorded
Hudson Hope - BCHPA Dam	22	74	141	144	highest recorded
Sikanni Chief	8	124	258	192	2nd highest recorded
Taylor Flats	38	70	136	127	2nd highest recorded
Chetwynd Airport	16	74	132	102	4th highest recorded
Dawson Creek Airport	31	78	173	119	6th highest recorded
Fort St. John Airport	60	67	155	111	9th highest recorded

The ranking of the events indicates that significant precipitation amounts occurred throughout the region in June 2001. Hudson Hope received the most precipitation ever for the month of June. Figure 3-3 (opposite) shows the large spatial variation in daily precipitation totals from the three regional stations listed in Table 3-3 that are the closest to the Halfway River watershed.

3.2 SNOW SURVEY DATA

RELEVANT SNOW SURVEY STATIONS

The BC Ministry of Sustainable Resource Management's Water Inventory Section manages a network of snow survey stations throughout the province. Snow survey data is published in March, April, May, and June of every year. The closest snow survey station to the Halfway River watershed is Pine Pass as shown on Figure 3-1. The Pine Pass station is located at an elevation of 1,400 metres and began recording data in 1989. The elevation of the Pine Pass snow survey station is representative of the Halfway River watershed because it is located at approximately the mid-basin elevation of Halfway River. However, the Pine Pass Station is approximately 150 kilometres from the centre of the Halfway River watershed and may not be totally representative of the snowpack conditions in the Halfway River basin. Analysis of data from a snow survey station within the basin or much closer to the basin would have been preferable because of the spatial variability of snowfall and the large size of the Halfway River basin.

ANALYSIS OF SNOW SURVEY DATA

Figure 3-4 (opposite) provides a comparison of the Pine Pass typical snow water equivalent to the snow water equivalent measured in 2001 at Pine Pass. Snow water equivalent is defined as a measure of the water content of the snowpack, expressed as a depth of water that would result from melting the snow.

The snowpack at Pine Pass in 2001 was characterized by below average snow accumulation through the winter months and higher than average amounts of rainfall in the spring months. The onset of the spring snowmelt was one to two weeks later than average. This indicates that the snowpack would have been very ripe at the beginning of June 2001 because the melt was underway. In addition, saturation of the snowpack would have occurred quickly with the above-average precipitation recorded in the region.

3.3 HYDROMETRIC DATA

RELEVANT HYDROMETRIC STATIONS

Water Survey of Canada operates a network of hydrometric stations throughout British Columbia to monitor streamflow. Hydrometric stations in the region around Halfway River are shown on Figure 3-1 and are summarized in Table 3-4. The hydrometric stations of most relevance to this study are those closest to Halfway River with similar physical characteristics.

Table 3-4
Hydrometric Stations

Station Name	Station Number	Drainage Area (km^2)	Period of Record	Flow Data Available
Halfway River near Farrell Creek	07FA006	9350	1983 – Present	17 years of max. daily, 15 years of max. inst.
Halfway River near Farrell Creek (Lower Station)	07FA001	9400	1917 – 1983	22 years of max. daily, 17 years of max. inst.
Graham River above Colt Creek	07FA005	2200	1981 – Present	18 years of max. daily, 17 years of max. inst.
Moberly River near Fort St. John	07FB008	1520	1980 – Present	21 years of max. daily, 21 years of max. inst.
Sikanni Chief River near Fort Nelson	10CB001	2160	1944 – Present	56 years of max. daily, 21 years of max. inst.
Blueberry River below Aitken Creek	07FC003	1750	1964 – Present	36 years of max. daily, 35 years of max. inst.
Beatton River near Fort St. John	07FC001	15,600	1961 – Present	40 years of max. daily, 24 years of max. inst.

Notes:
max. inst. is the maximum flow at any instant of time in the year of record
max. daily is the maximum average flow for one day in the year of record

ANALYSIS OF HYDROMETRIC DATA

This data is published up to 1999 and is available on CD-ROM in annual and historical summaries. Data from 2000 to the present must be ordered directly from Water Survey. Note that the data included in this report for 2001 had not yet been finalized by Water Survey of Canada. Preliminary data from 2001 has been used for analysis. Therefore, if the final numbers (when available) reported by Water Survey differ from the preliminary data, the analysis and results of this study should be reviewed at that time. Also, the 2001 results for Halfway River may be refined because WSC is updating the stage-discharge relationship for the station based on the large flow measured in 2001.

Based on a review of the Halfway River historical summary the annual streamflow patterns in this area can be summarized as follows:

- annual peak flows predominantly occur in the snowmelt freshet during the spring months of May through July;
- low to moderate base flows throughout the year;
- rainstorms in the summer and early fall may result in moderate to high streamflows;
- streamflows are low during the winter months while snowpacks accumulate; and

Section 4

Analysis of 2001 Hydrometric Data

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4. ANALYSIS OF 2001 HYDROMETRIC DATA

A regional analysis of hydrometric data can be used to determine the significance of a flood event. In particular, a comparison between the Halfway River and the surrounding watersheds can be completed to rate the magnitude of the June 2001 flood.

4.1 FLOW COMPARISONS

Hydrographs of all the rivers in the region have been plotted to determine the timing and magnitude of peak flows in the Halfway River region. These are shown in Figure 4-1.

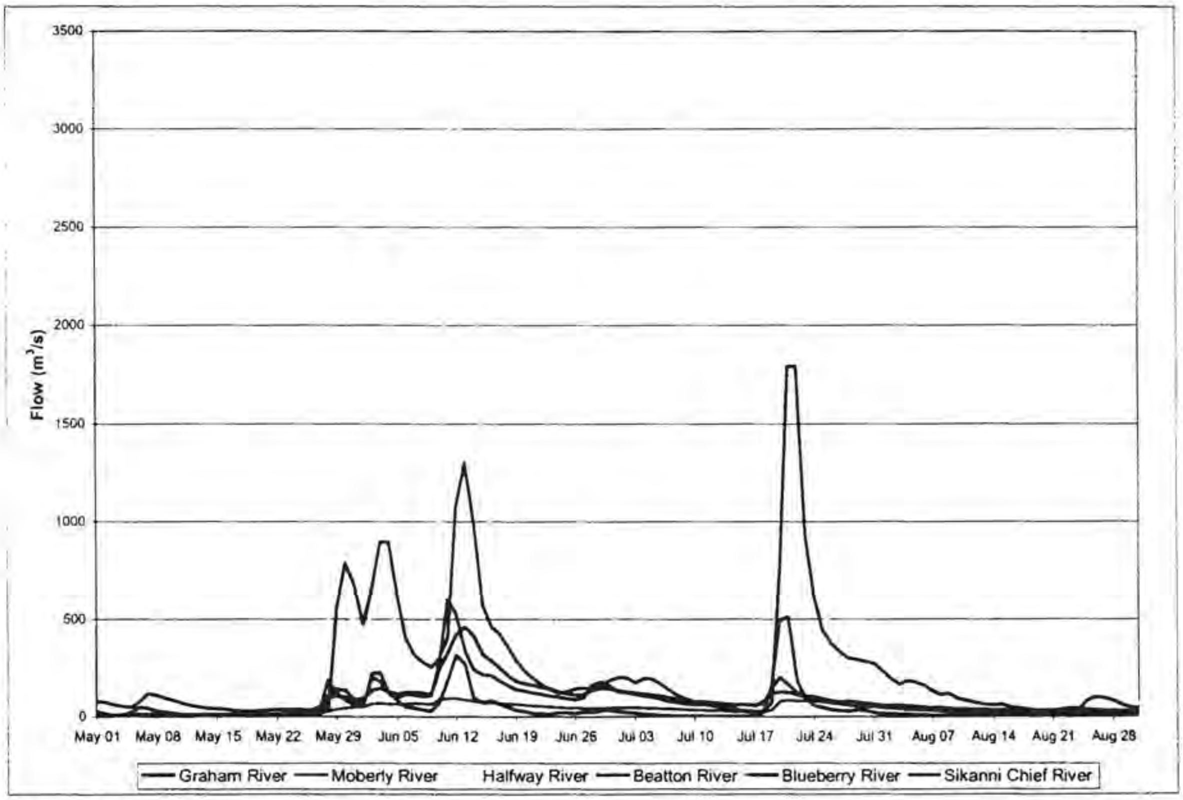


Figure 4-1
Hydrograph Comparison for 2001

The hydrograph comparison shows that Halfway River, Graham River, Moberly River, and Sikanni Chief River all peaked around June 12, while the Beatton River and Blueberry River had larger peaks later in July. Therefore, our hydrological analysis focuses on Halfway River, Graham River, Moberly River, and Sikanni Chief River.

4.2 METHODOLOGY FOR FREQUENCY ANALYSIS

Frequency analysis of maximum daily and maximum instantaneous flows for the available periods of record allows the June 2001 flood to be rated. Frequency analysis of the maximum instantaneous and maximum daily flows was completed using the Consolidated Frequency Analysis (CFA) version 3.1 software package. This package computes flood flow estimates for various return periods using the following methods:

- generalized extreme value distribution;
- three-parameter log-normal distribution;
- log Pearson type III distribution;
- Wakeby distribution; and
- non-parametric method.

The results from each type distribution was analyzed for the best fit to the data and the best fit method (or methods) was selected for reporting results. The results of the frequency analysis of Halfway River, Graham River, Moberly River, and Sikanni Chief River are discussed in the following sections.

4.3 HALFWAY RIVER FREQUENCY ANALYSIS

The Halfway River near Farrell Creek WSC hydrometric station was relocated in 1983 from the original site to a site upstream, with a resultant change in contributing drainage area from 9,400 km^2 to 9,350 km^2. The data collected prior to 1983 is valuable and should be used in conjunction with the data collected since 1983. The pre-1983 data can be shifted to the site upstream by applying a factor to the recorded maximum flows that is equivalent to the ratio of the contributing drainage areas of the two sites. This assumption is a reasonable one because of the very small difference in drainage area.

Maximum instantaneous peak flows are the most important peak flow data, yet there are years when only a maximum daily flow is recorded. An estimate of the difference between maximum instantaneous peak flows and maximum daily flows for the Halfway River system was determined from analysis of the years when both maximum instantaneous and maximum daily flow values exist. This difference, expressed as a multiplicative factor, is applied to the maximum daily peak flows to estimate the maximum instantaneous peaks. Table 4-1 (opposite) summarizes the complete flow record for Halfway River. All data has been shifted to the newer site and all missing instantaneous maximum flows have been estimated where possible.

The frequency analysis indicates that the generalized extreme value and the three-parameter log-normal distributions provide the best fit to the data; therefore, the results of the frequency analysis are summarized in Table 4-2 as an average of the generalized extreme value and the three-parameter log-normal distributions.

Table 4-2
Halfway River Frequency Analysis Results

Return Period (years)	Maximum Instantaneous Flow (m^3/s)	Maximum Daily Flow (m^3/s)
2	695	627
10	1,675	1,495
20	2,235	2,005
50	3,150	2,850
100	4,010	3,650
200	5,030	4,615
2001 event	**3,313**	**3,036**

This indicates that the June 2001 maximum instantaneous and maximum daily peak flows are in the range of a 50 to 100-year return period event for the Halfway River.

4.4 GRAHAM RIVER FREQUENCY ANALYSIS

The Graham River is one of the major tributaries of the Halfway River and therefore it would be expected that a similar flood peak would occur in June 2001 due to similar antecedent conditions, precipitation, and snowpack. Table 4-3 (opposite) summarizes the complete flow record for Graham River. The missing records were not estimated because the number of missing years was not significant in the period of record. The frequency analysis indicated that the generalized extreme value, the three-parameter log-normal, and the log Pearson type III distributions provide the best fit to the data; therefore, the results of the frequency analysis are summarized in Table 4-4 as an average of the three methods.

Table 4-4
Graham River Frequency Analysis Results

Return Period (years)	Maximum Instantaneous Flow (m^3/s)	Maximum Daily Flow (m^3/s)
2	174	161
10	340	301
20	445	378
50	629	503
100	814	619
200	1,050	757
2001 event	**461**	**418**

The results indicate that the June 2001 maximum instantaneous and maximum daily peak flows recorded on Graham River are in the range of a 20 to 50-year return period event.

4.5 MOBERLY RIVER FREQUENCY ANALYSIS

The Moberly River is directly south of the Halfway River and has similar topography. Therefore, it could also be expected that the June 2001 flood peak would be similar to Halfway River. Table 4-5 (opposite) summarizes the complete flow record for Moberly River. The frequency analysis indicates that the generalized extreme value, the three-parameter log-normal, and the log Pearson type III distributions provide the best fit to the data; therefore, the results of the frequency analysis are summarized in Table 4-6 as an average of the three methods.

Table 4-6
Moberly River Frequency Analysis Results

Return Period (years)	Maximum Instantaneous Flow (m^3/s)	Maximum Daily Flow (m^3/s)
2	69	68
5	94	92
10	112	108
20	130	125
50	153	146
100	172	163
200	192	179
2001 event	**93**	**90**

This indicates that the June 2001 maximum instantaneous and maximum daily peak flows are approximately equivalent to a 5-year return period event for the Moberly River.

4.6 SIKANNI CHIEF RIVER FREQUENCY ANALYSIS

The Sikanni Chief River is directly north of the Halfway River and has similar topography. Therefore, it would also be expected that the June 2001 flood peak would be similar to Halfway River. Table 4-7 summarizes the flow record for Sikanni Chief River. The missing records were not estimated because the number of missing years was not significant in the period of record. The frequency analysis indicated that the generalized extreme value distribution provided the best fit to the data; therefore, the results of the Sikanni Chief River frequency analysis as provided in Table 4-8 are only from the generalized extreme value distribution results.

Table 4-8
Sikanni Chief River Frequency Analysis Results

Return Period	Maximum Instantaneous Flow (m^3/s)	Maximum Daily Flow (m^3/s)
2	222	180
5	337	274
10	436	349
20	553	432
50	746	561
100	928	675
200	1,150	807
2001 event	**603**	**502**

This indicates that the June 2001 maximum instantaneous and maximum daily peak flows were a 20 to 50-year return period event for the Sikanni Chief River.

4.7 SUMMARY OF FREQUENCY ANALYSIS RESULTS

The results of the regional frequency analysis indicate that the region experienced significant flows, ranging from a 5-year return period event at one station to a 50 to 100-year return period event for Halfway River. Therefore, Halfway River experienced the most significant flood event when compared to the surrounding gauged watersheds.

Section 5

June 2001 Flood Analysis

KERR WOOD LEIDAL
associates limited
CONSULTING ENGINEERS

5. JUNE 2001 FLOOD ANALYSIS

5.1 WEATHER CONDITIONS

An Environment Canada weather analysis for June 2001 is provided in Appendix A. This analysis suggests that the June 2001 weather can be distinguished as three separate weather systems. However, it must be recognized that although there were three separate weather systems, the three systems occurred during a single snowmelt freshet event. In addition, it would be preferable to use hourly data recorded within the Halfway River basin for the analysis because of the large spatial variation that is typical of precipitation events.

5.2 CONTRIBUTING FACTORS

As determined in Section 4.3, the Halfway River watershed experienced a 50 to 100-year return period event in June 2001 that peaked on June 12, 2001 (3,313 m^3/s). The flooding that occurred was the result of a combination of above-average total precipitation and the snowmelt freshet. Figure 5-1 summarizes the regional climatic conditions during the spring of 2001.

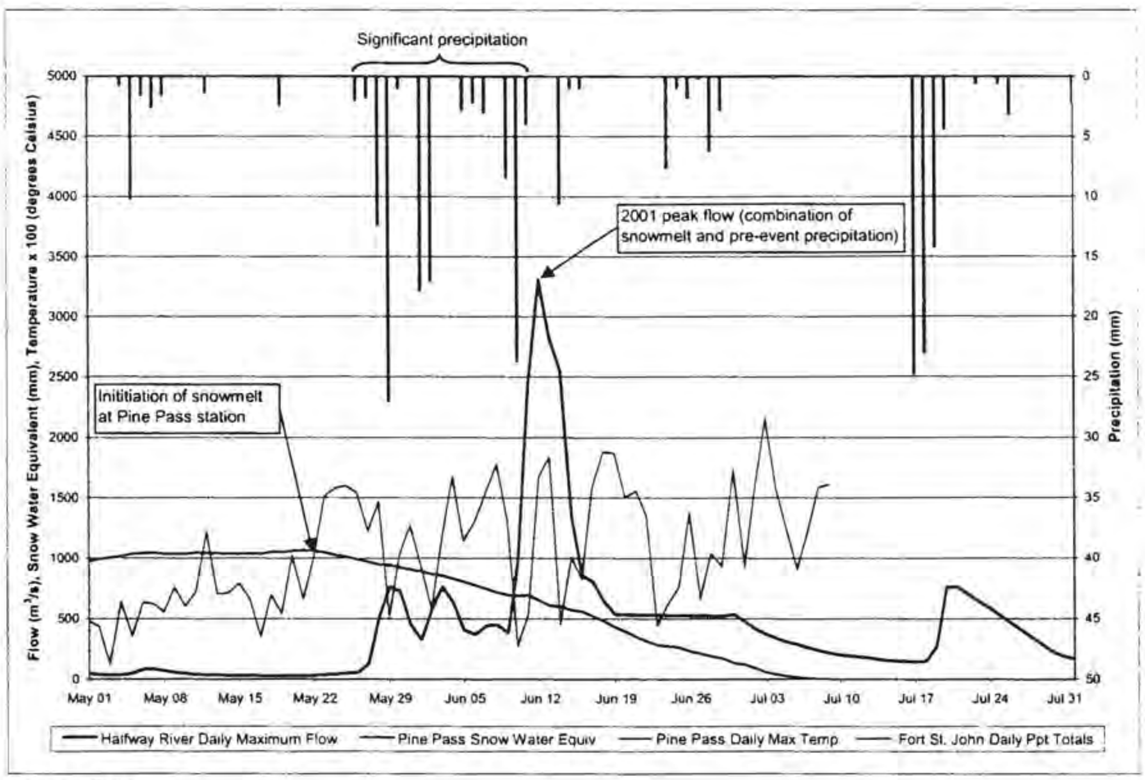


Figure 5-1
Halfway River Flood Event Data Summary

Figure 5-1 shows the following:

- Significant precipitation occurred in the two weeks preceding the June 12 flood peak. The majority of the precipitation recorded in June occurred within the first two weeks of the month.

- The temperature at the Pine Pass station was well above melting temperature in the two weeks preceding the flood event.

- The snow water equivalent indicates that the snowmelt at the Pine Pass station began approximately three weeks before June 12.

The Pine Pass snow survey station is approximately at the mid-basin elevation of the Halfway River watershed. Therefore, the majority of the basin would have been contributing snowmelt by the time of the flood event. Previous studies on other basins (Martinec, 1972 and Gaustka et al, 1958) have shown that peak runoff associated with snowmelt generally occurs after snow cover has melted off a portion of the watershed basin. These observations indicate that the snowpack was very ripe and snowmelt would have been contributing to the flood event.

5.3 SUMMARY OF ANALYSIS

The analysis of all of the hydrological data and consideration of the contributing factors indicate that the June 2001 flood event was the cumulative effect of above-average precipitation over an extended duration plus the snowmelt freshet. While the recorded daily precipitation was not greater than typical values, the above-average precipitation in the two weeks preceding the event and the timing of the snowmelt freshet combined to produce a significant freshet event with a return period of 50 to 100 years.

Section 6

Conclusions

KERR WOOD LEIDAL
associates limited
CONSULTING ENGINEERS

6. CONCLUSIONS

1. The precipitation records in the region of the Halfway River watershed indicate the occurrence of above-average precipitation in April to June 2001. In particular, significant precipitation was concentrated over the first two weeks of June 2001.

2. The precipitation records indicate multiple events spread out over several weeks.

3. The snowpack at Pine Pass in 2001 was characterized by lower than average amounts of snow accumulation through the winter months and a slightly later than average melting time. The snowpack was considered very ripe at the beginning of June 2001.

4. The rivers in the region recorded significant flows during June 2001, with the most significant event occurring on Halfway River.

5. The results of a frequency analysis estimate the June 2001 event to be a 50 to 100-year return period event for Halfway River.

6. The flood event in June 2001 on the Halfway River that produced significant flooding in the region can be characterized as the cumulative effect of above-average precipitation over an extended duration combined with the spring snowmelt freshet.

6.1 REPORT SUBMISSION

Prepared by:

KERR WOOD LEIDAL ASSOCIATES LTD.

Mar 22/02

Troy N. Jones, P.Eng.
Project Engineer

Mar 22/02

Dave N. Murray, P.Eng., A.Sc.T.
Project Manager

Reviewed by:

March 22, 2002

Mike V. Currie, M.Eng., P.Eng.
Senior Water Resources Engineer

Appendix A

Environment Canada
June 2001 Weather Analysis

KERR WOOD LEIDAL
associates limited
CONSULTING ENGINEERS

Applications and Services Division
Environment Canada
Suite 120, 1200 West 73rd Avenue
Vancouver, B.C.
V6P 6H9

December 3, 2001

Mr. Roberto Gonzalez
Office of Critical Infrastructure Protection and Emergency Preparedness
PO Box 10,000
Victoria, B.C.
V8W 3A5

Dear Mr. Gonzalez:

The Province of British Columbia requested Federal Disaster Financial Assistance with respect to a series of intense wet weather systems which occurred in the Peace River area of BC from June 1 to June 30, 2001. This letter is in response to a request for Environment Canada to provide an analysis of weather conditions which occurred in the North Peace region during June 2001. The climate records for the following Environment Canada stations were scrutinized: Dawson Creek Airport, Taylor, Fort St. John Airport, Sikanni Chief, Chetwynd Airport and Hudson Hope. These stations are marked on the map (Fig. I).

This daily precipitation data for June 2001 is displayed in Table I. An inspection of Table I reveals that there is a seven day break between June 17th and June 23rd where no significant precipitation was observed. This dry interval is the result of a high pressure area which provides a clear break between the wet weather systems which affected the Peace River Basin at other times in the month. As a result one cannot attribute the precipitation pattern of the entire month to one weather circulation.

A closer analysis of Table I and weather maps not included in the report reveals that there were three distinct weather systems which generated significant precipitation.

On the evening of June 1st a maritime cold front traversed the area and delivered 28 mm of rain to Fort St John Airport during a 12 hour period ending on the morning of June 2nd. Precipitation totals at Taylor Flats, Fort St. John and Sikanni Chief exceeded 35 mm during June 1st and 2nd.

Another cold front from the Pacific moved northeastward across the southern Peace region on the 9th of June. An associated low pressure area formed over Alberta on the 10th with a trough lingering westward across the Peace basin. A moist westward return circulation delivered precipitation over the area on the 10th and 11th. The precipitation dwindled as the trough filled thereafter. Dawson Creek received 21.6 mm on the 9th while Fort St. John Airport measured 15 mm in a 6 hour period during the morning of the 10th. The heaviest amounts were received north of Fort St. John with Sikanni Chief reporting 88 mm between the 8th and 11th.

On June 23rd a cold front slumped southward from the Mackenzie valley and formed a stationary east west trough of low pressure over the Peace Basin. This feature generated 27 mm at Sikanni Chief on the 23rd with modest amounts measured further to the south. Moisture was replenished to this trough between the 26th to 28th as a series of moist impulses from Washington State tracked into Southern Saskatchewan and re-circulated back to Northeastern B.C. Sikanni Chief reported an additional 35 mm between the 26th and 28th with lesser amounts being reported further to the south. The total of 65 mm recorded at Sikanni Chief were associated with the general circulation of one weather system.

In summary a number of weather systems generated significant rainfalls over the North Peace Basin (as delimited in Figure I) during June 2001. From a meteorological perspective, however, the precipitation was associated with 3 distinct weather systems and not one system.

TABLE I
Measurable Daily Precipitation [mm] June 2001

Date	Dawson Creek	Taylor Flats	Fort St John A	Sikanni Chief	Hudson Hope	Chetwynd
1	18.6	11.6	17.8	29.6	18.8	28.4
2	0.4	24.6	17	11.4	0.2	0.3
3	0	0	0	0	0	0
4	0	0	0	0	0	1.8
5	5.8	4	2.8	3	6	2.6
6	2.2	5.4	2.2	0	25	0
7	0	1.4	3	4		0.2
8	0	0	0	1.6		0.7
9	21.6	0.4	8.4	28		18.6
10	0.2	30.2	23.4			6.8
11	1.4	4.2	4	58.4	49.2 A	1.4
12	0	0	0	0.4	0	0
13	0.8	0	0	0	0	0.2
14	0.8	0	10.6	4.8	5	1.2
15	21.6	23.2	1	0	0	6.2
16	3.4	0.6	1	2.6	2	6.4
17	0	0	0	0	0.2	0
18	0	0	0	0	0	0
19	0	0	0	0	0	0
20	0	0	0	0	0	2.2
21	1	0	0	0	0	0.8
22	0	0	0	0	0	0
23	0	0	0	27.2	0	0.2
24	7	4.4	7.6	0.8	7.8	15.2
25	2.8	5.2	1	1.2	6	6.6
26	0	3	1.8	13.8	0.8	0.4
27	5.4	0	0.2	10	8.4	7.2
28	2	3.4	6.2	11	12	5.6
29	0.4	5.4	2.8	2.8	0.8	0.6
30	6.8	0	0	0.2	0	5.3
Total	102.2	127	111.1	191.9	143.6	118.9

R R Dunkley

Reg Dunkley
Forensic Meteorologist
Head, Data Management
Pacific and Yukon Region
Environment Canada

FORM H. 118

PROVINCE OF BRITISH COLUMBIA

DEPARTMENT OF HIGHWAYS

TO: J.Alton, Bridge Engineer, Victoria, B.C.	SENDER'S ADDRESS: Fort St.John, B.C.
	DATE: February 11th 1966.
	ELECTORAL DISTRICT: North Peace River,R.4
	HEADQUARTERS FILE:
	REGIONAL FILE:
ATTENTION:	DISTRICT FILE: N27-40-04
SUBJECT: HALFWAY RIVER BRIDGE:	REFERENCE: DATED:

Further to your teletype dated February 9th, we have to advise that high water has been 5 feet from the bridge deck. Driftwood during high water has been heavy, with trees of Spruce and black Poplar up to a length of 100 feet.

The ice is usually about 24" thick and as the Halfway River usually goes out before the Peace, water backs up at the mouth causing the ice to rot and lift.

Should the Peace go out first, the ice-flow is excessive.

Paul A. Tondevold
P.A.Tondevold,
District Superintendent.

JWL/lk

FOR DEPARTMENTAL CORRESPONDENCE ONLY.

KHA

GOVERNMENT OF BRITISH COLUMBIA

MEMORANDUM

TO Senior Bridge Engineer,

Department of Highways, Douglas Bldg.,

ATTENTION: L.C. Johnson

FROM

Director of Location

February 14, 1966.

SUBJECT

OUR FILE L4-M68-396

YOUR FILE

For your information, I attach a copy of Mr. Beaumont's teletype of February 10, 1966, referring the High Water Mark elevation and flow in Halfway River. Would you amend the site plan accordingly please.

N.R. Zapf,
Director of Location.

JWP/bkj

Encl.

5206

BR. ENGR.

MR N R ZAPF
DIR OF LOCATION
DEPT OF HIGHWAYS
VICTORIA B C

L4-M68-396 - HALFWAY RIVER SITE PLAN

REGRET TO ADVISE FURTHER MISTAKE IN SITE PLAN. HIGH WATER MARK WAS 1435 APPROXIMATELY. THIS WILL MAKE FLOW ABOUT 246,000 CFS LEAVING VELOCITIES MUCH THE SAME AS SHOWN. THIS WILL ACCOUNT FOR PEOPLE GETTING THEIR FEET WET CROSSING THE BAILEY BRIDGE BECAUSE ITS SUPPORT WAS WASHED OUT AND ITS CENTER SAGGED DOWN INTO THE WATER.

E A BEAUMONT
REG LOC ENGR

PRINCE GEORGE
FEB 10/66
11:49 AM
MSGE RCDVIC

LOCATION ENGINEER
Department of Highways
FEB 10 1966
VICTORIA, B. C.

BR. ENGR.

October 26, 1971.

Halfway River Bridge

Site visit, October 19, 1971, by W. A. Bowman, K. H. Arnott (Bridge Office), H. Good (District Superintendent), and the District Bridge Foreman.

The Peace River

Since the construction of the W.A.C. Bennett Dam, the Peace River is controlled at a constant flow of approximately 60% of the previous figure. This has eliminated the backing up of the Peace River into the Halfway River at high water. Consequently, the Halfway River is establishing a deeper channel on its east side. (See attached sketch.) The Bridge Foreman estimated that the east bank had scoured approximately 10 ft. in the past six years. The main stream of the river is diverted across to the west bank which has been badly scoured north and south of the existing bridge.

Conclusions

Due to the topographical changes at the site, it was concluded that a superior crossing could be achieved north of the existing bridge. It is recommended that the alignment be approximately as shown on the sketch, with the central pier and the abutments located in line with those of the existing bridge. It may be necessary to adopt a 'T' style pier to avoid impingement upon the flow of the river. The drop in high water level to approximate elevation 1427 allows the roadway grade to be lowered to elevation 1440. However, in order to obtain a more economical and desirable structure, it is possibly an advantage for the grade to remain at elevation 1447.95. An economy study comparing a multi-girder, two span continuous structure and the previously proposed "Stabbogen" bridge will be made.

It is understood that district forces will provide a revised site plan in accordance with the above proposals.

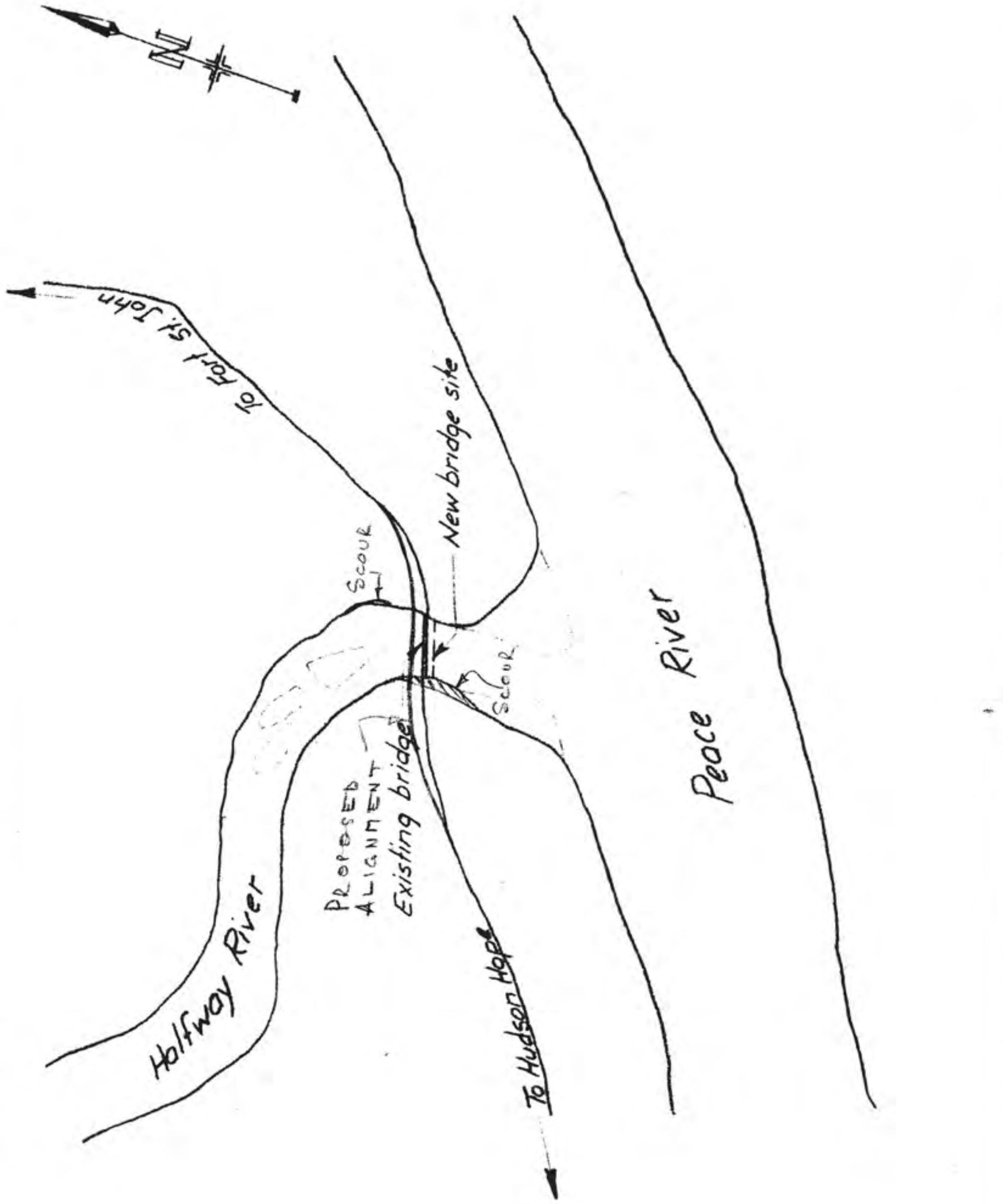
It is recommended that a Hydraulic Engineer be consulted for an opinion on the following.

(a) Is there any chance that the Peace River will ever reach its former high water level?

(b) Will the scour at the east and west banks continue?

K. H. Arnott,
Supervising Design Engineer.

KHA:lh



REVISIONS	B	A

BRITISH COLUMBIA
DEPARTMENT OF HIGHWAYS
BRIDGE DESIGN

HALFWAY RIVER BRIDGE
SITE PLAN Scale: 1" = 500'

DRAWN BY	RYC.	Oct. 25/71	APPROVED:	SKETCH No.
CHECKED BY				1042-18

CANCEL PRINTS BEARING EARLIER LETTER

GSK
BF

Victoria

September 6th, 1973.

L4-M68-396

Regional Highway Engineer,
Department of Highways,
Prince George.

Mr. E. A. Beaumont,
Reg. Hwy. Design & Surveys Eng.

Halfway River, Attachie Slide.

Attached is a print showing the area of the landslide on the Peace River which occurred on May 26th this year. A further slide in the distressed area immediately downstream could endanger the road and bridge at Halfway River.

Please investigate and report on the feasibility of relocating the bridge further upstream to avoid this danger.

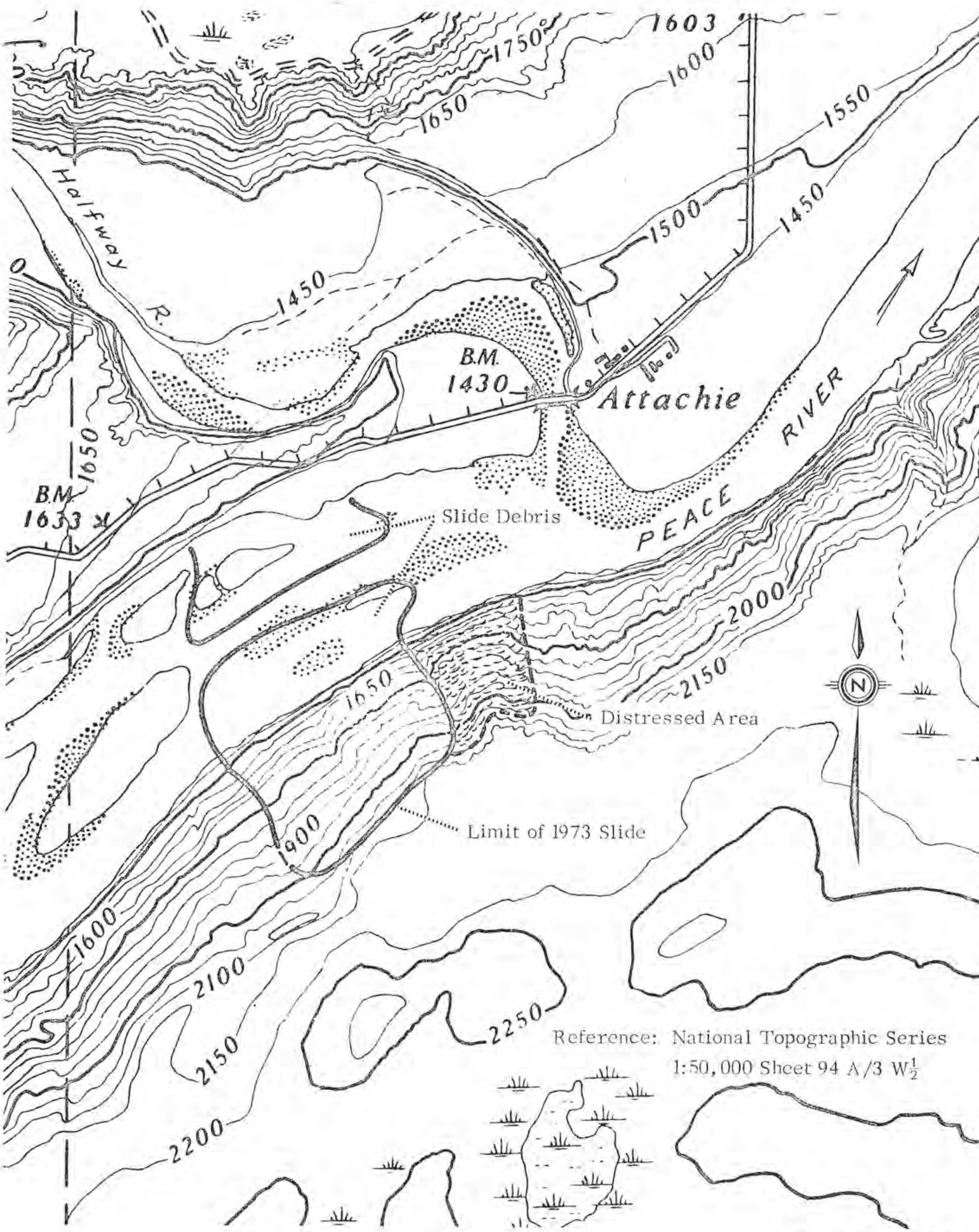
Sepias of the air photo mapping on the road were sent to you on April 8th, 1968.

E E Readshaw

E. E. Readshaw,
Director of Highway
Design and Surveys.

JWP:mdd
Encl.

→ c.c. Senior Bridge Engineer
Encl.



ATTACHIE SLIDE AREA
(PRE-SLIDE TOPOGRAPHY)

Scale: 1" - 2000'

Fig. 1

GOVERNMENT OF BRITISH COLUMBIA

MEMORANDUM

B. F.

TO Mr. W. A. Bowman, P. Eng.
Senior Bridge Engineer
Department of Highways

ATTENTION: Mr. K. Arnott

FROM

Geotechnical & Materials Branch

January 17th, 1974

SUBJECT Halfway River Bridge - Icebreaker
Highway #29, Fort St. John, Hudson Hope

OUR FILE 01-41-40

YOUR FILE

A steel H-pile icebreaker consisting of 12 piles is proposed to protect the existing bridge pier. It is understood that heavy ice conditions exist at the site.

The following general statements are applicable:

1) The river bottom consists of siltstone bedrock overlain by a 2 to 7 foot layer of medium dense sands.

2) The siltstone has substantial ultimate crushing strength (350-400 tons/ft^2).

3) Tensile strength of the siltstone is considerably lower than its crushing strength. However, due to the presence of discontinuities and general inhomogenity of the material is difficult to assess other than by direct field testing.

4) Because ice uplift on piles will be the primary force to be taken into consideration, the tensile strength of siltstone will be a governing factor.

5) Although it is difficult to accurately assess possible magnitude of ice uplift forces at the site, these may reach up to 25 tons per pile.

6) Owing to the high crushing strength and brittle nature of siltstone, driven piles are not recommended. Although under extremely hard driving conditions some penetration of rigid steel piles (1-3 feet) may be obtained, due to the brittleness of the siltstone, very low pullout capacity will result.

In view of the above, we recommend the use of 12" steel H piles socketed into holes prebored in siltstone for a length of 15 feet and filled with concrete. Pile spacing of 6 feet and over will be adequate.

. . . . 2

The holes should be predrilled, having a 20 inch diameter, to prescribed depth by a non-percussion method in order to minimize siltstone disturbance. Subsequently the holes must be cleaned from all loose material and remains of drilling.

It is preferable, but not essential, that concrete be placed in dry holes. Steel H piles should be inserted following concrete placement. Concrete should not protrude above the original siltstone surface.

Although the use of batter piles as shown in Drawing 1042-12 will be functional, the cost of drilling batter piles as opposed to vertical ones may prove significantly higher.

Consequently, it is preferable that vertical piles be installed and the desired icebreaker point be formed by welding additonal pile sections on to the horizontal piles. The proposed arrangement is schematically shown in the accompanying Drawing.

In the event that the particular river section is still considered as a potential future crossing site, perhaps, some pile arrangement that could eventually be used as a pier foundation should be devised.

J. W. G. Kerr, P. Eng.
Senior Geotechnical & Materials Eng.

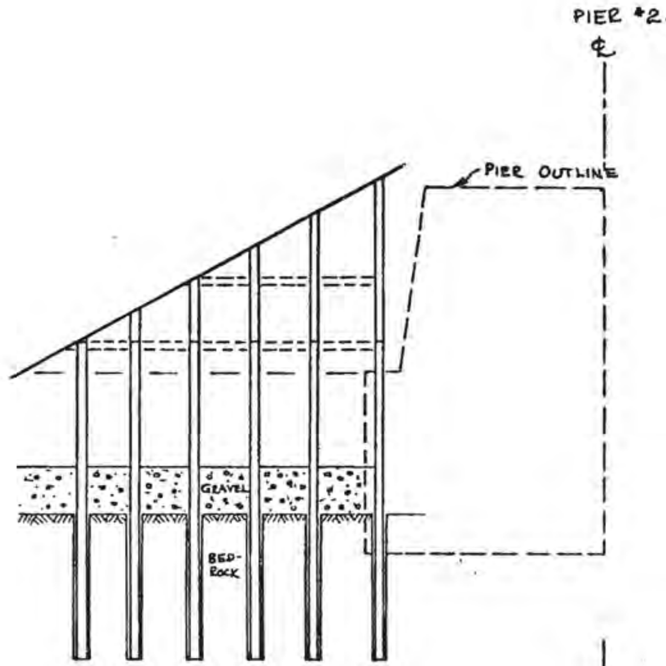
by: for [signature]

T. S. Coulter, P. Eng.
Evaluation & Design Engineer

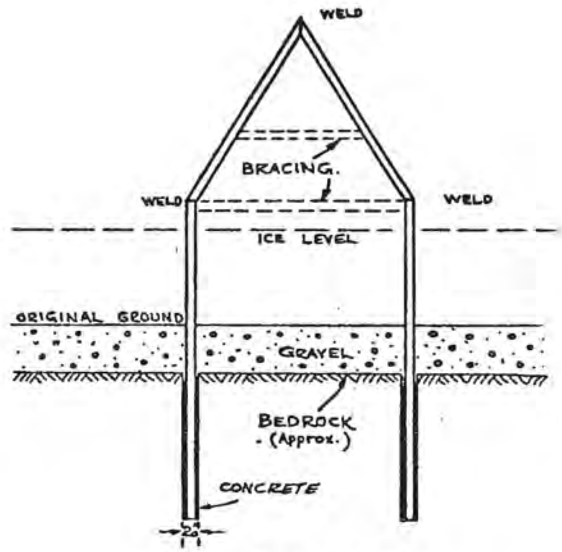
IG:jb

Attachment

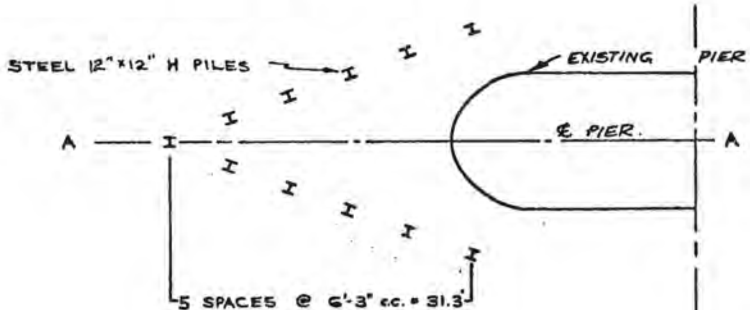
cc: Mr. B. C. McLeod, P. Eng.
Regional Geotechnical & Materials Eng.



X-SECT A-A.



END VIEW.



PLAN.

BRITISH COLUMBIA DEPARTMENT OF HIGHWAYS
GEOTECHNICAL AND MATERIALS BRANCH

HALFWAY RIVER BRIDGE
SUGGESTED ICE BREAKER LAYOUT.

DRAWN KFR.	SCALE Hor/Vert 1" = 10'	DATE 18/1/1974.	
TRACED	PROJ NO	REG NO H	DRAWING NO
CHECKED	FILE NO	Index H	Neg No 90507	Sheet No 1 of 1
APPROVED	APPROVED		

D	
C	
B	
A	
	REVISIONS

Mr. C. A. Beaumont
Regional Highway Design & Survey Engineer
1600 Third Avenue
Prince George, British Columbia

Bridge Engineer

June 27, 1974

4904

Halfway River Bridge #1042

We are enclosing herewith a print of Dwg. No. 1042-25 being profiles and notes traced from the hardshell which you prepared for the above crossing.

It will be noted that the centreline of the structure now proposed is not as originally intended, and shown on this drawing, but is now upstream of the existing bridge. A subsequent linen tracing prepared in Nov. 1971 was forwarded to us. A print of this tracing, Dwg. No. 1042-19 is also enclosed.

We would appreciate your advice, as soon as possible, with regard to the maximum stream velocity. As discussed with you on June 26th we are tentatively using 25 f.p.s.

W. A. Bowman
Bridge Engineer

by:

G. S. Kirkbride
Bridge Design Engineer

GSK:agm

Enclosures

FORM H. 118

B.F.

PROVINCE OF BRITISH COLUMBIA

DEPARTMENT OF HIGHWAYS

TO:
Mr. G.S. Kirkbride
Bridge Design Engineer
Victoria, B.C.

ATTENTION:

SENDER'S ADDRESS: Prince George
DATE: July 8, 1974
DISTRICT:
HEADQUARTERS FILE:
REGIONAL FILE: L4-M68-396
DISTRICT FILE:

SUBJECT:
Halfway River

REFERENCE: DATED:

Flow on June 29, 1965 was 70,000 cfs. corresponding to 9.5 ft./sec. mean velocity. Pleak flow (100 yr.) is estimated now by frequency studies (Water Survey of Canada) to be 100,000 cfs.

If high water was 1 ft. higher the cross-section area would be about 7,800 sq. ft. and mean velocity about 13 ft./sec.

Possibly the maximum velocity at midstream would approach 15 to 17 ft./sec.

I regret that these figures are substantially lower than those shown on site plan but I did not have them at hand at the time.

L.A. Broddy,
Regional Highway Engineer

by: [signature]
for E.A. Beaumont,
Regional Design and Surveys Engineer

EAB/1k

FOR DEPARTMENTAL CORRESPONDENCE ONLY.

FORM H. 118

~~465~~

BF

PROVINCE OF BRITISH COLUMBIA

DEPARTMENT OF HIGHWAYS

TO:

J. Alton,
Sr. Bridge Engineer,
Department of Highways,
Victoria, B.C.

ATTENTION:

SENDER'S ADDRESS: E.E. Readshaw, Sr. Materials Engineer, Victoria, B.C.

DATE: March 28, 1966.

ELECTORAL DISTRICT:

HEADQUARTERS FILE: M-690

REGIONAL FILE: Proj. 540

DISTRICT FILE:

SUBJECT:

Halfway River Bridge - Foundation Study

REFERENCE:

DATED:

Attached are the recommendations for foundations at this site. The influence of the Peace River Dam upon this site has not been considered in the scour portion.

E.B. Wilkins,
Design and Planning Engineer.

E E Readshaw

E.E. Readshaw,
Sr. Materials Engineer.

JHv/AFB/ek

2-cc -Bridge Engineer
cc - E.B. Wilkins
cc - N.R. Zapf
cc - R.G. Harvey, Prince George
cc - E.A. Lund, Prince George

FOR DEPARTMENTAL CORRESPONDENCE ONLY.

HALFWAY RIVER BRIDGE

Introduction

Originally this site was drilled at the request of Region 4 (30 August, 1965). A formal request for further drilling was received on February 11, 1966 from the Senior Bridge Engineer. The second stage of drilling was completed on March 4, 1966. The attached plan indicates the main details of the site and proposed structure.

Geology

The following is based on the cored bedrock from Test Hole No.6 but should be applicable to the bedrock underlying the entire site. A suggestion is that this material tends to part on bedding planes 2"-4" apart. It is sound shale rock with an allowable bearing load of 8 tons/sq.ft. There is very little sign of its being a "swelling shale", it is more of a cementation shale.

Such permeability as the rock possesses will be due to cracks and joint systems. The rock itself is impermeable.

This rock will only with difficulty be rippable - its rippability decreases with depth.

It should make a good foundation. To ensure stability (ice shove, etc.) set the central pier in an excavation in the shale rock 2' ± deep.

Foundation Considerations

To avoid confusion the logs of test holes 1 and 3 are not shown on the profile but are shown on log sheets attached to this report. The tentative bridge design indicated has one river pier and a support on each bank. The river pier which is to support a load of some 1125 tons, can be constructed on bedrock directly. To resist possible lateral forces (ice, driftwood) it is recommended that the pier base (or spread footing) be recessed at least 3 feet into the solid bedrock, as determined during excavation at time of construction. As stated previously, a vertical safe bearing capacity on solid rock of 8 tons per square foot can be utilized.

There is no reason to believe that the surface of the bedrock shale is other than flat. On this assumption, the west abutment can be constructed on piles driven to elevation 1400 ± 5 feet. The use of steel H piles is advocated. The steel H piles (12"x12" @ 54 lb.) can be loaded safely to 70 tons each; if the piles are properly seated on the solid bedrock, and if care is taken to ensure no rebound occurs during driving of adjacent piles. There are several other types of point bearing piles that can be used at this site. This Branch would welcome the opportunity of discussing any other types of piles that you may wish to utilize.

The east abutment can utilize the same type of foundation (steel H piles seated on bedrock at elevation 1410 ± 5 feet).

If desired both the abutments could be placed on spread footings founded on the solid bedrock but this would likely involve some 25 feet of excavation below present surface.

Scour

The estimated river velocity of 25 miles per hour is open to strong doubt. If the foundations are placed on solid bedrock river scour will not affect them. The west abutment is affected both by scour from the Halfway River and from the Peace River, dependent on the time of year.

The effect of the Peace River on the east abutment is unknown. For the present hydraulic river conditions, the existing fill scour-protection and its effectiveness should yield an answer on what is necessary to protect the new approach fills.

Conclusion

No soil problems are foreseen on the basis of present drilling.

H. 222 O

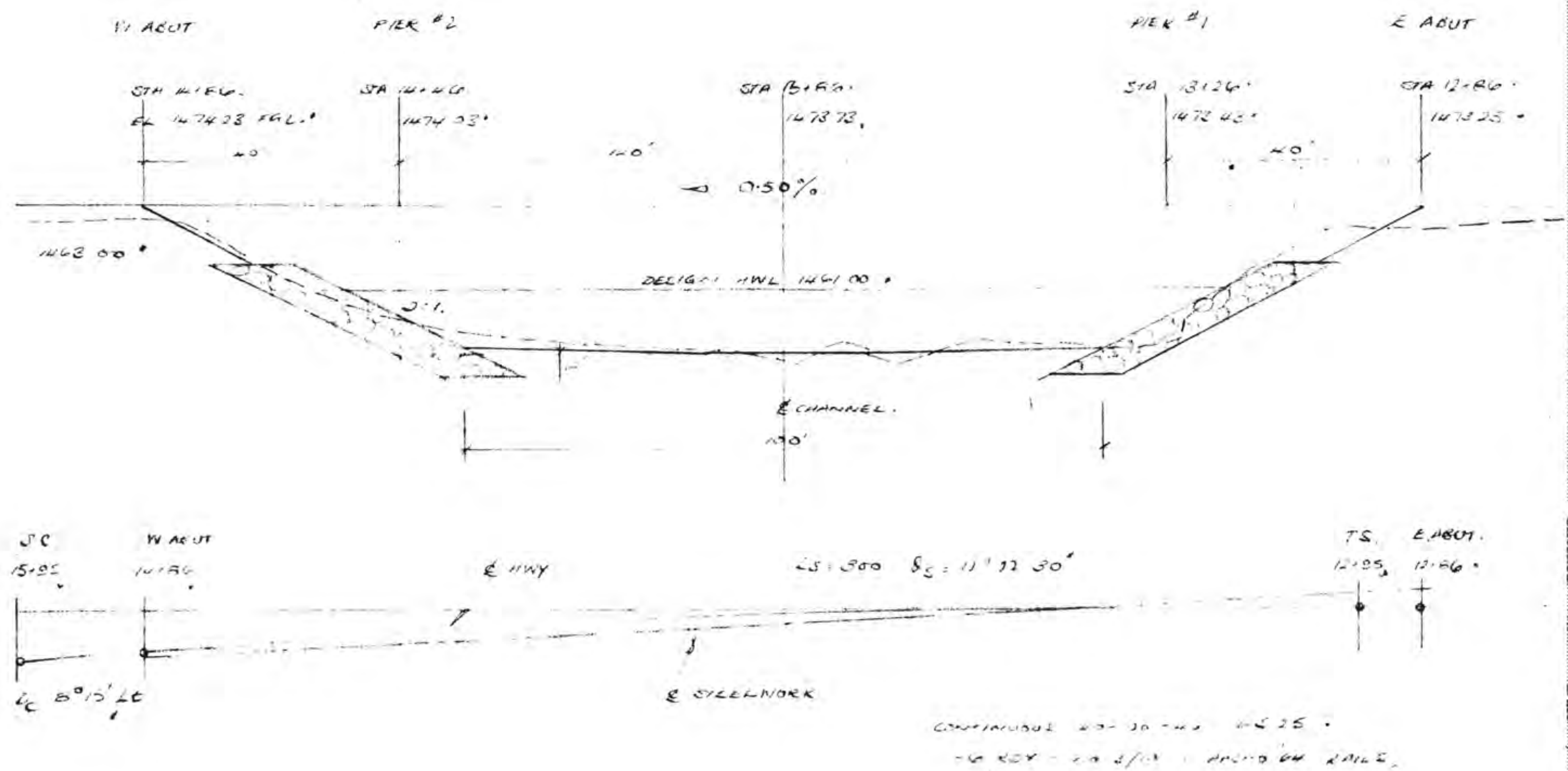
CALCULATIONS

Name of work: HALFWAY RIVER BRIDGE

Made: P.B. Date: JULY 69

Checked: Date:

Sheet No. 1



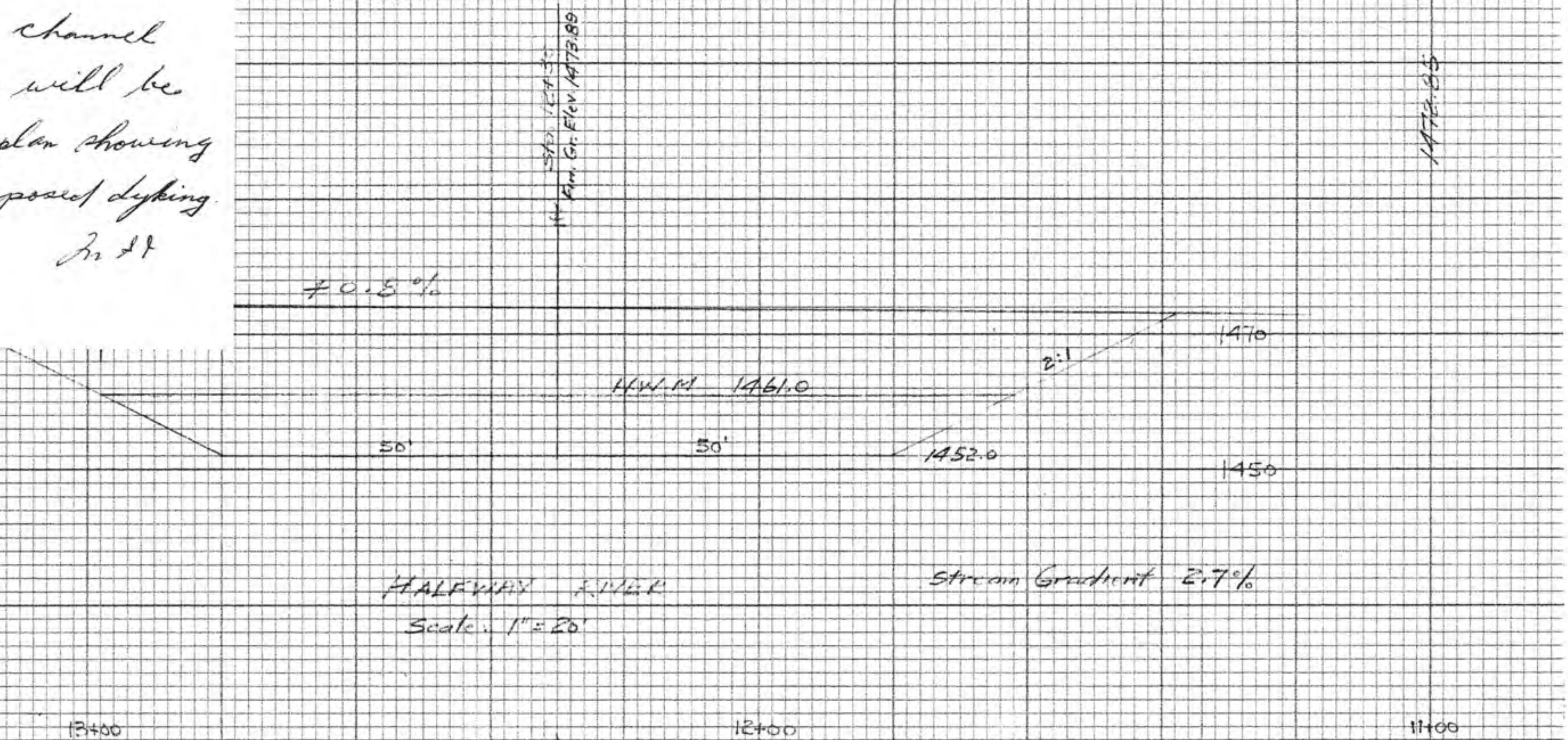
OFFICE OF DIRECTOR OF LOCATION

Mr. L. C. Johnson
Bridge Design Eng.

Halfway River channel
...n attached. We will be
...bmitting the site plan showing
...data and the proposed dyking.

M. H.

166-1586 (4)

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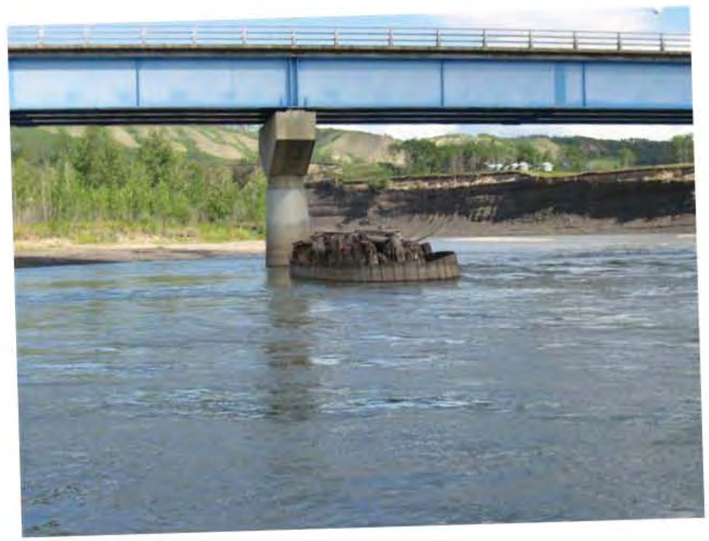



s22
19/04/2012


19/04/2012


19/04/2012

s22


19/04/2012


1042-75
19/04/2012

From: Sturrock, Ian F TRAN:EX
Sent: Thursday, September 10, 2009 8:42 AM
To: Odowichuk, Mike W TRAN:EX
Cc: Cienciala, Ed H TRAN:EX
Subject: FW: halfway river - Nearby River Obstructions
Attachments: IMG_0841.JPG; IMG_0842.JPG; IMG_0843.JPG; IMG_0840.JPG

FYI. Our hydrographic survey contractor has pointed out some river navigation dangers adjacent to the Halfway River Bridge on Highway 29. They look like foundations from an older bridge.

From: Alex Howden [mailto:ahowden@uniserve.com]
Sent: Tuesday, September 1, 2009 7:22 PM
To: Sturrock, Ian F TRAN:EX
Subject: halfway river

Ian,

Just wanted to make you aware of some dangerous obstructions near the Ha fway River Bridge, between Hudsons Hope and Fort St. John BC.

These areas are frequented by boaters, I thought you may wish to see these.

A ex

Province of British Columbia
Ministry of Transportation and Highways

MEMORANDUM

Sharlie Huffman
Regional Bridge Engineer
Central Northeast Region
Prince George, B.C.

Bridge Engineering
4D-940 Blanshard Street
Victoria, B. C. V8W 3E6
Fax: 387-7735
Phone: 356-9862
November 4, 1996

Re : 1996 Hydrographic Surveys - Region 4

Not Responsive

Halfway River Bridge No. 1042 Date of Survey: 96/08/02

This survey done at the request of the North Peace District. The survey indicates that the scour hole at the center pier, first observed in 1994, now is slightly deeper (0.4 meters) and wider. The pier is embedded in bedrock so there is no risk of undermining. Scour of up to 2.2 meters at the east abutment was observed.

Recommendations: Inspect the bank near the east abutment at low water.

. . . /2

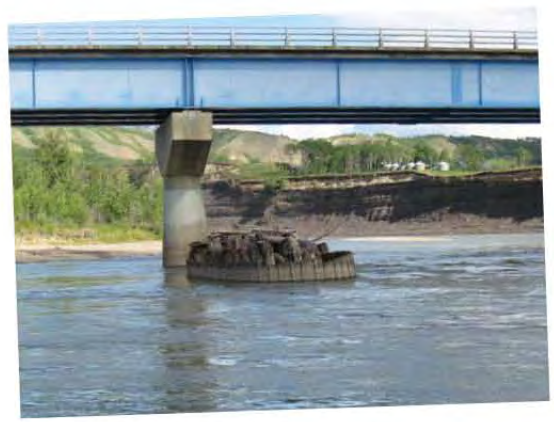

10/16/2008

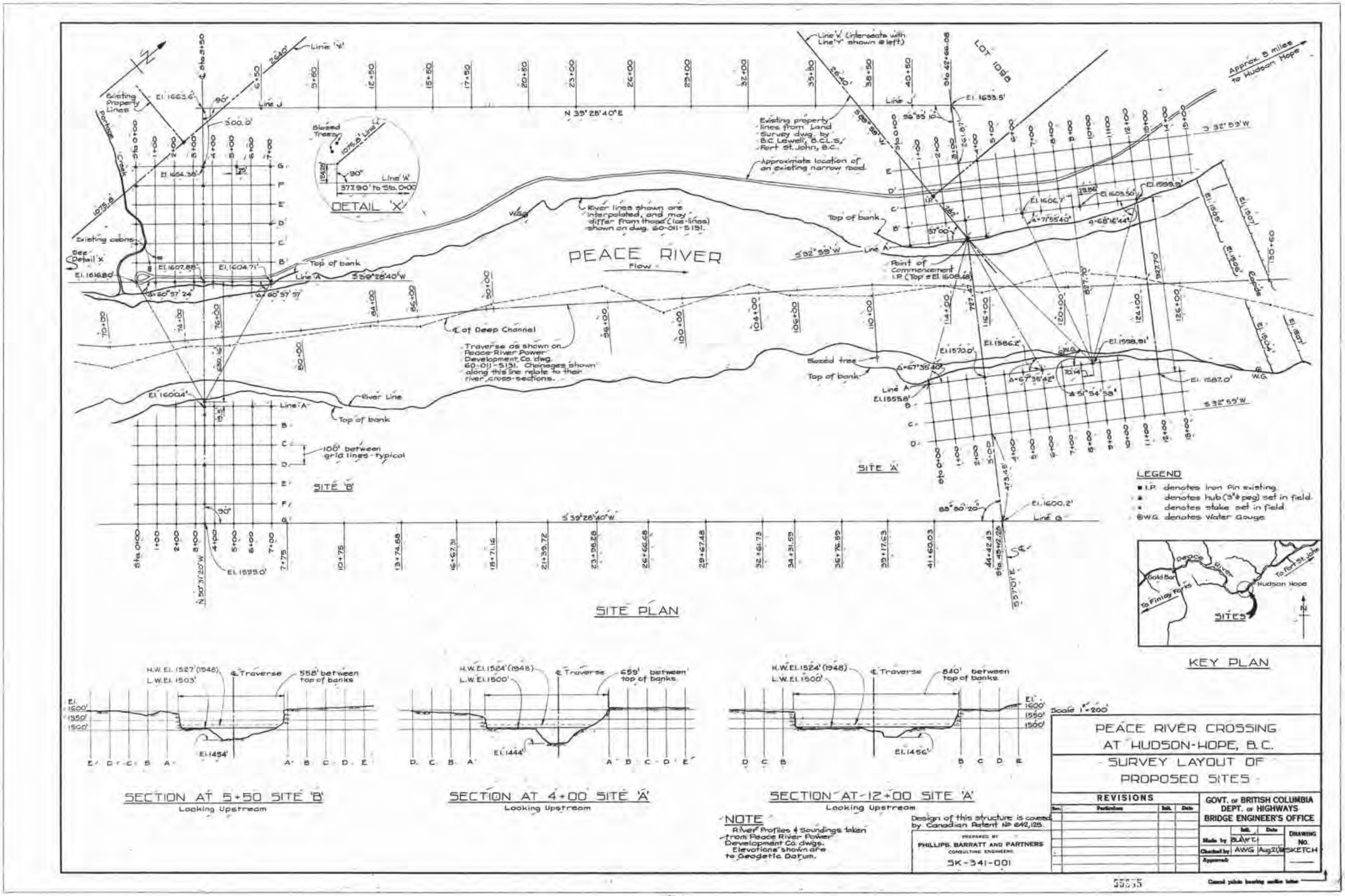

2004 10 05 15:57

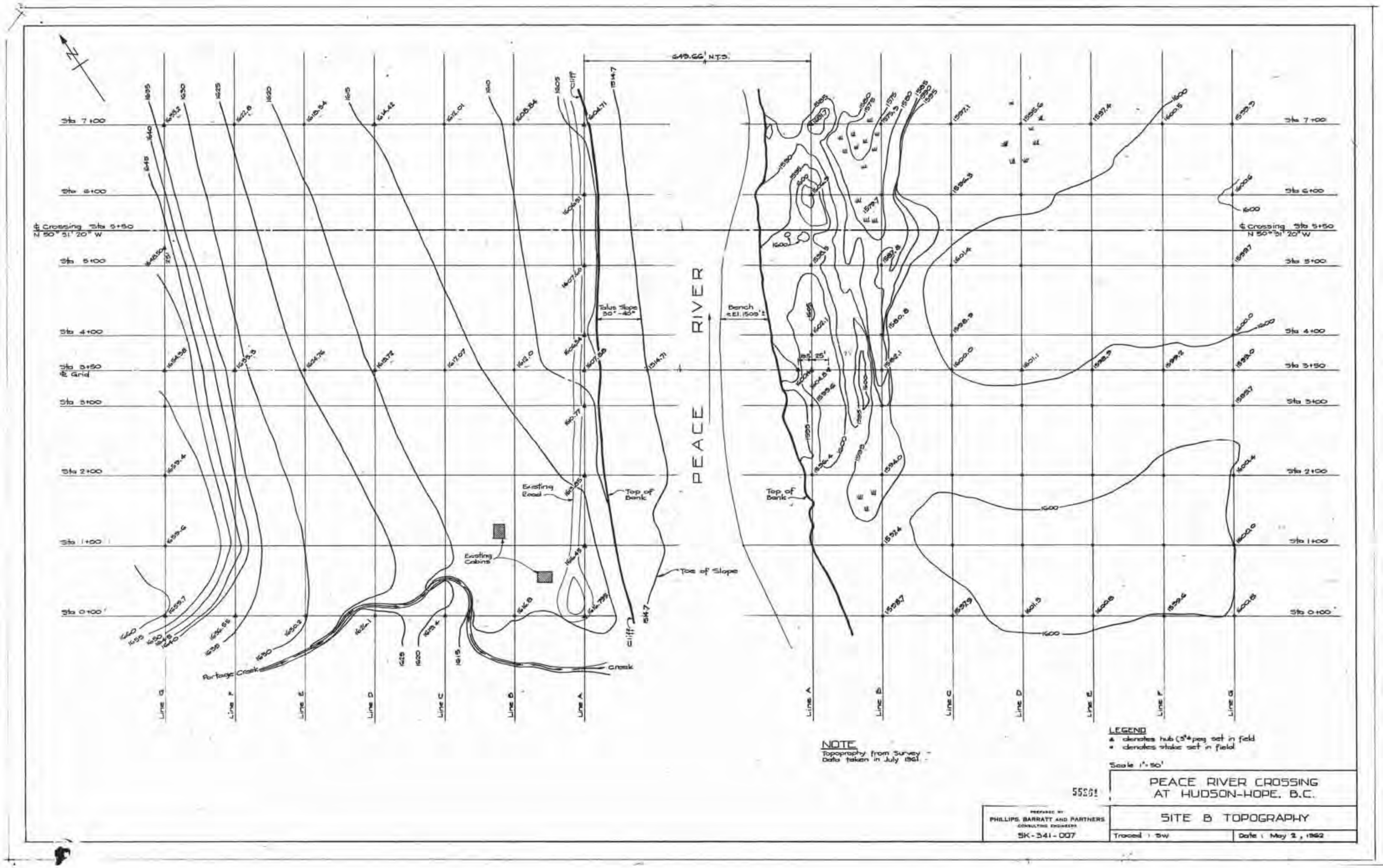

06.06.2007 15:11

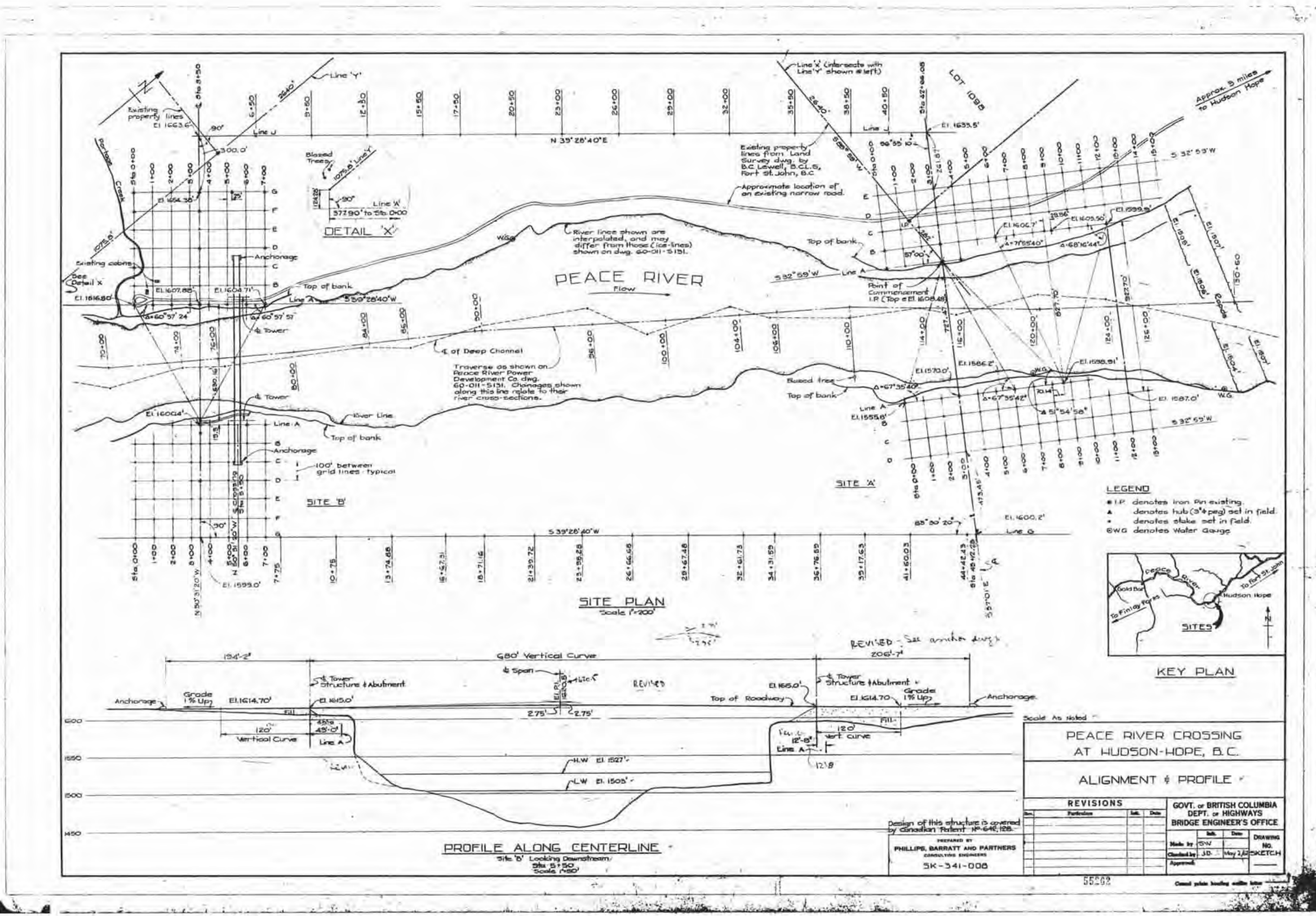

06.06.2007 15:12

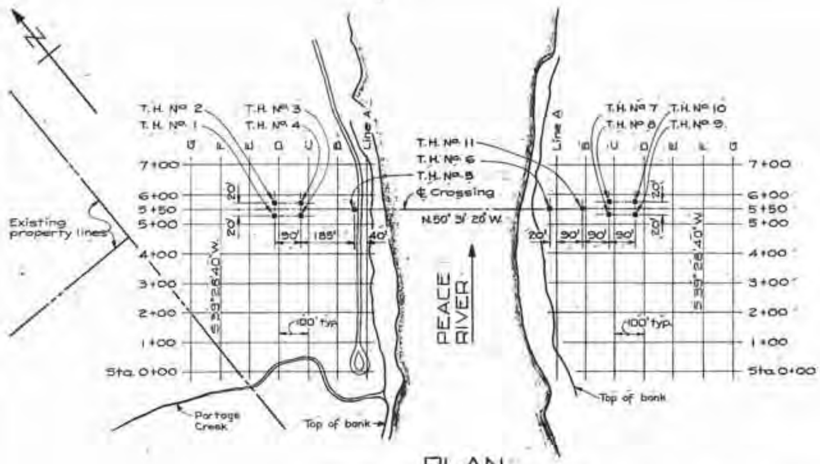

5/16/02 15:10




PEACE RIVER
Flow
SITE 'A'
SITE 'B'
DETAIL 'X'
SITE PLAN
KEY PLAN
LEGEND
I.P. denotes Iron Pin existing
denotes hub (3"ø peg) set in field
denotes stake set in field
W.G. denotes Water Gauge
Point of Commencement I.P. (Top = El. 1608.48)
Existing property lines from Land Survey dwg. by B.C. Lewell, B.C.L.S., Fort St. John, B.C.
Approximate location of an existing narrow road.
River lines shown are interpolated, and may differ from those (las-lines) shown on dwg. 60-011-5151.
Traverse as shown on Peace River Power Development Co. dwg. 60-011-5151. Chainages shown along this line relate to their river cross-sections.
℄ of Deep Channel
100' between grid lines - typical
Approx. 5 miles to Hudson Hope
LOT 1098
Peace River
Hudson Hope
To Fort St. John
To Finlay Forks
Gold Bar
SITES
SECTION AT 5+50 SITE 'B'
Looking Upstream
SECTION AT 4+00 SITE 'A'
Looking Upstream
SECTION AT 12+00 SITE 'A'
Looking Upstream
558' between top of banks
659' between top of banks
840' between top of banks
NOTE
River Profiles & Soundings taken from Peace River Power Development Co. dwgs.
Elevations shown are to Geodetic Datum.
Design of this structure is covered by Canadian Patent Nº 642,128.
PREPARED BY
PHILLIPS, BARRATT AND PARTNERS
CONSULTING ENGINEERS
SK-341-001
Scale 1" = 200'
PEACE RIVER CROSSING
AT HUDSON-HOPE, B.C.
SURVEY LAYOUT OF
PROPOSED SITES
REVISIONS
GOVT. of BRITISH COLUMBIA
DEPT. of HIGHWAYS
BRIDGE ENGINEER'S OFFICE
Made by B.A.W.T.
Checked by AWG
DRAWING No. SKETCH


PEACE RIVER
649.66' N.T.S.
Sta 7+00
Sta 6+00
¢ Crossing Sta 5+50
N 50° 51' 20" W
Sta 5+00
Sta 4+00
Sta 3+50
¢ Grid
Sta 3+00
Sta 2+00
Sta 1+00
Sta 0+00'
Talus Slope 30°-40°
Bench ±El. 1509'±
Existing Road
Existing Cabins
Top of Bank
Toe of Slope
Cliff
Portage Creek
Creek
Line G
Line F
Line E
Line D
Line C
Line B
Line A
NOTE
Topography from Survey - Data taken in July 1961 -
LEGEND
▲ denotes hub (3"x peg set in field
• denotes stake set in field
Scale 1" = 50'
PEACE RIVER CROSSING AT HUDSON-HOPE, B.C.
SITE B TOPOGRAPHY
PREPARED BY PHILLIPS, BARRATT AND PARTNERS CONSULTING ENGINEERS
SK-341-007
Traced : SW
Date : May 2, 1962


PEACE RIVER
Flow
SITE PLAN
Scale 1"=200'
DETAIL 'X'
SITE 'A'
SITE 'B'
LOT 1088
Approx. 5 miles to Hudson Hope
N 39°28'40"E
S 39°28'40"W
Line 'Y'
Line 'X' (intersects with Line 'Y' shown @ left)
Existing property lines
Existing property lines from Land Survey dwg. by B.C. Lewell, B.C.L.S., Fort St. John, B.C.
Approximate location of an existing narrow road.
River lines shown are interpolated, and may differ from those (ice-lines) shown on dwg. 60-011-5151.
Traverse as shown on Peace River Power Development Co. dwg. 60-011-5151. Chainages shown along this line relate to their river cross-sections.
℄ of Deep Channel
Top of bank
River Line
Existing cabins
Portage Creek
Blazed Trees
Blazed tree
Anchorage
℄ Tower
Point of Commencement I.P. (Top @ El. 1608.48)
100' between grid lines - typical
LEGEND
● I.P. denotes Iron Pin existing.
▲ denotes hub (3"± peg) set in field.
• denotes stake set in field.
⊕ W.G. denotes Water Gauge
KEY PLAN
Peace River
Gold Bar
To Finlay Forks
To Fort St. John
Hudson Hope
SITES
PROFILE ALONG CENTERLINE
Site 'B' Looking Downstream
Sta. 5+50
Scale 1"=50'
680' Vertical Curve
154'-2"
206'-7"
REVISED - See anchor dwgs.
℄ Span
℄ Tower Structure & Abutment
Anchorage
Grade 1% Up
El. 1614.70'
El. 1615.0'
Top of Roadway
El. 1615.0'
120' Vertical Curve
H.W. El. 1527'
L.W. El. 1503'
Line A
Fill
Scale As Noted
PEACE RIVER CROSSING
AT HUDSON-HOPE, B.C.
ALIGNMENT & PROFILE
REVISIONS
GOVT. of BRITISH COLUMBIA
DEPT. of HIGHWAYS
BRIDGE ENGINEER'S OFFICE
Design of this structure is covered by Canadian Patent No. 646,128.
PREPARED BY
PHILLIPS, BARRATT AND PARTNERS
CONSULTING ENGINEERS
SK-341-008
Drawing No. SKETCH
55262



PLAN

TEST HOLE LOCATIONS
Scale 1" = 200'

NOTE
Contractor is to satisfy himself as to the actual rock conditions. The borings are for his guidance only, see Special Provisions - "Test Borings".

DRAWING INDEX

NUMBER	TITLE
2169-2	Index, General Notes & Boring Data
2169-3	General Arrangement
2169-4	Existing Stripped Areas
2169-5	Layout - South Approaches
2169-6	Layout - Suspended Span - Sheet Nº 1
2169-7	Layout - Suspended Span - Sheet Nº 2
2169-8	Layout - Suspended Span - Sheet Nº 3
2169-9	Layout - North Approaches
2169-10	North Tower
2169-11	South Tower
2169-12	Tower Footings & Wing Walls
2169-13	Tower Cross Girder & Ladder
2169-14	Bearings
2169-15	Saddle Details
2169-16	South Anchorage & Housing
2169-17	North Anchorage & Housing
2169-18	Anchor Plates & Splay Collars
2169-19	Cable Rock-Anchorage
2169-20	Suspension Cables - Details
2169-21	Roadway Expansion Joints
2169-22	Precast Concrete Deck Units - Sh. Nº 1
2169-23	Precast Concrete Deck Units - Sh. Nº 2
2169-24	Deck Post-Tensioning - Sheet Nº 1
2169-25	Deck Post-Tensioning - Sheet Nº 2
2169-26	Camber & Load Diagram
2169-27	Miscellaneous Details
1810-7	Standard Roadway Fence

A

2169-45	Sculpture Bases
2169-47	Concrete Roadway Pavings
2169-48	Plaque Details
2169-49	Inventors Plaque

GENERAL NOTES

BASIS OF DESIGN

The basis of design has been the 1961 Standard Specifications for Highway Bridges of the American Association of State Highway Officials where it applies. The prestressed concrete segmental deck units have been designed in accordance with the "Tentative Recommendations for Prestressed Concrete" issued by the A.C.I.-A.S.C.E. Joint Committee 323"

DESIGN LIVE LOAD

1. Floor Slab, Cross Girders and Suspenders - H20-S16 + 30% impact.
2. Main Cables, Towers, Stiffening Girder and Anchorages - H20-S16 with no impact.

L.L. for each main cable:

Uniform load = 0.7k per lineal foot plus
Concentrated load = 19.8k for moment or
= 28.4k for shear.

DESIGN TEMPERATURE VARIATION

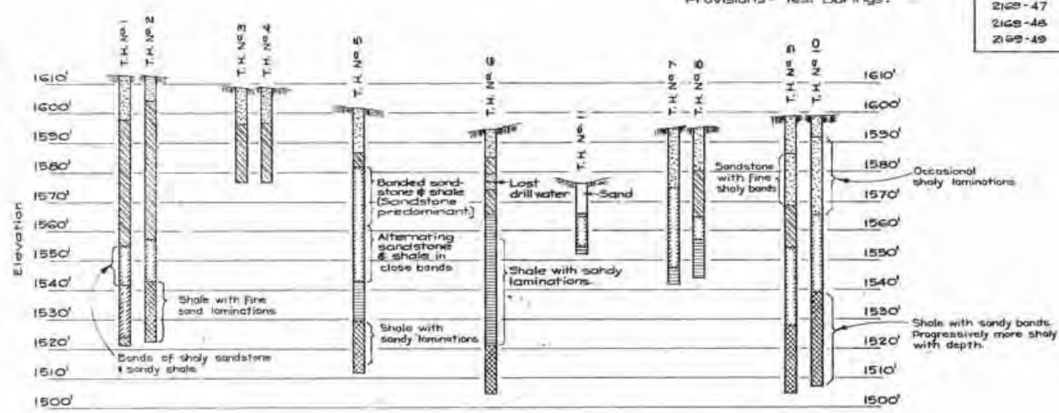
Mean Temperature = 30°F.
Maximum Temperature = 110°F.
Minimum Temperature = 50°F.

DESIGN THEORY FOR STIFFENING GIRDER

Deflection Theory for Suspension Bridges.

MAXIMUM CHANGE OF GRADE IN STIFFENING GIRDER

2.1% - (at tower - under L.L. condition as in 2 above and a temperature of + 110°F.)



LEGEND
- Sandstone
- Shaly sandstone
- Transition from shaly sandstone to sandy shale
- Sandy shale
- Shale

TEST HOLE BORINGS
Vert. Scale 1" = 20'

BORING DATA

NOTE
Test hole borings by "Materials Testing, Design & Planning Branch", Dept. of Hwys., Victoria B.C. in May, 1962.

GENERAL ARRANGEMENT APPROVED
CHIEF ENGINEER, DEPT. OF HIGHWAYS

Design of this structure is covered by Canadian Patent Nº 642,128

PREPARED BY
PHILLIPS, BARRATT AND PARTNERS
CONSULTING ENGINEERS
341-101

Scale As Noted

PEACE RIVER ELECTORAL DISTRICT
CHETWYND-HUDSON HOPE HIGHWAY
PEACE RIVER BRIDGE
AT HUDSON HOPE
INDEX, GEN. NOTES & BORING DATA

REVISIONS

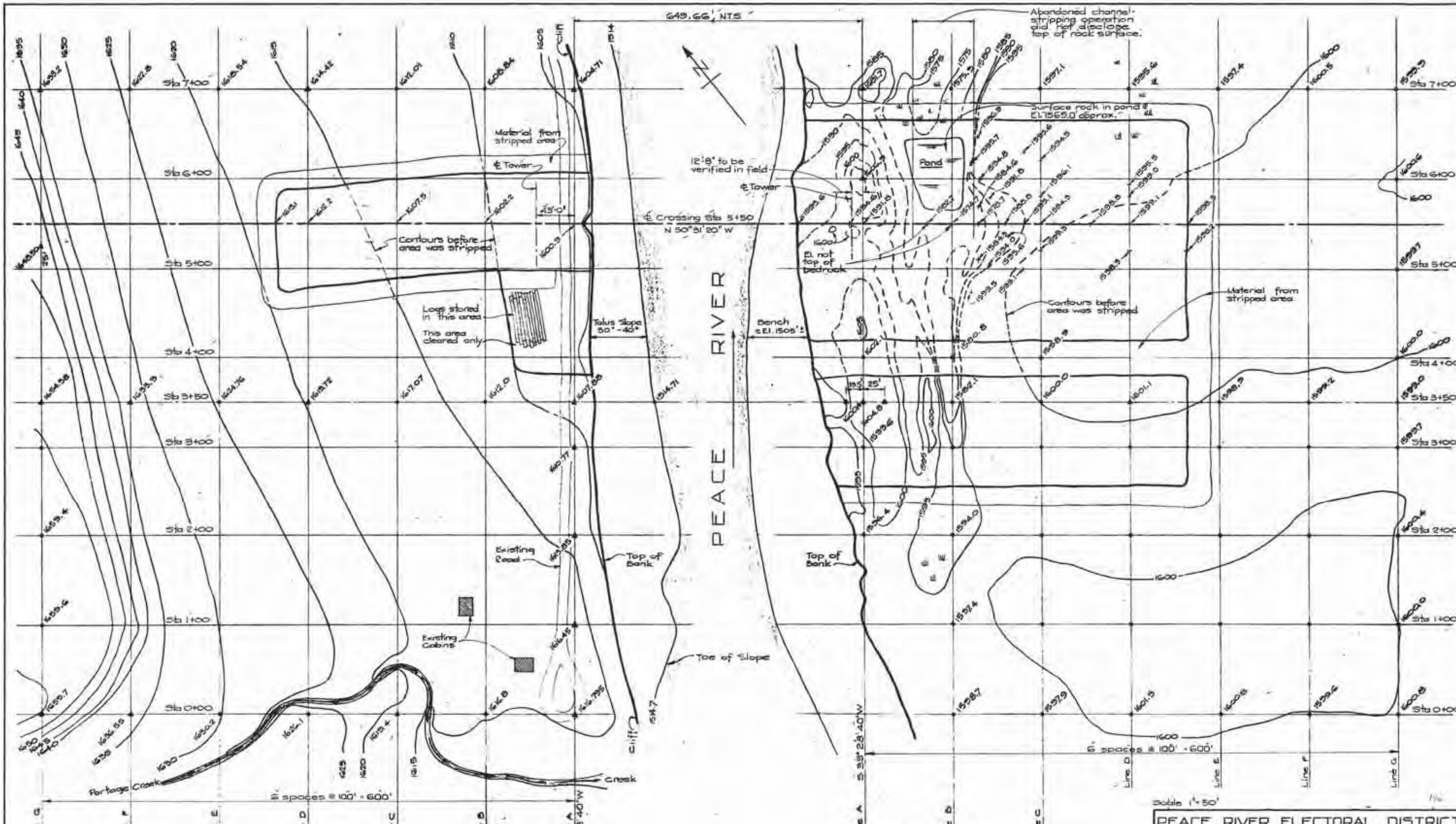
Rev.	Particulars	Init.	Date
A	Drawings added to Index	BLA	July/65

GOVT. OF BRITISH COLUMBIA
DEPT. OF HIGHWAYS
BRIDGE ENGINEER'S OFFICE

	Init.	Date
Made by	V.T.	
Checked by	R.A.	
Approved		

DRAWING No. BRIDGES 2169-2A

55267



NOTE

Topography from Survey Data taken in July 1961.
Elevations shown with arrow are to top of bedrock: i.e. 1584.6'→+
Elevations shown in stripped area are to top of bedrock as taken in May 1962.
The Contractor shall satisfy himself before tendering that conditions at the site as shown have not changed.

LEGEND

▲ denotes hub (3"4 peg set in field)
• denotes stake set in field

GENERAL ARRANGEMENT APPROVED

CHIEF ENGINEER, DEPT. OF HIGHWAYS

Design of this structure is covered by Canadian Patent No. 642,128.

PREPARED BY
PHILLIPS, BARRATT AND PARTNERS
CONSULTING ENGINEERS
341-103

PEACE RIVER ELECTORAL DISTRICT
CHETWYND-HUDSON HOPE HIGHWAY

PEACE RIVER BRIDGE AT HUDSON HOPE

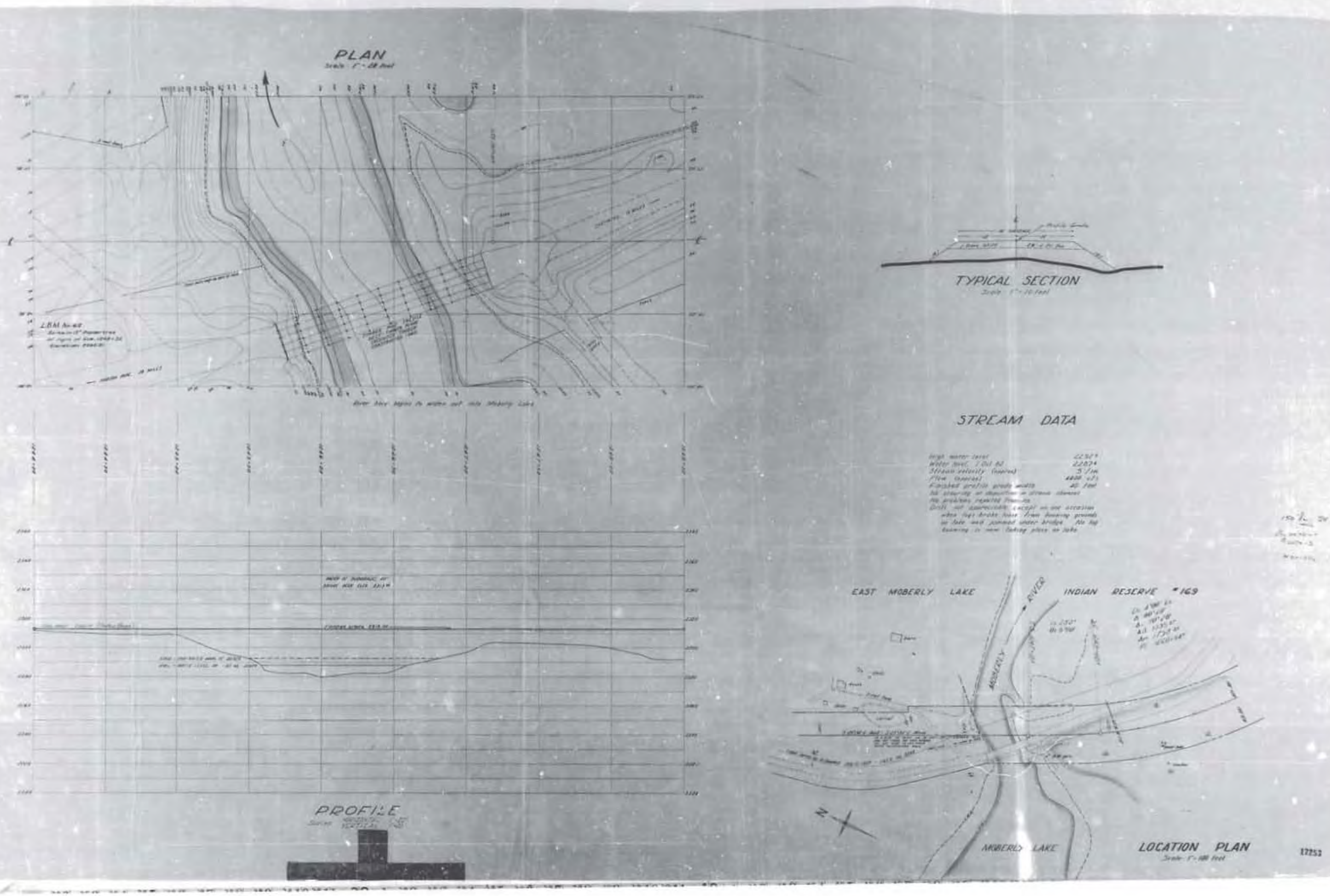
EXISTING STRIPPED AREAS

REVISIONS			
No.	Particulars	Init.	Date

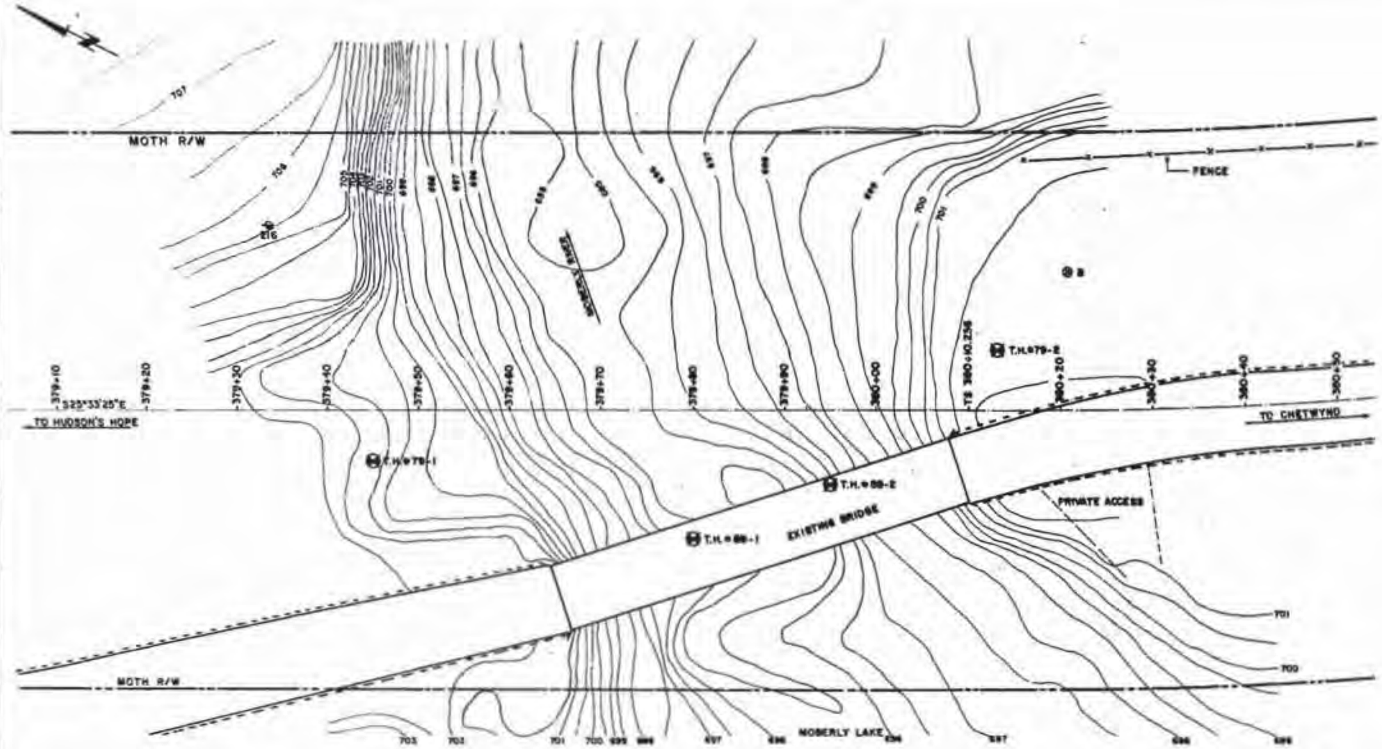
GOVT. OF BRITISH COLUMBIA
DEPT. OF HIGHWAYS
BRIDGE ENGINEER'S OFFICE

	Init.	Date	DRAWING No.
Made by	SW		BRIDGES
Checked by	J.D. E.R.A.	June 6 1962	2169-4
Approved			

55369

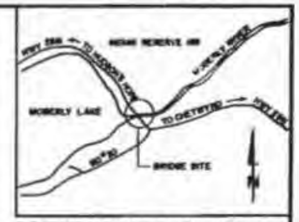

PLAN
TYPICAL SECTION
STREAM DATA
EAST MOBERLY LAKE
RIVER
INDIAN RESERVE #169
MOBERLY
PROFILE
MOBERLY LAKE
LOCATION PLAN
17253

[redacted]



SITE PLAN

1:250



THE BRIDGE SITE IS APPROX 20km NORTH OF CHETWYND B.C.

KEY MAP

1:50,000

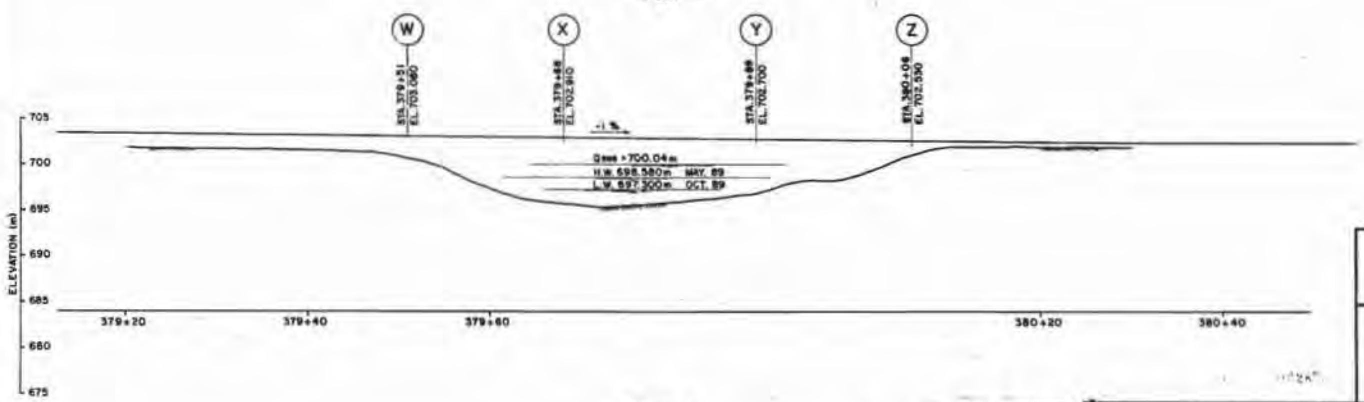
DRAWING LIST	
DRAWING NUMBER	TITLE
1176-1	SITE PLAN
1176-2	TEST HOLE LOGS SHEET 1
1176-3	TEST HOLE LOGS SHEET 2
1176-4	GENERAL ARRANGEMENT
1176-5	ABUTMENTS
1176-6	PIERS
1176-7	STRINGERS
1176-8	DECK DETAILS
1176-9	MISCELLANEOUS DETAILS
1176-10	BRIDGE APPROACHES PLAN
1176-11	PROFILE AND TYPICAL SECTIONS
2784-1K	STANDARD BRIDGE PARAPET
2784-2D	STANDARD PARAPET TRANSITION
2785-1K	STANDARD PARAPET STEEL RAILING
2891-1E	STANDARD STEEL SIDEWALK FENCE

NOTES:

1. SURVEY: M.O.T.H. SOUTH PEACE DISTRICT
2. BENCH MARK:
   GEODETIC B.M. # 1943J, ELEV. 707.453m (882+2321.040')
   STA. 400+66.9, 25.2m LT OF ℄
   *B.M. #1, ELEV. 704.144
   STA. 376+55, RT OF ℄, SPIKE IN HYDRO POLE
   *B.M. #2, ELEV. 701.712
   STA. 380+10, LT OF ℄, SPIKE IN POST AT END OF FENCE

* TO BE VERIFIED BY CONTRACTOR PRIOR TO USE

352875



PROFILE

1:250

ASSOCIATED ENGINEERING

GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH

SOUTH PEACE HIGHWAY DISTRICT
CHETWYND TO HUDSON'S HOPE HIGHWAY NO. 29 NORTH
MOBERLY BRIDGE
SITE PLAN

D			
C			
B			
A	AS BUILT	D.I.H.	11/91
	REVISIONS		

SCALE: AS NOTED | REG. No. 352875 | DRAWN C.C. | MAR. 90 | DRAWING No. 1176-1 | A

TEST HOLE LOG

Project: MOBERLY CREEK BRIDGE No. 1176 — Test Hole No. 79-1 — Sheet 01 of 02

TEST HOLE LOG

Project: MOBERLY CREEK BRIDGE No. 1176 — Test Hole No. 79-2 — Sheet 01 of 02

TEST HOLE LOG

Project: MOBERLY CREEK BRIDGE No. 1176 — Test Hole No. 79-1 — Sheet 02 of 02

TEST HOLE LOG

Project: MOBERLY CREEK BRIDGE No. 1176 — Test Hole No. 79-2 — Sheet 02 of 02

MATERIALS CLASSIFICATION LEGEND

MAJOR DIVISIONS		SYMBOL	SOIL TYPE
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
		GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
		GM*	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		GC*	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	SW	WELL-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
		SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
		SM*	SILTY SANDS SAND-SILT MIXTURES
		SC*	CLAYEY SANDS SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS W_L < 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS W_L > 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, PLASTIC SILTS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS
TOPSOIL		TS	TOPSOIL WITH ROOTS, ETC.
COBBLES		SB	ROCK FRAGMENTS AND COBBLES, PARTICLE SIZE 75mm TO 300mm
BOULDERS		LB	BOULDERS, PARTICLE SIZE OVER 300mm
BEDROCK		BR	BEDROCK

FOR SOILS HAVING 5 – 12% PASSING .075 SIEVE, USE DUAL SYMBOL

*GM1; GC1; SM1; SC1; 12 – 20%
GM2; GC2; SM2; SC2; 20 – 30%
GM3; GC3; SM3; SC3; 30 – 40%
GM4; GC4; SM4; SC4; 40 – 50% } PASSING .075mm SIEVE

REV. 89-12-01

GENERAL NOTES

NOTE 1) The soil and groundwater conditions shown are representative of the test hole locations only. Conditions encountered during construction may vary. Cobble and boulder sizes are driller's estimate of the dimension encountered.

NOTE 2) Field logs and laboratory logs are available for viewing at Geotechnical and Materials Engineering in Victoria. (Phone No. 387-1881)

NOTE 3) Gradation and classification are based on a visual estimate unless otherwise noted under "other tests".

NOTE 4) BR – Bedrock
Intact Rock Strength (See Canadian Foundation Manual, 2nd Edition, page 35.)

REVISIONS		
Date	Description	Initials

PROVINCE OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL AND MATERIALS ENGINEERING

SOUTH PEACE DISTRICT, HIGHWAY No. 29
MOBERLY BRIDGE No. 1176
TEST HOLE LOGS
CHETWYND TO HUDSON'S HOPE 362876

PREPARED UNDER THE DIRECTION OF | RECOMMENDED | ACCEPTED FOR CONSTRUCTION

SCALE 1:100 — NEGATIVE No. 92,711 — DRAWING No. 1176-2 — FILE No. 01-C-03

362876

TEST HOLE LOG

Project: MOBERLY BRIDGE No. 1176

Sheet 01 of 02

TEST HOLE LOG

Project: MOBERLY BRIDGE No. 1176

Sheet 01 of 02

TEST HOLE LOG

Project: MOBERLY BRIDGE No. 1176

Sheet 02 of 02

TEST HOLE LOG

Project: MOBERLY BRIDGE No. 1176

Sheet 02 of 02

MATERIALS CLASSIFICATION LEGEND

MAJOR DIVISIONS		SYMBOL	SOIL TYPE
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
		GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
		GM*	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		GC*	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	SW	WELL-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
		SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
		SM*	SILTY SANDS SAND-SILT MIXTURES
		SC*	CLAYEY SANDS SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS W_L < 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS W_L > 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, PLASTIC SILTS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS
TOPSOIL		TS	TOPSOIL, WITH ROOTS, ETC.
COBBLES		SB	ROCK FRAGMENTS AND COBBLES, PARTICLE SIZE 75mm TO 300mm
BOULDERS		LB	BOULDERS, PARTICLE SIZE OVER 300mm
BEDROCK		BR	BEDROCK

FOR SOILS HAVING 5 – 12% PASSING .075 SIEVE, USE DUAL SYMBOL

*GM1; GC1; SM1; SC1; 12 – 20%
GM2; GC2; SM2; SC2; 20 – 30%
GM3; GC3; SM3; SC3; 30 – 40%
GM4; GC4; SM4; SC4; 40 – 50%
} PASSING .075mm SIEVE

REV. 88-12-01

GENERAL NOTES

NOTE 1) The soil and groundwater conditions shown are representative of the test hole locations only. Conditions encountered during construction may vary. Cobble and boulder sizes are driller's estimate of the dimension encountered.

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NOTE 4) BR – Bedrock
Intact Rock Strength (See Canadian Foundation Manual, 2nd Edition, page 35.)

REVISIONS

Date	Description	Initials

PROVINCE OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL AND MATERIALS ENGINEERING

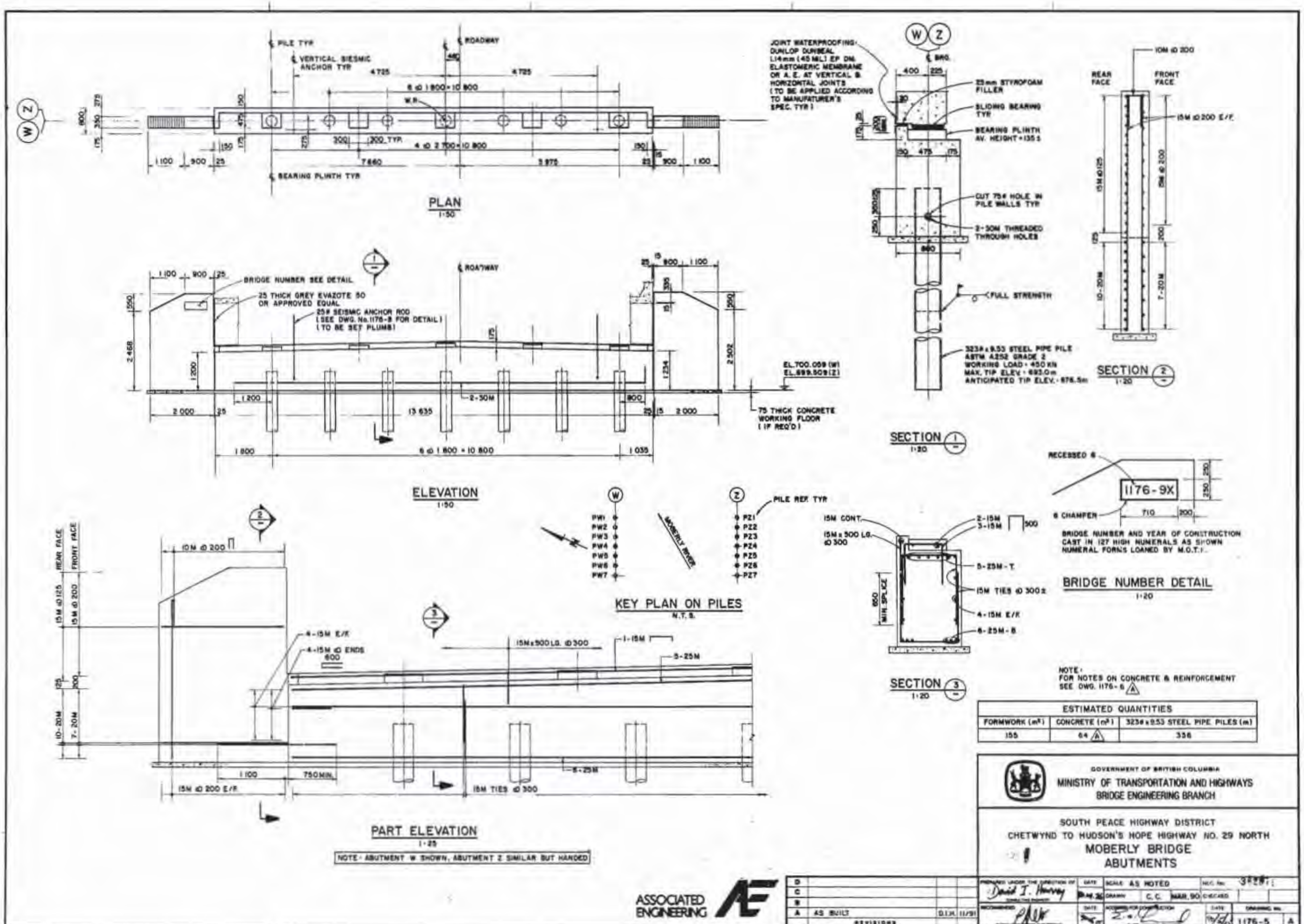
SOUTH PEACE DISTRICT, HIGHWAY No. 29
MOBERLY BRIDGE No. 1176
TEST HOLE LOGS
CHETWYND TO HUDSON'S HOPE

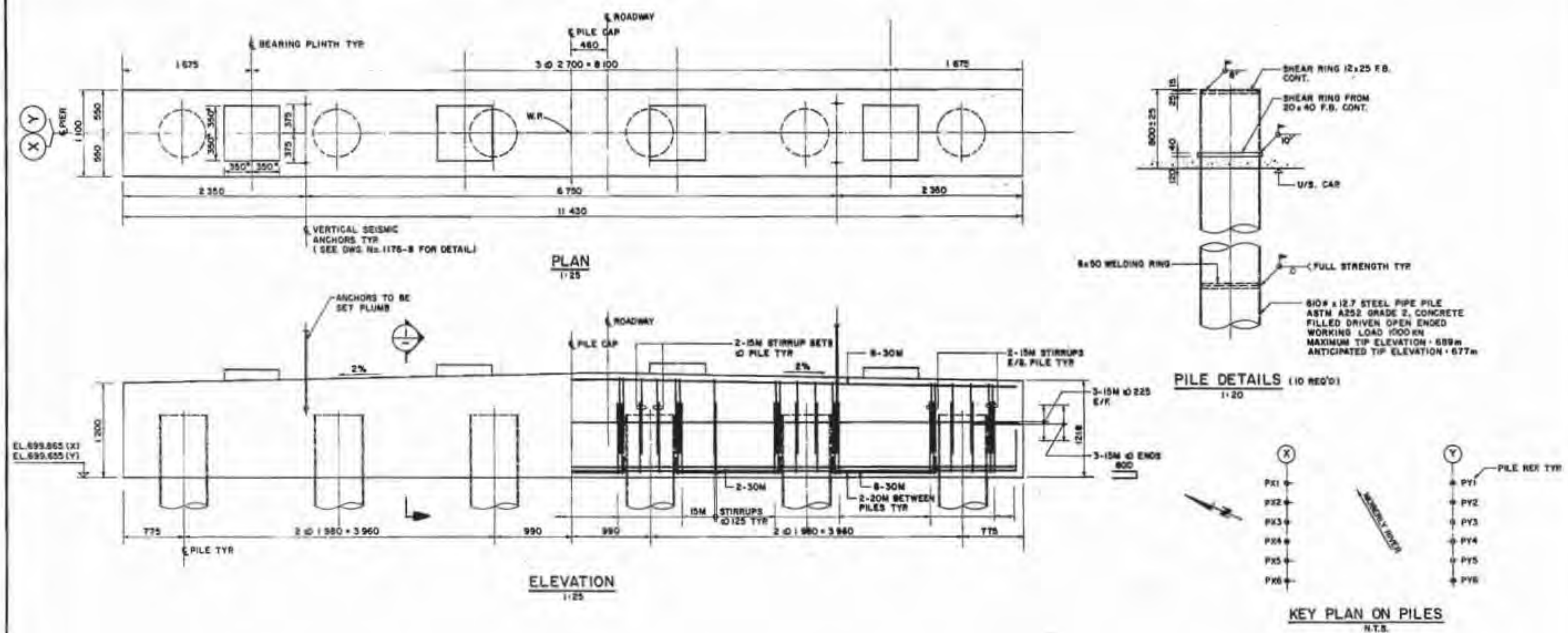
372279

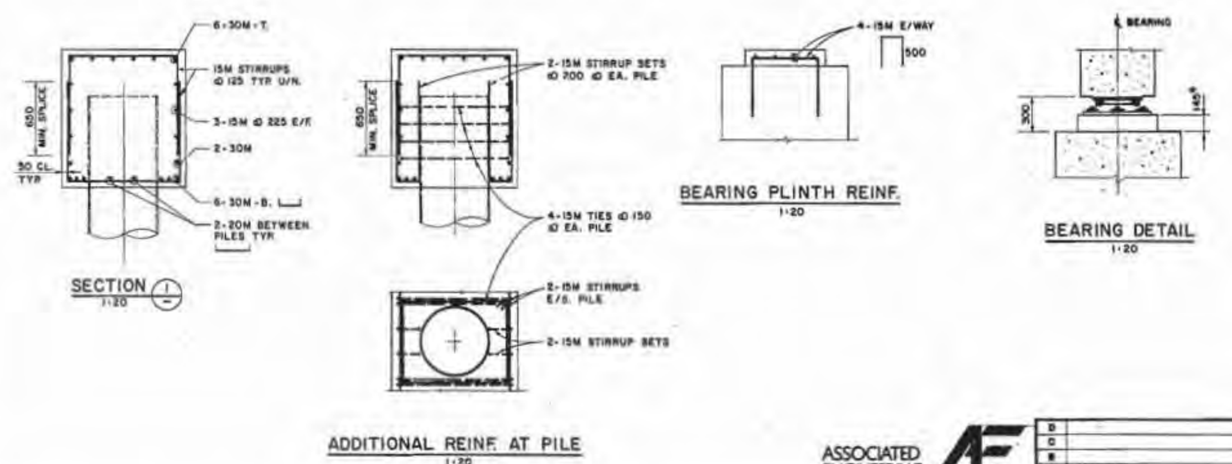
SCALE 1:100

92,712

1176-3


PLAN
1:50
ELEVATION
1:50
PART ELEVATION
1:25
NOTE: ABUTMENT W SHOWN, ABUTMENT Z SIMILAR BUT HANDED
KEY PLAN ON PILES
N.T.S.
SECTION 1
1:20
SECTION 2
1:20
SECTION 3
1:20
BRIDGE NUMBER DETAIL
1:20
1176-9X
BRIDGE NUMBER AND YEAR OF CONSTRUCTION CAST IN 127 HIGH NUMERALS AS SHOWN NUMERAL FORMS LOANED BY M.O.T.H.
NOTE:
FOR NOTES ON CONCRETE & REINFORCEMENT SEE DWG. 1176-6
ESTIMATED QUANTITIES
FORMWORK (m²) 155
CONCRETE (m³) 64
323ø x 9.53 STEEL PIPE PILES (m) 336
GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH
SOUTH PEACE HIGHWAY DISTRICT
CHETWYND TO HUDSON'S HOPE HIGHWAY NO. 29 NORTH
MOBERLY BRIDGE
ABUTMENTS
SCALE AS NOTED
1176-5
A AS BUILT
REVISIONS
ASSOCIATED ENGINEERING





NOTES:

1. CONCRETE: f'c = 30MPa @ 28 DAYS
2. REINFORCING STEEL: CSA G30-12M GRADE 400
3. COVER: 50mm TO MAIN REINFORCING U/N.
4. SPLICE: 40 x BAR DIAMETER MINIMUM, STAGGERED U/N.

* ASSUMED DIMENSION, VARY TO SUIT DESIGN OF BEARING AS REQUIRED

ESTIMATED QUANTITIES		
FORMWORK (m²)	CONCRETE (m³)	610ø x 12.7 STEEL PIPE PILES (m)
90	107 A	288

ASSOCIATED ENGINEERING

GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH

SOUTH PEACE HIGHWAY DISTRICT
CHETWYND TO HUDSON'S HOPE HIGHWAY NO. 29 NORTH
MOBERLY BRIDGE
PIERS

3528

REV.	DESCRIPTION	BY	DATE
D			
C			
B			
A	AS BUILT	D.I.H.	11/91
REVISIONS			

PREPARED UNDER THE DIRECTION OF: David I. Harvey
SCALE: AS NOTED
DRAWN: C. C. MAR. 90
DWG No.: 3528
RECOMMENDED
ACCEPTED FOR CONSTRUCTION
DRAWING No.: 1176-6 A

Province of
British Columbia

Ministry of
Transportation
and Highways

940 Blanshard Street
Victoria
British Columbia
V8W 3E6

Fax: (604)387-7735
January 16, 1990
Bridge Eng.
387-5377

Associated Engineering (B.C.) Ltd.
300-4940 Canada Way
Suite 300
Vancouver, British Columbia
V5G 4M5

Attention: Mr. David Harvey, P. Eng.

Dear Sirs:

Re: Moberly Bridge No. 1176

Enclosed is a copy of hydrology assessment of the above bridge site. Please send four copies of site plan and general arrangement to this office for fishery approval.

W. Szto
Assistant Consultant Liaison Engineer

WS/lo
Attachment

cc: Mr. R. W. Mathieson, Consultant Liaison Engineer
cc: Mr. J. H. Morley, Senior Hydraulic Engineer (+) Please review and comment.
cc: Miss Angela Abrams, Environmental Coordinator (+)

Province of
British Columbia

Ministry of
Environment

Water Management Branch
1011 Fourth Avenue
Prince George
British Columbia
V2L 3H9
Telephone (604) 565-6160

File: 55.5020
Moberly Lake

January 8, 1990

Ministry of Transportation
and Highways
213 - 1011 Fourth Avenue
Prince George, British Columbia
V2L 3H9

Attention: Ms. Sharlie Huffman, P.Eng.
Regional Bridge Engineer

Dear Ms. Huffman:

Re: Hydrology Assessment of Proposed Moberly Lake Bridge

I am responding to your September 20, 1989 request for a hydrology assessment of Moberly Lake/River in regards to the proposed Highway 29 bridge crossing. I must apologize for the delay in my response, resulting from other workload constraints.

After reviewing the site it was determined that the 1 in 200 year flood event would be appropriate in the design of this structure. This somewhat conservative level is required for this bridge due to the large number of private residences located on Moberly Lake, which could be affected by a flow restriction in the outlet of the lake.

Flood flow estimates (1 in 200 yr.) were made for the various watercourses within the Moberly Lake drainage and the resultant flows were routed through the lake. This produced a maximum lake level of 700.04 m G.S.C., based on an estimated stage/discharge relationship for the Moberly River at the outlet of the lake. This figure appears to be reasonable when compared to the measured normal high water level (natural boundary) at the site of 698.47 m G.S.C.

The normal Ministry of Environment practice is to require a 1.5 m freeboard allowance above the designated design level. In this case, I would recommend that this be reduced to 1.0 m due to the low velocities and reduced debris load of the system. This would require the bottom of the bridge beams to be placed no lower than 701.04 m G.S.C.

.../2

It should be noted that design flood is anticipated to take place in late June or early July. Since this event occurs after the normal ice breakup period, the flood threat is not expected to increase as a result of ice jamming at the bridge. However, during ice breakup it is not uncommon for ice to pile up at one end of Moberly Lake. While this phenomena, which generally occurs during May, is not expected to cause flooding, it may result in damage to the bridge structure. I would urge you to account for these forces in your design.

As I discussed with you, it is my intention to install a lake level gauge on the new bridge. I will contact you at a later date for assistance with this.

It is my understanding that your office has been in contact with Mr. Mike Lambert, in our Fort St. John office, regarding approval for instream work, and I encourage you to pursue this with him.

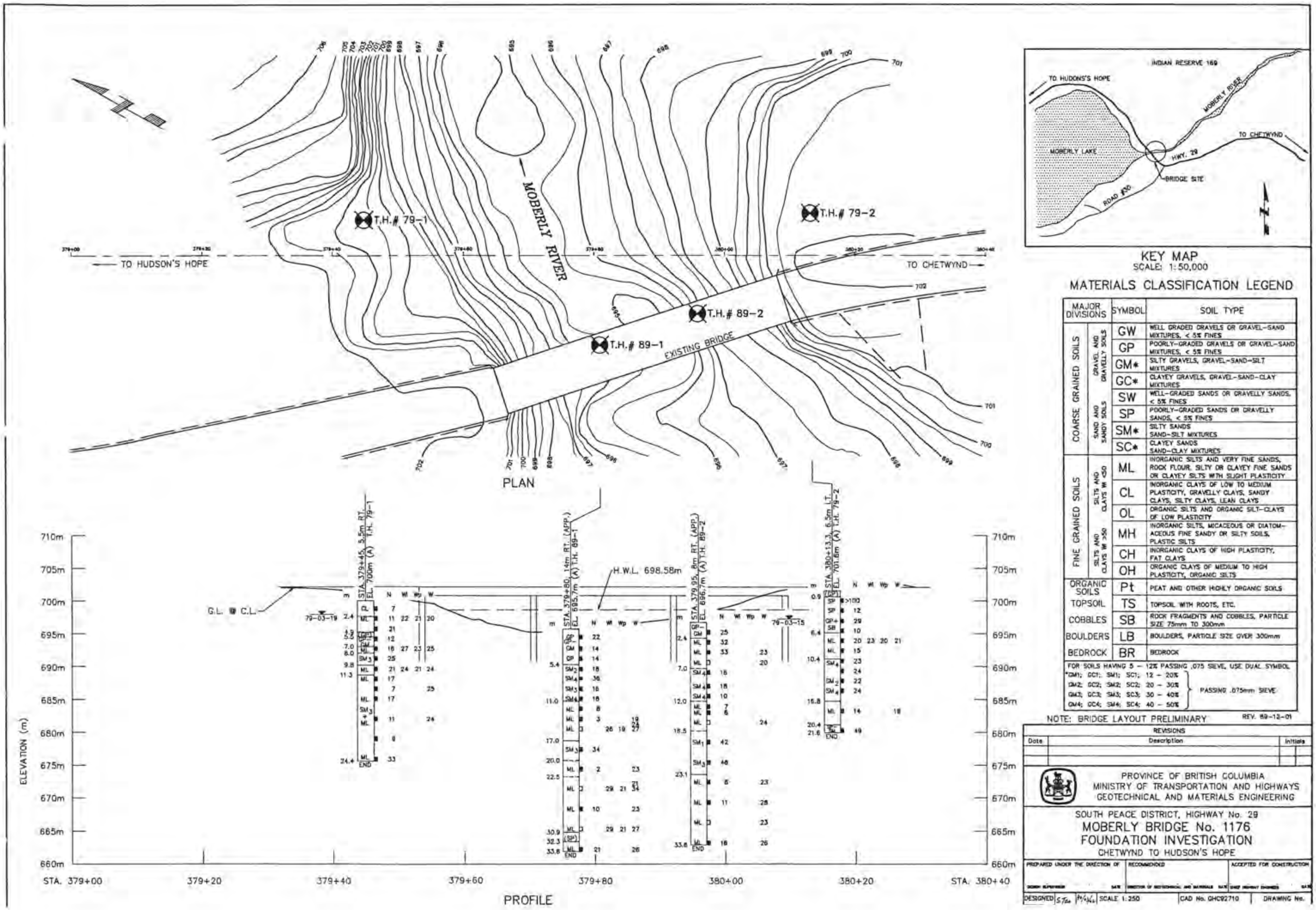
If you have any further questions, regarding the above information, please do not hesitate to contact me at 565-6436.

Yours truly,

for G.W. Davidson, P.Eng.
Head, Engineering Section
Northern Interior Region

GWD/dj

cc: Mike Lambert, Water Management, Fort St. John, B.C.



MOBERLY RIVER
T.H.# 79-1
T.H.# 79-2
T.H.# 89-1
T.H.# 89-2
EXISTING BRIDGE
TO HUDSON'S HOPE
TO CHETWYND
PLAN
KEY MAP
SCALE: 1:50,000
INDIAN RESERVE 169
TO HUDSON'S HOPE
MOBERLY RIVER
TO CHETWYND
MOBERLY LAKE
HWY. 29
BRIDGE SITE
ROAD #30
MATERIALS CLASSIFICATION LEGEND
MAJOR DIVISIONS
SYMBOL
SOIL TYPE
COARSE GRAINED SOILS
GRAVEL AND GRAVELLY SOILS
SAND AND SANDY SOILS
GW WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
GP POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, < 5% FINES
GM* SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
GC* CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
SW WELL-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
SP POORLY-GRADED SANDS OR GRAVELLY SANDS, < 5% FINES
SM* SILTY SANDS SAND-SILT MIXTURES
SC* CLAYEY SANDS SAND-CLAY MIXTURES
FINE GRAINED SOILS
ML INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
CL INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
OL ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
MH INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, PLASTIC SILTS
CH INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
OH ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
ORGANIC SOILS Pt PEAT AND OTHER HIGHLY ORGANIC SOILS
TOPSOIL TS TOPSOIL WITH ROOTS, ETC.
COBBLES SB ROCK FRAGMENTS AND COBBLES, PARTICLE SIZE 75mm TO 300mm
BOULDERS LB BOULDERS, PARTICLE SIZE OVER 300mm
BEDROCK BR BEDROCK
FOR SOILS HAVING 5 - 12% PASSING .075 SIEVE, USE DUAL SYMBOL
*GM1; GC1; SM1; SC1; 12 - 20%
GM2; GC2; SM2; SC2; 20 - 30%
GM3; GC3; SM3; SC3; 30 - 40%
GM4; GC4; SM4; SC4; 40 - 50%
PASSING .075mm SIEVE
NOTE: BRIDGE LAYOUT PRELIMINARY
REV. 89-12-01
STA. 379+45, 5.5m RT. EL. 700m (A) T.H. 79-1
STA. 379+80, 14m RT. (APP.) EL. 695.7m (A) T.H. 89-1
STA. 379.95, 8m RT. (APP.) EL. 696.7m (A) T.H. 89-2
STA. 380+13.3, 6.5m LT. EL. 701.8m (A) T.H. 79-2
H.W.L. 698.58m
G.L. @ C.L.
79-03-19
79-03-15
ELEVATION (m)
STA. 379+00
379+20
379+40
379+60
379+80
380+00
380+20
STA. 380+40
PROFILE
REVISIONS
Date
Description
Initials
PROVINCE OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL AND MATERIALS ENGINEERING
SOUTH PEACE DISTRICT, HIGHWAY No. 29
MOBERLY BRIDGE No. 1176
FOUNDATION INVESTIGATION
CHETWYND TO HUDSON'S HOPE
PREPARED UNDER THE DIRECTION OF
RECOMMENDED
ACCEPTED FOR CONSTRUCTION
DESIGNED
SCALE 1:250
CAD No. GHC92710
DRAWING No.


2011/05/25 14:29


2011/05/25 14:29


2011/05/25 14:29


2011/05/25 14:31


2011/05/25 14:31


2011/05/27 13:59


2011/06/17 15:28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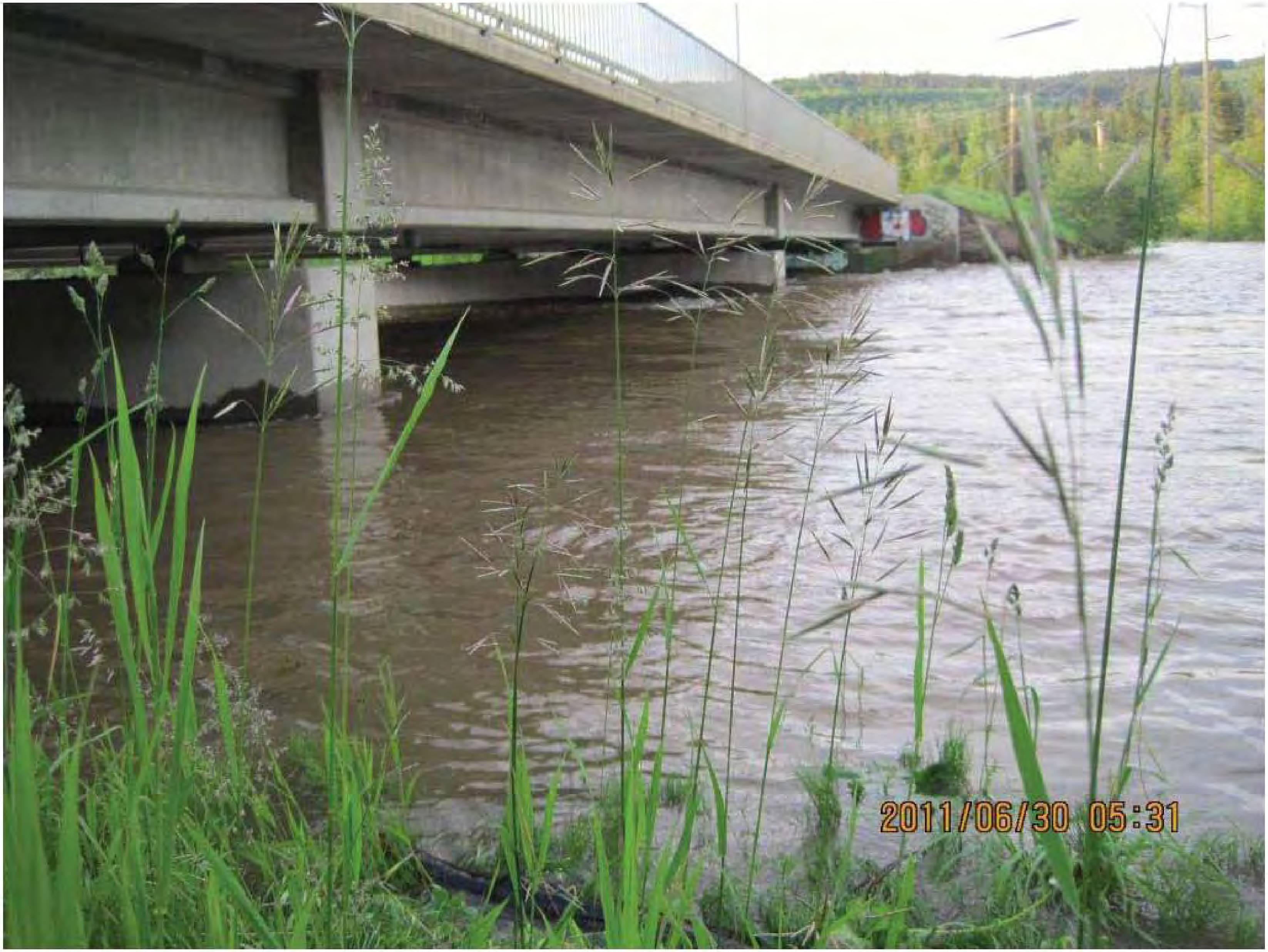
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CANADA
BEAVER
CAPITAL
2011/06/30 05:32


2011/07/01 05:38

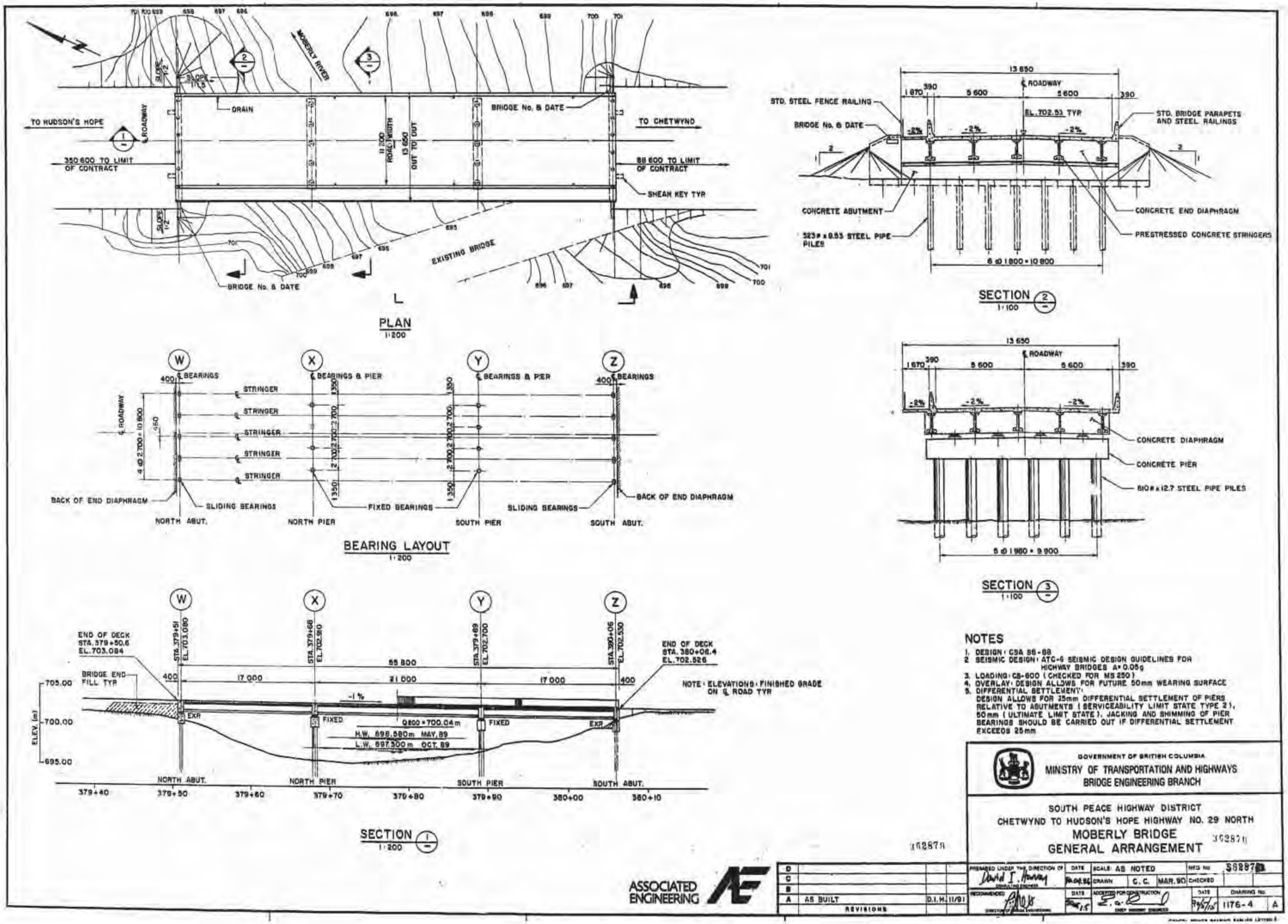

2011/07/01 05:38


16/04/2012


16/04/2012


BeAVeR


2/6/2000


PLAN
1:200
TO HUDSON'S HOPE
TO CHETWYND
350 600 TO LIMIT OF CONTRACT
88 600 TO LIMIT OF CONTRACT
MOBERLY RIVER
EXISTING BRIDGE
DRAIN
BRIDGE No. & DATE
SHEAR KEY TYP
BEARING LAYOUT
1:200
BACK OF END DIAPHRAGM
SLIDING BEARINGS
FIXED BEARINGS
NORTH ABUT.
NORTH PIER
SOUTH PIER
SOUTH ABUT.
SECTION 1
1:200
END OF DECK STA. 379+50.6 EL. 703.084
END OF DECK STA. 380+06.4 EL. 702.526
BRIDGE END FILL TYP
NOTE: ELEVATIONS: FINISHED GRADE ON ℄ ROAD TYP
H.W. 698.580m MAY.89
L.W. 697.300m OCT.89
SECTION 2
1:100
STD. STEEL FENCE RAILING
STD. BRIDGE PARAPETS AND STEEL RAILINGS
CONCRETE ABUTMENT
CONCRETE END DIAPHRAGM
PRESTRESSED CONCRETE STRINGERS
323ø x 9.53 STEEL PIPE PILES
SECTION 3
1:100
CONCRETE DIAPHRAGM
CONCRETE PIER
610ø x 12.7 STEEL PIPE PILES
NOTES
1. DESIGN: CSA S6-88
2. SEISMIC DESIGN: ATC-6 SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES A=0.05g
3. LOADING: CS-600 (CHECKED FOR MS 250)
4. OVERLAY: DESIGN ALLOWS FOR FUTURE 50mm WEARING SURFACE
5. DIFFERENTIAL SETTLEMENT: DESIGN ALLOWS FOR 25mm DIFFERENTIAL SETTLEMENT OF PIERS RELATIVE TO ABUTMENTS (SERVICEABILITY LIMIT STATE TYPE 2), 50mm (ULTIMATE LIMIT STATE). JACKING AND SHIMMING OF PIER BEARINGS SHOULD BE CARRIED OUT IF DIFFERENTIAL SETTLEMENT EXCEEDS 25mm
GOVERNMENT OF BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND HIGHWAYS
BRIDGE ENGINEERING BRANCH
SOUTH PEACE HIGHWAY DISTRICT
CHETWYND TO HUDSON'S HOPE HIGHWAY NO. 29 NORTH
MOBERLY BRIDGE
GENERAL ARRANGEMENT
ASSOCIATED ENGINEERING
A AS BUILT D.I.H. 11/91
REVISIONS
SCALE: AS NOTED
DRAWN C.C. MAR.90
DRAWING No. 1176-4 A


2/6/2000