Report Title: Site C Reservoir Shoreline Stability Assessment
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During Stage 2 of the Site C Project, studies are underway to update many of the historical studies and information known about the project.

The potential Site C project, as originally conceived, will be updated to reflect current information and to incorporate new ideas brought forward by communities, First Nations, regulatory agencies and stakeholders. Today's approach to Site C will consider environmental concerns, impacts to land, and opportunities for community benefits, and will update design, financial and technical work.

SITE C RESERVOIR
SHORELINE STABILITY ASSESSMENT

A report to

B.C. HYDRO & POWER AUTHORITY

APRIL 1978
15-2-62

THURBER CONSULTANTS LTD.
VICTORIA B.C.

SYNOPSIS

The Site C Development proposal involves the construction of an earth dam on the Peace River 4 miles southwest of Fort St. John. Approximately 50 miles of the Peace River would be flooded, the total shoreline length including tributaries would be 154 miles and the maximum depth of flooding would be approximately 165 feet.

The reservoir area has a history of slope failures. This report describes the effect that reservoir flooding would have on future shoreline stability.

The study is based on information gained from geological field work carried out during 1975-77, and a shoreline drilling program in 1977.

The findings of the study are summarized in Section 8.1 and recommendations for continuing work are provided in Section 8.2.

A separate album of drawings (Dwg.15-2-62-12 sheet 1 to 15) has been prepared showing the location of the safeline for all privately owned properties on the Peace River upstream of Tea Creek.

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Chapter 1

INTRODUCTION

1.1 PURPOSE

This report provides the results of a detailed assessment of the shoreline stability of the proposed Site C reservoir. The purpose of the work was covered in Mr. F.J. Patterson's letter dated July 15, 1976 to Thurber Consultants. Specifically, the work encompassed:

1. A detailed field examination of the entire shoreline upstream of the Site C axis, and updating of the previous shoreline stability classification particularly in those areas where there is a potential for large slides.

2. Establishing a final safeline along all privately owned land along the Peace River from the confluence with Tea Creek upstream to Site One.

3. Providing an assessment of the benefit (or otherwise) of lowering the F.S.L. from el. 1515 to el. 1505 or el. 1500. (Note, this requirement was added by Mr. H. Taylor on August 13, 1976 by telephone.)

1.2 AUTHORIZATION

The work was carried out in accordance with Thurber Consultants' proposal dated August 13, 1976. The work was authorized by B.C. Hydro P.O. 651-097.

1.3 DESCRIPTION OF PROJECT

The following description covers only those items which are pertinent to the study.

The proposed damsite is located on the Peace River about 4 miles southwest of Fort St. John and is 0.5 miles downstream of the mouth of the Moberly River (refer Fig.1 and Dwg. 1)*.

For a normal full supply level at el. 1515, the reservoir would have a surface area of 23,600 acres and a total volume of 1.87 million acre feet. Approximately 50 miles of the Peace River would be flooded, a total shoreline length including tributaries of 154 miles. Based on average flows the reservoir would take a little less than one month to fill. The maximum depth of inundation would be approximately 165 feet.

Since the energy output at Site C would be maximized when the reservoir is operated without drawdown, very small reservoir level changes would normally be made. Drawdown to accommodate optimum daily and weekly regulation should generally not exceed 2 or 3 feet. When passing floods, the reservoir might temporarily rise 1

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* Drawings have been provided in Appendix A. For brevity throughout the text, drawings have been identified by use of the last digit only of the 6 digit number on the title block (i.e. Dwg. 15-2-62-1 is referred to as Dwg. 1). All figures will be found inserted within the text of the report at suitable locations.

or 2 feet. To pass the "project flood" with a probability of occurrence of less than 1 in 10,000, would require a rise of 5 feet to el. 1520.

It is understood that the dam would be constructed of earth fill.

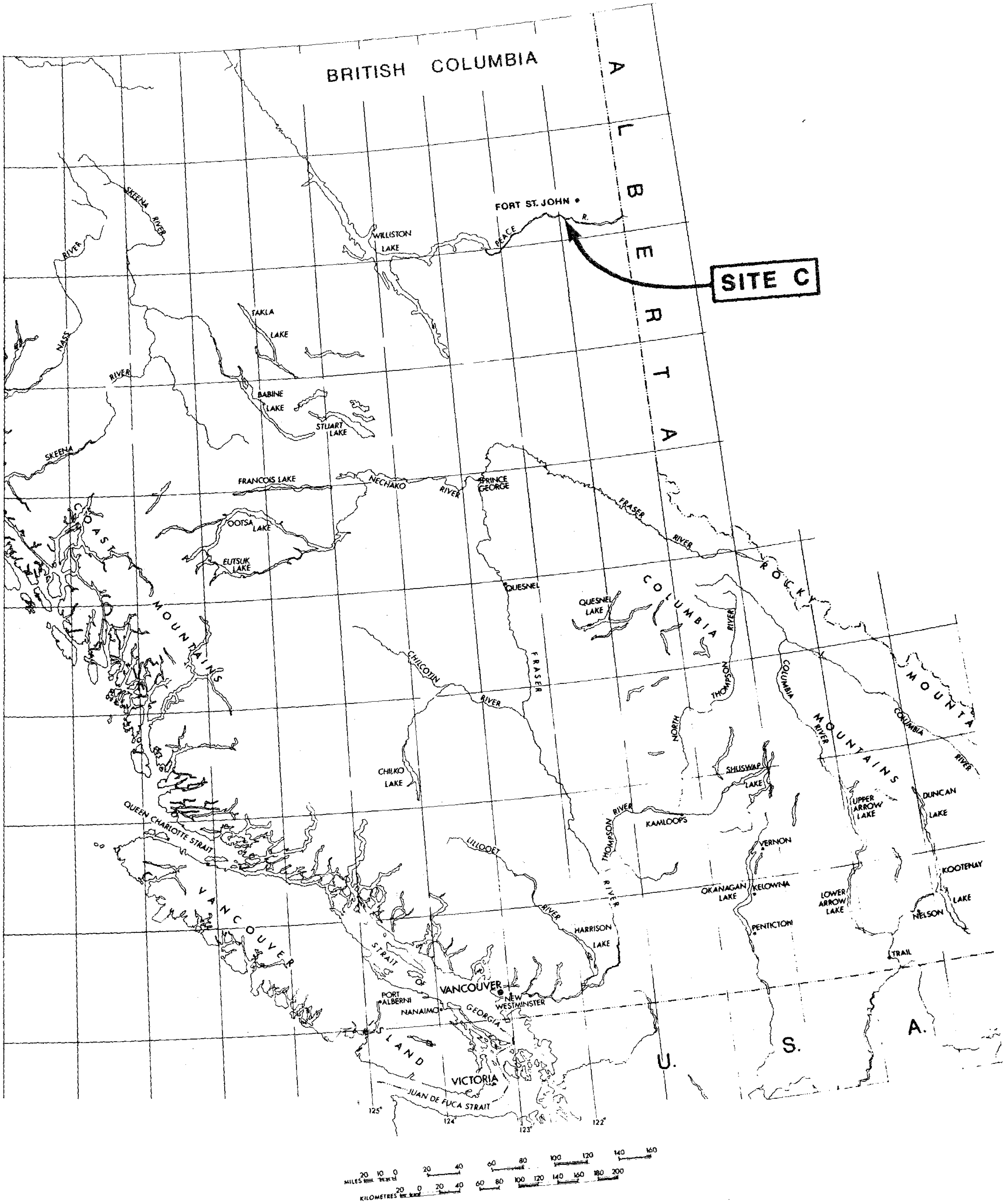
1.4 PREVIOUS WORK

The shoreline assessment work carried out to date for the Lower Peace River (downstream of Site One) has been reported in three separate reports by Thurber Consultants:

1. March 1974, "Lower Peace River Shoreline Study".
2. January 1976, "Site C and E, Lower Peace River and Tributaries Shoreline Assessment".
3. April 1976, "Shoreline Assessment, Moberly River, B.C".

The first two reports covered Site C in conjunction with possible downstream developments. The purpose of these studies was to generate an understanding and appreciation of the nature and magnitude of future reservoir shoreline stability problems. A generalised approach was taken throughout, although the detail of the work was sufficient to enable a comparison of alternative hydroelectric schemes to be made.

The third report is specific to Site C and was prepared because of the close proximity of the Moberly River to the damsite.



LOCATION OF PROJECT

SITE C RESERVOIR SHORELINE STABILITY ASSESSMENT

FIGURE 1

THURBER

1.5 SCOPE OF STUDY

The scope and methodology of the study are described in detail in Appendix D. A detailed geological study of the entire reservoir shoreline was carried out including an examination of most privately owned property. The field work was supplemented by a drilling program including installation of piezometers and inclinometers (at Cache Creek). This program is summarized on Table D-1 (Appendix D).

Chapter 2

DESCRIPTION OF PROJECT AREA

The selection of high level oblique air photographs included in this chapter (Figs. 2 to 9) portrays the physiography of the reservoir area. Drawing 2 also serves as an overall topographic plan of the region.

The area can be divided into three broad topographic units:

- Uplands
- Plateau Areas
- River Valleys

The uplands are typified by the country on the north side of the Peace River between Bear Flats and Charlie Lake. It consists of undulating, but rounded, topography occurring above el.2300 and ranging up to el.3000. It is underlain at shallow depth by horizontally bedded sandstones or shales. The ground surface slopes are typically in the order of 5:1* and 10:1 when underlain by a thinly bedded sandstone-shale sequence, but locally can be as steep as 2:1 when underlain by massive sandstone. The terrain is illustrated in Fig.8 (above the Cache Creek slide area).

The plateau areas constitute a very prominent unit as shown in Fig.6 and many other photographs. Throughout the unit the only break in topography is the occasional swampy hollow or shallow mound. The areas were formerly covered by glacial Lake Peace, shown on Dwg.2. The plateau, covered by clays with occasional gravel and underlain at shallow depth by Cretaceous

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* Throughout this report, slopes are described with the first number referring to the horizontal

shales and sandstone lies mostly between el.2000 and 2500. The present major streams tend to follow a pre-existing interglacial drainage pattern (Dwg.2) and as a consequence the overburden varies considerably in thickness adjacent to these streams and can be of the order of 600 feet thick.

The reservoir area is entirely contained within the river valleys which are trench-like features some 700 feet deep cut into the plateau (see Figs.3 and 4).

The Peace River has an average gradient of 3 ft./mile through the reservoir area. By contrast the gradient of the river through the canyon upstream of Site One averages 14 ft./mile.

The gradients of the tributary streams are given in the table below:

River or Creek	Distance inundated by proposed reservoir (Miles)	Average Gradient (ft./mile)
Peace	49.7	3
Moberly	6.0	27.8
Cache	3.6	36.1
Halfway	9.0	11.4
Farrel	1.0	60.0
	68.8	

Below Site One, the Peace River is well confined by either terraces or the valley slopes. The meanders are very slight, the sinuosity index (the ratio between distance measured along the river and distance measured along the centre-line of the valley) rarely exceeding 1.05. The width of the river channel is normally 1000 feet. There is no evidence of any significant changes in the channel pattern within the last 20 years or so other than those caused by river bank slides.

The bottom of the Peace River valley is typically 1 to 1.5 miles across. It has a maximum width of 3 miles at Lynx Creek (Mile 82* and has a minimum width of 2000 feet in the Site C canyon (Mile 37 to 42, Fig.3). The lower valley slopes are conspicuously terraced (Fig.4) often with steep river slopes 100 feet or so above river level. The main side slopes of the valley vary from 1.7:1 to as flat as 5:1. However the slopes in overburden generally range between 2.5:1 and 4:1.

Agriculture is a prominent land use throughout the area and many of the river terraces and plateau areas on the north shore have been cultivated (Fig.4).

* Miles are measured from the B.C. - Alberta Border

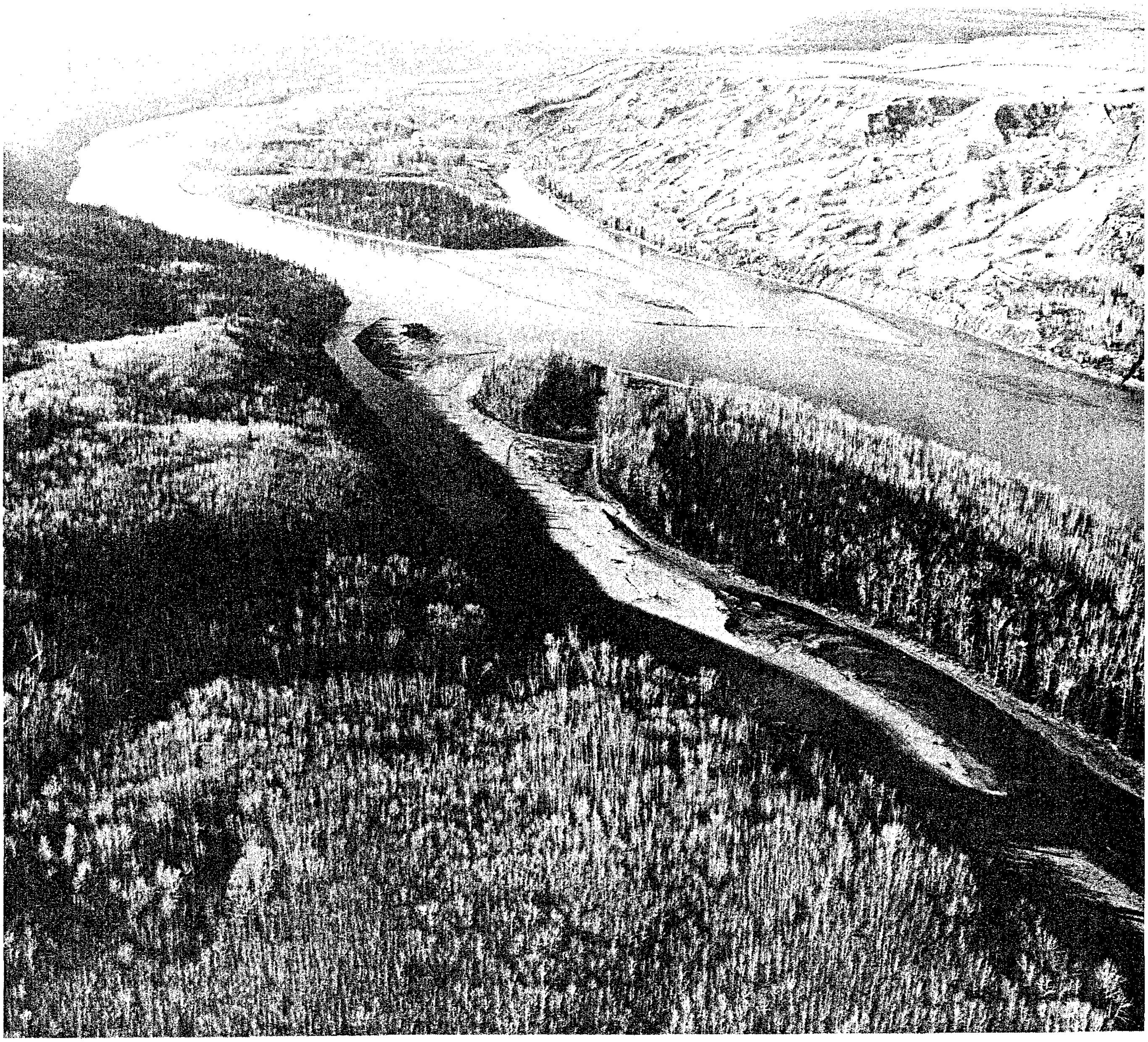


Fig. 2

Looking upstream from Mile 42. Tea Creek is off the right side of the photograph. Note the typical slump topography in silts and clays on the 700 feet high valley slope. The thickness of the slumped material is often quite shallow and relatively minor disturbance (e.g. road cuts and fills) can reactivate movement.

Fig. 3

Looking downstream from Tea Creek (Mile 42) towards Site C. Note the difference in the topography and vegetation of the valley slopes. The left bank shows more evidence of relatively recent slumping. Both slopes are in bedrock.

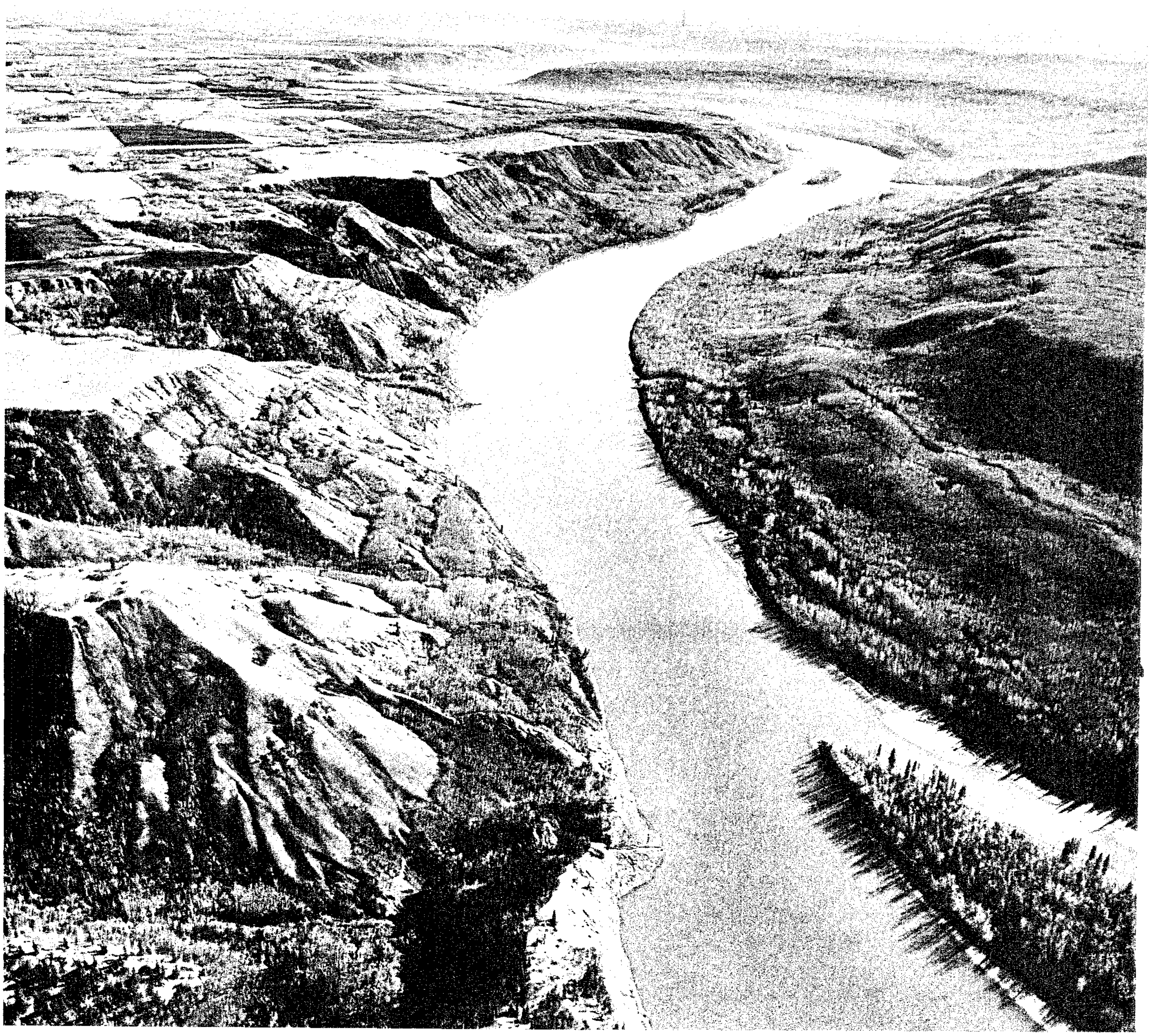


Fig. 4

A view downstream from Mile 46 looking towards the bedrock 'canyon' beginning at Tea Creek. (see Fig. 3).

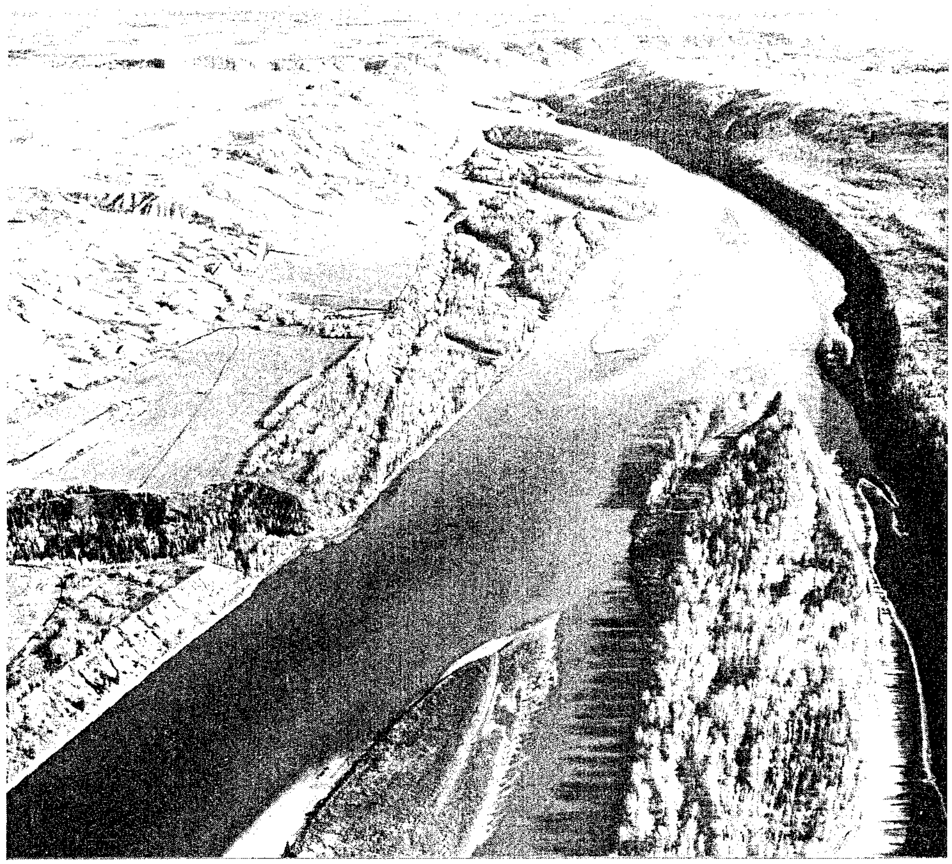


Fig. 5

Sandstone islands at Mile 77, marking the furthest downstream point of exposed Gates Sandstone. Beyond this point Ft. St. John Shales are found at river level.

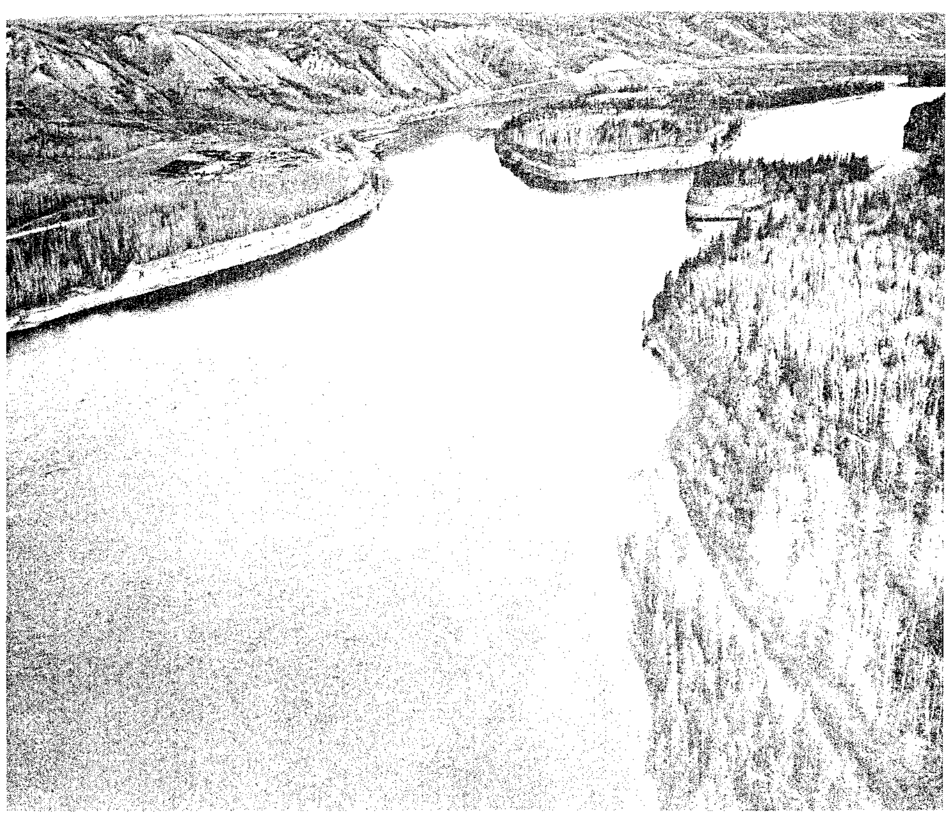


Fig. 6

A view of the Attachie Slide which occurred on May 26, 1972 blocking the river for several hours. The slide developed on the south bank of the Peace River at Mile 62. The bank extends up approximately 700 feet above river level.

Figs. 7 and 8

The Cache Creek Slide area at Mile 51 on the north bank. This massive slide occurred in shale and sandstone bedrock. Partial movements occurred at the beginning of this century. The slide, almost 1500 feet high and 1.5 miles across, has affected the entire slope.

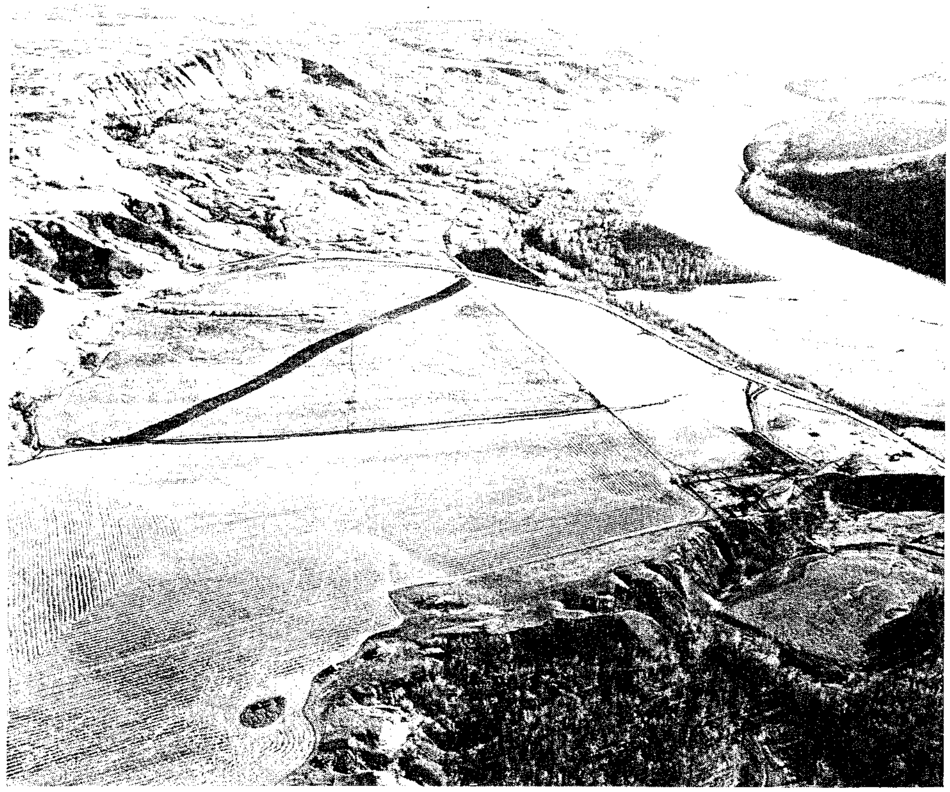
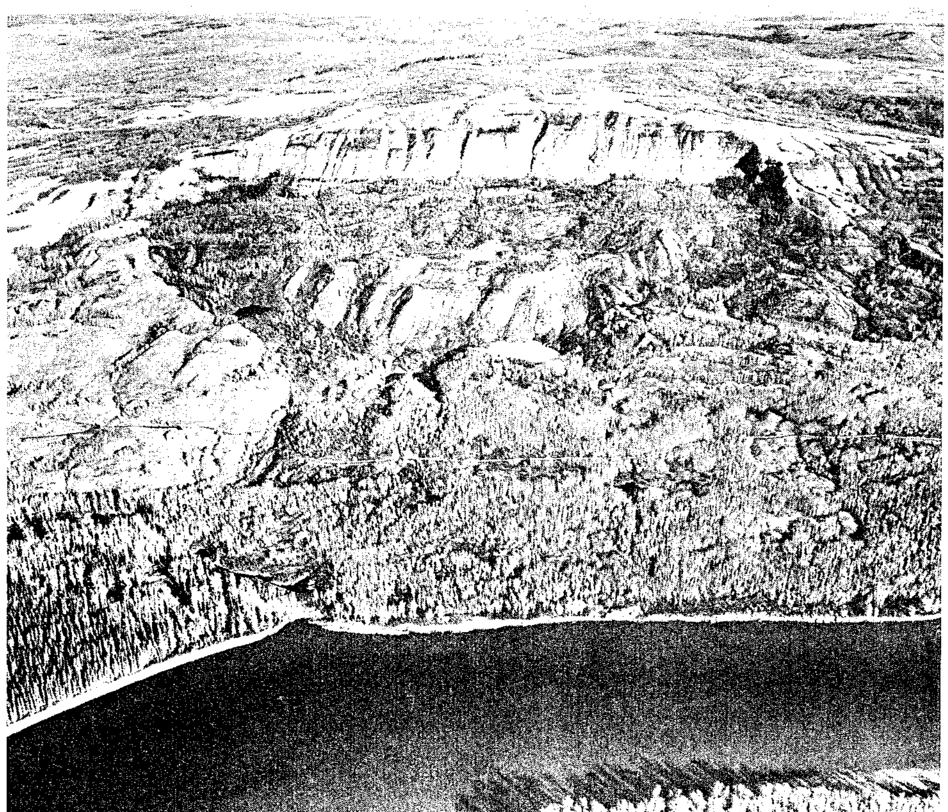


Fig. 7

Fig. 8



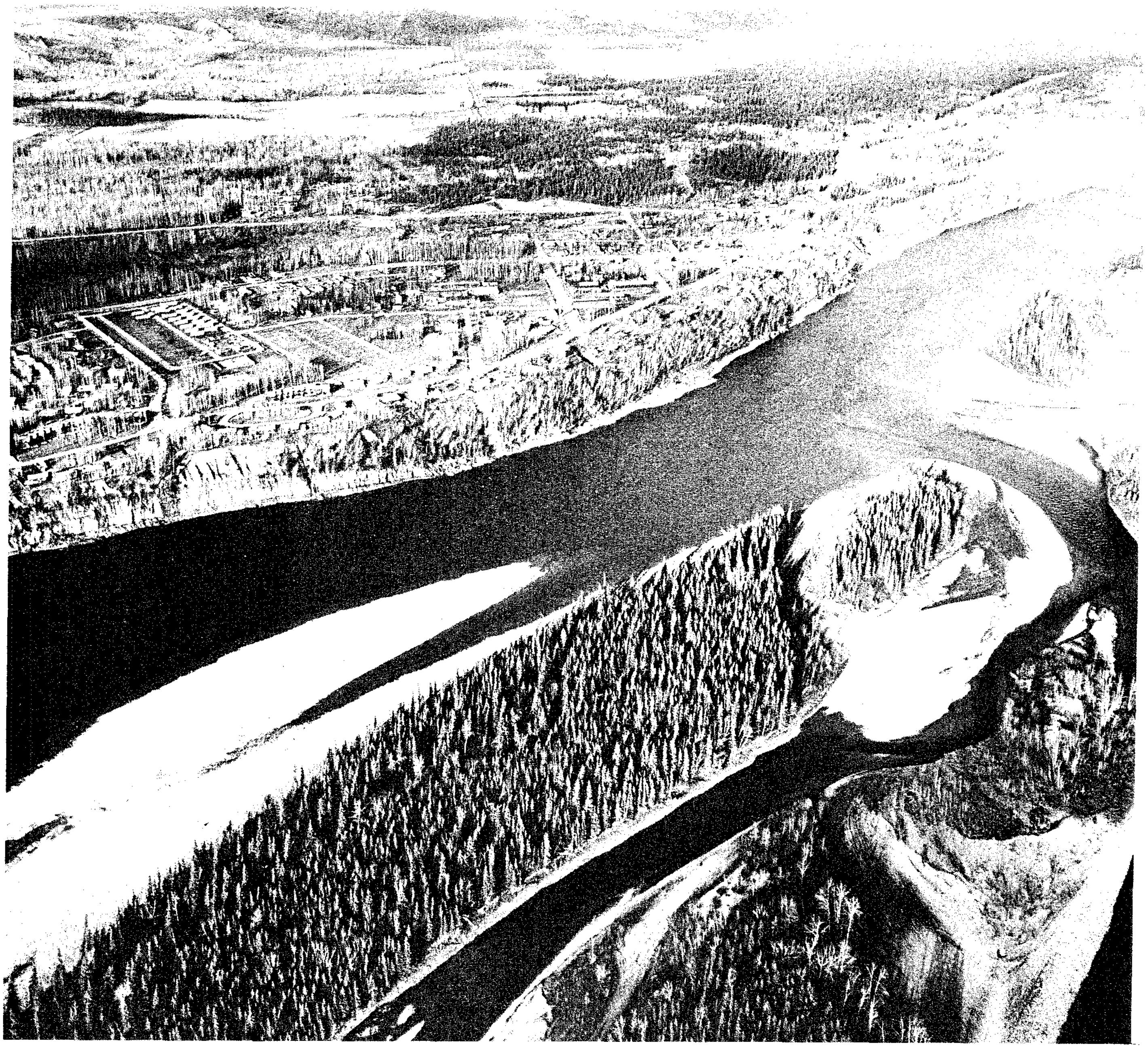


Fig. 9

The village of Hudson Hope on the north bank at Mile 84 is located on a terrace of gravel overlying clay silts. Bedrock dips well below river level at this point.

Chapter 3

GEOLOGICAL AND GEOTECHNICAL CONSIDERATIONS

3.1 GEOLOGY

3.1.2 Bedrock Geology

A map showing the bedrock geology of the reservoir area is provided on Dwg.1. Table 1 on the following page summarizes the stratigraphic sequence. The most abundant rock formation is Cretaceous Fort St. John Shale (Shaftesbury Formation) which is to be found frequently exposed along the banks of the Peace River (and its tributaries) from Mile 77 to the border. In sequence, it is overlain by the Dunvegan Sandstone which is very visible in the uplands (see Fig.8 of the Cache Creek Slide). Upstream of Mile 77 the older Gates Sandstone under the Fort St. John Shale occurs at river level (Figs.5 and 9). The shales exposed at Hudson Hope probably belong to the Moosebar Formation.

Structurally these bedrock formations are flat lying or very gently dipping to the east. The general low relief of the terrain reflects the uniformity of both lithology and structure of the underlying bedrock. At Mile 77 and at Hudson Hope, some variation from the gentle easterly dip occurs in the form of anticlines.

The Gates Sandstone consists of sandstone, shale and silty shale members which are relatively resistant to erosion and are exposed, up to heights of 50 feet, as islands and banks along the Peace River.

The Fort St. John (Shaftesbury) shales seldom exceed an exposed thickness of 175 feet. They are dark grey coloured and very fissile, and occasionally include interbedded siltstone.

TABLE 1: SUCCESSION OF CRETACEOUS BEDS IN THE VICINITY OF THE PINE AND PEACE RIVERS

SERIES	GROUP	FORMATION		THICKNESS IN FEET	LITHOLOGY
UPPER CRETACEOUS		*DUNVEGAN*		*300-1200*	*Marine and nonmarine sandstone and shale*
UPPER CRETACEOUS	*FORT ST. JOHN*	*SHAFTESBURY 400-900'*	*CRUISER*	*350-800*	*Dark grey marine shale with sideritic concretions; some sandstone*
LOWER CRETACEOUS	*FORT ST. JOHN*	*SHAFTESBURY 400-900'*	*GOODRICH*	*50-1,350*	*Fine-grained, crossbedded sandstone; shale and mudstone*
LOWER CRETACEOUS	*FORT ST. JOHN*	*SHAFTESBURY 400-900'*	*HASLER*	*500'? -1,500*	*Silty, dark grey marine shale with sideritic concretions, siltstone and sandstone in lower part; minor conglomerate*
LOWER CRETACEOUS	*FORT ST. JOHN*	*COMMOTION 1080-1600'*	*Boulder Creek Member*	*240-560*	*Fine-grained, well-sorted sandstone, massive conglomerate; nonmarine sandstone and mudstone*
LOWER CRETACEOUS	*FORT ST. JOHN*	*COMMOTION 1080-1600'*	*Hulcross Member*	*0-450*	*Dark grey marine shale with sideritic concretions*
LOWER CRETACEOUS	*FORT ST. JOHN*	*COMMOTION 1080-1600'*	*Gates Member*	*220-900*	*Fine-grained, marine and nonmarine sandstones; conglomerate; coal, shale and mudstone*
LOWER CRETACEOUS	*FORT ST. JOHN*	*MOOSEBAR*		*100-1,000*	*Dark grey marine shale with sideritic concretions; glauconitic sandstone and pebbles at base*
LOWER CRETACEOUS	*BULLHEAD*	*GETHING*		*75-1,000*	*Fine- to coarse-grained, brown, calcareous, carbonaceous sandstone; coal, carbonaceous shale, and conglomerate*
LOWER CRETACEOUS	*BULLHEAD*	*CADOMIN*		*45-600*	*Massive conglomerate containing chert and quartzite pebbles*

GSC

Ref: A.E. Foscolos and D.F. Stott, 1975. Degree of Diagenesis, Stratigraphic Correlations and Potential Sediment Sources of Lower Cretaceous Shale of Northeastern British Columbia; G.S.C. Bulletin 250.

Foscolos et al. (see reference, Table 2) have classified these shales on their clay minerological content and found them to be in the intermediate stage of diagenesis*. As a result they are fairly well-bonded shales, and on exposure they disintegrate into small angular fragments but retain the parent rock identity**. Cores of the shale must be logged soon after drilling since they soon develop abundant partings. There is evidence that these shales were overlain by at least 2500 feet of younger sedimentary rocks at the end of the late Tertiary***. The shales were subsequently again loaded by ice during the Pleistocene.

The Upper Cretaceous Dunvegan Formation consists predominantly of sandstone, is consistent in texture, and is commonly interbedded with

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* Diagenesis involves physical and chemical changes in sediment due primarily to burial and weight of subsequently accumulated overburden which convert it to a consolidated rock. It includes compaction, cementation, recrystallization and changes to clay mineralogy. Diagenesis has been considered to be incipient metamorphism.

** The shales have been described as a "low" slaking material (slaking is the tendency of a rock to disintegrate upon alternate wetting and drying, as discussed by Morgenstern and Eigenbrod 1973).

***Personal communication with W.H. Mathews.

shale. The most prominent, erosion resistant beds are relatively massive buff-coloured sandstone which range in thickness up to 20 feet. There are also thinly bedded, fine-grained siltstone and shale-claystone members in this formation.

3.1.2 Surficial Geology

The stratigraphy of the overburden overlying the bedrock although complex in detail, generally adheres to the following sequence:

- Post glacial river (and terrace) gravels, alluvial fans, slide debris, etc.

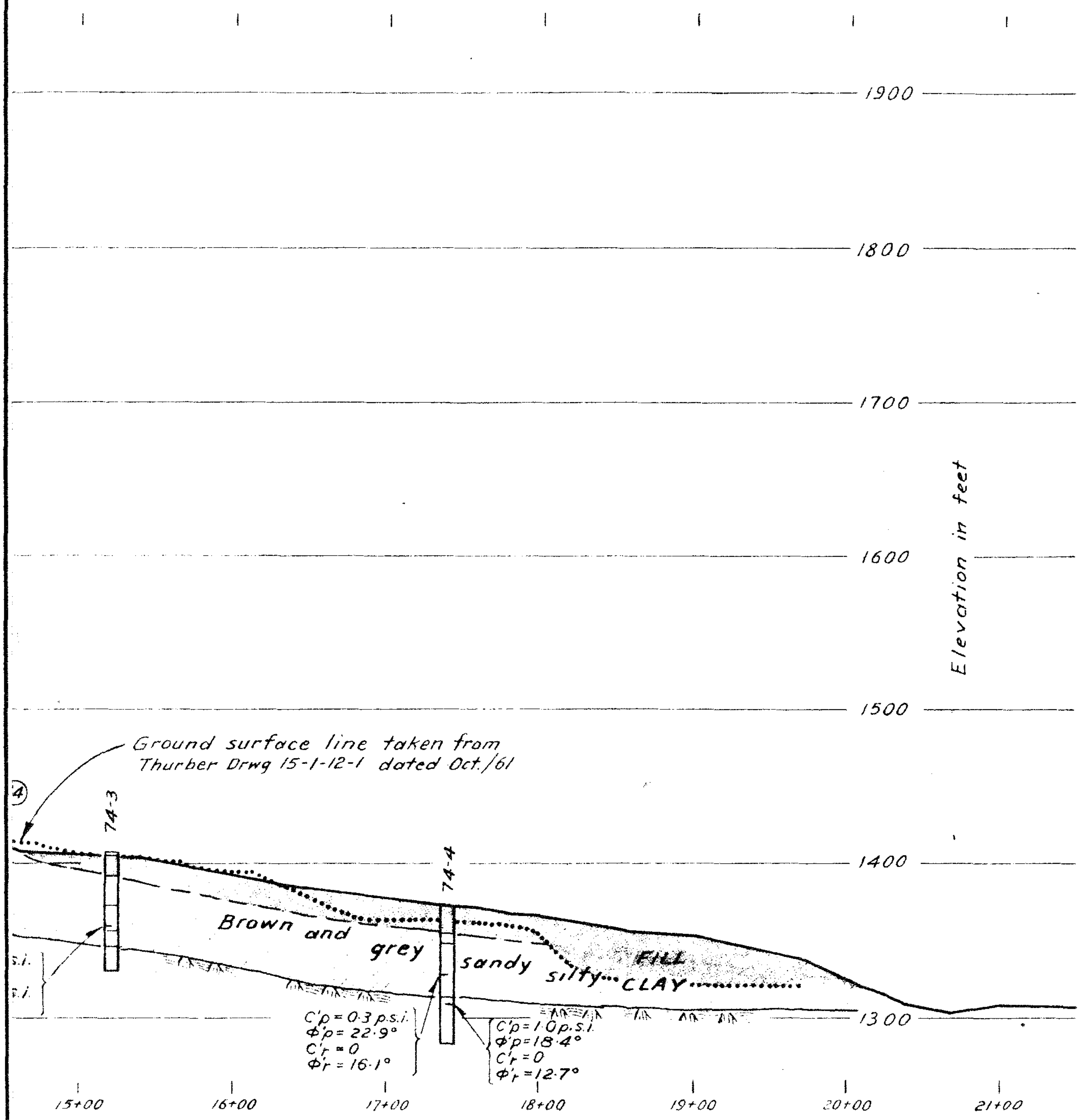
- Late glacial clays and silts up to 140 feet thick. These materials were deposited in "Lake Peace" up to the 2250 feet level. Occasionally gravelly. Commonly plastic.

- Glacial till (Wisconsin) up to 80 feet thick. Deposited during the last major advance of ice from the Canadian Shield.

- Interglacial river and lake deposits consisting typically of 400 feet or more silt, clay (sometimes stony) and minor sands.

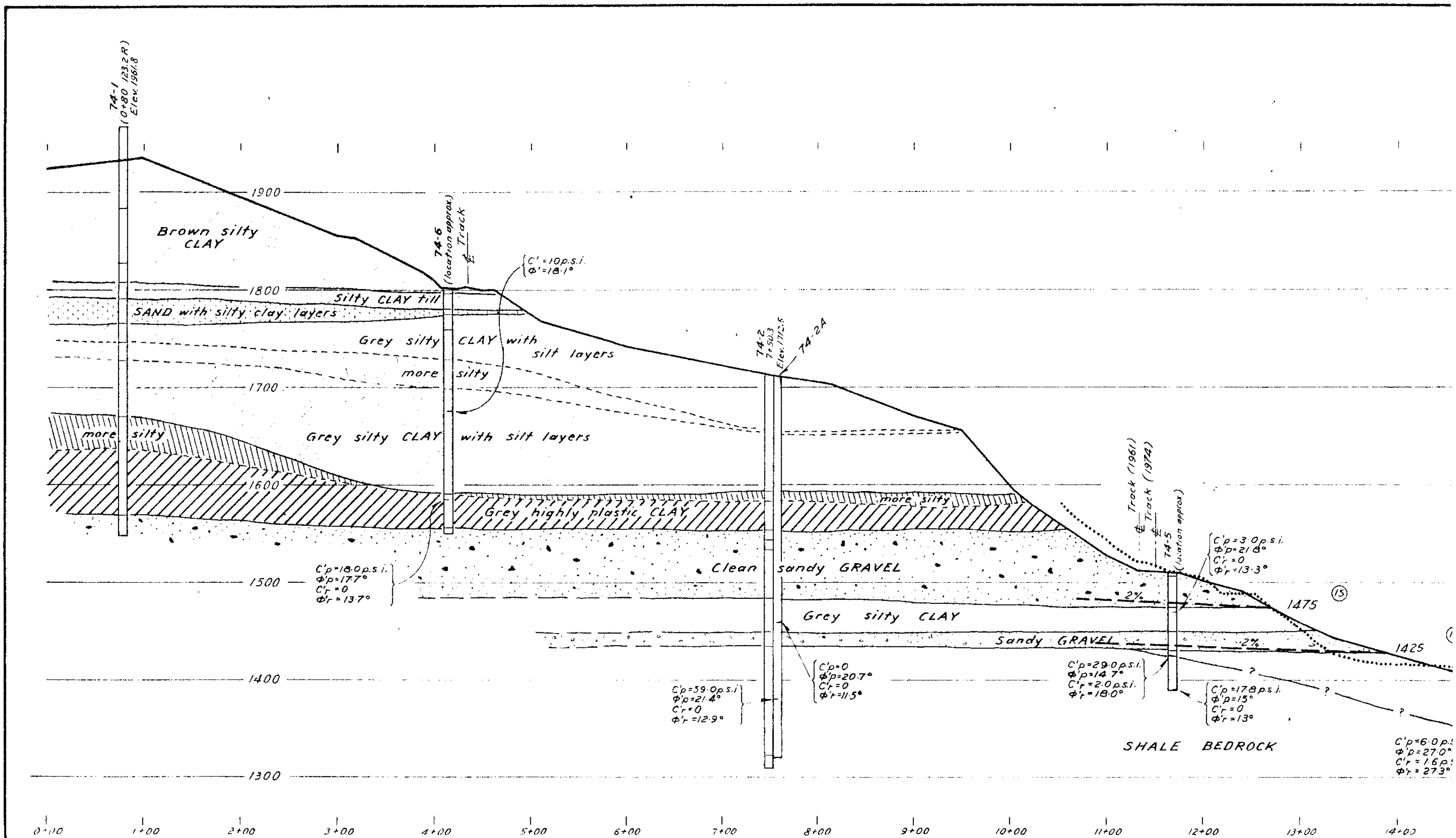
- Glacial till (Laurentide), rarely exposed and commonly missing in the reservoir area.

- Interglacial or preglacial basal river gravel deposits; typically not more than 100 feet thick. This material was deposited by a large river flowing through a bedrock channel.



SECTION THROUGH PEACE HILL SLIDE AREA
AT MILE 30 (NORTH BANK)

FIGURE 10



Cross-section ground line provided by B.C. Railway

REFERENCE: Report to B.C. Railway on Peace River Hill landslide investigation Mile 716 Ft. St. John, Thurber Consultants Ltd. File 15-1-105 April 1975. (used with permission)

The existing river drainage system follows approximately the former drainage courses. This is shown on Dwg.2. The former valley was four to five miles wide (somewhat wider than the present valley). An estimate of the location of the bottom of the valley can be made from the bedrock profiles given on Dwgs.6 and 7. At Hudson Hope, the bedrock dips below the present river level (from Mile 80 to 85) indicating a deeper former channel than at present. Elsewhere the bedrock contact rarely falls to within 50 feet of the river level and is usually greater than 100 feet above river level. Downstream of Mile 43 the present river deviates from the former valley and has eroded down through 600 feet of bedrock (Dwg.2 and Fig.3). At Mile 51 the site of the Cache Creek Slide, the river runs close to the north of the former valley and bedrock is exposed to the top of the bank.

Figure 10 shows a typical sequence in the overburden. It was prepared from a detailed drilling investigation of the north bank of the Peace River at Mile 30, downstream of Site C.

The terraces on the slopes of the Peace River portray the history of downcutting. Throughout the reservoir area there is a marked terrace about 150 to 200 feet above river level, designating a period when widening of the valley predominated over vertical downcutting. The community of Hudson Hope is located on this feature.

The recorded depth of the shale surface below the river bed is tabulated below for five locations on the Lower Peace River.

Location	Max. Depth of Alluvium (below river bed)
Mile 90 (Site One)	exposed bedrock
Mile 42 (Upstream Site C)	50'
Mile 39 (Site C)	32'
Mile 31 (BCR Bridge)	85'
Mile 1 (Site E)	150'

These results suggest increasing aggradation of the river in a downstream direction.

3.2 GEOTECHNICAL PROPERTIES

3.2.1 General

A range of typical index properties together with data on mineralogical composition is provided on Table 3 on the following page.

It is expected that the shale in the area would exhibit low slaking characteristics under unconfined conditions (Morgernstern and Eigenbrod, 1973).

The shales exhibit moderately high E-values. Moduli ranging between 65,000 and 100,000 psi have been measured by B.C. Hydro in-situ tests perpendicular to the bedding, and over 200,000 psi parallel to the bedding. (These values may be compared with those of the poorly bonded Upper Cretaceous clay shales of the Prairies, which are an order of magnitude less).

It is likely that the shales possess fairly high lateral stresses away from the valley slopes.

The highly variable consistency of the silty clays must be emphasized. They are frequently varved and some highly plastic layers exist.

The granular materials are all very dense with standard penetration N values frequently in excess of 100 blows per foot.

3.2.2 Shear Strength

The section from the B.C.R. investigation at Mile 31 shown on Fig.10 also shows the results of several direct shear tests on 2-inch square specimens parallel to the bedding. The range of values is tabulated below:

Soil Type	Peak		Residual	
	c'_p (psi)	ϕ'_p (degrees)	c'_r (psi)	ϕ'_r (degrees)
Shale	10-59(29)*	15-21(17)	0- 2(.5)	13 -18(14)
Silty Clay	0-18(6)	18-27(22)	0-16(3)	11 -27(16)

* Average values provided in brackets.

Direct shear tests carried out by B.C. Hydro on shale specimens up to 6 inches in size taken from the Site C area have provided a range of values of ϕ_r but with a reasonable minimum of 13° (a small cohesive intercept was also indicated).

The range of both peak and residual shear strength values in shale is so wide that the task of selecting suitable values presents a major difficulty. Another and often more reliable source is to compute values by analysis of well documented slides in the shale. The analysis of the Attachie and Cache Creek Slides described

TABLE 2 INDEX PROPERTIES OF SHALE, CLAY AND TILL

Soil Type	W_L%	W_P%	W_1%	W%	% Clay	% Silt	Unit Weight (P.C.F.)	Unified Soil Classiciation
Shale	37-42	20-21	17-22	6-14	40	-	130	CH
Silty Clay	38-49	22-28	17-23	20-36	32	62	120	CH
Till: upper	35	16	19	20-32(upper)	22	45	125	CL
lower	-	-		20-23(lower)	-	-		-

MINEROLOGICAL COMPOSITION OF CLAY FRACTION*

Soil Type	Montmorillonite	Illite	Kaolinite	Chlorite
Shale	5%	70%	25%	-
Silty Clay	trace	55%	35%	10%
Till	**	45%	25%	-

* Carried out and estimated by Alberta Research Council.
** 30% combined illite-montmorillonite (undifferentiated).

Note: The above information was obtained from the laboratory testing carried out on samples from DH 63-1 and DH 78-2, supplemented by data from Thurber files on investigations in the Mile 30 area.

in the next chapter suggests a $\phi'r$ equal to or greater than 13° for both shale and clay parallel to the bedding.

Triaxial shear tests on 1.5 ins. diameter specimens of silty clay taken at the Attachie Slide (DH 63-1) from below the failure plane gave the following peak values:

c'_p = 12 psi, ϕ'_p = 25° to 32° (across varves)

c'_p = 12 psi, ϕ'_p = 23° (parallel to varves)

3.3 GROUNDWATER CONDITIONS

Groundwater information in the Peace River region is at present very limited. Most recorded information is related to shallow wells which are commonly located on the low terraces adjacent to the Peace River where they have been dug for local use.

Piezometric data from this year's investigation is provided on Drawing 5 and in Appendix C.

A most important unit for groundwater is the basal gravel which serves as an aquifer of appreciable reservoir potential. Although overlain by several hundred feet of relatively impermeable silt, clay and till, the gravel does appear to be part of a surface recharge system.

Piezometers have been installed in the gravel this year in DH 63-1 (Attachie) and DH 78-2. They exhibit low to non-existant piezometric levels. However the analysis of the Attachie

Slide summarized in Appendix B indicates very high piezometric levels in the gravel immediately prior to the failure. This finding is discussed in detail later on page 24. The slide was inspected in 1973 three days after failure and strong springflows from the gravel were noted. In August 1977 very little spring flow was observed. It is probable that the water table in the gravel fluctuates seasonally very appreciably. An unconfirmed report by the British Columbia Railway indicates that in the vicinity of Peace Hill (Mile 30), after prolonged period of rainfall, the groundwater table in the gravel can rise significantly.

It is also reasonable to assume that the groundwater pattern in high banks of overburden is very complex. The overburden is a thick horizontally layered series with a high range of permeabilities. Perched water tables can be envisaged with higher piezometric pressures in the more pervious layers.

A similar pattern can ocur in the shale-siltstone sequence. The piezometers installed in the shale in DH 51-6 show different piezometric levels (Drawing 4). Both low and high shale banks at several locations were observed to be seeping water at various horizons.

Chapter 4

SHORELINE MORPHOLOGY

4.1 GENERAL

Vertical downcutting by a river, combined with widening of the valley by bank landsliding are natural processes of valley development. Most tributaries of the Peace River system are in a 'youthful' stage. The Peace River itself has reached a mature stage in its development but many of the valley slopes are still unstable and active.

A marked feature of the Peace River is the difference between the forest cover,valley slopes, and activity of the north versus the south sides of the valley in certain areas. Although there are several notable exceptions, the south slopes are generally flatter, more richly vegetated, and less active.

The valley downstream of Mile 42 (Fig.3) exhibits a steep north slope of 2:1 and a much flatter south slope of 4:1. Both slopes are in shale bedrock and there is no evidence of any change in the nature of the shales from one side of the river to the other. It can only be concluded that the groundwater conditions have been more adverse on the south side and has resulted in flattening of the slope by sliding.

It is conjectured that this difference is related to the exposure of the slopes to the sun. The south slope of the valley receive less solar heat than the north slope. Consequently, on the south side, frost penetration is greater and lasts longer. Hence the possibility of pore water pressure build up behind the frozen ground is greater. Also on the south side, the ground, after a snowmelt and rainfall, stays wetter for a longer period of time. This leads to saturation of the ground both above the frost layer and below it (where it can be penetrated) and increases the potential of pore water pressure build up. On the north side of the valley sublimation of winter snow, relatively rapid snowmelt, and evaporation of summer rains decrease the amount of water passing into the ground.

4.2 BEHAVIOUR OF EXISTING SLOPES

It is evident that landslides in the past have played a dominant role in widening the Peace River Valley. Since the beginning of the century the river has been blocked once and possibly twice.

- the Cache Creek rock slide (Mile 51), or more likely a portion of it, is reported to have moved in the early 1900's and may have temporarily blocked the river.*

- the Attachie slide which occurred on May 26, 1973 on the south shore opposite the mouth of the Halfway River (Mile 62) blocked the Peace River for 10 hours.

The P.G.E. Survey of Resources report issued in 1930 stated "about two miles below Farrell Creek, a slide from the south side of the valley is reported to have blocked the whole river only a few years ago." We have, however, searched the air photographs and have been unable to find confirming evidence of this slide, and have therefore discounted it.

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* This information was obtained through Mrs. E. Kyllo of Hudson Hope.

Other recently occurring slides a few miles downstream of the damsite have caused problems to structures,

- the failure of the bank at Taylor Flats which resulted in the collapse of the previous highway bridge (a suspension bridge) on October 16, 1957.

- failure of the north bank at Mile 31 which interrupted the B.C.R. mainline on June 13, 1974.

In addition, bank slides involving 100,000 to 500,000 c.y. have occurred recently (probably this decade):

- downstream of Taylor on the north bank

- close to the Alberta border on the north bank

- on the Beatton River

The four distinct types of land movement recognizable in the reservoir area are listed below:

(i) Surface erosion by water or gravity in either bedrock or overburden,

(ii) Flowslides in overburden,

(iii) Rotational slides in overburden,

(iv) Slides in bedrock

Erosion by flowing water or gravity (debris slides) are commonplace and are often initiated by man. Minor initial erosion can readily lead to a larger land movement.

In a flowslide, the movement of overburden material resembles that of a viscous fluid. There is no well defined failure surface and the slide geometry is generally characterized by a long narrow shape, becoming bulbous at its base where the debris spreads. Silt and clay deposits are readily susceptible to linear flowslide activity. This type of landslide usually originates in silt and clay deposits near the top of steep slopes or in fills constructed from these materials.

Rotational slides in overburden are characterized by either singular or multiple rotational sliding planes. There are very few high slopes that do not show the characteristic hummocky topography resulting from multiple sliding. A good example is shown on the north bank upstream of Tea Creek (Fig.2). It is this type of feature more than any other which creates the initial overall impression of large scale slope movement, whereas on closer examination the feature often has resulted from fairly shallow movements. However, the present equilibrium of these slump areas can be easily disturbed by the construction of roads and other utilities.

Where deep seated landslides in overburden have occurred, they are commonly associated with the basal gravel and bottom out at or near this gravel*. Some of the former slide areas are

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* This observation, and the relationship between fluctuating groundwater pressures in the gravel (p.15) and weather conditions (p.24 last para.) raises an interesting possibility of predicting on-coming large slide hazards based on weather records, much in the same way that flood hazar are predicted.

very large. The most recent large slide, the Attachie Slide is described on page 21. It is believed that many of the high steep overburden slopes have been failing progressively but imperceptibly for a considerable period of time prior to sliding. Slopes which fail in this manner have a safety factor close to unity immediately prior to failure and can be triggered by a relatively mild event (e.g. a heavy rainstorm filling tension cracks with water). The resulting failure can be quite rapid and unexpected.

It is significant that slides involving bedrock (shales) occur infrequently relative to slides in overburden. However, when they do occur, as in the case of the Cache Creek slide, they can be very large. It has been suggested (Hardy et al, 1962) that instability in the Shaftesbury Formation is a result of swelling upon reduction of overburden load. The development of swelling pressures is said to reduce the effective stress and thus the shearing resistance. An important part of this mechanism is the initial reversion of the shales to clay through the action of percolating groundwater, and it is argued that the additional fracturing of the shales due to rebound increases this action. However, as described earlier (p.9) these shales are fairly well bonded and although they deteriorate physically they generally retain their parent rock identity. Pre-existing shear planes paralleling the bedding of the shales are more likely to have played the dominant role. Extensive shear planes can result tectonically (e.g. from minor slippage between the beds with folding) and from differential movements occurring during relaxation or rebound resulting from valley erosion. The shearing resistance along these planes can approach the residual value and as such they constitute planes of weakness in otherwise stronger rock. The development of such a large slide as Cache Creek requires extensive pre-existing shear planes at depth.

The terrain analysis (airphoto interpretation) shown on Dwgs.3 and 4 depicts many of the former and presently active slide areas in the reservoir area.

A shoreline stability classification is described in Chapter 6 and the results shown on Dwg.11 are discussed relative to the predicted stability of the slopes after reservoir flooding.

4.3 THE ATTACHIE SLIDE (Miles 63, South bank)

The following account should be read in conjunction with Dwg.9. A photograph of the slide area is shown in Fig.6.

The slide occurred suddenly and rapidly on the night of May 26, 1973. It blocked the river for ten hours. By May 28 a well defined channel with a minimum width of 250 feet had been eroded through the slide debris. Prior to overflow, it is estimated that the slide caused the river to pond 23 to 24 feet above river level.

Using the before and after topography and an assumed location of the failure plane (based on field inspection and the results of DH 63-1), it is estimated that total quantities in the range of 15 to 23 million c.y. were involved in the slide. The valley bottom is covered by 8 to 11 million c.y., the remainder is on the slope.

The slope prior to failure was 3.3:1, and the top of the slope regressed 200 to 250 feet as a result of the slide.

The failure plane is well exposed at the head of a spring on the downstream side of the toe of the slide at el.1646, approximately 215 feet above present river level. It is a sharply defined knife edge plane located in a brittle plastic clay 12 to 17 inches above the basal gravel which is 60 feet thick at this point. The failure plane was not clearly determined in DH 63-1, but it appears to be located approximately 14 feet above the gravel. The peak shear strength properties of the clay beneath the failure plane have been measured. A minimum ϕ'_p value of 23° was recorded for failure parallel to the varves (c'_p= 12 psi). This is comparable to the average peak value ϕ'_p= 22° obtained from direct shear testing of clays from the slide area at Mile 30. Failure across the varves resulted in an average ϕ'_p= 28°. Residual values would be appreciably less; a minimum ϕ'= 11.5° was recorded at Mile 30 (figure 10).

A stability analysis was carried out to estimate the groundwater conditions in the gravel and overlying clay. A range of shear strength parameters was used, and a summary of the results are given in Appendix B. Air photos taken prior to the slide show evidence of movement and thus it is reasonable to assume different shear strength parameters acting along different segments of the failure plane immediately prior to failure. These would likely range from cross-bedding peak value to parallel bedding residual value. The analysis indicates that very high piezometric levels were probably acting at the time of failure. For example, referring to Figure B.1 strength parameters of c' = 0 and

Ø'a-b= 28, Ø'b-c= 25, Ø'c-d= 13, require a piezometric level at least as high as WL2, 60 to 100 feet below the ground surface in order for the slide to have occurred. If higher strength parameters are assumed, a higher pre-slide piezometric level would be required.

The slide probably failed progressively up the slope. The last phase of the slide may well have involved failure of the oversteepened back slope over the slide debris. This is portrayed by the dashed failure surfaces shown on Sections A-A and B-B of Dwg. 9. The depth of overburden over the gravel may have been sufficiently reduced by the slide to allow uplift and venting of groundwater from the gravel through the overlying clay and slide debris. This would have immediately reduced the water pressure downhill of the location of the venting and stopped the slide.

The geometry of the slide and the fact that relatively little debris accumulated on the 1:1 slope below the failure plane suggests a rapid movement. Nearby residents on Tomkins Farm reported hearing between 11.45 and 11.55 p.m. (May 26) a series of loud reports and thunder - like noises accompanied by the "sound of rushing water".* Computations, allowing for uplift equivalent to WL2 indicate a potential velocity of 20 mph at the toe of the slide (above the

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* Thurber Consultants, 1973. Inspection of the Attachie Landslide, File 15-6-12.

steep bank), and 70 to 80 mph at the bottom of the steep bank. The slide debris in moving across the 3,000 feet wide river had to overcome friction equivalent to a Ø-value of 3°, and thus must have generated substantial pore pressures.

Since the slide occurred well above river level and above the shales and basal gravel, it can be concluded that river action did not play a significant role. Other than seismic lines on the plateau above, there were few man-made changes in the immediate area. No significant earth tremors were recorded on the day of the slide.

Temperature and precipitation data from both Fort St. John Airport and Hudson Hope indicate that for at least two winters prior to the slide the snowfall had been greater than the long term average, whereas, the temperatures had been below or close to normal. With a slow snow melt this condition could well have resulted in higher than normal groundwater levels. The presence of brittle, highly plastic clays suggest that the slope may have been failing progressively for many years without showing conspicuous signs of distress other than local cracking. Slopes that fail in this manner have a very low safety factor immediately prior to failure and can be triggered off by a relatively mild event. Although April and May 1977 were very dry months on the day of the slide it rained steadily from 10 am to 2 pm and drizzled for the remainder of the day. This short but heavy period of rain could have filled or partially filled small cracks and fissures and triggered the slide.

The Attachie Slide thus illustrates the importance of the role of groundwater and indirectly, weather conditions, in occurrence and timing of slide activity.

The area immediately downstream of the slide (Section C) is traversed by a large number of fresh cracks and scarps. The probability of this area failing within the next few years is high. Should such a failure occur it could be of the same order of magnitude as the recent slide.

4.4 THE CACHE CREEK SLIDE (Mile 51, North Bank)

The following account should be read in conjunction with Dwg.10. Photographs of the slide are shown on Figs.7 and 8.

The Cache Creek slide is contained entirely within bedrock consisting of the Fort St. John Shales with a capping of Dunvegan Sandstone. The slide has affected the entire slope from river level at el.1390 to the top at el.2800. It has a maximum width parallel to the river of close to two miles and extends back 1.2 miles.

The slide is a multiple one with its youngest section in the upstream (west) half. The time of origin has not been established but there is a report of a substantial movement in the the early 1900's. This report is supported by the following field observations in the toe area downhill of the road.

- numerous sharply defined grabens, with few trees older than 50 years within the grabens.

- some older fir trees indicate a ground disturbace approximately 70 years ago.

- many ponds without apparent drainage that have not eutrophied.

- the flood plain is sharply truncated by toe of slide on the upstream side.

There is also evidence that part of the toe of the slide has not moved as recently as other sections:

- at one location the slide debris is overlain by intact river gravel.

- at other locations the slide debris is overlain by appreciable amounts of "over-bank" silt.

As part of the 1977 drilling program, six holes were put down in the slide. Four holes were drilled on the toe, five piezometers were installed, and two strings of slope indicator casing. The remaining two holes were drilled in the upper west section of the slide, and four piezometers were installed. Summary logs of these holes are included on Dwg.5, with tabulated piezometric data in Appendix C.

The profile of the river bank prior to failure has been estimated and plotted on Dwg. 10. Section C shows holes 51-2 and 51-4 straddling the bank which, like the existing one downstream, was a 100 feet or so high. The pre-slide profiles have been prepared by projecting the surface contours from each side of the slide and by comparing areas of slide debris at the toe with material removed from the top. The results are tabulated on the following page.

Section	Pre-slide Slope	Post-slide Slope	Regression of Top
A-A	2.8 :1 (5 : 1)*	3.8 : 1 (7 : 1)	300' - 350
C-C	3 : 1 (2 : 1)	4.8 : 1 (5.5 : 1)	500' - 600

* Bracketed values refer to toe area

The drilling program showed conclusively that sliding was initiated in the upper half of the slope. Section A-A shows a failure plane bottoming out between el.1900 and 2000. The shale - sandstone contact is believed to be at or around el.2200. Assuming that slippage between the beds due to tectonic folding or relaxation has occurred, the pre-slide existence of planes of weakness in the shale close to the contact with the sandstone is to be expected. The massive sandstone member would tend to concentrate slippage at its boundaries. The slide debris accumulated at the toe and the reported movement at the beginning of the century was probably activation of this toe debris. Quite possibly the river had eroded and steepened the debris causing the movement to occur. It is noted that clay seams were encountered in holes 51-1 and 51-3 in the shales near to the contact with the debris, although no clay seams are reported in holes 51-2 and 51-4 (Section C).

The estimated upper limit of the total volume of material involved in the slide is approximately 90 million c.y. This consists of 26 million c.y. on the upper slopes and 64 million c.y. at the toe of the slope.

Stability analyses have been carried out on the west half (Section A-A) for the upper and lower (toe) slide areas. The results are summarized in Appendix B.

The analysis of the upper west section of the slide suggests that a condition of marginal stability may exist, which is supported by the field observations noted on the plan (Dwg.10). It is reasonable to assume residual values for the entire failure plane which would range from $\emptyset = 40$ for sandstone to as low as $\emptyset = 13°$ for shale. For assumed piezometric levels which satisfy the current observations in hole 51-5 and 51-6, very low factors of safety can be obtained.

The upper slide areas in the east half are considered older than the west section, but it is noteworthy that scarps and tension features are very evident throughout these areas.

The analysis for the lower area, again assuming reasonable residual strength values and piezometric levels compatible with those observed to date, indicates a more stable condition. The lowest factor of safety was obtained for assumed reactivation of the slide debris over the old slide plane assuming a residual $\emptyset$ value (13°). It should be noted that an increase in piezometric level as much as 100 feet at the location of hole 51-1 would be required to lower the factor of safety to unity

The speed of movement of the slide mass when failure occurred is not known. The initial slides in intact material were undoubtedly quite rapid. The slope immediately below the toe of the upper slide in the west section (A-A) is as steep as 2.4:1 and 200 feet high. The velocity of the slide debris on reaching the bottom of this slope could have been in the order of 50 mph. Any later reactivated slide material may have moved quite slowly*.

The lower slopes (below road) immediately downstream of the slide are actively moving and exhibit many fresh features. As shown on the plan, they slope at about 4:1, are 100 to 150 feet high and are in shale.

In summary:

- the Cache Creek Slide originated with a failure of the upper half of the slope. Thus, although the slope is the highest (1400 feet) of any downstream of Site One, this height was not a factor in causing the slide.

- the upper section of the slide mass (particularly in the west half) appears marginally stable. Although the potential volume of any slide debris passing down the steep 100 to 200 ft. high slope below the upper slide is not likely to be great its velocity at the toe of this slope could be high

- the stability of the toe area is currently acceptable. A substantial increase in groundwater pressure along the failure

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* During the Public Information Meetings for the, environmental program, a report of slide debris reaching the south side of the river was received. In a subsequent field inspection the occasional angular boulder was noted in the beach material on that side of the river, but other than this no confirmation was obtained.

plane would likely be necessary to reactivate a major movement.

- should the lower slide be reactivated, it would probably move slowly.

4.5 LONG TERM SLOPES

4.5.1 Overburden

The flattest and most stable appearing slopes in overburden in or adjacent to the reservoir area are on the south bank at Mile 45 and at Mile 37. These slopes average 4.5:1 and 4.4:1 respectively. However, even at these slopes there is the occasional evidence of recent instability (see Dwg.3, Mile 45, south bank). It is concluded that the long-term stability of a slope of 5:1 would be reasonably assured.

4.5.2 Shale

The flattest and most stable appearing slopes in shale occur on the south bank from Mile 40 to 42. These slopes average 1:3.8. The existing flattest slope of the Cache Creek Slide is 1:4.8 (Section C-C). It is considered that the long-term stability of natural shale slopes of 5:1 would be reasonably assured.

Chapter 5

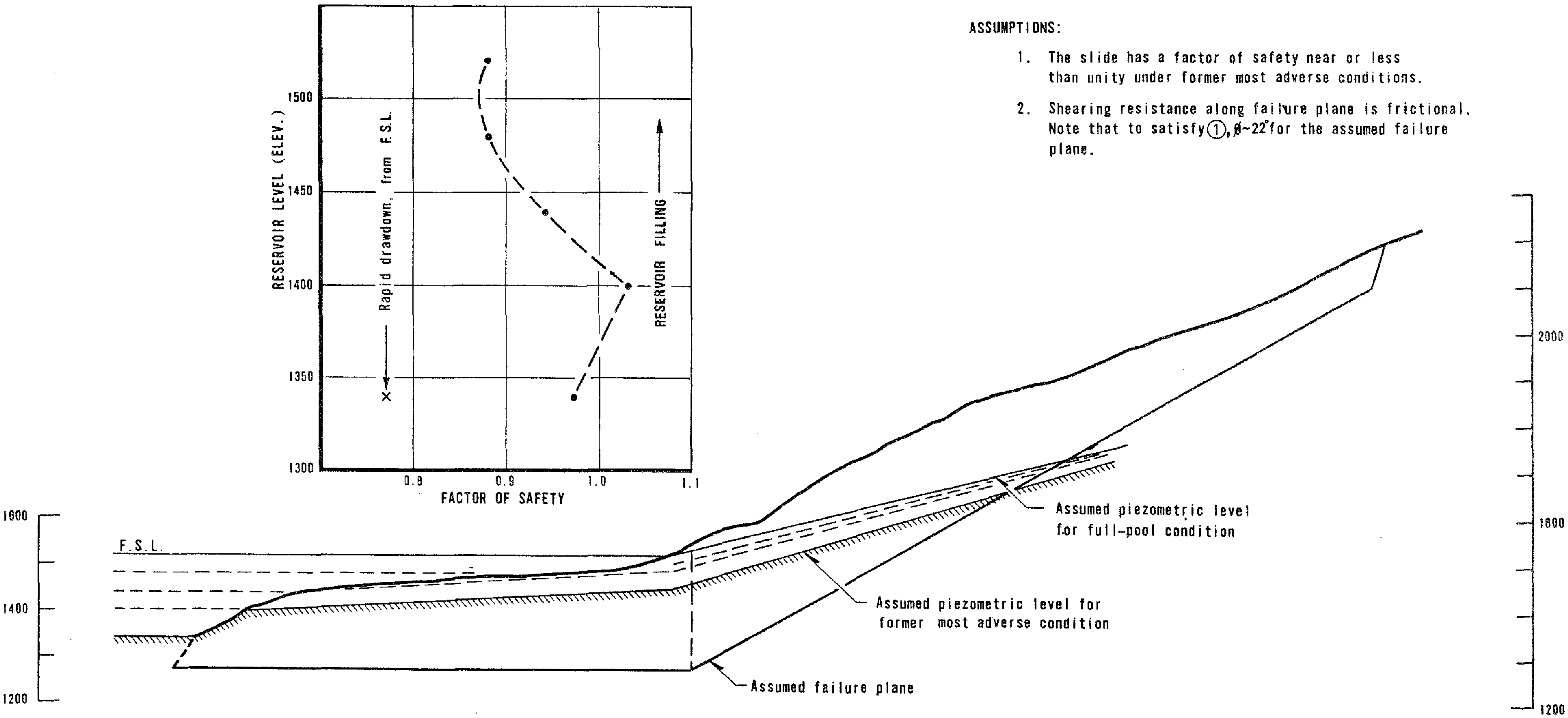
EFFECT OF THE PROPOSED RESERVOIR ON SLOPE STABILITY

5.1 GENERAL

The effects of partial flooding on the valley slopes of the Peace River and its tributaries are briefly discussed below.

Flooding of the toe of an existing slide or slump area, whether it be in overburden or bedrock, will generally reduce stability provided the flooding extends above the pre-flood water table. This results largely from the reduction in effective stresses at the toe of the slope. At the same time the forces tending to cause movement are relatively little affected by partial flooding, and thus overall stability is reduced. This mechanism is illustrated in Fig. 11 on the following page. The resistance to shearing along pre-existing slide planes can be assumed to be close to the residual value, and the rate of movement of any reactivated slide would probably be slow. However, rapid movements of the upper part of a slide together with intact material above it may occur.

Fig.11 portrays a homogeneous material with the existing groundwater table above the present river level, but below the final reservoir level. It is important to note that raising the reservoir level initially increases the factor of safety until the reservoir level is a little above the height of the existing groundwater. Any further rise in the reservoir level decreases the factor of safety. Very slow drawdown tends to increase the stability to that before filling, while rapid drawdown can decrease the stability. Although Fig.11 shows the case of a block slide along a pre-existing failure surface, colluvial, slumped or slide debris material overlying intact material are similar situations.



AN EXAMPLE OF THE EFFECT OF
RESERVOIR FILLING ON SLOPE STABILITY

FIGURE 11

This type of mechanism also applies to slopes which are close to sliding, although they may show little outward sign of being so. A reduction in shearing resistance at the toe of the slope along a plane of weakness which may already exist (e.g. a shear plane) or is in the process of developing by progressive failure, may be sufficient to cause a slide. However, rapid slides can result where this occurs.

As discussed in Section 3.3 the overburden and shale bedrock is not homogeneous but is in fact a layered system with horizontal permeabilities substantially greater than vertical permeabilities. Perched water tables are common. Thus Fig.6 is intended only to portray a principle, and runs the risk of being too simplistic. There is no substitute for considering each slope on its own merits and with a suitably conservative approach. The piezometric pressures along existing or potential failure planes must be measured before one can reliably predict the initial improvement in stability shown above.

Raising the groundwater table within or above the shales and the resulting reduction in effective normal stress may (as described on page 21) also result in some further disintegration of the shales, and reduction in shearing resistance by development of swelling pressures. It is debatable how far any significant changes would extend into the slopes, but presumably one factor upon which it would depend would be the extent of flooding. However, there is little doubt that flooding would adversely affect the highly disintegrated and weathered outer zone causing at least shallow slides to occur. Some slaking of the shale within the five feet operating range will also likely occur, caused by wetting and drying.

In general, the formation of a reservoir will reduce the erosive capability of the river because water velocities would be decreased. However, wave erosion effects would be increased. It is anticipated that beaching due to wave erosion will not be a major cause of shoreline regression in this reservoir, but allowance has been made for it (Section 7.2).

Based on experience elsewhere it can be expected that most of the slides directly resulting from inundation by the reservoir will occur within the first one to three years of initial filling.

5.2 EFFECT OF INUNDATION ON THE CACHE CREEK SLIDE

Based on observed piezometric pressures (Appendix C), a check on the effect of flooding on the stability of the lower slide area was included in the analysis (Appendix B. Fig.B2). With piezometric pressures along the failure plane compatible with those measured within the slide debris (i.e. W.L.1.), the analysis showed an increase in the factor of safety of 10 percent or more. With piezometric pressures along the failure plane compatible with those measured immediately below the slide debris (i.e. W.L.4.), the analysis showed a slight decrease in the factor of safety of 4 per cent or less. However assuming a residual Ø - value (13°) a minimum factor of safety of 1.50 was obtained for the latter case after flooding, which was greater than the corresponding value of 1.32 for the former case. Thus the analyses carried out to date have indicated little adverse affect on the toe area as a result of flooding.

Reservoir filling would have no effect on the upper slide area which would remain in its marginally stable state.

5.3 SIGNIFICANCE OF THE FSL ELEVATION

Profiles showing the location of the bedrock contact along the north and south banks the Peace River and the Moberly River are shown on Dwgs.6 to 8. The profiles are based on information gathered in the field programs carried out from 1975 on, including 40 measured geological sections and 116 helicopter measured sections. Further information was added from the low bank (privately owned) properties inspected in 1977. Although a reasonable tolerance should be assumed for inaccuracies in measuring elevations from section to section, the overall picture provided by the drawings is believed to be sufficiently reliable for the purposes of this report. The mileage on the drawings is a river mileage not floodline mileage.

The location of the bedrock contact and the overlying basal gravel aquifer is important required data for predicting the effect of reservoir flooding on bank stability. If the FSL is lower than the bedrock contact it can be expected to have no effect upon the stability of the overlying overburden. It can only affect the bedrock itself, and if this fails, the overburden indirectly. Whether the bedrock will fail will depend upon:

1. the existence of planes of weakness (e.g. pre-existing shear planes etc.), and whether they are on such an horizon as to allow failure to occur. It follows that the smaller the exposed height of rock actually inundated, the less there

is of the planes of weakness being at the most critical location.*

2. the piezometric pressures that have been experienced in the past, and whether they are exceeded as a result of flooding such that the safety factor is reduced to less than unity. It is likely that many though not all of the high slopes have experienced greater piezometric pressure in their toe areas than any included by the proposed flooding, and this becomes more probable as one proceeds upstream and the depth of flooding decreases.

To illustrate the above it is of interest to compare the potential effect of flooding upon two types of slopes. Type A is typified by the steep high shale slopes on the north side downstream of Tea Creek. Type B typified by the equally high and steep slopes on shale on the south bank opposite Attachie (Mile 61). In both cases the FSL is in shale.

Although some seepage is evident, the 2:1 slopes downstream of Tea Creek appear to be relatively dry. The lower piezometer in hole 40-1 was installed at el.1450 and has been dry since installation. Thus the slopes stay essentially in place either because of a permanently low water table or because the controlling shear strength at depth is greater than residual.

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* The fact that there are relatively few failure in bedrock within the reservoir area is strong evidence that, for the most part, where planes of weakness occur they are not in a critical horizon for present conditions.

Thus if there are planes of weakness in the slopes they may not be well developed or at a critical horizon (for present conditions)*. The reservoir will inundate the bottom of the slopes by 150 feet or so to el.1515 and lacking contrary evidence could effect one or more planes of weakness by introducing uplift pressures greater than those experienced in the past at this location. The effective permeability of the shale would govern the rate of build-up of these pressures, but with time the resulting decrease in stability could result in a slide.

The 2:1 and somewhat steep slopes opposite Attachie have a foundation of shale extending up to about el.1550. The overlying basal gravel is an acquifer which as discussed in Section 4.3 has likely experienced very high water pressures. It is reasonable to assume that some of these water pressures have in the past been transmitted to the shale. The fact that the shale has not already failed shows that the controlling shear strength must be greater than residual (in this case, a lot greater).

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* The occasional small well defined slide has developed on the steeper slopes suggesting that planes of weakness of <u>limited</u> extent do exist. The extensive debris covering the lower half of the slope at Mile 40 appears to have a multiple origin. Large slides in shale have occurred elsewhere on extensive failure planes close to the contact of the Dunvegan Sandstone (e.g. Cache Creek, and the east bank of the Pine River). However the field work to date in both Site C and E reservoir areas has revealed <u>no evidence of large slides in the body of the shale well below the contact.</u> The lower Cache Creek Slide is not deep seated.

The reservoir will inundate the shale by about 95 feet, and is unlikely to introduce uplift pressures greater than those already experienced.

Thus inundation to FSL 1515 is more likely to affect the toe of the Type A shale slopes downstream of Tea Creek than the Type B slopes opposite Attachie. On the other hand the risk of a slide unrelated to reservoir flooding (i.e. a slide effecting the upper part of the slope) is much greater at Attachie than downstream of Tea Creek.

The reservoir survey showed Type B slopes to be in the majority. Also for the most part FSL 1515 falls below the shale gravel contact along the high banks. Table 3 below has been derived from Dwgs. 6 to 8.

TABLE 3: RELATION OF FSL 1515 TO SHALE GRAVEL CONTACT

(Expressed as percentage of shoreline length).

	FSL below contact	FSL above contact
High* banks	(73 mls) 84%	16%
Low banks	(81 mls) 64%	36%

Thus the stability of the high banks should be relatively little affected by reservoir flooding.

Where the FSL inundates the shale contact and falls against the overburden it may induce

* High banks refer to those slopes which extend from the river up to the plateau (el.2000 and higher) without any conspicuous lower terraces Low banks refer to those sections of the shoreline which are terraced, the slope of the terrace usually being no greater than 150 feet high. There are virtually no banks of intermediate height.

piezometric pressures in the overburden higher than previously experienced and thus initiate a slide.Since many of the overburden slopes currently have a questionable stability (and many overburden slides have occurred) the risk of a slide occurring is proportionately higher.

Only 16 percent of the total length of high bank shoreline will be subject to inundation of the overburden (75% of this portion occurs in the Moberly River).

A little over one third of the low bank shoreline will be subject to inundation of the overburden, but half of this is in the Hudson Hope area (Section 5.4).

With the exception of the Hudson Hope area, at no location would the bedrock be inundated by more than 55 feet (and this would rarely occur); normally the contact would not be inundated by more than 15 to 20 feet. Also, since the top of the basal gravel is observed to be in the general range of 50 to 100 feet above the bedrock, the overlying clays and silts are unlikely to be inundated. Thus, even in those areas where the FSL is above the bedrock the direct effect on stability should be minimal. Some wave erosion of the gravel is expected depending on exposure, which could indirectly result in slope failures but these should rarely extend beyond classification B (see Section 6.2)

Where the FSL inundates landslide debris, some reactivation leading to mostly minor movement can be expected. The field survey showed that 18 miles of shoreline (i.e. 12% of the total) are covered with landslide debris at el. 1515. Most of this debris occurs on the Moberly River and the downstream 15 miles of the Peace River section.

5.4 THE HUDSON HOPE AREA

Borehole 85-1 is located in Hudson Hope; the location is shown on Dwg.11 sheet 3 and the log is summarized on Dwg.5. The hole was drilled to a depth of 223 feet, 120 feet below river level without encountering bedrock. Piezometric levels in the deeper horizons have been recorded at el. 1500 to 1507. (i.e. below the proposed FSL).

The banks are steep, approximately 100 feet high, and have a capping of 20 to 30 feet of gravel over silts and clayey silts. Several springs are emerging from the bank above river level, some at the gravel silt contact and others from within the silts (presumably water bearing sands within the silts). Calcareous tufa is being deposted on the banks below these springs.

The reservoir at FSL 1515 will result in inundation of the lower 25 to 30 feet of the bank. A nominal amount of erosion due to wave action must be expected. Further, the springs may represent perched water tables and the flooding could increase the piezometric levels at critical horizons above those previously experienced.* Thus pending further investigation it must be assumed at this time that bank protection (e.g. a berm) will be needed to ensure that the banks are not adversely affected.

A similar condition exists on the uninhabited south bank opposite Hudson Hope (Mile 80-85).

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* The maximum recorded HWL at Hudson Hope <u>prior</u> to the construction of the WAC Bennett Dam is el.1499. This was for a relatively short duration and is not comparable to permanent flooding.

Chapter 6

SHORELINE STABILITY CLASSIFICATION

6.1 GENERAL

Previous reports (p.3) have attempted to classify the shoreline in terms of the magnitude and type of slide hazard and the probability of its occurrence. The principles behind this classification have been maintained in this report although some refinements have been made in expressing probability and in presentation. Essentially three improvements have been made:

1. It is recognized that a given slope may be subject to more than one type of slide potential.

2. The previous probability rating tended to be misleading since it did not, for example, recognize that the probability of overburden slides should be rated on a different scale than the probability of bedrock slides. The rating system described below is simpler, easier to apply and readily permits comparison of predicted reservoir bank stability with that of the shoreline today.

3. The presentation has been changed to clearly show the classification of the shoreline after flooding in relation to the classification today.

These changes in no way invalidate the previous work. It served its purpose well in comparing alternative schemes and selecting a preferred one. However, having selected a preferred scheme we are now charged with showing the effect of flooding caused by this scheme in the clearest possible way; hence the changes.

6.2 DESCRIPTION OF SYSTEM

The classification used for describing slide potential (magnitude and type) is given below:

Type A indicates areas of shoreline where regression of the shoreline due to erosion and/or sloughing will be slight. It also covers those slopes where slides may occur but they are not expected to affect the slope above FSL.

Type B indicates those areas of the shoreline where small slides on terrace slopes (normally not higher than 150 feet) or minor slumping of higher slopes may occur (effecting not more than 150 feet of those slopes). It also includes reactivation of slide debris at the toe of higher slopes.

Type C indicates those areas of the shoreline where large slides may occur involving failure through bedrock either totally or in part. Such a slide would be greater than 150 feet high and would therefore likely be restricted to the high bank areas (slopes 500 feet high or more). However, the slide category could affect any portion of the slope and is not restricted to a toe failure. The category includes reactivation of existing slides, provided the other criteria are satisfied.

Type D indicates those areas of the shoreline where large slides occur affecting overburden only. Such a slide would be greater than 150 feet high and would therefore likely be restricted to the high bank areas (slopes 500 feet high or more).

However, the slide category could affect any portion of the slope, and is not restricted to a toe failure. The category includes reactivation of existing slides, provided the other criteria are satisfied.

Sections of shoreline which currently show no erosion or sloughing at el.1515 are designated with a hyphen.

The probability of occurrence of a given slide potential is designated using asterisks:

Two Asterisks (**) indicates high probability and is usually applied to those slopes which are known to be presently active.

One Asterisk (*) is applied to those areas where a slide or slides of a designated classification should, for the purpose of studies concerning wave hazards, land use and safelines, be assumed to occur during the next 70 years or within the life of the development. Overburden slopes showing no current signs of instability but which are appreciably steeper than long term slopes are usually put into this category. Bedrock slopes where there is reasonable evidence that their stability is uncertain or could become uncertain are also placed in this category.

No Asterisk indicates a possible potential, but one of low probability. Except for residential safeline studies it may be assumed that slide(s) would not occur within the next 70 years. Overburden slopes which are steeper than long term slopes but not appreciably so, and bedrock slopes which have a lengthy history of stability are placed in this category.

An example of a classification involving a large slope consisting of overburden overlying bedrock is given below:

B** C D*

With this slope there is a high probability of type B slides occurring (less than 150 feet high), and a possible but low probability of a type C slide (a large slide involving bedrock), also it should be assumed that a type D slide (a large slide in the overburden covering the slope) could occur during the next 70 years or within the life of the development.

The presentation given on Dwg.11 shows two classifications plotted within the same circle. The classification in the upper half refers to future stability after flooding to FSL 1515. The classification in the lower half refers to future stability under existing conditions.

The shoreline classification results given on Dwg.11 are conservative with respect to probability of occurrence, and should be regarded as interim. Depending on the results of further field exploration and studies, some shoreline areas may be redesignated in a lower risk category.

It is important to stress that the classification is not intended to indicate that an entire classified area will slide, but that the potential exists for a slide or slides to occur. It is expected that much of the shoreline area will continue to behave as it has in the recent past.

A detailed account of the predicted volume of future individual slides and their velocity does not fall within the terms of reference of this study. However, a brief and general account is given below to provide further clarification of types B, C, and D slides.

Type B slides would have an estimated upper limit of 100,000 c.y. of material falling to the bottom of the slopes, but would generally involve less than 25,000 c.y. The velocity would depend on whether the failure was a 'first time' slide or a reactivation, the height of fall, the nature of the slide material and several other factors. Velocities could range from barely perceptible movement for reactivated slides and slumps, to 10 to 20 mph for a low bank failure, to 70 to 80 mph for a failure from the top of the steep high bank falling through several hundred feet.

It is estimated that Type C slides, involving bedrock, would normally fall within the range 100,000 c.y. to 1 million c.y., but would generally be less than 500,000 c.y. It is recognized that large Cache Creek type slides involving close to 100 million c.y. have occurred in the past, but there is strong evidence that they are related to extensive planes of weakness which have developed in the upper shale horizons close to the Dunvegan Sandstone (see footnote, p.37).

Type C slides have a potential for high velocity; the height of fall will be a most important factor. Debris falling 200 feet down a 1:1 slope could reach speeds of 50 to 60 mph, and on a 2:1 slope 20 to 40 mph. Velocities would tend to be lowest for reservoir induced slides because the height of fall would be minimized.

Type D slides (involving overburden only) can be very large. The Attachie slide involved approximately 20 million c.y. with 10 million c.y. falling into the river. However, airphoto interpretation and field inspection has shown that, although this slide mechanism is common, overburden slides of this magnitude have rarely occurred in the past in the reservoir area.* Thus although the possibility of a future slide of such dimensions must be recognized, the following estimated quantities are believed to be more typical:

a) A moist, intermediate slope (e.g. 2.5:1 to 3.5:1), possibly with a hummocky surface but no topographic expression of a previous large slide. Slopes of this type occur on the south bank of the Peace River (Mile 61-65, and 52 - 57) and on the Moberly River. Type D slides developing on these slopes have a potential for up to 5 million c.y. of debris falling to the bottom of the slope.

b) Dry, steeper (e.g. 2:1) slopes such as at Mile 57 - 60 and Mile 78 on the north bank, or failed flatter slopes such as at Mile 44 have a potential for 500,000 to 2 million c.y. of debris falling to the bottom of the slope.

* Similar slide scars of lesser magnitude have been observed at the mouth of the Moberly River, on the Beatton River, and on some minor tributary streams on the north bank of the Peace downstream of Taylor.

Type D type slides tend to bottom out above gravel and thus there is a general relationship between potential velocity and the difference in elevation between FSL and the top of the gravel. Even so, should the Attachie Slide have occurred with the reservoir full it could have reached a velocity of 50 to 60 mph at FSL. Velocities would be lowest for reservoir induced slides when the height of fall would be minimal.

6.3 RESULTS OF CLASSIFICATION

The results of the classification for both present conditions and reservoir conditions as provided on Dwg.11 are summarized on the following Tables 4 and 5.

Considering the 73 miles of high bank shoreline the classification shows:

1. The direct effect of flooding on the stability of high banks is minimal, as would be expected from the discussion in Section 5.3.

2. The greatest change is in the type C size category. Approximately 8 miles of shoreline have changed from C to C*; this includes those sections of shoreline at the mouth of the Moberly River and on the northbank upstream of the damsite.

3. Over 50 miles of the existing high banks are classified D*, and it should be assumed that the overburden on some part of these slopes (not all) will fail within in the next 70 years. The classification shows that this risk exists at present

Table 4 SHORELINE CLASSIFICATION BY MILES

PRESENT CONDITIONS

RIVER	SHORELINE	LOW BANK (81.1)				HIGH BANK (73.1)					
		A	B	B*	B**	C	C*	C**	D	D*	D**
PEACE RIVER	R	0	13.7	9.6	2.1	20.1	0	0.8	0	16.2	1.2
	L	0	5.7	2.0	13.0	11.4	3.8	0	2.7	6.0	0
MOBERLY RIVER	R	0	0	0	0	6.7	0	0	0	6.7	0
	L	O	O	0	0	4.5	0	0	0	5.2	0
CACHE CREEK	R	0	1.1	0	0	1.8	0	0	0	1.8	0
	L	0	1.2	0	0	4.1	0	0	0	4.1	0
HALFWAY RIVER	R	0	0	0	0	6.0	0	0	0	6.0	0
	L	0	0	0	0	6.7	0	0	0	6.7	0
FARREL CREEK	R	0	0	0.8	0	1.5	0	0	0	1.5	0
	L	0	0	0	1.9	0	0	0	0	0	0
SUB - TOTAL	R	0	14.8	10.4	2.1	36.1	0	0.8	0	32.2	1.2
	L	0	6.9	2.0	14.9	26.7	3.8	0	2.7	22.0	0
TOTAL	(R & L)	0	21.7	12.4	17.0	62.8	3.8	0.8	2.7	54.2	1.2

Note: High bank classification B not included

Table 5 SHORELINE CLASSIFICATION BY MILES

AFTER FLOODING

RIVER	SHORELINE	LOW BANK (81.1)				HIGH BANK (73.1)					
		A	B	B*	B**	C	C*	C**	D	D*	D**
PEACE RIVER	R L	5.5* 9.8	7.6 7.8	19.2 8.6	2.1 13.0	20.1 5.7	0 9.5	0.8 0	0 0	16.2 8.7	1.2 0
MOBERLY RIVER	R L	0 0	0 0	0 0	0 0	5.0 3.3	1.7 1.1	0 0	0 0	6.1 3.0	0.5 2.2
CACHE CREEK	R L	0 0	1.1 1.2	0 0	0 0	1.8 4.1	0 0	0 0	0 0	1.8 4.1	0 0
HALFWAY RIVER	R L	0 0	0 0	1.1 0.7	0 0	6.0 6.7	0 0	0 0	0 0	6.0 6.7	0 0
FARREL CREEK	R L	0 0	0 0	0.8 0	0 1.9	1.5 0	0 0	0 0	0 0	1.5 0	0 0
SUB - TOTAL	R L	5.5 9.8	8.7 9.0	21.1 9.3	2.1 14.9	34.4 19.8	1.7 10.6	0.8 0	0 0	31.6 22.5	1.7 2.2
TOTAL	(R & L)	15.3	17.7	30.4	17.0	54.2	12.3	0.8	0	54.1	3.9

Note: High bank classification B not included and that it is largely unaffected by flooding. Approximately 40 percent of the shoreline so classified occurs along the Moberly and Halfway Rivers.

Considering the 81 miles of low bank shoreline, the classification shows:

1. Approximately 53 miles of the slopes are classified B, B* or B** (i.e. vulnerable to slides up to 150 feet high) under existing conditions. This increases to 65 miles after reservoir flooding.

2. The greatest change is in the B* category which increased from 12 to 31 miles. Again it must be stressed that not all of the slopes classified B* are expected to fail, but there is a reasonable risk of some of them failing during the next 70 years.

3. Seventeen miles of low bank shoreline have been classified B**. Most of these banks are currently distressed, and there is a high probability that they will fail during the next 70 years. The affect of flooding is to accelerate these failures

6.4 EFFECTS OF SHORELINE GROUND MOVEMENTS ON THE RESERVOIR

The effect of bank sliding on the reservoir does not fall within the terms of reference of this study.

It is understood that B.C. Hydro have initiated a study of slide-induced wave hazards.

Previous reports have commented upon the effect of a blockage caused by sliding which could back water up above the FSL. The possibility of such a blockage in the main stem of the reservoir along the Peace River now appears remote, but some risk does remain in the tributary streams including the Moberly (above Mile 3) and Halfway Rivers.

B.C. Hydro will carry out all necessary protective measures to eliminate any slide potential in close proximity to the dam abutments. For this reason the portion of shoreline on Dwg.11 adjacent to the dam has been left unclassified.

6.5 EFFECT OF REDUCING THE FULL SUPPLY LEVEL

The effect of reducing the full supply level from el.1515 to el.1500 as required by the terms of reference (p.1) is outlined below.

The extent of inundation above the bedrock contact (i.e. against overburden) on 'high' banks would be reduced from 12 to 9 miles*, and on 'low' banks (terraces) from 28 to 14 miles. The risk of reservoir induced overburden slides would be reduced accordingly.

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* This mileage has been estimated from Dwgs.6,7 and 8, and converted to shoreline mileages. Reminder: the total 'high' bank shoreline is 73 miles and the total 'low' bank shoreline mileages is 79 miles.

There would be little or no change in the following aspects:

- the risk of high bank slides and associated waves.

- any potential for large reservoir induced slides on the north bank in the vicinity of the dam and at the mouth of the Moberly River would be slightly reduced, but the potential would still require investigation.

- the protection of the Hudson Hope banks, pending further investigation.

Chapter 7

THE SAFELINE CONCEPT AND ITS APPLICATION

7.1 THE CONCEPT

The "safeline" is a conservatively located line beyond which the security of residents and residential improvements can be reasonably assured. It is not to be confused with the predicted extent of shoreline regression (sometimes called the "breakline"), which does not allow for any margin of safety. The "takeline" is a term used for land use; it is a line used to designate land which is to be purchased or restricted in future use as a direct result of the creation of the reservoir. Being governed by land use considerations, the location of the takeline can differ from the safeline location. If, for example, the reservoir flooding renders a farm unit uneconomic, the takeline may well be located beyond the safeline.

The terminology described above was established on the Arrow Lakes Development in cooperation with B.C. Hydro's Land Division. It has subsequently been applied to other reservoirs in British Columbia.

The distance between the safeline location and the predicted possible extent of shoreline regression represents the margin of safety. The selection of the margin of safety depends on:

- the cause of shoreline regression (i.e. erosion versus slides).

- the history of former landslides in the area (as it effects the probability of new slides occurring and the reactivation of old slides).

- the extent of the field investigation of the area (some properties are so heavily vegetated as to inhibit detailed surface inspection; other properties have been drilled, and the soil conditions are well established).

- the degree of uncertainty regarding future slope stability.

In this study the margin of safety was built into computations for the safeline location by using suitably conservative parameters. A more conservative approach was followed when shoreline regression by sliding was to be allowed for than when beaching erosion only was to be considered. The reason for this was two-fold. First, as a result of the Arrow Lakes Project a considerable amount of reliable experience on beaching experience has been obtained*. Second, it was reasoned that the regression caused by erosion would occur slowly, and in the unlikely event of an error on the unconservative side it would not constitute a threat to the security of residents.

Safelines are primarily concerned with residential use of land. Thus the actual or intended land use was not taken into account in establishing the location of the safeline.... it was simply assumed that the land would be used for residential purposes. Some roads and farm or industrial facilities may continue below

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* Thurber Consultants Ltd. (Jan. 1977). Shoreline Stability Assessment, Arrow Reservoir; a draft report for B.C. Hydro.

the safeline provided there is no permanent inhabitation.

A safeline represents the setback that would be required should a slide of a given size classification occur, thus the probability of occurrence of such a slide is not taken into account in establishing its location.

It is important to emphasize that failure and erosion of the entire land area falling below the safeline, or for that matter the breakline, would not occur. Occasionally parts of the area are expected to fail. It is not possible to predict where these failures would occur, and hence the entire suspect shoreline area is placed within the safeline.

Because the location of the safeline has been chosen conservatively, it should be understood that after a suitable period of operation, consideration can be given to an adjustment in the location. Since the reservoir drawdown interval of the Site C Development would be very small, based on experience elsewhere any low bank property that does not develop evidence of instability during the initial 5 years of reservoir operation could in general be considered as remaining stable throughout the life of the reservoir. In other words, with a few exceptions the 5-year period could be considered as a full scale test of the stability of the property. Accordingly inspections of the entire reservoir area should be carried out on an annual basis in the early summer.

Because slope instability is prevalent in the reservoir area, a safeline location has been established which allows for slope instability developing from natural causes unrelated to reservoir action. However, the safeline location does not allow for the possibility of any adverse effects resulting from man-made physical changes such as the placement of fill on the shoreline, excavation, and increased irrigation. These can result in rapid slope failures without any apparent warning on any body of water.

The term "minimum safeline" has been used where no future instability or erosion is predicted. The minimum safeline setback established by B.C. Hydro is 100 feet or that corresponding to el.1520, whichever is greater.

7.2 BASIS FOR LOCATING THE SAFELINE

A three-step procedure was used in establishing the safeline location:

1. Assembly of suitably detailed topographic plans. We have used 1"-200' B.C. Hydro plans (1967) for Hudson Hope (provided to us by the Village) reduced to 1"=400'; D.O.H. plans (1967) 1"-400' for the area between Hudson Hope and Bear Flats; and B.C. Surveys and Mapping Branch plans (1972) 1"-1000' with expanded coverage, for specific areas downstream of Bear Flats.

2. Obtaining suitably detailed geological data. Following the reconnaissance of 1973, field surveys were carried out in 1975, 1976 and 1977. The 1977 survey work was site specific; with one exception, every pertinent privately owned property was inspected, and pertinent data from previous surveys was checked.

The safeline was then located on the plans in the field (see item 3).

3. Establishing criteria which would readily permit establishing safelines on both low and highbank sections of the shoreline. These criteria draw fully on observations on the behaviour of slopes along the Peace River and in reservoirs elsewhere.

The procedure for establishing low bank safelines for various typical cases encountered around the reservoir is illustrated in Fig.12. Where applicable, beach slopes were selected by reference to Fig.13.

7.2.1 Low Bank Safeline

The normal procedure for locating a safeline above a slope consists of the following steps:

1) establish an after-slide profile compatible with known soil, rock, and groundwater conditions.

2) superimpose the after slide profile on the existing bank profile such that the area of material removed from the top of the slope equates with the area of slide material deposited at the toe of the slope (beyond the existing profile). This assumes that little or no slide material is carried out into the reservoir in suspension and that no bulking occurs. The point of intersection of the after slide profile above the existing slope is the breakline and represents the probable maximum regression of the slope resulting from slides.

3) Locate the safeline on the landward side of the breakline. The additional setback represents the safety margin.

The safeline setback from the top of the existing slope is commonly chosen to be 1.5 to 2 times the breakline setback*. The selection of twice the breakline setback permits the elimination of step 2, since the after slide profile drawn from the toe of the existing slope would locate (approximately) the safeline directly. This procedure is on the over-conservative side but is permissible for low banks where the safeline setback is normally small. If the setback appears excessive an adjustment is made. Thus the profiles for the various cases shown on Fig. 12 are after slide profiles not failure planes.

The eight cases shown on Fig.12 cover virtually all of combinations of soil and rock conditions, groundwater situations and extent of in undation applicable to the low banks or terraces around the Site C Reservoir Area. Of these, case 3 is by far the most common.

The selection of the 5:1 slope for shale is compatible with observed failed slopes (i.e. of shale slide material) and with theoretical analysis assuming a high groundwater table. The selection of 4:1 slopes for silt and clay can be similarly supported.

The selection of the beach slope for cases 4,6,7 and 8 is based on an extensive study of

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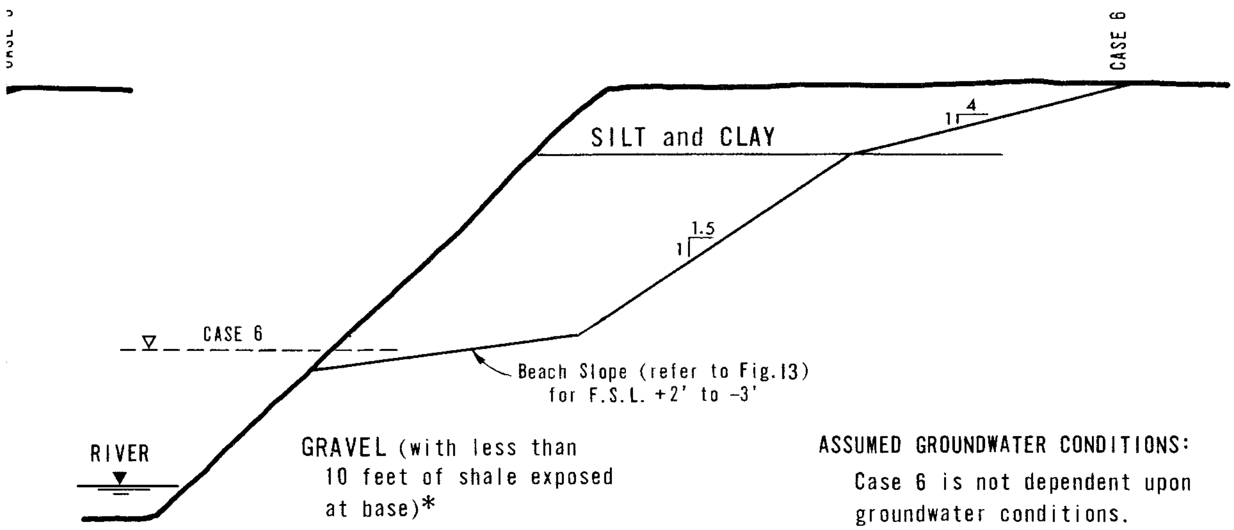
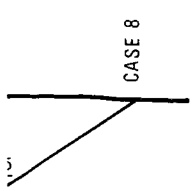
* Except where shoreline regression results from beach erosion only, when no additional setback beyond the breakline is required.

THURBER



* NOTE:
Where 10 to 20 ft. of shale is exposed, an additional 1:2 setback in shale was allowed (as in Case 2). Where more than 30 ft. of shale is exposed, Case 4 was applied.

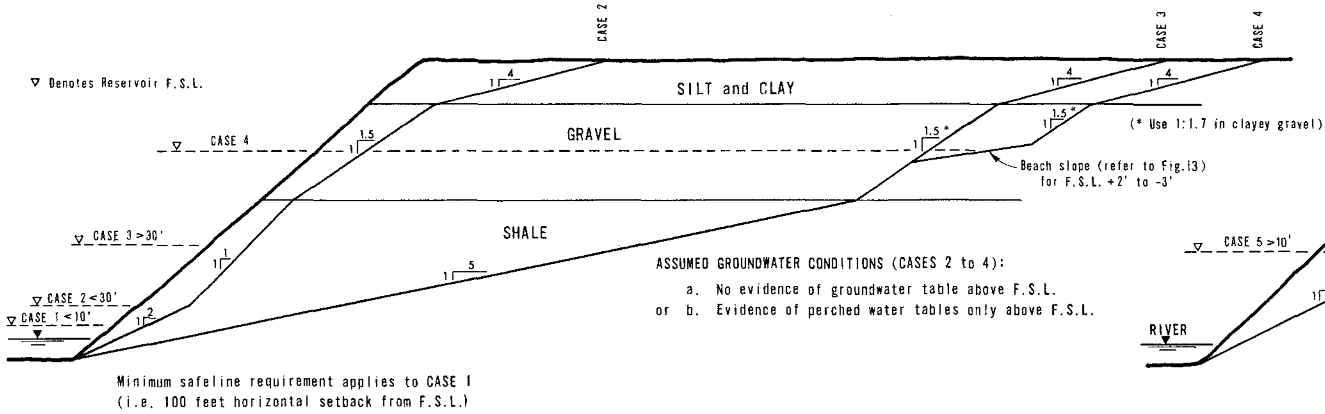


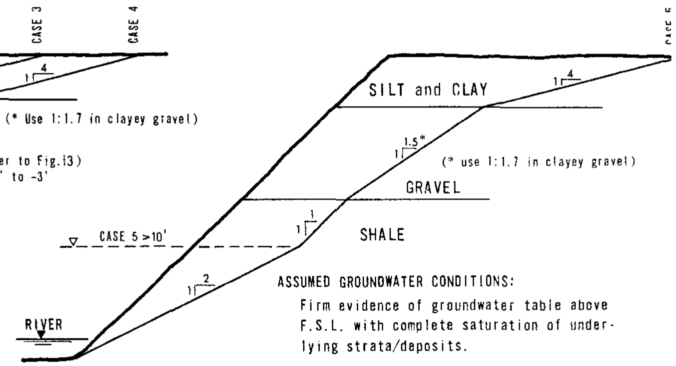
NOTES:

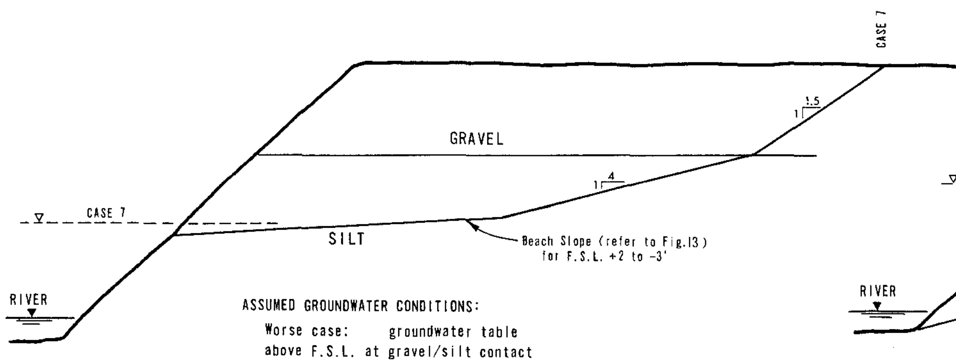
1. Where sloughed material masked the underlying shales at toe of slope, it was assumed (for the purpose of establishing a safeline) that the material formed a thin veneer, and no allowance for its presence was made.
2. Where the computed safeline fell on a steep slope the safeline was established (for Cases 2 to 8) at the break in slope above.
3. On flat slopes (such that no safeline intersect was obtained), the minimum safeline requirement was applied, i.e. 100 feet horizontal setback from F.S.L.

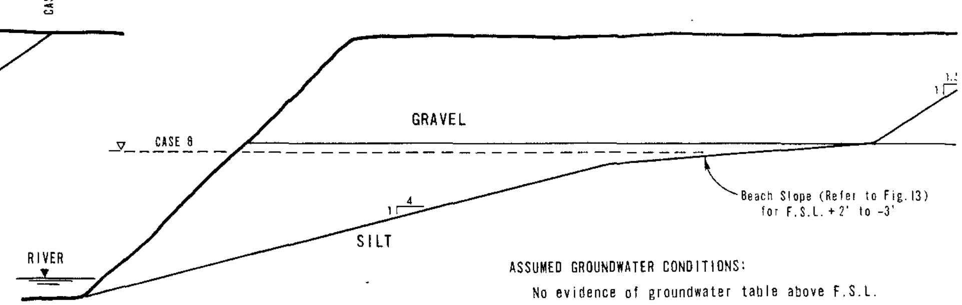
ALLOWANCE FOR REGRESSION OF LOW BANKS
SUBJECT TO SLIDES AND /OR EROSION BY BEACHING

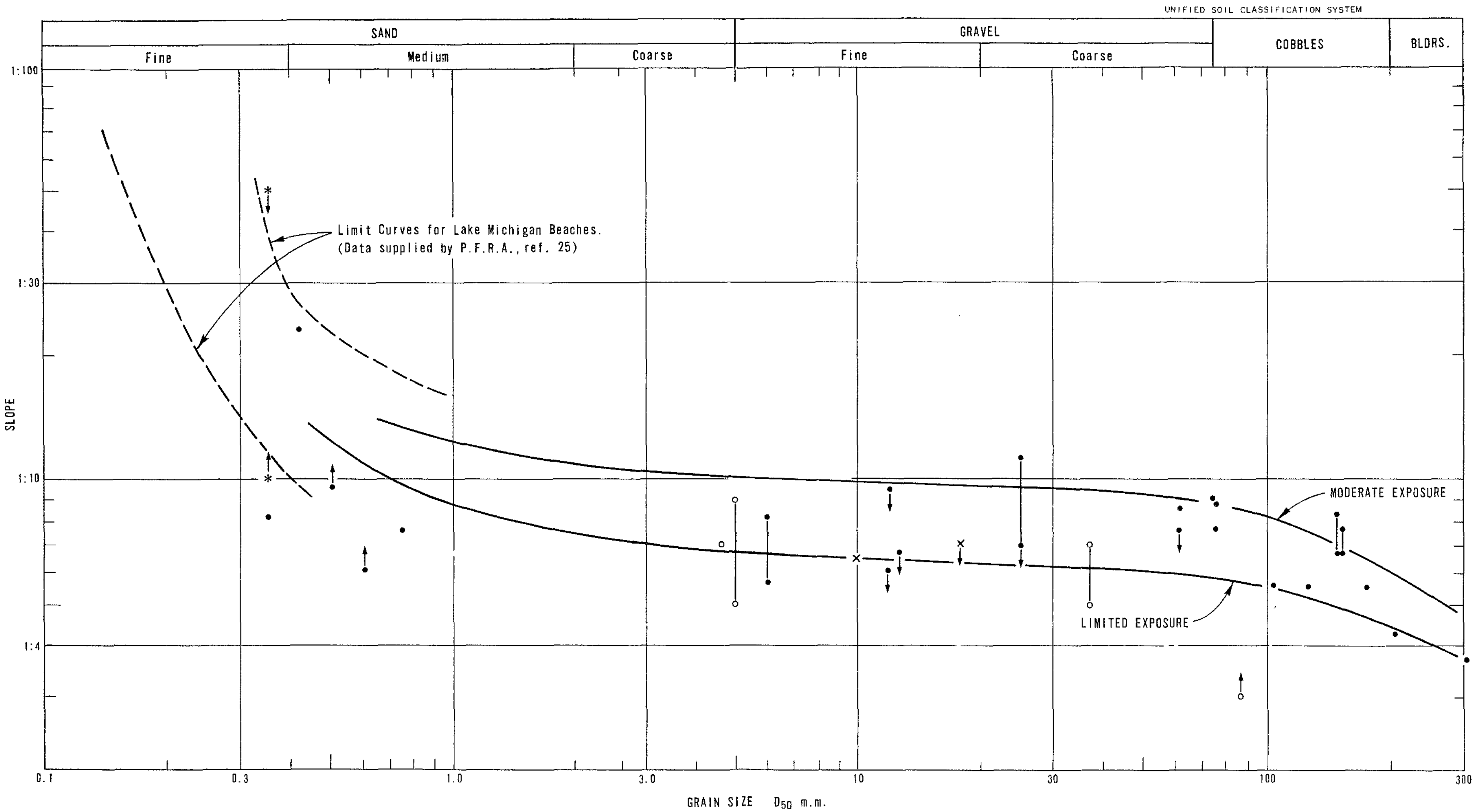
FIGURE 12


CASE 2
CASE 3
CASE 4
▽ Denotes Reservoir F.S.L.
SILT and CLAY
GRAVEL
SHALE
(* Use 1:1.7 in clayey gravel)
CASE 4
CASE 3 >30'
CASE 2 <30'
CASE 1 <10'
Beach slope (refer to Fig.13) for F.S.L. +2' to -3'
ASSUMED GROUNDWATER CONDITIONS (CASES 2 to 4):
a. No evidence of groundwater table above F.S.L.
or b. Evidence of perched water tables only above F.S.L.
Minimum safeline requirement applies to CASE 1
(i.e. 100 feet horizontal setback from F.S.L.)


SILT and CLAY
(* use 1:1.7 in clayey gravel)
GRAVEL
SHALE
CASE 5 >10'
RIVER
ASSUMED GROUNDWATER CONDITIONS:
Firm evidence of groundwater table above
F.S.L. with complete saturation of under-
lying strata/deposits.


CASE 7
GRAVEL
CASE 7
SILT
Beach Slope (refer to Fig.13) for F.S.L. +2 to -3'
RIVER
ASSUMED GROUNDWATER CONDITIONS:
Worse case: groundwater table
above F.S.L. at gravel/silt contact


GRAVEL
CASE 8
Beach Slope (Refer to Fig.13) for F.S.L. +2' to -3'
SILT
RIVER
ASSUMED GROUNDWATER CONDITIONS:
No evidence of groundwater table above F.S.L.



LEGEND:

- ● Data from Arrow Lakes
- × Data from Whatshan Lake
- ○ Data supplied by P.F.R.A. (ref. 25)
- ∗ Data supplied by U.S. Corps of Engineers (ref. 25)

NOTE:
The points with arrow indicate readings taken on beaches of "banded" materials and exhibiting a stepped profile. The D_{50} size of the coarser (arrow down) or finer (arrow up) material has been plotted against the average slope.

GRAIN SIZE (D_{50}) VERSUS BEACH SLOPES
FOR INLAND LAKES AND RESERVOIR

FIGURE 13

measured beach slopes in other lakes and reservoirs. The results of this study are summarized on Fig.13 which depicts a relationship between beach slope, grain size, and degree of exposure in inland waters*. The curve designated 'limited exposure' corresponds to an effective fetch of approximately 1.5 miles. The curve designated 'moderate exposure' corresponds to an effective fetch of approximately 5 miles. Note that for sediments within the D50 size range 1 to 100 mm, the degree of exposure is a critical factor governing the beach slope.

7.2.2 High Bank Safeline

Where geological and groundwater conditions have been reasonably established, the three-step procedure described under Section 7.2.1 is closely followed. The safeline setback from the top of the slope is normally taken as 1.5 times the breakline setback.

The after-slide profile is based on observations of failed slopes around the reservoir area and are similar to those given in Fig.12.

Where the geological and/or groundwater conditions are so uncertain as to make the above approach invalid, the observed long term stable profile of 5:1 is used. Step 2 procedure is followed to compute the breakline and this is taken as the safeline (i.e. no additional setback is added because of the conservative nature of the profile).

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* Taken from Thurber Consultants report on "Arrow Lakes Shoreline Stability Assessment", a draft report to B.C. Hydro, 1977.

7.3 LOCATION OF SAFELINE

The terms of reference of the study require a safeline to be established on all privately owned property on the Peace River upstream of Tea Creek. A separate album of prints (Dwg. 15-2-62-12, sheets 1-15) has been prepared showing the location of the safeline for these properties under reservoir conditions.

Most of these properties are in low bank areas, and the following summary facts applies to these areas. There would be almost 58 miles of low bank shoreline in private lands, corresponding to a total area of 1490 acres between the safeline and FSL 1515. Of this amount 693 acres are "compulsory" (i.e. equivalent to a minimum safeline with a 100 feet setback applied throughout). The average safeline setback is 215 feet from FSL 1515.

It is of interest to extrapolate from these values to produce a total estimate for the safeline area for all 81 miles of the low bank shoreline around the reservoir. A total of 2100 acres has been computed, assuming the same 215 feet average set back.

Dwg.15-2-62-12 also shows the safeline location for the high bank areas at Mile 78 and Mile 58 - 60. This location was established based on existing conditions in these areas, and is unaffected by reservoir flooding. (Flooding does increase the probability of occurrence of Type D slides at Mile 58). The setback of the safeline from the top of the bank is typically 500 to 600 feet.

Although this study does not include the location of a safeline on the high banks around the entire reservoir area, an approximate estimate of the area falling below such a safeline has been made assuming the 500 to 600 feet setback to be typical throughout (a reasonable assumption):

(i)	Area of high banks between FSL 1515 and top of bank........	approx.	10,000 acres
(ii)	Area between top of bank and safeline	approx.	5,000 acres
		TOTAL	15,000

Thus the total area falling below the safeline for both high and low banks is approximately 17,000 acres.

It is important to stress that the high bank safeline location for the 'without reservoir' condition is identical to that for the 'with reservoir' condition. Even without the reservoir, residents and residential development should not encroach below the safeline for high banks. Thus the safeline area for the low banks (2100 acres) represents the only area of land that would be restricted by reservoir flooding.

It is also emphasized that active slides are expected to effect only a small portion of the land falling below the safeline (Section 7.4) and, like today, much of it could be used for non-residential purposes (e.g. grazing and farming). Furthermore some relaxation of the low bank safeline can be expected after a period of 5 years operating experience.

7.4 PREDICTED EXTENT OF ACTIVE SLIDING

A discussion and estimate of typical volumes of individual slides, and the velocities that the moving slide masses could develop has been included in Section 6.2.

This section is concerned with the total area of active slides that would typically occur within the safeline at any given time. For this purpose an active slide is defined as one which either removes or distresses vegetative cover; it also includes land affected by wave erosion.

7.4.1 Low Banks

Of the safeline area (2100 acres) that portion falling below the breakline* is estimated to be approximately 1200 acres. This assumes that the setback from the FSL to the breakline averages 60% of the safeline setback; an assumption which is in accord with the discussion of low bank areas given in Section 7.2.

Slightly less than 17 miles out of a total of 81 miles of low banks show signs of current distress (i.e. are classified B**) and the effect of flooding would be to accelerate bank sliding in these areas. Thus approximately 20% or 240 acres of the land falling below the breakline could be so affected.

An additional allowance of up to 150 acres for beach erosion (where applicable) and the occasional B* slide should be provided.

Thus the total low bank area involved in active slides or erosion is expected to be in the range of 250 to 400 acres.

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* See Section 7.1, p 51, for definition

7.4.2 High Banks

Since the high banks will for the most part not be affected by inundation to FSL 1515 (refer Sections 5.3 and 6.3), the total area of active sliding will be similar to that existing today.

The estimated total currently active area on the high slopes is 300 acres. This is comprised of 14 slides on the Peace River and 10 slides on the tributary rivers, ranging in size from 100 acres (Attachie) to 5 acres and less. The extensive river eroded banks in shale were not included in this estimate because the formation of a reservoir will remove the erosive capabilities of the river.

A conservatively selected upper limit equal to three times the currently active area should adequately allow for:

- natural variation from present conditions

- some additional direct effect of the reservoir on bank stability. Approximately 11 miles out of a total 73 miles of high banks will be inundated above the bedrock contact. Most of these are on the Moberly River. No clays or silts will be flooded, only the basal gravel.

- some bank erosion caused by wave action

- human activities

Thus the total high bank area involved in active slides is predicted to be in the range 300 to 900 acres; a small fraction of the safeline area. However as discussed on p.20 many of these high bank slopes are presently marginally stable, particularly with regard to shallow slides, and this condition would remain after flooding (hence the safeline).

Chapter 8

SUMMARY AND RECOMMENDATIONS

8.1 SUMMARY

8.1.1 Existing Conditions

a) Landslides have played a significant role in the development of the Peace River Valley in the vicinity of the reservoir area. Some of the valley slopes are still marginally stable. Since the beginning of the century the following significant slides are known to have occurred (p.17-18)

- In the early 1900's, movement or reactivation of the Cache Creek Slide (Mile 51).

- In 1957, failure of the north bank at Taylor Flats resulting in collapse of the previous highway bridge. The slide occurred in shale.

- In 1973, the Attachie Slide on the south bank at Mile 62. The slide occurred in the overburden and blocked the river for 10 hours.

- In 1974, failure of the north bank at Mile 31, cutting off the B.C.R. mainline. The slide occurred in overburden.

b) Groundwater is and has been an important factor governing the occurrence of landslides along the Peace River. The generally flatter slopes in both overburden and bedrock on the south side of the river are believed to be the result of more adverse groundwater condition (p.16). The occurrence of deep seated overburden slides is commonly associated with the basal gravel overlying the bedrock (p.19). Groundwater pressures in the gravel are believed to fluctuate seasonally by considerable amounts (p.15). Quantitative data pertaining to groundwater conditions in the reservoir area is at present very limited.

c) Slides involving bedrock (shales) have occurred infrequently relative to slides in overburden. However, the largest slide to have occurred in the reservoir area (i.e. the Cache Creek Slide) developed in the shale bedrock. The requisites for the development of a large slide in shale are believed to be the presence of pre-existing shear planes in a critical location paralleling the bedding of the shales and sufficiently adverse groundwater conditions. The shearing resistance along these planes can approach residual values. Residual Ø-values as low as 13° and averaging 14° have been obtained from a detailed investigation at Mile 31 (p.13 and Fig.10).

d) Occasional small slides in shale have occurred on steep slopes suggesting that pre-existing shear planes of limited extent do exist at various horizons on exposed slopes (probably a result of relaxation). However where large slides have occurred on the Peace and Pine Rivers, the failure plane developed close to the contact with the Dunvegan Sandstone. The field work to date has revealed no evidence of large slides developing in the body of the shale well below this contact.

e) The Cache Creek Slide (p.27-31), involving up to 90 million c.y. of shale and sandstone, had a slope of approximately 3:1 before failure. Movement of slide debris had the potential for attaining an estimated maximum velocity of 50 mph. The top of the valley slope retrogressed up to 600 feet as a result of the slide. The slide originated with a failure of the upper half of the slope, depositing debris at the toe of the bank. The reported early 1900's movement is believed to have involved this toe debris. Analyses of the upper west section of the slide mass based upon measured piezometric levels showed this portion to be marginally stable (p.28). Similar analyses of the toe area indicate a more stable condition (p.28).

f) The Attachie Slide (p.21-25) involving up to 23 million c.y. of overburden had a slope of 3.3:1 before failure. The slide occurred rapidly and the slope regressed 200-250 feet. The failure plane followed brittle plastic clay close to the contact with the basal gravel. The slide bottomed out approximately 200 feet above river level and 8 to 11.5 million c.y. of slide debris fell to river level attaining an estimated velocity of 70-80 mph (p.24). Assuming reasonable peak piezometric pressures within the gravel, an analysis (Appendix B) indicated the shear strength at the time of failure to be considerably below peak-value (p.13). The slide area was probably progressively failing for many years and is believed to have been triggered by high piezometric pressures brought about by a slow melt of greater than normal snowfall peaked by heavy rain. The area immediately adjacent to the Attachie Slide is in a condition of incipient failure, depending on seasonal groundwater conditions.

g) The 750 feet high shale slopes downstream of Tea Creek (Mile 39 to 42) stand as steep as 2:1. A limited amount of bank sliding has occurred, but the slope has a relatively stable appearance. One borehole showed the slope to be dry to within 100 feet of river level. It is believed that the slopes remain essentially in place at this steep angle either because of a low water table or any existing shear planes on a critical horizon are not well developed (p.16 and 35).

h) From observations of existing slopes it is concluded that the long term stability of 5:1 slopes (within the reservoir area) can be assured under the most adverse conditions that can reasonably be expected.

8.1.2 Conditions after Flooding

a) Flooding the toe of an existing slide or slump area can increase or decrease stability depending on the magnitude of piezometric pressures along the failure plane prior to flooding. Where the piezometric pressures in the toe are greater than those that would be induced by flooding, an increase in stability can result (p.31). However, these piezometric pressures must be measured before one can reliably predict such a result.

b) Based upon measured piezometric pressures, the stability analyses carried out to date have indicated that flooding would have little adverse affect on the toe area of the Cache Creek Slide. Reservoir flooding would have no affect on the upper slide area, which would remain in its present marginally stable condition.

c) The location of the bedrock contact and the overlying basal gravel aquifer is important required data for predicting the effect of reservoir flooding on bank stability. If the proposed reservoir level is lower than the aquifer it can be expected to have no effect upon the stability of the overburden. The bedrock may fail (and indirectly any overlying overburden) depending upon:

   - the occurrence of pre-existing shear planes at a critical horizon, and,

   - induced piezometric pressures higher than those experienced in the past, sufficient to reduce the safey factor to less than unity.

   If the proposed reservoir level inundates the bedrock contact, flooding the gravel and possibly the overlying silts and clays, it may, (in addition to affecting the bedrock) induce piezometric pressures in the overburden higher than those previously experienced and thus initiate a slide.

d) The proposed reservoir shoreline will consist of approximately 73 miles of high banks (greater than 500 feet) and 81 miles of low banks (less than 150 feet). The reservoir level will fall below the bedrock contact for 84 percent of the 'high' bank shoreline and 64 percent of the 'low' bank shoreline (p.37). With the exception of the Hudson Hope area at no location would the bedrock be inundated by more than 55 feet and hence the silts and clays over the gravel are unlikely to be flooded.

e) With some noticeable exceptions (item f below), it is considered probable that where the reservoir level falls below the bedrock contact on 'high' banks flooding will have minimal adverse effect. It is likely that these high banks have experienced greater piezometric pressure in their toe areas than any to be induced by the proposed flooding, and this becomes more probable as one proceeds upstream and the depth of flooding decreases.

The risk of occasional large slides (mostly in overburden) unrelated to reservoir flooding is appreciable (refer 8.1.3b) and potential effects remain to be considered. The frequency of these slides can be expected to be similar to that experienced in the past.

f) The steep high shale banks on the north side downstream of Tea Creek to the damsite (Mile 39 to 42) are relatively dry. One piezometer installed at el.1450, 65 feet below the proposed reservoir level recorded no water. Lacking contrary evidence, flooding could induce piezometric pressures greater than those experienced to date and thus could initiate sliding along one or existing shear planes (p.35 to 37). The size of any slides would depend in part on the extent of existing shear planes. There is encouraging evidence that shear planes in the lower part of the slope at this location are not extensive (8.1.1d). However confirmation of this condition is required through a detailed exploration program of the area supplemented by the results of continuing exploration of the abutments at the dam site. A general account of the size and velocity potential of shale slides is provided on p.44 and summarized in the table following page 70.

g) The affect of the flooding by more than 10 feet against a 'low' bank, even if below the bedrock contact, should be assumed to be adverse, unless it can be shown that the existing piezometric levels in the bank are greater than those that would be induced by reservoir flooding.

h) Bedrock was not encountered in the Hudson Hope area. Drilling was terminated 120 feet below river level. Current piezometric levels in the deeper overburden stata are a few feet below the proposed reservoir level (FSL 1515). Springs are emerging from the bank above the proposed reservoir level, but they may represent perched water tables. Pending further investigation, it should be assumed that bank protection (e.g. berms) will be needed to ensure the banks are not adversely affected (p.39).

i) As a general rule, slide and slump areas which are reactivated by reservoir flooding can be expected to move relatively slowly and involve small volumes.

The field survey showed that 18 miles of shoreline (12% of the total) would be prone to this type of movement. Most of these slump areas are on the Moberly River and the downstream 15 miles of the Peace River. Only a fraction of this shoreline would actually move; inundation would improve stability of some sections (p.33).

j) The reservoir would be filled to FSL 1515 in a little less than one month and would be permanently operated within the range +2 to -3 feet. With this operation, it is expected that most slides directly resulting from the reservoir will occur within the first one to three years.

k) Reservoir flooding, in addition to affecting slope stability, may result in sloughing of the disintegrated and weathered outer zone of the shale banks. Beaching due to wave action on overburden would also occur although it would be a minor factor in shoreline regression.

8.1.3 Classification of Shoreline Stability

a) A revised classification system has been used (p.41). The system permits classification of the shoreline in terms of magnitude and type of slide potential and the probability of its occurrence (p.42). The presentation facilitates comparison of present conditions with 'after-flooding' conditions (Dwg.11).

The classification should be regarded as interim; with further field exploration. Some shoreline areas may be redesignated in a lower risk category.

b) The results of the classification are summarized in Tables 4 and 5 (following p.46).

The classification confirms that the 'high' bank sections would be little affected by the proposed flooding except for 7 miles of shale banks in the vicinity of the dam, including the mouth of the Moberly River (refer 8.1.2 f). Over 50 miles of the existing high banks are so classified that it should be assumed the overburden on some of these slopes may slide within the next 70 years irrespective of whether the reservoir is flooded or not.

The classification of 'low' banks indicates the the effect of flooding is an increase from 53 miles to 65 miles of banks vulnerable to slides up to 150 feet high. Seventeen miles of these banks are currently distressed and there is a high probability of some parts of them failing within the next 70 years (the effect of flooding is to accelerate these failures).

c) The potential size and velocities of various slide categories are discussed on p.44 to 46. The table on the following page summarizes this discussion.

d) The possibility of a blockage of the main stem of the reservoir along the Peace River caused by sliding which could back water up above FSL 1515 is now considered remote.

Some risk remains in the tributary streams (p.48).

e) The effect of a reduction in the full supply level from el.1515 to el.1500 is summarized below:

- any potential of large reservoir-induced slides in the vicinity of the damsite would be slightly reduced, but the potential would still require investigation.

- the shoreline mileage of inundation above the bedrock contact would be reduced from 12 miles of 'high' bank to 9 miles, and 28 miles of 'low' bank to 14 miles. The risk of reservoir induced overburden slides would be reduced accordingly.

- the high bank slide potential which presently exists and is not related to reservoir filling (8.1.3 b) would remain.

- the protection of the Hudson Hope banks would remain a requirement, pending further investigation.

8.1.4 The Safeline

a) The safeline is a conservatively located line beyond which the security of residents and residential improvements can be reasonably assured. A safeline has been established for all privately owned land upstream of Tea Creek. All of this land occurs on the north bank and the bulk of it is low bank shoreline. A separate album of prints (Dwg.15-2-62-12, sheets 1-15) has been prepared showing the safeline location for these properties under reservoir conditions.

b) Because instability of the existing slopes is prevalent, the safeline has been located to include slope instability from natural causes unrelated to reservoir action. (However, no allowance has been made for any future adverse effects resulting from man-made physical changes and slide induced wave hazards).

c) The terms of reference of this study did not require the location of the safeline

THURBER

Table 6 SUMMARY OF SLIDE VOLUME AND VELOCITY POTENTIAL

SLIDE HAZARD CATEGORY (1)	POTENTIAL VOLUME (2)	POTENTIAL VELOCITY	COMMENTS
A. No to minimal erosion or sloughing	-	-	-
B. Slides not higher than 150 feet	Upper limit 100,000 c.y., generally less than 25,000 c.y.	Low bank failure, slow to 20 mph. High bank failure, up to 80 mph.	Applicable to high and low banks
C. Slides higher than 150 feet involving failure of bedrock	Upper limit 1 million c.y. (3), lower limit 100,000 c.y. Generally less than 500,000 c.y.	Reservoir induced failure, slow to 40mph (depending on mass). Other failures, up to 60 mph.	Applicable to high banks only
D. Slides higher than 150 feet involving failure of overburden only	Upper limit 20-25 million c.y. Generally not more than 5 million c.y. falling to bottom. Dry, steep slopes; not more than 2 million c.y.	Up to 70 mph depending on height of fall	Applicable to high banks only

Notes: (1) For complete description, refer to p.45

(2) For single slides

(3) Assumes conditions for very large (Cache Creek type) slides do not now exist in reservoir area

not require the location of the safeline for crown land and land downstream of Tea Creek (Mile 42), which includes most of the 'high' bank shoreline. However, the safeline location for all 'high' bank shoreline under reservoir conditions would be identical to that for existing conditions. In this respect the impact of the reservoir ranges from minimal effect (refer 8.1.2 e) to an acceleration of instability that would occur with time under natural conditions.

d) Approximately 58 miles of 'low' bank shoreline is privately held. The total area of this land falling below the safeline is 1490 acres, equivalent to an average setback of 215 feet from FSL 1515. Note that the minimum required safeline location for land where no regression is predicted is a setback of 100 feet (p.53).

e) A total of 81 miles of 'low' bank shoreline exists. The safeline area for all low bank property (crown and private) is computed by extrapolation to be approximately 2100 acres. Thus in the light of conclusion 8.1.4 c, aside from the increase in flooded area of 11,500 acres, the impact of the project is an additional 2100 acres of land falling below the safeline.

f) It is important to emphasize that failure or erosion of the entire area below the safeline will not occur. Although it is not possible to predict the location of those parts of the area which will fail, a prediction of the total area of active slides under reservoir conditions has been made (p.58-61). The total low bank area involved in active slides is expected to be in the range 250 to 400 acres (a maximum of 19% of the low bank safeline area). The total high bank area involved in actual slides is expected to be of the same order as that existing today and in the range 300 to 900 acres (a maximum of 6% of the high bank safeline area). However many of these high bank slopes are presently marginally stable, particularly with regard to shallow slides, and this condition would remain after reservoir flooding.

g) In accord with 8.1.2 j and based on experience elsewhere those portions of low bank property that do not develop signs of instability during the inital 5 years of operation may in general be considered as remaining stable throughout the life of the reservoir, and the safeline relaxed accordingly.

8.2 RECOMMENDATIONS

a) The findings of this report have been based on limited readings of the instrumentation (piezometers and inclinometers) installed in the 1977 drilling program. To confirm these findings and provide further background data, a program of scheduled readings should be established and followed.

b) The status of investigation of the Cache Creek Slide area is considered preliminary. The extent of the stability analysis is compatible with the limited data which has so far been obtained from the site. In order to confirm the findings we recommend a more detailed determination of the subsurface and groundwater conditions.

c) In accordance with conclusions 8.1.2 f, it is recommended that a detailed geotechnical investigation be carried out of high banks in the vicinity of the damsite to assess any existing or large slide potential. Where applicable, it is recommended that hydraulic model studies be carried out to assess the effect of slide-induced waves on the earthdam.

d) In view of the significance of the apparent relationship between the occurrence of large slides in shale and the presence of the Dunvegan Sandstone, it is recommended that a field examination of all large slide areas in the Peace River area, and all areas where shale with overlying sandstone is exposed on steep slopes be carried out to confirm the relationship.

e) The location of a safeline for the privately owned lands downstream of Tea Creek should be deferred where practicable, until the completion of the investigation described 8.2 b.

f) Should any further refinement or extension of the safeline location shown on Dwg. 15-2-62-12 be required, some additional ground inspection and topographic survey may be necessary.

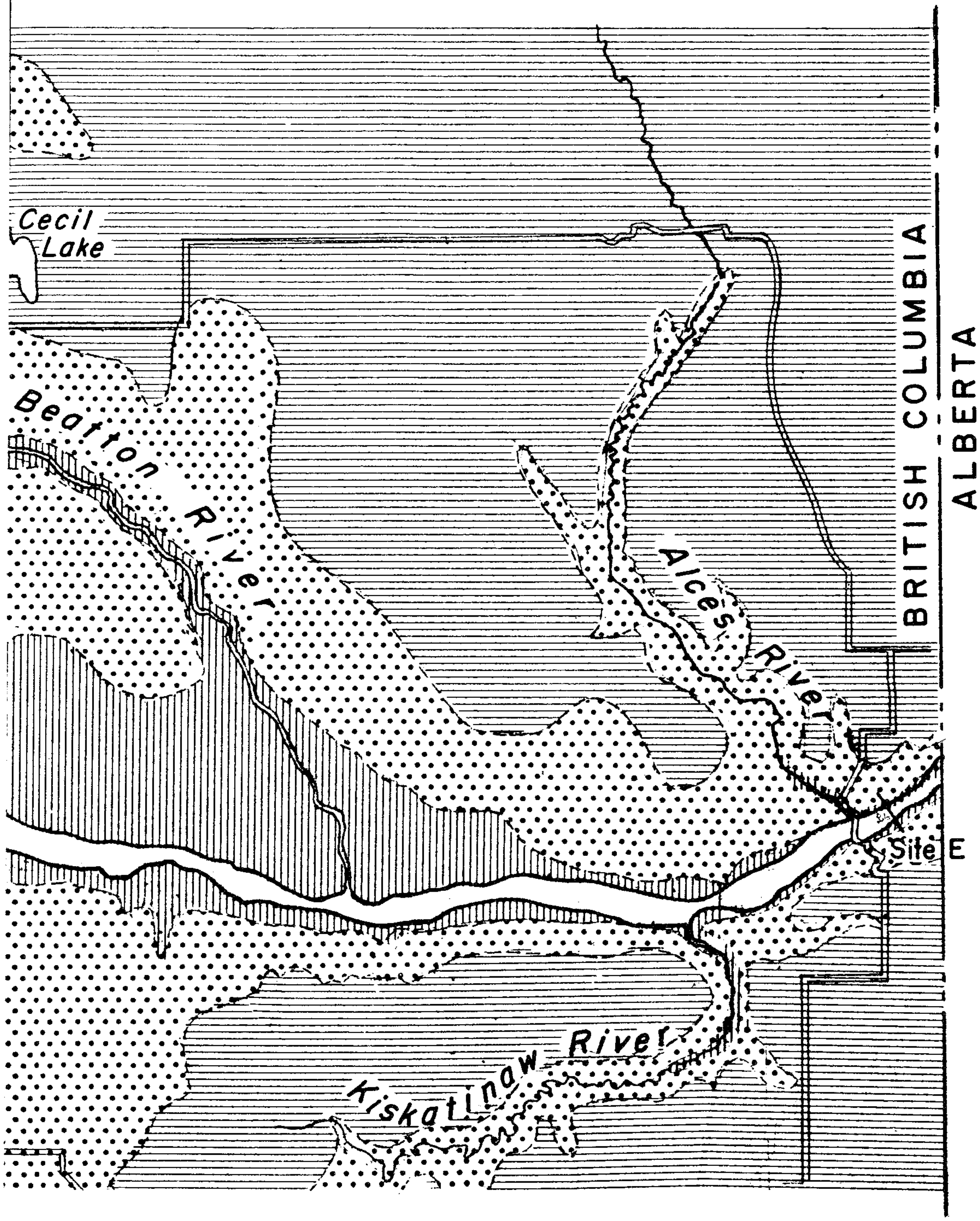
g) B.C. Hydro are carrying out a study of slide induced wave hazards. Information contained within this report and on supporting work sheets will provide a substantial basis for the required study data input on slide volumes and velocities. It is expected that as this study proceeds, detailed evaluation of specific localities will require further field work.

h) Following a decision to proceed with the Site C Development, a detailed geotechnical investigation of the Hudson Hope shoreline is recommended. The purpose of this investigation is to determine the requirements for shoreline protection. The investigation would include the preparation of underwater profiles.

i) In accordance with conclusion 8.1.3 f, and following the filling of the reservoir, a program of annual detailed inspections of the reservoir shoreline with particular emphasis on privately owned properties should be carried out. The purpose of these inspections would be to maintain a check on the performance of the reservoir shoreline and provide data for the 5-year revision of the safeline location.

A suitably designed program involving visual signs, delaying construction of boat ramps and campsites, etc. should be carried out to caution and deter people from using the reservoir for recreational activities during the first year of operation, particularly the spring season. Any campsites should be located with due reference to the findings of this report and any potential wave hazards. Industrial users of the reservoir (e.g. ferry crossings) should be similarly cautioned.

Safelines should be established for any lands passing from the crown to private or institutional hands.



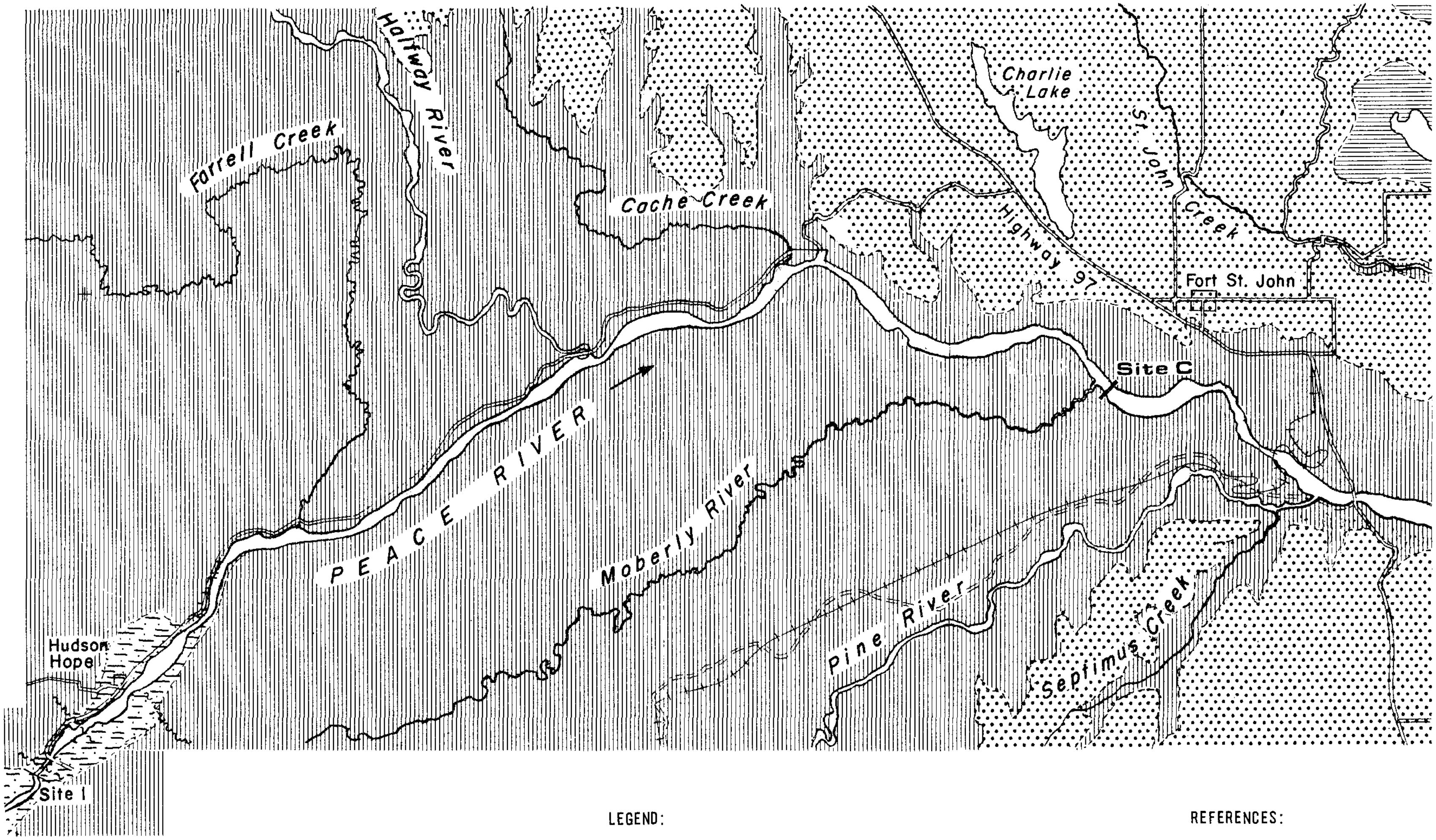
British Columbia Hydro and Power Authority

BEDROCK GEOLOGY

Site C Reservoir Shoreline Assessment

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	G. M.
TRACED	D. S.
DATE	Nov. 1977
APPROVED	E. C.M.
SCALE	1" to 4 Miles
DRAWING Nº	15-2-62-1

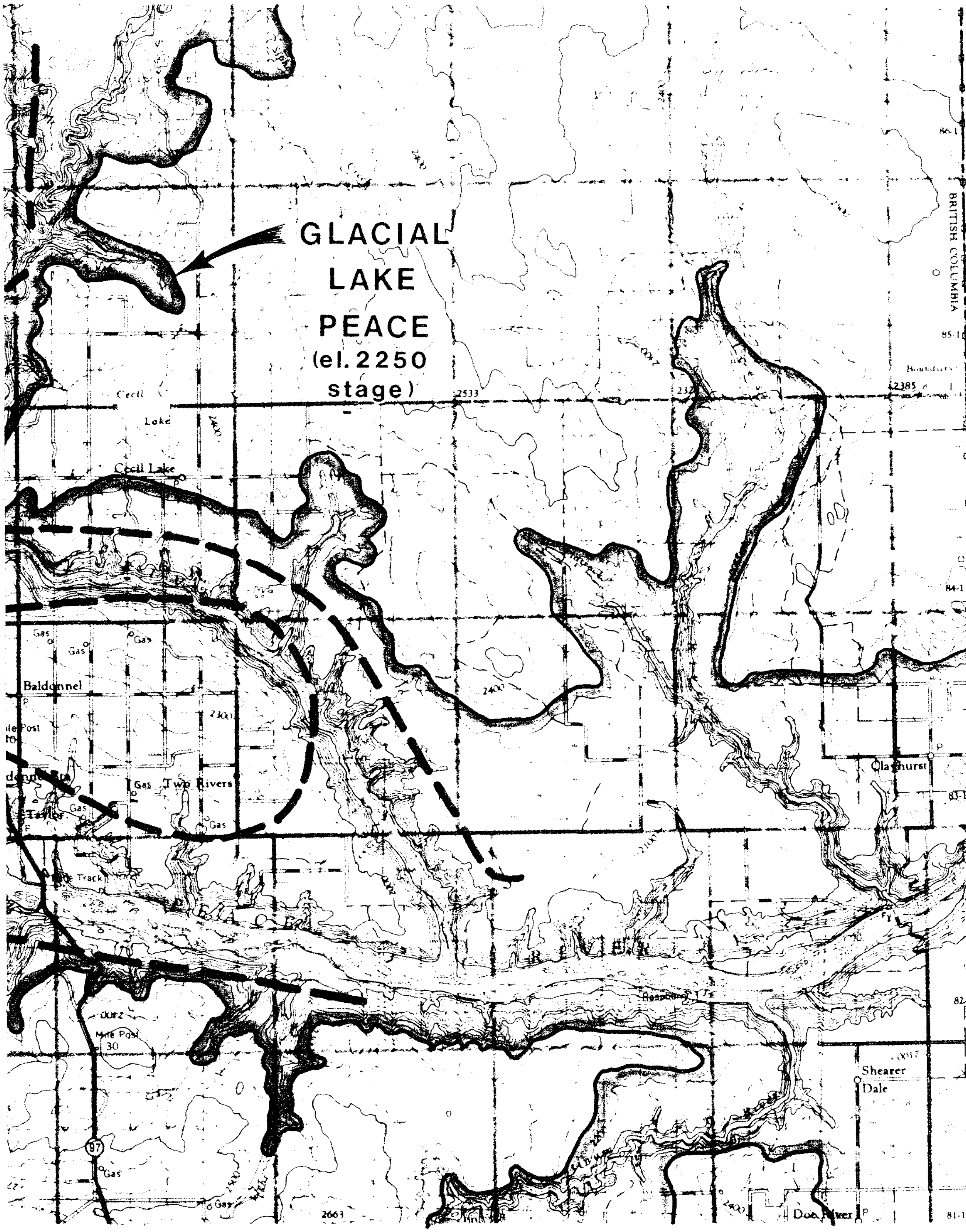


LEGEND:

- Shale with minor sandstone (Kaskapau For., Upper Cretaceous).
- Shale with minor thin sandstone members (Fort St. John Shale,) Shaftsbury For., Lower Cretaceous.
- Fine to coarse grained sandstone, with minor shale (Dunvegan For., Upper Cretaceous).
- Sandstone overlying shale (Gates and Moosebar Formation, Lower Cretaceous).
- ~~~~~ Fault or discontinuity.

REFERENCES:

This plan is a duplicate of IPEC Dwg. No. 1016-C14-D9, Plate II-2 (July 1972), but includes some modifications. It is based on G.S.C. Map 17-1958 (Charlie Lake) with original geology by E.J.W. Irish (1955-57).



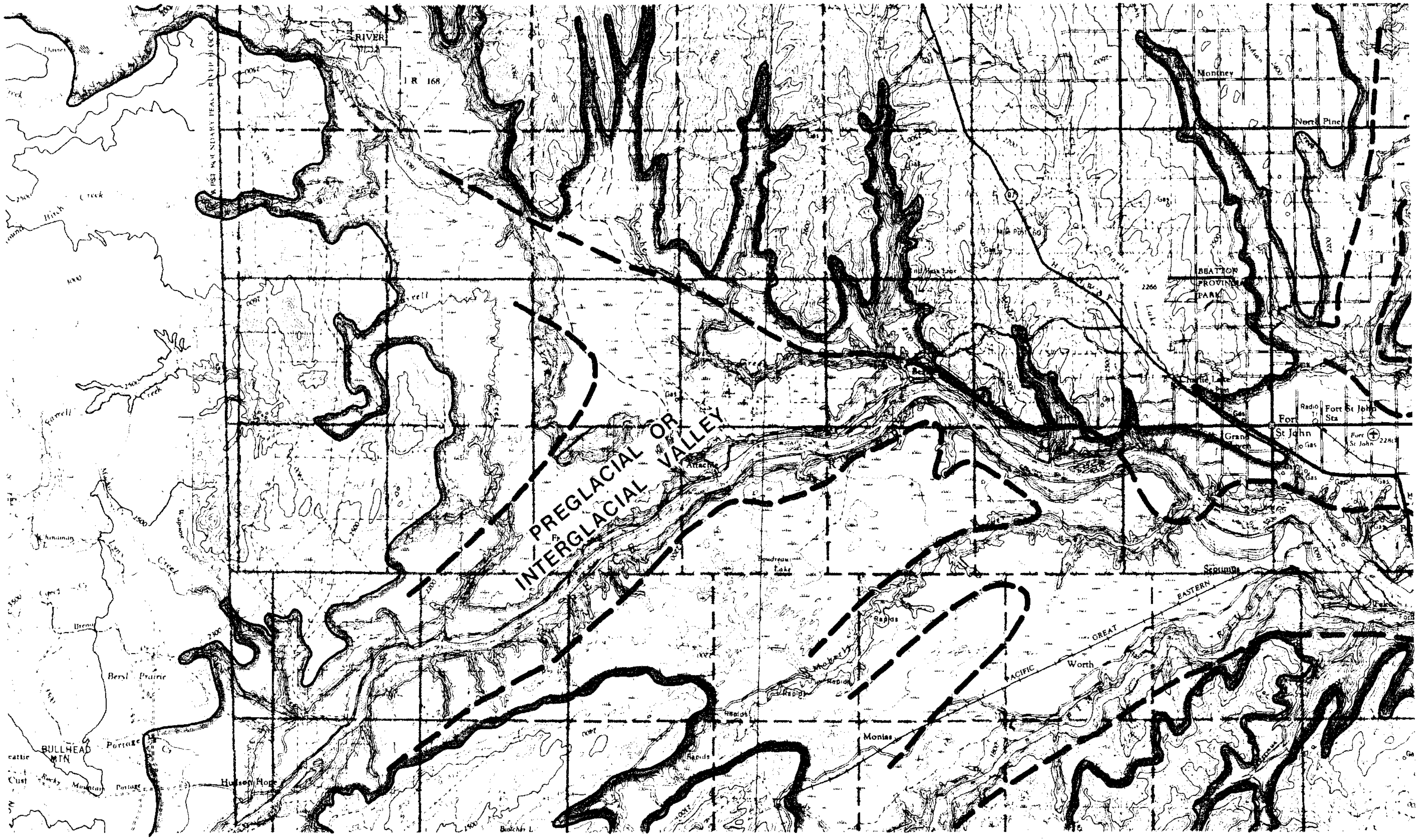
British Columbia Hydro and Power Authority

PREGLACIAL OR INTERGLACIAL PEACE RIVER
VALLEY AND EXTENT OF GLACIAL LAKE PEACE

Site C Reservoir Shoreline Assessment

D.W.S.	Feb. 1978
G.C.M.	1" = 4 Miles
15-2-62-2	

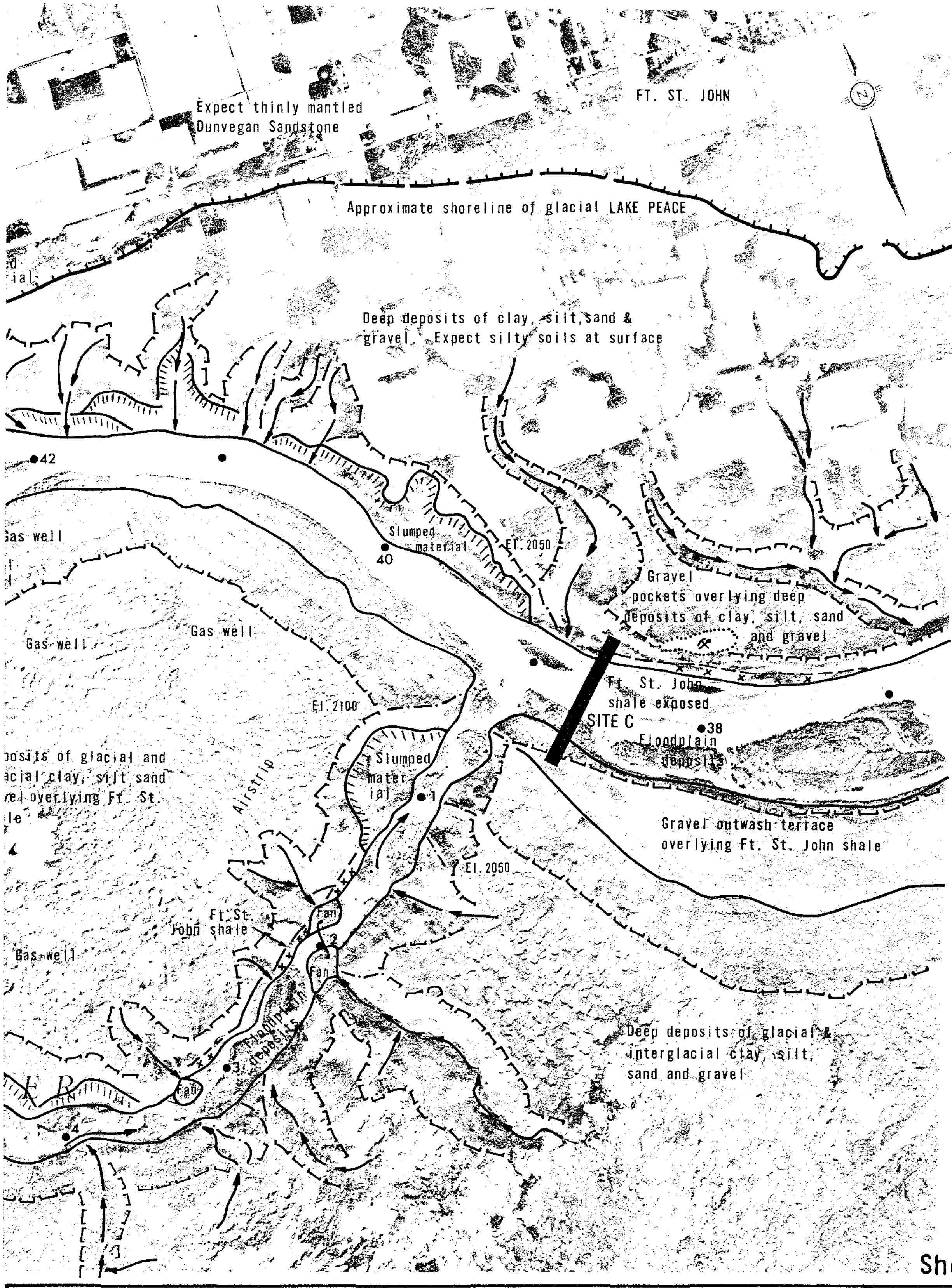
THURBER CONSULTANTS LTD., Geotechnical Engineers



REFERENCE:

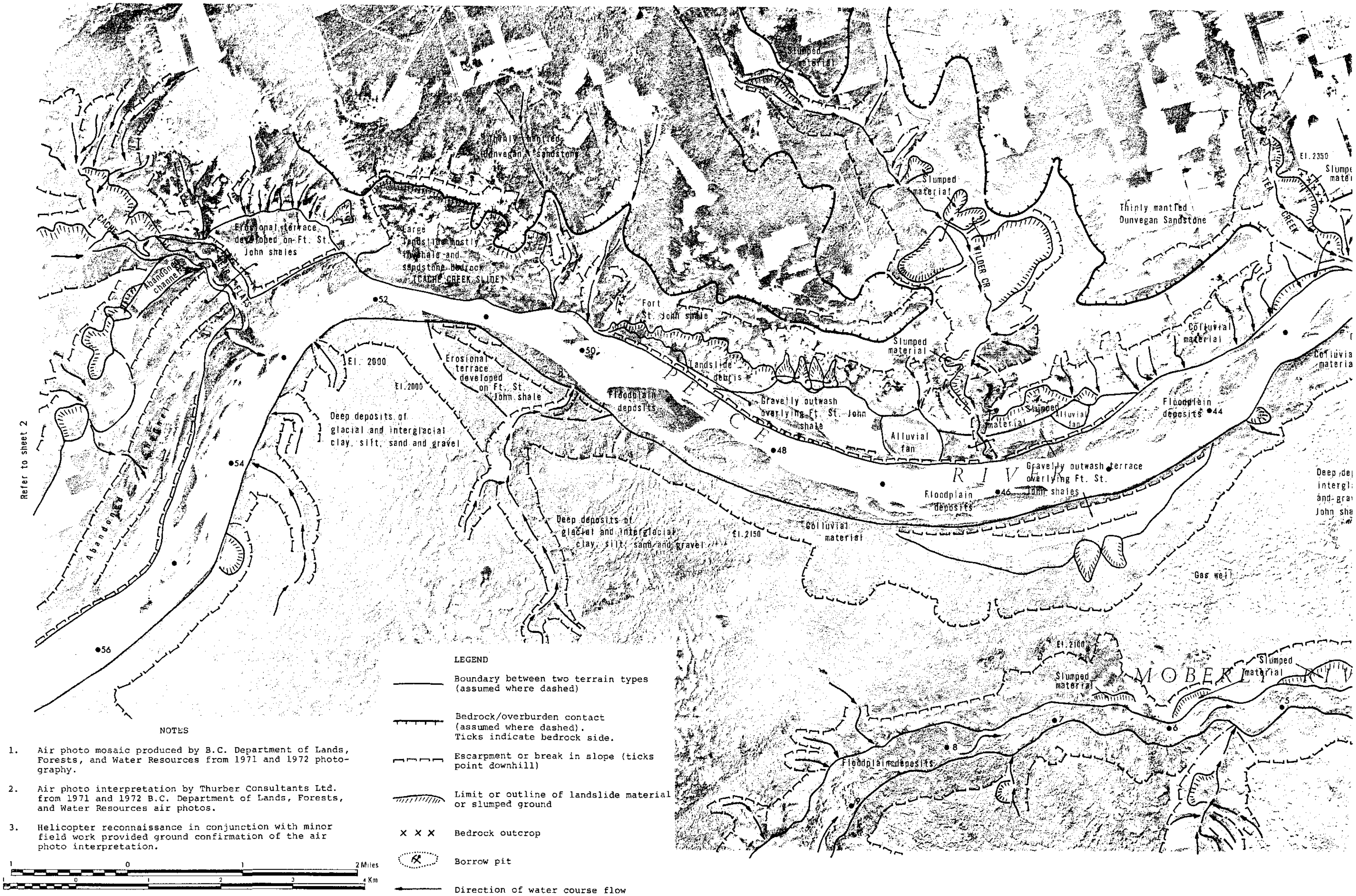
The base map has been duplicated from the 1:250,000 National Topographic Series.

Quaternary Stratigraphy and Geomorphology of the Fort St. John Area, by W.H. Mathews for Dept. of Mines and Petroleum Resources, Victoria, B.C. Nov. 1963. Some modifications by Thurber Consultants Ltd. from recent field work.



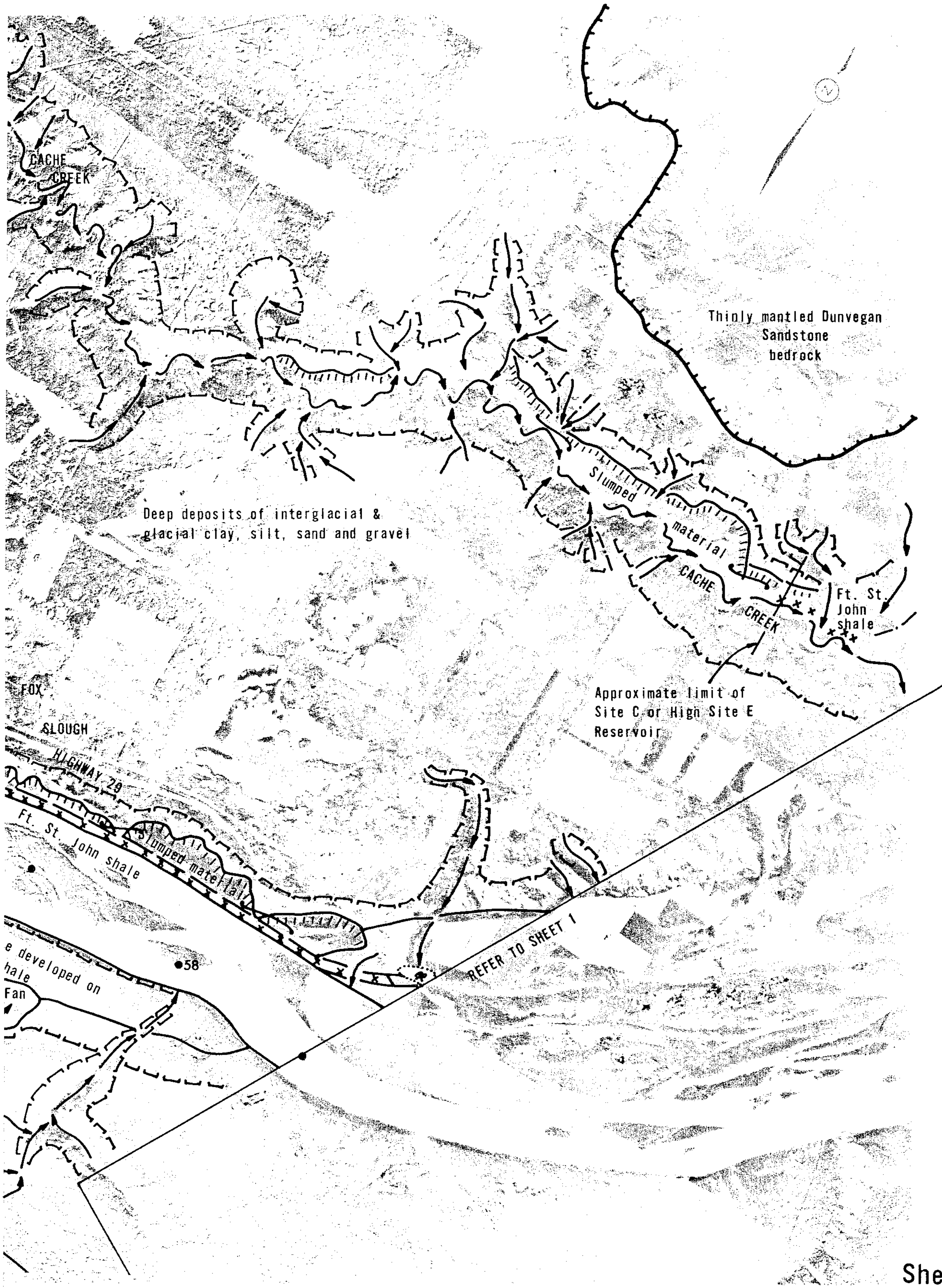
Sheet 1

	British Columbia Hydro and Power Authority	DRAWN DAM/DFV
		TRACED VEW
	TERRAIN ANALYSIS OF RESERVOIR AREA	DATE Nov. 1977
		APPROVED G.C.M.
	Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING No. 15-2-62-3



NOTES

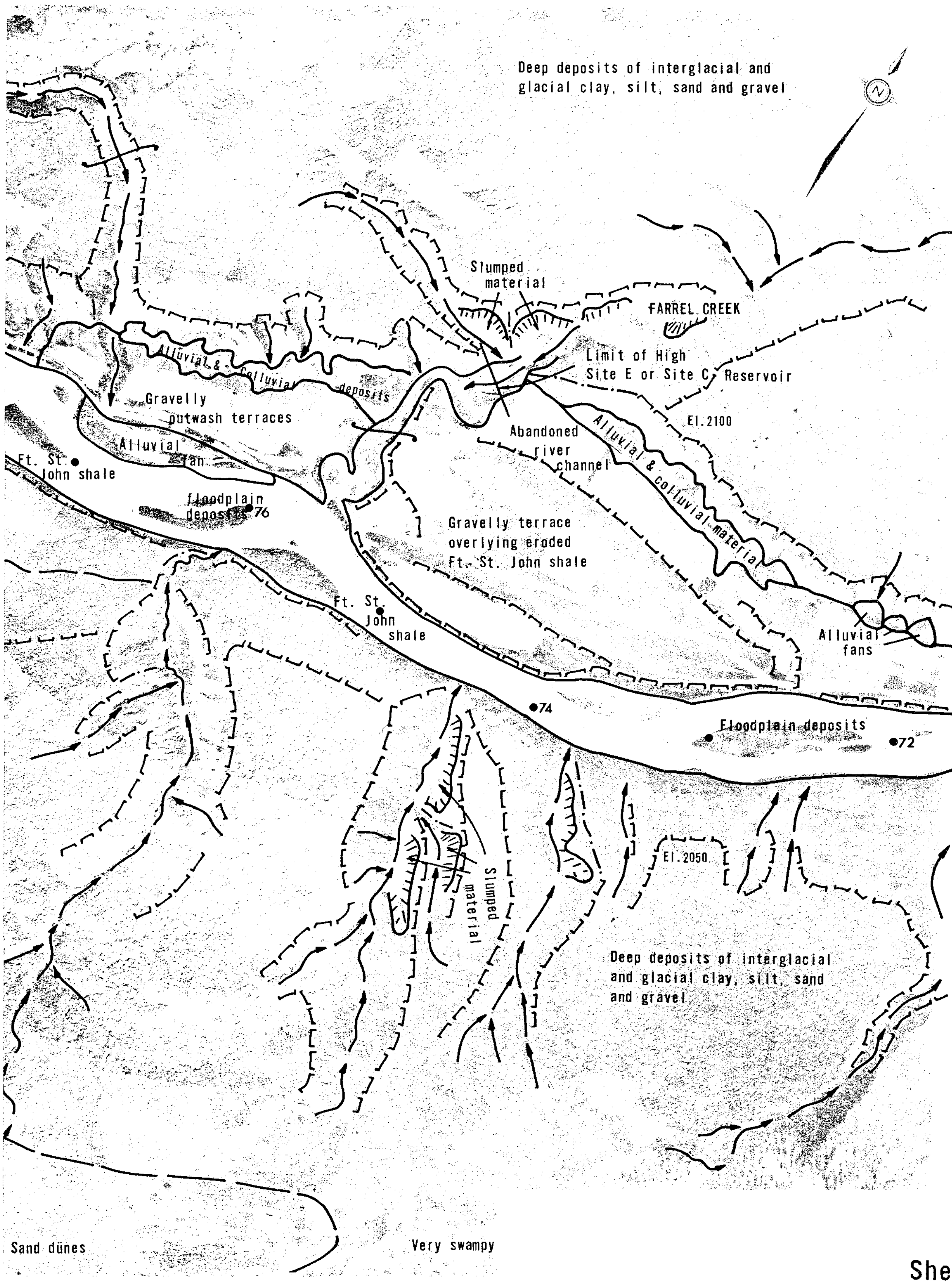
1. Air photo mosaic produced by B.C. Department of Lands, Forests, and Water Resources from 1971 and 1972 photography.
2. Air photo interpretation by Thurber Consultants Ltd. from 1971 and 1972 B.C. Department of Lands, Forests, and Water Resources air photos.
3. Helicopter reconnaissance in conjunction with minor field work provided ground confirmation of the air photo interpretation.



Sheet 2

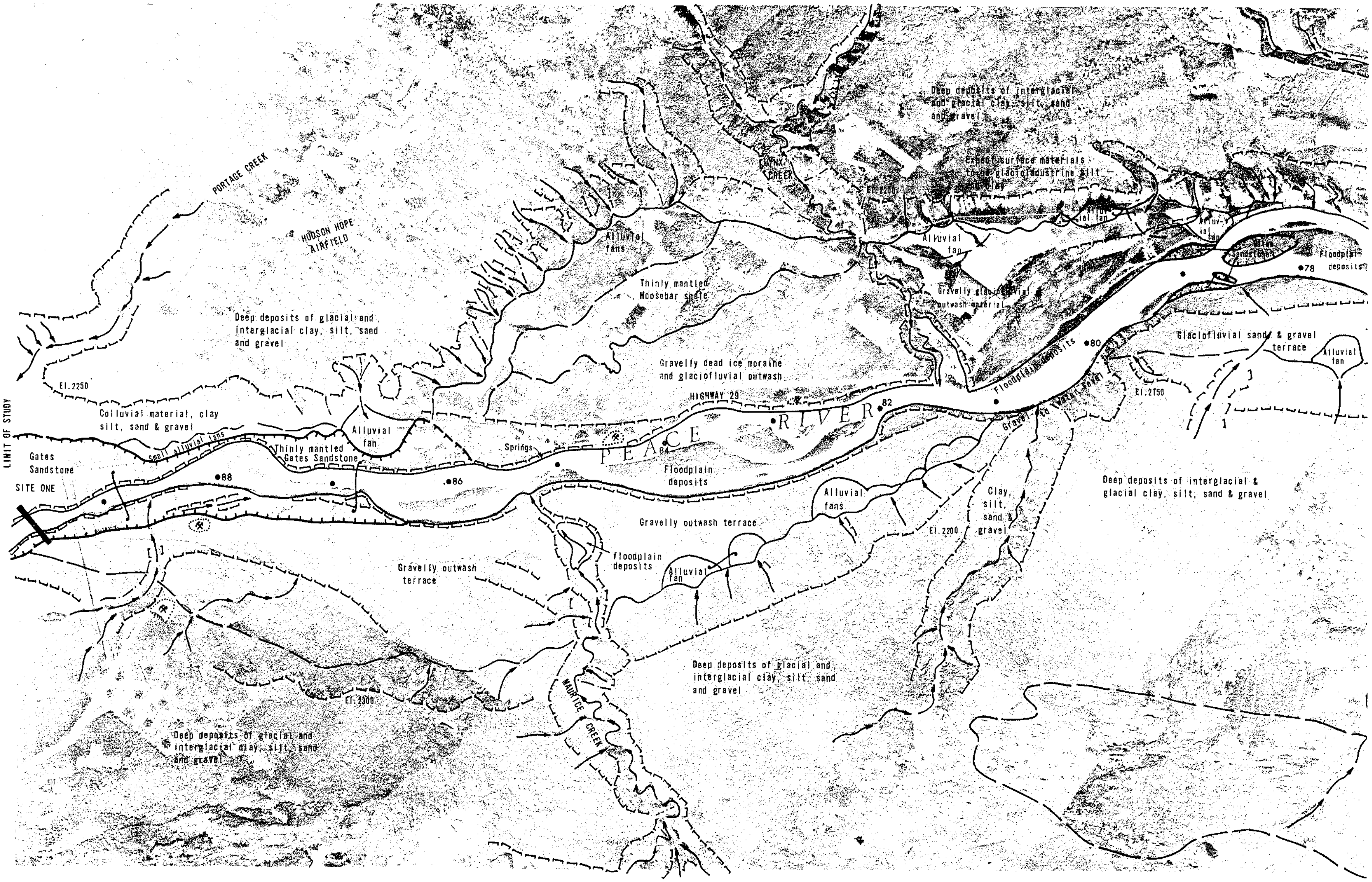
THURBER CONSULTANTS LTD., Geotechnical Engineers	British Columbia Hydro and Power Authority	DRAWN DAM/DFV
	TERRAIN ANALYSIS OF RESERVOIR AREA	TRACED VEW
		DATE Nov. 1977
		APPROVED G.C.M.
	Site C Reservoir Shoreline Assessment	SCALE Shown
		DRAWING No 15-2-62-3

REFER TO SHEET 3



Sheet 3

	British Columbia Hydro and Power Authority	DRAWN DAM/DFV
		TRACED VEW
	TERRAIN ANALYSIS OF RESERVOIR AREA	DATE Nov. 1977
		APPROVED G.C.M.
	Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING NO 15-2-62-3



LIMIT OF STUDY
SITE ONE
Gates Sandstone
PORTAGE CREEK
HUDSON HOPE AIRFIELD
Deep deposits of glacial and interglacial clay, silt, sand and gravel
El.2250
Colluvial material, clay silt, sand & gravel
small alluvial fans
Thinly mantled Gates Sandstone
Alluvial fan
●88
●86
Springs
Alluvial fans
Thinly mantled Moosebar shale
Gravelly dead ice moraine and glaciofluvial outwash
HIGHWAY 29
PEACE RIVER
84
●82
Floodplain deposits
Gravelly outwash terrace
floodplain deposits
Alluvial fan
Gravelly outwash terrace
El.2300
Deep deposits of glacial and interglacial clay, silt, sand and gravel
MAURICE CREEK
Deep deposits of glacial and interglacial clay, silt, sand and gravel
Alluvial fans
El.2200
Clay, silt, sand & gravel
LYNX CREEK
El.2200
Deep deposits of interglacial and glacial clay, silt, sand and gravel
Alluvial fan
Gravelly glaciofluvial outwash material
●80
Floodplain deposits
El.2150
Glaciofluvial sand & gravel terrace
Alluvial fan
Deep deposits of interglacial & glacial clay, silt, sand & gravel
●78
Floodplain deposits
Gates Sandstone

LEGEND

Escarpment

Contact between two terrain types; solid line indicates definite; broken line indicates approximate boundary.

Bedrock/overburden contact; ticks indicate bedrock side.

Slumped material

Currently active areas.

Topographic high or ridge.

Drainage course with direction of flow.

M 2 — Miles upstream on the Moberly R. from its confluence with the Peace R.

1 — Location of bedrock/overburden contact elevation measurement.

Field photo number and direction.

Cross-section location

Floodplain

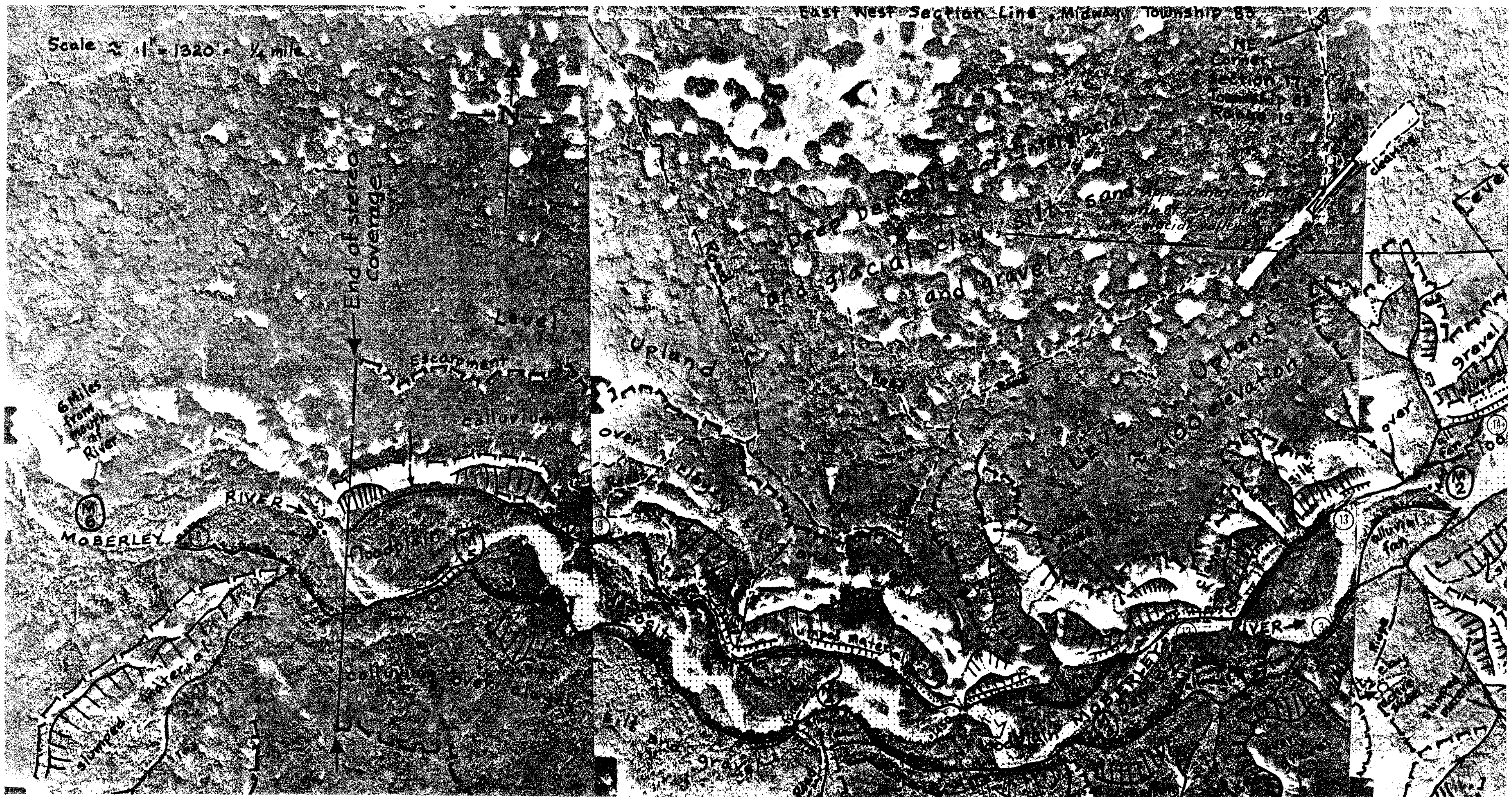
BEDROCK/OVERBURDEN CONTACT ELEVATIONS

Right Bank		Left Bank	
1	1560	9	1500
2	1540	10	1520
3	1500	11	1530
4	1540	12	1500
5	1550	13	1500
6	1570	14	1550
7	1535	15	1540
8	1580	16	1740
		17	1750

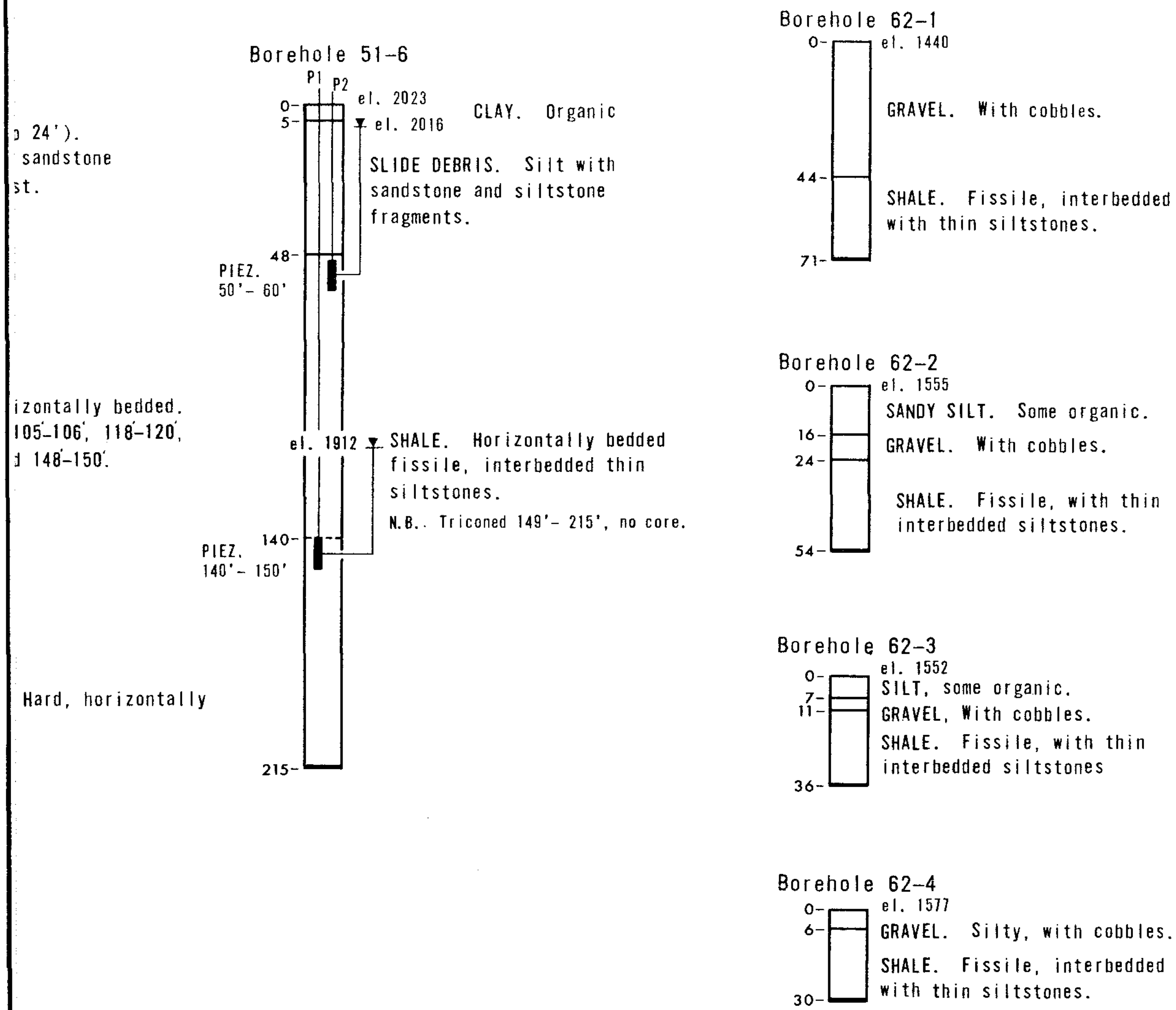
NOTES:

1. Mosaic produced from B.C. Lands, Forests and Water Resources Air Photos, BC 7278 230-234, 1'' to 1320'.
2. Contact elevations obtained from helicopter and ground measurements. Accurate to ± 10'.
3. Refer to Thurber dwgs. 15-2-37B-2, 3 & 4 for Cross-sections. (Not provided in this report).

British Columbia Hydro and Power Authority	DRAWN D.A.M./D.F.V.
DETAILED TERRAIN ANALYSIS FOR MOBERLY RIVER	TRACED V.E.W.
Site C Reservoir Shoreline Assessment	DATE Nov. 1977
	APPROVED
	SCALE 1''- 1320'
THURBER CONSULTANTS LTD., Geotechnical Engineers	DRAWING No 15-2-62-4



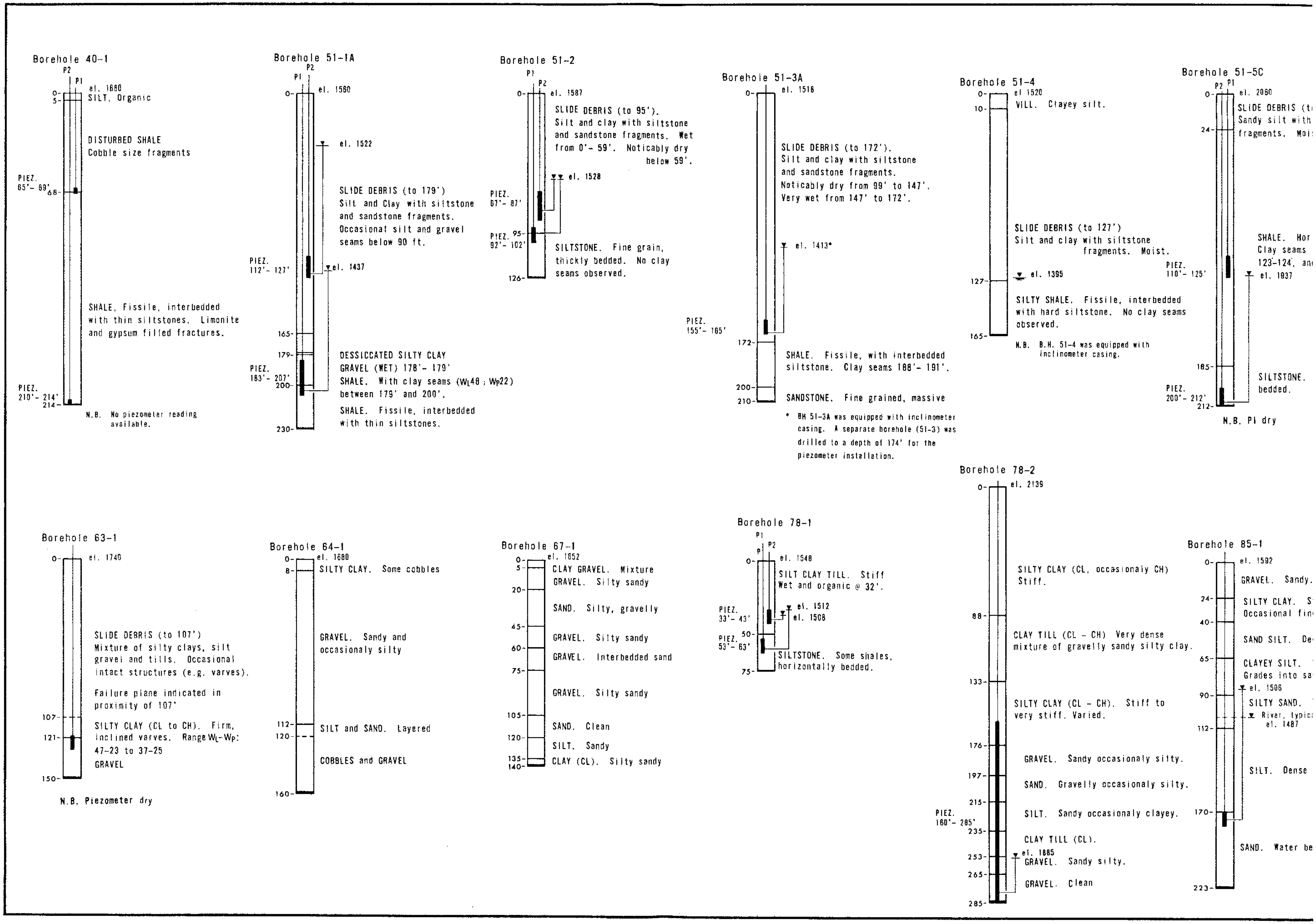
Scale ≈ 1" = 1320' = 1/4 mile
East West Section Line, Midway Township 83
NE Corner Section Township 83 Range 19
End of stereo coverage
Level
Escarpment
colluvium
Upland
Deep Deposits
and glacial clay
and gravel
Level
Upland
elevation
over
6 Miles from mouth of River
RIVER
MOBERLEY
floodplain
colluvium over clay
slumped material
silt and gravel
RIVER
alluvial fan
gravel
clearing



NOTE:

Piezometric readings shown on this drawing are those corresponding to the Nov. 28 - 30, 1977 readings as supplied by B.C. Hydro.

THURBER CONSULTANTS LTD., Geotechnical Engineers	British Columbia Hydro and Power Authority	DRAWN G.M.
	SUMMARY OF BOREHOLE DATA	TRACED D.B.
		DATE Nov. 1977
		APPROVED
	Site C Reservoir Shoreline Assessment	SCALE Shown
		DRAWING No 15-2-62-5


Borehole 40-1
P2
P1
el. 1680
SILT, Organic
DISTURBED SHALE
Cobble size fragments
PIEZ. 65'- 69'
SHALE, Fissile, interbedded with thin siltstones. Limonite and gypsum filled fractures.
PIEZ. 210'- 214'
N.B. No piezometer reading available.
Borehole 51-1A
el. 1560
el. 1522
SLIDE DEBRIS (to 179')
Silt and Clay with siltstone and sandstone fragments. Occasional silt and gravel seams below 90 ft.
PIEZ. 112'- 127'
el. 1437
DESSICCATED SILTY CLAY
GRAVEL (WET) 178'- 179'
SHALE. With clay seams (WL48 ; Wp22) between 179' and 200'.
SHALE. Fissile, interbedded with thin siltstones.
PIEZ. 183'- 207'
Borehole 51-2
el. 1587
SLIDE DEBRIS (to 95').
Silt and clay with siltstone and sandstone fragments. Wet from 0'- 59'. Noticably dry below 59'.
el. 1528
PIEZ. 67'- 87'
PIEZ. 92'- 102'
SILTSTONE. Fine grain, thickly bedded. No clay seams observed.
Borehole 51-3A
el. 1518
SLIDE DEBRIS (to 172').
Silt and clay with siltstone and sandstone fragments.
Noticably dry from 99' to 147'.
Very wet from 147' to 172'.
el. 1413*
PIEZ. 155'- 165'
SHALE. Fissile, with interbedded siltstone. Clay seams 188'- 191'.
SANDSTONE. Fine grained, massive
* BH 51-3A was equipped with inclinometer casing. A separate borehole (51-3) was drilled to a depth of 174' for the piezometer installation.
Borehole 51-4
el 1520
VILL. Clayey silt.
SLIDE DEBRIS (to 127')
Silt and clay with siltstone fragments. Moist.
el. 1395
SILTY SHALE. Fissile, interbedded with hard siltstone. No clay seams observed.
N.B. B.H. 51-4 was equipped with inclinometer casing.
Borehole 51-5C
el. 2060
SLIDE DEBRIS
Sandy silt with fragments. Moi
SHALE. Hor
Clay seams 123'-124', an
PIEZ. 110'- 125'
el. 1937
SILTSTONE. bedded.
PIEZ. 200'- 212'
N.B. P1 dry
Borehole 63-1
el. 1740
SLIDE DEBRIS (to 107')
Mixture of silty clays, silt gravel and tills. Occasional intact structures (e.g. varves).
Failure plane indicated in proximity of 107'
SILTY CLAY (CL to CH). Firm, inclined varves. Range WL-Wp: 47-23 to 37-25
GRAVEL
N.B. Piezometer dry
Borehole 64-1
el. 1680
SILTY CLAY. Some cobbles
GRAVEL. Sandy and occasionaly silty
SILT and SAND. Layered
COBBLES and GRAVEL
Borehole 67-1
el. 1652
CLAY GRAVEL. Mixture
GRAVEL. Silty sandy
SAND. Silty, gravelly
GRAVEL. Silty sandy
GRAVEL. Interbedded sand
GRAVEL. Silty sandy
SAND. Clean
SILT. Sandy
CLAY (CL). Silty sandy
Borehole 78-1
el. 1548
SILT CLAY TILL. Stiff
Wet and organic @ 32'.
PIEZ. 33'- 43'
el. 1512
el. 1508
PIEZ. 53'- 63'
SILTSTONE. Some shales, horizontally bedded.
Borehole 78-2
el. 2139
SILTY CLAY (CL, occasionaly CH) Stiff.
CLAY TILL (CL - CH) Very dense mixture of gravelly sandy silty clay.
SILTY CLAY (CL - CH). Stiff to very stiff. Varied.
GRAVEL. Sandy occasionaly silty.
SAND. Gravelly occasionaly silty.
SILT. Sandy occasionaly clayey.
PIEZ. 160'- 285'
CLAY TILL (CL).
el. 1885
GRAVEL. Sandy silty.
GRAVEL. Clean
Borehole 85-1
el. 1592
GRAVEL. Sandy.
SILTY CLAY.
Occasional fin
SAND SILT. De
CLAYEY SILT.
Grades into sa
el. 1506
SILTY SAND.
River, typic el. 1487
SILT. Dense
SAND. Water be

NOTES

1 The bedrock contact profiles shown on this drawing have been prepared from

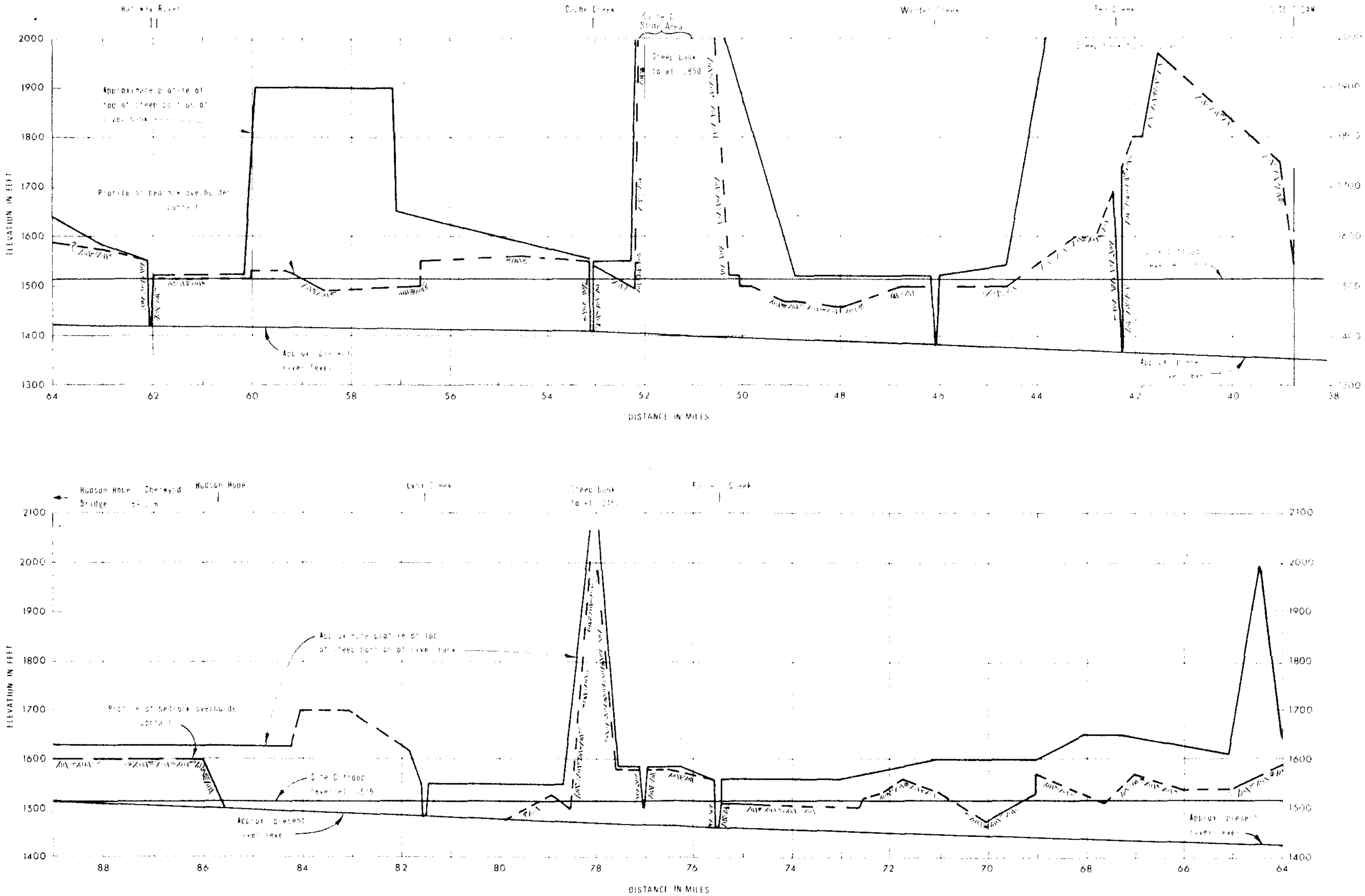
a) A total of 40 ground measured geological sections and 116 helicopter measured geological sections taken around the proposed reservoir's shoreline (with emphasis upon the high bank areas).

b) A helicopter inspection of the entire shoreline, including the preparation of a complete photographic record of the shoreline conditions. This record consisted of 339 35 mm. overlapping color photos taken at low angle

2 The topographic details shown on this drawing are approximate only. They have been taken from the 1 50 000 National Topographic Map Series."

3 The information presented on this drawing is intended only for the purpose of the report of which it is a part. It may be inadequate for other uses.

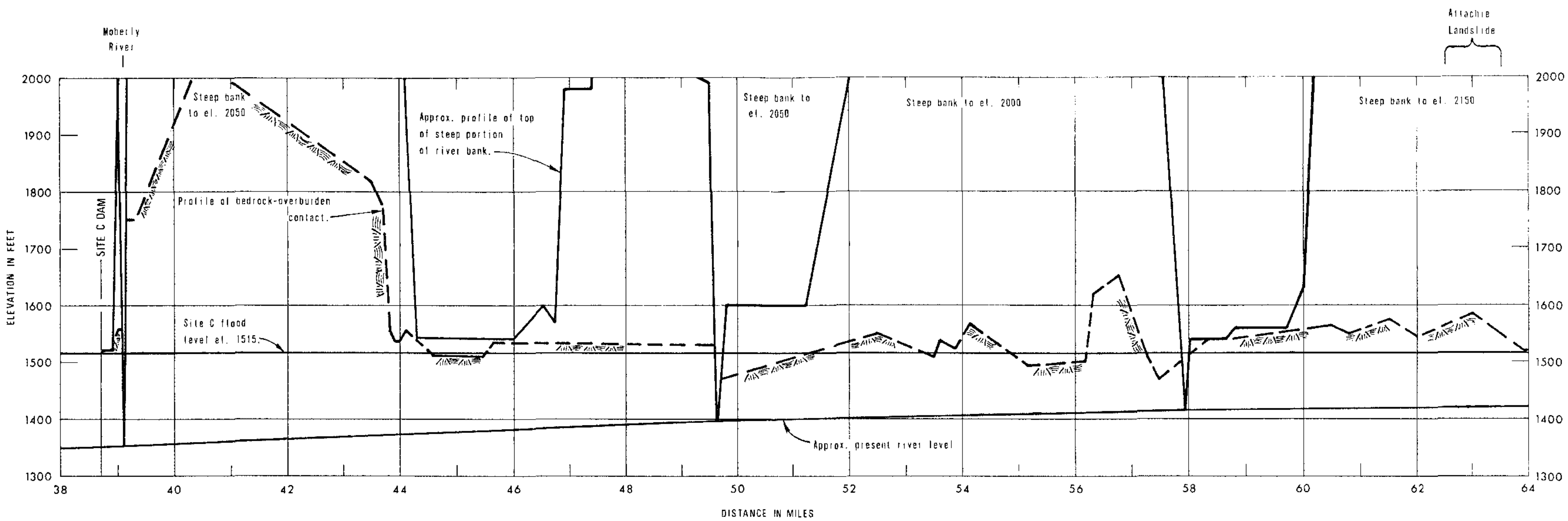
British Columbia Hydro and Power Authority	R.G.
	D.B.
NORTH BANK BEDROCK PROFILE	March 1978
	R.C.M.
Site C Reservoir Shoreline Assessment	Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers	15-2-62-6


Halfway River
Cache Creek
Wilder Creek
Approximate profile of top of steep portion of river bank
Profile of bedrock overburden contact
Steep bank to el. 1950
Appr. present river level
ELEVATION IN FEET
DISTANCE IN MILES
Hudson Hope
Hudson Hope - Chetwynd Bridge
Lynx Creek
Steep bank
Profile of bedrock overburden contact
Site C flood level el. 1515
Approx. present river level
Approximate profile of top of steep portion of river bank

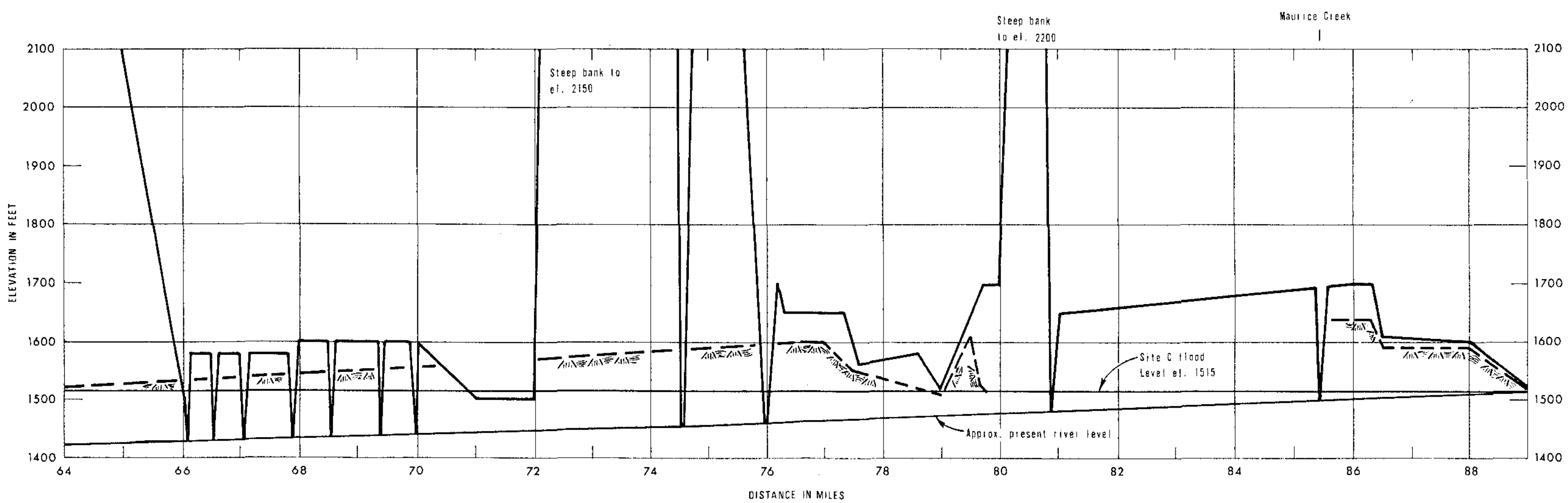
NOTES:

1. The bedrock contact profiles shown on this drawing have been prepared from:
   a) A total of 40 ground measured geological sections and 116 helicopter measured geological sections taken around the proposed reservoir's shoreline (with emphasis upon the high bank areas).
   b) A helicopter inspection of the entire shoreline, including the preparation of a complete photographic record of the shoreline conditions. This record consisted of 339, 35 mm. overlapping color photos taken at low angle.
2. The topographic details shown on this drawing are approximate only. They have been taken from the 1:50,000 National Topographic Map Series.
3. The information presented on this drawing is intended only for the purpose of the report of which it is a part. It may be inadequate for other uses.

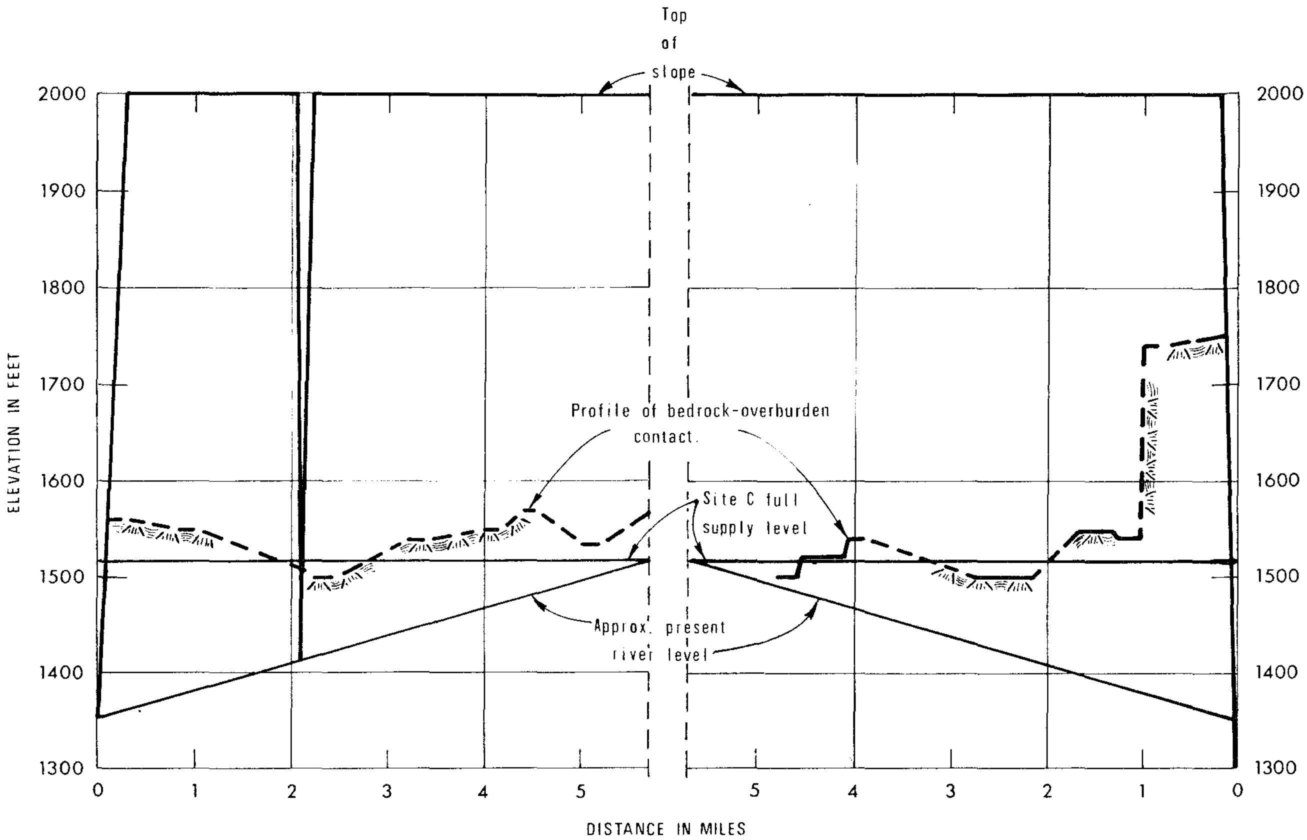
	British Columbia Hydro and Power Authority	DRAWN R. G.
		TRACED D. B.
	SOUTH BANK BEDROCK PROFILE	DATE March 1978
		APPROVED G. L.
	Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING No. 15-2-62-7



Moberly River
Attachie Landslide
SITE C DAM
Steep bank to el. 2050
Profile of bedrock-overburden contact.
Approx. profile of top of steep portion of river bank.
Steep bank to el. 2050
Steep bank to el. 2000
Steep bank to el. 2150
Site C flood level el. 1515.
Approx. present river level
ELEVATION IN FEET
DISTANCE IN MILES



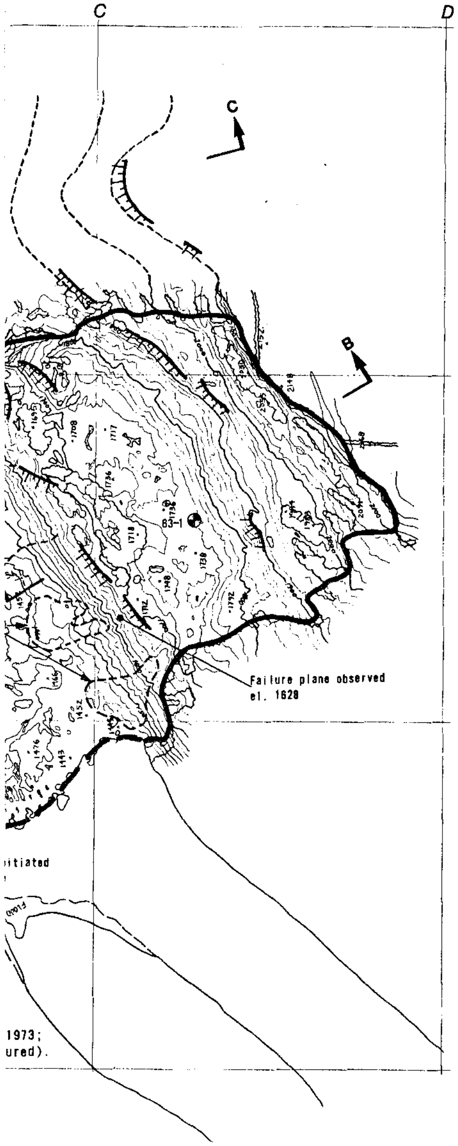
Steep bank to el. 2150
Steep bank to el. 2200
Maurice Creek
Site C flood Level el. 1515
Approx. present river level
ELEVATION IN FEET
DISTANCE IN MILES

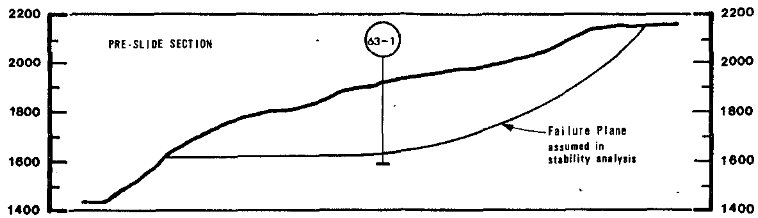


NOTES:

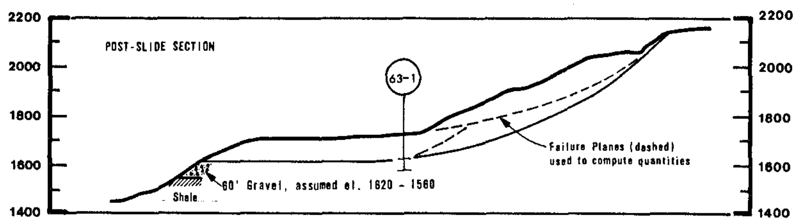
1. The bedrock contact profiles shown on this drawing have been prepared from:
   a) A total of 40 ground measured geological sections and 116 helicopter measured geological sections taken around the proposed reservoir's shoreline (with emphasis upon the high bank areas).
   b) A helicopter inspection of the entire shoreline, including the preparation of a complete photographic record of the shoreline conditions. This record consisted of 339, 35 mm. overlapping color photos taken at low angle.
2. The topographic details shown on this drawing are approximate only. They have been taken from the 1:50,000 National Topographic Map Series.
3. The information presented on this drawing is intended only for the purpose of the report of which it is a part. It may be inadequate for other uses.

British Columbia Hydro and Power Authority	DRAWN R.G.
	TRACED D.B.
MOBERLY RIVER BEDROCK PROFILE	DATE March 1978
	APPROVED ecm
Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers	DRAWING Nº 15-2-62-8

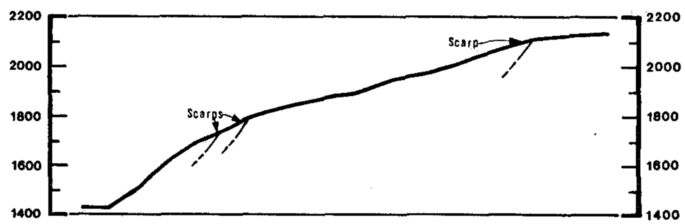




Section A-A



Section B-B



Section C-C

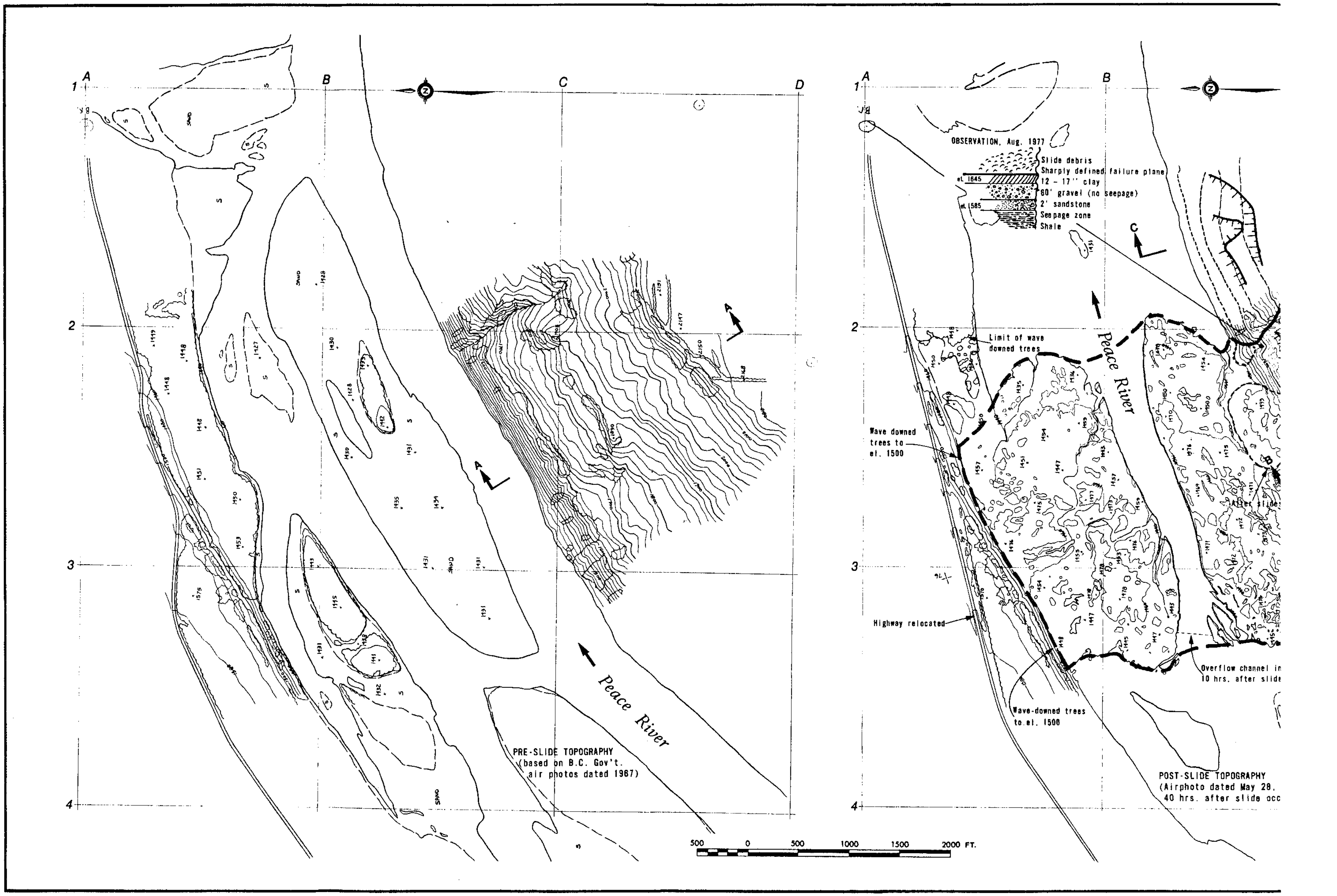
LEGEND:

- Limit of surface movement (observed, estimated)
- Scarp
- Boundary

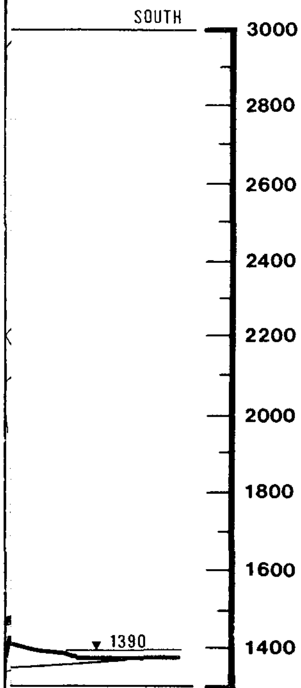
NOTES:

1. Geological and other surface data plotted by compass and pacing. At those locations where numerous closely spaced scarps or gullies occured, a representative number were plotted.
2. Elevation of DH 63-1 (ground) and field observations were established by helicopter; and are approximate only.
3. Refer to dwg. 15-2-62-5 for drill hole and groundwater data.

British Columbia Hydro and Power Authority	DRAWN	G.M.
	TRACED	K.R.C.-D.
ATTACHIE SLIDE AREA	DATE	Nov. 1977
	APPROVED	G.C.M.
Site C Reservoir Shoreline Assessment	SCALE	Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers	DRAWING No	15-2-62-9



Peace River
PRE-SLIDE TOPOGRAPHY
(based on B.C. Gov't.
air photos dated 1967)
OBSERVATION, Aug. 1977
Slide debris
Sharply defined failure plane
el. 1645
12 - 17'' clay
60' gravel (no seepage)
el. 1585
2' sandstone
Seepage zone
Shale
Limit of wave
downed trees
Peace River
Wave downed
trees to
el. 1500
Highway relocated
Wave-downed trees
to el. 1500
Overflow channel in
10 hrs. after slide
POST-SLIDE TOPOGRAPHY
(Airphoto dated May 28,
40 hrs. after slide occ
500 0 500 1000 1500 2000 FT.



LEGEND:

Limit of current or recent surface movement (observed, estimated).

High scarp (F_f indicates freshly exposed face).

Narrow gully believed to have resulted from down-faulting i.e. a graben. (F_f indicates freshly exposed faces. Depth of gully is indicated in feet).

Flowslide

Field observation location.

Drill hole

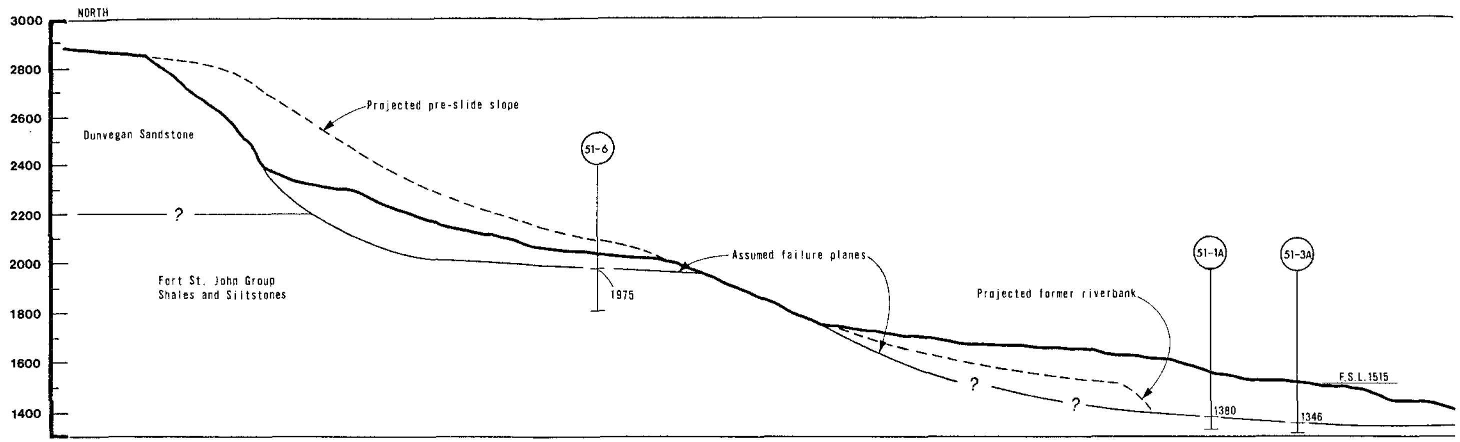
NOTES:

1. Geological and other surface data plotted by compass and pacing. At those locations where numerous closely spaced scarps or gullies occured, a representative number were plotted.
2. Refer to dwg. 15-2-62-5 for drill hole and groundwater data.
3. Borehole (ground) elevations are approximately.

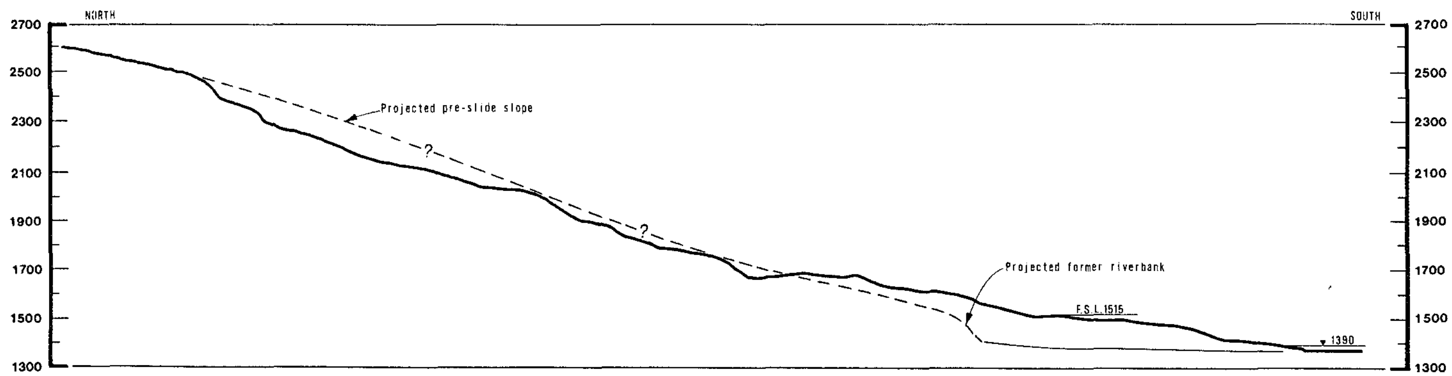
REFERENCE:

1. Topography based on B.C. Gov't. 1967 photography.

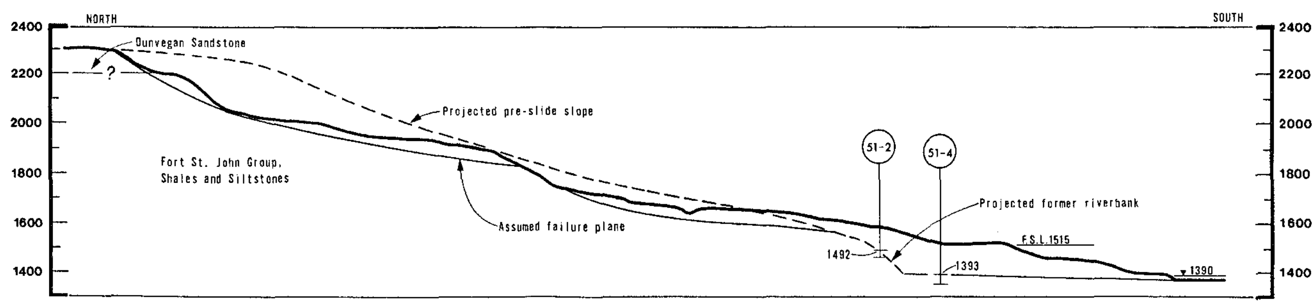
	British Columbia Hydro and Power Authority	DRAWN G.M.
		TRACED K.R.C.-D.
	CACHE CREEK SLIDE AREA	DATE Nov. 1977
		APPROVED G.C.M.
	Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING No 15-2-62-10



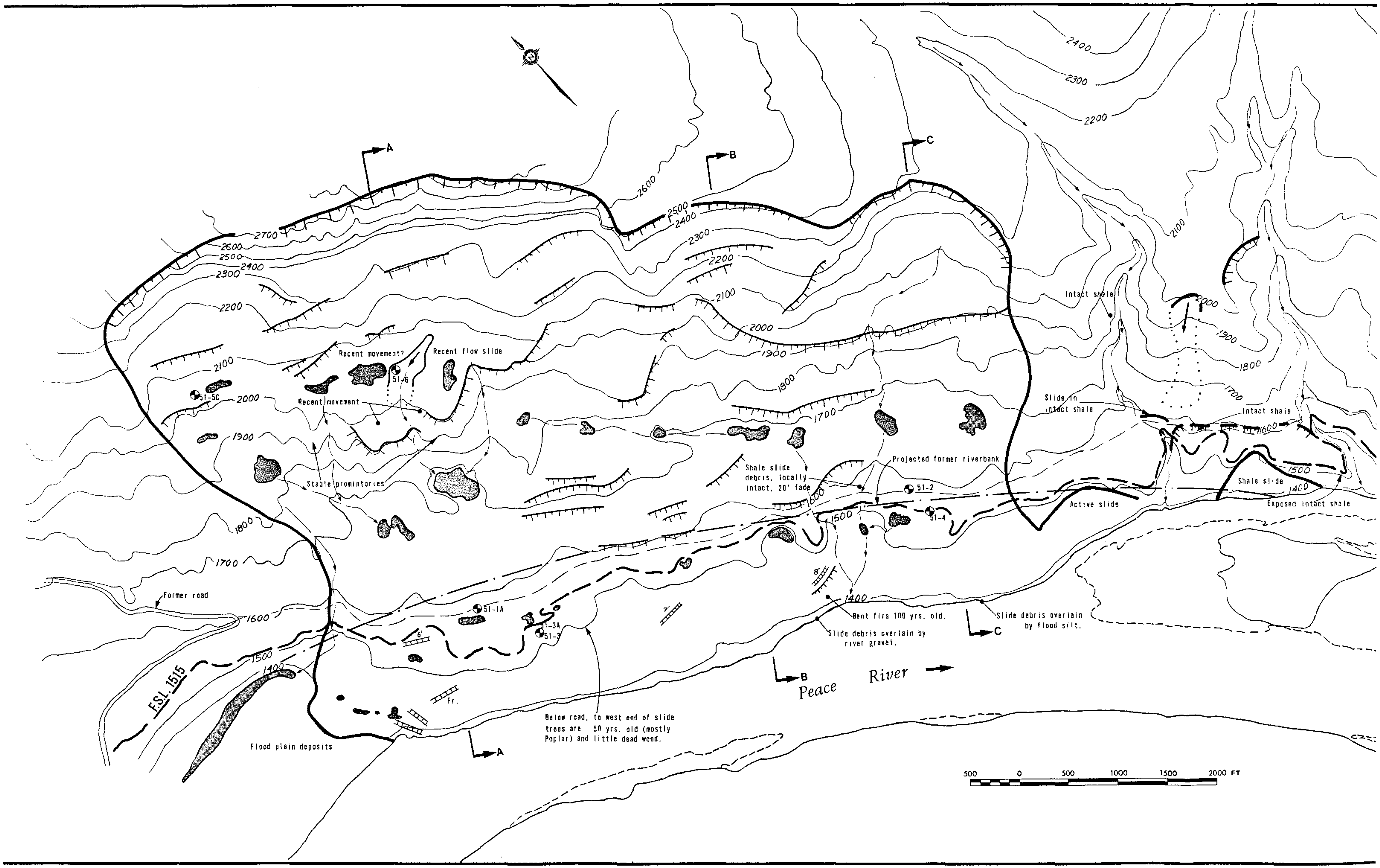
Section A-A

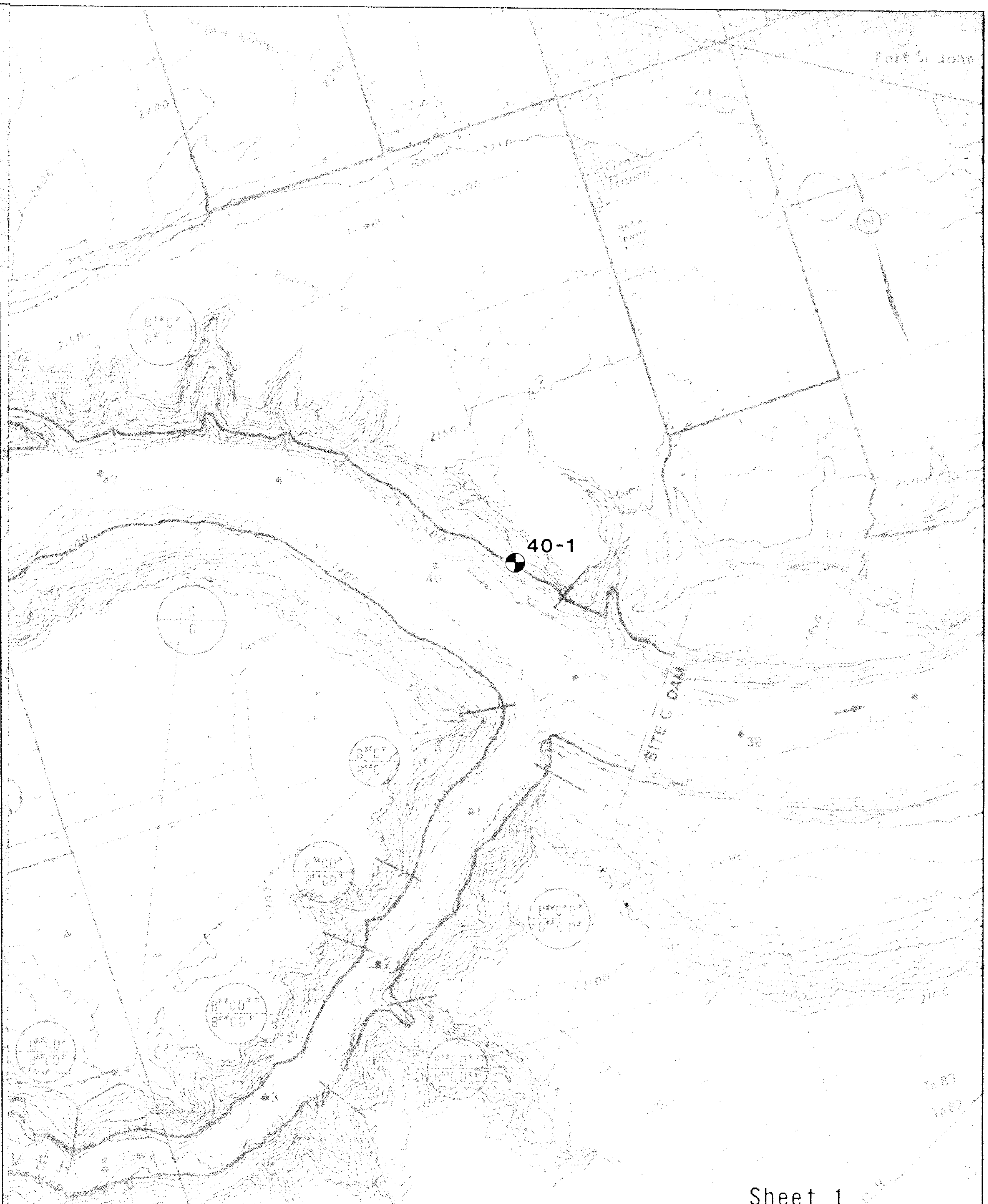


Section B-B



Section C-C


Recent movement?
Recent flow slide
Recent movement
Stable promintories
Shale slide debris, locally intact, 20' face
Projected former riverbank
Intact shale
Slide in intact shale
Intact shale
Shale slide
Active slide
Exposed intact shale
Bent firs 100 yrs. old.
Slide debris overlain by river gravel.
Slide debris overlain by flood silt.
Below road, to west end of slide trees are 50 yrs. old (mostly Poplar) and little dead wood.
Flood plain deposits
Former road
F.S.L. 1515
Peace River
Fr.
51-5C
51-6
51-1A
51-3A
51-3
51-2
51-4
A
B
C
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
1700
1600
1500
1400
500 0 500 1000 1500 2000 FT.



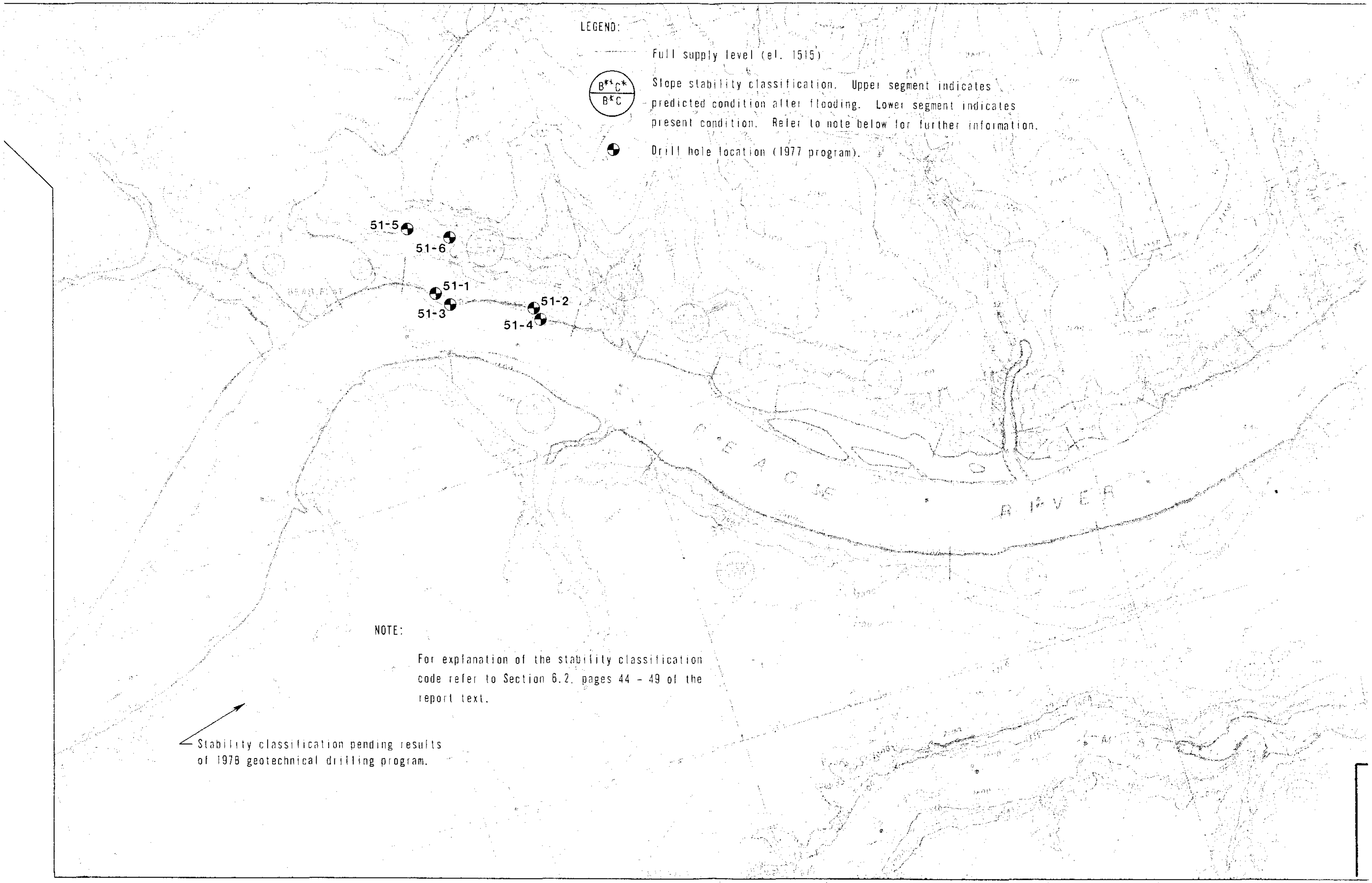
British Columbia Hydro and Power Authority

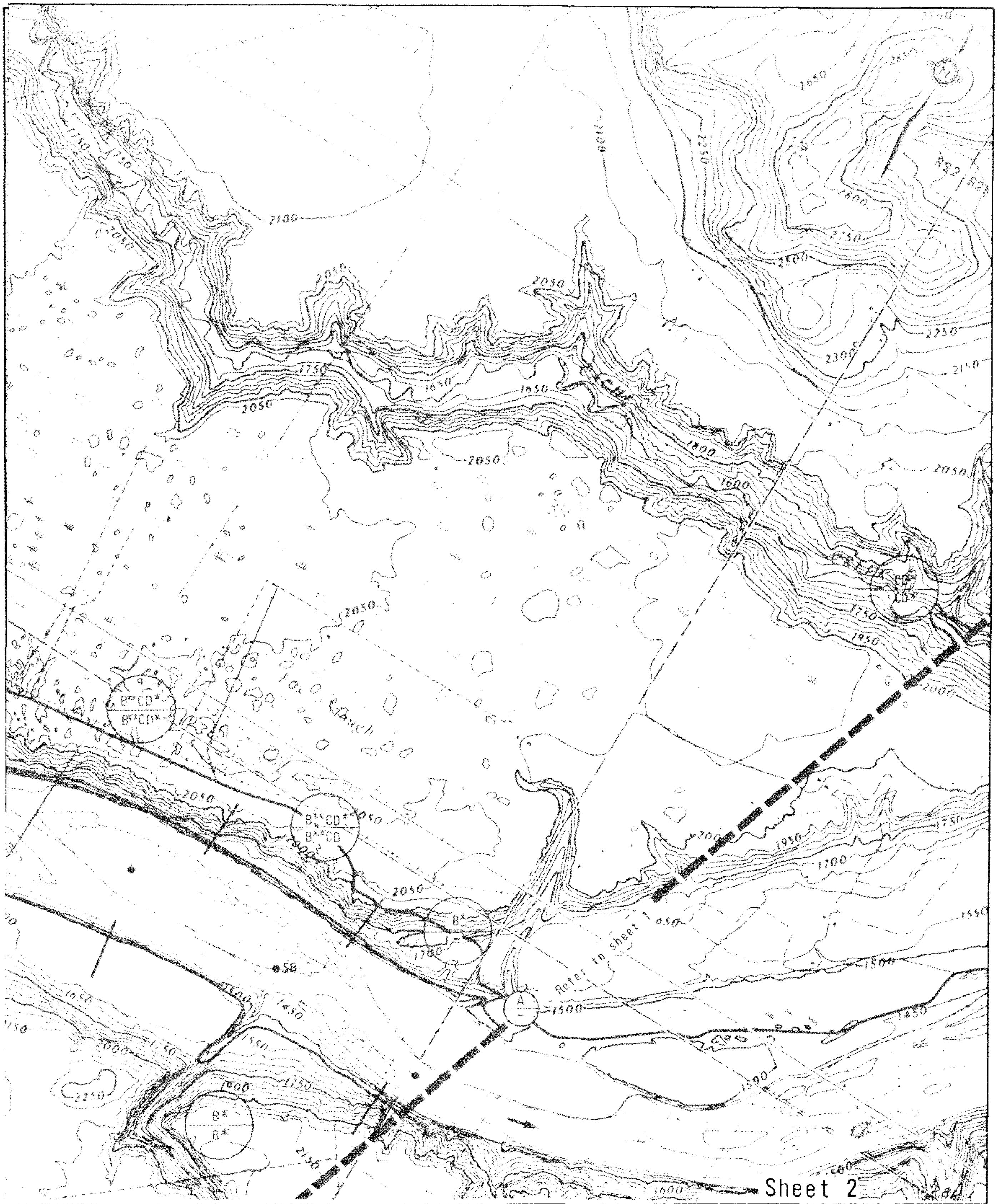
SHORELINE STABILITY CLASSIFICATION

Site C Reservoir Shoreline Assessment

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	G.M.
TRACED	D.B.
DATE	Nov. 1977
APPROVED	
SCALE	Shown
DRAWING Nº	15-2-62-11


LEGEND:
Full supply level (el. 1515)
B*C*
B*C
Slope stability classification. Upper segment indicates predicted condition after flooding. Lower segment indicates present condition. Refer to note below for further information.
Drill hole location (1977 program).
51-5
51-6
51-1
51-3
51-2
51-4
PEACE RIVER
NOTE:
For explanation of the stability classification code refer to Section 6.2, pages 44 - 49 of the report text.
Stability classification pending results of 1978 geotechnical drilling program.



British Columbia Hydro and Power Authority

SHORELINE STABILITY CLASSIFICATION

Site C Reservoir Shoreline Assessment

DRAWN	G.M.
TRACED	D.B.
DATE	Nov. 1977
APPROVED	
SCALE	Shown
DRAWING No	15-2-62-11

THURBER CONSULTANTS LTD., Geotechnical Engineers

West of Sixth Meridian

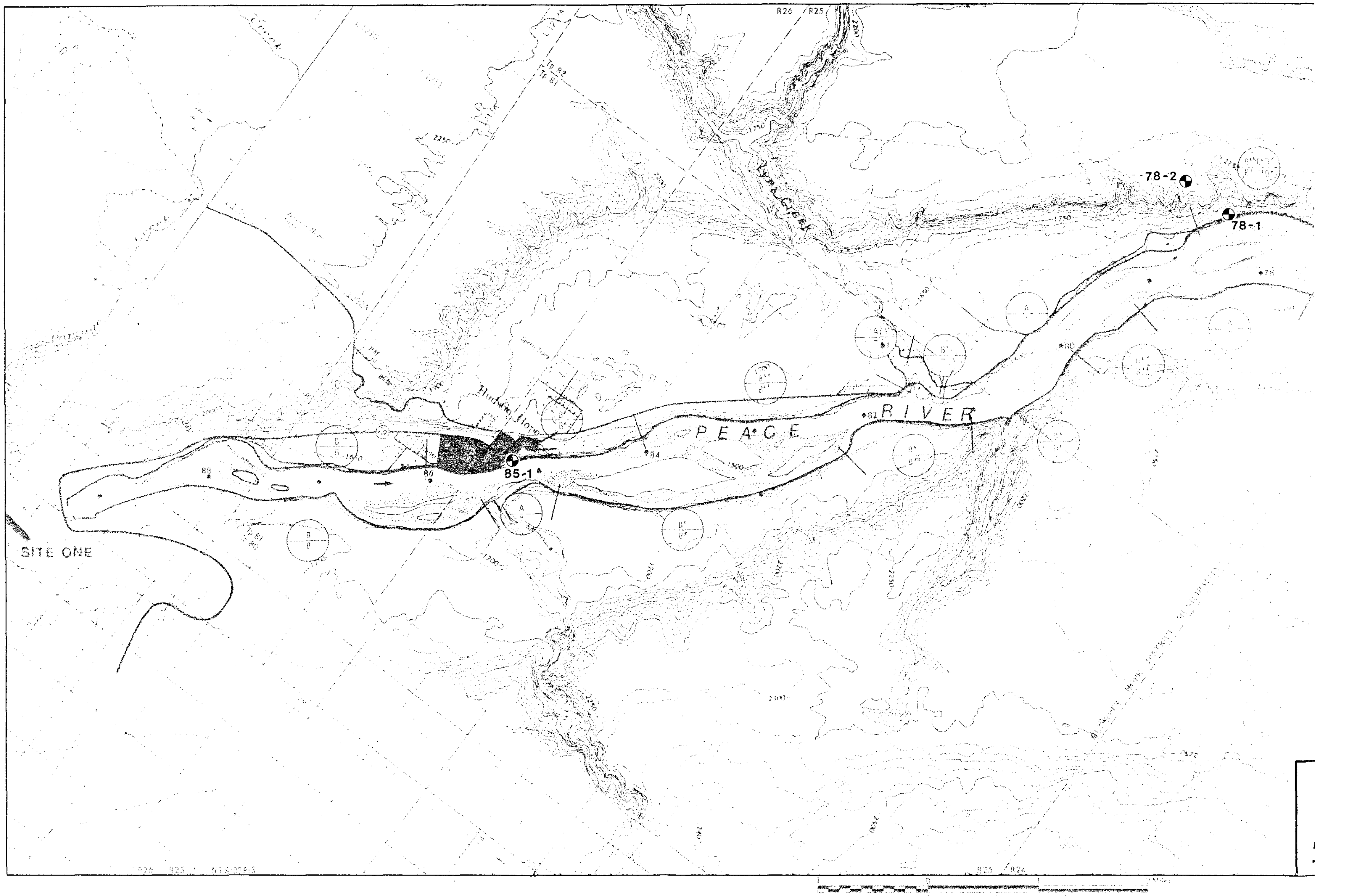

PEACE RIVER
HALFWAY RIVER
ATTACHIE
62-1
62-2
62-3
62-4
63-1
64-1
67-1
NTS 94A4
NTS 94A3
Refer to legend and note on Sheet 1.

Refer to legend and note on Sheet 1.

Sheet 3

	British Columbia Hydro and Power Authority	DRAWN G.M.
		TRACED D.B.
	SHORELINE STABILITY CLASSIFICATION	DATE Nov. 1977
		APPROVED
	Site C Reservoir Shoreline Assessment	SCALE Shown
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING № 15-2-62-11

West of Sixth Meridian



PEACE RIVER
Lynx Creek
Hudson Hope
SITE ONE
78-2
78-1
85-1

SUMMARY OF STABILITY ANALYSIS
FOR ATTACHIE CREEK (Section A-A)

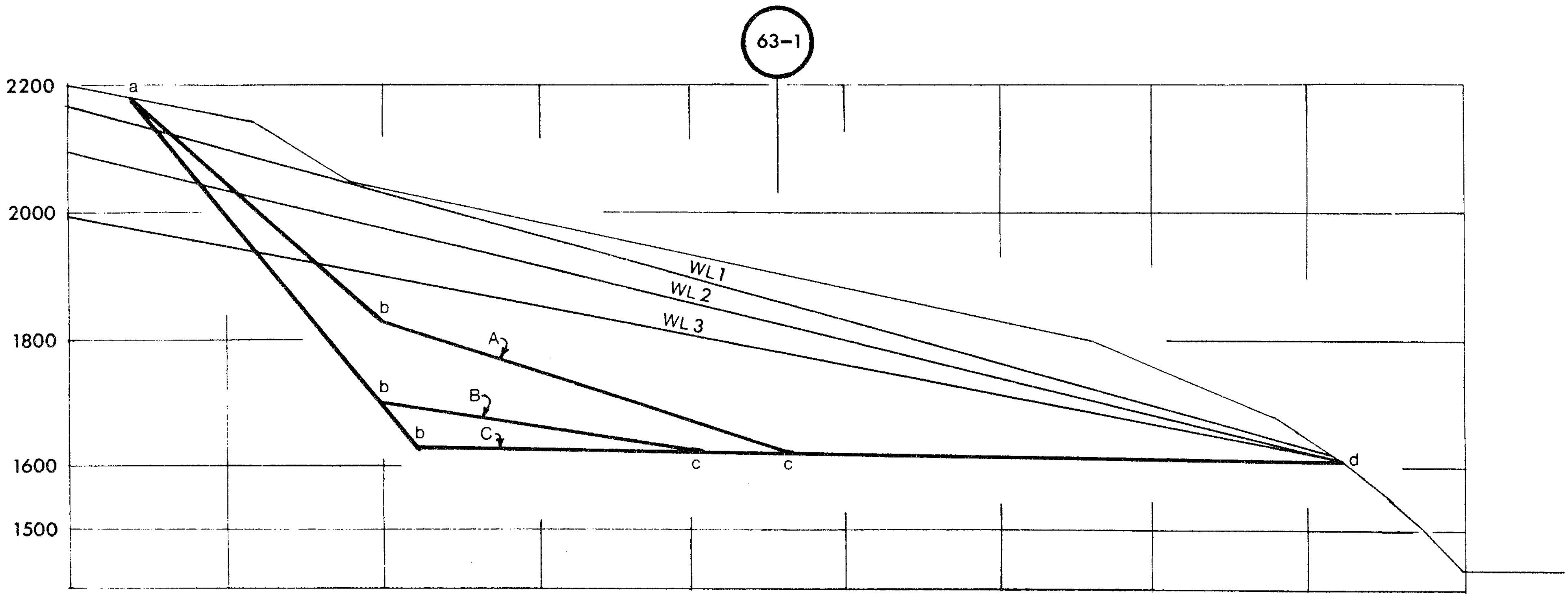
Refer Fig. B1

Notes:

- Analysis was carried out on the pre-slide Topography
- Water levels W.L.1,2 and 3 are assumed.
- A simple groundwater pattern has been assumed.
- Failure plane at el. 1633 satisfies findings of DH 63-1.
- The Morgernstern - Price method of analysis was used.

Strength Parameters(Ø)* (c=0 throughout)			Water Level	Factor of Safety		
				Plane A	Plane B	Plane C
$Ø_{a-b}$	$Ø_{b-c}$	$Ø_{c-d}$				
28	28	28	W.L.1	1.36	1.42	1.62
28	25	20	"	1.06	1.11	-
28	25	13	"	0.86	0.92	-
28	20	13	"	0.77	0.82	-
28	13	13	"	-	-	0.78
13	25	23	"	1.07	1.12	-
13	28	20	"	1.05	1.09	-
28	28	28	W.L.2	1.56	1.64	-
28	25	20	"	1.23	1.30	-
28	25	13	"	1.03	1.09	-
28	20	13	"	0.92	0.97	-
28	13	13	"	-	-	0.93
13	25	23		1.22	1.27	-
13	28	20		1.20	1.25	-
28	28	28	W.L.3	1.82	1.92	-
28	25	20	"	1.45	1.54	-
28	25	13	"	1.24	1.31	-
28	20	13	"	1.1	1.17	-
28	13	13	"	-	-	1.13
13	25	23	"	1.42	1.47	-
13	28	20	"	1.41	1.45	-

* Assumed Saturated Unit Weight = 120 PCF throughout.



STABILITY ANALYSIS OF ATTACHIE SLIDE (SECTION A-A)

SUMMARY OF STABILITY ANALYSIS FOR CACHE CREEK LOWER SLIDE (Section A-A)

Refer Fig. B2

Notes on Water Levels:

- W.L.4 satisfies observed piezometric readings immediately below the slide debris in D.H.51-1. W.L.1. is compatible with observed piezometer readings within the slide debris at a higher level (refer Dwg. 5).
- W.L.2 is an assumed water level higher than W.L.1.and 4.
- W.L.3 corresponds to F.S.L. 1515.
- A simple groundwater pattern has been assumed (i.e. no perched water tables).
- Present river level assumed at el. 1390.

Morgenstern Price Method

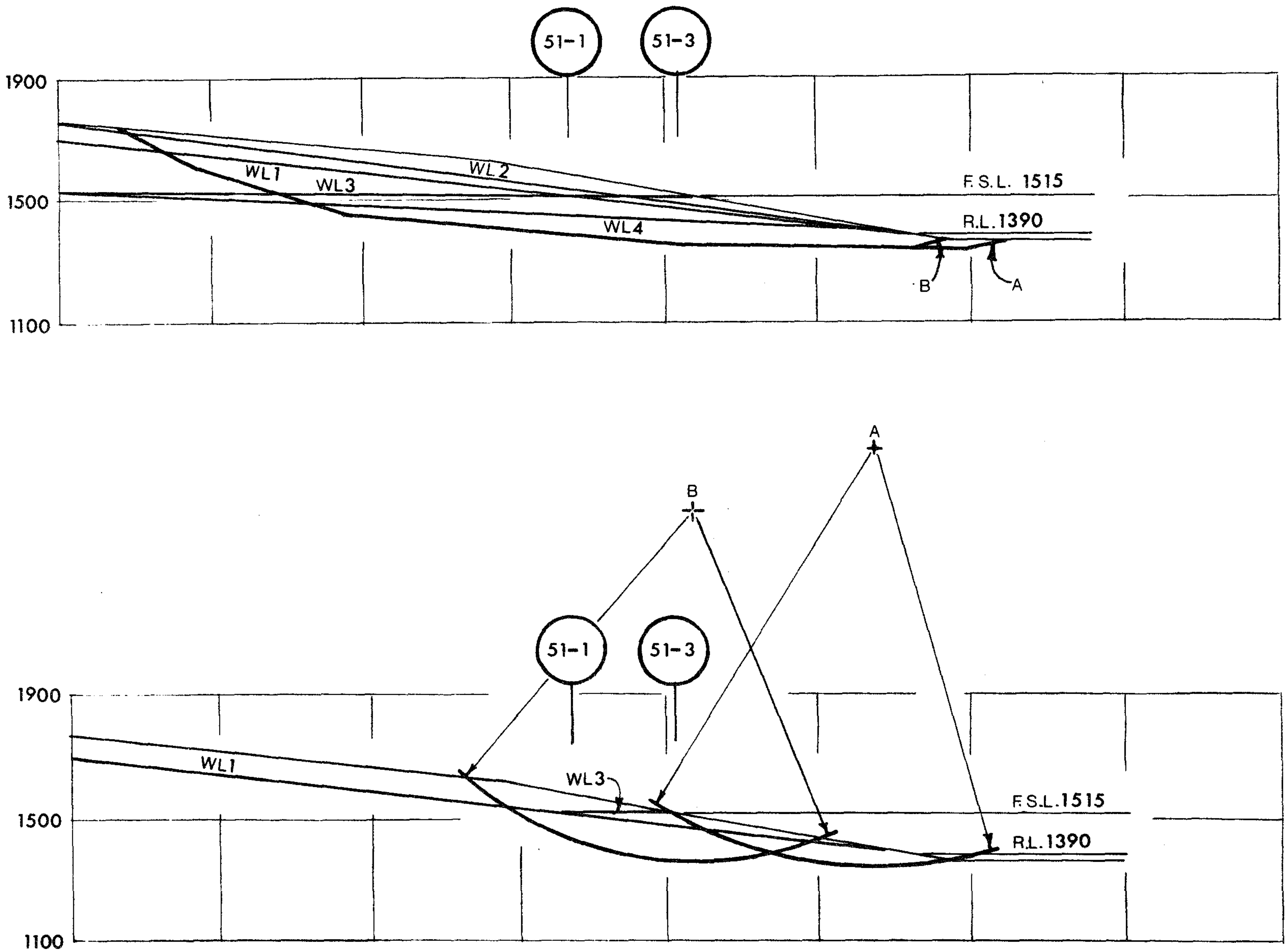
Strength Parameters(Ø)* (c=0 throughout)	Water Level	Factor of Safety Plane A	Factor of Safety Plane B
13°	W.L.1	1.21	1.20
22°	W.L.1	2.12	2.10
13°	W.L.2.	-	1.06
22°	W.L.2.	-	1.86
13°	W.L.4	-	1.56
22°	W.L.4	-	2.73
13°	W.L.1 and 3	-	1.32
20°	W.L.1 and 3	-	2.31
13°	W.L.4 and 3	-	1.50
22°	W.L.4 and 3	-	2.63

* Assumed Saturated Unit Weight = 120 PCF throughout

Circular Arc - Bishop Method

Strength Parameters(Ø)* (c=0 throughout)	Water Levels	Factor of Safety and Critical Circle
Ø = 13°	W.L.1	1.17 (Circle A)
Ø = 28°	W.L.1	2.29 (Circle A)
Ø = 13°	W.L.1 and 3	1.39 (Circle B)
Ø = 28°	W.L.1 and 3	2.76 (Circle B)

*Assumed Saturated Unit Weight = 120 PCF throughout.



STABILITY ANALYSIS OF CACHE CREEK LOWER SLIDE (SECTION A-A)

FIGURE B2

SUMMARY OF STABILITY ANALYSIS FOR CACHE CREEK UPPER SLIDE (Section A-A)

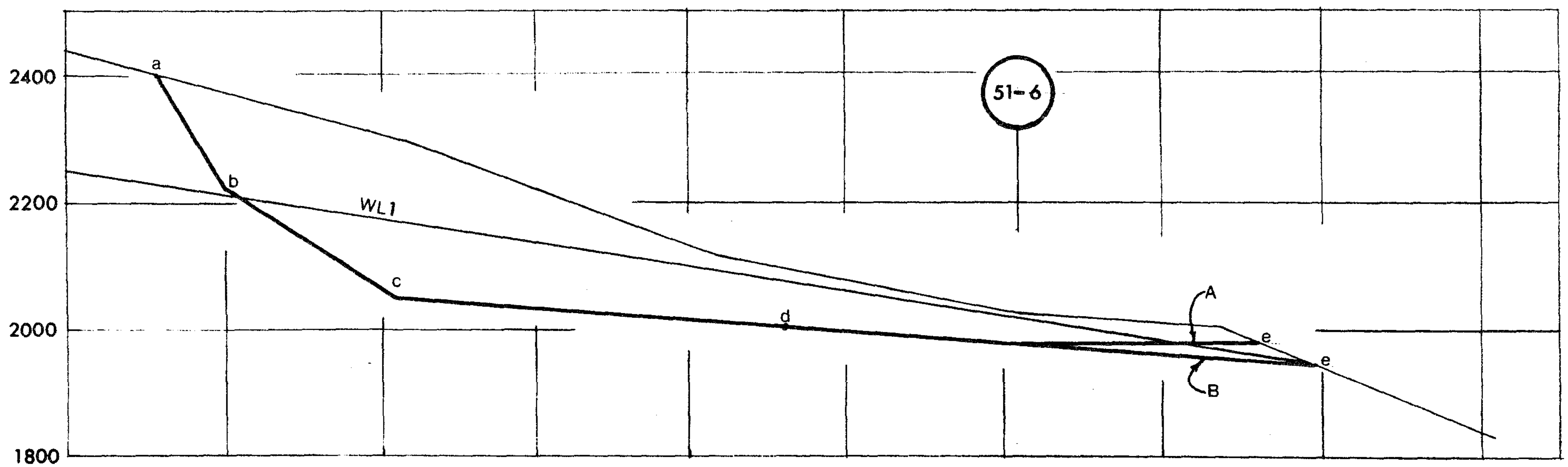
Refer Fig. B3

Notes:

- A simple groundwater pattern has been assumed (i.e. no perched water tables).
- The assumed water table WLI satisfies current observations in D.H. 51-6.
- The Morgenstern - Price method of analysis was used.

Strength Parameters(Ø)* (c=0 throughout)				Factor of Safety	
$Ø_{a-b}$	$Ø_{b-c}$	$Ø_{c-d}$	$Ø_{d-e}$	Plane A	Plane B
40	13	13	13	0.83	0.82
40	22	13	13	-	0.96
40	22	22	13	1.29	1.27
40	22	22	22	1.39	1.37
40	40	13	13	1.32	1.31
40	40	22	22	1.64	1.62
40	28	22	13	1.40	1.38

*Assumed Saturated Unit Weight = 130 PCF



STABILITY ANALYSIS OF CACHE CREEK UPPER SLIDE (SECTION A-A)

FIGURE B3

THURBER

SITE C RESERVOIR SHORELINE STABILITY ASSESSMENT
PIEZOMETRIC LEVELS* (1977)

Piezometer	Surface	Jul.22	Jul.26	Jul.29/30	Aug.8	Aug.13	Aug.26	Aug.30	Nov.30
DH85-1	El.1592	84.0'	86.3'	82.7'	92.4'	84.3'	85.3'	85.7'	84.0'
DH78-1(P-1)	El.1548	35.8'	35.4'	35.8'	35.6'	35.7'	37.7'	36.6'	39.9'
DH78-1(P-2)	El.1548	30.2'	30.6'	32.1'	38.7'	32.0'	33.5'	32.7'	36.4'
DH78-2	El.2139	-	-	-	-	-	174.0'	-	253.9
DH51-1A(P-1)	El.1560	-	116.2'	115.4'	-	-	111.5'	112.2'	122.7'
DH51-1A(P-2)	El.1560	-	27.7'	39.0'	-	-	34.6'	35.7'	37.1'
DH51-2(P-1)	El.1587	-	54.1'	53.5'	55.1'	-	56.1'	55.8'	59.1'
DH51-2(P-2)	El.1587	-	53.7'	54.4'	54.9'	-	55.9'	54.3'	59.0'
DH51-3A (Inclin)	El.1518	-	108.6'	109.8'	147.9'	-	-	-	144.4'
DH51-3	El.1518	-	-	-	-	-	103.7'	105.2'	104.5'
DH51-4 (Inclin)	El.1520	-	112.8'	112.5'	112.2'	-	79.2'	109.1'	124.1'
DH51-5(P-1)	El.2060	-	-	-	-	-		-	Dry
DH51-5(P-2)	El.2060	-	-	-	-	-		-	122.7'
DH51-6(P-1)	El.2023	-	-	-	-	-	-	82.9'	111.0'
DH51.6(P-2)	El.2023	-	-	-	-	-	-	7.4'	-**
DH63-1	El.1740	-	-	-	-	Dry		-	Dry
DH40-1(P-1)	El.1580(?)	-	-	-	-	-	48.8'	-	Dry
DH40-1(P-2)	El.1580(?)	-	-	-	-	-	Dry	-	Dry

* Data provided by B.C. Hydro

** Blocked at 2.5 from depth

Note: The above readings were taken by B.C. Hydro. The Hydroelectric Design Division has advised that some of the readings are in doubt and will be carrying out a thorough check of all piezometers early in 1978.

1. McElhanney Surveying and Engineering Ltd. were commissioned to:

   a) extend the topographic mapping of the existing 1"=1000' pondage sheets to cover the valley slopes above el. 1600 in those areas previously designated C and D stability potential*. This involved 33 miles of shoreline and an area of 10,600 acres.

   b) prepare topographic map of the Cache Creek slide area and the Attachie slide area (before and after the slide). These were done at a scale of 1"=500' and a 20' contour interval.

2. With the above additional topography covered under item 1(a), one complete set of pasted up plans was prepared using all available topography in the reservoir area (scale 1"-1,000'). These were used for study purposes and as a base for the safeline drawings downstream of Cache Creek.

3. A complete photographic record of the reservoir shoreline from Site C to Hudson Hope was prepared. This record consisted of 339, 35 mm. overlapping colour photos taken at a low angle from a helicopter. The photographs were assembled in book form and referenced with river mileage such that any portion of the shoreline may readily be inspected.

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* Refer: Thurber Consultants' report dated January 1976 entitled, "Sites C and E, Lower Peace River and Tributaries, Shoreline Assessment".

4. During the field program, additional geological and topographical sections were prepared and topographical sections were prepared (with particular emphasis on the location of the bedrock contact). The following table summarizes the status of this work.

	1975	Spring 1976 (Moberly)	Fall 1976
Topographic sections (with stratigraphy)	--	3	16
Helicopter measured) geological sections)	) 20	17	79
Ground measured) Geological sections)		4	17

The locations for measurement were selected from:

- areas previously classified C and D.

- sections of exposed strata where relatively accurate measurements could be made.

The field program also included a surface study of the Cache Creek and Attachie slide areas. An attempt was made to analyze the history and mode of past movements of the Cache Creek slide, and to check for any evidence of recent movement. Plans showing the geology of both slide areas were prepared from the observations made during the field work.

5. A set of pondage maps was prepared recording all observation and photo locations, geological sections etc. for both 1975 and 1976 field seasons.

6. A complete profile along the reservoir shoreline was compiled (scale 1"=1,000' horizontal; 1"=100' vertical) showing all available geological and pertinent topographical information. Particular emphasis was placed on plotting the bedrock contact.

7. A detailed geological/geotechnical inspection of all privately owned properties was carried out in 1977.

8. Table 1-1 on the following page summarizes the scope of the 1977 shoreline drilling program.

9. A laboratory test program was carried out on selected undisturbed overburden specimens from the Attachie Slide area.

10. Stability analyses of the Cache Creek Slide area and the Attachie Slide areas were completed.

11. The shoreline stability classification was revised and updated using the results of the foregoing work.

12. The safeline location was plotted for all privately owned properties using

plans to a scale of 1"=400' and where these were not available, 1"=1000'.

13. A comprehensive report was prepared which drew on the findings of all previous studies in addition to the present study.

Table D-1, SITE C SHORELINE DRILLING PROGRAM (1977)

AREA	DRILL HOLES	DEPTH	INSTRUMENTATION	PURPOSE
Upstream of damsite, north bank	40-1		Piezometers	Establish typical groundwater pattern and geological conditions in area of high bedrock slopes.
Cache Creek Slide	51-1**		Piezometers	Establish failure plane, groundwater conditions, and analysis parameters.
	51-2		Piezometers	
	51-3		Slope Indicator	
	51-4		Slope Indicator	
	51-5		Piezometers	
	51-6		Piezometers	
Attachie Slide	63-1		Piezometer	As for Cache Creek Slide.
Halfway River	62-1*		-	Determine foundation conditions for approaches.
	62-2*		-	
	62-3*		-	
	62-4*		-	
	64-1*		-	
Mile 67, northbank	67-1		-	Establish typical soil conditions for low overburden covered terrace.
Mile 78, northbank	78-1*		Piezometer	Establish stratigraphy and groundwater conditions in undisturbed high overburden slope.
	78-2*		Piezometer	
Hudson Hope	85-1		Piezometer	Establish underlying soil conditions. Also, groundwater conditions in overburden.

* These drill holes were requested by the Department of Highways.

** For various reasons, more than one hole may have been drilled at each drill hole location on the Cache Creek Slide; these were labelled a, b, c... etc.