PEACE RIVER DEVELOPMENT
SITE C PROJECT

REVIEW OF UPSTREAM AXES

Prepared by

Klohn Crippen Berger Ltd. and SNC-Lavalin Inc.

For

B.C. Hydro

Report No. 5032A01 02
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EXECUTIVE SUMMARY

Planning studies undertaken by BC Hydro evaluated several locations for the proposed Site C Project on the Peace River near the City of Fort St. John, British Columbia. These studies culminated in the selection of a dam axis just downstream of the confluence of the Moberly River and the Peace River.

As part of contingency planning by BC Hydro, Preparatory Engineering Studies for the Site C Project were undertaken in 1989 by a team of engineers from Klohn Crippen Consultants Ltd (now Klohn Crippen Berger Ltd), Shawinigan Integ (which was subsequently taken over by SNC-Lavalin Inc) and BC Hydro. Engineering activities on Site C continued in 1990 as part of the Shelf Ready Plan which was based on securing an earliest in service date of 1998. In early March 1990 the contingency plan was amended and engineering work on the Site C project was terminated by the end of March 1991.

This report has been prepared by senior staff of Klohn Crippen Berger Ltd and SNC Lavalin Inc, who are the designers of record for the Site C Project, in response to a request from BC Hydro to undertake at an overview level an order of magnitude assessment of the changes in cost and schedule, both positive and negative, if the dam axis for the Site C project was relocated upstream of the Moberly River to one of the previously considered axes known as C-1 and C2. In this report the current dam axis downstream of the Moberly River is referred to as Site C and the upstream axes are referred to as Axis C-1 and Axis C-2.

For this evaluation, it was assumed that the project layout would be based on that developed for Site C and that the overall dimensions of the structures would be the same.

Section 1 of this report briefly summarizes the history of the selection of the Site C axis, the stability of the valley slopes, the topography and the reconnaissance that was performed as part of this assessment.

Section 2 of this report describes the simplified geological sections at Axis C-1 and Axis C-2 that were developed from available information. These sections were required so that the order of magnitude of changes to the major quantities resulting from moving from Site C to one of the upstream axes could be assessed.

Section 3 of this report summarizes the changes to the energy that would be generated from the project if it was moved upstream. The head and flow at the upstream axes would be less than at Site C, which would reduce the average annual generation by 372 GW.h at Axis C-1 and 230 GW.h at Axis C-2. The present values of the reductions in average annual generation are in the order of $192.5 million at Axis C-1 and $119 million at Axis C-2.

Section 4 of this report summarizes the order of magnitude changes to the major quantities that would result from moving the dam to one of the upstream axes. It is estimated that due to the topography and geology at the upstream axes, the increase in excavation would be in the order of 100 million m^3 and the increase in earthfill for the dam would be in the order of 10 million m^3.

Section 5 of this report summarizes the order of magnitude of changes to the cost and schedule resulting from moving the dam to one of the upstream axes. Section 5.3, which gives the cost and schedule impacts of moving to one of the upstream axes was prepared by BC Hydro staff. It is estimated that the direct cost of the Project would nearly double from $3 billion at Site C to the order of $5.6 billion at one of the upstream axes and that the schedule would be increased by about 5 years. The total Project Capital Cost would increase from $4.2 billion at Site C for a March 2017 in-service date to the order of $9.7 billion at one of the upstream axes.

1. INTRODUCTION

1.1 History of the Selection of the Site C Axis

1.1.1 Selection of a Site in the Vicinity of Fort St John

In 1958, B.C. Engineering Ltd identified four potential dam sites named A, B, C and D in the reach of the Peace River between what is now Peace Canyon Dam and Taylor, B.C.

A surface geological reconnaissance in 1967 identified that Site C in the vicinity of Fort St. John was preferable to Sites A, B and D due to geological conditions and other considerations.

Subsequently, the following three axes for a dam in the vicinity of Fort St. John were investigated and studied:

- Axis C-1 located just downstream of Tea Creek;
- Axis C-2 located approximately 2 km upstream of the Moberly River; and
- Axis C-3 located approximately 1 km downstream of the Moberly River.

As described below, Axis C-3 was ultimately selected for the dam due to superior geological and topographical conditions. All of the work on the project since 1976 has been based on this axis; therefore Axis C-3 is hereinafter referred to as Site C. The locations of the three axes that have been investigated are shown on Figure 1.

1.1.2 1972 Feasibility Study

The results of an exploration program undertaken at Axis C-1 in 1971 are given in the BC Hydro 1972 feasibility study report[(1)]. The purpose of the exploration program was to determine: the depth and nature of the overburden; the type and characteristics of bedrock; and the sources of construction materials. Seven holes totalling 220 m were drilled.

The investigations determined that moderate depths of overburden and slide materials cover bedrock on the left slope and under the river, and deep overburden covers bedrock on the right bank. One drill hole on the right bank indicated a depth of nearly 30 m of overburden consisting of glacial till. The till is reported to extend approximately 400 m upstream and downstream of the drill hole.

Bedrock at Axis C-1 is reported to be shale of the Shaftesbury Formation with thin interbeds of silty and sandy shales. These interbeds were considered to be insignificant because they are relatively thin, lack continuity and are found only well below the anticipated foundation levels. A number of thin clay seams in the bedrock were also found.

The preliminary dam design consisted of concrete gravity structures for the powerhouse and spillway approximately 78 m high founded on bedrock in the river channel, with earthfill dams on each side. It was acknowledged that further exploration would be required to determine the extent of the clay seams and the feasibility of the selected layout.

1.1.3 1976 Feasibility Study

The BC Hydro 1976 feasibility study report(2) states that geological reconnaissance of Axis C-1 performed after the 1971 investigations indicated that the left slope was potentially unstable and that this unstable area extended high above the dam. It was also noted that there is considerable slumping on the right bank.

As part of the investigations for the 1976 feasibility study, geological reconnaissance and seismic methods were used to select a dam axis which would be geologically more favourable. Two potential alternative axes downstream from Axis C-1 were considered namely Axis C-2 and Site C.

The following significant geological features of Axis C-2 were reported:

- there is a considerable amount of slumped material on the right bank;
- slopes on both sides of the Peace River could produce large slides, both from the in-situ material and slumped material; and
- a buried channel of the Peace River connects the Peace River to the Moberly River valley less than 3 km southwest of Axis C-2.

The principal factors that were considered to make Site C preferable to Axis C-1 and C-2 were:

- the overburden thickness at Site C is less than at Axis C-1 or Axis C-2;
- the dam abutments at Site C appear to be stable whereas the abutments at Axis C-1 and Axis C-2 are unstable;
- terraces protect both abutments at Site C from potential slides originating from above the crest of the dam; and
- locating the dam downstream of the Moberly River provides slightly greater power flows and increased head, and avoids the problem of deposition of significant quantities of gravel from the Moberly River into the tailrace channel.

Based on these advantages Site C was selected as the axis for the 1976 feasibility study.

1.1.4 1978 Preliminary Design Study Phase I

The BC Hydro 1978 Preliminary Design Study Phase I(3), which incorporated the results of all investigations and studies up to June 1978, gives the following advantages of Site C over Axis C-2 in addition to those listed in the 1976 feasibility study:

- bedrock is exposed in the left bank at Site C up to the crest of the dam, and only shallow overburden overlies bedrock on the right bank; and

- a terrace on the right abutment at Site C, which is at an elevation close to the dam crest level, widens out downstream of the dam axis and provides an advantageous natural location for high level spillway headworks and power intakes.

The latter topographic advantage is of major significance. All of the prior studies had assumed a layout with concrete gravity structures located in the river bed. The shale bedrock throughout this reach of the Peace River has weak horizontal planes including thin clay seams and parted bedding planes under the riverbed. These features have very low shear strengths making the use of high gravity structures in the river bed unfeasible.

The hydraulic force acting on structures is proportional to the square of the head acting on the structures. In order to make the spillway headworks and power intakes stable, it is necessary to locate them as far up the bank as possible to reduce the hydraulic loading. The right bank terrace at Site C provides a natural location for these structures that minimizes the excavations required for the structures and for the channels required to convey water to and from them, and also minimizes the height of the concrete structures. On this terrace, the head acting on the spillway headworks and power intakes is about one half of that acting on concrete gravity structures located in the river, resulting in hydraulic forces approximately one quarter of the forces acting on structures located in the river.

There are no such terraces at Axis C-1 and Axis C-2, therefore very large excavations would be required to locate the concrete structures at high level on one of the banks.

1.2 Stability of Valley Slopes

Landslides have played a significant role in the development of the Peace River Valley. Some of the valley slopes are marginally stable and there are many historic and currently active landslides. Since the beginning of the century the following significant slides are known to have occurred in Site C:

- In the early 1900's, movement or reactivation of the Cache Creek Slide at Mile 51 (Site C is located at about Mile 39, measured from BC-Alberta border).

- 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. main line. The slide occurred in overburden.

A number of investigations have been undertaken to evaluate the stability of the valley walls in general and to investigate particularly significant active slides[(4) through (10)].

The 6 km length of the north bank of the Peace River between Tea Creek and the Moberly River is an area where the post glacial Peace River left its ancestral valley and cut down through bedrock to form a new bedrock valley 4 km to 5 km north of the old valley. The south side of the current valley is a relatively flat, vegetated slope formed for the most part by inactive slumps and river terraces. The north side of the valley is steeper, less vegetated and is actively eroding and slumping. Many of the slumps seem to be associated with a thin white clay layer near El. 432, which forms a boundary between disturbed rock above and undisturbed rock below. This clay layer is about 22 m above the riverbed at Site C and 30 m below the proposed reservoir level.

The inherent slope instability is a significant issue for the construction of a dam. Slopes can become destabilized due to the excavations required for access roads and structures. Major additional excavations are required to stabilize slopes and remove active slides.

Three old bedrock slides and one slope with potential for sliding have been identified on the north bank between Tea Creek and Site C.

Immediately downstream of Axis C-1, the north bank is 120 m high and slopes at about 40°. According to Ref. (8), this steep ravelling slope could be the result of either a slide of about 1 million m^3, or gradual sloughing of surface layers due to river erosion. A small amount of debris from this sloughing remains along the river's edge.

Ref. (8) identifies the following three slides on the north bank at or near Axis C-2:

1. A rotational bedrock slump consisting of about 3 million m^3 of disrupted shale.
2. A slide of about 2 million m^3 of overburden and rock that originated from the top of a steep slope and travelled to the river.
3. A slump of about 3 to 4 million m^3 of highly disrupted rock with similar features to Slide 1.

Slides 1 and 2 are contiguous and Slide 3 is a short distance downstream. The three slides form an unstable section of bank nearly 2 km long.

1.3 Topography

Downstream from Peace Canyon Dam the Peace River has eroded through glacial overburden and into the underlying sedimentary rock. The river has formed a broad, flat bottomed valley up to 230 m below the top of the surrounding plain, which is part of the Alberta Plateau.

The reservoir level would be fixed by the tailwater level requirements at the Peace Canyon Dam therefore the dam crest level at all three axes would be the same, El. 469.4.

The topographic mapping used for this overview was TRIM digital data of the Peace River between Peace Canyon and BC/Alberta Boundary in the form of a Digital Terrain Model.

As shown on Figures 2 and 3, at Axis C-1 and Axis C-2 the left bank rises steeply for nearly 230 m to the plateau at about El. 640. The lower part of the right bank has a relatively flat slope to El. 490 and then rises steeply to the plateau at about El. 625. At these two axes the valley walls would be up to 170 m above the crest of the dam.

As shown on Figure 4, at Site C the left bank rises steeply to about El. 570, 100 m above the crest of the dam and then rises relatively gently to El. 610. The right bank rises steeply to El. 450 where a broad terrace is located. The terrace rises gently to El. 480 followed by a relatively flat slope to El. 630.

The flatter slopes and the presence of the terrace make Site C more attractive topographically than Axis C-1 and C-2.

1.4 2005 Reconnaissance of Upstream Axes

At the request of BC Hydro a reconnaissance of the two upstream axes was undertaken in November 2005 by:

- John Boots, P. Eng. of BC Hydro;
- Tim Little, P. Eng. of BC Hydro;
- Dr. Alfred Hanna, P. Eng. of SNC-Lavalin; and
- John Nunn, P. Eng. of Klohn Crippen Berger.

Appendix A contains notes and photographs of the site reconnaissance.

Based on the observations made during the reconnaissance and an understanding of the geology and topography of the area, the engineers on the reconnaissance agreed that Site C is geologically and topographically superior to Axis C-1 and Axis C-2. Nevertheless, the dam could be constructed at either upstream axis although the excavation quantities would be significantly increased and there would be significantly greater problems with the stability of the valley slopes.

2. GEOLOGICAL SECTIONS

2.1 Approach

Simplified geological sections at Axis C-1 and Axis C-2 were prepared so that the order of magnitude of changes to the major quantities resulting from moving the dam from Site C to one of the upstream axes could be assessed.

The results of previous investigations and studies including drill hole and other data were reviewed and used to estimate the depth of overburden and the top of bedrock at each of the upstream axes. The profile of the bedrock across the valley was then established from the ground profile obtained from the topographic mapping by subtracting the overburden depth from the ground profile.

The cost of rock excavation by drilling and blasting is significantly greater than excavation by ripping using heavy bulldozers. The results of seismic refraction surveys were used to establish the boundary between rippable rock and non rippable rock at Site C(19). This data was used to estimate the depth of rippable rock at each upstream axis.

The investigations have shown that the stratigraphy of the shale is quite uniform and can be extrapolated reliably using key markers. The locations and dip of weak planes parallel to bedding were determined by extrapolating the geological section at Site C upstream to Axis C-1 and Axis C-2. A stratigraphic correlation between the Cache Creek slide and the Tea Creek Slide based on drill holes DD40-11 and LDH XI was used as the primary source of data.

The following sections describe the geological section at Site C and how the geological sections were determined at the two upstream axes.

2.2 Site C

Figure 5-12 of Ref. (13) shows that the bedrock in the river is approximately 18 m below the surface elevation of the island.

Figure 5-12 from Ref. (13), Figure 11 from Ref. (14) and Figure 5-12 from Ref. (13) show the bedrock surface under the left bank at El. 467. These two references also show that bedrock is exposed along the length of the left bank at El. 467.

Figure 5-12 from Ref. (13) shows the bedrock surface under the right bank at El. 445. The bedrock is also exposed at El. 445 on the right bank.

Ref. (13), Ref. (15) and Ref. (16) and KC35 (page 2-4) state that the bedding planes in the shale dip about 1° towards the north or northeast. A general bedding dip of 1° north was used to determine the apparent dip of the bedding planes in the geological model.

Significant bedding planes included in the geological section are:

- BP-8 which is a white clay possibly from volcanic ash;
- BP-12 which is a layer of marl;
- BP-18 which is significant due to its continuity and silty clay infill;
- BP-25 which is significant due to its weakness, continuity, and location; and
- BP-28 which is significant due to its weakness, continuity, and location.

Ref. (16) states that BP-28 is located approximately 2 m below the bedrock surface in the river channel. The bedding planes were located in the section with BP-25 at approximately El. 396 in the left abutment and BP-28 below the overburden in the river channel.

The bedding plane spacings from Ref. (17) were assumed for the geological section.

Based on the results of the seismic refractions surveys it has been assumed that the boundary between rippable rock and non rippable rock is at BP8. It is considered that the rock is rippable above this bedding plane due to fracturing of the bedrock due to stress relief from valley erosion.

The simplified geological section at Site C is shown in Figure 5.

2.3 Axis C-1

At Axis C-1 the following overburden depths were assumed:

- across the river the overburden depths shown on the borehole logs for holes C1 to C7 drilled at the axis in 1971;
- 15 m on the left slope as given in Ref. (2); and
- on the right slope 15 m at the river and 46 m at El. 457 as given in Ref. (2).

The bedrock surface beneath the top of the left bank was assumed to be at El. 580 as shown in Ref. (8).

The bedrock surface beneath the right bank was estimated to be at El. 550 by assuming the dip of the bedrock surface across the valley is the same as at Site C.

The following weak planes parallel to bedding were assumed:

- weak clay seams beneath the river, encountered in boreholes C3 to C6 as described in Ref. (1);
- a thin white clay layer BP-8 at El. 432 as described for the slide area immediately downstream of the axis in Ref. (8);
- the bedding planes found at Site C were assumed to occur at Axis C-1;

- the bedding plane dip and spacing were assumed to be the same as at Site C; and
- the locations of the bedding planes except for BP-8 were projected from boreholes LDH XI and DH40-11 onto Axis C-1.

It was assumed that the depth of rippable rock below the bedrock surface would be the same at Axis C-1 as at Site C. It was assumed that the upper 20 m of rock would be rippable and that this 20 m thick band of rippable rock runs parallel to the valley walls. BP8 was assumed to be the lower boundary of rippable rock.

The simplified geological section at Axis C-1 is shown in Figure 6.

2.4 Axis C-2

At Axis C-2 the following overburden depths were assumed:

- 24 m near the right bank of the river channel and 6 m near the left bank of the river based on information in Ref. (2);
- 24 m on the left slope and 6 m near the river based on information in Ref. (2); and
- 24 m on the right slope.

The bedrock surface beneath the top of the left bank was assumed to be at El. 590 as shown in Ref. (8). Photographs taken during the site reconnaissance in November 2005 show a shale outcrop between El. 540 and El. 590.

The right bank bedrock surface was determined by considering both the change in slope at El. 580, and checked by projecting the dip of the bedrock surface at Site C onto the C-2 Axis. The projected bedrock surface was at El. 557; however, this did not coincide with the change in slope. Based on the observations made during the site reconnaissance, it was considered that the most plausible bedrock surface elevation was coincident with the change in slope. Therefore, the bedrock surface was interpreted to be at El. 580.

The following weak planes parallel to bedding were assumed:

- a thin white clay layer BP-8 near El. 432 as confirmed by drilling in the nearby slide[(8)];
- the bedding planes found at Site C were assumed to occur at Axis C-1;
- the bedding plane dip and spacing were assumed to be the same as at Site C; and
- the locations of the bedding planes except for BP-8 were projected from boreholes LDH XI and DH40-11 onto Axis C-1.

The depth of rippable rock at Axis C-2 was assumed to be the same as at Axis C-1.

The simplified geological section at Axis C-2 is shown in Figure 7.

3. CHANGES TO GENERATION

Moving the dam axes upstream reduces the project energy due to the removal of the Moberly River inflow, and higher tailwater levels which reduce the overall head for generation.

3.1 Reduction in Mean Annual Flow

The mean annual flow in the Moberly River is estimated to be 0.9% of the mean annual flow in the Peace River at Site C. Construction of the project at one of the two upstream axes would reduce the available mean annual flow by this percentage.

3.2 Reduction in Head

The average gradient of the Peace River upstream of Site C is approximately 0.75 m per kilometre[18]. River levels at the upstream axes are higher than at Site C.

The Site C reservoir would back up to the tailwater level of the Peace Canyon Project. Therefore the reservoir level at the upstream axes cannot be raised to compensate for the higher river levels as this would reduce generation at Peace Canyon.

As a result the head available for generation would be 7% less at Axis C-1 and 4% less at Axis C-2.

3.3 Reduction in Generation

The average annual energy at Site C is estimated to be 4710 GW.h. Moving the project upstream of the Moberly River will reduce the flow and head available for generation.

Table 1 summarizes the reduction in average annual energy for the two upstream axes, assuming that the reduction in average annual energy is directly proportional to the reductions in mean annual flow and head.

Table1 shows that the present values of the lost generation over the life of the Project are estimated to be $192.5 million at Axis C-1 and $119.0 million at Axis C-2.

The terms of reference for this overview study were to assume the same overall dimensions of the structures at Site C. The lower head at the upstream axes would result in lower capacities for the equipment that would be installed at Site C. The installed capacities at two upstream axes would be:

- 837 MW at Axis C-1; and
- 864 MW at Axis C-2.

By comparison, the capacity at Site C is 900 MW.

The values of these capacity reductions have not been assessed.

4. CHANGES TO QUANTITIES

4.1 Methodology

4.1.1 Layout

The major components of the Site C Project shown schematically on Figures 8 and 9 are:

- the earthfill dam and cofferdams;
- the two tunnels required to divert the river during construction of the earthfill dam;
- the excavation to stabilize the left bank above the earthfill dam; and
- the right bank structures comprising the spillway and power generating facilities, including the excavations required to construct those structures and to convey water to and from them.

The scope of this overview is based on the assumption that the project layout at each of the upstream axes would be the same as developed for Site C and that the overall dimensions of the structures would be the same.

The incremental order of magnitude costs for developing the project at each upstream axis were determined by fitting the Site C layout to the topography at the axis and calculating the resulting changes to the major quantities required to construct the components listed above.

4.1.2 Earthfill Dam

The design of the earthfill dam at Site C is based on excavating the river bed alluvium and weathered bedrock under the centre of the dam and then backfilling the excavation with impervious fill to provide a cut-off (seepage barrier). The alluvium, which consists of cobbles, gravels and sands, will be left beneath the upstream and downstream shells of the dam as it provides an acceptable foundation for those portions of the dam. The relatively small amounts of colluvium and other slide material at the toes of the banks would be excavated as these materials would not provide a suitable foundation for the dam. Weathered bedrock on the valley walls will be excavated to expose sound rock suitable for the dam cut-off.

Weak bedding planes exist in the foundation of the earthfill dam at both upstream axes so the dam cross section will remain the same as at Site C.

At the upstream axes there are considerable thicknesses of colluvium and slide debris on the banks. Unlike the alluvium in the river bed at Site C, this material would not provide a suitable foundation for the earthfill dam and would have to be excavated.

The differences in earthfill dam quantities between Site C and the upstream axes were based on:

- excavating the colluvium and slide debris on the banks and replacing them with earthfill; and
- adjusting crest length of the earthfill dam to suit the assumed bedrock profile in the geological section.

4.1.3 Diversion Tunnels

At Site C, two 9.8 m diameter tunnels will be required to divert the river during construction of the earthfill dam. For the design at Site C the inside tunnel (closest to the river) has a length of about 690 m while the outside tunnel (furthest into the hillside) has a length of about 790 m.

The Moberly River contributes less than 10% of the flood flows in the Peace River at Site C, therefore moving the project to one of the two axes upstream of the Moberly River will not significantly reduce the diversion design flood and permit the use of significantly smaller diameter tunnels.

The diversion tunnels must be long enough to bypass the earthfill dam and cofferdams. Since the dam slopes and heights at the upstream axes will be the same as at Site C, the dams would have essentially the same base width. Preliminary tunnel layouts were established based on the topography at each of the axes.

In addition to the tunnel length, the diversion tunnel quantities that could be significantly affected by topography are the excavations for the inlets and outlets. The same excavation slopes were assumed as at Site C and the quantities of the excavations at the tunnel inlets and outlets were estimated for each upstream axis.

4.1.4 Left Bank Stabilization

The overburden in the left bank at Site C is a mainly a glacio-lacustrine (glacial lake bed) deposit. The design for Site C includes a major excavation to flatten and stabilize the slope above the dam.

The shoreline of the glacial Lake Peace is believed to have extended onto the plateau to approximately El. 670[(4)]. Therefore at the upstream axes the same glacio-lacustrine deposits are expected to occur in the left bank and the overburden in the left bank would have to be excavated to the same overall slopes as at Site C. At each upstream axis the excavation slopes were laid out to conform to the general topography.

4.1.5 Right Bank Structures

As stated in Section 4.1.3 moving the dam upstream of the Moberly River would not significantly reduce the flood flows. The spillway is designed to pass the probable maximum flood (PMF) which would occur as a result of a very large storm centred on the Halfway River basin which is located upstream of Axis C-1. Therefore moving to an axis upstream of the Moberly River would not result in a significant change to the spillway design.

The dimensions and layout of the right bank structures at the upstream axes were assumed to be the same as at Site C. As discussed in Section 1.3, the right bank structures at Site C are located on a natural terrace. There are no such terraces at the upstream axes and as a result significantly larger overburden and rock excavations would be required for the structures. Due to the very large increases in excavations, the layouts were modified by moving the right bank structures closer to the earthfill dam. This reduces the excavation quantities but increases the volume of concrete required in the retaining walls that separate the earthfill dam from the spillway.

4.1.6 Quantity Takeoffs

The available mapping that covered all three axes was less detailed than the mapping available for Site C. The same mapping was used for the layouts at all three axes so that the quantities were calculated on the same basis.

Several sections were taken through each excavation and the areas of overburden, rippable and non-rippable rock were estimated. The volume of each type of excavation was then calculated by the end area method.

Preliminary layouts of the diversion tunnels were produced to determine the lengths of the tunnels and the locations of the portals.

The quantities of dam fill were calculated using the end area method with several cross sections through the dam.

Due to the accuracy of the available topographic mapping, the simplified nature of the layouts and the relatively few cross sections taken in this overview, the accuracy of the quantities is expected to have a wide range. However, since the same topographic mapping and locations of cross sections for quantity calculations was used for all three axes, the quantities are comparable and suitable for making a reasonable comparison of order of magnitude costs between the three axes.

The quantities are based on the assumed geological sections at Site C and each upstream axis shown in Figures 5, 6 and 7, which have been based on available information as described in Section 2. The geological section at Site C is relatively well known whereas there are considerable uncertainties associated with the geological sections at the two upstream axes. Additional investigations at Axis C-1 and Axis C-2 would undoubtedly change the geological sections. However, the two upstream axes are fundamentally different to Site C – the slopes are higher, steeper and less stable. Therefore, while the actual quantities of excavation and fill could be significantly different, it is believed that the figures quoted in the following subsections give the order of the differences in work at C-1 and C-2 compared to Site C.

4.2 Axis C-1

Figure 10 shows the assumed layout of the structures at Axis C-1 and Figure 11 shows a section along Axis C-1.

Features that have a significant effect on the cost of the dam at Axis C-1 are as follows:

- the large amount of slide debris that would have to be excavated for the foundations of the earthfill dam and the right bank structures; and
- the excavated slope at the right bank structures would be about 200 m high, 115 m in rock and 85 m in overburden. In comparison the excavation at Site C has a total slope height of about 45 m.

Table 2 shows the major excavation and fill quantities for Axis C-1 and the differences between this axis and Site C. The total quantity of excavation at Axis C-1 would be over 100 million m^3 more than at Site C and the total volume of dam fill would be over 10 million m^3 more.

The diversion tunnels for this axis are complicated at the upstream end by Tea Creek. With the upstream portal as shown on Figure 10 it should be possible to pass under Tea Creek, however that would need to be confirmed, as the bedrock levels in that area are not known. The construction of the upstream cofferdam and tunnel portal may be complicated by any flow from Tea Creek; therefore a plan would be required for dealing with Tea Creek flows. The inside tunnel would have a length of about 926 m and the outside tunnel would have a length of about 1041 m.

4.3 Axis C-2

Figure 12 shows the assumed layout of the structures at Axis C-2 and Figure 13 shows a section along Axis C-3.

Features that have a significant effect on the cost of the dam at Axis C-2 are as follows:

- the large amount of slide debris that would have to be excavated for the left bank stabilization, the earthfill dam and the right bank structures; and

- the excavated slope at the right bank structures would be about 200 m high, 145 m in rock and 50 m in overburden, compared to a total slope height of about 45 m at Site C.

Table 3 shows the changes in the excavation and the fill quantities for Axis C-2. The total quantity of excavation at Axis C-2 would be about 95 million m^3 more than at Site C and the total volume of dam fill would be about 6 million m^3 more.

It was necessary to move the upstream and downstream tunnel portals further into the hillside to provide sufficient rock cover for the assumed overburden depths. The inside tunnel would have a length of about 457 m and the outside tunnel a length of about 657 m. This reduces the length of the tunnels but increases the volume of excavation for the portals.

There is a risk that the downstream end of the tunnels could be located in slide debris, in which case the tunnels would have to be extended up to 400 m downstream to avoid the slide area.

4.4 Alternate layouts

4.4.1 Moving the Structures Closer to the Earthfill Dam

Due to the large quantity of excavation for the right bank structures the layout at Axis C-1 was revised to move the structures 90 m closer to the earthfill dam. This would reduce the total volume of excavation by about

35 million m^3 but increase the volume of the concrete retaining walls between the spillway and the earthfill dam by about 150,000 m^3.

Even with this reduction in excavation, the total volume of excavations at Axis C-1 would still be about 70 million m^3 more than at Site C, and the additional concrete would cost in the order of $100 million. This indicates that while it would be possible to optimize the layout for the right bank structures at this axis to reduce the excavation quantities, the overall cost of the project would still be significantly greater than at Site C.

A similar change to the layout was considered at Axis C-2 where the layout was revised to move the structures 95 m closer to the earthfill dam. This would reduce the total volume of excavation by about 40 million m^3 and increase the volume of concrete by a similar amount as at Axis C-1. This relocation of the right bank structures would not reduce the slope height since the top of the cut would still daylight on the plateau.

Relocation of the structures even closer to the earthfill dam was considered to further reduce the excavation quantities. However, this would cause the left hand side of the spillway to drop off the side of the valley and require a complete re-configuration of the structures which was beyond the scope of this report.

Due to the topography, i.e. the absence of the right bank terrace, it would not be possible to re-configure the right bank structures at Axis C-2 so that the right bank excavation quantities were similar to those at Site C.

4.4.2 Relocating the Structures to the Left Bank

It would be possible to locate either the spillway or power facilities to the left bank in an attempt to reduce the total volume of the excavations. Such layout studies were outside the scope of this overview.

Given the topography of the left bank and the active and potential slides that have been identified, it is considered unlikely that such a change to the layout would result in a significant reduction in the cost of the project at the upstream axes.

5. CHANGES TO COST AND SCHEDULE

5.1 Cost Estimates

Based on the overview level of this study the cost estimates provided in this report are order of magnitude cost estimates. This means that when the words “in the order of” are followed by a cost figure it is expected that only the order of magnitude of the cost is correct, for example a cost in the order of $30 million means that the actual cost is expected to be in tens of millions, not millions and not hundreds of millions.

Since the cost estimates have been prepared by multiplication of the quantities derived as described in Section 4 by “all-in” unit costs derived as described in Section 5.3, costs are sometimes quoted with more than one figure. These figures should not be construed as significant, i.e. numbers that are known with any certainty.

5.2 Investigations and Preliminary Design

A considerable amount of time and money has been invested in the investigations and preliminary design for the Site C axis. Due to the similarities in the geology between the two upstream axes and Site C some of the knowledge would be transferable to the upstream axes. However, considerable investigations would have to be done to confirm the depth and characteristics of the overburden and bedrock, and the location and characteristics of the bedding planes in the bedrock.

It is apparent that the layout of the structures at Site C is not suited to the topography and geology at the upstream axes. Additional layout studies would be required to determine the optimum layout.

For the purposes of this assessment it is considered that 1% of the direct construction cost of the Site C Project would be required to undertake the investigations and preliminary design required to bring the project at one of the upstream axes to the same status as Site C. This work would cost in the order of $15 million and take 2 years or more.

5.3 Construction Costs and Schedule

Construction cost estimates at Axis C-1 and Axis C-2 were prepared based on similar arrangements as Site C. Variances in excavation and embankments increased the direct construction costs (in November 2005 constant dollars) as follows compared to Site C:

Site C	$2.967 billion
Axis C-1	$5.628 billion
Axis C-2	$5.607 billion

The major variances in the estimates are due to the increased volume of right bank excavation. The right bank excavation at Axis C-1 has increased by 131 million m^3 and at Axis C-2 the right bank excavation has increased by 103 million m3. In both cases the majority of the additional volume is rock excavation. Other variances in quantities were relatively minor and the structures (i.e. Powerhouse, spillway, intake, penstocks, switchgear building, and diversion tunnel structures) were constant in each arrangement.

The schedule also factors into the project cost and it is estimated that the in-service date schedules at Axis C-1 and Axis C-2 will be increased by 5 years; two years in investigation and preliminary design and three years in construction. Based on this schedule the fully loaded Project Capital Costs for Axis C-1 and Axis C-2 when compared to Site C is estimated as follows:

Site C	$4.188 billion (in-service of March 2017)
Axis C-1	$9.645 billion (in-service of March 2022)
Axis C-2	$9.660 billion (in-service of March 2022)

6. RISK FACTORS

6.1 Rebound

6.1.1 Allowance at Site C

Rebound of the shale bedrock in the foundations of the right bank structures at Site C has been identified as a significant design issue[20] with predicted ultimate rebound magnitudes in the order of:

- 0.2 m to 0.3 m under the spillway headworks and chute, and 0.3 m to 0.4 m under the spillway stilling basin;
- 0.3 m under the intakes and penstocks; and
- 0.4 m under the powerhouse with a differential of 0.1 m across the base of the powerhouse in the flow direction.

The rebound will result from the reduction of stress in the bedrock due to the excavations. The right bank structures will have to be designed so that they can accommodate the anticipated rebound without adversely effecting the operation of the project.

Measures to accommodate the rebound in the design of the right bank structures have not been evaluated in detail. A number of measures to mitigate the effects of the predicted rebound have been recommended to establish a special allowance in the cost estimate. Most of these measures were considered to be refinements of the design and covered by the contingency[20]. These design refinements were estimated to cost $41 million.

However, the recommendation[20] to move all of the right bank structures 30 m away from the earthfill dam to reduce the differential rebound across the structures was considered to be a change in design criteria, which added $15.3 million to the cost estimate.

6.1.2 Rebound at the Upstream Axes

Rebound is a linear function of the net unloading of the foundation (the weight of rock and overburden excavated for the structures less the loading applied by the structures). The depth of the excavation at Site C will be about 35 m (Figure 9) whereas it would be 100 m to 120 m deep at Axis C-1 (Figure 11), and 85 m to 120 m deep at Axis C-2 (Figure 13). This means that the rebound at the upstream axes would be 2.5 to 3.5 times greater than at Site C.

Further it can be seen from the sections that the slopes of the existing ground at Axis C-1 and Axis C-2 are much steeper than at Site C. Therefore the differential rebound at the upstream axes would be much greater than at Site C. The option for relocating the structures further away from the earthfill dam to limit differential rebound is not available at the upstream axes since there is no terrace at the upstream axes.

The greater rebound and differential rebound at the upstream axes will cause significant problems for the design of the structures. Assuming that the cost of accommodating the rebound is proportional to the amount of the predicted rebound the cost of accommodating rebound at the upstream axes could be in the order of $60 million to $100 million more than at Site C.

In the extreme it may not be economically feasible to build structures that can remain serviceable if the anticipated rebound occurs.

6.2 Earthquake Design Criteria

6.2.1 Allowance at Site C

Seismic studies undertaken since the suspension of engineering work on the project have resulted in increased design earthquake loadings. It is anticipated that the peak ground acceleration of the maximum design earthquake for the project will increase from 0.13 g to 0.2 g or higher[20].

Changes to the seismic design parameters will have impacts on the design of the various components of the project and hence may result in design changes that have cost and schedule impacts. A brief review of the components of the Site C Project indicated that the left bank stabilization could be adversely affected by the change in seismic design criteria[20]. A special allowance of $43 million was included in the cost estimate to allow for the effect of the increased design earthquake on the left bank stabilization.

6.2.2 Effects at the Upstream Axes

Due to the higher slopes and the presence of numerous existing and initiated potential slides, the change to the seismic criteria is likely to have a greater effect at the upstream axes than at Site C. Given the allowance made at Site C it seems reasonable to expect an increase in cost in the order of $10 million or more than at Site C.

6.3 Slope Heights

The angles to which slopes can be safely excavated in overburden and rock at Site C were determined based on the results of field investigations, laboratory testing and stability analyses. In order to lay out the required excavations at Axis C-1 and Axis C-2 it was assumed that the same excavated slope angles could be used.

However, the excavated slopes at Axis C-1 and C-2 would be considerably higher than at Site C, for example the right bank slope at Axis C-2 would be three times higher than at Site C. As a result it may be necessary to use flatter angles to achieve the required factors of safety for the slopes. This would further increase the quantities of the required excavations at Axis C-1 and Axis C-2.

The stability of the high slopes required at the upstream axes would be exacerbated by the increase in the seismic design criteria.

6.4 Access Roads

The entire existing road and highway infrastructure is on the north bank of the Peace River.

6.4.1 Permanent Access to Site C

Access to Site C from Fort St. John and the Alaska Highway (Highway 97) will be via existing municipal and provincial public roads. Two permanent access roads will connect Site C to Fort St. John.

The permanent left bank dam access road will connect to the public road system about 1 km north of the earthfill dam and traverse the left bank stabilization area.

The permanent powerplant access road will connect to the public road system about 1.5 km north of the earthfill dam. From there the road will go east for about 4 km before descending to the left bank of the river. It will cross the main channel of the Peace River on a bridge with a span of about 250 m to the large island downstream of the dam. From there the road will go along the north shore of the island to the right bank structures.

6.4.2 Permanent Access to Axis C-1 and Axis C-2

Since the left bank access road to Site C starts on the plateau and is integrated into the design of the left bank stabilization, it is reasonable to assume that there will be no significantly greater costs for left bank access at either Axis C-1 or Axis C-2.

Due to the high, steep topography and the active sides on the left bank in the vicinity of Axis C-1 and Axis C-2 it is likely that the right bank access road to Axis C-1 or Axis C-2 would follow the route of the Site C access road. West of Site C the road would cross the Moberly River on a bridge and then run along the lower part of the right bank slope. This would be a difficult and costly route due to the instability of the right bank of the valley.

An additional 4 km of road would be required to reach Axis C-2 and 6 km to reach Axis C-1.

To reflect the additional crossing of the Moberly River and additional length of access road, an allowance of $13 million was included in C-1 and C-2 estimates.

6.4.3 Temporary Access Roads

Numerous temporary access roads would be required during construction. For this overview no construction planning has been done to determine the temporary access roads that would be required to construct the project at Axis C-1 or Axis C-2. Given the terrain and slope instability, the construction of temporary access roads would be significantly more difficult and expensive than at Site C.

6.5 Disposal of Excavated Materials

Considerable construction planning has been done for the disposal of the excavated materials at Site C. There are few areas on the valley floor suitable for disposal and it would be prohibitively expensive to haul material up to the plateau for disposal. Therefore disposal areas will be developed by constructing earthfill dykes along the river bank to contain the excavated materials. Due to the low strength of the excavated material it will have to be placed at flat slopes (8H:1V or flatter) to provide stability. The dyke and the placed spoil will have to be protected against erosion during flood flows. The erosion protection at the upstream disposal areas will be temporary and placed up to the crest level of the cofferdam. The erosion protection at the downstream disposal areas will be permanent and will be placed to an appropriate level so that the risk of erosion over the life of the project is acceptably low.

Very large volumes of excavated material would have to be disposed of if the dam was built at one of the upstream axes. No studies have been done to determine how far up and down the river disposal areas would be required. Given the unstable nature of the slopes and the topography of both banks it is likely that disposal of the excavated materials in an acceptable manner would be a major problem.

6.6 River Diversion

For an earthfill dam the sequence for river diversion and construction in the river bed is a major consideration in the design. For the upstream axes it has been assumed that the diversion scheme developed for Site C will be applicable and that the only additional costs will be those for changes in tunnel length and the portal excavations.

The unstable slopes at the upstream axes present risks that may adversely affect the ability to implement the diversion scheme developed for Site C. For example, as stated in Section 4.3 the presence of a slide could require the tunnels to be extended by about 400 m each so the outlets would be located in stable ground. This change alone would increase the cost of the diversion works in the order of $28 million.

7. CONCLUSIONS

The feasibility studies and preliminary design by BC Hydro selected the axis at Site C because this location was topographically and geologically better than the two axes upstream of the Moberly River.

This overview has shown that construction of the Site C layout at the upstream axes would increase direct construction costs in the order of $2.7 billion.

A number of risk factors related to topography and geology have been identified. It is anticipated that while further studies might be able to find alternate layouts at the upstream axes that reduce the excavation quantities from those shown in this overview, the risk factors are likely to result in cost increases.

It is the opinion of the authors that due to the adverse topography and geology it would not be possible to develop a layout and design at the upstream axes that would have a similar cost to Site C.

8. REFERENCES

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TABLES

Table 1
Reduction in Generation Relative to Site C

Axis	Reduction in Energy (GW.h/annum)			Value
	Due to lower head	Due to lower flow	Total	Millions Cad $
C-1	330	42	372	192.5
C-2	188	42	230	119.0

The values given in the above table are based on the following:

- 13 to 15 March, 2006 midweek price of electricity in the Mid-Columbia market of US $45 per MW.h;
- exchange rate of $1.15 Cad to $1 US; and
- a present value factor of 10.

Table 2
Comparison of Major Quantities between Axis C-1 and Site C

	Axis C-1 (thousand m^3)	Site C (thousand m^3)	Difference (thousand m^3)
EXCAVATIONS			
Tunnel Portal Overburden	1,800	854	946
Tunnel Portal Bedrock:			
• rippable	1,260	155	1,105
• drill & blast	2,167	311	1,856
Left Bank Overburden	7,248	10,555	(3,307)
Dam Overburden	19,536	2,399	17,137
Dam Bedrock:			
• rippable	550	84	466
• drill & blast	371	795	(424)
Right Bank Overburden	40,094	14,624	25,470
Right Bank Bedrock:			
• rippable	11,902	3,229	8,673
• drill & blast	60,891	7,038	53,853
TOTAL EXCAVATION	145,819	40,043	105,775
EARTHFILL DAM			
Upstream Cofferdam	1,025	1,391	(366)

Table 2
Comparison of Major Quantities between Axis C-1 and Site C

Downstream Cofferdam	109	283	(175)
Dam	25,556	12,550	13,006
TOTAL FILL	26,690	14,224	12,465

Table 3
Comparison of Major Quantities between Axis C-2 and Site C

	Axis C-2 (thousand m^3)	Site C (thousand m^3)	Difference (thousand m^3)
EXCAVATIONS			
Tunnel Portal Overburden	2,054	854	1,200
Tunnel Portal Bedrock:			
• rippable	1,717	155	1,562
• drill & blast	337	311	27
Left Bank Overburden	7,525	10,555	(3,030)
Left Bank Bedrock:			
• rippable	1,997	0	1,997
• drill & blast	630	0	630
Dam Overburden	12,858	2,399	10,459
Dam Bedrock:			
• rippable	458	84	375
• drill & blast	595	795	(199)
Right Bank Overburden	22,523	14,624	7,900
Right Bank Bedrock:			
• rippable	11,811	3,229	8,582
• drill & blast	72,952	7,038	65,914

Table 3 (cont'd)
Comparison of Major Quantities between Axis C-2 and Site C

TOTAL EXCAVATION	135,459	40,043	95,415
EARTHFILL DAM			
Upstream Cofferdam	979	1,391	(411)
Downstream Cofferdam	101	283	(182)
Dam	20,696	12,550	8,146
TOTAL FILL	21,776	14,224	7,533