PEACE RIVER
SITE C HYDRO PROJECT

STAGE 2 ENGINEERING SERVICES
SUMMARY REPORT

Prepared by

Klohn Crippen Berger Ltd. and SNC-Lavalin Inc.

For

B. C. Hydro

Report No. P05032A02-A
November 2009

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SNC·LAVALIN

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PEACE RIVER DEVELOPMENT
SITE C PROJECT

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EXECUTIVE SUMMARY

As described in the Stage 1 Report, in 2005 the BC Hydro Board of Directors identified the need to review the design and cost estimate for Site C. The findings of the resulting design review by Klohn Crippen Berger Ltd. (KCBL) and SNC-Lavalin Inc. (SLI) were summarized in the Stage 1 Report. KCBL and SLI were engaged to address the design issues identified in the 2005 review as part of the Stage 2 engineering work.

The Probable Maximum Flood (PMF) estimate was updated in accordance with the Canadian Dam Association (CDA) Dam Safety Guidelines and found to be lower than the 1989 estimate because the meteorological extremes now used to estimate the PMF are less severe than used 1989.

The Maximum Design Earthquake (MDE) has increased significantly mainly due to increased seismic activity in the Fort St. John area and the 2001 M5.4 earthquake near Dawson Creek. As a result:

- additional excavation would be required to flatten and stabilize the north bank at the dam, significantly increasing the volume of excavation;
- substantial drainage works, including tunnels into the rock beneath the spillway, power intakes and penstocks, would be required; and
- the foundation of the spillway headworks would have to be reinforced with concrete piles.

The earthfill dam can safely withstand the updated MDE with small deformations.

The Diversion Design Flood estimate was updated using an additional 20 years of stream flow data and has not changed materially.

Site investigations were carried out to obtain more information on fundamental parameters for the design of the project.

Conceptual measures to mitigate the potential rebound of the spillway, power intakes, penstocks and powerhouse were assessed and it was concluded that it would be technically feasible to design these structures to accommodate any anticipated rebound.

Locations have been confirmed for the relocation of surplus excavated materials, including the additional material from the north bank excavation.

The layout of the road and bridge connecting the powerhouse on the south bank to the existing roads on the north bank was updated.

The methodology and criteria for determining five reservoir impact lines have been defined based on a review international practice for determining reservoir shoreline impacts. Preliminary impact lines have been developed but additional work is recommended in order to reduce uncertainties. These impact lines would be reviewed with potentially affected private land owners prior to public release.

Alignment and bridge options for the four segments of Highway 29 that would be flooded by the reservoir have been updated to current BC Ministry of Transportation and Infrastructure standards.

The pros and cons of changing from Francis turbines to Kaplan turbines were reviewed and Francis turbines are recommended based on a thorough analysis considering technical, environmental and operations perspectives.

The capability of the project to draw the reservoir down in an emergency was reviewed and it has been recommended that an assessment of low level outlet capability should be undertaken in conjunction with determination of the optimum project arrangement.

Opportunities for habitat creation in and around the reservoir shoreline were identified.

The general arrangement of the project was developed in 1978. There have been many changes over the last 30 years that could result in different design choices, in particular:

- the original design was governed by static loads (every day loads), now seismic loads govern some parts of the design because of the new estimate of the MDE;
- there is now more information and better understanding of potential rebound (the response of the bedrock foundation to excavation);
- resilience and redundancy are required due to increased seismic design loads and potential rebound
- societal and environmental values are different; and
- the costs of different types of construction have changed relative to each other, meaning that different design choices might now be less costly.

Therefore, it is recommended that the project be optimized to find the design that balances technical, risk, cost, safety and environmental considerations, based on the current understanding of the technical issues and current construction costs.

1. INTRODUCTION

1.1 2005 Review of Design Issues

As described in the Stage 1 Report, engineering work has been undertaken on the Site C Project several times throughout its history. Feasibility studies and preliminary design work by BC Hydro prior to 1981 resulted in the 1981 project design (the 1981 Design), which established the layout of the facilities for Site C. Preparatory engineering activities commenced in 1989 by a team of engineers from Klohn Crippen Berger Ltd. (KCBL), SNC-Lavalin Inc. (SLI) and BC Hydro. However, this engineering work was terminated in 1991 and a number of significant design issues remained unresolved.

In 2005, the BC Hydro Board of Directors identified the need to review the design and cost estimate for Site C. KCBL and SLI were engaged to provide an assessment of design issues that could affect project cost. These included:

- design issues outstanding since 1991;
- design standards that have changed since 1991; and
- design issues that have arisen since 1991.

The findings of the 2005 design review were summarized in the Stage 1 Report.

1.2 Stage 2 Engineering Work by KCBL and SLI

KCBL and SLI were engaged to undertake the following parts of the Stage 2 engineering work and address the design issues that had been identified in the 2005 review:

- update the estimate of the Probable Maximum Flood (PMF);
- prepare an interim estimate of the Maximum Design Earthquake (MDE);
- update the Diversion Design Flood;
- undertake site investigations to obtain more information on fundamental parameters for the design of the dam;
- undertake stability analysis of the north bank at the dam site;
- undertake stability analysis of the earthfill dam;
- undertake analysis of the potential rebound and stability of the spillway, power intakes, penstocks and powerhouse;

- determine locations for the relocation of surplus excavated materials;
- update the layout of the road and bridge that would connect the powerhouse on the south bank to the existing roads on the north bank;
- review international practice for determining reservoir shoreline impacts and make recommendations for the methods and criteria to be used for Site C;
- update the river crossings and alignment options for the four segments of Highway 29 that would be flooded by the reservoir to current standards;
- review the pros and cons of Francis turbines versus Kaplan turbines;
- review the capability to draw the reservoir down in an emergency; and
- assess opportunities for habitat creation in and around the reservoir shoreline.

In addition, engineers from KCBL and SLI participated in public consultation.

2. PROBABLE MAXIMUM FLOOD

2.1 Scope

The Probable Maximum Flood (PMF) is defined as the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a particular watershed. The PMF is determined using a calibrated mathematical model of the watershed.

The critical meteorological and hydrologic conditions that are required for determination of the PMF are:

Summer/fall PMF:

- the Probable Maximum Precipitation (PMP), which is defined as the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of the year, with no allowance for long-term climatic trends; and
- the 100-year return period summer/fall rainfall (the antecedent conditions).

Spring PMF Scenario 1:

- the melting of the Probable Maximum Snow Accumulation (PMSA), which is defined as the greatest meteorologically possible snow accumulation for a given winter season over a particular watershed; and
- the 100-year return period spring rainfall occurring when the runoff due to snowmelt is maximum.

Spring PMF Scenario 2:

- the melting of the 100-year return period snowpack; and
- the spring PMP occurring when the runoff due to snowmelt is maximum.

The spatial and temporal distributions of the PMP are required to calculate the PMF.

Antecedent conditions include snowpack, temperatures and rainfall.

The scope of this Stage 2 study was to review the factors affecting the magnitude of the PMF and update the flood estimate as required. The study was undertaken in accordance with the Canadian Dam Association (CDA) Dam Safety Guidelines, which represent current Canadian practice.

The study included:

- familiarization with previous PMP and PMF studies;
- updating the PMP using data from any large storms that have occurred since the 1989 PMP study;
- engaging a panel of external meteorological experts to assist in the development of the PMP;
- undertaking climatic studies to determine the antecedent conditions for the PMF in accordance with the CDA Dam Safety Guidelines, including:
    - the PMSA;
    - the 100 year snowpack;
    - the 100 year storm; and
    - temperature sequences;
- establishing a watershed model of the Site C local catchment (between Peace Canyon Dam and Site C), and calibrating the model using available streamflow data;
- engaging external hydrological experts to assist in the development of the PMF;
- using the calibrated watershed model to determine the PMF inflow from the Site C local catchment for the three cases recommended in the CDA Dam Safety Guidelines; and
- using the watershed model for the local Site C catchment to conduct sensitivity analyses to determine the possible effects of climate change on the PMF.

In conjunction with the above, BC Hydro would:

- use the available watershed model for Williston Reservoir with the rainfall over the Williston basin associated with the Site C PMP to determine the associated inflow into the Williston Reservoir during the Site C PMP; and
- route the associated inflow through the Williston Reservoir using the current Williston Reservoir operating rules to

determine the W.A.C. Bennett Dam discharges during the PMF from the Site C local catchment.

2.2 Summary

2.2.1 PMP

The PMP is an estimate of an upper physical bound to the precipitation that the atmosphere can produce. PMP calculations typically involve use of actual storms modified by applying moisture maximization and storm transposition. The PMP estimates for the Site C and Williston basins in British Columbia are based on weather station reports from Alberta, British Columbia, Saskatchewan, Northwest Territories and the U.S.

Figure 1 shows the Williston and Site C drainage basins which can be divided into the following four geographic zones:

- a Montane Cordillera[3] zone along the continental divide, which encompasses the Rocky Mountains and adjacent foothills and covers a large portion of southern British Columbia;
- a Boreal Cordillera zone covering the northwestern part of the province, including the northern part of the Williston basin;
- a Taiga Plains zone, forming a relatively small triangle in the northeast corner of the province; and
- a small Boreal Plains zone, encompassing most of the Site C local catchment in eastern British Columbia.

Eastern British Columbia experiences a continental climate with long cold winters and short cool summers and relatively low annual precipitation amounts. The prevailing west-east atmospheric circulation limits the arrival of moisture from the Atlantic or Gulf of Mexico and moist air from the Pacific loses much of its moisture crossing the mountains. As a result, away from the mountains and foothills the average annual precipitation is only in the 300 to 500 mm range, significantly less than half of annual amounts recorded on the British Columbia coast.

Most of the major river systems originate in the Montane and Boreal Cordillera zones, where significant snow-cover remains into May and June because of the high elevations and cool climate. The late spring snowmelt in the mountains corresponds with the June peak in monthly rainfall

[3] Cordillera is a Spanish term for a parallel set of mountains. The Cordillera is a Region in Western Canada that is covered with the Rocky Mountains.

creating the risk of combined snowmelt/rainfall flooding. Extensive heavy rainfall in the May-July period are usually the result of long-lived "cold lows" that persist in the foothill region for several days and draw moist Gulf of Mexico air into the area causing prolonged heavy rainfall due to the presence of the low and upslope winds in the foothills and mountains. With or without contributions from snowmelt, these cold lows are the most common cause of flooding along the eastward flowing rivers. Away from major rivers in the eastern part of the province, severe local thunderstorms are more likely to be the meteorological event associated with flooding.

The Williston and Site C basins are located in northeast British Columbia, approximately between latitudes 54º and 59ºN and between longitudes 121º and 127º W. The Williston basin is fully enclosed in the Rocky Mountains. The main rivers are the Finlay, flowing in a NNW-SSE direction and the Parsnip flowing from SSE to NNW. Both rivers flow into Williston Reservoir which was impounded by the W.A.C. Bennett Dam. The Peace River flows east from W.A.C. Bennett Dam, crosses the border between British Columbia and Alberta, and then flows in a northeast direction to Lake Athabaska. Major tributaries of the Peace River between Peace Canyon Dam and Site C are the Halfway and Moberly Rivers. The drainage areas for the Williston and Site C basins are shown in Table 1.

Table 1 Drainage Areas

Williston basin		
Sub basin	**Drainage area (km²)**	**Cumulative area (km²)**
Finlay	33,967	33,967
Natpak	10,402	44,369
Parsnip	4,800	49,169
Willoc	22,887	72,056
Total Williston basin		72,056
Site C		
Upper Halfway above Graham	3,774	3,774
Graham above Colt Creek	2,360	6,134
Lower Halfway near Farrell	3,338	9,472
Moberly near Fort St. John	1,813	11,285
Middle Peace	3,779	15,064
Total Site C local basin		15,064
Total Peace River at Site C		**87,120**

The accepted methodology for PMP determination involves identification of historical storms, maximizing them based upon the maximum moisture likely to be available to them and transposing them to the watershed of interest. In mountainous terrain this is problematic because rainfall amounts for these storms have significant contributions from local orographic effects which are not directly transposable to other regions. Restricting storm samples to those occurring directly over the basin does not provide a large enough sample of severe storms to ensure that a storm of maximum efficiency has been identified, leading to underestimation of the PMP. To overcome this deficiency, a technique referred to as storm separation methodology has been used wherein the precipitation from a storm occurring over an orographic region is divided into two components:

- a convergence component which is the precipitation not due to terrain features; and
- an orographic component which is caused by the relief.

Canadian precipitation and temperature data were obtained from Environment Canada. Climate data for Montana stations were obtained from the Western Regional Climate Center in Reno, Nevada.

Dew point measurements are used in PMP studies because:

- a good relationship has been found between dew point temperature and the measures of moisture in storms, particularly in the lowest layers;
- the network of dew point measuring stations is relatively dense; and
- dew point measurements are generally available for long periods of record.

The good relationship between dew point measurement and moisture content reflects the validity of certain atmospheric assumptions during storms, except for spring storms for which direct precipitable water measurements must be used based on data collected from upper air balloons.

Upper-air data were used in the following ways:

- precipitable water totals were calculated and used to compute average monthly precipitable water for the period of record, as well as extreme (10- and 100-year) totals; and
- the ratio of precipitable water values for each storm to 100-year values was used in calculations of in-place moisture maximization.

The various steps used in the storm separation method were:

- create a digital geographic information system (GIS) coverage of 100-year 24-hour precipitation for the area under study making use of station estimates of 100-year rainfall and relying on spatial patterns of seasonal rainfall developed using PRISM, a statistical model;
- create a digital GIS coverage of 100-year 24-hour convergence precipitation for the area;
- create a GIS coverage of T/C (100-year total precipitation divided by 100-year convergence-only precipitation) to determine the orographic component of precipitation by assuming that rainfall in the flat plains area of Alberta was unaffected by orographic enhancement;
- identify relevant historical storms during summer months (the largest rain storms) and spring (lower rainfall amounts but a higher potential for rain-on-snow events);
- determine the ratio of observed precipitation to 100-year precipitation for each storm and select the storms having the largest ratios;
- determine the convergence component of each storm using the T/C coverage;
- determine the precipitable water content (PWC) for each storm and create a GIS coverage of maximum precipitable water available over the whole study area;
- maximize storms by multiplying the observed convergence precipitation by the ratio of maximum to observed precipitable water;
- transpose the maximized storms from their storm centers to the basin area by multiplying the maximized convergence precipitation by the ratio of maximum precipitable water on the basin to that at the storm location;
- map maximized/transposed convergence PMP;
- convert convergence PMP to total PMP using T/C ratios; and
- map total PMP.

Software was developed that automatically computed the three day rainfall over the basins for a large number of storm centers and for changes in storm orientation up to 40° either clockwise or counterclockwise.

The governing storms were found to be:

- spring: the 1964-05-01 storm; and

- summer: the 1964-06-07 storm.

Figures 2 and 3 show the spatial distributions of the PMPs for the Williston and Site C drainage basins, respectively.

Tables 2 and 3 show the comparison between the rainfall amounts for the summer PMP with the former 1987 (Williston) and 1989 (Site C) PMP studies.

Table 2 Precipitation on each Williston sub basin for summer/fall PMP

Catchment	1987 estimate			2009 estimate		
	Day 1	Day 2	Day 3	Day 1	Day 2	Day 3
Finlay				45.5	125.1	9.2
Natpak				27.2	79.3	6.0
Parsnip				32.2	96.6	7.5
Willoc				33.3	100.9	7.4
Williston	**38.1**	**103.0**	**16.5**	**38.1**	**108.9**	**8.0**
3-day Total	**157.6**			**155.0**		

Table 3 Precipitation on each Site C sub basin for summer/fall PMP

Catchment	1989 estimate			2009 estimate		
	Day 1	Day 2	Day 3	Day 1	Day 2	Day 3
Upper Halfway				60.7	187.1	11.8
Lower Halfway				51.7	158.0	11.6
Middle Peace				52.5	160.2	11.7
Graham				54.4	189.3	11.0
Moberly				45.5	133.4	10.2
Site C	**87.3**	**236**	**37.8**	**53.8**	**167.5**	**11.4**
3-day total	**361.1**			**232.7**		

2.2.2 PMF

2.2.2.1 Watershed Model

The Site C local catchment (the drainage basin between Peace Canyon Dam and Site C) was modeled by the SSARR[4] model and the UBC Watershed Model.

The watershed was subdivided into the following five sub basins:

- the Upper Halfway River above the Graham River;
- the Graham River above Colt Creek;
- the Lower Halfway River above Farrell Creek;
- the Peace River between Bennett Dam and Site C; and
- the Moberly River.

Calibration was carried out on a daily basis using data for the period 1984 to 1994 and the period 2000-2001 except for the Halfway River which was calibrated using data for the period 1987 to 1994 due to lack of data for the years 1984 to 1986 and after 1995. Dependant on the availability of data, verification was subsequently performed for the year 1995 for the Upper Halfway, the period 1995 to 2002 for the Moberly, 1995 to 2004 for the Graham and 1995 to 2006 for the Lower Halfway. Hourly calibration was not possible due to lack of concomitant and reliable hourly precipitation and discharge data.

The UBC Watershed Model was selected to estimate the PMF as it was better at simulating flood events in the calibration results.

The calibrated UBC Watershed Model is being used to estimate the PMF based on the following three scenarios in accordance with the CDA Dam Safety Guidelines:

- the 100-year return period summer/fall rainfall followed by the summer/fall PMP;
- melting of the 100-year return period snowpack by a high temperature sequence followed by the spring PMP; and

[4] The Streamflow Synthesis and Reservoir Regulation (SSARR) Model was developed by the U.S. Army Corps of Engineers to provide mathematical hydrologic simulations for the planning, design, and operation of water control works and is also used for operational river forecasting and river management activities. As a general purpose mathematical model of a river system, SSARR is a useful tool for streamflow and runoff forecasting, as well as for long term studies of the hydrology of a river. Numerous river systems in the United States and abroad have been modeled with the SSARR Model by various agencies, organizations, and universities.

- melting of the PMSA followed by the 100-year return period spring rainfall.

A fifteen day high temperature sequence is normally sufficient to melt the entire snow cover in the boreal regions of Canada (most of the territory east of the Rocky Mountains). However, due to the large variations in altitude in the Site C local catchment, it can take longer than 15 days to melt a large snowpack. Therefore, a fourth scenario consisting of melting of the 100-year snowpack starting in early May followed by a thirty day high temperature sequence and then the summer/fall PMP in early June was considered.

2.2.2.2 100-year Snowpack

The 100-year return period snowpack was derived from data collected at snow course stations in and around the Site C local catchment. The snow water equivalent was estimated using the log Pearson Type III distribution.

2.2.2.3 100-year Rainfall

The 100-year return period rainfall was estimated based on an analysis of the maximum rainfall recorded at several climate stations in and around the Site C local catchment. The 100-year rainfalls were estimated for the spring (March 1 to May 31) and summer/fall (June 1 onwards).

Each 3-day 100-year storm rainfall distribution was assumed similar to the PMP distribution and was distributed on all sub basins forming the Site C local catchment.

GIS coverage of the 100-year return period summer/fall rainfall was derived from data recorded at several climate stations in and around the Site C local catchment. The 3-day rainfall is assumed to start seven days before the beginning of the PMP. There are therefore four days without rainfall between the 100-year rainfall and the PMP. The 3-day duration 100-year return period rainfall and the PMP are disaggregated over the five sub basins and the data entered in the UBC Watershed model. The summer PMP is assumed to start on August 1.

2.2.2.4 Snow Accumulation

The PMSA was determined by maximization of the snowstorms that occurred in particularly snowy winters. The three most snowy seasons were first determined and all snowstorms identified based on snow depths recorded at all climate stations in the region. Each snowstorm was maximized and the total maximized accumulation for each winter season was computed. The highest total corresponds to the PMSA.

Maximization was done using upper air data as these provide a more reliable estimate of moisture content during the winter and spring seasons.

The 100-year return period snowpack was estimated from data collected at snow course stations in and around the Site C catchment using the log Pearson Type III distribution.

GIS coverage of the PMSA and the 100-year return period snowpack were produced.

2.2.2.5 Hot Temperature Sequences

The objective of constructing critical temperature sequences is to obtain a realistic sequence of high temperatures to accelerate melting of the snowpack. Since the temperature sequences are applicable to snowmelt, they must be derived from data when there is snow on the ground, which sets a limit on the sequence. The sequences are built by estimating the 100-year return period of cumulated temperatures over 1, 2, 3, 4, 5, 10, 15 and 30 consecutive days. Each sequence is normally built in a progressively increasing manner culminating with the maximum temperature occurring just before the PMP is assumed to occur.

There are few climate stations for which snow on ground is measured around the Site C and Williston catchments. The station closest to Site C is Pine Pass Le Moray and this station was used to derive the hot temperature sequence. Hot temperature sequences for all Site C sub basins were derived from the Pine Pass Le Moray sequences based on the respective elevations of the climate stations used to calibrate the sub basins.

2.2.2.6 Estimated Flows

An analysis of hourly discharge at the gauging stations located in the Site C local catchment revealed that the ratio of peak instantaneous to maximum daily discharge should not exceed 1.10.

The UBC Watershed Model only provides an estimate of the inflows from the Site C local catchment. The outflow from the W.A.C. Bennett Dam, which will be determined by BC Hydro, will have to be added to the Site C local discharge to obtain the PMF at Site C.

It is anticipated that the Site C PMF will be lower than determined in the 1989 studies because:

- the PMP for the Site C local catchment is lower than previously estimated; and

- the antecedent conditions recommended by the CDA are less severe than used in the 1989 studies.

When this summary was prepared the watershed modeling of the local Site C catchment to conduct sensitivity analyses to determine the possible effects of climate change on the PMF had not yet been done.

2.2.3 External Contributors

The following external experts participated in the PMP study:

- Mr. William D. Hogg, retired hydrometeorologist from Environment Canada; and
- Mr. George Taylor, manager of the Oregon Climate Service at Oregon State University.

An external review board composed of experienced hydrologists and hydrometeorologists provided guidance to the team throughout the duration of the study. The members of the review board were:

- Mr. Ron Hopkinson, hydrometeorologist formerly from Environment Canada and who now works for Custom Climate Services Inc;
- Mr. Samuel L. Hui, hydrologist at Bechtel; and
- Mr. Ed Tomlinson, chief meteorologist at Applied Weather Associates LLC.

2.3 Key Findings and Next Steps

The PMF estimate for the Site C local catchment was updated in accordance with the CDA Dam Safety Guidelines and found to be lower than the 1989 estimate because the meteorological extremes now used to estimate the PMF are less severe than used 1989.

The discharges from W.A.C. Bennett Dam due to the PMP centered over the Site C local catchment have to be determined and added to the PMF from the Site C local catchment to give the total PMF inflow to Site C for design of the project.

Based on the results to date it is anticipated that the current spillway capacity of the Site C should be adequate for the most severe floods and may be oversized.

The updated PMF from the Site C local catchment with the concurrent discharges from W.A.C. Bennett Dam should be used for optimization of the project.

3. MAXIMUM DESIGN EARTHQUAKE

3.1 Scope

The Maximum Design Earthquake (MDE) is the level of earthquake ground motion for which a dam structure is designed. In accordance with the CDA Dam Safety Guidelines the MDE for the Site C Project would have a mean annual exceedance probability of 1/10,000.

After 1990, BC Hydro carried out an initial province wide seismic assessment study. A more comprehensive province wide seismic assessment study was initiated in 2008. Once it has been completed, the results of this province wide study will be used to establish the MDE for Site C.

The scope of the Stage 2 study was to determine an interim value of the MDE based on current knowledge and state of practice. This interim value will be used for the design of the Site C Project until a new value is established by the province wide seismic study.

The study included:

- reviewing previous seismic assessments of the site and seismic criteria and parameters;
- collecting and compiling additional seismic data available since 1990;
- establishing the MDE based on current methodologies; and
- engaging an external seismology expert to review the interim MDE.

3.2 Summary

The Site C Project would be located at the western edge of the Interior Plains and east of the southern Canadian Cordillera. The rate of seismic activity in the southern Cordillera quickly drops inland from the coast, and increases to the east in the region of the Rocky Mountain trench and western Alberta.

Notable large seismic events in the vicinity of Site C were:

- Magnitude 6 (M6) on 4 February 1918 near McNaughton Lake about 460 km southeast from Site C;
- M5 on 14 May 1978 near McNaughton Lake and about 420 km southeast from Site C;

- M5.4 on 21 March 1986 near Prince George about 230 km from project site; and
- M5.4 on 14 April 2001 near Dawson Creek, British Columbia, about 70 km from the project site.

Most of the seismic activity in the region is due to natural seismicity. However, there are regions where induced earthquake activity has been attributed to hydrocarbon extraction and associated high pressure water injection, including areas close to Site C.

Seismic events recorded within 50 km from Site C show that:

- the maximum historical magnitude was 4.3;
- all the recorded events occurred after 1984; and
- the reported depths of most events were less than 3 km.

The events recorded between 50 km and 100 km from Site C show that:

- all events were less than M3.5 other than the 2001 Dawson Creek Earthquake;
- all recorded events occurred after 1991; and
- the reported depths of most events were less than 3 km.

As shown in Figure 4, since 1985 there have been two clusters of seismic activity within 100 km of Site C, one located in the Fort St. John area just north of Site C and the other located about 70 to 100 km northwest of Site C. All events in the cluster located just north of Site C were recorded after 1984 and had a maximum magnitude of M4.3. All events in the second cluster located northwest of Site C were recorded after 2001 and had a maximum magnitude of M3.2.

Induced seismicity associated with oil and gas extraction in the Fort St. John area has to be considered in the seismic hazard assessment for Site C. The first gas discovery was made in this area in the early 1950s and the first oil well was drilled in the mid 1970s. However, significant oil production and enhanced recovery by fluid injection did not occur until the 1980s. A few years later, increased seismic activity was observed in this area, which was previously considered as a region of very low seismicity. This activity appears to continue until the present time and earthquakes with magnitude as high as 4.3 have been recorded in the Fort St. John area. No earthquakes had been either recorded or felt in this area prior to 1984. The May 22, 1994 M4.3, earthquake, which occurred 10 km northeast of Fort St. John, was considered to be induced. This earthquake was felt in the area bounded by Fort St. John, Charlie Lake, North Pine and Cecil Lake and is the largest recorded earthquake within 50 km of Fort St. John.

The two clusters of seismic activity close to Site C could be due to oil and gas extraction.

Probabilistic seismic hazard analyses were conducted to establish the ground motion parameters corresponding to a mean annual probability of 1/10,000. The probabilistic seismic hazard assessment was considered appropriate as there are no known active faults in the vicinity of Site C that can be correlated to the recorded seismicity in the region.

The Cornell-McGuire method embodied in the computer program Ez-Frisk was used; both aleatory[5] and epistemic[6] uncertainties in the ground motion estimates were considered.

The aleatory uncertainty was incorporated into the Cornell-McGuire analysis framework by integrating the statistical distribution in the ground motion relations and by considering the randomness in earthquake location. The epistemic uncertainties in the source zone models and ground motion prediction equations were treated using a logic tree approach.

Earthquake catalogs obtained from the Geological Survey of Canada, updated to December 2007, were used in the study. A set of alternative source zone models was developed for the study and a set of alternative ground motion prediction equations was used in the prediction of ground motions.

The seismic hazard analysis found that:

- the mean estimate of the 10,000 year return period peak ground acceleration at Site C is 0.23g, which is significantly greater than the 1990 estimate of 0.13g;
- local seismicity in the two clusters north and northwest of Site C and the 2001 Dawson Creek Earthquake are important factors in determining the MDE;
- the seismic hazard is dominated by magnitudes in the order of M6, i.e. the MDE would likely be a M6 earthquake close to Site C; and

[5] Aleatory uncertainty or random uncertainty is due to the physical variability of the earthquake processes such as the randomness of the location of the earthquakes and the scatter in the earthquake ground motions.
[6] Epistemic uncertainty or the professional uncertainty is due to incomplete understanding the physical models governing earthquake occurrence and ground motion generation, i.e. selection and characterization of sources zones, ground motion relations, etc.

- sensitivity analysis showed that the results were not sensitive to assumptions of focal depths and maximum magnitudes.

Dr. Gail Atkinson, Professor & Canada Research Chair in Earthquake Hazards and Ground Motions at the University of Western Ontario was the external reviewer for this study.

3.3 Key Findings and Next Steps

The MDE has increased significantly mainly due to increased seismic activity in the Fort St. John area and the 2001 M5.4 earthquake near Dawson Creek.

The interim MDE established during Stage 2 based on current state of practice should be used for the design of the project until a new MDE is established by BC Hydro in their province wide seismic hazard assessment.

4. DIVERSION DESIGN FLOOD

4.1 Scope

If the Site C Project were to be constructed, the Peace River would be diverted through tunnels in the north bank by the construction of cofferdams across the river channel upstream and downstream of the earthfill dam foundation area, so that the dam could be constructed in the dry. Floods at Site C are a combination of flood flows from the unregulated 15,150 km^2 catchment area between Peace Canyon Dam and Site C (the Site C local catchment) with regulated outflows from Williston Reservoir, which is very large. Discharges from W.A.C. Bennett Dam can be curtailed whenever there is a large flood on the Site C local catchment, reducing risk during construction of the cofferdams and earthfill dam.

The Halfway River, which has a catchment area of 9400 km^2 representing some 62% of the Site C local catchment, dominates flood flows at Site C. Studies in 1989 and 1990 recommended that the 50-year return period flood be used to determine the diameter of the diversion tunnels and the height of the cofferdams. The key characteristic of the 50-year flood at Site C required for the design of the diversion facilities (the diversion design flood) is the flood hydrograph (flow versus time over the duration of the flood). When the previous studies were done to establish the diversion design flood, limited stream flow data were available from within the Site C local catchment.

The scope of the Stage 2 study was to update the diversion design flood with the additional 20 years of stream flow data that are available since the previous studies were done.

The Stage 2 work included:

- reviewing previous estimates of the diversion design flood;
- updating the flood frequency analysis;
- updating the diversion design flood hydrograph;
- assessing the effect of alternative Williston Reservoir discharges on the upstream cofferdam freeboard;
- assessing the performance of the diversion tunnel and upstream cofferdam designs under updated diversion design flood and historic flood conditions; and
- assessing the downstream effect of a hypothetical breach of the upstream cofferdam.

4.2 Summary

A hydrograph for the Site C local catchment was determined from the highest recorded flood, which occurred in June 2001. The peak instantaneous (maximum mean hourly) flow in the June 2001 flood is estimated to be 3334 m^3/s, which indicates is 1.08 times the maximum mean daily flow. This is lower than the ratio of 1.24 used in the 1989 studies.

A flood frequency analysis was performed using the 24 years of available stream flow data (1984 to 2007). Based on this analysis, the 50-year diversion flood magnitude (maximum mean daily flow) from the Site C local catchment is estimated to be 2979 m^3/s, which is less than the 1989 estimate of 3100 m^3/s.

The flood volume is a significant parameter particularly when the flood is diverted through tunnels which have limited capacity. An inspection of flood hydrographs indicated that large flood durations were up to 11 days. Therefore, a linear relationship was established between maximum mean daily flow and the associated 11 day volume. Flood hydrographs can be obtained by scaling the June 2001 hydrograph so that the 11 day volume is appropriate for the maximum mean daily flow.

An analysis was performed of the ratio of instantaneous flow to maximum mean daily flow, which indicates a trend for a decreasing ratio with increasing flow. At a maximum mean daily flow of about 3000 m^3/s, this ratio is just less than 1.1, which is consistent with the June 2001 flood. This trend line can be used to determine the instantaneous flow for a given maximum mean daily flow. Based on this relationship the 50-year peak instantaneous discharge from the Site C local catchment is estimated to be 3217 m^3/s, which is less than the estimate of 3800 m^3/s in 1989.

The June 2001 flood peak (maximum mean daily flow of 3078 m^3/s) in the Site C local catchment was higher than the estimated 50-year flood magnitude. The 2001 hydrograph was pro-rated down to develop the hourly 50-year flood hydrograph for the Site C local catchment.

The 1989 studies were based on assumptions that Williston Reservoir discharges would be 1440 m^3/s prior to the flood and would be reduced to 600 m^3/s for five days during the flood peak. For the purposes of this study, the same discharges from Williston Reservoir were assumed and added to the 50-year hydrograph for the Site C local catchment to give the 50-year flood at Site C.

The 50-year flood at Site C routed through the two 9.8 m diameter diversion tunnels gave a calculated freeboard on the upstream cofferdam of 0.62 m, which is essentially the same as the freeboard of 0.58 m calculated in 1990.

Since the June 2001 flood was the highest flood of record at Site C, calculations were done to determine the upstream water levels that would occur if an identical flood occurred during construction. The calculations showed that passing the flood through the two 9.8 m diameter diversion tunnels would result in a freeboard of 1.53 m on the upstream cofferdam, largely due to the fact that the Williston Reservoir discharges before the flood were much lower than 1440 m^3/s.

The 2001 flood at Site C, adjusted to include the Williston Reservoir discharges assumed in the 1989 studies, routed through the two 9.8 m diameter diversion tunnels gave a calculated freeboard on the upstream cofferdam of zero. This flood has an estimated return period of 60-years. Increasing the duration of the flow reduction from Williston Reservoir to 8 and 10 days increased the freeboard to 0.45 m and 0.52 m, respectively.

Analyses were performed to assess the flooding at downstream locations that would result from a hypothetical breach of the upstream cofferdam due to piping or overtopping. A piping failure was found to produce the highest water levels at Old Fort, which is located on the north bank about 5 km downstream of Site C. Several outbuildings and trailers in Old Fort appear to be located below this flood inundation level of ~El. 413 m.

The other significantly populated area downstream of Site C is the Town of Taylor, where all buildings appear to be well above the flood elevations that would result from a failure of the upstream cofferdam.

4.3 Key Findings and Next Steps

The Diversion Design Flood estimate was updated using an additional 20 years of stream flow data and has not changed materially, therefore the diversion facilities at Site C have sufficient capacity to pass the updated 50-year diversion design flood.

The design of the diversion works should be optimized, taking into account the risk and consequences of a flood exceeding the selected diversion design flood. The Stage 2 studies provide the information required to establish the hydrographs of different return period floods for use in optimization studies, including confirming the nature of the outbuildings in the Old Fort area.

5. GEOTECHNICAL INVESTIGATIONS

5.1 Scope

5.1.1 North Bank

A large excavation of the north bank above the earthfill dam is required to stabilize the slope.

The field investigation of the north bank excavations to obtain data for the design of the slope stabilization was an outstanding task when the engineering work on project was terminated in 1991.

The scope of work for Stage 2 was to undertake the field investigation program recommended for 1990, including:

- a site visit to determine the current condition of the left bank slopes and to undertake a condition assessment of all previously installed instrumentation;
- drilling six or more drill holes with undisturbed and disturbed sampling of the soil horizons;
- installation of several pneumatic piezometers in each hole to provide information on groundwater conditions in the slope;
- geological mapping of the slope pitting; and
- testing of soil samples to determine the strength and other parameters required for the stability analyses.

5.1.2 Construction Materials

Approximately 17.5 million m^3 of material would be required for construction of the earthfill dam, including:

- 13.5 million m^3 of granular material (sand, gravel and fine rock);
- 3.5 million m^3 of impervious material; and
- 240,000 m^3 of coarse rock riprap.

In addition, approximately 5.0 million m^3 of fill materials and 405,000 m^3 riprap will be required for construction of the Highway 29 relocations (based in the short bridge and long causeway option). The fill materials for the highway will either be materials from the highway cuts or from local sources such as gravel deposits on the terraces, in river bars or islands. The riprap for erosion protection of the river crossings would have to come from quarries. It is expected that the Dunvegan sandstone from Tea

Creek would be suitable for temporary riprap on permanently submerged parts of the embankments, however the permanently exposed riprap would have to be more durable and likely come from a quarry in the Rockies. Due to the large volume of highway riprap, locating potential sources was included in the investigations for the construction materials.

The source, quantity and suitability of the materials affect the capital cost of the project. The location and extent of the sources must be identified and plans for the development, use and reclamation of those sources have to be known to evaluate the cost, schedule and environmental footprint of the project.

Previous studies have demonstrated that all of the granular materials would come from the dam site, mainly from the terrace on the south bank and from the excavations required for construction.

During the initial investigation phases of the project (1981), impervious material for the main dam core was expected to come from the north bank slope stabilization excavations. However, a reassessment of the north bank materials in 1989 found that the materials identified in 1981 were unsuitable impervious core material. This was due to the fact that 30% of the zone consists of granular material and the fine materials have a wide range of moisture contents. Therefore, suitable sources of impervious materials have to be identified.

The Shaftesbury shale bedrock that will be excavated for construction of the dam breaks down when exposed to the elements and therefore cannot be used for rip-rap. The sandstone bedrock in the vicinity of the dam site at Tea Creek is not sufficiently durable for riprap. Therefore, suitable sources of riprap have to be identified.

As noted in the Stage 1 Report, detailed field and laboratory investigation programs were required to define the most favourable sources for impervious material and riprap.

The purposes of the Stage 2 field investigations for construction materials were to define the most favourable sources of impervious material and to prove that there is sufficient quantity of suitable material for the impervious core of the dam; and to assess potential sources of riprap.

The scope of work included:

- collecting and reviewing all available studies and data that identify potential sources of impervious fill and riprap;
- defining a program for the additional field investigation and laboratory testing including:

  - o site reconnaissance by ground and helicopter to identify potential borrow areas for the impervious fill material and sources of riprap;
  - o geological mapping of the north bank extending from the dam site to Tea Creek to identify potential sources of impervious fill;
  - o developing a field test program for the investigations; and
  - o establishing laboratory testing procedures; and
- drilling and test pitting or trenching to identify the impervious fill material.

5.1.3 Bedding Plane Pore Pressure Parameters

Throughout the shale bedrock at Site C there are numerous weak horizontal layers known as bedding planes that influence the design of the dam. The bedding planes can weaken under the weight of an earthfill dam as increased pressure on the pores of the bedding planes causes a reduction in sliding resistance. This response to the weight of the dam is known as the pore pressure response and was addressed in the design of the dam.

The purpose of the Stage 2 investigations for the bedding plane pore pressure parameters was to undertake the field investigations required to confirm the pore pressure response of the bedding planes.

The scope of work included:

- collecting, reviewing and evaluating all available information relating to the determination of the foundation pore pressure response;
- undertaking a site reconnaissance to determine how much of the existing instrumentation can be salvaged and reinstated;
- developing the field investigation program; and
- installing instrumentation in the rock foundation, constructing the test embankment proposed in 1989, monitoring the pore pressure response, and interpreting the results.

Based on the review of the available information, the scope of the field work was revised to utilize pump tests to determine the pore pressure response rather than constructing the test embankment.

5.1.4 Rebound

Excavations of up to 65 m in depth are required for the construction of the south bank structures (spillway, power intakes, penstocks and powerhouse). Typically the load applied by the structures themselves would be less than the weight of the overburden and rock removed from the site, resulting in a net stress relief over much of the foundation area. This would allow the shale bedrock at the site to expand or rebound. Because of the importance of rebound for the design of the structures, a series of laboratory studies were conducted on samples of shale from one hole on the north bank and analyses were conducted in 1990 to determine the maximum amount and rate of rebound. Due to shelving of the project in early 1991, the results of the analysis were not incorporated into the design of the south bank structures, which would have to be designed to accommodate the predicted amounts and rates of rebound.

The purpose of the Stage 2 investigations for rebound was to obtain additional data for the prediction of the immediate elastic rebound, the ultimate rebound and the rebound over the economic life of the Project, which would be assumed to be 100 years.

The scope of work included:

- reviewing the results of previous investigations, planning the Stage 2 investigations and preparing technical specifications for all field work;
- drilling on the south bank to obtain bedrock samples; and
- testing bedrock samples to obtain the swelling index of each rock unit in the foundations of the south bank structures.

5.1.5 Reservoir Slopes

As part of previous investigations for the project, instrumentation had been installed in a number of unstable areas around the reservoir rim to monitor groundwater levels and movements.

The purpose of the Stage 2 investigations for these slopes was to resume monitoring of the unstable slopes.

The scope of work included:

- collecting and reviewing all available studies and data that identified and classified potential slide areas;
- undertaking a condition assessment of the slope monitoring instrumentation;

- resuming monitoring of existing useable instrumentation;
- rehabilitating existing instrumentation and installing new instrumentation as required; and
- undertaking periodic reading of the instrumentation.

5.1.6 Geological Information Database

Geological investigations for the project consisting of geological mapping, trenching, drilling, and the excavation of exploratory adits (tunnels) have been carried from 1975 to 1991. A considerable amount of geotechnical data was collected during these investigations, however, the data are not easily accessible and none is available in electronic format.

The purpose of this part of the Stage 2 work was to compile all available geological information into a database for the following purposes:

- make readily available the geological information for engineering studies and design;
- to avoid repeating work and/or investigations because historical information could not be located; and
- provide a database into which new geological investigation information can be entered.

The scope of work included:

- researching and evaluating commercially available geological databases and recommending a preferred software package;
- determining the appropriate framework and structure of the database;
- compiling all data, including checking to confirm any necessary adjustments required due to coordinate datum shifts, elevation datum adjustments, and conversion of units (i.e. imperial to metric); and
- entering the data into the database.

5.2 Summary

5.2.1 North Bank

Drilling, sampling, testing, and groundwater measurement were undertaken in 2008 in order to provide additional information for the determination of the seismic stability of the required north bank excavations; specifically, to confirm and refine the geological model developed during previous investigations.

Eight sonic holes and one rotary hole were drilled through the overburden. A total of 494 m of sonic drilling and 60 m of dual rotary drilling were completed. The sonic drilling provided continuous, relatively undisturbed 102 mm diameter core samples. The deepest sonic hole was 123 m. The soil logging was calibrated against the exposures on the slope and logs of drill holes completed during the 1989 investigations. Sonic soil core and the soil logging were checked by the geologist who mapped the overburden geology during the 1989 to 1990 program. Logging was carried out using the Unified Soils Classification System.

Detailed geological cross-sections of the north bank were revised with results from 2008 drilling, LiDAR topography data, and reinterpretation of select previous drill hole data. The sonic drilling enabled a detailed assessment to be made of the left bank stratigraphy and 15 different soil units have been indentified.

A total of 30 standpipe and 3 pneumatic piezometers were installed in overburden drill holes on the left bank, within 500 m upstream and downstream of the dam axis, during all phases of work prior to 2008. This instrumentation indicated that the regional water table was close to the top of the shale bedrock and two perched water tables exist in the overburden soils. An additional 30 vibrating wire piezometers were installed in the nine holes drilled in 2008 to supplement the data from the piezometers installed in previous investigations.

Groundwater data collected during late 2008 and early 2009 from the vibrating wire piezometers preliminarily confirmed the previously reported ground water regime. The piezometers will continue to be read periodically to establish seasonal variations in groundwater levels.

Laboratory test results provided confirmation of accurate soil classification of the different soil units. The testing consisted of index testing as well as a reassessment of the suitability of the cohesive soils for use as impervious fill. Laboratory testing was completed by Harder Associates Engineering Consulting Inc. in Fort St. John.

5.2.2 Construction Materials

5.2.2.1 General

The total material required for construction of the dam is about 17.5 million m^3. Previous studies estimate the breakdown of construction material volumes as follows:

Concrete Aggregate (for structures)	190,000 m^3
Granular Material (includes dam and cofferdams)	13,500,000 m^3
Impervious Material (for dam core material)	3,500,000 m^3
Riprap (large rock)	240,000 m^3
Total	**17,430,000 m^3**

5.2.2.2 Impervious Fill

Requirements for Impervious Fill

As described in Section 7.2, the earthfill dam would have a central impervious core. The basic requirement for a core material is that it should be practically impervious as it forms the water barrier of the dam. The material should have a coefficient of permeability preferably less than 1×10^{-6} cm/sec. For this basic requirement to be met, the material should have a minimum of 20% finer than 0.075 mm. In other words, it should have a minimum of 20% silt and clay content. The maximum particle size can generally be up to 150 mm provided the material is well graded.

Other important characteristics are:

- the material should have natural moisture content within about 2% of its optimum moisture content as determined by ASTM D-698 such that it can be compacted to a minimum density of 98% standard Proctor maximum dry density;
- adequate shear strength;
- low compressibility; and
- high resistance to internal erosion.

The naturally occurring fine, low permeability materials in the vicinity of Site C are glaciolacustrine[7] silts and clays and glacial tills[8].

The most common fine grained soils are glaciolacustrine silts and clays, which are exposed all along the sides of the Peace River Valley. It is evident from the slumping and sliding of the valley slopes in these materials that they are relatively weak. The glaciolacustrine silts are typically narrowly graded and are therefore likely to be susceptible to

[7] Glaciolacustrine is sediments deposited into lakes that have come from glaciers are called glaciolacustrine deposits.
[8] Glacial tills are materials that were deposited directly by a glacier or ice sheet and may vary from clays to mixtures of clay, sand, gravel and boulders.

internal erosion and piping. Based on their general characteristics, the glaciolacustrine silts and clays are unlikely to be suitable sources of impervious core material, whereas a well graded till is likely to have the required engineering characterizes.

Potential Sources at Site C

Samples of overburden material from the north bank slope were tested to confirm whether this material could be suitable for use as impervious fill. These soils were previously eliminated in part because most in situ moisture contents were more than 2% wet of optimum. Samples collected from sonic core, which was generally drilled dry, were used for in situ moisture content determination and were sufficiently large to provide adequate samples for standard Proctor compaction tests. In addition, gradations, Atterberg limits, and specific gravity tests were carried out.

Most of the north bank soils are fine-grained with more than 30% fines. The glaciolacustrine soils are mostly lean clays and the two most plastic stratigraphic units are the weakest of all of the north bank soils.

A total of nine standard compaction tests were carried out on soil samples. Only one sample had an acceptable in situ moisture content; of the other eight samples seven were more than 2% wet of optimum and one was more than 1% dry of optimum. It was found that in situ moisture contents can vary significantly within each glaciolacustrine unit; for example, the moisture content in one unit varies by approximately 12%.

The 2009 test results confirm the 1989 findings that the materials from the north bank are not suitable for use in the impervious core of the embankment dam due to the variability of the natural moisture contents of the glaciolacustrine soils.

Potential Sources within 10 km of Site C

Though the entire Site C area was glaciated at least three times during the Pleistocene (the last 2.5 million years) the surficial materials found in the area were predominantly derived from the last glaciation. Ice from the Laurentide and Cordilleran ice sheets eroded previous deposits and bedrock and deposited various sediments (silt, sand, gravel and till) in their paths. Either when the Cordilleran and Laurentide ice sheets met or simply when one crossed over the Peace River, around 11,000 to 10,500 years ago, the drainage was blocked forming glacial Lake Peace, which reached approximately El. 632. At some point between 10,400 and 9960 years ago, glacial Lake Peace drained, forming the present drainage pattern. In the Site C area, the deposits resulting from the last glaciation consist of glaciolacustrine silts and clays over till and interglacial river deposits.

The first stage of the investigations for potential sources within 10 km of Site C comprised air photo interpretation. The surficial geology of the study area was first mapped on 1:40,000 scale colour air photos to provide an overview of the area around Site C. This was followed by a review of published surficial geology of the Charlie Lake area. More detailed surficial mapping on 1:15,000 scale colour air photos was then done. Pre-typing (mapping on photos prior to fieldwork) using generally accepted conventions and definitions was completed in mid-April, 2008 and the resulting interpretations were used to guide the focus of the preliminary ground truthing in late April.

Based on the preliminary results of the air photo interpretation, a field reconnaissance was carried out in late April 2008. The reconnaissance was restricted to the south bank and public roadways on the north bank. Soils were investigated by digging shallow pits with a shovel and visually characterizing the soil. The information gathered during the field reconnaissance was then used to edit the initial air photo mapping and to assess the likely depth and extent of various deposits. The main objective of this study was to indicate areas in which more detailed investigations would be beneficial.

A field reconnaissance was carried out in June 2008 to identify potential drill hole locations for the north bank investigation program scheduled for the summer of 2008. The reconnaissance focused on the areas that were identified in 1989, as well as areas that were identified through air photo interpretation and field reconnaissance. Permission was obtained from land owners prior to accessing private land for the reconnaissance.

Further field reconnaissance was carried out in September and October 2008 after the geological model of the area had been refined. Areas where till had previously been observed were targeted in order to determine the extents of these till deposits, as well as areas at higher elevations where little or no glaciolacustrine deposits overlaid the till.

A review of water well data in the vicinity of the project area was carried since well data can provide general information about soil stratifications in the area, depth of ground water levels and the depth to bedrock. In the wells, till was logged in three wells located near the western edge of Fort St. John. However, the quality of the logs varies significantly and it is possible that material logged as gravel and sand with clay may also be till. Nevertheless, it appears that till is present near surface at higher levels and close to the western edge of Fort St. John.

No potential sources of till were identified within 10 km of the dam on the south bank. Potential sources of till within 10 km of the dam on the north bank are shown on Figure 3. A regional investigation of the potential till areas on the north bank commenced in September 2009 and by the end of the 2009 field season 104 auger holes had been drilled and 7 test pits had been excavated. Laboratory testing of samples from the drilling and test pitting for classification commenced in September. More sophisticated testing including permeability and strength tests will be undertaken through the winter of 2009/10. The preliminary results indicate that suitable till for the core of the dam can be found within 10 km of the dam site.

5.2.2.3 Riprap

Dunvegan Sandstone

Reconnaissance of potential sources of Dunvegan sandstone indicated that only the sandstones at Tea Creek, which is located near Site C, and the Charlie Lake (Wuthrich) Quarry are potentially suitable for use as riprap. All other potential sources of sandstone were found to be unsuitable for riprap for a variety of reasons including low strength, friability, abundant shale interbeds, thin sandstone beds, lack of calcareous materials, and access.

Laboratory testing indicated that the Dunvegan sandstone is unsuitable for permanent riprap and may not be suitable for temporary riprap in areas where strong durable riprap is required such as erosion protection for the diversion works. However, Dunvegan sandstone has been used as riprap on some bridge abutments, for example at the Halfway River where deterioration of the rock blocks due to weathering is evident. Temporary riprap is required on the relocated sections of Highway 29 to provide erosion protection on slopes that will be permanently submerged after reservoir filling. Dunvegan sandstone may be suitable for temporary riprap in such non-critical areas.

Cadomin Sandstone and Conglomerate

According to construction documents for the W.A.C. Bennett Dam, the sandstone riprap on the dam was obtained from local Cadomin or Gething Formation rocks about 1 km upstream from the dam. Rock strength and weathering data indicate that the material should be sufficiently durable to be used as riprap; however, low specific gravity test results cause some uncertainty about this conclusion. The existing riprap, which has been in place on the upstream face of W.A.C. Bennett Dam for 30 years needs replacement.

Sandstone from the Cadomin Formation in the area of Bullhead Mountain, if proved suitable, could be used for the permanent riprap on the realigned sections of Highway 29. It is estimated that about 338,000 m^3 of permanent riprap could be obtained from both the Castle and Pringle quarries combined.

Lemoray (West Pine) Limestone

The BC Ministry of Transportation presently operates within the Lemoray limestone quarry and suitably sized riprap rock has been produced. BC Hydro has recorded a Notation of Interest for part of the Lemoray quarry. The Lemoray quarry is approximately 145 km by rail from Site C (Septimus Siding).

Laboratory testing completed to date for the Site C Project indicate that the rock has an overall good quality rating with deficiencies relating only to localized presences of chert or flint. These rock mass deficiencies require additional investigation within the area of BC Hydro's present Notation of Interest by drilling investigations and experimental test blasting. Preliminary detailed mapping of the Notation of Interest area is recommended prior to any other investigations. It is difficult to accurately determine available riprap volumes and quarrying cost because there is no subsurface data within BC Hydro's Notation of Interest.

Presently, the Lemoray limestone is recommended as the source of permanent riprap. However, the logistics of transporting riprap to the Site C project are significant. Rail transport for permanent riprap is attractive as the CN Railway Septimus siding is within 4 km of the dam site. Rail transport for temporary and permanent riprap sources could be economically viable. Initial contacts with CN Rail representatives regarding rail transport of construction materials confirmed that this is a feasible option.

5.2.3 Bedding Plane Pore Pressures

Construction of a test embankment was planned for 1990 to estimate the pore pressure response of the bedding planes during construction of the Site C dam. A site for the test embankment was selected on an island in the Peace River, along the proposed dam alignment. The test was instrumented with vibrating wire and pneumatic piezometers in 1989, to provide information on pore pressure build-up and dissipation in response to construction of the test embankment. Piezometers installed on the site in 1980 and 1981 were also available for monitoring.

As part of the ongoing investigations, a large diameter drill hole located in the area of the planned test embankment was dewatered in August 1989 to provide access for personnel to log the stratigraphy in the hole, and to install piezometers. The hole was maintained dewatered from August 30 to September 28, 1989, and was cemented up after logging and piezometers installations were completed on September 29, 1989. Piezometers installed in the embankment area were monitored during the summer and fall of 1989, to document the drawdown that occurred while the large diameter drill hole was dewatered.

The test embankment was never constructed as all investigation work for the project was cancelled following the 1989 field program. A review of the originally proposed test embankment program in 2008 concluded that the results of such a field test might not be definitive. Therefore, it was proposed to undertake a pump test to determine the hydraulic characteristics of the bedding planes and allow an assessment of the pore pressure dissipation rate during construction of the dam.

Prior to committing to a pump test, and the development of a pump test program design, it was decided to assess the drawdown data collected in 1989. The purpose of the assessment was to provide some indication of the hydraulic character of the shale units and bedding planes; to confirm that a pump test would be a feasible method to measure the hydraulic conductivity of these units; and to predict the pore pressure dissipation rates that could be achieved during construction of the dam.

No flow data were recorded during the 1989 pump test, and the hole was cemented after the test. Well recovery data are therefore not available from the drawdown to estimate well inflows.

A two dimensional axisymmetric numerical model based on an existing stratigraphic section through the site was setup to represent the 1989 pump test site using the FEFLOW 5.3 finite-element modeling code, developed by the Institute for Water Resources Planning and Research. The model was calibrated in steady-state mode for both the initial (pre-drawdown) and end of drawdown conditions, and was then run in transient mode to simulate the drawdown trends monitored during the 1989 drawdown interval.

Hydraulic conductivities back-analyzed from the calibrated model are reasonable when compared to typical values for shale rock units and bedding planes in shale strata. The values indicated by the model calibration exercise also fall within the lower bound of the range of reported field testing results. The inflows predicted by the model were considered to fall within a reasonable range, based on the observed rate of water level recovery in the large diameter dill hole in early August 1989.

The 1989 drawdown monitoring, and the results of the numerical modeling, demonstrated that properly designed pump tests focused on the critical bedding planes will induce measurable responses that can be analyzed to determine the hydraulic character of the bedding planes and the bounding shale units. Therefore, a pilot pump test was designed to confirm the techniques that would be used in a full scale pump test. The piezometers required for the pilot pump test were installed in late 2008 and the test was completed in late September 2009. The information obtained from the pilot pump test was used to determine the details of the full scale pump test. Installation of 97 vibrating wire and 6 pneumatic piezometers for the full scale pump test commenced in late September 2009. The full scale pump test was completed in October 2009 and the results of the test will be evaluated over the winter 2009/2010. Early indications are that the permeability of the bedding planes is relatively high and would allow dissipation of pore pressures due to construction of the dam.

Piteau Associates Engineering Ltd. was retained as a subconsultant by SLI to conduct the assessment of the 1989 pump test data.

5.2.4 Rebound

Previous site investigations had obtained samples of shale bedrock and a number of laboratory tests were conducted to asses the swelling characteristics of the shale bedrock. However, samples were only obtained from one hole on the north bank and some of the shale units were not sampled at all.

An investigation program was developed to obtain samples for testing to obtain the swelling indices and properties necessary for analysis of time-dependant swelling from each shale unit on the south bank. Three drill holes were planned near the stilling basin, powerhouse and power intakes with hole depths ranging from 85 m to 97 m, with the deepest holes extending a depth of 15 m below the base of the deepest proposed excavation.

The shale rock deteriorates relatively quickly when exposed to air and moisture. Therefore, samples have to be preserved to provide good quality undisturbed samples for laboratory testing. Two different sample preservation techniques were used: sealing samples in wax; and vacuum sealing samples in PE-nylon and Aluminum PE-nylon bags. The two sample preservation techniques were used in case one method performed poorly with time. Vacuum sealing was considered likely to have the better potential to preserve the samples in their in situ state, specifically with respect to moisture content, for a longer period of time than the conventional wax sealing technique. Detailed procedures for each technique were developed for use in the field.

One hole was drilled in the 2008 field program from 29 September to 9 October to a depth of 92.6 m (El. 369.3). Samples from each shale unit were preserved in accordance with the written procedures. Samples were then placed and stored in insulated, cushioned, wooden shipping containers in an upright position and maintained at a temperature between 2°C and 10°C with freezer packs. The samples were shipped to Vancouver for testing.

Testing of eight samples commenced in January 2009. Each test is expected to take approximately eight months to complete. The results to date have confirmed the order of magnitude of the swelling characteristics obtained from previous investigations.

The remaining holes will be drilled in the late summer of 2009 and testing of samples from those holes will commence in the fall of 2009.

5.2.5 Reservoir Slopes

5.2.5.1 General

Generally the Site C reservoir slope instrumentation system has aged well. All of the 32 standpipe piezometers that were rehabilitated in 1989 were located and most were found to be in good working order. All 11 of the reservoir slope inclinometers were located and most were found to be operable to their previously recorded depths. Some new potential movement zones were identified at Cache Creek and Tea Creek while movements at Attachie and the Moberly River have followed historic trends.

Access to the various instrumentation sites has deteriorated since 1989. Helicopter landing pads are barely distinguishable and access trails have become overgrown with vegetation. Several small sloughs have occurred on the site access road from Highway 29 to the Cache Creek slide area. Several instrumentation sites require clearing and brushing to create space for working around the instruments.

5.2.5.2 Groundwater Levels

All but two standpipe piezometers were found open to their installation depths, which is encouraging as it indicates the majority of piezometers are still monitoring their zones of interest. Falling head tests conducted in each piezometer found that most piezometers have hydraulic conductivities greater than 1.0×10^{-6} cm/s. However, three piezometers displayed signs of potential blockages.

Piezometric pressures have generally continued to follow historic trends with a few instruments showing noteworthy changes since 1995. Only one standpipe has indicated piezometric pressure well above its historic trend. At Attachie, in August 2008, a piezometer indicated a pressure 6.4 m above its historic range of El. 605.5 to El. 611.9. This increase however was short lived; as in October 2008 a second reading shows piezometric pressures back down to El. 605.4.

Three piezometers have displayed significant pressure drops below historic levels:

- at Lynx Creek a piezometric pressure dropped 27.1 m since August 1995 and a review of this instrument's history shows that it is still dissipating standing water induced from flushing in May of 1981; and
- at Tea Creek two piezometers have shown pressure drops of 2.7 m and 18 m, respectively since August 1995 following a trend of decreasing pressures since installation in April of 1980.

The two piezometers at Tea Creek are located in slide areas below the white clay layer around El. 415. These ongoing pressure drops may be evidence of continued widespread pressure dissipation within the Tea Creek slopes although no evidence of this is apparent from piezometers at higher elevations.

5.2.5.3 Slope Movements

All the reservoir slope inclinometers were found to be functional to their installation depths with the exception of one at Tea Creek which was blocked about 0.7 m above the bottom of the hole. Since the last inclinometer profiles were measured in 1995, some new potential movement zones have been identified along weak layers at Cache Creek and Tea Creek. Previously observed slope movements at Attachie and Moberly River have continued to follow their historic trends.

The collar locations of the slope inclinometer casings were surveyed after installation to the North American Datum 27 (NAD27) and using traditional survey techniques. In 2008 the collar locations were surveyed again as the difference between the two surveys would indicate the overall movement of the slope since installation of the inclinometers. The 2008 survey was based on NAD83 and conducted using rapid static GPS. Before a comparison between the two surveys could be made, the results from the first survey had to be converted from NAD27 to NAD83. This was done using Geomatics Canada's software program "National Transformation Version 2" (NTv2), which provides a national standard for transforming coordinates between NAD27 and NAD83. Survey coordinates were entered in transverse Mercator coordinates (Northing & Easting) and a corresponding shift was computed along with accuracies of the shift. The estimated errors in the conversions (one standard deviation) were typically less than 0.1 m in the Northing and the Easting, although for two holes the errors were in the order of 0.3 m in the Northing and 0.2 m in the Easting.

The original survey did not record the survey location on the collar, e.g. center of cap, whereas this information was recorded in the 2008 survey to permit more accurate future surveys.

Significant differences were found between the surveys of some inclinometer locations. These differences could represent slope movements but are more likely due to differences between the surveys. The displacements observed at Tea Creek and Moberly River slopes are well outside the range of possible conversion error and are most likely the result of poor survey control. This brings the accuracy of all of the original survey work into question which should be considered when comparing results. Access to the slope monitoring locations is very difficult and using traditional techniques to establish accurate survey control would have been virtually impossible. It is likely that the intent of the original survey was to be able to locate the instrumentation on maps rather than to enable accurate survey of the movements of the casing collars. Further ground surveys using accurate GPS should be done to measure the movements of the casings.

5.2.6 Geological Information Database

5.2.6.1 Database Selection

The primary consideration was to find a suitable database that is interactive and allows users to readily access the available geological data. The secondary consideration is to be able to use the data for developing geological models. The software has to allow export of data to another modeling software program or the software itself needed the ability to create models.

As of June 2008, there were approximately 868 boreholes and 275 test pits, along with their associated logs, field and laboratory testing results from the previous geological site investigations carried out for the Site C Project. This information, while summarized in various reports, is not available in electronic format nor is the source data readily available.

The primary purpose of the database is therefore to act as a repository for all historical and new geological information so that it can be searched, sorted, filtered and cross-linked for the following purposes:

- make the geological information readily available so that sub-sets can be extracted for engineering studies and design;
- to avoid repeating work and/or investigations because historical information could not be located; and
- to display historical and new geological investigation information together in reports and graphics to provide new insights.

In addition, the database should provide a link to electronic copies of original documents, which is important for two reasons:

- to easily access the original documents related to specific boreholes in the database such as logs, laboratory test data and photographs; and
- to clarify entered data for all users when doubt exists or misinterpretation occurs.

The database will be used for the development of a geological model, which would contain the important information such as rock types, bedding planes, shear intersections, etc. It was recognized that some of the databases might not be capable of geological modeling, in which case it was a requirement that the data should be readily exportable from the database to other specialized software to develop a geological model.

A list of the desired features of the geotechnical database was prepared. A search of commercially-available database programs with log production capability began in early December 2007. Seven potential programs were identified and the capabilities of the seven programs were compared to the desired features list as the first stage of the evaluation process.

During the investigation of potential geological databases, the 1989 Site C database, which used software called PCXPLORE, was evaluated. The software publisher, Gemcom, was contacted and GEMS, which is the updated version of PCXPLORE, was added to the list of potential database packages for use on Site C. Electronic copies of the original database were obtained and submitted to Gemcom for evaluation.

Based on the desired features and the previous 1989 database, three programs were selected for further investigation. A detailed comparison of features, cost and client lists were then carried out.

Based on the evaluation of the commercially-available database systems, gINT Professional was selected because:

- it has all of the required features to easily and efficiently enter historical and newly-acquired data;
- it will allow searching all the previous data accumulated from the past 30 years of site investigation;
- is capable of linking the original documents to specific boreholes in the database; and
- will serve future investigation purposes as borehole data can be reported in various formats such as logs, fence diagrams, histograms, graphs and graphical tables.

5.2.6.2 Geological Modeling Software

The stratigraphy, geological features, bedding planes and groundwater levels are all important factors that affect the design of the dam, structures and excavations. Therefore, it is important to be able to create a three dimensional geological model from the database.

A three dimensional geological model must be able to

- model the geologic structure as a layered formation while keeping the stratigraphic hierarchy consistent;
- model the locations and extents of geological features such as shear zones and faults;
- allow for different interpretations of geological features to be made and considered;
- model the bedding planes, which extend throughout the site, between and within the various rock units;
- model the ground water table and water flow;
- assist in understanding the subsurface characteristics of the site, by producing geological sections and profiles; and
- export data into other programs for hydrogeological and geotechnical analyses.

Commercially available three dimensional geological modeling programs were evaluated according to the following requirements:

- ability to create complex geological features such as faults, shear zones, bedding planes and dykes;
- ability to model surface topography and sub-surface geological layers using borehole data;

- ability for three dimensional visualization and animation;
- ability to input/import data in a format that is compatible with gINT;
- output data format should be compatible with programs commonly used for hydrogeological and geotechnical numerical analysis;
- must have successful case histories in geological modeling and visualization in civil engineering applications; and
- must have high quality technical support service including regular software updates, readily available maintenance, and flexible training programs.

MVS by Ctech Development was selected for three dimensional geological modeling of the Site C Project.

5.2.6.3 Status

The status of data entry into gINT at the end of October 2009 was:

- 62% of the drill hole information had been entered;
- none of the test pit information had been entered;
- 33% of the data from the approximately 6000 field and laboratory tests had been entered; and
- overall the data entry was estimated to be 50% complete.

The data was gathered over a number of investigation programs over almost a 35 year period, and as the knowledge of the geology grew, the understanding of the complexity of the underlying geology increased from one investigation period to the next. For example, in the 1975 investigations the bedding planes were not identified, and the number of rock units identified in the shale were about half of the current classification. In order to make the best possible use of the all the information collected in the investigation programs, the information is being carefully reviewed to ensure consistency from one borehole to the next and from one time period to the next. For the borehole information to be useful in the geological visualization model, it is being evaluated and made consistent with the latest interpretation of the geology so it can be imported to the model.

There are also difficulties in locating all the information and some of it may be mislabeled. For example, a borehole may indicate samples taken for testing, however, the test results can not be found. Alternately test results are found, but the sample locations can not be correlated to a borehole or test-pit.

The development of the geological visualization model MVS is proving to be useful. Being able to have a three dimensional representation of the geology and outlining the excavations is proving to be an excellent tool in understanding the project foundations and the interaction of shears zones and bedding plans in the bedrock. It is envisioned that this will be a valuable tool during the design phase of the project. As discussed above, the borehole data has to be reviewed and made consistent with the latest interpretation of the geology before it can be entered in the model. This requires a geologist or engineering geologist to review the data and revise it to ensure the accuracy and consistency of the descriptions of the lithology and classification of the geological units. At present this work is about 20% complete.

5.3 Key Findings and Next Steps

The key findings from the Stage 2 geotechnical investigations are:

- it was confirmed that the excavated material from the north bank would not be suitable for the impervious core of the dam;
- no suitable impervious material has been found on the south bank within 10 km of the dam;
- it is believed that sufficient, suitable impervious material can be found on the north bank within 10 km of the dam, close to Fort St. John;
- suitable impervious material can likely be found on the south side of the river at considerable distance from Site C requiring rail transport;
- the Lemoray quarry, approximately 145 km from the site by rail, is a suitable location for the supply of durable riprap for the dam and other parts of the project that require permanent or high quality riprap;
- pump tests are likely to provide the information required on the bedding plane pore pressure response, but these tests could not be completed during Stage 2 due to permitting delays;
- sampling and testing to determine the rebound characteristics was commenced but not completed due to permitting delays;

- the majority of the reservoir slope instrumentation was found to be in good condition and operable; however work is required to improve access; and
- geological database and modeling software were selected and obtained, and data entry commenced to protect the investment.

Next steps include:

- monitoring the piezometers in the north bank to confirm groundwater levels in the various stratagraphic units;
- confirming the locations of suitable and sufficient impervious material for the core of the dam (scheduled for the late summer/fall of 2009 and the spring/summer of 2010);
- completing the pump tests and any other investigation work required to confirm the pore pressure response of the bedding planes in the foundation (scheduled for the fall of 2009);
- continuing to obtain samples and test shale bedrock for rebound characteristics;
- installing additional instrumentation in the reservoir slopes, monitoring all instrumentation and undertaking the surveys and analyses required to confirm the locations and extents of potential large slides around the reservoir; and
- completing the entry of historical data into the database and keeping the database up to date so that all data are available for use in the subsequent stages of the project should it proceed.

6. STABILITY OF THE LEFT BANK

6.1 Scope

A large excavation of the left (north) bank above the earthfill dam is required to stabilize the slope.

The Stage 2 work included:

- reviewing previous investigations, design criteria, analyses, stabilization measures and design;
- regular monitoring of the reservoir slope instrumentation;
- undertaking stability analyses for the left bank taking into account all subsurface information and updated seismic design criteria; and
- if the left bank excavation require modification to provide the required factors of safety, laying out the revised slopes and determining the excavation volumes.

6.2 Summary

The proposed Site C dam will be founded entirely on shale bedrock of the Shaftsbury Formation; however, a high slope composed of glacial and interglacial sediments overlies the bedrock on the left bank. This slope will have to be flattened to provide adequate stability.

The 1982 slope design required an excavation of some 15.6 million m^3 to achieve a stable configuration. The slope was redesigned in 1989 to a steeper overall angle, for a saving of about 5.1 million m^3. However, the 2005 design review assumed that the updated MDE would require re-adoption of the 1982 design, or possibly an even flatter slope.

Pseudo-static stability analyses were performed using the computer program Slide version 5. The same slope geometry, stratigraphy, shear strength parameters, and phreatic surfaces, as were used in 1989, were analyzed using Slide. The calculated factors of safety for all eight potential failure surfaces analyzed were within 0.1 of the 1989 results. The similarity in the results verified the program used in 1989 and Slide, which was then used to evaluate the effects of seismic loads on the slopes.

A pseudo-static stability analysis of the 1989 design slope was carried out using Slide for seismic accelerations of 0.05g, 0.10g, and 0.15g. Pseudo static analyses use two thirds of the peak ground acceleration to represent the sustained ground motion. Therefore, the seismic accelerations of 0.10g and 0.15g represent peak ground accelerations of 0.15g and 0.23g, respectively.

In the pseudo-static stability analysis, in addition to calculating the factor of safety for the eight predetermined failure surfaces, a search for the circular failure surface with the lowest factor of safety was also carried out.

The analysis showed that the 1989 slope design:

- is stable for a seismic acceleration of 0.05g;
- is nearly stable for a seismic acceleration of 0.10g, with two of the potential failure surfaces analyzed having a factor of safety less than 1.0; and
- is not stable for a seismic acceleration of 0.15g, and that the slope must be flattened significantly to be stable.

The critical failure planes that primarily control the overall stability of the slope pass through bedding planes BP8 and BP12.

Using the same assumptions for the stratigraphy and shear strength parameters as were used for the analysis of the 1989 slope design, further analysis was carried out to determine at which overall slope angle a factor of safety of 1.0 is achieved for both the 0.10g and 0.15g seismic loads. The slope angles that were analysed were 4H:1V, 5H:1V, 6H:1V, and 8H:1V.

The slope must be flattened to 4H:1V in order to achieve a factor of safety greater than 1.0 with a seismic acceleration of 0.10g.

Bedding planes BP8 and BP12 are the stratigraphic layers that control the stability of the slope for the 0.15g seismic acceleration. The slope must be flatter than 8H:1V to achieve a factor of safety greater than 1.0 for the failure planes through these two bedding planes. The volume of this excavation would be 60 million m^3 compared to 18.2 million m^3 for a slope of 4H:1V. It was concluded that flattening the slope to 8H:1V would not be economic and the slope of 4H:1V should be adopted.

The factor of safety of the 4H:1V slope would be less than 1.0 whenever the seismic acceleration exceeds 0.10g, which implies deformations of the slope during the MDE. These deformations are expected to be small as the durations that the seismic acceleration exceeds 0.10g are expected to be short. A deformation analysis should be undertaken during future stages of design to confirm that the expected deformations of the slope would be tolerable.

6.3 Key Findings and Next Steps

The key findings of the Stage 2 assessment of the north bank are:

- the slope has to be flattened to 4H:1V as a result of the increase in the MDE, increasing the excavation volume from 10.5 million m^3 to 18.2 million m^3; and
- small deformations of the 4H:1V slope are expected during the MDE, deformation analysis will be required in future stages of the project to confirm that the deformations are tolerable.

Next steps include:

- more detailed analysis using the results of the piezometric monitoring and the 2008 site investigations to refine the design of the slope determine whether the slope can be optimized; and
- deformation analyses to confirm that the deformations during the MDE would be tolerable.

7. STABILITY OF EARTHFILL DAM

7.1 Scope

An assessment of the seismic stability of the earthfill dam became necessary because of the higher MDE. The scope included:

- pseudo-static analyses for all the cases studied in 1990;
- investigating the effect of the vertical seismic acceleration acting simultaneously with horizontal seismic acceleration; and
- computing embankment deformations.

7.2 Summary

The proposed design of the earthfill dam at Site C comprises a central core of impervious material supported on each side by shells of granular material. Fine and coarse filters would be located between the core and the shells to prevent the migration of fines from the impervious fill.

The core would be founded on bedrock but the upstream and downstream shells would be founded on alluvium in the riverbed and on the right bank area, and by alluvium/colluvium in the left bank area. The thickness of alluvium/colluvium is generally in the order of 10 m.

The bedrock in the dam foundation is shale having adequate shear strength. However it is the presence of weak bedding planes in the shale that are critical to the stability of the dam.

The earthfill dam has the following main characteristics:

- the maximum dam height would be about 80 m above the base of the cutoff trench underneath the central impervious core;
- both the upstream and downstream slopes of the upper 30 m of dam (El. 469.4 to El. 440.0) would be at 2.5H:1.0V, and in the lower part of the dam the upstream and the downstream slopes would be 7H:1V and 6H:1V respectively;
- the area between the upstream slope and the upstream cofferdam would be filled with compacted excavated material from the north bank excavations (a non-structural fill that adds weight, thereby increasing the sliding stability);

- the central core is relatively wide; with a base width of about 75% of the dam height and 90% of the hydraulic head at maximum normal reservoir level;
- there would be a 4 m wide fine filter zone between the core and the upstream shell and between the core and the downstream shell there would be a 5 m wide fine filter and a 5 m wide coarse filter; and
- slope protection of rock riprap would be placed on the upstream face.

Static and pseudo-static stability analyses were carried out using the computer software SLOPEW. The 1990 stability analyses used only the Janbu method, however both Janbu and Simplified Bishop methods were used for the 2009 analyses. More rigorous analysis methods will be required if the project proceeds to further stages.

The analyses were carried out on the same cases analyzed in 1990, using the same shear strength parameters, reservoir level, the tailwater level and phreatic surfaces. Three sections were analyzed, one through the highest section in the river, one on the right bank and one on the left bank. The analysis showed:

- with the exception of the downstream slope of the dam on the left bank, the factors of safety for all cases under static loading were higher than those in the CDA Dam Safety Guidelines; and
- for the downstream slope of the left bank, the factors of safety at the end of construction and for steady seepage at full supply level were 1.21 and 1.34, respectively compared to the CDA Dam Safety Guidelines of 1.3 and 1.5, respectively, however, it was determined that required factors of safety values can be achieved by extending the core trench downstream by 20 m.

A pseudo-static horizontal seismic acceleration of 0.154g was used to represent the sustained ground motion during the MDE, which is 2/3 of the peak ground acceleration of 0.23g. A pseudo-static vertical seismic acceleration of 0.077g was use for the analyses to determine the significance of vertical acceleration.

The analyses found that simultaneous application of vertical and horizontal accelerations did not significantly change the factors of safety against sliding. For all cases the factors of safety were less than 1.0 implying that there would be permanent deformations during the MDE.

Four different methods were used to estimate embankment displacements during the MDE.

The following displacements were calculated using the Newmark method:

- river section 29 mm upstream and 53 mm downstream;
- left bank section 73 mm upstream and 106 mm downstream; and
- right bank section 22 mm downstream.

The following displacements were calculated using the Bray and Travasarou method for a M6 MDE:

- river section 32 mm upstream and 51 mm downstream;
- left bank section 66 mm upstream and 88 mm downstream; and
- right bank section 26 mm downstream.

The following crest settlements were calculated using the Jansen method for a M6 MDE:

- river section 40 mm upstream and 51 mm downstream;
- left bank section 59 mm upstream and 68 mm downstream; and
- right bank section 35 mm downstream.

A crest settlement of 33 mm was calculated for the highest section of the dam using the Swaisgood method for a M6 earthquake.

The predicted displacements and settlements are well within the tolerable limits for a well constructed earthfill dam with wide filters. If the project proceeds to future stages, detailed dynamic and deformation analyses should be undertaken using the dynamic properties of the fill materials and suitable earthquake time histories.

7.3 Key Findings and Next Steps

The key findings of the Stage 2 stability assessment of the earthfill dam are:

- static stability is adequate for all sections of the dam after a small modification to the left bank section;
- the pseudo-static factors of safety for the long term steady seepage condition would be less than 1.0 for the MDE; and

- preliminary seismic deformations computed using four simplified methods indicate that deformations and crest settlements would likely be small and within tolerable limits.

Next steps include:

- more detailed dynamic and deformation analyses to better define likely deformations during the MDE; and
- optimization studies of alternate core designs, in case sufficient, suitable impervious material cannot be found within an economic distance.

8. REBOUND AND STABILITY OF SPILLWAY AND POWERPLANT

8.1 Scope

The foundation rock at Site C is susceptible to short-term and long-term swelling (called rebound).

Short-term swelling due to the removal of the weight of soil and rock during excavation would be addressed during construction with careful foundation treatment and construction planning. In addition, the design of the structures would have to consider long-term swelling, which may occur over decades if the weight of the structure (spillway, power intakes, penstocks and powerhouse) is less than the weight of soil and rock excavated to reach the foundation of the structure. The majority of the concrete structures on the south bank are susceptible to rebound due to the depth of excavation.

The potential for foundation rebound was first indentified in the 1980s but the work on the project was postponed before assessment and field testing could be done. A preliminary evaluation of the potential long term rebound including laboratory testing of shale samples was done in 1990 but work on the project was terminated before the results of this evaluation could be integrated into the design.

The scope of work in Stage 2 was to undertake the field investigations, update the predictions of long-term rebound and evaluate options for accommodating the likely rebound of the south bank structures.

The Stage 2 work included:

- reviewing previous analyses and considerations for accommodating rebound;
- assessing rebound precedents at other dams, including obtaining additional data from Gardiner Dam and other dams on the measured rebound of the stilling basin and the bedrock properties;
- establishing an analytical model to predict the rebound at Site C, if possible using the data from Gardiner Dam and other dams to calibrate the analytical model;
- reviewing the design and performance of Gardiner Dam and other dams, and based on their performance, assess options for mitigating the amount of differential rebound and for accommodating the predicted rebound in the design of the right bank structures at Site C; and

- assessing the seismic stability of the south bank structures for the MDE.

8.2 Summary

8.2.1 Rebound Analysis

8.2.1.1 General

Long term rebound of a structure would occur when the pressure on the foundation from the weight of the structure is less than the pressure from the weight of soil and rock that was excavated to expose the foundation. The greatest concern is differential rebound, where the amount of rebound across a structure can vary due to different ground levels before excavation and/or to differences in loading under parts of the structure. Differential rebound could result in tilting, cracking or abrupt offsets that would adversely affect the performance of the structures.

The three dimensional consolidation analysis software package Settle 3D was used to model rebound based on the difference between the swelling pressure of the shale or the initial effective stress (whichever is less) and the final effective stress. Rebound calculations were done to predict the rebound at 100 years and rebound values quoted below are the predicted rebound 100 years after project completion.

One of the objectives of the design is to minimize differential rebound so that any deformations are within tolerable limits and do not adversely affect the performance or operation of the structure.

The spillway stilling basin would straddle the valley wall with the north side of the basin located on the valley floor and the south side of the basin located on the terrace. As a result the depth of excavation at the south side would be considerably greater than at the north side resulting in the potential for differential rebound across the basin.

Moving all of the structures to the southeast to eliminate the differential rebound was considered, however this would increase the excavation by 6.1 million m^3 (2 million m^3 of overburden and 4.1 million m^3 of rock) significantly increasing the cost and the amount of surplus excavated materials that would have to be relocated.

Other feasible methods for mitigating rebound that were considered are:

- increasing the foundation pressures by adding weight to the structures;
- adding anchors to load the foundation;

- changing the excavation profile and the shape of structures to minimize differential rebound; and
- articulating the structures to allow for movement without damage.

As described in the following subsections, preliminary assessments were made of the addition of weight and/or anchors to the structures to determine whether it would be technically feasible to use these methods to mitigate rebound. The assessments described below conceptual and if considered feasible, considerable analysis and design would be required to finalize the design details.

8.2.1.2 Power Intakes and Penstocks

Figure 4 shows a section through the power intakes and penstocks.

The foundation pressure under the power intakes would exceed the swelling pressure so these structures would not rebound.

The horizontal sections of penstock between the power intakes and the anchor block of the concrete intake structure were predicted to rebound where the height of backfill above the penstocks would be less than approximately 17 m. This would result in differential displacement of the penstock between the gravel backfill and the upstream side of the anchor block which would likely damage the penstock. Additional backfill to El. 469.4 to the anchor block would eliminate this problem as the foundation pressure due to the additional backfill would exceed the swelling pressure, thus preventing any rebound from occurring. A steep fill slope would be required at the downstream face of the anchor block, which would have to be constructed of reinforced earth or some other form of retaining structure such as precast concrete crib wall.

Anchors could be used to prevent undesirable differential rebound down the length of the 1.86H:1V inclined section of the concrete encased penstocks between the downstream toe of the anchor block and the coupling chambers at the upstream side of the powerhouse. The anchors would be designed so that the rebound of the downstream end of the penstocks would match the powerhouse rebound.

8.2.1.3 Powerhouse

Figure 5 shows a section through the powerhouse.

The foundation pressure under the powerhouse would be non-uniform due to the hydraulic thrust from the penstocks and the non-uniform distribution of weight in the powerhouse from the location of the generating equipment and associated concrete. As a result, differential rebound could tilt the powerhouse causing problems with the generating equipment. As shown in Figure 5, the foundation pressure could be made uniform by adding bins of gravel above the draft tubes and modifying the heel of the draft tubes to provide a flat base. Tilting of the powerhouse could still be caused by rebound of the lightly loaded rock under the coupling chambers at the upstream side of the powerhouse and by rebound in the unloaded tailrace area. However, anchors could be installed at these locations to mitigate tilting of the powerhouse due to rebound.

8.2.1.4 Spillway

Figure 6 shows a section through the spillway.

<u>Headworks and Chute</u>

The foundation pressure under the spillway headworks due to the weight of the structure and the water loads would exceed the swelling pressure so this structure would not rebound. However, immediately downstream of the intake, the lightly loaded spillway chute slab would rebound. Any abrupt offset of the chute slab into the flow could cause cavitation damage that could lead to failure and loss of the slab. Anchors could be used to prevent large differential displacements from occurring between the headworks and the chute slab, and within the chute slab itself. Where feasible, anchors would not be used throughout the entire length of the spillway chute to reduce cost. Instead anchors would only be used to prevent an abrupt offset from occurring at the junction between the headworks and chute and then gradually transition to allow free rebound to occur further downstream.

The lower portion of the chute slopes at 2.75H:1V and the swelling pressure in the shale increases further down the chute due to the greater depth of excavation and characteristics of the lower shale units, although the deeper shale units have a slower swelling rate. The largest rebound would occur near the center of the chute due to the slower swell rate of the deeper shale units. As with the upper part of the chute, the rebound could be controlled by anchors. The first 30 m of the 2.75H:1V inclined section of chute would be unanchored and allowed to rebound and anchors would be installed in the remainder of the inclined section. The anchors would be installed to limit the rebound.

The foundation pressure under the chute side walls would be higher than under the chute slab which would result in differential rebound. The foundation pressure under the walls would be minimized by using a cantilever wall design similar to Gardiner Dam and minimizing the fill placed behind the walls. Nevertheless, there would still be an abrupt change in foundation pressure at the junction between the walls and the chute. In addition, the stress relief across the chute (in the transverse direction) would vary due to the topography of the south bank terrace. As a result the rebound of the chute slab would vary across the chute with abrupt differences at the sides due to the weight of the side walls. Longitudinal walls would divide the chute into six separate bays between the ends of the gate piers and the jet deflector located at tailwater level for dissolved super saturation mitigation. Longitudinal joints would be located at each of these walls to articulate the structure. These joints would be designed to act as hinges and allow the slab to deform to conform to the rebound of the underlying rock. As described above the center portion of the chute is unanchored and allowed to freely rebound. However, anchors would be installed in the bays adjacent to the side walls to transition from the expected rebound to zero at the walls. Dowels located at the center of the slab would connect the chute slab to the walls forming a joint that would prevent any abrupt offset while allowing rotation. The dowels would also supplement the anchors by mobilizing the weight of the side wall.

As shown in Figure 6, the chute floor slab would be 1 m thick reinforced concrete except at the downstream end where the thickness would increase to match the thickness of the stilling basin floor slab. The chute slab would be designed for the loadings that could occur during operation of the spillway and due to the swelling pressures. The slab reinforcement would be continuous in the longitudinal (upstream to downstream) direction so that any differential loadings would be distributed and to prevent any abrupt offsets occurring into the flow. The slab reinforcement would be continuous in the transverse direction in each bay. Dowels located at the center of the slab would cross the longitudinal joints at the dividing walls to connect the chute slab sections together with a joint that would prevent any abrupt offset while allowing rotation. The longitudinal joints would contain compressible joint fillers that would allow the anticipated rotation and waterstops to limit leakage through the joints.

The slab would be underlain by a 3 m thick gravel drain blanket with transverse cutoff walls founded in the shale to prevent down slope migration of the drain gravel. Transverse perforated drains would carry any seepage from the rock into drainage galleries that would run beneath each chute side wall. The transverse drains would be located at the downstream end of each bedding plane as well as at regular intervals. The transverse drains would be located immediately upstream of the transverse cutoff walls. This would compartmentalize the seepage and allow monitoring of the seepage from each part of the chute. Drains would be provided beneath each longitudinal joint. These longitudinal drains would stop at each transverse cutoff wall to prevent down slope flow. The thickness of the blanket would prevent frost penetration of the shale, which has caused problems with spillway chutes constructed on Prairie shales. The drainage system would carry away any seepage, including any seepage through cracks and joints in the chute slab, limiting the access of water to the shale foundation.

Stilling Basin

A 1 m thick gravel drain layer would be installed beneath the stilling basin slab with a network of longitudinal and transverse perforated drain pipes connected to a drainage gallery running around the sides and downstream end of the slab. The drainage galleries would be constructed to cutoff seepage from the tailwater that would surround the basin. Two pump wells would be located in the stilling basin side walls to pump out all seepage water. The purpose of this drainage system would be to prevent uplift pressure acting on the base of the stilling basin slab.

The stilling basin would be founded in shale unit 1. Based on the limited testing to date this shale unit has the highest swelling pressure but has a very low coefficient of swelling and relatively large layer thickness. Therefore, the rebound under the stilling basin would be relatively low and an ultimate rebound of about 8 cm is expected. The stilling basin side walls would be massive mass concrete to prevent vibration from the turbulence during the spillway discharges. As a result there would be differential rebound between the slab and the side walls. The joints would be designed to allow for relative movement at this location. The stilling basin slab would be 3 m thick reinforced concrete with continuous reinforcement in each direction. Anchors would be installed to resist the large hydrodynamic forces that would occur during the PMF assuming the drainage system was not functioning. These anchors would not be designed to restrain rebound and would a low strength steel allowing them to deform elastically under the expected strain due to rebound.

8.2.2 Stability Analysis

8.2.2.1 General

The spillway headworks and the power intakes retain the reservoir and must be designed for the same extreme loads as the dam. The earlier design of these structures was governed by normal conditions, i.e. static loading. However, seismic loading now governs the stability of these structures due to the increase in the MDE.

The following weak bedding planes (BP) in the foundations of these structures control their stability:

- BP18, frictional strength 14º;
- BP25, frictional strength 10º;

- BP27, frictional strength 13.5º;
- BP28, frictional strength 13.5º; and
- BP31, frictional strength 13.5º.

The shear strength of hard, unweathered, intact shale was assumed to be:

- 250 kPa cohesion and a friction angle of 45º across bedding; and
- 250 kPa cohesion and a friction angle of 35º parallel to bedding.

Vertical and horizontal accelerations due to the MDE were applied simultaneously in the combination providing the most severe loading for the stability of the structure. Pseudo static analyses were done with ground accelerations equal to two thirds of the peak to represent the sustained ground motion.

The hydrodynamic loading created by the effect of the MDE on reservoir and tailwater were calculated using methods developed by Zangar. Hydrodynamic loads were assumed to only act above the foundation elevation.

The following load cases were evaluated:

- Normal Condition – maximum normal reservoir level with normal tailwater level;
- Earthquake - maximum normal reservoir level with normal tailwater level and the MDE; and
- Post Earthquake - maximum normal reservoir level with normal tailwater level after the MDE with changes to the uplift loading from cracking that could occur during the MDE.

8.2.2.2 Spillway

The stability of the spillway section shown in Figure 6 was analyzed assuming that movement could occur along one of the weak bedding planes; therefore sliding blocks consisting of the spillway structure and the rock above the bedding plane of interest were analyzed. The horizontal sliding planes pass through the chute concrete but the strength of the concrete was not taken into account. Sliding could not occur on BP18, which is the shallowest bedding plane in the spillway foundation, because part of the foundation of the headworks would extend below this bedding plane.

Key assumptions in stability analysis are the groundwater pressures and the resulting forces acting on the structure or the sliding block under consideration. Initial analyses for the MDE performed using assumptions conventionally used for gravity dams resulted in factors of safety significantly lower than acceptable values. These analyses indicated that BP25 governed the stability of the spillway and demonstrated the importance of the groundwater pressures assumed in the weak bedding planes.

Detailed seepage analyses were then performed to estimate the groundwater pressures in the rock mass beneath the spillway. The seepage model assumed that the shale foundation is a homogeneous isotropic mass, with equal vertical and horizontal permeabilities, and with bedding plane discontinuities ignored. These assumptions are considered to be conservative as the horizontal permeability of the shale is likely one order of magnitude greater than the vertical permeability and the bedding planes are believed to act as drains with horizontal permeabilities up to four orders of magnitude greater than the shale. The shale surface beneath the spillway chute and stilling basin slabs was assumed to be free draining, which is a reasonable assumption based on the drainage measures described in Section 8.2.1.4.

It was found that the groundwater pressures along the critical bedding planes could be significantly reduced by constructing a drainage tunnel along BP25 with vertical drain holes down through BP31. However these drainage measures alone would not be sufficient to provide adequate factors of safety against sliding along BP25 during the MDE. It was apparent that the sliding resistance of the foundation beneath the spillway headworks would have to be increased. The addition of 1.2 m diameter reinforced concrete piles at 5 m centers in both directions down to El. 411 beneath the spillway headworks in conjunction with the drainage measures would result in adequate factors for the MDE.

The stability of the spillway headworks was analyzed separately from the spillway chute. This was done to select a pile arrangement that would provide adequate factors of safety for the headworks without relying on the resistance of the large mass of rock beneath the chute. That is, the sliding stability of the headworks does not require mobilization the shear strength along BP25 beneath the chute to resist seismic loading from the MDE. Preliminary analysis of 1.2 m diameter concrete piles indicated that under seismic loading each pile would deflect in the order of 0.3 mm downstream.

The seepage analysis assumed that the shale foundation is a homogeneous isotropic mass, which ignored known discontinuities such as the bedding planes and cross cutting shears. Some seepage analyses were done assuming that the bedding planes have a significantly higher permeability than the shale. This analysis indicated that the bedding planes would act as drains resulting in lower groundwater pressures than used in the stability analysis, and that shale units 7 and 8 would be above the phreatic line indicating that these shale units would not have access to seepage water from the reservoir, which could significantly limit the potential rebound beneath the chute.

Earthquake shaking and small movements along the bedding planes during the MDE could disrupt the drainage and measures and the upstream impervious blanket. Therefore the post earthquake stability was analyzed assuming that a vertical tension crack forms from the reservoir down to the sliding surface after the MDE. The factors of safety for this case were adequate.

8.2.2.3 Power Intakes and Penstocks

The power intakes were checked for sliding along bedding planes 25, 27, 28 and 31. In order to achieve the desired factors of safety, additional drainage measures and backfill were required.

A total of 132,000 m^3 of additional backfill downstream of the power intakes, and 59,000 m^3 of additional backfill at the heel of the powerhouse were required to help stabilize the power intake structure. Drainage tunnels situated on BP25 and BP31 were also required to reduce the groundwater levels. These measures combined provided the required factors of safety against sliding.

8.2.3 Drainage and Seepage Control Works

As described above, drainage tunnels into the rock beneath the spillway, power intakes and penstocks, would be required to enable these structures to withstand the updated MDE.

The upper tunnel draining BP25 would be constructed as a loop so that there would be two entrances located near the crest of the downstream cofferdam. From the first entrance, the tunnel would extend in the upstream direction, follow BP25 under the spillway headworks and the power intakes, and then loop downstream back to the second entrance. The lower tunnel would branch down from the upper tunnel, follow BP31 under the spillway headworks and the power intakes, and then terminate beneath the south end of the penstock anchor block. An access shaft would be located at this end of the tunnel. A second access shaft would be located near the north end of the spillway headworks connecting the lower and upper tunnel to the dam crest level. Both access shafts would have an elevator to ground level and emergency stairs. The tunnels would have lighting, power, heating and ventilation to provide facilities for routine inspection and for maintenance when required. The extensive drainage tunnels would be a major part of the project.

A number of cross cutting shears would outcrop in the approach channel upstream of the spillway headworks and power intakes. These features could convey water beneath the structures and increase the groundwater pressures above those estimated by the seepage analysis. Grouting would be done beneath the upstream heel of the structures; however this might not be sufficient to prevent ingress of water along the shears as they could be difficult to grout. Therefore, an impervious blanket would be constructed for 100 m upstream of the spillway headworks and power intakes across the full width of the approach channel up to the maximum normal reservoir level to limit seepage into these features.

8.3 Key Findings and Next Steps

The key findings of the Stage 2 assessment of rebound and stability of the spillway and powerplant are:

- it would be technically feasible to limit the deformations of the structures due to rebound so that the structures would be serviceable for over 100 years;
- it would be technically feasible to design the structures to withstand the MDE; and
- the magnitudes of rebound model parameters, specifically the swelling index (Cs) and the coefficient of swelling (Cvs) have a significant impact on the amount of ultimate rebound and rate of rebound predicted.

Due to the sensitivity of the amount of rebound and rate of rebound to parameters such as Cs and Cvs, it is recommended that:

- shale core samples continue to be collected from the various right bank bedrock units;
- constant volume swell tests continue to be conducted on these samples from each rock unit in order to develop representative baseline estimates of critical rebound modeling parameters such as Cs and Cvs; and
- results from the constant volume swell testing be utilized to update ultimate amount and rate of rebound estimates.

9. RELOCATION OF SURPLUS EXCAVATED MATERIALS

9.1 Scope

Construction of the Site C Project would require the excavations of large volumes of earth and rock. Some of these materials can be used for the construction of the dam and the remainder will have to be relocated. Several relocation areas were identified in previous studies.

The purpose of this task was to review the potential relocation areas in light of current understanding of environmental issues, adjust the plan for relocating surplus materials as necessary and identifying potential mitigation options.

The scope included:

- reviewing previous studies and obtaining construction material estimates (for both the 1991 and 1982 dam design) and use the design that results in the larger impacts (i.e. the upper bound volumes of surplus materials) for environmental assessment and permitting;
- assessing the validity of the areas previously identified for relocation, reviewing the locations and confirm their validity in terms of volume and identifying potential environmental issues;
- defining new areas with minimal impacts (as required); and
- updating the material flow chart.

After the MDE was updated and it was apparent that the volume of material excavated from the north bank would increase significantly, it was necessary to indentify additional areas sufficient to contain all of the materials that would have to be relocated.

9.2 Summary

Figure 7 shows the construction facilities at the site and indentifies the areas that have been identified for the relocation of surplus excavated materials.

A total of 40.6 million m^3 of surplus excavated material would have to be relocated. Table 4 summarizes the capacities of the various relocation areas and the sources of the surplus excavated materials that would be placed in each area.

Area L3 was expanded to include the gully area to the east. The toe of the slope was selected to lie within the current BC Hydro property line.

Table 4 Relocation Areas – Capacities and Sources of Surplus Materials

Area	Capacity	Sources	Volume (m^3)
L3	**15,204,000**	North bank road	504,000
		North bank stabilization	12,946,000
		Access road	1,877,045
		Total	**15,327,045**
L5	**6,140,000**	L5 dyke waste	104,500
		Diversion inlet cofferdam foundation	33,000
		Diversion inlet & outlet breach	215,600
		Upstream cofferdam foundation (north side)	11,000
		North bank stabilization - Stage 1	3,462,800
		North bank stabilization - Stage 2	1,550,000
		Diversion inlet overburden & rock	604,400
		Diversion inlet channel	165,000
		Dam core trench	358,540
		Total	**6,504,840**
L6	**848,000**	L6 dyke waste	49,500
		Diversion outlet cofferdam foundation	70,400
		Dam downstream cofferdam foundation	11,000
		Tunnel rock	139,920
		Diversion outlet rock	378,000
		Total	**648,820**
Dam	**3,757,000**	North bank stabilization - Stage 2	**3,757,000**
R5	**1,300,000**	Approach channel	**1,300,000**
R1&R6	**13,000,000**	South bank cofferdam foundation	22,000
		South bank cofferdam removal	71,500
		Upstream cofferdam foundation (south side)	11,000
		North bank stabilization - Stage 2	-
		Approach channel	681,000
		Dam core trench	1,538,600
		Spillway rock	3,569,300
		Power intakes and penstocks	1,516,340
		Powerhouse	5,467,560
		Switchyard building foundation	21,560
		Railhead	176,000
		Total	**13,074,860**
		Overall Total	**40,612,565**

The revised excavation for the north bank stabilization reduces size of laydown Areas L2 and L10 as follows:

- Area L2 previous area 147,200 m^2, revised area 109,200 m^2; and
- Area L10 previous area 81,900 m^2, revised area 41,400 m^2.

As these areas are on top of a large cut, no stockpiling would be permitted in these areas.

The most significant change was the extension of Area L3 to the east, increasing the capacity from 661,000 m^3 to 15.2 million m^3.

The west side of Area L5 was extended to increase its capacity by one million m^3.

The west side of the Area R5 was modified to accommodate the downstream migration of the mouth of the Moberly River. This reduced the capacity by 900,000 m^3 to 1.3 million m^3.

9.3 Key Findings and Next Steps

Locations have been confirmed for the relocation of surplus excavated materials, including the additional material from the north bank excavation.

Next steps would include updating the distribution of materials into the indentified relocation areas based on the optimization studies, to reduce the total haul requirements.

10. POWERHOUSE ACCESS ROAD AND BRIDGE

10.1 Scope

As part of the Site C Project, access would be required from Fort St. John to the south bank of the Peace River for the construction of the spillway, power intakes, penstocks and powerhouse. After construction, this access would be used for operation and maintenance of the facility. The powerhouse access road would connect the existing Road 269 in the south of Fort St. John to the south bank via a bridge located about 5 km downstream of the dam and would connect to the Septimus siding on the rail line, where much of the heavy equipment and materials for construction of the Project would be delivered (Figure 8).

The purpose of this work was to update the 1981 road alignment and 1989 bridge design to current MoT standards.

10.2 Summary

10.2.1 Road Alignment

From Road 269, the powerhouse access road would go east and then down the north side of the valley partly following the alignment of the existing Old Fort Road to the valley bottom and then turn south to connect with the bridge. The distance from the connection to Road 269 to the north end of the new bridge would be approximately 5 km. From the south end of the bridge, the road would follow the north bank of a large island before crossing onto the south bank for access to the powerhouse. The distance from the south end of the bridge to the powerhouse would be approximately 4 km. The connection to Septimus siding would go from the powerhouse up the side of the valley for a distance of about 4 km.

The following design criteria were used for the road:

- MoT classification - low volume road;
- design speed 70 km/h;
- two 3.5 m wide lanes with 0.5 m wide shoulders;
- 190 minimum curve radius – actual minimum 200 m;
- 110 m minimum stopping sight distance; and
- 7% maximum grade – actual maximum 5.9%.

Cut and fill slopes of 3H:1V and 2H:1V, respectively were assumed to determine the limits of the rights of way (ROW). No specific geotechnical site investigations were done during this stage.

10.2.2 Bridge

The following design criteria were used for the bridge:

- the bridge cross-section would consist of two 3.5 m wide lanes with 0.5 m shoulders and a 1.5 m sidewalk plus cast-in-place concrete parapets for a total width of 10.4 m;
- the bridge design will meet the requirements of the Navigable Waterways Protection Act and have a minimum vertical clearance of 2.0 m above the 200-year water level;
- railings will consist of a standard combination bicycle/pedestrian rail mounted directly onto the bridge parapets; and
- live loads to be the greater of BCL-625 design truck plus lane load or off-highway vehicle load (dump trucks, transport trucks) to be determined for detailed design.

Two major changes were made to the 1989 bridge design:

- the vertical alignment of the road was raised to decrease the amount of cut required for the roadway on the north bank, resulting in a bridge deck elevation of approximately El. 435; and
- the river channel between the small island and the north bank would be spanned to preserve aquatic habitat.

These two changes result in a bridge structure that is substantially longer than the 1989 design, which had four spans totaling 280 m. The proposed new bridge would be a six span steel plate girder structure approximately 520 m long using “delta-frames”, which are large “V”-shaped columns at the intermediate piers that reduce the length of the main spans. A similar type of bridge structure is used for the new Simon Fraser River Bridge in Prince George.

In addition to the changes discussed above, the 1989 design was intended for access to the new powerhouse for BC Hydro personnel only; the bridge would not be open to the public. The current design allows for the future possibility that the bridge can be used by the public; therefore, the bridge has been designed to the MoT criteria for a low volume road.

The bridge has been designed with sufficient clearance to pass both the peak 200-year return period flood and the Probable Maximum Flood (PMF), however, some substructure elements such as the bottom of the delta-frame V-shaped columns may be submerged during a PMF. Although it is standard MoT practice for scour protection to be provided to the 200-year return flood level only, for the purposes of the conceptual design a full rip-rap blanket to an elevation approximately 2 m above the PMF level was assumed to prevent erosion of the bridge end-fills. The final size and distribution of the rip-rap protection would be determined in detailed design, but at this time a conservative Class 500 kg rip-rap with a blanket thickness of 2 m was assumed. The slope of the rip-rap protection is 1.5H:1V at the bridge abutments.

The preliminary foundation design was based on the limited geotechnical information available from the 1989 design, i.e. the soil conditions for the foundations are predominately silts and clays overlying shale bedrock. The competent shale bedrock was assumed to be within 2 to 3 m in depth from the surface. Rock-socketed concrete filled steel pipe piles would be used for the bridge abutments, outside piers and middle delta-frame piers.

The construction of the bridge would require the use of a temporary work bridge(s) with removable working platforms in order to construct the cofferdams required for the in-stream piers. The work bridge(s) could be supported on steel pipe piles driven into sound shale. The design of the temporary works would need to consider river levels and flows at the time of construction.

The conceptual design is based on cast-in-place concrete abutments supported by 610 mm diameter rock-socketed piles, however, simple spread footings may be used in detailed design if the results of the geotechnical investigation prove favorable.

The effect of ice loads on the piers was not been studied in detail for the conceptual design.

It is anticipated that the erection of the steel bridge girders would be done by cranes situated on the temporary work bridge.

11. RESERVOIR SHORELINE IMPACTS

11.1 Scope

Creation of the Site C reservoir would flood land and impact land use around the shoreline of the reservoir. The reservoir shoreline impacts would include: stability and erosion of slopes; changes to groundwater levels; and the potential for landslide generated waves.

In 1978, studies were conducted to establish a "residential safeline", or simply "safeline" on private land around the proposed Site C reservoir. This safeline was defined as *"a conservatively located line beyond which the security of residents and their belongings can be reasonably assured"*. The safeline was intended for residential and associated land use, and addressed safety from both:

- sudden landslides; and
- progressive, and relatively slow, shoreline erosion, regression and the development of beaches.

The safeline did not address existing natural landslides that would be unaffected by the reservoir, however a "high bank" safeline was identified around a portion of the reservoir that incorporated existing natural landslides. The safeline shown in the exhibits for the 1982 British Columbia Utilities Commission (BCUC) hearings incorporated existing natural landslides that would be unaffected by the reservoir.

The scope of work originally included:

- collecting and reviewing all available studies and data regarding the reservoir slope stability;
- compiling existing soil and bedrock strength data;
- performing surficial geological mapping along both banks of the river using airphoto interpretation combined with inspection of all reservoir slopes including recent slides; and
- undertaking a stability assessment for both static and seismic conditions using updated data from piezometers and inclinometers installed during the 1977 drilling program to confirm whether potential failure surfaces would be within the safeline.

The scope of work was subsequently revised to include a review of the current state of practice for determining reservoir shoreline impacts, and based on that review to prepare separate lines to identify each physical impact of the reservoir on the shoreline.

11.2 Summary

11.2.1 General

BC Hydro Report No H2293, *Geotechnical Guidelines for Determining Slope Stability and Groundwater Impacts on Reservoir Shorelines for Land Use Purposes, Provisional Issue* (BC Hydro 1993) proposed changing from a "safeline" concept to a "reservoir impact line" concept. The term "reservoir impact line" means a "*boundary beyond which lands adjacent to a reservoir are not expected to be affected by the creation, or normal operation, of the reservoir*".

The use of reservoir impact lines has two main benefits:

- impact lines are concerned only with technical issues that determine the landward extent of the effects of reservoir shoreline processes; and
- impact lines provide a standardized method of analysis for the entire reservoir shoreline, regardless of ownership.

After reservoir impact lines have been established, guidelines can be established to tailor uses of the land that lies between the reservoir and the impact lines.

International Commission on Large Dams (ICOLD)[9] Bulletin 124, *Reservoir Landslides: Investigation and Management, Guidelines and Case Histories*, (ICOLD 2002), adopted the reservoir impact line methodology verbatim from BC Hydro (1993).

Based on a literature review, it was considered that the use of reservoir impact lines in accordance with ICOLD (2002) represents the current state of practice, and this approach was recommended for determining the reservoir shoreline impacts of the Site C Project.

In accordance with ICOLD (2002) the reservoir shoreline impacts are characterized by establishing the following reservoir impact lines:

- Flooding Impact Line;
- Stability Impact Line;

[9] ICOLD is a non-governmental international organization that provides a forum for the exchange of knowledge and experience in dam engineering, with the objective of ensuring that dams are built safely, efficiently, economically, and without detrimental effects on the environment.

- Erosion Impact Line;
- Groundwater Impact Line; and
- Landslide Generated Wave Impact Line.

Establishing the five separate reservoir impact lines around the shoreline enables stakeholders, including the public, First Nations, and regulatory agencies, to better understand the different physical processes involved and the resulting reservoir shoreline impacts.

The purpose of establishing reservoir impact lines is to determine the effects of the reservoir on the adjacent land and land use. The effect of the reservoir is the difference between the existing natural conditions and the anticipated conditions after reservoir filling (i.e. post-project minus pre-project). Therefore, where applicable, a natural stability impact line and a natural erosion impact line will be established using the same methodology and criteria used to establish the reservoir impact lines.

The methods used to establish reservoir impacts have inherent uncertainties that must be understood and taken into account when both establishing and using reservoir impact lines. In addition, some of the physical processes that cause the reservoir shoreline impacts are time dependent. For example, progressive shoreline erosion, regression and beach development can continue for a very long time, if not indefinitely. The uncertainties in predicting both the extent and rate of the reservoir shoreline impacts lead to the proposal to adopt an observational approach for periodically reviewing and updating some of the reservoir impact lines if the reservoir is flooded.

The methodologies and criteria used are considered suitable for establishing reservoir impact lines at a regional level only. Areas found to be especially prone to flooding, landslides, erosion, groundwater, and landslide-generated waves, and where the consequences are considered potentially high, will require further investigation and analysis so that the reservoir impact lines in those areas can be defined with more confidence during future stages of the project should it proceed.

If the project proceeds to future stages, it is recommended that the reservoir impact lines be reviewed, and possibly modified in light of any new information, methodologies, or criteria available at that time, especially just before reservoir filling, during reservoir filling and periodically thereafter.

Mr. Doug VanDine, P.Eng., P.Geo., is the independent reviewer for the reservoir impact lines.

11.2.2 Flooding Impact Line

The Flooding Impact Line is the boundary beyond which the land adjacent to the reservoir is not expected to be flooded as a result of the creation or normal operation of the reservoir.

The Flooding Impact Line will be based on the maximum normal reservoir level with suitable allowances for floods and winds. Flood allowances will include any surcharge above the maximum normal reservoir level due to the passage of floods, and any gradient on the water surface due to flows through the reservoir, including the tributary arms. Wind allowances will include seiche[10], waves, and wave runup[11].

Based on a review of guidelines the following criteria were recommended for establishing the Flooding Impact Line for the proposed Site C reservoir:

- the 1000-year flood;
- the maximum normal reservoir level of El. 461.8, since the spillway will have a capacity greater than the 1000-year flood with the reservoir at this level; and
- the 200-year return period wind.

The Flooding Impact Line was established as follows:

- the water surface profile along the Peace River was calculated using the 1000-year flood discharge from Peace Canyon Dam and the water surface profiles were calculated along each tributary arm using the estimated 1000-year flood for that tributary;
- wind allowances were calculated using the 200-year maximum hourly wind speed values for Fort St. John airport provided by Environment Canada;
- seiche was calculated using a standard method assuming that the 200-year wind blows along the reservoir long enough to cause a seiche; and
- wave heights and wave runups were calculated using a standard method assuming that the 200-year wind blows along the reservoir long enough to develop the full wave heights.

[10] Seiche is the increase in water level along a reservoir due to wind blowing along the reservoir. The water level downwind is higher than the water level upwind.
[11] Wave runup is the maximum vertical extent of wave uprush on a beach or structure above the still water level.

The methodology described above provided elevations which were added to provide the elevation of the Flooding Impact Line at discrete locations along the reservoir, i.e. Flooding Impact Line elevations = water surface elevation for the 1000-year flood (from the water surface profile) + seiche + wave runup. The Flooding Impact Line was then plotted on topographic mapping produced during Stage 2 from aerial photography and aerial LiDAR survey using the calculated elevations.

Note that the only available wind data for the prediction of seiche, wave heights and wave runup are from the Fort St. John airport, which may not be representative of the winds that would exist in the valley after the reservoir has been filled, therefore during Stage 2 five wind stations were installed in the valley upstream of Site C to collect wind data near the proposed maximum normal reservoir level to provide more representative data for future stages of the project if it proceeds.

11.2.3 Stability Impact Line

The Stability Impact Line is the boundary beyond which the land adjacent to the reservoir is not expected to be affected by landslides resulting from the creation, or normal operation, of the reservoir.

The banks of the Peace River and its tributaries are prone to naturally occurring slumping and landslides, and therefore a natural stability line will be established to identify existing naturally unstable areas. While establishing the Stability Impact Line, existing naturally unstable areas, which will not be affected by the reservoir, will be noted, but such areas will not necessarily be included on the reservoir side of the Stability Impact Line. Existing naturally unstable areas that will be affected by the reservoir will be incorporated into the Stability Impact Line.

The Stability Impact Line is being established as follows:

- the bedrock, surficial materials, and geomorphic processes around have been mapped around the proposed reservoir shoreline using air photos and observations from a river based reconnaissance, supplemented by a review of existing geological mapping, geological sections, drill hole data, laboratory test data;
- existing natural landslides and unstable areas have been identified and characterized using air photos supplemented by information contained in existing reports;
- the reservoir shoreline has been divided into a number of representative reaches with similar topography; geology;

material properties; geomorphic[12] processes; and groundwater conditions, and prepare a geological model for each representative reach;

- the furthest distance from the reservoir shoreline where a landslide backscarp could extend (the break line) is being estimated for each representative reach, based on observations of existing natural landslides in similar terrain and geology; and
- the results of the above four steps are being used to establish the natural stability line and the Stability Impact Line as the lines that envelope all break lines for potential slope failures, and will include an assessment of whether the frequency and magnitude of landslides will change as a result of filling of the reservoir.

Considerable judgment is required for establishing both the natural stability line and the Stability Impact Line.

The Stability Impact Line will be based on the existing topography, which will change as a result of shoreline erosion, regression and beach development. To the extent possible, the effects of these shoreline changes will be taken into account in developing the Stability Impact Line.

As described above, a geological model will be prepared for each representative reach. At the maximum normal reservoir level, the proposed Site C reservoir would have a shoreline approximately 280 km long. Dividing a shoreline of this length into representative reaches inevitably involves some simplification and approximation, and therefore introduces uncertainty.

11.2.4 Erosion Impact Line

The Erosion Impact Line is the boundary beyond which the land adjacent to the reservoir is not expected to be affected by progressive shoreline erosion, regression and beach development as a result of the creation, or normal operation, of the reservoir.

Progressive shoreline erosion, regression and beach development does not typically have an immediate effect on property or threaten human life; however, sudden, relatively small, localized slope failures due to undercutting of the bank can be expected. Shoreline erosion is an ongoing process so the Erosion Impact Line should be predicted for

[12] Geomorphology is the scientific study of landforms and the processes that shape them. Geomorphologists seek to understand why landscapes look the way they do, understand landform history and dynamics, and predict future changes.

several time intervals to demonstrate the expected progressive nature of shoreline erosion. In some cases it may be possible to use protective works to control or limit shoreline erosion and regression.

The banks of the Peace River and its tributaries are naturally being eroded by river flows. A natural erosion line will therefore be established along the reaches where natural erosion is significant, particularly at the upstream end of the reservoir and on the tributaries where the reservoir depth will be shallow. While establishing the Erosion Impact Line, existing natural erosion that will not be affected by the reservoir will be noted but such areas will not necessarily be included on the reservoir side of the Erosion Impact Line. Areas of existing natural erosion that will be affected by the reservoir will be incorporated into the Erosion Impact Line.

Erosion of the reservoir shoreline would be caused by wind generated waves and long shore currents. Since this is a continual process, average annual wind data will be used to estimate the progressive shoreline erosion, regression and beach development. Sensitivity analyses will be done to determine the effects of unusual wind storms.

After reservoir filling, river bank erosion and beach development due to undercutting and the movement of material downstream by the Peace River and its tributaries will cease; but long-term progressive shoreline erosion, regression and beach development will begin within the normal operating range of the reservoir. Based on a literature review, the method proposed by Lynden Penner[13] is considered to represent the current state of practice for predicting progressive shoreline erosion, slope regression and beach development around reservoirs, and will be used to establish the Erosion Impact Line. Therefore, Lynden Penner was engaged as a subconsultant by KCBL to estimate the Erosion Impact Line.

According to Penner, shoreline erosion is caused by several interacting processes: wave erosion of the bluff toe; beach flattening and down-cutting by current and wave action; mass-wasting of the shoreline bluff from weathering and periodic failures of the bluff; removal of failed bluff slope debris by wave action; and offshore and alongshore transport of eroded sediment. For modeling purposes shoreline erosion is visualized in three stages of evolution:

[13] Lynden Penner, M.Sc., P.Eng., P.Geo., has specialized in airphoto and satellite remote sensing since 1986, and is a sessional lecturer in terrain analysis for the Faculty of Environmental Engineering, University of Regina. He authored the 1993 report "Shore Erosion and Slumping on Western Canadian Lakes and Reservoirs – a Methodology for Estimating Future Bank Recession Rates" for Environment Canada.

- waves wash and erode previously un-flooded slopes around a new reservoir, forming a narrow beach slope backed by an adjoining bluff face; erosion of the bluff toe and bluff mass wasting dominate the shoreline erosion process;
- the beach slope widens and the bluff face gets higher, the height of the evolving bluff depends on the topography of the pre-erosion shoreline zone, beach erosion becomes increasingly important, although erosion of the bluff toe and bluff mass wasting are still significant; and
- the beach slope widens and flattens and beach down-cutting begins to dominate the erosion process, bluff mass wasting continues, but is largely independent of wave erosion effects except for removal of bluff debris by wave action.

Penner developed a numerical model for predicting erosion rates on lakes and reservoirs by: synthesis of data in published papers and reports; measurements of historical erosion rates; and field studies of three reservoirs in the Prairies. The model predicts volumetric rates of erosion (m^3/yr per metre of shoreline) based on a linear relationship between the erodibility coefficient of the geological material being eroded and the annual wave energy. It is important to note that this method estimates the volume of material eroded per year, therefore the landward progression of a high bluff will be slower than a low bluff, as the higher bluff will produce more material that has to be transported away.

The impact of storms on long term erosion rates is accounted for by incorporating the full range of historical wind velocities and durations into the wave energy calculations. However, short term erosion rates due to storms, although temporary, may exceed long-term average rates predicted by the model. Similarly, short term erosion rates between storms may be much lower than long-term average rates.

The Erosion Impact Line is being established using the geology of the same representative reaches of reservoir shoreline used to establish the Stability Impact Line. Erodibility coefficients are being based on an evaluation of existing river bank erosion and slope recession rates measured from ortho-rectified historical air photos; experience on other reservoirs and lakes, including Williston Reservoir and Moberly Lake; and on judgment. The beach and bluff slope angles are based on observations from natural lakes and reservoirs. The Erosion Impact Line will be predicted for several time intervals up to 100 years, to demonstrate the expected progressive nature of shoreline erosion.

The Fort St. John wind data is being used for establishing the Erosion Impact Line until more sufficient wind data has been collected from these wind stations at which time the Erosion Impact Line should be updated. The lack of in-valley wind data results in uncertainty in the Erosion Impact Line established using the available data.

There are different methods that can be used to reduce the rate of or halt progressive shoreline erosion, regression and beach formation. As part of the Stage 2 studies, the Erosion Impact Line is not considering such methods, other than at Hudson's Hope.

11.2.5 Groundwater Impact Line

The Groundwater Impact line is the boundary beyond which groundwater levels in the land adjacent to the reservoir are not expected to be affected by the creation, or normal operation, of the reservoir.

The Groundwater Impact Line identifies areas where groundwater levels will be increased by the reservoir so that potential effects on water well supply, habitable buildings (e.g. basement flooding and septic systems), natural vegetation and agriculture can be indentified in future stages if the project proceeds.

The Groundwater Impact Line does not address effects of raised groundwater levels on slope stability, which will be included in the Stability Impact Line.

Groundwater recharge from the surface will be determined from the average annual precipitation at Fort St. John.

Regional groundwater flow in the overburden and bedrock adjacent to the Peace River is three dimensional in nature has been modeled using FEFLOW®, a three dimensional finite element program capable of modeling fracture flow, time-varying boundary conditions and material parameters, saturated and unsaturated flow. The program was used to prepare a regional numerical model which was then used to estimate the steady-state, pre-project, groundwater flow, and the lateral extent of groundwater effects of reservoir filling.

The data used to develop the numerical model included:

- surface topography derived from the project digital elevation model, developed from the LiDAR survey;
- geologic data used for the Stability Impact Lines and the Erosion Impact Lines;
- GIS information on land uses; and
- information available through the BC Ministry of Environment, Groundwater Section.

The ground surface and the surface of each regionally significant stratum (e.g. glaciolacustrine deposits, basal gravel, and shale bedrock) were modeled. These surfaces were based on the geological mapping of the river valley, supplemented by other available geological data (e.g. water well logs).

The model was calibrated to pre-project conditions and used to simulate the results of groundwater changes due to filling of the reservoir. Piezometric maps were produced to indicate the extent of the anticipated regional groundwater impacts.

Key sensitivities in numerical model parameters were identified and recommendations made for future work and site groundwater investigations to reduce uncertainty in the numerical model and the parameters used in the model.

11.2.6 Landslide Generated Wave Impact Line

The Landslide Generated Wave Impact Line is the boundary beyond which the land adjacent to the reservoir is not expected to be affected by waves generated by a landslide into the reservoir.

It is proposed that landslides considered for the Landslide Generated Wave Impact Line will include both:

- those that result from the creation, and normal operation of, the reservoir; and
- existing naturally unstable areas, that may not be affected by the reservoir, but whose effects could be increased by the presence of the reservoir.

In the latter case, it is proposed that the Landslide Generated Wave Impact Line will include only the area affected by the Landslide Generated wave, but not the slide area itself.

In areas where large slides have occurred in the past and where potential large slides have been identified, slide volumes and velocities identified in previous studies have been used to estimate the waves that could be generated by those landslides.

A hypothetical slide of 75,000 m^3 with a velocity of 3 m/s has been used to estimate the landslide generated waves in all other parts of the shoreline.

The Landslide Generated Wave Impact Line for the reservoir shoreline in the vicinity of the proposed Site C dam, at Bear Flat and at the Halfway River has been based on the landslide-generated wave data from 1981 and 1983 physical hydraulic model tests.

The Landslide Generated Wave Impact Line for the remainder of the reservoir shoreline has been based on published empirical methods for estimating landslide-generated wave heights. These methods require the landslide and reservoir characteristics to be simplified, and provide order of magnitude estimates of the wave heights generated and propagated by the simplified landslide.

11.3 Key Findings and Next Steps

The key finding of the Stage 2 assessment is that current international practice for characterizing reservoir shoreline impacts is to establish the following reservoir impact lines:

- Flooding Impact Line;
- Stability Impact Line;
- Erosion Impact Line;
- Groundwater Impact Line; and
- Landslide Generated Wave Impact Line.

In Stage 2, the methodology and criteria for establishing each of the above impact lines were established. Draft impact lines were developed that need to be ground truthed and validated, therefore next steps include:

- updating the estimates of the Flooding Impact Line Erosion Impact Line with wind data measured at the five in valley weather stations installed during Stage 2;
- undertaking more detailed investigations of the geology of the valley slopes to improve the estimates of the Stability Impact Line and the Erosion Impact Line; and
- consider the use of numerical modeling to improve the estimates of the Landslide Generated Wave Impact Line.

12. HIGHWAY 29 RELOCATIONS

12.1 Scope

Creation of the Site C reservoir would flood four sections of Highway 29 between Hudson Hope and Fort St. John. These four sections are shown on Figure 9. The realignment of approximately 25 kilometres of Highway 29 would be required in the Cache Creek, Halfway River, Farrell Creek and Lynx Creek areas. Studies in 1978 identified several alternate alignments at Cache Creek, Halfway River and Lynx Creek, and one at Farrell Creek.

The purpose of this work was to update the 1978 alignments to current MoT standards.

The scope of work included:

- reviewing 1978 alignments;
- collecting and compiling available mapping including LiDAR data;
- consulting with MoT to determine applicable highway design standards and local practice for cuts and fills;
- updating highway alignments using highway design software;
- determining the rights of way for each alignment;
- submitting the updated alignments to MoT for review and comment;
- preparing layouts for river crossings; and
- consulting with stakeholders including affected property owners.

Highway alignments were done by Urban Systems as a subconsultant to KCBL.

12.2 Summary

12.2.1 Road Alignments

The potential for wholesale realignment of the highway along the plateau, involving 60 km of new road construction, was discounted in the earlier study on the basis of high cost and impacts. This possibility was revisited again at this stage of design during the initial review process but was not pursued further as the rationale for the previous decision was still considered valid.

The following MoT design criteria were used for the roads:

- design speed 70 to 100 km/h, the preferred design speed is 100 km/h with lower speed design allowed in difficult terrain;
- two 3.6 m wide lanes with 2.0 m wide shoulders;
- 340 m minimum curve radius; and
- 7% maximum grade.

The following alignment options were updated:

- four options for the 7.7 km section at Lynx Creek;
- one option for the 2.5 km section at Farrell Creek;
- three options for the 4 km section at the Halfway River; and
- two options for the 10 km section at Bear Flat.

There have been numerous slope stability problems along some sections of Highway 29. The alignments were refined to try and avoid impact to known slide areas or areas identified during site visits as having unstable slopes. However, the requirements for stable slopes in both cut and fill will have to be investigated further during subsequent stages of the project, if it proceeds.

Based on discussions with MoT, 3H:1V cut slopes and 2H:1V fill slopes were assumed to determine the limits of the rights of way (ROW). No specific geotechnical site investigations were done during this stage. Approximate ROW boundaries were set using a 10 m offset from the calculated cut and fill toe lines which are expected to cover the required ROW for the alignments as currently shown. Due to the limits of the survey data and the design carried out at this stage, these ROW lines should not be used for anything other than a rough estimate of the actual property requirements.

Design of local road connections to Highway 29 realignments were not carried out during this stage of the highway design. There are however, no topographic constraints that have been identified to indicate local road connections to the realigned highway will be a concern.

Storm culverts along the road would be required for all realigned sections. Storm design was not carried out and quantities for storm culverts were based on cross culverts every 300 m based on MoT design guidelines.

BC Hydro distribution lines parallel the highway in many areas and these lines would need to be relocated to the new alignments.

No underground utility impacts were identified.

12.2.2 River Crossings

The following four bridges would have to be replaced as part of the realignment of Highway 29:

- Lynx Creek Bridge No. 2327;
- Farrell Creek Bridge No. 2184;
- Halfway River Bridge No. 1042; and
- Cache Creek Bridge No. 1025.

The following design criteria were used for the replacement bridges:

- the bridge cross-section would consist of two 3.6 m wide lanes with 1.5 m shoulders plus cast-in-place concrete parapets for a total width of 11.0 m;
- the bridge design will meet the requirements of the *Navigable Waterways Protection Act* and have a minimum vertical clearance of 2.0 m above the 200-year water level;
- railings will consist of a standard combination bicycle/pedestrian rail mounted directly onto the bridge parapets; and
- live loads BCL-625 design truck plus lane load.

Several possible river crossing types were examined as conceptual design alternatives. The close proximity of large volumes of granular material results in the following two most viable crossing types:

- a short bridge in combination with a large fill (causeway) constructed from granular fill; and
- a long bridge with no causeway.

The bridges have been sized for sufficient clearances and passage to carry the peak flood flows at the valley floor prior to reservoir filling. A full rip-rap blanket is assumed to be required to prevent erosion along the causeways and at the bridge end-fills. The final size and distribution of the rip-rap protection would be determined during detailed design, but at this stage a conservative Class 500 kg rip-rap with a blanket thickness of 2 m was assumed. The slope of the rip-rap protection varies from 1.5:1 at the bridge abutments to 2:1 on the causeways.

Preliminary foundation design was based on the limited geotechnical information (borehole log data) available for the existing bridges. The soil conditions for the foundations at the existing crossings are predominately silts and clays overlying shale bedrock. The competent shale bedrock is within 5 to 6 m in depth from the surface at the Lynx and Farrell Creek crossings and is approximately 2 to 3 m from the surface at the Halfway River and Cache Creek crossings. Rock-socketed concrete filled steel pipe piles are a common foundation solution for bridge abutments and piers in the region.

The conceptual designs are based on cast-in-place concrete abutments supported by 610 mm diameter rock-socketed piles; however simple spread footings may be used in detailed design if the results of the geotechnical investigation prove favorable. As previously discussed, the bridge end-fills will have rip-rap protection against scour regardless of what type of foundations are used.

The effect of ice loads on the piers was not taken into account for the conceptual design.

The steel bridge girders could be erected in pieces using mobile cranes in the dry or from temporary working platforms near the piers in the water on the Halfway River crossing, thereby allowing the use of different depth girders on shorter spans. However, for the conceptual design is based on launching constant-depth steel girders from one or both abutments.

The following alternative crossings of Lynx Creek were considered:

- a 90 m long two span bridge with a 328 m long causeway; and
- a continuous 390 m long six-span bridge with no causeway.

Lynx Creek is on the east side of the valley floor far enough away for the abutments of the short bridge to be constructed in the dry, with minimal environmental disturbance. The substructures for the long bridge alternative can also be constructed in the dry.

The following alternative crossings of Farrell Creek were considered:

- a 140 m long three span bridge with a 144 m long causeway; and
- a continuous 265 m long six-span bridge with no causeway.

Farrell Creek is on the east side of the valley floor far enough away for the abutments of the short bridge to be constructed in the dry, with minimal environmental disturbance. The substructures for the long bridge alternative can also be constructed in the dry.

The Halfway River is a wide, swift flowing river which can often carry a large amount of sediment and debris during floods. The following alternative crossings of the Halfway River were considered:

- a 320 m long five span bridge with a 637 m long causeway; and
- a 938 m long thirteen span bridge with no causeway.

The construction of the Halfway River crossing will be more difficult than the other three crossings. The waterway is wide and flows quickly during some periods of the year. A temporary work bridge may be required to cross parts of the channel with working platforms for construction of at least two of the piers. The piers can be constructed in the winter when the flow would be lowest and much of the channel would be dry. Scheduling installation of the substructures for the better months of the year would reduce the cost of the work bridge and driving platforms. The piers could then likely be constructed without cofferdams.

Cache Creek is a wide, meandering creek which can carry sediment and debris during floods. The original wood truss bridge was replaced in 2008 by a 40 m long single-span bridge with steel girders, pre-cast concrete deck panels and an asphalt wearing surface. The recently constructed bridge is located on the existing alignment, several hundred metres downstream of the potential Highway 29 re-alignments.

The following alternative crossings of Cache Creek were considered:

- a 210 m long four span bridge with a 227 m long causeway;
- a 410 m long six span bridge with no causeway; and
- a 210 m long four span bridge with a 227 m long causeway, with the existing 40 m span of the Cache Creek Bridge re-located and used as the first span.

Cache Creek is on the west side of the valley floor and far enough away for the abutments of the short bridge to be constructed in the dry, with minimal environmental disturbance. The substructures for the multiple span bridge could also be constructed in the dry.

12.3 Next Steps

Next steps include:

- preparation of alignments for feasible alternate highway relocations proposed during public and landowner consultation; and
- assessment of the social and environmental aspects of each alternate alignment to enable selection of the preferred alignment of each section.

13. TURBINE ALTERNATIVES

13.1 Scope

The purpose of this task was to determine whether changing to Kaplan turbines would provide economic benefits and/or potentially reduce the mortality of fish entrained in power flows.

The scope included:

- identifying options for Kaplan turbines;
- obtaining information from manufacturers and other sources and recommending the Kaplan turbine capacity for further study;
- providing turbine characteristics for power studies by BC Hydro;
- assessing changes required to the design of the intakes, penstocks and powerhouse;
- preparing a cost estimate; and
- considering the full life cycle including operating costs, maintenance and environmental risks.

Power studies for Kaplan and Francis turbines were performed by BC Hydro.

13.2 Summary

Francis turbines have a peak efficiency 0.5 to 1.5% higher than equivalent capacity Kaplan turbines, whereas Kaplan turbines have a flatter efficiency curve. The generation of a hydroelectric project depends on the number of units, and the range of heads and flows at the site. At projects with few units and a wide range of heads and flows, Kaplan units can generate more than Francis units. Water to wire efficiency data for both turbine types were provided to BC Hydro to calculate the annual generation from the Kaplan turbine and Francis turbine alternatives. The BC Hydro system studies estimated that six Francis turbines at Site C would produce 33 GW.h/year more energy than six Kaplan turbines.

An extensive literature review was carried out on the effects on fish of passage through the two turbine types. Based on this review, fish mortality is not expected to differ significantly between Francis and Kaplan turbines of equal capacity for the head and discharges at Site C.

Due to the greater complexity of Kaplan turbines it is estimated that the duration of scheduled maintenance outages for Kaplan turbines would be twice the duration of the scheduled maintenance outages for Francis turbines and the annual maintenance cost would be double. The longer scheduled maintenance duration would result in longer constraints on the operation of the GMS Shrum and Peace Canyon powerhouses.

Replacing the Francis turbines with Kaplan turbines would increase:

- the direct construction cost by over $45 million in 2002$, based on the 2002 cost estimate (the cost difference would be significantly greater in 2009$); and
- the construction schedule by 90 days.

Francis turbines are recommended for the Site C Project and recommendations are made for the design of the facility to reduce fish mortality.

13.3 Key Findings and Next Steps

The key findings of the Stage 2 assessment of turbine alternatives were that there were no advantages of changing from Francis to Kaplan turbines.

Next steps will incorporate the design changes recommended for the Francis turbines to reduce fish mortality into the project optimization.

14. RESERVOIR DRAWDOWN CAPABILITY

14.1 Scope

The scope of work included:

- summarizing previous studies for the conversion of the diversion tunnels into low level outlets, capable of drawing the Site C reservoir down below the spillway crest level;
- summarizing the capability of the Site C Project as currently configured to lower reservoir levels in an emergency; and
- recommending whether the need for permanent low level outlet capability should be reassessed based on current knowledge.

14.2 Summary

Studies in 1980 evaluated converting the diversion tunnels into permanent low level outlets by adding gated conduits through the concrete plugs used to seal the tunnels. These low level outlets would have had a capacity of approximately 40% of the diversion tunnel capacity. The studies concluded that adequate control over reservoir levels would be provided by:

- a temporary low level outlet installed in one diversion tunnel during reservoir filling; and
- the spillway and powerhouse during operations.

Studies in 1990 evaluated converting the diversion tunnels into permanent high capacity low level outlets by adding regulating gates in the tunnels and using the full capacity of the tunnels. These outlets would be capable of operation during the PMF and therefore the required spillway capacity would be reduced. The studies concluded that the savings in spillway cost would be less than the cost of converting the diversion tunnels into permanent high capacity low level outlets, and that converting the tunnels to low level outlets would have significant schedule impacts.

As currently configured the project has the capability to:

- use two regulating gates installed in the inlet of one diversion tunnel as a temporary low level outlet to control downstream discharges during reservoir filling, to control the rate of reservoir filling, and to lower the reservoir, if necessary, during filling or early in the life of the reservoir; and

- use the spillway and powerhouse to draw the reservoir down to the emergency reservoir level of El. 450 and then operate the powerhouse continually with the reservoir at that level.

A number of factors have changed since the current configuration was selected, particularly:

- there is now a greater understanding of the rebound potential of the foundation, as well as the uncertainties in predicting both the rebound (particularly the effects of discontinuities in the foundation) and the performance of the structures due to rebound;
- the MDE is considerably higher than previously considered; and
- a desire for greater safety by having redundancy of discharge facilities.

An emergency drawdown of the Site C reservoir could be required due to unexpected performance of the approach channel, intakes, spillway, penstocks, powerhouse, earthfill dam or reservoir slopes under normal or extreme loads such as earthquakes. In some circumstances, e.g. after an earthquake or if rebound causes damage, operation of some of the discharge facilities may not be possible or prudent.

A number of guidelines for reservoir evacuation have been published since the 1990 studies. The majority of the guidelines could not be met without low level outlets. Even with both diversion tunnels converted into permanent low level outlets it could be necessary to reduce discharges from Peace Canyon Dam to meet the guidelines for drawing the reservoir down to low levels.

Conversion of the diversion tunnels to permanent low level outlets appears technically feasible; however the required capacity, operational risks, additional costs and impacts on the construction schedule need to be identified.

Redundancy of discharge capability could be important even without the need for reservoir drawdown, for example if the spillway needs repairs or is deemed inoperable for even a short period. The provision of an additional discharge structure independent of the spillway and power intakes would provide redundancy, additional reliability and resilience.

As part of project optimization it is recommended that an assessment of low level outlet capability be undertaken to determine the optimum project arrangement for minimizing risk and cost. This would include a more rigorous comparison between the drawdown capability of the project and the various guidelines. The design discharge capacity of low level outlets would have to consider the system perspective, particularly the operation of the upstream powerplants, and the full life cycle implications such as the need to operate and maintain the outlet on a regular basis so that it can be relied upon in the long term.

14.3 Key Findings and Next Steps

The key findings of the Stage 2 assessment of reservoir drawdown capability were:

- it appears technically feasible to convert the diversion tunnels into permanent low level outlets; and
- a number of factors have changed since the 1981 design was selected and the provision of low level outlets needs to be reconsidered.

Next steps would include identifying the required low level outlet capacity, operational risks, any additional costs and impacts on the construction schedule as part of determining the optimum project arrangement.

15. OPPORTUNITIES FOR RESERVOIR SHORELINE HABITAT CREATION

15.1 Scope

The scope of work was to identify opportunities for habitat creation in and around the reservoir shoreline.

15.2 Summary

GIS software was utilized in conjunction with high-resolution orthophoto images and LiDAR data coverage of the potentially flooded reservoir area between Peace Canyon Dam and Site C to identify potential areas where shoreline habitat could be created of enhanced. Ground contours were identified and highlighted on maps in 1 m increments below the proposed maximum normal reservoir level of El. 461.8 for the entire reach between Peace Canyon Dam and Site C. Areas where water depths ranged from 0 to 3 m were flagged as locations for further evaluation. Potential habitat areas, potential habitat types (e.g. spawning, breeding, and rearing) and species that may benefit from habitat enhancement were identified for each location. Mitigation designs were developed for nine habitat areas.

In-reservoir mitigation options that were considered included:

- construction of gravel berms and deflectors to create back-channel habitat;
- construction of gravel terraces (benches) to increase shallow-water extent and habitat complexity;
- construction of gravel islands to increase in-channel habitat and shallow-water extent;
- excavation of low-lying areas (at or near maximum normal reservoir level) to increase shallow-water extent;
- excavation of ‘pothole lakes’ to increase shallow-water extent and provide additional wetland habitat;
- placement of fill material to increase shallow-water extent;
- placement of appropriate-sized gravels to increase spawning areas;
- placement of large woody debris for habitat complexing; and
- creation of a ‘perched lake’ using gravel berms to isolate a side-channel.

Borrow areas and earthworks would be vegetated with naturally occurring species.

15.3 Key Findings and Next Steps

The study found that diverse opportunities would exist for the creation of complex habitat around the reservoir shoreline.

Next steps would include:

- more detailed assessment of the opportunities as part of the environmental assessment of the project; and
- incorporation of the results of the studies on the impact lines, particularly the erosion impact line, so that the shoreline habitat is sustainable.

16. CONCLUSIONS

The general arrangement of the project was developed in 1978. There have been many changes over the last 30 years that could result in different design choices, in particular:

- the costs of different types of construction have changed relative to each other, meaning that different design choices might now be less costly;
- the original design was governed by static loads, now seismic loads govern;
- there is now more information and better understanding of potential rebound;
- resilience and redundancy are required due to increased seismic design loads and potential rebound; and
- societal and environmental values are different.

In 1978, the design choices would largely have been based on cost and the general arrangement would have been developed to have the least overall cost including interest during construction which is a function of the construction schedule. Construction costs have increased significantly since 1978, however some costs have increased more than others. For example, the increase in costs for earthmoving due to increases in the costs of wages, fuel and equipment ownership, operation and maintenance have been partly offset by the increased productivity of improved, more powerful equipment. This is particularly true for Site C where the large volumes of excavations and fills allow equipment to be used to its full capacity for long periods. On the other hand, the cost increases in the types of construction that are more labour intensive have generally not been offset by increases in productivity, for example formwork and placing concrete reinforcement. In fact, productivity may have decreased as more attention is now rightly paid to safety and environment.

A major design choice made in 1978 was to use 9.35 m diameter steel penstocks rather than tunnels to convey the water to the powerhouse. These penstocks, which have a diameter equal to the height of a three storey building, have to be constructed on site from relatively thin steel plate sections. This requires fit up, welding and quality control with difficult access, particularly on the inclined portion of the penstock, all of which is very labour intensive. There are few contractors with current experience in the erection of such large diameter steel penstocks and few manufacturers of the large diameter couplings required. Recent experience has shown that construction cost of large diameter penstocks has increased significantly.

The increase in the MDE has a significant affect on the stability of the 1981 design of the spillway headworks, the power intakes and penstocks. As described in Section 8.2.2 extensive drainage of the rock on the south bank and reinforcement of the foundation of the spillway headworks would be required to stabilize these structures. The power intakes and spillway headworks impound 31.8 m and 26.8 m of water, respectively. As the operating range of the reservoir is 1.8 m, it may be possible to lower these structures, i.e. raise their foundation level, to reduce the static and dynamic water loads acting on them. This would also reduce the amount of rock excavation which would mitigate rebound.

As described in Section 8.2.1, conceptually rebound of the 1981 Design of the spillway, power intakes and penstocks can be mitigated by adjusting the weight acting on the foundation, by the addition of anchors and by articulating the structures. The design of the anchors has considerable uncertainty due to the range of the swelling characteristics of the rock and the potential effects of discontinuities such as the cross cutting shears. Changing the general arrangement to minimize the depth of excavation at important structures would be a more certain method of addressing rebound, for example, relocating the powerhouse and spillway stilling basin to the valley floor.

An emergency drawdown of the Site C reservoir could be required due to unexpected performance of the approach channel, power intakes, spillway, penstocks, powerhouse, earthfill dam or reservoir slopes due to rebound or an earthquake. Therefore, given the current understanding of seismicity and rebound, a different decision might be made about the option for converting the diversion tunnels into permanent low level outlets.

Based on the above, it is apparent that different design choices might be made today than in 1978, meaning that the current arrangement may not be the optimum balance between risk, cost and environmental considerations.

17. RECOMMENDATIONS

KCBL and SLI recommend that the project be optimized to find the design that best balances risk, cost and environmental considerations, based on the current understanding of the technical issues and current construction costs.

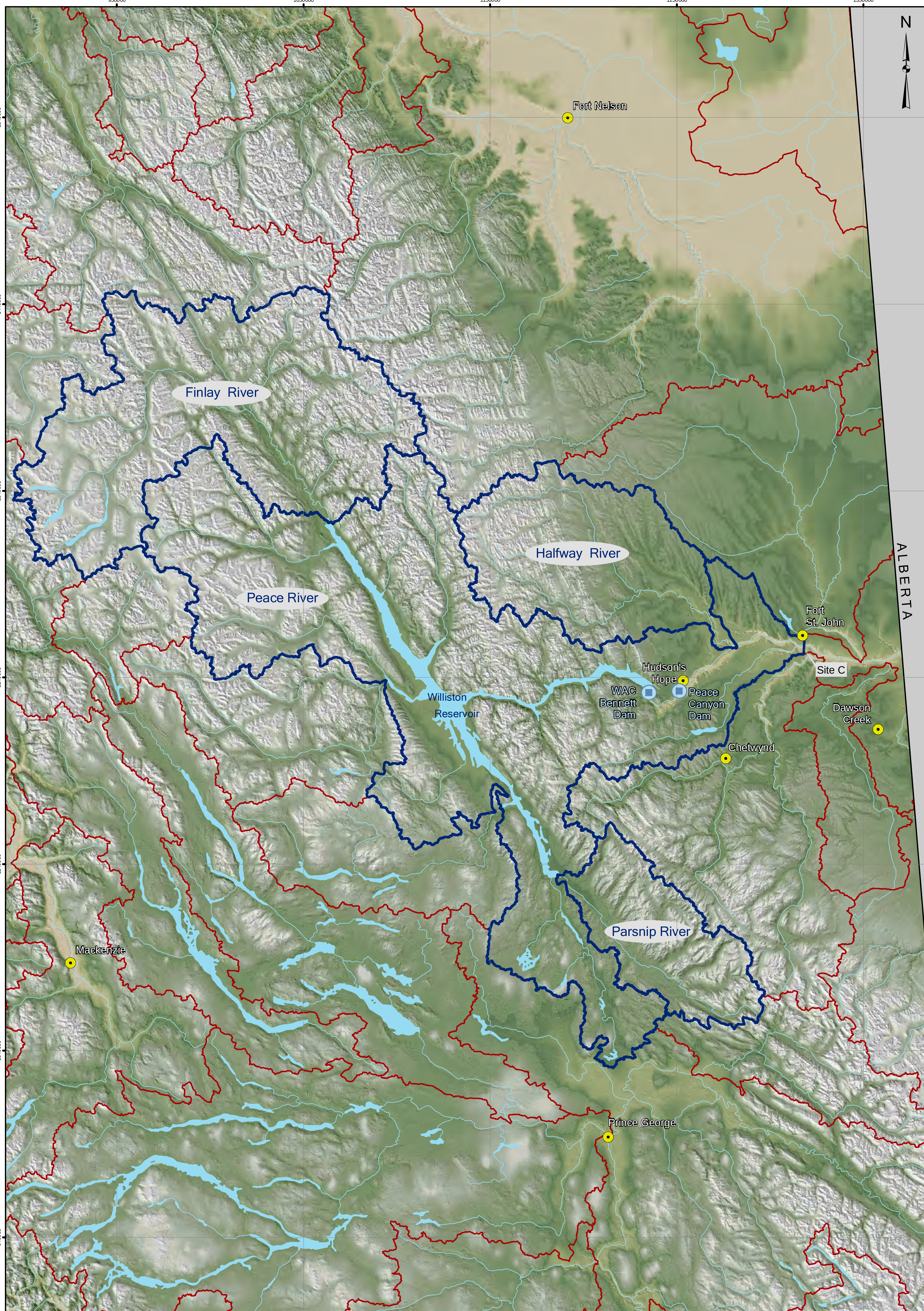
Optimization of the project would refine the design and confirm:

- type of dam
- general arrangement;
- installed capacity;
- number and type of units;
- maximum normal reservoir level;
- river diversion during construction;
- low level outlet requirements;
- power tunnels versus penstocks;
- diameter of power tunnels/penstocks;
- PMF passage; and
- location of power intakes, penstocks and powerhouse

Many of these features are interrelated.

Major inputs into the optimization studies would be the relative costs of alternate arrangements and configurations of the project components and future operating costs and benefits. In addition, where alternates have different environmental and social impacts, for example optimization of the maximum normal reservoir level, a multi criteria decision making assessment would be used to select the optimum.

FIGURES



Legend:

Figure 1

Site C Regional drainage basins

Watershed

Town

Notes:
1. Datum: NAD83
2. Projection: BC ALBERS
3. Data Source: BC Hillshade model courtesy BC Environment, Lands and Parks

Scale: 1:200,000 0 125 km

Klohn Crippen Berger

SNC·LAVALIN

BC hydro

Williston & Site C Drainage Areas

DATE	JULY, 2009	DWG NO	xxxx-xxx-xxxxx	R x

P:\P05032 A02 Site C - 2006 to 2008 Eng Studies\400 Drawings\GIS_data\MXD\watershed\Watersheds_Regional.mxd

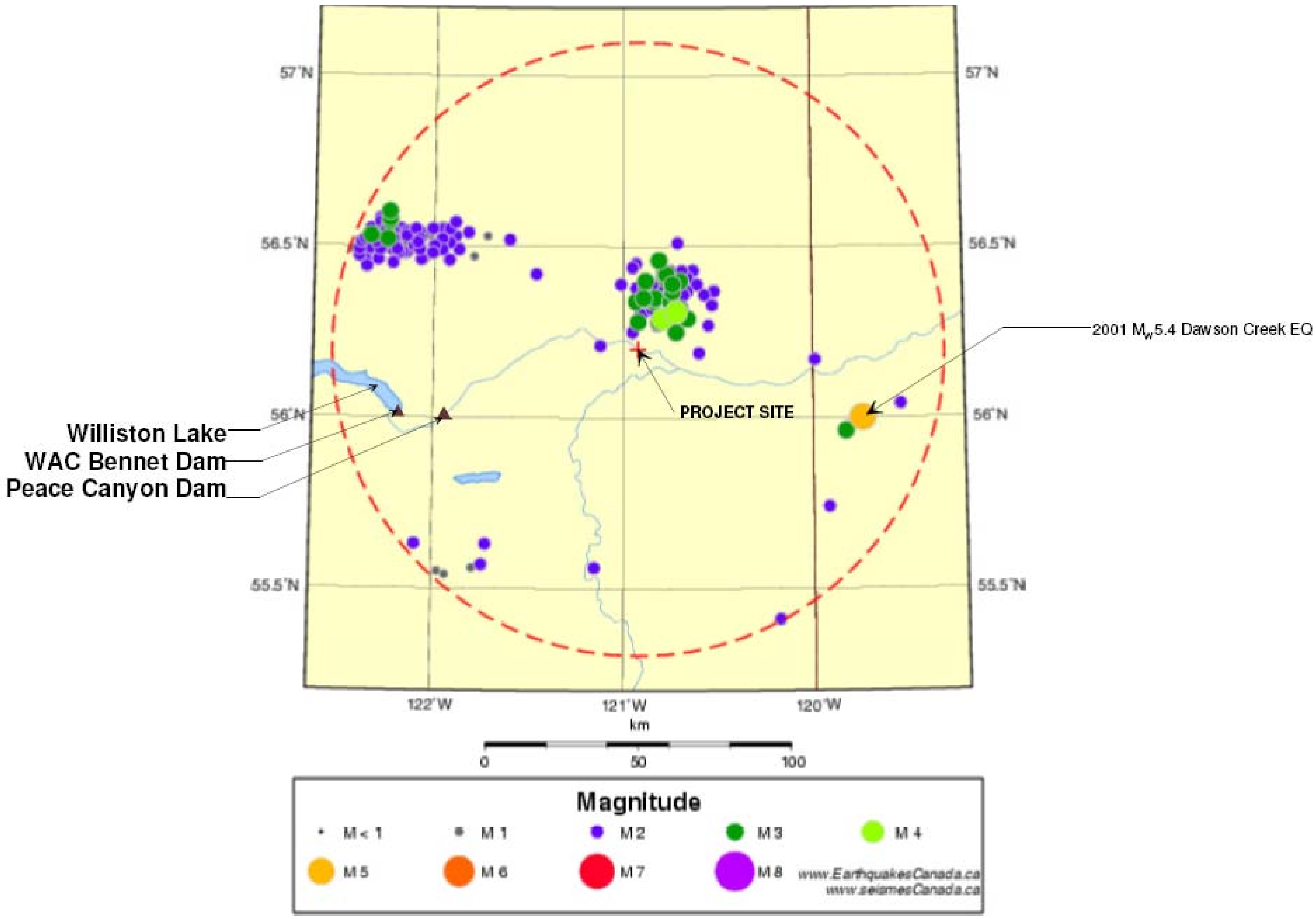


Figure 2
Epicentres of Earthquakes
Within 100km of Site C Since 1985

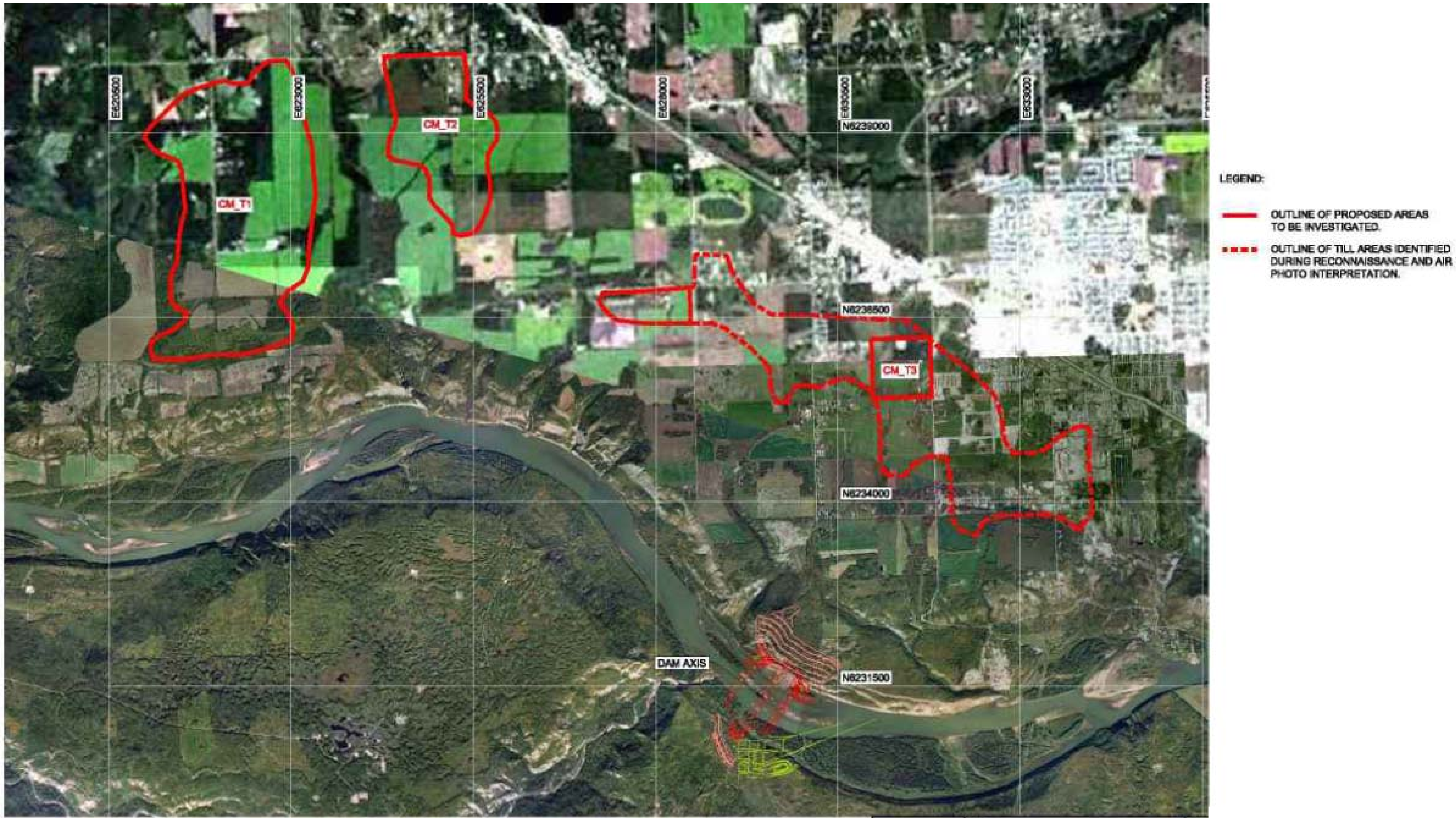


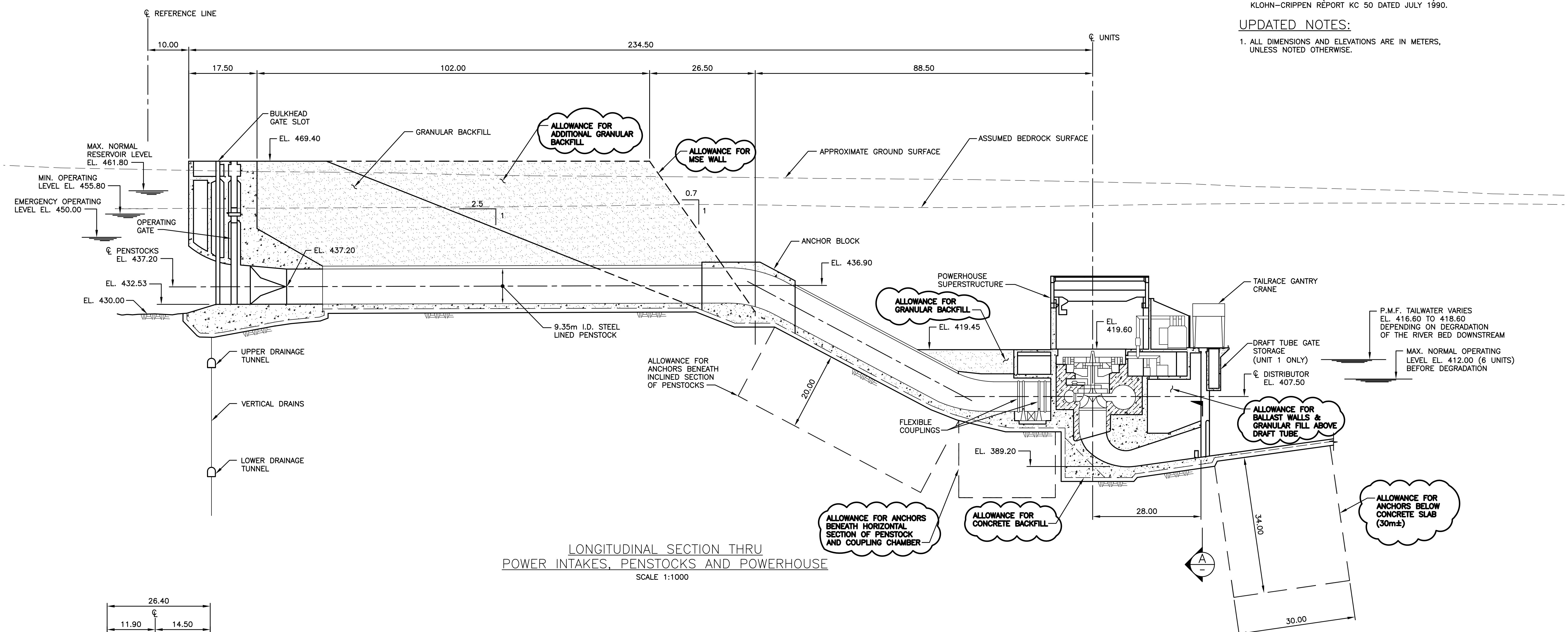
Figure 3
Outlines of Likely Till Areas Identified by
Air Photo Interpretation and Reconnaissance

ORIGINAL NOTES:

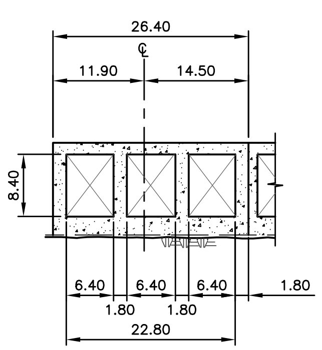
1. BASED ON FIG. 7 (DWG. No. 1016-C14-B2393) KLOHN-CRIPPEN REPORT KC 50 DATED JULY 1990.

UPDATED NOTES:

1. ALL DIMENSIONS AND ELEVATIONS ARE IN METERS, UNLESS NOTED OTHERWISE.



LONGITUDINAL SECTION THRU
POWER INTAKES, PENSTOCKS AND POWERHOUSE
SCALE 1:1000



SECTION A
1:1000

DRAWING LINEWORK AND ORIGINAL NOTES COPIED FROM DWG. 1016-C14-D2676 DATED JAN. 2003

0 25m

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PEACE RIVER SITE C PROJECT

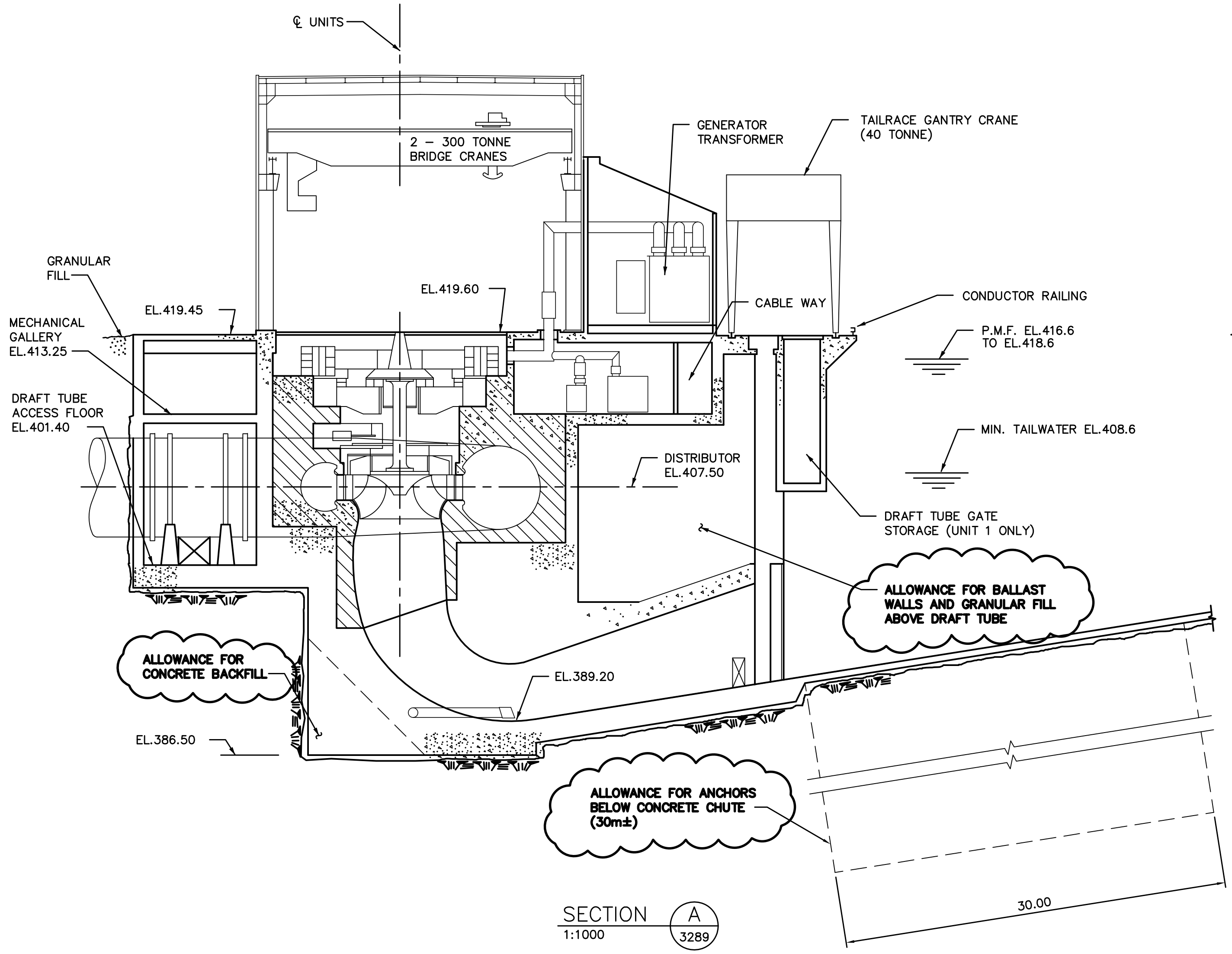
POWER INTAKES, PENSTOCKS AND POWERHOUSE
POSSIBLE MEASURES FOR STABILIZATION
AND REBOUND MITIGATION

DSGN	
INDEP CHK	
DFTG	WN (KCBL)
DFTG CHK	
REPORT NO.	
PROJ. NO.	

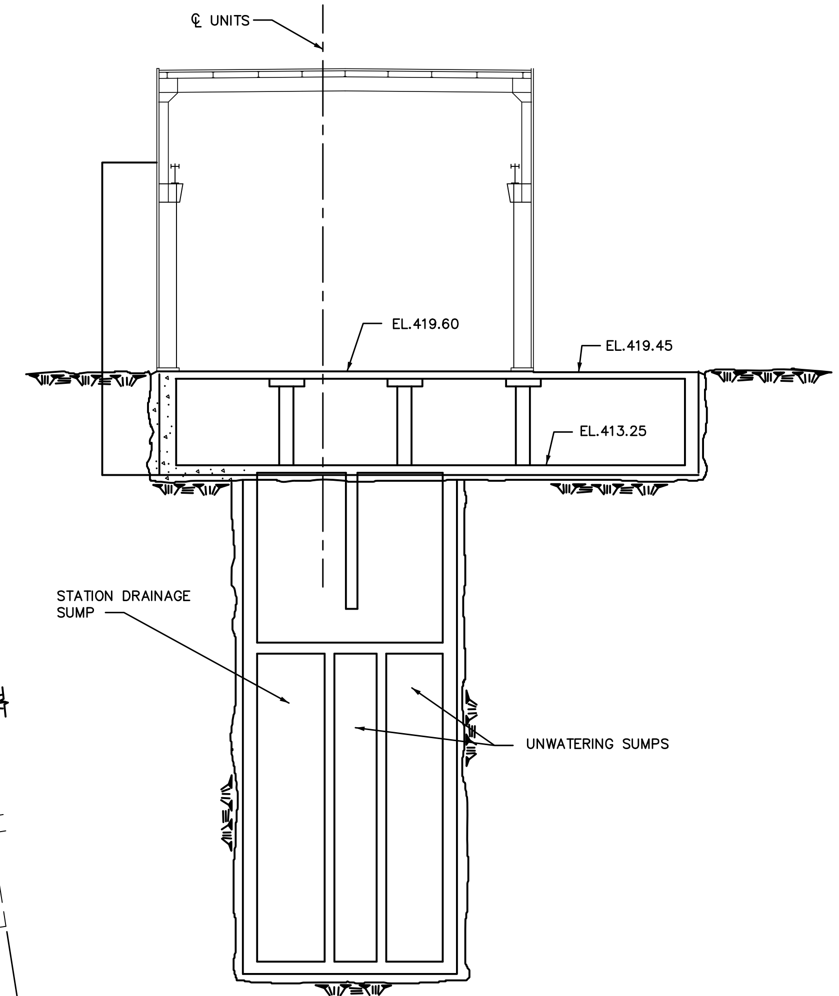
DATE MAY 2009	FIG NO FIGURE 4		
DWN WN	DWG NO 1016-C14-D3288	R	

 B

Douglas, Simon



SECTION A
1:1000
3289



SECTION B
1:1000
3290

ORIGINAL NOTES:

1. FOR LOCATION OF SECTIONS, SEE FIGS. 23 & 24.
2. BASED ON FIG.3–10 SHEET 1 (DWG,No.1016–C14–D914) B.C.HYDRO REPORT EP8 DATED SEPTEMBER 1986.
3. P.M.F. TAILWATER VARIES FROM EL.416.6 TO EL.418.6 DEPENDING ON DEGRATION OF THE RIVER BED DOWNSTREAM.

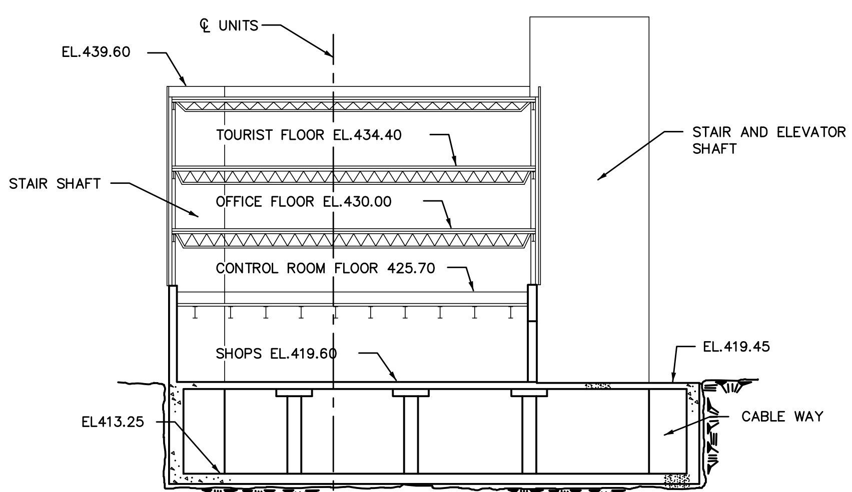
L E G E N D :

1st STAGE CONCRETE

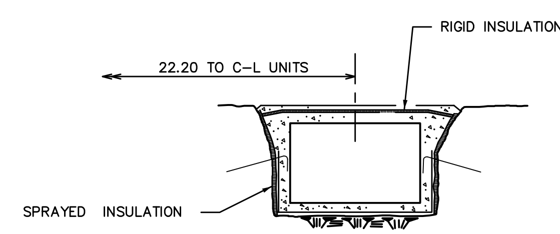
2nd STAGE CONCRETE

UPDATED NOTES:

1. DRAWING NUMBER REFERENCES ON SECTION MARKERS AND ADJACENT TO MATCH LINES REFLECT UPDATED DRAWING NUMBERS.



SECTION C
1:1000
3290



SECTION D
1:1000
3290

DRAWING LINEWORK AND ORIGINAL NOTES COPIED FROM DWG. 1016–C14–D2679 DATED JAN. 2003

0 5m

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PEACE RIVER SITE C PROJECT

POWERHOUSE
POSSIBLE MEASURES FOR REBOUND MITIGATION

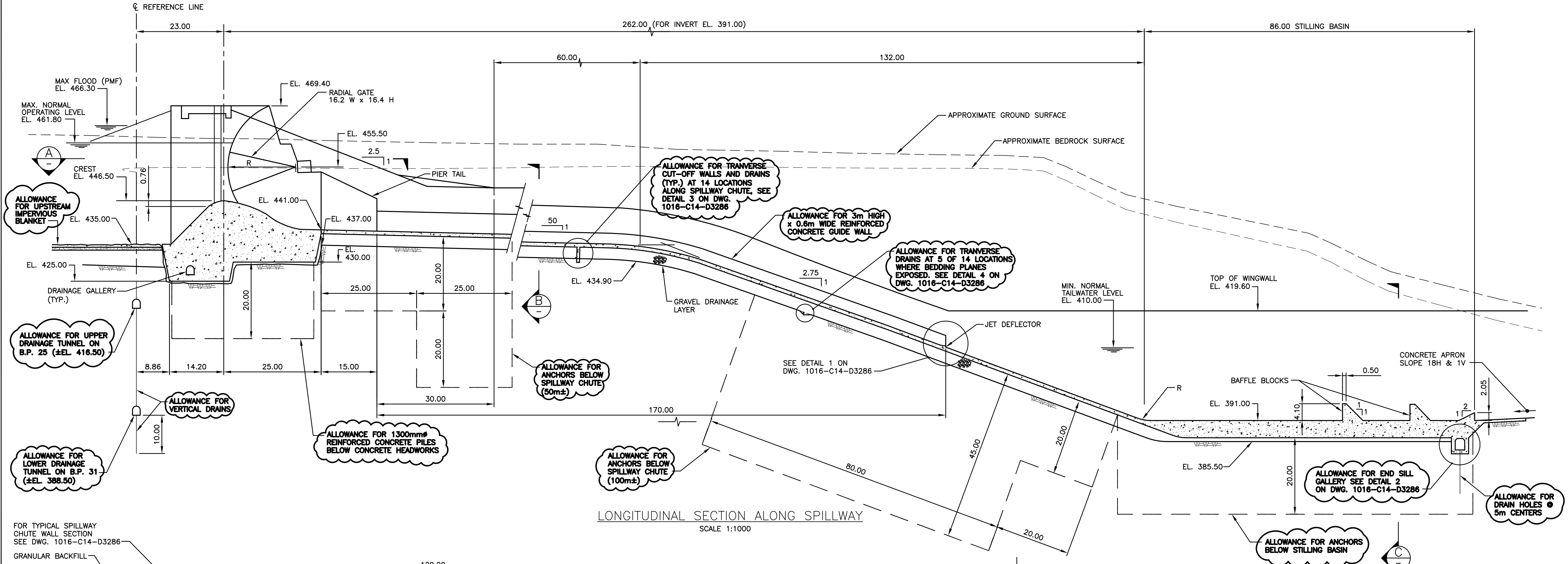
DSGN	
INDEP CHK	
DFTG	WN (KCBL)
DFTG CHK	
REPORT NO.	
PROJ. NO.	

DATE	MAY 2009	FIG NO	FIGURE 5
DWN	WN	DWG NO	1016-C14-D3291

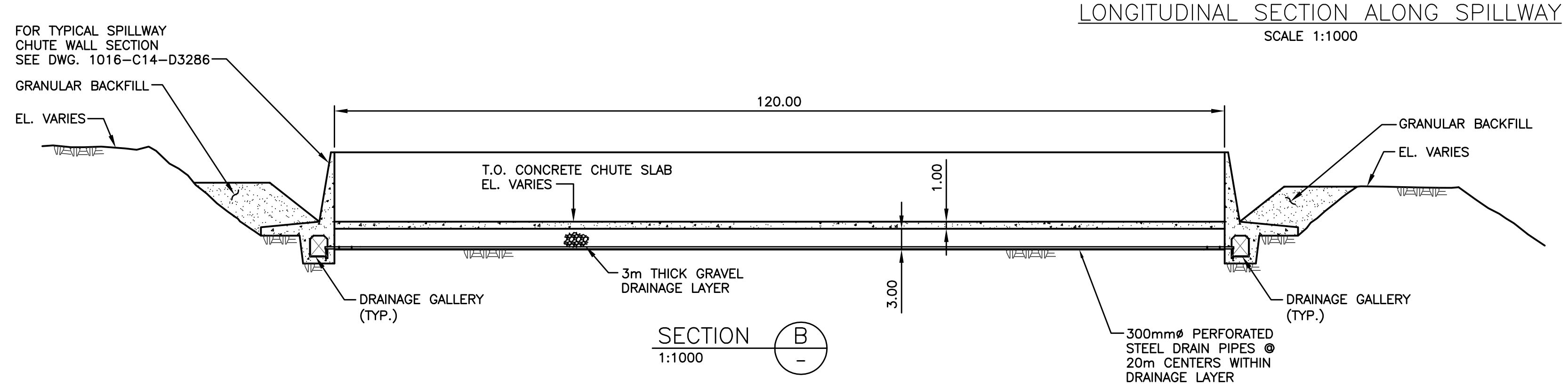
R

 B

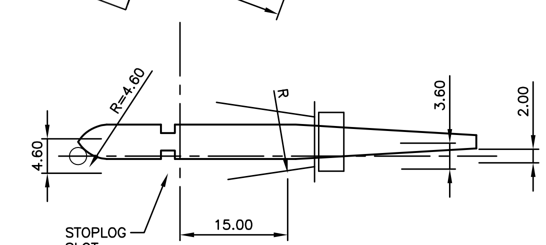
Douglas, Simon



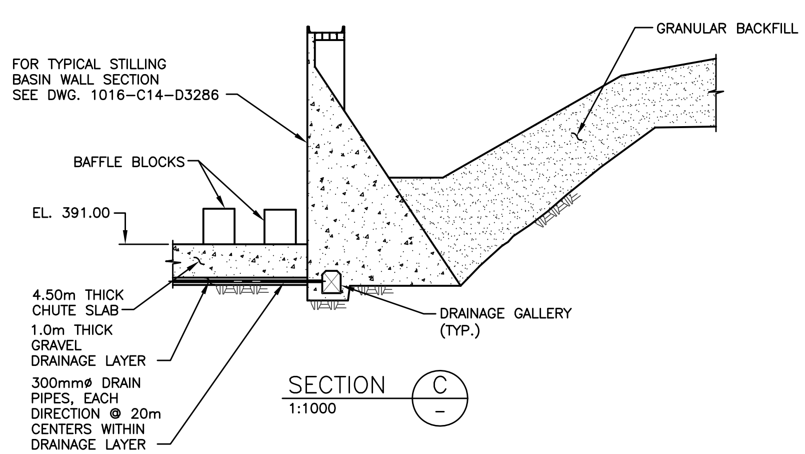
LONGITUDINAL SECTION ALONG SPILLWAY
SCALE 1:1000



SECTION B
1:1000



SECTION A
1:1000



SECTION C
1:1000

NOTES:
1. ALL DIMENSIONS AND ELEVATIONS ARE IN METERS, UNLESS NOTED OTHERWISE.

0 25m

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DSGN	
INDEP CHK	
DFTG	WN (KCBL)
DFTG CHK	
REPORT NO.	
PROJ. NO.	

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PEACE RIVER SITE C PROJECT

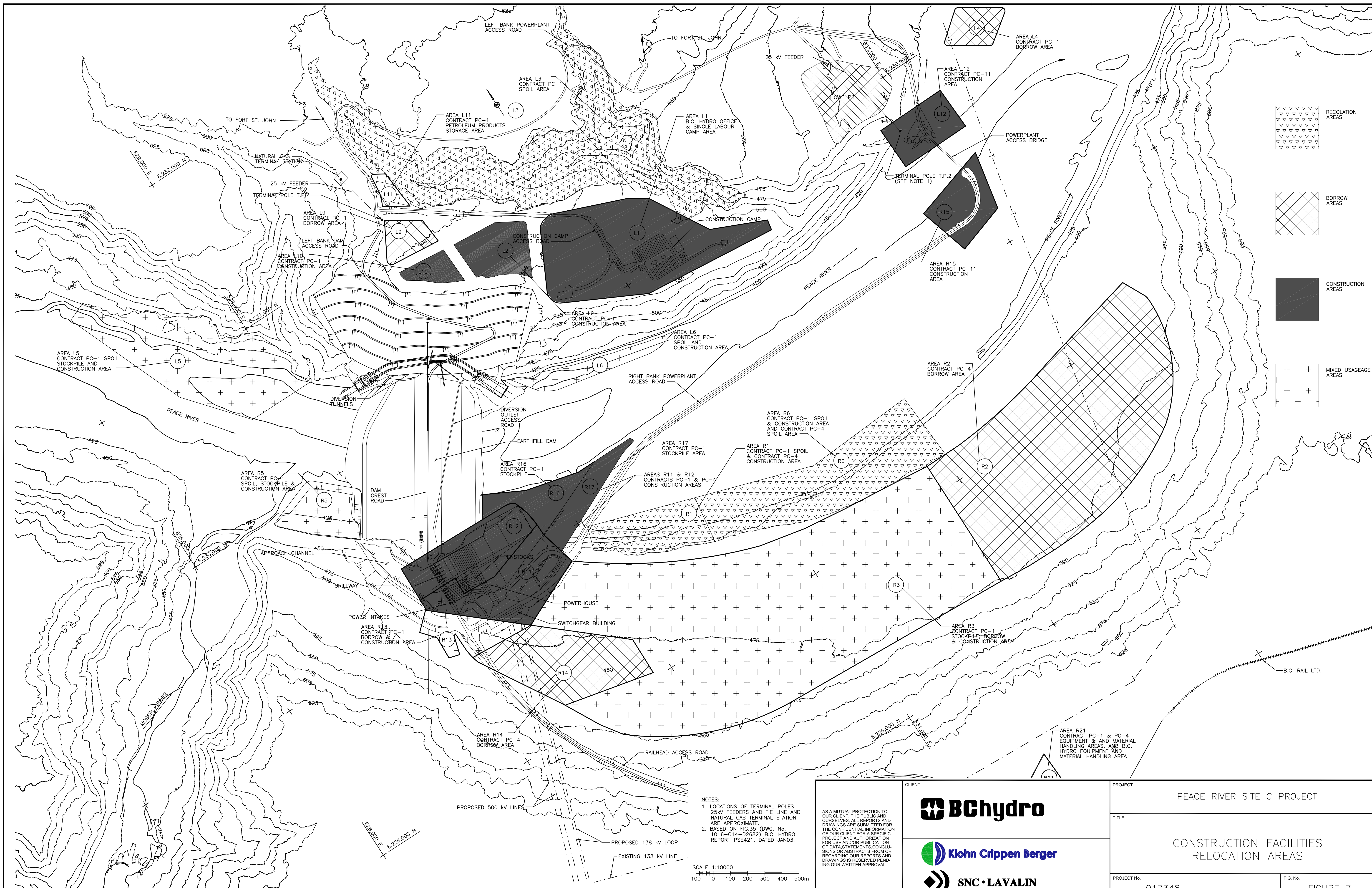
SPILLWAY
POSSIBLE MEASURES FOR STABILIZATION AND REBOUND MITIGATION

DATE	FIG NO
MAY 2009	FIGURE 6
DWN: WN	DWG NO: 1016-C14-D3285

R

 B

Douglas, Simon



NOTES:
1. LOCATIONS OF TERMINAL POLES. 25kV FEEDERS AND TIE LINE AND NATURAL GAS TERMINAL STATION ARE APPROXIMATE.
2. BASED ON FIG.35 (DWG. No. 1016-C14-D2682) B.C. HYDRO REPORT PSE421, DATED JAN03.

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CLIENT: BChydro
Klohn Crippen Berger
SNC • LAVALIN

PROJECT: PEACE RIVER SITE C PROJECT

TITLE: CONSTRUCTION FACILITIES RELOCATION AREAS

PROJECT No: 017348

FIG. No. FIGURE 7

DATE: 2009/11/30 - 1:44pm

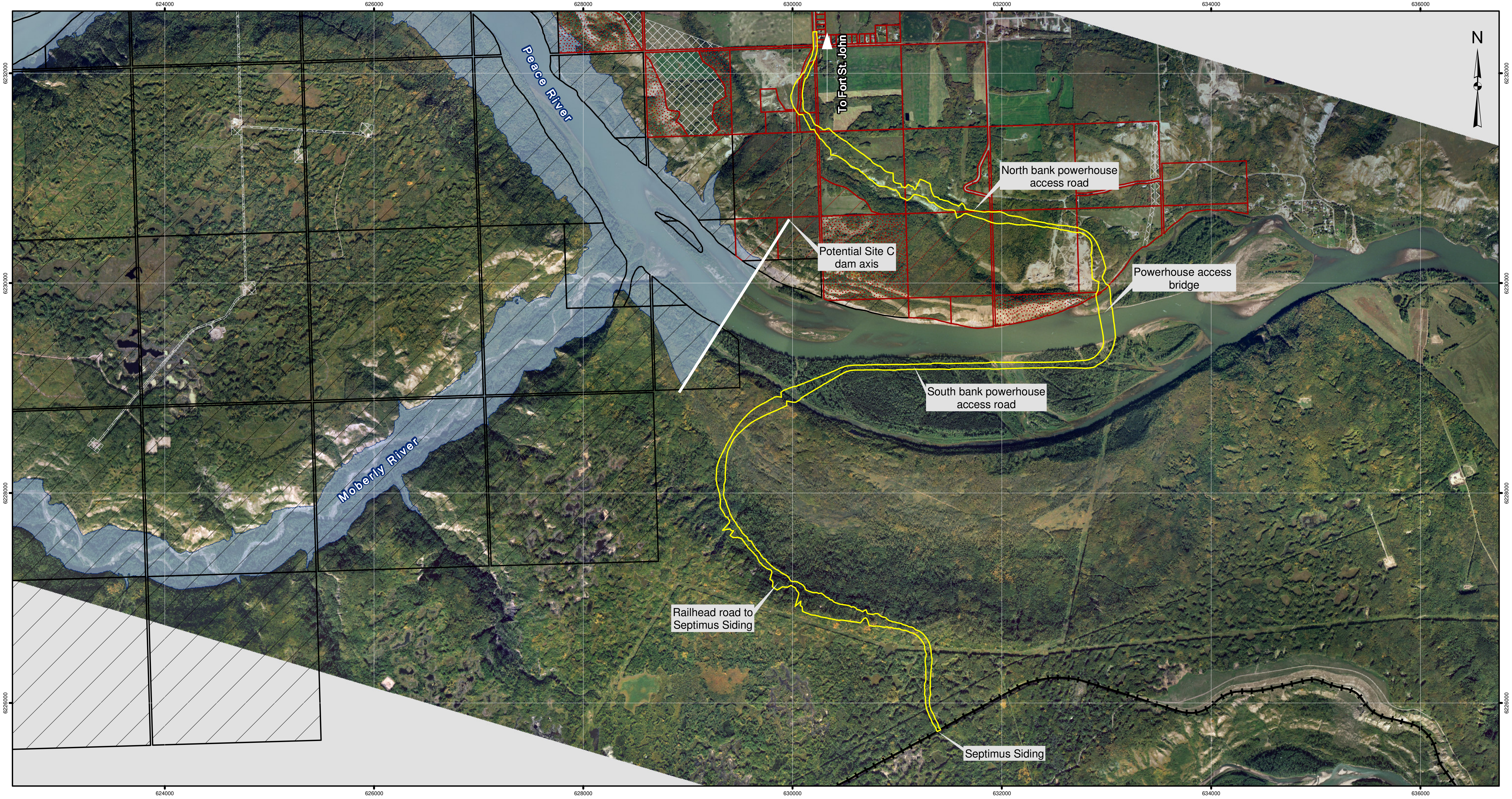
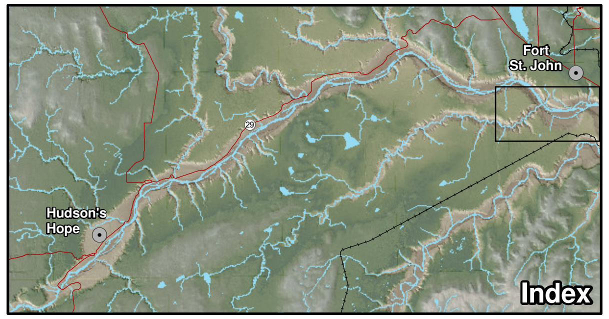


FIGURE 8



Map Notes:
1. Orthophotos created from 1:40,000, 1:20,000, and 1:5,000 scale photography taken June and Sept, 2007.
2. Proposed reservoir flood line (461.8m maximum derived normal elevation) from Digital Elevation Models (DEM) generated from LiDAR data acquired July and August, 2006.
3. Property status information represents BC Hydro's current ownership records.
4. Property boundaries shown are the result of computations and adjustment to GPS field observations. The estimate positional accuracy is less than 1.0 metre.
5. Datum/Projection: NAD83/UTM Zone 10N.

Legend:

- Potential flooded area (for reservoir at EL. 461.8m)
- Crown land
- BC Hydro owned land (leased)
- BC Hydro owned land
- Private land
- BC Hydo, Oil & Gas, Telephone, Municipal & Misc. ROWs
- Railway
- Powerhouse access road & railhead road to Septimus Siding

Scale: 1:37,000

0 1,500 m

BC hydro

Klohn Crippen Berger

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POWERHOUSE ACCESS ROAD & BRIDGE

DATE	JUNE, 2009	DWG NO	1016-C14-D3184	R 0

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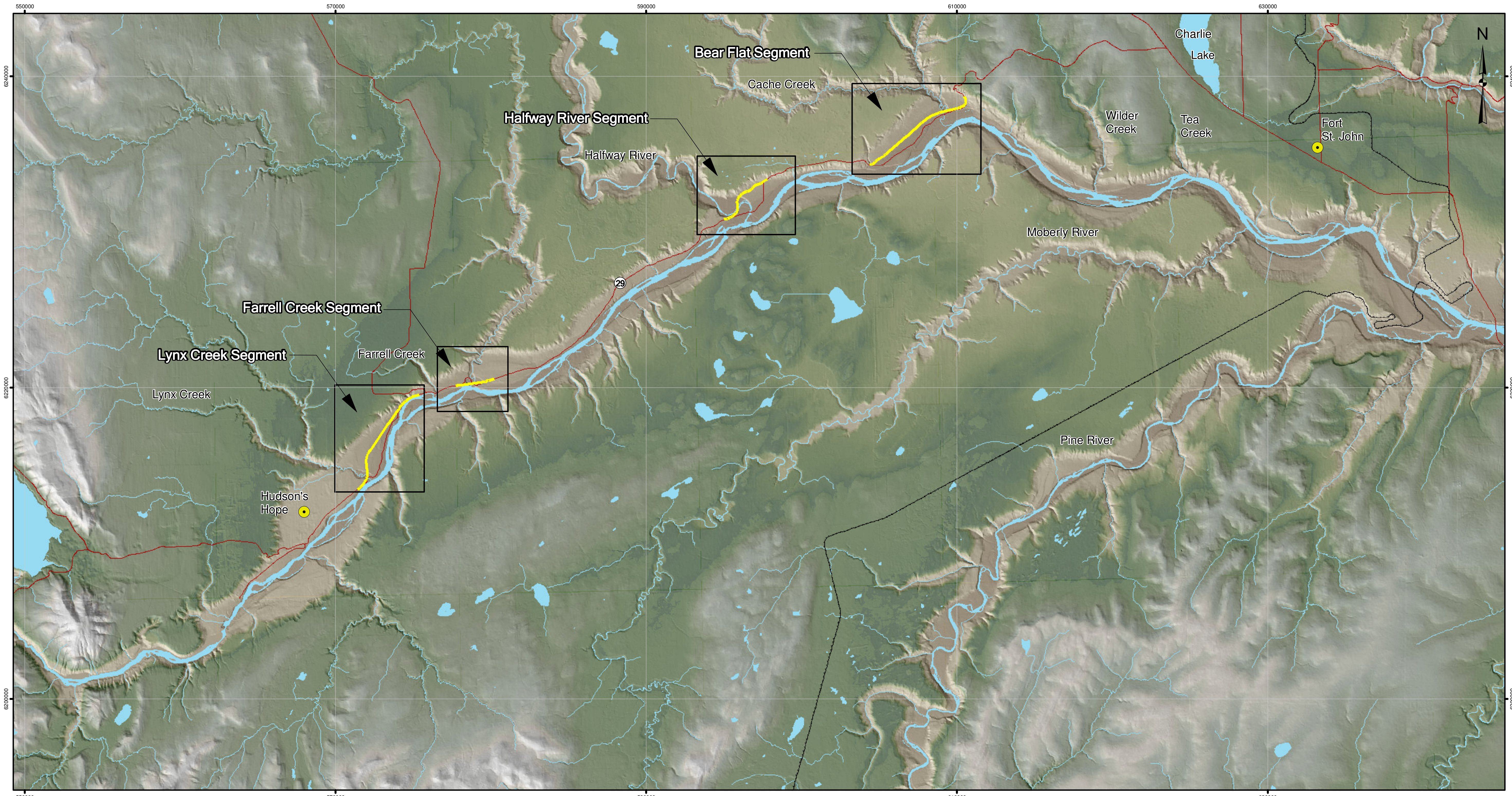

Figure 9

Legend:

29 Highway 29

Highway realignment section

Railway

Scale: 1:250,000 0 10km

Notes:
1. Datum: NAD83
2. Projection: UTM Zone 10N
3. Data Source: BC Hillshade model courtesy BC Environment, Lands and Parks

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BC hydro

Highway 29 Realignments

DATE	JUNE, 2008	µ	DWG NO CAD	xxxx-xxx-xxxxx	R X

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