Feasibility Study for Portland Creek Hydroelectric Project

FINAL REPORT

Prepared for:
Newfoundland & Labrador Hydro

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1 INTRODUCTION

1.1 PREAMBLE

This report on the Feasibility Study of the Portland Creek Hydroelectric Development was prepared on behalf of Newfoundland and Labrador Hydro (NL Hydro). The proposed development is located in Western Newfoundland and Labrador, near Daniel's Harbour on the west side of the Great Northern Peninsula. More specifically, it is located on Main Port Brook, a tributary of Portland Creek. A Pre-Feasibility Study[1] was completed in 1987 by our firm.

1.2 STUDY SCOPE AND OBJECTIVES

The Terms of Reference required field investigations and engineering studies to confirm the feasibility of the Project, and to complete a Construction Schedule, including a detailed Capital Cost Estimate and a Risk Analysis of its viability based on assumptions made and possible future economic impacts.

The Scope of Work was defined in the RFP, as follows:

- A comprehensive review of all available reports prepared by others and related to this Project;
- Hydrologic/hydraulic studies;
- A Geotechnical investigation, including test pitting, drilling, seismic or geophysical surveys at all structure sites as deemed necessary by the Consultant to assess the suitability of foundation conditions and/or depth of overburden and identify any areas of erosion or slope stabilization problems from a geotechnical perspective;
- An exploration program sufficient to identify sources and quantities of construction materials and to indicate the Project geology;

[1] 1987 Small Hydro Studies, SNC-BAE Joint Venture

- A field survey program to obtain more detailed topography of all structure sites;
- Preparation of mapping to identify any flooded zones and clearing requirements;
- Identification of areas with suitable size and terrain for disposal of excavated materials, including earth and rock;
- Identification of the optimum routes for the permanent and temporary Project access roads;
- Detailed office engineering and optimization studies using energy and capacity rates provided by NL Hydro;
- Identification of the location, layout, extent and methods for mitigation of the HADD (in conjunction with NL Hydro);
- Development of a detailed construction schedule, assuming a Project in-service date of October 31, 2010;
- Preparation of a Capital Cost Estimate (CCE) for the Project.

1.3 CHANGES TO STUDY SCOPE AND OBJECTIVES

The primary objective of the Geotechnical Investigation was changed to a review and comment on reports that were previously prepared for the Project and to carry out additional investigations, (i.e., test pits), at the proposed dam, powerhouse, and access roads to determine the surface condition and to comment on the geotechnical aspects of each area. In addition, sources for various grades of construction material were to be investigated and inventoried.

Geotechnical drilling of the sites of the major structures, including the powerhouse site was deleted from the Scope of Work. Changes were made in a Post Tender Addendum.

1.4 EXCLUDED WORK

Excluded from the Scope of the Services were:

- Environmental/biological studies to confirm the extent of fish habitat to be altered, disturbed or destroyed (HADD) that may result with the flooding of existing ponds and/or the excavation for the tailrace;
- Environmental/biological studies to confirm the general location and methods for mitigation of the HADD;
- Negotiations with Department of Fisheries and Oceans to agree on the HADD compensation;
- Environmental/biological studies related to caribou and other wildlife species;
- Transmission line size and route to be determined by NL Hydro; information will be provided to the Consultant for inclusion in the Project report;
- Telecontrol supervisory and communications, to be determined by NL Hydro; information will be provided to the Consultant for inclusion in the Project Report.

1.5 PREVIOUS STUDIES

1. Pre-feasibility study by SNC-BAE Joint Venture in 1987, and
2. Pre-feasibility study by Pinnacle Engineering and Design in 2004, as a study project by MUN engineering students.

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2 DESCRIPTION OF SITE

2.1 GENERAL

The proposed Portland Creek development would be located on Main Port Brook, a tributary of Portland Creek on the west side of the Great Northern Peninsula. The proposed development would utilize the available net head of approximately 394.9 m between the Head Pond and the outlet of Main Port Brook.

The development is composed of the following key components:

- 320 m long Diversion Canal that transfers flows from the Diversion Pond into the main storage reservoir. Maximum depth of excavation is 16.2 m;
- 110 m long concrete gravity Diversion Dam and overflow spillway with a crest length of 70 m. Maximum height of dam is 12.0 m;
- A 45 m long concrete gravity Storage Dam including a flow regulating structure fitted with a trash rack. Maximum height of dam is 16.0 m;
- A 143 m long concrete gravity Headpond Dam including a power intake structure fitted with a trash rack and an overflow spillway. Maximum height of dam is 15.0 m;
- A 2900 m long Penstock, 1.52 m in diameter, to convey water from the Headpond Dam to the powerhouse.
- A 17 m x 14 m powerhouse, constructed of concrete substructure and steel superstructure, and equipped with two Pelton turbine-generator units.
- A 66 kV switchyard, adjacent to the powerhouse.
- A 27 km transmission line (66 kV), connecting the switchyard to the existing substation at Peter's Barren.

The spillway for the storage reservoir will be located on the Diversion Dam. A small spillway is required on the Headpond Dam to handle natural drainage between the Headpond and Storage Dams plus the gated discharge from the Storage Dam in the event of a plant shut down during flood flow.

The principle parameters for the Portland Creek Development are discussed through this report and are summarized below:

- Installed Capacity - 23.0 MW;
- Number of Units - 2 (Pelton Turbines);
- Plant Flow (max) - 6.60 m^3/s;
- Net Head - 394.9 m;
- Annual Energy - 141.5 GWh;
- Plant Capacity Factor - 72%.

Access

Access to the site would be provided from the existing Daniel's Harbour Mine Road and would include upgrading of approximately 9.16 km of existing forest access road and 18.15 km of new construction from the end of the forest access roads along Inner Pond to the powerhouse. In addition, 11.62 km of construction access would be required to access the Storage, Diversion and Headpond Dam sites.

2.2 REGIONAL GEOLOGY

The Project area is within Zone A of the Appalachian Structural Province. This zone is characterized by a complex assemblage of basement rocks (gneiss, schist, anorthosite, granite and granodiorite) belonging to the Canadian Shield (Grenville Province), which are overlain by carbonate sedimentary rocks (limestone and dolomite) and by transported clastic sequences (siltstone, sandstone and conglomerate). In general, the Shield Rocks occupy the highlands and the carbonate/clastic rocks occupy the lowlands.[2]

[2] Geotechnical Investigation for Feasibility Study for the Portland Creek Hydroelectric Development, prepared by AMEC Earth & Environmental, January 2007.

2.3 SUMMARY OF SITE TOPOGRAPHY & SURFICIAL GEOLOGY

The development area is located within two physiographic divisions of the Appalachian Region. The high elevations at the power development site are within the Atlantic Uplands Physiographic Division of the Island of Newfoundland. More specifically it is within the Newfoundland Highlands, Long Range Mountains sub-division. The access road in the lower elevations crosses over the St. Lawrence Lowlands Region, which has been further divided into the East St. Lawrence Lowlands and then into the Newfoundland Coastal Lowland.

The whole of the area was glaciated by the last advance of the Wisconsin Glaciation and much of the pre-glacial surface has been scoured and subsequently covered by a discontinuous layer of till of varying thickness. The topography is generally characterized by features commonly seen in areas of continental glaciation and from alpine glaciation at the edges of the mountains. In areas away from high relief the ground is characterized by rolling hills and many thin glacial till deposits. Drainage is poor with numerous bogs formed and well-defined stream drainage patterns mimic the underlying bedrock structures. Ice flow direction in the study area is estimated to be generally from the east to west with local deviations in the steep sided valleys.

Glacially derived soils in the form of a thin veneer of glacial till covers about 30% of the Highlands portion of the site. Bedrock outcrop is generally continuous or covered by thin organic growth in areas of high relief. Colluvial soil in the form of talus covers many of the lower slopes in areas of high relief. Glacial till in the form of lateral moraines is present at the boundary of the Long Range Mountains and the Newfoundland Coastal Lowlands.

The Coastal Lowlands are generally covered with a thin layer of marine sediments in the form of raised beaches. Low-lying areas of the Coastal Lowlands contain marine clay and silt.

Periglacial sea water levels were higher than today. There is evidence in the Portland Creek/Inner Pond Valley that water levels stood at approximately 110 m above present levels. Glacial melting was still active with large amounts of material being carried in the streams and subsequently deposited as glaciofluvial material in the Valley. In fact, there are extensive glaciofluvial deposits of sand, gravel, cobbles and boulders filling the valley. The surface area of these deposits covers about 3 km^2. These deposits have been subsequently eroded with several erosion terraces recognized in the powerhouse area and north towards Inner Pond. About 0.5 km^3 of glaciofluvial soil may exist in this Valley.

Karst topography has developed in the carbonate sediments south of Brian's Pond along the proposed access road. At least two small streams disappear beneath the ground surface in this area and there are several ponds with internal drainage.[3]

2.4 BEDROCK GEOLOGY

In the area of the proposed dam sites and powerhouse the rocks are a mix of intrusive and metamorphic genesis and comprise mega-crystic granite, granite gneiss and quartz monzonite of Grenvillian to Helikian age. They are part of the Canadian Shield, which is a suite of somewhat ancient rocks forming the core of the North American continent.

On the coastal lowlands the rocks are a carbonate sequence of limestone, dolomite and some clastic sediments.[4]

[3] Ibid
[4] Ibid

3 SITE INVESTIGATIONS

3.1 GENERAL

A site investigation crew was mobilized to the site on September 28, 2006. Fieldwork was completed on October 4, 2006 and the crew demobilized on October 5, 2006. The crew consisted of:

1. David Brown, P. Eng., Project Manager, NL Hydro;
2. Brad Chaulk, P. Eng., Project Manager, SNC Lavalin Inc.;
3. Calvin Miles, P. Geo., Project Manager, AMEC, sub consultants to SNC Lavalin Inc.;
4. Bruce Chaulk, Logistics;
5. Brian Warren, Lead Surveyor;
6. Jason Morgan, Assistant Surveyor;
7. Three Cutters;
8. One Jack Hammer Operator.

Fieldwork included a geological assessment of the site(s), location of access roads, identification of locations of key structures, and topographic surveys of the sites, including the establishment of 14 points of geodetic control for the preparation of 1:2500 mapping of the Project site.

3.2 MAPPING

To prepare this study, new 1:2500 scale topographic mapping with 2 m contours was prepared by Robert Croucher based on color aerial photography taken in 1979 and obtained from the Mapping Division of the Provincial Department of Environment. To prepare the mapping, 14 control points were established in the field during the execution of the Field Program.

3.3 TOPOGRAPHIC SURVEYS

Topographic Surveys, using Topcon GPS GB-500 Bosc and Hiper Pro Roul were undertaken at each of the key structures, namely;

- Powerhouse;
- Headpond Dam;
- Storage Pond Dam;
- Diversion Canal;
- Diversion Dam.

The topographic surveys were incorporated with the contour mapping prepared by Croucher and are part of the Project drawings. Mapping Control and details of surveyed areas are included in Interim Report No. 1, which is included in Appendix C of this report.

3.4 SITE CONDITIONS

3.4.1 Diversion Dam and Diversion Canal

As these structures are in close proximity to each other and there was no discernable change in the rock conditions between the sites, they are treated as one area for the purpose of geotechnical discussions. See Figures 3-1 and 3-2.

The area is generally open barrens with few trees and about 20% exposed bedrock. Bogs have developed in the low, poorly drained areas between rock outcrops and on planar hillsides. Probing revealed that the bog was as much as 1.5 m in some areas; however, depths averaging about 1 m were encountered in the nine test locations.

Except for boulders no glacial derived soils were recognized in the area.

Colluvium in the form of frost heaved boulders and coarse boulder talus exists sporadically in the area. The channel of the existing stream contains many large boulders, which appear to be talus from the adjacent left bank, steep hillside.

Beneath the surficial layers of bog and sporadic colluvium, bedrock will be encountered. Massive, hard, and strong granodiorite and quartz monzonite were the dominant rock types recognized over the site. (These rocks are similar to granite in appearance but are characterized by containing more plagioclase feldspar for the granodiorite and about equal amounts of plagioclase and orthoclase for the quartz monzonite. Both rock types have reduced quartz content compared to pure granite). Mica and hornblende are common in both rock types. The rock is megacrystic with crystals generally a few centimetres across with occasional areas containing 5 cm crystals. Joint spacing was wide where observed with spacing over 2 m apart in many of the outcrops observed. Exfoliation jointing was poorly developed with only one joint recognized at the Diversion Dam axis.

A very weak schistosity exists in the rock with strike of about 80° AZ and near vertical dip.[5]

In summary, the area is well suited for a concrete gravity dam with only excavation of the overburden and loose or broken rock to be removed. Grouting of the bedrock will be required. Materials excavated from the Diversion Canal should be suitable for concrete aggregate subject to testing to evaluate its quality in accordance with applicable CSA and ASTM standards.

[5] Ibid

Figure 3-1: Map of Diversion Dam & Diversion Canal

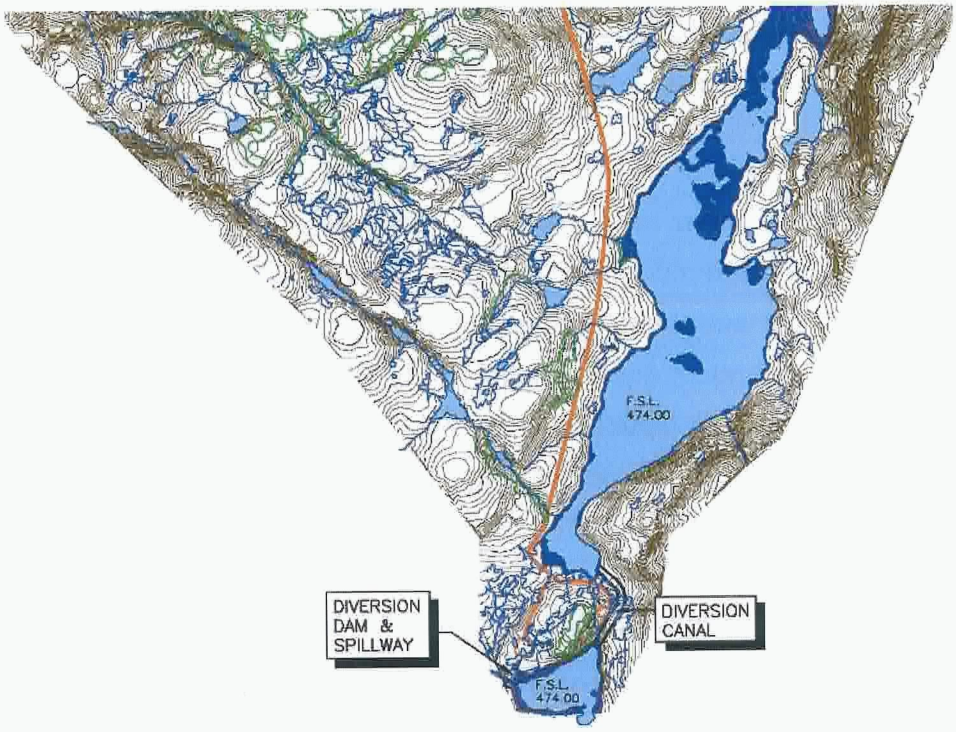
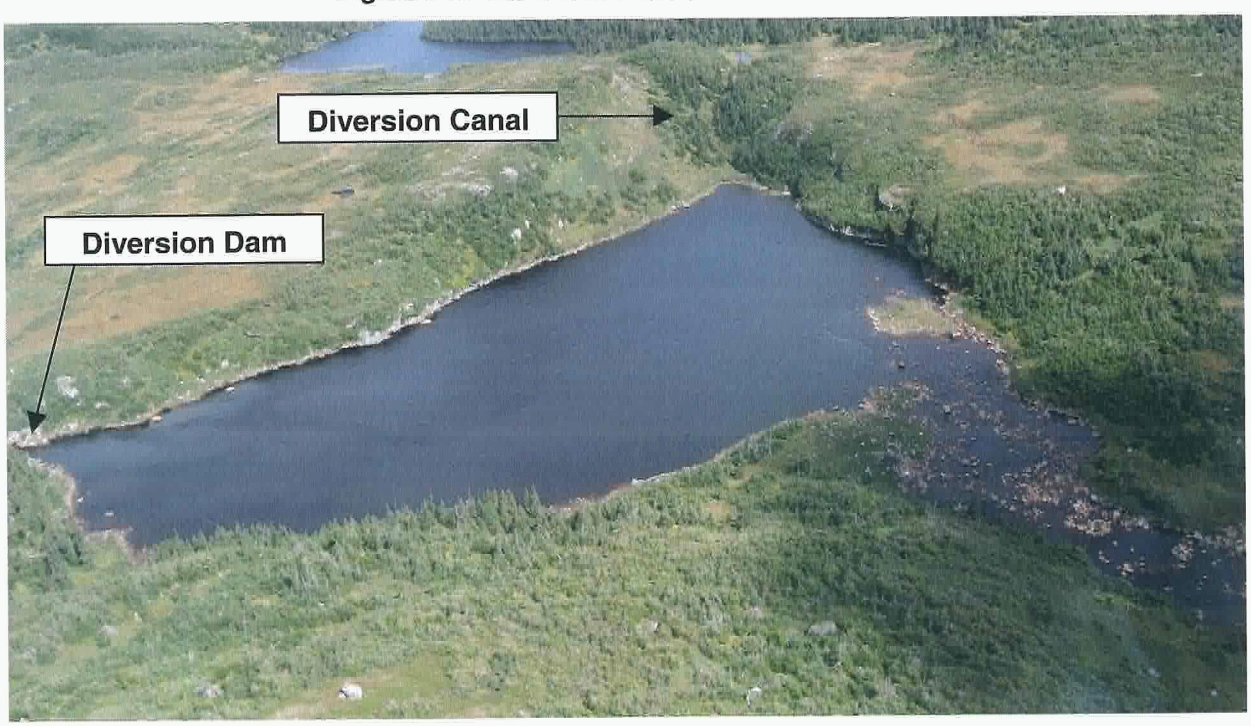


Figure 3-2: Diversion Dam & Diversion Canal



3.4.2 Storage Dam

The terrain in this area varies from gently rolling hills and low relief at the lake outlet to a steep sided valley at the river at the dam location. In general, the immediate area of the dam on the right bank is covered by occasional scrub (dwarf trees) and barrens occurring on the hillsides and tops. The left bank is covered with scrub and barrens. See Figures 3-3 and 3-4.

Except for boulders no glacial derived soils were recognized in the area.

Colluvium in the form of frost heaved boulders and coarse boulder talus exists sporadically in the area. The channel of the existing stream contains many large boulders, which appear to be talus from the adjacent steep hillsides. Bedrock outcrop is abundant in the riverbed and slopes of the river valley. Overall the immediate area has about 30% exposed rock and about 60% rock with a thin layer of organic growth. The remaining 10% is either colluvium or bog.

Beneath the surficial layers of bog and sporadic colluvium, bedrock will be encountered. Granodiorite was the dominant rock type recognized over the site. Mica and hornblende are common in all rock observed. The rock is coarse grained with crystals generally a centimetre across with occasional areas containing 2 cm crystals.

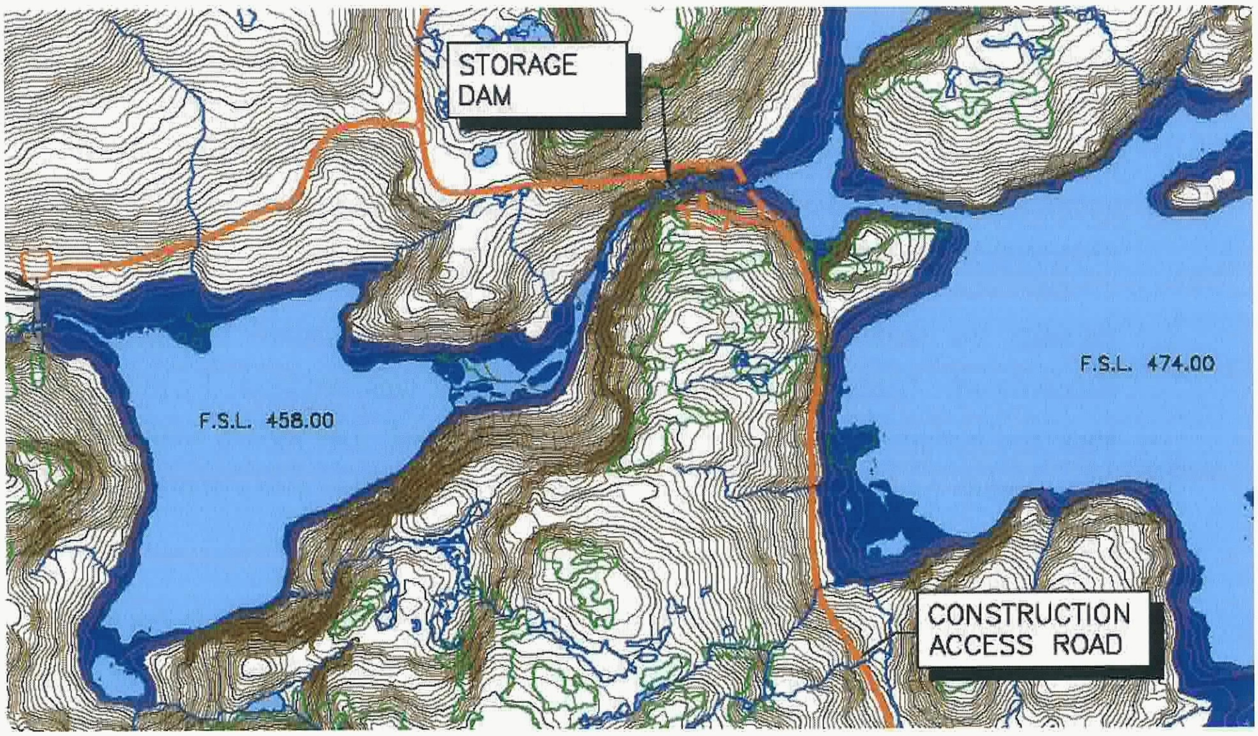
Joint spacing was generally close in rocks in the riverbed to very wide in the rocks on the adjoining slopes. A prominent, very close spaced joint set had strike of 227° to 255° AZ and dip 70° to 85° NW. A single joint, which showed some movement and is best described as a shear joint had strike of 255° AZ and dip of 85°N. Other joints, most of which are discontinuous and at random orientation occur throughout the area. All joints observed in the streambed where frost action was not prevalent were tight with little or no separation. In areas away from the stream, frost had opened up the joints several millimetres to several centimetres.

Exfoliation jointing was not observed in the rocks at this dam site. However, most granitic rocks do have fractures just below and parallel the ground surface.

A moderately strong gneissosity exists in the rock in the stream with strike of 225° to 275° AZ and dip of 80° NW.[6] This gneissosity generally lines up with an east-west lineament observed from the air photos and on topographic maps.

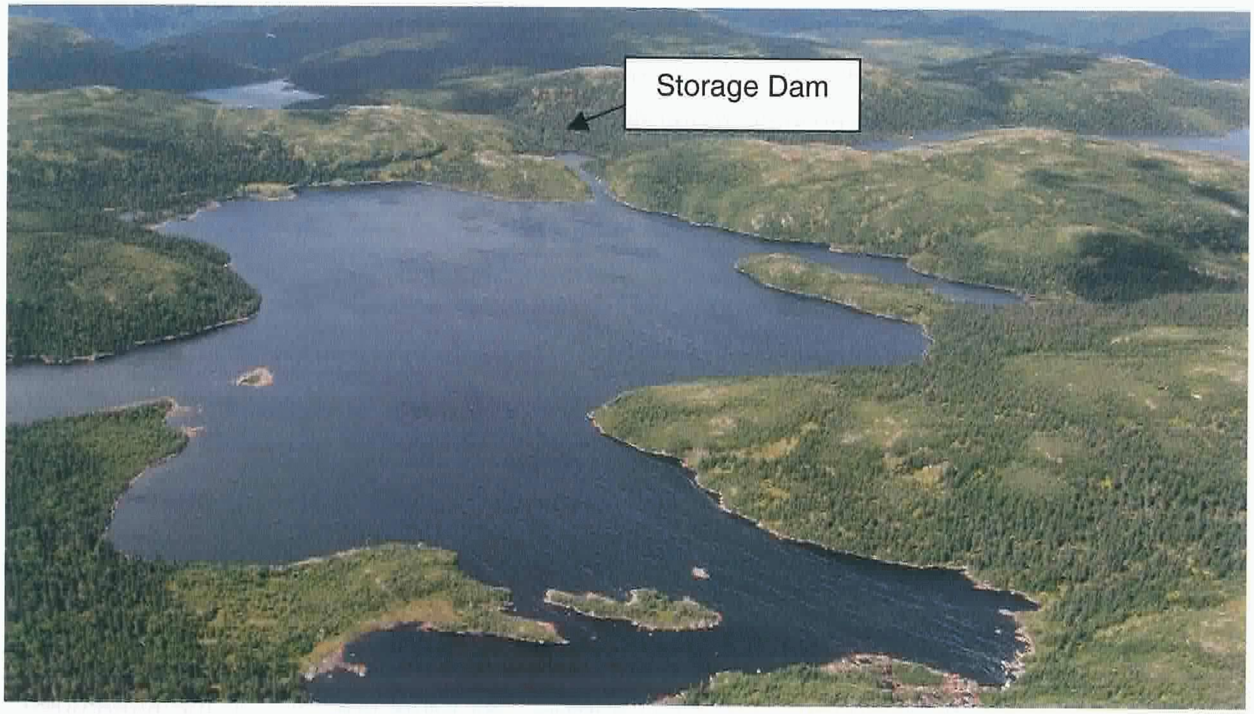
In summary, the rock is hard and similar to conditions at the Diversion Dam. The area is well suited for a concrete gravity dam with only removal of overburden together with loose or broken rock required. Grouting is required beneath and along the sides of the dam.

Figure 3-3: Map of Storage Dam



[6] Ibid

Figure 3-4: Storage Dam



3.4.3 Headpond Dam

The terrain in this area varies from gently rolling hills and low relief at the lake outlet, where the Dam and Spillway are proposed to be located, to a steep sided valley at the river just downstream of the dam location. In general, the immediate area of the dam on the right bank is covered by a light, mature forest with scrub and barrens occurring on the hill sides and tops. The left bank is covered with scrub and barrens. See Figures 3-5 and 3-6.

Except for boulders, no glacial derived soils were recognized in the immediate dam area.

Colluvium in the form of frost heaved boulders and coarse boulder talus exists sporadically in the area. The channel of the existing stream contains many large boulders, which appear to be talus from the adjacent steep hillsides.

Bedrock outcrop is abundant in the riverbed and slopes of the river valley. Overall the immediate area has about 30% exposed rock 70% rock with a thin layer of organic growth.

Beneath the surficial layers of organic material and occasional colluvium, bedrock will be encountered. Megacrystic granite was the dominant rock type recognized over the site. Mica and hornblende are common in all rocks observed. The rock is coarse grained with crystals generally a centimetre across with occasional areas containing 2 cm crystals. Occasional small pegmatite veins were recognized in the rock at the river's right bank. These veins were generally thin, less than 5 cm thick and contained a mix of quartz and potassium feldspar crystals.

Joint spacing was generally wide spaced in rocks in the riverbed to very wide in the rocks on the adjoining slopes. No discernable patterns were noted on the ground, however, plotting of the joint orientations on a Schmitt Equal Area Stereonet provided a joint set with strike of 265° AZ and dip of 85° N and another with Strike of 010° AZ and near vertical dip. Other random joints, most of which are discontinuous occur throughout the area. Several near horizontal joints, spaced from 1 to 5 m apart exist in the area of the proposed dam and at a location 300 m downstream. These are interpreted to be exfoliation joints. All joints observed in the streambed, where frost action was not prevalent, were tight with little or no separation. In areas away from the stream, frost had opened up the joints several mm to several cm.

Although not recognized in the rocks at the dam site, the published maps from the Geological Survey of Canada show schistosity on the nearby hill tops orientated in a general east to southeast direction with dips ranging from 65° N to 79° S.[7]

In summary, the bedrock is hard and the area is also well suited for a concrete gravity dam with removal of overburden and loose and broken rock. Grouting will be required.

[7] Ibid

Figure 3-5: Map of Headpond Dam

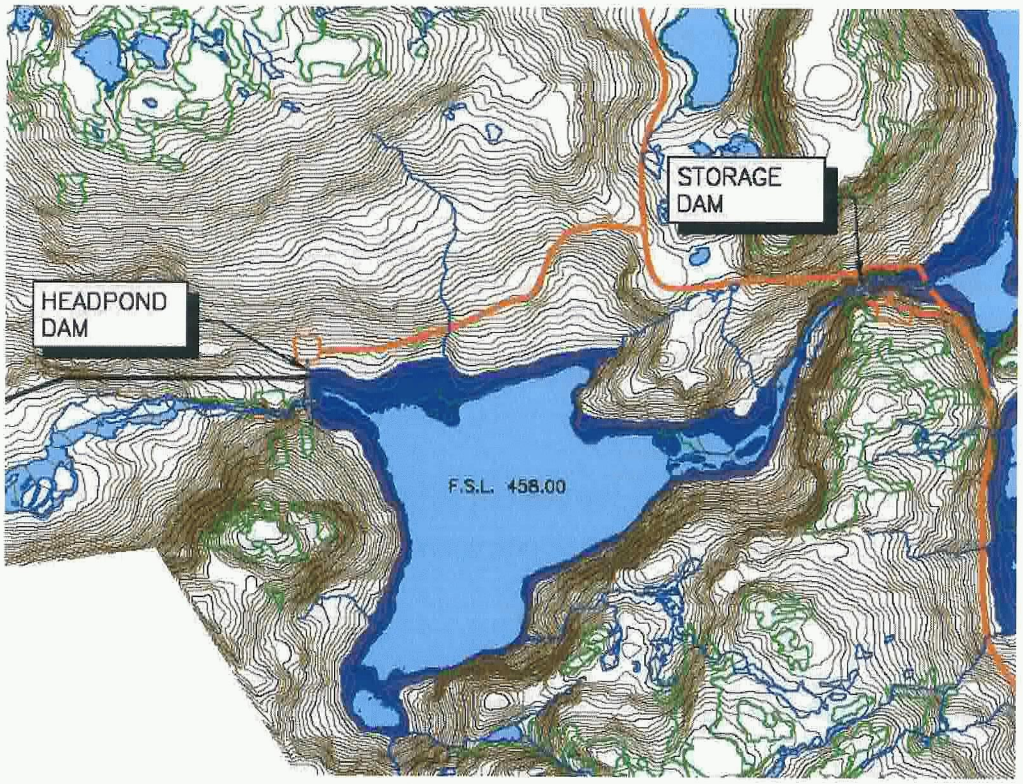
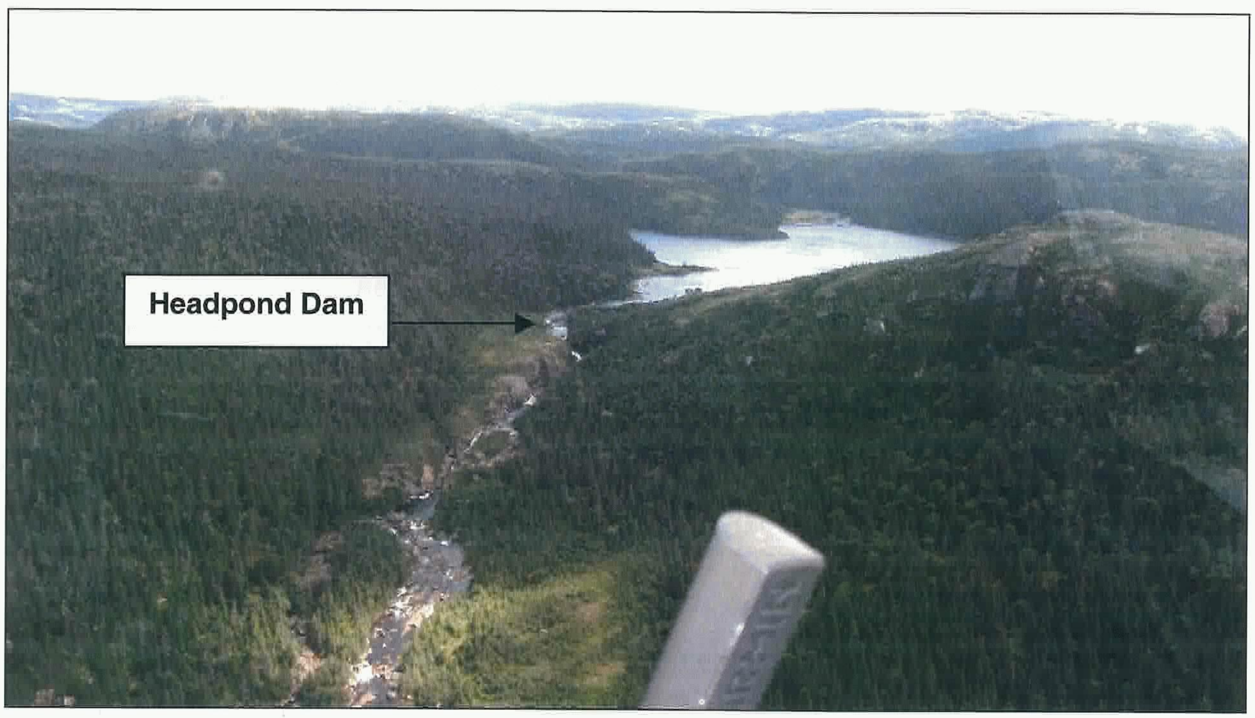


Figure 3-6: Headpond Dam



3.4.4 Penstock Route

The penstock will traverse three distinct terrains as follows:

- Km 0.0 to km 2.1 will traverse an uplands terrain consisting of a moderately steep side hill with bedrock ridges;
- Km 2.1 to km 2.5 will traverse an alpine terrain with very steep slopes. Overall slope angle approaches 14° with local areas at 33° over a length of 140 m. Several, short, near vertical, cliff faces, generally less than 15 m high, exist in the area;
- Km 2.5 to km 2.9 will traverse an uplands terrain with a combination of flat areas and relatively steep slopes approaching the powerhouse.

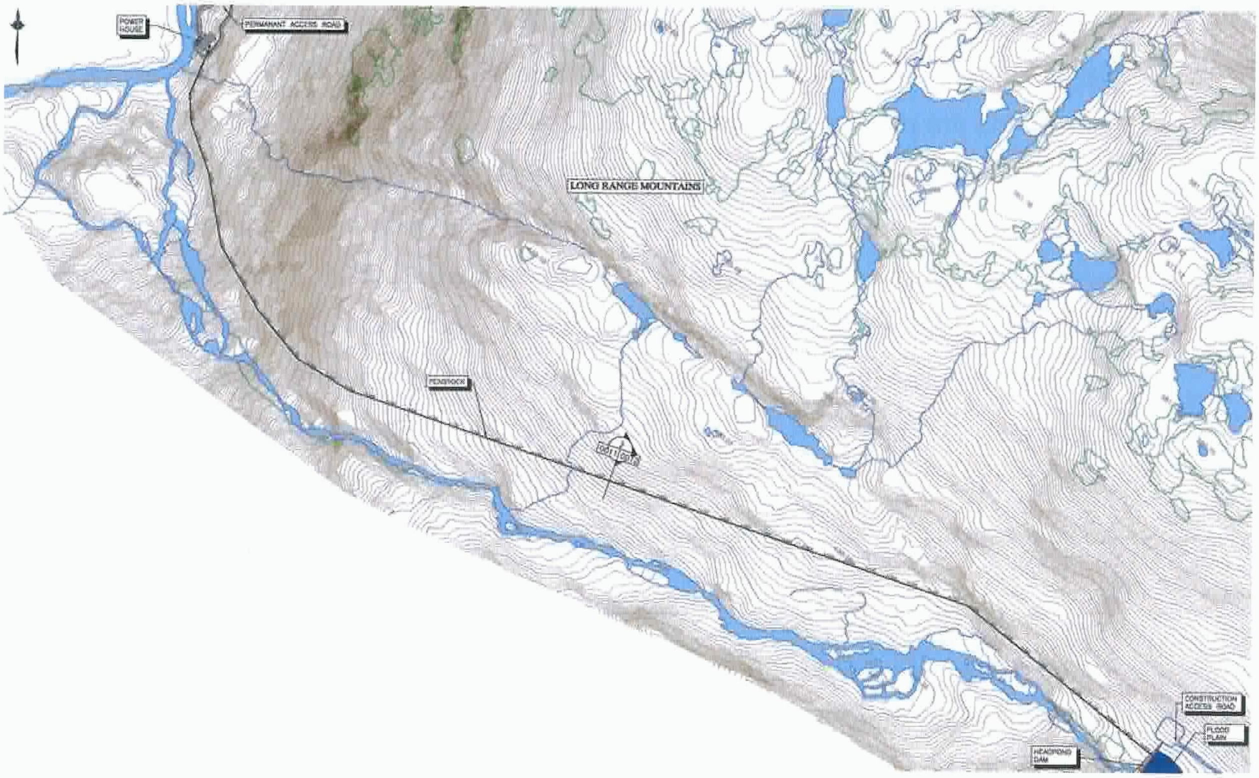
The subsurface conditions expected along the Penstock route are as follows:

- Between km 0.0 to km 2.1 km there is very little overburden with minor colluvium and glacial till in shallow valleys between the rock ridges. Bog has also developed in the shallow valleys;
- Between km 2.1 to km 2.5 the natural soil is practically nonexistent with some accumulations of colluvium in the form of talus at the bases of some of the steep cliffs along the route. Many large detached or semi-detached slabs of rock were noted in the nearby streambed of Main Port Brook. These are interpreted as material produced from detached blocks along exfoliation joints that are near parallel to the rock surface in the area;
- Between km 2.5 to km 2.9 the natural soil will range from talus at the toe of slope to relatively thick glaciofluvial sand, gravel, cobbles and boulders for most of this distance, then back to bedrock near the powerhouse. The glaciofluvial soil is expected to exist from approximate elevation 100 m down to 40 m.[8]

A map of the Penstock route is shown in Figure 3-7.

[8] Ibid

Figure 3-7: Map of Penstock Route



3.4.5 Powerhouse Site

The conditions at this site were visually examined only, both on the ground and through air photo interpretation. At the time of the fieldwork, it was expected that additional investigations would be performed, during the design stage, to firm up the visual interpretations. Digging test pits, drilling boreholes and geophysics profiling would be performed at that time.

Just upstream of the powerhouse location, bedrock outcrop was exposed on the right bank of Portland Creek. Its composition was megacrystic, granite with bands of quartz monzonite. All rock examined were hard, crystalline and strong. A very weak curved, gneissic fabric with general strike of 140° AZ and near vertical dip was visible in some outcrops. Several wide spaced joint sets were recognized. The most prominent had strike of 174° AZ and dip of 44° W. Other wide joint sets were: strike 126° AZ, dip 72° SE; 082° AZ, dip vertical; and, 090° AZ, dip 65° SE. Exfoliation jointing was prominent in the outcrop in the river with two joints recognized: one at 1.8 m; and another at 4.0 m beneath the surface of the rock knob.

The upland area immediately adjacent to Portland Creek comprises a series of terraces incised into a large deposit of glaciofluvial material. Within Portland Creek Valley, seven distinct terraces were recognized through air photo interpretation. Not all terraces are present everywhere in the Valley. West of the powerhouse, only four of the terraces were recognized. A narrow flood plane approximately 3 m above the present river level is the lowest at approximate elevation 42 m ± 2 m. Two narrow terraces were present at about 8 m and 18 m above Portland Creek at approximate elevations of 50 ± 2 m and 60 ± m respectively. The fourth terrace at the site was at approximate elevation 87 ± 2 m. Another higher terrace is present at and just southwest of Main Port Brook where it enters Portland Creek at approximate elevation of 100 ± 2 m. This terrace was not recognized near the powerhouse; however, glaciofluvial soil may exist up to elevation 100 m along the Penstock.

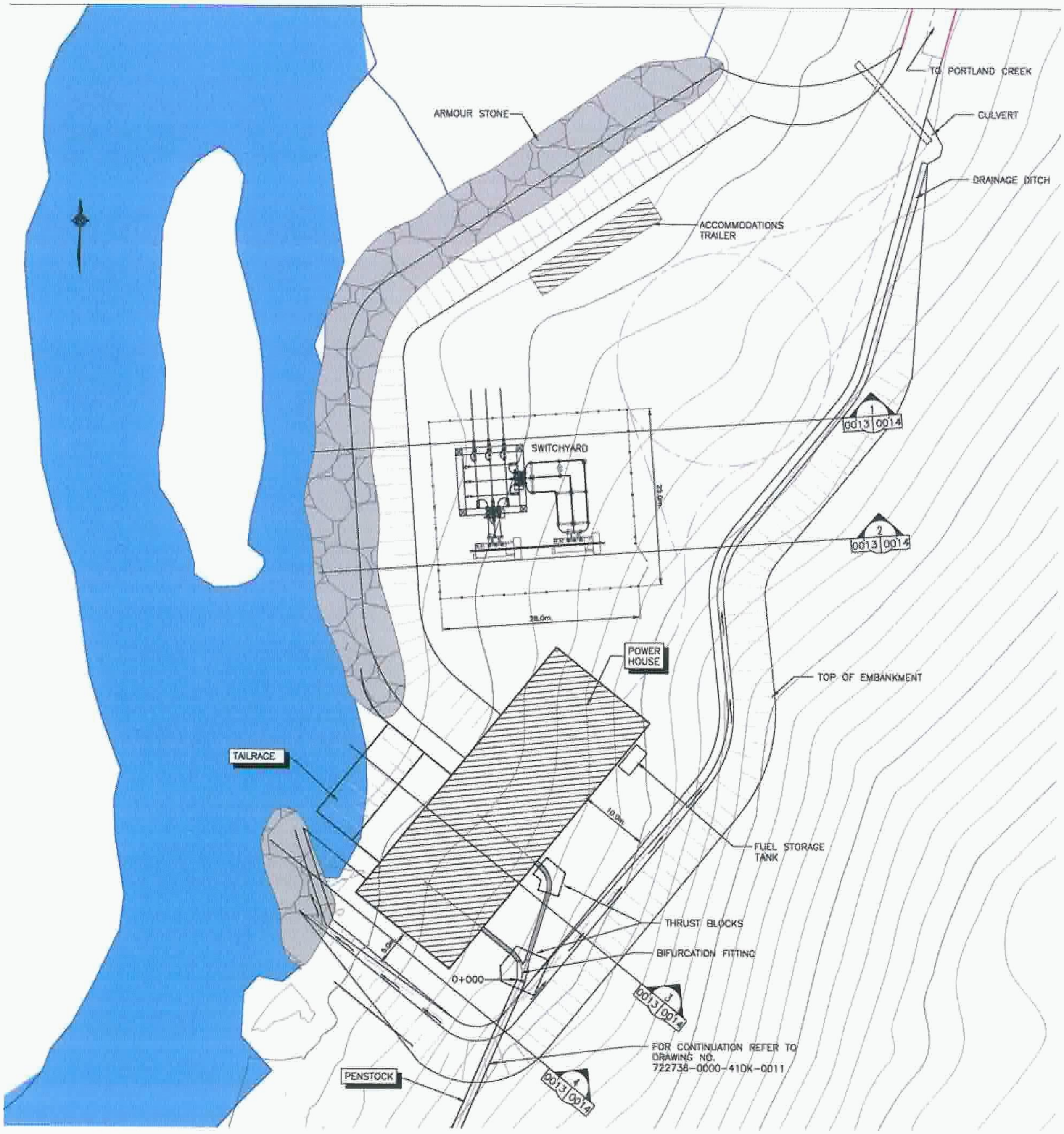
Based on observations of the material incised by the present Portland Creek channel, the material in the glaciofluvial terraces is a mix of sand, gravel, cobbles and boulders with a trace of fines. Most particles were well rounded to sub-rounded. Measured natural slope angles to the horizontal varied from 25° to 36° with an average of 31°. Soil creep was evident on all slopes observed in the glaciofluvial soil.

Just east of the proposed powerhouse location small amounts of groundwater were observed running out of the hillside at the toe of slope. The apparent gradation of the in situ soils would allow any water entering the ground to infiltrate and descend rapidly until either a less permeable layer is encountered or the water entering is more than can be released and groundwater level rises. At this particular location, it is interpreted that, as the bedrock surface rises into the area, through which excess water cannot penetrate, thus it flows horizontally and is released into the slope.[9]

A map of the powerhouse site follows in Figure 3-8.

[9] Ibid

Figure 3-8: Map of Powerhouse Site



4 HYDROLOGY

4.1 DRAINAGE AREA

The centroid of the combined main and the diversion drainage basins is located at 50°06'05" N, 57°14'06" W on the plateau of the Northern Peninsula at an average elevation of 550 m.

The drainage areas of the Main and Diversion Basins were derived from an AutoCAD map having 50 ft contour lines.

Figure 4-1 shows the location of the drainage basins, while Table 4-1 summarizes the drainage areas.

Figure 4-1: Location Map

Table 4-1: Drainage Areas

Drainage Area (DA)	Area (km^2)
Main Basin	58.0
Between Storage Pond and Intake	2.1
Diversion Basin	46.2
Project Drainage Area	**106.3**

4.2 SELECTION OF FLOW DATA

Greavett Brook above Portland Creek Pond (02YE001) was selected to represent the flow regime of Portland Creek. It has a drainage area of 95.7 km^2 and is located at 50°9'37" N, 57°34'45" W, about 19 km west of the Project drainage area – centroid to centroid.

The following are the reasons for the station selection:

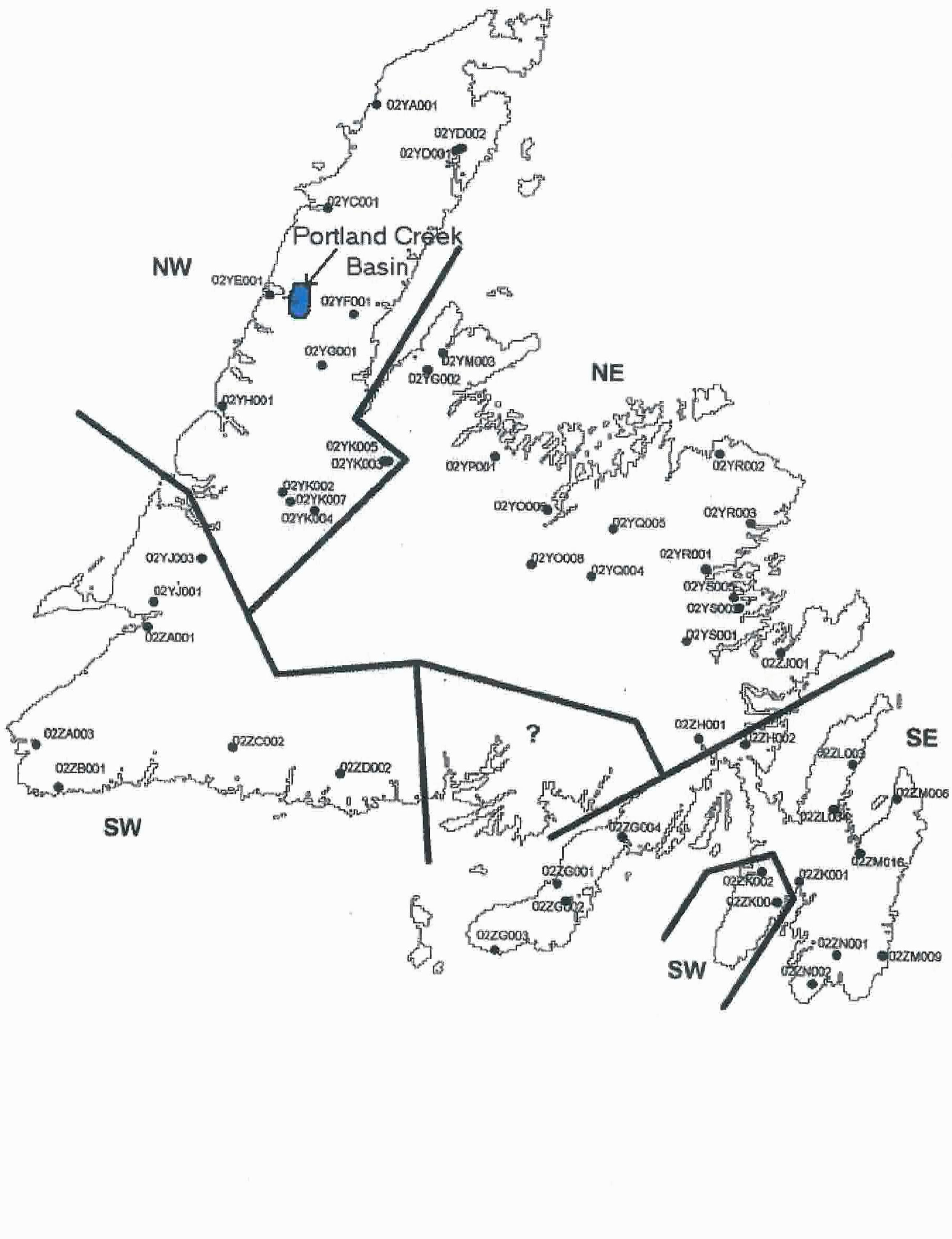
- Its drainage basin is located close to the Project's drainage basin;
- Its drainage area is of the same order of magnitude (106.3 km^2 vs 95.7 km^2);
- It has the same orientation (flows from East to West);
- It has similar physiographic characteristics;
- The river flow is not regulated;
- It has 22 years of daily records (1984 – 2005).

The second nearest station is Cat Arm River above Great Cat Arm (02YF001). It was used for assessing the power potential in the 1987 study. At that time, it was the nearest available station with a sufficient period of records. However, its drainage area is about three times the size of Portland Creek's and is oriented West-East. Greavett Brook Station has 22 years of records; 1984-2006. In temperate humid climates such as insular Newfoundland, a 20+ year flow record is generally considered adequate for evaluation of energy potential and estimation of design floods.

Figure 4-2 shows the geographical distribution of the available hydrological stations on the island of Newfoundland[10].

[10] Regional Water Resources Study, Humber Valley and Northern Peninsula, Acres, Department of Environment and Lands Water Resources Division (1999).

Figure 4-2: Hydrological Stations



4.3 RECONSTITUTION OF LONG-TERM INFLOW AT THE PROJECT SITE

The computed annual runoff of the selected station (Greavett Brook) is 1545 mm.

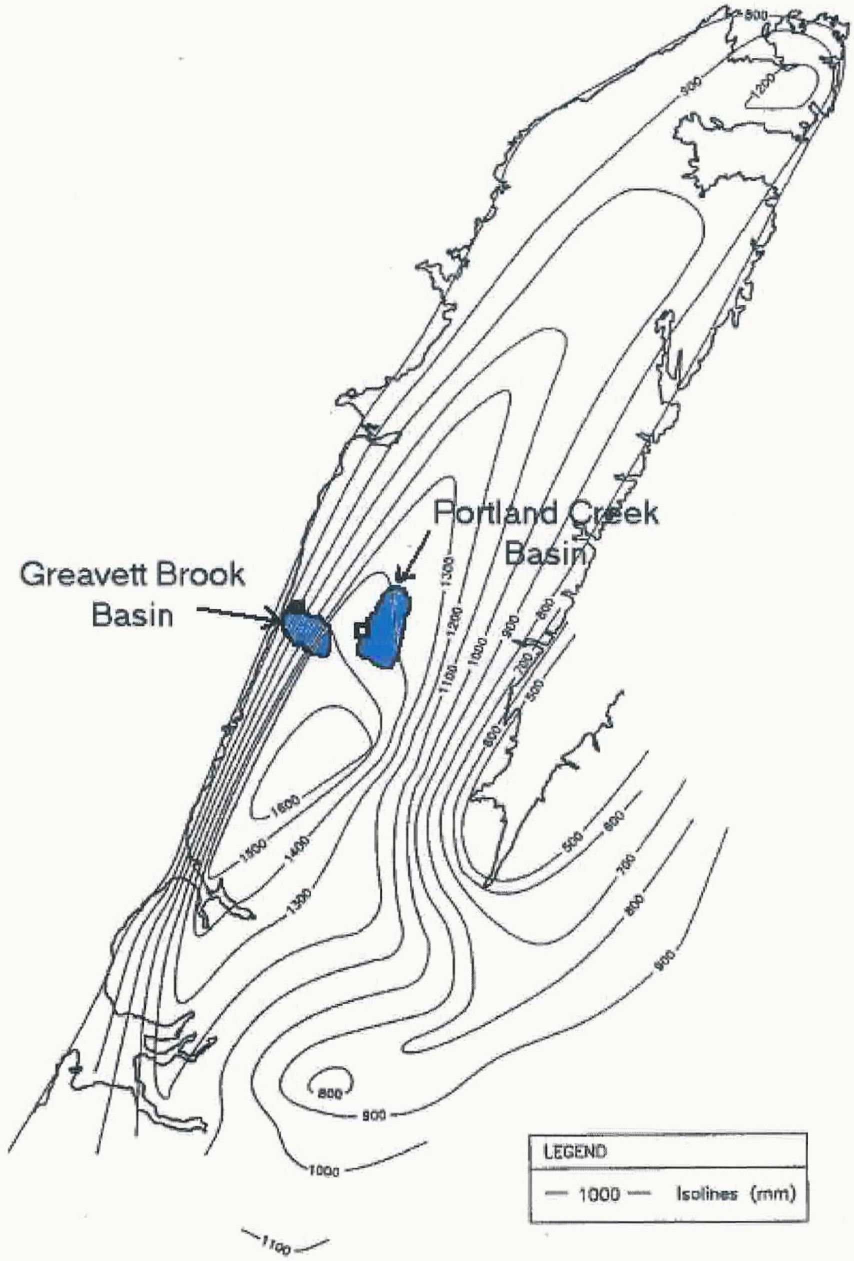
Figure 4-3 shows isolines of mean annual runoff derived by Acres[11] for the Northern Peninsula.

This figure indicates annual runoffs of 1,400 mm for Portland Creek and 1,200 mm for Greavett Brook. Given the scant data base and subjective methods employed in preparing this map, it is recommended, conservatively, that the Greavett data be used in this study pro-rated by drainage area only, i.e., assuming equal unit runoffs.

It is further recommended that NL Hydro consider installing a flow gauge in Portland Creek to confirm the yield of this basin. There is some anecdotal evidence of high precipitation (runoff) in Portland Creek vis-à-vis Greavett Brook – based on observations by SNC-Lavalin's Brad Chaulk who is familiar with the area.

[11] Regional Water Resources Study, Humber Valley and Northern Peninsula, Acres, Department of Environment and Lands Water Resources Division (1990).

Figure 4-3: Isolines of Mean Annual Runoff



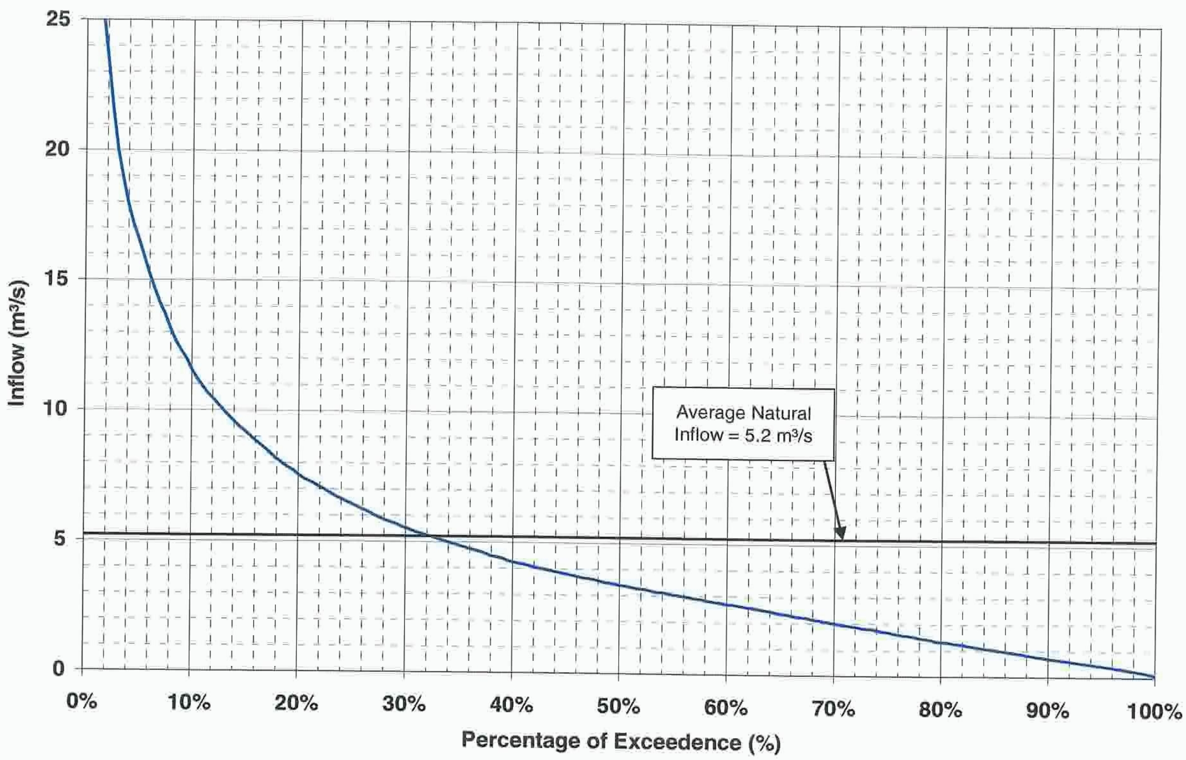
Natural inflow for Portland Creek was derived by prorating the recorded flow at the selected hydrometric station by the ratio of the drainage basin areas.

It is to be noted that the average inflow is 17% higher than the one derived in the 1987 study.

Table 4-2: Average Inflow

	Drainage Area	Source Data	Average Inflow
	km²	-	m³/s
1987 Study	108	Monthly Flow at station 02YF001 (Sept. 68 – Dec. 82) : 14.3 yrs	4.45
2006 Study	106.3	Daily flow at station 02YE001 (Jan. 84 – Dec. 05) : 22 yrs	5.20 (+17%)

Figure 4-4: Portland Creek - Natural Inflow Duration Curve (DA = 106.3 km²)



Note: Based on 1984 – 2005 reconstituted daily flow

This curve shows that the average inflow (5.20 m³/s) is exceeded 32% of the time.

Table 4-3 summarizes mean monthly flows while Figure 4-5 shows the mean annual flow pattern for the river.

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Table 4-3: Project Monthly Flows

D.A. = 106.3 km^2 Ratio Project/ Greavett Brook = 1.111

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
1984	1.40	1.92	5.33	4.73	16.44	16.55	3.78	2.85	3.41	3.75	4.37	6.99	5.96
1985	1.11	0.19	0.48	0.96	13.88	16.55	5.34	1.77	1.79	6.90	3.87	2.14	4.58
1986	4.37	2.43	0.78	12.66	5.64	6.15	1.91	2.41	2.19	4.77	4.31	1.58	4.10
1987	0.32	0.35	1.54	11.22	8.54	4.23	1.52	1.37	4.98	7.30	6.65	2.59	4.22
1988	0.86	0.43	3.53	5.73	13.88	6.19	2.41	2.09	4.65	6.15	7.40	1.90	4.60
1989	0.82	0.48	1.62	5.51	15.44	5.84	3.11	9.57	4.44	6.65	4.70	1.12	4.94
1990	1.73	2.84	1.48	4.35	9.89	13.00	5.86	7.20	5.49	8.92	7.65	8.96	6.45
1991	1.28	1.21	2.99	3.05	11.44	10.71	5.34	2.58	7.86	6.48	4.78	2.33	5.00
1992	1.16	0.64	1.43	2.04	12.33	7.75	2.72	7.28	4.81	7.71	5.75	1.31	4.58
1993	1.08	0.92	0.77	10.10	17.55	6.29	3.54	5.35	9.70	2.69	4.98	3.42	5.53
1994	1.99	0.69	1.80	8.68	12.33	15.44	7.69	4.40	3.74	4.94	9.19	1.39	6.02
1995	1.14	0.32	2.89	7.80	12.11	13.22	4.18	2.89	6.03	5.90	7.42	2.61	5.54
1996	3.87	5.96	4.08	4.91	10.15	6.92	7.70	1.46	2.78	6.53	4.69	5.44	5.37
1997	3.37	1.44	0.44	2.65	14.33	15.00	5.29	4.57	5.25	5.45	5.08	1.66	5.38
1998	1.00	0.69	4.28	5.24	14.33	4.24	3.44	4.31	12.33	5.02	3.13	4.45	5.21
1999	4.64	4.05	3.07	4.07	14.88	4.78	2.49	6.08	4.53	6.12	11.55	4.25	5.88
2000	3.33	1.53	2.88	7.75	13.44	9.74	4.11	1.46	3.79	7.26	3.18	2.97	5.12
2001	1.01	0.22	0.53	4.33	14.77	8.09	3.25	2.79	5.23	4.27	8.93	3.42	4.74
2002	2.13	0.76	1.90	4.39	11.44	10.45	4.39	5.09	7.95	4.70	8.34	1.90	5.29
2003	2.54	3.92	1.57	9.63	12.22	9.11	5.93	2.80	2.54	6.05	4.68	5.43	5.54
2004	2.21	0.80	0.42	10.33	12.44	5.86	1.73	4.35	9.07	3.20	4.12	5.90	5.04
2005	4.12	2.58	2.28	4.47	13.22	3.69	3.32	3.55	6.43	7.73	6.04	3.84	5.11
Means:	2.07	1.56	2.09	6.12	12.76	9.08	4.05	3.92	5.41	5.84	5.95	3.44	5.19

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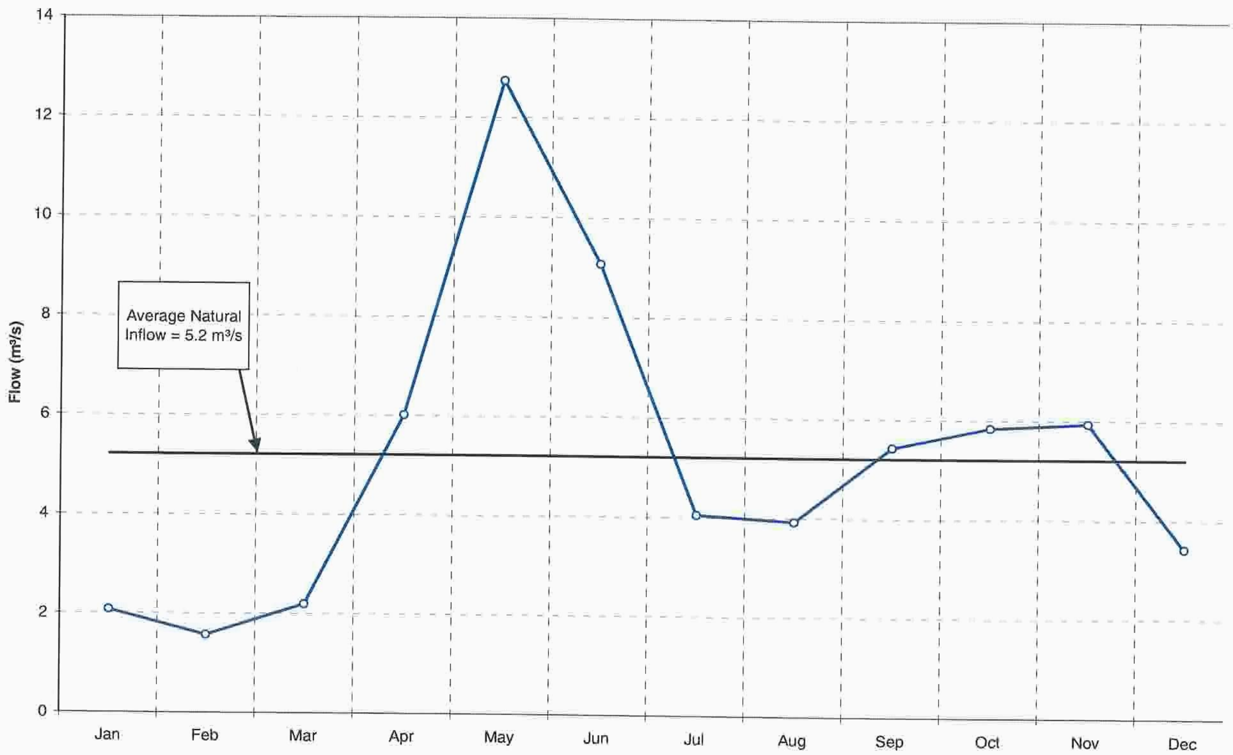
Figure 4-5: Portland Creek - Average Monthly Natural Inflow (DA = 106.3 km²)



Note: Based on 1984 – 2005 reconstituted daily flow

The lowest flows occur in the winter (January to March) and are about a third of the mean annual flow. This low flow period is followed by high spring flows, which may continue from April to June. The flow is typically highest in May, at which time the flow is typically two to three times the annual average.

4.4 FLOOD ANALYSIS

In order to carry out the flood analysis, it was decided to use the statistical method. The annual maximum instantaneous peaks at the selected station (02YE001: Greavett Brook) for the period 1984 – 2005 were obtained from the Environment Canada's website and are presented in Table 4-4.

Table 4-4: Instantaneous Peaks

Year	Date	Recorded Maximum Daily Flow (m^3/s)	Recorded Instantaneous Peak (m^3/s)	Peak Ratio	Selected Instantaneous Peak (m^3/s)
1984	3-Jun	58.0			68.3*
1985	19-May	39.9	54.8	1.373	54.8
1986	24-Apr	25.8	29.9	1.159	29.9
1987	22-Apr	30.2	34.9	1.156	34.9
1988	28-Mar	32.5	35.1	1.080	35.1
1989	8-Aug	35.6	39.7	1.115	39.7
1990	19-Aug	32.0	37.1	1.159	37.1
1991	13-May	22.5	24.5	1.089	24.5
1992	16-May	44.5	50.1	1.126	50.1
1993	7-May	42.6	46.6	1.094	46.6
1994	8-Jun	44.4	48.2	1.086	48.2
1995	9-Jun	55.3			65.1*
1996	26-Feb	29.5			34.7*
1997	20-Jun	29.3	36.1	1.232	36.1
1998	4-May	45.6	52.5	1.151	52.5
1999	28-Nov	39.6	57.3	1.447	57.3
2000	19-May	51.0	70.8	1.388	70.8
2001	14-May	35.7	38.4	1.076	38.4
2002	31-May	31.6	34.3	1.085	34.3
2003	1-Apr	58.3	66.2	1.136	66.2
2004	15-Apr	33.5	42.9	1.281	42.9
2005	28-Sep	29.6	33.4	1.128	33.4
Average		38.5	43.8	1.177	45.5

**Obtained by multiplying the maximum daily flow by the average peak ratio*

Four statistical distributions were applied (Log-Pearson 3, Normal, Log-Normal and Gumbel). The Log-Pearson 3 distribution was selected because it best fits the observed records. The results are presented in Figure 4-6 and Table 4-5.

Figure 4-6: Greavett Brook - Instantaneous Peak - Statistical Distributors

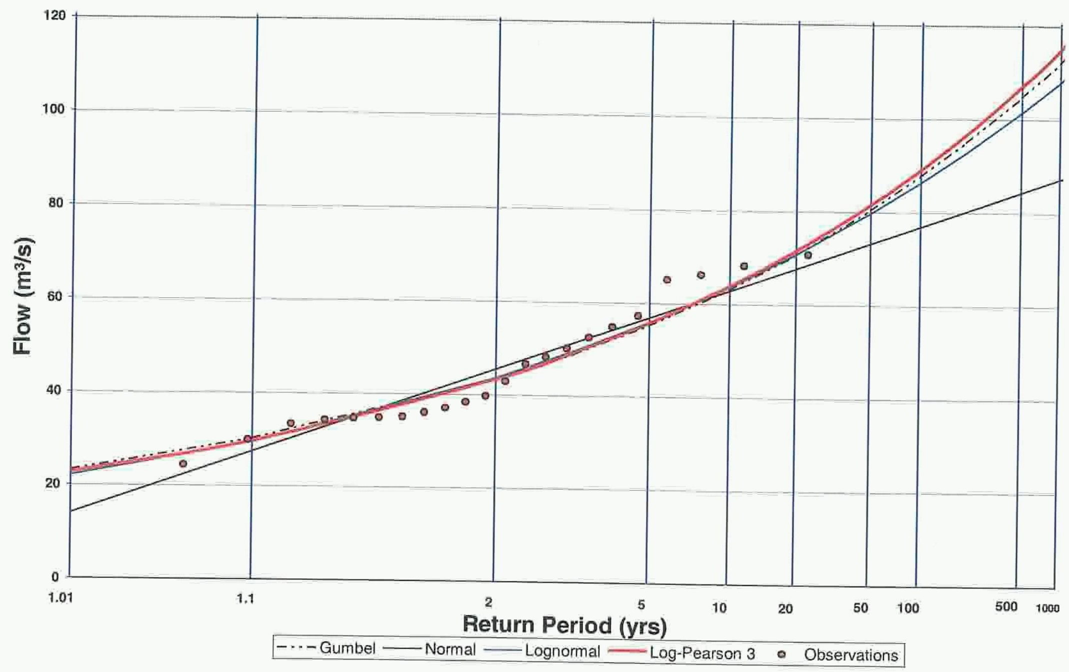


Table 4-5: Flood Flow Estimation

	Greavett Brook	Portland Creek
DA (km^2)	95.7	106.3
Return Period (yr)		
1000	115	130
100	90	100
15	70	75
Mean (1984 – 2005)	45	51

4.5 SELECTION OF THE DESIGN FLOOD

Flood flow will be discharged at the overflow spillway located at Diversion Dam via Diversion Brook, into Inner Pond and from there into Portland Creek Pond. Since the downstream flood zone is unoccupied consequences from a possible dam failure would be minimal. According to the CDA[12], either the 100-yr or 1000-yr criteria can

[12] Dam Safety Guidelines (Canadian Dam Association, 1999)

be selected as the design flood. The 1000-yr design flood was selected for the Project; it was estimated to be 130 m^3/s. The estimated construction flood is 75 m^3/s for a return period of 1 in 15 years.

Details of reservoir volumes and flooded areas are presented in Section 6-7 – Optimization of Reservoir Volume.

5 HYDRAULIC DESIGN

5.1 HEADLOSSES

The head losses in the penstock(s) were computed as the sum of frictional losses and form losses.

Friction losses were computed based on Manning's equation. The selected Manning's coefficients (n) for both steel and polyethylene pipes were 0.011. The following formula, derived from Manning's equation was used for calculation of friction losses:

$$h_f = L \times \left(\frac{V \cdot n}{R^{2/3}} \right)^2$$

Where:

h_f is the friction losses in m;

L is the penstock length in m;

V is flow velocity in m/s;

n is the Manning's coefficient;

R is the hydraulic radius in m.

Form losses at inlet bends, contractions and bifurcation were completed using the following equation:

$$h = K \frac{V^2}{2g}$$

K values were selected from SNC-Lavalin's files.

5.2 SPILLWAYS

The spillway discharge rating curve is given by the following equation[13].

$$Q = C\ L\ H^{3/2}$$

Where:

Q is the discharge over the spillway in m^3/s;

C is the discharge coefficient;

L is the length of the spillway in m;

H is the head above the crest in m.

The discharge coefficient of the ogee spillway is 2.2.

5.2.1 Spillway at Diversion Dam

The spillway length is 70 m. Figure 5-1 shows the rating curve of the spillway.

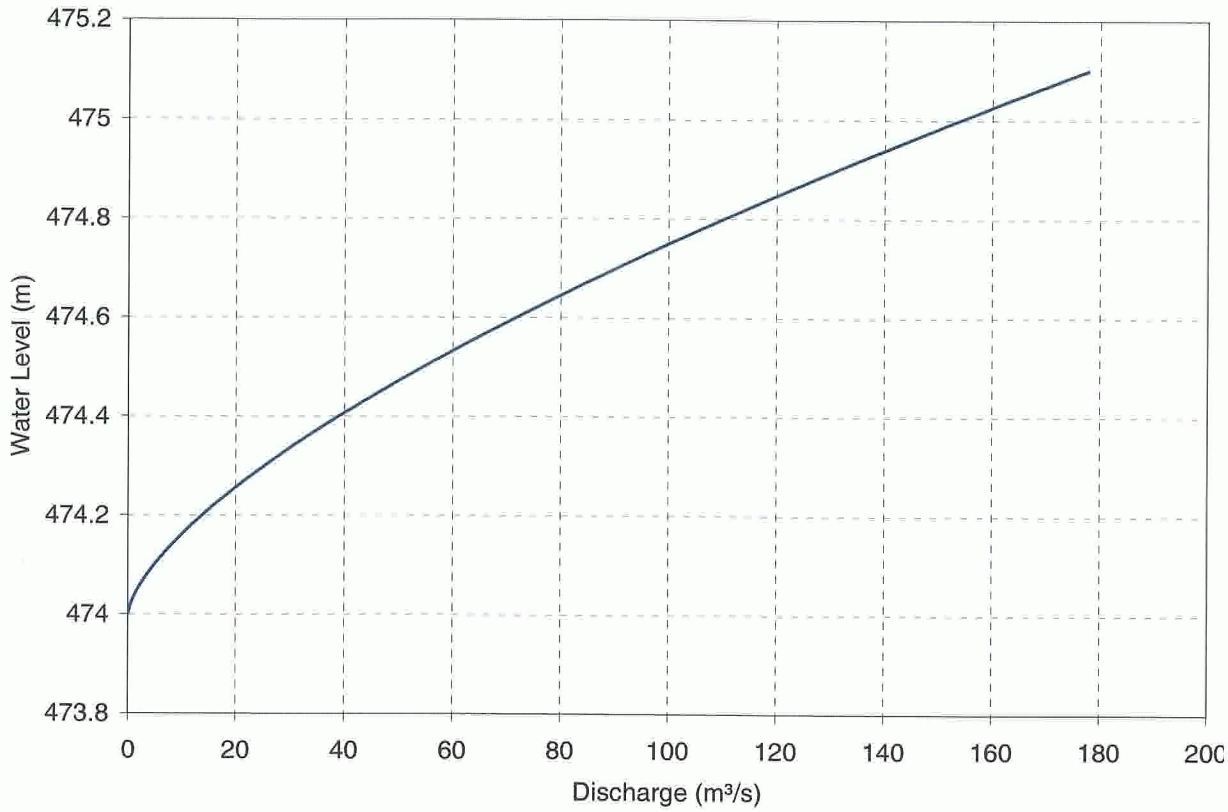
Water level would rise to elevation 474.89 m during the 1000-yr flood (130 m^3/s).

See calculation below.

$$\begin{aligned} Q &= C\ L\ H^{3/2} \\ &= 2.2\ x\ 70\ x\ (474.89 - 474.00)^{3/2} \\ &= 130\, m^3/s \end{aligned}$$

[13] "Open Channel Improvements" by Van T. Chow, McGraw-Hill Brook Company, New York, 1959

Figure 5-1: Diversion Dam Spillway: Rating Curve



5.2.2 Spillway at Headpond Dam

The spillway at the Headpond Dam is designed to pass the sum of the two following flows:

- The 1000-yr flood flow from the 2.1 km^2 additional catchments between the Main Reservoir and Headpond Dam: 3 m^3/s;
- The maximum outflow from the reservoir, i.e. 7 m^3/s.

The spillway length is 30 m; its crest elevation is at 458 m. Water level will rise to elevation 458.3 m when the outflow is 10 m^3/s, as below:

$$
\begin{aligned}
Q &= C\ L\ H^{3/2} \\
&= 2.2\ x\ 30\ x\ (458.3 - 458.0)^{3/2} \\
&= 10.8 m^3/s
\end{aligned}
$$

5.3 INTAKE (AT HEADPOND DAM)

The design submergence was designed according Gordon's criteria:

$$S = 0.54\ V\ D^{1/2}$$

Where:

S is the minimum submergence in m;

D is the height of the gate in m (= 2.0 m)

W is width of gate (= 1.6 m)

V is velocity at gate section (= 2.06 m/s)

Whence

$$
\begin{aligned}
S &= 0.54\ V\ D^{1/2} \\
&= 0.54\, x\, 2.06\, x\, 2.0^{1/2} \\
&= 1.57\, m
\end{aligned}
$$

If LOL	=	*457.3*	
Sill	=	*457.3 – 1.57 – 2.0*	
	=	*453.73 m*	*use 453.40 m*

5.4 WATERHAMMER DESIGN

A preliminary waterhammer design was derived for input to penstock layout studies and penstock strength design, as follows:

- Pressure rise or full load rejection was set at static head (at unit) plus 15% based on experience at Cat Arm. This assumes use of deflectors to divert jet flows from

Pelton runners in order to mitigate generator over speeds; while, jet valves would be closed slowly to control waterhammer. Valve closure times, in the order of 27 seconds, were estimated.

- Pressure drop on load additions was set at 20% of static head (at unit). This is based on loading units sequentially, at an equivalent valve opening time of 10 s.

The double action of a Pelton turbine on load rejection permits loads to be rejected rapidly via deflector deployment, while load additions at the rate of 1.15 MW/s are permissible without undue pressure drops in the penstock.

A rule-of-thumb suggests that a surge tank or other device for control of waterhammer may be required when the ratio of penstock length/head exceeds four to eight. For Portland Creek this ratio was found to be seven, indicating the design to be marginal with respect to control of waterhammer and (related) generator overspeed. However, the jet deflectors provide the same service as bypass valves and protect against both excessive waterhammer pressure increase and generator overspeed on load rejection; therefore a surge tank is not required.

5.5 FREQUENCY REGULATION

A preliminary assessment was made of the frequency regulation capability of the plant, using an empirical method developed by T.L. Gordon[14]. This method classifies frequency regulation as a function of water starting time (T_w), mechanical starting time T_m and governor closing times (T_e and T_g). Further Gordon established a classification system as a function of a parameter K, where:

$$K = \frac{T_m}{T_g\left(1 + T_w \cdot T_e^{-1}\right)}$$

[14] "Generator Inertia for Isolated Hydropower Systems", T.L. Gordon & D.H. Whitman, Canadian Journal of Civil Engineering, Vol. 12, No. 4, 1985.

Considering two units in operation:

T_m = *9.52 s (normal generator inertia)*
= *19.0 s (2 x normal generator inertia)*
= *28.5 s (3 x normal generator inertia)*
T_w = *2.72 s*
T_e = *19.5 s (effective governor time)*
T_g = *21.0 s (total governor time)*

Whence:

Assumptions	K	Regulation Capability	
Normal Generator Inertia	0.40	K<0.55	Nil
2 x Normal	0.79	0.55<K<0.82	Regulation on large systems
3 x Normal	1.19	K>1.10	Regulations on small systems medium/large load changes

This analysis concluded that the proposed design, intended primarily to produce energy, would not contribute significantly to system frequency regulation. However, NL Hydro may consider designs with additional inertia (addition of flywheels) to enhance frequency regulation capability. It is recommended that this question be reviewed during final design at which time more refined analyses could be employed.

6 LAYOUT STUDIES AND OPTIMIZATION

6.1 INTRODUCTION

This section summarizes the studies undertaken to develop an optimal Project concept, including the optimization of key Project components.

6.2 APPROACH

Layout studies were undertaken in which a variety of alternative layouts or component layouts were assessed with the objective of developing an optimal conceptual design; taking into account site characteristics, constructability, operating concerns and the like. Optimal plant features were decided either on the basis of economic analysis or by engineering judgement taking into account site constraints and relevant SNC-Lavalin experience on similar projects.

6.3 ECONOMIC PARAMETERS

NL Hydro provided the following economic factors for input to optimization analyses:

- Energy at $0.70/kWh;
- Capacity at $1,144/kW, based on the use of gas turbines for peaking;
- Service life of Project – 60 years;
- Discount rate – 8.4%;
- Current interest rate for computation of interest during construction – 7.8%.

Using the above parameters, the value of 1.0 m of head loss was calculated to be $318,000.

6.4 ACCESS ROADS

Various access road routings were investigated considering alternative starting points and routes from the powerhouse to upstream structures. Routings considered topography and ground conditions, as assessed from 1:50,000 topo maps and aerial photos. Further details on the alternative studies and the selected permanent and construction access road routes are given in Section 8.1.

6.5 WATERWAY LAYOUT

The 1987 Portland Creek Pre-Feasibility Study proposed separate storage and Headpond Dams. This concept was reviewed considering the following alternative arrangements:

- Alternative 1 – "Basic Arrangement"

  This alternative assumes separate Headpond and Storage Dams as in the 1987 study.

- Alternative 2 – "High Headpond Dam".

  In this alternative the crest of the Headpond Dam would be raised to 476.0 m to control the water level in the reservoir. This would permit elimination of the storage dam.

- Alternative 3 – "Extended Penstock"

  In this alternative the penstock would be lengthened by 1,500 m to an intake at the Storage Dam. This would permit the elimination of Headpond Dam and exploitation of the head difference between Headpond Reservoir and Storage Reservoir.

Preliminary layouts and order-of-magnitude costs were prepared for each of these alternatives. Comparison of these cost estimates showed:

1. The increase in cost of a high Headpond Dam was much greater than the savings obtained by elimination of the Storage Dam.
2. The cost of lengthening the Penstock was found to be much greater than the savings obtained by eliminating the Headpond Dam, even after considering the

benefits of increased head. The benefits of increased head were found to be substantially reduced due to additional friction losses in the penstock. Additionally, the mean gross head would be based on the mean water level in the reservoir, substantially below the Full Supply Level (FSL) of 474.0 m. This exercise confirmed the merits of the selected layout – Alternative 1.

6.6 OPTIMIZATION OF CAPACITY

Approach

The economic analysis involved comparison of benefits and costs for a range of assumed plant flow capacities (and plant installed capacities). In all, five alternative project capacities were considered.

Determination of Benefits

A regulation model was set up in Excel to simulate operation of the plant. The plant was operated as a run-of-river plant with temporary storage, with the primary objective of maximizing energy output (or minimizing spills). A secondary objective was to ensure sufficient firm flow to permit operation of the plant at full capacity for four hours daily, while maintaining a minimum flow via a single jet at 30% open for the remaining 20 hours. (This assumes a plant configuration of two 2-jet horizontal axis Pelton turbines or one vertical axis 4-jet Pelton turbine). With this operating mode, the full plant capacity can be credited as firm capacity. To ensure this requirement, rule curves were developed for each alternative. The model simulation was done in monthly steps for 21 years from October 1984 to September 2005, inclusive. "Optimum" penstock diameters were computed using Falbusch's formula + 10%. Energy output was calculated each month, taking into account hydraulic losses in the penstock as a function of Q^2.

Benefits were calculated using economic factors provided by NL Hydro, as noted in Section 6.3. Benefits were also evaluated for alternative energy at $1.40/kWh for input to sensitivity analyses.

The results of these regulation studies are summarized in Table 6-1.

Table 6-1: Summary of Regulation Study

Description	Units	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5
Plant Flow Capacity (Q_P)	m^3/s	4.45	5.20	6.50	8.00	10.00
Capacity (at generator terminals)	MW	14.6	17.1	21.5	26.7	33.5
Firm Capacity (at generator terminals)	MW	14.6	17.1	21.5	26.7	33.5
Minimum Flow	m^3/s	0.33	0.39	0.49	0.60	0.75
Firm Flow	m^3/s	1.02	1.19	1.49	1.83	2.29
Turbinable Flow	m^3/s	4.22	4.62	4.90	4.99	5.11
Spill	m^3/s	0.94	0.54	0.25	0.17	0.05
Total Flow	m^3/s	5.15	5.15	5.15	5.15	5.15
Firm Capacity delivered	MW	14.3	16.8	21.1	26.1	32.8
Energy at generator	GWh	116.7	134.2	144.2	149.2	154.5
Energy delivered	GWh	112.0	128.8	138.4	143.2	148.3
Firm Plant Load Factor		22.9%	22.9%	22.9%	22.9%	22.9%
Average Plant Load Factor		95.1%	88.3%	76.5%	63.8%	52.7%

BENEFITS

..capacity at $ 1,144 / kW		$16 364 989	$19 212 633	$24 104 080	$29 908 118	$37 501 464
..energy at $ 0.70 / kWh		$78 422 400	$90 182 400	$96 902 400	$100 262 400	$103 824 000
..energy at $ 1.40 / kWh		*$156 844 800*	*$180 364 800*	*$193 804 800*	*$200 524 800*	*$207 648 000*
Basic with energy at $ 0.70 / kWh		$94 787 389	$109 395 033	$121 006 480	$130 170 518	$141 325 464
Alternative with energy at $ 1.40 / kWh		*$173 209 789*	*$199 577 433*	*$217 908 880*	*$230 432 918*	*$245 149 464*

Notes:

1/ Plant operated as run-of-river with temporary storage.
2/ Minimum flow based on one jet at 30% opening.
3/ Firm flow determined to provide 20 hrs at Q_{min} + 4 hrs at Q_P in the most severe low flow period.
4/ Optimum penstock diameters estimated by Falbusch's formula.
5/ Plant capacity calculated assuming turbine effy. = 90% and generator effy. = 98%.
6/ Plant delivered capacity includes plant effy = 99% and transformer effy = 99%.
7/ Plant delivered energy includes plant effy = 99%, transformer effy = 99% and water utilization factor = 98%
8/ Water utilization factor corrects for assumptions in model, outages and off best efficiency operation.
9/ Reservoir FSL = 474.0 and LSL = 462.0 m with live storage = 23,400,000 m^3 (= 8.90 m^3/s.mos).

Determination of Relative Capital Costs

The significant costs in this analysis were costs that vary as a function of plant capacity, notably:

- Powerhouse, including water to wire equipment;
- Penstock;
- Intake;
- Control Structure;
- Switchyard.

Relative costs were determined using parametric equations and quantity take-offs, as appropriate. Unit prices for civil works, structural steel, penstock steel and gate steel were based on recent Newfoundland experience. Cost of water-to-wine equipment was estimated using a computer program developed by J.L. Gordon, assuming a 2005 - 2006 price escalation of 6%.

The resulting relative capital cost estimates are shown in Table 6.2.

Economic Analysis

Economic Analysis examined Project lifetime costs and benefits expressed in present value terms, as derived from the economic parameters provided by NL Hydro, as in Section 6.3. Net benefits (B-C), benefit cost ratio (B/C) and incremental benefit cost ratios ($\triangle B/\triangle C$) were determined for each alternative and for several cases, so as to assess the sensitivity of the selected alternative to plausible variations in the basic assumptions. The cases studied were:

Case 1 - Basic Case including both energy and capacity benefits;

Case 2 - Considering Energy Benefits only at $0.70/kWh as stipulated by NL Hydro;

Table 6-2: Summary of Relative Capital Costs

STRUCTURE	COSTS ($ x 10^3)					REMARKS
	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	
INSTALLED CAPACITY (MW)						
POWERHOUSE						
.. water-to-wire equipment						
...erection of equipment						
Electrical:						
...remainder of plant services						
Mechanical:						
..shut-off valves (2)						
..P/H crane						
..draft tube stoplogs						
..mechanical auxiliaries						
Civil:						
..excavation (rock + overburden)						
..substructure concrete						
..structural steel						
..building envelope						
..architectural features						
SUBTOTAL POWERHOUSE	***$8 987***	***$10 355***	***$13 185***	***$15 467***	***$19 925***	
PENSTOCK						
..pipe supply and install						
..earthwork						
..corrosion protection						
SUBTOTAL PENSTOCK	***$7 149***	***$7 802***	***$8 813***	***$9 972***	***$11 272***	
INTAKE						
..common items						
..service gate						
..bulkhead gate						
..trashrack						
SUBTOTAL INTAKE	***$1 038***	***$1 047***	***$1 059***	***$1 074***	***$1 095***	
CONTROL STRUCTURE	$1 018	$1 021	$1 027	$1 034	$1 044	
SWITCHYARD						
..power transformer						
..common items						
SUBTOTAL SWITCHYARD	***$673***	***$754***	***$897***	***$1 065***	***$1 285***	
INFRASTRUCTURE COSTS						
SUBTOTAL DIRECT COSTS	$18 864	$20 979	$24 981	$28 613	$34 622	
E&M						
Owner's Costs:						
EDC @ 2.4%						
IDC @ 7.53%						
GRAND TOTAL						

Case 3 - Same as Case 2 but with energy valued at \$1.40/kWh;

Case 4 - Same as Base Case but with CCE increased by 20%.

As explained in Gulliver and Arndt[15], the appropriate measure for evaluating the economics of a hydroplant operating as part of a utility system is to determine the point at which the incremental benefit/cost ratio become unity. Beyond this point, $\triangle B/\triangle C<1.0$, power benefits can be obtained more cheaply at an alternative plant.

The results of these analyses are given in Table 6.3

The following observations are noted:

- Optima were not found within the range of alternatives studied for either Case 1 or Case 4 that included capacity benefits. It is thought that this finding results from the definition of firm capacity based on an assumed four-hour peaking operation – perhaps unrepresentative of NL Hydro's system load characteristics.

- Optima were found for Cases 2 and 3 that considered energy benefits only. Case 2 based on energy benefits evaluated at \$0.70/kWh is judged to be the most plausible result giving an optimal plant flow of 6.6 m^3/s, see Figure 6.1. This value is consistent with recent Canadian experience, notably at Paradise River G.S. where the plant flow capacity is approximately 1.5 x Q_m (mean flow) versus Portland Creek with a ratio of 6.6/5.2 = 1.27. The larger capacity values from Case 3 and implied in Cases 1 and 4 are considered implausible. Accordingly, the recommended optimum plant capacity is 6.6 m^3/s. This corresponds to a plant capacity of 23.0 MW, based on the optimized penstock diameter.

[15] Gulliver, J.S., R.E.A. Arndt. Hydropower Engineering Handbook. New York, McGraw-Hill, Inc. New York (1991)

Table 6-3: Summary of Economic Analyses

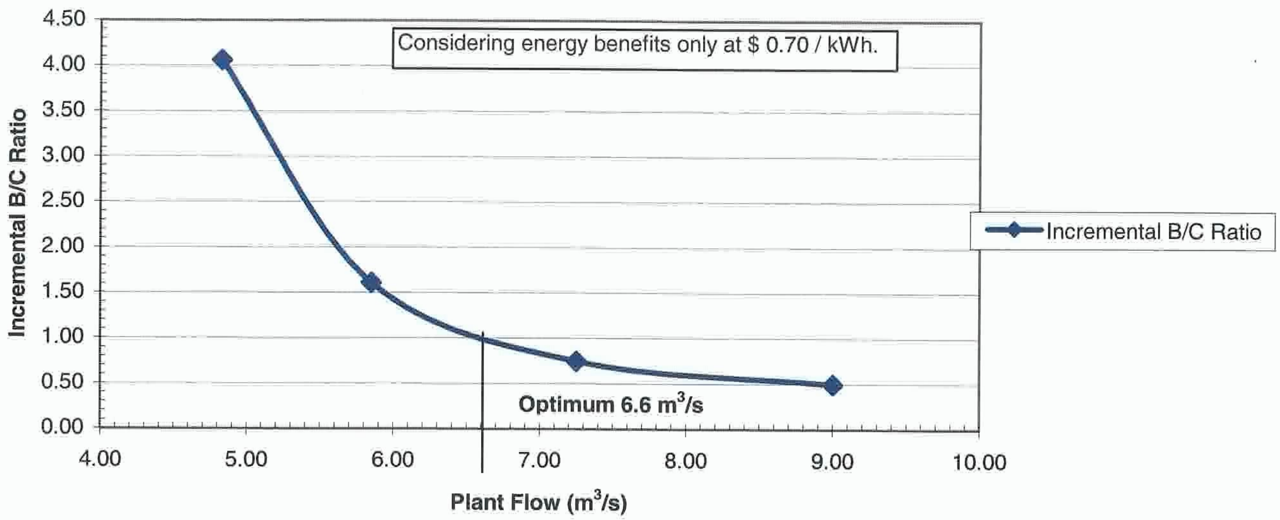
	Analyses	Economic Parameters Capacity (MW)	Alternative 1 14.6	Alternative 2 17.1	Alternative 3 21.5	Alternative 4 26.7	Alternative 5 33.5	Remarks
	BASE CASE							
					($ x 1,000)			
1/	**Benefit**	**Capacity = $ 1144 / kW**	**$94 787**	**$109 395**	**$121 006**	**$130 171**	**$141 325**	**Sum of energy and**
		Energy = $ 0.70 / kWh						**capacity benefits.**
	CCE		$20 977	$23 336	$27 788	$31 828	$38 512	
	PV O&M etc		$5 229	$5 487	$5 924	$6 354	$6 927	**Note PV of annual recurring**
	COSTS		**$26 206**	**$28 823**	**$33 712**	**$38 182**	**$45 439**	**costs are included.**
	B - C		**$68 581**	**$80 572**	**$87 294**	**$91 988**	**$95 886**	
	B/C		**3.62**	**3.80**	**3.59**	**3.41**	**3.11**	
	Incremental B/C			5.58	2.38	2.05	1.54	
	CONSIDERING ENERGY ONLY							
2/	Benefit	Energy = $ 0.70 / kWh	$78 422	$90 182	$96 902	$100 262	$103 824	Energy Benefits only.
	Cost		$26 206	$28 823	$33 712	$38 182	$45 439	
	B - C		$52 216	$60 217	$63 190	$62 080	$58 385	
	B/C		2.99	3.09	2.87	2.63	2.28	
	Incremental B/C			4.06	1.61	0.75	0.49	
	Plant Flow	**(m^3/s)**		4.83	5.85	7.25	9.00	Mid range values
3/	Benefit	Energy = $ 1.40 / kWh	$156 845	$180 364	$193 805	$200 525	$207 648	Energy Benefits only.
	Cost		$26 206	$28 823	$33 712	$38 182	$45 439	
	B - C		$130 639	$151 541	$160 093	$162 342	$162 209	
	B/C		5.99	6.26	5.75	5.25	4.57	
	Incremental B/C			8.99	2.75	1.50	0.98	
	Plant Flow	**(m^3/s)**		4.83	5.85	7.25	9.00	Mid range values
	BASE CASE CCE + 20%							
4/	**Benefit**	**Capacity = $ 1144 / kW**	**$94 787**	**$109 395**	**$121 006**	**$130 171**	**$141 325**	
		Energy = $ 0.70 / kWh						
	CCE	**Base + 20%**	$25 172	$28 003	$33 346	$38 194	$46 214	
	PV O&M etc		$5 229	$5 487	$5 924	$6 354	$6 927	
	COSTS		**$30 401**	**$33 490**	**$39 270**	**$44 548**	**$53 141**	
	B - C		**$64 386**	**$75 905**	**$81 737**	**$85 623**	**$88 184**	
	B/C		**3.12**	**3.27**	**3.08**	**2.92**	**2.66**	
	Incremental B/C			4.73	2.01	1.74	1.30	
	Plant Flow	**(m^3/s)**		4.83	5.85	7.25	9.00	Mid range values

NOTE:

Determination of Annual Recurring Costs

Annual Cost					
O&M per $/kW	*$23*	*$21*	*$17*	*$15*	*$13*
O&M total	*$335*	*$351*	*$375*	*$400*	*$427*
Interim at 0.22%	*$46*	*$51*	*$61*	*$70*	*$85*
Insurance at 0.10%	*$21*	*$23*	*$28*	*$32*	*$39*
Subtotal:	*$425*	*$446*	*$482*	*$517*	*$563*
Cumulative PV:	***$5 229***	***$5 487***	***$5 924***	***$6 354***	***$6 927***

Figure 6-1: Incremental B/C Ratio



6.7 OPTIMIZATION OF RESERVOIR VOLUME

The merits of raising (or lowering) the reservoir full supply level (FSL) were investigated by comparing the cost of raising the FSL from 473.0 m to 474.0 m versus the benefit in improved water utilization (reduced spill). The water use benefit was estimated using a regulation model of the selected scheme in which the reservoir capacity was varied by the volume between 473.0 m and 474.0 m. The energy saving was found to be 1.39 GWh annually. This analysis indicated it would be desirable to raise the reservoir FSL further. However, layout studies on the latest mapping and visual appraisal of site conditions concluded that to raise the FSL above 474.0 m would probably require construction of an additional saddle dam and substantial (disproportionate) lengthening of the Diversion Dam. For these reasons, the design FSL level was fixed at 474.0 m.

At a FSL of 474.0, the flooded areas associated with the Storage Pond and Headpond may be computed as follows:

Storage Pond:

- Future Surface Area - 273.6 ha
- Existing Surface Area - 195.8 ha
- New Flooded Area - 77.8 ha

Headpond:

- Future Surface Area - 41.5 ha
- Existing Surface Area - 26.0 ha
- New Flooded Area - 15.5 ha

6.8 PENSTOCK CONCEPTUAL DESIGN

Routing of the penstock was determined, respecting hydraulic design requirements and taking into account site topography and ground conditions. The resulting penstock profile is characterized as below:

- Along the upper 1800 m the slope is low (about 6%);
- Along the lower 1100 m the slope is steep (about 30%).

Two different penstock arrangements were then studied. In the upper section, it was assumed that a single penstock would be installed. As this penstock section will operate under low head, a polyethylene penstock offered significant economics; while for the lower section steel would be required for strength reasons.

For the Base Case (Case 1), a single steel penstock installed in the lower section was assumed, followed by a bifurcation to the two units at the powerhouse.

Table 6-4: Layout Selection

Case:	1	2
	Base Case	**Parallel Penstocks**
Upper Section	One 63" 1800 m polyethylene penstock	One 63" 1800 m polyethylene penstock
Lower Section	One 60" 1100 m steel penstock	Two 48" 1100 m steel penstock
Natural Inflow (m^3/s)	5.2	5.2
Average Spilled Flow (m^3/s)	0.34	0.34
Flow Capacity (m^3/s)	6.6	6.6
Average Head Water Level (m)	458	458
Turbine (m)	42.6	42.6
Gross Head (m)	415.4	415.4
Maximum Head Losses	18.7	19.6
Rated Net Head (m)	396.7	395.8
Installed Capacity (MW)	23.0	23.0
Annual Energy (GWh)	141.4	141.1

For Case 2, with a view to facilitating the installation of the penstocks in the lower section, the possibility of having two steel penstocks of smaller size was studies. In this case, the bifurcation would be at the downstream end of the polyethylene section.

Case 1 vs Case 2: Number of Penstocks

In the steel section, two penstocks of 48" diameter were assumed. This diameter was determined by an optimization analysis using incremental benefit-cost analysis methodology, as described in Section 6.6.

The comparison shows that having two penstocks in the steel section results in a slightly lower annual energy. Case 2 is, therefore, not recommended.

6.9 PENSTOCK DIAMETER OPTIMIZATION

In order to determine the optimum penstock diameter, the following calculations were carried out for different penstock diameters:

- Compute the hydraulic gradient along the penstock;
- Select steel strength class for each section;
- Determine the required thickness of pipe for each section;
- Estimate the cost of pipes, excavation, earth fill and wrapping;
- Compute the head losses coefficient;
- Evaluate the annual energy with the corresponding head losses.

For polyethylene pipe, the following standard diameters were considered: 40, 42, 48, 54, 55 and 63 inches. Steel strength design assumed CSA G40.2 350WT steel plate and an allowable stress of 166MPa (24 ksi).

The cost of polyethylene pipe was obtained from suppliers. The cost of steel was estimated to be 7000 $ / tonne installed.

Incremental benefit-cost ratios were computed with the same methodology as described in Section 6.6. Figures 6-2 and 6-3 show the results of these analyses.

For the polyethylene pipe, all values of pipe diameter gave benefit-cost ratios higher than one. The selected diameter for the polyethylene section was therefore the largest available standard pipe diameter, which is 63 inches (1.60 m).

A similar analysis was done for the steel section, which found the increment benefit-cost of 1.0 to be between 60" and 61". The recommended optimum diameter for the steel penstock is, therefore, 60" (1.52 m).

Figure 6-2: Incremental BCR vs Diameter - Polyethylene Section

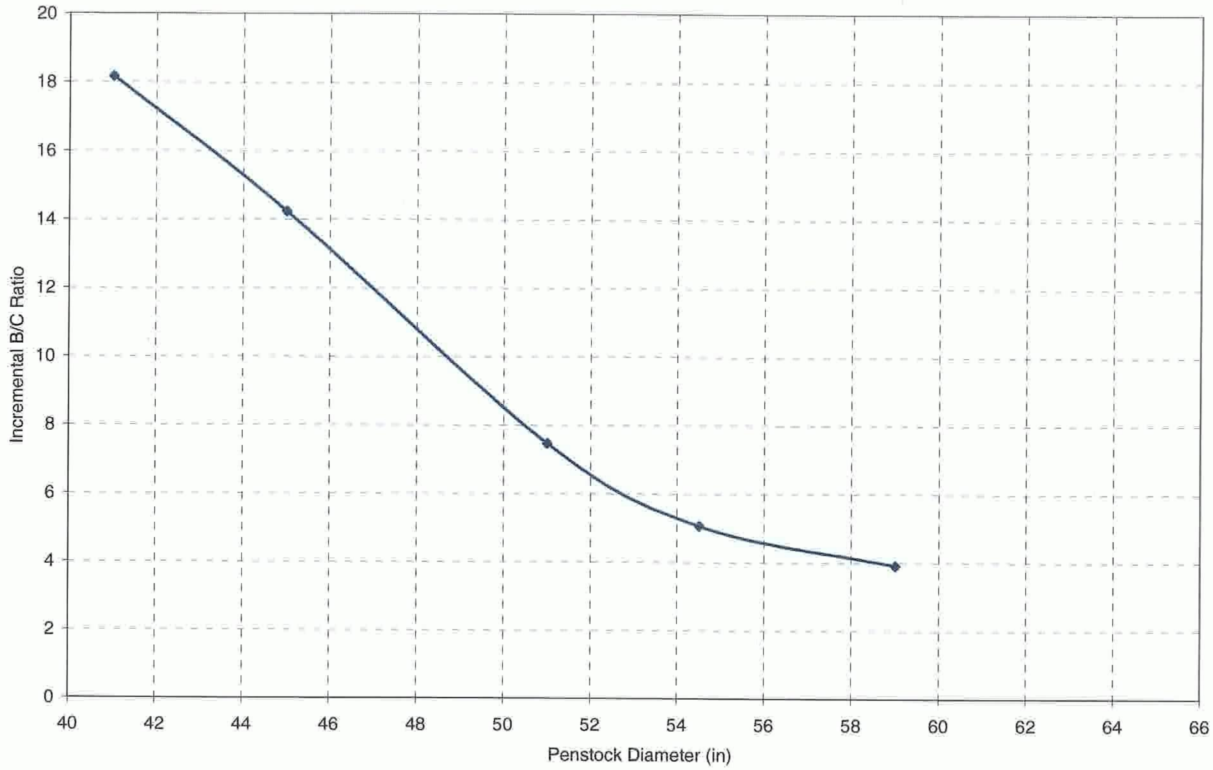
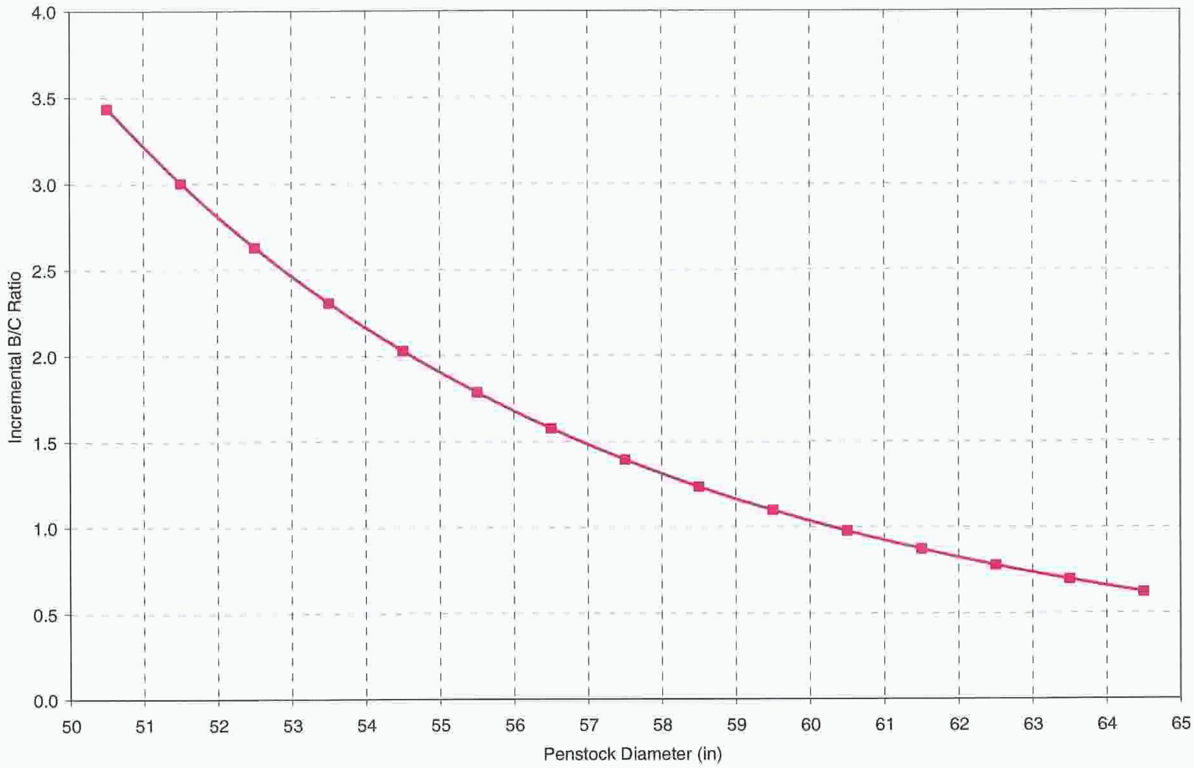


Table 6-2 presents the main results.

Figure 6-3: Incremental BCR vs Diameter - Steel Section



7 POWER POTENTIAL

7.1 APPROACH

This section summarizes the computations carried out to establish the energy and capacity benefits of the Portland Creek Development.

7.2 ENERGY BENEFITS

Energy benefits were determined by means of a regulation model that simulated plant operation on a daily basis for the available period of records; 1984 – 2005.

The regulation model accounts for daily inflows, changes in reservoir storage, power flows, spillway flows and computed daily energy production. Power flows are determined as a function of a rule curve. The rule curve was developed to ensure that the minimum environmental flows would be reliably provided. These environmental flows, as recommended by NL Hydro were:

- 3.5 m^3/s from May 1st to September 30th;
- 2.0 m^3/s from October 1st to April 30th.

Model inputs are:

- Plant and reservoir characteristics;
- Inflow;
- Operating rules (rule curve).

Model outputs are:

- Turbinable flow (Q_T);
- Spilled flow (Q_S);
- Reservoir volume;

- Reservoir level;
- Energy produced.

7.2.1 Assumptions and Design Criteria

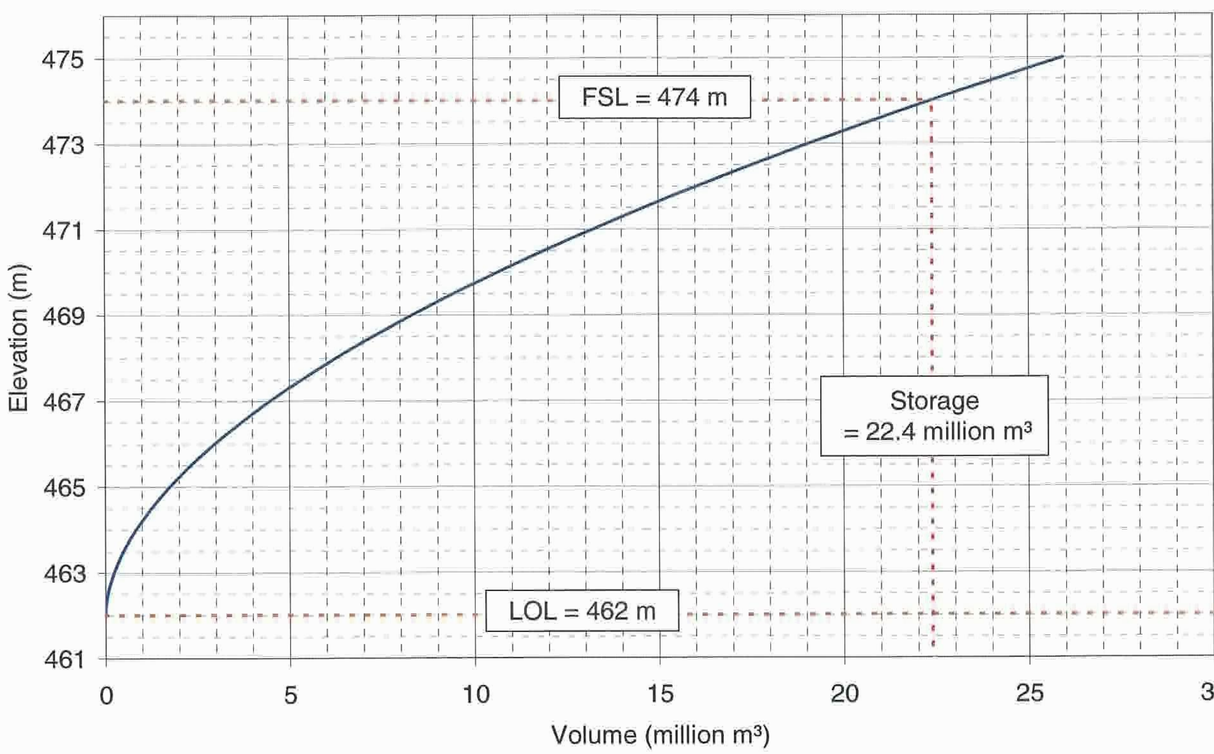
Inflow Data

The reconstituted flow series described in Section 4.0 was used to represent the inflow in the reservoir.

Storage Curve

The storage-elevation curve for the Storage Pond Reservoir was derived from 2.0 m contour mapping commissioned for this study. The curve is shown on Figure 7-1.

Figure 7-1: Portland Creek - Storage - Elevation Curve



Net Head

The head water level was assumed to be 458.0 m, i.e. the spillway crest elevation of the Headpond Dam. Water level in the Headpond will be kept constant at its maximum level, by balancing releases from the Storage Dam with power flows.

As Pelton units are considered, the downstream head level corresponds to the turbine centerline elevation, at elevation 42.63 m.

The head losses in the penstock were computed as a function of Q_T^2 (see Section 5.1). The optimized penstock diameters were assumed.

Operating Rules

A rule curve was developed by trial and error to ensure that the required minimum daily environmental flows would be reliably provided, as noted previously these flows are:

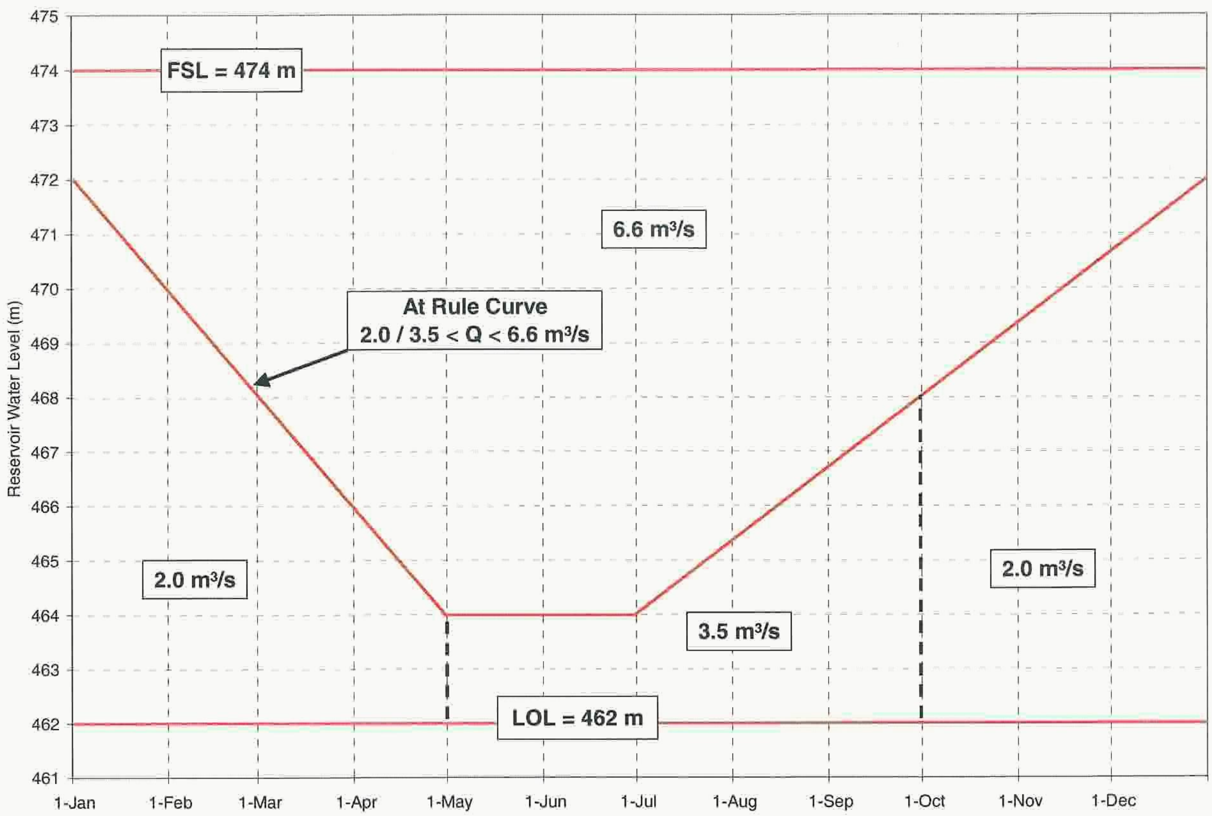
- 3.5 m^3/s from May 1st to September 30th;
- 2.0 m^3/s from October 1st to April 30th.

The rule curve divides the reservoir into two zones:

- Above rule curve, a potential surplus is indicated - operate turbines at full capacity (6.6 m^3/s);
- Below rule curve, a potential shortage is indicated - operate turbines at stipulated environmental flows (3.5 m^3/s or 2.0 m^3/s, depending on the month);
- At rule curve, intermediate flow between above limits, as calculated with reservoir at rule curve elevation.

Figure 7-2 illustrates this operating strategy.

Figure 7-2: Preliminary Rule Curve



Equipment Efficiency

- Typically, the turbine efficiency curve of Pelton turbines is almost flat over the normal operating range, therefore efficiency was assumed to be constant and equal to 90% (average over operating range);
- Generator efficiency was assumed to be 98%;
- Total unit efficiency was 90% X 98% = 88.2% (at generator terminals).

Maintenance and Outages

Unit outages can be of different types:

- Unscheduled outages due to any failure on the network;
- Forced outages due to a component failure;

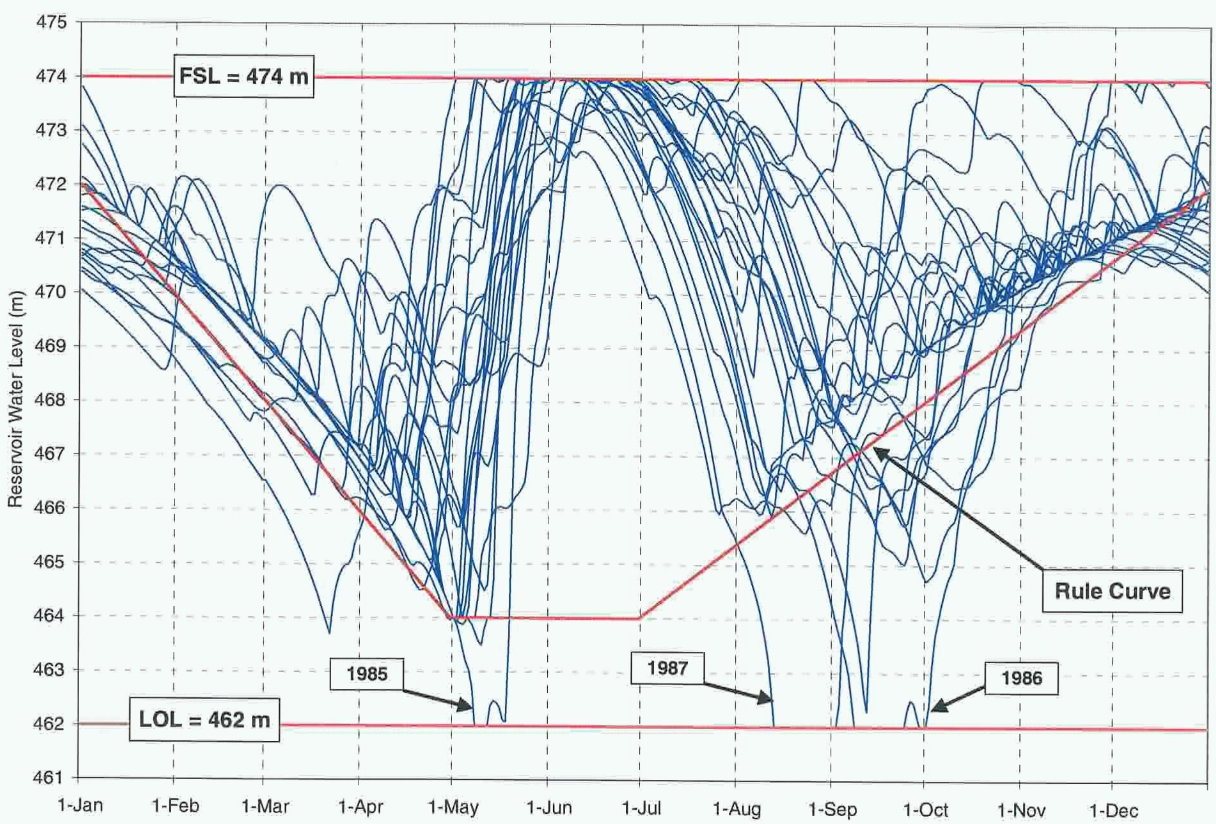
- Scheduled outages for periodical maintenance.

In the case of Portland Creek, it is anticipated that loss of energy due to outages and maintenance can be mitigated on most occasions by holding unused flow in the reservoir. A water utilization factor of 97% will be applied to theoretical energy to adjust for losses in energy production due to outages and "off-peak" operation.

7.2.2 Findings from Simulation Study

Figure 7-3 shows the simulated reservoir water levels over the simulation period, 1984 to 2005.

Figure 7-3: Simulated Water Level (1984 - 2005)



During this period, the reservoir level reached the lower operating level (LOL) on three occasions in 1985, 1986 and in 1987. During 1985, the operation failed to provide the stipulated environmental flows for 5 days, 20 days in 1986 and 21 days in 1987 for a total of 46 days in the simulation period. In this period, the plant flow was equal to the reservoir inflow reaching a minimum flow of 0.5 m^3/s. This represents a reliability of 99.4%, which is very satisfactory (1.5 months failure in 264 months).

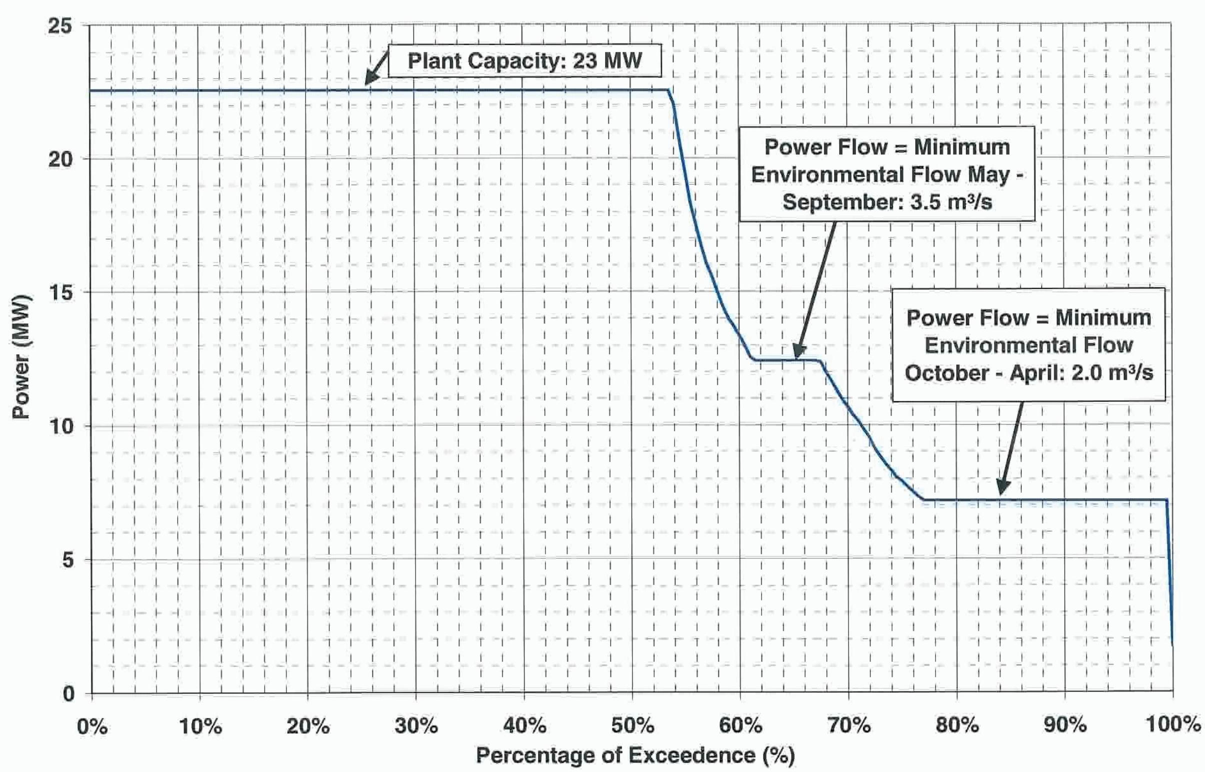
Figure 7-4 shows the power duration curve over the period of simulation. It must be noted that the plant maximum capacity (23.0 MW) is produced 54% of the time.

During 23% of the time, the plant output is equal to 7.2 MW, which corresponds to a flow of 2.0 m^3/s, i.e. the required minimum environmental flow between October and April.

During 6% of the time, the plant capacity is equal to 12.4 MW, which corresponds to a flow of 3.5 m^3/s, i.e. the required minimum environmental flow between May and September.

During the rest of the time (17%), the power flow is such that the reservoir level matches the rule curve.

Figure 7-4: Power Duration Curve



The principal results of the simulation study are summarized in Table 7-1.

Table 7-1: Main Energy Parameters

Average Natural Inflow (m^3/s)	5.20
Plant Capacity Flow (m^3/s)	6.60
Spilled Flow (m^3/s)	0.37
Head Water Level (m)	458.0
Turbine Centreline (m)	42.6
Head Losses at Maximum Flow (m)	20.4
Net Head (m)	394.9
Plant Capacity (MW)	23.0
Annual Energy (GWh)	141.5
Plant Average Capacity Factor	72%

7.2.3 Monthly Energy

Monthly average energy was computed for the period of daily simulation. It is presented in Table 7-2.

Table 7-2: Monthly Energy (at Generator Terminals)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1984	5.2	7.2	10.8	15.8	16.3	15.8	16.3	15.1	8.7	5.2	6.9	14.9	138.0
1985	6.6	4.7	5.2	5.1	11.6	15.8	16.3	16.0	8.7	7.0	9.9	5.2	111.9
1986	7.3	12.5	5.9	14.1	16.3	15.8	16.3	9.3	6.3	5.2	5.0	5.2	119.1
1987	5.2	4.7	5.2	15.0	16.3	15.8	15.0	5.6	8.5	11.0	13.7	7.0	122.8
1988	5.2	4.8	11.0	15.8	16.3	15.8	16.3	11.1	8.7	10.0	13.5	7.7	136.0
1989	5.2	4.7	5.2	14.9	16.3	15.8	16.3	16.3	15.8	16.3	10.5	5.2	142.2
1990	5.2	7.8	7.6	14.0	15.9	15.8	16.3	16.3	15.8	16.3	15.8	16.3	162.9
1991	12.0	6.2	11.3	8.8	16.3	15.8	16.3	16.3	14.4	16.3	9.7	5.4	148.6
1992	5.2	4.8	7.2	8.7	14.0	15.8	16.3	16.3	15.0	15.7	12.3	5.2	136.5
1993	5.2	4.7	5.2	11.6	16.3	15.8	16.3	16.3	15.8	12.3	9.5	6.4	135.1
1994	8.6	5.1	8.4	13.7	16.3	15.8	16.3	16.3	15.6	10.0	14.8	9.1	149.8
1995	5.2	4.7	6.9	15.6	16.3	15.8	16.3	15.8	12.1	12.2	14.9	6.3	141.8
1996	10.7	12.3	16.3	13.8	16.3	15.8	16.3	16.2	8.7	9.9	9.2	10.8	156.1
1997	12.4	6.7	5.3	9.8	16.0	15.8	16.3	16.3	14.6	13.5	10.7	5.2	142.4
1998	5.2	4.7	11.9	12.1	16.3	15.8	16.3	12.0	14.7	16.3	11.4	8.9	145.4
1999	12.7	14.7	11.4	13.5	15.3	15.8	16.3	14.4	9.8	12.4	14.1	15.6	165.9
2000	10.6	8.3	11.0	15.8	16.3	15.8	16.3	13.7	8.7	10.4	7.8	5.2	139.6
2001	5.8	4.7	5.2	7.8	16.3	15.8	16.3	13.6	10.2	8.4	14.7	10.6	129.2
2002	9.1	5.2	8.7	13.3	15.6	15.8	16.3	16.3	15.8	16.0	12.6	9.8	154.3
2003	6.2	13.7	6.5	15.8	16.3	15.8	16.3	16.3	11.2	6.6	11.4	8.1	144.1
2004	12.2	5.5	5.2	14.3	16.3	15.8	16.3	12.1	15.1	10.3	6.4	10.8	140.3
2005	15.8	9.3	9.7	14.2	16.1	15.8	15.6	9.0	9.4	15.7	14.9	5.6	151.0
Avg.	**8.0**	**7.1**	**8.2**	**12.9**	**15.9**	**15.8**	**16.2**	**14.1**	**12.0**	**11.7**	**11.3**	**8.4**	**141.5**

7.3 CAPACITY BENEFITS

The plant installed capacity 23.0 MW (at generator terminals) was determined on the basis of an optimization study as described in Section 6.6 and corresponds to a maximum plant flow of 6.6 m^3/s.

In a mixed hydrothermal system such as the Island System of NL Hydro, firm capacity is the capacity that would be readily available to provide peak power during critical low flow periods. During such periods, thermal capacity at Holyrood Generating Station would be base loaded. In the absence of a system study to evaluate the role of Portland Creek, the following pattern of daily operation was assumed:

- 4 hours at 6.6 m^3/s giving 23.0 MW;
- 20 hours at 1.08 m^3/s giving 3.9 MW.

  Average over 24 hours = 2.0 m^3/s.

In this operating mode, the minimum flow is 1.08 m^3/s, which is greater than the operating limit of 30% of single jet flow, 0.5 m^3/s, and also greater than the minimum monthly natural flow of 0.22 m^3/s. On the basis of the above assumptions, the firm capacity is evaluated as equal to installed capacity of 23.0 MW (at generator terminals).

7.4 PROJECT BENEFITS

As previously noted (Section 6.3), Project benefits were evaluated using the following economic parameters:

- Capacity at $1,144 per kW;
- Energy at $ 0.70 per kWh.

Benefits were evaluated as below:

• Capacity	23,000 kW	@ $1,144/kW	=	$ 26.3 M;
• Energy	141.5 x 10^6 kWh	@ $0.70/kWh	=	$ 99.1 M.
Total				$125.4 M

7.5 COMPARISON TO PREVIOUS STUDY

These energy results were compared to the previous study made in 1987. The higher selected installed capacity and the higher annual energy can be explained by:

- Greater values for natural inflow were used;

- The head increased due to an increase of the headwater level and decreased head losses with a larger penstock diameter;

- The value of energy escalated more rapidly than construction costs, favouring a larger installed capacity;

- The fact that the spilled flow is less, which is explained by the higher installed capacity and the utilization of a rule curve to manage the reservoir.

Table 7-3 presents a comparison between this study and the study made in 1987.

Table 7-3: Comparison to Previous Study

		1987 Study	2006 Study	Diff.
a	Natural Inflow (m^3/s)	4.44	5.2	17%
b	Installed Capacity (MW)	12.1	23.0	90%
c	Spilled Flow (m^3/s)	1.18	0.37	-69%
d	Average Power Flow (m^3/s)	3.26	4.82	48%
e	Average Net Head (m)	375	402.5	7%
f	Equipment Efficiency and Outages	86%	85.6%	0%
g	Annual Energy (GWh)	89.8	141.5	58%

8 PROJECT COMPONENTS

8.1 ACCESS ROADS

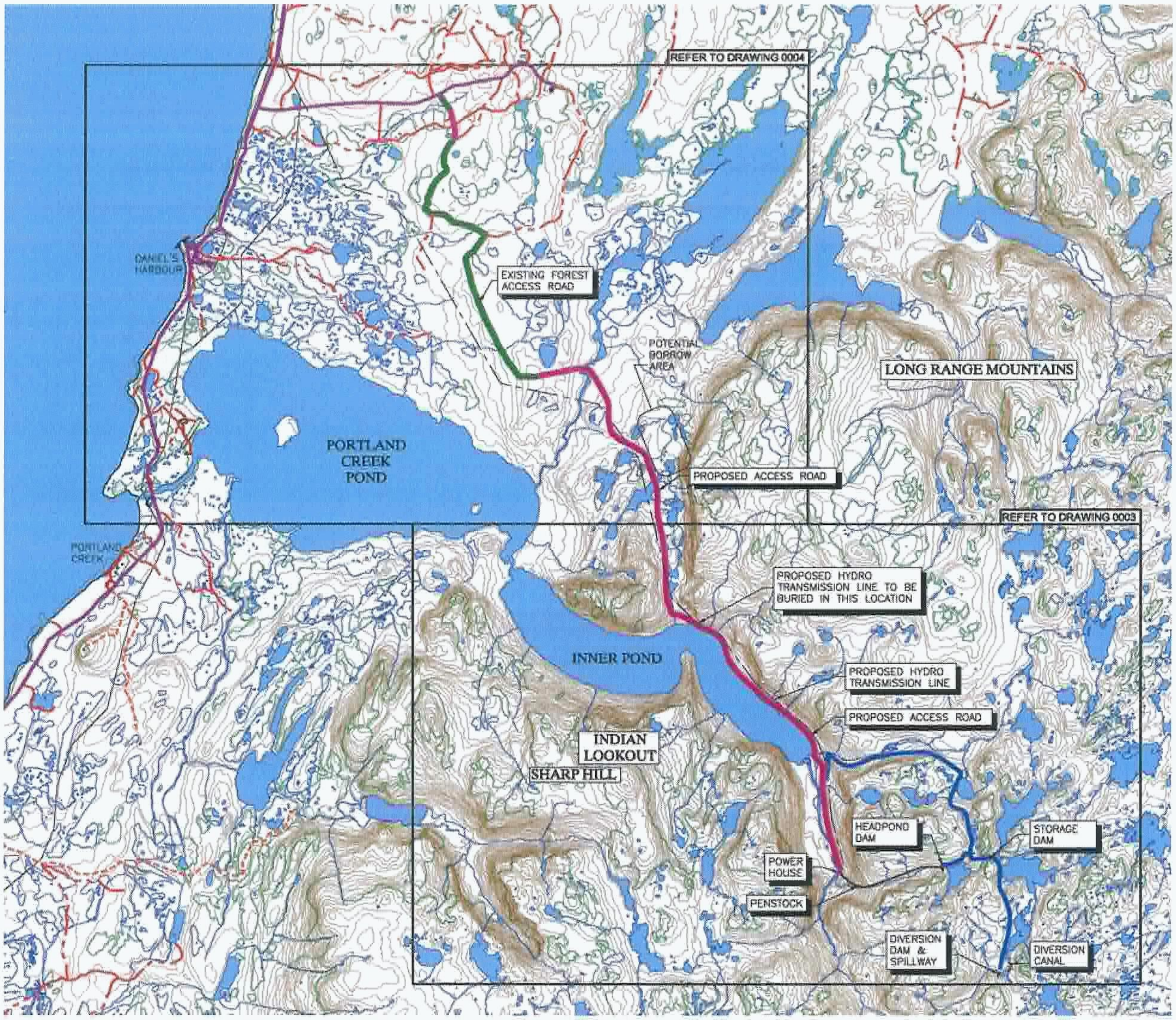
To access the site, various alternatives for roadway access were considered.

The recommended route includes:

- Permanent access from the existing Mine Road. This includes upgrading of existing forest access roads and new construction to Inner Pond and south to the powerhouse site;
- Temporary construction access roads from the powerhouse intersection heading east before turning south to access storage, Headpond and Diversion Dams.

The following Figure 8-1 and Drawing Number 722736-0000-41DD-0001 shows the proposed access route for the Project. The route includes 9.16 km of upgrading existing roads and 18.15 km of new road construction to permanent standard. Also, it includes 11.62 km of new road construction to a temporary construction standard. The route includes two permanent and one temporary bridge structure. The temporary bridge structure, located at the storage dam, will be relocated to a permanent location on the dam at the end of construction. The cost to provide access to the site is estimated at $12,380,000.

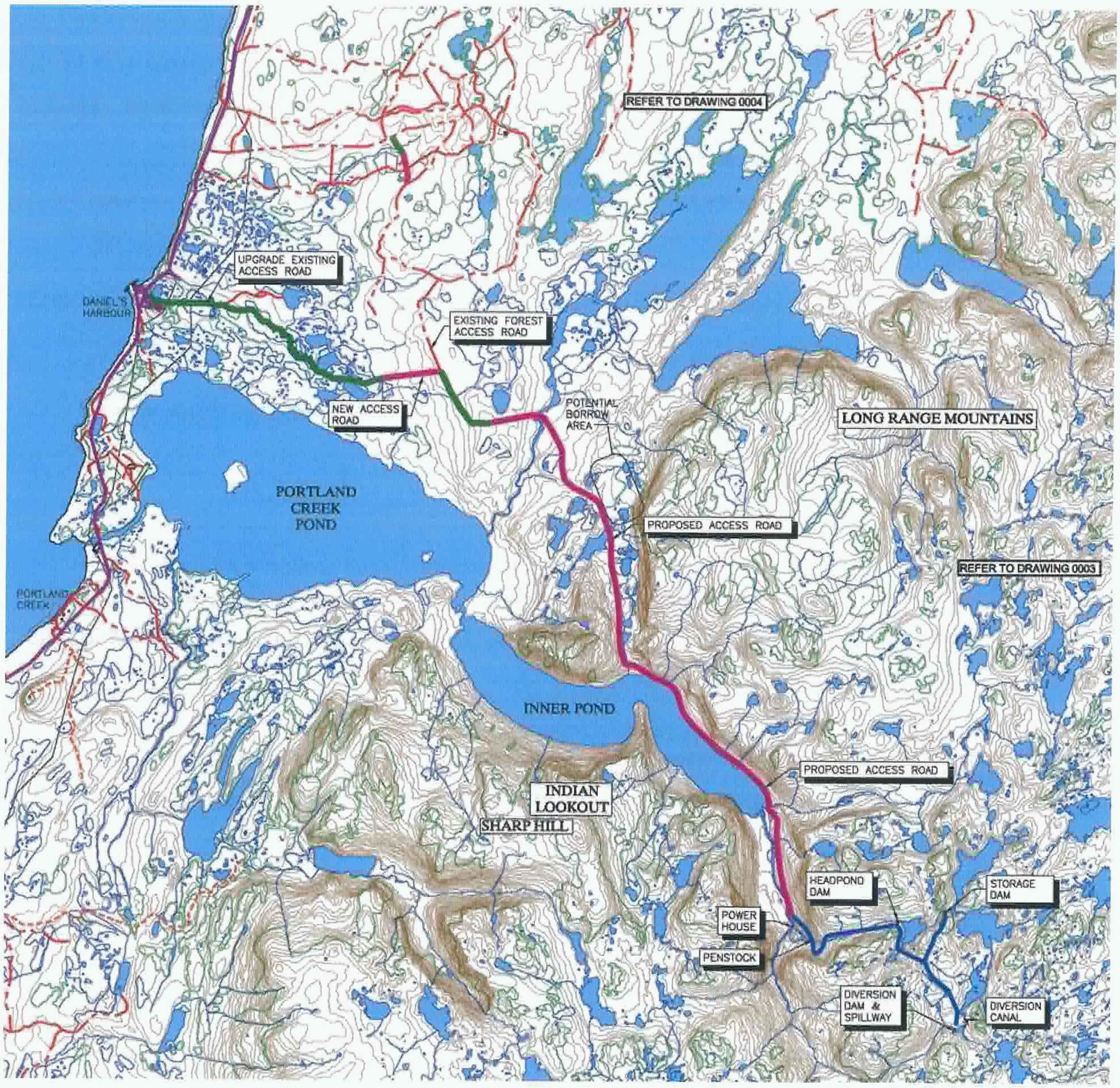
Figure 8-1: Access Roads



Alternative Considerations

1. General site access. In addition to the recommended routes, two alternatives were considered. Consideration was given to providing access from the community of Daniel's Harbour heading east before turning south towards Inner Pond as shown in Figure 8-2. This route included 7.32 km of upgrading existing roads and 1.6 km of new road construction to a permanent standard. When compared to upgrading of the existing forest access roads from Mine Road, the later route was the most feasible and favourable of the two.

Figure 8-2: Alternative Considerations



2. Access to the Headpond, Storage and Diversion Dams from the powerhouse. Consideration was given to providing access to these sites from the powerhouse heading east for approximately 3.6 km to the Headpond Dam adjacent to the selected penstock route. From the Headpond Dam the route would continue approximately 3.4 km southeast to the Diversion Dam. A separate access approximately 1.5 km north from this section of road would be provided to the storage dam as per Figure 8-2. This alternative route crosses very rough terrain

and the road would have sections with very steep grades. Also, the route would require at least three bridges as switchbacks over the existing river would be required to achieve suitable grades for road construction. In comparison to the selected route, this route would be more difficult and costly to construct. Hence, the selected route was most feasible and favourable of the two.

Bridges

A total of three bridges are required for the permanent and temporary access roads as follows:

1. River Crossing Permanent Access in to Portland Creek Pond - $365,750;
2. River Crossing Permanent Access in to Inner Pond - $239,800;
3. Storage Dam – Temporary Bridge required for Construction - $220,000.

After construction, the temporary bridge would be relocated to a permanent location on the dam - $432,850.

Bridge estimates are based on panel-type bridges, which are more commonly referred to as Bailey Bridges. Panel Bridges are the most economical and can be installed quickly.

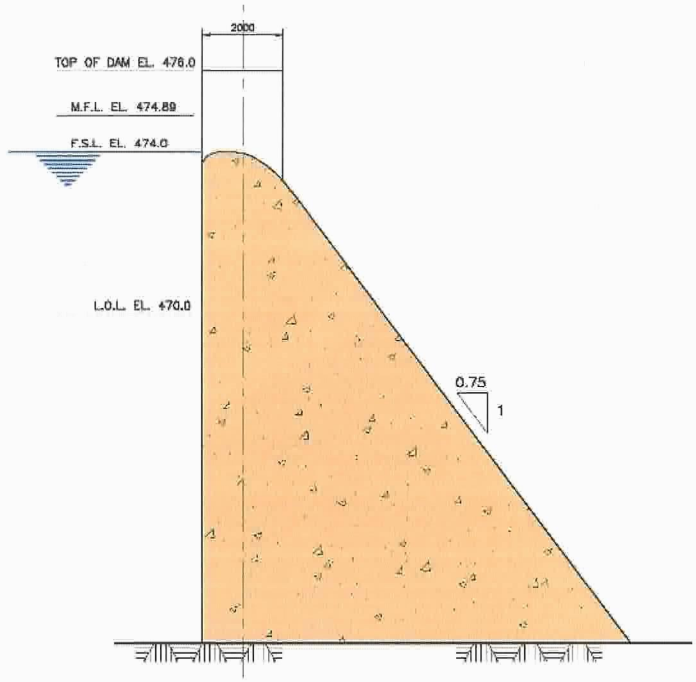
8.2 CONTROL STRUCTURES

8.2.1 Diversion Dam

The diversion dam structure is of concrete gravity dam construction with a maximum height of 12 m. See Figures 8-3 and 8-4. The dam is approximately 110 m long and includes an overflow spillway with a crest length of 70 m set at elevation 474.0 m. The spillway is designed to pass 130 m^3/sec (1/1000 flood). This is the combined flood flow from both the main drainage area and the diversion drainage area. The flood flow produces a maximum upstream water elevation of 474.89 m. The top of the non-overflow dam is set at 476.00 m providing a free board of approximately 1.11 m above the design flood level.

Figure 8-3: Diversion Dam

Figure 8-4: Diversion Dam Spillway Section



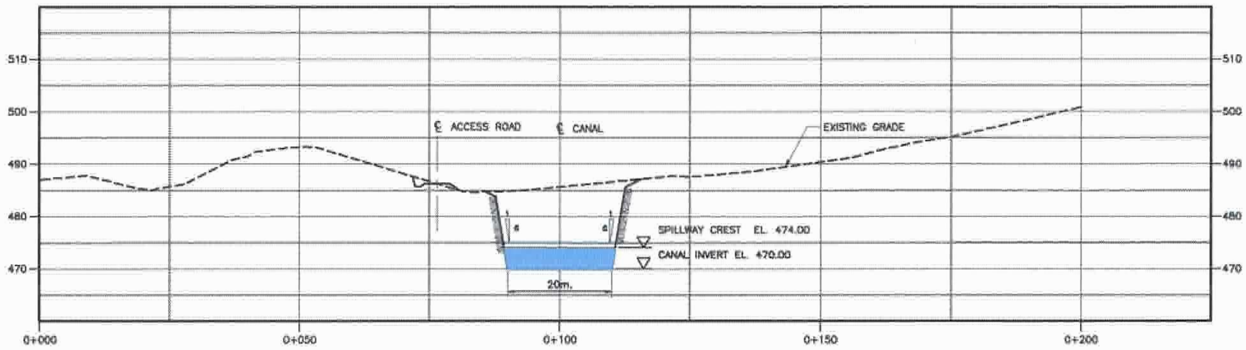
8.2.2 Diversion Canal

8.2.2.1 General

The diversion canal serves two purposes. It transfers the normal inflows from the diversion pond into the main storage reservoir and allows flood flows into the main storage reservoir to flow into the diversion pond where they are discharged over the diversion dam spillway. The canal is approximately 320 m long with its invert elevation at 470.00 m. See Figures 8-5 and 8-6.

Figure 8-5: Diversion Canal

Figure 8-6: Diversion Canal Section



8.2.3 Storage Dam

8.2.3.1 General

The storage dam is of concrete gravity dam construction. The dam is approximately 45.0 m long and has a maximum height of 16.0 m. The top of the dam is set at elevation 476.00 m providing a free board of 1.11 m above the design flood level of 474.89 m. The storage dam includes a flow regulating structure fitted with a trash rack. Flow regulation through the structure is achieved by a bottom sluice equipped with a 1.6 m wide by 2.0 m high vertical lift gate. For gate maintenance the trash rack will be removed and replaced by stop logs. A small heated building will be provided to house the control and operating systems. There will be no spillway at the storage dam since it is intended to route all flood flows over the diversion dam spillway. See Figures 8-7, 8-8, and 8-9.

Figure 8-7: Storage Dam

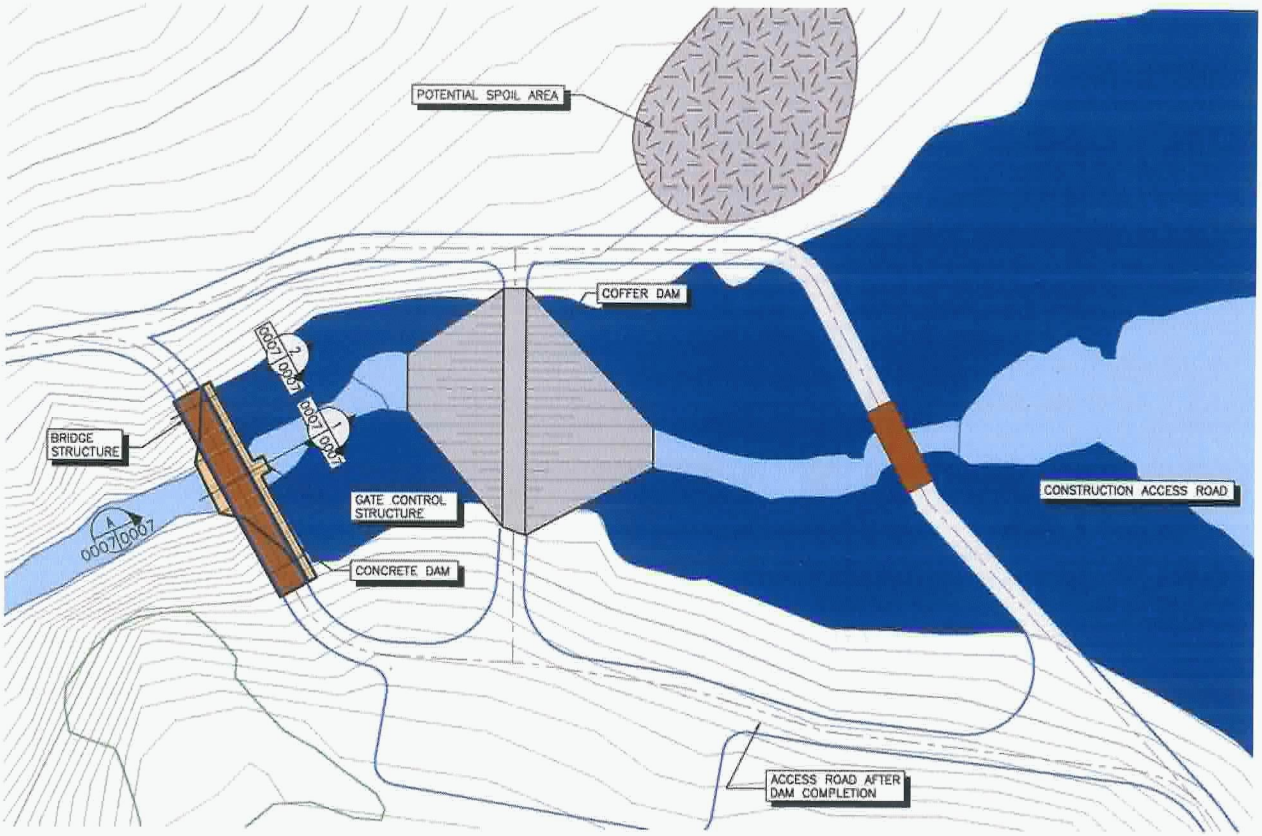


Figure 8-8: Section at Control Structure

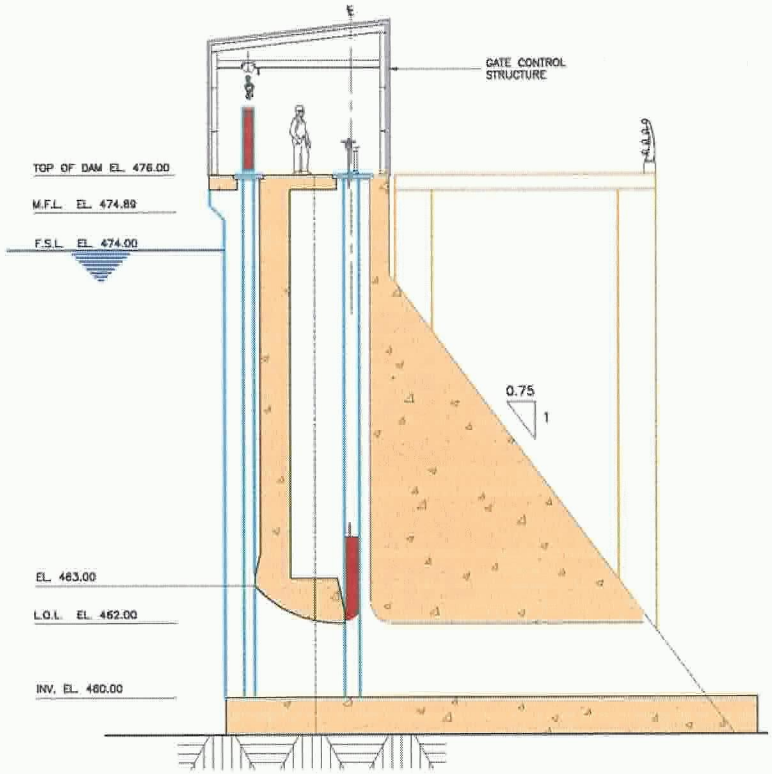
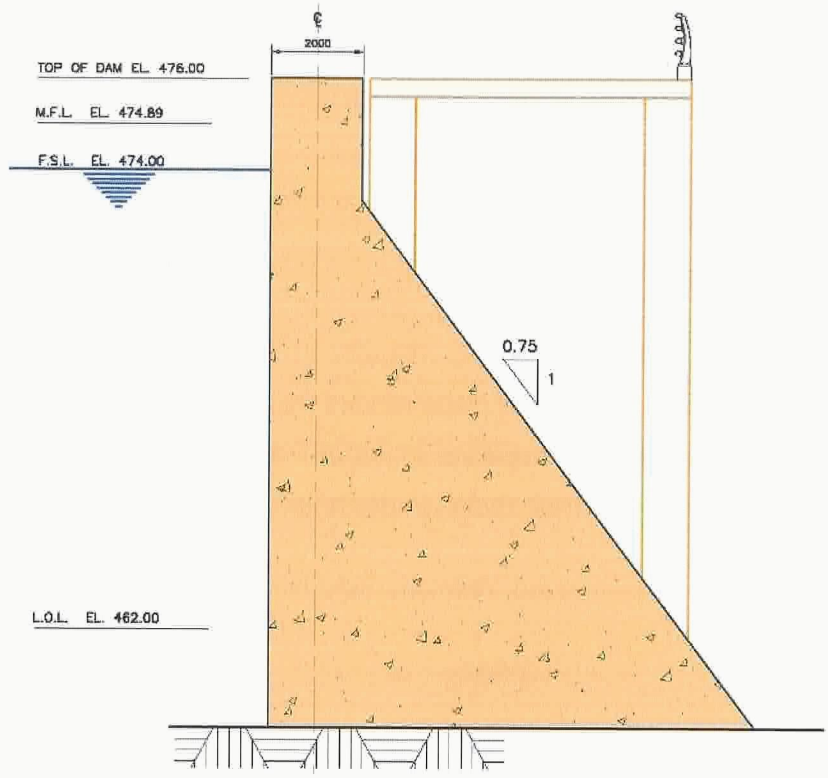


Figure 8-9: Typical Dam Section



8.2.3.2 *Mechanical Equipment*

Trash racks

Trash racks will be of the removable type to be installed in steel embedded parts on the face of the intake and will be handled using an electric cable monorail to be installed inside the building above the slots.

Following are estimated main characteristics of the trash racks:

- Quantity - 1 Set;
- Hydraulic Passage Width - 2.5 m;
- Hydraulic Passage Height - 3.0 m;
- Sill Elevation - 460.0 m.

A differential pressure measuring system will be provided to monitor the degree of obstruction of the trash racks.

Since reservoir minimum level, which will normally be met in wintertime, will be below the trash rack lintel elevation, trash racks will require design in order to prevent formation of ice on the exposed portion of the trash racks.

Stop logs

When required, stop logs will be installed in the trash rack embedded parts in case the control gate's embedded parts require inspection and/or maintenance. They will be provided in multiple sections to be individually handled by the same monorail used for handling the trash racks, under balanced hydraulic conditions.

Following are the estimated main characteristics of the stop logs:

- Quantity - 1 Set;
- Design Head - 15 m.

Control Gate

The control gate will be of the fixed-wheel type, with an upstream sealing system. For removal and maintenance the gate will be operated by an electric cable monorail, which will also be used for trash rack and stop log handling and maintenance. The gate will come complete with steel embedded parts and will be designed to cut the flow under emergency conditions. The gate well and embedded parts will be heated to prevent formation of ice in winter.

Following are the estimated main characteristics of the control gate:

- Quantity - 1;
- Hydraulic Passage Width - 1.6 m;
- Hydraulic Passage Height - 2.0 m;
- Sill Elevation - 460.0 m;

- Monorail Lifting Capacity - <10 tonne;
- Design Head - 15 m.

8.2.4 Headpond Dam

8.2.4.1 General

The Headpond Dam is of concrete gravity dam construction with a total length of 143.0 m and a maximum height of 15.0 m. The Headpond Dam includes a power intake structure fitted with a trash rack and an overflow spillway. The power intake structure is located on the right bank of the river. Flows through the structure to the penstock will be regulated by a 1.6 m wide by 2.0 m high vertical lift gate. For gate maintenance the trash rack will be removed and replaced by stop logs. A 30 m long overflow spillway with its crest set at elevation 458.00 m is designed to pass a maximum flow of 10 m^3/sec. This flow produces a maximum upstream water level of 458.30 m. The top of the non-overflow dam is set at elevation 459.30 m providing a free board of approximately 1.0 m above the maximum water level in the Headpond. See Figures 8-10, 8-11 and 8-12.

Figure 8-10: Headpond Dam

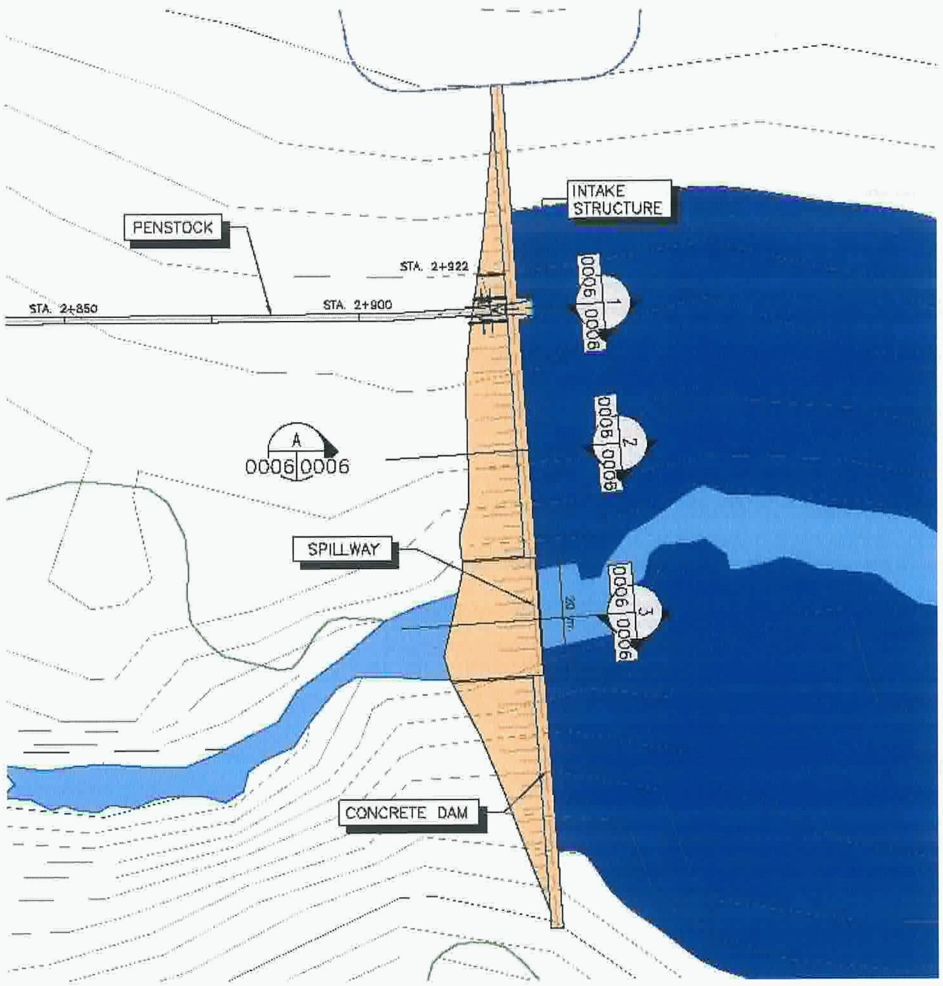


Figure 8-11: Headpond Dam: Intake

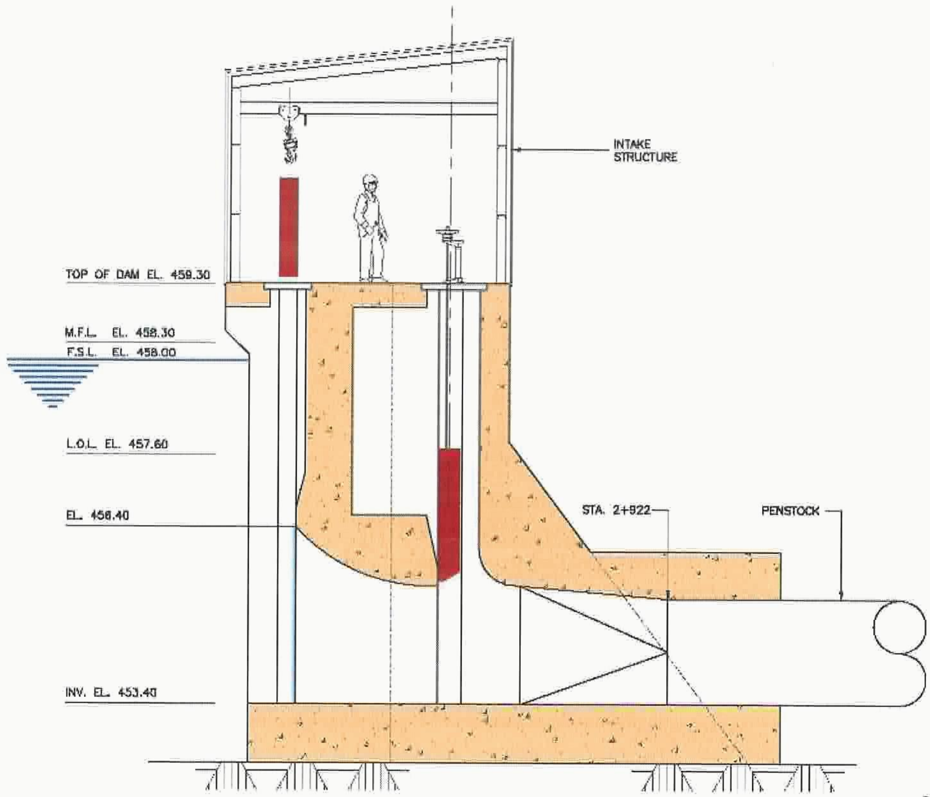
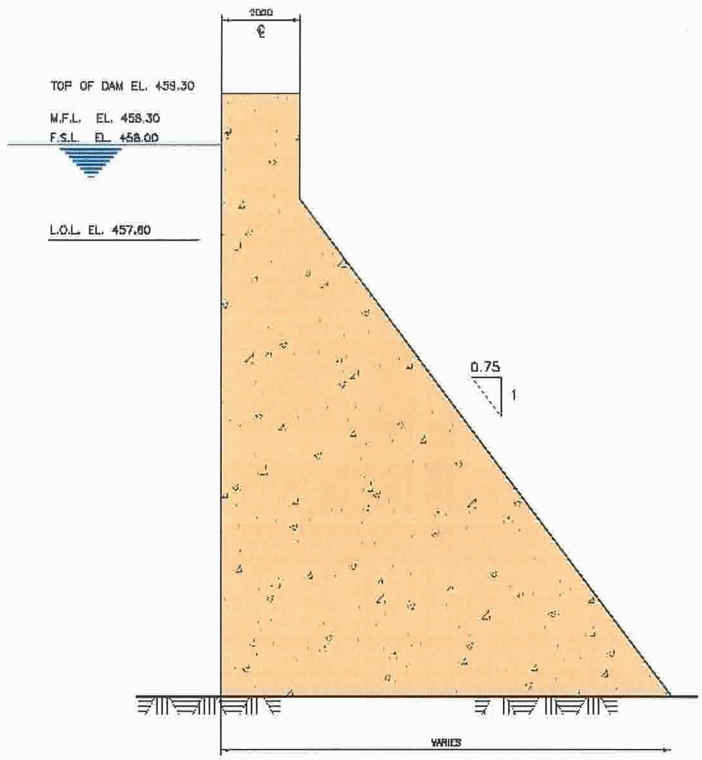


Figure 8-12: Headpond Dam: Typical Dam Section



8.2.4.2 *Mechanical Equipment*

Trash racks

Trash racks of the Headpond Dam will be of the same arrangement as at the Storage Dam, with the following estimated main characteristics:

- Quantity - 1 set;
- Hydraulic Passage Width - 2.5 m;
- Hydraulic Passage Height - 3.0 m;
- Sill Elevation - 453.4 m.

No special design will be required to prevent ice formation on the trash racks as reservoir minimum level will remain above the trash rack lintel elevation at all times.

Stop logs

Stop logs of the Headpond Dam will be of the same arrangement as at the Storage Dam, with the following estimated main characteristics:

- Quantity - 1 Set;
- Design Head - 4.9 m.

Control Gate

The control gate of the Headpond Dam will be of the same arrangement as at the Storage Dam, with the following estimated main characteristics:

- Quantity - 1;
- Hydraulic Passage Width - 1.6 m;
- Hydraulic Passage Height - 2.0 m;
- Sill Elevation - 453.4 m;
- Monorail Lifting Capacity - <10 tonne;
- Design Head - 4.9 m.

8.3 POWERHOUSE

8.3.1 General

The powerhouse structure is located on bedrock on the right bank of Portland Creek. A short tailrace will be excavated through bedrock and boulders.

The powerhouse substructure is primarily of flat slab construction placed on a rock foundation. The superstructure is of braced steel construction with insulated metal siding. The building is approximately 17 m wide by 44 m long. See Figures 8-13, 8-14 and 8-15.

Space for the various functional areas of the powerhouse such as warehouse, office, repair bay, washroom, control room etc., have not been laid out at this stage but are included in the Cost Estimate.

Figure 8-13: Powerhouse Site Plan

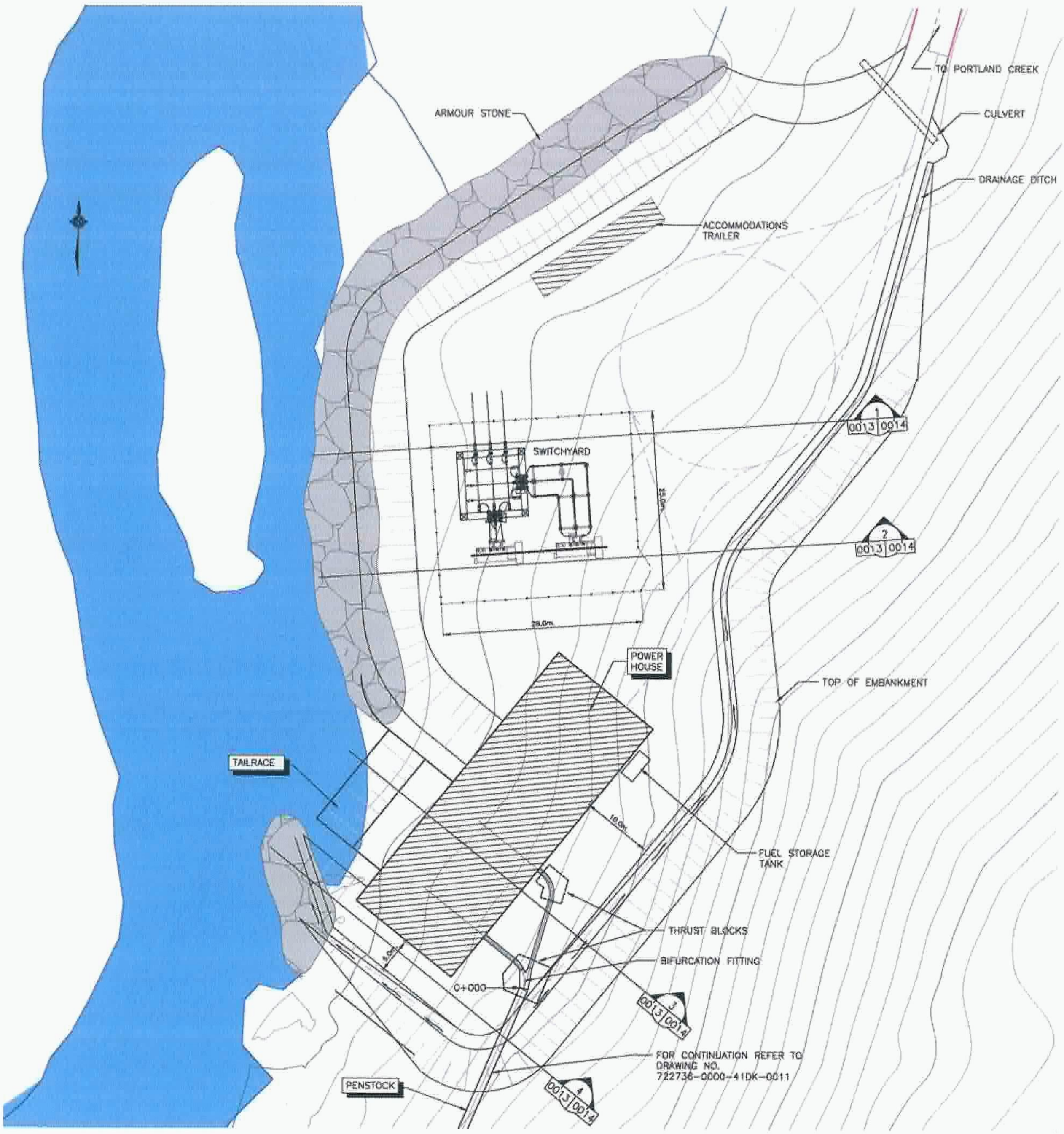


Figure 8-14: Powerhouse Floor Plan

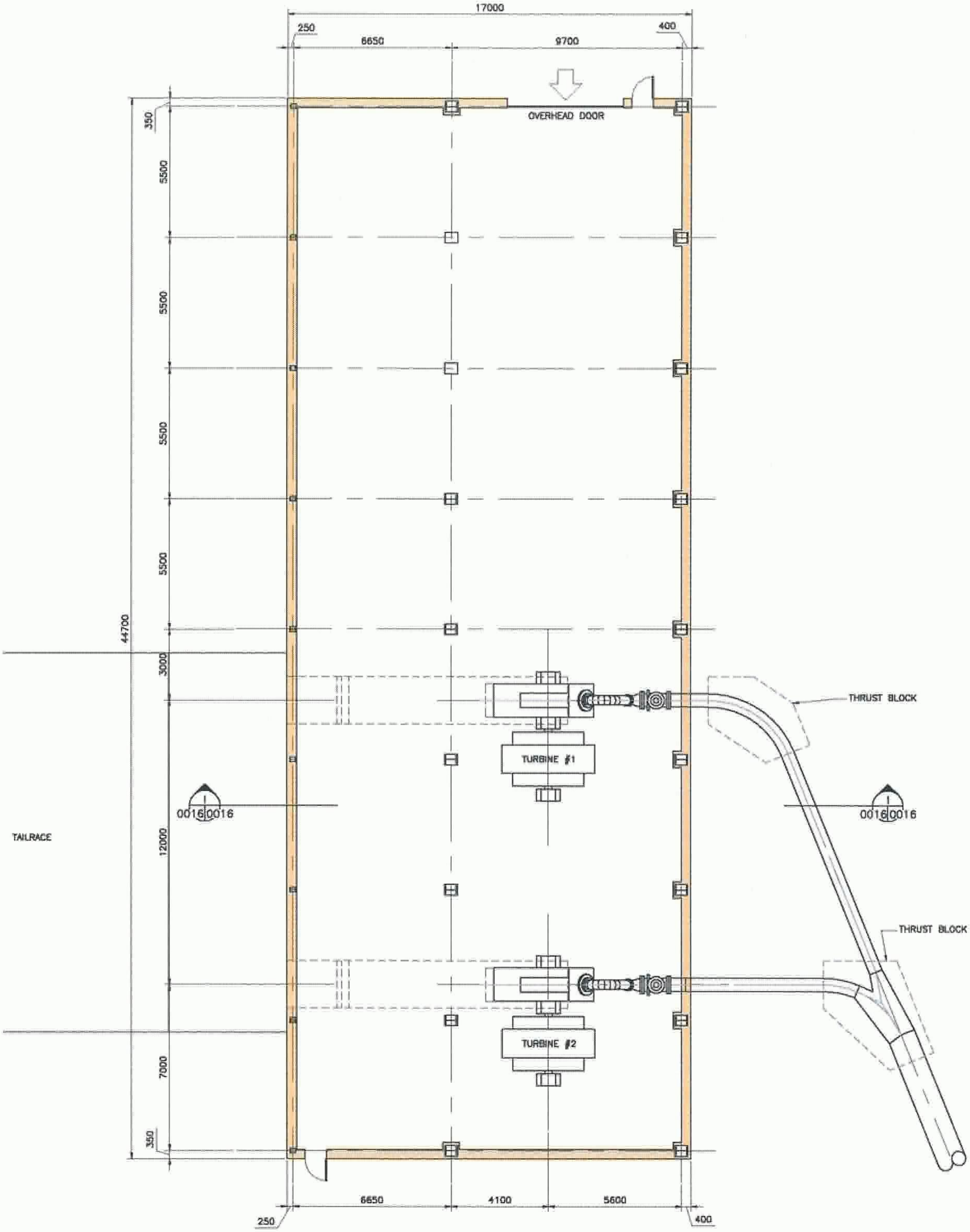
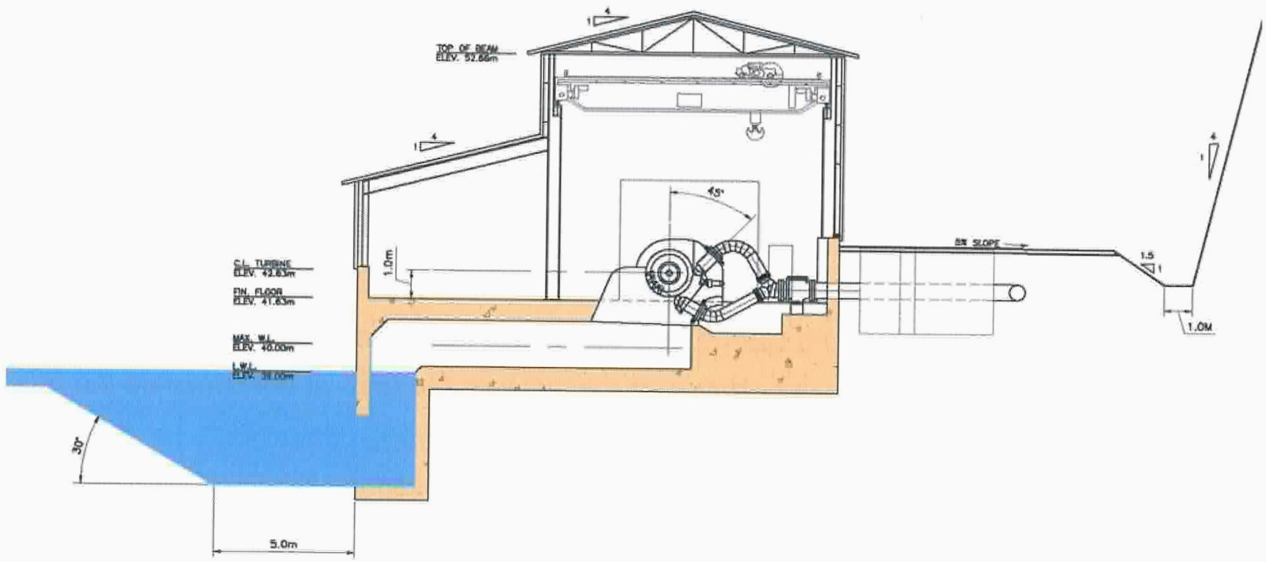


Figure 8-15: Building Section



8.3.2 Mechanical Equipment

8.3.2.1 Turbine, Main Inlet Valve and Governor

The powerhouse will be equipped with two horizontal-axis Pelton turbines with the following estimated main characteristics:

• Number of Units:	2
• Type:	Horizontal-axis, single runner, double jet Pelton
• Rated Head:	394.9 m
• Rated Output/Unit:	11.5 MW
• Turbine Discharge/Unit:	3.3 m^3/s
• Synchronous Speed:	514.28 rpm
• Runner Outside Diameter:	1.5 m

Each unit will be isolated with the help of a main inlet valve of the following estimated main characteristics:

Number:	2
Type:	Spherical
Nominal diameter:	750 mm

Each unit will consist of a single runner coupled to the generator. The runner will be driven by two jets using needle valves. The needle valves will be controlled by the governor action through a system of operating mechanism and hydraulic servomotors. Jet deflectors will be used to prevent excessive over speed during load rejection. The control system will allow operation of one or two nozzles depending on the flow.

The components of the turbine will be designed to withstand all static and dynamic loads resulting from continuous operation within the full range of heads and outputs.

An electro-hydraulic digital type governor, suitable for fully automatic control, will control unit speed during start-up and synchronizing sequences as well as generator output power after synchronization of the unit with the grid. It will comprise the PID controls and adjustable (0-10%) speed droop operating components for load and frequency controls and logical functions for start-stop sequences.

Due to the limited size of the units, it is not envisaged to carry out any model tests.

8.3.2.2 Powerhouse Overhead Crane

The powerhouse will be equipped with an overhead traveling crane. It will be provided with a main lifting hook, for handling the turbine-generator unit's heaviest components and an auxiliary lifting hook for handling other minor equipment. The main hook lifting capacity will be selected based on the weight of the rotor, which will be the heaviest piece of equipment to be handled by the crane.

Following are the estimated main characteristics of the powerhouse overhead crane:

- Main Lifting Capacity - 35 Tonne;
- Auxiliary Lifting Capacity - 5 Tonne;
- Span - 16.5 m.

8.3.2.3 Mechanical Auxiliary Systems

Following mechanical auxiliary systems will be provided inside the powerhouse:

- Plumbing, sanitary;

- Fire protection system;
- Cooling water system;
- Low pressure compressed air system;
- Powerhouse HVAC system;
- Oil handling and purification system.

8.3.3 Electrical Equipment

8.3.3.1 Generator

The two synchronous machines at the Portland Creek powerhouse will have the following characteristics: 12 MVA, 0.95 p.f., 13.8 kV, 3 phase, 60 Hz, 514.28 r.p.m., vertical shaft, hydro-electric generators, air cooled by air to water heat exchangers, and directly coupled to the Pelton turbine. A system interconnection study by NL Hydro will determine the ultimate power factor of the machines at the final feasibility stage.

8.3.3.2 Static Excitation System

The excitation system will be of the static type, matching the field requirements of the 12 MVA generators. The excitation system will consist of a dry-type excitation transformer, thyristor rectifier bridges, voltage regulator, DC field breaker and discharge resistor.

The power supply for the excitation system will be taken from the generator terminals through the excitation transformer.

8.3.3.3 Generator Phase Connection

The connection between the generators and the step-up transformers will be done via two parallel single-core 15 kV XLPE cables per phase. The cable connection will be made between the generator phase cubicle terminals and step-up transformer terminals located in the switchyard.

The nominal current, at rated generator output and rated voltage flowing, in the cables will be 502 A. With a 10 % overload, the current will increase to 552 A. According to our evaluation, by taking into account the applicable de-rating factors, the cable with 2 x 350 MCM per phase will be suitable for 552 A continuous service.

8.3.3.4 13.8 kV Generator Terminal Cubicle

Inside the Portland Creek powerhouse, the three-phase taps to the generator phase cubicle and to the station auxiliary transformers will be done according to the main single line Drawing No. 722736-0000-47DD-0001.

In line with the main single line diagram, each generator terminal cubicle will be equipped with generator phase current transformers, withdrawable voltage transformers, surge arresters, surge capacitors, disconnect switch and grounding switch.

8.3.3.5 Neutral Grounding Cubicle

The neutral terminals of the generators will be connected to the generator neutral grounding cubicles. The cubicles will be equipped with a single-pole, single throw disconnect switch, a single-phase, dry-type, 13800 – 240/120 V neutral grounding transformer, and an earthing resistor connected to the secondary of the transformer. The resistor value will be calculated during the design phase to limit the single-phase fault current of the generator to a maximum of 15 A.

The generator neutral grounding cubicle will be installed next to the neutral terminals of the generator on the generator floor.

8.3.3.6 Generator Step-Up Transformer

The generator terminal voltage of 13.8 kV will be stepped-up to 66 kV through generator step-up (GSU) transformers directly connected with the units. The GSU transformers will be oil immersed, outdoor type, 10/13.3 MVA with ONAN/ONAF air-cooling system.

The low voltage bushings will be directly connected to the corresponding generator through 15 kV XLPE power cables. The H.V. terminations will be connected to switchyard overhead conductors.

The high voltage side of the transformer will be equipped with bushings and H.V. surge arresters will be mounted on the brackets of the transformer tank. The neutral on the high voltage side of the transformers will be solidly grounded. The neutral and H.V. bushings of the transformers will be equipped with bushing type current transformers.

The transformers will be designed to ensure satisfactory operation under the working conditions in the system. An off-load tap changer will be provided to compensate for changes in the system operating characteristics throughout the life of the power plant. The tap range will be ± 5% in 2.5 % steps.

A fire protection system, including fire detectors and sprinklers, will be installed around the transformers to provide adequate fire protection. A firewall, which will exceed the height of the transformers by about one meter, will separate one transformer from the next.

The transformers will have their own oil recuperation basin connected to a drainage system feeding a common oil–water separation pit. The net volume of the basin will be able to contain the volume of oil of one transformer.

8.3.3.7 Powerhouse AC and DC Auxiliary Electrical System

Powerhouse AC Auxiliaries

The powerhouse ac auxiliaries will be fed from the generating units through two 13.8 kV: 600 V, dry-type auxiliary transformers.

The primary of the auxiliary transformers will be connected through medium voltage power cables to the 13.8 kV generator terminal cubicle. The secondary of the auxiliary transformers will be connected to the 600 V, 3-phase, 4-wire station services switchboard through low voltage power cables. The station services switchboard will be sectionalized and interconnected through a bus-tie circuit breaker.

A diesel generator (DG) set, sized to feed the essential loads, will provide station emergency power in the event of the unavailability of the generating units and of the grid supply. The DG set will be connected to the 600 V station service switchboard through three-phase power cables.

The DG set will be complete with all accessories for local and remote control starting and stopping.

On loss of station supply, an under voltage relay will start the diesel generator set after a suitable time, to supply the essential loads for periods when the infeed from the system is not available. Upon restoration of the main supply, the switchover to the main will occur and the DG set will stop automatically.

The three incoming circuit breakers feeding the 600 V station services switchboard are electrically interlocked to prevent their inadvertent paralleling onto the station service bus.

For the supply of power to the Headpond and Storage Dams, one step-up 25 kV-600V, remote area services transformer, connected to the 600V station service bus of the powerhouse will be provided. A 25 kV overhead line will supply power from the powerhouse to the remote area feeders. At the Headpond and Storage Dams, 25 kV – 600V step-down transformers will feed the local loads at the dams.

Standby DG sets, one for each dam, will provide an emergency source in the event of a power outage at the dams. The diesel generator sets will be installed in a housing near their respective dams.

Powerhouse DC Auxiliaries

For the utmost reliability, the control and protection equipment will be fed from a dc supply. Supply for the 129 V dc auxiliaries of the Portland Creek powerhouse will be derived from a single 129 V battery set. The battery set will be supplied with its own charger.

One main dc distribution board will also be provided to feed the powerhouse auxiliary dc loads.

The 129 V battery set and charger will be located in the battery room and charger room respectively.

The communication system will cover all plant locations. 48 V dc battery systems will be provided at the powerhouse, Headpond Dam and Storage Dam.

Battery and charger sizes will be determined during the detailed engineering stage.

An uninterruptible power supply (UPS) will also provide the ac supply derived from 129 V dc supply.

8.3.3.8 Control, Protection and Monitoring System

The Portland Creek powerhouse will be unmanned. Therefore, a Remote Terminal Unit, (RTU) will be installed at the powerhouse, the Storage dam and at the Headpond Dam to allow the Energy Control Center (ECC) the capability to monitor and control the complete generating station.

Plant operation philosophy will be based on unit output dependent on the water level at the Storage dam.

The control and protection system of the powerhouse will consist of a modern distributed control system compatible with an automatic controller for each unit, intake gate controller, automatic synchronizing equipment, generator protection system, main transformer protection system, and auxiliary system protection.

The 66 kV bus and outgoing transmission line protection will be supplied and coordinated by NL Hydro.

The Distributed Control System (DCS) will be provided to control and monitor the two units, the auxiliary systems, the electrical power system and the hydraulic structures associated with the powerhouse scheme. The DCS will include indication and alarm monitoring functions and full remote control of the powerhouse.

The DCS structure will use modern electronic device equipment in accordance with the latest available technology for power plant control systems capable of giving satisfactory performance under the specified conditions.

The protection system (electrical and mechanical) will interface with the DCS system at a number of locations throughout the plant for the exchange of alarms, status and commands.

8.3.3.9 Communication System

NL Hydro's Energy Control Center will have remote communications for monitoring and control of the powerhouse, intake, switchyard, and associated facilities. The Portland Creek communication infrastructure will be a combination of fibre optic cable, leased facilities, and NL Hydro's private microwave system.

Portland Creek telecommunications will include the following: VHF mobile radio, high speed administrative and internet access, telephone service, Supervisory Control and Data Acquisition (SCADA), operational LAN and Teleprotection for the new 66 kV transmission line.

A new 27 km fibre optic cable will be installed between Portland Creek Powerhouse and Peter's Barren Terminal Station. This cable will be attached to the new 66 kV transmission structures.

In addition, a new 10 km fibre optic cable will be installed between Portland Creek Powerhouse and the Storage Dam/Headpond Dam control structures. This cable will be attached to the new station service line.

8.3.3.10 Earthing System

The earthing system of the Portland Creek power station will be interconnected to the earthing grid of the 66 kV switchyard.

The earthing grid will be designed to obtain an earth resistance of 1 ohm or less. The step and touch potentials will be limited to acceptable limits as indicated in IEEE 80 Standard.

The earthing conductors will be dimensioned for carrying earth-fault current in any part of the plant for a minimum of half a second (0.5 s) without harm to the conductors.

During the detailed design phase, ground resistivity measurements will have to be done for the design of the earthing grid.

8.3.3.11 Indoor, Outdoor and Emergency Lighting

The lighting will be designed in line with industry practices and in conformance with the IES (Illuminating Engineering Society) lighting recommendations. The system will consist of the following:

- Indoor lighting system;
- Outdoor lighting system;
- Emergency lighting system.

8.4 PENSTOCK

The buried penstock is shown on Drawings 722736-0000-41DD-0011 and 722736-0000-41DD-0012 and is discussed in Section 6. The overall length of the penstock is 2,900 m. The penstock route will parallel the right bank of the river until it meets the powerhouse. The upper section of the penstock is approximately 1,900 m long and has an average slope of 5.5%. The lower section of the penstock has an average slope of 29.5%. The first 1,500 m of penstock will be constructed of high-density polyethylene and has a nominal diameter of 1.6 m. The remaining 1,400 m of penstock up to the bifurcation will be constructed of steel and is 1.52 m in diameter. A steel bifurcation is located just upstream of the powerhouse and will provide the required flow to each of the turbine generating units. Anchor blocks will be provided at critical changes in direction along the route of the penstock. No surge tank is required due to the turbine unit selected. See Figures 8-16 and 8-17.

Figure 8-16: Penstock Location Plan

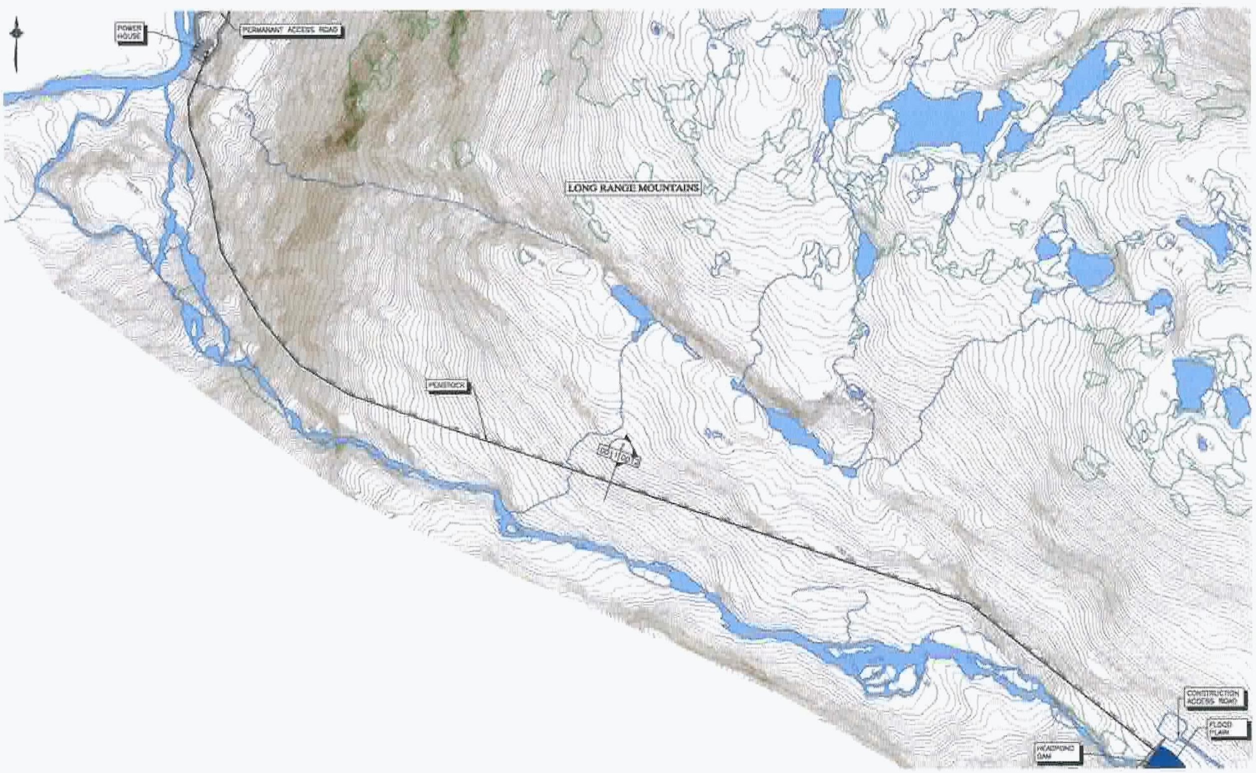
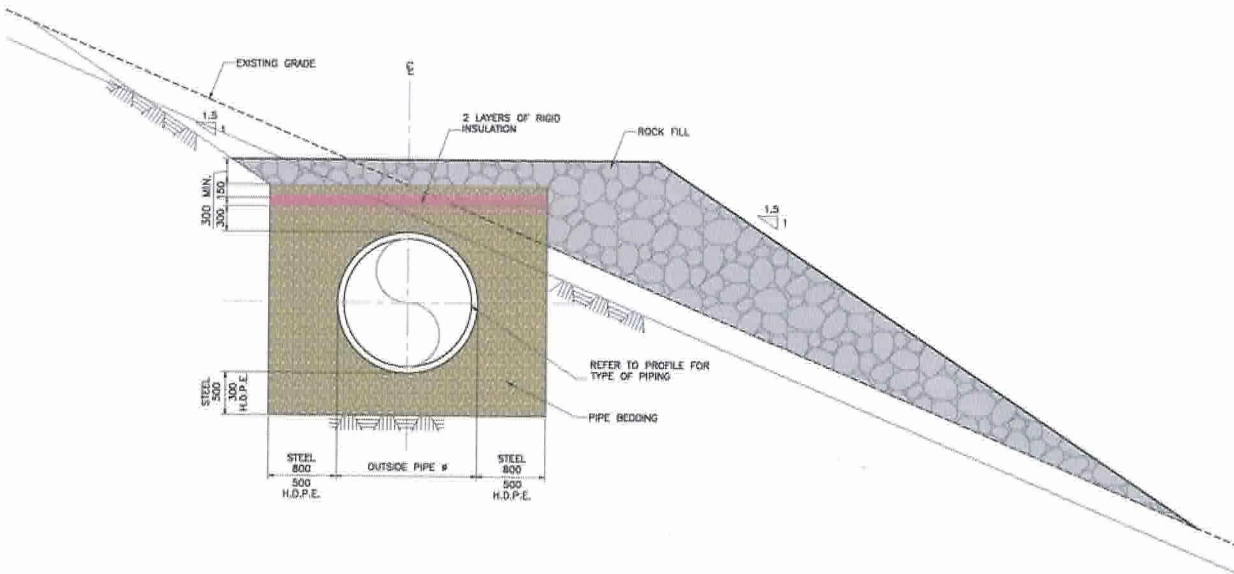


Figure 8-17: Penstock Section

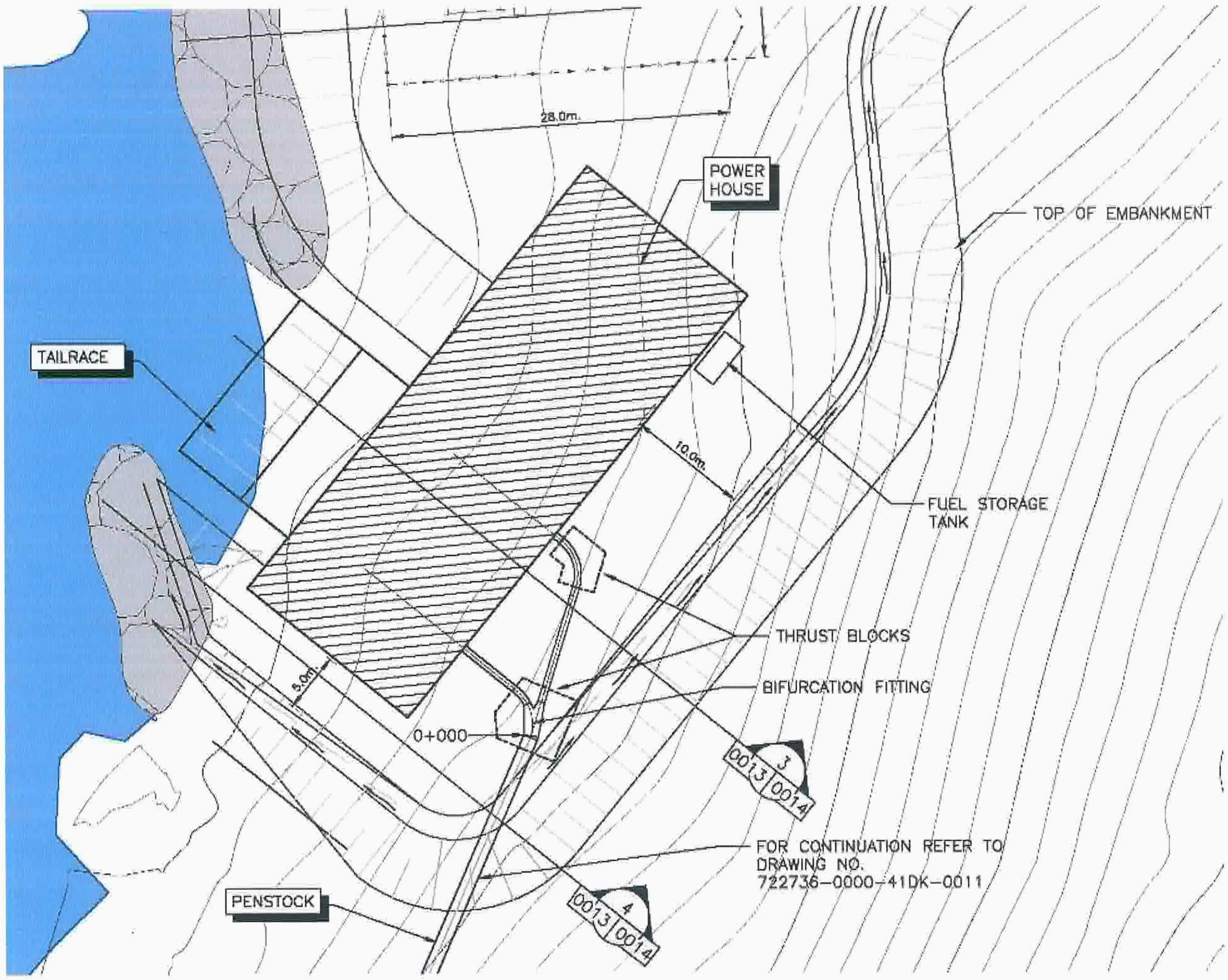


8.5 TAILRACE

8.5.1 General

The tailrace discharges the water from the turbines into Portland Creek. The water level in the tailrace is controlled in order to provide a sufficient volume of water for fire protection of the powerhouse. See Figure 8-18.

Figure 8-18: Tailrace



8.6 FISH HABITAT

The impact on Fish Habitat caused by the footprint of the Project has not yet been determined. For the purposes of this report, $0.5 million plus a conservative contingency of $0.5 million has been added to the capital cost estimate as an allowance for replacing habitat altered, disturbed or destroyed (HADD) by the Project.

8.7 SWITCHYARD

The switchyard will be located to the west of the powerhouse and will provide a line termination point for the new transmission line to Peter's Barren Terminal Station. A level area approximately 35 m x 30 m will be required to be constructed by the main civil contractor. See Figure 8-19.

Figure 8-19: Switchyard

The 66 kV switchyard apparatus will be provided and coordinated by the client and will have two step-up transformers, two 66 kV SF6 circuit breakers, five 66 kV disconnect switches, associated bus work, VT's and CT's, surge arresters, 66 kV cable potheads, grounding, foundations and fence. The output of each unit will be connected to the outdoor substation through its own transformer.

The AC and DC circuits required to power equipment in the yard will be incorporated in the powerhouse design.

The embedded earthing grid at outdoor switchyard and its surrounding area will be interconnected. The earthing grid will be designed in accordance with IEEE 80 standard and constructed for the operating voltages and short circuit capacity associated with short circuit and earth fault current levels.

8.8 TRANSMISSION LINE

One 27 km long, three-phase, single circuit, 66 kV transmission line will connect the switchyard to the existing substation at Peter's Barren. The route of the transmission line leaves the Portland Creek Plant, follows the proposed road along the north side of Inner Pond, and then runs relatively straight northwest to Peter's Barren. It is anticipated that a short section of line will be buried along Inner Pond as shown on Drawing No. 722736-0000-41DD-0001.

The line will comprise single wood pole structures with wood cross arms, davit arms, 559 MCM AASC conductor, line post and suspension insulators. A steel tower, or other special structure, will be required where the line diverges away from Inner Pond, where wood pole construction will be difficult. Span lengths will be approximately 60 m including the requirement of fibre optic communication cable attachment.

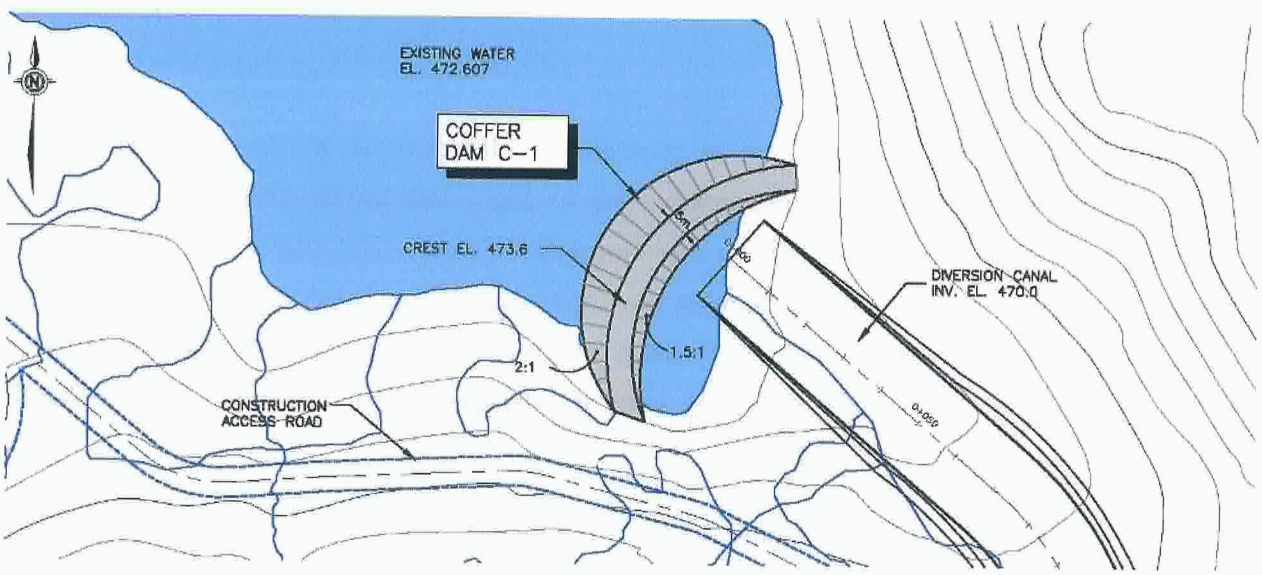
In addition, a distribution line will connect the powerhouse to the Head Pond Dam/Storage Dam structures. This three-phase, 25 kV line will follow the route of the construction access road, approximately 10 km.

8.9 DE-WATERING SCHEME

De-watering of the key Project components is shown on Drawing Number 722736-0000-41DD-0015, an excerpt of which appears below, and is recommended as follows.

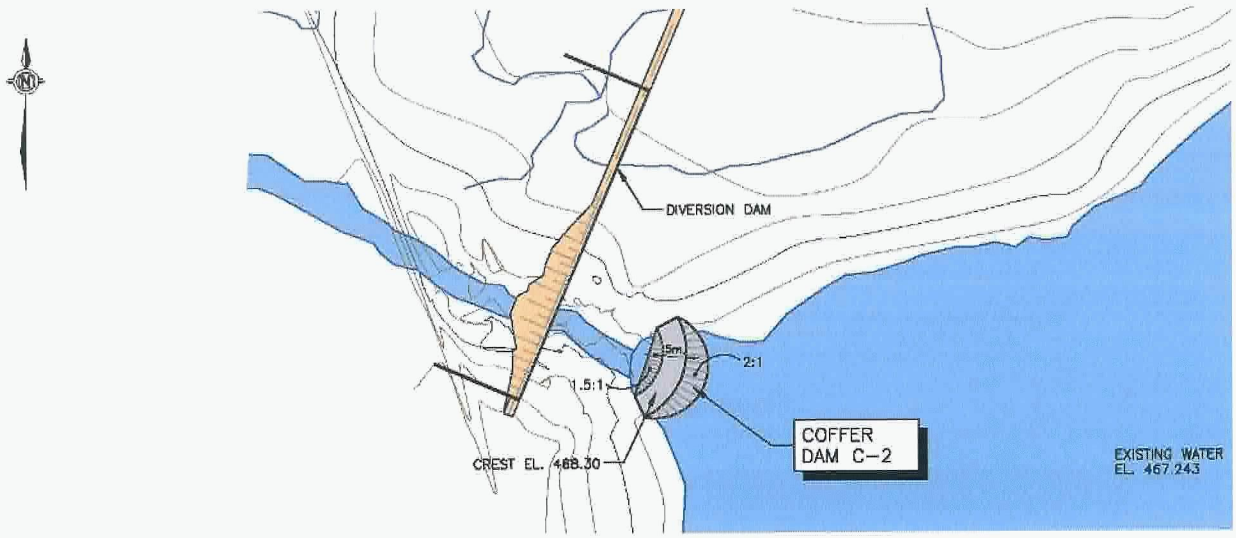
1. Construct a small cofferdam at the north end of the diversion canal (C1 – approximately 4 m high) to facilitate excavation of the canal and to prevent water from getting into the diversion pond from the storage pond. See Figure 8-20.

Figure 8-20: Cofferdam C1



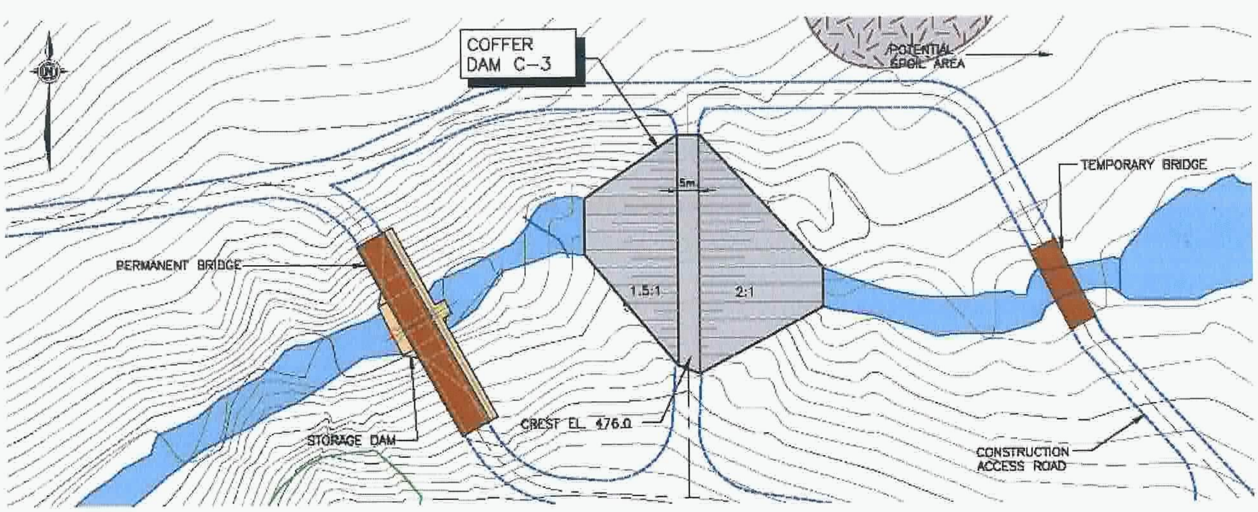
2. Construct a cofferdam to facilitate construction of the Diversion Dam (C2 – approximately 7 m high). When water levels equalize, cofferdam C1 will be removed to allow flow back through the storage pond. The diversion dam will be constructed in the dry. A portion of the dam could be completed while the diversion canal is being excavated. See Figure 8-21.

Figure 8-21: Cofferdam C2



3. When the diversion dam is completed, cofferdam C2 will be removed and a cofferdam (C3 – approximately 16 m high) will be constructed to facilitate construction of the storage and Headpond Dams. This cofferdam would divert water through the diversion canal. Spill would be over the diversion dam, which is the main spillway for the Project. The local drainage entering Headpond would be handled by pumping and settling basins during construction of Headpond Dam. See Figure 8-22.

Figure 8-22: Cofferdam C3



8.10 SUPPORT FACILITIES

It is recommended that the decision to construct a camp, or drive workers to the site, should be left to the contractors to decide. It is envisaged that the contractor engaged to do the access roads would build a temporary construction camp or avail of accommodations in Daniel's Harbour. $500,000 is included in the CCE for this purpose.

In the second year during construction of the powerhouse, penstock and dams it is envisaged that the contractor would construct a new camp. A site has been selected for the camp and is indicated on Drawing Number 722736-0000-41DD-0003. A cost estimate for developing a 60-man camp at this site is included in the CCE. The camp would consist of bunkhouses and a kitchen complete with treated water and sewer systems. As the camp would be for one construction season, recreation facilities would not be provided. A four-bedroom house-style trailer would be provided for NL Hydro staff, and would be turned over to the Owner upon completion of the Project.

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9 CONSTRUCTION METHODOLOGY AND SCHEDULE

9.1 CONSTRUCTION METHODOLOGY

The construction methodology for this Project is typical for heavy civil construction projects involving various types of earthworks, concrete structures, etc., and is based on two key activities:

- The first season would involve the construction of all support infrastructures, ie: access roads, bridges, excavation of the powerhouse/switchyard area and preparation of a campsite.
- The second season would see the installation of the camp, penstock, powerhouse, turbine and generator, the construction of the three dams and the excavation of the diversion canal.

The above works would be awarded under two tenders, with the owner providing detailed designs for the key Project components and the contractor to provide the site access, and related support operations required during construction.

9.2 CONSTRUCTION SCHEDULE

A proposed construction schedule as prepared in Microsoft Project is outlined on the following page, the basis for which is summarized as follows.

Year 1: Environmental approval process initiated, and completed early in Year 2.

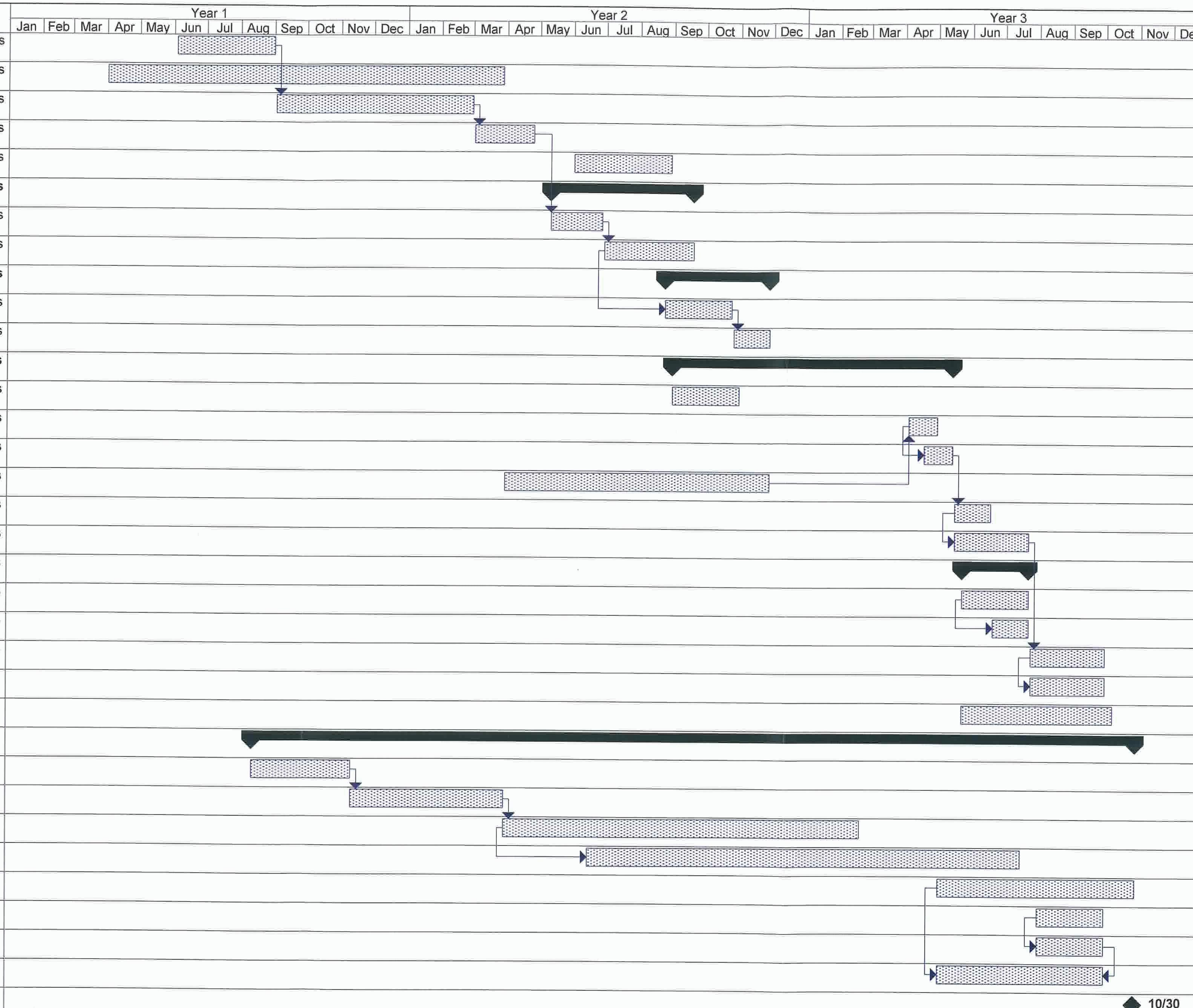
Undertake the final field program, final Project optimization, detailed design/engineering, and update the capital cost estimate.

Appoint the EPCM contractor.

Year 2: Construct access roads, excavate and backfill the powerhouse/switchyard area, construct HADD compensation structures and prepare the campsite.

Portland Creek Construction Schedule

ID	Task Name	Duration
1	**Field Program**	65 days
2	**Environmental Approval**	260 days
3	**Design/Engineering**	130 days
4	**Tender & Award: Package 1 - Items 5 to 13**	40 days
5	**HADD Compensation Structures**	65 days
6	**Permanent Access**	**95 days**
7	Existing Mine Road to Inner Pond	35 days
8	Inner Pond to Powerhouse	60 days
9	**Construction Access Roads**	**70 days**
10	P.H. Int. to Storage & Headpond Dams	45 days
11	Storage Dam to Diversion Dam	25 days
12	**Construction Camp**	**185 days**
13	Site Preparation	45 days
14	Water and Sanitation	20 days
15	Camp Trailers (Installation)	20 days
16	**Design, Tender & Award: Package 2 - Items 14 to 32**	174 days
17	**Diversion Canal**	25 days
18	**Diversion Dam**	50 days
19	**Reservoir Clearing**	**45 days**
20	Clearing Storage Reservoir	45 days
21	Clearing Headpond Reservoir	25 days
22	**Storage Dam**	50 days
23	**Headpond Dam**	50 days
24	**Penstock**	100 days
25	**Power Complex**	**579 days**
26	Turbine & Generator/Transformer Engineering & Package Definition	65 days
27	Turbine & Generator/Transformer Procurement Cycle to Award	100 days
28	Turbine & Generator/Transformer Supplier Engineering	234 days
29	Turbine & Generator/Transformer Manufacturing & Del'y	284 days
30	Powerhouse	130 days
31	Tailrace	45 days
32	Switchyard	45 days
33	**Transmission Line**	110 days
34	**On-Power Date**	1 day?



Project: Portland Creek Hydro Development
Date: Tue 3/6/07

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

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Year 3: Construct the powerhouse, switchyard, penstock, Headpond Dam, storage dam, diversion canal, Diversion Dam and reservoir clearing.

Construct the campsite including installation of water and sanitation facilities.

Construct transmission line.

The turbines proposed for this Project are basically catalogue items and model testing is not required. The key to completing the Project over a two-season construction period is establishing access to all key Project areas in the first year (i.e. the access roads will have to begin by May 1, Year 2). In order to achieve the proposed schedule, the field program and some engineering design must run concurrently with the Environmental Assessment Process. The design and tender period for Package 1 and the engineering/procurement period for the turbine-generator package may have to start before environmental release of the Project.

In summary, the schedule shows a 32-month Project with a power-on date of October 30 of Year 3. This assumes a Project release date no later than April 1st of Year 1.

9.3 CONSTRUCTION PLANNING

To undertake this Project, it is suggested that it be separated into three packages namely:

Package 1: Access Roads, Excavation for Powerhouse/Switchyard, HADD Compensation, and Preparation of Camp Site.

Package 2: Powerhouse, Switchyard, Penstock, Headpond Dam, Storage Dam, Diversion Canal, Diversion Dam, Reservoir Clearing and Construction of Camp Site including installation of water and sanitation facilities.

A tender for the turbine/generator package would be called after project sanction. The successful contractor would become a designated sub-contractor to the main civil work contract as part of this package.

Package 3: Hydro Contracts

Transmission Line

Switchyard

Protection and Control

Telecommunications

10 CONSTRUCTION COST ESTIMATES

10.1 BASIS FOR ESTIMATES

The cost estimates are based on historical unit prices for earthwork construction, and the cost is based on quantities calculated from survey information and concept design as presented in this Report.

10.2 CAPITAL COST ESTIMATE

Description	2006 Estimate
Reservoir Clearing	
Access Roads	
Diversion Canal	
Diversion Dam	
Storage Dam	
Headpond Dam	
Penstock	
Powerhouse	
Tailrace	
Switchyard	
Project Support	
HADD Compensation	
Sub-Total Before Contingency	54,396,396
Contingency	
Total	
Hydro Contracts	
Transmission Line	
Switchyard	
Protection & Control	
Telecommunications	
Sub-Total Before Contingency	5,047,768
Contingency (10%)	
Total	
Total Direct Cost	
Management & Engineering (13.5%)	
Owner's Costs (10% of Direct Cost)	
Escalation	
AFUDC	
Total Indirect Costs	
TOTAL ESTIMATED CAPITAL COST	

The detailed CCE is included in Appendix A. The CCE is based on December 2006 pricing with escalation effective the beginning of 2007. The CCE is based on the Project schedule included in Section 9.2 of this report. Escalation factors applied to the CCE are as follows:

- 2007 - 1.4%;
- 2008 - 2.6%;
- 2009 - 2.6%.

Allowance for funds used during construction is calculated at 7.53% for 2007-2009.

The following summarizes the Capital Cost Estimate for the Project and provides the cost per kilowatt (kW) of installed capacity and the cost per kilowatt hour (kWh) of energy.

- Total Direct Cost (including contingency) $65.84 M
- Total Indirect Cost $24.62 M
- Total Estimated Capital Cost $90.46 M
- Installed Capacity (MW) 23.0 MW
- Cost per MW of installed capacity $ 3.933 M/MW
- Cost per kWh of Annual Energy $0.639/kWh

The total estimated capital cost includes transmission line and switchyard cost.

10.3 CONTRACT VALUES AND CASH FLOW

The entire Project can be broken into the following contract packages as described in Section 9.3.

1. Package 1 - $14,752,000
2. Package 2 - $45,533,580
3. Package 3 - $ 5,553,000

10.4 RISK ANALYSIS

10.4.1 Introduction

NL Hydro and SNC-Lavalin agreed, during post proposal negotiations, that a Risk Analysis of the Capital Cost Estimate should be prepared. The exercise would facilitate the decision making process for NL Hydro in its assessment of the Project and the Project risk factors. It would also assist in the determination of the Project Budget Allocation.

The analysis is based on the identification of the main anticipated risk areas and quantifying the maximum overall range of likely outcomes for each. This should pull out the very best and very worst scenarios for a particular cost centre. The elements and the risk factors are entered in a program called "@ Risk" where they are processed by iteration using Monte Carlo Simulation.

The types of Risks to be considered were grouped into two categories:

- Technical Risks:
    - Level of Engineering completed at the time of the estimate;
    - Proven / Unproven Technology;
    - Geotechnical / Site Conditions;
    - Scheduling Considerations;

  - o Environmental Considerations;
  - o Major Equipment – Supply and Demand Considerations.
- Commercial Risks:
  - o Labour Market Conditions;
  - o Financing Variables;
  - o Insurance and Bonds;
  - o Legal Considerations;
  - o Labour Union Issues.

The Risk Analysis consisted of three key steps:

A. An assessment, by the Project team, of the Cost Estimate line-by-line, to establish the "least cost" and "highest cost" cases, with the "most likely" cost case being the estimate itself; no contingency considered.

B. An analysis of the two categories of risk as noted above.

C. An analysis of the above information using a Monte Carlo Simulation using "@ Risk" software.

10.4.2 The Risk Analysis

10.4.2.1 Step One: Assessment of Cost Estimate

A line-by-line analysis of the Capital Cost Estimate is presented in the following table.

RISK IDENTIFICATION TABLE

NOTE: ALL COST IN CAN$ AS PER DECEMBER 2006

RISK IDENTIFICATION	MINIMUM VALUE %	MOST LIKELY COST $	MAXIMUM VALUE %	MINIMUM COST $	MAXIMUM COST $
RESERVOIR CLEARING					
ACCESS ROADS					
DIVERSION CANAL					
DIVERSION DAM					
DAM					
HEADPOND DAM					
PENSTOCK					
POWERHOUSE					
MECHANICAL					
ELECTRICAL					
TAILRACE					
SWITCHYARD					
PROJECT SUPPORT CONSTRUCTION CAMP (200 Men)					
PROJECT SUPPORT PROVISIONAL ALLOWANCES					
HADD COMPENSATION					
DIRECT COSTS CONTINGENCIES					
HYDRO CONTRACTS					
HYDRO CONTRACT CONTINGENCIES					
INDIRECT COSTS					
OTHER RISKS					
Environmental Approval					
Penstock					
Construction labour					
ADJUSTMENTS					
TOTAL		90 461 053		78 870 965	106 697 995
		[illegible]			
Most probable cost					
Percent increase					

10.4.3 Technical Risk

The Technical Risk for this Project is assessed as follows.

<u>Level of Engineering</u>

The level of engineering for this Project is considered to be close to Feasibility Stage, which is sufficient to provide an accurate estimate for budgeting purposes.

<u>Proven / Unproven Technology</u>

The technology incorporated into this Project is consistent with that of previously completed projects of a similar nature. The risk related to this factor is considered minimal.

Geotechnical / Site Conditions

Some geotechnical information is missing, but the AMEC Geotechnical Report indicates an overall confidence in the soundness and quality of the rock. It is likely that further geotechnical drilling will not produce results that will adversely affect the budget and overall assessment of the Project.

Powerhouse Rock

Indications are that the rock in the area of the powerhouse is of good quality and not likely to be acid generating. The powerhouse location appears to have a suitable outcrop of rock, unto which the powerhouse sub structure can be built. Further investigation will be required to confirm this, but the risk of finding unsuitable rock conditions are considered very minimal.

Scheduling

Scheduling is the key risk Item identified in this Project and is one that particular attention should be paid too, both by the Owner and the engineers. In detail, the key items are as follows.

Access Roads

Six months are allowed for the construction and/or upgrading of access roads (37 km of road of which 27 km are new build) and site preparation in the first year. This may be tight. Any delay will push the access road construction into the next summer and could result in a two-three month delay in the final completion date. Allow a 20% factor.

Penstock

The recommended penstock scheme will have 1,500 m of 63" polyethylene pipe and 1,400 m of 63" steel pipe. The period from award to completion of the main civil works contract is about 12 months. Allowing two months for installation, this leaves ten months for fabrication and delivery of the pipe. This period should be adequate, however, the steel pipe is of a size, which cannot currently be rolled in

Newfoundland, and the polyethylene pipe will have to come from a plant in Montreal. There is also the issue of the availability of a welding machine for the polyethylene. This is a specialized machine, which may not be available without significant advance notice. Any delay will result in a similar delay to the completion (20%).

Environmental Approval (EA)

This could delay the start of the Project. Engineering already has to start in advance of environmental approval. A significant delay in EA could cause the Project to slip a year. Since there are some significant issues (e.g., Atlantic salmon, caribou, etc.) this risk should be assigned a probability of 50%.

Turbine Generator Package (T/G)

16 months are allowed for design, fabrication and delivery of the turbine generator assembly. This is based on the unit being a simple "catalogue item" with no model testing required. A delay in this item, or if model testing becomes necessary, could impact the completion date of the T/G erection activities, the powerhouse mechanical and electrical, commissioning and hence overall Project completion.

Environmental Considerations

Fisheries Issues

No detailed work was carried out on the HADD. Due to the anticipated minimal affect on fish that this Project will have, it is not seen as a major environmental or cost consideration. Contingent amounts are in the estimate, based on historical cost on other projects.

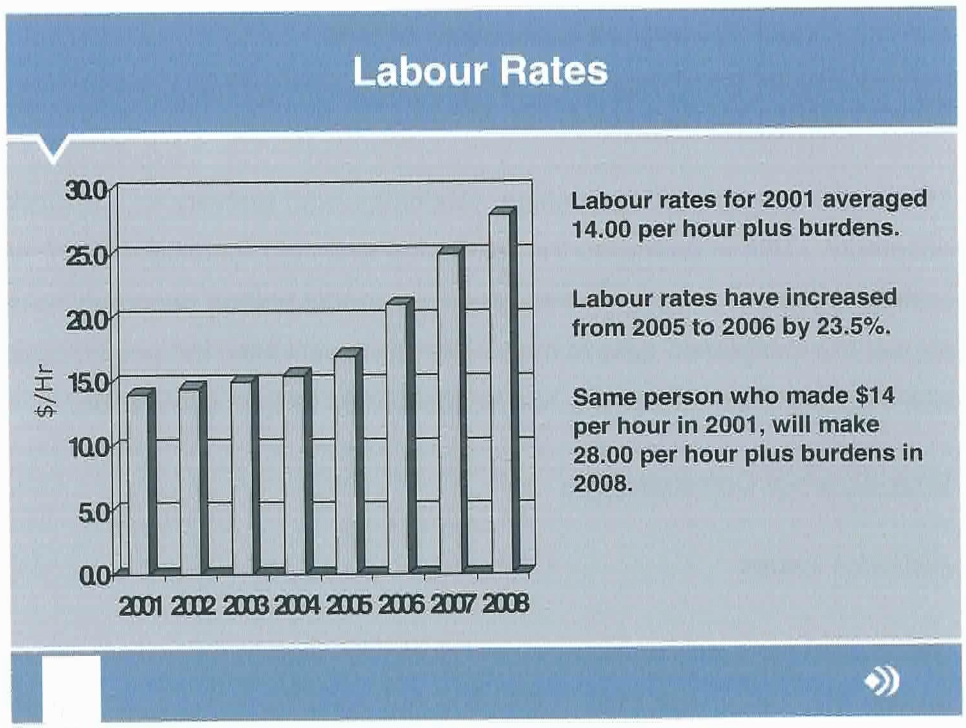
Major Equipment – Supply and Demand Considerations

Given the major energy and infrastructure construction boom that is going on around the world, the manufacturers of major components for this Project are very busy, and on time delivery of components could be a problem. This will be difficult to assess until the detailed engineering stage. However, it can be mitigated by an early start, and thus an early assessment of the problem and preordering of the affected components where necessary.

10.4.4 Commercial Risk

Labour Market Conditions

Labour Market conditions have changed considerably in the last 12 to 18 months in this Province, as our labour force comes under the effect of the significant labour shortage in Western Canada. This effect is outlined on the following chart:



To mitigate this, we have allowed for a significant cost of wages increase in the Capital Cost Estimate. This was done in consultation with our construction industry advisors. It is also expected that there will be several major projects proposed to be built in this province, (i.e., new nickel processing plant at Long Harbour, a new refinery at Come By Chance, etc.) that may compete with this Project for available skilled labour. Some element of cost is built into the estimate to cover this but it may be insufficient if all these projects occur simultaneously.

Financing Variables

The financial information used in the economic evaluation of this Project was limited to information supplied by the Owner. Other more detailed analysis we assume is the responsibility of the Owner.

Insurance and Bonds

This item is also the responsibility of the Owner, who provided a number in the Owner's cost of the Project.

Legal Considerations

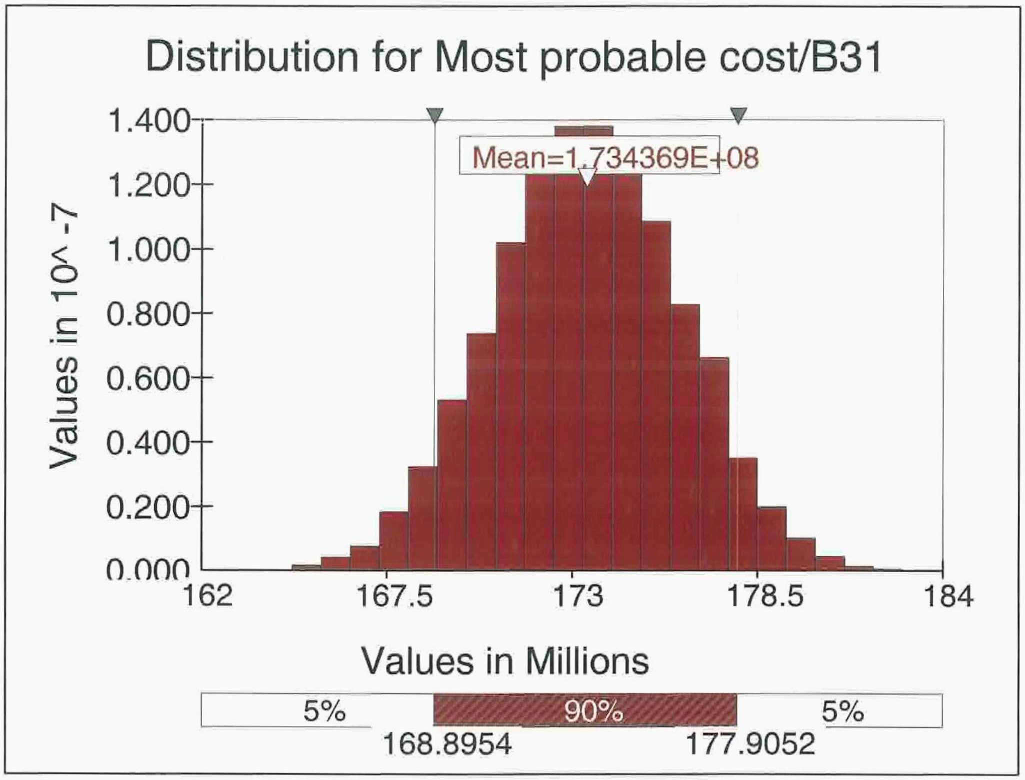
Same as above.

Labour Union Issues

The Owner instructed the team to assume that the Building Trades Council Agreement would apply to this Project.

10.4.5 Simulation Results to Determine Most Probable Cost

The Monte Carlo Simulation was preformed based on the above noted information and the simulation produced the following results.



Cost Estimate Probability
Values of Capital Cost Estimate

Probability Not to Exceed	Cdn $
5%	87,661,304
10%	88,208,344
15%	88,631,280
20%	88,950,992
25%	89,211,376
30%	89,459,056
35%	89,683,424
40%	89,901,488
45%	90,117,968
50%	90,350,344
55%	90,561,888
60%	90,775,080
65%	90,998,144
70%	91,228,992
75%	91,493,632
80%	91,768,288
85%	92,075,944
90%	92,474,664
95%	93,011,312

11 CONCLUSIONS AND RECOMMENDATIONS

11.1 CONCLUSIONS

It is concluded that:

1. This Project is technically feasible and can be readily constructed for an estimated capital cost of $90.46 Million including Owner's cost, allowance for funds used during construction, and allowance for escalation.

2. The estimated annual energy production for the development is 141.5 GWh/yr, with an optimum installed capacity of 23.0 MW.

3. The recommended arrangement for the primary Project structures is as follows:

   - Three concrete dams; Headpond, Storage and Diversion Dams;
   - A single penstock; 2.9 km long, 1.6 m diameter, with a combination of high density polyethylene and steel pipe;
   - Powerhouse with two Pelton turbines.

4. Geotechnical information provided in the AMEC Report does not appear to present any technical challenges outside the norm for a hydro development of this type.

5. Concrete coarse aggregate can be produced from rock excavated from the Diversion Canal and fine aggregate from borrow sources identified in the AMEC Report.

6. Access to the site should be constructed from the existing Daniel's Harbour Mine Road and consists of upgrading sections of existing forest access roads, constructing a new road along Inner Pond to the powerhouse site and construction access to the dam sites.

7. The impact the Project will have on fish habitat is to be determined.

8. The Project construction schedule is presented as a three-year schedule based on one year for Environmental Assessment and two years to construct the Project.

9. The turbines proposed for the Project are basically catalogue items, which reduce the delivery schedule.

11.2 RECOMMENDATIONS

It is recommended, should the Owner decide to proceed to Front-End Engineering, that the following engineering activities be initiated, namely;

1. A detailed field program be carried out this winter. Winter fieldwork would eliminate the need for helicopter support and reduce cost considerably.

2. Front-End Engineering should be carried out on all components of the Project.

3. Based on the additional field data and design information, cost estimates should be updated and the Project viability finalized.

Assuming Project viability, an EPCM Contract should be put in place and final engineering and Project implementation carried out.

In addition to the engineering activities identified above, it is also recommended that NL Hydro proceed with preliminary assessments of potential environmental/social/cultural concerns associated with construction of the Project and operation of the development.