Davies, James W FLNR:EX

From: Steeves, George <gsteeves@ameresco.com>
Sent: Wednesday, August 13, 2008 1:07 PM
To: Bennett, Timothy A ENV:EX; Davies, James W ENV:EX
Cc: s.22
Subject: Tyson Creek - leave to Commence Construction - Ammended (File no. 2002777; Licence No. C120523)
Attachments: Leave to Commence Construction - Tyson - Amended v2.doc; Monitoring Plan - Version 2.pdf; Drill Waste - CEMP.pdf; 2401_(0).pdf; 2301_(A).pdf

Tim/Jim

Please find attached the above Leave to Commence Construction. I have also appended the tunnel plan and profile as well as a drawing showing the proposed IFR infrastructure. I will send you the balance of the tunnel drawings tomorrow when i am back in my office. As part of the process i requested and received an updated Civil Design Basis as well as an amenedment to the CEMP to reflect the inclusion of the tunnel.

I also requested the completion of the final draft of the OEMP to reflect the tunnel (after a cryptic reminder from Scott B). I was surprised to discover this document was still in the draft stage. It is my understanding that the proponents consultant Dave Bates is working with Scott to get it into a close to final form, something that should have been done sooner.

They are continuing to move forward with the construction of the project based on the intitial Leave to Commence Construction, but are anxious to get going on the tunnel which requires the issuance of the second Leave to Commence Construction.

George

Davies, James W FLNR:EX

From:	Steeves, George <gsteeves@ameresco.com>
Sent:	Thursday, August 14, 2008 5:05 AM
To:	Bennett, Timothy A ENV:EX; Davies, James W ENV:EX
Cc:	s.22
Subject:	Tyson Creek - Leave to Commence Construction - Ammended (File no. 2002777; Licence No. C1205230
Attachments:	Drill Waste.pdf; 210_(0).pdf; 2008-06-11 IFR Tunnel Options.pdf; 02290 Tunneling.pdf; 2301_(A).pdf; 02390 - Soil and Rock Anchors_0.pdf; 2401_(0).pdf; 2411_(0).pdf; 2431 _(C).pdf; 2432_(C).pdf; 2455_(C).pdf; 02491 - Rock Grouting_A.pdf; Design Basis - Rev F 2008-07-18.pdf; Drill Waste - CEMP.pdf; Technical Specifications - Tunnel.pdf; Monitoring Plan - Version 2.pdf; Leave to Commence Construction - Tyson - Amended v2.doc

Tim/jim

Further to the documents I sent you yesterday, following are the additional drawings and documents wrt to the leave to Commence Construction. Some may be repeats.

George

George L. Steeves, P.Eng.

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WATER STEWARDSHIP DIVISION
MINISTRY OF ENVIRONMENT

TYSON CREEK HYDROELECTRIC CORPORATION
TYSON CREEK HYDROELECTRIC PROJECT
LEAVE TO COMMENCE CONSTRUCTION – AMENDED

George L. Steeves, P. Eng.
Independent Engineer
30 Catherine Ave.
Aurora, Ontario
L4G 1K5

TRUE NORTH ENERGY

August 8, 2008

File # 2002777
CWL # C120523

Water Stewardship – Lower Mainland
Ministry of the Environment
10470 152nd Street
Surrey, BC
V3R 0Y3

Attention: Mr. Timothy Bennett, P. Eng.
Section Head, Water Allocation

Dear Sir:

RE: **TYSON CREEK HYDROELECTRIC CORPORATION**
TYSON CREEK HYDROELECTRIC PROJECT
LEAVE TO COMMENCE CONSTRUCTION – AMENDED

Further to prior correspondence, please find attached a copy of the Leave to Commence Construction – Amended Report for your review and comments.

As you are aware, in the time since we prepared our first Leave to Commence Construction – Part 1 Report, the proponents of the Tyson Creek project have changed the project layout for economic and environmental reasons to include a direct lake tap and underground tunnel. This arrangement eliminates approximately 600 metres of penstock and the weir and intake structures at the outflow of Tyson Lake. The elimination of the weir and intake structures requires additional works to deliver the 70 L/s of Instream Flow Release ("IFR") stipulated in the conditional water licence to the original Point of Diversion ("POD") at the lake. In order to comply with the IFR requirements, the proponents have proposed a pump and conduit system be installed that will deliver water from the downstream tunnel portal overland back up to the POD. In addition the plant equipment design incorporates the provision to maintain a further bypass flow though the turbine in the instance the headpond water level is below the crest elevation at the discharge from the lake.

This amended Leave to Commence Construction is with respect to the required construction activities for the project up to the point of commencement of the diversion of water for the commissioning of the plant. This report reviews the transmission line and interconnection, access roads, powerhouse, penstock, headworks, tunnel, lake tap and IFR pumping system. This report specifically deals with the project General Arrangement and Design and Operating Criteria in accordance with the guidance provided by the "Scope of Information and Reports by the Independent Engineer".

Attached is the Civil Design Basis Report – Revision F prepared jointly by the Tyson Creek Hydro Corporation ("TCHC"), Kerr Wood Leidal Associates Limited ("KWL") and Gygax Engineering Associates Limited ("GEA") with respect to the project. Within the report is the relevant information with respect to the design requirements and construction standards to be utilized in the detailed engineering and construction process. Further attached are the Construction Environmental Management Plan and a technical memorandum that deals with care and control of tunnel drillwater, again prepared jointly by TCHC and FSCI Biological Consultants ("FSCI"). FSCI is the proposed Environmental Monitor for the project.

TCHC has entered into an agreement with Three Point Equipment ("Three Point") for the civil works, Canyon Turbines ("Canyon") for the turbine generator and Unit Electrical Engineering Limited ("UEE") for electrical works associated with the construction of the Tyson Creek generating station. I am in receipt of 31 sealed engineering drawings for Tyson Creek of sufficient detail to identify conformance of the proposed general arrangement with the conditions of the Conditional Water Licence. These drawings will be supplemented as the detailed design proceeds and the Independent Engineer will be reviewing the sealed engineering drawings as part of the review process for the issuance of the respective Leaves to Construct for the various elements of the project.

I am also in receipt of a technical design memorandum prepared by KWL that rationalizes the IFR pump system design and general arrangement. The Independent Engineer has reviewed the adequacy of the proposed system for delivering the mandated IFR's to Tyson Creek at the outfall of the lake as stipulated in the conditional water licence. The system has been designed with the appropriate ruggedness and redundancies required. The proposed system has been designed to operate in an ambient temperature of -20°C. The design provides for two separate pumps, each capable of delivering the entire IFR to the POD through a 470 m long, 250 mm diameter pipeline. Primary power will be provided from the powerhouse through a cable buried with the penstock.

The use of redundant pumps and a buried cable will reduce of the likelihood of system failure. With adequate control functions built into the plant's operating system and a rigorous preventive maintenance and testing regime, neither of which have been prepared or reviewed at this time, the Independent Engineer is of the opinion that the proposed system is adequate to deliver the required IFR's to the lake outfall.

In the event of plant shutdown during periods when Tyson Lake has been drawn down, flows downstream of the powerhouse will comprise the IFR, natural inflows into Tyson Creek downstream of the lake and partial flows through the powerhouse. The turbine deflectors have been designed for extended continuous use, essentially maintaining the plant flow at the time the plant went off line.

The proponent is in the process of completing the Operating Environmental Monitoring Program ("OEMP") and has initiated the collection of background data consistent with the requirements of the draft OEMP. It is the understanding of the IE that TCHC are moving forward with ongoing discussions with MOE-ESD with the objective of completing an appropriate OEMP, reflecting the environmental requirements of the site and the proposed operating regime.

Please accept this report as confirmation that it is the Independent Engineer's opinion that the General Arrangement, Design and Operating criteria are consistent with the terms of the Conditional Water Licence and meet accepted industry standards.

Yours very truly,

George Steeves

George L. Steeves, P. Eng
Independent Engineer
BC Licence #27528

WATER STEWARDSHIP DIVISION
MINISTRY OF ENVIRONMENT

TYSON CREEK HYDROELECTRIC CORPORATION
TYSON CREEK GENERATING STATION
LEAVE TO COMMENCE CONSTRUCTION - AMENDED

Table of Contents

LIST OF APPENDICES

1.0 INTRODUCTION

George Steeves, P. Eng. has been retained to serve as the Independent Engineer ("IE"), supporting the Ministry of Environment in the review of the design and construction of the 9.3 MW Tyson Creek Generating Station.

1.1 Objectives and Scope of Review

The role of the Independent Engineer is to (1) review the General Arrangement Plans and Design Criteria for conformance to the objectives of the Engineer under the Water Act; (2) review the detailed plans for conformance with the General Arrangement and the Design Criteria; (3) monitor the construction of the works for conformance with the plans reviewed under (2); (4) periodically submit reports to the Ministry of Environment on the results of the reviews and of the plans and the monitoring of the construction of the works; (5) provide Project Summary reports summarizing the construction of the works, including as-built drawings, and confirming the works as constructed are in accordance with the conditions of the Conditional Water License. This report is intended to respond to item (1) Review General Arrangement Plans and Design Criteria.

1.2 Project Description and General Arrangement

The Tyson Creek project is located in the southern section of the Tzoonie River valley. The Twoonie River flows south easterly to its confluence with Narrows Inlet which in turn is part of Sechelt inlet/ Tyson creek is a high alpine watershed with a northwest exposure, approximately 70 km northwest of Vancouver.

The proposed project develops the hydraulic resources of the lower reaches of Tyson Creek, with the point of diversion ("POD") 4.7 km upstream from the powerhouse which in turn is 500 m upstream of the confluence of Tyson Creek with the Twoonie River, 6 km upstream of its discharge into Narrows Inlet.

With a gross head of 868 m and a drainage area of 11.06 km^2, the project has a total installed capacity of 9.3 MW, and includes a submerged lake tap in the bed of Tyson Lake, a tunnel with concrete plug, penstock, powerhouse, substation and transmission line to the interconnection.

1.2.1 Access Road

To provide access to Tyson Creek, 15 km of existing roads will need to be upgraded and approximately 1 km of new access roads constructed to provide access to the powerhouse and tunnel portal locations.

The roads and required drainage works will be designed, constructed and maintained in accordance with the requirements of Ministry of Forests for road construction, and the appropriate recommendations of the Forest Road Engineering Guidebook, and the Forest Service Bridge Design and Construction Manual.

1.2.2 Lake Tap and Tunnel

Water will be withdrawn from the lake through a 400 m long tunnel driven into the lake bottom using lake tap principles. The tunnel will have a nominal 3 x 3 m horseshoe cross-section, enlarged locally to accommodate rock traps and chambers. The tunnel will fall at an 8% grade from the lake tap to the tunnel portal located on the access road, some 200 m west of the top of the steep penstock section.

The tunnel will be sealed with a concrete plug approximately 80 m upstream of the portal. The remaining 317 m of tunnel will convey water directly, under pressure. Upstream of the plug, there will be two rock traps excavated in the floor of the tunnel. The first will be directly below the lake tap and will be designed to capture the bulk of the rock from the final blast that will break into the lake. A secondary rock trap is located about 320 m downstream and just upstream of the concrete tunnel plug.

The concrete plug will be an 11 m long mass of concrete poured in two stages. A 1.2 m diameter steel pipe that will serve as the inlet to the penstock. The first stage of concrete will form the interface between the concrete and the rock wall of the tunnel. This will be pressure grouted through the concrete into the rock. The second stage of concrete will complete the plug and hold the penstock inlet pipe.

Downstream of the plug, there will be a chamber that houses the maintenance and penstock inlet gate. The water conveyance downstream of the plug will be through a 750 mm diameter steel pipe supported on concrete cradles. Finally, a portal chamber, containing the IFR pumping equipment and tunnel ventilation system will be excavated near the downstream end of the tunnel. The portal will be sealed by an exterior bulkhead wall incorporating a door for maintenance access.

The dry portions of the tunnel, downstream of the plug, will be lined with shotcrete to provide additional support and protection from damage of the steel penstock. A crushed gravel leveling course will be provided to ease maintenance traffic and a drainage ditch will evacuate any ground water weeping in.

The lake tap will be excavated from the tunnel in a single blast designed to drop the result waste rock into a rock trap below the lake. The tunnel will be advanced using standard drill and blast procedures to within 30 m of the lake. At this time, several pilot holes will be drilled through the rock in and to the lake in order to accurately map the rock profile in the bed of the lake. The pilot holes will be then sealed with grout. The tunnel will then be carefully advanced to within 5 m of the lake, the rock trap excavated and the final charges set. The concrete plug will then be constructed and the tunnel flooded with water. Once the tunnel is flooded, the final charge will be shot to break the final rock into the lake. By filling the tunnel with water, or using a "wet blast" technique, an onrush of water into the tunnel, which would have a tendency of washing blast rock down the length of the tunnel, will be avoided.

1.2.3 Instream Flow Release – Pump and Conduit

During periods of high flow, when the lake has not been drawn down below its natural outfall elevation of 1040 m, IFR's will flow naturally from the lake. During periods of lower inflows, when the project has drawn the lake level down below the sill of the outfall, IFR's will be pumped back up to the to the lake's outflow through a surface mounted pipeline from the tunnel portal.

The IFR pumping system will consist of a pair of variable frequency, horizontal split case style pumps located inside a chamber excavated at the portal of the tunnel. Each pump will be capable of lifting the full 70 L/second to the lake outfall over the entire expected range of net heads. The pumps will be attached to 470 m long, surface mounted HDPE (Sclairpipe) pipe with a diameter of 250 mm. The pipeline will be secured to the ground at nominal spacing that will be determined during detailed design.

Primary power will be supplied to the pumps through a buried cable from the powerhouse laid next to the penstock.

The system will require up to 57 kW (75 HP) of capacity and the range of dynamic net head will vary, depending on plant flows and lake levels, between 5 and 43 m.

1.2.4 Penstock

The high pressure steel penstock ("HPP") will originate in the concrete tunnel plug, 80 m upstream of the tunnel portal. The penstock within the tunnel will be mounted on concrete cradles. Once outside the tunnel portal, the balance of the penstock will run 4,040 m to the powerhouse. The average gradient over the penstock length is 19.4 %. The steepest section of the penstock is a short 440 m section, 250 m downstream of the tunnel portal. Over the 440 m, the penstock falls a total of 290 m with an average gradient of 65 %. It has a short 30 m section that has a slope in excess of 200 %.

The first 250 metres of penstock outside the tunnel will be buried in the access road. It then drops down an extremely steep slope with grades in excess of 200 %. The penstock will be supported by concrete of steel cradles with appropriately designed concrete thrust blocks at all vertical or horizontal direction changes. This section of the penstock will be above grade. The design will incorporate the required expansion joints.

For the balance of the steel penstock alignment (St. 1+210 to Sta. 4+620), the penstock will be buried. The design incorporates the interaction between the soil and the pipe to resist the thermal and hydraulic transient loads. The IE will be reviewing the detailed design for the high pressure penstock sections both with respect to the selected design criteria as well as the application to the actual design. Currently there are nine concrete thrust blocks contemplated in the lower high pressure penstock section.

Corrosion protection of the HPP will be provided by an external and internal epoxy coating which a known and reliable corrosion protection system.

At the tunnel plug, the steel penstock is under 75 m of static head. Over its length of 4040 m as it progresses down the valley to the powerhouse, the internal static head increases to a maximum of 868 m. It is anticipated that the steel pipe will be a combination of butt welded and bell and spigot welded steel section with bell and spigot being used for the initial low pressure sections and butt welds used for high pressure sections.

The design of the HPP and all the associated elements will be in accordance with accepted industry standards. Due to the public safety endangerment in the instance of HPP failure it is assumed that the most stringent of criteria will apply.

1.2.5 Powerhouse

The powerhouse will be constructed adjacent to Tyson Creek, approximately 600 m upstream from its confluence with the Twoonie River. The powerhouse will be a 20 m by 10 m pre-engineered building founded on a concrete foundation. At the powerhouse, the 700 mm diameter penstock will be reduced to 600 mm, pass through the turbine inlet valve ("TIV") and then be bifurcated to feed the two nozzles of the horizontal pelton turbines with a rated generator output of 9.3 MW.

The powerhouse generator floor is at elevation 173.8 m, well above the 1:200 year flood level of the creek at this location.

The powerhouse is a steel framed building with a building envelope consistent with the local building code requirements and operating conditions.

The powerhouse has sufficient space to accommodate the ancillary equipment required to support the operation of the generating equipment. In addition to the switch gear, protection and controls the powerhouse houses the hydraulic pressure units ("HPU"), TIV, and an assortment of smaller electrical equipment. The powerhouse will also house the dry type main transformer.

The powerhouse will have the required mechanical and electrical station services including exhaust fans, unit heaters, etc. The small office will house the computers which are used to operate the plant and will be the depository for one copy of all operating and maintenance manuals. The office will also house the various communication systems for contact with BC Hydro ("BCH").

The powerhouse will have a large rolling overhead door to facilitate the delivery of larger items. A 5 ton overhead crane services the entire building. In accordance with the applicable Building Codes it will have the required number of exits. The powerhouse will not have potable water or sanitary facilities.

1.2.6 Tailrace

The tailrace delivers the plant flow back to Tyson Creek. The initial 5 m section is concrete which is followed by a 15 m trapezoidal rip rap channel.

1.2.7 Generating Equipment

The selected generating equipment arrangement includes one two jet horizontal pelton turbines, directly coupled to a synchronous generator. The selection of the two jet pelton turbine allows the plant to operate at very high efficiencies over a wide range of flows varying from as little as 0.3 m^3/s

up to the maximum unit flow of 1.3 m^3/s. This is especially significant for smaller watersheds such as Tyson Creek because it can experience a wide range of flows on a daily and monthly basis.

The selection of Canyon, a recognized leading manufacturer of this type of equipment will ensure on time delivery and performance of the equipment when installed and commissioned. The protection, controls and switchgear are designed to meet industry standards as well as the interconnection requirements of BCH.

The turbine will be equipped with deflectors designed for extended continuous use, enabling flows to continue through the system and downstream of the powerhouse even during periods of extended shutdown.

1.2.8 Substation and Interconnection

An outdoor switchyard will be constructed adjacent to the powerhouse. Approximately 20.5 km of 25 kV wood pole transmission line from the switchyard to the interconnection with BC Hydro at Earle Creek. The route of the transmission lines follows existing FSRs where feasible. The last 2.2 km of the transmission line crosses over private lands for which the required easements have been obtained.

2.0 DESIGN & OPERATING CRITERIA

2.1 Design Criteria

The evaluation of the design criteria has two distinct aspects. First is the selection of the appropriate design processes, and the second is the technical specifications of the materials to be incorporated in the project. The technical specifications identify all the relevant codes to be utilized and material specifications and are consistent with industry standards. The applicable building standards have been referenced with respect to occupancy type, applicable building loads, fire protection, etc. The applicable design codes with respect to the design of structural elements have also been referenced.

The design of certain elements of a hydroelectric generating station are, however, unique, especially with respect to the design of the hydraulic elements including the weir, intake, penstocks and powerhouse. Further, the design and selection of the generating equipment including the turbine, generator, protection controls and switchgear is very site specific.

The referenced national design standards and the technical specifications are consistent with accepted utility standards and adhere to the conditions of the Conditional Water Licence.

2.2 Operating Criteria

Within the Design Basis Report and the appended Technical Memorandum – Tyson Creek IFR System, the proponent has described the operating procedures. The following discussion is a broad brush summary of those procedures

The tunnel portal will be equipped with an IFR pumping chamber that will house two horizontal split case style pumps, each capable of pumping the entire IFR up 75 m, through 470 m long, surface mounted pipeline to the natural outflow of the lake. The redundancy of the pump system has been designed to guarantee the release of the 70 L/sIFR of Tyson Creek. Inflows to Tyson Lake greater than the IFR and the plant flow will first fill the lake to its natural water level of 1040 m and then flow out over the natural lake outlet.

The generating plant will respond to the available inflows to the lake and any available drawdown storage in the lake as its normal operating regime. It will, however, experience faults both internal and external to the plant. To respond to the fault and to not damage the equipment, the plant will follow specified shutdown sequences depending on the type of fault. In the instance of an emergency shutdown, deflector plates will be deployed, which redirect the flows away from the runner, allowing the turbine/generator unit to come to a halt. With the flows diverted by the deflector plates, the flows in the system can be ramped down slowly.

The overall flow diversion and associated operating criteria is consistent with the maximum plant flow of the Conditional Water Licence and the other operating conditions of the licence. It is the conclusion of the IE that the operating criteria, with respect to flow utilization and diversion, is consistent with accepted industry standards, meets the technical requirements of the project and adheres to the conditions of the Conditional Water Licence.

2.3 Protection of the Environment

2.3.1 Construction Environmental Management Plan

The Construction Environmental Management Plan ("CEMP") provides for an extensive construction mitigation program, of which a significant portion is applicable to clearing and grubbing activities. These include defining the cutting limits, providing sediment control structures down slope of all activities, hand clearing on sensitive slopes, protection of riparian zones, elimination of grubbing activities next to wetlands, controls on stockpiles of soil and overburden, soil and overburden replacement, burning and disposal of trash, and reclamation/rehabilitation of work areas.

The environmental monitoring program will provide a continuous review of the construction activities to ensure all the requirements of the Forest Practices Code of BC Act are complied with.

2.3.2 Operating Environmental Monitoring Program

The Operational Environmental Management Plan ("OEMP") has been prepared by FSCI Biological Consultants on behalf of the project and reviewed by BC Ministry of Environment – Environmental Stewardship Division (ESD). It described the goals and methodology for the ongoing monitoring program that will take place over the next six years. Collection of pre-diversion baseline data will continue during seasonally appropriate windows through construction. Once the plant achieves commercial operation, collection of post-diversion values will continue for five years. At that point, statistically relevant impacts beyond a scientifically accepted range will bring about further agency consultation and appropriate remedial action.

The OEMP described reporting protocols and procedures and outlines specific biological values that will be measured. These include:

- Water Quality
- Stage versus Channel Geometry to develop wetted width, depth and velocity response to varying low flows
- Discharge versus Useable Area
- Macro-invertebrates

3.0 DOCUMENTS REVIEWED

3.1 Civil Design Basis Report – Tyson Creek Hydroelectric Project

The Civil Design Basis Report – Revision F prepared by KWL and GEA is comprehensive and covers the fundamental Design and Operating Criteria.

The report is divided into Section 1 – Executive Summary; Section 2 – Introduction; Section 3 – Description of Facilities; Section 4 – Site, Environment and Design Parameters; Section 5 – Facility Sizing; Section 6 – Civil and Structural Design; Section 7 – Powerhouse Mechanical Systems and Appendices.

The document sets out the design and operating criteria in the appropriate manner detail. This level of detail and the format of the document have been used in the successful implementation of other recent similar sized hydroelectric projects. It is the opinion of the Independent Engineer that the design criteria incorporated within the document is consistent with the requirements of the Conditional Water Licence and accepted industry practice.

3.2 Construction Environmental Management Plan – FSCI Biological Consultants

The CEMP prepared by FSCI is intended to provide the broad framework under which onsite construction activities are to be managed. The CEMP is intended to identify environmental issues and the monitoring and mitigation procedures to be followed during the construction period. It is complimentary to all environmental mitigation measures required by relevant legislation with the higher standard to be applied in each instance.

The report is structured by activity and divided into Section 1- Introduction; Section 2 – Best Management Practices; Section 3 – Environmental Management, Section 4 – Incident Preparedness and Emergency Response; Section 5 – Compliance Monitoring; Section 6 – Reporting; Section 7 – Commissioning.

An addendum memorandum was prepared that discusses the care and control of drill wastewater that will exit from the tunnel portal during the tunnel's excavation.

The CEMP is very comprehensive and exceeds the standards set with other plans developed and implemented on other recent hydroelectric projects of similar scale.

3.3 Project Drawings

The following listing of project drawings comprises the sealed general arrangement drawings submitted by the proponent is support of the Leave to Commence Construction. With each subsequent submission by the proponent for a Leave to Construct, there will be a revised and expanded list of engineering drawings which will reflect the specific Leave to Construct.

3.3.1 Kerr Wood Leidal

2468-001-001 – General Arrangement – Location Plan & Drawing List

2468-001-002 B – General Arrangement – Key Plan

2468-001-003 – Civil - General Arrangements

2468-001-010 – General Arrangement - Piping & Instrumentation Standards – Sheet 1 of 2

2468-001-011 – General Arrangement - Piping & Instrumentation Standards – Sheet 2 of 2

2468-001-012 – General Arrangements – Hydraulic Profile

2468-001-013 – General Arrangement - Piping & Instrumentation – Intake - Process Flow Diagram

2468-001-014 – General Arrangement - Piping & Instrumentation – Powerhouse - Process Flow Diagram

2468-001-015 – General Arrangement - Piping & Instrumentation – IFR Pumping System

2468-001-016 – General Arrangement - Piping & Instrumentation – Powerhouse

2468-001-310 B – General Arrangement – Intake – Tunnel Alternative – Direct to Lake

2468-001-321 C – General Arrangement – Intake – Headworks & Dam – Longitudinal Section

2468-001-401 A – General Arrangement – Penstock Alignment – Sheet 1 of 4

2468-001-402 A – General Arrangement – Penstock Alignment – Sheet 2 of 4

2468-001-403 A – General Arrangement – Penstock Alignment – Sheet 3 of 4

2468-001-404 A – General Arrangement – Penstock Alignment – Sheet 4 of 4

2468-001-410 C – General Arrangement – Penstock Sitework – Sections and Details

2468-001-451 B – General Arrangement – Penstock

2468-001-501 A – General Arrangement – Powerhouse – Site Plan

2468-001-505-1 A – General Arrangement – Powerhouse Elevations

2468-001-505 A – General Arrangement – Powerhouse Elevations

2468-001-511 F – General Arrangement – Powerhouse – Plans

2468-001-521 F – General Arrangement – Powerhouse – Longitudinal Section

2468-001-522 F – General Arrangement – Powerhouse – Transverse Section

2468-002-210 0 – General Arrangements – Tunnel – Plan, Profile and Typical Section

2468-002-2301 A – Tunnel – Overall Plan and General Arrangement

2468-002-2401 0 – Tunnel – Excavation – Plan and Profile

2468-002-2411 0 – Tunnel – Excavation and Support – Sections

2468-002-2431 C – Tunnel – Concrete Plug and Headworks Chamber – Structure Outline – Profile, Plan

2468-002-2432 C – Tunnel – Concrete Plug and Headworks Chamber – Structure Outline – Sections, Details

2468-002-2455 C – Tunnel Portal IFR Equipment Chamber – Structure Outline

3.4 Conditional Water Licence – Ministry of Environment

The Conditional Water Licence provides a number of conditions which must be met before the licensee can generate electricity utilizing the hydraulic resources of the stream for which the licence has been granted. It further defines the obligations of the Licensee to the Province of British Columbia.

4.0 RECOMMENDATIONS

The review of the General Arrangements, Design and Operating Criteria for the Tyson Creek Hydroelectric Project, as amended to include a lake tap and tunnel conveyance from Tyson Lake, as opposed to the previously reviewed weir and low pressure conduit, has confirmed that they are consistent with accepted industry standards and adhere to the conditions of the Conditional Water Licence. Further, they reflect the appropriate consideration for the environment, in that the works proposed are to be executed in a logical, safe and organized manner. It is the opinion of the Independent Engineer that the Engineer should provide the Leave to Commence Construction (Amended) for the overall General Arrangement, Design and Operating Criteria.

FSCI Biological Consultants

Memorandum

DATE: July 18, 2008

To: Peter Schober
Tyson Creek Hydro Corporation

From: D. Bates

RE: **Tunnel Construction and Drill Water Sump**

It is our understanding Tyson Creek Hydro will be constructing a tunnel for the intake and penstock from Tyson Lake. The tunnel will require drill waste water management on the downstream end of Tyson Lake. The following amendment is suggested for the Tyson Creek Hydro CEMP:

3.20 Tunnel Drill Waste Water

A tunnel will be constructed from Tyson Lake that connects the lake to the penstock. The tunnel will require extensive rock drilling and excavation. The drill process uses water for cooling that may be supplemented with water seeping into the tunnel from surrounding sources (groundwater). In order to protect surrounding environmental values the following processes will be implemented:

- *A sump measuring at least 3-m and 4-m and 1-m deep will be constructed at the tunnel entrance. The sump size will provide a collection and settling structure for the drill water, groundwater seepage and any associated suspended drill flour;*

- *The sump should not have a surface connection to drainage channels that may directly enter Tyson Creek. The onsite EM should be consulted if this presents problems:*

- *The sump should be lined and impervious as required. It may be constructed in impervious rock. The sump should have a downstream surface outlet that moves settled water into an adequately designed ditch constructed of pervious materials. The water should go to ground at this point or within a short distance downstream;*

- *Sediment control structures such as rock plugs and/or silt fencing may be required at the sump outlet in the event of heavy suspended sediment loading. The onsite EM should be consulted;*

- *Drill operations and associated fuel tanks must have appropriate spill response equipment on-site to deal with fuel and/or lubricants that enter the drill waste water sump and/or downstream ditch;*

- *The EM will monitor the condition and water quality within the constructed sump. Observations and recommendations will be reported in the environmental reporting.*

In discussion with the contractor (Procon Mining and Tunnelling Ltd) they felt confident waste water and suspended sediments would be minimal. This opinion is based on the drill core results reviewed.

FSCI Biological Consultants

Memorandum

DATE: July 18, 2008

To: Peter Schober
Tyson Creek Hydro Corporation

From: D. Bates

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- *The sump should be lined and impervious as required. It may be constructed in impervious rock. The sump should have a downstream surface outlet that moves settled water into an adequately designed ditch constructed of pervious materials. The water should go to ground at this point or within a short distance downstream;*

- *Sediment control structures such as rock plugs and/or silt fencing may be required at the sump outlet in the event of heavy suspended sediment loading. The onsite EM should be consulted;*

- *Drill operations and associated fuel tanks must have appropriate spill response equipment on-site to deal with fuel and/or lubricants that enter the drill waste water sump and/or downstream ditch;*

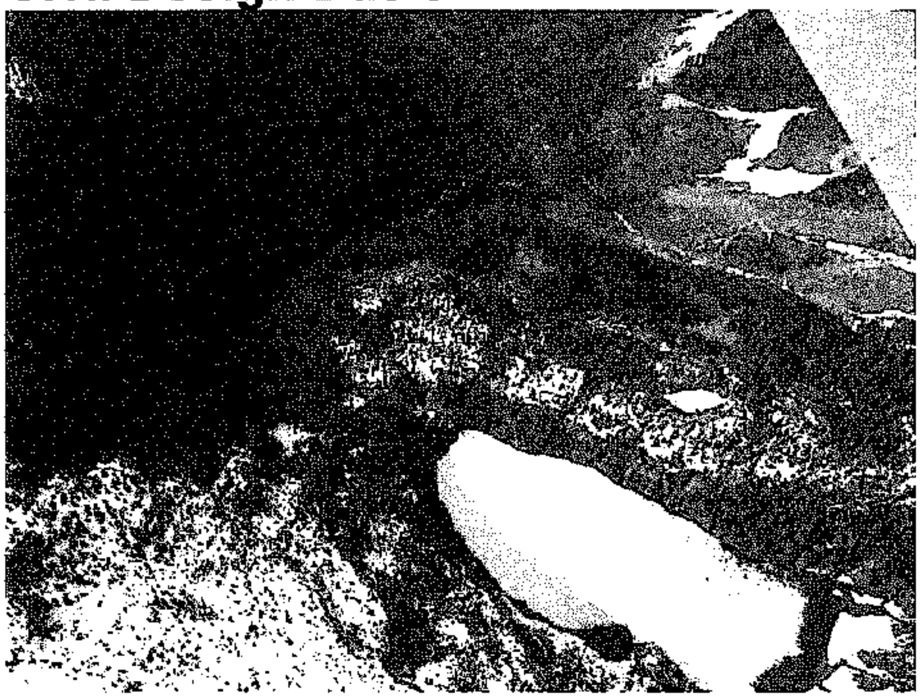
- *The EM will monitor the condition and water quality within the constructed sump. Observations and recommendations will be reported in the environmental reporting.*

In discussion with the contractor (Procon Mining and Tunnelling Ltd) they felt confident waste water and suspended sediments would be minimal. This opinion is based on the drill core results reviewed.

Tyson Creek Hydro Corporation

Tyson Creek Hydroelectric Project

Civil Design Basis



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CONTENTS

Appendices

1 EXECUTIVE SUMMARY

The Tyson Creek Hydroelectric Project is a proposed 9.3 MW plant located north of Narrows Inlet, a branch of the Sechelt Inlet approximately 70 km northwest of Vancouver, British Columbia. This Civil Design Basis establishes the basic criteria for the design and construction of the civil portions of the Tyson Creek Hydroelectric Project, specifically the intake, penstock, and powerhouse. The turbine, generator and electrical equipment, power line and access roads are being designed, supplied and constructed by others.

Preliminary Civil Design is now complete for the penstock and powerhouse. Further work is underway for the intake based on a request by Tyson Creek Hydro Corporation to revise the live storage design to 1.25 million m^3.

Based on geotechnical investigations undertaken in May and June of 2008, including a 420 m long pilot hole, a pressure tunnel with a lake tap design has been developed. The tunnel would be about 390 m long. Approximately 60 m from the downstream portal, a concrete plug would seal the tunnel. The penstock would start at this plug and the tunnel would be enlarged there to provide space for the maintenance and isolation gates.

The penstock runs approximately 4.25 km from the tunnel plug to the powerhouse. The first 200 m below tunnel portal will be buried in the access road. At about 250 m from the tunnel plug, the penstock transitions to a DN 750 mm steel section. The steel penstock traverses a steep slope above ground for 470 m before dropping back below grade to follow the access road for a further 3,180 m. The final 270 m above the powerhouse will be DN 700 mm buried steel penstock.

The powerhouse has a 20 m x 10 m concrete foundation and steel superstructure. It will contain the turbine inlet valve, turbine, generator, switchgear, transformer, controls and auxiliaries. Below the powerhouse, discharge is returned to Tyson Creek via a short tailrace.

Geotechnical investigations have been conducted and demonstrate acceptable conditions for construction of the facilities.

Hydrology reports by others estimate the mean annual flow for Tyson Creek as approximately 1.5 m^3/s, which exceeds the 1.4 m^3/s facility design flow. An independent review has found that these reports provide a reasonable indication of Tyson Creek streamflow. Based on preliminary facility design and hydrology, the plant is expected to generate approximately 57 GWh in a median flow year and about 48 GWh in a year with a 5^{th} percentile mean annual discharge.

The project is technically feasible and can likely be developed for commercial operation by late 2009. Table 1.1 provides summary data for the project.

Table 1-1: Project Summary Data

Intake		
Design Flow		1.4 m^3/s
Intake type		storage
Minimum Operating Water Level		EL 1,033 m
Maximum Operating Water Level		EL 1,043 m
Live Storage		1.25 million m^3
Dam		concrete
Spillway		sharp-crested 15 m wide passage
200 Year Flood Discharge		115 m^3/s
200 Year Flood Level		EL 1,046 m
Penstock		
Upper	DN 750 in tunnel, DN 750 buried,	50 m 200 m
Middle	DN 750 above ground steel	470 m
Lower	DN 750 buried steel DN 700 buried steel	3,180 m 270 m
Powerhouse		
Building Type		free-standing, concrete substructure, steel superstructure
Plan Dimensions		20 m x 10 m
Crane		traveling bridge crane 5 tonne hook height to main floor 6 m hook height to mezzanine floor 4.2 m
Main Floor Elevation		EL 174.20 m
Mezzanine Floor Elevation		EL 176.00 m
Generation		
No./Type of Units		1 @ 9.3 MW
		twin-jet horizontal axis Pelton
Rated Capacity		9.3 MW
Runner Elevation		EL 175.00
Control Room Location		powerhouse
Runner Maintenance Provision		runner removal from above
Isolation Systems		main gate at penstock inlet turbine inlet valve

2 INTRODUCTION

2.1 PURPOSE AND SCOPE

This Civil Design Basis establishes criteria for the design and construction of the civil portions of the Tyson Creek Hydroelectric Project. The document describes the project at the completion of the preliminary design phase and summarizes basic project site data, specified design criteria, relevant design guidelines, codes, and other key project information. The basic project features outlined in this Design Basis are intended to provide a foundation for the development of the detailed design. This document will also serve as the basis for negotiation with contractors to establish a construction contract for the project. It is recognized that this document may be revised during the detailed design development and construction as a result of site conditions.

The scope of this document includes the main project civil works, including the tunnel, penstock and powerhouse. The following components of the project are being completed by others and are not included in the scope of this document:

- turbine inlet valve, turbine, generator, control, switch gear, transformer and associated electrical and mechanical works;
- power line to connect the plant to BC Hydro near Egmont; and
- access roads.

This section of the Civil Design Basis includes an introduction and overview of the project, while Section 3 provides a more detailed description of proposed project facilities. Site conditions are described in Section 4. Section 5 addresses facility sizing and optimization, while parameters for civil and structural design are given in Section 6. Mechanical design requirements for the powerhouse are given in Section 7.

2.2 PROJECT OVERVIEW

The proposed Tyson Creek Hydroelectric Project is a 9.3 MW plant located on Tyson Creek, a tributary of the Tzoonie River. The Tzoonie River flows into the upper end of Narrows Inlet, a branch arm of Sechelt Inlet. The intake site was selected to take advantage of natural storage provided by a natural high-elevation lake at EL 1,040 m.

The powerhouse will be located at EL 175. With 865 m of available head, this is expected to be the highest-head project in North America. A peak turbine design flow of 1.4 m^3/s will be withdrawn from Tyson Creek to produce 57 GWh in a median flow year and about 48 GWh in a year with a 5th percentile mean annual discharge.

Tyson Creek Hydro Corporation (TCHC) has an Electricity Purchase Agreement (EPA) with BC Hydro and the project will connect to the BC Hydro grid near Egmont on west end of the Sechelt Peninsula.

Lake levels will be controlled to obtain a live storage range of about 10 m. The natural lake outlet will serve as the spillway. A pressure tunnel will draw water from near the lake bottom an daylight on the north slope of the ridge that forms the depression in which the lake has collected. The penstock will be approximately 4.25 km long and will include an upper lower-pressure section of buried steel pipe, a steep above-ground steel section, and buried steel pipe over the remaining alignment. The powerhouse will be a single-storey, semi-buried structure and will house the turbine inlet valve, twin-jet Pelton turbine, generator, switchgear, controls and balance-of-plant. Water will discharge back to Tyson Creek through a short tailrace channel.

Five major contracts are envisioned:

<u>civil engineering</u>: covering detailed design engineering for the intake, penstock, powerhouse, tailrace and site works. This report covers preliminary civil engineering.

<u>civil works construction</u>: covering construction of the intake, supply and fabrication of the penstock, powerhouse, tailrace, and site works. The contract would also cover associated hydro-mechanical and building services, mechanical and electrical components, and erection and commissioning of the water-to-wire equipment under the supervision of the equipment supplier.

<u>water-to-wire equipment supply</u>: covering energy production equipment including the turbine inlet valve, turbine, generator, and hydraulic governor. TCHC is currently finalizing a contract for this package with Canyon Hydro.

<u>electrical equipment</u>: covering specific elements of the electrical system including the transformer, switchgear, protection and control systems, and ancillary electrical equipment. TCHC is currently finalizing a contract for this package with Unit Electrical Engineering Ltd. (UEE).

<u>power line</u>: covering the 25 kV power line to the interconnection point with BC Hydro.

Access roads have been designed by others and are under construction.

The EPA between TCHC and BC Hydro requires revenue service by December 31, 2010. The target in-service date is November 2009.

3 DESCRIPTION OF FACILITIES

3.1 TUNNEL

Water will be withdrawn from the lake through a pressure tunnel driven into the lake bottom using lake tap principles. The tunnel will have a nominal 3 x 3 m horseshoe profile, enlarged locally to accommodate rock traps and chambers. The tunnel will fall at about 8% from the lake tap to the tunnel portal located on the access road, some 200 m west of the top of the steep penstock section.

The tunnel comprises the following features (see Drawing 210):

- A primary rock trap, located below the short riser section leading up to the tunnel mouth in the lake bottom.
- A secondary rock trap, located about 320 m downstream and just above the concrete tunnel plug.
- A concrete tunnel plug, that serves to seal the tunnel. The plug will be penetrated by a DN1200 pipe that serves as the inlet to the penstock and also allows access, by means of a manhole, through the plug into the tunnel upstream.
- A chamber, just below the tunnel plug, that houses the maintenance and penstock inlet gate.
- A portal chamber, containing the instream-flow release (IFR) equipment and tunnel ventilation system. The portal will be sealed by an exterior bulkhead wall provided with aa double door for access.

The unwatered portions of the tunnel downstream of the plug will be permanently lined with shotcrete. Aa crush gravel levelling course will be provided to ease maintenance traffic and a drainage ditch will evacuate any ground water weeping in. The DN750 penstock will be carried on concrete saddles along the right wall of the tunnel.

3.2 PENSTOCK

3.2.1 TYSON CREEK CROSSING AND LOW-PRESSURE SECTION

After exiting the trench, the penstock will cross Tyson Creek by means of a self-supporting steel pipe bridge with a span of about 24 m. Steel bents will be provided at both abutments to support the pipe bridge.

From the bridge to approximately Station 0+840, the penstock comprises DN 900 mm HDPE pipe laid in a trench within the bed of the access road. Careful attention will be given to ensuring drainage of the penstock trench to avoid erosion and to the construction of the access road to ensure stability of the penstock within the road.

3.2.2 EXPOSED SECTION

From Station 0+840 to about 1+210, the penstock drops down a steep slope. As this slope is largely bedrock with thin to no surface cover, the penstock will comprise an exposed steel pipe supported on concrete saddles. All bends will be harnessed by thrust blocks anchored to bedrock as required. The location of movement joints will be optimized to provide the most efficient control of thermal displacements. The exposed section is expected to have eight anchored bends.

3.2.3 HIGH-PRESSURE BURIED SECTION

From approximately Station 1+210 to the powerhouse (Station 4+620), the penstock will be buried steel pipe with a nominal diameter of 760 mm, decreasing to 700 mm for the last 270 metres. The pipe sections will be joined by butt welds and bends will be mitred to a radius of about 5 m. The buried section is expected to include 18 bends and a total of nine anchor blocks.

3.2.4 CONSTRUCTION AND FILLING

The construction of the buried sections is expected to progress concurrently on numerous fronts. For the steel pipe sections, attention must be paid to weld-up temperatures. Typically, solar protection will be required during mid-summer, and sections will need to be backfilled as quickly as is practicable.

It is anticipated that the steep, exposed section will be constructed by highline from the bottom up. This will allow the expansion joints to be set at the correct location to accommodate movements in both directions from the lock-in point.

The filling of high-pressure water conveyance systems must occur slowly under carefully controlled conditions to avoid dangerous transient pressure waves, air blockages, and blow-back. The filling operation is expected to take 6 to 8 hours. The intake gate will be opened gradually to control the filling rate. Air displaced by the water during filling will be exhausted through the air vent just downstream of the intake gate.

It is expected that the system will be emptied and refilled for inspection every 2 to 5 years. Complete operating procedures for control of the penstock head gate operation will be developed and discussed with the Owner prior to being set out in detail in a Operations and Maintenance Manual.

3.3 POWERHOUSE AND TAILRACE

The powerhouse will be located on the right bank of Tyson Creek where the stream branches approximately 800 m upstream of the Tzoonie River.

The turbine runner axis will be at EL 175.00 m, with the inlet pipe axis slightly higher at EL 175.50 m. A twin-jet horizontal-axis Pelton turbine will be connected to a generator to provide a nominal capacity of 9.3 MW. A turbine inlet valve will provide upstream isolation of the turbine assembly. Water will be discharged into a 3 m deep plunge pit from where it will drop into a tailrace channel and return to Tyson Creek. The powerhouse and tailrace are shown in Drawings 501 through 522.

The powerhouse structure has a plan footprint of 20 m x 10 m at the foundation and 23 x 13 m at the roof level. It comprises a single split-level storey, with the upper level (at EL 176.00 m) providing space for the indoor-type switchgear and substation equipment as well as mass-concrete anchorage of the inlet pipe. The lower level at EL 174.20 m provides space for the turbine/generator, lay down area, control room, control cabinets, and batteries. Other auxiliaries, such as hydraulic power units and the turbine bearing cooling system will be arranged along the walls. Access is provided through two roll-up doors and a number of person doors. A 5 tonne overhead crane will travel the entire length of the building.

The powerhouse will be constructed in an open-cast excavation and founded on native soils. The substructure will be of reinforced concrete. The superstructure will comprise conventionally-braced steel frames of moderate ductility with light gauge steel purlins and girts supporting the envelope system. The envelope system will comprise an inner steel lining, vapour barrier, insulation and exterior metal cladding or roofing. The roof will be mono-pitched and overhang at the eaves and gable ends. Flow through ventilation will be provided from low-set wall louvers on the south wall to high-set exhaust louvers on the north wall. Airflow will operate on negative pressures.

An adequate laydown and truck-manoeuvring area will be provided around the building. Exterior access to all sides of the structure will be provided.

4 SITE, ENVIRONMENT AND DESIGN PARAMETERS

4.1 GEOTECHNICAL CONDITIONS

4.1.1 GENERAL

Geotechnical conditions at the intake are discussed in detail in Appendix A. Geotechnical assessments for the balance of the site are discussed in Appendices B and C, which include the Geotechnical Investigation Report and Addendum to the Geotechnical Investigation Report, respectively. A summary of the intake, penstock, and powerhouse geotechnical conditions is provided below.

4.1.2 TUNNEL ALIGNMENT

The tunnel will run roughly north to south through a rock ridge that rises above the north shore of the lake. In May and June 2008, a pilot hole was drilled adjacent to the proposed alignment and generally competent granite was found along most of its length. Very little ground water was encountered except along the final 30 to 50 m as the pilot hole passed below the lake bottom. Ground level outcrop mapping was also carried out. The lake bed was investigated with a remote operated vehicle (ROV) in early July 2008, and the lake bed appears to generally be characterised by fine silts. Considerable woody debris is also present. From the apparent depth of penetration of boulders that have fallen into the lake, it can be suggested that the silt overburden is fairly thin.

The site investigations and tunnel construction recommendations are presented in Appendix H.

4.1.3 PENSTOCK ALIGNMENT

Below the tunnel portal, terrain along the penstock alignment is characterized by steep bedrock slopes partly overlain with colluvium. From the pipe bridge to approximately Station 0+840, the penstock will be placed in a trench that will be fully contained within the bed of the access road.

From Station 0+840 to approximately Station 1+210, the penstock will descend straight down a slope with an average grade of nearly 100%. Numerous sound bedrock outcrops will provide locations for supports and bend anchorages.

Between Station 1+210 (approximately EL 650 m) and the powerhouse, the penstock alignment follows the current forest road, which in turn generally follows the right bank of Tyson Creek. The soil conditions encountered at 18 test pit locations along this lower penstock alignment generally consist of thin to thick colluvial deposits comprising sand, gravel, cobble and boulders of variable proportions. The test pit logs indicate that these deposits range from dense to compact. In places, boulders may be too large to practicably excavate and may have to be split or blasted.

4.1.4 Powerhouse Site

The powerhouse will be located on a bench five to ten metres above the creek bed and approximately 30 m from the creek channel. Two test pits near the creek exposed a colluvium material generally consisting of clast-supported cobbles and boulders with a matrix of moist to wet sand, gravel, and silt with some organics. These soil deposits were inferred to range from compact to dense based on the difficulty of excavation.

Two additional test pits within the proposed building footprint exposed slightly looser and sandier materials.

4.2 Seismicity

A site-specific seismic risk assessment was obtained from the National Research Council's seismic zoning website. The seismic parameters for the governing return period of 2,500 years are:

- Peak ground acceleration PGA = 0.324 g
- Importance factor I = 1.0
- Site Class:
  - Intake A (hard rock)
  - Powerhouse C (dense soil)
- Spectral parameters:

Period, T	0.2 s	0.5 s	1.0 s	2.0 s
S(T)	0.70	0.51	0.294	0.16

4.3 Climate

4.3.1 Overview

The peninsula between Howe Sound and Egmont is subject to a maritime climate with moderate summers and mild winters, during which the area receives heavy precipitation from Pacific storms. At lower elevations, winter precipitation typically falls as rain and substantial snowpacks do not develop. As ground elevation increases, an increasing percentage of the annual precipitation falls as snow. The Tyson Creek watershed lies in a transition zone between lower-elevation watersheds (where runoff patterns are dominated by fall and winter rains) and higher-elevation watersheds (where runoff patterns are dominated by snow accumulation and a strong spring freshet).

Regional climate normal values indicate that temperature values are reasonably consistent in the Howe Sound region, but that precipitation values vary significantly by location. Tyson Creek is located in an area of relatively high precipitation.

4.3.2 TEMPERATURE

Design Temperatures

The 2006 BC Building Code (Appendix C, Division B) provides 2.5% exceedance design temperatures for several urban locations relatively close to the Tyson Creek project. The BCBC design temperatures represent values commonly used for the design of heating and cooling systems, with the assumption that these design conditions should be occasionally (but not regularly or greatly) exceeded.

The BCBC design temperatures are based on hourly ambient temperature records and do not include the effects of wind or solar radiation.

Transposing the nearby BCBC design temperatures to the project site using conservative seasonal lapse rates and an allowance for climate change yields the results below.

Table 4-1: January and July Design Temperatures for Tyson Creek

Location	Elevation (m GSC)	January (°C)	July (°C)
Powerhouse	170	-13	30
Intake	1,040	-21	27

If necessary, site-specific values can be obtained directly from Environment Canada's Atmospheric Environment Service during detailed design.

Average Summer and Winter Temperatures

To provide an idea of more typical summer and winter temperatures, 1971-2000 climate normal average daily maximum (August) and minimum (January) temperatures have been transposed to the project facility elevations using conservative seasonal lapse rates. The results are given in Table 4-2 below.

Table 4-2: Estimated Average Daily Temperatures for Tyson Creek

Location	Elevation (m GSC)	Average Daily Minimum (January, °C)	Average Daily Maximum (August, °C)
Tyson Creek Powerhouse	170	-2	23
Tyson Creek Intake	1040	-11	20

The above data will be used to estimate average-day heating and ventilation loads, as well as in establishing construction requirements such as maximum weld-up temperatures and concrete curing procedures.

4.3.3 SNOW AND RAIN

Typical precipitation values considered applicable to the Tyson Creek powerhouse location are shown below. These are approximately representative of the most conservative data provided in the Sunshine Coast Regional District building bylaw.

Total annual snowfall (equivalent depth of water):	115 mm
Total annual rainfall:	3,200 mm
Total annual precipitation:	3,315 mm
Maximum daily rainfall	330 mm

Based on the above data, the BCBC values for nearby stations and the relationship derived at Whistler for snow load as a function elevation, the following design snow and rain loads are applicable:

- Powerhouse:
  - Ground snow load $S_s = 5.0$ kPa
  - Associated rain load $S_r = 0.5$ kPa
  - Depth of snow pack 2 m
- Intake:
  - Ground snow load $S_s = 15.0$ kPa
  - Associated rain load $S_r = 0.5$ kPa
  - Depth of snow pack 5 m

4.3.4 WIND

Based on wind values for nearby stations, the following values are applicable at both the powerhouse and intake sites.

Table 4-3: Design Wind Values for Tyson Creek

Return Period	Reference Velocity		Reference Pressure
	v_{ref} (m/s)	v_{ref} (km/hr)	q_{ref}, (kPa)
10 years	25	85	0.35
50 years	30	105	0.55

4.4 HYDROLOGY AND FLOWS

4.4.1 BASELINE HYDROLOGY

Baseline hydrometric and hydrology studies for Tyson Creek were carried out by Gabe Sentlinger, M.A.Sc., EIT, of Aquarius R&D. The Instream Flow Measurement Program is attached as Appendix C and the Hydrologic Analysis as Appendix D. A review of these studies by Kerr Wood Leidal Associates Ltd. (KWL) concludes that the approach used by Aquarius R&D is generally consistent with accepted industry practices, and that the results provide a reasonable indication of streamflow on Tyson Creek.

Shown below is a summary of the typical streamflow results reported by Aquarius R&D. The data in the table are taken from Appendices B and C of Aquarius' Hydrologic Analysis report. Mean and median values are calculated from a 23-year series of synthetic daily average discharges for the 11.8 km^2 watershed above the proposed intake. The synthetic streamflow series is based on monthly multiple linear regression analysis of concurrent measured discharges at Tyson Creek and nearby Water Survey of Canada hydrometric stations on the Clowhom River and the Cheakamus River.

Table 4-4: Estimated Mean and Median Streamflow Data for Tyson Creek at the Intake

Period	Median Discharge (m^3/s)	Mean Discharge (m^3/s)
January	0.20	0.35
February	0.23	0.33
March	0.26	0.37
April	0.35	0.51
May	1.72	2.18
June	3.69	3.82
July	3.23	3.34
August	1.59	1.76
September	0.92	1.24
October	1.17	2.30
November	0.51	1.15
December	0.32	0.53
Annual	0.79	1.50

The Hydrologic Analysis prepared by Aquarius R&D includes a semi-quantitative analysis of uncertainty in the mean annual discharge. Based on this analysis, hydrologic uncertainty in the mean annual discharge is estimated to range from approximately ± 6.5% to 13%. This uncertainty is distinct from year-over-year hydrologic variability, which is implicitly considered in monthly and annual mean and median values for the 23-year synthetic flow series.

It is noted that total annual runoff for Tyson Creek is quite high for a watershed of this size. The high runoff is a response to the high precipitation experienced at higher elevations along BC's south coast and is similar to published values for other nearby watersheds.

4.4.2 Water Licence

As of September 2007, the BC Ministry of Environment is still adjudicating the water licence application for Tyson Creek (Application No. Z120523, priority date March 8, 2005). The water licence application requests a diversion flow of 2.5 m^3/s and storage of up to 1.25 million m^3.

4.4.3 Minimum Instream Flow Requirements

Based on 5% of mean annual discharge, Aquarius R&D has proposed a continuous minimum instream flow release of 0.07 m^3/s. The minimum flow release will be provided via a bypass valve off the penstock.

4.4.4 Flood Hydrology

The hydrology report prepared by Aquarius R&D presents several approaches for estimating the 200-year discharge at the intake site. The resulting estimates range from 62 m^3/s to 114 m^3/s with a recommended value of 87 m^3/s. KWL carried out an independent analysis of the synthetic streamflow record provided by Aquarius R&D and obtained a 200-year discharge estimate of 115 m^3/s.

The results of the KWL analysis generally concur with the more conservative values calculated by Aquarius R&D at all return periods from two years to 200 years. To ensure that the facility is not underdesigned, these more conservative estimates for clear-water design flows have been adopted for preliminary design. The resulting flow quantiles are shown in the table below.

Table 4-5: Recommended Clear-Water Design Floods at the Tyson Creek Intake

Return Period (years)	Average Daily Flow (m^3/s)	Instantaneous Flow (m^3/s)
2	12.5	18
5	20	30
10	25	40
20	30	50
50	40	70
100	45	90
200	55	115

Differences between the Aquarius R&D and KWL approaches will be discussed with the goal of reaching consensus during the detailed design phase.

4.4.5 Debris Flood and Debris Flow Hazards

Based on a field inspection and desktop review of available information, Frank Baumann, P.Eng., has concluded that the proposed works on Tyson Creek are not subject to debris flow or debris flood hazards (see Appendix B). The gradient of the wide channel upstream of Tyson Lake is insufficient to support debris flow and debris flood events initiated in the upper watershed.

4.4.6 Landslide-Generated Wave

Frank Baumann, P.Eng., carried out a field inspection and desktop review to determine whether mass wastage into the lake could result in waves capable of damaging or overtopping the intake area.

His review concludes that the only possible source of such activity is a bluffy area along the middle south shore of the lake; however, this area is found to consist of competent granitic rock cut by widely-spaced, near-vertical joints. The area does not exhibit any evidence of instability (e.g., tension cracking or sagging). The bluffs are therefore unlikely to be a source of rockfall or rockslide activity of sufficient volume to create significant waves in the lake.

Given the time that has elapsed since glacial recession, the absence of any evidence of recent rockfall activity below these bluffs also implies that rockfall activity is not a significant issue in this area.

4.5 MAPPING, SURVEYS AND CONTROLS

Initial planning and feasibility assessments for the Tyson Creek Hydroelectric Project utilized provincial TRIM mapping and forestry mapping provided by Interfor Forest Products Ltd. The Interfor mapping provided 10 m contour interval mapping for the lake, penstock, and powerhouse sites. Provincial TRIM mapping provided 20 m contours for the remaining areas from Narrows Inlet to Earle Creek.

Lake soundings were taken during the early stages of the project and served as the basis for extrapolated bathymetric contours prepared by Aquarius R&D.

The above mapping provided a good basis for the project feasibility studies but is insufficient for preliminary and detailed design engineering. LIDAR mapping has subsequently been completed for the project site, including the area around the lake as well as the penstock and powerhouse areas.

4.5.1 LIDAR MAPPING

Terra Remote Sensing Inc. completed data acquisition for the LIDAR (LIght Detection And Ranging) mapping on July 10, 2007. The total area of mapping and digital imagery was approximately three square kilometres. The data collection specifications provided by Terra Remote Sensing include a relative accuracy of better then ±0.15 m on hard surfaces and absolute accuracy for individual points on the order of 0.30 m. The digital orthomosaic was created from the aerial photography at 0.15 m pixel resolution.

4.5.2 GPS SURVEY CONTROL

Under the direction of KWL, Bazett Land Surveying Inc. provided survey control for the project as well as an accurate bathymetric survey of the headpond lake.

During their first site visit, Bazett provided GPS control to the area. Lake soundings were then collected using a boat-mounted digital depth sounder connected to a laptop computer and a

Real-Time Kinematic (RTK) GPS receiver. Several photo targets were also set during this site visit to confirm the accuracy of subsequent air photo and LIDAR data.

All project mapping has been prepared in ground-level measured distances. To obtain ground level distances from UTM coordinates, the following equations can be applied (based on conversion from UTM Zone 10 to Local-Ground Level at 600 m):

1. Scale Adjustment: Multiply Northing and Easting coordinates by 1 / 0.99953759
2. Northing Shift: N - 5502552.645
3. Easting Shift: E - 400209.324

The equations given above were sent to Terra Remote Sensing to facilitate production of the LIDAR mapping in the project's ground-level coordinate system. Additional mapping parameters provided to Terra Remote Sensing included the following:

- Horizontal Datum: NAD83 (CSRS)
- Vertical Datum: CGVD28
- Projection: UTM Zone 10
- Height Transformation (ellipsoidal to Geodetic): HTv2.0 (Geodetic Survey of Canada)
- Active Control Stations used:
  NANO (Nanoose)
  WSLR (Whistler)
  BCOV (Beaver Cove)
  CHWK (Chilliwack)

Accuracy of GPS

All 3D-GPS positions were transformed to UTM Zone 10-Orthometric. Measured ellipsoidal heights are transformed to orthometric using the Geodetic Survey of Canada's HT2.0 height transformation. The Geodetic Survey of Canada provides a quoted absolute accuracy for this transformation of "±5 cm in the southern regions".

The positions of the local GPS base stations (IPRock #10000 and SPK #9928) were established by processing 9 hours and 8 hours, respectively, of static observations to the four Western Canadian Deformation Array (WCDA) stations listed above (NANO, WSLR, BCOV, and CHWK). The reported absolute accuracy (95% confidence) of the final positions for IPRock#10000 and SPK#9928 is 0.015 m horizontal and 0.030 m vertical. All other GPS measurements are based on these two positions with a redundant check tie between them at SPK #9926.

4.5.3 TOPOGRAPHIC SURVEYS

On July 4, 2007 the first of three site visits by KWL completed the topographic survey at the proposed intake site. Subsequent site surveys were completed on August 13-17 (Powerhouse) and August 28-31 (steep portion of penstock from Access Road Station 0+550 to 1+944).

The resulting topographic surveys have confirmed that the vertical accuracy of the LIDAR mapping is within 1 m in most areas.

4.5.4 FUTURE SURVEYS

A survey of the newly-constructed intake access road will be necessary to facilitate the detailed design.

KWL also plans to perform a control survey from the intake to the powerhouse prior to construction. The purpose of the control survey will be to "tighten up" measured observations obtained under difficult circumstances. This work would include the use of RTK GPS, Robotic Total Stations, and differential levelling equipment. These surveys will also provide additional survey control between the bottom end of the above-ground penstock section (Station 1+210) and the powerhouse site (Station 4+600). There is presently no survey control in this area.

4.6 SITE ACCESS

Access to the project area is by boat, barge or seaplane to the head of Narrows Inlet. Direct access to the intake site is also possible by helicopter. In 2007, the existing forest service roads along the east side of the Tzoonie River valley were upgraded and extended from Narrows Inlet to the powerhouse site. This work was undertaken directly by TCHC.

Construction of the access road from the powerhouse to the intake was also undertaken directly by TCHC in 2007. Where the penstock is buried in the access road, the construction of the access roads is critical to the stability and structural integrity and performance of the penstock. To ensure that the road design can accommodate the penstock, KWL has worked with the road designers to develop the following general requirements:

- Maximum grade 20 %
- Minimum grade, where penstock shares alignment 1%
- Minimum roadway width, where penstock shares alignment 7 m

Frank Baumann, P.Eng., was retained by TCHC to review the proposed road alignment from EL 600 m to the intake. His review found that the new road crosses terrain where construction is not likely to be very difficult. No areas where unstable ground could impact road construction were identified. While there is some rockfall hazard in areas of steep talus and bedrock, Baumann's review notes that it will not affect road construction or create an unacceptable slope hazard.

This section of road has been designed by others according to the general requirements outlined above and the following summary of additional specifications proposed by Frank Baumann, P.Eng. Detailed requirements are listed in Appendix B.

- Several potentially volatile creeks cross the lower stretch of the road. Every watercourse will be culverted or, preferably, will have a combination of a culvert and swale. A sufficient number of cross-drain culverts will be installed to prevent excessive runoff retention on the upslope shoulder, and the road will be seasonally deactivated with a sufficient number of cross-ditches when it is not being used.

- The creek crossing between road chainage 1+160 and 1+230 is particularly prone to flash floods and will be crossed using a broad swale or ford with a culvert pipe to convey normal runoff.

- From approximate road chainage 1+230 to 1+350 m, a full bench cut is needed to accommodate the very steep terrain.

- The penstock will be located in the road prism from about road chainage 1+900 up to the intake at road chainage 2+794. Depending the equipment required to install the pipe, the road should be widened in this section to at least seven metres. It should also be kept as straight as possible in sections where a steel penstock will be used.

At the request of TCHC, KWL prepared a preliminary hydraulic model of Tyson Creek at the proposed upper road crossing near the intake. The goal of the model was to establish an approximate minimum low chord elevation at the bridge structure so that the road profile could be designed all the way up to the intake. KWL found that the permanent bridge crossing at Tyson Creek should have a minimum low chord of EL 1,016.5 m, corresponding to the water surface of the 200-year reservoir outflow plus 1.5 m freeboard. This elevation applies to the bridge location shown in the preliminary design drawings just downstream of the steel pipe bridge across Tyson Creek and will have to be updated if the bridge location is changed.

During the detailed design phase, KWL will refine hydraulic design criteria for the upper and lower bridge crossings of Tyson Creek to limit potential impacts to the penstock bridge (at the upper crossing) and the powerhouse (downstream of the lower crossing).

5 FACILITY SIZING

5.1 FINANCIAL PARAMETERS

This section outlines the process by which major facility components are sized and optimized. This includes the selection of a live storage amount based on the time value of stored water, which leads to selection of spillway and intake elevations. These elevations in turn can be set by a combination of rock cut (for the intake) and concrete fill (for the weir and/or dam).

The financial analyses are based on the following basic parameters:

- return period 20 years
- discount rates (net of inflation): 17 %, 12%, and 7.5%

The value of energy is set by the BC Hydro F2006 Open Call for Power – Small Project EPA, included as Appendix F to this report. The Initial Bid Price for this project is defined in Appendix F as $101/MWh.

5.2 LIVE STORAGE VOLUME AND RESERVOIR OPERATING LEVELS

The live storage volume for the Tyson Creek Hydroelectric Project is selected based on an analysis of the relationship between live storage volume, construction costs, and project revenue. For a given quantity of storage, Present Value (PV) construction costs are calculated and compared to the PV revenue generated over the 20-year design life. The analysis was completed in 500,000 m^3 storage increments up to and including 3,500,000 m^3.

The resulting series of cost curves (one for each discount rate) can be used as a decision making tool by the Owner. Each curve compares gross revenue, net revenue, and capital cost with storage volume. Costs are clearly identified as actual or incremental.

Detailed bathymetric and topographic data are used to determine the actual storage volume at various combinations of intake elevation and spillway elevation. The combinations are evaluated at minimum 1 m increments in elevation. The least-cost combination of surcharge (dam) and drawdown (trench) storage will be used to evaluate the cost for each increment of storage.

Based on the above analysis and an evaluation of acceptable risk, the Owner has selected a live storage volume of approximately 1.25 million m^3. The least-cost alternative for 1.25 million m^3 combined surcharge and drawdown storage results in a maximum normal operating level EL 1,043 m and a minimum operating level of EL 1,033 m.

5.3 PENSTOCK SIZING

The penstock will be sized to meet the following two criteria:

- produce at least 9.3 MW at full pond, measured at the tail leads of the generator; and
- maximize revenue vs. construction cost over the project return period.

System curves are developed to describe how the net available head at the powerhouse varies under different conditions. The net available head is converted into energy, and then into revenue on a net present value (NPV) basis. Additional details on the approach to penstock sizing are provided in Appendix G.

Friction losses in the penstock were estimated using the Darcy-Weisbach method. The roughness coefficients used are shown in Table 5-1.

Table 5-1: Roughness Coefficients for Pipe Friction Calculation

Pipe Material	Smooth Coefficient (mm)	Rough Coefficient (mm)
HDPE	0.002	0.009
Steel	0.031	0.061

Based on the above analysis, the penstock will consist of varying lengths of DN 900, DN 760 and DN 700 pipe.

5.4 TURBINE/GENERATOR SIZING

The turbine, generator, and appurtenant works will be sized by the supplier based on a nominal capacity of 9.3 MW.

6 CIVIL AND STRUCTURAL DESIGN

6.1 DESIGN APPROACH

All engineering design will be carried out in SI units (i.e. kN, MPa, mm, etc.). Engineering design will use a Limit States Design approach. All elevations will be in metres above Geodetic Survey of Canada (GSC) datum CGVD28.

6.1.1 FACILITY CLASSIFICATION

The power project is designed as a post-disaster facility. The BCBC 2006 classification is Division F Class 3. The design life of the facility is 50 years. The overall downstream consequences of failure, per Canadian Dam Association Guidelines and BC Water Regulations (44/00), are rated as LOW.

6.1.2 REQUIRED PERFORMANCE LEVELS

The civil structures will be designed to perform as specified under the following performance levels:

- Normal Operation Condition: The facilities operate normally.
- Emergency Operation Condition: The facilities operate without damage but components may be at their design limit.
- Survivability Condition: The facilities may suffer local permanent movement or deformations, as well as minor, repairable damage.

The design of the facilities will consider the component performance limit states appropriate for each performance level.

Where a limit state check requires factored loads, the load factors and combinations stipulated by BCBC will be used. Examples of factored loads include:

- gravity loads due to self weight, usage, and equipment;
- creep and shrinkage effects;
- rock and soil pressures;
- environmental loads due to snow and wind; and
- equipment operation loads.

For certain loads and effects, discrete load levels are defined for the each performance level and no further load factors are appropriate. Examples of unfactored loads include:

- water levels and flow at the intake;
- pipe pressures and thermal effects; and
- sesmic loads.

6.1.3 MATERIAL PARAMETERS

The following is a summary of the principal material parameters assumed for the preliminary design. More detailed information and requirements can be found in the relevant material specifications and actual test results.

Geotechnical Design Parameters

Geotechnical design parameters for the penstock and structures in loose ground are derived from twenty test pits and eight plate bearing tests carried out in the summer of 2007.

- temporary excavations:

  maximum slopes for temporary unshored excavations less than 6 m deep 3H : 4V

  Grade all areas adjacent to excavations to drain water away from the excavation.

  Store and stockpile materials more than 2 m or half the excavation depth away from excavation edges.

- backfill pressures:

  Select native free-draining granular materials, compacted in controlled lifts, will be used for structural backfills. The following strength parameters are appropriate:

active soil pressure coefficient	$k_a = 0.22$
at-rest soil pressure coefficient	$k_o = 0.45$
but not less than the following compaction pressure	20 kPa
passive soil pressure coefficient	$k_p = 8.0$
drained shear strength	$ø' = 38°$
total unit weight of backfill material	20 kN/m^3

  The active seismic pressure component on vertical faces of height H [m] by the Monobe-Okaba relationship is:

  an inverted triangle with a maximum value of 2.7 H [kPa]

- penstock lateral support:

  The penstock will be DN 900, DN 760 or DN 700 pipe. Subgrade reactions of soil on the pipe will be within the following ranges:

up to EL 600 m	$k_h = k_v$ = 5 to 15 MPa/m
above EL 600 m	$k_h = k_v$ = 15 to 30 MPa/m

  For thrust blocks where thrust is into the ground, values equal to one-half of those specified above are appropriate. Ultimate resistance should be within the following limits:

horizontal component, at depth z [m] below grade	p_{hu} = 160 z [kPa]
vertical component	p_{vu} = 600 kPa

  A resistance factor, $ø_{soil}$ of 0.5 should be applied to the above when comparing with factored or survivability performance level loads.

- penstock bend anchorage:

  For anchored bends, double-corrosion protected anchors will be used. Anchor sizes will be based on the following:

matching performance level for prestress	Normal operation
pre-stress level	0.6 ultimate tensile strength
soil-grout transfer strength at prestress level	160 kN/m

- powerhouse foundations:

  The powerhouse will be founded on colluvium that is generally free-draining. Foundation design parameters are:

frost protection depth	450 mm
minimum spread footing dimension	600 mm
minimum strip footing dimension	450 mm
factored bearing resistance	p_r = 600 kPa
modulus of subgrade reaction (vertical)	k_v = 15 MPa/m
seismic site classification	Class C

- intake foundations

  The dam and headworks structure are founded on competent granitic rock. Foundation design parameters are:

rock excavation slope	0.2H : 1V

factored bearing resistance	p_r = 2000 kPa
interface friction coefficient for concrete cast directly against rock	μ = 1.2
seismic site classification	Class A

Concrete Design Parameters

Concrete mix types and corresponding design properties are as follows:

- Concrete Mix C15 (for concrete blinding on foundation inverts):

specified cylinder compression strength at 28 days	f'_c = 15 MPa

- Concrete Mix C25 (for mass concrete):

specified cylinder compression strength at 28 days	f'_c = 25 MPa
fly ash portion of cementiceous materials	10 - 40 %
95th percentile tensile strength	$f_{ctk,0.95}$ = 3.0 MPa
5th percentile tensile strength	$f_{ctk,0.05}$ = 1.5 MPa
modulus of elasticity	E_{cm} = 21 000 MPa
Poisson's ratio (cracked section)	ν = 0.17

- Concrete Mix C35 (for structural reinforced concrete):

specified cylinder compression strength	f_{ck} = 35 MPa
fly ash portion of cementiceous materials	5 – 40 %
characteristic flexural tensile strength	$f_{ctk,0.95}$ = 4.2 MPa
5th percentile tensile strength	$f_{ctk,0.05}$ = 2.2 MPa
modulus of elasticity	E_{cm} = 28'000 MPa
Poisson's ratio (cracked section)	ν = 0.17

Reinforcement Design Parameters

All reinforcing steel will be CSA G30.18 Grade 400W, weldable. Design parameters are:

yield strength	f_y = 400 MPa
ultimate tensile strength	f_{tk} = 550 MPa
elongation at yield	γ_y > 0.2 %
ductility	$\gamma_{50\ mm}$ > 12 %
modulus of elasticity	E_{sk} = 200'000 Mpa

Penstock Materials

Penstock steel work, including line pipe, bends, appurtenances and integral stiffeners, ring girders and other supports will be of grades suitable for variable pressure and temperature conditions. Pipe will be manufactured and shop-tested per AWWA C200.

- HDPE line pipe Type III, Category 5, Class C Grade P34 to ASTM D1248:

cell classification to ASTM D3350	PE3408
design stress	15 MPa

- steel line pipe, longitudinal or spiral welded, to ASTM A139 Grade E or API 5L X52, ASTM A1011 HSLAS Grade 55, or ASTM A1018 HSLAS Grade 55:

yield strength	$f_y = 360$ MPa
ultimate tensile strength	$f_u = 455$ MPa
elongation at yield	$\gamma_y > 0.18$ %
ductility	$\gamma_{50\ mm} > 22$ %
modulus of elasticity	$E_s = 200'000$ MPa
Poisson's ratio	$\nu = 0.3$

In addition, for pipe walls greater than 16 mm, the required impact energy, measured per ASTM A370 at 0° C on three specimens is:

16 mm ≤ wall thickness < 25 mm	$CV_{average} = 27$ J
	$CV_{lowest} = 20$ J
25 mm ≤ wall thickness < 38 mm	$CV_{average} = 34$ J
	$CV_{lowest} = 27$ J

- plate work, including fabricated appurtenances, integral stiffeners, ring girders and other supports, plate to CSA G40.21 Grade 350 WT, Category 1:

yield strength	$f_y = 350$ MPa
ultimate tensile strength	$f_u = 450$ MPa
elongation at yield	$\gamma_y > 0.18$ %
ductility	$\gamma_{50\ mm} > 22$ %
modulus of elasticity	$E_s = 200'000$ MPa
Poisson's ratio	$\nu = 0.3$

- pipe lining:

  Line pipe will be internally coated with urethane epoxy to AWWA C210.

- pipe coating:

  All penstock components will be externally coated with urethane epoxy to AWWA C210.

Backfill Materials

Pipe bedding and pipe zone backfill will consist of a single material that is either 19 mm minus or bedding sand. Adequate drainage will be provided and will consist of a combination of trench dams, perforated drainage pipe, filter fabric and drain rock.

Structural Steel

Steel sections and corresponding design parameters are as follows:

- I sections, welded wide flange members, structural platework CSA G40.21 Grade 350W:

yield strength	f_y = 350 MPa
ultimate tensile strength	f_{tk} = 450 to 650 MPa
modulus of elasticity	E_s = 200'000 MPa

- square hollow structural shapes (HSS) CSA G40.21 ASTM A 500, Grade C:

yield strength	f_y = 345 MPa
ultimate tensile strength	f_{tk} = 450 to 600 MPa
modulus of elasticity	E_s = 200'000 MPa

- C and L sections and anchor rods, CSA G40.21 Grade 300W:

yield strength	f_y = 300 MPa
ultimate tensile strength	f_{tk} = 450 - 620 MPa
modulus of elasticity	E_s = 200'000 MPa

- high-strength bolts, ASTM A325:

yield strength	f_y = 417 MPa
tensile strength	f_u = 830 Mpa

- weld material:

  Weld material to match the base metal per CSA W59:

for Grade 350W, 350WT and A139 steels	E49XX
for Grade 300W steels	E43XX

Timber

Timber will be Douglas Fir Larch species, possibly locally sourced and visually graded.

- glulam members

grade of flexural members	Grade 24f-E
grade of compression members	Grade 16c-E

- structural timber members — DF-L Select Structural

6.1.4 GOVERNING DESIGN CODES AND STANDARDS

[1] British Columbia Building Code, 2006

[2] CSA A165 Standards on Concrete masonry Units, 1994

[3] CSA A23.1 Concrete Materials and Methods of Concrete Construction, Canadian Standards Association, 2004

[4] CSA A23.2 Methods of Test for Concrete, 2004

[5] CSA A23.3 Design of Concrete Structures, Canadian Standards Association, 2004

[6] CSA G30.18M Billet Steel Bars for Concrete Reinforcement, 1992 (R2002)

[7] CSA O86 Engineering Design in Wood Canadian Standards Association, 2001

[8] CSA S16 Design of Steel, Canadian Standards Association, 2001 (R2005)

[9] CSA S304.1 Masonry Design for Buildings (Limit States Design), 1994 (R2001)

[10] CSA W47.1 Certification of Companies for Fusion Welding of Steel Structures, 2003

[11] CSA W59 Welded Steel Construction (Metal Arc Welding), Canadian Standards Association, 2003

[12] CISC Canadian Institute for Steel Construction, Handbook of Steel Construction, Eight Edition, 2004

[13] CPCA Concrete Design Handbook , 3rd edition, 2006.

[14] Canadian Wood Council, Wood Design Manual, 2005

[15] Canadian Foundation Engineering Manual, Canadian Geotechnical Society, 1992

[16] ASCE – Manuals and Reports on Engineering Practice No. 79 -Steel Pipeworks (for tunnel liner and general conveyance piping design).

[17] AISI - Steel Plate Engineering Data, Volume 4: Buried Steel Penstocks (for general conveyance piping design).

[18] AWWA – Manual M11: Steel Pipe, a Guide for Design and Installation (for coating and lining design).

[19] Eurocode 2: Structural Concrete (for crack control limit state).

6.2 HYDRAULIC AND HYDROMECHANICAL DESIGN

6.2.1 PERFORMANCE REQUIREMENTS

Design Water Levels

The following elevations will be used for design based on the operating range set out in Section 5.2:

Maximum (1 in 200 year) Water Level	EL 1,043.0 m
Natural lake outlet	approx.EL 1,040.0 m
Maximum Normal Operating Level	EL 1,040.0 m
Minimum Normal Operating Level	EL 1,030.0 m

Operating Rules

The Minimum Normal Operating Level is based on assumed seasonal power flows, a realistic estimate of monthly inflow, and an acceptable (but non-zero) probability of exceedance. Table 4-4 shows that expected monthly inflow is less than the plant design capacity of 1.4 m^3/s for seven months of the year. Operating rules will therefore be necessary to ensure that the water levels do not fall below the Minimum Normal Operating Level.

Energy production estimates quoted in this report have been produced by hindcasting using the synthetic streamflow data provided by Aquarius R&D. Actual energy production is expected to vary based on the operating rules adopted.

Submergence

In the event that the lake is drawn down below the minimum operating level, minimum submergence of the tunnel inlet will dictate the lowest lake level. The minimum submergence will be calculated by the Hecker (1987) equation:

$(h''/B)_{critical} = 1.0 + 2.3F_r$

where h'' is the submergence above the soffit of a horizontal intake of height B, and F_r is the Froude Number.

If the required depth is excessive, it may be slightly reduced by increasing the intake diameter. If the intake is oversized, a long-taper reducer would be used. The will be eccentric with constant crown elevation [BC Hydro Section 3.5.2 (a) (v)].

6.2.2 TUNNEL DESIGN

The tunnel will be designed in accordance with accepted rock mechanics and tunnelling design practices as well as Worksafe BC and Mines Act requirements.

Conventional Tunnel Drive

The tunnel will be driven from the portal on the north side of the rock ridge towards the lake in approximately a north-south direction and at a rise of about 8%. On open trench will be used to develop a portal headwall of adequate height to safely start the drive and to allow for later widening of the portal for the IFR chamber. It is anticipated that a nominal 3 x 3 m horseshoe profile will first be advanced about 340 m. Three classes of temporary rock support are foreseen, ranging from:

Class I: spot bolting as required in strong, jointed, rock, to

Class II: rock bolts in the arch with shotcrete and or chain link mesh as required in strong rock with localized shears and closely jointed seams, to

Class III: pattern rock bolts in the arch and walls with shotcrete on the full arch and on the walls as required in extensively sheared, closely jointed rock.

The Rock Tunnelling Quality Index, Q, per Barton (1974) will be used.

Lake Tap

The lake tap procedure has been developed in general terms, but will be finalized using an Observational Approach as the tunnel is advanced over the final 50 m or so. Over this stretch, groundwater inflow will be controlled through injection grouting and probe holes will be drilled in advance of the face to ensure that the ground conditions, and the precise location of the lake bottom are well understood.

In general terms, a semi-wet tap is foreseen. Once the tunnel has been advanced to the point where one final blast remains and the upstream rock trap has been excavated, construction will focus on establishing the headworks chamber, concrete tunnel plug and the secondary rock trap. Once the concrete plug has been constructed (see below), the final balst will be charged and the tunnel upstream of it will be flooded leaving only a pressurized air cushion just below the final lake tap rock plug. This air cushion will help attenuate the blast pressures travelling down the flooded tunnel and acting on the concrete plug, while the flooding of the tunnel itself avoids a sudden in-rush of water that would carry blast rock and debris down to the concrete plug and potentially block the penstock inlet. The air cushion pressure will be carefully maintained prior to the blast through compressed air lines from the headworks chamber.

Concrete Plug

The concrete plug upstream of the headworks chamber keeps the lake from draining out of the tunnel. The tunnel will be enlarged so that the concrete plug is wedged against the rock surface. The plug geometry will be selected so that it can safely withstand blast and hydrostatic pressures.

Pattern rock dowels will be used to enhance interface friction between the plug and the rock further. Once the enlarged excavation is complete, a vertical grout curtain will be established to consolidate the rock and ensure the plug cannot be bypassed by substantial seepage. The length and spacing of the grout curtain boreholes will be established once the rock at that location has been mapped from within the initial tunnel drive.

The plug will then be cast in two stages. The first stage comprises a concrete ring 500 to 1000 mm thick cast against the rock excavation. Once this ring has cured and aged about 28 days contact grouting will be undertaken through a pattern of holes drilled through the concrete into the rock. This serves to further consolidate the rock at the excavation boundary and fill any gaps that may have formed due to concrete shrinkage or incomplete air evacuation at the crown during concrete placing. The inner surface of the first stage ring will be roughened. A system of injection pipes and collection vents will installed respectively in the invert and crown of this ring and two circumferential waterstops installed near the upstream and downstream limits of the plug.

After the penstock inlet pipe, bypass, compressed air and other piping that will penetrate the concrete plug is installed, the second stage will be cast. Once this has cured, the interface between the two stages is grouted through the injection and vent pipes previously installed in the first stage. The up and downstream waterstops will serve to confine the grouted surface to a know area.

Unwatered Tunnel and Chambers

A permanent shotcrete lining will be provided on the arch and walls of the headworks chamber, the downstream unwatered tunnel and IFR equipment chamber. The lining will impede loosening and popping of rock that would damage the penstock and pose a safety risk for man access.

The penstock will be supported on concrete cradles spaced to limit bending stresses to safe levels. A sliding seat will be provided between the concrete and the steel pipe. About 200 mm of crushed gravel will be placed in the tunnel invert to provide a safe walking surface. A permanent drain pipe will be installed to evacuate any water that seeps into the tunnel.

6.2.3 Intake Controls and Isolation

Intake Gate

A butterfly-type intake gate will allow draining of the penstock for periodic inspections. The intake gate will be designed to operate with the lake at full pool. An air inlet pipe will be provided downstream of the intake gate to prevent the development of high negative pressures. The gate will close automatically in the event of a penstock rupture.

A heavy, cast-iron slide gate will be provided above the intake gate as the a maintenance isolation device. The gate will be provided with a flap that automatically seals the leaf casing as the leaf is withdrawn from the flow. This allows inspection and maintenance of the leaf seals.

6.3 Penstock Design

6.3.1 Transient Pressure Analysis

Transient analyses will be completed for the following scenarios:

- load rejection at full flow;
- manual closure of either the turbine inlet valve or intake gate from fully open to fully closed in 60 seconds under full flow;
- slam closure of one needle valve during full flow.

6.3.2 Structural Analysis

Permissible Steel Stresses

Permissible stresses of the steel penstock are limited to:

- Normal Operation Performance Level:

  The lesser of $0.67\ f_y$ or $0.33\ f_u$
  so for $f_y = 360$ MPa and $f_u = 455$ MPa $f_{permissible} = 150$ MPa

- Emergency Operation Performance Level:

The lesser of f_y or $0.50\ f_u$
so for $f_y = 360$ MPa and $f_u = 455$ MPa $f_{permissible} = 228$ MPa

- Survivability Performance Level:

The lesser of $1.67\ f_y$ **or** $0.83\ f_u$
so for $f_y = 360$ MPa and $f_u = 455$ MPa $f_{permissible} = 378$ MPa

Penstock Design Loads

The load conditions relevant for design are defined as follows, using the ASCE notation:

- Internal Pressure:

maximum static head $P_1 = 1{,}046 - z$ [m]
where z [m] is the elevation of the section under consideration.

maximum static head, including head loss, normal valve closure water hammer, and surge for a plant load rejection closure $P_2 = 1.15\ P_1$ [m]

extreme pressure due to a spear valve stem failure causing slam closure is determined based on a transient analysis P_3 [m]

vacuum pressure $P_4 = -100$ kPa

- Dead Loads:

Dead loads comprise all permanently acting gravity loads due to self weight of structure, contained water and surcharge soil or rock, and are defined as follows:

weight of penstock D_1
weight of water contained within penstock D_2
weight of soil or rock surcharge and groundwater D_3

- Live Loads

wind load, normal operation L_1 = 1/30 year wind loads
wind load, extreme L_2 = 1.4 x 1/50 year wind loads
ground snow load, per Section, normal operation $L_3 = S_s$
ground snow load, per Section, extreme $L_4 = 1.5 \times (S_s + S_r)$
traffic load, logging truck loading, per Section 0 L_5

- Earthquake Loads

quasi-static earthquake loads, determined for the 2500 year return event EQ

- Thermal Loads

  Thermal loads will be calculated for the buried and exposed sections of penstock, assuming site temperature variations as provided in Section 4.3.2 and accounting for wind and solar radiation where appropriate.

  Thermal analysis will be undertaken for exposed sections of penstock to determine the requirements for minimum flows, insulation, or other protective measures to prevent freezing in the penstock.

Difference between temperature at weld-up and cold water	$T_1 = 15 - 4 = 11°C$
Difference between weld-up and extreme annual maxima	$T_{2,1} = 15 - T_{min}$
	$T_{2,2} = T_{max} - 15$

Load Combinations

The relevant load combinations for each Performance Level are presented in the table overleaf.

Table 6-1: Relevant Penstock Load Combinations for Various Performance Levels

Performance Level	Loads														
	Internal Pressure				Dead Loads			Live Loads					Earthquake Load	Temperature Variation	
	static head	normal trans-lent	water hammer	vacuum	pipe	internal water	soil	1/30 wind	fact. 1/50 wind	snow	fact. snow	traffic		normal	extreme
	P1	P2	P3	P4	D1	D2	D3	L1	L2	L3	L4	L5	EQ	T1	T2
Normal Operation														●	
Emply pipe					●		●	●		●		●			
Full pipe, plant is shut down	●				●	●	●	●		●		●		●	
Normal operation		●			●	●	●	●		●		●		●	
Filling	●				●	●	●							●	
Emergency Operation															
Wind storm		●			●	●	●		●					●	
Extreme winter		●			●	●					●			●	
Extreme temperature change					●										●
Sudden draining				●	●	●	●							●	
Survivability															
Earthquake, operating		●			●	●	●			●		●	●	●	
Earthquake, empty					●	●	●			●		●	●	●	
Slam closure			●	●	●	●	●							●	

6.3.3 PENSTOCK STRUCTURAL DESIGN

The penstock will be DN 750 steel pipe on concrete cradles witthin the tunnel, then buried DN 750 steel for following 200 meters, transitioning downstream to a DN 750 mm steel section that runs above ground for 470 m and below ground for a further 3,180 m. The penstock reduces to DN 700 m buried steel section approximately 270 m above the powerhouse.

The penstock has been aligned using the LIDAR survey supplemented by ground-level surveys at the steep section. The number of bends has been minimized and a minimum fall of 1% maintained over the entire alignment.

Steep Above-Ground Section

The exposed portion of the penstock will be subjected to the broadest range of loads, including:

- internal pressure;
- self weight;
- environmental loads, including snow, wind and extreme temperatures; and
- seismic loads.

The Hinckey-von Mises combined stress approach will be used for the strength design of this portion of the penstock. All bends will be restrained by anchorages and intermediate supports will be provided to maintain combined stresses below the permissible stress limits set out above. A minimum number of movement joints will be provided to maintain axial stresses due to thermal variation within tolerable limits.

Buried Section

The buried penstock will be designed as an isostatic pipe, with hydrostatic thrusts at bends and axial movements due to temperature changes and Poisson's effects controlled by soil-pipe interaction. All bends with thrusts out of the ground (combined vertical and horizontal components) will be designed as fully-restrained anchor blocks with double-corrosion protected ground anchors. The ground anchors will be pre-stressed to accommodate normal operating pressures, but the penstock may be allowed to behave fully autostatically for pressures above this level. For bends with thrusts into the ground, the pipe bedding will be relied on to transfer the forces into the ground in bearing and friction.

Pipe wall sizes will generally be selected based on hoop stress alone, as the hoop stress will always be greater than the combined stress with axial stresses included in the Hinckey-von Mises relationship.

A three-dimensional line-element computer model will be used with the programme MultiFrame 4D to design for adequate control of displacements by the pipe bedding and anchor blocks and to check the axial and bending stresses that occur locally in combination with the hoop stresses. This allows ground anchors at the bend anchorages to be modelled as tension-only springs and the lateral subgrade reaction of the soil bedding as compression-only springs. Ground anchors for bend anchorages will be prestressed to 60% of ultimate tensile strength ($0.6f_u$) for the governing thrust of the Normal and Emergency Operation Performance Levels. At thrusts exceeding the pre-stress level, anchorages will move, with the ground anchors acting as tension spring members. At the Survivability Performance Level, anchor stresses must not exceed the 0.9 times the tensile strength.

The loads considered in this model include:

- hydraulic thrusts at bends for Normal Operation, Emergency Operation, and Survivability performance levels.
- temperature change from weld-up to cold water.
- Poisson's effect as an equivalent temperature change.

Pipe sizes will also be checked for wall stability under full vacuum pressures wtih overburden surcharge. Minimum shell thickness will be the lesser of:

PG&E formula	$t_{min} = D/7315$
or USBR formula	$t_{min} = (D+508)/10160$

where D and tmin are in mm units.

Section In Powerhouse

The short section of penstock within the powerhouse, up to the flange at the Turbine Inlet Valve (TIV), will be designed in the same way as described for the buried sections. Piping downstream of the TIV will be the responsibility of the supplier.

High-pressure mechanical piping connected to the penstock, such as drain and bypass valve piping, as well as the associated flanges, branches, and valves, will be designed to the ASME B31.3 – Process Piping code. In this code, the Design Pressure (P_D) is the pressure at the most extreme condition of temperature and pressure expected during service, except for infrequent exceedances that are set out in the code or by experience. In any case the most extreme pressure (P_3) shall not exceed the test pressure, and the test pressure shall not be less than $1.5P_D$. Therefore P_D shall not be less than P_3 / 1.5.

Flanges will be selected using ASME/ANSI B16.5 – Pipe Flanges and Flanged Fittings. Valves will be cast or fabricated steel.

6.3.4 Penstock Drain and Energy Dissipation

A penstock drain will be provided to allow periodic de-watering of the penstock for inspection. The penstock drain will be equipped with an energy dissipating system to discharge water into the tailrace at an outlet pressure of approximately 345 kPa (50 psi). The 100 mm (4") drain system will have flanged fittings. All valves and energy dissipaters will be located in a buried chamber near the penstock tee.

The energy dissipater system will consist of three valves that will be opened in succession as the pressure in the penstock drops. The drain time to full dewater the penstock will be approximately five days.

6.3.5 Stream Crossings

The penstock crosses numerous streams, most notably Tyson Creek. The design flood for all water crossings will be the 200-year design flood, plus any applicable allowance for debris flow or debris flood hazards.

Based on hydraulic modelling of the 200-year design flood plus 1.5 m freeboard, the low chord of the Tyson Creek crossing along the preliminary penstock alignment (adjacent to the road bridge) must be at a minimum elevation of EL 1,017 m. The low chord elevation will be updated if the location of the proposed bridge is changed.

The steel pipe bridge will be designed during the detailed design phase.

As reported in Appendix B, Frank Baumann, P.Eng., recommends that culverts and swales be provided at all other stream crossings. Sufficient cover, hydraulic capacity, and erosion protection will be provided at each crossing to minimize the potential for high streamflows to affect the penstock.

6.3.6 Erosion and Flood Protection Works

The penstock alignment follows the intake trench from the headworks to just upstream of the proposed Tyson Creek crossing. Erosion protection is required to protect the backfilled trench from erosion during flood events on Tyson Creek.

The proposed flood wall and creek training structure will begin about 40 m downstream of the dam with a crest elevation of EL 1,024 m. The height of the structure should be minimum 4 m (3 m water depth plus 1 m freeboard) above the adjacent channel invert.

The flood wall training structure will tie into the left abutment erosion protection at the steel pipe bridge with a top elevation equal to the low chord of the bridge.

Erosion protection at other locations will be addressed as required during detailed design.

6.4 INTAKE AND POWERHOUSE STRUCTURAL DESIGN

The following is a summary of the applicable general loads for structural design specified by the BCBC and other sources.

6.4.1 LIMIT STATES

Limit States And Performance Levels

Appropriate load combinations will be formed for the Normal Operation and Survivability conditions. The table below lists the loads to be considered for each condition and whether or not a load factor, per BCBC 2006, is to be applied.

Table 6-2: Loads and Load Factors

Load Type	Normal Operation Condition		Emergency Condition		Survivability Condition	
	Cracking	**Deflection**	**Stability**	**Strength**	**Stability**	**Strength**
Self weight	U	U	F	F	F	F
Usage and equipment	U	U	F	F		F
Creep and shrinkage	U					
Soil pressures	U	U	F	F	F	F
Environmental	U	U	F	F		
Seismic					E	E
Turbine and generator	U	U	F	F		
Intake water levels	E	E	E	E	E	E

Legend: U: unfactored load
F: factored load to BCBC 2006
E: explicit load defined for condition

Cracking Limit State

Cracking of concrete sections will be limited by providing sufficient reinforcement that the cracks are controlled over the effective tension zone of the concrete section. Limiting characteristic crack widths are:

Surfaces not continuously exposed to water	$w_{k1} \leq 0.30$ mm
Surfaces continuously exposed to water	$w_{k2} \leq 0.20$ mm
Surfaces of water-tight elements	$w_{k3} \leq 0.10$ mm

Deflection Limit State

Unless otherwise stipulated by equipment suppliers, characteristic deflections (δ_k) will be kept below the following limits:

Deflection under permanent and storage loads	$\delta_{kp} \leq$ span length/240
Deflection under other variable loads	$\delta_{ki} \leq$ span length/300
Deflection of crane rails, under permanent and variable loads (5 tonne capacity)	$\delta_{kc} \leq$ span length/600

Stability Limit State

To ensure sufficient safety against overturning, sliding and uplift of the structure:

$$\frac{\text{Factored Restoring Effects}}{\text{Factored Driving Actions}} \geq 1.0$$

Strength Limit State

To ensure sufficient member and structure strength:

$$\frac{\text{Factored Resistance}}{\text{Factored Action}} \geq 1.0$$

The factored resistance of concrete, masonry, structural steel and steel platework will be in accordance with the relevant material codes.

6.4.2 DESIGN PARAMETERS

Self Weight Loads

Unit weights of materials for calculating dead loads due to self-weight of structural members and building components are as follows:

Mass concrete	23 kN/m^3
Reinforced concrete	24 kN/m^3
Steel	78.5 kN/m^3
Structural steel deck	0.30 kPa

Self-weights of finishes and non-structural elements are as follows:

Hollow concrete block masonry (reinforced and partially grouted)	3.0 kPa
Roofing (insulated)	0.25 kPa

Wall cladding (insulated)	0.30 kPa
Partition walls (drywall and steel studs)	0.30 kPa
Suspended ceilings, including luminaires and HVAC	0.50 kPa

Creep And Shrinkage

Creep and shrinkage effects will be considered for the crack-control limit state. The parameters of Section 1.2.3 of the CPCA Concrete Design Handbook and the approach of Eurocode 2 will be followed for crack control in mass concrete.

Usage Loads

Usage loads due to human and vehicular traffic, equipment (pending equipment suppliers' loading drawings) and lay-down of parts are as follows:

- for the powerhouse floor:

the greater of	10.0 kPa
or	10 kN
or	equipment loads

- roof areas

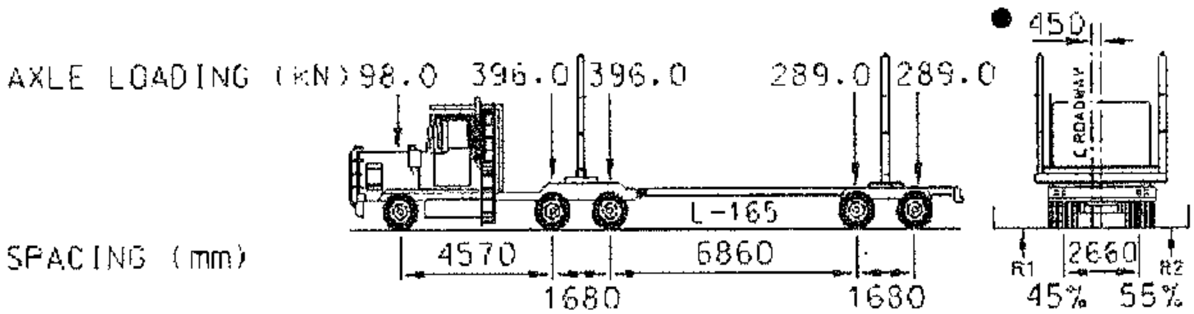
the greater of	1.0 kPa
or	1.3 kN
or	snow load

- horizontal usage loads due to persons:

railings and partition walls, applied at 1.0 m above floor	0.75 kN/m
or	1.0 kN

- vehicle loads, based on a logging truck with the following characteristics:

L165 axel loads	GVW 149 700 kg
	45%/55% unbalance
	450 mm eccentricity



L-165 (OFF HIGHWAY) GVW 149,700 kg

Soil And Rock Pressures

Soil and rock pressures will be determined using the final geotechnical parameters determined from the additional field investigations, using established principles of soil mechanics. Lateral earth pressures will be determined using the following criteria:

Gravity retaining walls and substructure walls braced by floor slabs	at-rest condition
Cantilever retaining walls	average of at-rest and active condition

Structures located at sufficient distance from streams that they are not influenced by stream water levels will be designed for geohydrostatic loads obtained using presumed groundwater tables. The factored geohydrostatic load in this case will be limited to that obtained by assuming the groundwater table is coincident with the ground level.

Environmental and Seismic Loads

Loads due to the effects of temperature change, wind, rain, snow and earthquake will be determined using the general design parameters given in Section 4.

Active lateral seismic soil pressures will be determined using Wood's approach for rigid basement walls. Passive lateral seismic soil pressures will be determined from static equilibrium, but will not be greater than those determined using the Monobe-Okabe approach.

Water Levels and Flow

All impoundment structures, including the tunnel plug and control gates will be designed for hydraulic loads including the following:

- Differential pressure heads upstream and downstream of the structure.
- Uplift pressures on structures due to seepage through the foundation. The pressure gradient is assumed to vary linearly between the upstream and the downstream water level heads.
- Hydrodynamic effects, such as dynamic pressure head on structures in the stream flow.
- Dynamic effects on slender structures in the stream flow, such as trashrack bars.

The tunnel plug will also be designed for blast pressures.

Turbine/Generator Loads

Turbine generator loads include:

- Weight of the assembled unit, acting at the anchorages.
- Weight of disassembled components on the powerhouse floor in the laydown area.
- Normal and emergency hydraulic thrust loads
- Normal and emergency rotational loads
- Short-circuit torque acting on the generator stator anchorages.

6.4.3 Key Design Details

Concrete Cover

The minimum distance from the face of a concrete member to the edge of reinforcement (cover) will be as follows:

- surfaces exposed to air

Stirrups and ties of 10M and 15M	40 mm
Principal reinforcement	50 mm

- reinforcement at faces in contact with soil backfill, groundwater or blinding concrete 50 mm
- all reinforcement at faces in contact with running water 60 mm
- all reinforcement at faces cast against soil or rock 75 mm

Minimum Reinforcement

The following requirements apply over and above those specified in CSA A23.3:

- Beam flexural reinforcement contents will be greater than $A_{smin} = .003\ b_t\ h$
- Free edges of slabs will be reinforced with U-bars whose legs extend at least 2h away from the edge.
- Every second longitudinal column bar will be enclosed by a tie bend or hook and no bar will be more than 150 mm clear either side from such an enclosed bar.

Concrete Formwork And Finishes

The following formed (F) and unformed finishes (U) will be used:

- For surfaces that will have fill placed against them:

  Finish F1: Forms built with a minimum of refinement. Form surface in contact with concrete to be standard formply that will not leak mortar or yield beyond specified tolerances when the concrete is vibrated.

  Finish U1: Even, uniform finish. Consolidate concrete level and screed concrete to obtain an even, uniform surface. Surplus concrete to be removed immediately after consolidation by striking off with a sawing motion of the straight edge or template across wood or metal guide strips. When the surface is curved, use screen strips at approved intervals. For long, narrow stretches of curved surfaces such as on invert paving, a heavy slip form may be used. In the case of extensive flat paving, a paving and finishing machine is preferred.

- General application for surfaces exposed to view without stringent aesthetic requirements:

  Finish F2: Form surface in contact with concrete to be plywood, steel or pre-approved form liner.

  Finish U2: Wood float finish. Follow treatment specified for Finish U1 by floating either by hand, or by power driven equipment. Started floating after some stiffening has taken place in the surface concrete and the moisture or "shine" has disappeared. Work the concrete no more than necessary to produce a surface known as "wood float finish", which is uniform in texture and free of screen marks. Do any necessary cutting and filling during the floating operations.

- Surfaces where aesthetic or mechanical performance requires a high quality finish such as control buildings and interior surfaces of the powerhouse:

  Finish F3: Form surface in contact with concrete to be plywood, steel or pre-approved form liner. Plan form construction so that any residual pattern from the forms will harmonize with the structure or building. All joints shall be horizontal or vertical.

  Finish U3: Steel troweled finish. Follow the treatment specified for Finish U2, but leave a small amount of mortar without excess water at the surface to permit effective troweling. Start steel troweling after the moisture film or "shine" has disappeared from the float surface and after the concrete has hardened enough to prevent an excess of fine material and water from being worked to the surface. Trowel with firm pressure that will flatten the sand surface left by the floating and produce a dense, uniform surface free of blemishes, ripples and trowel marks.

  Finish U5: Broom finish: Follow the treatment specified for Finish U3 by roughening the surface immediately after troweling with a fiber bristle broom in a direction perpendicular to the direction of traffic. Broom grooves not more than 1.60mm deep. After brooming, neatly tool all joints and edges to configuration.

  Finish U6: Hardened finish: Follow the treatment specified for Finish U3 and immediately after troweling, apply floor hardener per manufacturer's instructions.

- Surfaces with severe tolerance limits because they are exposed to flowing water with a velocity exceeding 0.5 m/s:

  Finish F4: As for Finish F3, but with severe tolerance limits. Smooth sanded plywood to be used for form surfaces in contact with concrete. Steel or impermeably lined plywood forms are not permitted unless lined with at least 5 mm thin smooth sanded plywood sheathing.

  Finish U4: A hard, burnished steel troweled finish. Follow the treatment specified for Finish U3 with additional steel troweling after the surface has nearly hardened, using firm pressure and troweling until the surface has a burnished appearance.

Mass Concrete

For the mass concrete sections, the provisions CSA A23.3 will be modified to take into account the absence (or low ratio) of reinforcement. Where reinforcement is provided to balance overturning moments of the section above a presumed crack or construction joint, the steel strain will be checked to assure the specified elongation at yield is not exceeded.

For preliminary design, the following reinforcement will be assumed for mass concrete faces requiring reinforcement for crack control:

Walls and slabs with 400 < t < 600	15M @ 150 each way
Gravity walls, sills and other mass pours, t > 1000 mm	20M @ 200 each way

Joints

Where movement joints are unavoidable, they will be designed to accommodate the full anticipated movement, with water-tightness assured by a combination of internal waterstop and surface sealant. Water-stops will be bulb-type, sized for the required transverse and shear movement capacities.

Continuous reinforcement and internal water-stops will be used for construction joints. Concrete joint surfaces will be water/air pressure blasted to an amplitude of 5 mm and to reveal the coarse aggregate. Prior to casting the continuing pour, a bonding agent will be applied to the hardened joint surface. Where hydrostatic pressures on the joint exceed 15 kPa, a hydrophillic compound will be applied in lieu of a bonding agent.

Benchmark materials to be used are:

Internal water-stops	Sternson Durajoint Type 7
Bonding agent	Sika Albitol Concentrate
Hydrophillic compound	Xypex Concentrate

Connections

All bolted connections will be designed as bearing connections. Bolts will be tensioned by the turn-of-nut method to ensure slip-free behaviour under service loads.

Fillet weld dimensions will fall within the following limits:

Minimum weld leg	$a_{leg} \geq 6$ mm
Minimum weld length	$L_{weld} \geq 4\ a_{leg}$ or ≥ 40 mm
Angle included between weld legs:	
Minimum	60°
Maximum	120°
Maximum pitch of discontinuous welds	$6\ t_{min} \geq 300$

where t_{min} is the thickness of the smaller piece being connected.

7 POWERHOUSE MECHANICAL SERVICES

7.1 DESIGN PARAMETERS

7.1.1 PERFORMANCE REQUIREMENTS

Mechanical services, including water supply, drainage, heating and ventilation will be provided to facilitate operation and maintenance of the powerhouse, and to protect the facility.

In general, water supply is for washdown and maintenance; both hot and cold water will be required. The water provided will not be treated to potable standards. Sanitary facilities will consist of a portable toilet.

Cooling will be by means of ambient air ventilation. Heating will be by means of electric unit heaters; these will rarely be required to operate because of the heat radiated by the generator and indoor transformer.

Fire Protection will be provided by means of hand-held extinguishers located in the powerhouse.

7.1.2 GOVERNING CODES AND STANDARDS

The mechanical services in the powerhouse shall conform to applicable codes, standards and other standards referenced:

- American National Standards Institute, ANSI
- American Society of Heating, Refrigerating and Air-Conditioning Engineers, ASHRAE
- American Water Works Association, AWWA
- Canadian Standards Association, CSA

The latest editions of relevant standards shall govern.

7.2 VENTILATION

Ventilation shall be designed in accordance with the ASHRAE Handbook, HVAC Applications, particularly Chapter 24 Power Plants and Chapter 28 Ventilation of the Industrial Environment.

7.2.1 Control Room Heating and Ventilation

The Control Room will be heated by electric baseboard heaters and cooled by a small supply fan drawing air from outside (not from the machine hall). The fan will be two-speed, capable of providing at least 5 air changes per hour (ACH).

7.2.2 Machine Hall Heating and Ventilation

The Machine Hall will normally be heated by the generator. When the generator is not operating electric unit heaters will be used to heat the Machine Hall. The Machine Hall will be cooled by a series of exhaust fans drawing air from outside via intake louvres. The intakes will be located low and the exhausts high, to facilitate air flow across the equipment.

Design criteria for Machine Hall Ventilation are set out below.

7.2.3 Machine Hall Ventilation

Design Basis

The air flow supplied shall be the largest of the flows required for:

1. Cooling (heat radiated from equipment);
2. Dilution (air changes to eliminate condensation, vapours, etc.); and
3. Replacement (such as combustion air).

These air-flows are not additive.

Cooling Air

Cooling air carries away the heat radiated from equipment, primarily the main generator and transformer but also miscellaneous equipment. It is expected outside air cooling will be sufficient so air conditioning will not be provided.

With outside air cooling, the temperature in the building can never be less than the temperature outside. Therefore, the design uses differential temperature above ambient, rather than an absolute maximum. A differential of 10 C is used, which is within the range of 6 to 12 C set by ASHRAE HVAC Applications 1999, p. 24.2, Table 1.

The main sources of heat in the building are the generator and transformer. TCHC has provided the following design criteria in these regards, for units rated 9.3 MW:

1. The generator efficiencies are given as follows: 97.1% at full load; 96.9% at 3/4 load; 96.7% at 1/2 load; and 94.1% at 1/4 load.

2. For dry type power transformer (Class F), the radiated heat is given as 58.8 kW at full load, equivalent to an efficiency of 99.4%.

The losses are assumed to be all heat.

The foregoing criteria result in a maximum cooling airflow of 28,000 L/s.

Dilution Air

The minimum dilution airflow criterion is set by the larger of the following:

1. 7.5 L/s/m^2, based on ASHRAE Standard 62, Table 2. This results in a flow of approximately 1,500 L/s or 4.5 ACH.

3. The flow required to control condensation on the turbine casing. This is set by the manufacturer's specification sheet.

Replacement Air

Combustion air is not required as there is no significant consumption of air (e.g. diesel generator) in the powerhouse.

Design Cases

The design cases include the following:

1. Generator full load on a hot day (cooling flow governs).

   Maximum generating capacity on a hot day. The design generating capacity is 9.3 MW as defined by the generator manufacturer. The design temperature is set out in Section 4. This case is likely to govern for cooling air design.

2. Low load on a cold day (dilution flow governs).

   In this case dilution air flow typically governs and hence this case is likely to govern for heating design. With minimal load on the generator, the radiated heat may approximately balance the heat-loss from the building, meaning that the dilution air must be heated.

3. Generator not operating on a winter day. Station service would power the building loads including electric heating. The ventilation system should be set to its minimum setting. It should be assumed that the temperature is no colder than the average minimum winter day temperature.

Temperature Limits

A temperature differential (ΔT) of 10 C will be used for Design Case No. 1. This will keep the machine-hall temperatures below 25 C except during the extreme hottest summer days when temperatures may rise to 35 C.

The plant will be designed for an average minimum temperature of 15 C for Design Case No. 2.

The plant will be designed for an absolute minimum temperature of 7 C for Design Case No. 3 [Ref: ASHRAE HVAC Applications 1999, p. 24.2, Table 1].

Air Intake and Exhaust Systems

For simplicity the machine hall will be ventilated with exhaust fans. This will provide a negative pressure in the building; the negative pressure shall not exceed 5 Pa as recommended in ASHRAE HVAC Applications 1999, p. 28.8, Table 4. Inlet filters will not be provided.

Air will enter the building through weather louvers located near the bottom of the south wall. The intake air system shall take special consideration of wind-driven rain. Louvers shall be selected to minimize water ingress as well as pressure drop. They shall be extruded aluminum at least 150 mm deep. Exterior hoods may be required.

The inlet weather louvers shall be backed up by control dampers to control wind blowing through the building. The control dampers shall be high-quality with aluminum airfoil blades. The combined pressure drop across the intake louvers and control dampers shall not exceed the allowable negative pressure in the building.

The exhaust air shall be collected from high in the machine hall and exhausted by four fans, one small and three identical large fans. The fans are sized such that the small fan provides the minimum dilution air flow (air changes). The three large fans operating together provide the maximum cooling air flow.

The fans will be wall-mounted high on the north façade. Access will be provided for maintenance. An alternative is to collect the air through ceiling-hung ductwork and exhaust it through low-mounted centrifugal fans. The fans will have single-speed, 600 V, 3 phase motors in accordance with the project electrical design criteria.

Silencers will not be provided on either the inlet or exhaust openings.

Air Recirculation

There shall be an air circulation system in the machine hall to maintain a uniform temperature. If wall-mounted exhaust fans are used, air circulation can be provided by two ceiling fans. If air-collection ducting is used, bypasses can be provided on the fan discharges to recirculate air; the fans shall be increased in size accordingly.

Air Heating

The supply of outside air for dilution must be as stated above for condensation control and contaminant dilution, and the heating system must be capable of heating this air-flow during periods of low plant demand.

Heating will be by means of electric unit heaters (minimum 4).

Air System Controls

The control system will automatically control the fans and heaters to maintain airflows, temperatures and humidity within design limits. The optimum temperature band is to be initially set at 18-22 C but this is to be adjustable in the control system. The optimum humidity is to be initially set to 70-80% relative humidity but this is to be adjustable in the control system.

The default (start up) setting is for one exhaust fan to operate to exhaust the minimum dilution air flow. As long as the temperature remains in the optimum band, control is through humidistats in the machine room. If the humidity rises to the high set-point, the second fan is turned on. When the humidity falls to the low set-point, the second fan is turned off.

If at any time the temperature rises to the high temperature set-point, the second fan is turned on if it is not already operating. When the temperature falls to the low set-point, the second fan is turned off. If the temperature stays within the optimum range, the system reverts to humidity control. If the temperature continues to fall, heaters are turned on to maintain the temperature in the optimum range.

If the temperature falls to the minimum allowable, the fan is turned off and the inlet control damper is closed. A building air low-temperature alarm is sent.

When the temperature again rises to an intermediate set point 5 C above the minimum allowable, the fan is turned on again and the system reverts to temperature control. When the temperature rises to the optimum band, the system reverts to humidity control.

A control narrative based on the foregoing will be provided in Division 15 of the technical specifications.

7.3 OVERHEAD CRANE

The powerhouse will be equipped with a 5 tonne bridge crane for maintenance of equipment. The crane will be provided with electric hoist and travel.

7.4 WATER SUPPLY AND DRAINAGE

The powerhouse will be equipped with a non-potable water supply system. Water will be supplied (pumped) from the tailrace by a submerged pump to a 2,000 L storage tank above the control room in the powerhouse. Water will be heated by an electric tankless heater and pressurized by means of a booster pump and hydropneumatic tank.

Hot and cold water will be provided to at least two hose bibs so that warm water can be used for cleaning. The hose bibs will be heavy-duty janitorial type with bucket hooks. Heavy washdown hoses and a two-basin janitor sink will also be provided. A portable toilet will be provided.

Concrete floors will be sloped to floor drains. A duplex automatic sump pump system will be provided at the lowest point if necessary. Floor drainage will be directed to an oil/water separator and then to a rock pit located in an area where it can drain to soil or other suitable disposal.

Proposed Pre and Post Environmental Monitoring Program for the Tyson Creek Run-of-the-river Hydro Project 2008 - 2015

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1. ENVIRONMENTAL COMMITMENT

Development of small run-of-river hydroelectric energy generation is being developed worldwide increasing clean, sustainable energy and helping to reduce greenhouse gas emissions. According to the International Energy Agency's World Energy Outlook 2004, hydropower will remain the largest source of renewable "Green" electricity generation until 2030.

The Government of Canada recognizes that hydropower is a clean and renewable source of energy that offers many co-benefits for the environment. Thanks to hydropower, Canada has the lowest cost of electricity production and one of the most reliable generating systems in the world.

In Canada, the Environmental Choice^{M} Program has made a commitment to promote electrical energy sources that greatly reduce environmental impacts. The Program recognizes technologies that use naturally occurring energy sources such as the wind and sun, and power sources that, with the proper controls, add little in the way of environmental burdens including small run-of-river hydro. If the stringent conditions of the certification criteria are met a project attains the EcoLogo endorsement.

Renewable Power Corporation (RPC) is a BC based company working to develop and deliver "Green" hydro power to British Columbians. The company based in Gibsons, BC built its first small hydro facility on McNair Creek in Howe Sound, BC. The project which has EcoLogo certification was also given the "award of excellence" by the Consulting Engineers of British Columbia. This facility provides a model for the development of other company run-of-the-river projects.

In addition to the McNair Creek hydro project, RPC continues to develop and provide expertise other areas of hydro development, including:

- Permitting, design, construction supervision and plant operation and maintenance for the McCannell Creek Hydro Project. This facility has allowed the world renowned Camp Malibu eliminating their dependency on diesel generator electrical power.

- The operation and maintenance for the Brandywine "Green" Power Project. The company is responsible for bringing the plant to optimum operating performance.

- Participation in a joint research project with UBC that resulted in the development of an industry leading hydro power reconnaissance software tool. This tool will provide the company with a technological advantage in selecting potential sites for low impact run-of-the-river hydro.

2. PROPOSED MONITORING PROGRAM

Tyson Creek is a tributary of the Tzoonie River located at the head of Narrows Inlet near Sechelt, BC. It is a lake fed tributary that spills over a rock sill into a steep bedrock and boulder strewn channel. The upper diversion reaches (immediately downstream of the lake) are characterized by subsurface flows and large boulder fields. The lower diversion reaches are characterized by steep, boulder cascade and step pool morphology. There are limited off-channel habitat opportunities that are connected to the main channel and numerous indicators of unstable and violent channel events evident by the accumulations of wood debris and large diameter bedload materials.

The proposed monitoring program will begin in the spring and summer of 2007. This will allow an additional 1 possibly 2 years of pre-diversion data collection and 5 years post diversion for specific environmental/biological parameters. This additional information will be supplemented with the data collected in 2005 and 2006 for the preparation of the water license application.

The purpose of the monitoring program is the continued collection of biological and physical data on Tyson Creek. The emphasis is placed on the proposed diversion reaches and will help RPC develop and manage the facility with as little impacts on the environment.

2.1 Reporting Protocol

All monitoring programs will result in detailed results from field activities. Data collected from each monitoring program will be georeferenced for future works with the information provided on site maps. Permanent sample sites will be fixed using permanent survey markers in order to identify the location on the ground in subsequent years.

Data from each years sampling and monitoring programs will be summarized and where applicable analyzed. Inferences will then be made with respect to the pre and post diversion conditions. Results of the data collection and analysis will be submitted to the appropriate government agency by January 31 of each year and will include the previous years' data. All monitoring data will be accompanied by known operating, natural and or man-made incidents that may impact the environment and possibly affect the monitoring results.

In order to ensure monitoring procedures are adequate the results from each year will be reviewed and recommendations regarding changes to the monitoring program will be submitted to the regulatory agencies. A broader review of the monitoring data will occur in year 2, post diversion. At this time the pre and post monitoring data will be analyzed and adjustments to the project operation recommended. This may result in changes to the water license identified in the initial license and could result in the identified IFR moving up or down or adjustment the monthly IFR's. The purpose of this is to ensure project operation has minimal impacts and that both positive and negative effects of the project are identified.

The proposed review agencies for the Tyson Creek project should include:

- Sechelt First Nation
- BC Ministry of Environment-Water Stewardship Division
- Integrated Land Management Branch
- Fisheries and Oceans Canada

3. ENVIRONMENTAL MONITORING

3.1 Water Quality

Water quality baseline data was first collected in 2005 and will continue for the pre-diversion and 5 year post diversion. The purpose of the water quality monitoring is to identify changes in "key" water quality parameters that may adversely impact aquatic and riparian habitats. The "key" water quality parameters follow the guidelines identified in *Lewis et al.* (2004) and are shown in the following table along with the proposed sampling frequency.

Parameter	Frequency
Dissolve Oxygen (%sat)	Quarterly
Total Gas Pressure	Quarterly
Temperature	2 hours
TSS	Quarterly
Conductance	Yearly (low summer flow)
Alkalinity	Yearly (low summer flow)
pH	Quarterly
Phosphorous	Quarterly
Orthophosphate	Quarterly
Ammonia	Quarterly
Nitrite	Quarterly
Nitrate	Quarterly

Water samples will be collected at predetermined sample locations both above and below the proposed tailrace (diversion), and at the outflow from the headwater lake. Water quality data for 2005 and 2006 is reported in *FSCI* (2006). Water temperature is logged using an Onset temperature logger. An additional 3 temperature loggers will be installed at the lake, midway through the diversion reach section and immediately above the tailrace. These will be installed in the summer of 2007. There is currently one data logger operating near the mouth of Tyson Creek. A second data logger located in the diversion section was lost during a storm event.

Dissolved oxygen, TGP and pH will be measured in the field during collection of water samples. The remaining parameters will be determined from water samples collected at the assigned (georeferenced) and benchmarked water stations, then shipped to a certified water quality testing laboratory in Vancouver, BC.

Additional pre and post diversion water quality data will be added to the existing data set and comparisons made between years. Results will be summarized annually and a final report prepared upon completion of the 5 years post diversion.

3.2 Stage versus Channel Geometry - Modify this based on reality of working in difficult reaches???!!!!

In 2006, a set of 5 cross sections were benchmarked and surveyed in the lower portion of Reach 4 (diversion reach) immediately above the proposed tailrace. The purpose of the cross sections was to provide baseline channel geometry measurements versus stream discharge. This information would then be used to develop site specific stage discharge relationships with changes in wetted width, depth and velocity. It is expected that the information will help in the "fine tuning" the IFR and possibly correlate with the invertebrate production in the diversion reaches. It is anticipated that the results would be applied to the operation of the power project in an adaptive management approach that ensured acceptable benthic production to downstream fish bearing reaches.

The stage versus geometry method is referred to as At-A-Station Geometry (ASG) (*Stewardson*, 2005) and provides a quantitative measure and understanding of changes in the stream channel with changing stream discharge.

At-a station geometry data collection will continue in 2008 with the following activities planned:

- Additional cross-sections (5) to be benchmarked and surveyed further upstream in Reach 4 and/or 5.

- Development of stage relationship curves for each of the established cross sections and their wetted width, depth and velocity.

- Photo record of each cross section at sampled discharge

- Evaluate and determine changes in the wetted perimeter under restricted diversion flows and attempt to relate this to biological parameters measured.

It is expected that the ASG data will help support the current IFR and/or allow quantitative data to adjust the initial IFR when coupled with expected aquatic macroinvertebrate data.

3.3 Discharge versus Useable Area (Added)

Add details of velocity depth transect process. Transects will be measured using Swoffer 2100 and using weighted Useable Area model provided by Ron Ptolemy – MoE. Current model has HIS for inverts.

Conditions (Q) to be measured or targeted is 2.5%, 5, 10, 20, 40 and 100?? MAD. Similar model relationship to be built as used on Chapman Cteek in 2007 (Bates)

3.4 Macroinvertebrates

Salmonids are drift feeding fish with their growth and abundance correlated to the quality of habitat and abundance of food presented as insect drift. This applies to stream reaches that support salmonid population directly as well as non fish bearing reaches that "feed" fish bearing reaches. As a result the Tyson Creek Hydro Power is committed to designing and implementing an invertebrate sampling program that will continue to sample benthic invertebrates and determination of B-IBI for the Tyson Creek watershed as well as sample invertebrate drift.

3.4.1 Benthic Index of Biological Integrity (B-IBI)- May be removed

In 2006, initial B-IBI data was collected and presented in *FSCI* (2006). This program will continue and follow the following design.

- Samples will be collected from riffle habitats using a Hess sampler. Replicates of 3 - 5 samples will be collected from each sample site. The sample sites will be located in the diversion reach, downstream of the tailrace (non-fish bearing) and the fish bearing reach. These sites will be georeferenced.

- Samples will be preserved and processed by a qualified invertebrate taxonomist who will enumerate by species (or genus).

- Results of the benthic sampling will then be used to determine the B-IBI based on criteria presented in *Karr and Chu* (1999). In addition to the B-IBI the data will be used to establish an estimate of benthic invertebrate density (number/m2).

- Sampling will begin in the summer of 2007 and include two periods of time during the growth period. The first early summer under moderate flows and the second in late summer at summer flows.

- Data will be compiled and compared to data collected in 2006. The data will be summarized and presented to the regulatory agencies.

- The period of monitoring for invertebrates will include 1 -2 years post diversion and 5 years post diversion. The period of sampling will be reviewed by the proponent and regulatory agencies following year 2 post diversion.

The B-IBI is considered a reliable index of general stream health and a reflection of anthropogenic impacts within a watershed. The monitoring of the B-IBI will achieve two purposes. First, monitor changes positive and negative in the B-IBI that may be a result of the hydro power project. Secondly, to determine area specific B-IBI values that could assist in developing guidelines that would help direct "green" hydro development in the area.

3.4.2 Invertebrate Drift

As previously stated invertebrate drift is important in both fish-bearing streams and non-fish bearing streams and specific information does exist that shows measurable changes in drift with changes in discharge. As result a monitoring program to address the question of insect drift versus changes in discharge is proposed. Information from the drift program would also be used with the changes in stream geometry to better understand the IFR.

This program is expected to begin in the summer of 2008 and will include the following design:

- The sampling of insect drift will begin in the summer of 2008 and be continued for 1-2 years pre-diversion and extend 5 years post development. This should meet the requirements of a BACI design and power analysis recommended by MoE.

- Sample sites will be located in riffle habitats and include georeferenced sites; above the tailrace (diversion Reach), below the tailrace, fish bearing reach (Reach 1) and in an adjacent non-fish-bearing stream with similar geomorphic and topographic features (between stream comparison).

- Sampling will be conducted 2 times per year with the first sample in early summer and the second late summer and low summer flows. An effort to replicate conditions (discharge, weather) will be made each year. This is an attempt to reduce variation in sampling conditions between years.

- The samples will be collected with a series of 2 sets of 3 - 250 micron drift nets anchored to the stream bottom. The nets will sample a predetermined cross section. The area and velocity at the mouth of the net will be recorded.

- Sampling duration will be set between 2 – 3 hours starting within 4 hours after sun-up and the time the sampling begins will be recorded.

- Samples will be preserved and processed. The samples will be filtered, sorted by size classes then photographed and sample invertebrates measured. Length weight relationships will be applied to the samples using information provided by MoE (J. Rosenfeld – UBC).

- All abundance and biomass data will be expressed per volume of water, tabulated and graphed and presented to regulatory agencies at the end of each sample year (January 31).
- Invertebrate drift results will also be compared to changes in useable area and where possible the IFR.

The purpose of the macroinvertebrate monitoring will be to address specific hypotheses regarding measurable effects of small hydro.

4. ADAPTIVE MANAGEMENT

Renewable Power Corporation is committed to responsible environmentally sound "Green" power. This monitoring plan is evidence of RPC's commitment. In the event results of the monitoring program indicate change beyond a scientifically acceptable range, appropriate remedial action will be made. This will only occur after complete consultation with the key regulatory agency.

5. REPORTING

Renewable Power Corporation will provide yearly reports summarizing the years' monitoring results from each aspect of the program. The yearly reports will be submitted to the regulatory agency pertinent to each area of the program. A detailed report highlighting the pre and post diversion data will be completed at the end of year 2 post diversion. This will then be followed by a final detailed analysis and report at the completion of the monitoring program

6. REFERENCES

FSCI, 2006. Salmonid distribution within the Tyson Creek watershed (900-195000-31800). Results of the 2006 field sampling. Renewable Power, Corp., Gibsons, BC.

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Lewis A., Hatfield, T., Chilibeck, B. and Roberts, C. 2004. Assessment methods for aquatic habitat and instream flow characteristics in support of applications to dams divert or extract water from streams in British Columbia. Ministry of Water, Land and Air. Protection, Victoria, BC.

Stewardson, M. 2005. Hydraulic geometry of stream reaches. Journal of hydrology, 306:97-111.

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Technical Memorandum

DATE: June 11, 2008

TO: Peter Schober, P.Eng
Tyson Creek Hydro Corporation

FROM: Robin Parker

RE: **TYSON CREEK HYDRO POWER PROJECT**
IFR Pump Options for Tunnel Intake Scenario
Our File 2468.002-450

1. BACKGROUND

Tyson Creek Hydro Corporation (TCHC) is developing a hydro power project on the Tyson Lake watershed, near the Tzoonie Valley. Water will be drawn from Tyson Lake through a tunnel, which will cause the water level in the lake to vary between 1040 m and 1003 m. The natural outlet is at 1040 m.

The conditions of the water license require that and minimum of 70 L/s is present in the creek at all times at the point of the natural outlet, and particularly when the lake level is below 1040 masl.

This memorandum summarizes the design options that would pump the required flow back into Tyson Creek.

1.1 DESIGN CONSIDERATIONS

The following considerations are critical to the successful operation of the system, and are incorporated into the analysis:

- Robust: The system must operate 24/7/365 in a hostile mountain environment where access is difficult many months of the year.
- Reliability: The system must be reliable, with redundancy built into it. Should there be a failure of one component, a secondary, backup system must be present to take over.
- Optimize for efficiency: Energy consumption should be minimuzed.
- Freezing: For components located in the elements, the ambient design temperature is -20° C.

2. OVERVIEW OF OPTIONS

Four options are considered and detailed herein, including:

- A submersible well style pump on the east lake shore hillside;
- A pump at the tunnel portal, with the pipeline running down the tunnel, to the pumps, onto the access road, and over the hillside to the natural lake outlet;
- A pump in the tunnel, with the pipeline running from the pumps, up through the tunnel, into the lake, and up to the natural lake outlet; and
- A submersible pump in the lake.

2.1 OPTION 1: WELL STYLE PUMP ON HILLSIDE

Option 1 would be a well style submersible pump station located on the hillside above the tunnel in a small container style building. Redundancy would be provided by two pumps. Each pump would be placed above two vertical 350 mm diameter shafts that would be drilled down through the rock and into the top of the tunnel. Each pump would be 250 mm diameter, capable of pumping the full 70 L/s design flow at the full range in heads (about 10 m to 43 m TDH). Due to the extreme range, variable frequency drives (VFDs) would be necessary.

Approximately 100 m of 250 mm diameter HDPE piping would run from the station westward and parallel to the lake. It would remain slightly above the high water level until it would reach the natural lake outlet where it would discharge.

This system would require between 40 and 45 kW (52 – 60 Hp). Electrical power and control cable would be supplied from the tunnel portal, up and over the hillside, and into the station.

Hydraulically, this option would be the most beneficial; well-style pumps are best suited to large ranges in static head. However, the system would be difficult to maintain or access. Removing the pump from the well would require a complex lifting mechanism as a crane would not be able to access the area. Further, the building would be exposed on a steep rocky mountainside and would need to be designed to withstand high winds, storms, and falling debris.

For these reasons Option 1 is not considered any further.

2.2 Option 2: Pump from Tunnel, Pipeline on Hillside

Option 2 would locate the pump system inside the tunnel at the portal, downstream of the tunnel plug. The water would be supplied via a nozzle installed on the penstock upstream of the penstock isolation valve, and run through down the tunnel the pumps.

There would be approximately 100 m of pipeline from the tunnel plug to the pumps, then the pipe would be run overland on the hillside and terminate at the natural lake outlet. The total run would be about 470 m of 250 mm diameter HDPE pipe. The system would require up to 56 kW (75 HP). The range in total dynamic head would vary between 5 and 43 m. To manage the range, the pumps would be run by VFD.

Option 2 is the favoured option.

2.3 Option 3: Pump in Tunnel, Pipeline in Tunnel

Option 3 would place the pumps inside the tunnel just downstream of the tunnel plug. The actual pumping system would be the same as in Option 2, as would the overall length of pipe.

However, rather than run the pipeline down the tunnel and around the hillside, this option proposes to run the pipeline through the tunnel plug, into the lake, along the lake shore, and to the natural lake outlet. Due to the destructive nature of the lake tap blast, the pipe would likely need to be installed after the tunnel is watered up; this would require divers and man-handling pipe under water.

Further, the pipeline running along the lake shore would have some significant construction challenges; it would need to be protected from falling rocks and debris on the hillside, and access could be difficult.

An alternative would have the pipeline running on the lake bottom (weighted down to prevent floatation). However, as the lake level rises and lowers in the winter during freezing conditions, significant ice loading would be placed on the pipeline, particularly at the point of interface between lake level and atmosphere.

Due to the construction challenges this option is not considered any further.

2.4 Option 4: Pump in Lake

Option 4 would be a submersible pump located in the lake below the low water level, with a pipeline running along the lake bottom to the outlet. However, there are significant drawbacks to this option including:

- The pump would require a VFD, but the distance between the MCC and the pump would be at least 130 m; this length pushes the limits of successful VFD control.
- The pump would not be accessible during the winter when it would be below the ice.
- During non-frozen conditions the pump would also be difficult to access. It could weigh over 800 lbs and so would require a complex lifting system.

An alternative to this option could be a floating structure or other type of permanent wharf structure founded to the lake bottom. The floating structure is not recommended due to the uneven ice loading potentially crushing the structure. The permanent wharf structure would be very costly due to the length and height requirement (over 40 m tall).

Based on the above, Option 4 or any version of it is not considered any further.

3. IFR PUMP STATION

Option 2 (pump from tunnel, pipeline on hillside) is the recommended option. The following sections detail the proposed pump station.

3.1 GENERAL DESCRIPTION

From discussions with TCHC, the pump station would be located inside the tunnel, just upstream of the portal. A larger, widened area would be blasted into the rock, and concrete foundations would be poured as pump bases. Motor control centres including VFDs and PLCs would all be located adjacent to the pumps. A monorail or other suitable lifting mechanism would be provided.

The 250 mm diameter supply pipeline from the penstock would be steel, as would the station piping and discharge piping. From the portal onwards, the pipeline would be HDPE, also 250 mm diameter.

PUMP STYLES

Two style of pumps are considered, including spilt case horizontal shaft pumps, and end suction horizontal shaft pumps.

In general, splitcase pumps last longer and are more reliable than end suction pumps, assuming best practices are employed during installations. In contrast end suction pumps are considered "industrial workhorses", where they can continue to operate under less than ideal conditions, but often require more servicing. End suction pumps can also be installed on smaller footprints.

Due to the remoteness and requirement for an extremely robust design, split case pumps are recommended. Further, space limitations are not expected to be an issue. A brief comparison was completed to confirm that there are no significant energy or other cost savings between the two styles.

ENERGY COSTS

Energy costs were developed based on the following assumptions:

- Energy value of 101 $/MWh;
- Pump operating 9 months of the year, 24/7 (the reservoir would typically spill 3 months of the year);
- An average annual lake level of 1021 masl, or a static head of 19 m (a daily flow model confirms the average lake level);
- 250 mm diameter HDPE pipeline, equating to 5.7 m of friction loss; and
- A total dynamic head requirement of 24.7 m (i.e. 19 m plus 5.7 m).

Based on the foregoing, the annual energy cost is approximately $15,000 in today's dollars, based on an average pumping efficiency of 80%.

3.2 PUMP SYSTEMS

A detailed analysis of potential pumping systems was completed. Two basic systems were considered. They include:

1. A single pump (with a backup pump) capable of pumping the design flow at the full total dynamic head (TDH) range (i.e. from 5.7 m to 43 m), or

2. A main pump (with a backup pump) capable of pumping the design flow for approximately 70% of the TDH, and a second "jockey" pump (with backup) capable of pumping the lower head flows.

PUMP SYSTEM 1 (SINGLE PUMP WITH BACKUP)

This option would have a single horizontal split case pump operate over the full range of head at the design flow. A second, backup pump would be provided. Detailed system and pump curves were developed for two different pumps and compared. The one with the highest overall efficiency (80%) and best reputation was selected.

Because the pump would be required to operate over such a large range, the minimum efficiency would be 72%; this would require a motor running at 56 kW (75 Hp).

The budget price for the supply of two pumps including pump, coupling, guard, driver, and base plate is $61,000. Taxes, markup, installation, MCCs, controls, and all pipng and appurtenances would be extra.

Pump System 2 (Main Pump plus Jockey plus Backup)

This system would require four pumps in total which in turn would require a larger area inside the tunnel. There would be more mechanical components that could fail, and the controls would be more complex.

However, the power requirements would be much less, i.e. a 37 kW main pump, and an 18 kW jockey pump, compared with the 56 kW pumps for System 1.

The budget price for the supply of all four pumps including pump, coupling, guard, driver, and base plate is $45,000. Taxes, markup, installation, MCCs, controls, and all pipng and appurtenances would be extra.

We note that this pump is $16,000 less exensive than System 1. However, there would be double the valves, instrumentation, pipe supports, and pump supply and discharge piping; this complete would be comparable if not more expensive.

Pump Selection

System 1 is preferred over System 2, primarily for its simplicity and overall reliability. Specifically, Flowserve Pump model 6LR-18SA as supplied by Smith Cameron Industrial.

However, TCHC should comment on the incremental cost of power supply between 37 kW and 56 kW to the tunnel portal. An allowance for an extra 5 kW should be provided to account for other power draws in the tunnel (HVAC, instrumentation, and valve actuators).

4. SUMMARY

Based on the foregoing, we recommend the following:

General Arrangment: Option 2 (pump from the tunnel portal, onto the access road, and over the hillside to the natural lake outlet.)

Pumping System: Two horizontal split case style pumps inside the tunnel, each capable of pumping the design flow at all heads.

We trust the foregoing is satisfactory. We will wait for your comments prior to proceeding with detailed design of the IFR pump station.

KERR WOOD LEIDAL ASSOCIATES LTD.

Prepared by:

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Reviewed by:

11 JUN 08

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RDP/

STATEMENT OF LIMITATIONS

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02390 ROCK ANCHORS, BOLTS AND DOWELS

1. General

1.1. Related Work

.1	Tunneling	Section 02290
.2	Rock Removal	Section 02311
.3	Excavation and Backfill	Section 02315
.4	Penstock Erection	Section 02651
.5	Cast-in-Place Concrete	Section 03300
.6	Structural Steel	Section 05120

1.2. Work Included

.1 All labour, materials, accessories and equipment necessary for the fabrication and installation of soil and rock anchor work as indicated on the drawings and specified herein.

.2 Drilling of holes in loose ground, open-cast rock excavations and within the tunnel.

.3 Grouting and testing of soil and rock anchors.

1.3. Reference Standards

.1 Conform to the current version of the following standards for work in this section:

.2 British Columbia Building Code.

.3 Canadian Standards Association (CSA) Standards:

a) CSA A3001: Portland Cement and Blended Hydraulic Cement

b) CSA A23.2: Methods of Test for Concrete

c) CSA A23.5: Supplementary Cementing Materials

d) CAN/CSA-G30.18: Billet-Steel Bars for Concrete Reinforcement

e) CAN/CSA-G40.20: General Requirements for Rolled or Welded Structural Quality Steel

f) CAN/CSA-G40.21: Structural Quality Steels

g) CSA G164-92: Hot Dip Galvanizing of Irregularly Shaped Articles

h) CSA W59: Welded Steel Construction (Metal Arc Welding)

.4 American Society for Testing and Materials (ASTM) Standards:

a) ASTM A722: Hot-rolled, Proof Stressed and Stress Relieved Post-tensioning Threadbar

b) ASTM C494: Chemical Admixtures for Concrete

c) ASTM C1017: Chemical Admixtures for Use in Producing Flowing Concrete

.5 Post Tensioning Institute

a) Recommendations for Prestressed Rock and Soil Anchors, 1996.

1.4. Submittals

.1 Mill Certificates: Submit two (2) certified copy of mill reports covering chemical and physical properties of steel used in this work.

.2 Submit shop drawings in Adobe pdf format to the Engineer for review prior to fabrication. Shop drawings to show:

a) Drilling methods and hole support.

b) Complete anchor assemblies, including centralisers, grout tubes, couplers and sheaths.

c) Bar schedules with sizes, grades and lengths.

d) Detailed construction sequence.

e) Grout mix design.

f) Detailed testing procedure.

.3 Shop drawings to be prepared in SI metric units.

.4 Review of shop drawings to be for size and arrangement only. Such review will not relieve the anchor fabricator of his responsibility for general and detail dimensions, correct fit, and any errors or omissions.

.5 Allow a minimum of 15 working days for the Engineer to review drawings in addition to the time required for other consultants.

.6 Submission of anchor overdrill lengths and drillhole diameters during installation.

.7 Proof of equipment set-up calibration within two months of testing.

1.5. Quality Assurance

.1 Appoint an independent testing agency that will:

a) Take grout samples and test them in the field for flowability, bleeding and expansion. Prepare three compression test samples for each batch of grout prepared or each day's grouting, whichever is less.

b) Test grout samples in the laboratory for compressive strength.

c) Witness and certify anchor proof tests.

.2 The Owner reserves the right to carry out random tests and inspections of the work, and will designate testing authority and pay costs for same.

2. PRODUCTS

2.1. Materials

.1 Steel

a) Reinforcing bar: to CSA G30.18, Grade 400W

b) Threadbar: CSA G30.18 Grade 500W as noted herein or
pre-stressing steel to ASTM A722, Grade 835/1030 as noted herein.

.2 Accessories

a) Anchorage head, wedges and other accessories: per manufacturer's requirements to suit anchor steel.

b) Corrosion protection: hard polyethylene (PE) or polyvinylchloride (PVC) sheath, 48 MPa tensile strength, 103 MPa compressive strength, free of water-soluble chlorides and any other material causing corrosion, hydrogen embrittlement or stress corrosion. Minimum sheath/grout bond strength at 27 MPa grout compressive strength: 4.8 MPa.

.3 Grout Materials

a) Cement: CSA A3001 Type GU or HE. Alkalai content less than 0.6%, tricalcium aluminate content between 4 and 7%. Use same brand throughout work.

b) Silica fume: CSA A23.5 Type U.

c) Fine aggregate: sand to CSA A23.1.

2.2. Types

.1 Rock Dowels

a) For tying concrete structures to rock foundation and for excavation support.

b) Anchor steel: reinforcing bar G330.18 Grade 400W.

c) Design prestress load: $P_D = 0$ kN

.2 Rock Bolts

a) For anchoring penstock supports and excavation support within the tunnel.

b) Anchor steel: threadbar G30.18 Grade 500W.

c) Pre-approved supplier: Dywidag Systems International Canada Ltd.

d) Galvanized to CSA G164.

e) Design prestress load range: $P_D = 0$ kN

.3 DCP Rock Anchors

a) For anchoring penstock bends.

b) Anchor steel: Threadbar to ASTM A722 Grade 835/1030.

c) Corrosion protection: double corrosion protection (DCP).

d) Pre-approved supplier: Dywidag Systems International Canada Ltd.

e) Design prestress load range: P_D = 500 to 5000 kN.

2.3. Grout

.1 Mix limitations:

a) Silica fume content; 12 and 15% of cement mass.

b) Admixtures: superplasticiser, thixotropic additive and expansion agent.

.2 Plastic properties:

a) Expansion: 1 to 3%., admixture properties to suit grouting times.

b) Bleeding: 1% maximum

.3 Water/cement ratio: 0.40

.4 Flow: 12 to 30 sec per CSA A23.2-1B

.5 Compressive strength:

a) At 7 days: 25 MPa

b) At 28 days: 40 MPa

3. EXECUTION

3.1. Examination

.1 Examine all surfaces and conditions to which the work of this section shall be applied. Ensure that all conditions are suitable to provide a complete and satisfactory installation. Report any deficiencies to the Engineer.

.2 Before commencing installation of the dowels and anchors, ensure that the ground conditions encountered during drilling substantially correspond with the geotechnical information on which the dowel and anchor design was based.

3.2. Drilling

.1 Before drilling, obtain familiarity with all available geotechnical information describing the anticipated ground conditions.

.2 Adopt the drilling method best suited to the anticipated ground conditions.

.3 Use suitable measures to ensure against collapse of the borehole during drilling and anchor installation..

3.3. Rock Dowel and Rock Bolt Installation (Open Excavation)

.1 Fill the borehole with grout by pumping to the lowest point of the borehole through an injection tube.

.2 Fill the entire borehole with grout in one continuous operation.

.3 Lower dowel or bolt into the borehole by self weight.

.4 If the dowel or bolt cannot be completely inserted, remove the anchor and redrill the borehole or otherwise clean it out.

3.4. Rock Bolt Installation (Tunnel)

.1 Comply with Sepcification Section 02290.

3.5. Grouting of Anchors

.1 Lower anchors into the borehole by self weight.

.2 Ensure that smooth sheaths limiting the anchorage zone are properly and securely located.

.3 If the anchor cannot be completely inserted, remove the anchor and redrill the borehole or otherwise clean it out.

.4 Inject the grout by pumping to the lowest point of the borehole through the injection tube.

.5 Inject the entire borehole in one continuous operation.

.6 Carefully control and record the quantity of grout used. The minimum quantity required must be sufficient to completely fill the borehole interstice, voids and fissures anticipated in the ground, as well as expected radial deformations due to grouting pressure.

.7 Ensure that grouting is complete before expanding action of the expansion admixture commences.

.8 For anchors in ground requiring double injection (claquage), clean out injection tube by low pressure flushing with water.

.9 If stipulated on the drawings, re-inject the anchorage zone (claquage) under high pressure after 3 days.

3.6. Stressing and Proof Testing of Anchors

.1 Equipment: Submit proof of set-up calibration within two months of testing.

a) Centre-hole jack and hydraulic pressure pump with calibrated pressure gauge.

b) Annular load cell.

c) Equipment for elongation measurement, capable of recording to the nearest 0.025 mm.

.2 Do not perform stressing and testing until the grout has attained sufficient strength to transfer 1.5 times the design load to the ground.

.3 Perform proof tests for all anchors, unless otherwise instructed by the Engineer.

.4 Load increments: Refer to Post Tensioning Institute, Recommendations for Prestressed Rock and Soil Anchors, 1996.

Incrementally load the anchor by the following increments of the design prestress load (P_D). Hold each increment for at least one minute:

a) 0

b) 0.25 P_D

c) 0.50 P_D

d) 0.75 P_D

e) P_D

f) 1.20 P_D

g) 1.30 P_D Hold for creep test

h) 1.00 P_D Lock-off

i) Lift-off test

.5 Take duplicate readings on the load cell and pump dial gauge at all load increment levels. Record the elongation and load at each increment.

.6 Creep test: perform as part of all proof tests:

a) Hold the proof load (1.30 P_D) for at least 10 minutes.

b) Record the elongation at 0 sec, 30 sec, 1 min., 5 min. and 10 min.

.7 The anchor is acceptable if all the following are satisfied:

a) There is less than 1 mm of movement between the 1 and 10 minute mark, the creep rate is less than 2 mm per log-cycle of time <u>and</u> the creep rate is linear or decreasing throughout the load-hold period.

b) The total movement of the anchor head at all load levels is within 80 and 100% of the theoretical elastic elongation of the free anchor length at that load.

.8 Anchors that fail the proof test must be replaced per the Engineer's instructions at no extra cost to the Owner.

.9 After lock-off, but before removing the jack and load cell, determine the load required to lift off the end anchorage. If this load is less than 95% of P_D, repeat the lock-off and lift-off test.

3.7. Records

.1 Keep an anchor record during the installation period for each anchor.

.2 For each anchor, record:

a) Contractor's name

b) Drill rig operator's name

c) Anchor location

d) Deviation from specified tolerances

e) Design anchor length

f) Installed anchor length

g) Date, start and finish time of drilling

h) Borehole log, including groundwater conditions, casing or bentonite requirements, caving or sloughing experienced, drilling difficulties encountered.

i) Date, time and method of grouting.

j) Quantity of grouts placed.

k) Record of stressing and proof testing.

l) Design modifications made

.3 Maintain and sign all records in ink.

3.8. Protection and Cleanup

.1 At completion and during progress of the work maintain premises in a neat and orderly manner. Dispose of all rubbish, construction debris and surplus materials at least on a weekly basis.

.2 Cover and protect the work from damage by work of other sections.

.3 Protect the work of other sections from damage resulting from the work of this section.

END OF SECTION 02390

18 July 2008

02491 ROCK GROUTING

1 GENERAL

1.1 Related Work

.1 Rock Excavation — Section 02311

.2 Cast-in-Place Concrete — Section 03300

1.2 Work Included

.1 Consolidation grouting to reduce the permeability of the rock mass.

.2 Contact grouting between concrete structures and rock to reduce the permability, and increase the adhesion of the contact.

.3 Coring and drilling through rock and concrete.

.4 Grouting of joints, fissures cracks and cavities in the rock.

1.3 Reference Standards

.1 Do work in accordance with the current versions of the following standards except where specified otherwise.

.2 British Columbia Building Code.

.3 Canadian Standards Association (CSA):

a) CSA A23.1: Concrete Materials and Methods of Concrete Construction

b) CSA A23.2: Methods of Test for Concrete

c) CSA A179: Masonry Mortars and Grouts

d) CAN/CSA A3000: Cementitious Materials Compendium

.4 American Society for Testing and Materials (ASTM) Standards

a) ASTM C494: Chemical Admixtures for Concrete

b) ASTM C1017: Chemical Admixtures for Use in Producing Flowing Concrete

1.4 Submittals

.1 Mix Design: Submit four (4) copies of the proposed grout mix design to the Engineer at least ten working days before any grout is placed on site. Include:

a) Name of Supplier

b) Specification and proportions by weight of each design component.

c) All admixtures

.2 Delivery tickets: Submit copies of delivery ticket for each load of grout delivered to the site together with progress draw each month.

1.5 Quality Assurance

.1 The Owner will appoint a testing agent, who at the Owner's cost, will:

a) Review the proposed mix designs.

b) Take samples of grout as it is delivered to the point of final deposit. Make one set of three test cubes for every 3 cubic metres of grout placed but not less than one set each day if less than 3 cubic metres are placed.

c) Record the grout temperature, air temperature, location of pour and mix number for each set of test cubes made.

d) Make one additional grout cube during cold weather when air temperature is forecast below 5°C within 72 hours of concrete placement. Test additional cube as directed by the Engineer.

e) Test grout for density and viscosity from each day's pour at the time that grout cubes are made.

f) Test one cube at 7 days and two cubes at 28 days. Compressive strength is defined as the average strength of 2-28 day cubes taken from a single batch.

g) Notify the Contractor and Engineer immediately if the results of these tests are not in accordance with the specifications.

h) Send copies of all results to the Engineer and Owner's representative and Contractor.

.2 If testing of grout cubes indicates grout below the specified strength, core samples of the in-situ grout shall be taken and tested to verify the grout meets the specifications. Costs of all retesting and remedial work to repair grout, if required, to be borne by the Contractor.

.3 In the event that the grout does not meet the specified requirements after retesting, the Engineer shall have the right to require remedial measures or to require the grout be removed and replaced at no extra cost to the Owner.

1.6 Site Conditions

.1 Low Temperature Conditions: Comply with the requirements of CSA A23.1 for work when the air temperature falls below 5°C. Do not place grout if temperature is less than 0°C.

.2 Hot Weather Conditions: Comply with CSA A23.1 for work when the air temperature is above 30°C.

1.7 Coordination

.1 Be responsible for correctness of measurements and report to the Engineer, in writing, all discrepancies between measurements and those shown on drawings prior to commencing work.

.2 The Engineer may modify the injection programme at any time in order to suit actual site conditions as they are revealed during the construction activities.

1.8 Inspection

.1 Obtain the Engineer's approval before drilling injection holes or injecting grout.

.2 Provide at least 24 hours notice to the Engineer for required inspections.

2 PRODUCTS

2.1 Grout Materials

.1 Cement: Type GU or HE, conforming to CSA 3000.

.2 Bentonite: sodium montmorillonite with following properties:

a) Retention on #200 sieve $\leq$ 2.5%

b) Moisture $\leq$ 10%

c) Fan reading $\geq$ 30 at 600 rpm

d) Yield point 3 x plastic viscosity

e) Plasticity index ≥ 400%

.3 Aggregates: Fine aggregates comprising natural round-grained sand conforming to CSA A179. Maximum size of coarse aggregate 2.4 mm. Grading:

Particle size (mm)	% finer
2.4	100
1.2	95 to 100
0.6	60 to 85
0.3	20 to 50
0.15	10 to 30
0.075	0 to 5

.4 Water: potable. Use hot water in the mix when forecasted temperature is below 5°C.

.5 Accelerating, retarding and water-reducing admixtures: to ASTM C494 and only with prior approval of the Engineer.

.6 Superplasticizers: to ASTM C1017.

2.2 Grout Mixes

.1 Conform to CSA A179 and British Columbia Building Code.

.2 Grout mixes are to be designed by the Contractor to meet the requirements listed herein and in the Notes on the structural drawings. Grouts are principally expected to comprise cement, water and plasticizing admixtures as well as small amounts of bentonite. Grouts containing aggregates and accelerating admixtures may be used to fill large voids near to the injection boreholes, and when grout takes are high.

2.3 Equipment

.1 Select mixing, pumping, transport and injection equipment as required to suit the proposed method of execution.

.2 Injection equipment must be capable of injecting grouts at the following pressures:

a) Consolidation grouting up to 3000 kPa.

b) Contact grouting up to 500 kPa].

.3 Injection equipment must be furnished with gauges capable of reading injection pressures to an accuracy of 5% of the design injection pressure.

.4 Injection pipes must be capable of reaching each injection stage of the borehole and be furnished with packers or manchettes capable of providing a tight seal between the injection pipe and the borehole.

.5 Packers shall consist of mechanically expanded rings or pneumatically expanded sleeves of rubber or other suitable material, which can be set tightly in a drillhole at any depth required. Packers shall be capable of withstanding pressures of up to 10000 kPa without leakage. Keep at the site single packers, as well as double packers spaced 3 and 6 m apart to allow to isolate a section of a hole, and shall have an adequate supply of spare packers of a size to suit the various drill hole diameters.

2.4 Water-Pressure Testing Equipment

.1 Provide a sufficient number of complete sets of pressure testing equipment (with spares) to allow simultaneous testing at the various drilling locations.

.2 Unless otherwise directed by the Engineer, the pumps shall have a capacity of 100 liters per minute at a pressure of 2500 kPa and shall be capable of exerting a pressure of at least 5000 kPa. The pumping system shall be capable of maintaining any desired pressure without fluctuation and the pressure and discharge must be continuously adjustable.

.3 Water pressure shall be measured by means of a pressure gauge with an accuracy of 50 kPa and a range of 5000 kPa. Discharge shall be measured with an accuracy of 2%. The water meter shall measure the discharge from 0.5 l/min. Water meters as well as all pipes, hoses and couplings shall be designed to resist a pressure of 6000 kPa.

.4 Water meters and pressure gauges shall be calibrated and certified by an independent laboratory prior to installation at the Site and shall be subject to periodic verification. One pressure gauge and one meter shall, after independent checking, remain at the disposal of the Engineer for further checking purposes. The Contractor may be requested to establish, by way of tests, correction graphs for pressure losses occurring in the pipes. Pressure gauges shall be installed directly at the collar of the drillhole.

.5 For water level measurements, electrical probes with an accuracy of 10 mm shall be used. They shall be provided with measuring tapes marked with centimeter gradations.

3 EXECUTION

3.1 General

.1 All work of this section must take place in the presence of the representative of the Engineer, unless that person decides, on a day by day basis, that their presence is not required.

.2 Consolidation grouting is expected to be either in single or in multiple stages, using the successive withdrawl method, depending on the rock conditions encountered.

.3 Contact grouting is expected to be in a single stage.

.4 Single-stage Grouting: Single-stage grouting is carried out by introducing the grout at either the collar of the hole through a nipple or by means of a grout supply pipe at the bottom of the hole. The entire length of the hole is grouted in one operation.

.5 Multi-stage Grouting using the successive withdrawl method: the hole is drilled to its full depth, washed out, and the packer is set at the top of the deepest section be grouted. The section is then water-pressure tested and grouted at the required pressure through the grout supply pipe. The packer is allowed to remain in place until there is no back pressure and then withdrawn to the top of the next deepest section to be grouted, and thus successively water-pressure testing and grouting the entire length of the hole.

.6 Design injection pressures and pump injection pressures will be confirmed by the Engineer after injection hole drilling and will be based on:

a) The anticipated grout takes.

b) The number of injection stages.

c) The depth of the packer for a given injection stage.

d) Any other relevant geotechnical considerations.

.7 Design injection pressures are defined as the pressure in the borehole beyond the packer. For deep packer placements, the design injection pressure is equal to the injection pump pressure plus the product of the vertical distance between the pump pressure gauge and the packer and the density of the fluid grout.

3.2 Injection Hole Drilling

.1 Consolidation Grouting:

a) Hole diameter to suit injection pipe packer diameter, but not less than 45 mm.

b) Drill holes no closer than 8 m to holes previously grouted within 24 hours.

c) Carefully log cuttings or core recoveries.

d) Drill hole to slopes, depths and/or lengths specified.

e) Perform Water Pressure Tests in all consolidation grouting injection holes.

.2 Contact grouting

a) Hole diameter to suit injection pipe packer diameter, but not less than 25 mm.

b) Drill contact grout holes after completing the concrete structures.

c) Inlieu of drilling through concrete, pre-placed pvc sleeves of equal diameter to the borehole diameter may be used.

d) In general, drill all contact grouting holes for a given structure prior to commencing grout injection.

e) Carefully log cuttings or core recoveries.

f) Drill hole to slopes, depths and/or lengths specified.

3.3 Grout Mixing and Transport

.1 Do not mix concrete while the air temperature is below 5°C. The Contractor is responsible for any defective work resulting from freezing or injury in any manner during placing and curing and shall replace concrete not meeting these specifications at no extra cost to the Owner.

.2 Truck-mixed and mechanical small batch mixed grout: mix for a minimum of 3 minutes and a maximum of 5 minutes.

.3 Discharge grout at the placing site within

a) 1 hours after cement was first introduced into the mix.

b) Prior to any set of the mix.

.4 Comply with the requirements of CSA A23.1 when the air temperature rises above 30°C.

.5 Comply with the requirements of CSA A23.1 when the air temperature falls below 5°C.

3.4 Injection

.1 Inject grout using the single stage or multi-stage injection method defined above.

.2 Definition of completion of grouting:

a) The grout take, at the maximum injection pressure, is less than 30 litres per 10 minutes.

b) Stop contact grouting immediately if grout is observed exiting adjacent construction joints, concrete-rock contact joints or from the face of the rock mass. Move the to the next injection hole and continue injecting from this hole.

3.5 Control Borings

.1 Perform control borings as required by the Engineer.

.2 Carefully log cuttings/recovered cores.

.3 Perform Water Pressure Tests in all control borings.

.4 Fully grout the control borehole after Water Pressure Tests are complete.

3.6 Water Pressure Testing (Lugeon Method)

.1 Perform water-pressure tests by the Lugeon Test Method in grout and exploratory holes or section of holes, as the drilling proceeds or after completion of a drillhole, as and where directed by the Engineer.

.2 The permeabilty as measured in the drillholes shall be expressed in Lugeon units, 1 Lugeon unit is equal to a water take of 1 liter per minute per linear meter of hole at a pressure of 1000 kPa.

.3 Seal the hole or section of a hole to be subject to testing with single packers, or double packers with the packers 3 m apart. Measure the groundwater table, if any, before and after each test.

.4 After the packer is installed, flush the drillhole throughly and check that the flow through the packer is not obstructed.

.5 Pump water shall be pumped into the hole through a header apply pressure in steps up to the maximum pressure. The maximum pressure shall be determined by the

Engineer, but shall in general not exceed 1000 kPa. Where water-pressure tests are carried out in grout holes, the maximum pessure shall not exceed the maximum grouting pressure to be used. Unless otherwise directed by the Engineer, the pressure steps shall be as follows:

Step	Pressure
1	0.2 P_{max}
2	0.5 P_{max}
3	P_{max}
4	0.5 P_{max}
5	0.2 P_{max}

.6 Water take measurements shall be started only after a stable pressure has been established. For each pressure step, measure water takes 3 times for periods of 3 minutes each. Each pressure step will therefore require approximately 9 minutes and the total time requirement for each Lugeon Test will be 45 minutes to 1 hour.

.7 If during water take measurements the rate of take or pressure changes, extend the test until discharge and pressure remain constant over a period of 5 consecutive minutes.

.8 If, due to high takes, it is not possible to maintain the required pressure, operate the pump at its maximum discharge rate for 10 minutes and measure the pressure at 2 minute intervals.

3.7 Protection and Clean-up

.1 At completion and during progress of the work maintain premises in a neat and orderly manner. Dispose of all rubbish, construction debris and surplus materials at least on a weekly basis.

.2 Cover and protect the work from damage by work of other sections.

.3 Protect the work of other sections from damage resulting from the work of this section.

END OF SECTION 02495

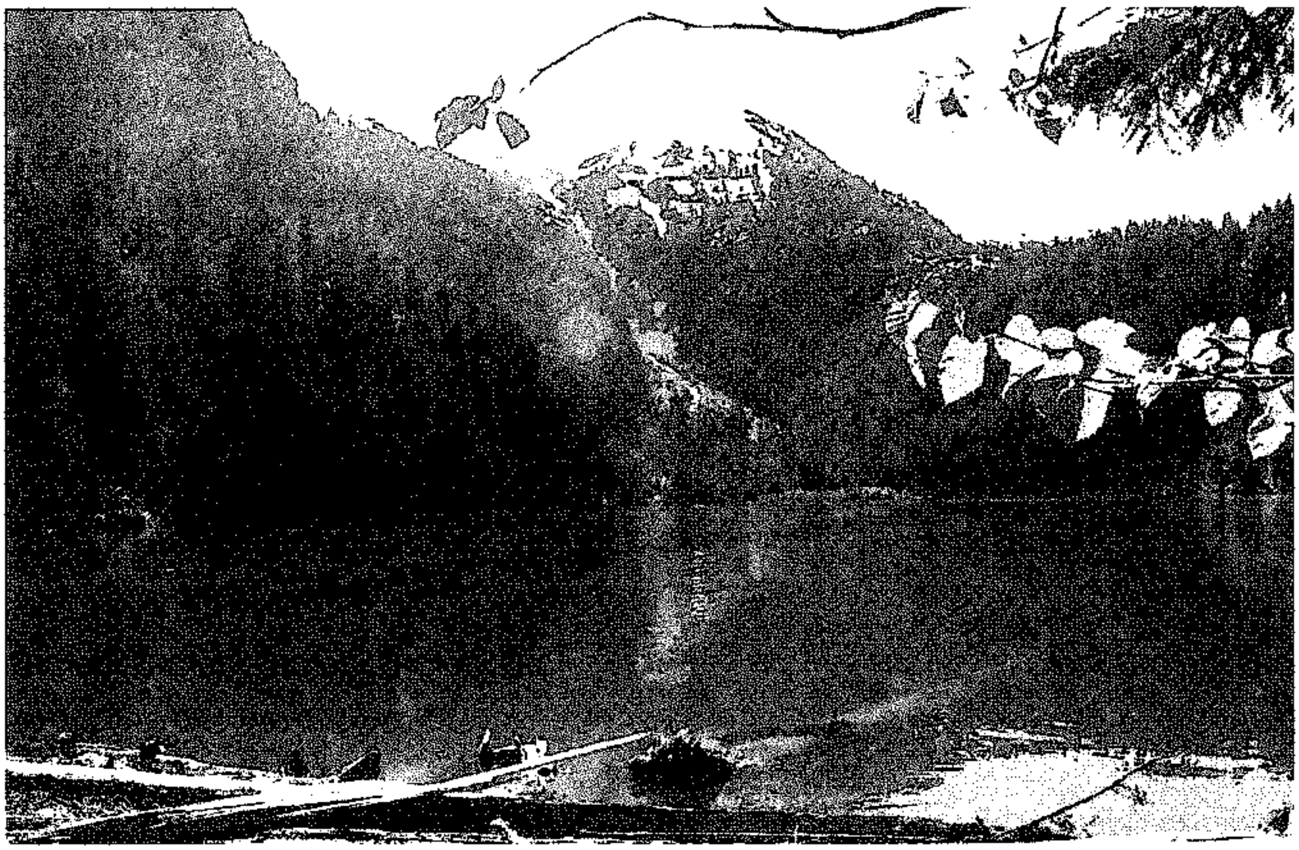
19 September 2007

Tyson Creek Hydro Corporation

Tyson Creek Hydroelectric Project

Civil Works Technical Specifications

Tunnel Contract



July 2008 DRAFT Rev. A

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Technical Specifications

Section		Rev.
02290	Tunneling	
02390	Rock Anchors, Bolts and Dowels	
02491	Rock Grouting	
02650	Steel Pipe	
02660	Pipeline and Penstock Installation	
03100	Concrete Formwork and Falsework	
03200	Concrete Reinforcement	
03300	Concrete	
05090	Pipe Welding	
05120	Structural Steel	
08010	Door & Frame Schedule	
08110	Standard Steel Doors	
08111	Standard Steel Frames	
08710	Finish Hardware	
09801	Protective Coatings - Penstock	
09900	Painting	
09970	Field Applied Coatings	
11620	Butterfly Valves	
11145	Packaged Pump Station	
15050	Basic Mechanical Materials and Methods	
15075	Identification Systems	
15100	Building Services Piping	
15200	Process Piping	
15210	Valves	

02491 ROCK GROUTING

1 GENERAL

1.1 Related Work

.1 Rock Excavation — Section 02311

.2 Cast-in-Place Concrete — Section 03300

1.2 Work Included

.1 Consolidation grouting to reduce the permeability of the rock mass.

.2 Contact grouting between concrete structures and rock to reduce the permability, and increase the adhesion of the contact.

.3 Coring and drilling through rock and concrete.

.4 Grouting of joints, fissures cracks and cavities in the rock.

1.3 Reference Standards

.1 Do work in accordance with the current versions of the following standards except where specified otherwise.

.2 British Columbia Building Code.

.3 Canadian Standards Association (CSA):

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1.4 Submittals

.1 Mix Design: Submit four (4) copies of the proposed grout mix design to the Engineer at least ten working days before any grout is placed on site. Include:

a) Name of Supplier

b) Specification and proportions by weight of each design component.

c) All admixtures

.2 Delivery tickets: Submit copies of delivery ticket for each load of grout delivered to the site together with progress draw each month.

1.5 Quality Assurance

.1 The Owner will appoint a testing agent, who at the Owner's cost, will:

a) Review the proposed mix designs.

b) Take samples of grout as it is delivered to the point of final deposit. Make one set of three test cubes for every 3 cubic metres of grout placed but not less than one set each day if less than 3 cubic metres are placed.

c) Record the grout temperature, air temperature, location of pour and mix number for each set of test cubes made.

d) Make one additional grout cube during cold weather when air temperature is forecast below 5°C within 72 hours of concrete placement. Test additional cube as directed by the Engineer.

e) Test grout for density and viscosity from each day's pour at the time that grout cubes are made.

f) Test one cube at 7 days and two cubes at 28 days. Compressive strength is defined as the average strength of 2-28 day cubes taken from a single batch.

g) Notify the Contractor and Engineer immediately if the results of these tests are not in accordance with the specifications.

h) Send copies of all results to the Engineer and Owner's representative and Contractor.

.2 If testing of grout cubes indicates grout below the specified strength, core samples of the in-situ grout shall be taken and tested to verify the grout meets the specifications. Costs of all retesting and remedial work to repair grout, if required, to be borne by the Contractor.

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1.6 Site Conditions

.1 Low Temperature Conditions: Comply with the requirements of CSA A23.1 for work when the air temperature falls below 5°C. Do not place grout if temperature is less than 0°C.

.2 Hot Weather Conditions: Comply with CSA A23.1 for work when the air temperature is above 30°C.

1.7 Coordination

.1 Be responsible for correctness of measurements and report to the Engineer, in writing, all discrepancies between measurements and those shown on drawings prior to commencing work.

.2 The Engineer may modify the injection programme at any time in order to suit actual site conditions as they are revealed during the construction activities.

1.8 Inspection

.1 Obtain the Engineer's approval before drilling injection holes or injecting grout.

.2 Provide at least 24 hours notice to the Engineer for required inspections.

2 PRODUCTS

2.1 Grout Materials

.1 Cement: Type GU or HE, conforming to CSA 3000.

.2 Bentonite: sodium montmorillonite with following properties:

a)	Retention on #200 sieve	≤ 2.5%
b)	Moisture	≤ 10%
c)	Fan reading	≥ 30 at 600 rpm

d) Yield point 3 x plastic viscosity

e) Plasticity index ≥ 400%

.3 Aggregates: Fine aggregates comprising natural round-grained sand conforming to CSA A179. Maximum size of coarse aggregate 2.4 mm. Grading:

Particle size (mm)	% finer
2.4	100
1.2	95 to 100
0.6	60 to 85
0.3	20 to 50
0.15	10 to 30
0.075	0 to 5

.4 Water: potable. Use hot water in the mix when forecasted temperature is below 5°C.

.5 Accelerating, retarding and water-reducing admixtures: to ASTM C494 and only with prior approval of the Engineer.

.6 Superplasticizers: to ASTM C1017.

2.2 Grout Mixes

.1 Conform to CSA A179 and British Columbia Building Code.

.2 Grout mixes are to be designed by the Contractor to meet the requirements listed herein and in the Notes on the structural drawings. Grouts are principally expected to comprise cement, water and plasticizing admixtures as well as small amounts of bentonite. Grouts containing aggregates and accelerating admixtures may be used to fill large voids near to the injection boreholes, and when grout takes are high.

2.3 Equipment

.1 Select mixing, pumping, transport and injection equipment as required to suit the proposed method of execution.

.2 Injection equipment must be capable of injecting grouts at the following pressures:

a) Consolidation grouting up to 3000 kPa.

b) Contact grouting up to 500 kPa].

.3 Injection equipment must be furnished with gauges capable of reading injection pressures to an accuracy of 5% of the design injection pressure.

.4 Injection pipes must be capable of reaching each injection stage of the borehole and be furnished with packers or manchettes capable of providing a tight seal between the injection pipe and the borehole.

.5 Packers shall consist of mechanically expanded rings or pneumatically expanded sleeves of rubber or other suitable material, which can be set tightly in a drillhole at any depth required. Packers shall be capable of withstanding pressures of up to 10000 kPa without leakage. Keep at the site single packers, as well as double packers spaced 3 and 6 m apart to allow to isolate a section of a hole, and shall have an adequate supply of spare packers of a size to suit the various drill hole diameters.

2.4 Water-Pressure Testing Equipment

.1 Provide a sufficient number of complete sets of pressure testing equipment (with spares) to allow simultaneous testing at the various drilling locations.

.2 Unless otherwise directed by the Engineer, the pumps shall have a capacity of 100 liters per minute at a pressure of 2500 kPa and shall be capable of exerting a pressure of at least 5000 kPa. The pumping system shall be capable of maintaining any desired pressure without fluctuation and the pressure and discharge must be continuously adjustable.

.3 Water pressure shall be measured by means of a pressure gauge with an accuracy of 50 kPa and a range of 5000 kPa. Discharge shall be measured with an accuracy of 2%. The water meter shall measure the discharge from 0.5 l/min. Water meters as well as all pipes, hoses and couplings shall be designed to resist a pressure of 6000 kPa.

.4 Water meters and pressure gauges shall be calibrated and certified by an independent laboratory prior to installation at the Site and shall be subject to periodic verification. One pressure gauge and one meter shall, after independent checking, remain at the disposal of the Engineer for further checking purposes. The Contractor may be requested to establish, by way of tests, correction graphs for pressure losses occurring in the pipes. Pressure gauges shall be installed directly at the collar of the drillhole.

.5 For water level measurements, electrical probes with an accuracy of 10 mm shall be used. They shall be provided with measuring tapes marked with centimeter gradations.

3 EXECUTION

3.1 General

.1 All work of this section must take place in the presence of the representative of the Engineer, unless that person decides, on a day by day basis, that their presence is not required.

.2 Consolidation grouting is expected to be either in single or in multiple stages, using the successive withdrawl method, depending on the rock conditions encountered.

.3 Contact grouting is expected to be in a single stage.

.4 <u>Single-stage Grouting</u>: Single-stage grouting is carried out by introducing the grout at either the collar of the hole through a nipple or by means of a grout supply pipe at the bottom of the hole. The entire length of the hole is grouted in one operation.

.5 <u>Multi-stage Grouting using the successive withdrawl method</u>: the hole is drilled to its full depth, washed out, and the packer is set at the top of the deepest section be grouted. The section is then water-pressure tested and grouted at the required pressure through the grout supply pipe. The packer is allowed to remain in place until there is no back pressure and then withdrawn to the top of the next deepest section to be grouted, and thus successively water-pressure testing and grouting the entire length of the hole.

.6 Design injection pressures and pump injection pressures will be confirmed by the Engineer after injection hole drilling and will be based on:

a) The anticipated grout takes.

b) The number of injection stages.

c) The depth of the packer for a given injection stage.

d) Any other relevant geotechnical considerations.

.7 Design injection pressures are defined as the pressure in the borehole beyond the packer. For deep packer placements, the design injection pressure is equal to the injection pump pressure plus the product of the vertical distance between the pump pressure gauge and the packer and the density of the fluid grout.

3.2 Injection Hole Drilling

.1 Consolidation Grouting:

a) Hole diameter to suit injection pipe packer diameter, but not less than 45 mm.

b) Drill holes no closer than 8 m to holes previously grouted within 24 hours.

c) Carefully log cuttings or core recoveries.

d) Drill hole to slopes, depths and/or lengths specified.

e) Perform Water Pressure Tests in all consolidation grouting injection holes.

.2 Contact grouting

a) Hole diameter to suit injection pipe packer diameter, but not less than 25 mm.

b) Drill contact grout holes after completing the concrete structures.

c) Inlieu of drilling through concrete, pre-placed pvc sleeves of equal diameter to the borehole diameter may be used.

d) In general, drill all contact grouting holes for a given structure prior to commencing grout injection.

e) Carefully log cuttings or core recoveries.

f) Drill hole to slopes, depths and/or lengths specified.

3.3 Grout Mixing and Transport

.1 Do not mix concrete while the air temperature is below 5°C. The Contractor is responsible for any defective work resulting from freezing or injury in any manner during placing and curing and shall replace concrete not meeting these specifications at no extra cost to the Owner.

.2 Truck-mixed and mechanical small batch mixed grout: mix for a minimum of 3 minutes and a maximum of 5 minutes.

.3 Discharge grout at the placing site within

a) 1 hours after cement was first introduced into the mix.

b) Prior to any set of the mix.

.4 Comply with the requirements of CSA A23.1 when the air temperature rises above 30°C.

.5 Comply with the requirements of CSA A23.1 when the air temperature falls below 5°C.

3.4 Injection

.1 Inject grout using the single stage or multi-stage injection method defined above.

.2 Definition of completion of grouting:

a) The grout take, at the maximum injection pressure, is less than 30 litres per 10 minutes.

b) Stop contact grouting immediately if grout is observed exiting adjacent construction joints, concrete-rock contact joints or from the face of the rock mass. Move the to the next injection hole and continue injecting from this hole.

3.5 Control Borings

.1 Perform control borings as required by the Engineer.

.2 Carefully log cuttings/recovered cores.

.3 Perform Water Pressure Tests in all control borings.

.4 Fully grout the control borehole after Water Pressure Tests are complete.

3.6 Water Pressure Testing (Lugeon Method)

.1 Perform water-pressure tests by the Lugeon Test Method in grout and exploratory holes or section of holes, as the drilling proceeds or after completion of a drillhole, as and where directed by the Engineer.

.2 The permeabilty as measured in the drillholes shall be expressed in Lugeon units, 1 Lugeon unit is equal to a water take of 1 liter per minute per linear meter of hole at a pressure of 1000 kPa.

.3 Seal the hole or section of a hole to be subject to testing with single packers, or double packers with the packers 3 m apart. Measure the groundwater table, if any, before and after each test.

.4 After the packer is installed, flush the drillhole throughly and check that the flow through the packer is not obstructed.

.5 Pump water shall be pumped into the hole through a header apply pressure in steps up to the maximum pressure. The maximum pressure shall be determined by the

Engineer, but shall in general not exceed 1000 kPa. Where water-pressure tests are carried out in grout holes, the maximum pessure shall not exceed the maximum grouting pressure to be used. Unless otherwise directed by the Engineer, the pressure steps shall be as follows:

Step	Pressure
1	0.2 P_{max}
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3	P_{max}
4	0.5 P_{max}
5	0.2 P_{max}

.6 Water take measurements shall be started only after a stable pressure has been established. For each pressure step, measure water takes 3 times for periods of 3 minutes each. Each pressure step will therefore require approximately 9 minutes and the total time requirement for each Lugeon Test will be 45 minutes to 1 hour.

.7 If during water take measurements the rate of take or pressure changes, extend the test until discharge and pressure remain constant over a period of 5 consecutive minutes.

.8 If, due to high takes, it is not possible to maintain the required pressure, operate the pump at its maximum discharge rate for 10 minutes and measure the pressure at 2 minute intervals.

3.7 Protection and Clean-up

.1 At completion and during progress of the work maintain premises in a neat and orderly manner. Dispose of all rubbish, construction debris and surplus materials at least on a weekly basis.

.2 Cover and protect the work from damage by work of other sections.

.3 Protect the work of other sections from damage resulting from the work of this section.

END OF SECTION 02495

19 September 2007

Tyson Creek Hydro Corporation

Tyson Creek Hydroelectric Project

Civil Works Technical Specifications

Tunnel Contract

July 2008 DRAFT Rev. A

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GEA Project No. 07012

Technical Specifications

02290 TUNNELING

1. GENERAL

1.1. Related Work

.1 Excavation, Trenching and Backfill	Section 02200
.2 Roads	Section 02224
.3 Cast-in-place Concrete	Section 03330
.4 Shotcrete	Section 03361

1.2. Work Included

.1 Mobilization of tunnelling equipment and supplies, ventilation equipment, and all portal facilities required for the safe and efficient execution of the work.

.2 Scaling and rock-bolting at tunnel portals.

.3 Excavation and temporary support of a 2.75 m high by 2.5 m wide tunnel of approx. 1055 m length in dioritic rock with some known and other unknown fracture and shear zones.

.4 Placement of tunnel spoil in a designated spoil area at the tunnel portal.

.5 Control of water and compliance with discharge water quality requirements.

.6 Installation of permanent support in the tunnel.

1.3. Reference Standards

.1 Do work in accordance with the standards except where specified otherwise.

.2 British Columbia Building Code 1992.

.3 Workers Compensation Board Industrial Health and Safety Regulations.

1.4. Additional Definitions

.1 Drawings: the plans, schedules and sketches prepared by the Engineer.

.2 Shop drawings: the plans, schedules and sketches prepared by the Contractor as a part of the work of this Section.

1.5. Submittals

.1 Safety Program, complete with names and copies of mine rescue certificates and first aid certificates, proposed organization and safety equipment.

.2 Proposed Ventilation Design

.3 Equipment List, of all production equipment for tunnel excavation and support.

.4 Shotcrete Mix Design: Submit four (4) copies of the proposed shotcrete mix design to the Engineer at least ten working days before any shotcrete is placed on site. Include:

a) Name of Supplier of all purchased materials

b) Specification and proportions by weight of each design component.

c) All admixtures

d) Name and certificate of nozzleman assigned to the work. The Engineer will require that test panels are made prior to and at various times during the work for the purpose of quality control tests.

.5. Portal Facilities Layout, to include details of proposed equipment, fuel storage, explosives storage, lubricants and waste oil storage, office and change room, tunnel water treatment complete with oil separator, and spoil area layout.

.6 Grouting method statement, complete with description of proposed equipment.

1.6. Quality Assurance

.1 The Owner will appoint a testing agent, who at the Owner's cost, will:

a) Carry out rock bolt pull tests.

b) Notify the Contractor and Engineer immediately if the results of these tests are not in accordance with the specifications.

c) Send copies of all results to the Engineer and Owner's representative and Contractor.

1.7. Site Testing

.1 The Contractor shall carry out rock bolt pull tests to 60% of the yield strength of the bolts on randomly selected bolts as agreed with the Engineer, testing 2% of the total number of bolts installed.

1.8. Site Conditions

.1 Tunnel construction operations shall comply with applicable laws and regulations governing ventilation; fuel, lubricant and waste oil storage; explosives handling and storage; and treatment and discharge of water.

1.9. Coordination

.1Be responsible for correctness of tunnel alignment surveys.

1.10. Inspection

.1 The Contractor shall be responsible for installation of all temporary support measures in the tunnel and shall carry out such evaluations of the ground conditions as he deems necessary to assure safe and efficient progress of the work.

.2 The Contractor shall notify the Engineer whenever Class IV or Class V ground conditions are encountered to give the Engineer an opportunity to inspect the tunnel prior to covering with shotcrete.

.3 The Engineer will carry out mapping for the purpose of rock classification and will designate in writing to the Contractor, the limits of the rock class designations and permanent support types to be installed by the Contractor.

.4 Provide at least 4 hours notice to the Engineer for required inspections.

1.11. Measurement and Payment

.1 Unit prices submitted shall be used to calculate the cost of the work.

.2 Mobilization for tunnel construction will be paid at the lump sum price entered for this item in the Schedule of Estimated Quantities and Prices and shall include for all costs of establishment of the portal facilities in accordance with the portal facilities layout plan submitted by the Contractor and approved by the Engineer.

.3 Demobilization for tunnel construction will be paid at the lump sum price entered for this item in the Schedule and shall include for all costs of removal of temporary facilities and final grading of the tunnel spoil area.

.4 Excavation of the tunnel will be paid on the basis of linear metres of tunnel requiring each of the 5 support classes as designated by the Engineer. Payment on a linear meter basis shall include for all costs of excavation to the lines, grades and dimensions shown on the Drawings, for drilling of the probe holes required by the specifications, for disposal of tunnel spoil and for treatment and discharge of water from the tunnel. No separate payment will be made for passing bays, equipment recesses, or other enlargements of the tunnel which the Contractor may elect to excavate for his own convenience. No separate payment will be made for installation, operation and final removal of tunnel ventilation, lighting, power, water, compressed air and drainage systems.

.5 Temporary support installed by the contractor for safety and efficiency of production will, if accepted upon inspection by the Engineer as correctly installed in accordance with the requirements of permanent support, be paid at the unit rates for permanent support.

.6 Permanent support measures installed in the tunnel and at the portals will be paid at the unit rates entered for the relevant items in the Schedule and shall include for all costs of cleaning of rock surfaces for shotcrete and concrete.

.7 Drilling for drainage in Class V ground and grouting of spiling pipes, if required by the Engineer, will be paid on the basis of actual and necessary net cost plus 15% as demonstrated by invoices for materials and equipment rental, transport, and manpower timesheets from the Provisional Sum established by the Engineer for this purpose. Work under provisional sums shall be carried out only under the direction of the Engineer and shall be carried out based on an agreed written method statement for each occurrence.

2. PRODUCTS

2.1. Concrete Materials

.1 Refer to Section 03300.

2.2. Concrete Mixes

.1 Concrete Type C30, for tunnel invert in Class IV and V ground support, refer to Section 03300.

.2 Shotcrete – refer to Section 03361.

2.3. Grouts

.1 Neat cement grout shall be Portland cement Type 10 with addition of 3% pre-hydrated bentonite by weight of cement, mixed in a high shear mixer to 1:1 w/c ratio or such other consistency as the Engineer may require.

.2 Epoxy resin cartridges for rock bolt anchoring shall be a commercial product approved by the Engineer.

2.4. Rockbolts

.1 Type 1 rockbolts as designated on the Drawings shall be 22 mm nominal diameter Dywidag threaded bar with steel strength fy/fu of 413/620 MPa. Rockbolts shall be 1.2 m length for use in the tunnel and 3 m length for use at the portals and shall be supplied with 100 x 100 mm x 6 mm thick deformable face plates and standard nuts as supplied by Dywidag.

2.5. Welded Wire Mesh

.1 Welded wire mesh shall be carbon steel welded wire mesh as designated on the Drawings, 152 x 152 x MW18.7 x MW18.7.

2.6. Tunnel Support Sets

.1 Tunnel support sets shall be W100 x 17 wide flange I-beams rolled to the dimensions shown on the drawings and provided with splice plates and foot plates and welded angle brackets for spreader bars as shown on the Drawings. The Contractor shall submit a detailed fabrication sketch for review by the Engineer prior to ordering the steel sets.

.2 Spreader bars shall be 25 mm nominal diameter reinforcing steel, cut and bent on site to lengths as needed in the work.

.3 Spiling pipes shall be 75 mm dia steel pipe of minimum 8 mm wall thickness.

3. EXECUTION

3.1. Portal Facilities

.1 The portal facilities established by the Contractor shall be constructed in a neat and orderly manner and shall be maintained in full compliance with the requirements of the General Conditions and Special Conditions, with particular emphasis on safety and environmental considerations.

.2 The excavated face of the rock bluff for the south portal of the tunnel shall be scaled, rockbolted and shotcreted for safety prior to the start of tunnel excavation. The excavated face for the north portal of the tunnel shall be scaled, rockbolted and shotcreted for safety prior to the tunnel face coming within 20 m of holing through to the north portal.

3.2. Tunnel Excavation and Support

.1 The majority of the length of the tunnel is expected to be excavated in very good and good dioritic rock, Class I and Class II. Notwithstanding this, a number of shear zones of varying width and strength and with unknown water conditions are expected to be intersected during the course of the work as shown on the Drawings.

.2 The Contractor shall drill a pilot hole or probe hole to 30 m ahead of the face and shall redrill this probe hole after each 15 m of advance of the tunnel face. The Engineer may require that additional probe holes be drilled if very soft ground and/or high inflows of water are encountered.

.3 In the event that significant water inflows and or ground conditions indicative of Class V ground are intersected in the probe hole(s), the Contractor shall immediately notify the Engineer. The Contractor shall have on 48 hr maximum mobilization leadtime standby, grouting equipment for pressure grouting with normal cement grout or such other chemical grouts as may be approved by the Engineer to reduce inflows and stabilize the ground prior to excavation. Such work shall be carried out in accordance with a method statement prepared by the Contractor and submitted to the Engineer for approval. If required, pregrouting of the tunnel will be paid under the Provisional Sum item established for this contingency.

.4 Drilling and blasting of the tunnel advance rounds shall be carried out in accordance with commonly accepted standards of good workmanship for civil tunnelling and in full compliance with applicable safety regulations.

.5 The contractor shall adjust his drilling pattern in accordance with the ground conditions to produce a regular and clean profile and shall avoid damage to the rock to the greatest degree reasonably possible by appropriate loading and sequential firing of the holes.

.6 Scaling and temporary support shall be completed concurrently with excavation advance and shall not lag behind the face by more than the length of one round.

.7 Permanent support shall be installed concurrently with the advance of the face and shall not lag behind the face by more than 50 m.

.8 Passing bays, turnouts, equipment recesses and any other enlargements of the tunnel cross-section which the Contractor elects to construct for his own convenience shall be excavated only in Class I and Class II ground to the dimensions and at the locations approved by the Engineer. Such approval will not relieve the Contractor of his obligations for construction of a safe and stable tunnel. Support installed in approved enlargements will be paid at the unit rates in the Schedule.

.9 All rock surfaces and surfaces of previously installed shotcrete shall be pressure washed to assure a good bond of the shotcrete to the substrate.

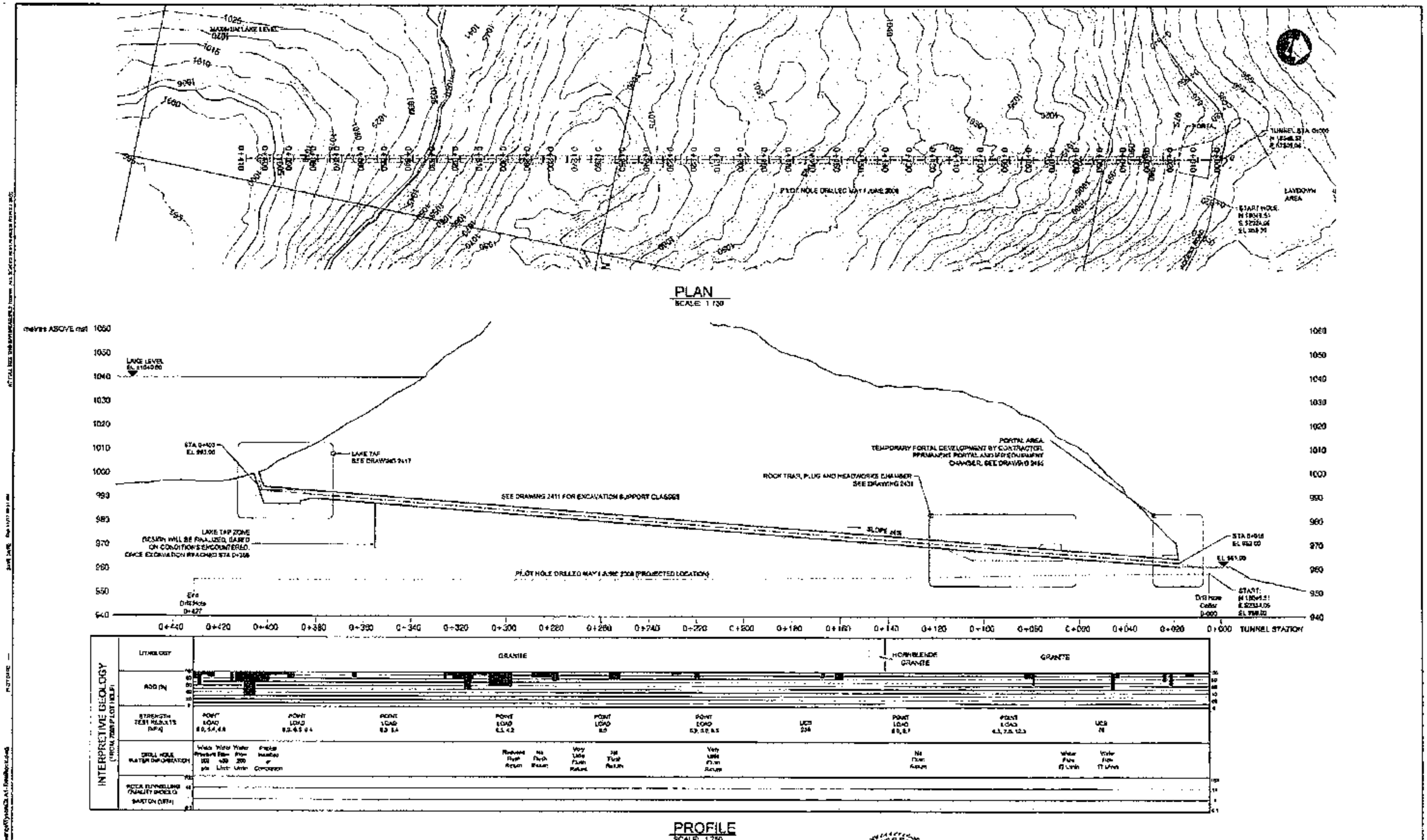
.10 Drummy shotcrete, howsoever caused, shall be removed and replaced at the expense of the Contractor.

.11 Rock bolts shall be installed as recommended by the material suppliers, including flushing of the holes.

.12 Adjacent panels of wire mesh shall be overlapped by one full mesh or 152 mm, whichever is greater.

.13 Steel sets shall be installed with temporary footblocks and shall be brought to line and grade prior to shotcreting. Footblocks shall be removed for installation of invert bracing and pouring of the concrete invert slab.

END OF SECTION 02290



Issued for	Issue	Date	Issued By
Reference	R1	[illegible]	AG
Approvals	A1	[illegible]	GD
Tender	T1	[illegible]	GD
Permits			
Construction			
Record Drawings			

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
0	[illegible]	D. DYER	AG	D. WYLLIE	

KERR WOOD LEIDAL
CONSULTING ENGINEERS

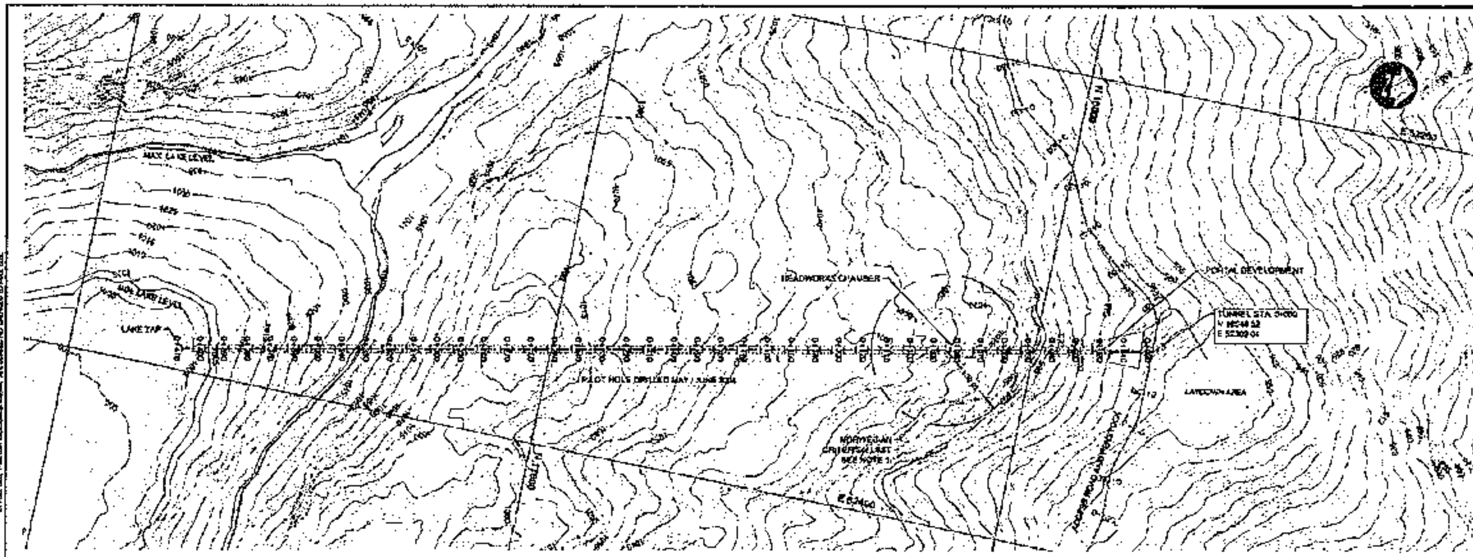
GEA Dyer Engineering Associates Ltd.

wyllie & norrish

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION
PLAN AND PROFILE

Scale	Project No.	Discipline	Drawing Number	Rev.
1:750	2469-002	STRUCT	2401	0

Client: Tyson Creek Hydro Corp.



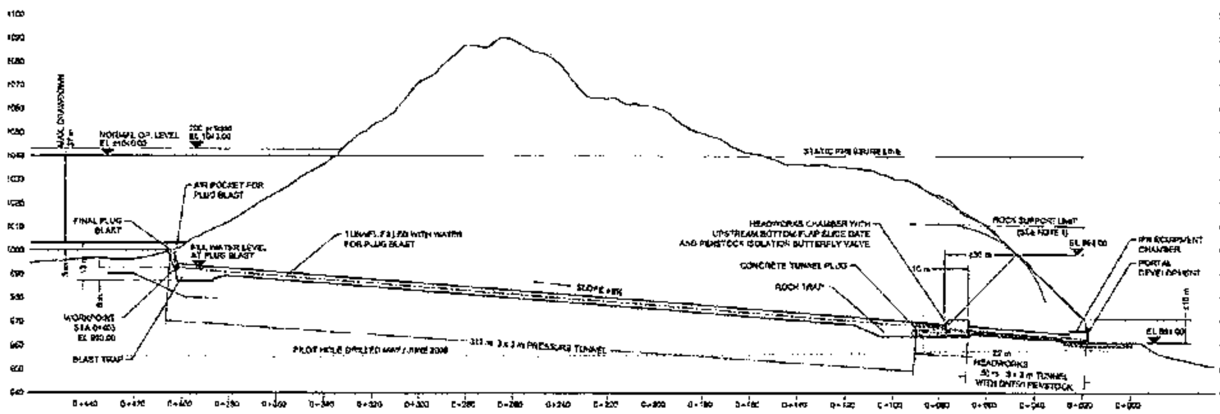
PLAN

SCALE: 1:1000

NOTES:

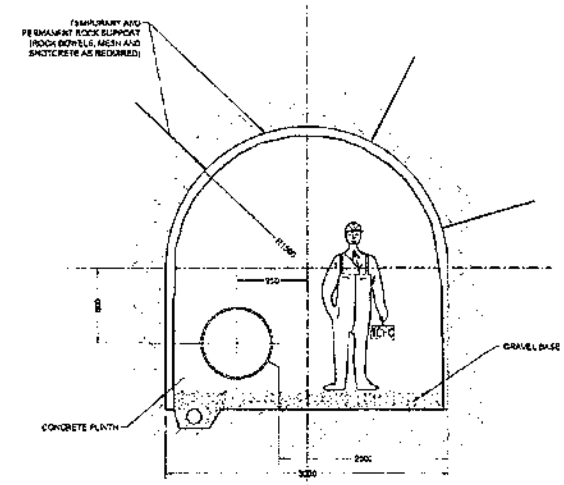
1. LOCATION OF TUNNEL PLUG DETERMINED USING THE THREE-DIMENSIONAL NORWEGIAN CRITERION FOR ROCK SUPPORT AGAINST HYDROJACKING. ASSUMED ROCK UNIT WEIGHT: 26.5 kN/m³
2. FOR FINAL PLUG BLAST (DETAILS TO BE FINALIZED ONCE EXCAVATION HAS ADVANCED TO THE LAKE TAP ZONE AND PROBE BORINGS ARE COMPLETE):
   - 2.1 CONCRETE TUNNEL PLUG MUST BE COMPLETE AND ALL MANWAYS AND PENSTOCK INLET SEALED.
   - 2.2 FILL TUNNEL WITH WATER MAINTAINING HEAD LESS THAN LAKE WATER LEVEL.
   - 2.3 MAINTAIN PRESSURIZED AIR POCKET BELOW ROCK PLUG.
   - 2.4 FIRE PLUG.

1:1000 0 62.5



PROFILE

SCALE: 1:1000



(A) SECTION - DOWNSTREAM OF CHAMBER

SCALE: 1:25

Issued for	Issue	Date	Issued By	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference	R0	20MAY08	AG	0	18JUL08	A. GYGAX	AG	D. WYLLIE	
Approvals	A2	18JUL08	AG						
Tender	T1	18JUL08	AG						
Permits									
Construction									
Record Drawings									

JUL 18 2008

KERR WOOD LEIDAL CONSULTING ENGINEERS

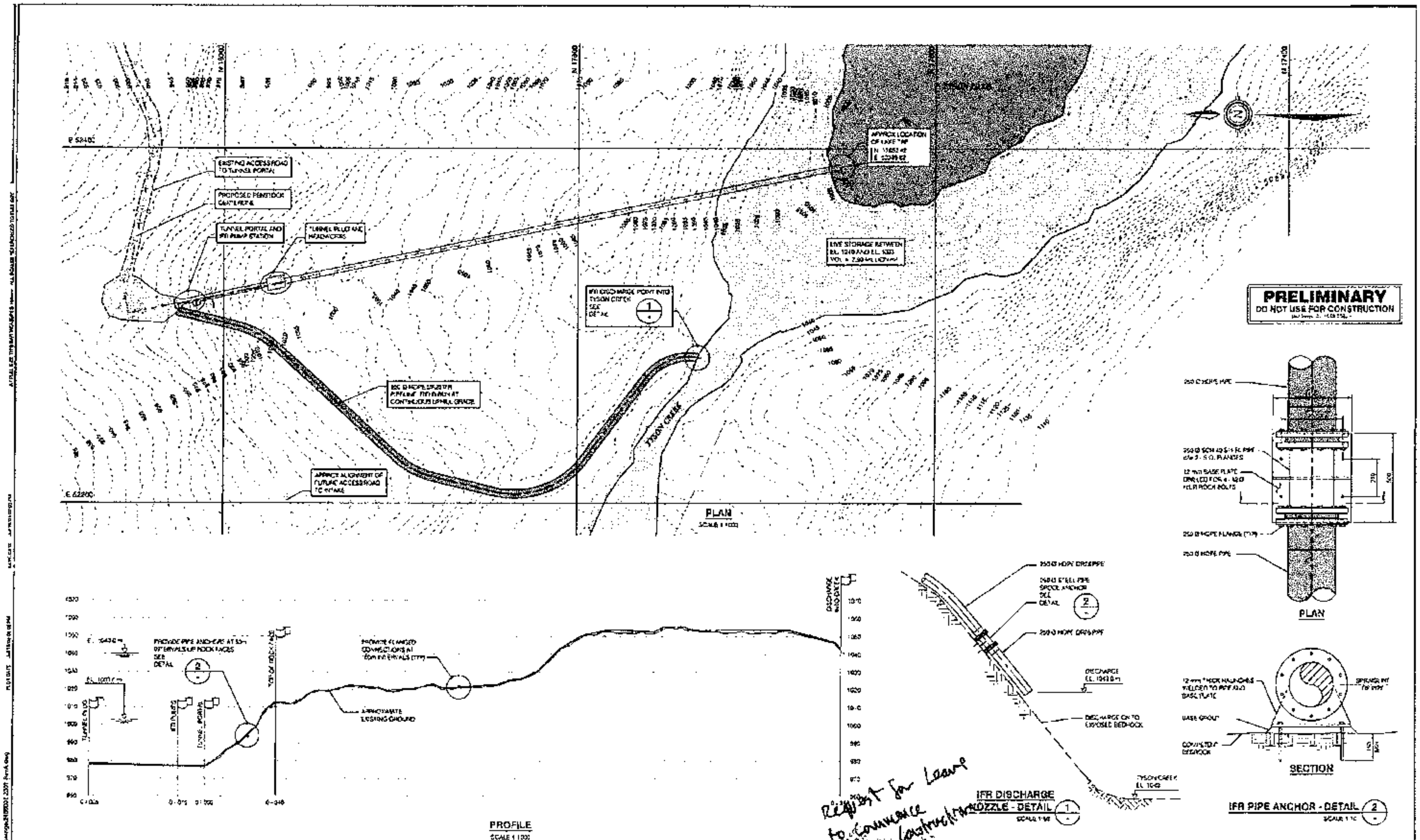
Gygax Engineering Associates Ltd.

wyllie & norrish ROCK ENGINEERS

TYSON CREEK HYDROELECTRIC PROJECT
GENERAL ARRANGEMENTS
TUNNEL
PLAN, PROFILE AND TYPICAL SECTION

Scale	Project No.	Discipline	Drawing Number	Rev.
1:1000	2468-002	STRUCT	210	0

Client: Tyson Creek Hydro Corp.



Request for Leave to Commence Construction

A. BRONSTEIN
ENGINEER
18 July 08

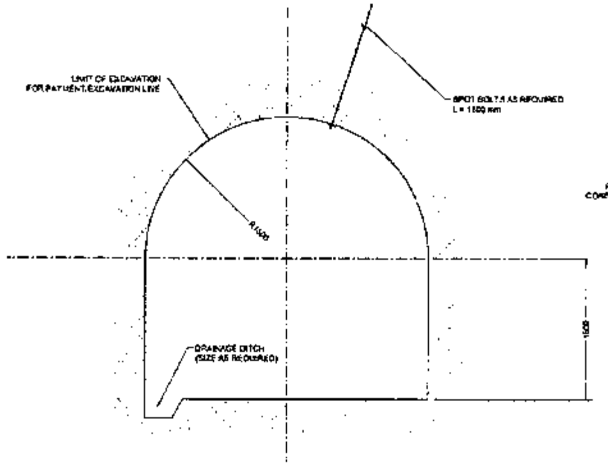
KERR WOOD LEIDAL CONSULTING ENGINEERS

GEA Gygax Engineering Associates Ltd.

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
OVERALL PLAN AND GENERAL ARRANGEMENT

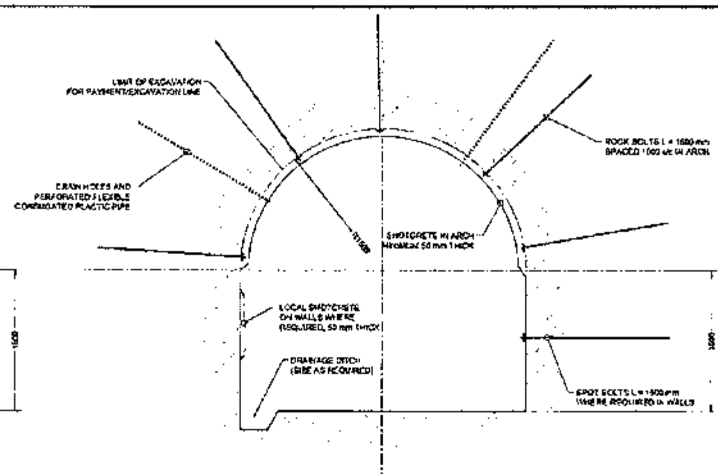
Scale	Project No.	Discipline	Drawing Number	Rev
SHOWN	2469-002	CIVIL	2301	A

Client: Tyson Creek Hydro Corp.



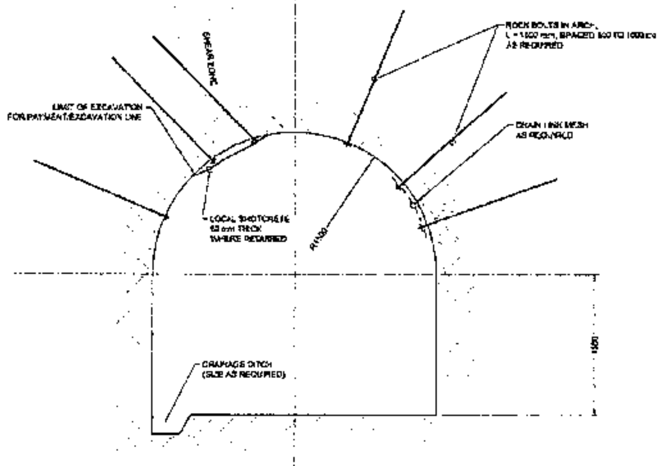
SUPPORT CLASS I - SECTION

SCALE 1:25
STRONG, JOINTED ROCK. SPOT BOLTING AS REQUIRED.



SUPPORT CLASS III - SECTION

SCALE 1:25
EXTENSIVELY SHEARED, CLOSELY JOINTED, ROCK.
PATTERN ROCK BOLTS IN ARCH AND WALLS, SHOTCRETE ON FULL ARCH AND ON WALLS AS REQUIRED.



SUPPORT CLASS II - SECTION

SCALE 1:25
STRONG ROCK, WITH LOCALIZED SHEARS AND CLOSELY JOINTED SEAMS. ROCK BOLTS IN ARCH, MESH AND SHOTCRETE AS REQUIRED.

NOTES:

1. GENERAL
1.1 REFER TO GEOLOGY REPORT
1.2 REFER TO DRAWINGS 2521 AND 2541 FOR PERMANENT WORKS REQUIREMENTS
2. PRODUCTS
2.1 ROCK BOLTS
 2.1.1 GALVANIZED THREADBAR TO CSA G30.18 GRADE 400W, DIAMETER 22M, L = 1500 mm
 2.1.2 STEEL NUTS AND PLATES COMPATIBLE WITH ROCK BOLTS
2.2 WIRE MESH: GALVANIZED WIRE MESH REINFORCEMENT TO CSA G30.3, PRE-BENT TO 1.5 m DIAMETER
2.3 SHOTCRETE: STEEL FIBRE REINFORCED FINE AGGREGATE SHOTCRETE TO CSA A23.1-04
3. EXECUTION
3.1 SUPPORT TYPES SHOWN ASSUME DRILL AND BLAST EXCAVATION.
3.2 SUPPORT REQUIRED AT ANY STRETCH OF THE TUNNEL IS AS DIRECTED BY THE ENGINEER. CLASSES SHOWN ARE FOR GENERAL REFERENCE ONLY.
3.3 LAKE TAP ZONE (APPROX. STA. 0+100 TO 0+000):
 3.3.1 ANTICIPATE CONSOLIDATION GROUT NO WORKS TO INCREASE AS LAKE IS APPROACHED.
 3.3.2 TEMPORARY ROCK SUPPORT, DRAINAGE AND GROUTING REQUIREMENTS MAY CHANGE WITH EACH ADVANCE.
 3.3.3 TAP ALIGNMENT, PROFILE, THICKNESS OF FINAL ROCK PLUG AND WATER CONTROL MEASURES WILL BE ADJUSTED TO SUIT THE ROCK CONDITIONS ENCOUNTERED.

1:25 0 2.0 m

Issued for	Issue	Date	Issued By
Reference	R2	[illegible]	AG
Approvals	A1	[illegible]	AG
Tender	T1	[illegible]	AG
Permits			
Construction			
Record Drawings			

REVISION INFORMATION

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
0	[illegible]	D. WYLLIE	GN	A. GYGAX	

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision

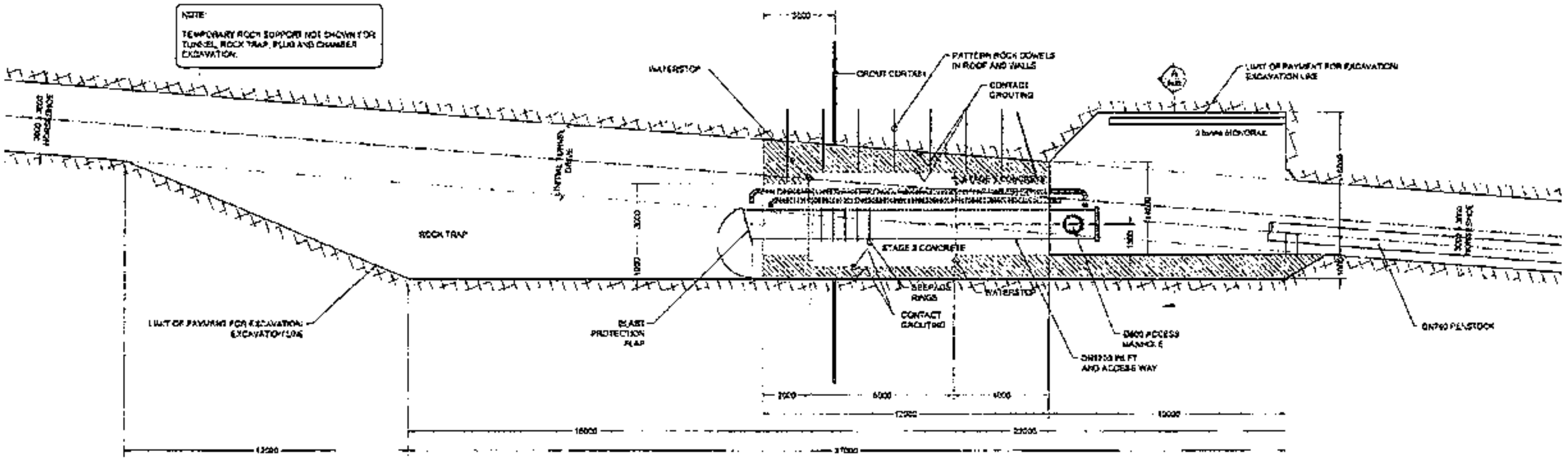
PROFESSIONAL ENGINEER K.A. GYGAX
JUL 1 5 2008

KERR WOOD LEIDAL associates limited CONSULTING ENGINEERS
Gygax Engineering Associates Ltd.
wyllie & norrish ROCK ENGINEERS

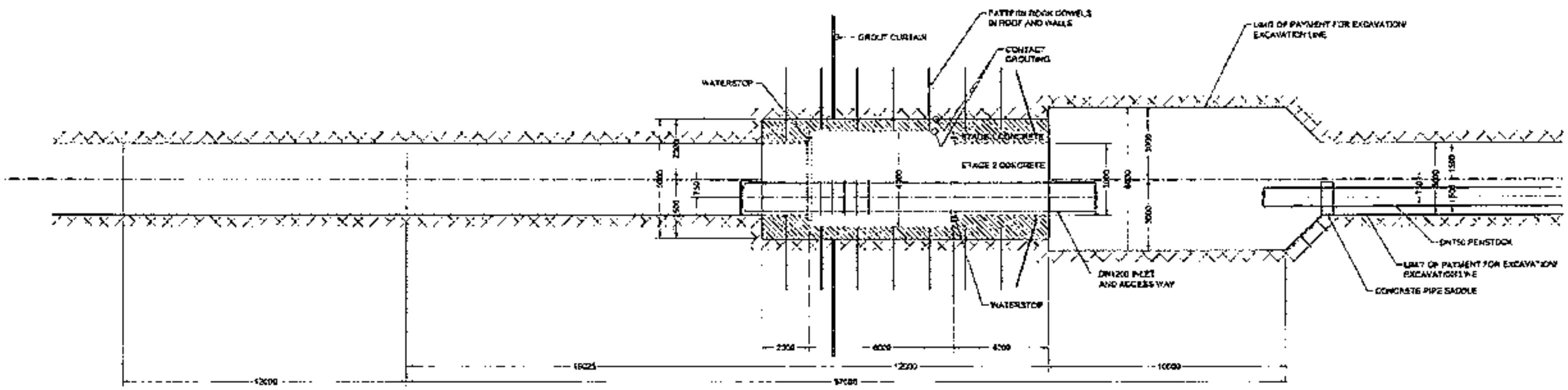
TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION AND SUPPORT
SECTIONS

Scale	Project No.	Discipline	Drawing Number	Rev.
1:25	2468-002	STRUCT	2411	0

Client: Tyson Creek Hydro Corp.



LONGITUDINAL PROFILE - TUNNEL PLUG AND CHAMBER
SCALE: 1:100



PLAN
SCALE: 1:100

Request for leave to commence construction

Issued for	Initial	Date		Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference				C		A. GYGAX	AG		UPDATED
Approvals									
Tender									
Permits									
Construction									
Record Drawings									

JUL 1 6 2008

PRELIMINARY
NOT FOR CONSTRUCTION

KERR WOOD LEIDAL
CONSULTING ENGINEERS

Gygax Engineering Associates Ltd.

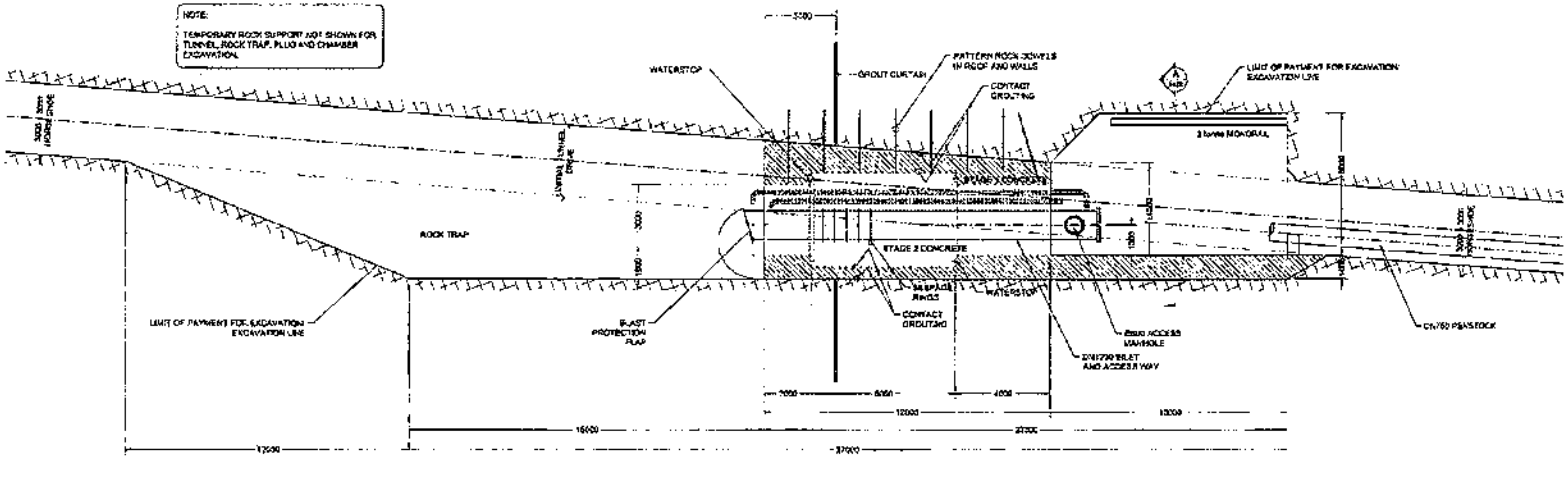
wyllie & norrish
ROCK ENGINEERS

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
CONCRETE PLUG AND HEADWORKS CHAMBER
STRUCTURE OUTLINE - PROFILE, PLAN

Scale	Project No.	Discipline	Drawing Number	Rev
1:100	2468-002	STRUCT	2431	C

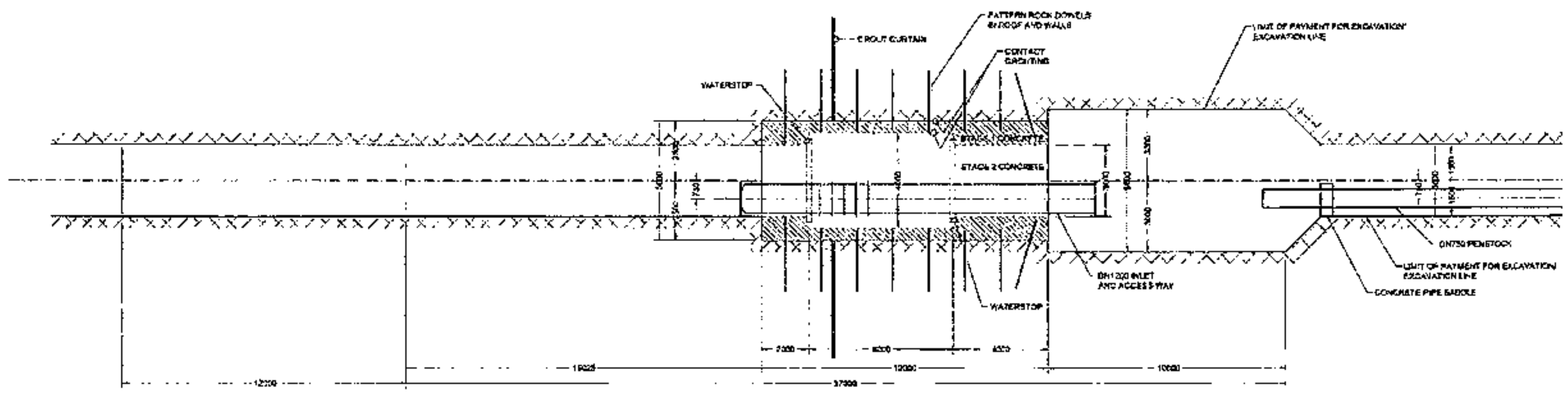
Client: Tyson Creek Hydro Corp.

NOTE:
TEMPORARY ROCK SUPPORT NOT SHOWN FOR TUNNEL, ROCK TRAP, PLUG AND CHAMBER EXCAVATION.



LONGITUDINAL PROFILE - TUNNEL PLUG AND CHAMBER

SCALE: 1:100



PLAN

SCALE: 1:100

Request for
leave to commence
construction

PRELIMINARY
NOT FOR CONSTRUCTION

KERR WOOD LEIDAL
CONSULTING ENGINEERS

Gygax Engineering Associates Ltd.

wyllie & norrish

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
CONCRETE PLUG AND HEADWORKS CHAMBER
STRUCTURE OUTLINE - PROFILE, PLAN

Scale	Project No.	Discipline	Drawing Number	Rev.
1:100	2468-002	STRUCT	2431	C

Client: Tyson Creek Hydro Corp.

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
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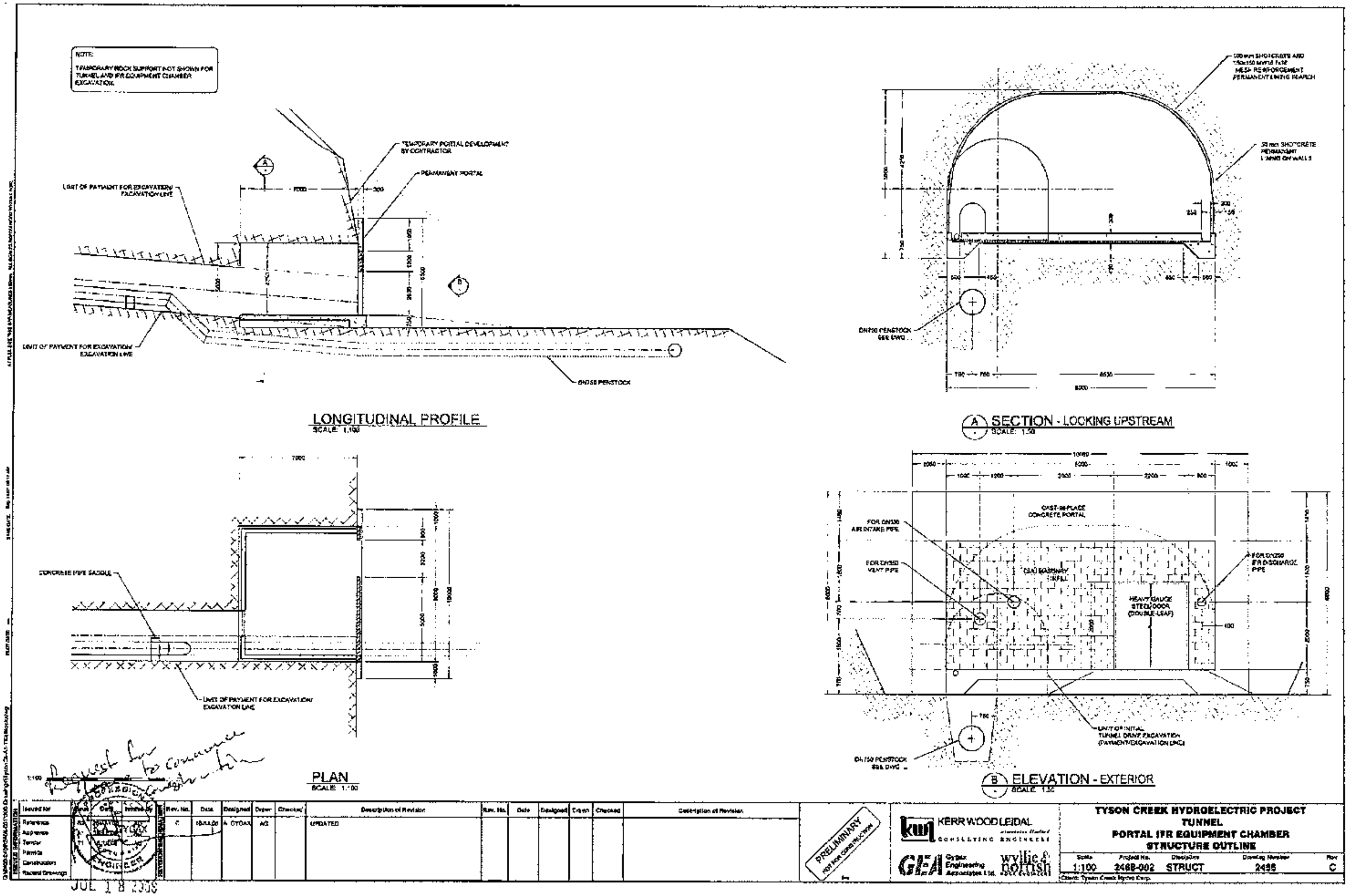

NOTE:
TEMPORARY ROCK SUPPORT NOT SHOWN FOR TUNNEL AND IFR EQUIPMENT CHAMBER EXCAVATION.
TEMPORARY PORTAL DEVELOPMENT BY CONTRACTOR
PERMANENT PORTAL
LIMIT OF PAYMENT FOR EXCAVATION/ EXCAVATION LINE
DN750 PENSTOCK
LONGITUDINAL PROFILE
SCALE: 1:100
A SECTION - LOOKING UPSTREAM
SCALE: 1:50
CONCRETE PIPE SADDLE
PLAN
SCALE: 1:100
CAST-IN-PLACE CONCRETE PORTAL
CMU MASONRY INFILL
HEAVY GAUGE STEEL DOOR (DOUBLE-LEAF)
B ELEVATION - EXTERIOR
SCALE: 1:50
Issued for
Reference
Approval
Tender
Permits
Construction
Record Drawings
Rev. No.
Date
Designed
Drawn
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Description of Revision
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KERR WOOD LEIDAL
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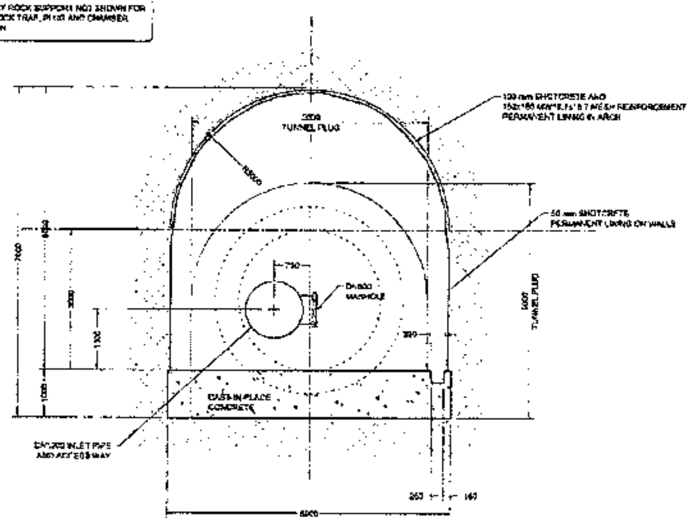
wyllie & norrish
TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
PORTAL IFR EQUIPMENT CHAMBER
STRUCTURE OUTLINE
Scale
1:100
Project No.
2468-002
Discipline
STRUCT
Drawing Number
2495
Rev
C
Client: Tyson Creek Hydro Corp.
JUL 18 2008

NOTE

TEMPORARY ROCK SUPPORT NOT SHOWN FOR TUNNEL, ROCK TRAP, PLUG AND CHAMBER EXCAVATION



A SECTION - LOOKING UPSTREAM
2431 SCALE: 1:50

Request for leave to commence construction

Issued for	Initials	Date	Issued By	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference	[illegible]	[illegible]	AG	C	[illegible]	A. GYGAX	AG		UPDATED						
Approval															
Tender															
Permits															
Construction															
Record Drawings															

JUL 1 0 2008

PRELIMINARY
NOT FOR CONSTRUCTION

KERR WOOD LEIDAL
CONSULTING ENGINEERS

Gygax Engineering Associates Ltd.

wyllie & norrish

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
CONCRETE PLUG AND HEADWORKS CHAMBER
STRUCTURE OUTLINE - SECTIONS, DETAILS

Scale	Project No.	Discipline	Drawing Number	Rev.
1:50	2468-002	STRUCT	2432	C

Client: Tyson Creek Hydro Corp.

TYSON CREEK HYDRO PROJECT

RECORD DRAWINGS

3 March 2010

GENERAL ARRANGEMENTS

DWG No.	ISSUED	DISC	TITLE
000 - 3	20-Apr-09	GEN	DRAWING LIST
001 - 0	21-Jul-08	GEN	LOCATION PLAN
002 - 0	21-Jul-08	GEN	KEY PLAN
004 - 1	10-May-08	GEN	LAND TENURE AND CLEARING BOUNDARY
007 - 0	16-Sep-08	GEN	SURVEY CONTROL
010 - 0	12-Feb-09	GEN	PIPING AND INSTRUMENTATION - STANDARDS SHEET 1 OF 2
011 - 0	12-Feb-09	GEN	PIPING AND INSTRUMENTATION - STANDARDS SHEET 2 OF 2
012 - 0	01-Apr-09	GEN	PIPING AND INSTRUMENTATION - HYDRAULIC PROFILE
015 - 1	16-Apr-09	GEN	PIPING AND INSTRUMENTATION - IFR PUMPING SYSTEM
016 - 0	03-Apr-09	GEN	PIPING AND INSTRUMENTATION - POWERHOUSE
200 - 2	27-Nov-08	GEN	TUNNEL - PLAN, PROFILE, AND TYPICAL SECTION
500 - 0	05-Dec-08	GEN	POWERHOUSE - STRUCTURAL NOTES
501 - 1	29-Sep-08	GEN	POWERHOUSE - OVERALL SITE PLAN
505 - 0	05-Sep-08	GEN	POWERHOUSE - ELEVATIONS
511 - 0	05-Sep-08	GEN	POWERHOUSE - PLANS
521 - 0	05-Sep-08	GEN	POWERHOUSE - LONGITUDINAL SECTION
522 - 0	05-Sep-08	GEN	POWERHOUSE - TRANSVERSE SECTION

TUNNEL

DWG No.	ISSUED	DISC	TITLE
2002 - 0	12-Feb-09	CIVIL	INSTREAM FLOW RELEASE - OVERALL PLAN PROFILE
2401 - 5	03-Nov-08	STRUC	EXCAVATION - PLAN AND PROFILE
2411 - 1	22-Aug-08	STRUC	EXCAVATION AND SUPPORT - SECTIONS
2415 - 1	21-Nov-08	STRUC	EXCAVATION - LAKE TAP - ACTUAL ADVANCE
2416 - 1	14-Nov-08	STRUC	EXCAVATION - PLUG AND CHAMBERS - PROFILE AND PLAN
2417 - 0	06-Nov-08	STRUC	EXCAVATION - PLUG AND CHAMBERS - SECTIONS
2418 - 0		STRUC	EXCAVATION - LAKE TAP FINAL BLAST
2431 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE - FIRST STAGE
2432 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE - SECOND STAGE
2433 - 0	04-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE AND REINF. - PLINTHS
2435 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE - CONSOLIDATION GROUTING
2436 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE - CONTACT GROUTING
2437 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - STRUCTURE OUTLINE - HYDRAULIC GROUTING
2441 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - REINFORCEMENT - FIRST STAGE
2442 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - REINFORCEMENT - FIRST STAGE
2443 - 0	03-Mar-09	STRUC	CONCRETE PLUG AND HEADWORKS CHAMBER - REINFORCEMENT - SECOND STAGE
2451 - 0	04-Mar-09	STRUC	DOWNSTREAM SECTION AND PORTAL - STRUCTURE OUTLINE AND REINF. - PLINTHS
2500 - 2	16-Apr-09	MECH	CONCEPTUAL PLAN
2501 - 3	20-Apr-09	MECH	HEADWORKS PLAN
2502 - 3	20-Apr-09	MECH	HEADWORKS SECTIONS SHEET 1 OF 2
2503 - 3	20-Apr-09	MECH	HEADWORKS SECTIONS SHEET 2 OF 2
2504 - 1	20-Apr-09	MECH	EMBEDDED PIPING FOR TUNNEL PLUG
2505 - 1	16-Apr-09	MECH	PORTAL PLAN
2506 - 1	16-Apr-09	MECH	PORTAL SECTIONS
2507 - 0	03-Apr-09	MECH	DEBRIS MANAGEMENT DETAILS
2810 - 2	16-Apr-09	MECH	IFR PUMP STATION PLAN
2811 - 2	16-Apr-09	MECH	IFR PUMP STATION SECTIONS
2812 - 2	16-Apr-09	MECH	IFR PUMP STATION SECTION AND TUNNEL SECTIONS
2813 - 3	16-Apr-09	MECH	TUNNEL MECHANICAL DETAILS - PIPE DETAILS AND SUPPORTS
2816 - 2	16-Apr-09	MECH	TUNNEL MECHANICAL DETAILS - VENTILATION DETAILS
2817 - 2	20-Apr-09	MECH	TUNNEL MECHANICAL DETAILS - PENSTOCK DETAILS

PENSTOCK

DWG No.	ISSUED	DISC	TITLE
4300 - 1	13-Mar-09	CIVIL	KEY PLAN AND NOTES
4302 - 3	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 1 OF 13
4303 - 1	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 2 OF 13
4304 - 1	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 3 OF 13
4305 - 3	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 4 OF 13
4306 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 5 OF 13
4307 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 6 OF 13
4308 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 7 OF 13
4309 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 8 OF 13
4310 - 3	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 9 OF 13
4311 - 1	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 10 OF 13
4312 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 11 OF 13
4313 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 12 OF 13
4314 - 2	[illegible]	CIVIL	DETAILED PLAN AND PROFILE - SHEET 13 OF 13
4322 - 0	[illegible]	CIVIL	CROSS SECTIONS - SHEET 1 OF 23
4323 - 1	[illegible]	CIVIL	CROSS SECTIONS - SHEET 2 OF 23
4324 - 1	[illegible]	CIVIL	CROSS SECTIONS - SHEET 3 OF 23
4325 - 1	[illegible]	CIVIL	CROSS SECTIONS - SHEET 4 OF 23
4326 - 1	[illegible]	CIVIL	CROSS SECTIONS - SHEET 5 OF 23
4327 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 6 OF 23
4328 - 3	[illegible]	CIVIL	CROSS SECTIONS - SHEET 7 OF 23
4329 - 3	[illegible]	CIVIL	CROSS SECTIONS - SHEET 8 OF 23
4330 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 9 OF 23
4331 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 10 OF 23
4332 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 11 OF 23
4333 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 12 OF 23
4334 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 13 OF 23
4335 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 14 OF 23
4336 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 15 OF 23
4337 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 16 OF 23
4338 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 17 OF 23
4339 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 18 OF 23
4340 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 19 OF 23
4341 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 20 OF 23
4342 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 21 OF 23
4343 - 3	[illegible]	CIVIL	CROSS SECTIONS - SHEET 22 OF 23
4344 - 2	[illegible]	CIVIL	CROSS SECTIONS - SHEET 23 OF 23
4345 - 2	[illegible]	CIVIL	CROSS SECTIONS - TYPICAL DETAILS
4350 - 0	[illegible]	STRUC	LOW PRESSURE SECTION - BEND BACKFILL DETAILS
4390 - 0	[illegible]	CIVIL	TYPICAL CREEK CROSSINGS
4400 - 1	[illegible]	CIVIL	TRENCH DAMS - TYPICAL DETAILS
4406 - 1	[illegible]	CIVIL	CROSS CULVERTS - STATION 2+920
4407 - 0	[illegible]	CIVIL	CREEK CROSSINGS - CROSSING AT STA 3+917
4421 - 2	[illegible]	STRUC	GROUND ANCHORS - GENERAL REQUIREMENTS
4431 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINES - AB 1 - PLAN AND SECTIONS
4432 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINES - AB 2 - PLAN AND SECTIONS
4433 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINES - AB 3 - PLAN AND SECTIONS
4434 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINES - AB 4 - PLAN AND SECTIONS
4435 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINES - AB 5 - PLAN AND SECTIONS
4437 - 0	[illegible]	STRUC	EXPOSED ANCHOR BLOCK STRUCTURE OUTLINE - DETAILS
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 10
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 11
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 12
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 13
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 14
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 15
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK STRUCTURE OUTLINE - AB 16
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 1 - PLANS AND ELEVATION
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 1 - ELEVATION AND SECTIONS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 2 - PLAN AND SECTIONS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 3 - PLAN AND SECTIONS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 4 - PLAN AND SECTIONS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - AB 5 - PLAN AND SECTIONS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - DETAILS
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK REINFORCEMENT - DETAILS
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 10
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 11
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 12
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 13
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 14
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 15
[illegible]	[illegible]	STRUC	BURIED ANCHOR BLOCK REINFORCEMENT - AB 16
[illegible]	[illegible]	STRUC	EXPOSED INTERMEDIATE SUPPORTS ON ROCK - TYPES 1 AND 2 STRUCTURE OUTLINE
[illegible]	[illegible]	STRUC	EXPOSED INTERMEDIATE SUPPORTS ON SOIL - TYPES 3 AND 4 - STRUCTURE OUTLINE
[illegible]	[illegible]	STRUC	EXPOSED INTERMEDIATE SUPPORTS - TYPE 1 AND TYPE 3 - REINFORCEMENT
[illegible]	[illegible]	STRUC	EXPOSED INTERMEDIATE SUPPORTS - TYPE 4 - REINFORCEMENT
[illegible]	[illegible]	STRUC	EXPOSED INTERMEDIATE SUPPORTS - TYPE 4 - STEEL CRADLE DETAILS
[illegible]	[illegible]	STRUC	LOW PRESSURE SECTION BURIED PENSTOCK MITRE BENDS - SHEET 1 OF 2
[illegible]	[illegible]	STRUC	LOW PRESSURE SECTION BURIED PENSTOCK MITRE BENDS - SHEET 2 OF 2
[illegible]	[illegible]	STRUC	EXPOSED ANCHOR BLOCK MITRE BENDS - SHEET 1 OF 1
[illegible]	[illegible]	STRUC	MITRED BENDS - SHEET 1 OF 5
[illegible]	[illegible]	STRUC	MITRED BENDS - SHEET 2 OF 5
[illegible]	[illegible]	STRUC	MITRED BENDS - SHEET 3 OF 5
[illegible]	[illegible]	STRUC	MITRED BENDS - SHEET 5 OF 5
[illegible]	[illegible]	STRUC	MANHOLE TYPE 1 - DETAILS
[illegible]	[illegible]	STRUC	MANHOLE TYPE 2 - DETAILS
[illegible]	[illegible]	STRUC	POWERHOUSE REDUCER - DETAILS

POWERHOUSE

DWG No.	ISSUED	DISC	TITLE
5301 - 1	17-Sep-08	CIVIL	EXCAVATION - PLAN
5302 - 1	17-Sep-08	CIVIL	EXCAVATION - SECTIONS
5303 - 0	21-Jul-08	CIVIL	BOREHOLE LOGS
5304 - 3	05-Mar-09	CIVIL	DETAILED SITE PLAN
5305 - 1	17-Sep-08	CIVIL	FINISHED GRADE SECTIONS
5306 - 0	21-Jul-08	CIVIL	SITEWORK DETAILS - SHEET 1 OF 2
5307 - 0	21-Jul-08	CIVIL	SITEWORK DETAILS - SHEET 2 OF 2
5310 - 1	17-Sep-08	CIVIL	TAILRACE - PLAN
5311 - 0	17-Sep-08	CIVIL	TAILRACE - PROFILE AND SECTIONS
5401 - 1	24-Oct-08	STRUC	CONCRETE OUTLINE - SUBSTRUCTURE TAILRACE PLAN
5402 - 2	19-Dec-08	STRUC	CONCRETE OUTLINE - SUBSTRUCTURE FOOTINGS PLAN
5403 - 0	05-Sep-08	STRUC	CONCRETE OUTLINE - FLOOR PLAN
5405 - 1	24-Oct-08	STRUC	CONCRETE OUTLINE - EMBEDMENTS
5411 - 1	24-Oct-08	STRUC	CONCRETE OUTLINE - TAILRACE SECTIONS
5412 - 1	19-Dec-08	STRUC	CONCRETE OUTLINE - SECTIONS
5413 - 0	05-Sep-08	STRUC	CONCRETE OUTLINE - SECTIONS
5414 - 0	05-Sep-08	STRUC	CONCRETE OUTLINE - SECTIONS
5415 - 0	05-Sep-08	STRUC	CONCRETE OUTLINE - SECTIONS
5421 - 1	19-Dec-08	STRUC	CONCRETE OUTLINE - 2ND STAGE - FLOOR PLAN
5425 - 0	31-Oct-08	STRUC	CONCRETE OUTLINE - 2ND STAGE - EMBEDMENTS
5431 - 1	24-Oct-08	STRUC	REINFORCEMENT - SUBSTRUCTURE TAILRACE PLAN
5432 - 0	05-Sep-08	STRUC	REINFORCEMENT - SUBSTRUCTURE FOOTINGS PLAN
5433 - 1	31-Oct-08	STRUC	REINFORCEMENT - FLOOR PLAN
5441 - 0	05-Sep-08	STRUC	REINFORCEMENT - TAILRACE SECTIONS
5442 - 0	05-Sep-08	STRUC	REINFORCEMENT - TAILRACE SECTIONS
5443 - 1	31-Oct-08	STRUC	REINFORCEMENT - TAILRACE SECTIONS
5444 - 0	05-Sep-08	STRUC	REINFORCEMENT - LONGITUDINAL SECTIONS
5445 - 1	19-Dec-08	STRUC	REINFORCEMENT - LONGITUDINAL SECTIONS
5446 - 0	05-Sep-08	STRUC	REINFORCEMENT - SECTIONS
5447 - 0	05-Sep-08	STRUC	REINFORCEMENT - SECTIONS
5448 - 0	05-Sep-08	STRUC	REINFORCEMENT - TRANSVERSE SECTIONS
5449 - 0	05-Sep-08	STRUC	REINFORCEMENT - TRANSVERSE SECTIONS
5450 - 0	05-Sep-08	STRUC	REINFORCEMENT - SECTIONS
5471 - 1	19-Dec-08	STRUC	REINFORCEMENT 2ND STAGE - FLOOR PLAN
5474 - 0	31-Oct-08	STRUC	REINFORCEMENT 2ND STAGE - SECTIONS
5475 - 1	19-Dec-08	STRUC	REINFORCEMENT 2ND STAGE - SECTIONS
5481 - 0	31-Oct-08	STRUC	CONCRETE OUTLINE - REINFORCEMENT - GENSET PAD - PLANS AND SECTIONS
5501 - 0	05-Sep-08	STRUC	STRUCTURAL STEELWORK - ROOF FRAMING
5511 - 0	05-Sep-08	STRUC	STRUCTURAL STEELWORK - FRAMING SECTIONS
5512 - 0	05-Sep-08	STRUC	STRUCTURAL STEELWORK - FRAMING SECTIONS
5513 - 0	05-Sep-08	STRUC	STRUCTURAL STEELWORK - FRAMING SECTIONS
5521 - 1	24-Oct-08	STRUC	STRUCTURAL STEELWORK - CONNECTIONS DETAILS
5522 - 1	24-Oct-08	STRUC	STRUCTURAL STEELWORK - CONNECTIONS DETAILS
5531 - 0	05-Sep-08	STRUC	STRUCTURAL STEELWORK - CRANE RUNWAY
5532 - 0	24-Oct-08	STRUC	MISCELLANEOUS METALS - STAIR, PLATFORM, & GUARDRAILS - PLAN AND SECTIONS
5533 - 0	24-Oct-08	STRUC	MISCELLANEOUS METALS - STAIR, PLATFORM, & GUARDRAILS - DETAILS
5561 - 1	24-Oct-08	STRUC	STRUCTURAL TIMBER - PLANS, SECTIONS, AND DETAILS
5571 - 1	11-Dec-08	STRUC	SECONDARY STEEL WORK - PURLIN ARRANGEMENT PLAN
5572 - 2	11-Dec-08	STRUC	SECONDARY STEEL WORK - GIRT ARRANGEMENT PLAN
5573 - 2	11-Dec-08	STRUC	SECONDARY STEEL WORK - GIRT ARRANGEMENT SECTIONS
5575 - 1	11-Dec-08	STRUC	SECONDARY STEEL WORK - PURLIN AND GIRT SECTIONS AND DETAILS
5576 - 0	24-Oct-08	STRUC	SECONDARY STEEL WORK - CONTROL ROOM - PLANS, ELEVATIONS
5580 - 0	11-Dec-08	STRUC	ARCHITECTURAL - RENDERING
5581 - 0	11-Dec-08	STRUC	BUILDING ENVELOPE - ELEVATIONS
5582 - 0	11-Dec-08	STRUC	BUILDING ENVELOPE - SECTIONS AND FLASHING DETAILS
5601 - 0	03-Apr-09	MECH	WASHDOWN WATER SYSTEM - PLAN AND SECTIONS
5602 - 0	03-Apr-09	MECH	WASHDOWN WATER SYSTEM - DETAILS
5610 - 1	02-May-09	MECH	BUILDING DRAINAGE - PLAN AND PROFILE
5620 - 0	10-Sep-08	MECH	VENTILATION - PLAN AND ELEVATIONS
5621 - 0	10-Sep-08	MECH	VENTILATION - DETAILS

ISSUED INFORMATION

Issued for	Issue	Date	Issued By
Reference			
Approvals	A1	Apr.23/04	RDP
Tender			
Permits			
Construction	C0	APR.23/08	AG
Record Drawings			

REVISION INFORMATION

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
A	APR.23/04	RDP	RDP	RL	ISSUED FOR LEAVE TO COMMENCE CONSTRUCTION
0	Jul.21/08	RDP	RDP	RL	ISSUED FOR CONSTRUCTION
1	Aug.25/08	RDP	EV	RL	GENERAL REVISION, IFC
2	Sep.18/08	RDP	PC	RL	GENERAL REVISION, IFC
3	APR.20/09	RDP	JT	RL	GENERAL REVISION

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision

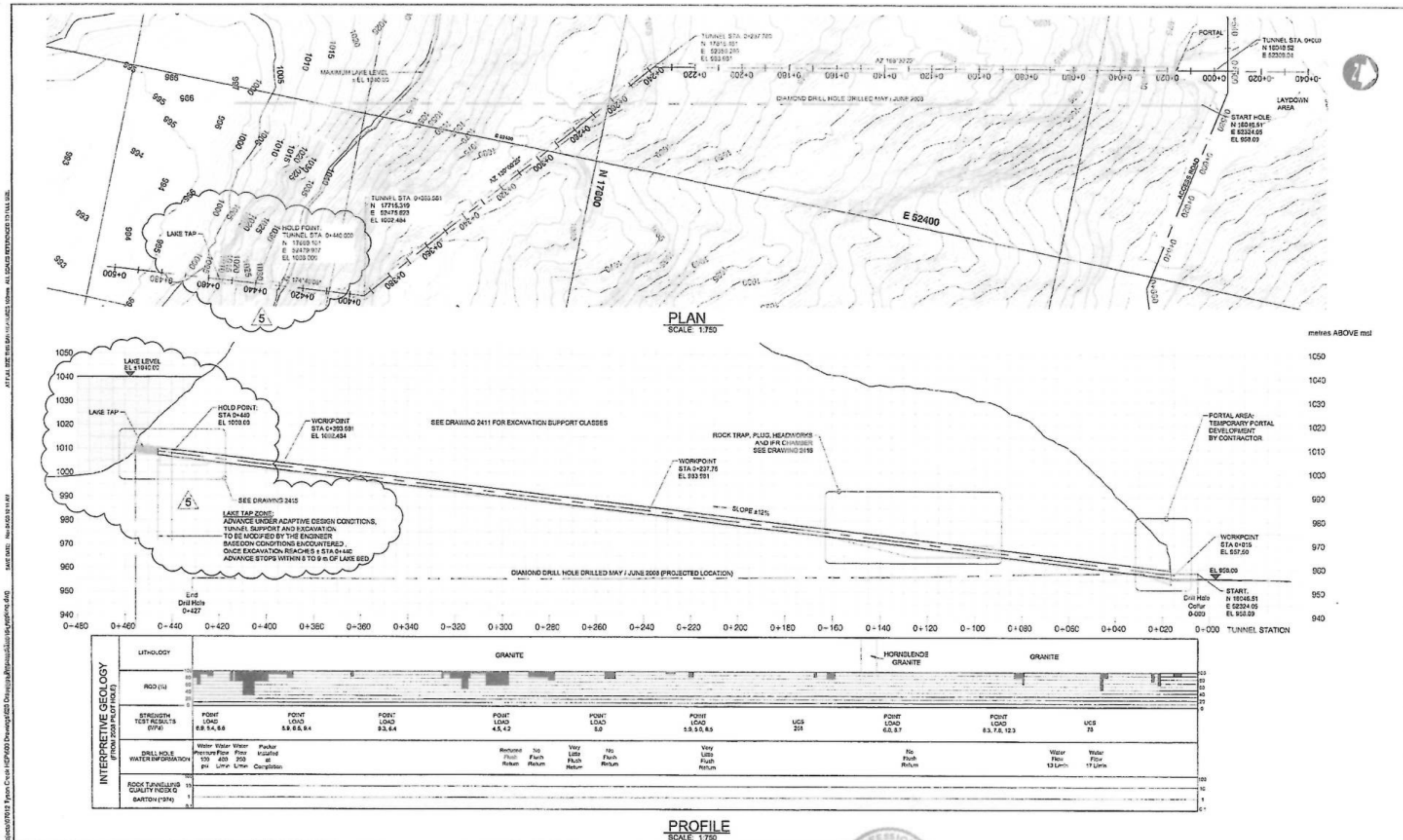
KERR WOOD LEIDAL CONSULTING ENGINEERS

Gygax Engineering Associates Ltd.

TYSON CREEK HYDROELECTRIC PROJECT
GENERAL ARRANGEMENTS
DRAWING LIST

Scale	Project No.	Discipline	Drawing Number	Rev
NONE	2466-002	GEN	000	3

Client: Tyson Creek Hydro Corp.



Issued for	Issue	Date	Issued By
Reference	R1	20JUN08	AG
Approvals	A1	18JUL08	GD
Tender	T1	18JUL08	GD
Permits			
Construction	C4	03NOV08	DW
Record Drawings			

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
0	18JUL08	G. DYER	AG	D. WYLLIE	
1	22AUG08	G. DYER	AG	D. WYLLIE	
2	10SEP08	G. DYER	AG	D. WYLLIE	BATHYMETRY & PROFILE INTERMEDIATE WORKPOINT
3	03OCT08	D. WYLLIE	AG	D. WYLLIE	1ST HORIZONTAL BEND AT STATION 0+240
4	23OCT08	D. WYLLIE	TL	A. GYGAX	GENERAL ALIGNMENT REVISION
5	03NOV08	D. WYLLIE	TL	A. GYGAX	LAKE TAP ZONE

Rev. No.	Date	Designed	Drawn	Checked	Description of Revision

KERR WOOD LEIDAL associates limited CONSULTING ENGINEERS

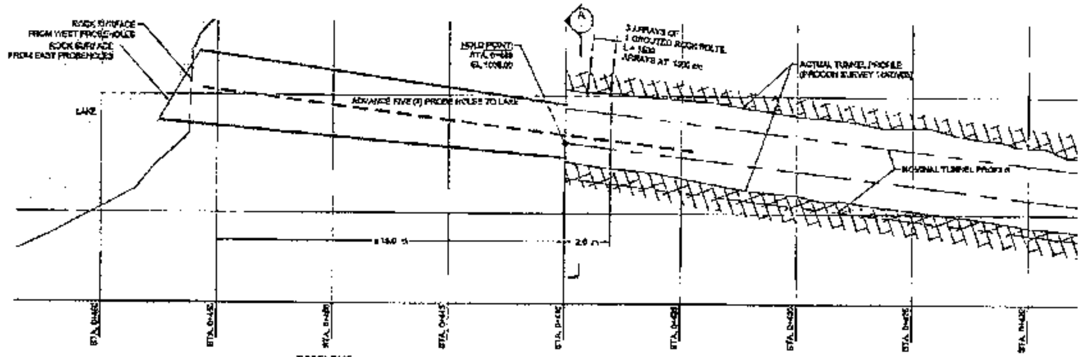
Gygax Engineering Associates Ltd.

wyllie norrish ROCK ENGINEERS

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION
PLAN AND PROFILE

Scale	Project No.	Discipline	Drawing Number	Rev.
1:750	2468-002	STRUCT	2401	5

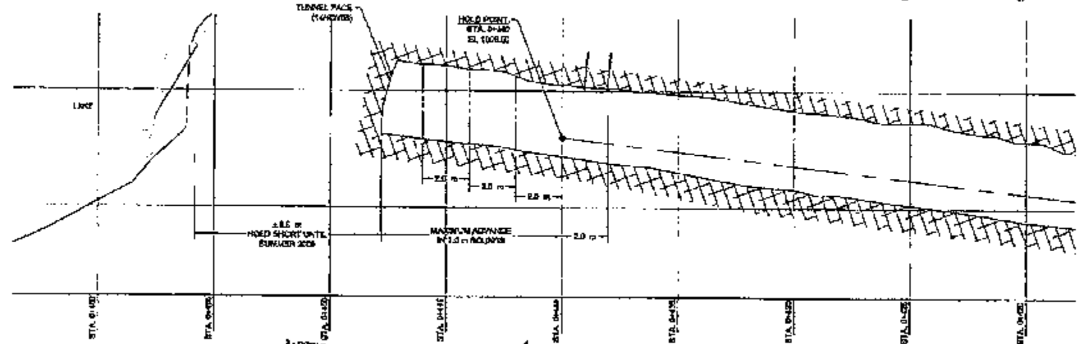
Client: Tyson Creek Hydro Corp.



PROFILE - STEP 1
SCALE: 1:100

Excavation Step 1:

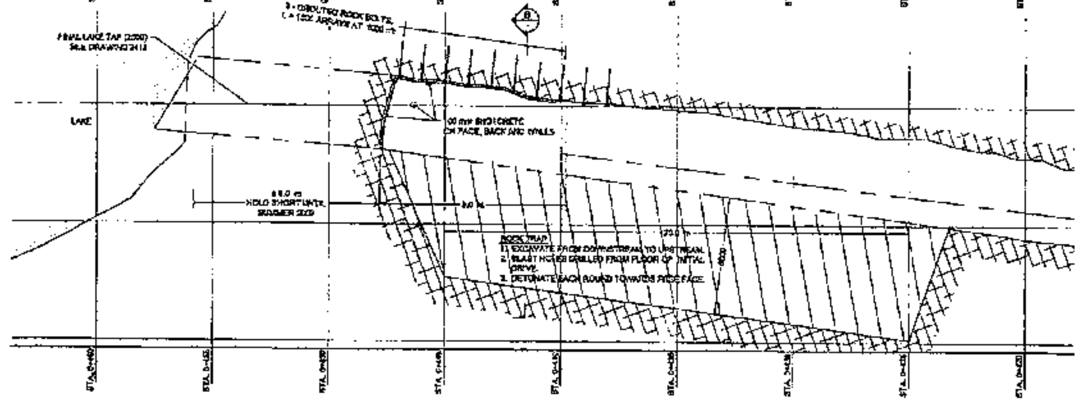
1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]



PROFILE - STEP 2
SCALE: 1:100

Excavation Step 2:

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
   - [illegible]
   - [illegible]



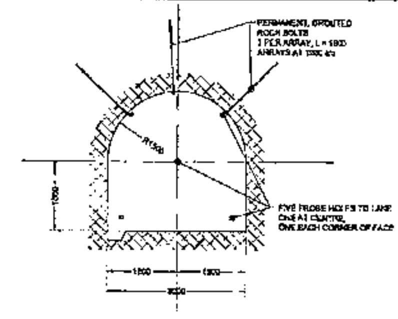
PROFILE - STEP 3
SCALE: 1:100

Excavation Step 3:

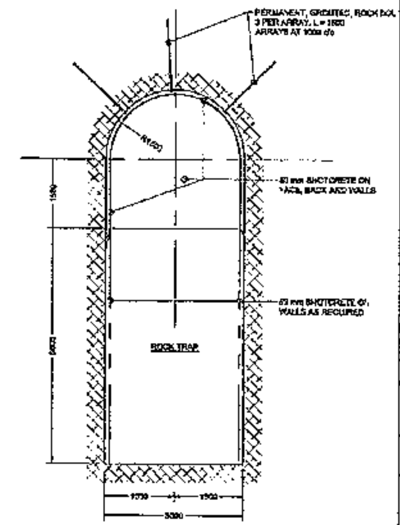
1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]

Required quantities:

- [illegible]
- [illegible]



A SECTION - STEP 1
SCALE: 1:50



B SECTION - STEP 3
SCALE: 1:50

Issued for	Issue	Date	Issued By	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference	R1	[illegible]	AG	0	[illegible]	[illegible]	AG	D. WYLLIE	ISSUED FOR CONSTRUCTION
Approvals				1	21NOV[illegible]	[illegible]	AG	D. WYLLIE	TO CONSTRUCTION SURVEYS & FINAL OBSERVATIONS
Tender									
Permits									
Construction	C2	[illegible]	AG						
Record Drawings									

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
D. C. WYLLIE

2022-11-28

KERR WOOD LEIDAL
CONSULTING ENGINEERS

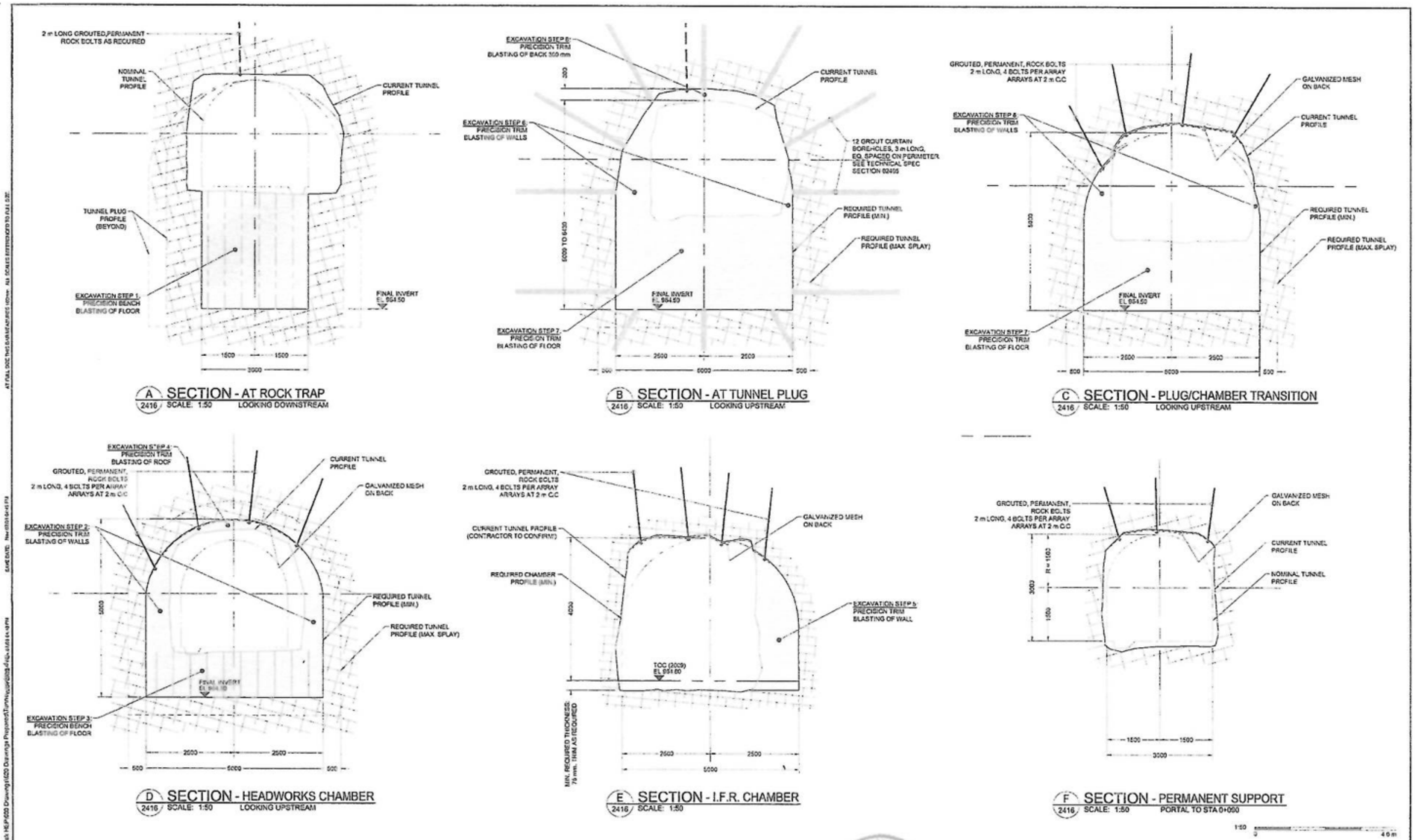
GEA Engineering Associates Ltd.

wyllie & norrish

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION
LAKE TAP - INITIAL ADVANCE

Scale	Project No.	Discipline	Drawing Number	Rev.
1:100, 1:50	2468-002	STRUCT	2415	1

Client: Tyson Creek Hydro Corp.



Issued for	Issue	Date	Issued By	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference				0	06NOV08	D. WYLLIE	AG	D. WYLLIE	ISSUED FOR CONSTRUCTION						
Approvals															
Tender															
Permits															
Construction	C1	07NOV08	D. WYLLIE												
Record Drawings															

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA D. C. WYLLIE 2008 11 07

KERR WOOD LEIDAL associates limited CONSULTING ENGINEERS

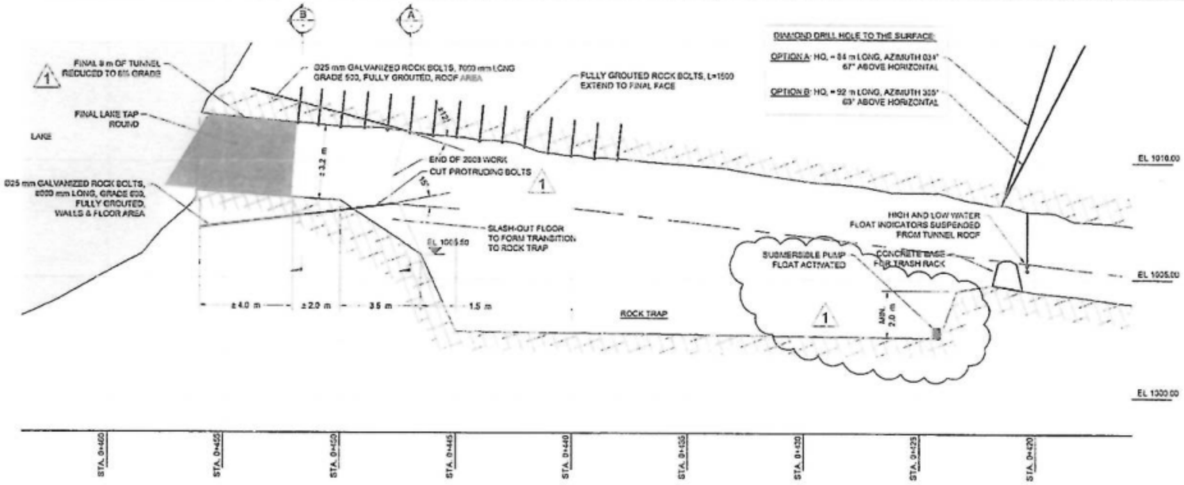
GEA Gygax Engineering Associates Ltd.

wyllie norrish rock engineers

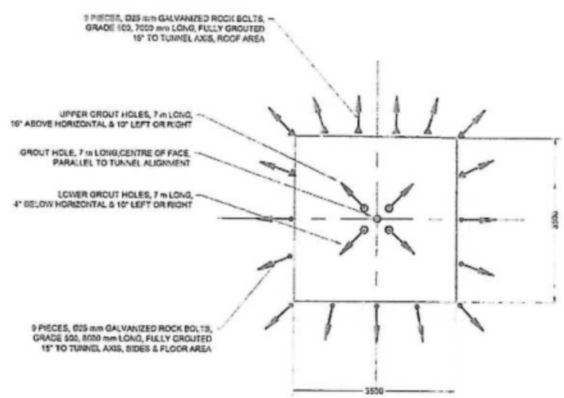
TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION
PLUG AND CHAMBERS - SECTIONS

Scale	Project No.	Discipline	Drawing Number	Rev.
1:50	2468-002	STRUCT	2417	0

Client: Tyson Creek Hydro Corp.



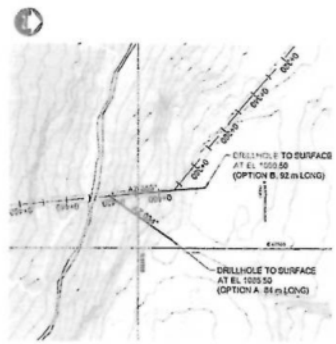
PROFILE - 2009 WORK
SCALE: 1:100



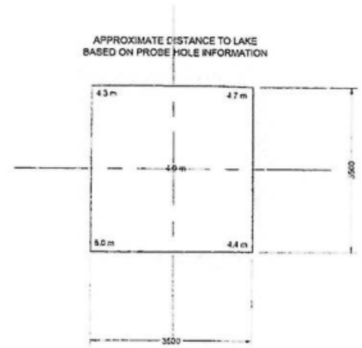
A SECTION - FACE 8 m FROM LAKE
SCALE: 1:50

2009 WORK TASK SEQUENCE

1. Check scale full length of tunnel and install permanent bolts and shotcrete as required for long term stability.
2. Drill two HQ diamond drill holes from tunnel to surface, each about 90 m long, at Station 0+422.
3. Drill five grout holes 7 m long in spray pattern in tunnel face at 8 m from lake bed (Section A) and pressure grout.
4. Install array of 17 bolts around full periphery of tunnel using custom lengths so that bolts do not penetrate into lake (Section A, 8 m from lake bed).
5. Drill and blast Round 1 (2 m long). Install bolts and shotcrete as required.
6. Drill probe holes into lake bed (optional - observe holes with ROV).
7. Drill and blast Round 2 (1 m long). Install bolts and shotcrete on face and walls/roof/floor as required.
8. Drill and blast Round 3 (1 m long). Install bolts and shotcrete on face and walls/roof/floor as required.
9. Drill final round of blast holes in face; grout holes as required.
10. Upper rock trap: complete blasting and muck-out.
11. Trim blast cone in floor of tunnel.
12. Place (wooden) ladders in end of tunnel to provide access to face for loading explosives in final round.
13. Machine clean floor of tunnel.
14. Install trash rack in tunnel, incorporating concrete weir, just downstream of upper rock trap.
15. Muck out lower rock trap but leave ramp of loose rock at upstream end to facilitate access to upper tunnel.
16. Install submersible pump in upper rock trap.
17. Install water level monitors at upper rock trap. Pull power line for pump, blast lines and water level monitoring cables up diamond drill hole from lake tap area to surface. Set up communication system from top of diamond drill hole to tunnel portal.
18. Strip services from tunnel.
19. Install temporary ventilation line to lake tap.
20. Install permanent support at portal - bolts and shotcrete.
21. Construct tunnel plug and IPR pump.
22. Load explosives in final face and strip ventilation line.
23. Pump water into tunnel to predetermined level, while allowing air to escape from the tunnel through diamond drill hole.
24. Blast lake tap.
25. Use ROV to inspect tunnel face lake bed.



PLAN - DRILLHOLES TO SURFACE
SCALE: 1:1000



B SECTION - FACE 4 m FROM LAKE
SCALE: 1:50

1:100 0 — 6.0 m
1:50 0 — 4.0 m

Issued for	Issue	Date	Issued By	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision	Rev. No.	Date	Designed	Drawn	Checked	Description of Revision
Reference	R1	21NOV08	AG	0	17MAR09	SR / DW	GD	D. WYLLIE	ISSUED FOR CONSTRUCTION						
Approvals				1	22MAY09	SR / DW	GD	D. WYLLIE	REVISED AS NOTED						
Tender															
Permits															
Construction	C2	22MAY09	DW												
Record Drawings															

PROFESSIONAL ENGINEER D. C. WYLLIE

KERR WOOD LEIDAL associates limited CONSULTING ENGINEERS

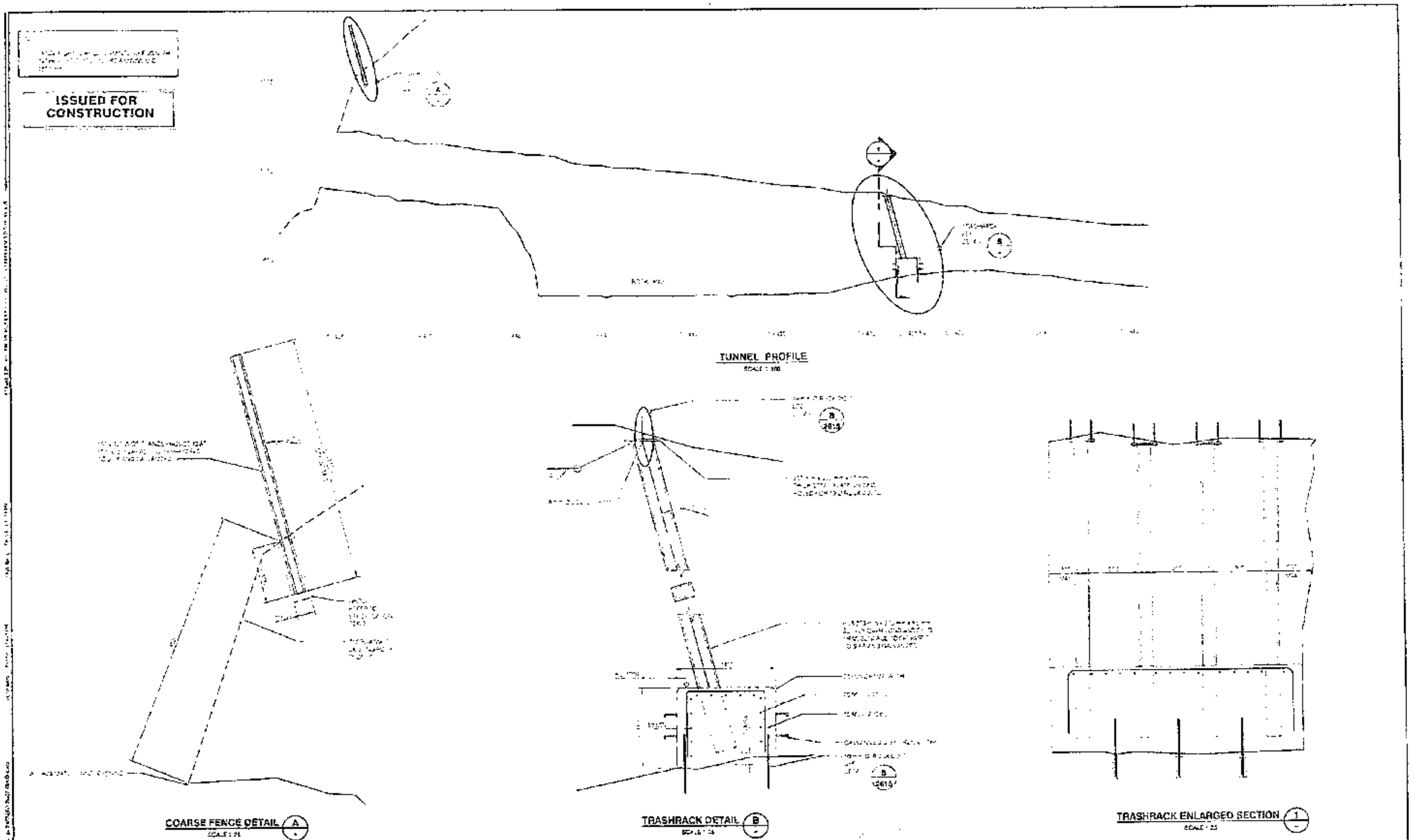
GEA Gygax Engineering Associates Ltd.

wyllie norrish ROCK ENGINEERS

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
EXCAVATION
LAKE TAP - FINAL BLAST

Scale	Project No.	Discipline	Drawing Number	Rev.
1:100, 1:50	2468-002	STRUCT	2418	1

Client: Tyson Creek Hydro Corp.



ISSUED FOR CONSTRUCTION

B
03APR'06

KERR WOOD LEIDAL

GEA Gygax Engineering Associates Ltd

TYSON CREEK HYDROELECTRIC PROJECT
TUNNEL
CIVIL
DEBRIS MANAGEMENT DETAILS

Scale	Project No.	Discipline	Drawing Number	Rev
AS SHOWN	2466-002	CIVIL	2607	0

Client: Tyson Creek Hydro Corp.