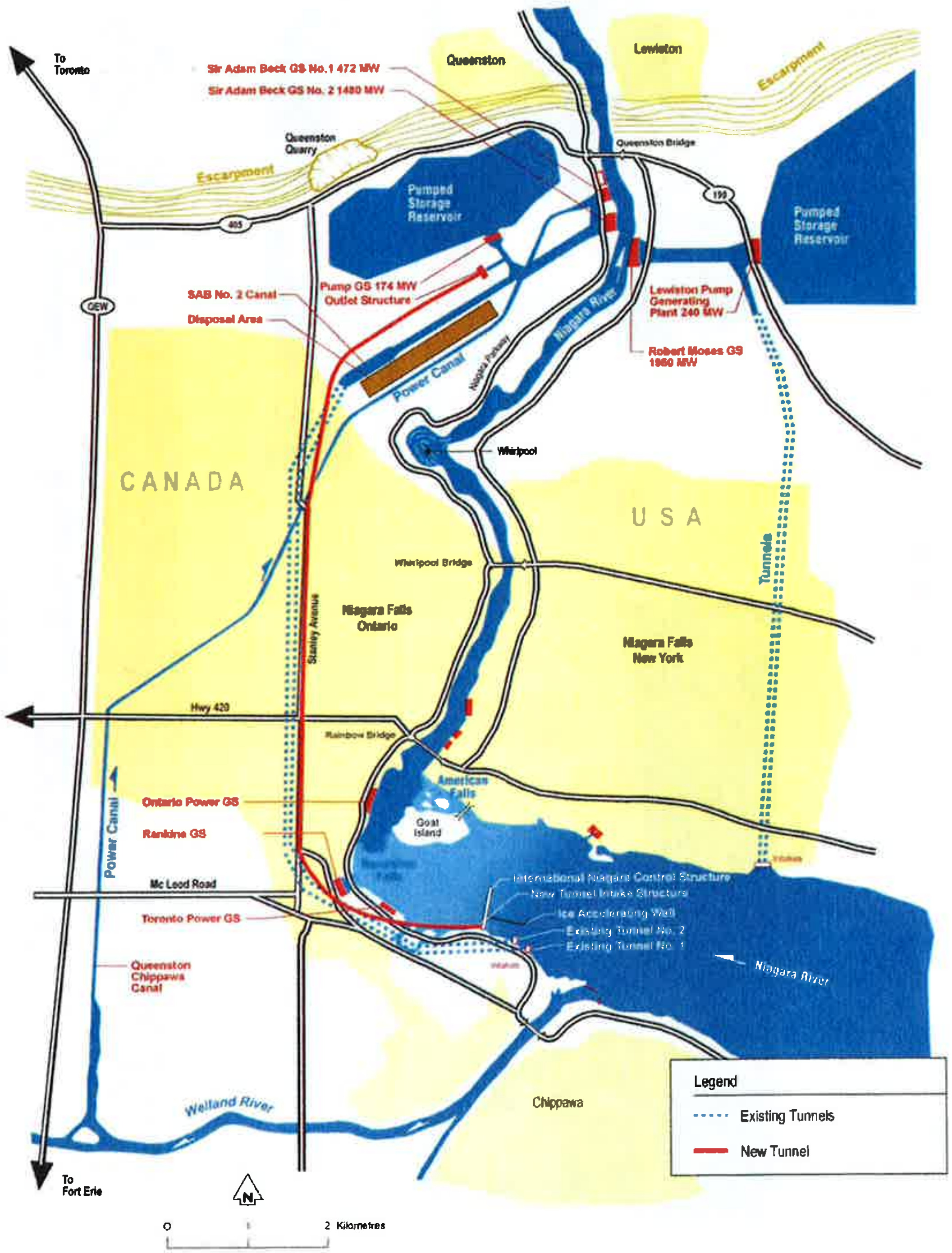
OPG 2014/2015 Payment Amounts Application

AMPCO Compendium

Panel 3 - Niagara Tunnel Project

June 12, 2014

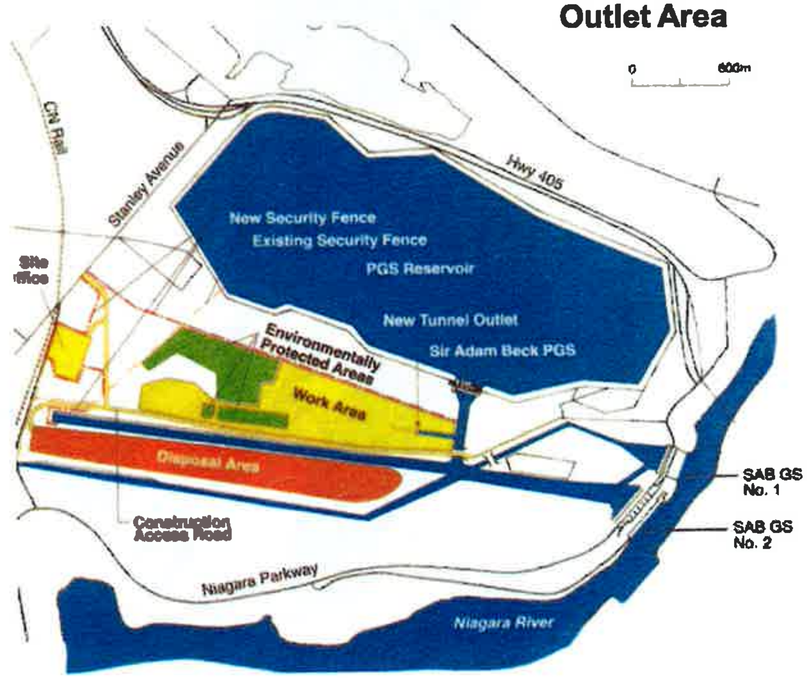
Figure 1 - Project Map

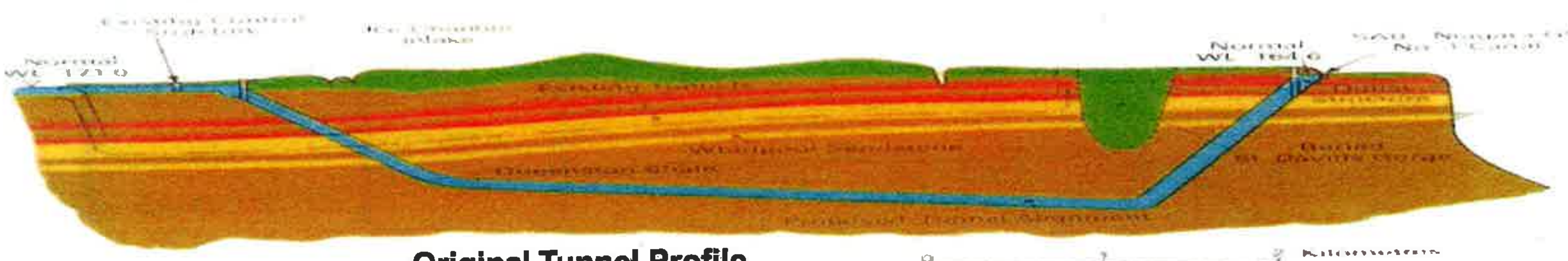


General Arrangement

Intake Area

Outlet Area





Original Tunnel Profile

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ONTARIO POWER GENERATION

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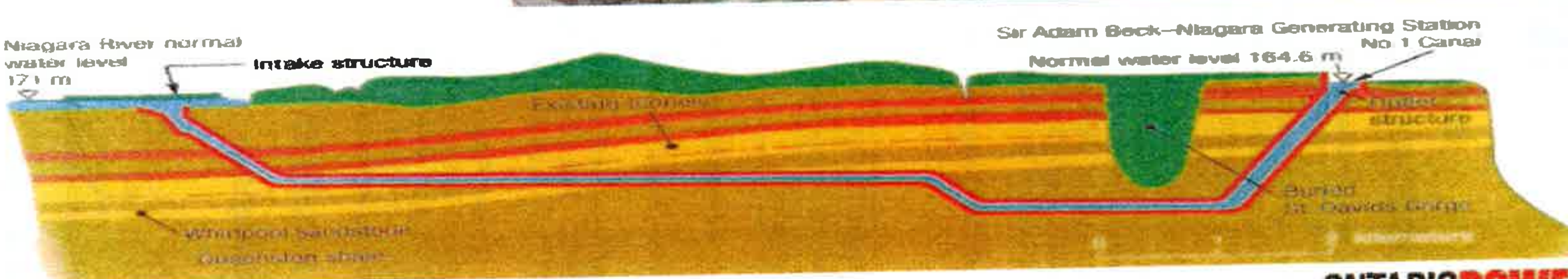
Changes in the Tunnel Alignment

- Amended DBA facilitated the required tunnel realignment.
- Horizontal alignment shifted under Stanley Avenue (yellow line to red line) starting at Sta. 3,000 m, reducing the tunnel length by about 200 m.
- Vertical alignment raised about 40 m from Sta. 4,000 m to Sta. 9,500 m minimizing the tunnel length in the Queenston shale formation.
- Required acquisition of additional subsurface property rights mostly from the Regional Municipality of Niagara (under Stanley Av).



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ONTARIO POWER GENERATION

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ONTARIOPOWER GENERATION

BUSINESS CASE SUMMARY
Niagara Tunnel Project (EXEC0007)
May 2009 (Confidential)

SUPERSEDING RELEASE FOR NIAGARA TUNNEL PROJECT (EXEC0007)

1. RECOMMENDATION:

Approve the release of $615 M additional funding for design and construction of the Niagara Tunnel Project (the "Project"), bringing the total Project cost estimate to $1,600 M including $985 M previously approved. Based on the amended design / build agreement, the tunnel will be in-service by December 2013, will increase the diversion capacity of the Sir Adam Beck Niagara GS complex by 500 m^3/s and facilitate a 1.6 TWh increase in average annual energy output from the Sir Adam Beck generating stations.

The Niagara Tunnel Project has been delayed due primarily to difficulties encountered by the contractor, Strabag Inc. (Strabag) in excavating the tunnel through the Queenston shale formation. Following an unsuccessful attempt to resolve Strabag's claim for cost and schedule relief, the parties submitted the dispute to the Dispute Review Board (DRB), as provided in the Design Build Agreement between OPG and Strabag. Following receipt of the DRB's recommendations OPG and Strabag have negotiated a settlement to ensure the tunnel is completed both safely and expeditiously.

Total Investment Cost: $1,600 M (including $985 M previously approved)

Year	To 2008	2009	2010	2011	2012	2013	2014	Totals
Project Capital	435	200	275	274	206	216	(6)	1,600
2009 Business Plan	432	173	235	143	2	-	-	985
Variance	3	27	40	131	204	216	(6)	615

Type of Investment: Strategic Projects (OAR - Section 1.3)

Release Type: Superseding

Funding: The financing for the project is arranged through the Ontario Electricity Financial Corporation (OEFC). The amended agreement increasing the facility limit of $1B to $1.6B will be executed following the OEFC's third quarter Board meeting in September 2009.

Investment Financial Measures: The increased energy output resulting from the Project will receive a regulated rate as part of OPG's regulated hydroelectric assets. With a Levelized Unit Energy Cost of under 7 ¢/kWh and an equivalent Power Purchase Agreement price of less than 10 ¢/kWh, the Niagara Tunnel Project continues to remain attractive and economic relative to other generation alternatives. Other project financial metrics and sensitivities are presented in the Financial Analysis section of this BCS.

2. SIGNATURES

Submitted by:

Carlo Crozzoli
Vice President
Hydro Development

Recommended By:

John Murphy
EVP, VP
Hydro

Approved By:

Donn Hanbidge
Chief Financial Officer

Approved By:

Tom Mitchell
President and CEO

3. BACKGROUND & ISSUES

Background

- On July 28, 2005, OPG's Board of Directors approved the Execution Phase of the Niagara Tunnel Project. The approved budget and in service date were $985 M and June 2010, respectively. This new water diversion tunnel will increase the amount of water flowing to existing turbines at the Sir Adam Beck generating stations in Niagara Falls. This tunnel will allow the Sir Adam Beck generating facilities to utilize available water more effectively and is expected to increase annual generation on average by about 1.6 TWh (14%).

- The decision to proceed with the Execution Phase was taken after comprehensive geological studies, engaging an international tunnelling/mining consulting expert (Hatch Mott MacDonald) as OPG's Owner's Representative (OR), engaging Torys to provide legal oversight and advice, and conducting an international competition to select a Design Build contractor (Strabag).

- Preparation for the new Niagara Tunnel commenced more than 25 years ago, in 1982, when Ontario Hydro (predecessor of OPG) began to study the possible expansion of its hydroelectric facilities on the Niagara River. Detailed engineering, environmental and socioeconomic studies were conducted from 1988 through 1994 with an environmental assessment (EA) submitted in 1991 for the then planned project (two 500 m^3/s water diversion tunnels, a three-unit 900-MW underground generating station and transmission improvements between Niagara Falls and Hamilton). Among the commitments made through the EA process, was to utilize a tunnel boring machine (TBM) to excavate the tunnels from the outlet end, under the buried St. Davids gorge and following the route of the existing SAB2 tunnels through the City of Niagara Falls. The EA received approval from Ontario's Minister of the Environment in 1998, including provisions to begin with construction of one tunnel, the Niagara Tunnel Project.

- Through an international proposal competition, a fixed price Design Build Agreement (DBA) was awarded to Strabag AG on August 18, 2005 and construction commenced in September 2005. The TBM was acquired and assembled within 12 months and it commenced excavation of the tunnel on September 1, 2006.

- Significant challenges excavating and supporting the Queenston shale formation, due to overstressing and insufficient, unsupported stand-up time, resulted in excessive overbreak of rock from the tunnel crown, impeded TBM advance and required significant modifications to the initial support area immediately behind the TBM cutterhead.

- Upon entering the Queenston shale formation in April 2007, Strabag encountered subsurface conditions that resulted in significantly slower than planned progress. Strabag alleged large block failures, insufficient stand-up time and excessive overbreak encountered were not consistent with the conditions described in the DBA. Strabag alleged these claims constituted a Differing Subsurface Condition (DSC), and as a result, it should be entitled to cost and schedule relief.

- Following unsuccessful attempts to resolve the issue, Strabag submitted the claim to the Dispute Review Board (DRB). The DRB is part of the dispute resolution process set out in the DBA and consists of three tunnelling experts who were regularly updated on project progress and issues. The claim was heard over four days in June 2008.

- The DRB issued its non-binding recommendations in August 2008. The DRB ruled that the excessive overbreak encountered during the tunnel drive constituted a Differing Subsurface Condition and recommended that:

  "There is a DSC with respect to excessive overbreak" (and) "both Parties must accept responsibility for some portion of the additional cost, but at the same time the Contractor must have adequate incentives to complete the Work as soon as possible."

5

Niagara Tunnel Project

The Niagara Tunnel began operation on March 9, 2013. The Niagara Tunnel Project ("NTP") was an extremely large, complex and challenging construction project that OPG completed safely and cost effectively given the conditions encountered. The emissions free electricity produced from the water flowing through the NTP will benefit the people of Ontario into the next century. Information contained within Ex. D1-2-1 will support the inclusion of the approximately $1,500M of costs associated with the NTP into regulated hydroelectric rate base.

Darlington Refurbishment Project

The continuation of the definition phase of the Darlington Refurbishment Project ("DRP") will allow OPG to develop release-quality estimates for the cost and scope of activities necessary to allow Darlington to operate for an additional 30 years. Included as part of this application is a request for a finding that the commercial and contracting strategies used by OPG in respect of the DRP are reasonable, a request for approval of the proposed test period capital ($837.4M in 2014 and $631.8M in 2015) and OM&A expenditures ($19.6M in 2014 and $18.2M in 2015), and a request for approval of in-service additions to rate base ($5.0M in 2012, $104.2M in 2013, $18.7M in 2014, and $209.4M in 2015). The Darlington Refurbishment Project is discussed in Ex. D2-2-1.

Deferral and Variance Accounts

OPG proposes to clear the audited, year-end 2013 balances only for those accounts where review was deferred to a future proceeding in EB-2012-0002. These are: 1) Hydroelectric Incentive Mechanism Variance Account, 2) Hydroelectric Surplus Baseload Generation Variance Account, 3) Capacity Refurbishment Variance and the 4) Nuclear Development Variance Accounts. Details regarding proposed account clearance and riders are presented in Ex. H1-2-1, and details regarding the continuation of accounts are found in Ex. H1-3-1. OPG intends to seek review and clearance of the audited year-end December 31, 2014 balances in all of its deferral and variance accounts through a separate application to be filed in 2014.

the variance account additions sum to the "Variance Account Total Balance (c/b)" line. All variance account amounts were recorded in the Capacity Refurbishment Variance Account.

OPG currently estimates that the total cost of the Niagara Tunnel Project will be $1,476.6M ($1,472.0M in capital expenditures with an additional $4.6M in removal costs). In 2013, a total of $1,439.2M in capital costs was brought into service. This consisted of $1,424.9M placed in-service in March 2013 and an additional $14.3M placed in-service at the end of November 2013.

Chart 1

Numbers may not add due to rounding

Niagara Tunnel Project

(in millions$)	Pre-2008 actual	2008 actual	2009 actual	2010 actual	2011 actual	2012 actual	2013 actual	2014 Test Year	2015 Test Year	Total 2008-2015	Estimate at Completion
Project Budget Approved/Revised by OPG Board[1]	985.0	985.0	1,600.0	1,600.0	1,600.0	1,600.0	1,600.0	1,600.0	1,600.0	1,600.0	1,600.0
Capital Expenditures	300.2	131.3	213.5	231.8	264.2	231.2	86.6	13.0	0.4	1,171.8	1,472.0
Running Total Accumulated Capital Expenditures	300.2	431.6	645.0	876.8	1,140.9	1,372.1	1,458.7	1,471.7	1,472.0	1,171.8	1,472.0
Gross Plant In-service (o/b)	19.2	19.2	19.2	19.2	19.2	19.2	19.2	1,458.4	1,471.5	19.2	-
Gross Plant Additions	-	-	-	-	-	-	1,439.2	13.0	0.4	1,452.8	1,472.0
Gross Plant in-service (c/b)	19.2	19.2	19.2	19.2	19.2	19.2	1,458.4	1,471.5	1,472.0	1,472.0	1,472.0
Accumulated Depreciation (o/b)	-	0.3	0.5	0.8	1.0	1.3	1.5	15.8	29.8	0.3	0.3
Accumulated Depreciation (c/b)	0.3	0.5	0.8	1.0	1.3	1.5	14.5	29.8	45.6	45.6	45.6
Net Plant In-service (o/b)	19.2	18.9	18.7	18.4	18.2	17.9	17.7	1,442.6	1,441.7	18.9	-
Net Plant In-service (c/b)	18.9	18.7	18.4	18.2	17.9	17.7	1,443.9	1,441.7	1,426.4	1,426.4	1,426.4
Operating Costs Expensed (Removal Costs)[2]	3.0	-	-	-	1.4	0.2	-	-	-	1.6	4.6
Operating Costs Recorded in Variance Account [*, 2]	3.0	-	-	-	1.4	0.2	-	-	-	1.6	4.6
Rate Base Related Costs Recorded in Variance Account	-	-	-	-	(2.3)	1.8	115.4	-	-	114.9	114.9
Interest Improvement on Variance Account Balance	-	-	-	-	-	-	0.6	1.7	1.3	3.6	3.6
Variance Account Total Balance (o/b)	-	-	-	-	-	(0.9)	1.0	117.1	118.8	120.1	120.1
Variance Account Amount Cleared[4]	-	-	-	-	-	-	-	-	58.5	58.5	58.5
Variance Account Total Balance (c/b)	-	-	-	-	(0.9)	1.0	117.1	118.8	61.6	61.6	61.6

Note: * Capacity Refurbishment Variance Account or equivalent

o/b= opening balance, c/b = closing balance

Notes:
1 Project Budget Approved is as per Superseding Business Case Summary in Ex. D1-2-1, Attachment 8a.
2 Per Ex. D1-2-1 page 4, lines 11-16.
3 Includes income tax impacts as shown in Ex. L-9-1 Schedule 17, SEC-132, Attachment 1, Table 7, line 10.
4 Represents 12/24 of the actual 2013 balance consistent with OPG's proposal to recover the balance over 24-months ending December 31, 2016.

Board Staff Interrogatory #021

Ref: Exh D1-1-2 page 13, Exh D1-2-1 page 2, Attachment 8B and EB-2007-0905/Exh D1-1-2 Attachment A Appendix C page 3

Issue Number: 4.4
Issue: Do the costs associated with the Niagara Tunnel Project that are subject to section 6(2)4 of O. Reg. 53/05 and proposed for recovery, meet the requirements of that section?

Interrogatory

OPG indicates that it placed $1,474.2M in service in 2013 for the NTP. OPG also states that O. Reg. 53/05, section 6(2)4 requires the Board to ensure that OPG recovers the capital and non-capital costs of the NTP approved by the OPG Board of Directors prior to the first payment amounts order and to determine the prudence of any expenditures beyond the OPG Board approved amount.

In the Recommendation for Submission to the Board of Directors, dated May 21, 2009, OPG states:

> Once in-service, the NTP will form part of OPG's regulated rate base. Under O.Reg 53/05 the OEB is required to ensure that OPG recovers the original project budget of $985M approved by OPG's Board and this amount will not be subject to a prudence review by the OEB. However, the incremental project costs above the original approval will be subject to a prudence test. Under the OEB's prudence test, OPG's actions are assumed to be prudent unless challenged on reasonable grounds. In assessing prudence, the OEB will consider what information was known or should have been known at the time key decisions were made and what third-party expert advice was sought to assist in decision making. Hindsight is not to be used in determining prudence. Given the extensive volume of studies conducted prior to project execution and the nature of independent advice sought throughout the process (leading international consultants, academia, Dispute Review Board, Contract Oversight Committee, etc.), OPG is well positioned to make the case that the entire capital cost should be recoverable. OPG will, of course, have to demonstrate ongoing diligence in project execution as part of its case for recoverability. However, given the significant cost over-runs associated with the project, the OEB will be likely to review the matter in detail and therefore regulatory risk remains.

In the original Full Release Business Case Summary ("BCS"), dated July 28, 2005, filed in the 2008-09 Payments Amounts proceeding, at page 3 OPG indicated that "Under Ontario Regulation 53/05, effective April 1, 2005, the Project will become part of OPG's regulated hydroelectric assets and OPG will be given a fair opportunity to recover prudently incurred costs through regulated rates."

8

a) Of the total NTP related costs that have been or are proposed to be recovered from ratepayers, please confirm whether $985M is the amount that OPG considers as "OPG Board of Directors approved". What is the exact amount that OPG views as in excess of the OPG Board approved amount?

b) Appendix C of the BCS, dated July 28, 2005, provides a project risk profile for the NTP. Mitigating activity is identified regarding the risk that the contractor may encounter subsurface conditions that are more adverse than described in the Geotechnical Baseline Report ("GBR"). Mitigating activities include "The GBR is based on extensive field investigations carried out over a 10-year period and knowledge gained through the construction of the SAB2 tunnels." and "The 3-stage GBR process used facilitates contractor input and concurrence before construction begins".
   i. Are the SAB2 tunnels at the same depth as the NTP?
   ii. To what extent, as compared to the planned route for the NTP, do the SAB2 tunnels travel through the same Queenston shale environment?

c) Please compare and contrast the excavation or boring technique used for SAB2 with that used in the NTP. Is it the case that the only risk mentioned in Appendix C of the BCS regarding Queenston shale, the host rock formation for the majority of the tunnel, is its swelling properties when exposed to fresh water? At the time the Business Case was prepared was OPG aware of any other geotechnical risks that could be associated with Queenston shale?

d) In OPG's view how successful were the aforementioned mitigating activities in reducing, if not eliminating the noted risk?

e) To what extent would the costs in excess of $985M be greater had the mitigating activities not taken place?

Response

a) The original budget of $985.2M was approved by the OPG Board of Directors ("OPG Board") prior to the OEB's first order with respect to payment amounts for OPG's prescribed facilities under Section 78.1 of the Ontario Energy Board Act (see Ex. D1-2-1, page 2, and Ex. D1-2-1, Attachment 5, BCS July 28, 2005). This is the amount that OPG considers to be the "OPG Board of Directors approved" for purposes of section 6.(2)4. of O. Reg. 53/05. The OPG Board subsequently approved a revised budget of $1,600M (see Ex. D1-2-1, page 115, and Ex. D1-2-1, Attachment 8a, Superseding BCS May 21, 2009). The actual project cost is currently estimated at $1,476.6M which is $491.4M over the original OPG Board approval but $123.4M below the superseding OPG Board approval.

b) I) No, the SAB2 tunnels are not as deep as the NTP.
   ii) No portion of the SAB2 tunnels is in the Queenston shale formation.

c) A tunnel boring machine ("TBM") was not used for the SAB2 tunnels. Instead, the 15.55m diameter SAB2 tunnels were blasted through the rock in two stages. First, the top 9.0m was excavated and supported with steel ribs. Second, the bottom 6.55m was excavated. For additional information please see the response to Ex. L-6.12-1 Staff IR-160 c). The NTP was

9

Ontario Energy Board Act, 1998
Loi de 1998 sur la Commission de l'énergie de l'Ontario

ONTARIO REGULATION 53/05

PAYMENTS UNDER SECTION 78.1 OF THE ACT

Consolidation Period: From November 29, 2013 to the e-Laws currency date.

Last amendment: O. Reg. 312/13.

This Regulation is made in English only.

Definition

0.1 In this Regulation,

"approved reference plan" means a reference plan, as defined in the Ontario Nuclear Funds Agreement, that has been approved by Her Majesty the Queen in right of Ontario in accordance with that agreement;

"nuclear decommissioning liability" means the liability of Ontario Power Generation Inc. for decommissioning its nuclear generation facilities and the management of its nuclear waste and used fuel;

"Ontario Nuclear Funds Agreement" means the agreement entered into as of April 1, 1999 by Her Majesty the Queen in right of Ontario, Ontario Power Generation Inc. and certain subsidiaries of Ontario Power Generation Inc., including any amendments to the agreement. O. Reg. 23/07, s. 1.

Note: On July 1, 2014, section 0.1 is amended by adding the following subsection: (See: O. Reg. 312/13, ss. 1, 6)

(2) For the purposes of this Regulation, the output of a generation facility shall be measured at the facility's delivery points, as determined in accordance with the market rules. O. Reg. 312/13. s. 1.

Prescribed generator

1. Ontario Power Generation Inc. is prescribed as a generator for the purposes of section 78.1 of the Act. O. Reg. 53/05, s. 1.

Prescribed generation facilities

2. The following generation facilities of Ontario Power Generation Inc. are prescribed for the purposes of section 78.1 of the Act:

1. The following hydroelectric generating stations located in The Regional Municipality of Niagara:
   i. Sir Adam Beck I.
   ii. Sir Adam Beck II.
   iii. Sir Adam Beck Pump Generating Station.
   iv. De Cew Falls I.
   v. De Cew Falls II.
2. The R. H. Saunders hydroelectric generating station on the St. Lawrence River.
3. Pickering A Nuclear Generating Station.
4. Pickering B Nuclear Generating Station.
5. Darlington Nuclear Generating Station. O. Reg. 53/05, s. 2; O. Reg. 23/07, s. 2.

Note: On July 1, 2014, section 2 is amended by adding the following paragraph: (See: O. Reg. 312/13, ss. 2, 6)

6. As of July 1, 2014, the generation facilities of Ontario Power Generation Inc. that are set out in the Schedule.

Prescribed date for s. 78.1 (2) of the Act

3. April 1, 2008 is prescribed for the purposes of subsection 78.1 (2) of the Act. O. Reg. 53/05, s. 3.

Payment amounts under s. 78.1 (2) (a) of the Act

2. In setting payment amounts for the assets prescribed under section 2, the Board shall not adopt any methodologies, assumptions or calculations that are based upon the contracting for all or any portion of the output of those assets.

3. The Board shall ensure that Ontario Power Generation Inc. recovers the balance recorded in the deferral account established under subsection 5 (4). The Board shall authorize recovery of the balance on a straight line basis over a period not to exceed 15 years.

4. The Board shall ensure that Ontario Power Generation Inc. recovers capital and non-capital costs, and firm financial commitments incurred to increase the output of, refurbish or add operating capacity to a generation facility referred to in section 2, including, but not limited to, assessment costs and pre-engineering costs and commitments,

   i. if the costs and financial commitments were within the project budgets approved for that purpose by the board of directors of Ontario Power Generation Inc. before the making of the Board's first order under section 78.1 of the Act in respect of Ontario Power Generation Inc., or

   ii. if the costs and financial commitments were not approved by the board of directors of Ontario Power Generation Inc. before the making of the Board's first order under section 78.1 of the Act in respect of Ontario Power Generation Inc., if the Board is satisfied that the costs were prudently incurred and that the financial commitments were prudently made.

4.1 The Board shall ensure that Ontario Power Generation Inc. recovers the costs incurred and firm financial commitments made in the course of planning and preparation for the development of proposed new nuclear generation facilities, to the extent the Board is satisfied that,

   i. the costs were prudently incurred, and

   ii. the financial commitments were prudently made.

5. In making its first order under section 78.1 of the Act in respect of Ontario Power Generation Inc., the Board shall accept the amounts for the following matters as set out in Ontario Power Generation Inc.'s most recently audited financial statements that were approved by the board of directors of Ontario Power Generation Inc. before the effective date of that order:

   i. Ontario Power Generation Inc.'s assets and liabilities, other than the variance account referred to in subsection 5 (1), which shall be determined in accordance with paragraph 1.

   ii. Ontario Power Generation Inc.'s revenues earned with respect to any lease of the Bruce Nuclear Generating Stations.

   iii. Ontario Power Generation Inc.'s costs with respect to the Bruce Nuclear Generating Stations.

6. Without limiting the generality of paragraph 5, that paragraph applies to values relating to,

   i. capital cost allowances,

   ii. the revenue requirement impact of accounting and tax policy decisions, and

   iii. capital and non-capital costs and firm financial commitments to increase the output of, refurbish or add operating capacity to a generation facility referred to in section 2.

7. The Board shall ensure that the balances recorded in the deferral accounts established under subsections 5.1 (1) and 5.2 (1) are recovered on a straight line basis over a period not to exceed three years, to the extent that the Board is satisfied that revenue requirement impacts are accurately recorded in the accounts, based on the following items, as reflected in the audited financial statements approved by the board of directors of Ontario Power Generation Inc.,

Note: On July 1, 2014, paragraph 7 is amended by striking out the portion before subparagraph i and substituting the following: (See: O. Reg. 312/13, ss. 4 (1), 6)

7. The Board shall ensure that the balance recorded in the deferral account established under subsection 5.2 (1) is recovered on a straight line basis over a period not to exceed three years, to the extent that the Board is satisfied that revenue requirement impacts are accurately recorded in the account, based on the following items, as reflected in the audited financial statements approved by the board of directors of Ontario Power Generation Inc.,

   i. return on rate base,

   ii. depreciation expense,

   iii. income and capital taxes, and

   iv. fuel expense.

7.1 The Board shall ensure the balances recorded in the deferral account established under subsection 5.3 (1) and the variance account established under subsection 5.4 (1) are recovered on a straight line basis over a period not to exceed three years, to the extent the Board is satisfied that,

SEC Interrogatory #033

Ref: D1/2/1/p.28

Issue Number: 4.5
Issue: Are the proposed test period in-service additions for the Niagara Tunnel Project appropriate?

Interrogatory

How did OPG determine the appropriate contingency for the Niagara Tunnel Project?

Response

OPG determined the appropriate (90% confidence level) cost and schedule contingencies for the tunnel design build contract based on OPG's update (Ex. D1-2-1 Attachment 4) of the URS quantitative risk assessment (Ex. D1-2-1 Attachment 3). On this basis, the appropriate cost contingency was $96M and the appropriate schedule contingency was 36 weeks for the tunnel design build contract. Engineering judgment was used to determine the $5M contingency added for the Guaranteed Flow Amount incentive and the $11M contingency added for cost risks associated with other elements of the project. Accordingly, OPG included cost contingency of $112M and schedule contingency of 36 weeks in the originally approved Business Case (Ex. D1-2-1 Attachment 5) for the Niagara Tunnel Project.

f) Beginning in 1988, Ontario Hydro (now OPG) engaged Acres (now Hatch) to provide engineering services that included geotechnical investigations and analysis as outlined in Ex. D1-2-1 Appendix B – Summary of Geological Investigations and in Ex. F5-6-1 Niagara Diversion Tunnel Report prepared by Roger Ilsley. Based on these geotechnical investigations and analysis, Hatch (formerly Acres) prepared the Geotechnical Baseline Report ("GBR") included in the Design Build Agreement (Ex. D1-2-1 Attachment 6). The GBR captures the results of the extensive geotechnical investigations and analysis to detail the subsurface conditions expected to be encountered during design and construction of the Niagara Tunnel.

g) OPG considers that 100% of the variance relative to the originally approved budget of $985.2M is due to the more adverse subsurface conditions experienced during the tunnel construction. This includes direct increases in tunnel contract costs and additional time related costs in categories such as interest during construction, OPG Project Management and Owner's Representative costs.

Project Cost Flow Estimate ($M) (including Contingency)	**Original Approval (DBA)**	**Revised Estimate (ADBA)**	**Estimated Capital Cost at Completion** [1]	**Costs Associated with Adverse Subsurface Conditions**
OPG Project Management	4.4	6.0	5.0	0.6
Owner's Representative	25.4	40.4	36.2	10.8
Other Consultants	4.0	5.9	6.5	2.5
Environmental / Compensation	12.0	9.6	8.7	(3.3)
Tunnel Contract (including Incentives)	723.6	1,181.7	1,112.9	389.3
Other Contracts / Costs	78.9	69.8	68.4	(10.5)
Interest	136.8	286.6	234.5	97.7
Total Project Capital	985.2	1,600.0	1,472.0	486.8

Notes: 1) Estimated Capital Cost at Completion as noted in response to Board Staff Interrogatory #28.
2) Numbers may not calculate due to rounding.

h) OPG did not consider any cost sharing arrangements for the costs above the $985.2 M approved by OPG's Board of Directors prior to OEB regulation. As fully documented in the evidence, the amount OPG spent on the NTP represents the true cost of completing the project given the subsurface conditions actually encountered. OPG acted prudently in planning and executing this project and in addressing the differing subsurface conditions encountered. Since any cost sharing arrangements would amount to a disallowance of prudently incurred costs, OPG did not consider them.

as a living document that would be frequently updated as the project moved from conceptualization to completion.

Following the completion of the qualitative risk assessment, URS undertook the quantitative assessment. The quantitative assessment was performed using a Monte Carlo simulation based analysis. The methodology consisted of identifying the conceivable hazards that could occur during the project, and assessing a probability of occurrence for each hazard as well as their potential cost and schedule impacts. The probabilities and consequences were then combined to identify potential outcomes in 5,000 scenarios for the project and to obtain probability distributions of possible outcomes. Based on these distributions, the probability, cost and schedule values were established by members of an expert panel, which included NTP team members from OPG and Hatch. The expert panel's efforts were facilitated by URS. The analysis only addressed the costs and risks impacts for the project (i.e., to the time of commissioning) and did not include risks associated with post-project operation.

As both the qualitative and quantitative risk evaluations undertaken by URS were done prior to completing the solicitation for a design-build contractor, OPG recognized the need to update the quantitative risk evaluation once the final proposals were received from the design-build proponents. This update was undertaken by an expert panel of NTP team members consisting of personnel from OPG, Hatch and Torys LLP ("Torys"), OPG's external legal counsel. It was completed on July 27, 2005, the day before the selection of the successful proponent was approved by the OPG Board. OPG used the model that had been developed by URS and updated it to:

- confirm analytical assumptions and numerical inputs;
- add any additional hazards identified and remove any that were no longer relevant; and
- reflect any differences among the proposals submitted.

In the OPG update, the top two contributors to potential cost increases were: 1) "Dispute Review Board interpretation of Agreement unfavourable" and 2) "DSC [Differing Subsurface Conditions] claim due to rock strength." These same two factors, in reverse order, were also identified as the top two contributors to potential schedule delay for which OPG, rather than the contractor, would be responsible. Based on the results of the updated quantitative risk assessment model, OPG estimated that for the tunnel construction portion of Strabag's proposal, a $96M cost contingency and a 36 week schedule contingency were required to achieve a 90 per cent probability that the project would remain within its budget and schedule.[12] OPG then determined the overall cost contingency to be $112M for the project as a whole.

3.4 Invitation to Submit Design-Build Proposals

In late December 2004 invitations to respond to the RFP were sent to the four firms identified in the preceding section with the proposals due on April 15, 2005. The RFP consisted of three volumes: the first contained the invitation letter, instructions, the draft DBA and various appendices; the second volume contained concept drawings; and the third contained construction labour agreements from the Electrical Power Systems Construction Association ("EPSCA"). The RFP requested that the proponents return a form indicating whether they would be submitting a proposal.

Three of the four invitees, namely Niagara Tunnel Constructors, Niagara Tunnelers and Strabag AG, indicated that they would submit a proposal. In January 2005, these three proponents participated in a mandatory site visit. In association with the visit, the proponents also reviewed background documents in a data room that had been established by OPG near the project site.

Amendments to the invitation documents were distributed starting February 2005. In total, five amendments were issued reflecting changes made in response to questions or issues raised by the proponents. Based on a request from all three proponents, the deadline for

[12] As noted in the OPG risk update (page 2): "The schedule contingency only took into consideration OPG-accountable schedule risks, as the DBA compensated OPG for contractor-accountable delays through the payment of Liquidated Damages. Moreover, the schedule contingency assumed that the project schedule, which was set by the contractor, included some contingency as determined by the contractor."

With respect to safety, where Strabag is the "constructor" (as that term is defined under the *Occupational Health and Safety Act*, Ontario), the OR monitored and audited Strabag's safety performance. At the intake area, when OPG is the constructor (explained more fully in Section 6.5.3), the OR was responsible for managing project site safety on OPG's behalf in accordance with OPG's policies and procedures.

6.3 Project Risk Management

In addition to the PEP, OPG periodically updated the OPG Risk Management Plan ("RMP"). The RMP was prepared at the onset of the project by building on and extending the risk assessment work initially developed by URS prior to contract award as discussed above in Section 3.3. It documented how risk management is performed for the NTP, as well as the roles and responsibilities of the project team members, the methodology and tools to be used, and the schedule for risk management activities. The RMP summarized the NTP risk management process as consisting of the following activities: risk identification, risk assessment, risk response planning, risk monitoring and control, and risk reporting.

Strabag independently conducted risk assessments as part of its proposal preparation and submitted a summary risk register with its proposal. Both OPG and Strabag continued independent risk management initiatives during the design/construction phase of the NTP so as to protect their proprietary information. However, OPG and Strabag were required to adopt significant portions of the "Code of Practice for Risk Management of Tunnel Works" (referenced above in Section 3.3 as a condition of obtaining insurance coverage for the project). These provisions required OPG and Strabag to share details of their respective risk assessments and to systematically coordinate construction phase risk management efforts to identify risks and mitigate them to the extent possible.

As a result of these requirements, two risk registers are discussed in the OPG Risk Management Plan: the OPG Qualitative Risk Register ("OPG Risk Register"), which later evolved into the NTP Key Risk Register as discussed below, and the Construction Phase Qualitative Risk Register ("Combined Risk Register").

16

- Technical / Operational Considerations
  - The Niagara Tunnel design life is 90 years without the need for any planned maintenance.

- Health & Safety
  - Safety program / performance was a significant factor in contractor pre-qualification.
  - The Design / Build Contractor has implemented comprehensive project site specific plans for construction safety and for public safety and security.
  - Strabag and its subcontractors have achieved commendable Health and Safety performance to date with a Lost Time Injury Frequency of 0.8 per 200,000 hours worked, less than half of the average for Ontario's heavy civil construction industry.

- Staff Relations
  - An agreement was reached with The Society of Energy Professionals regarding "purchased services" required for the Niagara Tunnel Project. Further discussions are expected in regard to additional services required for the extended project duration.
  - Purchased Services Agreement discussions were completed with the Power Workers Union.
  - In accordance with the Chestnut Park Accord Addendum, trades work has been assigned to the Building Trades Unions.
  - Electric Power Systems Construction Association (EPSCA) conditions apply to the performance of this work.

7. RISKS

- Prior to project execution, OPG, with the assistance of URS (a specialist consultant), conducted a comprehensive risk assessment (qualitative and quantitative) for design and construction of the Niagara Tunnel. Major project risks were identified through a series of workshops involving the project team and key stakeholders. During project execution, a Risk Register and associated Risk Management Plan have been maintained to manage residual risks.

- As required by the underwriters of the builder's all risk insurance policy, OPG (represented by OR) and the Contractor developed and maintain a Combined Risk Register for management of the tunnel construction risks.

- OPG's Risk Services Group facilitated the updating of the original risk registers. The input data was gathered through five separate facilitated workshops involving OPG project team and OR representatives who were asked to provide individual estimates of both the likelihood and the impact of 13 key risks that they had previously identified. Further details on the key risks are summarized in Appendix C.

- In addition, six schedule uncertainty risks (TBM mining, invert concreting, infill shotcreting, arch concreting, contact grouting and pre-stress grouting) were similarly assessed.

- These cost and schedule uncertainties were combined using Monte Carlo simulations to generate estimates of possible cost and schedule outcomes at various levels of confidence. The results indicated that a cost contingency of $164 million would likely be sufficient to cover the cost uncertainties at a 90% confidence level for the 13 identified risks and six schedule uncertainty risks.

- The estimated in-service date is December 31, 2013, including a 6.5 month schedule contingency beyond Target Schedule date of June 15, 2013. The schedule contingency was based on management judgement.

- The financial analysis completed for the recommended alternative is based on spending the entire cost and schedule contingency and is therefore considered to be conservative and robust.

14 Risk Assessment and Risk Management

14.1 Overview

Major projects generally face significant technical and other challenges during their planning, design, construction, and commissioning phases. Effective risk management is critical to the success of these projects and will allow for informed communication with project stakeholders such as owners, funding partners, insurers, designers, contractors, and the regulatory authorities, with regard to issues and expectations.

The Risk Management Plan (RMP) documents how risk management will be performed for the Project. It documents the roles and responsibilities for project team members, the methodology and tools to be used, and the schedule for risk management..

The OPG risk management process used for the NTP is based on a standardized methodology as detailed in Project Risk Management, OPG-PROC-0025 (superseded by Project Risk Management Standard, OPG-STD-0062) and is consistent with industry best practices. In addition, as a condition of providing insurance coverage for the NTP, the underwriters insisted that significant portions of the International Tunneling Insurance Group "Code of Practice for Risk Management of Tunnel Works" (the "Code") be adopted by the Project. As a result of this Code adoption, OPG and the Design/Build Contractor were required to share details of their respective risk assessments and to systematically coordinate construction phase risk management efforts. These coordinated efforts are documented in the Construction Phase Qualitative Risk Register ("Combined Risk Register"). In summary, the NTP risk management process consists of the following activities:

- Risk Identification
- Risk Assessment
- Risk Response Planning
- Risk Monitoring and Control.

It is important to note that the risk management process is iterative, so as the project progresses, the RMP and corresponding documents (e.g., Risk Registers) continue to be revised.

7 The documents which will be used to carry out the risk management process are as follows:

- Risk Management Plan (RMP)
- Execution Phase Business Case Summary (BCS)
- Execution Phase Project Execution Plan (PEP)
- Monthly Reports
- Key Risk Register
- Key Risk Register Summary
- Combined Risk Register (OPG/Contractor)
- Quantitative Risk Analysis reports.

Section	Title
Volume 2	
Appendix 1.1(vv)	Owner's Mandatory Requirements
Appendix 1.1(hhh)	Project Change Directive Form
Appendix 1.1(iii)	Project Change Notice
Appendix 1.1(sss)	Summary of Work
Appendix 1.1(hhhh)	On-Site Total Monthly Trade Labour Hours
Appendix 2.2(a)	Organizational Chart
Appendix 2.2(b)	Scopes of Authority for Contractor's Delegates
Appendix 2.4(c)	INTENTIONALLY DELETED
Volume 3	
Appendix 2.4(d)	Preliminary Project Specific Site Safety, Security, Public Safety and Emergency Response Plan
	Letter and Pre-Start Project Specific Safety Plan (table)
	Contactor Management Environment, Health & Safety Qualification
	Annex A – Work Safety Information
	Annex C – Industrial Safety Information
	Annex D – Supervisors Qualification Program
REPORTS IN GERMAN ?	Annex E – Risks Evaluation on Site
	Annex F – Example Accident Investigation Program Manapouri
	Annex G – Examples Job Safety Program
	Annex H – Safety Personnel
	McNally Health & Safety Policy and Program
	- Sections 1 to 4
	- Sections 5 to 7

German

Annex E
Risks Evaluation on Site

Inhaltsverzeichnis ASA 2002

	Ersteller: Administrator
	Erstelldatum: 22.11.02

Arbeitsvorgänge

Arbeitsplätze

Arbeitsmittel

Arbeitsstoffe

BAUSTELLE

Besonderheiten - ASA 2002

	Ersteller: Administrator
	Erstelldatum: 22.11.02

Arbeitsvorgänge

Arbeitsplätze

Arbeitsmittel

Arbeitsstoffe

PSA: Sicherheitsschuhe/-stiefel * Ggf. Schutzhandschuhe

Fachkunde: Für Jugendliche ist das Manipulieren von schweren Lasten verboten

Unterlagen: Baumappe D4 * Baumappe 2002 D23 Transport / Ladungssicherung

■ Transport/Laden maschinell

Maßnahmen: Ladung ausreichend sichern (Verspannen, Verkeilen, Versperren). * Transportfahrzeug max. Breite/Höhe 2,5 m/4 m, sonst Ausnahmegenehmigung. * Sichere Auffahrtsrampen bzw. geeignetes Hebegerät einsetzen. * Keine Personen im Bereich der schwebenden Last. * Nicht neben oder hinter zu ladenden Fahrzeugen (Abroll-/ Kippgefahr). * Zwischenlastfälle beachten (Dreh-/ Kippbewegungen).

Gefahren: Quetschen * Kippen/Herabfallen

PSA: Sicherheitsschuhe/-stiefel * Ggf. Schutzhelm * Ggf. Schutzhandschuhe

Fachkunde: Ggf. Führerschein

Unterlagen: Baumappe D4 * Baumappe 2002 D23 Transport / Ladungssicherung

Arbeitsplätze

■ Arbeiten in Baugruben/Arbeitsgräben

Maßnahmen: Fachgerechten Verbau oder Böschung sicherstellen (ab 1,25 m Tiefe jedenfalls). * Schutzstreifen 50 cm freilassen. * Absturzsicherung vorsehen. * Zugang über Treppen oder Leitern. * Auf Schadstoffansammlung achten. * Auf ausreichende Arbeitsraumbreite achten. * Bei Ausführung steilerer Neigung als Regelböschung: Standsicherheitsnachweis.

Sonstiges: Böschung/Verbau regelmäßig prüfen (jedenfalls nach starken Regenfällen, Tauwetter, Sprengungen oder wesentlichen Belastungsänderungen).

Gefahren: Verschütten * Rollen/Gleiten/Abrutschen * Stürzen / fallen * Ausrutschen

PSA: Sicherheitsschuhe/-stiefel * Ggf. Schutzhelm

Unterlagen: Baumappe C3 * Baumappe C4 * Baumappe C5 * AUVA M 223 * BauV 6. Abschnitt * Baumappe 2002 D1 Böschungen * Baumappe 2002 D3 Leitungssicherung * Baumappe 2002 D4 Baugrubenverbau * Baumappe 2002 D5 Arbeitsraumbreiten dulden. * Mitfahrt von Personen nur auf geeigneten Plätzen. * Personenhub nur mit geprüften Arbeitsmitteln. * Grabwerkzeug gegen unbeabsichtigtes Lösen sichern. * Auf tragfähigen Untergrund und Geländestufen achten (Sicherheitsabstände). * Bei Sichteinschränkung Einweiser. * Für Wartung: Motor abstellen; Geräteteile absenken oder stützen. * Gefahren aus Antrieb beachten. * Zurücklaufen der Last verhindern, durch: Rückschlagventil zwischen Pumpe und Hubzylinder oder Windwerk mit selbsthemmendem Getriebe, oder selbsttätig wirkende Bremse. * Beim Heben von Personen Zusatzbestimmungen beachten.

Gefahren: Quetschen * Kippen/Herabfallen * Überfahren

PSA: Ggf. Schutzhelm * Ggf. Sicherheitsschuhe / -stiefel

Prüfungen: Abnahmeprüfung * Jährlich * Vor Inbetriebnahme

Fachkunde: Ausbildung/betriebliche Erlaubnis * Für Jugendliche mit Lernfahrausweis oder Lenkerberechtigung (kraftrechtliche Vorschriften) erlaubt

Unterlagen: Bedienungsanleitung * BauV 21. Abschnitt * Prüfbuch aufliegend * Baumappe D1 * AUVA M 250 * Arbeitsmittelverordnung - AM-VO §§ 24, 53 selbstfahrende Arbeitsmittel * Baumappe 2002 E1 Baumaschinen

Arbeitsstoffe

Bauchemie allgemein

Maßnahmen: Sicherheitsdatenblätter sammeln und betroffenen Mitarbeitern zugänglich machen. * Einkauf von Schutzausrüstung entsprechend Anforderungen laut Sicherheitsdatenblatt z.B. spez. Material beim Schutzhandschuh informieren. * Unterweisung bezüglich spezieller Maßnahmen. * Einsatz des jeweiligen am wenigsten gefährlichen Arbeitsstoffes anstreben.

Sonstiges: Jugendschutzbestimmungen beachten. * Bei Be- und Entladen stets geschlossene Gebinde. * Verschmutzung der Hände durch anhaftende Inhaltsstoffe vermeiden.

Gefahren: lt. Sicherheitsdatenblatt

PSA: lt. Sicherheitsdatenblatt

Prüfungen: lt. ASCHG 4. Abschnitt

Fachkunde: Präventivdienste: Mitwirken bei Auswahl und Verwendung

Unterlagen: BauV 2. Abschnitt * Sicherheitsdatenblätter * Gefahrenhinweise auf dem

BETRIEBSANWEISUNG

STRABAG

SEILBAGGER

Hinweis für den aufsichtspflichtigen Bauleiter / Polier

Für die Bedienung des übernommenen Baggers sind nur unterwiesene und bereits mit dem Gerät vertraute Arbeitnehmer heranzuziehen und eine Fahrbewilligung zu erteilen.
Vor der Verwendung eines für den Benützer neuen Gerätes hat dieser die Betriebsanleitung des Herstellers zu lesen und es ist eine gerätebezogene Unterweisung durch eine fachkundige Person zu veranlassen.(Besonderer Augenmerk bei fremdsprachigen Arbeitnehmern.)
Bagger mit vorhandener Überlasteinrichtung (Kranbetrieb) unterliegen bzgl. Abnahme- sowie wiederkehrender Überprüfung den Auflagen eines Autokranes und dürfen auch nur von Arbeitnehmern mit dem gesetzlichen Nachweis dieser Fachkenntnisse bedient werden.
Baustellenbezogene Besonderheiten sind im Zuge der Evaluierung zu erfassen.
Diese Betriebsanweisung ist Bestandteil der Evaluierung und ist dem Fahrer im Zuge der Unterweisung zur Kenntnis zu bringen.

SCHUTZMASSNAHMEN UND VERHALTENSREGELN

Kein Gerät ohne Fahrbewilligung durch den Arbeitgeber in Betrieb nehmen.
Machen Sie sich vor der Aufnahme der Arbeiten mit den Besonderheiten der Baustelle und der Arbeitsumgebung vertraut und informieren Sie sich insbesondere über Bodenbeschaffenheit, Erd und Freileitungen sowie bestehende oder zu erwartende Windgeschwindigkeiten.
Beim Überschreiten der max.zulässigen Windgeschwindigkeit ist die Last abzusetzen und der Bagger in Parkposition zu bringen. Bei zu erwartendem Sturm über 20 m/sec bzw. Windstärke 8 muss der gesamte Ausleger flach auf dem Boden abgelegt werden.
Nehmen Sie nie ein Gerät ohne vorherigen Inspektionsrundgang in Betrieb.
Vergewissern Sie sich, daß alle Hauben und Deckeln geschlossen und alle Warnschilder montiert sind und kontrollieren Sie das Gerät auf augenscheinliche Mängel. Keine beschädigten oder in ihrer Tragfähigkeit unzureichenden Drahtseile oder Ketten verwenden.
Die Bedienungselemente dürfen nur vom Fahrersitz aus betätigt werden. Dulden Sie keine Beifahrer.
Im Gefahrenbereich des Baggers dürfen sich keine Personen aufhalten, der Aufenthalt unter schwebender Last ist verboten.
Bagger nur auf ebenem und festem Boden abstellen. Ausrüstung absetzen oder ablegen, Gerät gegen unbefugte Inbetriebnahme sichern.
Gerät nur bei abgestelltem Motor und ausgeschalteter Heizung tanken. Kein offenes Feuer, nicht rauchen, sich vom Standort des nächsten Feuerlöschers überzeugen.
Auf- und Abbau des Gerätes sowie Verladung nur nach den Angaben des Herstellers unter Beiziehung von diesbezüglich geschultem Personal auf ebenem und festem Untergrund mit ausreichender Bodenbeschaffenheit. Rampenverladung nur mit Einweiser.

WARTUNG und INSTANDHALTUNG

Wartungs- und Reparaturarbeiten dürfen nur von fachkundigen und dazu beauftragten Personen durchgeführt werden.
Für Arbeiten am Hydraulik- und Steuerungssystem sowie an den div. Bremsen sind spezielle Fachkenntnisse Voraussetzung.
Wartungsarbeiten nie an fahrender Maschine oder laufendem Motor vornehmen.
Vorhandene Wartungssperren verwenden.
Vor jeder Arbeit an Hydraulikleitungen diese drucklos machen. Vom Hersteller vorgesehene Wartungsschutzsperren verwenden.

ACHTUNG

Diese Anweisung ist nur ein Auszug der wesentlichsten Schutzmaßnahmen und Betriebsauflagen.
Die lückenlose Einhaltung der Betriebsanleitung des Geräteherstellers ist Voraussetzung für jeden sicheren und wirtschaftlichen Geräteeinsatz.

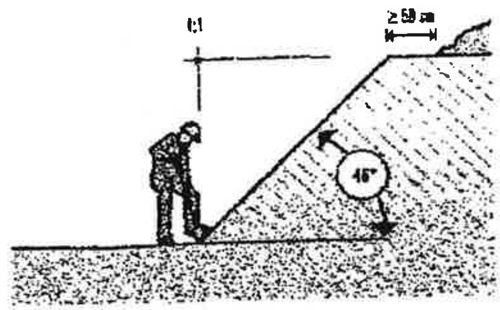
Böschungen

Böschungsneigung

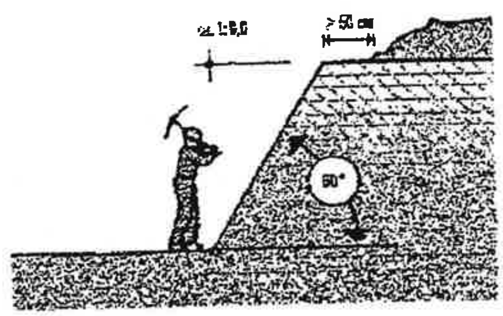
- Die Böschungsneigung richtet sich unter anderem nach
  - der Bodenart,
  - den vorhandenen Auflasten (z. B. Verkehr, Geräte, Aushub, angrenzende Bauwerke),
  - den möglichen Erschütterungen,
  - den Grundwasserverhältnissen,
  - den Witterungsverhältnissen,
  - den geologischen Verhältnissen.

- Ohne rechnerischen Nachweis dürfen die untenstehenden Böschungswinkel nicht überschritten werden.

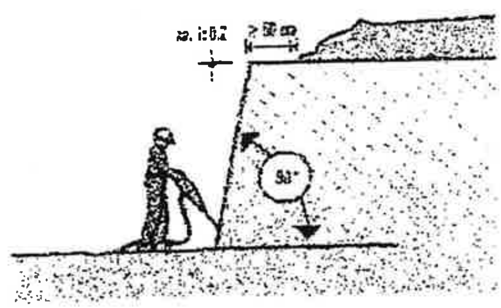
- **Nicht bindiger oder weicher bindiger Boden**
  z. B. Sande, Kiese, Mutterboden



- **Steifer oder halbfester bindiger Boden**
  z. B. Lehm, Mergel, fester Ton, Böden mit festem Zusammenhalt



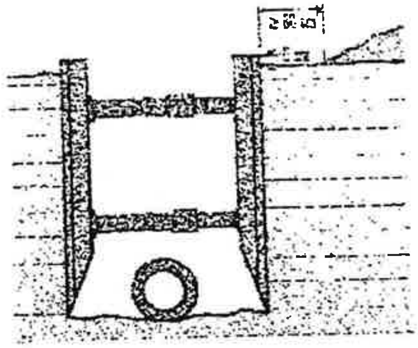
- **Leichter Fels**
  nicht gebräch und nicht verwittert, keine zur Baugrube einfallenden Schichten, ohne Klüfte



- **Schwerer Fels**
  nur durch Sprengen lösbar

Böschungswinkel 90° erlaubt.

Künettenverbau



Allgemeine Forderungen

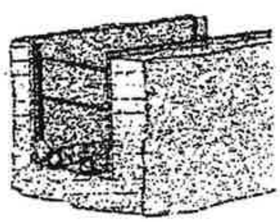
- Senkrechte Künettenwände.
- Beidseitig lastfreier Schutzstreifen mindestens 60 cm.
- Ungesicherte Künettenwände nicht durch Baugeräte und Fahrzeuge belasten.
- Künetten mit ungesicherten Wänden nicht betreten.
- Zufluss von Oberflächenwasser verhindern.
- Sich nicht an ungesicherten Künettenwänden aufhalten (weder oben noch unten).

D 2

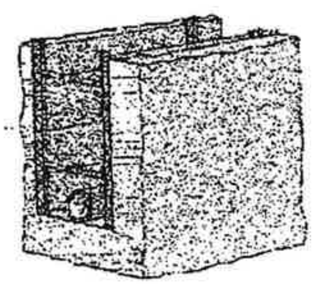
Der Verbau

- Der Verbau muss für die anstehende Bodenart geeignet sein.
- Er muss die auftretende Erddruckbelastung aufnehmen können.
- Er muss nach der ungünstigsten Beanspruchung bemessen werden.
- Er muss in allen Bauzuständen (Einbau und Rückbau) standsicher sein.
- Er muss ausreichend dicht sein und von der Künettensohle bis mindestens 5 cm über die Geländeoberkante reichen.
- Er muss ganzflächig am Erdreich anliegen und einwandfrei hinterfüllt sein (keine Hohlräume).
- Die Künette muss über Leitern o. Ä. begangen werden können.

Verbaugeräte (Beispiele)



Randgestützte Verbaugeräte



Gleitschienen-Verbaugeräte

Voraussetzungen für den Einsatz von Verbaugeräten

- Verwendungsanleitung des Herstellers beachten (Tragfähigkeit bei verschiedenen Künettenbreiten und -tiefen, Montage).
- Belastung ermitteln, z. B. aus Erddruck oder Gebäuden, und mit der Belastbarkeit des Verbaugerätes vergleichen.
- Keine ausfließenden Böden.

Appendix C – Project Risk Profile

Description of Risk	Description of Consequence	Risk Before Mitigation	Mitigation Activity	Risk After Mitigation
Cost				
The contractor may encounter subsurface conditions that are more adverse than described in the Geotechnical Baseline Report (GBR)	Unexpected, adverse subsurface conditions could slow tunnel construction and require the contractor to undertake remedial / extra work resulting in legitimate claims for extra costs and / or schedule extension for differing subsurface conditions (DSC).	High	• The GBR is based on extensive field investigations carried out over a 10-year period and knowledge gained through construction of the existing SAB2 tunnels. • The 3-stage GBR process used facilitates contractor input and concurrence before construction begins. • Residual tunnel construction risk to OPG is addressed by a contingency allowance of $96 M in the project release estimate and a contingency allowance of 8 months in the scheduled in-service date, both based on a 90% confidence level.	Medium
Insurance coverage is inadequate or unavailable because underground construction has developed a reputation for cost over-runs and a negative perception from insurers.	Establishing an Owner Controlled Insurance Program (OCIP) to mitigate insurable risks for OPG, the Owner's Representative, the contractor and affected third parties.	Medium	• Engagement of key underwriters through project presentations. • Following, in principle, the UK Code of Practice for Risk Management of Tunnel Works. • A conservative estimate for insurance costs is included in the release estimate.	Low
The design / build contractor may not complete the tunnel due to non-performance or default.	OPG would need to engage another contractor to complete the tunnel construction.	Medium	• Requirements in the design / build contract for the contractor to provide bonds and / or letters of credit as security for non-performance or default. • Requirements in the design / build contract for the contractor to provide a parental guarantee.	Low

28

shall be recorded. The resolution of any disagreements will be held in abeyance ..., unless the parties mutually agree that the issue is sufficiently material that the issue should be referred to dispute resolution in which event the matter be resolved in accordance with Section 11;..."

Section 5.5 (e) states "No request by the Contractor for relief for differing subsurface conditions will be allowed in respect of Work under the St. Davids Gorge to the extent that the width of the gorge is within the width defined in the GBR."

2.4 Contract

2.4.1 Design Build

Tunnels in North America have traditionally been constmcted using Design-Bid-Build contracts, in which the Contractor has no involvement in preparing the contract documents, including the GBR. All bidders tender to the identical contract provisions, GBR conditions and design.

Design-Build (DB) contracting is becoming a more frequently used form of contract on large, challenging construction projects primarily to reduce the pre-bid time spent on design efforts and equipment procurement, thereby facilitating earlier completion. DB is used on this Project and four main parties are involved: the Owner, the Owner's Representative (OR), the Contractor, and the Designer, ILF Consulting Engineers, of Austria, who is retained by the Contractor. The three contractors that proposed for this Work and their designers prepared preliminary designs, design basis and methods statements, specifications, drawings and payment provisions in general accordance with the Owner's bidding requirements, mandatory requirements and conceptual design. However, after evaluating the conceptual tunnel design, Strabag proposed a different lining design that required a different type ofTBM. This was accepted by the Owner and is being used to construct the tunnel.

On this contract the Owner's team prepared an initial GBR, called a GBR-A. Each proposal included a GBR-B, in which the tenderers supplemented and revised GBR-A, to be consistent with the bidder's proposed design approach and planned means and methods of constmction. The GBR-C was negotiated with the selected tenderer and became the contractually binding GBR.

The Contractor is responsible for design and constmction of the Work. The Owner is responsible for more adverse subsurface conditions than are represented in the GBR. The Owner **and** the Contractor are **jointly** responsible for preparation of the GBR.

2.4.2 Contractor's Proposal

The Contractor proposed a prestressed tunnel lining method, and listed nine hydroelectric tunnels where the method had been used between 1963 and 1988. This lining approach was judged by the Owner's team to be significantly superior, for the unique requirements of the Niagara project, to the methods proposed by the other two tenderers, each of which involved a fully-shielded TBM with a single pass, pre-cast segmental lining. The price and duration of the Strabag proposal, as negotiated, were acceptable. Therefore the Owner contracted with this Contractor to do the Work.

As the DRB understands it, Strabag was not the low bidder and acknowledged in their proposal that using a shielded TBM with a pre-cast segmental liner would make constmction easier. However, Strabag considered a segmental liner too umeliable, under the unique site conditions, to meet the required service life of90-years without unwatering the tunnel for repairs.

Ontario. This project was referred to as the Niagara River Hydroelectric Development ("NRHD").

Among the commitments made through the EA process was to utilize a TBM to excavate the tunnels starting from the outlet end, proceeding under the buried St. Davids Gorge and following the route of the existing SAB 2 tunnels through the City of Niagara Falls. A TBM was required in light of the development that had occurred in Niagara Falls since the original two diversion tunnels were constructed using the drill and blast method in the 1950s, and to minimize the amount of excavated materials from the project requiring disposal. Other commitments included re-use of excavated materials where feasible and an agreement to compensate the host municipalities, the Regional Municipality of Niagara, City of Niagara Falls and Town of Niagara-on-the-Lake, for forecasted project impacts on tourism, roads and municipal services.

2.2.2 1998 Decision to Pursue Third Tunnel

Early in February 1998, in anticipation of receiving EA approval, Ontario Hydro initiated a review of the viability of proceeding with the first phase of the NRHD (i.e., the construction of one additional 500 m^3/s tunnel). This review included the solicitation and evaluation of bids for the construction of the tunnel during the summer and fall of 1998 using a design-build approach.

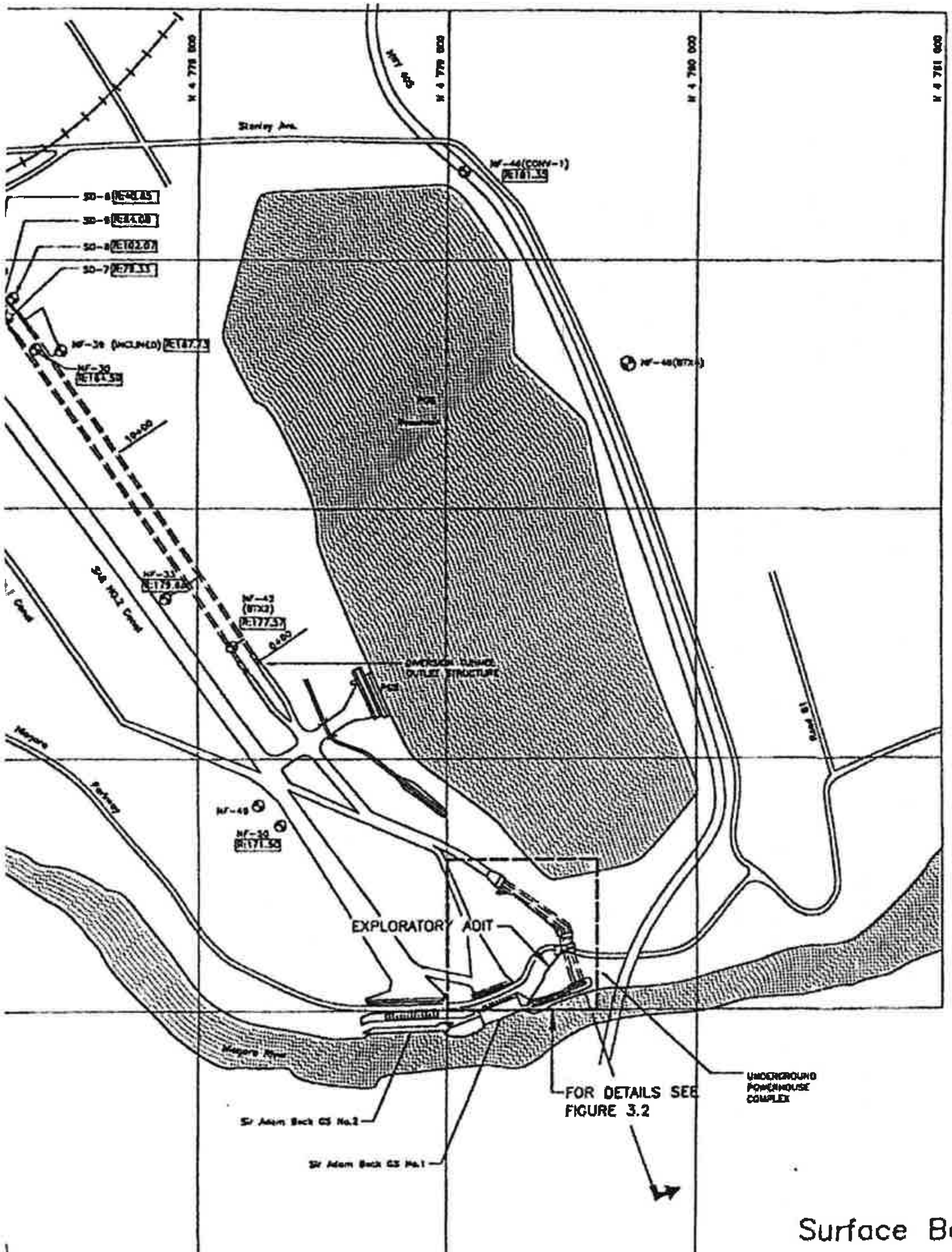
In October 1998, the Minister of Environment provided approval under the *Environmental Assessment Act* for the complete NRHD as outlined above. The EA approval stipulated that it would "terminate if construction has not commenced within ten years from the date of this approval." This stipulation could be extended a further five years "based on the review and approval of an environmental review assessment status report." It provided Ontario Hydro with the flexibility to undertake the development in phases (i.e., initial construction of one tunnel); but did require that no construction extend "beyond twenty years following the commencement of construction."

13.2 Appendix B – Summary of Geologic Investigations

Beginning in 1983, extensive geotechnical investigations were undertaken during concept and definition phases for the expansion of OPG's Niagara hydroelectric facilities, which at that time contemplated two additional tunnels and a new underground generating station ("Beck 3"). These investigations were heavily focused on the Queenston shale formation because drilling in this formation was required by the plans to excavate the new tunnels under the existing Sir Adam Beck No. 2 tunnels with sufficient separation to allow the use of the existing rights of way (i.e., tunnel at greater depth in the same corridor). Because the plan also involved tunneling under the buried St. Davids Gorge (to reduce excavated material disposal relative to an open canal) and constructing the planned underground powerhouse, the investigations also focused on the buried St. Davids Gorge area and the planned powerhouse area.

As indicated in Table 1 below, the geotechnical investigations were carried out in stages and included a total of 59 boreholes and a geotechnical test adit (small test tunnel). Rock cores were retrieved from the boreholes to determine physical and engineering properties (chemical composition, strength, in-situ stress, joints, swelling potential, etc.). This investigation work involved internal staff, experienced engineering consultants (i.e., Acres, Golder), geotechnical engineering faculty from the University of Western Ontario, University of Toronto, Laurentian University, University of Michigan, and other international geotechnical engineering and construction experts from universities in Florida and Germany who participated through technical review panels (see Table 2 below).

Twenty of the 59 boreholes were along the 10 kilometre tunnel route with the remainder in the area of the proposed powerhouses, along other potential tunnel alignments and around the Pump Generating Station reservoir. Besides core retrieval for testing, in-situ stress measurements were conducted in some boreholes to assess the magnitude and orientation of the horizontal stress regime. Piezometers were also installed in many of the boreholes to assess groundwater conditions.



LEGEND

- NF-38 R:164.59 — BOREHOLE – 1992 HOLES – WITH ROCK SURFACE ELEVATION
- BOREHOLE – INCLINED
- NF-9 R:69.53 — BOREHOLE – PREVIOUS INVESTIGATIONS WITH ROCK SURFACE ELEVATION
- PROPOSED TUNNEL
- EXISTING TUNNEL

MAGNETIC NORTH
TRUE NORTH

NOTES

1. GRID IS (6° UTM) NORTH AMERICAN DATUM

1:20000 0 500 1000 m

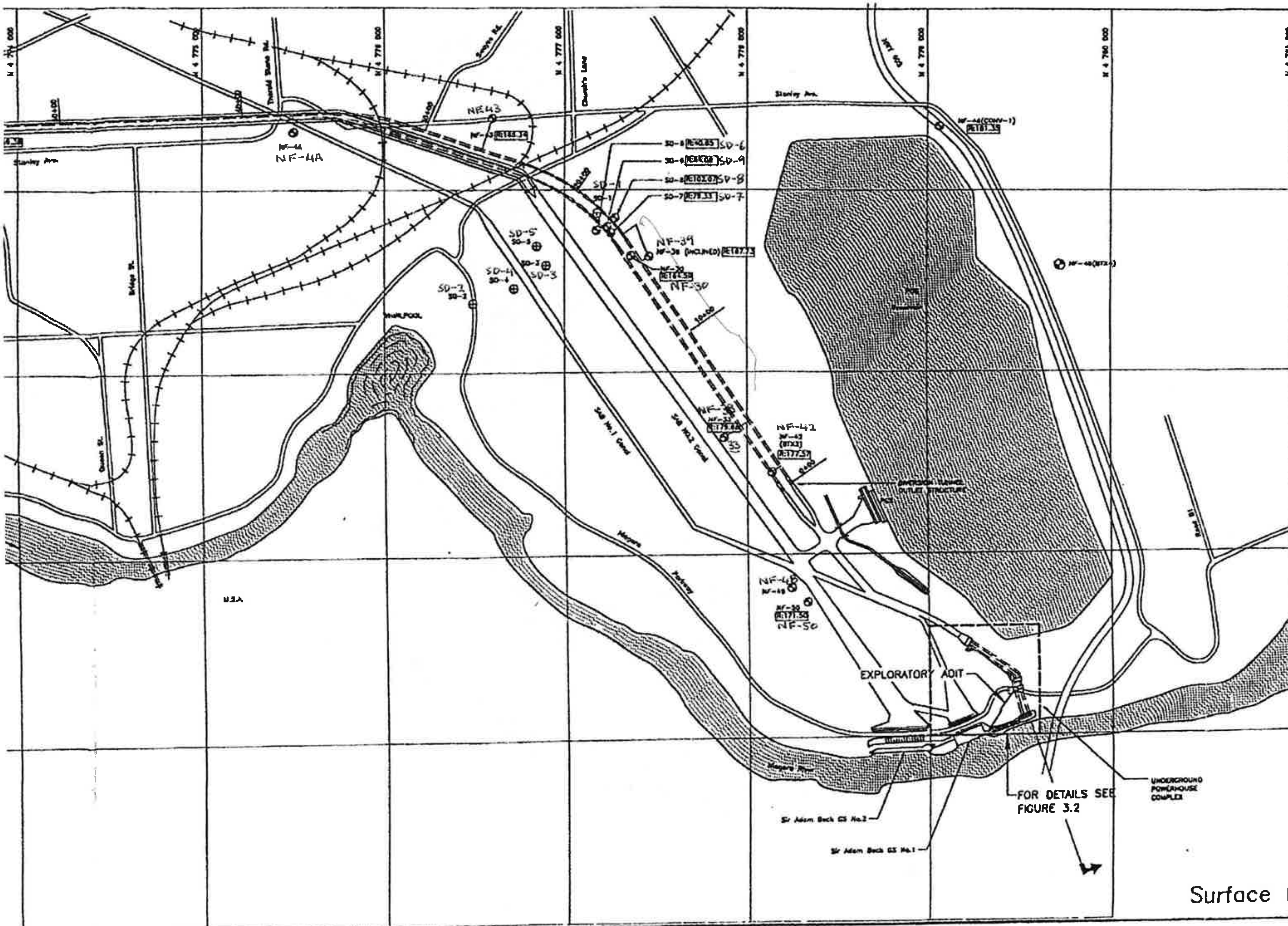
Figure 1.1

Ontario Hydro
NRHO – Definition Engineering Phase 2
Additional Geotechnical Investigations

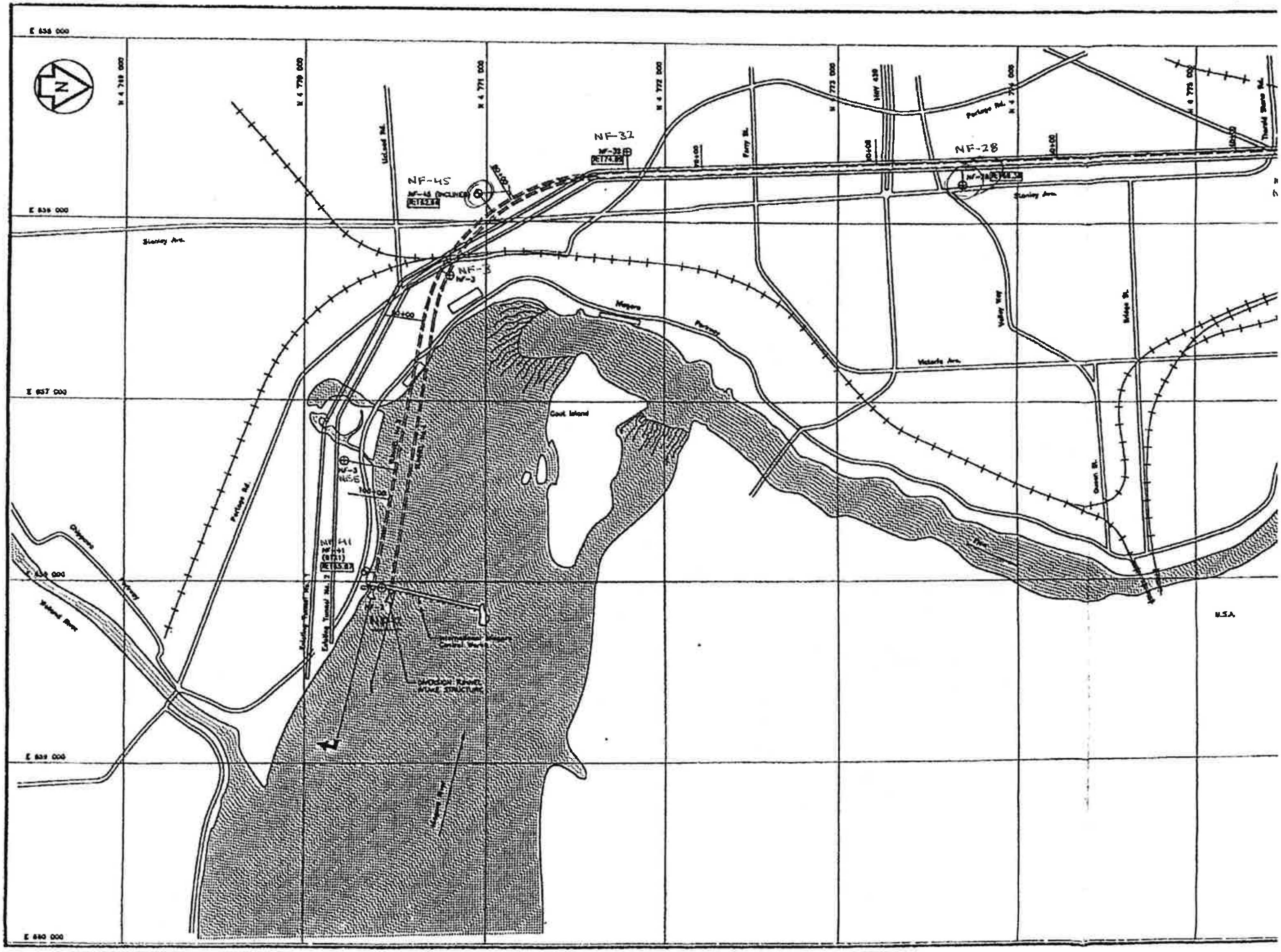
Surface Borehole Locations – Diversion Facilities

Acres Bechtel Canada

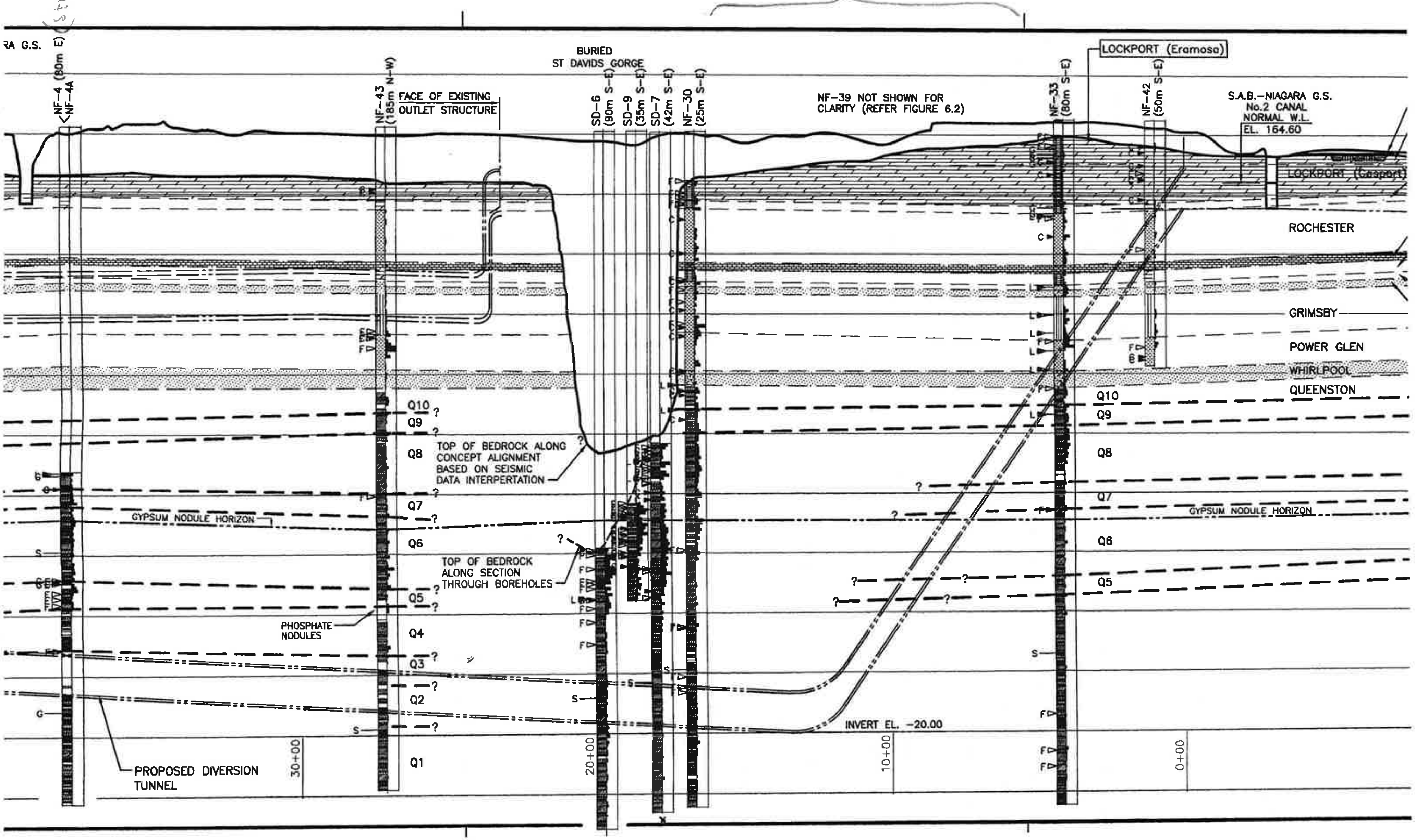
32


EXPLORATORY ADIT
FOR DETAILS SEE FIGURE 3.2
UNDERGROUND POWERHOUSE COMPLEX
Sir Adam Beck GS No.2
Sir Adam Beck GS No.1
Stanley Ave.
Niagara River
Niagara Parkway
WHIRLPOOL
U.S.A.
SAB No.1 Canal
SAB No.2 Canal
Bridge St.
Queen St.
Church's Lane
Thorold Stone Rd.
NF-43
NF-4A
SD-6
SD-9
SD-8
SD-7
SD-1
NF-39
NF-30
NF-33
NF-42
NF-48
NF-50
SD-5
SD-3
SD-4
SD-2

Surface Bor



N
NF-45
NF-32
NF-28
NF-3
NF-41
NSG
Stanley Ave.
McLeod Rd.
Portage Rd.
Ferry St.
Hwy 420
Victoria Ave.
Valley Way
Bridge St.
Queen St.
Niagara Parkway
Goat Island
U.S.A.



RA G.S.
NF-4 (80m E)
NF-4A
(stress)
NF-43 (185m N-W)
FACE OF EXISTING OUTLET STRUCTURE
BURIED ST DAVIDS GORGE
SD-6 (90m S-E)
SD-9 (35m S-E)
SD-7 (42m S-E)
NF-30 (25m S-E)
NF-39 NOT SHOWN FOR CLARITY (REFER FIGURE 6.2)
NF-33 (80m S-E)
LOCKPORT (Eramosa)
NF-42 (50m S-E)
S.A.B.-NIAGARA G.S. No.2 CANAL NORMAL W.L. EL. 164.60
LOCKPORT (Gasport)
ROCHESTER
GRIMSBY
POWER GLEN
WHIRLPOOL
QUEENSTON
Q10
Q9
Q8
Q7
Q6
Q5
Q4
Q3
Q2
Q1
TOP OF BEDROCK ALONG CONCEPT ALIGNMENT BASED ON SEISMIC DATA INTERPERTATION
GYPSUM NODULE HORIZON
TOP OF BEDROCK ALONG SECTION THROUGH BOREHOLES
PHOSPHATE NODULES
PROPOSED DIVERSION TUNNEL
INVERT EL. -20.00
30+00
20+00
10+00
0+00

35

At 14.4 metres in diameter, the Niagara Tunnel is precedent setting for excavation by an open full-face tunnel boring machine in rock. Rock is not a uniform material and subsurface conditions can vary considerably over a short distance. Despite extensive investigations, rock behaviour during tunneling cannot be precisely predicted from boreholes and adits that provide representative data for only a small percentage of the rock to be excavated. Consequently, tunnel designs are based on experience and interpretation of the geotechnical parameters. Actual rock conditions and its behaviour during tunnel construction cannot be fully known before the excavation is complete. Sub-surface conditions always remain a significant risk to both design and construction of tunneling projects.

Table 1 - Work Completed During Various Stages of Geotechnical Investigations

Stage / Work Completed	Timeline
Concept Phase • Drilled 5 boreholes (SD-1 to SD-5) in buried St. Davids Gorge • Drilled 25 boreholes (NF-1 to NF-26, excluding NF-16 – was not drilled) along potential tunnel alignments, surface and underground powerhouse locations and around the PGS reservoir	1983 - 1989
Definition Engineering Phase 1 • Drilled 16 boreholes. Five in the Diversion Facilities area (NF-4A, NF-28, NF-30, NF-32 and NF-33), four in the St. Davids Gorge area (SD-6 to SD-9), and seven in the Generation Facilities area (NF-27, NF-29, NF-31, NF-34 to NF-37)	1990
Definition Engineering Phase 2 • Drilled 13 boreholes (NF-38 to NF-50) • Exploratory adit program	1992-1993

01-2-1 Attach 5 (BOD MEMO)

3. BACKGROUND & ISSUES

Background

- The Sir Adam Beck (SAB) hydroelectric complex at Niagara consists of two generating stations (SAB1 and SAB2), and a pumping / generating station (SAB PGS). SAB1 and SAB2 have a total generating capacity of 1,960 MW. SAB PGS has a capacity of 174 MW and is generally utilized to pump / store water during off-peak periods for use during periods of peak electricity demand. The SAB complex currently produces average annual energy output of approximately 12 TWh.

- The Niagara Tunnel development is a unique, site-specific opportunity for OPG to produce additional, low-cost, renewable and environmentally sustainable energy for its customers, enhancing the existing Sir Adam Beck – Niagara hydroelectric facilities in the efficient use of Niagara River flow available to Canada for power generation with a resultant 14% increase in average annual energy output.

- The Canadian streamflow share of the Niagara River has been calculated as ranging from about 600 to 3000 m^3/s, averages about 2000 m^3/s and exceeds the capacity of the existing SAB diversion facilities (canal and two tunnels) about 65% of the time.

- Feasibility studies for expansion of Ontario Hydro's hydroelectric facilities at Niagara commenced in 1982. Definition phase engineering and environmental assessment work started in 1988 and was suspended in 1993. The Environmental Assessment (EA) was submitted in March 1991 and approval was obtained on October 14, 1998.

- The Environmental Assessment (EA) approved the Niagara River Hydroelectric Development consisting of two new tunnels, an underground powerhouse and transmission improvements in the Niagara Peninsula. The EA approval provided Ontario Hydro with the flexibility to undertake the development in phases. A plan to proceed with only one tunnel was initiated in 1998, and tenders were called for detailed design and construction, but work was suspended in 1999 due to uncertain market conditions and imminent corporate reorganization. Expenditures in 1998/99 totalled $2.5 M and are included in the estimated total project cost. Earlier definition phase expenditures of $57 M on the Niagara River Hydroelectric Development were written off by Ontario Hydro.

- In November 2002, the Province announced that it had directed OPG to proceed with a new water diversion tunnel at Niagara and subsequently indicated a strong desire to have the project completed in the shortest possible timeframe.

- The timing for completion of the new tunnel is also linked to the required rehabilitation of the 83-year old SAB1 canal, which delivers over one third of the water used at the SAB complex. The canal rehabilitation work is expected to start in 2011 and will require taking the canal out of service for approximately 8-12 months. Having the new tunnel in place will avoid an energy generation loss of 2.7 to 4.0 TWh caused by the canal outage (depending on available Niagara River flow and outage duration).

- On June 24, 2004, the OPG Board of Directors approved a preliminary release of $10 M to conduct a Request For Proposal process and to carry out such preconstruction activities as OPG deems necessary. Commitments for this work, to the end of June 2005, total $8.7 M.

- Provisions of an agreement between the Niagara Parks Commission (NPC) and OPG, dated February 18, 2005 (which agreement forms part of the larger Niagara Exchange transaction concerning the long term disposition of water rights on the Niagara River), committed OPG to undertake remedial work at the retired Ontario Power and Toronto Power generating stations as

In 1983 a single borehole (SD-1) was drilled into Queenston bedrock sufficiently to define top of rock. In 1988/89 four vertical holes (SD-2 to SD-5) were drilled east of the alignment to the top of rock to define the deepest part of the Gorge. A Gravity Survey was also done to attempt to define the bedrock surface and gave indications of the deepest part of the Gorge. In addition a seismic reflection survey was completed but was ineffective as the energy source was too low.

A second seismic survey was done in 1988 which gave insufficient definition resulting in a third survey in 1989 using explosives as the energy source. Based on the seismic and borehole data an inferred bedrock surface plan was produced along with several profiles.

3.2.3 In Situ Stress Measurements

The identification of the stress magnitude and direction was an important objective due to the resulting high stresses that develop around the tunnel periphery during excavation.

In 1983 in situ stress measurements were made in Borehole NF-1 using overcoring methods, located at the SAB GS 1 access shaft. Although not on the tunnel alignment all in situ stress measurements were useful in an attempt to gain an overall picture of both magnitude and direction of the principal stresses; especially because of the inferred effects of the Niagara River Gorge and St. David's Gorge on these parameters. In 1983/84 hydro-fracturing stress measurements were made in boreholes NF-3 and NF-4. In 1988 a single piezometer was placed in the Queenston in boring SD-3.

3.2.4 Laboratory Testing of Rock Core Samples

In order to conduct appropriate analyses for the design, rock material parameters were provided from a comprehensive laboratory testing program of the rock core recovered from the boreholes.

In 1983 samples from the Whirlpool and Queenston Formations were tested. Values were measured for the following parameters; uniaxial compressive strength (UCS); static elastic modulus; Poisson's Ratio; compressive wave velocity, dynamic elastic modulus, water content; density; free swell rate and calcite content.

3.3 Definition Engineering Phase 1

Phase 1 site investigations related to the Diversion Tunnel were carried out in 1990 and included drilling boreholes with core recovery for laboratory testing, a geophysical program, and in-situ stress measurement.

Phase 2 consisted primarily of the excavation of an Exploratory Adit (Adit) located in the area of the power generation complex; also additional borings were completed as well as some additional long term swell tests.

The objectives of the program were as follows:

- Further definition of the bedrock surface location in the Gorge;
- Additional in-situ stress measurements, especially the Queenston;
- Further definition of the lateral and vertical variations in the Queenston along the tunnel alignment; and
- Investigation of potential for inflows of groundwater and methane gas.

The results of the Phase 1 investigations were presented in Report No. 91150 consisting of five volumes issued in May 1991. The results of the Adit related investigations were issued as Definition Engineering Phase 2 Geotechnical Investigations and Evaluation in seven volumes in December 1993 (Report NAW130-P4D-10120-0005-00).

A review of the investigative reports indicates that the rock characterization along the alignment, better definition of the bottom of the St. David's Gorge, measurement of the in-situ stresses, definition of the groundwater regime and groundwater quality analysis and measurement of rock material parameters, were accomplished in regard to the three principal areas (see section 3.1.1, 2, and 3 above) of design issues for the tunnel.

3.3.1 Drilling Along Tunnel Alignment

The following five vertical borings to the tunnel level were done in Phase 1: NF-4A, NF-28, NF-30, NF-32 and NF-33; also four borings at the Gorge of which SD-7 and SD-8 penetrated to the tunnel level and SD-5 and SD- 6 ended at the top of rock. In Phase 2 the following borings were done: existing borehole NF-31 was extended from el. 41 m to

1.3 Preliminary Design and Construction Considerations for the Diversion Tunnel

1.3.1 Diversion Tunnel Alignment

The Proposal design follows the concept alignment in principle. Only below the buried St' David's Gorge, the alignment is slightly relocated to the north-west to gain maximum rock cover, which is predicted close to the location of geotechnical borehole SD-8. Horizontal and vertical curvature is arranged such to maintain a min. 1000 m radius for to facilitate muck transportation by conveyor belt systems. In addition the alignment close to the existing outlet structure is moved away from underneath the existing Delivery Tunnel No. 1, to facilitate the drilling of the borehole for tunnel piezometers.

The overall depth of the tunnel has been slightly reduced as compared to the concept design. The inclination of fall and raise of the grade near the outlet and intake of the facility is arranged slightly shallower as in the concept design. The dewatering structure has also been moved further away from the buried St' David's gorge as compared to the concept design. A potential fourth Diversion Tunnel may be arranged in parallel to the proposed alignment route.

1.3.2 Diversion Tunnel Lining

Originally two lining alternatives for the Diversion Tunnel have been investigated by the Proponent:

- Single shell lining with precast concrete segments
- Double shell lining with an initial lining of shotcrete, ribs and rock bolts and a final lining of cast in place concrete. Both linings being separated by a waterproofing membrane system.

Although easier to apply in combination with a Tunnel Boring Machine (TBM), the single shell lining alternative has been abandoned for the following reasons:

- The surface roughness of precast segments (K_s = 75 – 80) is inferior to cast in place concrete (K_s = 85 – 90) according to Strickler (see chapter 2.3).
- Although compressible annular grouting mortar is available to compensate deformations resulting from rock squeeze, it is not possible to hold the operational internal water pressure in segmental lining rings. Water could escape through segment joints at pressures up to 14 bar and could adversely affect the rock of formations, which are sensitive to water. High swelling pressures or even worse, erosion of ground around the tunnel would be the undesirable result.
- It cannot be guaranteed, that uniform grouting of the annulus around the segmental lining ring is achieved, since rocks falling from behind the shield of the TBM into the

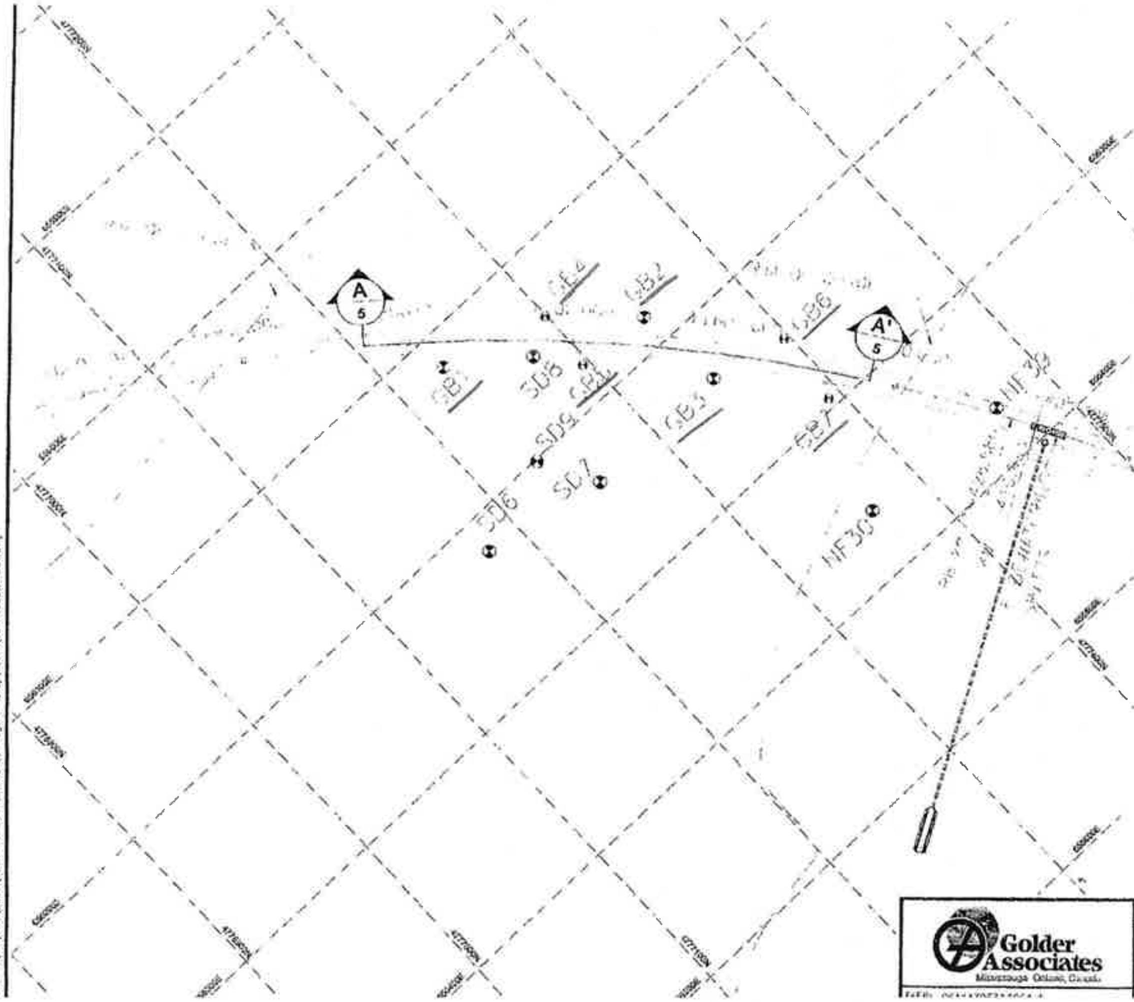
40

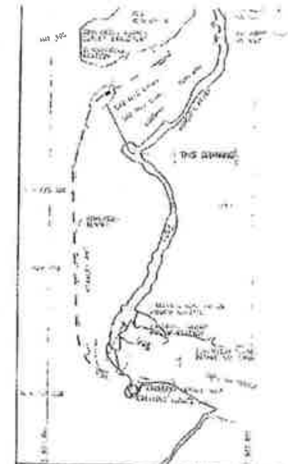
The geotechnical adit was originally 580 metres long and three metres in diameter. It was subsequently enlarged on a trial basis to 12 metres in diameter over its last 30 metres. The adit was excavated at the Sir Adam Beck complex by Thyssen Mining Corporation of Canada Ltd (subcontractors to Acres Bechtel Canada). Excavation occurred between August 1992 and July 1993 (see Figure 1 below). The adit was tested and observed as part of the investigation program, and monitoring continued through March 1994.

Construction of a geotechnical adit is not typically done for tunnel projects because of the associated time and cost. The trial enlargement was specifically designed and constructed to simulate the excavation of the planned diversion tunnels in the Queenston shale formation using a full-face tunnel boring machine. In consultation with engaged experts on the Specialist Consulting Board, the adit helped OPG conclude that rapid, full-face tunnel excavation in the Queenston shale formation on the planned scale was technically feasible and cost-effective.

The relevant geotechnical parameters were summarized in the draft Geotechnical Baseline Report ("GBR") and included in OPG's Design Build Request for Proposal documents. The contractor, Strabag, refined the GBR to incorporate its interpretation of the data and rock behaviour expected relative to its planned means and methods of construction. The collaboratively negotiated 3-stage GBR was included in the Design Build Agreement as the agreed baseline for expected geotechnical conditions.

After contract award, Strabag drilled seven additional boreholes to verify the rock conditions in the vicinity of the buried St. Davids Gorge. These boreholes confirmed that the Queenston shale was intact and that Strabag's proposed alignment (which was higher than the concept alignment in the RFP) was feasible.





KEY PLAN
SCALE 1 : 100000

LEGEND:

- AS BUILT LOCATION OF BOREHOLE
- EXISTING LOCATION OF BOREHOLE

NOTES:

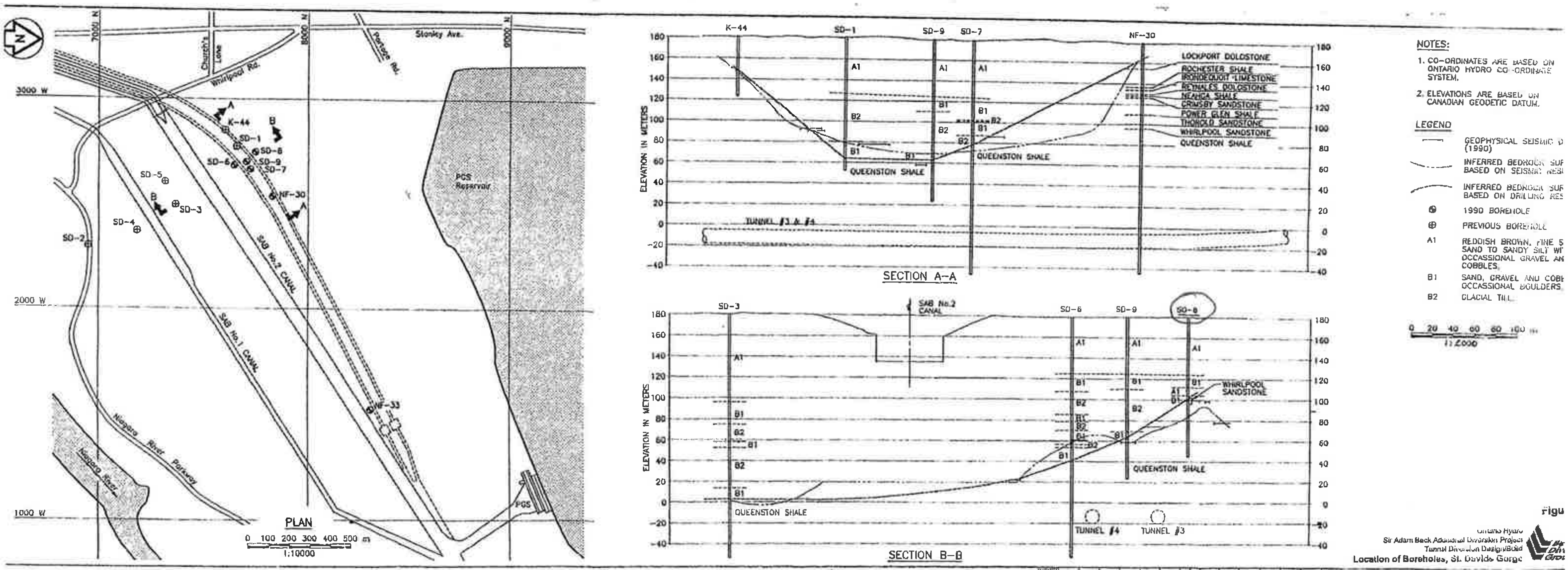
1. DATUM IS NAD 83

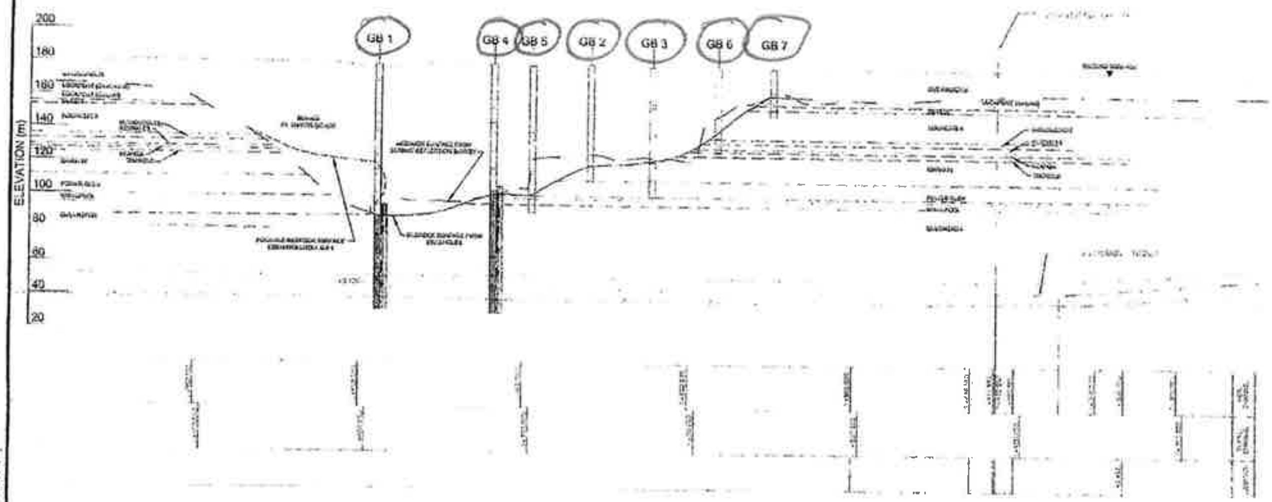
REFERENCES:

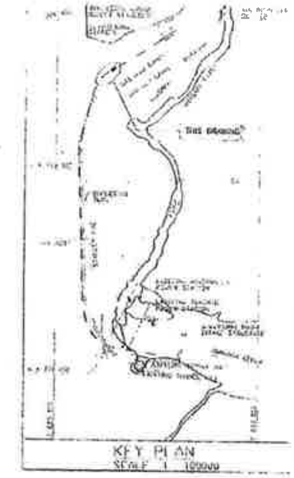
1. MAPPING BASED ON DRAWING PROVIDED BY STRABAG DATED OCTOBER 11, 2005

Golder Associates, Mississauga, Ontario, Canada	SCALE: AS SHOWN DATE: MAR. 2006 DESIGN: CAD: JFC	TITLE: **SUMMARY OF BOREHOLE LOCATIONS**

DATE March 07, 2006
NAME T:\Projects\2005\05-1117-053 (STRABAG, Niagara Tunnel)\-M-\05117053A601.dwg


PLAN
0 100 200 300 400 500 m
1:10000
Stonley Ave.
Portage Rd.
Church's Lane
Whirlpool Rd.
7000 N
8000 N
9000 N
3000 W
2000 W
1000 W
K-44
SD-1
SD-6
SD-8
SD-9
SD-7
NF-30
NF-33
SD-5
SD-3
SD-4
SD-2
SAB No.2 CANAL
SAB No.1 CANAL
Niagara River Parkway
Niagara River
PGS Reservoir
PGS
SECTION A-A
ELEVATION IN METERS
LOCKPORT DOLOSTONE
ROCHESTER SHALE
IRONDEQUOIT LIMESTONE
REYNALES DOLOSTONE
NEAHGA SHALE
GRIMSBY SANDSTONE
POWER GLEN SHALE
THOROLD SANDSTONE
WHIRLPOOL SANDSTONE
QUEENSTON SHALE
TUNNEL #3 & #4
SECTION B-B
SAB No.2 CANAL
TUNNEL #4
TUNNEL #3
NOTES:
1. CO-ORDINATES ARE BASED ON ONTARIO HYDRO CO-ORDINATE SYSTEM.
2. ELEVATIONS ARE BASED ON CANADIAN GEODETIC DATUM.
LEGEND
GEOPHYSICAL SEISMIC (1990)
INFERRED BEDROCK SUR BASED ON SEISMIC RES
INFERRED BEDROCK SUR BASED ON DRILLING RES
1990 BOREHOLE
PREVIOUS BOREHOLE
A1 REDDISH BROWN, FINE S SAND TO SANDY SILT WI OCCASSIONAL GRAVEL AN COBBLES.
B1 SAND, GRAVEL AND COB OCCASSIONAL BOULDERS.
B2 GLACIAL TILL.
0 20 40 60 80 100
1:2000
Figu
Ontario Hydro
Sir Adam Beck Additional Diversion Project
Tunnel Diversion Design/Build
Location of Boreholes, St. Davids Gorge





A-A'
1

CROSS SECTION

50 0 50 100

SCALE 1:2500 METRES

NOTES:

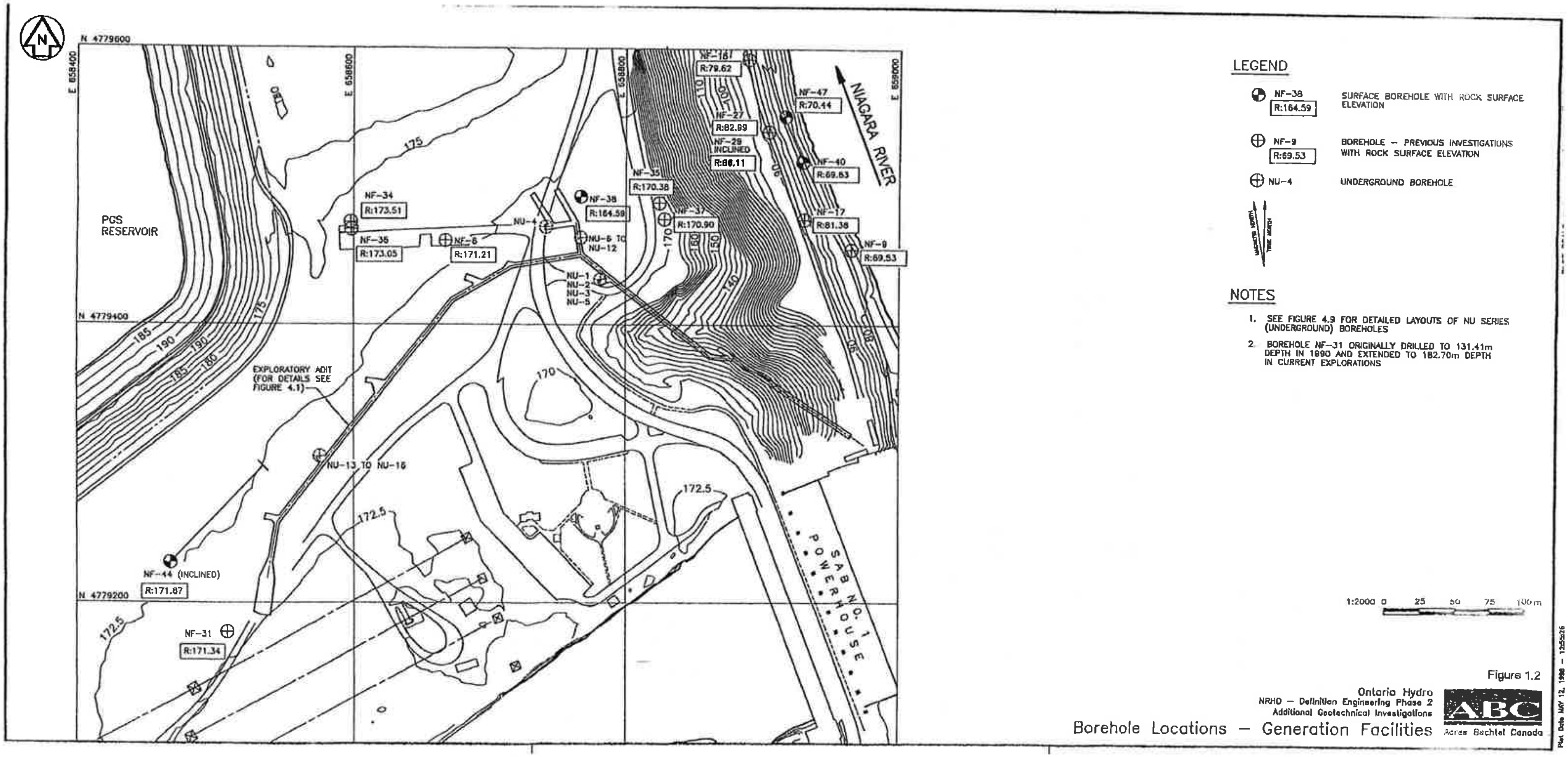
1 DATUM IS NAD 83

REFERENCES:

1 TUNNEL ALIGNMENT BASED ON DRAWING BD 01 1164 PROVIDED BY STRABAG DATED OCTOBER 11, 2005

Golder Associates Mississauga, Ontario, Canada	SCALE	AS SHOWN	CROSS-SECTION SHOWING BEDROCK ELEVATIONS FOR THE BURIED ST. DAVID'S GORGE
	DATE	MAR. 2006	
	DESIGN		
	CAD	JFC	
FILE: 051117057AA005.dwg	CHECK	JC	NIAGARA TUNNEL FACILITY PROJECT

ADIT


N
N 4779600
N 4779400
N 4779200
E 658400
E 658600
E 658800
E 659000
PGS RESERVOIR
NIAGARA RIVER
NF-34
R:173.51
NF-36
R:173.05
NF-6
R:171.21
NU-4
NF-38
R:164.59
NU-6 TO NU-12
NU-1
NU-2
NU-3
NU-5
NF-35
R:170.38
NF-37
R:170.90
NF-16
R:79.62
NF-27
R:82.99
NF-29 INCLINED
R:86.11
NF-47
R:70.44
NF-40
R:69.63
NF-17
R:81.36
NF-9
R:69.53
EXPLORATORY ADIT (FOR DETAILS SEE FIGURE 4.1)
NU-13 TO NU-16
NF-44 (INCLINED)
R:171.87
NF-31
R:171.34
SAB NO. 1 POWERHOUSE
175
170
172.5
185
190
180
160
150
140
110
100
90
80
LEGEND
NF-38
R:164.59
SURFACE BOREHOLE WITH ROCK SURFACE ELEVATION
NF-9
R:69.53
BOREHOLE – PREVIOUS INVESTIGATIONS WITH ROCK SURFACE ELEVATION
NU-4
UNDERGROUND BOREHOLE
MAGNETIC NORTH
TRUE NORTH
NOTES
1. SEE FIGURE 4.9 FOR DETAILED LAYOUTS OF NU SERIES (UNDERGROUND) BOREHOLES
2. BOREHOLE NF-31 ORIGINALLY DRILLED TO 131.41m DEPTH IN 1990 AND EXTENDED TO 182.70m DEPTH IN CURRENT EXPLORATIONS
1:2000 0 25 50 75 100m
Figure 1.2
Ontario Hydro
NRHD – Definition Engineering Phase 2
Additional Geotechnical Investigations
Borehole Locations – Generation Facilities
ABC
Acres Bechtel Canada
Plot Date MAY 12, 1998 – 1255026

45

theses buckets before it can be supported by the TBM roof shield. Even with stress induced fractures, such a condition may not have been anticipated if the rock was believed to be "generally **massive"**.

In the DRB's opinion, the Contractor's original plan to use steel ribs as a regular means of initial support in the QF suggests that it anticipated the rock to be "generally massive" with reasonably good stand up time throughout much of the QF formation. Under such a scenario, the need for full circle steel ribs to resist sidewall spalling and invert heave would make sense, while feeling that stress induced fracturing in a "generally massive" rock would not produce serious crown stability problems or loosening of crown rock to a degree that would raise concern over performance of the final liner under high interface grouting pressures.

It appears to the Board that there was a serious misunderstanding between the Parties with respect to the anticipated rock conditions and rock behavior at the time the contract GBR was being negotiated. Since both Parties developed the GBR jointly, any misunderstanding is the shared responsibility of both Parties.

3.5 Geotechnical Baseline Report

It is noteworthy that Appendix 5.4- Geotechnical Baseline Report states in item 1.4 that "the GBR will be used during the execution of the Contract for comparison of the *assumed subswface conditions with actual subsurface conditions* as encountered during construction." The wording contained in this Appendix 5.4 is consistent with the usual concept of a GBR on a Design-Bid-Build project.

Section 5.4 of the DBA, however, states the GBR "describes *anticipated behaviors and conditions that are dependent on the Contractor's selected designs, means. methods ....anticipated or implied* at the date of this Agreement." The wording in the DBA expands and complicates the GBR concept and purpose by (1) changing *"assumed"* to *"anticipated"* or *"implied"* and (2) by including *"behaviors and conditions that are dependent on the Contractor's selected designs, means, methods ...* ", both of which require a mutual understanding between the Parties. The DRB assumes the objective of these modifications is to avoid DSCs based on subsurface conditions set by one party to the contract. This may seem achievable, especially when the GBR is ')ointly developed" by the Owner and Contractor. However, neither Party is likely to anticipate all of the conditions and behaviours that will be encountered and would influence the performance of the Work, let alone have a clear mutual understanding of those conditions and behaviours. In the Board's opinion, the wording in the DBA makes the application of the GBR concept much more complex and increases the likelihood of misunderstandings.

The GBR concept was originally developed and generally used as a risk allocation tool. It should be noted that rock behavior is generally dependant on both the ground conditions (Owner's responsibility) and the means and methods (Contractor's responsibility) and, therefore, identification of a DSC based on behavior makes allocation of the risk inherent in the work extremely difficult, if not impossible.

The Owner's conceptual design assumed that a precast segment lining would be used. Thus, at the time the GBR-A was prepared, the Owner's team anticipated that a precast, gasketed segmental liner would be used, erected within a fully shielded TBM. Under such conditions, the rock surrounding the excavation is never exposed; the rock is allowed to slab, loose rock is not removed, and continuous support is provided by the shield, segments and annular backfill. Consequently,

- 8.1.3.2 "... initial support must be installed ... immediately ... and must provide full coverage to the rock surface." Initial support cannot be installed immediately when using a main beam TBM. This apparently is also written for a TBM with a full circle shield.

The statement that stress induced spalling *will* occur at the sidewalls within Yz hour of excavation, in addition to the statement that invert heave is expected, could have led the Contractor to accept steel sets as the predominant support method within the QF, considering this to be the only method to effectively support both the sidewall spalling and invert heave.

There are also potentially misleading portions in Section 7.4.1.2 of the GBR "Observed Performance of the Trial Enlargement", such as:

- (a) "numerous incidences of ...sidewall spalling developed." Sidewall spalling in the Trial Enlargement probably occurred because it was excavated in four levels. Sidewall spalling would not be expected in a circular tunnel, excavated with a TBM in rock expected to fail due to high horizontal overstress. Sidewall spalling has not occurred in the QF; although some joint controlled and gripper induced fallout has occurred.

- (b) "The depth of crown slabbing (up to 0.5 m) was controlled by the presence of the overlying bedding plane." The fact that rock bolts were promptly installed to support the rock above the bedding plane may have limited the depth of crown slabbing and the degree of loosening of the crown rock. In addition, testimony noted that crown-slabbing observations were minimized because the roadheader operator over-excavated the crown to remove slabbing as it formed. Crown slabbing in the QF to Sta. 2+200 has varied from <0.5 m to 3 m in depth and is expected to continue.

- (c) "...slabbing of rock in the invert, up to 1.4 min depth, was noted ... when the invert was excavated to a horizontal ... profile." The wide flat invert was most prone to invert heave in the high horizontal overstress environment; whereas the circular invert of a TBM tunnel might show only minor invert cracking under the same subsurface conditions. Only fracturing and minor slabbing ofrock in the invert has occurred.

The Board considers that the Contractor's design, means and methods for support were changed based on the subsurface conditions encountered (4R & 4S) and as a result of serious misunderstandings as to the rock characteristics and behaviour within the QF.

The DRB believes that during preparation of the GBR, the Owner, the OR, the Contractor and the Designer did not realize these misunderstandings. Further, the DRB believes that these misunderstandings led to misinterpretations that resulted in the current dispute over the subsurface conditions that were anticipated in the QF and delineated in the GBR. Since both Parties worked together to develop the GBR, the consequences of the resulting misunderstandings should be shared between the Parties.

As noted previously, the DBA states "the GBR shall serve as the only basis for determining changes in or differing geotechnical subsurface conditions". However, the GBR states under Rock Support Requirements (Section 8.1.3.7) that "the in-situ Rock Condition shall be determined based on the closest **match** to the Rock Characteristics within each Rock Condition defined below" (in the Rock

(i) The horizontal stress values are plotted against elevation as shown in Figure 6.17. The maximum and minimum horizontal stresses in this section are about 17 and 10 MPa, respectively, and are relatively constant in the Queenston Formation.

(c) Downstream Section (Approximately Sta 0+000 to Sta 2+000)

(i) Results shown in Figure 6.18, including one test from Borehole NF-33, indicate that the maximum and minimum horizontal stresses at the elevation of the tunnel alignment as shown on the Concept Drawings are about 24 and 14 MPa, respectively.

(d) Stress Regime near the Trial Enlargement

(i) The boreholes for stress measurement in the downstream area are located in an area bounded by the Niagara River, the Niagara Escarpment and the St. Davids Gorge. The measured stress near the trial enlargement is lower than values in the diversion tunnels area due primarily to the stress relief effects of the Niagara River Gorge as all the measurements in the generation area are above the river bed. Results from this area are assumed to be relevant to the Queenston Formation in the tunnel outlet area.

(ii) The three-dimensional (3D) in situ stress components were determined by the overcoring technique at the powerhouse area and at a stub near the trial enlargement area. The average in situ stresses in the area close to the trial enlargement are as follows:

Principal Stresses	Azimuth (deg)	Dip (deg)
$\sigma 1 = 11.9$ MPa	133	-13
$\sigma 2 = 9.6$ MPa	050	-15
$\sigma 3 = 4.6$ MPa	008	-70

(iii) The resolved vertical stress from the overcoring tests is 5.3 MPa which is about 30% higher than the overburden stress calculated by the weight of the overburden material. This difference in magnitude is considered to be within the expected range of variation of vertical stresses from the overburden pressure in sedimentary deposits.

(e) Stresses Above the Queenston Formation

(i) In the upstream sections, maximum and minimum horizontal stresses above the Queenston Formation are about 10.5 and 4.5 MPa, respectively, measured in the Power Glen Formation in Borehole NF-3. Stresses are higher in the central segment; up to 18 and 6.5 MPa for maximum and minimum magnitudes of the maximum and minimum stress in the Queenston Formation below the bottom of the gorge are comparable to those in this section.

(b) Upstream Section (Approximately Sta 7+600 to Sta 10+000)

(i) The horizontal stress values are plotted against elevation as shown in Figure 6.17. The maximum and minimum horizontal stresses in this section are about 17 and 10 MPa, respectively, and are relatively constant in the Queenston Formation.

(c) Downstream Section (Approximately Sta 0+000 to Sta 2+000)

(i) Results shown in Figure 6.18, including one test from Borehole NF-33, indicate that the maximum and minimum horizontal stresses at the elevation of the tunnel alignment as shown on the Concept Drawings are about 24 and 14 MPa, respectively.

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(iii) The resolved vertical stress from the overcoring tests is 5.3 MPa which is about 30% higher than the overburden stress calculated by the weight of the overburden material. This difference in magnitude is considered to be within the expected range of variation of vertical stresses from the overburden pressure in sedimentary rock deposits. This result is considered to be applicable to the entire tunnel alignment.

19 Records Management

During execution of the Niagara Tunnel Project, most project records will be kept at the Project Records Centre at the Hatch Acres offices in Niagara Falls. Exceptions to this will be confidential and legal documents that will be kept at OPG headquarters in Toronto. Upon completion of the Project, all Project records will be transmitted to the Niagara Plant Group Records Centre.

Documents and records are organized in accordance with the SCI system.

19.1 Data Room

A Data Room was assembled and open to prequalified proponents intending to submit a proposal for the Niagara Tunnel Facility Project. In compiling the material for the Data Room, OPG and its Representatives elected to make all available information, of which they are aware, that is potentially relevant to the Niagara Tunnel Project, available to proponents. The material in the Data Room represented work done since the 1980s by various parties. Proponents were advised of risk that material in the Data Room may have been outdated, irrelevant, inaccurate or incomplete.

The Data Room was located at the Project Records Centre in Niagara Falls. The OR Data Room Coordinator was responsible for developing Data Room operation procedures and for facilitating access to the Data Room for Proponents. All documents have been stored at the Project Records Centre in Niagara Falls.

19.2 Core Samples

The core samples are located at the OPG Niagara Transformer Station, 1900 Murray Street (at Main Street) in Niagara Falls, and are available for viewing by the Contractor. Visits can be arranged by contacting Peter Pahl, Telephone 905-357-6721, email: peter.pahl@opg.com.

19.3 Project Documents and Correspondence

All Project documents, including correspondence, Purchase Requisitions, Purchase Orders (including amendments), reports, drawings, bills of material and the like must include proper document numbers and must be provided for filing with Project Records Centre.

Project drawings will be produced following OPG drawing standards and will include an approved title block. Project drawings are to be produced in electronic format preferably using the latest approved version of Autocad.

Proper document numbers, include the Property Designation (NAW130), Document Type, SCI, Serial Number and Revision Number. The Niagara Plant Group Records Centre manages the assignment of document and drawing numbers.

Proper file numbers, including the following, must appear on all Project correspondence:

Property Designation **NAW130**
SCI (5-number code).. **XXXXX**

7 PREVIOUS CONSTRUCTION EXPERIENCE

7.1 Time-Dependent Deformations Observed in Surface Excavations

1 The phenomenon of time dependent deformation in surface excavations (often referred to as 'rock squeeze') was first recognized in the early 1900s during the construction of the wheel pits of the Canadian Niagara and Toronto Power Plants in Niagara Falls. The wheel pits are 5.5 m wide, 50 m deep slots to house the penstocks and turbines. The wheel pits extend through the upper carbonate units into the Rochester Formation. Measurements of the closure of the pit walls at the Canadian Niagara Plant began in 1903. Sum total inward movement of both walls over a 68-yr period at the turbine deck opposite the DeCew/Lower Gasport units was 7.2 cm.

2 Extensive concrete cracking occurred in the Thorold Tunnel west bulkhead wall shortly after construction associated with the shaly limestone bed of the Gasport member. A major remedial program involving excavation of a slot in the rock and backfilling with a clay/bentonite mixture was carried out.

7.2 Grouting at the International Control Works and PGS Dyke

1 A review of the existing grouting records compiled during foundation grouting for the excavation of the International Control Works indicates that the average grout take of the primary holes, spaced at 6-m centres drilled to about 10 m below rock surface was 30 bags/m with much larger takes over particular intervals.

2 Grouting for the construction of the PGS dyke is considered applicable to the tunnel outlet area. Primary and secondary grout holes were spaced 12 m apart and extended 3 m into the Rochester Formation. Tertiary and fourth stage holes were split-spaced. Overall average grout takes were about 8.9 and 3.7 bags/m for the primary and secondary holes, respectively, with about 1.6 bags/m take in the tertiary and fourth stage holes. Grout takes varied significantly from interval to interval, with up to 82 bags/m take being recorded.

7.3 Gas Encounters

1 There is a long history of natural gas occurrence and exploitation in the Niagara Peninsula. Records of gas occurrences have been compiled from previous boreholes drilled for the construction of existing tunnels and the SAB2 Generating Station and from the observations made during the recent investigations. Pockets of gas were

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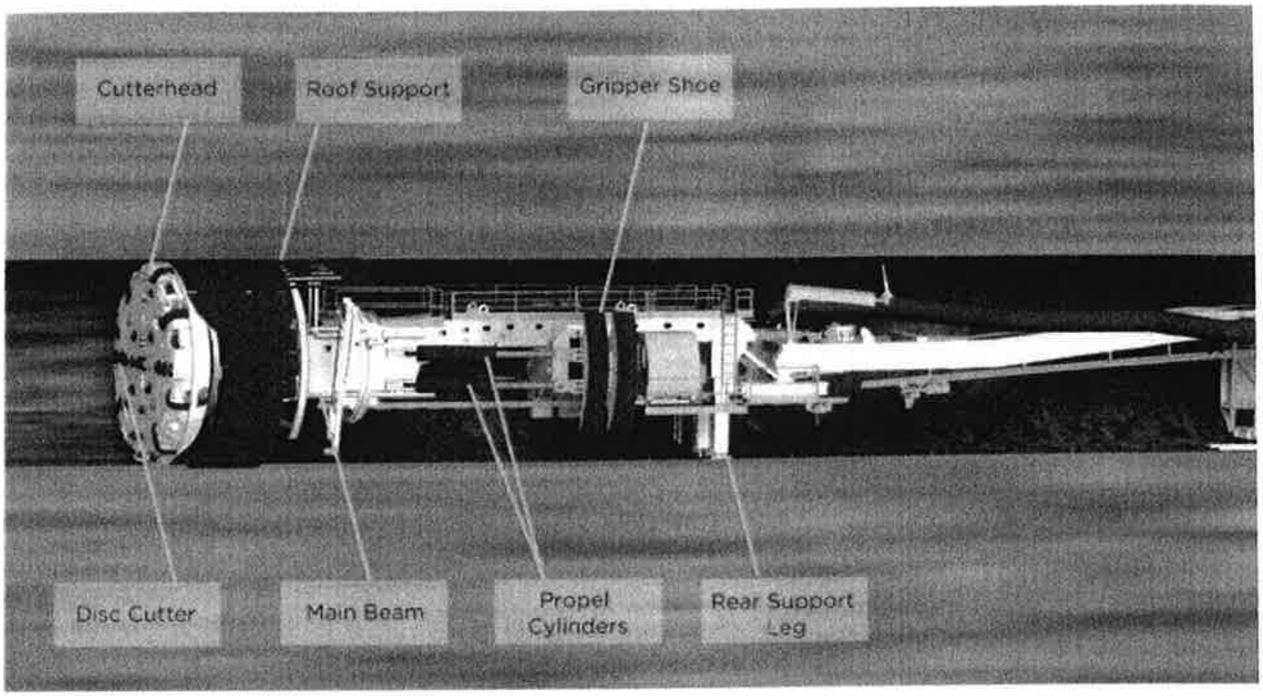
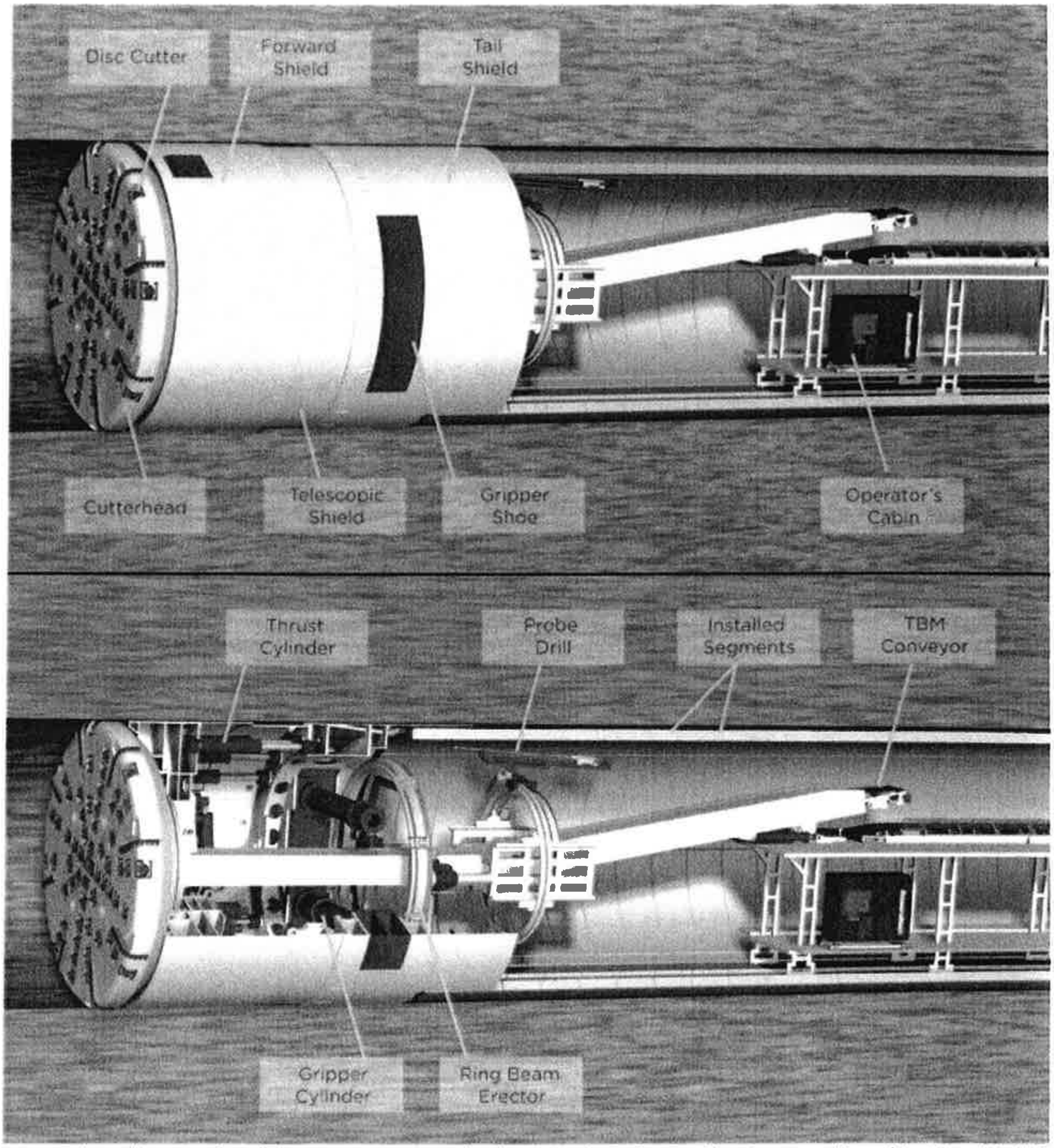
Main Beam



Source: http://www.therobbinscompany.com/en/our-products/tunnel-boring-machines/main-beam/

Double Shield



Source: http://www.therobbinscompany.com/en/our-products/tunnel-boring-machines/double-shield/

Ontario. This project was referred to as the Niagara River Hydroelectric Development ("NRHD").

Among the commitments made through the EA process was to utilize a TBM to excavate the tunnels starting from the outlet end, proceeding under the buried St. Davids Gorge and following the route of the existing SAB 2 tunnels through the City of Niagara Falls. A TBM was required in light of the development that had occurred in Niagara Falls since the original two diversion tunnels were constructed using the drill and blast method in the 1950s, and to minimize the amount of excavated materials from the project requiring disposal. Other commitments included re-use of excavated materials where feasible and an agreement to compensate the host municipalities, the Regional Municipality of Niagara, City of Niagara Falls and Town of Niagara-on-the-Lake, for forecasted project impacts on tourism, roads and municipal services.

2.2.2 1998 Decision to Pursue Third Tunnel

Early in February 1998, in anticipation of receiving EA approval, Ontario Hydro initiated a review of the viability of proceeding with the first phase of the NRHD (i.e., the construction of one additional 500 m^3/s tunnel). This review included the solicitation and evaluation of bids for the construction of the tunnel during the summer and fall of 1998 using a design-build approach.

In October 1998, the Minister of Environment provided approval under the *Environmental Assessment Act* for the complete NRHD as outlined above. The EA approval stipulated that it would "terminate if construction has not commenced within ten years from the date of this approval." This stipulation could be extended a further five years "based on the review and approval of an environmental review assessment status report." It provided Ontario Hydro with the flexibility to undertake the development in phases (i.e., initial construction of one tunnel); but did require that no construction extend "beyond twenty years following the commencement of construction."

Use of an open TBM was designed to allow for the installation of Strabag's proposed pre-stressed cast in place concrete liner with an impermeable waterproof membrane.[23] With an open TBM, the initial lining consists of rock bolts, friction anchors, wire mesh, steel channels, and shotcrete, which are used in various combinations depending on the conditions encountered.[24] This initial lining is intended to support the rock until the waterproof membrane is placed and the final concrete lining is cast. The TBM was configured to permit initial support adjustments as required during construction based on the rock conditions encountered.

Strabag's construction methodology was scored higher by the Evaluation Team because Strabag was the only contractor that proposed a cast-in-place liner with an impermeable membrane to protect it from water egress or ingress. This was an important feature not only because it enhanced the life expectancy of the tunnel liner, but also because geological tests indicated that the Queenston shale has the potential to swell if exposed to fresh water. The waterproof membrane proposed by Strabag increased the Evaluation Team's confidence that Strabag's tunnel design would be able to meet the required 90-year lifespan. The cast-in-place liner also reduced the potential for voids to develop between the liner and the surrounding rock as could have occurred with a closed (shielded) TBM and a precast liner. Finally, with fewer construction joints, a cast-in-place concrete liner is smoother than a precast concrete liner, which leads to increased water flow because of reduced friction.

[23] During the 1998 bidding process, all of the qualified contractors had proposed a closed TBM with a precast concrete segmental lining. For this reason, the 2005 Invitation to Submit Design/Build Proposal anticipated a closed TBM with a one-pass concrete liner. Unlike the other respondents, however, Strabag considered both open and closed TBMs before arriving at their proposed approach of using an open TBM with a cast-in-place concrete lining as the most effective method of meeting the requirements of the project including the 90 year life, impermeability and target flow.

[24] The initial lining was installed in two stages using the two primary areas for installing rock support behind the TBM cutterhead, which were known as L1 and L2. Initial support in the tunnel crown was installed immediately behind the TBM cutterhead in the L1 position, and shotcrete was placed about 40 metres behind the face at the L2 position. Initial support was generally comprised of 4 metre-long Swellex friction anchors, 150 mm C-channels, and welded wire mesh. As the TBM progressed and overbreak increased, shotcrete was placed between approximately the 10 o'clock and 2 o'clock locations in the tunnel crown from additional portable sprayers at the L1 position. A shotcrete layer was sprayed in a full circle at the L2 position.

550 tonne crane. The scrap steel leftover from the equipment removed was sold for approximately $800k.

The last intake gate section was installed on November 13, 2012. MOL was on site the same day to discuss the transfer of control from Strabag to OPG for the purposes of completing the third phase of intake work: the removal of the cofferdam and ice groyne, and the placement of approach wall blocks. On November 15, 2012, OPG resumed the role of constructor at the intake and the intake channel (area within the cofferdam) was flooded.

Cofferdam removal work commenced on November 19, 2012 and was completed on February 3, 2013. The ice groyne was then removed by excavation in still water commencing February 23, 2013 and was completed on March 3. As of March 8, the third phase of intake site work was complete and OPG was no longer the constructor at the intake site. The MOL was then informed on March 11 that Strabag was the constructor until the end of the project.

6.5.4 Tunnel Construction

6.5.4.1 Tunnel Boring Machine

When the Tunnel Boring Machine ("TBM") used for the NTP was put into service, it was the largest open gripper main beam TBM in the world with a diameter of 14.44 metres.[22] The TBM and back-up was 150 metres long and weighed about 4,000 tonnes. It was named "Big Becky," the winning entry from a naming contest among local schools. The name reflects the contributions of Sir Adam Beck in hydroelectric development and the size of the TBM.

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[22] There are two main types of TBMs: open (unshielded) and closed (shielded). Open TBMs require systematic rock-support behind the cutter head because the final lining is installed later. They use a gripper system that pushes against the tunnel side walls to advance. Where a concrete liner is required, it is installed by means of second pass operation after the TBM has completed mining. Closed TBMs are equipped with a shielded body under which supporting operations, including installation of a precast concrete lining system, are carried out. They advance via thrust cylinders that push off against the tunnel lining segments installed behind the machine. The entire tunnel is excavated and lined in one-pass.

committed to install temporary signalization at the Niagara Parkway at Portage Road to minimize impacts on through traffic during construction.

At certain times, the Contractor requires access to the INCW bridge deck and will have to work within the river to undertake in-water excavation of the intake channel, installation and removal of the cofferdam, removal of the existing ice accelerating wall and construction of a new wall, closure of the downstream Bay 1 and construction of portions of the intake approach wall. During these periods of work, OPG will be the "Constructor" under the Occupational Heath and Safety Act when work is performed under the 'INCW Part Project' designation. This approach has received approval from the Ministry of Labour.

3.3.2 Outlet Area

The main construction facilities are on OPG's lands, located between the PGS Reservoir and the existing Sir Adam Beck 2 canal. Access is provided by a new road connection to Stanley Avenue. Temporary signalization is required at the intersection with Stanley Avenue and is being installed by the Regional Municipality of Niagara on behalf of OPG.

3.3.3 Intake Structure

The intake structure is a reinforced concrete structure that will be constructed underneath the INCW, located upstream from the Niagara Falls. The design of the intake (through the use of numerical and physical models) has been examined extensively to optimize flow conditions and minimize ice entrainment. The structure will house sectional service gates for closure of the diversion tunnel at the upstream end. Ice management during intake construction (cofferdam in place) has also been numerically modeled and determined to be comparable to existing conditions.

The majority of the intake excavation will be done within a cofferdam that must be completed prior to the break-through of the TBM. Prior to cofferdam construction, a new accelerating wall, used to facilitate ice management at the intake, will be constructed and the existing accelerating wall will be demolished. Following completion of the concrete works, the cofferdam will be removed.

It is expected that extensive grouting will be required of the upper rock formations to minimize water inflows into the tunnel during the TBM drive through these formations. In addition, underwater excavation of an intake channel is required upstream from the intake structure and beyond the confines of the cofferdam.

3.3.4 Diversion Tunnel

The tunnel is to be excavated from the downstream end through limestones, sandstones and shales using a 14.4 m excavated diameter TBM to be supplied by the Contractor. The tunnel will be constructed in two passes with the first pass consisting of excavation and an initial lining to support the excavation consisting of shotcrete, mesh, bolts and ribs. Once the complete tunnel is excavated and the TBM removed, a cast-in-place concrete final lining between 600 and 700 mm thick will be constructed. An impermeable membrane will be placed between the initial and final lining to ensure watertightness of the tunnel. The final lining will be prestressed using high pressure grout injected between the impermeable membrane and the initial lining.

submitting proposals was extended from April 15 to May 13, 2005 on the understanding that no further extensions would be authorized.

3.5 Proposal Evaluation and Negotiation

OPG prepared a detailed evaluation process as described in the first sub-section below. The second sub-section discusses the actual evaluation of the proposals received and the negotiations with the various proponents to refine the proposals prior to selecting the successful firm.

3.5.1 Evaluation/Negotiation Process Overview

OPG used a structured evaluation process developed jointly with the OR to evaluate the three proposals submitted. The Evaluation Team consisted of experienced personnel from OPG, Hatch and Torys. The team used evaluation criteria and scoring that were established for this project based on input from the both OPG and external members of the project team and documented before the proposals were received. A summary of the evaluation categories and their relative scoring is shown in
Table **3** below.

Table 3 - Evaluation Categories and Scoring

Summary Evaluation Categories	Score (#)	Percent (%)
Compliance with Owner's Mandatory Requirements	Yes/ No	Yes/ No
Design & Construction Approach	80	16%
Response to GBR	45	9%
Price/Schedule/Flow Guarantee	150	30%
Adherence to Invitation and Agreement	45	9%
Risk Management Approach/Impact on OPG Risk Profile	65	13%
Project Team & Key Personnel	45	9%
Preliminary Project-Specific Safety/Security/Emergency Plans	35	7%
Environmental Compliance Plan and QA/QC Program	35	7%
Total	**500**	**100%**

Rock mass parameters for the design analysis are derived by the Hoek/Brown method, utilizing the software package RocLab version 1.010 (October 2004) with the following input parameters:

1. *UCS of intact rock*
2. *GSI*
3. *mi-Index*

Data for UCS of the intact rock is obtained from GBR-A, table 6.3 (average value and range; if no range is indicated, the range is calculated according to GBR-A, chapter 1, 8).

The GSI values stated in GBR-A (tables 6.9 and 6.10) are found to be optimistic compared to the joint spacing data. An evaluation of the stated RMR and GBR values cannot be carried out due to the lack of information concerning the RMR input parameters.

Therefore GSI values are defined in evaluating the rock mass spacing of the individual rock mass formations encountered in the boreholes along the tunnel alignment and by evaluating information concerning discontinuity roughness available in the GBR-A.

The GSI values are obtained from the GSI chart provided by Cai and Kaiser 2002. The range of the GSI is estimated based on engineering judgement.

The mi-index is obtained from the mi chart provided with the RocLab software. The range of the mi index is estimated based on engineering judgement.

The average values and the ranges of the rock mass are based on the average values and ranges of the intact rock. The table below lists the input parameters for the calculations utilizing the Hoek/Brown Method and the rock mass parameters such derived.

A detailed description of the applied methodology is summarized in chapter 3 of the design basis and method statements document PR-00-3001 submitted with the Proposal.

60

Table 6.9
Rock Mass Strength Parameters for Rock Formation Above Queenston Shale

Formation	RMR	Adjusted RMR*	Compressive Strength (MPa)	m_i	m	s
Lockport Dolostone						
- Eramosa	69	79	151	7.0	3.3	0.0970
- Goat Island	69	79		7.0	3.3	0.0970
- Gasport	72	82		7.0	3.7	0.1353
DeCew Dolostone	69	79	128	7.0	3.3	0.0970
Rochester Shale	64	77	42	10.0	4.4	0.0777
Irondequoit Limestone	72	82	106	7.0	3.7	0.1353
Reynales Dolostone	67	77	95	7.0	3.1	0.0777
Neahga Shale	56	66	14	10.0	3.0	0.0229
Thorold Sandstone	78	83	163	15.0	8.2	0.1524
Grimsby Sandstone Shale	70	75	155 33	10.0	4.1	0.0622
Power Glen						
• Sandstone/Shale	61	66	172	10.0	3.0	0.0229
• Shale	65	70	24	10.0	3.4	0.0357
Whirlpool Sandstone	85	87	216	15.0	9.4	0.2359

* Adjusted RMR values are equivalent to GSI.

Table 6.10
Rock Mass Strength of Queenston Formation

Area	RMR	σ_c (MPa)	m_i	m	s
Inlet area	66	33	6.5	1.93	.0229
Tunnel alignment (general)					
Q10	55	33	6.5	1.30	.0067
Q8,9	65	33	6.5	1.86	.0205
Q6,7	71	33	6.5	2.31	.0399
Q4,5	67	46	14.5	4.46	.0256
Q1,2,3	82	46	14.5	7.62	.1353
Tunnel Alignment in area of St. Davids					
Gorge	67	33	6.5	2.00	.0256
Q6	67	46	14.5	4.46	.0256
Q5	73	46	14.5	5.53	.0498
Q3,4	76	46	14.5	6.15	.0695
Q1,2					
Outlet Area					
Q7-10	57	33	6.5	1.40	.0084
Q5-6	77	46	14.5	6.38	.0776

σ_c = uniaxial compressive strength
m,s = Hoek-Brown constants for rock mass
m_i = Hoek-Brown constants for intact rock

Notes:

1. Above values based on Definition Engineering Phase 2 investigation results for intact core. Phase 1 results of $m_i = 10$ and $\sigma_c = 45$ MPa were superseded by this work.
2. RMR values have been adjusted and are equivalent to GSI.

6.4 Assessment of Rock Mass Strength

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pressure and structural orientation terms. The modified value is denoted the Geological Strength Index (GSI). A set of empirical relationships are then used relating GSI values and the constants 'm' and 's'.

(c) Mohr-Coulomb parameters will be estimated from the constants 'm' and 's' following the instantaneous approach suggested by Hoek (1997), at the applicable actual effective horizontal stresses, in consideration of pore water pressures.

6.4.2 Rock Formations Above Queenston Formation

1 The rock mass strengths were estimated on the basis of the average uniaxial compressive strength of the rock and m_i values recommended by Hoek (1988) for the various rock types. The resulting 'm' and 's' values given in Table 6.9 were based on RMR values that were adjusted for the purpose of rock mass strength estimates as per Hoek (1988).

6.4.3 Rock Mass Strength of Queenston Formation

The Queenston rock mass strength has been evaluated in the Definition Engineering Phase 2 investigations, based on the 'm_i' (intact) values from triaxial testing and RMR values. Results of laboratory triaxial strength testing were used to estimate the intact rock strength as previously discussed. Rock mass strengths are given in Table 6.10.

1 The RMR values noted in Table 6.8 were similarly grouped into simplified 'generic' classes to provide approximate values for specific areas. These RMR values were then combined with the 'm_i' and compressive strength evaluations to estimate the strength of the in situ rock mass as given in Table 6.10.

2 The subdivision of the Queenston rock mass strengths into particular depths in Table 6.10 does not take into account any weaker or close jointed zones such as those under the St. Davids Gorge.

6.5 Groundwater and Gas

6.5.1 Hydrogeology

1 The rock strata form an interlayered succession of relatively pervious and relatively impervious rocks. The impervious formations impede flow, whereas the more permeable formations serve either as recharge or discharge horizons for adjacent formations. Within the more permeable formations, the hydraulic conductivity is principally related to the presence of a few open fractures which are predominantly horizontal. Vertical connectivity of these fractures is low, except in the upper rock units. Thus, formations which exhibit high hydraulic conductivity from packer testing may have a low vertical hydraulic connectivity.

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Neahga Shale	56	66	14	10.0	3.0	0.0229
Thorold Sandstone	78	83	163	15.0	8.2	0.1524
Grimsby Sandstone	70	75	155	10.0	4.1	0.0622
Shale			33			
Power Glen						
• Sandstone/Shale	61	66	172	10.0	3.0	0.0229
• Shale	65	70	24	10.0	3.4	0.0357
Whirlpool Sandstone	85	87	216	15.0	9.4	0.2359

* Adjusted RMR values are equivalent to GSI.

64

Table 6.10
Rock Mass Strength of Queenston Formation

Area	RMR	σ_c (MPa)	m_i	m	s
Inlet area	66	33	6.5	1.93	.0229
Tunnel alignment (general)					
Q10	55	33	6.5	1.30	.0067
Q8,9	65	33	6.5	1.86	.0205
Q6,7	71	33	6.5	2.31	.0399
Q4,5	67	46	14.5	4.46	.0256
Q1,2,3	82	46	14.5	7.62	.1353
Tunnel Alignment in area of St. Davids					
Gorge	67	33	6.5	2.00	.0256
Q6	67	46	14.5	4.46	.0256
Q5	73	46	14.5	5.53	.0498
Q3,4	76	46	14.5	6.15	.0695
Q1,2					
Outlet Area					
Q7-10	57	33	6.5	1.40	.0084
Q5-6	77	46	14.5	6.38	.0776

σ_c = uniaxial compressive strength
m,s = Hoek-Brown constants for rock mass
m_i = Hoek-Brown constants for intact rock

Notes:

1 Above values based on Definition Engineering Phase 2 investigation results for intact core. Phase 1 results of $m_i = 10$ and $\sigma_c = 45$ MPa were superseded by this work.

2 RMR values have been adjusted and are equivalent to GSI.

(c) Mohr-Coulomb parameters can be estimated from the constants 'm' and 's' following the instantaneous approach suggested by Hoek (1997), at the applicable actual effective horizontal stresses, in consideration of pore water pressures.

6.4.2 Rock Formations Above Queenston Formation

1 The uniaxial compressive strength (UCS) and m_i values of the rock and estimated RMR of the rock mass are given in Table 6.9. RMR values were adjusted for the purpose of rock mass strength estimates as per Hoek (1988) and m_i values. were estimated on the basis of the average values recommended by Hoek (1988) for the various rock types.

6.4.3 Rock Mass Strength of Queenston Formation

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1 The RMR and m_i values noted in Table 6.8 were similarly grouped into simplified 'generic' classes in Tables 6.9 and 6.10 to provide approximate values for specific areas. .

6.5 Groundwater and Gas

6.5.1 Hydrogeology

1 The rock strata form an interlayered succession of relatively pervious and relatively impervious rocks. The impervious formations impede flow, whereas the more permeable formations serve either as recharge or discharge horizons for adjacent formations. Within the more permeable formations, the hydraulic conductivity is principally related to the presence of a few open fractures which are predominantly horizontal. Vertical connectivity of these fractures is low, except in the upper rock units. Thus, formations which exhibit high hydraulic conductivity from packer testing may have a low vertical hydraulic connectivity.

2 In addition to areas of increased weathering and discontinuities as given in Section 4, zones of increased jointing and higher hydraulic conductivity in the area will potentially occur where the tunnel alignment crosses the trend line of the crest of Horseshoe Falls (the east-west trending jointing at the Canadian Falls area is parallel to this trend line).

3 Piezometric levels in the Guelph and Upper Lockport formations are controlled by recharge from nearby bodies of water such as the Niagara River, the PGS reservoir, and the existing power canals into which these strata daylight. High hydraulic conductivity was measured for some of these rocks and the flow is largely confined to

Table 6.9
Rock Mass Strength Parameters for Rock Formation above Queenston Shale

*

Formation	RMR	Adjusted RMR*	Unconfined Compressive Strength (MPa)	m_i
Lockport Dolostone			151	
- Eramosa	69	79		7.0
- Goat Island	69	79		7.0
- Gasport	72	82		7.0
DeCew Dolostone	69	79	128	7.0
Rochester Shale	64	77	42	10.0
Irondequoit Limestone	72	82	106	7.0
Reynales Dolostone	67	77	95	7.0
Neahga Shale	56	66	14	10.0
Thorold Sandstone	78	83	163	15.0
Grimsby Sandstone	70	75	155	10.0
Shale			33	
Power Glen				
• Sandstone/Shale	61	66	172	10.0
• Shale	65	70	24	10.0
Whirlpool Sandstone	85	87	216	15.0

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Tunnel alignment (general)			
Q10	55	33	6.5
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Q6,7	71	33	6.5
Q4,5	67	46	14.5
Q1,2,3	82	46	14.5
Tunnel Alignment in area of St. Davids Gorge			
Q6	67	33	6.5
Q5	67	46	14.5
Q3,4	73	46	14.5
Q1,2	76	46	14.5
Outlet Area			
Q7-10	57	33	6.5
Q5-6	77	46	14.5

σ_c = uniaxial compressive strength
m,s = Hoek-Brown constants for rock mass
m_i = Hoek-Brown constants for intact rock

Notes:

1 Above values based on Definition Engineering Phase 2 investigation results for intact core. Phase 1 results of $m_i = 10$ and $\sigma_c = 45$ MPa were superseded by this work.

2 RMR values have been adjusted and are equivalent to GSI.

3 The primary bedding planes will affect the excavation of the tunnel as many are clay rich and form weak discontinuity surfaces that, because of the shallow dip of the tunnels, may follow the excavation for considerable distances. Their locations can be estimated from Figure 4.1. However, because only two boreholes are available with geophysical trace information, detailed correlation of all the bedding planes within the Queenston Formation across the complete length of the tunnel alignment has not proved possible.

4.4.2 Faulting and Discontinuities

1 There are no known occurrences or reports of any major faulting within the Project area. Some near-surface, low angle thrusts with minor vertical displacement are known to occur and are probably related to stress relief associated with the gorge formation and the high horizontal residual stresses in the area. Some shearing of this type can be expected in the area of the St. Davids Gorge.

2 Regional joint measurements indicate the jointing to be high angle or vertical with the dominance of three major joint directions and a subordinate fourth set. In addition to these high angle sets, there is another set parallel to bedding. Based on strike directions the most prominent subvertical joint sets are

(a) a 005deg joint set which parallels the general trend of the Niagara River, particularly in the area of the tunnel outlet

(b) a 045deg joint set which approximately parallels the Niagara River, downstream from the Whirlpool

(c) a 085deg joint set which approximately parallels the Niagara Escarpment

(d) a 135deg joint set which approximately parallels the buried St. Davids Gorge.

3 Gypsum and calcite, and dolomite mineralization occur along joint sets of 085deg and 135deg orientations.

4 The joint sets vary in spacing, frequency and continuity depending on location and lithology. Vertical joints are generally widely spaced. The joint surfaces are generally rough and fresh to slightly weathered.

4.4.3 In Situ Stresses

1 High in situ stresses exist in the Project area bedrock. Measurements show that maximum horizontal stress in the Queenston Formation range from 10 to 24 MPA, with a maximum horizontal/vertical stress ratio varying from 3 to 5. Higher stress ratios are measured in the overlying rock units. In general, the orientations of the maximum horizontal stresses along the alignment of the diversion tunnel lie within the NE-SW quadrant. The orientations of the local stresses are influenced by the

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el. 10 m; NF-45 inclined at 53 degrees; NF-43 vertical boring; NF-39 inclined at 53 degrees at the Gorge.

Core recovery, RQD and the character of the discontinuities encountered, were recorded on the log for each borehole. The inclined borings were done to intersect sub-vertical to vertical joints. Also borehole photography with core orientation and permeability testing were done in NF-45, NF-39 and geophysical logging in NF-43 to further define the orientation, frequency and character of discontinuities. Permeability tests were done in borings NF-45 and NF-39 and ground water samples retrieved for water chemistry tests and piezometric heads in the various formations measured.

3.3.2 Exploration in the St David's Gorge Area

Phase 1 Testing

It was ascertained that within a zone of 15 to 25 m below the bedrock surface, the rock was slightly weathered with RQD values varying from 31 to 71 %. Bedding joints were frequent and some slickensides (surfaces of discontinuities with evidence of former movement and therefore of very low shear strength) were present. At depths greater than 30m below the bedrock surface, the RQD values improved significantly and were generally higher than 90% generally indicating that with increasing depth below the bedrock surface, rock conditions improved significantly.

3.3.3 In-Situ Stress Measurement

Hydro-fracture tests were done in borehole NF-31 (at a distance of 400 m from the Niagara River gorge) and NF-38 (powerhouse area) in order to locate the proposed Adit enlargement in an area where the in-situ stresses would be similar to those anticipated in the deep section of the diversion tunnels, as well as for the design of the underground powerhouse.

3.3.4 Laboratory Testing of Rock Core Samples

The testing for the Definition Engineering Phase 1 investigations was focused primarily on the Queenston along the diversion tunnels and at the underground powerhouse locations.

The laboratory test program consisted of the following components:

DBA

GBR-C

Table 4.1
Major Stratigraphic Units

Formation Name	Thickness (m)	Petrographic Description
Thorold	2 - 3.5	Sandstone, light grey to greenish-grey; medium-bedded to massive; irregular green shale partings occur throughout. The sandstone is orthoquartzitic. The texture of the formation is very fine-grained. Silt-size to fine-grained quartz particles are cemented with secondary silica.
Grimsby	12.5 - 12	Sandstone, to reddish-brown; thin- to thick-bedded, often calcareous with interbedded shale. The sandstone texture varies from fine to medium grained. A weathered zone frequently occurs at the top of the formation.
Power Glen	10 – 13	Shale with siltstone beds and stringers; dark grey to greyish-green shale and siltstone, and light grey limestone and dolomite. Quartz is the most abundant non-clay mineral. Clay minerals consist of illite, chlorite and small amounts of montmorillonite and mixed layered clays.
Whirlpool	4.9 - 8.5	Sandstone, light grey to white; medium-bedded and cross-bedded; fine- to medium-grained. The quartz grains are well rounded, and are well cemented by secondary silica. Feldspar grains altered to kaolinite are abundant. Occasional green shale inclusions and chloritic shale partings occur throughout.
Queenston	>300	Shale (technically classified as a silty mudstone or siltstone), reddish-brown with interbeds and nodules of green. The shale is silty and is cemented in many situations by dolomite and calcite. In many places it is massive to blocky, however some fissile sections occur. Scattered gypsum nodules occur throughout lower sections of the unit; quartz is a common constituent. Clay minerals are illite, chlorite, kaolinite, montmorillonite and other clays. Numerous small, high angle slickensides occur, often stained with iron oxide.
Subdivisions of the Queenston Formation		
Q10 Q9 Q8 Q7	45 - 50	Generally upwards fining sequence of reddish brown mudstones and silty mudstones with about 30% green muddy siltstone interbeds and blebs. Division Q10 commonly shows weathered surfaces within which numerous slickensided partings occur.
Q6 Q5	30 - 35	Reddish brown muddy siltstones with distinct bedding partings and marked bands of green siltstone and occasional bands and areas of distinctive gypsum nodules. Some zones contain slickensided compaction features. A zone of phosphate nodules occurs at base.
Q4	15 - 20	Reddish brown muddy siltstone with frequent green siltstone.

72

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Power Glen	10 – 13	Shale with siltstone beds and stringers; dark grey to greyish-green shale and siltstone, and light grey limestone and dolomite. Quartz is the most abundant non-clay mineral. Clay minerals consist of illite, chlorite and small amounts of montmorillonite and mixed layered clays.
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Q4	15 - 20	Reddish brown muddy siltstone with frequent green siltstone.

1.3.2 Owner's Position

The Owner maintains that the stand-up time relationships with RMR values, as developed by Bieniawski, are for ground conditions not subjected to high in situ stresses and therefore are not applicable to this situation. Further, the Owner maintains that stress induced failure in the QF, where tangential stresses are a high proportion of the rock's unconfined compressive strength, will occur at or immediately behind the cutterhead and, if not controlled by the TBM roof shield and immediate rock support, will continue into the rock mass and result in excessive overbreak. The Owner maintains that the Contractor agreed to install full and immediate support and closely spaced steel sets over -75% of the QF to mitigate this. Therefore, if the Contractor recognized the need for full and immediate support, stand-up time could not have been expected. The Owner maintains that stress-induced failure has been the primary failure mechanism within the QF, exactly as indicated in the GBR, and therefore no DSC was encountered.

1.4 Excessive Overbreak

1.4.1 Contractor's Position

The Contractor maintains that the QF did not behave as a "generally massive" rock, as indicated in the GBR, and therefore, that the originally agreed on support method using steel sets could not be practically installed in a manner that would limit loosening of the remaining rock to the degree deemed necessary by its Designer. Also, "the principal reason for using steel sets were indications in the GBR of a high stress environment and significant potential for swelling and squeezing in the QF, with invert heave and sidewall distress". Further, the final liner approach with a prestressed unreinforced cast-in-place liner and a water tight membrane was a key factor in the selection of this Contractor. Considering the extraordinary 90-year service life specified in the Owner's Mandatory Requirements, combined with practical limitations on the ability to grout any remaining voids, the Contractor had to change its support means and methods to reliably and practically limit the amount of loosened rock left in place. Also, the reduced squeeze, sidewall spalling and invert heave actually encountered made the use of steel sets less important. The change in means and methods was driven by the DSCs, not vice versa, and the resulting excessive overbreak (several times greater than the average amount per meter that was anticipated) is, in itself, sufficiently material to entitle the Contractor to immediate relief under the contract provisions for DSCs.

1.4.2 Owner's Position

The Owner maintains that the features originally provided on the TBM should have been sufficient to provide the necessary rock support until steel sets could be placed immediately behind the TBM shield and expanded behind the fingers. The Contractor removed the equipment on the TBM that was needed to install steel sets before reaching the QF and, hence, never attempted to install steel sets in the QF as stipulated in the GBR, let alone document an unacceptable degree of loosening of the remaining rock as required by Section 5.7(b) of the DBA. The Owner maintains that if the steel sets were properly installed, including the intermediate bolts, the resulting loosening of the rock could have been limited to levels that met the design requirements. Further, the conditions encountered were as defined in the GBR and it was the Contractor's decision to change its means and methods that caused the excessive overbreak. The DBA specifically states that the Contractor will not be entitled to make any claim for the impacts resulting from a change or deficiency in the designs, means and methods that causes a difference in the behaviour of the geotechnical subsurface conditions.

Although this might be construed to mean that no DSC has been encountered (i.e. the Contractor had correctly anticipated the ground conditions prior to encountering the ground within the tunnel), the DBA clearly states that the identification of a DSC shall be based on the information contained in the GBR. If the GBR is ambiguous or imprecise in its description of the subsurface conditions such that the Contractor reasonably misunderstood those conditions at the time the DBA was signed, then a DSC would exist. In this regard, one of the main differences in Rock Characteristics between Rock Condition 4 and Rock Conditions 5 & 6 as presented in the GBR is the inclusion of "rock pressure generally exceeding rock mass strength" for Types 5 & 6, but not for Type 4. Nonetheless, over 25% of the tunnel length in the QF is identified in the GBR as Rock Condition 4Q. This is inconsistent with the conditions actually encountered in the QF where stress induced fracturing has been encountered throughout, as evidenced by its classification as Type 5 by the Owner.

The addition of shotcrete in the Ll area to the Type 4 support described above is called Type 4R support. In the Board's opinion, this addition of shotcrete does not constitute a change in means and methods that would justify invoking the provision of DBA Section 5.5(b)(2) regarding" ...a change or deficiency in the Contractor's designs, means, methods ...".

Type 4S is a new support method necessitated, based on subsurface conditions actually encountered, by the QF overbreaking higher than the Contractor anticipated from the descriptions provided in the GBR. Types 4R and 4S are required by the design note: "loose rock to be **removed"**.

The DRB believes that loose rock formed faster than the Contractor anticipated, largely due to the stress induced fracturing, and the Board is also of the opinion that full circle steel sets are unnecessary and impractical to use to support only the crown (i.e. no significant sidewall spalling or invert heave). In the Board's opinion, rock bolts and steel channels, following removal of loose rock, are the optimum initial support in the QF in this tunnel under the actual ground conditions encountered and the final lining requirements, although this will probably result in greater overbreak quantities than indicated in the GBR.

3.4 Insufficient Stand-Up Time

The Contractor testified that RMR values stated in the GBR led it to believe the QF would not fail so fast that adequate initial support could not be installed within the Ll and L2 areas. Although GBR 6.3 states that RMR values were used to assess rock mass strengths in the concept design, it neglected to point out that the RMR method of rock mass classification was not applicable as an indicator of stand-up time in rock subject to stress-induced failure, such as the QF. Even for rock not subjected to a high horizontal stress, the reported RMR values, when compared to Bieniawski graphs showing opening spans, should have raised serious concerns over stand up times when installing initial support

However, the configuration of the selected TBM suggests to the DRB that the Contractor did not expect that rock in the crown in the QF (over 80% of the tunnel) would fail almost immediately due to overstress. If immediate overstress failures had been anticipated, the DRB believes the TBM would have been designed so all passages for muck to enter the cutterhead would have been radial openings in the cutterhead faceplate without peripheral buckets. With the TBM used on this project, there is an unsupported distance of 1.2 m over the cutterhead with the peripheral buckets comprising some 0.6 m of this distance. The rock can relax, fracture, break apart and fall into theses buckets before it can be supported by the TBM roof shield. Even with stress induced fractures, such a condition may not have been anticipated if the rock was believed to be "generally massive".

In the DRB's opinion, the Contractor's original plan to use steel ribs as a regular means of initial support in the QF suggests that it anticipated the rock to be "generally massive" with reasonably good stand up time throughout much of the QF formation. Under such a scenario, the need for full circle steel ribs to resist sidewall spalling and invert heave would make sense, while feeling that stress induced fracturing in a "generally massive" rock would not produce serious crown stability problems or loosening of crown rock to a degree that would raise concern over performance of the final liner under high interface grouting pressures.

It appears to the Board that there was a serious misunderstanding between the Parties with respect to the anticipated rock conditions and rock behavior at the time the contract GBR was being negotiated. Since both Parties developed the GBR jointly, any misunderstanding is the shared responsibility of both Parties.

3.5 Geotechnical Baseline Report

It is noteworthy that Appendix 5.4- Geotechnical Baseline Report states in item 1.4 that "the GBR will be used during the execution of the Contract for comparison of the *assumed subswface conditions with actual subsurface conditions* as encountered during construction." The wording contained in this Appendix 5.4 is consistent with the usual concept of a GBR on a Design-Bid-Build project.

Section 5.4 of the DBA, however, states the GBR "describes *anticipated behaviors and conditions that are dependent on the Contractor's selected designs, means. methods ....anticipated or implied* at the date of this Agreement." The wording in the DBA expands and complicates the GBR concept and purpose by (1) changing *"assumed"* to *"anticipated"* or *"implied"* and (2) by including *"behaviors and conditions that are dependent on the Contractor's selected designs, means, methods ...* ", both of which require a mutual understanding between the Parties. The DRB assumes the objective of these modifications is to avoid DSCs based on subsurface conditions set by one party to the contract. This may seem achievable, especially when the GBR is ')ointly developed" by the Owner and Contractor. However, neither Party is likely to anticipate all of the conditions and behaviours that will be encountered and would influence the performance of the Work, let alone have a clear mutual understanding of those conditions and behaviours. In the Board's opinion, the wording in the DBA makes the application of the GBR concept much more complex and increases the likelihood of misunderstandings.

The GBR concept was originally developed and generally used as a risk allocation tool. It should be noted that rock behavior is generally dependant on both the ground conditions (Owner's responsibility) and the means and methods (Contractor's responsibility) and, therefore, identification of a DSC based on behavior makes allocation of the risk inherent in the work extremely difficult, if not impossible.

The Owner's conceptual design assumed that a precast segment lining would be used. Thus, at the time the GBR-A was prepared, the Owner's team anticipated that a precast, gasketed segmental liner would be used, erected within a fully shielded TBM. Under such conditions, the rock surrounding the excavation is never exposed; the rock is allowed to slab, loose rock is not removed, and continuous support is provided by the shield, segments and annular backfill. Consequently, presence of major physiographic features, namely the buried St. Davids Gorge and the Niagara River Gorge.

4.4.4 Bedrock at St. Davids Gorge

1 The geological profile of and below the buried St. Davids Gorge, interpreted from boreholes and geophysical investigations, is shown in Figure 4.2.

2 For the purposes of this GBR, the width of the St. Davids Gorge is 800 m.

3 Figure 4.3 represents the baseline for the bottom of the St. Davids Gorge. This figure is based on available seismic (Niagara River Hydroelectric Development, Seismic Reflection Survey, Niagara Falls, Ontario, multiVIEW Geoservices Inc., January 1991) and borehole data from the St. Davids Gorge area. Elevations shown are equal to the interpreted seismic elevations minus an amount equal to a 20% error in depth calculations (as compared to 15% that was recommended in the seismic report). Elevations are given as ellipses consistent with the original seismic report. Borehole information is given as top of rock minus 5 m. The baseline represents spot elevations of the bottom of the gorge, defined as the top of bedrock (fractured or otherwise). Contouring of this data does not represent a baseline.

4 The bedrock (Queenston Formation) over the width of the St. Davids Gorge is slightly weathered and relatively more fractured to a depth of between 15 to 25 m below the bottom of the gorge. Below this depth, the rock is generally fresh and of excellent quality. No evidence of a major fault or other major discontinuities underlying the St. Davids Gorge has been found to date either by drilling or from geophysical surveys.

4.4.5 Geological Profile

1 The geological profile and the lithology as shown in Figures 4.1 and 4.2 of the GBR has been projected horizontally and is applicable to the alignment selected by the Contractor.

4.5 Hydrogeologic Setting

1 Groundwater conditions in the Project area are influenced by depth and lithology, and vary between the rock formations above the Queenston Formation, but are relatively consistent in the Queenston formation. The only known aquifers are the Lockport and DeCew (dolostone) Formations, whereas the remaining strata below the DeCew are generally considered to be aquitards. The groundwater below the DeCew Formation is highly corrosive.

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2 Figure 4.3 indicates the interpreted top of bedrock at the St. Davids gorge and is based on available seismic (Niagara River Hydroelectric Development, Seismic Reflection Survey, Niagara Falls, Ontario, multiVIEW Geoservices Inc., January 1991) and borehole data from the St. Davids Gorge area. Elevations shown are equal to the interpreted seismic elevations minus an amount equal to a 20% error in depth calculations (as compared to 15% that was recommended in the seismic report). Elevations are given as ellipses consistent with the original seismic report. Borehole information is given as top of rock minus 5 m.

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4.6 Natural Gas

1 Natural gas has been encountered in some of the formations, particularly in the Rochester and Grimsby Formations, with some minor amounts of gas being encountered in other formations, including the Queenston.

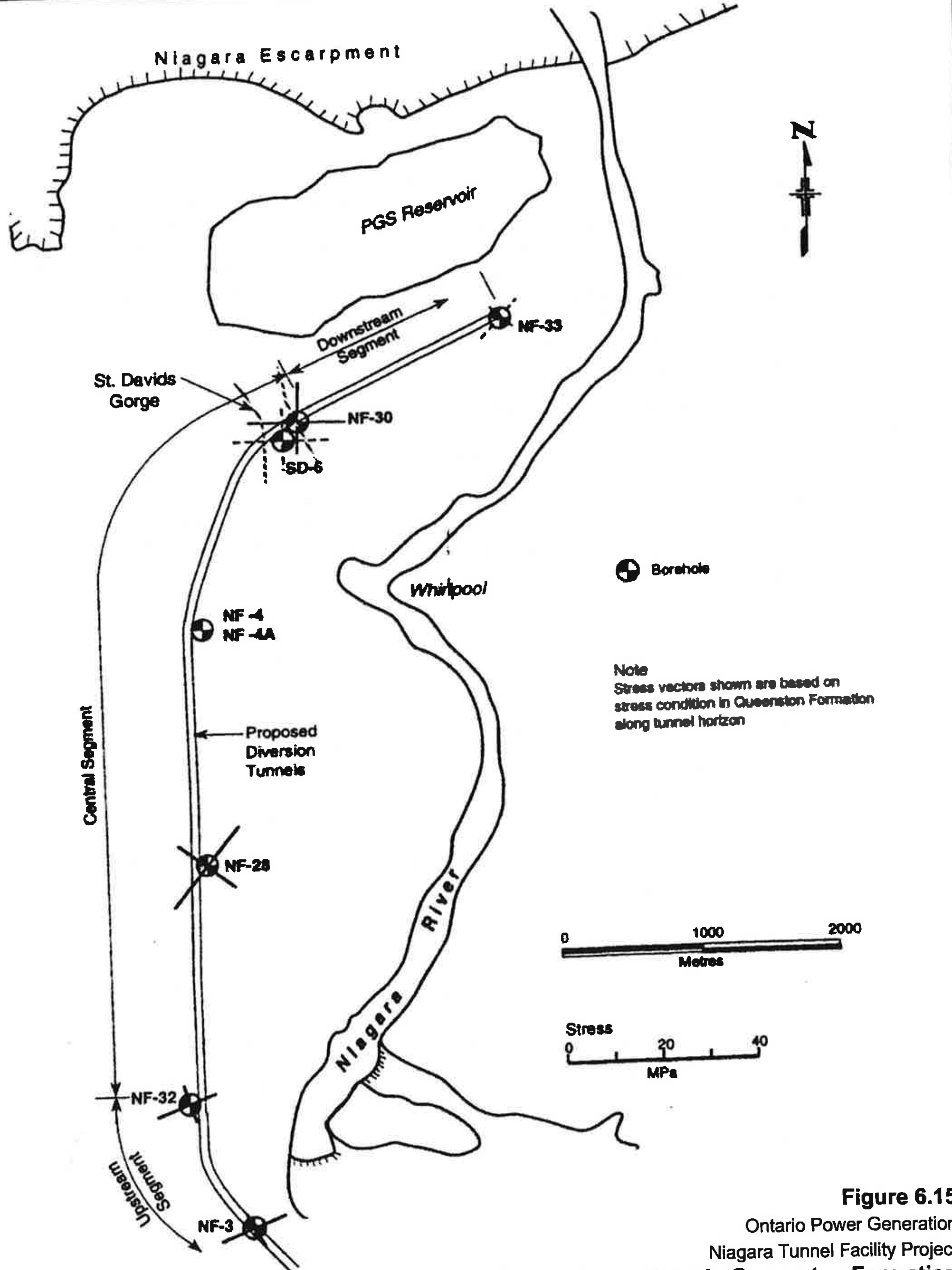

Figure 6.15
Ontario Power Generation
Niagara Tunnel Facility Project
Stress Orientations in Queenston Formation

6.5.2 Groundwater Quality

1 Groundwater from the primary bedding planes in the Queenston Formation is generally of connate origin. This connate water is supersaturated with salts. Seepage waters are acidic (lowest measured pH of 4.65) and have high chloride and sulphate levels, as well as high concentrations of some metals (including iron, magnesium, manganese, potassium, aluminum), ammonia, calcium, fluoride and phosphate. Chloride contents up to 296 000 mg/L and sulphate contents up to 1860 mg/L have been measured. Significant salt precipitation occurred along some primary bedding planes and also formed hollow stalactite-like precipitation features hanging from the crown of the adit in areas where bedding planes were exposed.

2 Generally, the percent difference between cations and anions in groundwater testing is less than 5%. However, in these brines, the differences in some cases are much greater, probably due to supersaturated conditions. In general, the chloride and metals levels were related to the amount of seepage at any location, with higher levels associated with less seepage: the higher the chloride concentration, the lower the sulphate concentration. The high chloride and sulphate contents are indicative of very corrosive groundwater conditions. Table 6.13 summarizes the groundwater quality.

3 Adjacent to the Niagara Gorge, the groundwater is relatively fresh and percolates from the surface through a system of open jointing into the rock formations.

6.5.3 Gas

1 During investigations for this project, methane gas was encountered along all those primary bedding planes encountered below the elevation of the Niagara River and St. Davids Gorges as shown in Figure 4.2. Gas pressures, however, were insignificant and flow usually reduced to insignificant levels within a few hours. However, minimal gas seepage was ongoing from some of the bedding planes intersected in the test adit for some months after its completion. Pockets of gases were encountered in Borehole NF-32 in the Rochester Formation and in the upper Lockport Formation near the proposed intake area.

6.6 In Situ Stress Conditions

1 The sedimentary rock strata in the Niagara Region are known to possess relatively high horizontal in situ stresses. The in situ stresses in the project area were determined using the overcoring method for shallow measurements of up to 40 m and the hydrofracturing method for tests at greater depth. In general, the horizontal stresses in the Niagara area are three to five times greater than the overburden stresses for the majority of the tunnel but the ratio can be greater than 5 at the inlet and outlet ends due to reduced overburden pressure.

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6.6.2 In Situ Stress Orientations

1 Figure 6.15 shows locations of boreholes with stress determinations and illustrates the average orientation of the stresses within the Queenston Formation. Results indicate that the orientation of the maximum principal stress in the central north-south segment of the diversion tunnel, obtained from impression packer tests on unambiguous vertical fractures, consistently lies within the northeast quadrant for all the rock formations. This is consistent with the regional stress trend in the Niagara area. Modifications of the regional stress regime by significant topographic features are evident from the results of measurements near the buried St. Davids Gorge and the Niagara River Gorge. These features tend to align the maximum horizontal stress parallel to the gorges.

6.6.3 Stress Magnitudes

magnitudes of the maximum and minimum stress in the Queenston Formation below the bottom of the gorge are comparable to those in this section.

(b) Upstream Section (Approximately Sta 7+600 to Sta 10+000)

(i) The horizontal stress values are plotted against elevation as shown in Figure 6.17. The maximum and minimum horizontal stresses in this section are about 17 and 10 MPa, respectively, and are relatively constant in the Queenston Formation.

(c) Downstream Section (Approximately Sta 0+000 to Sta 2+000)

(i) Results shown in Figure 6.18, including one test from Borehole NF-33, indicate that the maximum and minimum horizontal stresses at the elevation of the tunnel alignment as shown on the Concept Drawings are about 24 and 14 MPa, respectively.

(d) Stress Regime near the Trial Enlargement

(i) The boreholes for stress measurement in the downstream area are located in an area bounded by the Niagara River, the Niagara Escarpment and the St. Davids Gorge. The measured stress near the trial enlargement is lower than values in the diversion tunnels area due primarily to the stress relief effects of the Niagara River Gorge as all the measurements in the generation area are above the river bed

(ii) The three-dimensional (3D) in situ stress components were determined by the overcoring technique at the powerhouse area and at a stub near the trial enlargement area. The average in situ stresses in the area close to the trial enlargement are as follows:

Principal Stresses	Azimuth (deg)	Dip (deg)
$\sigma 1 = 11.9$ MPa	133	-13
$\sigma 2 = 9.6$ MPa	050	-15
$\sigma 3 = 4.6$ MPa	008	-70

(iii) The resolved vertical stress from the overcoring tests is 5.3 MPa which is about 30% higher than the overburden stress calculated by the weight of the overburden material. This difference in magnitude is considered to be within the expected range of variation of vertical stresses from the overburden pressure in sedimentary rock deposits. This result is considered to be applicable to the entire tunnel alignment.

(e) Stresses Above the Queenston Formation

(i) In the upstream sections, maximum and minimum horizontal stresses above the Queenston Formation are about 10.5 and 4.5 MPa, respectively, measured in the Power Glen Formation in Borehole NF-3. Stresses are higher in the central segment; up to 18 and 6.5 MPa for maximum and minimum horizontal stresses, respectively, measured in the Grimsby Formation. No stress measurements were made in the upper formations at the outlet.

4 Based on the selection of principal stresses along the designated sections of the tunnel alignment, four stress regimes were developed for purposes of various studies. These regimes were based on the following criteria:

(a) magnitude and orientation of the measured principal stresses given in Figures 6.16 to 6.18

(b) direction of the tunnel with respect to the stress field

* (c) confidence level in the data available

(d) major topographic features such as the St. Davids Gorge

(e) changes in tunnel vertical and horizontal alignments

5 The stress regimes for tunnel design purposes are presented in Table 6.14. Where there is greater confidence in orientation data, the stresses have been resolved with respect to tunnel orientation. However in some of the regimes, the maximum stress magnitudes are presented in the table.

6 Above the Queenston formation, stresses at the intake area will be a maximum horizontal stress of 10.5 MPA and a minimum stress of 4.5 MPA. Stresses in the outlet area will be 17 and 11 MPA (maximum and minimum horizontal values).

7 The vertical stresses are 30% higher than stresses calculated on the basis of overburden pressure. The horizontal stress values given in this section will be used as input into analyses and then reduced appropriately until no overall plastification of the rock mass occurs. These modified values for horizontal stress will be used in subsequent analyses.

6.7 Particular Characteristics of Shale Units

6.7.1 BTEX Occurrences

1 Four shale units were tested for the presence of naturally occurring hydrocarbons, in particular benzenes, toluenes, and xylenes (BTEX, also formerly referred to in the literature as BTX). The tests indicated that Queenston Formation appears to be inert with respect to BTEX but that the Rochester, Power Glen and Grimsby Formations

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(c) Downstream Section (Approximately Sta 0+000 to Sta 2+000)

(i) Results shown in Figure 6.18, including one test from Borehole NF-33, indicate that the maximum and minimum horizontal effective stresses in the Queenston formation near the St Davids Gorge and at the elevation of the tunnel alignment as shown on the Concept Drawings is about 24 and 14 MPa, respectively. No stress measurements were undertaken in the upper units or at higher Queenston Formation elevations near the outlet.

(d) Stress Regime near the Trial Enlargement

(i) The boreholes for stress measurement in the downstream area are located in an area bounded by the Niagara River, the Niagara Escarpment and the St. Davids Gorge. The measured stress near the trial enlargement is lower than values in the diversion tunnel central and upstream areas due primarily to the stress relief effects of the Niagara River Gorge as all the measurements in the generation area are above the river bed

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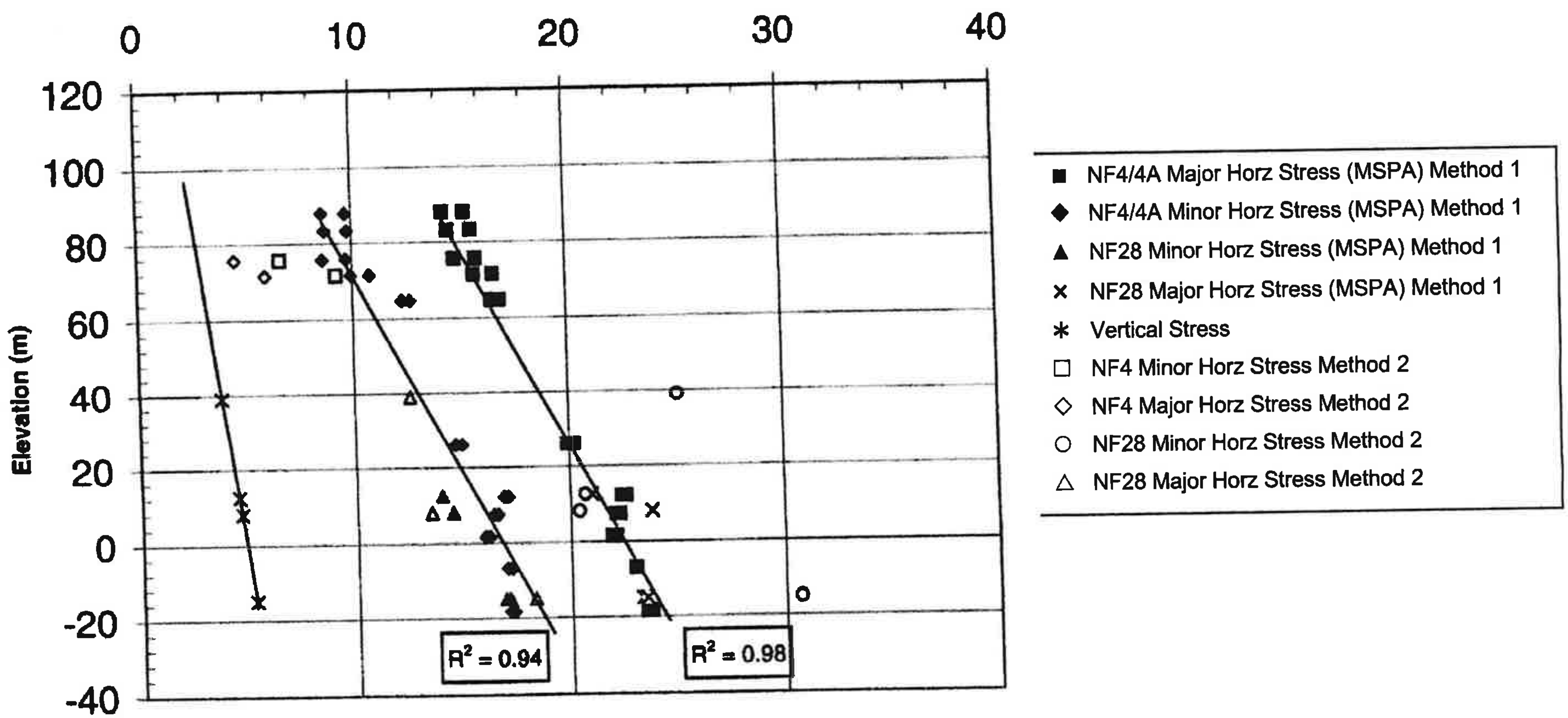
5-4 OBR-C



Method of Analysis:
1. Modified Stress Path Analysis (MSPA) (Hefney and Lo, 1992)
2. Conventional stress analysis on vertical fractures only.

Figure 6.16
Ontario Power Generation
Niagara Tunnel Facility Project
Sta 2+000 to 7+600

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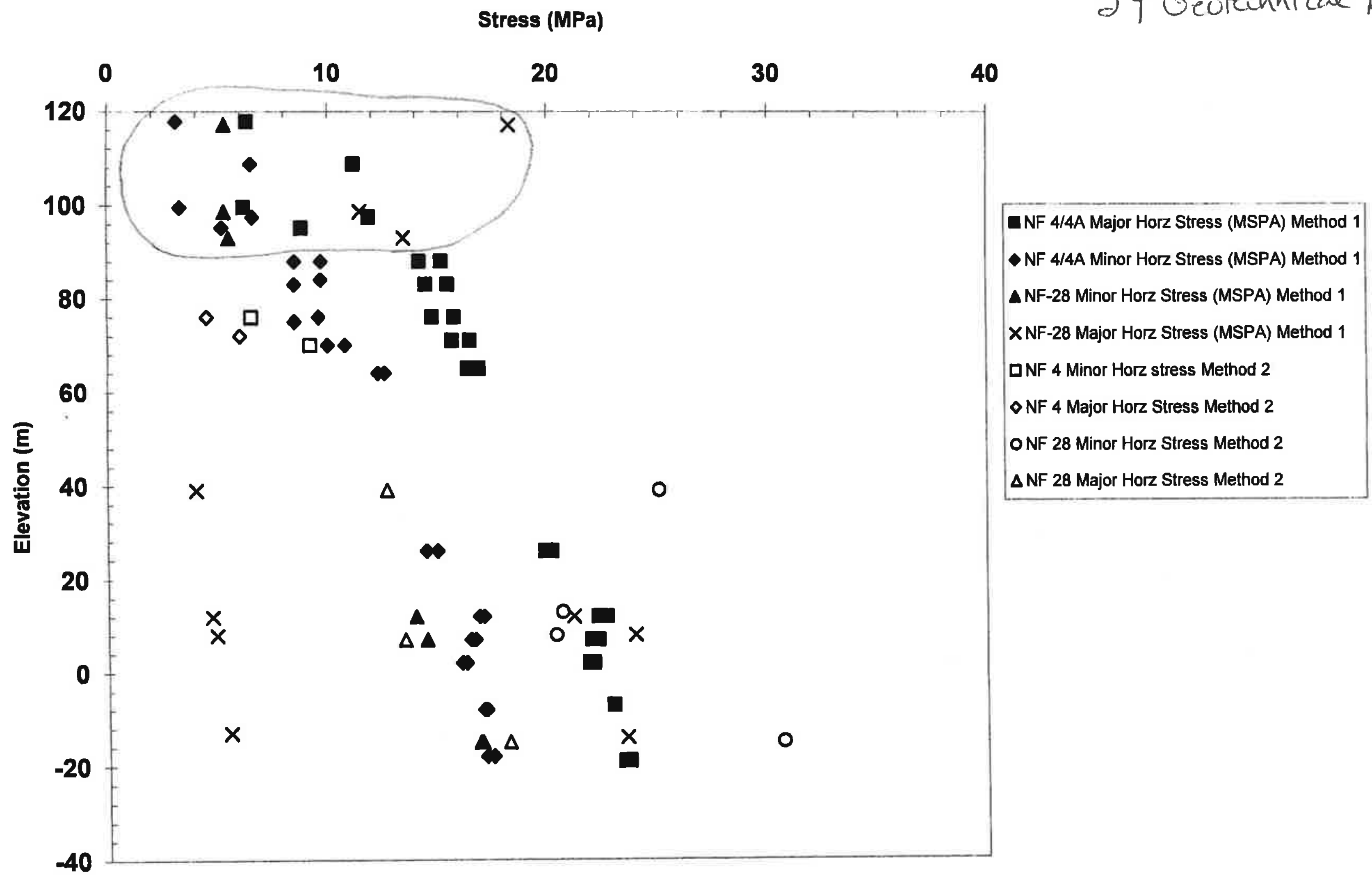
54 Geotechnical Report



Method of Analysis:

1. Modified Stress Path Analysis (MSPA) (Hefney and Lo, 1992)
2. Conventional stress analysis on vertical fractures only.

Figure 6.16
Ontario Power Generation
Niagara Tunnel Facility Project
Sta 2+000 to 7+600

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14504-2090

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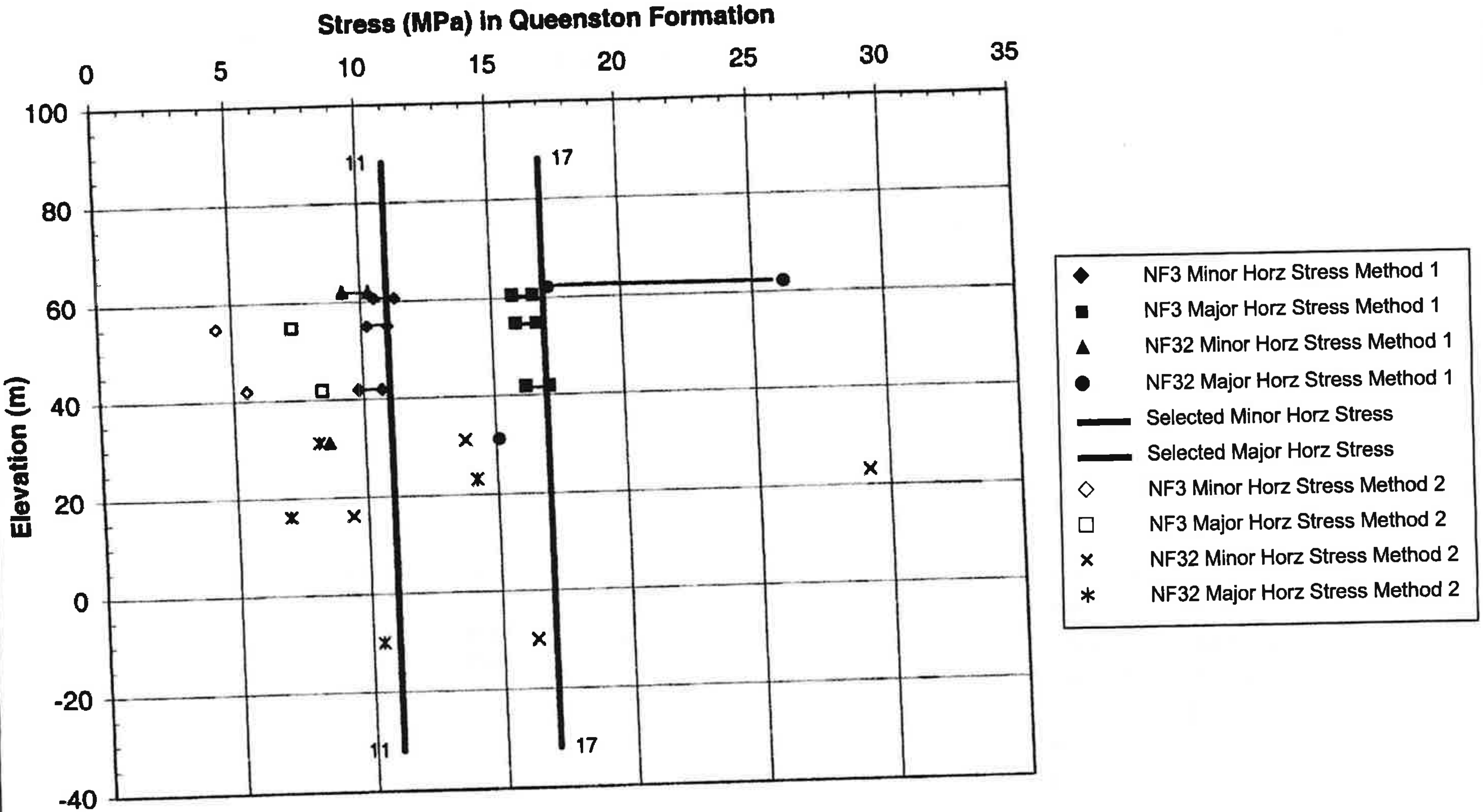
5.4 GBR-C



Method of Analysis:
1. Modified Stress Path Analysis (MSPA) (Hefney and Lo, 1992)
2. Conventional stress analysis on vertical fractures only.

Figure 6.17
Ontario Power Generation
Niagara Tunnel Facility Project
Sta 7+600 to ~10+000

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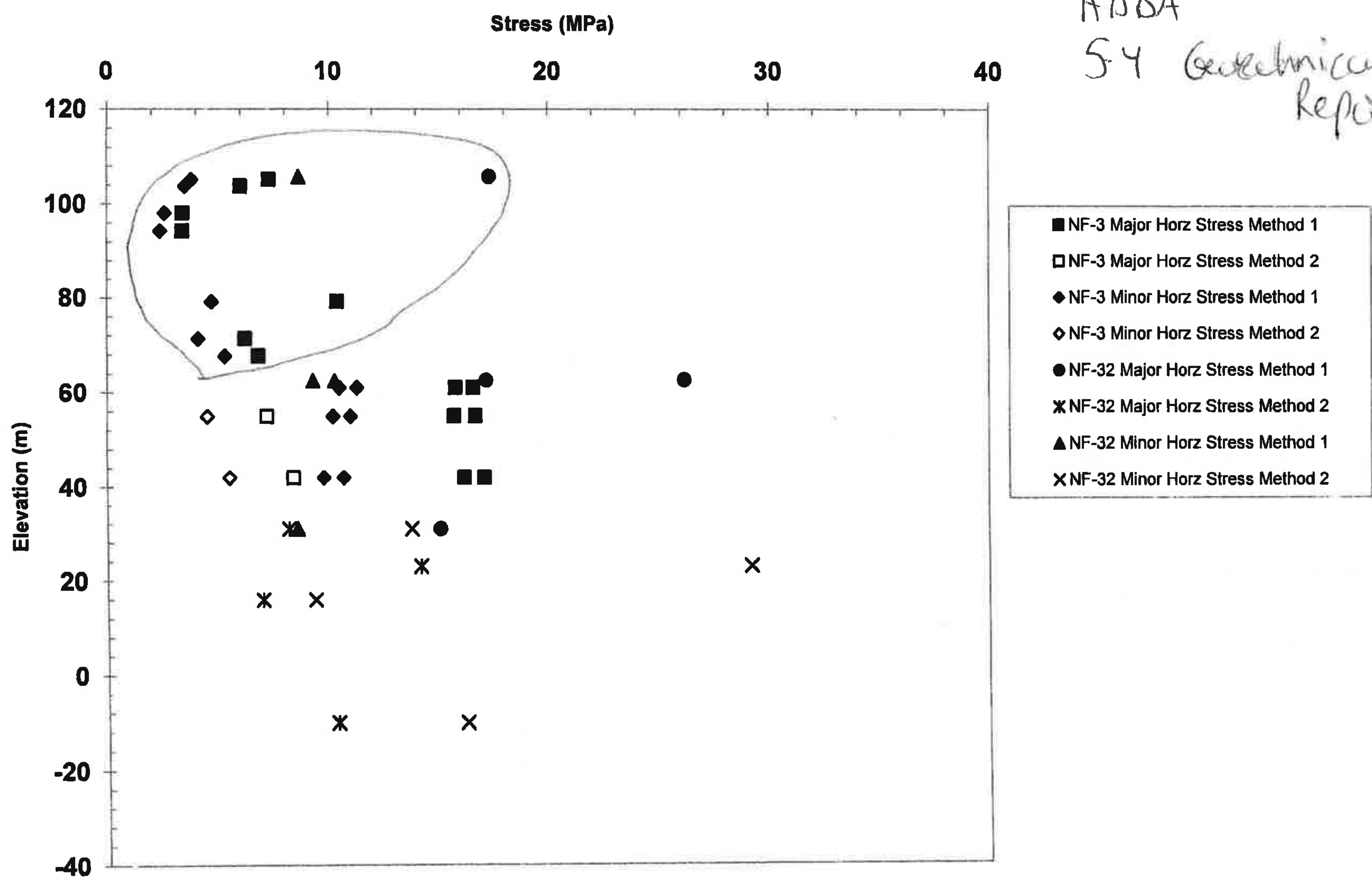
ADBA
5.4 Geotechnical Report



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1. Modified Stress Path Analysis (MSPA) (Hefney and Lo, 1992)
2. Conventional stress analysis on vertical fractures only.

Figure 6.17
Ontario Power Generation
Niagara Tunnel Facility Project
Sta 7+600 to ~10+000

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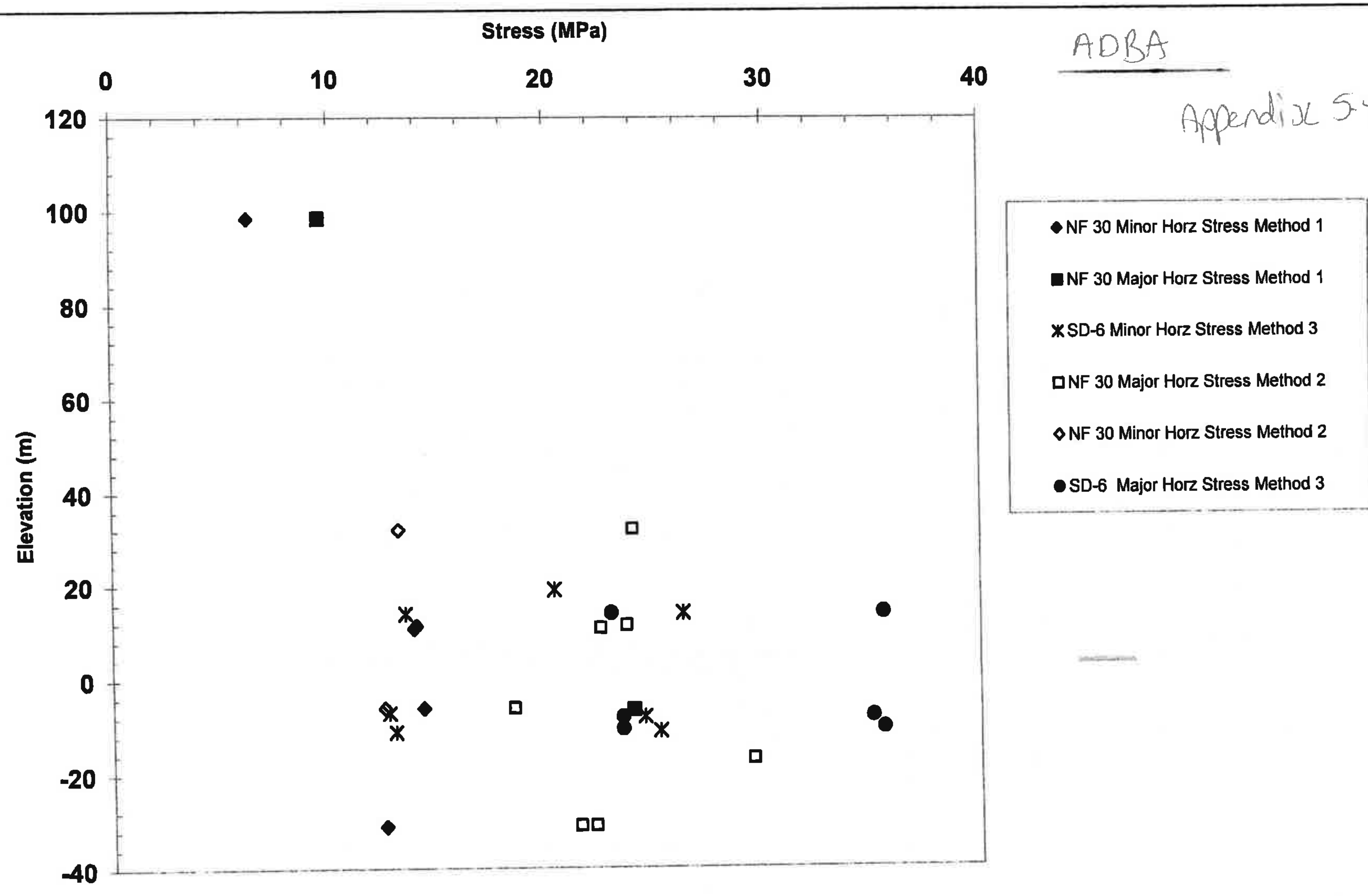
Appendix 5.4 Geotechnical Report



Method of Analysis:

1. Modified Stress Path Analysis (MSPA) (Hefney and Lo, 1992)
2. Conventional stress analysis on vertical fractures only.
3. Ljungguen and Anadei (1989) based on horizontal fracture data.

Figure 6.18
Ontario Power Generation
Niagara Tunnel Facility Project
Sta 0+000 to 2+000

9606446.3
14504-2000

(ii) both unwatered and operational tunnel conditions.

(c) The following parameters shall be included in the analyses:

(i) appropriate rockmass and bedding plane strength and deformability values as given in the GBR

(ii) appropriate in situ stresses as given in the GBR

(iii) Hoek-Brown residual rock mass strength parameters: $m_r = 1.0$, $s_r = 0.001$ (or equivalent)

(iv) plastic shear strain in rock for peak to post-peak: ranging from 0.5% to 2.0%

(v) design line for rock swelling rates as shown in Figure 8.1 (maximum free swell potential of 0.3% per log cycle is based on overall average of all free swell test results)

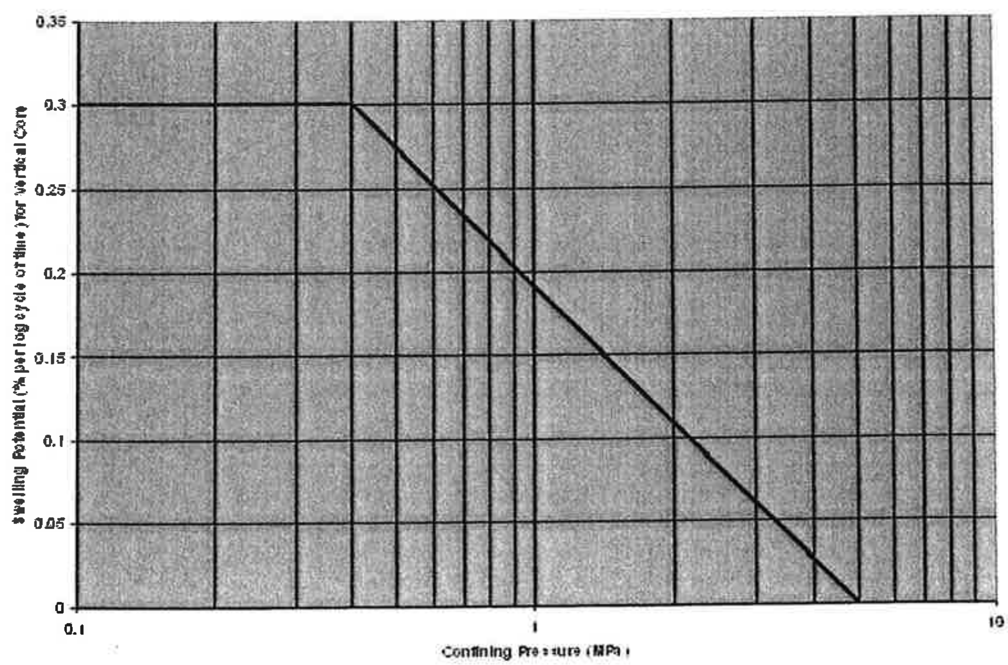


Figure 8.1 - Design Line for Rock Swelling

(vi) time steps up to 4.5 log cycles of time in increments of days

Table 6.14
Stress Regimes for Design Purposes

Approximate Station	Queenston Subunits	Horizontal Stress (respect to tunnel) (MPa)		Remarks
		Radial	Axial	
0+000 to 1+700	Q2 to Q10	15	23	tunnel is nearly parallel to minimum stress, transformed stresses quoted
1+700 to 3+800	Q2 to Q3	22	16	orientation of stress field uncertain and tunnel curves in this section; maximum values quoted
3+800 to 7+800	Q4 to Q5	19	17	stress orientation is known and consistent with regional stress field; transformed values quoted
7+600 to 10+000	Q6 to Q10	17	11	stress orientation uncertain and tunnel curves, maximum values quoted

(b) The modeling shall include

(i) analyses for both the deepest and shallowest tunnel sections in the Queenston formation

(ii) both unwatered and operational tunnel conditions.

(c) The following parameters shall be included in the analyses:

(i) appropriate rockmass and bedding plane strength and deformability values as given in the GR

(ii) The horizontal effective stress values given in Section 6.6 of the GR shall be used as input into an analysis that considers the relative stiffnesses of the various rock formations. The input in situ stresses shall then be reduced appropriately until no overall plastification of the rock mass occurs. These modified values for horizontal stress will be used in subsequent analyses.

For the Queenston Formation the following horizontal effective stresses are to be considered as input into the design.

Approxi-mate Station	Queenston Subunits	Horizontal Effective Stress (respect to tunnel) (MPa)		Remarks
		Radial	Axial	
0+000 to 1+700	Q2 to Q10	15	23	tunnel is nearly parallel to maximum stress, transformed stresses quoted
1+700 to 3+800	Q2 to Q3	22	16	orientation of stress field uncertain and tunnel curves in this section; maximum values quoted
3+800 to 7+800	Q4 to Q5	19	17	stress orientation is known and consistent with regional stress field; transformed values quoted
7+600 to 10+000	Q6 to Q10	17	11	stress orientation uncertain and tunnel curves, maximum values quoted

(iii) Hoek-Brown residual rock mass strength parameters: $m_r = 1.0$, $s_r = 0.001$ (or equivalent)

3.3.4 Rock Mass Types

The rock mass types (RT) are defined using relevant geotechnical rock volumes including lithology, discontinuities and tectonic structures. The characteristics of the rock mass types are governed by:

- Lithology
- Properties of discontinuities
- Strength parameters of intact rock
- Conditions affecting parameters of intact rock and of rock mass

Six characteristic geotechnical parameters are used to define thirteen rock mass types for the Niagara Facility Tunnel project which are summarized in the following table 3.1.

3.3.5 Geotechnical Parameters

The geotechnical rock mass parameters are derived based on Hoek-Brown's mass law described in detail in [3.6]. The general form of the Hoek-Brown's failure criterion is:

$$\sigma_1 = \sigma_3 + \sigma_{ci} \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s \right)^a$$

σ_1, σ_3 are the major and minor principal effective stresses

m_b is the Hoek-Brown constant for rock masses

s, *a* are parameters describing rock mass properties

σ_{ci} is the uniaxial compressive strength of the intact rock (obtained from [3.19], table 6.3)

The Hoek-Brown criterion thus establishes a connection between the principal effective stresses. The rock mass parameters m_b, *a* and *s* can be derived by means of the following parameters:

- Hoek-Brown constant for intact rock *mi*
- Geological Strength Index *GSI*

Values for Hoek-Brown constant *mi* were derived using the *mi*-chart provided with the software RocLab [3.12]. The *GSI* is a parameter introduced by Hoek in 1994, providing a numerical rating of the rock masses based on the structure and discontinuity surfaces of the rock mass. The *GSI* values were derived by evaluating average joint spacing and surface conditions of the individual rock formations, using the *GSI*-chart provided in [3.1] (see also Appendix 3.2).

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s, a are parameters describing rock mass properties

σ_{ci} is the uniaxial compressive strength of the intact rock (obtained from [3.19], table 6.3)

The Hoek-Brown criterion thus establishes a connection between the principal effective stresses. The effective stresses will take into account 100% of the pore water pressure in the rock. The calculation of rock mass strength will be based on confining pressures that account for effective stresses.

The rock mass parameters m_b, a and s can be derived by means of the following parameters:

- Hoek-Brown constant for intact rock *mi*
- Geological Strength Index *GSI*

Values for Hoek-Brown constant *mi* were derived using the *mi*-chart provided with the software RocLab [3.12]. The *GSI* is a parameter introduced by Hoek in 1994, providing a numerical rating of the rock masses based on the structure and discontinuity surfaces of the rock mass. The *GSI* values were derived by evaluating average joint spacing and surface

94

3.4 Rock Mass

Rock mass behaviour is decisive for the design of the required initial support and final lining of a tunnel. Various methods have been applied in order to determine rock mass behaviour along the proposed tunnel alignment including block stability analyses and FE-modelling. Details of the applied methodology are summarized in [3.17].

3.4.1 Boundary Conditions

The boundary conditions influencing the rock mass behaviour can be listed as follows:

- Rock mass properties
- In situ stress conditions
- Groundwater conditions
- Orientation of the opening
- Dimension and shape of the opening

3.4.1.1 Rock Mass Properties

Rock mass properties to be encountered along the tunnel are presented in chapters 3.3.4 and 3.3.5.

3.4.1.2 In Situ Stress Conditions

Extensive in situ testing was carried out in order to determine stress conditions along the tunnel alignment. The results to be considered for the tunnel design are presented in table 6.14 of the [3.19]. These results cover the Concept Alignment which is basically situated within the Queenston Formation.

In situ horizontal stress conditions included in GBR will be adopted for design. Vertical stress is assumed to be governed by the overburden only with the exception of the outlet section. There 3D in situ stress measurements indicate, that vertical stresses are 30% higher than stresses induced by overburden only.

The following table 3.3 summarizes the in situ stress conditions considered for the design of the proposal and will be updated in the detail design.

Table 3.3: Stress Regimes for Design Purposes

In Situ Stress Conditions along the Proposed Tunnel Alignment					
Tunnel section	Horizontal Stress (respect to tunnel)		Vertical Stress		
	[MPa]	[MPa]	[MPa]		
	Radial	Axial	min	max	mean
0+000-2+840	17	11	0,5	2,3	1,4
2+840-7+070	19	17	2,3	3,9	3,1
7+070-8+900	22	16	3,3	3,4	3,3
8+900-10+421,380	15	23	0,6	4,5	2,6

95

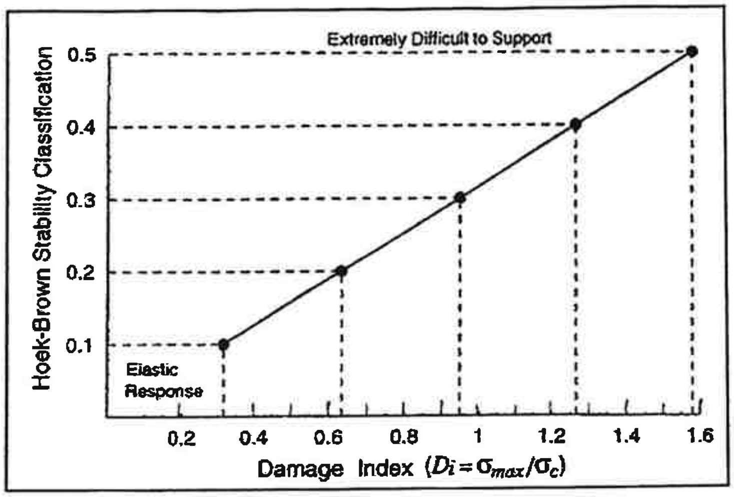


Figure 3.2: Correlation between Damage Index (*Di*) and the Hoek-Brown Stability Classification

The depth of stress induced failure was estimated using the below formula.

$$\frac{Rf}{a} = 0{,}49(\pm 0{,}1) + 1{,}25 * Di$$

Rf is the depth of failure measured from the tunnel center

a is the tunnel radius.

Both procedures are derived from [3.8]. The results of the calculations are summarised in Appendix 3.3 for the four tunnel sections shown in Table 3.3. They were used for choosing the necessary model for evaluating rock mass behaviour (see Chapter 3.4.2).

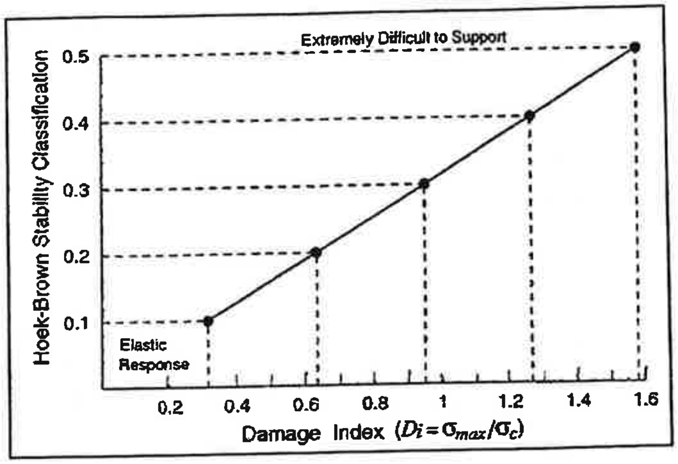
3.4.1.3 Groundwater Conditions

Groundwater can have a major impact on rock mass behaviour during tunnel construction. Groundwater conditions along the proposed tunnel alignment are expected to vary significantly due to the encountered rock mass properties. Significant groundwater inflow is to be expected within the Lockport and the De Cew Formations. It is assumed that groundwater has no influence on rock mass behaviour of those formations, due to their rock mass properties.

Groundwater inflow in the below situated formations is very limited.

Within shale formations, groundwater can trigger rock mass swelling if appropriate clay minerals are available.

96



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Groundwater inflow in the below situated formations is very limited. Thus it is assumed that groundwater has, despite rock mass swelling, no influence on rock mass behaviour.

Within shale formations, groundwater can trigger rock mass swelling if appropriate clay minerals are available.

significantly lower. It is assumed that surface water inflow from the existing channel is causing dilution of the groundwater.

The Lockport formation shows highly variable chloride and sulphate contents due to the changing influence of surface water all along the tunnel alignment.

The results of the evaluation of the hydrochemical testing with respect to the tunnel alignment are summarized in Appendix 3.1. The assumed distribution of the hydrochemical properties along the tunnel alignment is shown in [3.14].

3.4.1.4 Orientation of Opening

The orientation of the opening relative to the major discontinuity sets governs the stress relevant for the tunnel design. It also has a major impact on size, shape and stability of rock wedges formed by the intersection of discontinuities and the tunnel opening. Therefore the orientation of the opening is considered in the block stability analysis as well as the FE-analysis.

3.4.1.5 Dimension and Shape of Opening

The distribution of stress around the tunnel opening is governed to a large extent by the size and shape of the opening. They also affect size and shape of potentially unstable blocks during tunnel excavation. Therefore the size and shape of the opening are considered in the block stability analysis as well as in the FE-analysis.

The bored part of the Diversion Tunnel has a circular excavation cross section of 14.44 m diameter. The circular cross section is favourable for redistribution of stresses, which develop in the rock mass around the excavation opening. Rock mass loosening will such be minimized.

A short section of tunnel, adjacent to the Intake and Outlet structures, is excavated by mining methods. The tunnel cross section has to be changed from circular to square on a length, which corresponds to approximately one tunnel diameter. The square end of excavation is up to 19 m wide. The excavation cross section at the interface to the bored tunnel is horse-shaped and 16 m wide and 17 m high at its top.

The cross sections for channels at the Intake and the Outlet area is generally rectangular.

3.4.2 Rock Mass Behaviour Types

In total 8 basic rock mass behaviour types have been identified along the tunnel alignment. It has to be mentioned that some rock mass behaviour types can coexist along a tunnel section since some types represent short term rock mass behaviour (e.g. wedge failure) and some types represent long term rock mass behaviour (e.g. swelling or squeezing rock). During future design phases it may be found reasonable to refine this rock mass classification by partitioning the identified rock mass behaviour types into more subtypes.

classified as highly corrosive and concrete aggressive. Excluded from this general assumption has to be a tunnel section around borehole NF4 where chloride and sulphate contents are significantly lower. It is assumed that surface water inflow from the existing channel is causing dilution of the groundwater.

The Lockport formation shows highly variable chloride and sulphate contents due to the changing influence of surface water all along the tunnel alignment.

The results of the evaluation of the hydrochemical testing with respect to the tunnel alignment are summarized in Appendix 3.1. The assumed distribution of the hydrochemical properties along the tunnel alignment is shown in [3.14].

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The cross sections for channels at the Intake and the Outlet area is generally rectangular.

3.4.2 Rock Mass Behaviour Types

* [The rock mass behaviour types in this section were developed during the proposal and are for information only]

99

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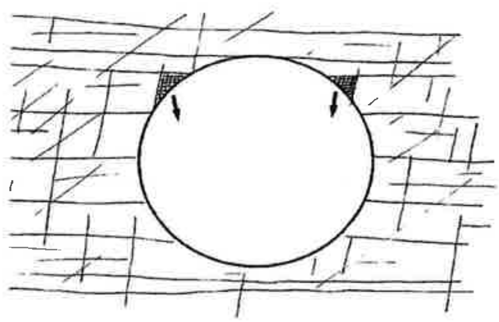
Note that rock mass behaviour types are defined considering an endless long tunnel without any construction stages and support measures.

3.4.2.1 Behaviour Type 1: Stable Rock

Rock mass behaviour was analysed using block theory. The block modelling was carried out applying the software UNWEDGE [3.13].

<table>
<tr><th colspan="3">Rock Mass Behaviour: Stable Rock Mass</th></tr>
<tr><td colspan="3">
Sketch of assumed rock mass failure; wedges not in scale</td></tr>
<tr><td>Formations</td><td colspan="2">Lockport, De Cew, Irondequoit, Reynales, Whirlpool</td></tr>
<tr><td>Characteristics of Discontinuities</td><td><u>Bedding:</u>
Persistence: >20m
Spacing: dm - m
Roughness: rough to slightly rough, fresh to slightly weathered</td><td><u>Joints:</u>
Persistence: <10m
Spacing: dm - m
Roughness: rough to slightly rough, fresh to slightly weathered</td></tr>
<tr><td>In Situ Stress Conditions</td><td colspan="2">in situ stresses do not exceed rock mass strength</td></tr>
<tr><td>Groundwater Conditions</td><td colspan="2">groundwater conditions are varying from wet to flowing, significant inflow will occur close to ground surface (Lockport and De Cew Formation)</td></tr>
<tr><td>Rock Mass Behaviour</td><td colspan="2">local, gravity controlled failure of rock wedges induced by discontinuities; max. wedge size up to several dm³; groundwater has no influence on rock mass behaviour</td></tr>
<tr><td>Deformations</td><td colspan="2">minor deformation < 5mm, which stabilize quickly</td></tr>
</table>

100

(4) **Effects of Non-Conformance.** If the Contractor identifies anything which does not conform to the quality assurance program set out in Section 2.12(c)(1), the Contractor will promptly correct such non-conformance (unless the Contractor proposes to "use as is") and deliver a Notice in the form of Appendix 2.12(c)(4) to OPG's Representative reporting the corrective action taken by the Contractor or that the Contractor proposes to "use as is". OPG's Representative will return the Notice in the form of Appendix 2.12(c)(4) to the Contractor indicating OPG's agreement with the proposed disposition (with or without additional terms detailed in Appendix B to the Notice) or directing the Contractor to comply with the Contractor's Proposal Documents or the Final Submittals, as the case may be.

2.13 Construction

(a) **Direction and Competent Supervision.** The Contractor will perform (including all direction, supervision and inspection of) the Work competently and efficiently, devoting such attention and applying such skills and expertise as may be necessary to perform the Work in accordance with this Agreement. The Contractor will at all times maintain good discipline and order at the Site. The Contractor will be solely responsible for the means, methods, techniques, sequences and procedures used to perform the Work (except with respect to the INCW Part Project, in which case OPG will be the "constructor" and will have the control necessary to effectively carry out that role, as described more particularly in Section 2.20). The Contractor will keep OPG advised as to the quality and progress of the Work and the Tunnel Facility Project in such manner and at such times as OPG may request from time to time.

1

(b) **Temporary Structures and Facilities.** Except with respect to the INCW Part Project (as discussed on Section 2.20), the Contractor will have the sole responsibility for:

(1) the design, erection, operation, maintenance and removal of all temporary structures and facilities at the Site; and

(2) the design and execution of construction methods required in the use of such structures and facilities.

(c) **Time for Performance of the Work.** The Contractor may perform the Work at any time except to the extent that performing the Work is prohibited or restricted:

(1) by Applicable Laws;

(2) by the Approvals;

(3) in the Summary of Work; or

(4) elsewhere in this Agreement.

2 **3.3 No OPG Control Over the Work**

Except as may be necessary to fulfill its role as "Constructor" with respect to the INCW Part Project, OPG will not supervise, direct, have control or authority over, or otherwise be responsible for:

(a) the Contractor's means, methods, techniques, sequences or procedures respecting the Work; or

(b) the safety programs and precautions used in respect of the Work, subject to OPG's rights and obligations under the *Occupational Health and Safety Act* (Ontario).

OPG will not be responsible for any failure of the Contractor to comply with any Applicable Laws, Approvals or this Agreement in performing the Work. The Contractor acknowledges exclusive control over and commercial responsibility for any and all means, methods, techniques, sequences or procedures employed or necessary to complete the Work, for the Contract Price and in accordance with the Contract Schedule.

3.4 Hazardous Conditions

(a) **Division of Responsibility.** OPG will be responsible for the costs of dealing with any Hazardous Materials to the extent such Hazardous Material presents a material danger to any Person performing the Work (a "**Hazardous Condition**") encountered at the Site that was not generally or specifically identified in this Agreement to be part of the Work. To the extent OPG is responsible for such costs and such Hazardous Condition has the effect of materially increasing the cost or time of performing the Work, then such change will be treated as a Project Change Directive issued by OPG under Section 5.1. Notwithstanding the previous sentence, the Contractor will be responsible for any Hazardous Condition caused by or resulting or arising from the performance of the Work or brought on the Site by or on behalf of the Contractor and no adjustment will be made to the Contract Price or Contract Schedule in respect of such Hazardous Condition.

(b) **Actions on Discovery.** Immediately on the discovery of a Hazardous Condition on the Site, the Contractor will:

(1) in accordance with prudent practices, act to contain the Hazardous Condition in order to minimize the impact of the Hazardous Condition;

(2) stop all Work in the area that could reasonably be affected by the Hazardous Condition, subject to Section 2.4(j); and

(3) verbally notify OPG of the discovery and confirm by Notice within 24 hours of the discovery.

3

(1) impose on OPG responsibility for the sequencing, scheduling or progress of the Work;

(2) be deemed to confirm that any schedule is a reasonable plan for performing the Work in accordance with the detailed contract schedule;

(3) affect or change the Contractor's obligation to perform the Work in accordance with this Agreement; or

(4) otherwise have the effect of transferring any obligation under this Agreement from the Contractor to OPG or otherwise have the effect of amending this Agreement.

(c) **Adherence to Schedules.** The Contractor will adhere to the Contract Schedule. The Contractor will provide OPG with a monthly progress schedule, setting out the status and progress of the Work and any deviations or anticipated deviations from the Contract Schedule or the detailed contract schedule described in Section 2.7(a)(3). To the extent that the Contract Schedule has not been, or is anticipated not to be, satisfied, the Contractor will indicate the total number of days set aside for contingencies that will be used and will provide OPG with satisfactory assurances, including, at the Contractor's cost, recovery plans, involving all necessary additional resources, acceptable to OPG, that the Contract Schedule will be restored. There will be no changes to the Contract Schedule except as provided by Section 5 and Section 6 of this Agreement. OPG may refuse to approve any changes to the Contract Schedule in its sole and absolute discretion.

(d) **Daily Record.** The Contractor will maintain a detailed daily record of the progress of the Work, the number of personnel of all categories at the Site, the Goods delivered to the Site and all such other items deemed necessary to record.

(e) **Continuing the Work.** Notwithstanding any term in this Agreement, the Contractor will not stop or delay the performance of Work, in whole or in part, on account of any Dispute or pending resolution of any such Dispute between the Contractor and OPG or between the Contractor and any other Person and will continue to perform the Work in a timely manner and continue to adhere to the Contract Schedule, except to the extent, if any, expressly directed to do so by OPG in a Project Change Directive, provided that the Contractor may suspend the performance of the Work, if OPG has not paid the amounts required under Section 7.3(b) within 30 days of the date OPG is required to make payment under Section 7.3(b).

2.8 Submittals

(a) **Submission of Submittals.** The Contractor agrees to provide each Submittal described in Appendix 2.8(a), in addition to other Submittals as required pursuant to this Agreement all in accordance with the requirements set out in Appendix 2.8(a). The Contractor agrees to provide each Submittal to OPG (as set

8 Execution and Delivery Strategy

8.1 Project Phasing

As indicated in Section 1 of this plan, the Niagara Tunnel Project will be executed in two distinct phases as follows:

- **Phase 1 (Planning and Procurement Phase)** – Project activation, procurement of construction and service contracts, liaison and coordination with approving agencies and others, agreements with stakeholders.
- **Phase 2 (Design/Construction and Commissioning Phase)** – Detail design and construction of the diversion tunnel and related works, commissioning of the tunnel and project closeout.

8.2 Project Resources

The Niagara Tunnel Project is being designed and constructed by a Design/Build Contractor. OPG has not designed or constructed hydroelectric facilities, including major diversion tunnels, for several decades and as a result the specialist skills required for this work are not available within the organization. Therefore, for this project, some of the Owner's activities have been assigned to an outside consultant, acting as OR. It may be necessary for the OR to engage specialist contractors to perform specific assignments.

OPG has labour agreements with the Power Workers Union and the Society of Energy Professionals. These collective agreements include requirements for OPG to gain agreement and/or to engage in discussions with the union representatives to contract out work. Approval/discussions to contract out this work have been completed with both unions.

8.3 Contracting Approach

OPG had previously determined that the diversion tunnel be implemented through a Design/Build Contract. This approach has been reviewed and refined based on lessons learned on other projects, current contracting practices, the latest information on tunnel technology, and the objectives of the Project. The Design/Build approach was maintained in order to maximize the degree of certainty of cost outcome due to single point accountability for both design and construction. The Design/Build approach also would permit OPG to canvass the design creativity of the marketplace instead of being restricted to a single design, which would result in cost or schedule savings. The contract was structured to reward early project completion, and better than target tunnel flow performance while providing for competitive pricing from the contractors.

8.4 Risk Allocation

At the time that this Project was re-activated in June 2004, OPG management intended to pursue a fixed price contract that allocated the risk of differing site conditions (subsurface geological) to the Contractor. Following significant discussion on this subject, including review of industry "norms", it was concluded that such risk should be borne by OPG. Other risk allocation decisions were determined through extensive discussed between OPG, OR and outside legal counsel.

A Geotechnical Baseline Report (GBR), initiated during the proposal stage and finalized prior to contract award, will form the basis for evaluating claims for Differing Site Conditions (DSCs). An innovative multi-step process has been adopted for the preparation of the GBR. An initial

V2
DBA

9 Organization, Roles and Responsibilities

9.1 General

This section of the PEP identifies the organizational approach envisaged for overall management of the Project and describes roles and responsibilities for key members of the project team.

A functionally integrated project management team, consisting of OPG and consultant (OR) staff, has been formed to manage the project. This management team will be empowered with adequate authority and have access to appropriate resources to successfully execute the project. They will be responsible for accomplishing Project goals by undertaking project planning and project configuration and by overseeing and monitoring all aspects of design, construction, commissioning and project closeout.

9.2 Division of Work

A coordinated and sustained multi-disciplinary effort by OPG staff, the OR and the Contractor will be essential for the successful execution of the Project. The division of functional responsibility must be clearly understood and adhered to by all project participants. Table 9.1 provides a summary of the allocation of project responsibilities.

Table 9.1 Functional Responsibilities

Function	Responsibility		
	OPG	OR	Contractor
Project Setup			
Project Direction and Oversight	R		
Risk Assessment – Phase 1	R	C	
Risk Assessment – Phase 2	C	R	
Risk Management Plan	A	R	
Risk Management Plan – Contractor			R
Legal – Corporate and Project	R		
Legal – Real Estate	R	C	
Real Estate Acquisition	R	S	
Financial Modeling Economic Evaluations/Business Case	R	S	
Project Charter	R	S	
Project Execution Plan	A	R	
Engage OR and administer OR contract	R		
OPG Union Agreements	R		
Insurance/Bonding/Tax Requirements	R	C	
EOI response	C	R	
Project Procurement Planning/Execution			
Contract Execution and Other Commitments	R	C	
Procurement – Policy/Strategy	R	C	
Procurement – Execution	C	R	
Functional Requirements (engineering)	A	R	
Geotechnical Baseline Report Preparation	C	R	C
Evaluation of Proposals	A	R	
Coordination of proposal invitation process	I	R	

105

Function	Responsibility		
	OPG	OR	Contractor
Project Permits/Approvals			
EA Conditions of Approvals (see Table 7.1a)			
Permit Applications (see Table 7.1b)			
Community Impact Agreement	R	C	C
Project Design/Construction			
Preliminary/Detail Design	C	C	R
Construction		C	R
Construction Monitoring including environmental	I	R	C
Construction Safety Management (OPG Owner Only)			R
Construction Safety Compliance Monitoring (OPG Owner Only)	I	R	C
Construction Safety Management (Part Project)	C	R	
Tunnel Flow Test	A	C	R
Start Up and Commissioning (commitments beyond Project execution must be approved by NPG	C	C	R
Engineering Support to OPG	C	R	
Facilitate DRB Establishment	I	R	C
Construction monitoring/claims management	I	R	
Quality monitoring	I	R	
Environmental compliance monitoring	I	R	
Project Controls/Reporting			
Contract Administration	C	R	
Project Cost Estimate	A	R	
Project Scheduling	A	R	
Contract Scheduling	A	R	
Project Controls including Change Control	A	R	C
Project Accounting	R	C	
Projection of Cash Requirements	R	C	
Project Payments	R	C	
Project Reporting (Except Cash Reporting)	C	R	
Project Closeout Documentation	C	R	C
Document Management	C	R	
Action Tracking	C	R	C
Financial reporting requirements	R	C	
WBS development	A	R	
Design/Build contract administration	I	R	C
Change Control Board (CCB) setup	C	R	
Change management	A	R	
Project Communications			
Third Party Liaison (see MOU and Protocols)			
Third Party Agreements – Liaison	C	R	
Public and Media Relations/Shareholder Contact	R	C	C
Public communications support	R	I	I
Communication plan	R	C	I
Citizen Complaints (see MOU Protocol)			

R = Responsible for executing the work
A = Must approve (including review)
C = Must be consulted (includes support, review and other input)
I = Must be informed (for information only, no action needed)

	Rock Condition	Rock Characteristics	% of Total Bored Tunnel Length
Formations above Queenston Formation	1	• stable rock	0.16
	2	• loosening of rock in crown or localized area	2.73
	3	• unstable or closely broken rock • frequent overbreak due to discontinuities	10.59
	4	• unstable or closely broken rock • continuous overbreak due to any of: o discontinuities o sidewall spalling o invert heave	5.28
Queenston Formation	4Q	• continuous overbreak due any of: o sidewall spalling o invert heave • crown is more than 3m from bedding plane	23.69
	5	• continuous overbreak due to any of: o sidewall spalling o invert heave o slabbing • squeezing rock conditions • rock pressure generally exceeding rock mass strength • crown is within 3m of bedding plane	46.90
	6	• continuous overbreak due to any of: o sidewall spalling o invert heave o slabbing • squeezing rock conditions • rock pressure generally exceeding rock mass strength • closely broken shear and thrust zones • crown is within 3m of bedding plane • all other conditions requiring greater support than under Conditions 4Q and 5	10.65

8.1.4 Rock Mass Boreability

1 A linear cutter test was carried out on a single block sample from the Queenston Formation to evaluate the basic requirements of a TBM system. Note that the block sample was taken from the test adit excavation, and that no uniaxial strength data is available for this particular block sample for comparative purposes.

2 This cutter test is not considered to be representative for the full range of conditions in the Queenston Formation and obviously does not address the rocks above the Queenston. The the linear cutter test shall not be used solely to assess the boreability.

3 Instantaneous penetration rate above the Queenston is an average of 2.70 m/h; instantaneous penetration rate within the Queenston is an average of 2.40 m/h. Average penetration rates are based on average thrust values of 150 kN per cutter..

Appendix 1.1(j)
Baseline

Rock Support Table

	A	B	C	D	E	F	G	H	I	J	K	L
1												
2	NIAGARA TUNNEL FACILITY PROJECT :											
3	Calculation of Aggregate Rock Conditions											
4												
5	BASELINED ROCK CONDITIONS (Ref: GBR Section 8.1.3.7)		Calculated Baseline Lengths of Each Rock Condition		Contract Unit Rates (Ref: Appendix 1.1(g))		Calculated Baseline		Actual Measured Lengths of Each Rock Condition		Calculated Actual	
6	Rock Condition	Percentage of Total TBM Bored Tunnel Length					Days	Total Cost			Days	Total Cost
7		(%)	(m)		(m/d)	($/m)	(d)	($)	(m)	(%)	(d)	($)
8												
9	1	0.16%	=+B9*$B$19		28.35	$9383	=+C9/$E9	=+C9*$F9	INPUT	=+I9/$I$17	=(J9*$B$19)/$E9	=(J9*$B$19)*$F9
10	2	2.73%	=+B10*$B$19		27.00	$10289	=+C10/$E10	=+C10*$F10	INPUT	=+I10/$I$17	=(J10*$B$19)/$E10	=(J10*$B$19)*$F10
11	3	10.59%	=+B11*$B$19		24.30	$11866	=+C11/$E11	=+C11*$F11	INPUT	=+I11/$I$17	=(J11*$B$19)/$E11	=(J11*$B$19)*$F11
12	4	5.28%	=+B12*$B$19		20.25	$13953	=+C12/$E12	=+C12*$F12	INPUT	=+I12/$I$17	=(J12*$B$19)/$E12	=(J12*$B$19)*$F12
13	4O	23.69%	=+B13*$B$19		17.00	$14933	=+C13/$E13	=+C13*$F13	INPUT	=+I13/$I$17	=(J13*$B$19)/$E13	=(J13*$B$19)*$F13
14	5	46.90%	=+B14*$B$19		15.60	$17073	=+C14/$E14	=+C14*$F14	INPUT	=+I14/$I$17	=(J14*$B$19)/$E14	=(J14*$B$19)*$F14
15	6	10.65%	=+B15*$B$19		12.00	$26272	=+C15/$E15	=+C15*$F15	INPUT	=+I15/$I$17	=(J15*$B$19)/$E15	=(J15*$B$19)*$F15
16												
17	Aggregate Totals	100%	=SUM(C9:C15)		Aggregate Totals		=SUM(G9:G15)	=SUM(H9:H15)	=SUM(I9:I15)	=SUM(J9:J15)	=SUM(K9:K15)	=SUM(L9:L15)
18												
19	Actual Measured Overall TBM Bored Tunnel	INPUT			plus 5% variance		=+G17+G17*0.05	=+H17+H17*0.05	difference above or below 5% variance		If (K17-G19)>0 = K17-G19 else If(K17-G20)<0 = K17-G20, else=0	If (L17-H19)>0 = L17-H19 else If(L17-H20)<0 = L17-H20, else=0
20					minus 5% variance		=+G17-G17*0.05	=+H17-H17*0.05				
21												

22 NOTES:
23 1. Cells for input INPUT
24 2. For measurement purposes, the portion of the tunnel under the St. Davids Gorge, 400m of Type 5 and 400m of Type 6 Conditions will be deemed to have been encountered
25 3. Actual Measured Tunnel Lengths based on metres of rock conditions as measured on a daily basis during tunnel boring
26 4. Actual Measured Overall TBM Bored Tunnel Length shall be the as-built length as measured by survey
27 5. All Contract Unit Rates include cavity grouting

EE

108

optimize the water flow and ice-flushing capability of the INCW structure inside the accelerating channel.[21]

6.5.3.2 Scheduling

Mobilization of marine equipment (barges, tugs, cranes, etc.) started in April 2006. In-water blasting for the new intake channel started in May 2006. Replacement of the accelerating wall started in June 2006 along with construction of the cofferdam. Accelerating wall replacement was essentially completed in December 2006. Cofferdam foundation grouting and dewatering were completed in July 2007.

6.5.3.3 Intake DSC Dispute

Prior to TBM in ground (before Sept 1/06)

Starting in May 2006 a series of project change notices were filed by Strabag on behalf of its sub-contractors based on claims of DSC and other changes to the work required at the intake. The intake DSC disputes alleged various differences between the actual conditions experienced during construction of the intake channel, accelerating wall and approach wall and those presented in the GBR. Specific DSC claims included the discovery of a greater amount of overburden on the riverbed, a difference in the riverbed elevation and the presence of boulders within the riverbed.

Disputes also arose with respect to other aspects of the work at the intake site. These included the identification of "fractured rock seams" found in the intake channel, inefficiencies claimed to have resulted from the schedule acceleration requested by Strabag, the re-alignment and lengthening of the new acceleration wall, and obstructions encountered while installing the grout curtain for the cofferdam.

[21] In addition to freezing water from the Niagara River itself, masses of ice can form in Lake Erie and float down the river. This situation may create blockages, ice damage, or reduction of flow into the power plant intakes. Chunks of ice may even enter intake tunnels causing potentially serious damage, unless ice-flushing measures are taken.

OPG and Strabag could not agree on the scope of the changes in work that resulted from these differences or on the appropriate change in contract price to reflect the additional work. OPG requested documentation supporting the claimed amount of about $19.3M in extra costs. After reviewing the documentation, OPG estimated the cost of these changes at roughly $5M and provided a change directive increasing the value of the contract by this amount. Eventually, one of the sub-contractors, McNally Construction, filed a lien and commenced a lien action against OPG and others.

OPG and the parties negotiated a compromise in settlement of all issues, claims and actions relating to the disputes over work at the intake, and any other potential claims related to intake work performed prior to July 25, 2007. Under this settlement, OPG agreed to change the contract price by a total of $7.5M, which represented an additional $2.5M above the $5M contract change already agreed to by OPG. A settlement agreement and a full and final release to this effect were signed on September 20, 2007. A court order was subsequently registered to vacate the lien and the lien action.

6.5.3.4 Intake Completion

Following the removal and disassembly of the TBM and BU from October 2011 to March 2012, work at the intake focused on the completion of the concrete pour for the intake structure. This was achieved by the end of April 2012.

Once the intake structure was completed, the work associated with the installation of the intake gate commenced. The intake gate consists of a sectional steel service gate and guide tower. However, unlike the outlet gate which is a permanent structure, the intake gate sections and guide tower are installed only when the tunnel is to be dewatered, and will be stored at a nearby location when not in use. The intake gate underwent dry fit testing, and installation and removal of the guide tower to ensure it functioned as designed.

While the intake gate was installed and commissioned during much of 2012, tunnel equipment (i.e., invert bridge system) continued to be disassembled and removed using a

Oct 10, 2007

Attachment 6a
Amendment # 3

AMENDMENT AGREEMENT NUMBER 3

This Agreement is made as of October 10, 2007, between

ONTARIO POWER GENERATION INC., a corporation existing under the laws of Ontario ("**OPG**"),

and

STRABAG INC., a corporation existing under the laws of Ontario (the "**Inc.**").

RECITALS

A. OPG and Strabag AG ("AG") entered into a design/build agreement dated as of August 18, 2005, (the "**Original Agreement**");

B. AG has assigned the Original Agreement to Inc., its wholly owned subsidiary;

C. The Original Agreement has been amended by amendment agreements dated as of March 15, 2006, and July 5, 2006; and

D. OPG and Inc. (the "**Parties**") have agreed to amend the Original Agreement to allow for certain changes to the Contract Price and the Work described in the Settlement Agreement dated as of September 20, 2007.

For value received, the Parties agree as follows:

1. Interpretation

Any defined term used in this Amendment Agreement (hereinafter the "**Agreement**") that is not defined in this Agreement has the meaning given to that term in the Original Agreement.

This Agreement supersedes and replaces Project Change Notices 0004, 0005, 0007, 0008, 0009 and 0019 and Project Change Directives 0003, 0005, 0010, 0016 and 0019 in their entirety and is intended to reflect the Contract Price increase and changes to the Work as described in the Settlement Agreement dated September 20, 2007 between the Parties.

2. Change to Appendix 1.1(j) – Contract Price

In the Breakdown of Contract Price table set out in Appendix 1.1(j) of the Original Agreement, Item 1.6 – Accelerating Wall, Intake Channel and Approach Wall is deleted in its entirety and replaced with the following:

1.6 Accelerating Wall, Intake Channel and Approach Wall	$1,007,528.00	$62,362,211.00

In the Breakdown of Contract Price table set out in Appendix 1.1(j) of the Original Agreement, the Item "Total Contract Price" is deleted in its entirety and replaced with the following:

Total Contract Price	$14,609,850.00	$630,135,171.00

3. Change to Appendix 1.1(h) – Concept Drawings

Concept Drawing NAW130-D0E-29230-0015-05, "Diversion Tunnel – General Arrangement," is deleted in its entirety and replaced with Concept Drawing NAW130-D0E-29230-0015-06, "Diversion Tunnel – General Arrangement," as attached to this Agreement.

Concept Drawing NAW130-D0E-29310-0007-05, "Intake Works – Intake Channel and Accelerating Wall Arrangement and Details," is deleted in its entirety and replaced with Concept Drawing NAW130-D0E-29310-0007-09, "Intake Works – Intake Channel and Accelerating Wall Arrangement and Details," as attached to this Agreement.

Concept Drawing NAW130-D0E-29310-0008-03, "Intake Works – Modifications to INCW Control Structure," is deleted in its entirety and replaced with Concept Drawing NAW130-D0E-29310-0008-04, "Intake Works – Modifications to INCW Control Structure," as attached to this Agreement.

Concept Drawing NAW130-D0E-80000-0014-04, "Construction Facilities – Intake Area Plan and Section," is deleted in its entirety and replaced with Concept Drawing NAW130-D0E-80000-0014-09, "Construction Facilities – Intake Area Plan and Section," as attached to this Agreement.

4. Change to Appendix 1.1(sss) – Summary of Work

In Appendix 1.1(sss), Section 1.2.1(x) is deleted in its entirety and replaced with the following:

1.2.1(x) in-river approach channel at the intake area including removal of overburden and glacial till from the riverbed, disposal of such overburden and glacial till, and all other ancillary work required to complete the in-river approach channel

In Appendix 1.1(sss), Section 1.2.1(y) is deleted in its entirety and replaced with the following:

1.2.1(y) in-river approach wall along the river bank at the intake area including removal of overburden and glacial till from the riverbed, disposal of such overburden and glacial till, supply and placement of tremie concrete below precast concrete units and all other ancillary work required to complete the approach wall

In Appendix 1.1(sss), Section 1.2.1(dd) is deleted in its entirety and replaced with the following:

1.2.1(dd) in-river accelerating wall at the intake area, including removal of overburden and glacial till, boulders and other obstructions from the riverbed on the alignment of the accelerating wall, disposal of such overburden and glacial till, boulders and other obstructions, supply and placement of tremie concrete below precast concrete units, and all other ancillary work required to complete the accelerating wall. Also to include installation of navigation strobe light on new wall with associated cabling and connection to power supply at INCW and installation of fall arrest on new wall

5. Original Agreement Remains in Full Force

Except for changes to the Original Agreement set out in this Agreement and any previous Amendment Agreement, the Original Agreement remains in full force, unamended, including the provisions relating to Contract Price and Contract Schedule.

Characteristics was defective. Table 5 below shows the specific issues identified by the DRB and its findings on each issue.

Table 5 - DRB Conclusions and Recommendations

Issue	Finding
Large Block Failures	There is no DSC. The actual conditions were adequately described in the GBR.
St. Davids Gorge	Given the provision of the DBA Section 5.5 (e), the Contractor has no claim for any DSC in this section of the tunnel.
Insufficient Stand-Up Time	There is no DSC based on insufficient stand-up time, as the Contractor's reliance on Rock Mass Rating values stated in the GBR was inappropriate.
Excessive Overbreak	"There is a DSC with respect to the excessive overbreak, provided the defective provisions of the GBR are overlooked, because the GBR contained potentially misleading statements that make the Contractor's position reasonable. Any substantial changes in the designs, means and methods of the support (i.e., Type 4S) were the result of DSCs encountered and not vice versa. Since the development of the GBR was the mutual responsibility of both Parties, we recommend that the Parties negotiate a reasonable resolution based on a fair and equitable sharing of the cost and time impacts resulting from the overbreak conditions that have been encountered and the support measures that have been employed. Both Parties must accept responsibility for some portion of the additional cost, but at the same time the Contractor must have adequate incentives to complete the Work as soon as possible." DRB Report, pages 18-19
Inadequate Table of Rock Conditions and Rock Characteristics	"The Table of Rock Conditions and Rock Characteristics is inadequate to define the subsurface conditions that were encountered. More importantly, the classification of support types based on the "closest match" to rock conditions and rock characteristics given in this Table, together with rock characteristics defined as "all other conditions", renders the concept of DSCs essentially meaningless and the GBR defective. The DRB recommends that the Parties jointly revise the Table of Rock Conditions and Rock Characteristics in such a manner that it describes the rock characteristics to be assumed in terms that are mappable (or otherwise quantifiable) so that it can serve as a clear basis for defining DSCs throughout the remainder of the tunnel excavation. The DRB also recommends that the terms 'closest match' and 'all other conditions' be removed from the GBR." DRB Report, page19

On September 11, 2009, about 100m^3 of Queenston shale and temporary tunnel lining (shotcrete, wire mesh and steel channels) fell from the right side of the tunnel between 3,605 metres and 3,625 metres, about two kilometres behind where the TBM was then located.[30] Work was stopped immediately. There were no injuries and all workers were safely evacuated from the tunnel. The Ministry of Labour ("MOL") subsequently issued a Stop-Work Order stopping all tunnel work beyond 3,500 metres pending an investigation, remedial work and verification of the adequacy of the tunnel crown support.

Photo 11 - Fall of Ground 2009

[30] Measurements used to describe locations in the tunnel represent the distance from the outlet where tunnel boring began. This fall of ground occurred approximately 3.6 kilometres from the outlet. These measurements are often referred to as "chainage" or "station" measurements.

Remedial work involved installing a rock-fill ramp to gain access to the fall area and scaling and installation of new rock support measures in the area of the fall. In accordance with the remediation plan reviewed and accepted by the MOL, Strabag began clean up and repair of the primary rock support and lining on September 20, and continued to install wire mesh, steel channel ribs, rockbolts and shotcrete until October 12. The MOL lifted the Stop-Work Order on October 16 and the contractor proceeded to scale loose shotcrete from the tunnel crown (from 3,700 metres to the TBM) and began applying a precautionary layer of wire mesh to prevent falling shotcrete and enhance worker safety.

A full investigation of the fall of ground was conducted by Strabag and the OR. The investigations concluded that a loosening of the rock support dowels put more pressure on the face plates for the dowels than they could hold, which led to the fall. The investigations also concluded that Boreholes NF-4 and NF-4A contributed to the loosening of the dowels by allowing relatively fresh water to penetrate and degrade the rock surrounding the dowels. These boreholes were drilled in 1984 and 1990, respectively as part of the geotechnical investigation for the NRHD. Owing to the horizontal realignment, the tunnel excavation had intersected with the borehole on February 27, 2009. The boreholes were a source of groundwater inflow before being plugged with grout in March 2009.

The investigation also revealed that Strabag needed to improve monitoring procedures, protocols and frequency, as there were indications that excessive movement was detected on September 10, 2009 at a monitoring point just five metres from where the fall occurred, and that no alert was sounded and no action was taken to check on the stability of the area. Following the fall of ground incident, Strabag reported to the OR that it implemented new monitoring software, installed additional measuring stations and tunnel support enhancements, established tighter trigger levels and adopted more rigorous procedures to monitor and respond to ground movements. Strabag also noted that some of these measures either had been planned or were initiated prior to the fall of ground incident.

lane has been provided at the south end of Portage Road for the new intake access road. A new turning lane has also been constructed at the intersection of Stanley Avenue and Thorold Stone Road to minimize the impact of construction traffic traveling south from the outlet construction area. The roadworks were carried out by the RMON on behalf of OPG under the terms of the CIA.

3.3.8 Pre-Construction Condition Surveys

A number of structures in the vicinity of the site were surveyed before commencement of construction to establish a basis for determining if the structure has been affected by the construction activities and to determine responsibility for repair, if necessary. Pre-construction survey work was undertaken by OPG/OR and involved a third party consultant. OPG/NPG and the Contractor were required to endorse the pre-construction survey before commencement of construction activities that could result in damages to the existing SAB PGS or INCW facilities. RMON conducted a pre-construction pavement survey of all roads to be used by heavy construction traffic and will undertake a post-construction survey to determine Project impacts.

Added

3.3.9 Other Contracts

In addition to the main contract for the design and construction of the diversion tunnel, a number of smaller contracts are necessary for the implementation of the Project. These include contracts for

- examination of flow conditions in the Welland River
- installation of survey control monuments and third-party audits of the tunnel survey work carried out by the tunnel Contractor
- compilation of video footage of the Project progress
- a third-party consultant to assist the MOE with Project related issues
- a third-party facilitator to support the Project team building and alignment activities
- implementation of fish habitat compensation scheme at Drapers Creek
- decommissioning of wells and boreholes that were installed prior to Phase 1 and could interfere with tunnel construction
- relocation of fibre optic cable at the Intake area which would have been affected by the construction work
- installation and monitoring of groundwater control boreholes (monitoring transferred to Contractor during the construction period)
- pre- and post-blasting inspections of existing structures
- a third party accountant to assist with accounting system setup and review of Contractor expenditures/billings/reconciliations
- technical experts if/when required
- safety incident investigation services.

3.4 Project Management

Management of the Project is a combined responsibility of OPG and the OR as defined later in the PEP. These two parties work together as a team to enable the successful completion of the Project. Management activities are assigned to one of the parties as the primary responsible party. The other party may provide specific support or may be consulted on certain activities as indicated in Table 9.1 and detailed in Appendix E. In either case, all parties will be informed of

Due to the fall of ground and associated remedial work, tunnel boring was suspended for a total of 46 days, from September 11 to October 26, 2009. Once the remedial work was completed, Strabag undertook a planned TBM maintenance shutdown, primarily to overhaul the cutterhead, which lasted until December 8, 2009.

While the TBM was stopped due to the fall of ground, remedial work and planned maintenance shutdown, work continued on other aspects of the tunnel. This work included lining and profile restoration in the area before 3,500 metres, construction at the intake and outlet, equipment modifications, and work on the conveyor and dust enclosure.

Ultimately the fall of ground in 2009 only set back the schedule for overall NTP completion by approximately 17 days because the parties agreed under Appendix 5.3C of the ADBA that a one day delay to TBM mining translated into 0.375 days delay to the critical path.

At the time of this event, a decision was made to forego a claim under the Builder's All Risk ("BAR") insurance because Strabag's estimate to execute the remedial work was comparable to the $2M insurance deductible. Strabag's subsequent request for a Target Cost increase of $4.5M could not be substantiated by the OR records that valued the actual costs for the remedial work at $2.1M. Based on the decision to forego a BAR insurance claim, OPG offered, and Strabag accepted, a Target Cost increase by $2M. Altogether, the final impact of the 2009 fall of ground was an increase to the target schedule by 17 days and an increase to the Target Cost by $2M.

In the first part of 2010, tunnelling progress improved, but the advance rate remained below the target established in the ADBA. Strabag took measures to remove loose shotcrete and install protective wire mesh. Overbreak amounts varied, but were generally less than what had been experienced while tunnelling in the Queenston shale.

By spring 2010, the TBM was making good progress and the gap between targeted and actual performance began to significantly decrease. Progress improved further in the

In May 2012, the OR submitted a summary of the costs associated with the fall of ground work to the adjuster. The costs totalled approximately $17.6M, and included work done outside of the MOL mandated area, where reinforcement of the rock support was considered necessary to ensure the safety of the workers and equipment before entering and repairing the MOL mandated and fall of ground areas.

OPG received a letter from the insurance adjuster on August 13, 2012, which noted that, on the basis that the fall of ground itself did not exceed 100 metres, there is a $10M limit to the loss at hand. The adjuster's evaluation report attached to the letter found that substantiated costs based on the documentation received by the OR were only about $7.5M. In June 2013, after several information exchanges with the adjusters, the OR submitted a final revised cost summary, which reduced the claim amount to approximately $12.1M. Regarding the $10M limit, the OR pointed out that although the fall of ground may have been less than 100 metres, the area of damage associated with this loss significantly exceeded 100 metres. Ultimately, however, the insurers rejected this position, invoked the $10M limit and are expected to pay this amount by October 2013. This amount is relatively close (within $400k) to the amount by which the Target Cost in the ADBA was increased due to the July 2, 2011 fall of ground.

6.5.5.6 Swelling at Low Point

In the fall of 2009, it was noted that water from construction activities and surface water from the outlet portal was migrating under the invert concrete at the low point in the tunnel. The ingress of water had caused the invert liner to float, and created a concern for the potential swelling of the rock, a phenomenon that occurs when rocks of the Queenston formation come into contact with fresh water. A Notice of Defective Project and a Disallowed Cost Notice[33] were consequently issued to Strabag in November 2009 by OPG. As a temporary measure, Strabag installed sumps at the low point to remove the water.

[33] Under s. 1.1(O)(1)(ii) of the ADBA, any cost arising from or incurred as a result of repair or remediation of the Work to be carried out prior to Substantial Completion and due to the previous or ongoing presence of fresh water outside the impermeable membrane liner in any part of the tunnel contained in the Queenston, is a Disallowed Cost, and is not payable by OPG.

on December 7, 2012 to facilitate the removal of the grouting and arch concrete carriers in the tunnel. Pre-stress grouting was completed on February 4, 2013, almost two months ahead of the ADBA target schedule.

6.5.5.5 July 2011 Fall of Ground 6+033 - 6+080

On July 2, 2011, a portion of the tunnel roof partially collapsed between 6,033 metres and 6,080 metres, resulting in about 1,200 m^3 of fallen rock and initial lining and rock support materials. No one was injured. The tunnel was initially shutdown from 5,933 metres to 6,130 metres to prevent access to the area. Strabag's consulting engineer and the MOL inspected the site along with the OR and Strabag staff. Following the MOL inspection, a Stop-Work Order was issued for the area between 5,983 metres and 6,130 metres, pending Strabag's submission of its engineering assessment and plans for safe remediation of the area. The Stop-Work Order for this area of the tunnel was in effect from July 5 to September 27, 2011.

The upper limit of the failure occurred in the Grimsby formation between 6,050 metres and 6,060 meters to a depth of approximately seven metres above the tunnel crown. Most of the failure was within a thinning wedge of the Power Glen shale/sandstone layer, which is comparatively stiffer than the overlying Grimsby shale rock mass and the underlying Power Glen shale. Horizontal stresses concentrate in this formation because the surrounding rock does not have the stiffness to withstand such stresses. Strabag's consulting engineer cited the overload of the initial support systems caused by these rock conditions as the primary cause of this fall of ground.

118

Photo 17 - Fall of Ground 2011

During the original excavation of the area in March 2010, stress-induced deformation occurred in the form of a small notch at about the 11:30 position. Rock support installed at the time consisted of the following elements:

- 4 metre friction anchors;
- steel channels in crown ("C-channels");
- welded wire mesh;
- shotcrete, with a "slot" left in the shotcrete arch to allow deformation to occur without causing spalling, as had been a problem in other areas of the tunnel; and
- additional 4 metre field bolts.

3D monitoring arrays were also installed through this portion of the tunnel in March 2010. In association with these arrays, the following three threshold "trigger" levels were established to assess the stability of the excavation:

- at the "design" level, deformations were within the expected level and no action was required.
- at the "review" level, Strabag was to evaluate the specific situation and assess if any further action was required.
- at the "action" level, the stability of the tunnel excavation was jeopardized and immediate action was required to install additional support.

In November 2010, analysis of the survey monitoring data indicated that deformations in the fall of ground area were at the "review" trigger level. As a result, Strabag reviewed the situation and installed additional Swellex anchor bolts and mesh as a remedial measure. In December 2010, Strabag's routine inspection revealed that there was more convergence in the tunnel roof and monitoring data indicated accelerating movement. In addition, shotcrete cracking was observed on the crown. As a result of this deformation, additional review and geotechnical assessment of the rock reinforcement requirements was undertaken. Following this review, Strabag developed a supplemental construction drawing for the installation of additional support between 5,690 and 5,710 metres and between 6,000 and 6,160 metres. The drawing indicated that six metre long grouted "hollow bar dowels" on a two metre staggered pattern with an additional 130mm shotcrete layer and wire mesh were to be installed.

Areas approaching the "action" trigger level and areas showing acceleration were given priority for the installation of additional support. Before the fall of ground occurred, the additional support shown in the supplemental drawings was installed between 5,690 metres and 5,710 metres. By January 2011, monitoring data revealed movement between 6,000 metres and 6,160 metres, the area where the fall ultimately occurred, had decreased. This data was interpreted as indicating stabilization. Consequently, Strabag determined that, unless new movement occurred, installation of additional support in this area was not immediately required. The additional support work was scheduled for a planned shutdown starting on July 4, 2011.

Monitoring frequency for this area changed according to the rate of deformations recorded. Before the fall occurred, monitoring frequency had increased to twice a week and the area was kept under frequent visual observation. The last few readings at some arrays did indicate some acceleration of movement, but the established "action" trigger level was never reached before the fall occurred.

Bolts removed from the fall of ground area were tested in December 2011, and results indicated that the breakage was not an installation or manufacturing issue. Based on the information available, Strabag concluded that the most probable cause of the July 2, 2011 fall was the unique geological conditions at the local boundary between the Grimsby and Power Glen formations, in particular, the thickness, relative stiffness and redistribution of high horizontal stresses in the rock immediately above the tunnel excavation. This conclusion is supported by the fact that the bolts broke close to the Grimsby shale and Power Glen shale/sandstone interface. However, inadequate rock support measures and response to visual and survey monitoring signs of instability may have also contributed to the incident.

Strabag divided the required remediation into phases. Phase 1 involved stabilization of the tunnel on both sides of the fall between 5,900 metres and 6,170 metres. Phase 2 was rehabilitation and replacement of the tunnel rock support where it was damaged by the fall. Work on the two phases overlapped with the remediation being completed at the end of December 2011.

An insurance claim was submitted under the Builder's All Risk policy to recover the cost of remedial work associated with the July 2011 fall of ground. The claim was subject to a $2M deductible.

In May 2012, the OR submitted a summary of the costs associated with the fall of ground work to the adjuster. The costs totalled approximately \$17.6M, and included work done outside of the MOL mandated area, where reinforcement of the rock support was considered necessary to ensure the safety of the workers and equipment before entering and repairing the MOL mandated and fall of ground areas.

OPG received a letter from the insurance adjuster on August 13, 2012, which noted that, on the basis that the fall of ground itself did not exceed 100 metres, there is a \$10M limit to the loss at hand. The adjuster's evaluation report attached to the letter found that substantiated costs based on the documentation received by the OR were only about \$7.5M. In June 2013, after several information exchanges with the adjusters, the OR submitted a final revised cost summary, which reduced the claim amount to approximately \$12.1M. Regarding the \$10M limit, the OR pointed out that although the fall of ground may have been less than 100 metres, the area of damage associated with this loss significantly exceeded 100 metres. Ultimately, however, the insurers rejected this position, invoked the \$10M limit and are expected to pay this amount by October 2013. This amount is relatively close (within \$400k) to the amount by which the Target Cost in the ADBA was increased due to the July 2, 2011 fall of ground.

6.5.5.6 Swelling at Low Point

In the fall of 2009, it was noted that water from construction activities and surface water from the outlet portal was migrating under the invert concrete at the low point in the tunnel. The ingress of water had caused the invert liner to float, and created a concern for the potential swelling of the rock, a phenomenon that occurs when rocks of the Queenston formation come into contact with fresh water. A Notice of Defective Project and a Disallowed Cost Notice[33] were consequently issued to Strabag in November 2009 by OPG. As a temporary measure, Strabag installed sumps at the low point to remove the water.

[33] Under s. 1.1(O)(1)(ii) of the ADBA, any cost arising from or incurred as a result of repair or remediation of the Work to be carried out prior to Substantial Completion and due to the previous or ongoing presence of fresh water outside the impermeable membrane liner in any part of the tunnel contained in the Queenston, is a Disallowed Cost, and is not payable by OPG.

6. Other Agreed Changes to Target Cost

6.1 PCD048, Crown Overbreak Adjustment

Adjustment to the Target Cost in accordance with Sections 5.3C and 5.3D of the ADBA due to crown overbreak as calculated in accordance with Table 1 of Appendix 5.3C and Table 4 of Appendix 1.1 (UUU).

6.2 PCD055, Dufferin Construction Company Global Claim Settlement

Adjustment to the Target Cost in accordance with Section 5.3D of the ADBA due to claims agreed between Strabag and Dufferin Construction Company relating to Intake Channel Walls and Outlet Structure as baselined in Appendix 1.1 (UUU) of the ADBA.

6.3 PCD056, DCC Stand-by and Double Shift Work Claim at PGS Dewatering Structure Removal

Adjustment to the Target Cost in accordance with Sections 5.1(a) and 2.14(c) of the ADBA due to claims agreed between Strabag and Dufferin Construction Company relating to stand-by and double shift work carried out as part of the PGS Dewatering Structure removal.

7. Change to Target Cost

Appendix 1.1(TTT) – Target Cost of the ADBA is deleted in its entirety and replaced with the Attachment A attached to this Agreement.

8. Summary of Target Cost Adjustments

8.1 The Parties acknowledge that the breakdown of change to the Target Cost resulting from the changes described in Sections 2 through 6 above are as follows:

PCD035	Item 10	Intake Gates	+$23,392.00
	Item 12	Outlet Gate and Hoist	+$110,594.00
PCD037 Rev1	Item 13	Diversion Tunnel	+$1,736,952.00
PCD042	Item 19	Scope Changes	+$185,000.00
PCD045	Item 19	Scope Changes	+$185,000.00
PCD046	Item 13	Diversion Tunnel	-$2,485,671.79 (for 2010 & 2011)
PCD047	Item 13	Diversion Tunnel	$0.00
PCD048	Item 13	Diversion Tunnel	+$10,454,848.58
PCD049 Rev1	Item 13	Diversion Tunnel	-$1,527,120.00
PCD051	Item 12	Outlet Gate and Hoist	+$19,115.00
PCD055	Item 06	Intake Channel & Walls	-$10,000.00
	Item 11	Outlet Structure	+$86,000.00
PCD056	Item 16	Dewatering Structure	+$167,657.12.

123

Attachment A

Appendix 1.1(TTT) – Target Cost

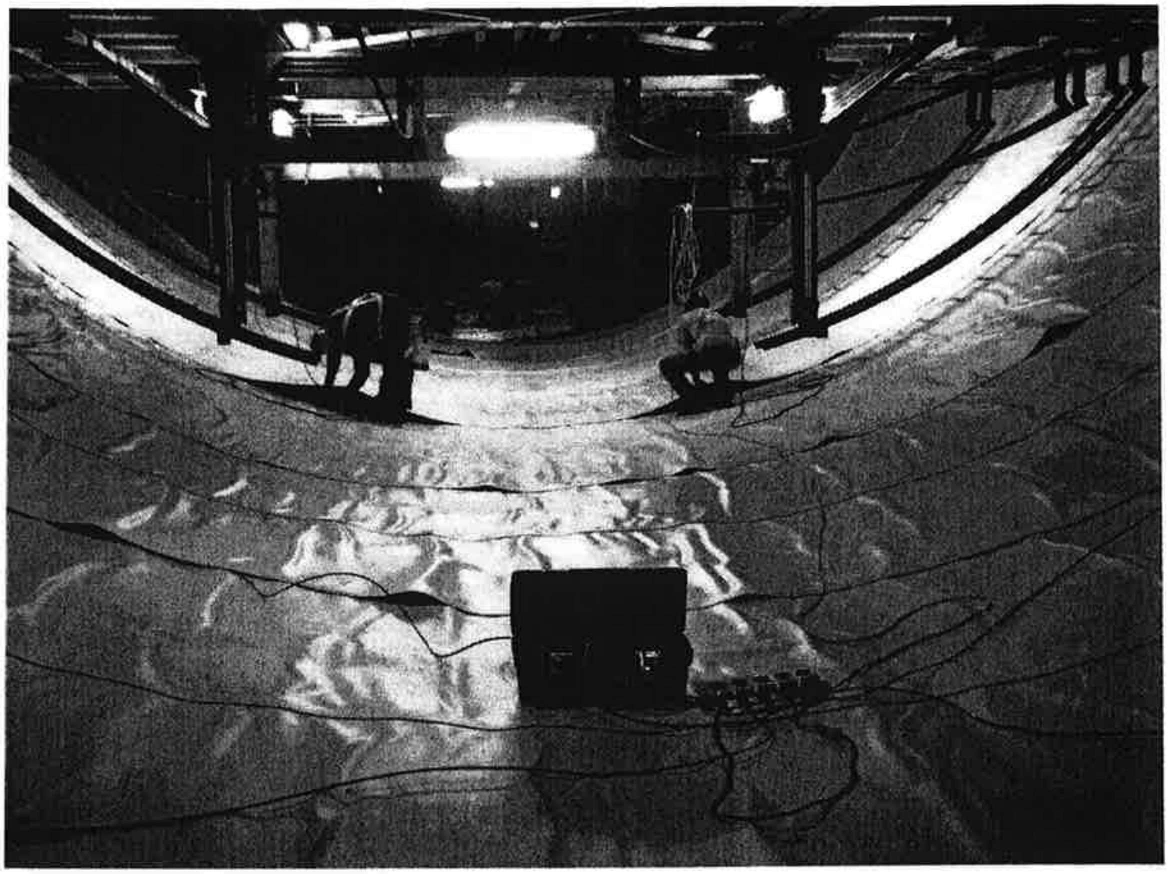
The Target Cost is $994,003,566.91.

For the purposes of cost control, cost projection and cost performance indices only, the Target Cost will be allocated in the following Manner:

		Revised Target Cost Data		
Item	Description	Pre Effective Date	Post Effective Date	Total
1	Mobilize/Demobilize	$25,037,603.51	$5,940,000.00	$30,977,603.51
2	Maintenance Bond	$0.00	$700,000.00	$700,000.00
3	Performance LC	$2,135,833.33	$4,341,458.00	$6,477,291.33
4	Insurance Premium	$2,293,333.33	$550,047.00	$2,843,380.33
5	Design	$5,425,340.78	$5,277,000.00	$10,702,340.78
6	Intake Channel and Walls	$58,386,649.32	$4,612,932.00	$62,999,581.32
7	Diversion Outlet Canal	$11,395,047.88	$1,621,734.00	$13,016,781.88
8	Dewatering Shafts	$3,159,097.60	$630,728.10	$3,789,825.70
9	Intake Structure	$304,440.00	$5,631,354.00	$5,935,794.00
10	Intake Gates	$0.00	$3,001,530.00	$3,001,530.00
11	Outlet Structure	$2,292,196.28	$9,613,698.00	$11,905,894.28
12	Outlet Gate and Hoist	$0.00	$3,782,821.00	$3,782,821.00
13	Diversion Tunnel	$112,171,914.06	$563,430,137.24	$675,602,051.30
14	Tunnel Boring Machine	$78,242,470.00	$0.00	$78,242,470.00
15	Flow Verification Test	$0.00	$319,097.00	$319,097.00
16	Dewatering Structure	$0.00	$1,677,491.12	$1,677,491.12
17	DRB Estimated Cost	$291,671.11	$75,000.00	$366,671.11
18	Item not used	$0.00	$0.00	$0.00
19	Scope Changes	$739,235.99	$12,843.38	$752,079.37
20	Provisional Sum	$206,152.03	$0.00	$206,152.03
21	Changes in Applicable Law	$117,500.00	$17,500.00	$135,000.00
22	Warranty Administration Fee	$0.00	$100,000.00	$100,000.00
23	Office and General Cost	$0.00	$80,469,710.85	$80,469,710.85
	Target Price	**$302,198,485.22**	**$691,805,081.69**	**$994,003,566.91**

[illegible] target cost

Photo 12 - Testing of the Invert Lining



6.5.5.2 Profile Restoration

Profile restoration is the process of recreating the tunnel's circular shape. In some parts of the tunnel, considerable overbreak in the arch along the tunnel's top significantly altered the circular shape produced by the TBM. Profile restoration on the scale required for the NTP is not typical in tunnel construction, but was required because of the amount of overbreak experienced. As neither party anticipated this scale of restoration work, it was not included in the DBA. The amount of restoration work required the development of specialized equipment during the execution of the project.

the cofferdam will be removed. The works yard/laydown site will be restored to meet Niagara Parks Commission (NPC) requirements.

Extensive grouting of the upper rock formations at the intake end of the tunnel has been undertaken to minimize water inflows into the tunnel during the TBM drive through these formations. In addition, underwater excavation of an intake channel has been carried out upstream from the intake structure and beyond the confines of the cofferdam.

3.2.5 Diversion Tunnel

The tunnel is being excavated from the downstream end through limestones, sandstones and shales using a 14.44 m excavated diameter TBM supplied by the Contractor. The tunnel was initially constructed using an initial support of full-ring steel ribs, mesh, rock bolts and shotcrete. Subsequently, the Contractor changed its means and methods so that as the tunnel is being bored, workers behind the TBM cutterhead install various combinations of steel ribs, wire mesh, and rock bolts to reinforce the rock in the upper (approximately) third of the tunnel only. The surrounding surface is then sprayed with a layer of shotcrete to cover the exposed rock and form a protective shell. The tunnel is subsequently lined with a polyolefin waterproofing membrane in order to prevent fresh water in the tunnel from entering the host rock thus eliminating the well documented swelling potential in these formations. The final lining consists of cast-in-place unreinforced concrete at least 600 mm thick. The final lining is prestressed using high pressure grout injected between the impermeable membrane and the initial lining.

Excessive overbreak in the tunnel crown, particularly in the Queenston shale formation, has necessitated the addition of an infill operation to restore the tunnel profile to a circular shape prior to installing the membrane and arch concrete. Elevated mobile, structural steel work platforms accommodated drill jumbos, grouting facilities, shotcrete robots and material handling equipment needed for the arch profile restoration.

The tunnel crosses various geological formations. Tunnel lining design must address time dependent deformation characteristics of the host strata. The swelling component of the time dependent deformations have been eliminated by providing a watertight membrane as discussed above that will prevent contact of fresh water with the swelling shales and prevent diffusion of chloride ions out of the pore water of the shales. This will eliminate the advection and diffusion process necessary to mobilize swelling.

On completion of the tunnel and following tunnel water-up, a flow test will be performed to establish whether the tunnel meets the GFA. The testing will be done by a tester jointly agreed by OPG and the Contractor. The results of the tests will be used to determine the final GFA on which disincentives or incentives will be based.

3.2.6 Outlet Structure and Channel

The outlet structure is a reinforced concrete structure, housing the closure gate and provisions for sectional service gates for closure of the diversion tunnel.

Water from the diversion tunnel will be discharged into the existing canal system feeding the forebays of the SAB generating stations.

7.4 **PCD-027 Rev. 1 – Grout Investigative Boreholes NF-39 and SD-8**

The following Section is added as a new Section 1.2(1) (sss) in Appendix 1.1(sss) of the Original Agreement:

"(sss) grout investigative boreholes NF-39 and SD-8."

7.5 **PCD-028 – Clean-up of Dust at Butterfly Conservatory**

The following Section is added as a new Section 1.2(1) (ttt) in Appendix 1.1(sss) of the Original Agreement:

"(ttt) perform remedial and maintenance work for the cleaning of the Butterfly Conservatory after the 2007 dust events."

7.6 **PCD-029 – Provide Radio Communication System for Owner's Representative**

The following Section is added as a new Section 1.2(1) (uuu) in Appendix 1.1(sss) of the Original Agreement:

"(uuu) supply, install, and maintain a tunnel radio communications system for use by the Owner's Representative while travelling and working within the tunnel."

7.7 **PCD-030 – Remove Concrete Structure and Contaminated Soil Interfering with Installation of the Dewatering System Outfall Pipe**

The following Section is added as a new Section 1.2(1) (vvv) in Appendix 1.1(sss) of the Original Agreement:

"(vvv) demolish a portion of an existing buried concrete structure that conflicted with the dewatering system pipeline alignment, and excavate, contain stockpiling and dispose off- site hydrocarbon contaminated soil from beneath the concrete structure."

8. Change to Appendix 1.1 (j) – Breakdown of Contract Price

8.1 The following Measurement Payment Item is added as new Measurement Payment Item 1.21 in the Breakdown of Contract Price of Appendix 1.1(j) of the Original Agreement:

"1.21	***Changes in Applicable Laws***	*$0*	*$235,000.00"*

8.2 Measurement Payment Item 1.19 in the Breakdown of Contract Price is deleted in its entirety and replaced with the following:

"1.19	*Scope Changes*	*$26,763*	*$678,116.88"*

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8.3 **PCD032, Increase to OPG's 50% Portion of DRB Estimated Costs**

In the Breakdown of Contract Price table set out in Appendix 1.1(j), the item 1.17 "OPG's 50% Portion of DRB Estimated Cost" is deleted in its entirety and replaced with the following, to reflect an increase in the price of $228,443:

"1.17 OPG's 50% Portion of DRB Estimated Cost $0 $450,000.00"

8.4 The Breakdown of Contract Price table in Appendix 1.1(j) is deleted in its entirety and replaced with the table shown below.

BREAKDOWN OF CONTRACT PRICE

MEASUREMENT PAYMENT ITEM	DESCRIPTION OF WORK	ORST INCLUDED	TOTAL
	Insurance Premium	170,000	2,724,181
1.1	Mobilization/Demobilization	871,824	31,693,169
1.2	Maintenance Bond in the form of Appendix 4.1(f)	0	610,749
1.3	Performance LC	0	2,544,789
1.5	Design	0	5,870,313
1.6	Accelerating Wall, Intake Channel and Approach Wall	1,007,528	62,362,211
1.7	Diversion Outlet Canal	33,520	12,730,052
1.8	Dewatering System Shafts	145,367	3,787,251
1.9	Intake Structure	56,789	5,334,935
1.10	Intake Gates	8,389	2,325,461
1.11	Outlet Structure	60,149	7,222,558
1.12	Outlet Structure Gate and Hoist	16,729	5,957,260
1.13	Diversion Tunnel	7,489,430	406,881,138
1.14	Tunnel Boring Machine	4,738,617	78,242,470
1.15	Flow Verification Test	0	126,948
1.16	Demolition and Disposal of Dewatering Structure (optional)	11,490	1,495,595
1.17	OPG's 50% portion of DRB Estimated Cost	**0**	**450,000**
1.18	Item not used	0	0
1.19	Scope Changes	**26,763**	**678,117**
1.20	Provisional Sum	23,000	400,000
1.21	Changes in Applicable Laws	**0**	**235,000**
	Total Contract Price	**14,659,595**	**631,672,197**

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128

Project Management

- A strong team remains in place for management and execution of the Niagara Tunnel Project and includes:
    - The OPG Project Director empowered to ensure effective integration of internal and external resources and timely communications between the project team and other stakeholders
    - Other OPG personnel representing Niagara Plant Group, Water Resources, Law Division, Supply Chain, Finance, Real Estate, Health & Safety and Risk Services
    - Hatch Mott MacDonald (HMM), an Ontario-based consultant with considerable experience in tunnel design and construction, has been engaged as Owner's Representative and holds primary responsibility for project management, design review and construction oversight with Hatch Energy providing assistance in the areas of geotechnical and hydraulic engineering, environmental agency liaison and third party liaison
    - Torys has been engaged as external legal counsel and has been part of the core project team providing advice on contractual, procedural fairness, environmental, real estate and regulatory matters
    - Strabag (a large Austrian construction group, supported by ILF Beratende Ingenieure of Austria, Morrison Hershfield of Toronto, Dufferin Construction of Oakville, and other speciality subcontractors), the engaged Design / Build Contractor, has extensive international experience in tunnelling and heavy civil underground works.
    - Expert consultants and contractors are engaged, as required, to provide support in areas such as project risk assessment, financial modeling, teambuilding, field investigations, surveying, geotechnical engineering, TBM tunnel construction, construction litigation, ICC arbitration, etc.

- Decision authority for this Project remains with OPG and delegation will be in accordance with OPG's Organization Authority Register (OAR).

- A Project Execution Plan has been developed and issued to provide the framework for management of the Niagara Tunnel Project, and it will be reviewed and revised as necessary during project execution.

4. ALTERNATIVES AND ECONOMIC ANALYSIS

Key Project and Financial Assumptions:

- The Project is estimated to cost $1,600 M, including the previously released funding.
- The sunk cost on the Project to date (to the end of April 2009) is $463 M.
- The Project will receive a 10-year "holiday" for Gross Revenue Charge (GRC) payments.
- The Project will be funded through financing arranged with the OEFC.
- Other Assumptions are listed in Appendix B.

Status Quo – Proceed Under the Existing DBA (Not Recommended)

- Considering the significant schedule delay, contractor claims regarding differing subsurface conditions (primarily in the Queenston shale formation), recommendations of the Dispute Review Board in August 2008 that OPG and Strabag should equitably share the cost and schedule impacts, difficulties experienced in excavating and supporting the Queenston shale, and significant liquidated damages included in the existing DBA, there is a high risk that the contractor would abandon the project, requiring completion of the tunnel by another contractor with higher costs and a significant delay (see Alternative 2), and causing OPG to expend considerable resources on legal proceedings. This alternative is not recommended.

on the cutterhead that necessitated reducing the penetration rate to less than 1.5 m/hr in order to avoid damaging the TBM main bearing.

- Restoring the tunnel to a circular profile ("profile restoration") is an additional task that was not included in the original schedule. Profile restoration must be completed prior to installing the arch membrane and concrete lining. Undertaking this operation concurrent with the mining, invert lining and arch lining operations added significant complication and risk to project logistics.
- Additional time to allow for removal of tunneling equipment before removal of the cofferdam at the intake structure.

The forecast cost changes between the DBA and the ADBA are shown in Table 6 below. The bulk of the increase is attributable to the tunnel contract (including contingency), but the longer schedule also increases the cost of maintaining the OR on site and interest cost.

Table 6 - Cost Changes between the DBA and the ADBA

Project Cost Flow Estimate ($M) (including Contingency)	Original Approval (DBA)	Revised Estimate (ADBA)	Variance	Variance (%)
OPG Project Management	4.4	6.0	1.6	36
Owner's Representative	25.4	40.4	15.0	59
Other Consultants	4.0	5.9	1.9	48
Environmental / Compensation	12.0	9.6	(2.4)	-20
Tunnel Contract (including Incentives)	723.6	1,181.7	458.1	63
Other Contracts / Costs	78.9	69.8	(9.1)	-11
Interest	136.8	286.6	149.8	110
Total Project Capital	985.2	1,600.0	614.8	62

There were four alternatives presented in the Superseding BCS. In addition to the recommended alternative of proceeding under the target cost and schedule approach negotiated in the ADBA, the following three alternatives were considered and rejected:

- Continue under the DBA – This alternative was rejected because OPG concluded that it would lead to Strabag abandoning the project based on projected costs of over $300M more than the contract price under the DBA. Under this approach, Strabag would have been expected to continue tunneling under difficult conditions and to experience an ongoing revenue loss in the hope of receiving some unspecified additional compensation

Alternative 1 – Proceed Under a Target Cost Amended DBA (Preferred Alternative)

- Complete design, construction and commissioning of the Niagara Tunnel under an amended DBA that features a target cost / target schedule with cost and schedule incentives and disincentives and incorporates changes in the tunnel alignment to minimize further excavation with the tunnel crown in the Queenston shale formation. This approach settles all of Strabag's outstanding claims to November 30, 2008, establishes a sharing of incremental costs and provides incentives for Strabag to complete the tunnel in a timely manner. The remaining cost for this alternative is $1,137 M and the total cost is $1,600 M. This is considered to be the least cost alternative for completion of the Project and is the recommended alternative. Appendix A provides a more detailed breakdown of the Project costs.

Alternative 2 – Engage another Contractor to Complete the Project (Not Recommended)

- Complete design, construction and commissioning of the Niagara Tunnel by terminating the existing DBA with Strabag and engaging another contractor. This approach would result in a further delay of 18 to 24 months to engage another contractor, unknown higher costs (actual plus mark-up), loss of experience gained to date and key personnel (contractor, designers and subcontractors) and require OPG to expend considerable resources on legal proceedings to recover damages from Strabag. This alternative is not recommended.

Alternative 3 – Cancel the Project (Not Recommended)

- Abandon design, construction and commissioning of the Niagara Tunnel, incurring additional costs in the order of $100 M to secure the site in a safe and environmentally acceptable state, and forego the opportunity to generate additional clean, renewable hydroelectric energy averaging 1.6 TWh per year for at least 90 years at the Sir Adam Beck generating stations. With this alternative, there is a low likelihood of recovering any of the $563 M incurred costs through the regulated rates. This alternative is not recommended.

Financial Analysis

- While the Niagara Tunnel is expected to be part of OPG's regulated hydroelectric assets and receive a regulated rate reflecting cost recovery and a return on capital, it is appropriate to consider several financial metrics, as follows, to ensure that this is an economic investment relative to other generation options:

    - Levelized Unit Energy Cost (LUEC) represents the price required to cover all forecast costs, including a return on capital over the service life, escalates over time at the rate of inflation, and it permits a consistent cost comparison between generation options with different service lives and cost flow characteristics.
    - Equivalent Power Purchase Agreement (PPA) represents the price required if one were to bid the project into the renewable RFP. It is similar to LUEC except only 20% of the PPA escalates at the Consumer Price Index.
    - Revenue Requirement is a measure that represents the annual accounting cost of this project including an allowed return on capital employed. Revenue Requirement generally declines over time as the rate base is depreciated.
    - These metrics are equivalent in present value terms over the life of the asset and reflect full recovery of costs including a return on the investment.

agreement.[35] Both parties agreed that their joint focus over the next few months would be on negotiating a mutually satisfactory resolution of their disagreements and a path forward to project completion. To this end, Strabag agreed to bring forward two proposals to resolve existing disputes and move the project forward.

In early October 2008, Strabag submitted two options to OPG for resolving the current dispute and moving forward. Option A involved continuing the fixed priced approach in the DBA with additional cost included to reflect the rock conditions encountered and anticipated going forward. The bulk of the cost increase came from the addition of two new rock support types (4R and 4S) to reflect areas of substantial overbreak. Option A included per metre costs and estimated quantities (in metres) for each of these new rock support types. In addition, Strabag included its claimed cost for modifications to the TBM and a contingency amount for future TBM risks. Finally, this option included compensation for the extension of the project schedule. Taken together these costs were estimated at approximately $190M.

Strabag also estimated that the cost of pending claims, profile restoration and other future modifications would total an additional $90M, but indicated that this figure was only a preliminary estimate. Strabag proposed the elimination or renegotiation of the liquidated damage and early completion bonus provisions. All told, Strabag estimated the revised fixed price of the tunnel at approximately $910M under Option A.

In Option B, Strabag proposed converting the contract to a target price and reducing the overhead fee from 19 per cent to 12 per cent. OPG and Strabag would agree on a target price and schedule under this approach with the benefits of any cost savings and early completion to be shared equally between Strabag and OPG. This option also included two disincentives: the overhead fee would decrease as contract cost increased reaching zero per

[35] The DBA provided that a party who was dissatisfied with one or more DRB recommendations had 30 days to notify the other party in writing of its intent to commence arbitration (DBA section 11.1 (f) as amended). In order to preserve its right to seek arbitration if necessary, OPG provided the required notice of intent to commence arbitration because it disagreed with the DRB recommendations concerning excessive overbreak and the need to revise the Table of Rock Conditions and Rock Characteristics. Strabag similarly notified OPG in writing that it rejected all 5 DRB recommendations and intended to pursue arbitration.

cent at $1B; and the overhead fee would also be reduced for late completion reaching zero per cent if the project was six months late. The target price under this option would be $856M, a figure derived by reducing the price estimated for Option A to account for the reduction in overhead fee from 19 per cent to 12 per cent.

Strabag saw the following benefits from adopting Option B:

- It eliminates ongoing concerns about deficiencies in the GBR.
- It includes sufficient incentives to encourage the contractor to complete the project as quickly and cost effectively as possible.
- It allows all available resources, including the expertise of the OR, to be fully dedicated to optimizing project execution and developing innovative solutions to emerging issues.

Strabag's proposals were thoroughly considered by OPG, the OPG Board and the CLOC. OPG, in consultation with the OR, noted that neither of Strabag's proposals adequately captured the notion of a "fair and equitable sharing of the cost and time impacts" as recommended by the DRB. However, OPG also noted that as Strabag continued to do a good job and work safely on the project despite the difficult rock conditions in the tunnel, it was in OPG's interest to attempt to settle with Strabag. To that end, OPG's management recommended adopting a three-part negotiation strategy and counter-proposal:

- a lump sum payment to be made by OPG to settle Strabag's costs and claims to November 30, 2008;
- a revised contract effective from December 1, 2008 forward with a negotiated target price and schedule (similar to Strabag's proposal B); and
- incentives and disincentives to ensure completion of work.

Strabag and OPG had a number of meetings throughout October and early November of 2008. At these meetings the various options tabled by Strabag and OPG were discussed. Ultimately, the parties agreed to the approach reflected in the Principals of Agreement that captured both the advantages of Strabag's proposal B as well as OPG's attempt to

Strabag Contract Renegotiation Budgets ($ M): Options A & B vs ADBA
Ref: D1-2-1-104 & 105

Strabag Option A	Cost	Strabag Option B	Cost
Continue Fixed Price contract (DBA)	$630	**Convert to Target Price contract**	
+ Plus Additional cost to reflect rock conditions encountered and anticipated going forward Bulk of cost addition 2 new rock support types (4R & 4S) for areas of substantial overbreak ($/m for each rock support type& estimated quantities in metres for each new rock support type) Claimed costs for TBM modifications Contingency for future TBM risks Compensation for extension to project schedule	? lump sum ? ? ? ?	**Reduce O/H fee from 19% to 12%** OPG & Strabag to agree on target price & schedule Benefits & cost savings & early completion shared equally between Strabag & OPG Eliminates concerns re: deficiencies GBR	?
Sub-total	**$190**		
Cost Pending claims, profile restoration & other future modifications	**$90**		
Elimination or renegotiation of liquidated damage & early completion bonus provisions ($125 M)	$0 ?	• Two disincentives: O/H fee decreases as contract cost increased reaching 0% at $1B • O/H fee reduced for late completion to 0% if project 6 mos late	?
Revised tunnel costs	**$910 Fixed**	Derived by reducing price estimate in Option A to account for reduction in O/H fee from 19% to 12%	**$856**

134

- To settle the dispute concerning the alleged differing subsurface conditions in the Queenston shale formation and all other outstanding claims prior to November 30, 2008, OPG and Strabag agreed to convert the fixed price DBA into a target cost DBA with cost and schedule incentives and disincentives, and incorporate changes in the tunnel route to minimize further excavation with the crown in the challenging Queenston shale formation. Negotiated changes to the DBA include a target in-service date of June 15, 2013, target cost of $985 M and a significant shift in the risk profile for completion of the tunnel construction.

Financing

- In 2005, financing for the project was arranged through the OEFC with a facility limit of $1B. Preliminary discussions have taken place with the OEFC regarding an increase in the facility, to $1.6B, as well as a timing extension. However, staff have indicated that given their current priorities it would be difficult to expedite the required "Minister Directive" because OPG's Niagara Tunnel Project spend is currently well below the $1B facility limit. OEFC currently plans to have the final amendment executed after its third quarter Board meeting in September 2009.

Project Execution Strategy

- During October and November 2008, the parties negotiated a non-binding Principles of Agreement that would settle all claims up to November 30, 2008 and move to a Target Cost Contract for the remainder of the project with schedule and cost incentives and disincentives. The key tenets of the Principles of Agreement were as follows:
    - Strabag claimed that it had incurred a loss of $90M up to November 30, 2008. Under the Principles of Agreement, OPG would pay Strabag $40M to settle all claims up to November 30, 2008, leaving Strabag with a loss of approximately $50M.
    - Should the $90M loss not be substantiated, the agreement allows OPG to claw back the $40M on a prorated basis.
    - From December 1, 2008 onwards, Strabag could earn a $20M completion fee plus maximum cost and schedule incentives of $40M. If both Target Cost and Schedule are met, Strabag's loss will be reduced from $50M to $30M. Maximum incentives for early completion and lower cost will result in Strabag making a profit of $10M. If the project is late or cost is exceeded, Strabag will incur a $50M loss.
    - The incentive (bonus / liquidated damages) associated with the Guaranteed Flow Amount[1] (tunnel flow capacity more or less than 500 m^3/s) remains unchanged.
- On November 19, 2008, OPG's Major Projects Committee reviewed the Principles of Agreement and endorsed management's plan to proceed to build upon the Principles of Agreement by negotiating a Term Sheet followed by an Amended Design Build Agreement with Strabag. On February 9, 2009, OPG and Strabag executed a non-binding Term Sheet that further elaborates on the Principles of Agreement.
- Since then, the parties negotiated a Target Schedule of June 15, 2013 and a Target Cost of $985M. Both of these targets were developed on an open book basis with the OR and OPG auditors having access required to verify the reasonableness of key inputs. The Target Schedule is premised on a horizontal realignment that reduces the tunnel length by approximately 200 m, and a vertical realignment to exit the Queenston shale and move to the overlying rock formations where tunnelling conditions are expected to improve.

[1] Guaranteed Flow Amount means the tunnel flow capacity guaranteed by the contractor at the reference hydraulic head and the reference elevation of energy grade line defined in the Design / Build Agreement.

9.2 Amended Agreement

The original DBA was used as the base for the Amended Design Build Agreement ("ADBA"). Most DBA provisions were retained unchanged except as necessary to convert the agreement to a target cost contract.[37] Under the ADBA, OPG and Strabag agreed on a Target Cost of $985M, a contract schedule with Substantial Completion by June 15, 2013 and changes to the allocation of risk. Strabag will be entitled to its costs to complete the project and incentives will apply if it completes the project for less than the Target Cost or before the agreed Substantial Completion date. Conversely, disincentives will apply if the costs exceed the Target Cost or the project is late.

The ADBA defines Actual Cost as the $302M paid to Strabag prior to December 1, 2008 plus the accumulated Allowed Costs (defined below) from December 1, 2008 onwards, minus any proceeds from the sale of assets and any insurance payments received by Strabag. Actual Cost will be used to calculate the applicable cost incentives and disincentives which apply to Strabag. Strabag will be reimbursed for all costs it incurs to complete the project ("Allowed Costs") that are not specified to be Disallowed Costs in the ADBA. Disallowed Costs include items such as costs arising from Strabag's negligence, wilful misconduct or breach of Applicable Law, head office costs, interest costs, certain insurance deductibles, costs for warranty work, costs to correct or remove a defective part of the project and third party liability. Strabag also will be entitled to apply an overhead recovery fee of 5 per cent to Allowed Costs from December 1, 2008 onwards to cover the costs of head office support. OPG is to make monthly payments under the contract based on anticipated Allowed Costs for the coming month and true up the prior month's payments.

The Target Cost will be adjusted to reflect changes in costs for certain items, as baseline assumptions were included in the calculation of the Target Cost with the expectation that the Target Cost would be adjusted up or down to reflect actual circumstances such as, for example, changes in the baseline inflation assumption or diesel fuel costs.

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[37] Capitalized terms in this section are defined in the ADBA, which is included in the CD of NTP Key Documents accompanying this Exhibit.

136

Cost Description	Pre-2008 Actual	2008 Actual	2009 Actual	2010 Actual	2011 Actual	2012 Actual	2013 Actual	2014-2015 Projected	Estimated Capital Cost To Complete
Tunnel Contract (including incentives):									
Mobilize / Demobilize	18.1	3.5	3.6	(0.1)	-	2.3	3.7	-	**31.0**
Performance Letter of Credit	1.8	0.4	1.6	0.7	0.9	1.0	1.0	0.3	**7.6**
Insurance Premium	1.9	0.4	0.3	-	-	-	-	-	**2.6**
Design	5.2	0.3	1.5	1.0	1.2	1.7	0.8	-	**11.6**
Intake Channel and Walls	53.7	1.7	2.1	-	1.4	1.7	3.7	-	**64.4**
Diversion Outlet Canal	11.2	-	0.2	0.5	-	0.8	2.7	-	**15.4**
Dewatering Shafts	0.3	2.9	0.5	0.1	-	-	-	-	**3.8**
Intake Structure	-	0.3	0.6	4.9	(0.2)	0.5	-	-	**6.1**
Intake Gates	-	-	0.5	0.1	0.6	2.7	0.9	-	**4.7**
Outlet Structure	1.4	0.9	-	-	7.8	1.7	-	-	**11.7**
Outlet Gates and Hoist	-	-	0.7	-	0.6	2.5	0.9	-	**4.8**
Diversion Tunnel	32.7	89.8	128.4	140.2	156.8	132.7	6.4	-	**687.2**
Tunnel Boring Machine	78.2	-	-	-	-	-	-	-	**78.2**
Flow Verification Test	-	-	-	-	-	-	0.3	-	**0.3**
Demolish Dewatering Structure	-	-	-	0.1	-	-	-	-	**0.1**
Dispute Review Board Cost	0.1	0.2	-	-	-	-	-	-	**0.3**
Scope Changes	0.3	0.3	0.3	-	(0.1)	-	-	-	**0.7**
Provisional Sum	0.1	0.1	-	-	-	-	-	-	**0.2**
Changes in Applicable Law	-	0.1	-	-	-	-	-	-	**0.1**
Warranty Administration Fee	-	-	-	-	-	-	0.1	-	**0.1**
Office and General Cost	-	-	28.2	18.8	14.5	13.5	(0.8)	4.4	**78.4**
Overhead Recovery	-	-	8.8	8.4	9.3	8.2	1.5	-	**36.2**
Interim Completion Fee	-	-	-	-	10.0	-	-	-	**10.0**
Substantial Completion Fee	-	-	-	-	-	-	10.0	-	**10.0**
Schedule Incentive	-	-	-	-	-	-	40.0	-	**40.0**
One-time Settlement Interest	-	-	-	1.4	-	-	-	-	**1.4**
Contingency	-	-	-	-	-	-	-	5.8	**5.8**
Total Tunnel Contract Costs	***205.0***	***100.9***	***177.4***	***176.1***	***202.8***	***169.2***	***71.2***	***10.4***	***1,112.9***
OPG Project Management	2.0	0.4	0.4	0.4	0.5	0.4	0.4	0.3	**5.0**
Owner's Representative	10.9	4.4	4.5	4.9	4.8	4.2	2.2	0.2	**36.2**
Other Consultants	3.2	1.3	0.5	0.3	0.6	0.2	0.3	0.1	**6.5**
Environmental / Compensation	8.2	0.1	0.1	0.1	0.2	0.1	-	-	**8.7**
Other Contracts / Costs	50.5	7.1	3.2	8.2	4.7	(3.1)	(4.5)	2.3	**68.4**
Interest	20.4	17.2	27.4	41.8	50.6	60.1	17.0	-	**234.5**
Total Capital Project Costs	**300.2**	**131.3**	**213.5**	**231.8**	**264.2**	**231.2**	**86.6**	**13.3**	**1,472.0**

6.2.3 Owners Representative

The Owner's Representative ("OR"), Hatch Mott MacDonald in association with Hatch Acres ("Hatch"), provides independent monitoring, review, auditing, testing, and reporting of the contractor's designs, activities and products. Hatch administers the contract, performs continuous review of contract performance and coordinates project meetings and documents. Hatch has a full-time onsite organization whose main objective is to ensure the contractor's compliance with the DBA/ADBA and to facilitate achievement by OPG of the project's safety, cost, schedule and quality objectives.

OPG chose Hatch to be the OR for the following reasons:

- Hatch Mott MacDonald is one of the top tunneling firms worldwide.
- Hatch, working with Acres Bechtel, acted as the Owner's Representatives when this project was tendered in 1998 and OPG was very positive about Hatch's performance.
- Acres had provided engineering support on Beck 3 and the tunnel design since 1991. Hatch purchased Acres in June 2004.
- The sub-surface risks of this project were investigated and analyzed by Acres and Hatch. As a result, Hatch has considerable knowledge about the project, including geological risks, permitting and costs. To transfer this information to another firm would have required substantial time and effort.
- Hatch is Canadian owned and headquartered in Mississauga. As a result, OPG has excellent access to senior personnel at Hatch.

Hatch has acted as the OR through both phases of the NTP. In Phase One, the planning and procurement phase, the OR was active in all aspects of the solicitation including pre-qualification of bidders and the RFP process. At the pre-qualification stage, the OR developed the evaluation criteria, reviewed submissions and made recommendations to OPG as to which entities should be pre-qualified. In collaboration with OPG and external legal counsel, the OR prepared the RFP documents provided to prospective bidders, including the proposed contract and the GBR, and administered the bidding process.

Appendix 1.1(j) - Contract Price

BREAKDOWN OF CONTRACT PRICE

MEASUREMENT PAYMENT ITEM	DESCRIPTION OF WORK	ORST INCLUDED	TOTAL
	Insurance Premium	170,000	2,724,181
1.1	Mobilization/Demobilization	871,842	31,729,969
1.2	Maintenance Bond in the form of Appendix 4.1(f)	0	610,749
1.3	Performance LC	0	2,544,789
1.5	Design	0	5,870,313
1.6	Accelerating Wall, Intake Channel and Approach Wall	952,100	54,862,211
1.7	Diversion Outlet Canal	33,520	12,730,052
1.8	Dewatering System Shafts	145,367	3,787,251
1.9	Intake Structure	56,789	5,334,935
1.10	Intake Gates	8,389	2,325,461
1.11	Outlet Structure	60,149	7,222,558
1.12	Outlet Structure Gate and Hoist	16,729	5,957,260
1.13	Diversion Tunnel	7,489,430	406,881,138
1.14	Tunnel Boring Machine	4,738,617	78,242,470
1.15	Flow Verification Test	0	94,682
1.16	Demolition and Disposal of Dewatering Structure (optional)	11,490	1,495,595
	Proponent's Estimate of its DRB Cost (50% of overall cost)	0	221,557
	Total Contract Price	14,554,422	**622,635,171**

This Contract Price is to divert a GFA of 500 m^3/s (at the reference hydraulic head and the reference elevation of the energy grade line as defined in Appendix 1.1(aa), Flow Verification Test) of the flow of the Niagara River from an intake located under the International Niagara Control Works, to an outlet that will discharge into the existing canal system that feeds the existing Sir Adam Beck hydroelectric plants at Queenston, Ontario.

DF2-1
ADBA Attach#9 Part 1

Appendix 1.1(TTT) – Target Cost

The Target Cost is $985,000,000.00

For the purposes of cost control, cost projection and cost performance indices only, the Target Cost will be allocated in the following manner:

Item	Description	Pre Effective Date	Post Effective Date	Total
1	Mobilize/Demobilize	$25,037,603.51	$5,940,000.00	$30,977,603.51
2	Maintenance Bond	$ 0.00	$700,000.00	$700,000.00
3	Performance LC	$2,135,833.33	$3,291,458.00	$5,427,291.33
4	Insurance Premium	$2,293,333.33	$2,000,047.00	$4,293,380.33
5	Design	$5,425,340.78	$4,277,000.00	$9,702,340.78
6	Intake Channel and Walls	$58,386,649.32	$6,372,932.00	$64,759,581.32
7	Diversion Outlet Canal	$11,395,047.88	$1,511,734.00	$12,906,781.88
8	Dewatering Shafts	$3,159,097.60	$490,035.00	$3,649,132.60
9	Intake Structure	$304,440.00	$8,331,354.00	$8,635,794.00
10	Intake Gates	$ 0.00	$2,478,138.00	$2,478,138.00
11	Outlet Structure	$2,292,196.28	$10,527,698.00	$12,819,894.28
12	Outlet Gate and Hoist	$ 0.00	$3,603,112.00	$3,603,112.00
13	Diversion Tunnel	$112,171,914.06	$576,844,664.93	$689,016,578.99
14	Tunnel Boring Machine	$78,242,470.00	$ 0.00	$78,242,470.00
15	Flow Verification Test	$ 0.00	$569,097.00	$569,097.00
16	Dewatering Structure	$ 0.00	$1,452,034.00	$1,452,034.00
17	DRB Estimated Cost	$291,671.11	$75,000.00	$366,671.11
18	Item not used	$ 0.00	$ 0.00	$ 0.00
19	Scope Changes	$739,235.99	$ 0.00	$739,235.99
20	Provisional Sum	$206,152.03	$ 0.00	$206,152.03
21	Changes in Applicable Law	$117,500.00	$117,500.00	$235,000.00
22	Warranty Administration Fee	$ 0.00	$100,000.00	$100,000.00
23	Office and General Cost	$ 0.00	$54,119,710.85	$54,119,710.85
	Target Price	**$302,198,485.22**	**$682,801,514.78**	**$985,000,000.00**

Cost Description	Pre-2008 Actual	2008 Actual	2009 Actual	2010 Actual	2011 Actual	2012 Actual	2013 Actual	2014-2015 Projected	Estimated Capital Cost To Complete
Tunnel Contract (including incentives):									
Mobilize / Demobilize	18.1	3.5	3.6	(0.1)	-	2.3	3.7	-	**31.0**
Performance Letter of Credit	1.8	0.4	1.6	0.7	0.9	1.0	1.0	0.3	**7.6**
Insurance Premium	1.9	0.4	0.3	-	-	-	-	-	**2.6**
Design	5.2	0.3	1.5	1.0	1.2	1.7	0.8	-	**11.6**
Intake Channel and Walls	53.7	1.7	2.1	-	1.4	1.7	3.7	-	**64.4**
Diversion Outlet Canal	11.2	-	0.2	0.5	-	0.8	2.7	-	**15.4**
Dewatering Shafts	0.3	2.9	0.5	0.1	-	-	-	-	**3.8**
Intake Structure	-	0.3	0.6	4.9	(0.2)	0.5	-	-	**6.1**
Intake Gates	-	-	0.5	0.1	0.6	2.7	0.9	-	**4.7**
Outlet Structure	1.4	0.9	-	-	7.8	1.7	-	-	**11.7**
Outlet Gates and Hoist	-	-	0.7	-	0.6	2.5	0.9	-	**4.8**
Diversion Tunnel	32.7	89.8	128.4	140.2	156.8	132.7	6.4	-	**687.2**
Tunnel Boring Machine	78.2	-	-	-	-	-	-	-	**78.2**
Flow Verification Test	-	-	-	-	-	-	0.3	-	**0.3**
Demolish Dewatering Structure	-	-	-	0.1	-	-	-	-	**0.1**
Dispute Review Board Cost	0.1	0.2	-	-	-	-	-	-	**0.3**
Scope Changes	0.3	0.3	0.3	-	(0.1)	-	-	-	**0.7**
Provisional Sum	0.1	0.1	-	-	-	-	-	-	**0.2**
Changes in Applicable Law	-	0.1	-	-	-	-	-	-	**0.1**
Warranty Administration Fee	-	-	-	-	-	-	0.1	-	**0.1**
Office and General Cost	-	-	28.2	18.8	14.5	13.5	(0.8)	4.4	**78.4**
Overhead Recovery	-	-	8.8	8.4	9.3	8.2	1.5	-	**36.2**
Interim Completion Fee	-	-	-	-	10.0	-	-	-	**10.0**
Substantial Completion Fee	-	-	-	-	-	-	10.0	-	**10.0**
Schedule Incentive	-	-	-	-	-	-	40.0	-	**40.0**
One-time Settlement Interest	-	-	-	1.4	-	-	-	-	**1.4**
Contingency	-	-	-	-	-	-	-	5.8	**5.8**
Total Tunnel Contract Costs	***205.0***	***100.9***	***177.4***	***176.1***	***202.8***	***169.2***	***71.2***	***10.4***	***1,112.9***
OPG Project Management	2.0	0.4	0.4	0.4	0.5	0.4	0.4	0.3	**5.0**
Owner's Representative	10.9	4.4	4.5	4.9	4.8	4.2	2.2	0.2	**36.2**
Other Consultants	3.2	1.3	0.5	0.3	0.6	0.2	0.3	0.1	**6.5**
Environmental / Compensation	8.2	0.1	0.1	0.1	0.2	0.1	-	-	**8.7**
Other Contracts / Costs	50.5	7.1	3.2	8.2	4.7	(3.1)	(4.5)	2.3	**68.4**
Interest	20.4	17.2	27.4	41.8	50.6	60.1	17.0	-	**234.5**
Total Capital Project Costs	**300.2**	**131.3**	**213.5**	**231.8**	**264.2**	**231.2**	**86.6**	**13.3**	**1,472.0**

2 Purpose of Project and Objectives

2.1 Project Purpose

The new diversion tunnel is intended to facilitate more efficient utilization of available water in the existing Sir Adam Beck generating complex, increasing the average annual energy production by about 1.6 TWh. At an estimated Levelized Unit Energy Cost (LUEC) of approximately 4.8 ¢/kWh (2005 dollars), the Project provides a competitive alternative for meeting the needs of the Province.

2.2 Objectives

The objective of the Project is the successful design and construction of a diversion tunnel to divert at least an additional 500 m^3/s of flow from the upper Niagara River to the Sir Adam Beck generating complex, executed in a safe, environmentally responsible, economic and timely manner as described below and to the extent practical and possible, in a manner that reflects and meets the requirements of the primary stakeholders.

2.2.1 Safety

OPG considers safety as a primary objective with a Project goal to maintain a safe working environment that results in completion of the Project with zero fatalities, zero critical injuries, and zero lost time injuries while maintaining the safety of the public at all times. In OPG's "Owner Only" capacity on this project, the Contractor will be responsible for safety within its controlled areas. For Part Project Area (as described below) activities carried out at the International Control Works (INCW), however, OPG will assume the role of "Constructor" at which times the Contractor will execute the work in a manner that is consistent with OPG/NPG safety procedures and the OR will manage safety on OPG's behalf.

2.2.2 Environmental Protection

The Project is to be executed to meet the commitments contained in the Environmental Assessment (EA) and the conditions of the EA Approval, all legislated environmental and mitigation requirements and to provide at project completion, minimal long-term environmental obligations to the OPG Niagara Plant Group.

2.2.3 Quality

The Project is to achieve a high overall quality of design and construction and meet all specified performance requirements. It is intended that the design and construction of the project provide for a 90-year service life for key elements of the facility such as the tunnel, intake structure and outlet structure, and will not result in any forced outages during that period. Other components of the project will be designed and constructed to meet, at a minimum, existing legal requirements. The Design/Build Agreement requires the Contractor to demonstrate that the Guaranteed Flow Amount (GFA) of 500 m^3/s through the diversion tunnel has been achieved by conducting specified flow tests. Should the GFA not be met, the Contractor will be liable for liquated damages or if it is exceeded, a bonus will be available.

2 Purpose of Project and Objectives

2.1 Project Purpose

The new diversion tunnel is intended to facilitate more efficient utilization of available water in the existing Sir Adam Beck (**SAB**) generating complex, increasing the average annual energy production by about 1.6 TWh. The Project provides a competitive alternative for meeting the needs of the Province with clean, renewable hydroelectric energy.

2.2 Objectives

The overall objective of the Project is the successful design, construction, commissioning and placing into service of a tunnel to divert at least an additional 500 m^3/s of flow from the upper Niagara River to the SAB generating complex, executed in a safe, environmentally responsible, timely and economic manner as described below and to the extent practical and possible, in a manner that reflects and meets the requirements of the primary stakeholders.

2.2.1 Health and Safety

OPG considers health and safety as a primary objective with a Project goal to maintain a safe working environment that results in completion of the Project with zero fatalities, zero critical injuries, and zero lost time injuries while maintaining the safety of the public at all times. In OPG's "Owner Only" capacity on this project, the Contractor is responsible for safety within its controlled areas. For the Part Project Area (as described below) during the execution of some of the work carried out at the International Niagara Control Works (**INCW**), OPG will assume the role of "Constructor" at which times the Contractor will execute the work in a manner that is consistent with OPG/NPG safety procedures and the OR will manage safety on OPG's behalf.

2.2.2 Environmental Protection

The Project is to be executed to meet the commitments contained in the EA and the conditions of the EA Approval, all legislated environmental and mitigation requirements and to provide at project completion, minimal long-term environmental obligations to the OPG/NPG.

2.2.3 Quality

The Project is to achieve a high overall quality of design and construction and meet all specified performance requirements. It is intended that the design and construction of the project provide for a 90-yr service life for key elements of the facility such as the tunnel, intake structure and outlet structure, and will not result in any forced outages during that period. Other components of the project will be designed and constructed to meet, at a minimum, existing legal requirements. The ADBA requires the Contractor to demonstrate that the Guaranteed Flow Amount (**GFA**) of 500 m^3/s through the diversion tunnel has been achieved by conducting specified flow tests. Should the GFA not be met, the Contractor will pay OPG a disincentive or if it is exceeded, an incentive amount will be paid by OPG to the Contractor.

2.2.4 Schedule and Cost

The project is to be maintained within the approved schedule and budget. Decisions regarding any deviation from approved budget and/or schedule will be based on the net business impact,