

NIAGARA DIVERSION TUNNEL REPORT

Prepared for Ontario Power Generation

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Page of Contents

1	Executive Summary
2	Project Site Investigation Overview and Scope of Document Review
	2.1 Concept Phase Geotechnical Investigations
	2.2 Definition Engineering Phases
3	Site Conditions
	3.1 Design Challenges
	3.1.1 Ground Characterization Along Alignment
	3.1.2 High Horizontal Stresses
	3.1.2 Time Dependent Deformations
	3.2 Conceptual Phase Investigation
	3.2.1 Drilling Along Tunnel Alignment
	3.2.2 Explorations in the St. David's Gorge Area
	3.2.3 In Situ Stress Measurements
	3.2.4 Laboratory Testing of Rock Core Samples
	3.3 Definition Engineering Phase 1
	3.3.1 Drilling Along Tunnel Alignment
	3.3.2 Explorations in the St. David's Gorge Area
	3.3.3 In Situ Stress Measurements
	3.3.4 Laboratory Testing of Rock Core Samples
	3.4 Definition Engineering Phase 2 Exploratory Adit
	3.4.1 Adit Enlargement
4	Site Investigation Results
	4.1 Ground Characterization for Design Analyses Along the Alignment
	4.2 Rock Mass Strength for Design Analysis
	4.2.1 FEM Analysis
	4.2.2 Wedge Analysis
	4.2.3 Convergence-Confinement Method
	4.2.4 Beam-Spring Model
	4.3 High Horizontal Stress
	4.4 Time Dependent Deformations
	4.5 Constructability Considerations
	4.5.1 Excavation and Support
5	Conclusions in Regard to the Scope and Quality of the Tunnel Site Investigations
6	OPG Decision to Bring the Dispute to DRB for a Hearing
	6.1 Design Build Agreement
	6.2 GDR and GBR
	6.3 Dispute Review Board
	6.4 DRB Hearing on Strabag Claim for Differing Site Conditions
	6.5 Conclusions
7	OPG Performance at the DRB Hearing
	7.1 OPG Position and Rebuttal Papers
	7.2 DRB Recommendations
8	OPG Decision to Renegotiate a Revised Contract with Strabag
	8.1 Discussion and Conclusions
9	Summary and Conclusions
	Glossary

NIAGARA DIVERSION TUNNEL PROJECT

1.0 Executive Summary

I was requested by Tory's to review all pertinent geotechnical investigations conducted and reports prepared for the design and construction of the 14.4 m excavated diameter, approximately 10.4 Km long (as designed vs. 10.2 Km as constructed), Niagara Diversion Tunnel.

I have done so and formed an opinion that these site investigations addressed the appropriate design and construction issues and that the studies undertaken were completed to professional standards and exceeded those standards in some cases.

I was also requested to review the design work undertaken by Strabag during their proposal preparation and subsequently during the work. I have done so and formed an opinion that the design work performed was conducted to an appropriate professional standard.

In addition, I was requested to form an opinion as to whether it was appropriate to refer the dispute between OPG and the contractor Strabag for a hearing conducted by the Dispute Review Board (DRB) and to form an opinion as to the way OPG conducted the hearing. I have done so and found that it was appropriate to take the dispute before the DRB and further that OPG conducted the hearing in a proper manner.

Finally, after review of the subsequent DRB recommendations coupled with my own evaluation of the circumstances, I formed the opinion that the decision to re-negotiate a revised contract with Strabag was appropriate and reasonable given the circumstances of the dispute and the status of the project.

2.0 Project Investigations Overview and Scope of Document Review

The design and construction of the Niagara Diversion Tunnel as part of the Niagara River Hydroelectric Development was the culmination of various geotechnical investigations and design efforts beginning in 1983.

2.1 Concept Phase Geotechnical Investigations

The main objectives were to provide the essential geotechnical data for conducting technical feasibility and economical comparison of various development alternatives being studied for increasing the generating capacity at the Sir Adam Beck (SAB) complex.

The investigations were initiated in 1983 and conducted successively in 1984, 1984/85, 1986 and 1988/89. The geotechnical data collected during this period were for various project arrangements considered at that time and were not solely for the project actually constructed. Refer pages 2-1 to 2-18 of the Geotechnical Data Report (GDR) for a comprehensive description of the various studies done. (GDR is discussed below at the end of section 2.2)

In addition geological and geotechnical data were acquired by OPG during the construction of the SAB Generating Station (GS) 2 in the 1950s.

The results of these investigations were summarized in Feasibility Report 87269 Rev.1 dated March 1989.

2.2 Definition Engineering Phases

In the fall of 1988 OPG advanced the project into the Definition Engineering Phase in which environmental assessment and preliminary engineering were carried out. Phase 1 was completed in 1990 and included various site investigations. [Refer to GDR pages 3-1 to 3-20 for a full description]. A final report [Report 91150] consisting of five volumes was issued in May 1991.

This was followed by Phase 2 consisting of an Exploratory Adit (Adit) excavated in the Queenston Formation (Queenston) to the elevation of the proposed tunnels and enlarged at the end to the approximate diameter of the proposed tunnels. This work was completed in 1992/93 [Refer GDR pages 4-1 to 4-44 for a full description] and a seven volume draft report [Report NAW130-P4D-10120-0005-00] issued to OPG in December 1993.

Additional laboratory testing was done from 1994 to 1996 on samples of core from the Adit and a final draft report [Report NAW130-P4D-10120-007-00] issued in February 1997.

The geological and design issues studied in these investigations are addressed in detail below in Section 3.0 as is the manner in which the work was completed.

The GDR was prepared for inclusion in the document package issued to the selected Design-Build teams for their use in the preparation of their proposals. The GDR consisted of 12 volumes and incorporated all of the pertinent data collected during the phases of the work described above. It included a bibliography listing all of the investigation reports. A Geotechnical Baseline Report (GBR) was also prepared and issued in the RFP as GBR A. The GBR is discussed further below in section 6.1

3.0 Site Investigations

The primary aim of site investigations for a rock tunnel is to characterize the rock mass conditions sufficiently so that the design approach and selected construction methods can address the indicated ground conditions. The appropriate approach was adopted for the Niagara Tunnel, which was to phase the investigations beginning with general studies for the Conceptual Phase that began to define rock mass properties, overall stratigraphy, in situ stress conditions, the groundwater regime and other geologic hazards such as the presence of gas. Based on these results and preliminary analyses, a second phase of investigation was done for the Definition Engineering Phase which included additional borings with field and laboratory testing that resolved data gaps and focused on acquiring data to address design issues. Additional phases were completed as necessary until an appropriate level of confidence was reached that the geotechnical related risk issues had been mitigated to an acceptable level.

3.1 Design Challenges

It was recognized from the beginning that the tunnel design and construction presented several design challenges; chiefly the high horizontal stress, the presence of the St. David's Gorge, time dependent deformation of the rock mass and the presence of sulphates in the groundwater. The various site investigations were directed at recovering

physical data and making qualitative geological assessments for preliminary analyses and so address these challenges. The related design and constructability issues are discussed below. The subsequent discussion will cover the actual investigations performed during the Concept and Definition Phases of the work.

3.1.1 Ground Characterization Along Alignment

The tunnel length of 10.4 km results in a natural variability in the rock mass characteristics of the rock formations to be excavated, including; rock mass strength, rock structure (presence and character of discontinuities such as bedding and joints), lithology (nature of the rock material such as siltstone, mudstone or shale) and the piezometric level of groundwater as well as its quality, in the formations to be excavated.

All of these characteristics needed to be quantified and the tunnel length characterized appropriately, with differences identified. The rock mass was generally known to vary from weak to moderately strong.

The depth of the tunnel was dictated by the necessity of passing beneath the glacial soil filled ancestral river channel some 800 m wide, named the St. David's Gorge (Gorge). The location and character of the top of rock in the Gorge and in relation to the tunnel roof (crown) was therefore an issue.

3.1.2 High Horizontal Stresses

The presence of high horizontal stress had been recognized in the region and on previous OPG construction at the site. The identification of the stress magnitude and direction was an important objective due to the high stresses that develop around the excavation perimeter upon excavation and the resulting potential for overstress of the rock mass. The nature of the failure which would occur if the rock remained unsupported after excavation is termed the 'rock mass behavior'. This relates to the type of initial support to be placed and the timing of placement of the support – the elapsed time of stable rock conditions is commonly referred to as the 'stand-up time' and is the window for erection of the initial tunnel support. Because the stand-up time is affected by the chosen construction method it is deemed to be a constructability issue as well as a design

issue. The large size of the proposed tunnels would also be part of the concern regarding tunnel stability upon excavation.

3.1.3 Time Dependent Deformation

During the construction of the vertical shaft Wheel Pit of the Canadian Niagara and Toronto Power Plants, the 5.5 m wide and 50 m deep slots showed an inward movement of both walls. The total maximum inward movement of both walls over a 68 year period was approximately 7 cm. The data shows a general trend of decreasing rate of rock movement with time. These long term deformations were in the rock formations above the Queenston and it was known that the Queenston was prone to swelling, hence both of these mechanisms could potentially generate long term loading on the lining. The presence of saline and sulphate bearing groundwater with the resulting potential for corrosion effects on steel and sulphate attack on concrete, plus high operating pressures in the finished tunnel; all became issues bearing on the design of the tunnel lining. These factors and the requirement for a 90 year design life would define the design and eventual thickness of the concrete lining, in itself a very significant challenge.

3.2 Conceptual Phase Investigations

As described in general above, this phase occurred in the period from 1983 to 1989. During this period the investigations were broadly based so only the parts relating to the tunnel alignment will be discussed. A list of the activities is presented below:

- Geological mapping including joint measurements of rock outcrops;
- Drilling and core recovery of 5 boreholes, SD-1 to 5, coupled with seismic reflection surveys to determine the location of the top of rock in the Gorge;
- Drilling of 25 borings NF-1 to NF -26 (not NF-16) along potential tunnel alignments, surface and underground power house locations and around the PGS reservoir;
- Installation and monitoring of multi-level Westbay piezometers in 4 borings (NF-2 to NF-4 and NF-6);
- In-situ stress measurements by over-coring in boring NF-1 and by hydraulic fracturing in boreholes NF-3 and NF-4; and

- Laboratory testing on rock samples including physical and mechanical properties, compression and tensile strength tests; also tests on the time dependent deformation characteristics of core samples from the Queenston.

The results of these Conceptual Phase investigations were presented in Volume 11 of the GDR. A review of these investigative reports indicates that in general the following important activities (Sections 3.2.1, 3.2.2, 3.2.3, 3.2.4) were accomplished in regard to the three principal areas (described in Sections 3.1.1, 2, and 3 above) of design issues for the tunnel.

3.2.1 Drilling Along Tunnel Alignment

To quantify the natural variability of the rock mass along the alignment it was necessary to drill exploratory holes, conduct field tests, recover core for the purpose of identification of the lithology, to identify stratigraphic relations between different rock formations, to identify groundwater levels and groundwater quality and to provide core for various laboratory tests.

In 1983 four vertical boreholes (NF-2 to NF-5) were drilled south of the Gorge using wireline core recovery methods, each penetrating 30m into the Queenston. The core lithology was logged to define stratigraphic relations between formations; also Core Recovery (CR) and Rock Quality Designation (RQD) were recorded and the character of the discontinuities logged. Constant head permeability tests were carried out as the holes advanced. In situ permeability tests were done in borings NF-2 and NF-4 in 1984 in the various rock strata to be excavated by the tunnel. Also a series of Westbay multi-level piezometers were installed in boreholes NF-2 to NF-4. These were designed to allow groundwater samples to be taken at any of the ports located in the various strata for water quality (chemistry) testing.

3.2.2 Exploration in the St. David's Gorge Area

It was necessary to define the bottom of the glacial sediment filled gorge so that the tunnel could be optimally located in the most favorable rock conditions.

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.

In 1984/85 core samples for the rock formations to be excavated from boreholes NF-5 to NF-7 were tested. Values were measured for the following parameters: UCS; tensile strength, Schimdt hammer hardness and free swell tests. Also core samples from various formations in borehole NF-7 were tested. Values were measured for the following parameters: anisotropic Poisson's Ratio and elastic modulus; UCS; free swell and semi-confined time dependent deformation of Queenston samples.

In 1986, seventeen core samples from the Queenston containing one or more clay seams were tested. Values were measured for the shear strength of the clay seams in both multi-stage direct shear and biaxial tests. Index testing consisting of grain size analysis and Atterberg Limits of the clay fillings were also done and mineralogical analyses of the clay. These results were used in the Wedge Analysis described below in Section 4.2.2.

In 1988/89 core samples from the Queenston were tested. Values were measured for the following parameters: anisotropic Poisson's Ratio; elastic modulus; UCS; tensile strength and free swell tests. A time dependent deformation test program on samples from the Queenston was also completed. The purpose of these tests was to evaluate the swell pressures that could be experienced by the finished lining system and so allow for them in the design.

The results of these laboratory test programs were incorporated in a data base of engineering and index parameters for the overall purpose of characterizing the rock formations present with respect to rock mass strength, modulus and swelling characteristics.

Initial stability analyses were performed using closed formed solutions with elastic properties and preliminary numerical modeling using finite element analysis using an early (1980) Hoek and Brown constitutive model with assumed rock mass factors based broadly on evaluations of Rock Mass Rating by Z.T. Bieniawski. These initial studies indicated that generally for the diameters considered, the Queenston rock would not be overstressed. It was recognized that UCS test values declined in proportion to the shale vs. siltstone content in the samples tested leading to a division of the Queenston into sub-units based on changes in lithology, particularly the proportion of siltstone versus mudstone and shale present. Also the UCS values of core box dried samples of Queenston were significantly stronger than saturated samples.

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

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

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:

- Strength and deformation testing;
- Time dependent deformation testing-swell tests;
- Petrographic analyses of thin sections from the Lockport and Queenston Formations;
- Point Load Strength Index testing;
- Chemical analysis of groundwater samples from piezometer installations; and
- Testing for hydraulic fracturing tensile strength and biaxial testing for deformation modulus of samples from over-coring tests (for in situ stress measurement).

A summary of the tests completed was presented in Tables 3.5 and 3.6 in the GDR and described in more detail in Section 12.1.2 of the GDR.

The 1992/3 Definition Engineering Phase 2 laboratory testing program addressed the following key issues regarding the engineering properties of the Queenston:

- Uniaxial (UCS), triaxial and tensile strength of intact rock;
- Direct shear strength tests of the major (very persistent and clay filled showing signs of movement) bedding planes sampled in the Adit; and
- Time dependent swelling characteristics of the Queenston in confined and unconfined tests to ascertain potential load on the final lining of the tunnel.

The scope of the testing program was presented in Table 4.9 of the GDR. Particular emphasis was placed on the proper sealing and storage of the rock core, with early testing of the samples to preserve the in situ moisture content. The results of the program were described in more detail in Section 12.4.3 of the GDR.

Additional testing on the time dependent deformation characteristics of the Queenston was done from 1994 to 1996 to further define the pressures to limit swelling; to investigate the effects of increasing axial load (analogous to swelling pressure build up on the tunnel lining); to investigate anisotropy by providing results for horizontal cores and to determine the swelling characteristics with pore water of different saline concentrations. The results of the program were presented in Section 12.4.4 of the GDR.

3.4 Definition Engineering Phase 2 Exploratory Adit

The excavation of the Adit represents a level of exploration rarely achieved due to the cost and was therefore a significant commitment to achieving the objective of ascertaining the rock behavior in an excavation of comparable size to the planned tunnel.

The Adit, excavated in 1992/93 entirely within the Queenston, was located in the vicinity of the power generating complex in order to provide access to and to develop the powerhouse test area and to allow over-coring stress measurements. The objectives of the program were to:

- To record qualitative observations of rock mass behaviour and to measure rock mass behaviour with instrumentation;
- Conduct in situ stress measurements;
- Record geological data by mapping of the excavation, photography and coring of the exposed rock; and
- Conduct in situ testing.

3.4.1 Adit Enlargement

Stage 3B Excavation mainly comprised of widening the end of the Adit as part of a trial enlargement. The main objective was to evaluate the full face Tunnel Boring Machine (TBM) excavation method by observing and measuring the rock mass response around an opening similar in span to the final excavation dimension. The test program was as follows:

- Developed an opening 12m wide and 4m high with a circular arch of radius 6.8m to simulate the upper part of the diversion tunnel;
- Rock deformations and the extent of the overstressed zone were measured with rod extensometers and surface convergence points. Stress changes at the roof were also measured;
- The excavation was supported with dowels and mesh; and
- The last approximately 5m of the enlargement was left unsupported for 48 hours to assess stand-up time of the arch.

Stage 3C Excavation consisted of further deepening of the opening by benching downwards to a full height of 12m to further observe the effects on stability of the crown, invert and face. Additional extensometers were installed at the springline of the full depth excavation and in the end wall invert to monitor deformation.

A detailed description of the Stage 2 Exploratory Adit Investigation Program was provided in Section 4.3 of the GDR.

4.0 Site Investigation Results

The results of all the phased site investigations conducted for the Conceptual Phase and the Definition Engineering Phases 1 and 2 were presented in the GDR as follows:

- Section 6 Surface Geological Mapping;
- Section 7 Results of Surface Drilling: Logging and Downhole Testing;
- Section 8 Results of Underground and In Situ Testing;
- Section 9 Exploratory Excavations- Geotechnical Conditions and Observations;
- Section 10 Results of Adit Enlargement and Field Instrumentation and Testing;
- Section 11 Groundwater and Gas; and
- Section 12 Results of Laboratory Testing of Rock Samples.

The GDR was a comprehensive document which gathered all of the data from numerous studies for a variety of concepts of power generation with various configurations and included detailed studies for the diversion tunnels inlet and outlet works and for underground power stations.

In my review I have focused on the site investigations related to the diversion tunnels which remained within a defined corridor from the start of the studies. The number of borings was appropriate given the relative uniformity of the Queenston. I have reviewed a sufficient number of examples of laboratory and field test results to form an opinion that they were completed in an appropriate manner. I will discuss below how the objectives of the investigations were met, in that the necessary data was provided for the appropriate design analyses and for evaluation of the perceived constructability issues.

The issues listed below are broadly described above in section 3.1:

- 3.1.1 Ground Characterization Along the Alignment;

- 3.1.2 High Horizontal Stresses; and
- 3.1.3 Time Dependent Deformations.

4.1 Ground Characterization for Design Analyses Along the Alignment

In order to characterize the ground conditions, the rock mass characteristics were required including intact rock UCS and triaxial strength and elastic modulus; rock structure, including RQD, core recovery, frequency of discontinuities such as bedding planes and joints; the characterization of the discontinuities including type of filling, roughness and persistence; the shear strength of prominent bedding planes; groundwater levels and the presence of gas; the chemistry of the ground water and logging of the rock type (lithology). All of this data was incorporated on a geologic profile prepared for the approximately 10.4 km long, 14.4 m diameter tunnel (Refer Strabag ILF Drawing No. PD-0101002). In this manner the alignment was split up into sections with similar properties for the purposes of analysis and subsequent support design.

4.2 Rock Mass Strength for Design Analysis

Strabag's designer ILF conducted design analyses including elastic beam-spring models, wedge analysis and convergence-confinement methods. These analyses, completed by ILF as part of the Strabag design-build proposal, were incorporated in two reports titled "Outline Design Basis and Method Statements" and "Structural Analysis for the Diversion Tunnel", both dated April 2005. Figure 3.1 Flow Chart for the Geotechnical Design was presented in the "Outline of Design Basis" and shows the steps and inputs required to arrive at the type of support to be used, beginning with the geotechnical parameters provided in the GDR and derived from the GDR data. The geotechnical data used in the various analyses was based on data from Table 6.16 of the GBR for rock mass strength and deformation and GDR Volume 2 and Fig. 12.1 for major bedding plane shear strength parameters.

4.2.1 FEM Analysis

The 2D and 3D finite element modeling (FEM) conducted by ILF as part of the design, enabled analysis of rock overstress at the tunnel periphery during excavation. The

method used a rock mass constitutive model (or rock mass failure criteria) derived from Hoek and Brown in 1980 and then successively improved by them with the last iteration issued in 2002. The model assumed isotropic conditions as evaluated by consideration of the block size formed in the mass by the discontinuities present. The inputs to the model were derived from the intact rock UCS and triaxial test data and a Geological Strength Index (GSI) that incorporated the rock structure. The model provided rock mass strength parameters for the numerical analysis. Also the Owner's Mandatory Requirements, chapter 8.3.4 of the Design Build Agreement (DBA) stipulated Hoek-Brown residual rock mass strength parameters ' $m_r = 1$ ' and ' $s_r = 0.001$ ' and a plastic shear strain in rock for peak to post-peak strength ranging from 0.5 to 2.0%.

The FEM modeling incorporated the measured existing high rock stress and could allow for the presence of the identified major bedding planes, as well as the delay in placing rock support sufficiently stiff to prevent further convergence and loosening of the rock mass. Direct shear tests on the major bedding planes provided the necessary shear strength parameters for this analysis.

Strabag's designer ILF conducted other analyses including elastic beam-spring models, wedge analysis and convergence-confinement methods for the purpose of initial and final support design which are described below.

4.2.2 Wedge Analysis

The wedge analysis identified kinematically feasible blocks that may slide or fall into the excavation. Logging of the discontinuities in the core and mapping of outcrops, coupled with evaluation of downhole photography, provided the characterization, orientation and frequency of joint sets and bedding present. Direct shear tests on discontinuities from recovered core provided the shear strength parameters for limit equilibrium analyses of the resulting blocks. This approach allowed appropriate initial support to be designed for a given set of geologic conditions in the Queenston and for the formations above.

4.2.3 Convergence-Confinement Method

The convergence-confinement method of analysis provides the interaction of ground behaviour, represented by a ground-reaction curve and tunnel support, represented by a support reaction curve. The available support pressure was evaluated from computations of the initial lining characteristics including rock reinforcement, shotcrete, and steel ribs. An essential input was the convergence measured during the excavation of the Exploratory Adit enlargement to the approximate planned tunnel diameter. The method was used for preliminary tunnel support design; short term time dependent load distribution of ground load to the initial support and long term time dependent load distribution of ground load to the final lining.

4.2.4 Beam-Spring Model

The Beam-Spring Model used linear elastic analyses to evaluate static loading of the tunnel lining from self weight, hydrostatic pressures, temperature, shrinkage and live loads from post lining grouting. The rock mass strength and deformation properties used were based on Table 6.16 from the GBR.

4.3 High Horizontal Stress

Extensive in situ testing was done to determine the stress regime along the alignment which enabled the tunnel to be divided into three parts and stress magnitudes and directions assigned to each. The results presented in the GDR and summarized in Table 6.14 of the GBR B covered the concept alignment in the RFP. Table 3.3 Stress Regimes for Design Purposes in “Outline Design Basis and Method Statements”, lists the in-situ stress conditions for different tunnel sections that were used in the proposal design.

4.4 Time Dependent Deformations

In their structural design analysis ILF analyzed swell pressure data using FEM and concluded that the area with swelling potential was small. This was mainly based on the advantages of the proposed dual shell lining system. In addition the existence of high horizontal stresses >5 MPa suppressed the swelling potential. This conclusion was based upon laboratory test results which reported that application of stress in one direction not only suppressed the swelling in that direction but reduced it in the orthogonal direction.

ILF concluded that the swelling potential was negligible due to the secondary stress field and the dual shell lining system.

ILF also considered in the final lining design the recognized long term deformation (which they termed rock squeeze behaviour) that had been observed and measured in previously constructed OPG underground facilities. The long term rock mass behaviour was considered by calculating a reduced stiffness modulus for the design life of 90 years using a creep rate based on the measured deformations.

4.5 Constructability Considerations

The presence of high horizontal stress had been recognized in the region and on previous OPG construction at the site. The identification of the stress magnitude and direction was an important objective due to the resulting high stresses which develop around the excavation perimeter upon excavation and to the potential for overstress of the rock mass. The nature of the failure which would occur if the rock remained unsupported after excavation is termed the ‘rock mass behavior’.

This relates to the type of initial support to be placed and the timing of placement of the support – the elapsed time of stable rock conditions is commonly referred to as the ‘stand-up time’ and is the window for erection of the initial tunnel support. Because the stand-up time is affected by the chosen construction method it is deemed to be a constructability issue as well as a design issue. The large size of the tunnel would also be part of the concern regarding tunnel stability upon excavation.

A review of the various design documents prepared by ILF and described above in 4.2 shows that these considerations were evaluated in detail by the contractor as described below. This in turn was made possible by the sufficiency and appropriateness of the geotechnical and geological data gathered in the site investigations described in Section 3 above and provided in the GDR and GBR for the contract.

4.5.1 Excavation and Support

As described by ILF in Section 3.5 of “Outline Design Basis and Method Statements” the requirements for excavation methods and support were based on the following factors:

- Worker safety;
- Structural stability of support system;
- Avoidance of rock mass loosening;
- Initial lining capacity; and
- Allowable deformations.

Section 3.5.4 Tunnel Support Application of the ILF report, describes the planned locations and type of support to be placed. These were carried through into the actual TBM configuration used and the detailed support designs provided.

5.0 Conclusions in Regard to the Scope and Quality of the Tunnel Site Investigations

It is my opinion that both the quality and standard of the site investigations met the generally recognized professional standards for work of a similar type and magnitude.

The natural variability of the 10.4 km alignment as manifested by variable lithology, high horizontal stresses in varying directions, rock strength anisotropy, adverse groundwater chemistry, methane gas potential, swelling pressures and long term deformation, provided significant challenges to OPG in providing the necessary and sufficient data to the Strabag design-build team for their use in the design and construction of the work. The geotechnical and geologic data gathered in the various site investigations as previously described, was sufficient and appropriate to meet these challenges. The field and laboratory testing provided appropriate data for the empirical and numerical analyses conducted. The excavation and instrumentation of the Exploratory Adit provided key data on the ground characterization and behavior. In conclusion, the appropriate and comprehensive designs and construction procedures developed by Strabag (summarized above) were based upon the geological and geotechnical data provided to them in the GDR and GBR.

6.0 OPG Decision to Bring the Dispute to the DRB for a Hearing

This section describes the background leading up to the decision to resolve a dispute on differing sub-surface (ground) conditions by taking it before the Dispute Review Board (DRB) for a hearing and the appropriateness of OPG actions in doing so.

6.1 Design Build Agreement

The Design-Build Agreement (DBA) between OPG and Strabag included Section 11 Dispute Resolution, which described the establishment and operation of a DRB as an alternative method of dispute resolution in that it provided a means of resolving disputes without resorting to arbitration or litigation. This was part of a risk sharing initiative provided by OPG; other elements included the provision of a GDR and a jointly negotiated GBR C (as discussed below) in the contract and for the contractor to place in escrow at the time of bid, data pertinent to the development of the cost estimate for the work. The DBA also included Section 5 Changes in Work with sub-sections 5.5 Differing Subsurface Conditions and 5.7 Resolution of Claims.

To further assist the parties in the resolution of any issues arising from the encountered ground conditions, Section 5.5 Differing Subsurface Conditions was included in the DBA. In particular Section 5.5(c) which states: “Notwithstanding Sections 5.5(a) and 5.5(b) and in lieu of the procedures described in Sections 5.5(a) and 5.5(b), the following procedure shall apply in full satisfaction of any change to the Contract Price and Contract Schedule relating to rock support resulting from differing subsurface conditions (the “Rock Support Adjustment”):

(1) on a continuous basis during the performance of the Work, the contractor will record the rock conditions (as defined in the GBR) encountered during the performance of the Work and measure the tunnel lengths thereof and OPG will review and verify such determinations. If the parties cannot agree, the positions of both parties shall be recorded. The resolution of any disagreements will be held in abeyance until the step described in section (4) below has been completed, 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;

...

(4) OPG shall promptly thereafter issue a one-time Project Change Directive^{F5-6-1} setting out the net change to the Contract price and Contract Schedule determined by completing the Rock Support Table as set out in (3) above.”

The referenced table was included in Section 8.1.3.7 of the GBR C as follows:

“Tunnel rock support will be designed to accommodate the Rock Conditions as given below. The in-situ Rock Conditions shall be determined based on the closest match to the Rock Characteristics within each Rock Condition defined below.”

The table is presented on page 37 of GBR C.

By this means the parties intended to provide a way to avoid protracted disagreements in regard to the type and placement of appropriate support for the encountered conditions. Note that Section 3.3 No OPG Control of the Work of the DBA, expressly makes Strabag responsible for “the Contractor’s means, methods, techniques, sequences or procedures respecting the work”.

The interpretation of these clauses by the parties in relation to the referral of a dispute for Differing Site Conditions to the DRB for a hearing is discussed further in section 6.4 below.

6.2 GDR and GBR

The geological and geotechnical aspects of the project were fully developed to the 100% level for inclusion in the RFP issued to the pre-selected design-build teams. The twelve volumes of the GDR consisted of material excerpted and summarized from the numerous studies and reports completed from 1983 to 1997. Version A of the GBR, termed GBR A, was also included in the RFP. GBR A provided baselines which were an assessment by OPG of the various geological and geotechnical risks to be encountered on the project; these baselines were distilled to a quantification of the physical parameters governing a particular risk, coupled with assessments of parameters such as ground behaviour, based on professional judgment. Version B of the GBR termed GBR B was provided by Strabag in their proposal. Version C of the GBR, termed GBR C was prepared and negotiated by both parties and included in the DBA for the work.

In providing these documents in the contract, OPG shared the risk with Strabag #5-6-1 that any differences that could potentially occur in regard to the actually encountered ground conditions were limited in that if ground conditions encountered were more adverse, the baselines would provide the means for resolving the ensuing claims. If the parties could not negotiate a resolution of the claim then it could be brought before the DRB for a hearing and the issuance of recommendations as to resolution. By sharing the risk in this manner, OPG benefitted in that the cost estimate for the work did not include contingencies for these risks which otherwise would be included.

6.3 Dispute Review Board

The DRB was formed at the start of the project and manned with recognized experts in the field of tunnel construction. The DRB visited the job on a regular basis and received documentation related to the progress of the work and the issues that arose during the course of the work. In this manner the DRB became familiar with the site staff and with the construction progress and the problems which arose in the course of the work. The contract required the DRB, if requested by either party, to hold a hearing on a particular dispute and to then issue non-binding recommendations.

6.4 DRB Hearing on Strabag Claim for a Differing Site Condition

Strabag filed a claim (PCN 017) for differing site conditions on 18/05/07, related to the ground conditions encountered at the Whirlpool/Queenston Formation contact, from chainage 0+806.5 to 0+839.7. The claim was filed under Section 5.5 (a) of the DBA, and was rejected by OPG on the basis that as it was a clearly a claim relating to rock support resulting from differing subsurface conditions, it must be resolved through the procedure as negotiated and agreed by the parties in Section 5.5 (c) and cannot fall within Section 5.5 (a). The procedure in Section 5.5 (c) is to apply “in full satisfaction of any change to the Contract Price and Contract Schedule relating to rock support resulting from differing subsurface conditions.”

Subsequently after several exchanges on the issue Strabag filed Dispute Notice No. 001 under Section 5.7 (a) of the DBA on 05/11/07. OPG again responded on 12/11/07 to the effect that the dispute notice was premature because the claim in PCN 017 must be

resolved through the procedure as negotiated and agreed by the parties in Section 5.5(a) and cannot fall under Section 5.5 (a). However OPG indicated that the first issue to put before the DRB was if they had jurisdiction of the dispute.

Strabag responded on 14/11/07 as follows: “The “rock support adjustment” clause allows for contract price and schedule adjustments to be made relative to the variation in the distribution of the Encountered vs. Expected GBR denoted rock conditions. The PCN 017 encountered rock conditions are clearly not within those denoted rock conditions nor were they anticipated by the GBR and are subsequently a Differing Subsurface Condition. Contrary to OPG’s stated position (letter of 31 October 2007), it was not the intention nor would it be reasonable to expect that the Rock Condition 6 would become a “catch all” for any possible rock condition ever encountered in the tunnel that did not fit into the conditions 4Q or 5 whether anticipated in the GBR or not.”

OPG responded on 28/11/07 in a memo confirming an agreement, reached in the meeting of 27/11/07, the following: “If the parties are unable to achieve a mutually acceptable plan for tunnel realignment within the next 3 months, that also resolves PCN 017 and as many other open issues under the contract as possible, the threshold issue will be the first to go to the DRB as soon as possible after February 29, 2008.”

On 27/02/08 Strabag issued Dispute Notice No.002 as per Section 5.7(a) and Section 5.7(c) of the DBA regarding PCN 017 in which they noted that the parties had agreed that the dispute should be placed before the DRB for resolution under Section 11 of the DBA.

On 05/03/08 OPG responded as follows:

“1.Strabag’s inability to achieve the agreed TBM advance rates and any “excessive” overbreak described in Strabag’s Proposal for Optimized Alignment and Revised Schedule are a direct consequence of the design, means and methods of construction eventually adopted by Strabag on this project. Pursuant to Section 5.4 of the DBA, Strabag accepted sole and exclusive responsibility and commercial risk for its choice of design, means and methods. Section 5.4 therefore precludes Strabag’s claim in its entirety. This is the preliminary issue for the DRB’s consideration under Section 11 of the DBA before any possibility of differing subsurface conditions under Section 5.5 of the DBA may be considered;

2. To the extent Strabag's claim is not fully disposed of under Section 5.4 of the DBA, Strabag's claim is a Rock Support Adjustment claim under Section 5.5(c) of the DBA and is premature. The parties agreed at the time of contract that the procedures set out in Section 5.5(c) were in full satisfaction and in lieu of any change to the Contract Price or Contract Schedule. Section 5.5(c) is mandatory.

Consequently no Section 5.7(a) "Dispute" is properly before the DRB at this time."

Eventually the parties agreed to hold a hearing starting on 23/06/08, on the differing site condition issue which had grown to include the following issues:

- Large Block Failures;
- Ground Conditions beneath St. David's Gorge;
- Insufficient Stand-up time;
- Excessive Overbreak; and
- Inadequate Table of Rock Conditions and Rock Characteristics.

6.5 Conclusions

In my opinion OPG's decision to go before the DRB with the issue was appropriate because of the following reasons. Section 5.5(c) (1) of the DBA provided that: "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." [Emphasis added]. It was eventually apparent that the ground conditions and support methods were severely impacting the work and would continue to do so as long as the tunnel excavation was in the Queenston Formation.

Given the merits of OPG's position a consideration of forcing Strabag to comply with the contract by invoking arbitration and bypassing the dispute resolution laid out in Section 11 of the DBA was a possibility. However, given the losses being sustained by Strabag at the time they would likely have stopped work and spent their project management efforts on the dispute thereby piling up additional substantive costs in addition to those being experienced. Also an adversarial relationship would inevitably have arisen between the parties, a further detriment to the completion of the work.

OPG may also have considered termination of Strabag's contract in order to cure its problems. This would have resulted in a long delay to allow preparation of new contract documents and procurement of a new contractor and afterwards a protracted litigation between the parties. All of which would have delayed the contract completion with concomitant revenue loss and the further unknowns of the re-bid amount and the litigation costs and outcomes.

I was on the DRB on a major tunneling project in Canada and have direct experience where such a course of action was adopted in that the differing site condition issue was not brought before the DRB and the contractor was terminated after stopping work for six months. This led to about a year delay in re-bidding and the new bid coming in at about 1.8 times the original bid with about 60 % of the work completed; plus litigation is ongoing 5 years afterwards.

I have formed the opinion after my extensive review of the circumstances pertaining to this dispute, that OPG's decision to bring it before the DRB was appropriate.

7.0 OPG Performance at the DRB Hearing

Section 11-Dispute Resolution of the DBA provides general guidelines as to the procedures to be adopted by the DRB when conducting a hearing and the preparation of their subsequent recommendations. It is also made clear that the DRB is in charge of the proceedings.

Following the provisions in Section 11, in preparation for the hearing, each party submitted Position Papers on the dispute to the DRB and each other, followed by Rebuttal Papers. All this was done on a mutually agreed timetable.

The hearing was convened by the DRB and conducted from 23/06/08 to 26/06/08. The DRB issued their recommendations dated 30/08/08.

Importantly the construction of the work continued throughout this period with the parties cooperating fully in its prosecution.

7.1 OPG Position and Rebuttal Papers

The principal arguments put forth by OPG are those bulleted in section 6.4 above and were prepared in the main by Hatch Mott McDonald staff (Owner Representative of

OPG) with retained experts and oversight from OPG. The experts were Dr. Dougal F5-6-1 McCreath for rock mechanics and design issues, Dr. Ed Cording for design and constructability issues and Larry Snyder for TBM design related issues; all of whom are very experienced and experts in their fields as evidenced by the reports provided as part of the Position and Rebuttal Papers.

The Position Paper prepared by OPG was clear and comprehensive in its presentation of the issues; the history of development of the design and the construction history; the discussion related to the collaborative effort with Strabag in the preparation of the DBA and the GBR included in the contract. Similarly, the Rebuttal Paper further clarified OPG's position.

7.2 DRB Recommendations

The recommendations provided by the DRB on the five issues listed in section 6.4 are summarized below.

Large Block Failures: The DRB indicated that this condition was adequately forewarned in the GBR and no DSC was warranted.

St. David's Gorge: The DRB found that the Contractor was not entitled to make a claim of DSCs within the 800m width of the Gorge as stipulated in the GBR.

Insufficient Stand-Up Time: The DRB indicated that there was a serious misunderstanding between the parties with respect to the anticipated rock conditions and rock behaviour at the time the contract GBR Version C was being negotiated. Since both parties developed the GBR jointly, any misunderstanding was the shared responsibility of both Parties.

Excessive Overbreak: The large overbreak quantity encountered throughout much of the Queenston Formation mined at that time, had impacted the rate of advance of the TBM and it appeared that the total quantity of overbreak would exceed the GBR quantity by a significant amount. Although the DBA indicated that if DSCs are encountered, the resolution of such claims should be held in abeyance until tunnel excavation was complete, the DRB believed that the consequences of the misunderstandings that had led to both the large overbreak quantities and the related impacts had been so material that some form of resolution was needed.

Whether the GBR was defective or simply misleading, both Parties developed the GBR jointly and therefore both Parties must share in the consequences in resolving the issue.

Inadequate Table of Rock Conditions and Rock Characteristics: The DRB agreed that the Table of Rock Conditions and Rock Characteristics was inadequate to be used for the identification of DSCs and, further, that the inclusion of such terms as the “closest match” and “all other conditions” essentially rendered the concepts of DSCs meaningless and made the GBR defective. In this Design-Build contract, both parties jointly developed the GBR document and both parties should share the shortcomings of the resulting document.

8.0 OPG Decision to Renegotiate a Revised Contract with Strabag

In my opinion there was sufficient weight to Strabag’s positions, particularly regarding the issues relating to ground behaviour and the removal of loose rock, to engender acceptance of the DRB’s recommendations, at least in part. In addition the first three major issues were resolved in OPG’s favour. Taking into account the DRB recommendations and their delineation of the various joint areas of responsibility for the encountered conditions and the subsequent mitigating actions of the parties, in my opinion the decision of OPG to renegotiate a new contract with Strabag was appropriate. The alternatives of arbitration or termination discussed above in section 6.5, would have very likely led to protracted delays and unknown cost expansion in order to complete the project.

9.0 Summary and Conclusions

There were significant challenges to OPG in providing the necessary and sufficient data for the design and construction of the proposed 10.4 km Diversion Tunnel. The natural variability of the alignment was manifested by variable lithology, high horizontal stresses in varying directions, rock strength anisotropy, adverse groundwater chemistry, methane gas potential, rock swelling pressures and long term deformation of the rock mass. OPG conducted a series of phased site investigations from 1983 to 1997. The results of all the investigations conducted for the Conceptual Phase and the Definition

Engineering Phase 1 and Phase 2, were presented or referenced in the twelve volumes of the GDR which was included in the proposal issued to the design-build teams as well as GBR Version A. It is my opinion that the site investigations addressed the appropriate design and construction issues and that the studies undertaken were professionally completed and met or exceeded in some cases, the professional standards for work of similar type and magnitude.

As part of the DBA, Strabag was required to conduct appropriate analyses for the initial support and final lining design; the final lining had a mandatory 90 year design life. Strabag's designer ILF conducted design analyses including Finite Element Modeling, Wedge Analysis, Convergence-Confinement Analysis, and Beam- Spring Model Analysis. Constructability issues were also evaluated in relation to the timing of placement of the initial support. I concluded that the geotechnical and geological data gathered from the various site investigations was sufficient and appropriate for ILF's comprehensive design analyses and further that the analyses were conducted to an appropriate professional standard.

In my opinion the decision to present the disputes to the DRB was appropriate because it was apparent that the ground conditions and support methods were severely impacting the work. I believe that bypassing the DRB process and proceeding to arbitration or terminating Strabag would have resulted in long delays with protracted litigation. All of which would have delayed the contract completion with related revenue loss and the further unknowns of the re-bid amount and the litigation costs and outcomes. I also formed the opinion that OPG's conduct of the hearing was appropriate.

Finally, after review of the subsequent DRB recommendations coupled with my own evaluation of the circumstances, I formed the opinion that the decision to re-negotiate a revised contract with Strabag was appropriate and reasonable given the circumstances of the disputes and the status of the project.

GLOSSARY

Anisotropic: The material properties are different in different directions.

Atterberg Limits: Laboratory tests measuring the moisture content of a clay soil at its consistency (resistance to deformation) limits, termed the liquid and plastic limits.

Closed Formed Solution: A calculation method which assumes that the rock is a F5-6-1 homogenous, isotropic, linearly elastic material.

Core Recovery: The length of actual core recovered during core drilling of a measured interval, referred to as a core run, expressed as a percentage of the core run length, which is typically 3m. It is an indirect measure of core loss which is indicative of general rock quality.

Dynamic Elastic Modulus of Elasticity: The Modulus of Elasticity derived from the measured sonic velocity of sound waves propagated in the rock sample.

Free Swell Test: Test for determining the swelling strain developed in an unconfined rock sample submersed in water as described in the International Society of Rock Mechanics Suggested Test Methods 1979.

Geotechnical Baseline Report (GBR): A report that is part of the contract documents, the purpose of which is to mitigate contingencies in the bid amount and to prevent litigation by promoting dispute resolution in a timely way at the site level. The report incorporates values of the rock's physical parameters as measured during the site investigations, ground characterization and an assessment of rock behavior, which are termed baselines. Generally speaking if the presented baselines are found to be materially different during the work then the resulting Differing Site Condition forms the basis for a contract modification.

Geotechnical Data Report (GDR): The GDR incorporates all of the geotechnical and geotechnical data gathered for the project and/or refers to documents containing such data.

In-Situ Stress Measurement: The existing stresses in the rock mass are measured by hydro-fracture field tests in which water is injected into a discrete section of a borehole isolated by packers, at a pressure sufficient to induce a vertical fracture in the rock. From the data collected, the magnitude and direction of the principal field stresses in the rock mass are estimated.

Isotropic: The material properties are the same in all directions.

Limit Equilibrium Analysis: Analytical method which compares the induced shear stresses on a given set of discontinuities forming a block, to the shear strength of the

discontinuities for the purpose of ascertaining the stability of the block in the tunnel crown.

Lithology: The nature of the rock material such as siltstone, mudstone, shale, sandstone.

Numerical Modeling: Calculation methods for numerical stress analysis using computer models such as the Finite Element Method.

Over-Coring Method of In-Situ Stress Measurement: Another method of measuring in-situ stresses in the rock mass, in which a series of strain gauges attached to a plug are inserted in a core hole and the hole over-cored; during this process the induced strains are measured. From this data, the magnitude and direction of the principal field stresses are calculated.

Permeability Testing: A field test conducted in a borehole in which the rate of water injected into a discrete interval isolated by packers under a given pressure is measured; from this data the rock permeability or hydraulic conductivity is calculated.

Petrographic Analysis: Examination of very thin sections of rock under a polarizing light microscope which enables the identification of the minerals present.

Point Load Strength Index Testing: A measure of rock strength using a testing device consisting of two opposing pointed platens actuated by a hydraulic ram. The load at failure and the distance between the platen points at the start of the test is measured. The Point Load Strength Index is calculated by dividing the load at failure by the square of the initial distance between the points of the platens and expressed in Mpa. It can be normalized to the equivalent distance for a 50 mm diameter core. The test is principally conducted axially or diametrically on core samples but can be used on lumps of rock.

Rock Mass Behaviour: The performance of the rock mass after it is excavated; the term is usually applied to the unsupported condition.

Rock Mass Rating (RMR): An empirical, quantitative measure of a rock mass as initially proposed by Z.T. Bieniawski in 1976 and subsequently revised.

Rock Quality Designation (RQD): The total length of core pieces greater than 10 cm expressed as a percentage of the core run length, generally of 300 cm.

Rock Structure: General term referring to the presence of discontinuities in the rock mass such as bedding planes, joints, faults.

Poisson's Ratio: The ratio of the axial and radial strains as measured during the Uniaxial Compressive Strength Test.

Seismic Reflection Survey: A field test in which an array of geophones are used to record reflected seismic waves emanating from a surface of interest as a result of an energy input on the ground surface.

Stand-up Time: The elapsed time of stable rock conditions is referred to as the stand-up time and is the window for erection of the initial tunnel support.

Modulus of Elasticity: A measure of the rock stiffness expressed as the ratio of the axial stress and the axial strain, as measured in the Uniaxial Compressive Strength test.

Stratigraphy: Describes the spatial relationships between the various rock formations identified by core logging from boreholes spaced along the alignment.

Triaxial Strength Test: A compressive strength test conducted on a specially prepared rock sample placed in a cell which is capable of applying a radial pressure to the sample to simulate in-situ stress. An axial load is applied to the sample through end platens.

Tunnel Crown: Roof of tunnel.

Tunnel Invert: Floor of tunnel.

Tunnel Springline: The location on the tunnel wall which is intersected by a horizontal plane through the center of the tunnel.

Uniaxial Compressive Strength: A compressive strength test of a properly prepared rock core sample conducted by applying an axial load to each end of sample through the platens of the testing machine. The axial load and axial deformation are recorded in real time until failure occurs. The uniaxial compressive strength is calculated by dividing the load at failure by the initial cross sectional area of the sample expressed as Mpa. The axial deformation is used to calculate the Modulus of Elasticity. Radial deformation can also be recorded if the Poisson's Ratio is required.

Westbury Piezometer: Instrument located in a borehole which enables recording of water levels and recovery of water samples at selected elevations within the borehole.

Wireline Core Recovery: A drilling method in which the core is recovered from the borehole by a wireline for each core run without removing the drill string and core barrel.