ROCKMASS BEHAVIOURAL UNCERTAINTY:
IMPLICATIONS FOR HARD ROCK TUNNEL
GEOTECHNICAL BASELINE REPORTS

by

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A thesis submitted to the Department of Geological Sciences and Geological Engineering
In conformity with the requirements for
the degree of Master of Applied Science

Queen’s University
Kingston, Ontario, Canada
(November, 2014)

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Abstract

Geotechnical Baseline Reports (GBRs) have become a prevalent risk sharing mechanism on North American tunneling projects as they are based on the following risk allocation concept: the subsurface ground conditions described within the GBR are the financial responsibility of the Contractor, whereas encountered conditions which exceed those described belong to the Owner. This interpretation is intended to reduce project bid prices due to subsurface ground conditions uncertainty and the geotechnical rationale for a differing site conditions claim. However, recent tunnel project case studies have used the GBR as a risk transfer mechanism by presenting a conservative and/or limited interpretation of the expected ground conditions. In particular, the expected ground conditions are described with a summary of the intact and rockmass properties and empirical rockmass classification systems.

This research has shown that the application of intact rock properties and rockmass classification systems to describe the various rockmasses along the tunnel alignment leads to rockmass behavioural uncertainty. Empirical rockmass classification systems are not able to adequately capture the effects of geologic uncertainty and the collective impact of the individual controls on rockmass behaviour.

A new rock engineering design tool was developed which utilized geologic uncertainty and the capabilities of numerical modelling methods to predict and quantify rockmass behaviours. The 3D Rockmass Behaviour Map reduces subsurface ground conditions uncertainty as the range of possible rockmass behaviours is presented as a function of the three critical geomechanics parameters. Quantifying rockmass behaviours per tunnel domain demonstrates the effects of geologic uncertainty with rockmass behaviour mode switching.

GBRs should include this 3D Rockmass Behaviour Map and quantified rockmass behaviours as these tools reduce uncertainty in the expected ground conditions and provide a greater understanding of the anticipated rockmass behaviours. Rather than using a conservative
GBR which shifts subsurface ground conditions risk to a Contractor, this improved prediction of the expected ground conditions may result in better subsurface risk allocation, reduced construction contingencies, aid excavation means and methods selection, reduced geotechnical basis for a differing site condition claim, and provide greater certainty in the final project price and schedule.
Co-Authorship

This thesis is the product of research conducted by the author, Michelle van der Pouw Kraan. Complete references for conference proceedings resulting from this research are included in the References section and included contributions from Dr. Mark Diederichs, Dr. Jean Hutchinson, and Ms. Jennifer Day. However, the written work is solely that of Michelle van der Pouw Kraan.
Acknowledgements

First off, many thanks are due to the National Science and Energy Research Board, the Center for Excellence in Mining Innovation, the Nuclear Waste Management Organization, and the Geological Sciences and Geological Engineering Department for the financial support to complete this research.

To my supervisor, Dr. Mark Diederichs: the technical discussions, guidance, and the significant number of opportunities to broaden and enhance my understanding of tunneling and rock mechanics through the attendance of courses, workshops, and conferences both nationally and internationally, has taken this master’s further than where I had ever expected; thank you.

Many thanks are due to Dr. Jean Hutchinson for reviewing my thesis, and to Stephanie Fekete and Cortney Palleske for their insightful comments for Chapters 2 and 3, respectively.

A special thank you is also owed to Randall J. Essex, for enhancing my comprehension of Geotechnical Baseline Reports during the World Tunnel Conference in Iguazu, Brazil.

To Ehsan and Felipe, thank you for your friendship, support, and generosity. Your assistance in answering questions (often taking the form of mini ‘lessons’), technical discussions, and always being available for a coffee run were invaluable for this thesis.

To Cara, Chrysothemis, Gabe, Jenn, Ryan, and the past and present members of the Geomechanics Group, your assistance and support to various degrees for this work is gratefully acknowledged. Thank you for being such great friends.

Finally, to my family: Mom and Dad, Mark and Tonja and little Max, and Ashley, thank you for your constant support and encouragement.
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Chapter 1

Introduction

1.1 Project Background

The subsurface nature of tunneling projects, where the ground conditions may remain significantly unknown up until excavation, means that these projects are amongst the most challenging engineering structures to analyze, design, and construct. However, the expected ground conditions are one of the most important aspects to any tunnel project, as the tunnel design, construction, cost, schedule, and occurrence of differing site conditions claims depend on this interpretation. In this regard, Geotechnical Baseline Reports (GBRs) have become prevalent on North American tunneling projects as the interpretation of the expected ground conditions is intended to advise the Contractor of anticipated construction difficulties, aid the Contractor in their selection of an excavation methodology, and reduce Contractor construction contingencies. Furthermore, the description of the expected ground conditions is intended to share the subsurface ground conditions risk amongst the project parties, and decreases the geotechnical rationale for differing site conditions claims (Essex, 2007). Subsurface ground conditions risk sharing is based on the following concept: the encountered ground conditions less adverse or consistent with the GBR are the financial responsibility of the Contractor, whereas the encountered conditions which exceed those described conditions in the GBR are the financial responsibility of the Owner (Essex, 2007).

However, due to geologic uncertainty and the increasingly litigious nature of the North American tunneling industry, project Owners have used the GBR as a risk transfer mechanism by presenting a conservative and/or limited interpretation of the expected ground conditions. Contractors may or may not recognize their increased risk, which can result in overbidding with a large construction contingency (which the Owner pays for regardless of whether the adverse
subsurface ground conditions are encountered), or underbidding and potentially resulting in an increased number of site conditions claims. Differing site conditions claims can unpredictably increase the project cost and delay the project schedule.

1.2 Project Objective and Scope
This research was conducted to determine the challenges currently facing GBRs in the tunneling industry and to investigate how the expected ground conditions description in GBRs could potentially be improved.

The first part of this study was to determine how geologic uncertainty and a project’s contractual setting (i.e. the project delivery and financing method) impact the interpretation of the expected ground conditions in the GBR. A literature review of the challenges introduced by geologic uncertainty, current rock engineering design tools and contracting methods, and a GBR Industry Survey formed the basis for a demonstration of the implications of a conservative expected ground conditions interpretation.

The second part was to determine how the interpretation of the expected ground conditions in a GBR could be improved by considering the influence of the three major controlling parameters on rockmass behaviour: the intact rock strength, rockmass structure, and the in-situ stress condition. As rockmass behaviour in jointed rockmasses is significantly controlled by the discontinuities, a parametric study of the assumptions and limitations of modelling discontinuities in a continuum numerical modelling tool was performed. A new methodology is proposed where geologic uncertainty is utilized to predict and quantify the probable rockmass behaviours along a tunnel alignment.

1.3 Thesis Outline
This thesis has been prepared in accordance with the requirements outlined by the School of Graduate Studies at Queen’s University, in Kingston, Ontario. This thesis consists of six chapters, as outlined below.
Chapter Two consists of a literature review of the challenges of geologic uncertainty on tunnel projects and the rock engineering design tools developed as design aids to manage this uncertainty. Rockmass classification systems use and misuse, characterization systems, and numerical modelling tools are discussed.

Chapter Three presents an overview of the project delivery and financing methods relevant to GBRs. Additionally, the Differing Site Conditions clause and the Geotechnical Baseline Report (GBR) are defined and their applications explained.

Chapter Four discusses the results from the Geotechnical Baseline Report Industry Survey, and demonstrates how the use of rockmass properties and rockmass classification systems in GBRs promotes rockmass behavioural uncertainty. The latter half of the material presented in this chapter is based on a paper submitted to the 2013 American Rock Mechanics Association conference.

The implications of discontinuity uncertainty and numerical modelling of discontinuities in a continuum software program on rockmass behaviour predictions are discussed in Chapter 5. Additionally, a new rock engineering design tool which utilizes geologic uncertainty to predict and quantify the expected rockmass behaviours along a tunnel alignment is presented. The material presented in this chapter is based on three separate submitted conference papers to: the Discrete Fracture Network Engineering Conference (in 2014 in collaboration with Ms. Jennifer Day), the World Tunnel Conference (2014), and the American Rock Mechanics Association conference (2014).

Chapter Six discusses the key findings from this work and provides recommendations for future work.

1.4 Summary of Key Findings

The key findings from this work are summarized in the subsections below.
1.4.1 The Geotechnical Baseline Report Industry Survey

The GBR Industry Survey produced several interesting trends regarding the contents and perceived effectiveness of GBRs in the tunneling industry. Prevalent means to define the various rockmasses along a tunnel alignment included the intact rock and rockmass properties and rockmass classification systems, whereas the in-situ stress was the least defined ground conditions parameter. A relatively significant number of GBRs reported one to two rockmass behaviours. While based on a limited number of responses, approximately half of the respondents reported that the GBR was ineffective or somewhat ineffective at mitigating differing site conditions claims, and that excavation means and methods recommendations should not be present in a GBR.

The prevalent use of intact rock properties and rockmass classification systems to define the various rockmasses along a tunnel alignment leads to rockmass behavioural uncertainty. The low perceived effectiveness of GBRs in mitigating a differing site conditions claim may be the culmination of this and the other aforementioned data trends.

1.4.2 Rockmass Behavioural Uncertainty

GBRs which use rockmass properties and rockmass classification systems to define the expected ground conditions promote rockmass behavioural uncertainty along a tunnel alignment. This was demonstrated with the empirical rockmass classification system, the Q System (Barton et al., 1974), where five physically different rockmasses with the same rock quality rating were evaluated with respect to two different excavation methodologies. Five mechanically different rockmass behaviours were produced: squeezing, block failure, spalling, buckling, and flowing ground. The excavation methodology affected the failure magnitude and the measured effectiveness of the recommended rock support. In the majority of cases the recommended rock support was inadequate for the rockmass behaviour.
The GBR is typically the only interpretive document of the expected subsurface ground conditions in which to explain the basis for design and the anticipated construction issues. GBRs which use rockmass classification systems alone to describe the expected ground conditions promote uncertainty in the expected rockmass behaviours as several interpretations are possible for the same quality rating. During the tender stage this can result in Contractor over or under bidding due to conservatism or neglect of rockmass behaviours, respectively. Additionally, during construction this may provide justification for a differing site conditions claim, which unpredictably increases the project cost and schedule.

1.4.3 Numerical Modelling of Discontinuities

Rockmass behaviour in jointed rockmasses is significantly controlled by the discontinuities; however discontinuity uncertainty and numerical modelling assumptions affect the prediction of rockmass behaviours. A parametric study of joint shear strength failure criterions and joint stiffness in the finite element program Phase2 (RocScience, 2014) demonstrated that at low stress the Mohr-Coulomb cohesion term had the greatest effect controlling effect on the rockmass behaviour. Varying cohesion produced three different rockmass behaviours, whereas the Barton-Bandis shear strength criterion produced one rockmass behaviour. Under high stress conditions all the Mohr-Coulomb and Barton-Bandis models had the same rockmass behaviour as the joint shear strength was exceeded in all the models. The joint stiffness controlled the amount of deformation that occurred. While the joint shear strength criterion should represent the actual joint surface condition, discontinuity uncertainty may make selection difficult. Numerical modelling of rockmass behaviours at low stress should be performed with at least two joint shear strength criterions to evaluate the potential differences in rockmass behaviour.

1.4.4 Quantifying Rockmass Behavioural Uncertainty

A new rock engineering design tool was developed which utilizes geologic uncertainty to predict and quantify rockmass behaviours along a tunnel alignment. A parametric study of the three
critical geomechanics parameters which significantly control rockmass behaviour: the intact strength, the rockmass structure, and the in-situ stress condition, generated the 3D Rockmass Behaviour Map. This map reduces subsurface ground conditions uncertainty as the entire range of possible rockmass behaviours as a function of these parameters is revealed.

Rockmass behavioural uncertainty within tunnel domains can be quantified by applying probabilistic ranges of the intact strength, rockmass structure, and in-situ stress condition along a tunnel alignment to this map. This was demonstrated with a case study. The effects of geologic uncertainty are highlighted with rockmass behaviour mode switching occurring within each tunnel domain.

GBRs should include this 3D Rockmass Behaviour Map and the quantified rockmass behaviours as these tools reduce uncertainty in the expected ground conditions and provide a greater understanding of the anticipated rockmass behaviours. Rather than using a conservative GBR which shifts subsurface allocation risk to a Contractor, utilizing geologic uncertainty to generate a definition of the expected ground conditions may result in improved allocation of subsurface geotechnical risk, decreased construction contingencies, improved excavation means and methods selection, reduced geotechnical basis for a Type 1 differing site condition claim, and possibly greater certainty in the final project price and schedule.
Chapter 2

Geologic Uncertainty and Rock Engineering Design Tools

2.1 Geologic Uncertainty

The subsurface nature of tunneling projects, where the ground conditions may remain significantly unknown up until excavation, means that these projects are among the most challenging engineering structures to investigate, analyze, design, and construct. As a site investigation can never fully determine the entire range of subsurface conditions to be encountered, interpretations and assumptions must be made throughout the project process based on an incomplete geologic database in order for the project to progress. In this regard, numerous quantitative and qualitative rock engineering design tools have been developed to manage geologic uncertainty and provide analysis and design guidance. Through assessment of certain key rockmass characteristics, engineers are able to obtain either a rock support design or predict rockmass behaviours. Additionally, numerical modelling methods adapted for rock mechanics have arisen as a powerful tool to analyze complex rock engineering design problems.

2.1.1 Site Investigation Data Collection Limitations

A site investigation is one of the initial steps on a tunneling project designed to develop an understanding of the ground conditions. Development of a geologic database begins with desk studies where air photos, regional geologic maps, previous geological, geotechnical, and hydrogeological reports, construction experience, and other documents (e.g. the World Stress Map (Heidbach et al., 2008)) are reviewed and analyzed for relevant geological information. Site visits build and refine this database with surface mapping and subsurface exploration techniques to obtain further information on the geologic units, structural features, groundwater conditions, and the in-situ stress conditions.
While the purpose of a site investigation is to obtain all the geologic information required for engineering analysis, design, and construction purposes, several constraints make obtaining a representative database on which to base subsequent project work difficult (Hoek and Palmeiri, 1998). The geologic complexity along a tunnel alignment, including the spatial variability of geologic materials, structures, groundwater and in-situ stress conditions, means that these features can never be fully investigated. Furthermore, the geometrical tunnel properties (e.g. length, depth) with respect to financial and scheduling restraints often necessitates that site exploration efforts are allocated according to ease of site accessibility and/or are concentrated on anticipated problematic areas (e.g. fault zones) (Hoek and Palmeiri, 1998).

Additional complications and uncertainty arise from sampling bias and field and laboratory testing procedures. For example, structures oriented relatively parallel to the borehole orientation are underrepresented in core logs and during sample selection for strength testing (Carter, 1992). Rock sample selection for strength testing requires intact pieces, and is often biased towards competent pieces with no veining or fractures as typically these are considered as planes of weakness and preferential breakage locations. In-situ stress, a key component in determining rockmass behaviour, may be one of the most difficult parameters to determine, as it requires expensive, complex equipment (e.g. USBM deformation gauges), is relatively time consuming, and the interpretation of test outcomes can be challenging or give ambiguous results. Field testing of the groundwater conditions (e.g. packer testing, falling head test, piezometers) is similarly challenging (Carter, 1992, Stille and Palmstrom, 2008).

While internationally recognized testing procedures are available for field and laboratory testing of physical and mechanical intact and rockmass parameters (e.g. the American Society for Testing and Materials (ASTM), the International Society for Rock Mechanics (ISRM)), the actual sample handling and preparation, testing procedures, measurement process, and human error may result in variant, inaccurate or unrepresentative results. Additionally, project testing budgets may
limit the number of tests able to be performed. This limits the reliance and confidence of the collected data, and adds uncertainty to the geologic database (e.g. Carter, 1992, Fookes, 1997, Hoek and Palmeiri, 1998).

2.1.2 Data Interpretation and Geologic Model Development

Geologic models are required on tunnel projects as it is not possible to precisely determine all aspects of the subsurface ground conditions, and therefore, a representation of the subsurface conditions is required (Bowden, 2004). Data limitations, including inconsistent and/or sparse data, the inherent variability of the geologic materials and forces acting on a rockmass, and the bias and uncertainty introduced during the testing stages makes the construction of this geologic model difficult (Bowden, 2004).

Interpreting and synthesizing all the collected material, structural, in-situ stress, and groundwater site investigation data into a geological model to subsequently analyze rockmass behaviours is challenging. Difficulties include the inherent spatial variability of geologic materials (making determination of representative properties difficult), and interpreting structural data in between surface outcrops and widely spaced boreholes. While guidelines are available to assist Engineers in projecting surface units and structural features to depth (e.g. Hoek, 1982), depending on the knowledge and complexity of the geology, extrapolating information obtained from surface and often only a few widely spaced boreholes to locations where little or no information is available can result in multiple plausible geologic interpretations of the continuity and locations of the geologic units and structures. Supplemental measures may be required where testing (if completed) yields uncertain or unreliable results, or knowledge of other geologic features may be required to interpret and determine geologic conditions where data may not be easily obtainable. For example, in-situ stress testing can be integrated (or replaced) with an analysis of the gravitational, topographic, tectonic, and/or the residual stress components, and
understanding of the structural network and the rockmass permeability can assist in the prediction of groundwater inflows (Stille and Palmstrom, 2008).

2.1.3 Rockmass Behaviour Analysis and Rock Support Design
Geologic uncertainty also challenges the rockmass behaviour analysis and rock support design process. Tools to manage parameter uncertainty include conservatism, quantifying uncertainty with statistical procedures, and the observational approach (Christian, 2004). Conservative input parameters may be selected to ensure that a design is sufficiently robust to withstand loads greater than expected, however this results in costly design solutions (Christian, 2004). Statistical methods allow for input parameter uncertainty to be propagated through the design process and its effect on the selected rock support design to be quantified. Additionally, reliability analyses can calculate a probability of failure based on a design performance criterion (Christian, 2004). A prevalent alternative is the observational approach, where several alternative design solutions are developed for the range of anticipated rockmass behaviours and are modified as necessary during construction (Christian, 2004).

There are many kinematic, analytical, and numerical modelling tools available to analyze the rockmass behaviour and determine the appropriate rock support requirements, each with its own advantages and limitations. However, geologic uncertainty may cause difficulties in identifying the anticipated rockmass behaviours, and thus, also in selecting the appropriate modelling tool. Furthermore, many kinematic, analytical, and numerical modelling tools cannot incorporate geologic uncertainty in that they are essentially deterministic, requiring input as single values (Carter, 1992). While parametric studies and Monte Carlo analyses can aid the design Engineer in this regard, these studies can be relatively time consuming and/or they do not quantitatively define the effect of geologic uncertainty (Carter, 1992).

Geologic uncertainty also affects the measurement of the acceptability and performance of the rock support design. Available tools to define satisfactory system behaviour include factors
of safety, statistical analyses, and/or engineering judgment (Carter, 1992, Hoek, 2007). However, uncertainty in the model may affect the measured confidence of the overall system behaviour (Bowden, 2004).

Consequently, while several methods and tools are available throughout the site investigation, analysis, and design process, often the subjective nature of prior experience, engineering judgment, and conservatism will play a large role in the geologic model development, rockmass behaviour analysis, and the rock support design process (Bowden, 2004).

In this regard, several different rock engineering design tools have been developed to assist Engineers to determine the rockmass quality, behaviours, and the tunnel excavation and support requirements. One set of tools are empirical rockmass classification systems which simplify this complex design problem and provide guidance during the site investigation and design stages by quantifying certain rockmass characteristics considered critical to tunnel stability. As these systems are based on case studies, they provide a measure of confidence in the prediction of actual conditions and the recommended rock support design (Carter, 1992).

2.2 Empirical Classification Systems

Empirical classification systems are design tools which through examination of certain rockmass characteristics and external forces, either ranks the rockmass quality (e.g. based on the intact rock strength, the degree and/or condition of jointing, and/or the external forces acting on the rockmass) according to a defined scale, and/or correlates the rockmass quality to a rock support design (Bieniawski, 1989).

Since their introduction, rockmass classification systems have become very popular due to their relative simplicity (e.g. in comparison to analytical and numerical modelling methods), practicality, and wide applicability (Riedmuller and Schubert, 1999). Throughout a project, classification systems can be applied (Hoek and Palmeiri, 1998, Hoek, 2007, Stille and Palmstrom, 2003):
• as part of the project feasibility studies (especially when limited information is available);
• to obtain an indication of the rockmass quality;
• as a checklist to verify that the relevant data has been collected during the site investigation;
• as a means to obtain intact and rockmass properties through empirical correlations (rather than with expensive in-situ or laboratory testing);
• to quantitatively compare different design options;
• to predict rockmass behaviours;
• to obtain rock support requirements and quantities for design and contracting purposes;
• to determine or verify support at the tunnel face during construction.

Four of the most pertinent classification systems for determining the rock support requirements for civil tunnel applications include the Rock Load Classification by Terzaghi (1946), the Rock Mass Rating by Bieniawski (1989), and the Rock Tunnelling Quality Index, by Barton, Lien, and Lunde (1974). The Rock Quality Designation, by Deere et al. (1967), which provides an estimate of the degree of jointing, is also discussed as it is incorporated into the Rock Mass Rating and the Rock Tunneling Quality Index. The following discussion will describe each of these systems as they were published.

2.2.1 The Rock Load Classification
In 1946 the first rockmass classification system was published by Terzaghi; it used qualitative rockmass descriptions and the tunnel width (B) and depth (Ht) to estimate the rock loads for steel set design (Terzaghi, 1946, Hoek, 2007). A reproduction of the original classification system is shown in Table 2-1. The rockmass descriptions include geologic and structural descriptions of the rockmass, the external forces driving deformation (e.g. gravity, in-situ stress, groundwater), the rockmass behaviour, and practical notes for excavation (Terzaghi, 1946, Hoek, 2007). Modifications occurred in 1970 and 1982, with the incorporation of the Rock Quality Designation
(by Deere et al., 1970), and reduction of the original rock support loads, to account for groundwater (by Rose, 1982), respectively.
Table 2-1: The Rock Load Classification (from Bieniawski, 1989, after Terzaghi, 1946).

<table>
<thead>
<tr>
<th>Rock Condition</th>
<th>Rock Load Hp (ft)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard and Intact</td>
<td>Zero</td>
<td>Light lining required only if spalling or popping occurs</td>
</tr>
<tr>
<td>Hard stratified or schistose</td>
<td>0 – 0.5B</td>
<td>Light support, mainly for protection against spalls. Load may change erratically from point to point</td>
</tr>
<tr>
<td>Massive, moderately jointed</td>
<td>0 – 0.25B</td>
<td></td>
</tr>
<tr>
<td>Moderately blocky and seamy</td>
<td>0.25B – 0.35(B + Ht)</td>
<td>No side pressure</td>
</tr>
<tr>
<td>Very blocky and seamy</td>
<td>(0.35 – 1.10)(B + Ht)</td>
<td>Little or no side pressure</td>
</tr>
<tr>
<td>Completely crushed</td>
<td>1.10(B + Ht)</td>
<td>Considerable side pressure. Softening effects of seepage toward bottom of tunnel require either continuous support for lower ends of ribs or circular ribs</td>
</tr>
<tr>
<td>Squeezing rock, moderate depth</td>
<td>(1.10 – 2.10)(B + Ht)</td>
<td>Heavy side pressure, invert struts required. Circular ribs are recommended.</td>
</tr>
<tr>
<td>Squeezing rock, great depth</td>
<td>(2.10 – 4.50)(B + Ht)</td>
<td></td>
</tr>
<tr>
<td>Swelling rock</td>
<td>Up to 250ft, irrespective of the value of (B + Ht)</td>
<td>Circular ribs are required. In extreme cases, use yielding support.</td>
</tr>
</tbody>
</table>

Definitions:

- **Intact rock** contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a spalling condition. Hard, intact rock may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.

- **Stratified rock** consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.

- **Moderately jointed rock** contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.

- **Blocky and seamy rock** consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.

- **Crushed but chemically intact rock** has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

- **Squeezing rock** slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

- **Swelling rock** advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.
2.2.2 The Rock Quality Designation Index

The Rock Quality Designation (RQD) index was developed by Deere et al. in 1967 to obtain a quantitative estimate of the rockmass quality from drill core (Hoek, 2007, Hoek et al., 1995). The length of pieces of intact, competent core greater than 100 mm in length are summed together and divided by the overall run length to obtain a rock quality percent rating which correlates to a ranking scale varying from very poor, to excellent rock (see Figure 2-1) (Bieniawski, 1989).

ASTM International Standard recommends using a core diameter of NQ (47.5 mm) to NX (54.7 mm) (2014), drilled using double-tube core barrels (ASTM International, 2014, Bieniawski, 1989). RQD has since been applied to surface outcrop mapping (Palmstrom, 2005).

Measurements should be recorded in various orientations to obtain a representative RQD range.
Figure 2-1: Methodology to calculate RQD on drill core and the associated quality ratings (from Palmstrom, 2005, modified after Deere, 1989).

The RQD value can indicate whether the intact rock material or the discontinuities control the rockmass behaviour. For a rockmass with a RQD of less than 50%, consisting of only a few intact pieces greater than 100 mm, behaviour in response to gravity and in-situ stress is controlled by the discontinuities. Rockmasses with an RQD greater than 95% exhibit discontinuity controlled failure under low stress, whereas under high stress the discontinuities may have little impact on the rockmass behaviour (Hutchinson and Diederichs, 1996).
2.2.3 Rock Mass Rating

The Rock Mass Rating (RMR), also known as the Geomechanics Classification system, correlates a quantitative rockmass quality rating based on the evaluation of several rockmass characteristics to a rock support design. RMR was first published in 1973 based on 49 case studies (Bieniawski, 1989). As additional studies became available, the system was revised and updated several times. The 1989 version is based on 351 case studies, of which 227 are in sedimentary rock, 68 are in igneous rock, and 56 are in metamorphic rock. Of these, 147 case studies are tunnels, 135 are mines, 54 are chambers, 8 are foundations, 5 are slopes, and 2 are shafts (Bieniawski, 1989). The RMR system was recently modified in 2014 by adding 2,298 RMR 1989 tunnel face case studies and modifying the input parameters (Celada et al., 2014). However, the work for this thesis used RMR 1989.

The RMR system is based on quantitative quality estimates of 5 parameters: the rock uniaxial compressive strength (from unconfined compressive strength (UCS) tests or point load tests (PLT)), RQD, joint spacing, joint condition, and the groundwater condition. These ratings are summed to obtain a rockmass quality rating from 8 to 100 (Bieniawski, 1989). The rating can be adjusted by incorporating the orientation of the critical discontinuity set with respect to the tunnel orientation (Bieniawski, 1989). The final rating falls into one of five equally divided rockmass classes (i.e. 20 points each), ranging from very poor (0 – 20) to very good (81 – 100) rock. Each rockmass class has cohesion and friction values for the rockmass, stand-up times, excavation rates and number of headings, and final rock support requirements, in the form of rockbolts, shotcrete, and/or steel sets (Bieniawski, 1989). It is important to note that the rock support recommendations are based on a 10m span drill and blast horseshoe shaped tunnel with a maximum vertical stress of 25 MPa (Bieniawski, 1989). Figure 2-2 and Figure 2-3 show the parameter ratings and rock support recommendations tables.
Figure 2-2: the RMR rock quality ratings (from Hoek, 2007, after Bieniawski, 1989).
<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Rock bolts (20 mm diameter, fully grouted)</th>
<th>Shotcrete</th>
<th>Steel sets</th>
</tr>
</thead>
<tbody>
<tr>
<td>I - Very good rock</td>
<td>Full face, 3 m advance.</td>
<td>Generally no support required except spot bolting.</td>
<td>None.</td>
<td></td>
</tr>
<tr>
<td>II - Good rock</td>
<td>Full face, 1-1.5 m advance. Complete support 20 m from face.</td>
<td>Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.</td>
<td>50 mm in crown where required.</td>
<td></td>
</tr>
<tr>
<td>III - Fair rock</td>
<td>Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.</td>
<td>Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.</td>
<td>50-100 mm in crown and 30 mm in sides.</td>
<td></td>
</tr>
<tr>
<td>IV - Poor rock</td>
<td>Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.</td>
<td>Systematic bolts 4.5 m long, spaced 1-1.5 m in crown and walls with wire mesh.</td>
<td>100-150 mm in crown and 100 mm in sides.</td>
<td></td>
</tr>
<tr>
<td>V – Very poor rock</td>
<td>Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.</td>
<td>Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.</td>
<td>150-200 mm in crown, 150 mm in sides, and 50 mm on face.</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 2-3:** RMR rock support recommendations per rockmass class (from Hoek, 2007, after Bieniawski, 1989).

### 2.2.4 The Rock Tunneling Quality Index

The Rock Tunneling Quality Index, or Q system, was published by Barton, Lien, and Lunde at the Norwegian Geotechnical Institute (NGI) in 1974 and is similar to RMR, in that a quantitative estimate of the rockmass quality leads to a rock support recommendation. The Q system is based on approximately 200 case studies, the majority of which were drill and blast tunnels in Scandinavia. A range of geologies were included in these case studies, including 13 igneous types, 24 metamorphic types, and 9 sedimentary types (Barton et al., 1974). Updates to the Q system occurred in 1994 and 2013 with 1,050 and approximately 900 case studies added, respectively (NGI, 2013). The 1994 updates included adding SRF parameters and modifying the support recommendations to include shotcrete with steel fiber reinforcement, and in 2013 the majority of the support categories were modified to recommend stiffer support, including rebar.
quantities within shotcrete (NGI, 2013). As work for this thesis preceded the release of the 2013 version, this thesis used the 1994 version of the Q system.

The Q system combines the parameters RQD, $J_n$ (number of joint sets), $J_r$ (joint roughness), $J_a$ (joint alteration), $J_w$ (joint water reduction factor), and the stress reduction factor (SRF). Representing the ratio of intact strength to the in-situ stress condition, the SRF subcategories are divided according to whether the intact material or the in-situ stress controls the rockmass behaviour, with subcategories and associated ratings for different types of “weakness zones”, “competent rock, rock stress problems”, “squeezing rock”, and “swelling rock” conditions (Hutchinson and Diederichs, 1996, Barton and Grimstad, 1994). Together these parameters are combined into the following equation (Barton et al., 1974):

$$Q = \frac{\text{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}}$$  \hspace{1cm} (2-1)

The quotients RQD/$J_n$, $J_r/J_a$, and $J_w$/SRF are considered to be measures of the block size, joint shear strength, and the active stresses, respectively, and, taken together, are intended to predict the rockmass behaviour (Barton et al., 1974, Barton and Bieniawski, 2008). The final rating falls within one of nine rockmass quality categories, ranging from 0.001 (exceptionally poor) to 1000 (exceptionally good) (Barton and Grimstad, 1994). Parameter ratings are shown in Figure 2-4 and Figure 2-5.
### A. Rock quality designation (RQD)

<table>
<thead>
<tr>
<th>Quality</th>
<th>RQD Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very poor</td>
<td>0 - 25%</td>
</tr>
<tr>
<td>Poor</td>
<td>25 - 50</td>
</tr>
<tr>
<td>Fair</td>
<td>50 - 75</td>
</tr>
<tr>
<td>Good</td>
<td>75 - 90</td>
</tr>
<tr>
<td>Excellent</td>
<td>90 - 100</td>
</tr>
</tbody>
</table>

**Notes:**
(i) Where RQD is reported or measured as < 10 (including 0), a nominal value of 10 is used to evaluate Q.
(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

### B. Classification with ratings for the Joint set number (Jn)

<table>
<thead>
<tr>
<th>Joint Set Number (Jn)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive, no or few</td>
<td>One joint set</td>
</tr>
<tr>
<td>joints</td>
<td>One joint set plus random</td>
</tr>
<tr>
<td></td>
<td>Two joint sets</td>
</tr>
<tr>
<td></td>
<td>Two joint sets plus random</td>
</tr>
<tr>
<td></td>
<td>Three joint sets</td>
</tr>
<tr>
<td></td>
<td>Three joint sets plus random</td>
</tr>
<tr>
<td></td>
<td>Four or more joint sets, heavily jointed, &quot;sugar-cube&quot;, etc.</td>
</tr>
<tr>
<td></td>
<td>Crushed rock, earth-like</td>
</tr>
</tbody>
</table>

**Notes:**
(i) For tunnel intersections, use (3.0 x Jn).
(ii) For portals, use (2.0 x Jn).

### C. Classification with ratings for the Joint roughness number (Jr)

#### a) Rock-wall contact, b) rock-wall contact before 10 cm shear

<table>
<thead>
<tr>
<th>Joint Roughness Number (Jr)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>JR = 4</td>
<td>Zone containing clay minerals thick enough to prevent rock-wall contact</td>
</tr>
<tr>
<td>JR = 10</td>
<td>Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact</td>
</tr>
</tbody>
</table>

**Notes:**
(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
(ii) JR = 0.5 can be used for planar, slickensided joints having linestains, provided all lineasions are orientated for minimum strength.

### D. Classification with ratings for the Joint alteration number (Ja)

<table>
<thead>
<tr>
<th>Joint Alteration Number (Ja)</th>
<th>Condition</th>
<th>Wall Contact</th>
</tr>
</thead>
<tbody>
<tr>
<td>JA = 0.75</td>
<td>Healed or welded joints: filling of quartz, epidote, etc.</td>
<td>JA = 0.75</td>
</tr>
<tr>
<td>JA = 1</td>
<td>Fresh joint walls: no coating or filling, except from staining (rust)</td>
<td>1</td>
</tr>
<tr>
<td>JA = 2</td>
<td>Slightly altered joint walls: non-softening mineral coatings, clay-free particles, etc.</td>
<td>2</td>
</tr>
<tr>
<td>JA = 3</td>
<td>Friction materials: sand, silt calcite, etc. (non-softening)</td>
<td>3</td>
</tr>
<tr>
<td>JA = 4</td>
<td>Cohesive materials: clay, chlorite talc, etc. (softening)</td>
<td>4</td>
</tr>
</tbody>
</table>

### E. Classification with ratings for the Joint water reduction factor (Jw)

<table>
<thead>
<tr>
<th>Joint Water Reduction Factor (Jw)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>JW = 1</td>
<td>Dry excavations or minor inflow, i.e., &lt; 5 l/min locally</td>
</tr>
<tr>
<td>JW = 0.66</td>
<td>Medium inflow or pressure, occasional outwash of joint fillings</td>
</tr>
<tr>
<td>JW = 0.5</td>
<td>Large inflow or high pressure in competent rock with unfilled joints</td>
</tr>
<tr>
<td>JW = 0.3</td>
<td>Large inflow or high pressure, considerable outwash of joint fillings</td>
</tr>
<tr>
<td>JW = 0.2</td>
<td>Exceptionally high inflow or water pressure at blasting, decay, with time</td>
</tr>
<tr>
<td>JW = 0.1</td>
<td>Exceptionally high inflow or water pressure continuing without noticeable decay</td>
</tr>
</tbody>
</table>

**Note:**
(i) The last four factors are crude estimates. Increase Jw if drainage measures are installed.
(ii) Special problems caused by ice formation are not considered.

---

Figure 2-4: Q system ratings for RQD, Jn, Jr, Ja, and Jw (from Palmstrom, 2009, after Barton and Grimstad, 1994).
Figure 2-5: Q system ratings for SRF (from Palmstrom, 2009, after Barton and Grimstad, 1994).

This quality rating is then combined with the quotient of the facility’s span or height divided by the Excavation Support Ratio (ESR) (capturing the facility’s purpose: temporary or permanent, relative importance, and safety level) to obtain the rock support (Barton and Grimstad, 1994). Parameter ratings and rock support recommendations are shown in Figure 2-6.

The Q system has been expanded several times, most notably with the Modified Rock Quality Index, Q’, and QTBM. Q’ is used where the groundwater and in-situ stress conditions are not considered to detrimentally affect the tunnel stability, and are set to one. Q’ is often used when the objective of using the classification system is to obtain rockmass properties for analytical or numerical design (Hutchinson and Diederichs, 1996). QTBM is used to predict TBM advance rates. To do so, QTBM incorporates the rock mass strength, the average cutter load, the cutter life index, the rockmass quartz content, and the induced biaxial stress occurring on the tunnel face into the original Q equation (Barton, 1999).
Figure 2-6: The Q System support recommendations (from Palmstrom and Broch, 2006, modified from Grimstad and Barton, 1994).

2.2.5 Critical Empirical Classification Systems Comments

While rockmass classification systems are popular and have been applied successfully on thousands of tunnel projects worldwide, a large amount of literature has been published in approximately the last 20 years on general and specific insufficiencies of rockmass classification systems (e.g. Milne et al., 1998, Riedmuller and Schubert, 1999, Stille and Palmstrom, 2003, Palmstrom 2005, Palmstrom and Broch, 2006, Schubert, 2013). This section will summarize general inadequacies, with particular attention to RQD, RMR and Q.

The greatest limitations for RQD are that it is a one dimensional measurement of rockmass structure and it contains a rather arbitrary cut-off value to distinguish between competent and incompetent rock. To obtain a 3D representation of the rockmass structure,
multiple boreholes or scanlines oriented in several directions are required. Palmstrom (2005) discussed that the variance in the measurement orientation (especially in anisotropic rocks) and rockmasses with average joint spacings of 90 mm or 110 mm can both result in 0% or 100% RQD. RQD is inapplicable to the full range of jointing, as it has been shown to be insensitive to discontinuity spacings greater than 0.3 m (Hutchinson and Diederichs, 1996, Palmstrom and Broch, 2006). When applied to drill core, fracture origins are subject to interpretation as drill breaks can be mistaken for natural fractures, thereby decreasing the RQD value and perceived rock competence (Carter, 1992). These limitations are then incorporated into the RMR and Q systems.

While Bieniawski stated that rockmass classification systems (e.g. RMR, Q) “were not intended to replace analytical considerations, field observations, or engineering judgment, they were simply to be design aids” (1989), due to their relative simplicity, classification systems have often replaced these more rigorous and experience based analysis and design methods, and/or applied to projects for which they were not designed for (Riedmuller and Schubert, 1999). In focusing only on certain rockmass characteristics the user may overlook other rockmass features potentially critical to design (Riedmuller and Schubert, 1999). Classification systems decrease appreciation of special rockmass characteristics (e.g. anisotropy, time dependency), how the external forces influence the rockmass behaviour, and how the rockmass behaviour interacts with the tunnel geometry and rock support (Riedmuller and Schubert, 1999, Cai, 2011).

The case studies on which the RMR and Q rock support recommendations are based, likely contained support for contractual or safety applications (Palmstrom and Stille, 2007). As the current rock support recommendations are based on the average rock support requirements as determined from these case studies (Palmstrom and Stille, 2007), these rock support recommendations must be conservative (Riedmuller and Schubert, 1999). Classification system users may not know how appropriate or accurate the rock support recommendations are to the
rockmass behaviour (Stille and Palmstrom, 2003), as significant discrepancies may exist between
the weakest and stiffest rock support case studies for a given rock quality rating (Palmstrom and
Stille, 2007). It has also been asked how classification systems can accurately characterize
complex rockmass conditions and behaviours such as brittle spalling, squeezing, or swelling,
through quantitative ratings of a limited number of rockmass characteristics (Stille and Palmstrom
2003, Palmstrom and Broch, 2006).

Engineers using classification systems as part of a tunnel design may run into legal
implications as complex ground conditions coupled with the increasing litigious nature of tunnel
engineering entails that the sophistication of a tunnel design must match the expected ground
conditions, for which empirical systems may be too simplistic (Schubert, 2013). Design
Engineers must demonstrate that they have adhered to the standard of care in their design
approach (Samuels and Sanders, 2007).

2.3 Rockmass Characterization and Behaviour Prediction Methods
In response to the use and misuse of empirical rockmass classification methods, rockmass
characterization and qualitative classification methods predicting rockmass behaviours have been
developed. Rather than ranking specific rockmass features and external forces to obtain a quality
rating and rock support design, the ground conditions (including the intact material, structural
characteristics, in-situ stress, groundwater, and project related features) are characterized or
classified together to determine the rockmass behaviour, as illustrated in Figure 2-7 (Stille and
Palmstrom, 2003). Methods have been proposed by Hoek, Kaiser, and Bawden (1995), Schubert
and Goricki (2004), Stille and Palmstrom (2008), and Marinos (2012). Rockmass behaviour
definitions often accompany the characterization and/or classification systems.
Figure 2-7: Rockmass behaviour analysis approach as proposed by Stille and Palmstrom (2003).

The Geologic Strength Index, by Hoek (1994), while not a directly a classification system, represents a further development by applying a qualitative rockmass assessment to improve rockmass behaviour analyses with analytical or numerical modelling tools. The following discussion is ordered chronologically in terms of characterization or classification system development.

2.3.1 The Geologic Strength Index

The Geologic Strength Index (GSI) was initially developed by Hoek (1994) to correlate field observations of a jointed rockmass to the Hoek-Brown failure criterion, to obtain an estimate of the rockmass strength (Hoek et al., 1995). In this respect, the GSI system is significantly different from the previously described empirical classification systems and the subsequent behaviour determination systems as it does not lead to a rock support design or a behaviour prediction.
Initially, the GSI rock quality ratings varied from 10 – 85, representing very poor, to intact rock, respectively. After several modifications, including an expansion for weak heterogeneous rockmasses (Marinos and Hoek, 2001), the most commonly used GSI chart varies from 10 – 100 (Marinos et al., 2005) (Figure 2-8). In 2013 the GSI chart was updated to include correlations to RQD and the RMR joint condition parameter, and the ‘intact or massive’ and ‘laminated/sheared’ structure categories were removed (Hoek et al., 2013).
Figure 2-8: The Geologic Strength Index for jointed rocks chart (from Marinos et al., 2005).
For isotropic rockmasses, a visual assessment of the geology, rockmass structure, and discontinuity surface condition leads to a qualitative description of the rockmass blockiness, discontinuity condition, and subsequently a rating value (Marinos et al., 2005). The rockmass strength is calculated through empirically derived equations linking the GSI rating to the material constants \( m_i, m_b, s, a \), and the laboratory obtained intact rock strength, \( \sigma_c \) (Hoek et al., 1995). This rockmass strength is applied to analytical and numerical analyses of rockmass behaviours (Marinos et al., 2005).

### 2.3.2 Hard Rock Rockmass Behaviours

The book “Support of Underground Excavations in Hard Rock”, by Hoek, Kaiser and Bawden (1995) describes rock engineering design tools and rock support options based on experience in Canadian underground hard rock mines. Figure 2-9 demonstrates the effects of the rockmass structure and the in-situ stress on the rockmass behaviour, and describes the rock support requirements. In the left column, under low stress, as the structural density increases, the rockmass behaviour transforms from a stable condition in a massive rockmass, to where the blocks are free to slide or rotate in a heavily jointed rockmass. Under high stress, rockmass failure changes from brittle spalling in a massive rockmass, to near ductile behaviour in a heavily jointed rockmass.
<table>
<thead>
<tr>
<th></th>
<th>Low stress levels</th>
<th>High stress levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive rock</td>
<td>Massive rock subjected to low in situ stress levels. No support or ‘safety bolts’ or dowels and mesh.</td>
<td>Massive rock subjected to high in situ stress levels. Pattern rockbolts or dowels with mesh or shotcrete to inhibit fracturing and to keep broken rock in place.</td>
</tr>
<tr>
<td>Jointed rock</td>
<td>Massive rock with relatively few discontinuities subjected to low in situ stress conditions. ‘Spot’ bolts located to prevent failure of individual blocks and wedges. Bolts must be tensioned.</td>
<td>Massive rock with relatively few discontinuities subjected to high in situ stress conditions. Heavy bolts or dowels, inclined to cross rock structure, with mesh or steel fibre reinforced shotcrete on roof and sidewalls.</td>
</tr>
<tr>
<td>Heavily jointed rock</td>
<td>Heavily jointed rock subjected to low in situ stress conditions. Light pattern bolts with mesh and/or shotcrete will control ravelling of near surface rock pieces.</td>
<td>Heavily jointed rock subjected to high in situ stress conditions. Heavy rockbolt or dowel pattern with steel fibre reinforced shotcrete. In extreme cases, steel sets with sliding joints may be required. Invert struts or concrete floor slabs may be required to control floor heave.</td>
</tr>
</tbody>
</table>

Figure 2-9: Effect of the in-situ stress and structure on hard rock rockmass behaviours (from Hoek et al., 2005).
This work was later modified by Martin et al. (2001), by adding an intermediate in-situ stress column, and the rock support requirements were removed. Equations relating the maximum stress to the intact rock strength, and GSI ranges as structural boundaries, were added to act as behaviour limits for each row and column.

2.3.3 Probabilistic Assessment of Rockmass Behaviour

In 2004 Schubert and Goricki published a quantitative rockmass behaviour prediction tool where the probability of rockmass behaviours occurring along a tunnel alignment are quantified using the Monte Carlo analysis method (Schubert and Goricki, 2004). This method follows the Austrian Society for Geomechanics geotechnical design procedure (Austrian Society for Geomechanics, 2010).

To begin, based on the site investigation results and other sources of geologic data, the tunnel alignment is divided into relatively geologically homogenous domains. Within each domain the uncertainty of the physical and mechanical intact, rockmass, and structural parameters are quantified with an appropriate fitted or assumed probability distribution curve. These quantified parameters are entered into a Monte Carlo analysis to identify and quantify one or more Rock Mass Types (RMTs) within each domain (see Figure 2-10) (Schubert and Goricki, 2004).
Figure 2-10: Probabilistic determination to determine the Rock Mass Types (from Schubert and Goricki, 2004).

Once the RMTs are established, the rockmass Behaviour Types (BTs) are determined by applying factors affecting the rockmass behaviour including the tunnel geometry, the in-situ stress, and the groundwater conditions (Schubert and Goricki, 2004). These factors are quantified where possible, and input into analytical or numerical models. Several analytical methods which calculate the measurable effects of rockmass behaviours (e.g. tunnel convergence, depth and volume of overbreak, rock strain energy build-up) are suggested, as analytical models can be directly input into a Monte Carlo analysis (Schubert and Goricki, 2004). Once the Monte Carlo
analysis is complete, the probabilistic output is analyzed to identify the BTs, their probability per tunnel domain, and if required, along the entire tunnel alignment (Figure 2-11).

![Figure 2-11: Analytical models used in the Monte Carlo analysis generate a probability distribution of the measurable effects of rockmass behaviours. This is used to identify and determine the probability of rockmass Behaviour Types (from Goricki et al., 2002).](image)

Propagating geologic uncertainty throughout the rockmass behaviour analysis reveals how geologic uncertainty affects the RMTs, BTs, and also the project scheduling and costing analyses (Schubert and Goricki, 2004). The nature of the Monte Carlo analysis method also allows for calculations to be rapidly repeated should a design change occur (e.g. if the tunnel diameter changes), allowing for the user to quickly determine the effects on the rockmass BTs (Schubert and Goricki, 2004).

### 2.3.4 Initial and Long-Term Behaviours based on the Rockmass Composition

The qualitative Initial and Long-Term Behaviours based on the Rockmass Composition chart developed by Stille and Palmstrom (2008), correlates rockmass composition to several possible initial and long-term rockmass behaviours. This work appears to be the culmination of three previously published papers on rockmass classification and characterization methods (Stille and Palmstrom, 2003, Palmstrom and Broch, 2006, Palmstrom and Stille, 2007).
This chart demonstrates the relationship between rockmass composition, the in-situ stress, groundwater conditions, and fourteen different initial and long-term rockmass behaviours (Figure 2-12). Rockmass compositions are categorized according to whether the rockmass behaviour is controlled by the intact material, the rockmass structure, or by a combination of both, with the subcategories “massive rocks”, “jointed rock or blocky materials”, “particulate materials”, and “special materials” (highly anisotropic or noticeably weak rock materials) (Stille and Palmstrom, 2008). Initial and long-term behaviours are shown to either remain constant (e.g. structurally controlled behaviours), or to vary with time (e.g. initial plastic deformation transitioning to squeezing) (Stille and Palmstrom, 2008). Rockmass behaviour variability and/or combinations of behaviours are emphasized per rockmass composition, with two different proposed behaviours commonly listed. The influence of the construction methodology and the excavation geometry on the rockmass behaviour is discussed in the paper, however, this is not considered in the chart.
Figure 2-12: Rockmass composition and behaviour chart: anticipated initial and long term behaviour as a function of the rockmass composition, in-situ stress, and groundwater condition (from Stille and Palmström, 2008).
2.3.5 The Tunnel Behaviour Chart and the Ground Characterization, Behaviour, and Support for Tunnels system

The Tunnel Behaviour Chart and the Ground Characterization, Behaviour, and Support for Tunnels system are a rockmass classification and characterization system respectively, developed by V. Marinos, that is based on experience gained from the construction of 62 rural and alpine tunnels as part of the new Egnatia Odos Highway across northern Greece (2012). With drill and blast top heading and bench construction, these 12 m diameter tunnels were constructed through a wide range of geologic conditions, varying from undisturbed sedimentary packages to heavily tectonized and metamorphosed rock units, with intact rock strengths and overburden varying from 5 – 100 MPa, and 30 – 500 m, respectively (Marinos, 2012). This project produced the TIAS database, which consists of tens of thousands of project records (from site investigation through to construction), which was analyzed for correlations between design parameters, the rockmass characteristics and behaviours, and the temporary support works (Marinos et al., 2010). This work resulted in the Tunnel Behaviour Chart (with supporting rockmass behaviour definitions) and the Ground Characterization, Behaviour, and Support for Tunnels system (Marinos, 2012).

The qualitative Tunnel Behaviour Chart (TBC) classifies rockmass behaviour based on the intact rock strength, the in-situ stress, and the rockmass structure (based on the GSI chart) (Figure 2-13). In the TBC, the influence of each parameter, and parameter relationships, on the rockmass behaviour are demonstrated. In many cases, one or more rockmass behaviours are possible for a given combination of the aforementioned parameters. The TBC does not consider brittle rock behaviours as these were not encountered during tunnel construction (Marinos, 2012). The Notes following the TBC are very important and should be considered in application of the chart.
Figure 2-13: The Tunnel Behaviour Chart (from Marinos, 2012).
The Ground Characterization, Behaviour, and Support for Tunnels (GCBST) system is a step-by-step analysis and design procedure where the user is guided from characterizing the rockmass and the ground conditions to quantifying the intact and rockmass analysis and design parameters (Figure 2-14 and Figure 2-15). This then leads to determining the rockmass behaviour and the corresponding ‘tunnel support philosophy’, based on rock support suggestions previously outlined in his paper (Marinos, 2012).
Figure 2-14: The Ground Characterization, Behaviour and Support for Tunnels methodology (1/2) (from Marinos, 2012).
Together, the TBC and the GCBST promote users to anticipate rockmass behaviours. In predicting rockmass behaviours, these systems emphasize the importance of understanding how the ground type, structure, and external forces contribute to the rockmass behaviour, and subsequently, based on the rockmass behaviour, which rockmass parameters are important for determining a proper excavation and rock support methodology (Marinos, 2012).

2.4 Numerical Modelling Tools
As computers have become more powerful and rock engineering problems have become more complex numerical modelling has become a prevailing analysis and design tool. There are several types of numerical modelling methods, including but not limited to: boundary element, finite element, finite difference, and discrete element, and also hybrids of these, such as the finite element method/discrete element method. These tools are used to analyze the system behaviour consisting of the rock mass, the boundary conditions, and the rock support. This section will briefly describe the implementation of the finite element method, as this method was used for all the research presented in this thesis.

2.4.1 The Finite Element Method
The finite element method (FEM) is a continuum method which approximates a solution to the partial differential equations of equilibrium governing physical behaviour by applying a series of algebraic equations. The problem domain is simplified by dividing it into a finite number of similarly shaped subdomains, called elements, which are connected by nodes to form a mesh (Wittke, 1990).
Figure 2-16: FEM discretization of a domain into a number of finite elements (reproduced and modified after Hume, 2011).

The major steps for the FEM are as follows (Puzrin, 2012):

1. Discretize a problem domain into a number of finite elements (e) connected with a number of nodes (n). The displacements at the nodes \( \{u_n\} \) are the main unknowns.

2. Select a shape function \([N^e(x_i)]\) which calculates the element displacement \( u_i^e(x_i) \) as a function of the nodal displacements:

\[
\{u_n\} = u_i^e(x_i) = [N^e(x_i)]\{u_n\}
\] (2-1)

3. Compatibility equations and constitutive relationships are used to calculate the strains \( \varepsilon_{ij}^e(x_i) \) and the stresses \( \sigma_{ij}^e(x_i) \) in regard to the nodal displacements, respectively:

\[
\varepsilon_{ij}^e(x_i) = [B^e(x_i)]\{u_n\}
\] (2-2)

\[
\sigma_{ij}^e(x_i) = [D][B^e]\{u_n\}
\] (2-3)

4. Relate the strains to the stresses to derive the element stiffness matrix \([K^e]\), by integrating over the element volume \( V^e \):

\[
[K^e] = \int_{V^e}[B^e]^T[D][B^e]dV
\] (2-4)

5. Assemble the forces acting at the nodes \( \{q_n^e\} \) in each element in terms of \( \{u_n\} \):

\[
\{q_n^e\} = [K^e]\{u_n\} + \{f_{bn}^e\} + \{f_{sn}^e\}
\] (2-5)

Where \( \{f_{bn}^e\} \) are the body forces and \( \{f_{sn}^e\} \) are the surface forces within each element.
6. Sum the nodal forces \( \{ F_n \} = \sum_e q_n^e \) to assemble the element stiffness matrices into the global stiffness matrix \([K]\), to calculate the unknown nodal displacements:

\[
\{ F_n \} = [K]\{ u_n \} + \{ f_{bn} \} + \{ f_{sn} \} \tag{2-6}
\]

Where \( \{ f_{bn} \} \) and \( \{ f_{sn} \} \) sum all the element body and surface forces, respectively.

7. The nodal displacements are used to calculate the strains and stresses for each element.

Since the 1960’s, the FEM has become a very popular numerical modelling tool due to its ability to model problems with complex boundary conditions, geometry, and heterogeneous materials (i.e. the DIANE materials: discontinuous, inhomogeneous, anisotropic, and non-elastic) (Jing, 2003). Material heterogeneity is accommodated through the assignment of distinct elastic material properties to specific discretized areas within the problem domain. Additionally, different types of elements can be incorporated into the FEM matrix formulation, such as the Goodman joint element for rockmass structures (Goodman et al., 1968), and the Timoshenko beam element for concrete or shotcrete liners (Owen et al., 1980).
Chapter 3

Contracts, Differing Site Conditions, and Geotechnical Baseline Reports

3.1 Introduction

In the traditional design-bid-build method of tunnel construction, tunnel project development proceeds in a linear fashion with construction following design. These projects are relatively simple for an Owner to manage, in terms of contracts, party responsibilities, and allocation of risk. As projects become located in more geologically complex ground conditions, this project delivery approach can result in very long project delivery schedules and significant cost overruns, as, for example, should an adverse subsurface ground condition occur, the design Engineer and the Contractor may hold opposing views as to the cause of the event and the appropriate mitigation measures (Lamb and Henk, 2010). New project delivery and financing methods were introduced to the tunneling industry in attempts to improve project delivery schedules and decrease the chance of cost escalation due to unexpected ground conditions, by allocating risk to those best able to manage it. One of these new methods is the design-build approach, where the design Engineer works with the Contractor. In merging the final design and construction, the risk of the final design, subsurface ground conditions, and/or the project financing can be transferred to the design-build team. In addition to modification in the project delivery and financing method, risk sharing tools, such as the Differing Site Conditions clause and the Geotechnical Baseline Report, became prevalent measures to share the risk associated with the subsurface conditions and reduce the overall project costs between the Owner and the Contractor or design-build team. As projects become increasingly complex, contracting methods are continually evolving as the contracting parties attempt to determine the best combination of the final design and construction activities, project financing, and allocation of risk.
3.2 Project Delivery Methods

Project delivery methods are categorized according to the final design and construction sequencing, the role of the Engineer, the Contractor’s scope of work, the financing method, and the allocation of risk. The two main project delivery methods are design-bid-build, and design-build and its variations. The alliance contracting method will also be discussed.

Selection of the project delivery and financing method is the responsibility of the Owner. To determine which methods are the most compatible with a proposed project, Owners may hire an Engineer to determine the scope of work and project complexity, and perform feasibility and cost-analysis studies. Other significant influencing factors for this decision include the Owner’s familiarity with these methods, schedule and financial constraints, degree of risk aversion, in-house capabilities, and the desired level of involvement (Gordon, 1994). External factors, such as market conditions and contractor capabilities also contribute (Gordon, 1994). A breakdown of the project delivery and financing methods is shown in Figure 3-1, which will be discussed further in this section. For the discussion that follows, it should be noted that many variations of these contracting methods are possible, and they are often customized to a project.

![Figure 3-1: Project scope, delivery, and financing methods (modified after Gordon, 1994).](image)

### 3.2.1 Separate Design and Construction

Projects which follow a sequential design and construction schedule are called ‘design-bid-build’. On these projects the design Engineer works with the Owner to develop and design the project.
Once the design is complete, a Contractor is hired to construct the works. The following sections will discuss the roles of the Engineer, Owner, and Contractor on these projects.

3.2.1.1 Role of the Engineer
On a design-bid-build project, the design Engineer works for, and with, the Owner. Throughout the project duration, the Owner may decide to contract with one engineering firm, or with multiple engineering firms for different portions or stages of the work.

Design Engineers have several responsibilities, including developing, managing, and interpreting the results from the site investigation program, producing the final design, evaluating construction methodologies, writing the Geotechnical Data and Baseline Reports (Sections 3.5.1 and 3.5.3), and generating contract documents (e.g. reports, drawings, specifications). Engineers are responsible for ensuring that the final design has been developed to the Engineer’s ‘standard of care’ (Samuels and Sanders, 2007). Design Engineers may also become involved in the procurement process through the development of a request for proposals (RFP), and once Contractor tender documents have been received, evaluating the tender documents to select a Contractor (Hatem, 2010).

Tunnels are a challenging structure to design as the temporary works (which are the responsibility of the Contractor, as discussed in Section 3.2.1.3), may influence the permanent (or final design) works (which are the responsibility of the design Engineer). For example, in a concrete lined tunnel, if the temporary liner is to be integrated into the final liner, the axial forces and bending moments in the temporary liner are required for the final liner design calculations (Hoek, 2003). The design Engineer may specify a temporary liner design which corresponds with the final liner design in the contract documents (Carpenter, 2012). Alternatively, if the Contractor is required to design the temporary liner, they may request a copy of Engineer’s final liner design calculations (Illingworth, 2000).
During construction, Owners often hire an Engineer responsible for ensuring that the project is constructed to the design and protecting the Owner’s interests with a construction quality assurance program. This role is typically known as ‘the Owner’s Engineer’, which can be performed by the original design Engineer, or another engineering firm. Quality assurance services often include documenting construction progress, reviewing Contractor submittals, performing construction inspections, and assisting the Contractor in planning the construction of the works. If any of the Contractor’s construction activities (e.g. construction submittals, constructed work) is found deviant from the contract documents, the Engineer works to resolve the problem, with actions varying from formal identification notices to designing remediation works (Lamb and Henk, 2010). In working for the Owner, the Engineer also has the difficult position of administering the Differing Site Conditions clause (as discussed in Section 3.4) in accordance with the Geotechnical Baseline Report (Section 3.5.3).

An Owner may also hire a construction manager to manage the day-to-day construction activities and provide cost services. The construction manager can be the design Engineer, or another engineering or contractor firm (International Tunneling Association (ITA), 1996). The construction manager performs minimal or no physical work on the construction site, rather, they coordinate between the Owner, the Owner’s Engineer, and the Contractor, to administer the contract, coordinate the site activities, and administer payments (Samuels and Sanders, 2007, ITA, 1996). Owners can also hire a construction manager earlier in the project development process, to assist with issuing tender documents, evaluating tenders, and negotiating Contractor contracts (ITA, 1996).

A Contractor may also hire an Engineer to act as part of their construction team in the design and construction of the work. This may be required as part of the contract specifications (Levy, 2000).
3.2.1.2 Role of the Owner
As previously mentioned, the Owner is responsible for selecting the project delivery and financing methods. These decisions determine and affect the entire project delivery process.

The design-bid-build format entails that an Owner has separate contracts with a design Engineer and a Contractor, as shown in Figure 3-2. The Owner is responsible for contract management and payment provisions. The Owner is also responsible for answering to the public and dealing with third party impacts (Peyton and Harrison, 2010).

![Figure 3-2: Simplified contract structure of a design-bid-build project.](image)

The Owner is responsible for several high risk items on design-bid-build projects. This includes the site investigation (Hatem, 2010) and the final design, because the Owner is able to tailor the project to their specific objectives (ITA, 1996). For example, the Owner controls how much money is spent on the site investigation, and during the design process the Owner may request design changes (which typically occur at low cost as the Engineer is contracted to the Owner) (ITA, 1996). The Owner warrants to the Contractor that the design is buildable and free from defects (Peyton and Harrison, 2010). Additionally, the Owner is responsible for differing site conditions (Section 3.4).

3.2.1.3 Role of the Contractor
The Contractor is responsible for the tender (Eddleston et al., 1995), the construction means and methods, and constructing the project to the contract specifications (Peyton and Harrison, 2010).
Tender documents are developed based on the information provided in the RFP. These documents outline the Contractor’s proposed schedule, construction methodology, and bid price to construct the works (Hatem, 2010).

Construction means and methods include selecting equipment, organizing manpower, design and construction of temporary works, and the overall construction logistics required to build the project (Biggart et al., 2010). During construction, Contractors are also required to have a quality control program to ensure that the works adhere to the project specifications, typically through a quality control testing and documentation program (Lamb and Henk, 2010).

While the Contractor is responsible for constructing the project to the contract documents and contract specifications, this does not include the guarantee that the project will perform as designed (Peyton and Harrison, 2010).

3.2.2 Combined Design and Construction
Projects following a combined final design and construction process are known as design-build. This section will focus on the changed role of the design Engineer in relation to the Owner and the Contractor responsibilities.

3.2.2.1 Role of the Engineer
On design-build projects, more than one engineering firm may act on the project at the same time, representing or working with the Owner or Contractor, respectively. In general, design Engineers working with the Owner conceptualize the project and monitor the final design and construction development, whereas Engineers working with the Contractor produce the final design and construct the project as a design-build team.

3.2.2.1.1 With the Owner
The Engineer working for the Owner on a design-build project has similar, yet reduced responsibilities in comparison to the design Engineer on a design-bid-build project. Similar
responsibilities include the project scope determination and feasibility study, developing RFP documents, and evaluating Contractor tenders. As the responsibility for the final design is transferred to the design-build team, the extent to which the site investigation is completed by the Owner (e.g. preliminary, or complete) is project specific, however, Essex (2007) recommends that a complete site investigation be completed, with the same scope as that for design-bid-build projects. Design Engineers are responsible for developing a conceptual design and/or a performance specification for an RFP (Peyton and Harrison, 2010). Transferring additional responsibilities to a design-build team requires that the RFP documents should make it clear to all parties the project requirements and how risk has been allocated (Peyton and Harrison, 2010). Selection of a design-build team takes on greater importance than in design-bid-build due to the additional project responsibilities of this team (Lamb and Henk, 2010).

In comparison to design-bid-build, the Owner plays a similar, yet reduced role throughout the project development. As only a conceptual design and/or performance specification is required to be developed, Owners are not significantly involved in the final design process. With the design Engineer working with the Contractor, the Owner no longer has direct contact with the Engineer developing the final design, and is thus restricted as to their degree of influence over the design. If changes are required, Owners must go through a formal, expensive change order process (ITA, 1996). Design-build projects work best if the scope of the work and the allocation of risk have been clearly defined and the Owner is willing to take a ‘hands-off’ approach to the project development (Lamb and Henk, 2010). Similar to design-bid-build, Owners retain the financial responsibility of unexpected adverse site conditions.

Once a design-build team has been selected, the Owner may retain an Engineer to act as an independent reviewer to ensure that the Owner’s objectives and interests are being met and protected (Hatem, 2010). This may involve reviewing design-build team design submittals,
providing technical support, construction monitoring, and dispute and/or claim assistance (Peyton and Harrison, 2010).

3.2.2.1.2 With the Contractor
As previously mentioned, a design-build team is composed of a design Engineer and a Contractor. This team can be either Engineer or Contractor led, or it can operate as a joint venture. Often design-build teams are Contractor led, as Contractors are experienced in assuming risk, have greater financial net worth and bonding capacity, are skilled in estimating construction costs, and are practiced in working with unions (Peyton and Harrison, 2010).

During the procurement process, design-build teams develop a tender based on the site investigation data, conceptual design and/or the performance specification as supplied by the Owner (Hatem, 2010). This tender is composed of a detailed design, construction methodology, schedule, and price to construct the works (Essex, 2007). During the procurement process, design-build teams may request the Owner to complete additional site investigation or laboratory testing to obtain data pertinent to completing their tender design, or to include additional required site investigation works as part of their tender (Brierley and Dill, 2010).

Once the project has been awarded, design Engineers develop the final design which conforms to both the Owner’s project requirements and the Contractor’s preferred construction methodology or build history. The design-build framework allows for the final design to become integrated with the construction methodology and the expected ground behaviours (Hatem, 2010). In contrast to design-bid-build where the Owner retains responsibility for the final design, as the Engineer now works with the Contractor, the responsibility for this item is frequently allocated to the design-build team.

3.2.3 Variants of the Combined Design and Construction Project Delivery Method
Variations to the design-build approach include turnkey or engineer-procure-construct (EPC), and public-private-partnership (PPP) or design-build-operate (DBO) project delivery methods. On
these projects the design-build team is assigned additional financial responsibility (Section 3.3) and additional project scope after construction is complete. In turnkey or EPC (different names for the same contracting method), the design-build team is also responsible for project commissioning, in that the project is considered complete once the Owner can simply ‘turn the key’ to begin operating the project (FIDIC, 2011). On PPP or DBO (also different names for the same contracting method), the Contractor is additionally responsible for operating and maintaining the project for a contractually defined number of years after construction is complete (Gordon, 1994).

These project delivery methods are typically reserved for complex projects where it is difficult to define the full project scope, the Owner does not wish to be involved in the day to day operations, or the Owner does not have the funding to finance the project (Section 3.3.2) (FIDIC, 1999). The entire project works can be placed into one contract package with a performance specification. This entails that while the project is built to the Owner’s objectives, the Owner has no control over any aspect of the project development (ITA, 1996). As such, often the risk of adverse subsurface conditions is also assigned to the design-build team (FIDIC, 2011). Transferring or allocating the majority of the project risk to the design-build team will result in a higher project price as the design-build team will include a larger contingency to account for this increased risk. However, in return, the project delivery schedule is significantly compressed, as the design-build team has complete control over the project development, and Owners are often guaranteed the certainty of the final project price and the completion date (FIDIC, 2011). Owners may include financial incentives in the contract documents if the project is delivered early, or penalties if delivered late (Gordon, 1994).
3.2.4 Comparing Design-Bid-Build to Design-Build

In addition to the differing role of the design Engineer on design-bid-build and design-build projects, the contrasting delivery process and responsibilities of the contracting parties entails significant variations in the overall project cost, schedule, and risk allocation.

Design-bid-build projects are often favored by Owners seeking to retain a significant amount of control throughout the project delivery process. During design, Owners are able to tailor the project to their specific objectives, and during construction, the Owner’s Engineer acts to protect the Owner’s interests. This latter point stems from the fact that on design-bid-build projects, Contractors are often viewed as attempting to ‘cut corners’ to save money, especially if the project was won in a competitive bidding environment, where they were the low bidder (Lamb and Henk, 2010). This situation, amongst others, promotes the traditional adversarial attitudes between design Engineers and Contractors (Lamb and Henk, 2010). Conversely, design-build may be considered advantageous for an Owner, as a design-build team operates as one source of contact and responsibility (Lamb and Henk, 2010).

Engineering costs are reduced on design-build projects as the final design is developed according to the Contractor’s preferred construction methodology, and the design Engineer does not need to monitor the Contractor’s construction activities in the same fashion as on design-bid-build projects (Hatem, 2010, Lamb and Henk, 2010). In contrast, on design-bid-build, the design Engineer has to evaluate multiple construction methodologies in preparing the final design, and oversee the Contractor’s construction activities (Hatem, 2010).

Design-bid-build projects have a longer overall schedule as the project progresses in a linear fashion, with construction following design, and thus are easier to manage than design-build. On design-build projects, initial construction works which do not require significant design input, such as the construction of a tunnel staging area, can be completed to help gain schedule (Lamb and Henk, 2010). Often design proceeds nearly concurrently with construction, with construction drawings often issued ‘just in time’ (Lamb and Henk, 2010). However, ‘just-
in-time' drawing issues can create complications, and actually add to the schedule. For example, if a particularly complex project has been divided into several design segments for ease of management, conflicts can occur if the segment teams do not effectively communicate near segment boundaries, with one segment team possible affecting the construction progress of another. While design-build projects are more difficult to manage, significant time and thus cost savings can be achieved if managed effectively.

While design-build projects are typically initially more expensive than design-bid-build projects as additional risk and responsibilities are allotted to the design-build team, there is less chance of cost growth throughout the project (Lamb and Henk, 2010). As design-build teams are responsible for both the final design and construction, in theory claims should only occur for differing site conditions (Section 3.4), and not for design-bid-build Owner responsibilities or changes, such as design inaccuracies, changed quantities, or delays (Halligan et al., 1987). In contrast, on design-bid-build in competitive bidding environments, Contractors may bid low or even underbid the work in attempts to win the work, and then intend to make up the deficit with claims during construction (Lamb and Henk, 2010).

### 3.2.5 Alliance Contracting

Alliance contracting is an alternative project delivery method in which the project risk is minimized through collective project team management, rather than being allocated to a particular project party (Henneveld, 2006).

In alliance contracting, a project Owner selects a design Engineer and a Contractor based on competence and the perceived ability to complete the proposed project, not price (Henneveld, 2006). Once the Owner has selected these team members, commercial aspects regarding the expected profit, risks, and costs are discussed, which typically results in a lump-sum contract with incentives (Section 3.3) (Henneveld, 2006, Samuels and Sanders, 2007). A senior manager from
each party composes the project board which governs the project development (Henneveld, 2006).

This integrated team (called an ‘alliance’) approach allows for the expertise and resources of each team member to be collectively applied throughout the entire project process. This is intended to reduce the project schedule, improve the identification and mitigation of risks, and aids to eliminate the traditional adversarial attitudes between project parties (e.g. as on design-bid-build projects) (Henneveld, 2006, Samuels and Sanders, 2007). The alliance carries all the risk associated with the project together. Risk management is allocated to the party best able to manage it; if an event occurs, the alliance is focused on mitigating the event impact and remedial measures, rather than on blame assignment (e.g. similar to a mining environment) (Henneveld, 2006).

This contracting method is growing in popularity in Australia, with Henneveld reporting nearly $2 billion dollars of projects completed with alliance contracting (2006).

3.3 Project Financing
There are no formal guidelines to correlate a project delivery method with a financing approach (ITA, 1996). Rather, the selection of a financing method is influenced by the certainty of the site conditions, the project complexity, and the degree to which the project scope is defined (ITA, 1996). Project financing is divided according to which party provides financing for the construction: either by the Owner, or the Contractor (Gordon, 1994).

3.3.1 Owner Financed
Owners finance projects typically through public funds, loans from financial institutions, or other money lenders (ITA, 1996, FIDIC, 2011). Payments to the Contractor are typically scheduled to occur during construction (often targeting specific project milestones, and/or on a unit rate basis, see below), and/or after the completion of the works (ITA, 1996). Owner financed payment methods include (combinations are possible):
• Lump sum or fixed price: the contracting parties agree to a price for construction of the project, including an amount for overhead and profit (Samuels and Sanders, 2007). This method, while encouraging efficient work, is the most risky for a Contractor, and as such, may include a large contingency in their tender which the Owner pays for, irrespective of whether this contingency is used (ITA, 1996). Unless measures have been included in the contract for project changes or claims which may occur during construction, this method should only be used where the site conditions are well known and the works are well defined (Samuels and Sanders, 2007, ITA, 1996).

• Cost-reimbursable or cost-plus: where a Contractor is paid for all the costs required to construct the project, plus an amount to cover overhead and profit (ITA, 1996). This method should only be used where the subsurface conditions and the project requirements are well known, as the price to the Owner is unknown until the project is complete (ITA, 1996). Similar to this are time and materials contracts, in which a Contractor is paid for all the time and materials required to construct the works.

• Target price or guaranteed maximum price: similar to cost-reimbursable, where the Contractor is paid for all the costs to construct a project, up to a predefined maximum price (Gordon, 1994). The Contractor pays for all costs exceeding this price, and shares in cost-savings if the final cost is below this price (Gordon, 1994).

• Unit price, bill of quantities, or admeasurement: this method is often used when the extent of the construction works cannot be accurately determined prior to contract formation, or as a means to reduce the financial risk for both parties (ITA, 1996, Samuels and Sanders, 2007). In the contract documents, the construction works are divided into measurable quantities, each with an associated unit cost (e.g. per meter of tunnel excavation, quantities of materials used), on which payment to the Contractor is based (Gordon, 1994). This allows for a more accurate representation for the total cost of the
works (e.g. in comparison to lump sum). Furthermore, unit-price provisions can be put in place as a precautionary measure should a part of the works exceed what is specified in the contract (Samuels and Sanders, 2007).

3.3.2 Contractor Financed
Owners which select a Contractor financing method transfer the short and/or long term financial risk to a Contractor (FIDIC, 1999). These projects require the Contractor to develop a financial tender in which the construction costs (e.g. turnkey/EPC), long term (maintenance) costs, and/or assessment of expected project revenues as a means to pay for the construction of the works are evaluated (e.g. PPP or DBO) (Gordon, 1994, ITA, 1996, Hobden, 1998). In turnkey/EPC, short term construction financing is provided by the Contractor and the Owners pays the Contractor upon project completion (Gordon, 1994). On PPP or DBO, the Owner (e.g. the government) does not pay for the project; rather the Contractor is responsible for both the short and long term financing. Contractors construct the project with their own internal funds (or with funds from other money lenders, such as banks (FIDIC, 2011)), and are given a long term lease to operate the project once it is complete (Samuels and Sanders, 2007). This long term lease period may include a maintenance contract where the Contractor is responsible for maintaining the project. The lease period also allows for Contractors to obtain their return on investment through project use (e.g. revenues generated from a hydroelectric tunnel, or users pay tolls to drive through a new highway tunnel) (Samuels and Sanders, 2007). Figure 3-3 summarizes the levels of risk for Owners and Contractors per contract delivery method.
3.4 The Differing Site Conditions Clause

3.4.1 Purpose and History
The Differing Site Conditions (DSC) clause is a contractual risk sharing mechanism to reduce the Contractor’s financial risk of encountering an unexpected adverse subsurface ground conditions during construction (Gould, 1995). Owners are financially responsible for unanticipated adverse subsurface conditions which qualify under the clause (Gould, 1995). In response, bid prices are reduced as Contractors do not include large contingencies to account for this risk (Essex, 2007). Owners are allocated this risk as the Owner ‘owns the ground’, meaning that the Owner physically owns the ground in which the project is to be constructed, and more importantly, stands to benefit from the project once completed (Heuer, 1997). The DSC clause has become very prevalent on construction projects, with its use mandated in U.S. Government contracts, and it has been included in standard contract documents developed by several professional organizations (e.g. the American Institute of Architects, the American Society of Civil Engineers,

Prior to the 1970’s, Owners had two risk allocation options for unexpected subsurface conditions: either to the Contractor, or to assume this risk themselves (Halligan et al., 1987). As unexpected, adverse subsurface conditions are one of the costliest uncertain price increases on any tunneling project, risk averse Owners on traditional design-bid-build projects assigned this risk to the Contractor to avoid responsibility and have a firmer certainty of the final project cost (Halligan et al., 1987). To do so, Owners either did not consider constructability requirements in the site investigation program (Essex, 2007), disclaimed the validity and interpretation (if provided) of all geologic data provided in the contract, and/or excluded the DSC clause (Halligan et al., 1987). As the tender duration is insufficient to complete a reasonable site investigation to obtain their own geologic data, Contractors were left to develop their own interpretation of the provided data (Samuels and Sanders, 2007). Then, depending on the market conditions and the need to be competitive, Contractors would either a) provide for very little contingency for subsurface conditions in their bid, with the intent to claim later (Halligan et al., 1987), or b) include large bid contingencies to cover this risk (Gould, 1995). With regards to a), regardless of the contract language, Contractors believed that the risk of unexpected site conditions either belonged to the Owner, or was a shared risk (Halligan et al., 1987). If there was no clause in the contract documents to manage the process for encountering an unexpected subsurface condition, Contractors could frame a claim under another clause, and often succeeded in obtaining compensation (Halligan et al., 1987). With regards to b), by including large contingencies in their bids, Owners paid for this risk, whether it was encountered or not (Essex, 2007).

In the 1970’s and 1980’s the U.S. National Committee on Tunneling Technology published a series of recommendations to improve contracting practices (Gould, 1995). These risk management recommendations included: disclosing all geologic data in the contract without
the use of disclaimers (e.g. the Geotechnical Data Report, Section 3.5.1); provide an interpretive subsurface ground conditions report (e.g. the Geotechnical Baseline Report, Section 3.5.3); incorporating a changed conditions clause (the Differing Site Conditions clause); and establishing a dispute review board in the contract (Gould, 1995).

3.4.2 The DSC Clause

As of April 1984, the Differing Site Conditions clause appears in the US Federal Code of Regulations (48 CFR 52.236-2) as follows:

(a) The Contractor shall promptly, and before the conditions are disturbed, give a written notice to the Contracting Officer of (1) subsurface or latent physical conditions at the site which differ materially from those indicated in this contract, or (2) unknown physical conditions at the site, of an unusual nature, which differ materially from those ordinarily encountered and generally recognized as inhering in work of the character provided for in the contract.

(b) The Contracting Officer shall investigate the site conditions promptly after receiving the notice. If the conditions do materially so differ and cause an increase or decrease in the Contractor's cost of, or the time required for, performing any part of the work under this contract, whether or not changed as a result of the conditions, an equitable adjustment shall be made under this clause and the contract modified in writing accordingly.

(c) No request by the Contractor for an equitable adjustment to the contract under this clause shall be allowed, unless the Contractor has given the written notice required; provided, that the time prescribed in (a) above for giving written notice may be extended by the Contracting Officer.

(d) No request by the Contractor for an equitable adjustment to the contract for differing site conditions shall be allowed if made after final payment under this contract (U.S. Government Printing Office (GPO), 2014).

3.4.3 Definitions

There are two types of differing site conditions described in part (a) of the clause. In part (a), (1), “subsurface or latent physical conditions at the site which differ materially from those indicated in this contract” are subsurface conditions which materially exceed those stated or indicated in the contract documents (U.S. GPO, 2014, Gould, 1995). These are referred to as ‘Type 1’ or ‘unforeseen’ conditions. In part (a), (2), “unknown physical conditions at the site, of an unusual
nature, which differ materially from those ordinarily encountered and generally recognized as inhering in work of the character provided for in the contract” are subsurface conditions which are a ‘surprise’ to all parties on a project (U.S. GPO, 2014, Gould, 1995). These are referred to as ‘Type 2’ or ‘unforeseeable’ conditions. FIDIC provides further definition of an unforeseeable site condition, as one which is “not reasonably foreseeable by an experienced Contractor” (FIDIC, 2010).

3.4.4 Application

Unforeseen ground conditions may arise due to an insufficient site investigation, and/or inadequate geologic and/or geotechnical interpretation (e.g. human error) (Baynes, 2010). Unforeseeable conditions are often a result of being unable to investigate every geologic detail on a project, or are those conditions which are excluded from the contract documents as they are considered unlikely to occur (Baynes, 2010, Samuels and Sanders, 2007).

This clause allows for DSCs to be handled within the contract as an administration item (with predefined payment provisions), rather than proceeding with other costly dispute resolution measures, such as litigation or arbitration (Essex 2007, Essex 1996).

An unexpected adverse subsurface condition qualifies as a Type 1 condition if it is materially different from the conditions described in the contract documents. For a Contractor to receive compensation for a Type 1 condition, they must prove that they relied on the subsurface conditions presented in the contract documents in preparing their tender, the condition materially exceeds those described in the contract documents, the condition was not able to be foreseen, and that their increased construction costs are solely the result of the unforeseen condition (Kutil and Silverburg, 2007). Determining the material difference involves comparing the physical and mechanical characteristics and/or quantity of the encountered condition, to what was presented in the contract documents (McClure, 1985). Suggestions in the literature as to a starting point for when a material difference has been encountered are if the encountered condition is 15 – 20%
greater than what was anticipated (e.g. strength, volume, etc.) (McClure, 1985, Levy, 2000). However, as it may be difficult to quantify a material change, the effect of the material changes – namely, if the Contractor’s means and methods have changed, can provide a basis for determining whether a material change has occurred (McClure, 1985). In these cases, the financial impact to a Contractor (e.g. increased construction costs, schedule delays) is evaluated to determine an equitable adjustment under the DSC clause (Gould, 1995, McClure, 1985). Type 1, or unforeseen conditions, is the focus of this thesis.

Type 2 conditions are those not mentioned or described in the contract documents. To receive financial compensation, the Contractor must prove the existence of the unusual or unexpected condition and that the condition negatively affected the construction (Kutil and Silverburg, 2007).

As previously discussed, unexpected adverse site conditions are one of the costliest uncertain price increases, and account for a significant percentage of all claims on tunnel projects. While published in 1988, a review of 87 projects in a study by the U.S. National Committee on Tunneling Technology (USNCTT) determined that unforeseen conditions amounted to 55% of all claims (USNCTT, 1984). Of these claims, 95% were for a significant amount, and 30% were settled for the full amount (R. Robinson et al., 2001, after UNSCTT, 1984). To obtain an indication of the magnitude that DSC claims can currently reach, in early 2014, Seattle Tunnel Partners, the design-build team constructing the Alaska Way Viaduct Tunnel, after completing approximately 10% of the TBM drive, submitted a DSC claim worth US $20 million to the Washington State Department of Transportation, on a contract valued at US $1.44 billion (The Seattle Times, 2014).
3.5 Project Geotechnical Reports

There are several types of geologic and geotechnical report formats possible on tunneling projects: Geotechnical Data Reports, Geotechnical Interpretive Reports, Ground Reference Reports, and Geotechnical Baseline Reports.

3.5.1 The Geotechnical Data Report

As previously mentioned in Section 3.4.1, the U.S. National Committee on Tunneling Technology recommended that all geologic data must be disclosed to the Contractor for tender evaluation. The standard document for this purpose in North America is the Geotechnical Data Report (GDR), and forms part of the contract documents. The GDR is written by the design Engineer or as a collaboration between the Owner and the design-build team, and contains all the factual site investigation data for the project (Essex, 2007). This includes a description and/or discussion of: the site geology, the site investigation program methodology and data (including any actual borehole or other logs), and the laboratory testing program procedures and results (Essex, 2007).

3.5.2 The Geotechnical Interpretive Report and the Ground Reference Report

The Geotechnical Interpretive Report (GIR) and the Ground Reference Report (GRR) are both interpretive geotechnical documents of the ground conditions. Formal guidelines as to their contents are unknown. In the GIR, the interpretation of the ground conditions is from a design perspective, where the project issues and design requirements are presented (Chapman et al., 2010). The GRR is simply a geological and geotechnical interpretation of the expected ground conditions – it does not include design or construction considerations or requirements (The International Tunnelling Insurance Group (ITIG), 2006). Both documents can be used in the tender process (Chapman et al., 2010, ITIG, 2006).
These documents may be similar to a Geotechnical Baseline Report (discussed in the following section), in their level of interpretation of the expected ground conditions. However, both of these reports do not have construction issues as their main focus.

3.5.3 The Geotechnical Baseline Report

3.5.3.1 Publications
As previously mentioned in Section 3.4.1, the Geotechnical Baseline Report (GBR) evolved as part of the efforts to reduce the litigious nature of the North American underground construction industry. Prior to the publication of the initial GBR guidelines, there were no formal content guidelines for interpretive geotechnical reports for underground projects, resulting in inconsistent, ambiguous reports, which possibly conflicted with other contract documents (Essex, 2007). Furthermore, these interpretive reports often did not have construction issues as their main focus. The first formal guidelines were published in 1997: “Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices” by the American Society of Civil Engineers (ASCE), with editor Randall J. Essex. This booklet describes how to write a contractual description of the expected subsurface ground conditions, known as the ‘baseline’, with the primary focus on construction based issues (Essex, 2007). Following 10 years of industry application, the second version was published in 2007: “Geotechnical Baseline Reports for Construction: Suggested Guidelines” (Essex, 2007). While similar to the first version, the second version was expanded to include guidelines for other project types (e.g. pipelines, shafts, highways), applications to design-build procurement, and ‘lessons learned’. This later reference is used for this thesis.

3.5.3.2 Purpose of the GBR
The GBR is primarily a risk sharing contract document which describes the expected physical ground conditions for construction along a tunnel alignment (Hatem, 1998). Risk due to the
expected ground conditions is allocated to either the Owner or Contractor with the baseline. Expected ground conditions less adverse or consistent with the baseline are the contractual and financial responsibility of the Contractor, whereas ground conditions exceeding the baseline are the financial responsibility of the Owner (Hatem, 1998, Essex, 2007).

The GBR is a multi-purpose document used throughout the design and construction process. During design, the GBR provides assistance for project costing and assessment of the Owner’s financial risk (Essex, 2007). Bidders have a better understanding of the main project issues and restrictions which influenced the requirements for design and construction, as well as the allocated geotechnical risks, on which they can base their construction methodology and bid price (Essex, 2007). During tunnel construction, the GBR provides contract administration assistance, guidelines for construction monitoring, and either prevents ground conditions disputes, or aids in resolving disputes with the DSC clause (Essex, 2007). Due to its wide range of applications, GBRs have become a popular risk management tool on tunnel projects in North America.

3.5.3.3 GBR Document Contents
As presented in “Geotechnical Baseline Reports for Construction: Suggested Guidelines” (Essex, 2007) for traditional design-bid-build underground projects, a GBR has eight main topical sections, summarized as follows:

1. Introduction: project name, Owner, design team, and the contractual order of documents.
2. Project Description: location, purpose, general project dimensions, etc.
3. Sources of Geologic and Geotechnical Information: list and describe information sources.
4. Project Geologic Setting: description of the site geology, groundwater, topography, environmental concerns, and the site investigation and testing program.
5. Previous Construction Experience: for similar relevant projects, identify and describe the
evacuation means and methods, ground behaviours, groundwater conditions, rock
support, and if construction problems were encountered, how they were overcome.

6. Ground Characterization: qualitative and quantitative baseline descriptions of the
physical and mechanical properties of the individual ground types (e.g. soil or rock units),
including excavation lengths, and other ground conditions which may affect construction
(e.g. groundwater condition, boulders, material disposal requirements).

7. Design Considerations: description of the requirements and procedures utilized for the
final support design, including rockmass classification systems employed, ground type
support and monitoring requirements, and groundwater and environmental
considerations.

8. Construction Considerations: description of the expected ground behaviour in regards to
the excavation methodology, description and explanation for Contractor means and
methods prescriptions or restrictions, explanation for groundwater condition baseline
estimates, and anticipated construction difficulties and potential delays.

3.5.3.4 GBR Development and Project Delivery Methods
On design-bid-build projects, the GBR is developed by the Owner’s design Engineer in
accordance with the Owner’s subsurface risk tolerance (Essex, 2007). In the GBR, the design
Engineer describes the expected physical and behavioural ground conditions based on a
construction methodology (Essex and Warren, 2010). In working with the Owner to develop the
baseline, design Engineers should ensure that the Owner understands what the baselines mean,
and the consequences and financial implications of where the baseline is set (Essex, 2007).

Baselines can be difficult to develop due to complex geologic conditions, inadequate site
investigation data, and unknown Contractor means and methods (Freeman et al., 2009). As the
construction methodology is the Contractor’s responsibility, means and methods prescriptions by
the design Engineer and Owner are often limited to prescribing an excavation philosophy (e.g. Contractors must use a pressurized face TBM), or with a performance specification (Biggart et al., 2010). Prescribed or restricted construction means and methods should be explained in the GBR, as construction risk is significantly influenced by the construction methodology (Biggart et al., 2010).

On design-build projects, as recommended by Essex (2007), the GBR should be a collaborative document developed by the Owner and the design-build team, based on: the site investigation results obtained by either party, the Owner’s risk tolerance, and the preferred construction methodology of the design-build team. The GBR may be developed as part of the procurement process, where Owners supply potential design-build teams with a baseline containing only the physical ground characteristics. Bidding design-build teams then interpret these characteristics in correlation with their construction methodology. The GBR may become part of a design-build team’s tender documents, and if selected, that GBR is incorporated into the contract documents (Essex, 2007).

For both project delivery formats, baselines should be quantified where possible, and be measurable during construction (Essex, 2007). The baseline does not need to be based on data obtained solely from the project site. While requiring additional explanation, baseline descriptions can be developed for geologic conditions known to exist (e.g. based on local previous construction experience), but not encountered during the project site investigation program (Essex, 2007).

3.5.3.5 GBR Risk Allocation
As it is impossible to precisely and accurately predict the subsurface ground conditions, the GBR is one possible interpretation of the available geologic data (Essex, 2007), and not a warranty of conditions which will be encountered (Hatem, 1998). Where the interpretation or baseline is set significantly influences the bid price, the quantity and cost of change orders and/or claims, and
the degree of final project cost fluctuation (Essex, 2007). A reasonable, or balanced baseline, where ground conditions risk is assigned to those best able to manage it, results in lower Contractor bid prices as the Contractor is allocated the risk over which they have control (Biggart et al., 2010).

A conservative GBR which portrays an adverse subsurface condition (in comparison to the available geologic data) transfers the majority of the subsurface risk to the Contractor. A conservative baseline may be desirable from the Owner’s perspective due to the uncertainty associated with limited geologic data and/or high geologic variability (Freeman et al., 2009). However, a conservative baseline results in greater Contractor bid prices as Contractors will include larger contingencies to account for this increased risk (Essex, 2007). For example, for a project with prescribed tunnel boring machine (TBM) excavation, if the highest Cerchar abrasivity value from laboratory testing was 2.0 (medium abrasive), yet the baseline was set at 4 (very abrasive), this represents greater TBM cutter wear rates and possibly slower TBM advance rates (Cerchar, 1986). Here, the Owner will pay for a greater number of predicted cutter changes, and possibly a longer schedule, regardless of whether they occur. If the subsurface conditions are portrayed in a particularly adverse manner, a Contractor may claim additional time and/or cost impacts due to an incorrect construction methodology, and misrepresentation of the geologic data (Hatem, 1998).

A conservative baseline may result in greater certainty in the bid price, as by transferring geotechnical risk to the Contractor, the Contractor may account for this risk in their bid price, and DSC claims may become less likely to occur. For public Owners, a conservative baseline with a higher bid price may be more politically favorable than incurring change orders and Contractor claims for adverse site conditions, as these conditions unpredictably increase the total project cost (Essex, 2007). However, public Owners are often mandated to select the low bidder, which may
negate any perceived final project cost advantages in having a conservative baseline (Samuels and Sanders, 2007).

Contractors are not required to bid to the baseline. Contractors may bid below the baseline either in an attempt to be competitive and become the low bidder (Essex, 2007). Contractors can also take on the risk of bidding below the baseline, or exclude the baseline from their bid, if they believe that the baseline is conservative or not representative of the actual conditions. In doing so, Contractors assume the risk of being mistaken (Essex, 2007).

3.5.3.6 The GBR and the DSC Clause
As mentioned in Section 3.5.3.2, another primary purpose of the GBR is to administer the DSC clause (Essex, 2007). The DSC clause works best with clear, unambiguous, measureable baselines (Essex, 2007).

To prevent confusion, the GBR should be the only interpretive geotechnical contract document on a project, and it should be stated in the contract documents that in the event of a dispute, the GBR should take contractual precedence over the GDR (Essex, 2007). For conditions which exceed the baseline, while it is more expensive to pay for these conditions under the DSC clause, these costs to the Owner are only incurred if the adverse condition occurs (Biggart et al., 2010).

The ground behaviour encountered during excavation represents the majority of disputes on underground construction projects (Stolz, 2012, Essex, 2007). While the ground conditions presented in the GBR are the financial responsibility of the Contractor, and those exceeding the baseline are the financial responsibility of the Owner, classifying adverse site conditions as within the Owners or Contractors responsibility may be difficult, as rockmass behaviour is a function of both the Owner represented subsurface conditions (requiring interpretation from the contract documents), and the Contractor’s construction methodology (Stolz, 2012).
3.5.3.7 Current Perception in Industry

In the recent literature there have been a few papers and discussions analyzing the effectiveness of GBRs on tunnel projects (e.g. Edgerton et al., 2012, Freeman et al., 2009, Heslop and Caruso, 2013, Tunnel Business Magazine, 2009). The overall consensus appears to be that GBRs are accepted by the tunneling industry as a good risk sharing tool. The main points (which often reiterate that which is presented in the ASCE GBR guidelines), are briefly summarized as follows:

- Baselines not consistent with the data presented in the GDR should be explained.
- Baselines should be unambiguous and not conflict with other baselines
- Baselines should relevant to construction and measureable during construction.
- GBRs should not be conservative and act as a risk transfer tool, rather GBRs should describe the anticipated ground conditions and construction issues.
Chapter 4

GBR Current State of Practice and Failure Mode Uncertainty

4.1 Introduction

Since their introduction in 1997, GBRs have become a popular risk sharing mechanism on North American tunnel projects as they are based on a simple concept: ground conditions described within the GBR are the financial responsibility of the Contractor, whereas those conditions which exceed the GBR are the financial responsibility of the Owner. However, as described in the previous chapter, risk allocation in the GBR is not always equitable or allocated to those best able to manage it, with risk-averse Owners using the GBR as a risk transfer mechanism instead.

To determine the current status and effectiveness of GBRs in the tunneling industry, the author conducted a review of nine GBRs obtained through various means, and developed a survey. This survey, titled the GBR Industry Survey, had several main topical sections including the tools, methods, and properties used to describe various rockmasses along the tunnel alignment, the use of rockmass classification systems, anticipated rockmass behaviours, excavation means and methods, and differing site conditions claims. Several trends were identified as a result of the author’s GBR review and these will be discussed in the first half of this chapter. The impact of these trends, specifically, the use of material properties and rockmass classification systems to describe rockmasses along a tunnel alignment, is analyzed in the second half of this chapter.

4.2 GBR Review

To understand how risk can be allocated in GBRs, the author reviewed nine GBRs obtained from online sources and the author’s supervisor, Dr. Mark Diederichs. This review categorized the GBR contents with respect to the project geometry, previous relevant construction experience,  

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1 Included in part in the conference paper by van der Pouw Kraan, Hutchinson, and Diederichs (2013).
ground characterization methods, how soil and/or rock properties were specified, the use of rockmass classification systems, and whether design and construction considerations were included. Additionally, the project delivery and financing method for each project were researched to further define how the subsurface risk was allocated. The results of this review were incorporated where possible into the GBR Industry Survey results, as discussed in the next section. (Note: while a few of these GBRs are publicly available online, all the GBRs will remain confidential.)

4.3 The GBR Industry Survey

An online survey was developed with the online survey generation website SurveyMonkey to determine the current status quo and effectiveness of GBRs in the tunneling industry by those who use them: Owners, Engineers, Contractors, and Academics (2014). The survey appears in Appendix A. To conduct this survey approval was required from the Queen’s University General Research Ethics Board, as the survey involved human subjects (Queen’s University, 2014). The General Research Ethics Board Delegated Clearance Review Letter appears in Appendix B. As GBRs are often confidential documents, the survey was designed to be completely anonymous, in that it did not ask for any respondent or tunnel project identifying information, nor did it track IP addresses. To ensure anonymity, data is only publishable in a statistical format, with no individual results. The survey was also wholly voluntary, in that respondents did not have to answer any questions they found objectionable.

The survey consisted of 40 multiple choice and short answer questions about a tunnel project of the respondent’s choice. Respondents were able to answer the survey up to a maximum of five times, each time with a different tunnel project. The survey questions were ordered to guide a respondent through a project, beginning with the basic respondent and project environment and geometry information, then the contract information, GBR ground conditions contents (e.g. how soil and/or rock, and rockmass properties were specified, rockmass
classification systems, descriptions of the various rockmasses along the tunnel alignment, anticipated rockmass behaviours, tunnel domains, excavation means and methods), and ending with DSC claims (e.g. whether a claim occurred, what type of claim, the perceived effectiveness of the GBR in mitigating claims).

The survey was emailed to fifty contacts of the author located in North America, Australia, Europe, and the Middle East. These contacts are engineering consultants, contractors, academics, and owners, with industry experience varying from a few years to decades.

The number of responses per question ranged from one to seventeen. In the first half of the questionnaire, there was an average of twelve responses per question, whereas in the second half, this average dropped to six. Many questions were also partly answered. Of the total seventeen projects referred to in the survey, eleven projects from ten respondents were identified as having a significant number of commonly answered questions. These eleven projects form the basis of the discussion that follows. In order to obtain a more encompassing view of the current use of GBRs in the tunneling industry, these responses were compared to the results from the author’s GBR review, and then compiled. For the majority of subsections that follow, the results and discussion are based on these twenty projects. Within each section it is stated how many responses are from the survey and/or the review.

4.3.1 GBR Industry Survey and Review Results

4.3.1.1 Respondent Data
The first set of questions asked basic classifying information about the respondents: the number of years of experience and in what roles, the number of GBRs worked with, and their role on the project being reported on (questions 1, 2, and 5). Summarized results are presented in Table 4-1 to Table 4-3 below. While thirteen to seventeen respondents answered these questions, only the ten respondents who answered a significant number of survey questions were considered here.
Table 4-1: Respondent years of experience.

<table>
<thead>
<tr>
<th>Role on Project</th>
<th>Years of experience</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 – 5</td>
<td>5 – 10</td>
</tr>
<tr>
<td>Consultant</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>Contractor</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Owner</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Academia</td>
<td>1</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 4-2: Respondent GBR experience – number of GBRs worked with.

<table>
<thead>
<tr>
<th>No. of GBRs</th>
<th>1 – 3</th>
<th>3 – 5</th>
<th>5 – 10</th>
<th>10 – 15</th>
<th>15 – 20</th>
<th>20 – 30</th>
<th>&gt; 30</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Respondents</td>
<td>3</td>
<td>1</td>
<td>4</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 4-3: Respondent role on the project selected.

<table>
<thead>
<tr>
<th>Role</th>
<th>Consultant</th>
<th>Contractor</th>
<th>Owner</th>
<th>Academia</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Respondents</td>
<td>7</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>1</td>
</tr>
</tbody>
</table>

In Table 4-1 as several respondents worked in more than one capacity during their career, this results in a greater number of roles than respondents. The majority of the respondents were consultants (likely the result as the author worked in consulting for 3 years) with a significant amount of experience – four of the respondents have greater than thirty years of experience. Similarly, many of the respondents are experienced with GBRs, with six of the ten respondents indicating experience with greater than five to ten GBRs (Table 4-2). Within this survey, the majority of respondents acted as a consultant on their project of choice (Table 4-3).

4.3.1.2 Basic Tunnel Project Information

Questions 6 to 11 asked for the tunnel project environment and purpose, excavation means and methods used, tunnel geometry, the ground type, and the groundwater conditions.

4.3.1.2.1 Tunnel Project Environment and Purpose

Questions 6 and 7 asked about the tunnel project environment and purpose, the results of which are summarized in Table 4-4. Of the twenty projects (eleven from the survey and nine from the
review), 65% were located in urban environments with the primary purposes of water conveyance and rail. The remaining projects were located in rural and alpine settings, at 25% and 10% respectively, with applications including rail, water conveyance, and mining.

Table 4-4: Project Environment and Purpose.

<table>
<thead>
<tr>
<th>Purpose</th>
<th>Urban Survey</th>
<th>Urban Review</th>
<th>Rural Survey</th>
<th>Rural Review</th>
<th>Alpine Survey</th>
<th>Alpine Review</th>
<th>Total (No.)</th>
<th>Total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>Water conveyance</td>
<td>4</td>
<td>1</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>2</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>Wastewater</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Mining</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Highway/road</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td><strong>Total No.</strong></td>
<td><strong>13</strong></td>
<td><strong>5</strong></td>
<td><strong>2</strong></td>
<td><strong>-</strong></td>
<td><strong>-</strong></td>
<td><strong>-</strong></td>
<td><strong>20</strong></td>
<td><strong>100</strong></td>
</tr>
<tr>
<td><strong>Total (%)</strong></td>
<td><strong>65</strong></td>
<td><strong>25</strong></td>
<td><strong>10</strong></td>
<td><strong>-</strong></td>
<td><strong>-</strong></td>
<td><strong>-</strong></td>
<td><strong>20</strong></td>
<td><strong>100</strong></td>
</tr>
</tbody>
</table>

4.3.1.2.2 Rock Type and Excavation Means and Methods

Question 8 asked respondents to select from a list all the excavation means and methods used on the project. This list included: earth pressure balance (EPB) TBMs, slurry TBMs, compressed air TBMs, single shield TBMs, double shield TBMs, open beam TBMs, drill and blast (D&B), road headers/excavators/rippers, New Austrian Tunneling Method (NATM)/Sequential Excavation Method (SEM), and cut-and-cover. The twenty projects (eleven from the survey and nine from the review), produced twenty-three responses as several projects had more than one excavation methodology, which are summarized by source (survey or review) and cross referenced with the ground type (as further discussed in the following section) in Table 4-5. In hard rock conditions, open beam TBM and drill and blast were the most prevalent, whereas the other three ground types each had three of the following methods: EPB TBMs, drill and blast, road header/excavator/rippers, and NATM/SEM.
Table 4-5: Ground type and Excavation Means and Methods.

<table>
<thead>
<tr>
<th>Rock Type and Source</th>
<th>EPB TBM</th>
<th>Open beam TBM</th>
<th>D&amp;B</th>
<th>Road headers/ excavators/ rippers</th>
<th>NATM/ SEM</th>
<th>Total (No.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard Rock Survey</td>
<td>-</td>
<td>1</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft Rock Survey</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Soil Survey</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mixed Survey</td>
<td>2</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Total (No.) (%)</td>
<td>6</td>
<td>3</td>
<td>8</td>
<td>2</td>
<td>4</td>
<td>23</td>
</tr>
</tbody>
</table>

4.3.1.2.3 Tunnel Project Geometry, In-Situ Stress, and Groundwater Conditions

Questions 9 to 11 asked further basic tunnel project information including the number of bores, tunnel geometry, minimum and maximum ground cover, ground type, and the in-situ stress and groundwater conditions. Eleven survey responses and nine reviewed GBRs were considered. In this section not all the questions were fully answered, or able to be answered.

Ground types for these twenty projects included eight projects in hard rock, two in soft rock, six in soil, and four in mixed conditions. ‘Mixed conditions’ is considered a mix of hard rock, soft rock, and/or soil conditions. Summarized project geometry and groundwater conditions data per ground type and source is presented in Table 4-6 and Table 4-7 below.
Table 4-6: Summarized project geometry by source ('Sur.' is from the survey, ‘Rev.’ is from the review).

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Number of bores</th>
<th>Source</th>
<th>Tunnel Length (km)</th>
<th>Tunnel Span (m)</th>
<th>Maximum cover (m)</th>
<th>Minimum Cover (m)</th>
<th>In-situ stress ratio (H:V)</th>
<th>Topography Present</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>&lt;2</td>
<td>&lt;5</td>
<td>&lt;50</td>
<td>&lt;20</td>
<td>&lt;1</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 - 5</td>
<td>5 - 10</td>
<td>50 - 100</td>
<td>20 - 400</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt; 10</td>
<td>&gt; 10</td>
<td>&gt; 1000</td>
<td>&gt; 4000</td>
<td>&gt; 1</td>
<td>Yes</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Single</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>-</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Double</td>
<td>2</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Multi</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
</tr>
</tbody>
</table>
Table 4-7: Summarized project groundwater conditions (‘Sur.’ is from the survey, ‘Rev.’ is from the review).

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Source</th>
<th>Hard Rock</th>
<th>Soft Rock</th>
<th>Mixed</th>
<th>Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below the groundwater table</td>
<td>Y</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>N</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average groundwater pressure (m)</td>
<td>&lt; 20</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>20 - 40</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>&gt; 40</td>
<td>2</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average hydraulic conductivity (m/s)</td>
<td>10-5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>10-7 - 10-8</td>
<td>3</td>
<td>2</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>Localized groundwater inflows</td>
<td>Y</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>N</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average localized groundwater inflow (L/s)</td>
<td>&lt; 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1 - 5</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>&gt; 5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Groundwater flow</td>
<td>Through discrete fractures</td>
<td>4</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Through the groundmass</td>
<td>2</td>
<td>-</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

While there were not a significant number of projects per ground type, a few trends are observed. Hard rock projects were on average the longest, with the greatest maximum cover. It is interesting that three of the six projects noted an in-situ stress ratio (horizontal:vertical) of 1, as topography was also recorded for all these projects. Groundwater conditions consisted of localized groundwater inflows occurring through discrete fractures, surrounded by a low hydraulically conductive material. The six soil projects were generally shorter in length than the hard rock projects, and located close to the surface. An in-situ stress ratio was not reported for the majority of these projects (this could either be the result of an unknown in-situ stress ratio, or it is not applicable). Groundwater flowed through the low hydraulically conductive groundmass. Projects located in mixed and soft rock conditions either had a wide range of answers, and/or
occurred infrequently (four and two projects, respectively). Data trends were not able to be observed for the projects in these ground types.

As previously mentioned in Chapter 2, the in-situ stress and groundwater conditions are two of the most difficult conditions to determine. That is reflected here, as these parameters had the lowest response rate out of all these questions, with a largest number of blank or ‘unknown’ responses. The in-situ stress ratio and groundwater conditions questions were missing responses in eight and seven of the twenty projects, respectively. However, it is noted that the in-situ stress and groundwater conditions may not be applicable or critical to every project, and therefore may not have been investigated.

4.3.1.3 Tunnel Project Contract Information
Questions 12 to 16 asked about the project delivery and financing method, and the types of geotechnical reports used on the project, as discussed below.

4.3.1.3.1 Project Delivery and Financing Method
The project delivery method was cross referenced with the financing method for nineteen projects (eleven from the survey, and eight from the review). Within the author’s review, the project delivery and/or financing method were not always obtainable: the project delivery and financing method could not be determined for one project, and financing methods could not be determined for two design-bid-projects (these are shown as ‘unknown’). Additionally, a few projects recorded more than one financing method. Summarized results by source are presented in Table 4-8.
A relatively similar number of project delivery and financing methods occurred in both the GBR survey and in the review. Design-bid-build and design-build projects represented the significant majority of projects, at 55% and 40%, respectively. The low occurrence of turnkey/EPC/design-build-operate-finance (DBOF) projects (5%) is not surprising, given that on these projects Owners may allocate all the subsurface ground conditions risk to a Contractor, therefore potentially not requiring a GBR, and thus not applicable to this survey.

Design-bid-build and design-build each were each associated with three financing methods. Lump-sum and unit-price were the most prevalent, with application on 35% and 40% of the projects, respectively. Time and materials, and target price occurred on 5% and 10% of projects, with ‘unknown’ as the remaining 10%.

4.3.1.3.2 Use of Geotechnical Reports
Questions 14 to 16 enquired about the use of GDRs, GBRs, and other geotechnical interpretive reports on the project. Due to the current state of practice of disclosing all geologic data to project bidders, and the subject of this survey, GDRs and GBRs occurred on all twenty projects, respectively. As to whether another form of interpretive report was available on the project (the
example given was a GRR), data was only available from the eleven survey respondents as this information was not available from the author’s review. Within the survey data, 45% had another interpretive document, whereas 55% did not. No further questions were asked regarding the nature of these interpretive reports.

4.3.1.4 Geotechnical Baseline Reports

The next series of questions were regarding how the soil and rock properties were specified in the GBR and the use of rockmass classification systems.

4.3.1.4.1 Specification of Soil and Rock Properties

Tools and methods used to determine the soil and rock properties in the GBR for twenty projects are summarized by source in Table 4-9 (question 17). Laboratory testing was the most popular method (nineteen projects), followed by in-situ testing (thirteen projects), then by site investigation field estimates (e.g. Schmidt hammers) and correlations from empirical methods (e.g. rockmass classification systems) (both on eleven projects). Application of a strength criterion (e.g. Mohr-Coulomb) and prior knowledge were the least popular options, with seven and five projects, respectively.

Table 4-9: Specification of soil and rock properties.

<table>
<thead>
<tr>
<th>Tool or Method</th>
<th>Survey</th>
<th>Review</th>
<th>Total (No.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratory Testing</td>
<td>10</td>
<td>9</td>
<td>19</td>
</tr>
<tr>
<td>In-situ testing</td>
<td>6</td>
<td>7</td>
<td>13</td>
</tr>
<tr>
<td>Site investigation field estimates</td>
<td>6</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>Correlations from empirical methods</td>
<td>10</td>
<td>1</td>
<td>11</td>
</tr>
<tr>
<td>Application of a strength criterion</td>
<td>5</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>Prior knowledge</td>
<td>5</td>
<td>-</td>
<td>5</td>
</tr>
</tbody>
</table>
The number of tools and methods used to determine the soil and rock properties per project was also counted: 20% of projects used 5 or 6 methods, 15% used 4 methods, 35% used 3 methods, and 30% used less than 3 methods.

4.3.1.4.2 Use of Rockmass Classification Systems
The next two questions were regarding rockmass classification systems. First, respondents were asked to select from a list of rockmass classification systems which systems were applied to the project, and the order of preference. Then for the most preferential system, respondents were asked what the contributing factors were in selecting this system, and the importance of each factor (questions 18 and 19). This list of classification systems included: RQD, RMR, Q, Q’, QTBM, GSI, the Rock Load Classification, Rock Mass Index (RMi), Rock Structure Rating (RSR), Mining Rock Mass Rating (MRMR), and the Mathews/Potvin Stability Graph Method. While a few of these classification systems are more applicable to mining purposes, they were included for thoroughness.

Eleven and seven respondents answered questions 18 and 19, respectively. It was only possible to supplement the first part of question 18 with data from the GBR review (i.e. which classification systems were used). Due to the small number of responses for the remaining questions, only trends can be discussed.

4.3.1.4.2.1 Rockmass classification systems occurrence: GBR review and survey data
The frequency of occurrence of rockmass classification systems for twenty projects (eleven from the survey and nine from the review) is shown in Figure 4-1. RQD, RMR, and Q were the most prevalent systems, with application of RQD on 60% of the projects, and 30% for both RMR and Q. As RQD is a parameter within RMR and Q, this is not surprising. GSI, Q’, the Rock Load Classification, and QTBM were applied on 20%, 15%, 15%, and 5% of projects, respectively. RMi, RSR, MRMR, and the Mathews/Potvin Stability Graph Method were not applied to any of the projects.
Overall, rockmass classification systems were applied to 65% of the projects, with the remaining 35% selecting ‘Not Used’ to all the classification systems in the list. The number of rockmass classification systems applied per project was also tabulated: 15% of projects used one system, 20% used two systems, 20% used three systems, and 10% used greater than five systems.

The rockmass classification systems were correlated to the ground type. Unsurprisingly, classification systems were the most prevalent on hard and soft rock projects, with 100% of these projects using at least one of the following classification systems: RQD, RMR, Q, Q’, QTBM, GSI, and/or the Rock Load classification system. In the soil and mixed ground conditions, classification systems occurred on 33% and 25% of projects, respectively. For these ground types, three projects used RQD and one used Q. The 35% of projects which did not use a classification system were all located in soil or mixed ground conditions (as these systems are mainly applicable to hard or soft rock conditions, this is not surprising).
4.3.1.4.2 Rockmass classification systems rankings and preference: GBR survey data

The analysis in the preceding section was repeated and furthered with just the survey data in order to correlate the respondent’s preferred classification system with the factors used to select the system. Only seven survey responses are discussed as only seven respondents answered questions 18 and 19 (i.e. four respondents selected ‘Not Used’ to all the classification systems).

For the first part of question 18, within the respondent data similar trends are observed as in the preceding section, in that for the seven projects which applied classification systems, RQD, Q, and RMR remain the most popular classification systems, both in terms of frequency of occurrence (Figure 4-2) and preference of use. RQD occurred on 55% of projects, Q on 45%, and RMR on 36%. The Rock Load Classification System, GSI, Q’, and \( Q_{\text{TBM}} \) occurred on 27%, 18%, 18%, and 9% of projects, respectively. Together, RQD, Q, and RMR occupied 100% of the No. 1 rankings and 83% of the second place rankings. Of these three systems, Q was ranked No. 1 three times, and RQD and RMR both twice. Q’ was ranked No. 2 once. To summarize the overall ranking data for all the systems, weightings were assigned to each ranking: No. 1 was assigned a weight of seven, No. 2 was assigned a weight of six, etc. The results are shown in Figure 4-3.
Figure 4-2: Frequency of occurrence of rockmass classification systems from the survey data.

Figure 4-3: Rockmass classification systems ranking for the seven respondents.

The top factors for selecting these three systems (RQD, RMR, and Q) are not as distinct, as data is only available for seven respondents. Respondents were able to rank up to eight
contributing factors in selecting a particular classification system; however three of the seven respondents only ranked half of the contributing factors, and ignored the remainder, and furthermore, one respondent ranked multiple factors as ‘1’ or ‘2’ for one classification system. Subsequently, the process to determine the top factors for selecting each classification system was done twice: first with the top two factors, and then with all the factors.

For each instance in which these three systems were ranked as ‘1’, the top two factors in selecting these system were tabulated, weighted, and normalized to a scale out of ten, as shown in Figure 4-4. The top factor in selecting RQD was to determine the rockmass behaviours, for RMR it was to determine the preliminary support requirements, while the Q system had a range: ease of use, to determine the expected rockmass behaviours, and to determine the preliminary support requirements.

![Figure 4-4: Preferred top two factors for selecting either the RQD, RMR, or Q systems.](image)

When all of the ranked factors are considered, the top four contributing factors for selecting a classification system were prior experience, to obtain the preliminary support
requirements, and a tie between the relevance of the input parameters to the project, and to determine the expected rockmass behaviours (Figure 4-5). The remaining options: ease of use, as a parameter checklist, the correlation capabilities for obtaining physical or mechanical properties, and to obtain the final support requirements did not appear as influential.

Figure 4-5: Preferred factors for selecting either the RQD, RMR, or Q system.

4.3.1.5 Tunnel alignment rockmass descriptions

The next series of survey questions asked respondents about the most and least favorable, and most dominant rockmasses occurring along the tunnel alignment. Question 20 asked respondents to select from a list all the properties and tools used to define these rockmasses, and questions 21 – 23 asked how the respondent would picture each rockmass along the alignment by ranking a list of rockmass characteristics from very high, high, medium, low, to very low (e.g. ‘very high’ equates to very high strength, very high in-situ stress condition (i.e. $K > 3$), etc., whereas ‘very low’ equates to very low strength, very low in-situ stress condition (i.e. $K < 1$), etc.). These
questions do not ask about GBR contents, rather these questions were posed to be able to ask subsequent questions regarding these rockmasses and the expected rockmass behaviours, DSC claims, and the effectiveness of the GBR.

There were ten survey responses regarding the properties and tools used to describe the most and least favorable, and most dominant rockmasses. The frequency of occurrence of each of these properties and tools on the ten projects is presented in Table 4-10. Geology, alteration, and/or weathering characteristics, intact rock properties, joint characteristics, groundwater characteristics, and classification systems were defined and applied on 70% or more of the projects. It is interesting that the in-situ stress condition and the expected deformation magnitude were the least defined properties as the majority of the tunnels were located in hard rock ground conditions with depths up to 1,700 m. ‘Other’ properties were recorded for 1 project.

**Table 4-10: Frequency of occurrence of the properties and tools used to describe the most and least favorable, and most dominant rockmasses.**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Percent Occurrence (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact rock properties</td>
<td>90</td>
</tr>
<tr>
<td>Classification systems</td>
<td>70</td>
</tr>
<tr>
<td>Geology, alteration, and/or weathering</td>
<td>90</td>
</tr>
<tr>
<td>Joint characteristics</td>
<td>80</td>
</tr>
<tr>
<td>In-situ stress</td>
<td>20</td>
</tr>
<tr>
<td>Groundwater characteristics</td>
<td>80</td>
</tr>
<tr>
<td>Expected deformation magnitude</td>
<td>-</td>
</tr>
<tr>
<td>Other</td>
<td>10</td>
</tr>
</tbody>
</table>

Of these projects, 20% used 6 or 7 of the aforementioned properties and tools in Table 4-10 to define the rockmasses, 50% used 5 properties, and 30% used 3 or less properties.

For questions 21 to 23, ten respondents answered questions 21 and 22 and nine respondents answered question 23. As discussed, for the most and least favorable, and most dominant rockmasses along the tunnel alignment, respondents were asked to rank a series of rockmass characteristics according to a scale from ‘very high’ to ‘very low’, or select ‘Not
applicable’ or ‘Unknown’. These rockmass characteristics included: rock strength, rock stiffness, joint surface quality, degree of rockmass ‘blockiness’, rockmass permeability, groundwater inflows, in-situ stress magnitude, in-situ stress ratio, and the tunnel face scale heterogeneity.

The results were summarized according to the three answer types: the respondent answered using either the ‘very high’ to ‘very low’ scale, or selected ‘Not applicable’, or ‘Unknown’, as shown in Figure 4-6 below. For the majority of the rockmass characteristics the respondents applied the ‘very high’ to ‘very low’ scale. The in-situ stress, both in terms of magnitude and ratio, and the groundwater inflows were the most infrequently defined, with the most ‘unknown’ responses. The least favorable rockmass also had the highest frequency of ‘Not applicable’ responses, potentially indicating that this rockmass may be of poor rock quality.
Figure 4-6: Frequency of occurrence for nine rockmass properties for the most and least favorable, and most dominant rockmasses along the tunnel alignment.
4.3.1.6 Anticipated Rockmass Behaviours

Question 24 asked respondents whether the most and least favorable, and most dominant rockmasses were included in the GBR, question 25 asked whether rockmass behaviours were described for these rockmasses in the GBR, and then question 26 asked respondents to select all the rockmass behaviours which were described in the GBR.

All three questions had a limited number of responses: eight, nine, and seven responses for questions 24, 25, and 26, respectively. Trends for these responses are discussed below. Question 26 was further supplemented with the rockmass behaviours obtained from the nine GBRs from the author’s review, and is also discussed below.

The majority of GBRs described the most and least favorable, and most dominant rockmasses along the tunnel alignment in the GBR, as shown in Table 4-11. The majority of GBRs also included rockmass behaviours for these rockmasses, as shown in Table 4-12.

Table 4-11: Specification of various rockmasses along the tunnel alignment in the GBR.

<table>
<thead>
<tr>
<th>Rockmass</th>
<th>Described in GBR (No. of Responses, %)</th>
<th>Overall (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most favorable</td>
<td>Yes 6, No 1, Partly described 1 12.5%</td>
<td>Yes 72%</td>
</tr>
<tr>
<td>Least favorable</td>
<td>Yes 6, No 1, Partly described 1 12.5%</td>
<td>No 12%</td>
</tr>
<tr>
<td>Most dominant</td>
<td>Yes 6, No 1, Partly described 2 22%</td>
<td>Partly described 16%</td>
</tr>
</tbody>
</table>
Table 4-12: Question 25: whether rockmass behaviours were included in the GBR for the most and least favorable, and most dominant rockmasses.

<table>
<thead>
<tr>
<th>Rockmass</th>
<th>Rockmass behaviours in the GBR (No. of Responses, %)</th>
<th>Overall (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Most favorable</td>
<td>Yes</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>No</td>
<td>2</td>
</tr>
<tr>
<td>Least favorable</td>
<td>Yes</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>No</td>
<td>2</td>
</tr>
<tr>
<td>Most dominant</td>
<td>Yes</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>No</td>
<td>2</td>
</tr>
</tbody>
</table>

Question 26 asked respondents to select from a list all the different rockmass behaviours which were described in the GBR. There were only seven respondents for in question 25 two respondents answered ‘no’ as to whether rockmass behaviours were described in the GBR for these rockmasses. Gravity driven failure modes were reported on 86% of the seven projects, whereas stress driven and water influenced were reported on 29% and 14% of projects, respectively. A text box was supplied at the end of the question for ‘Other’ failure modes, resulting in two soil behaviours recorded. Figure 4-7 shows the reporting frequency for all the different rockmass behaviours in the seven projects.
The seven survey responses were then supplemented with the nine GBRs from the author’s review to obtain a more representative view of the occurrence of rockmass behaviours in GBRs. Within this data set, 68% of projects reported gravity driven failure modes, 38% reported stress driven failure modes, and 31% reported water influenced failure modes. Of the gravity driven behaviours, wedge type failures were the most frequently reported, occurring in approximately 60% of the sixteen GBRs. Stress induced structural failure and squeezing were the most frequently reported stress driven rockmass behaviours, while swelling was the most frequently reported water influenced behaviour. The frequency of occurrence of each rockmass behaviour in these GBRs is shown in Figure 4-8. Rockmass behaviour definitions are discussed in Chapter 5.

**Figure 4-7: Reporting frequency of rockmass behaviour types for seven survey projects.**
As a further comparison, Table 4-13 compares per source the number of rockmass behaviours reported per rockmass behaviour ‘family’: gravity driven, stress driven, and water influenced. Furthermore, the number of projects which reported one to two, and four to nine rockmass behaviours is compared. Totals in the ‘Survey’ and ‘Review’ columns were calculated vertically, and totals in the ‘Survey + Review’ column were calculated horizontally.

**Table 4-13: GBR Survey and Review Rockmass Behaviour Comparison.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Survey (No.)</th>
<th>Review (No.)</th>
<th>Survey + Review (No.)</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Projects</td>
<td>7</td>
<td>9</td>
<td>16</td>
<td>-</td>
</tr>
<tr>
<td>Rockmass Behaviour Families</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravity driven</td>
<td>9</td>
<td>9</td>
<td>18</td>
<td>40</td>
</tr>
<tr>
<td>Stress driven</td>
<td>8</td>
<td>8</td>
<td>16</td>
<td>36</td>
</tr>
<tr>
<td>Water influenced</td>
<td>2</td>
<td>3</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>Other</td>
<td>2</td>
<td>4</td>
<td>6</td>
<td>13</td>
</tr>
<tr>
<td>Total</td>
<td>21</td>
<td>24</td>
<td>45</td>
<td>100</td>
</tr>
<tr>
<td>No. of Projects</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With 1 – 2 behaviours</td>
<td>5</td>
<td>6</td>
<td>11</td>
<td>69</td>
</tr>
<tr>
<td>With 4 – 9 behaviours</td>
<td>2</td>
<td>3</td>
<td>5</td>
<td>31</td>
</tr>
<tr>
<td>Total</td>
<td>7</td>
<td>9</td>
<td>16</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 4-13 shows that the trends from the seven survey results are similar to the results obtained from the GBR review, in the number of rockmass behaviours reported per rockmass behaviour family, and the number of projects with one to two, and four to nine behaviours. Both sources noticeably had approximately two-thirds of projects report one to two behaviours, with the remaining third reporting four or more behaviours. This will be discussed further in Section 4.3.2.

4.3.1.7 Tunnel Domains and Excavation Means and Methods
The next set of questions was regarding tunnel domains and excavation means and methods (questions 27 – 32). Nine respondents answered question 27 as to whether the tunnel alignment was divided into domains for excavation means and methods selection, and two to three respondents answered the subsequent questions about how the domains were specified. This response rate is because if a respondent answered ‘no’ to question 27, they would be automatically skipped over to the next series of questions regarding differing site condition claims (questions 33 – 40). The answers for questions 28 – 32 are not discussed due to the insufficient number of responses.

The nine responses for question 27 were supplemented with the nine projects from the author’s review. As seen in Table 4-14 below, the majority of projects did not domain the tunnel alignment for the purposes of specifying excavation means and methods.

Table 4-14: Results for whether the tunnel alignment was divided into domains for means and methods selection.

<table>
<thead>
<tr>
<th>Answer</th>
<th>Answer Source</th>
<th>No. of Responses</th>
<th>Responses (%)</th>
<th>Total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yes</td>
<td>Survey</td>
<td>3</td>
<td>33</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td>2</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>Survey</td>
<td>6</td>
<td>66</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>Review</td>
<td>7</td>
<td>78</td>
<td></td>
</tr>
</tbody>
</table>
The large percentage of ‘No’ responses may be part of a greater trend in the hesitation to interpret or discuss the anticipated rockmass behaviours and the excavation means and methods. This will be discussed further in Section 4.3.2.

4.3.1.8 Differing Site Conditions Claims
The last section of survey questions were regarding the suitability of GBR ground conditions descriptions, DSC claims, the effectiveness of the GBR in preventing DSC claims, why DSC claims occurred, the effectiveness of dispute resolution methods, whether excavations means and methods should be prescribed in a GBR, and two opinion based questions regarding excavation means and methods and GBR suggestions and/or recommendations (questions 33 – 40). As only up to seven respondents fully or partially answered these questions, and it was not possible to supplement these questions with data from the GBR review (e.g. GBRs may be confidential documents, whereas DSC claims typically are confidential proceedings), there is limited confidence in these results. Trends will be discussed for the questions which had at least 6 responses.

4.3.1.8.1 Suitability of GBR descriptions
Continuing with the questioning regarding the most and least favorable, and most dominant rockmasses along the tunnel alignment, question 33 asked about the adequacy of the GBR description of the rockmass properties, behaviours, and construction issues for construction purposes for these rockmasses. Six and seven respondents answered this question. As shown in Figure 4-9, the most favorable and most dominant rockmasses had nearly 100% adequate descriptions of the rockmass properties, rockmass behaviours, and the construction considerations in the GBR. For the least favorable rockmass these descriptions were not nearly as effective, with ‘inadequate’ and ‘unknown’ responses on two projects.
4.3.1.8.2 Differing Site Conditions Claim Occurrence

Question 34 asked whether a Type 1 or Type 2 claim occurred on the project. The majority of the six respondents reported that a Type 1 or 2 DSC did not occur, as shown in Table 4-15. While there is limited data, the relatively low occurrence of DSC claims on these projects may be linked to the adequacy of the rockmass descriptions as discussed in the previous question.

Table 4-15: Frequency of occurrence of Type 1 and 2 DSC claims.

<table>
<thead>
<tr>
<th>DSC Claim Type</th>
<th>Yes (%)</th>
<th>No (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td>33</td>
<td>67</td>
</tr>
<tr>
<td>Type 2</td>
<td>17</td>
<td>83</td>
</tr>
</tbody>
</table>

4.3.1.8.3 Effectiveness of the GBR in describing the ground conditions for construction purposes

The next question (No. 35) related the effectiveness of the GBR in describing the most and least favorable, and most dominant rockmasses along the tunnel alignment with whether a DSC occurred. Of the nine respondents who fully or partially answered the question, approximately 60% considered the GBR effective, in that a DSC did not occur for these rockmasses (Figure 4-10). For the most dominant and least favorable rockmasses these descriptions were considered ineffective on two separate projects. ‘Unknown’ responses occurred on three projects.
Figure 4-10: Reporting frequency of the perceived effectiveness of the GBR in mitigating DSC claims for the most and least favorable, and most dominant rockmasses.

4.3.1.8.4 Reasons for DSCs and Dispute Resolution Methods
The next two questions (36 and 37) asked the reasons for whether a DSC occurred and the effectiveness of four dispute resolution methods (dispute review board, mediation, arbitration, litigation, or other), respectively. These questions were only answered by the two respondents who reported in question 35 that the GBR contained an inadequate description of the ground conditions. A trend cannot be determined from so few answers; however it is interesting to note that for question 36, both answers were regarding an inadequate understanding of the rockmass behaviour and/or inappropriate excavation means and methods.

4.3.1.8.5 Effectiveness of the GBR in mitigating DSC claims
Question 38 asked whether the GBR was an effective tool for mitigating a Type 1 or Type 2 DSC claim on a project. The results for six respondents are presented in Figure 4-11 with two of these six only partially answering the question. While this is a very limited data set, it appears that the GBR was nearly equally effective in preventing Type 1 and Type 2 DSCs, at 40% and 60%,
respectively (the 20% difference in effectiveness between Type 1 and Type 2 is one data point). Approximately one-half of respondents reported that the GBR was a somewhat, or ineffective tool to mitigate a DSC claim.

![Graph showing reporting frequency for whether the GBR was considered effective in mitigating Type 1 and/or Type 2 DSC claims.]

**Figure 4-11: Reporting frequency for whether the GBR was considered effective in mitigating Type 1 and/or Type 2 DSC claims.**

4.3.1.8.6 Excavation Means and Method in a GBR

The last two questions of this survey were opinion and experience based questions. Question 39 asked whether excavation means and methods should be included in the GBR for two project delivery methods: design-bid-build, and design-build. As shown in Table 4-16 below, the seven respondents are near evenly split for design-bid-build projects as to whether excavation means and methods should be specified, but for design-build, the data suggests that the excavation means and methods should not be specified in a GBR. The higher ‘No’ response for design-build projects may be related to the fact that typically on design-build projects the Owner has less control over the project development, as the final design is developed in accordance with the Contractor’s preferred excavation methodology.
Table 4-16: Respondent answers as to whether excavation means and methods should be specified in a GBR for two project delivery methods.

<table>
<thead>
<tr>
<th>Project Delivery Method</th>
<th>Yes (%)</th>
<th>No (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design-bid-build</td>
<td>43</td>
<td>57</td>
</tr>
<tr>
<td>Design-build</td>
<td>33</td>
<td>67</td>
</tr>
</tbody>
</table>

4.3.1.8.7 Respondent Suggestions and Recommendations

The last question (No. 40) asked whether respondents had any suggestions or recommendations for improving GBRs, based on the answers provided and their experience. All the text answers from the six respondents were centered on risk allocation in the GBR. The main themes from these answers were:

- GBRs should remain as factual as possible, and only consider the issues relevant to the project;

- In a GBR the excavation means and methods should either not be specified, or limited to a construction philosophy;

- The GBR should clearly define how risk has been allocated to all project parties.

4.3.1.9 Study comparison: GBR Industry Survey to Heslop and Caruso (2013)

A similar study was conducted by Heslop and Caruso, titled “Recommendations on how Geotechnical Baseline Reports can be prepared for rock tunnel projects”, who reviewed twenty-five rock tunnel GBRs for projects located in the US, and a number of rock tunnel GBRs for projects located internationally (2013). This study was conducted to identify the current issues with GBRs and provide recommendations on how GBRs can be improved. In an attempt to verify the GBR survey results, the survey results were compared to a portion of the study results obtained by Heslop and Caruso.

Heslop and Caruso analyzed the occurrence of numerous baseline parameters in GBRs including intact and rockmass properties, rockmass classification systems, structural characteristics, in-situ stress and groundwater conditions, anticipated rockmass behaviours, and
construction considerations (2013). These results are shown in Figure 4-12. These results were compared where possible to the fourteen hard rock, soft rock, and mixed tunneling projects in the GBR Industry Survey and review (i.e. the six projects in soil ground conditions were not considered). For simplicity, the y-axis category labels are the same as those appearing in Heslop and Caruso. In Figure 4-13 it can be seen that rockmass classification systems occurred more frequently in the GBR survey, however the usage trend of these systems is the same, with RQD as the most prevalent and GSI as the least. The occurrence of ‘Failure Types’ in Heslop and Caruso was also compared to the number of rock projects which had at least one rockmass behaviour described. For both studies failure types were reported on a fairly similar percentage of projects, at approximately 60% and 70%, respectively.
Figure 4-12: Heslop and Caruso (2013) results for the occurrence of numerous baseline properties in GBRs. Realistic and unrealistic or irrelevant baselines are defined later in the paper.
Other categories which were not directly comparable in Figure 4-12 due to differences in how parameters were specified, are briefly discussed. Heslop and Caruso categories “(A) Geological Interpretation” and “(B) Intact Rock Properties” had, on average, the highest occurrence rates, whereas the category “(G) Insitu Stress” had the lowest occurrence rate (2013). This is similar to the GBR survey results. Categories “(D) Rock Jointing”, “(E) Faulting and Shear Zones”, “(F) Groundwater”, “(H) Specific Ground Risks” and “(J) Construction Considerations” could not be directly compared to the GBR survey, as these categories either appeared as part of questions asking about the most and least favorable, or most dominant rockmasses (i.e. and not the entire GBR), were not asked in the GBR survey, or there was an insufficient amount of data (Heslop and Caruso, 2013).

4.3.2 GBR Industry Survey and Review Discussion

The GBR Industry Survey and the author’s GBR review produced a large variety of interesting results and trends regarding GBR contents, risk allocation, and DSC claims. It is unfortunate that the survey had a low overall response rate. Reasons for this may include time restrictions,
inapplicable questions, confidentiality reasons, unknown answers, and lost interest. However, despite the low response rate, several trends can be observed in the data.

4.3.2.1 Contracting Methodology
Design-bid-build, design-build, lump-sum, and unit-price contracting methods were the most prevalent project delivery and financing methods within the survey, although it is noted that contracts which shift additional risk to a design-build team or Contractor, such as turnkey/EPC or PPP/DBOF, may not use a GBR, and are therefore not applicable to this survey. As the traditional, familiar contracting approach, design-bid-build remains a popular choice for Owners, while design-build projects appear to have become the prevailing alternative. For design-bid-build, unit price contracts occurred more frequently than lump sum. This is not surprising for design-bid-build projects in a competitive bidding environment, as unit price financing works to minimize the risk to both the Owner and Contractor. On design-build, lump sum and target price financing were employed more often, likely as the design-build team retains greater control over the overall project development and delivery.

4.3.2.2 Methods used to define rock and rockmass properties
Laboratory testing remains one of the most prevalent methods to define the soil and/or rock properties along the tunnel alignment, likely as it is one of the easiest and most repeatable methods. The majority of the remaining options (in-situ testing, site investigation field estimates, correlations from empirical methods, and a strength criterion) are either more difficult, costly, time consuming, and produce potentially uncertain results.

Rockmass classification systems remain a popular rock engineering design tool, with application for a wide variety of purposes. Prevailing reasons for selecting a classification system included prior experience, to obtain the preliminary support requirements, the relevance of the classification input parameters, and to determine the rockmass behaviours. Rockmass classification systems were also used to define the soil and/or rock properties, and as a means to
quantify the rockmass quality for the most and least favorable, and most dominant rockmasses along the tunnel alignment. These results indicate that prior experience (either belonging to the design Engineer, or relying on that of others) remains heavily favored for tunnel analysis and design.

With regards to the parameters used to define the most and least favorable, and the most dominant rockmasses along the tunnel alignment, nearly all the respondents selected the intact rock properties, rockmass classification systems, geology, weathering, and alteration, joint characteristics, and the groundwater conditions. This may indicate that these properties are standard in a rockmass description. The in-situ stress condition (as discussed further below) and the expected deformation magnitude were not prevalent in this description (the lack of definition of the in-situ stress condition likely impacts the definition of the expected deformation magnitude, as these properties are related). Additionally, when asked to picture the most and least favorable, and most dominant rockmasses along the tunnel alignment according to a series of rock properties, most respondents were able to rank the rock strength, stiffness, joint surface quality, degree of rockmass ‘blockiness’, rockmass permeability, groundwater inflows, and the tunnel face scale heterogeneity.

A frequently occurring theme within this questionnaire is that the in-situ stress condition is the most infrequently defined parameter for a tunnel project. In three separate sections: basic tunnel information, the properties used to define the most and least favorable, and most dominant rockmasses along the tunnel alignment, and then in the questions used to picture these rockmasses, the in-situ stress condition (magnitude and ratio), received the most blank or ‘unknown’ responses. This is likely the result of the in-situ stress being one of the most challenging parameters to define along a tunnel alignment due to its inherent variability. This may make selection of a representative value for baseline purposes difficult.
4.3.2.3 Impact on Rockmass Behaviour Definitions

An attempt was made to determine whether there was a correlation between the use of rockmass classification systems and the number of rockmass behaviours (i.e. whether classification systems were being used to predict behaviours). However, there was an insufficient amount of data to identify a trend. Of the four respondents who ranked ‘determining the rockmass behaviours’ as either the first, second, or third contributing factor in selecting a rockmass classification system, these projects reported one, two (twice), and nine rockmass behaviours.

The in-situ stress is one of the most important parameters in determining the anticipated rockmass behaviours, yet within this survey, the in-situ stress condition was the most rarely defined parameter. This is interesting, for when the rockmass behaviour types are examined, stress driven failure types occur in nearly the same frequency as gravity driven rockmass behaviours. It appears as if the effect of the in-situ stress condition on the rockmass is considered in GBRs, but the actual in-situ stress magnitude is not being baselined.

A significant number of rock properties and tools appear to be defined and used on tunnel projects; however this is not necessarily translating into rockmass behaviours. When considering the properties used to define the most and least favorable, and most dominant rockmasses along the tunnel alignment, the majority of respondents used the intact rock properties, rockmass classification systems, geology, alteration, and/or weathering characteristics, joint characteristics, and the groundwater characteristic to define these rockmasses. The in-situ stress condition was only considered on two projects. Subsequently, while the majority of respondents confirmed that these three rockmasses and their associated rockmass behaviours were included in the GBR, when the actual rockmass behaviours are questioned, approximately 70% of these projects only reported one to two rockmass behaviours. Furthermore, there is low variation in the types of reported rockmass behaviours, in that the majority of projects only considered gravity driven behaviours. Only two projects reporting a combination of gravity driven, stress driven, and/or water influenced failure modes. It is noted that this is a small data set, and it is possible that these
projects may be relatively geologically homogenous, or the in-situ stress was not considered to affect tunnel stability. However, the low occurrence of rockmass behaviours within the respondent data corresponds to the results obtained from the nine GBRs that the author reviewed.

The number of anticipated rockmass behaviours in GBRs appears to be divided into approximately two groups: one group with one to two rockmass behaviours, and the second group with four or more behaviours (Table 4-13). While only fifteen projects were considered, these two groups may represent two types of GBRs: those that are conservative, with additional risk being allocated to the Contractor by not including rockmass behaviours (e.g. similar to the 1970’s, when Contractors were required to develop their own interpretation of the provided geologic data), and those GBRs which are following the ASCE GBR guidelines, with the inclusion of construction considerations in the form of anticipated rockmass behaviours.

4.3.2.4 Differing Site Conditions Claims
For this section of questions, the limited number of responses, which was not supplemented with information from the author’s GBR review, necessitates that only data trends can be observed.

The majority of respondents indicated that while the description of the rockmass properties, behaviours, and construction considerations were adequate for the most favorable and most dominant rockmasses, they were not as effective for the least favorable rockmass. This is not surprising, as the least favorable rockmass had the highest occurrence of ‘unknown’ answers regarding the rockmass properties used to define this rockmass (Figure 4-9). Unknown rockmass parameters, especially the in-situ stress condition, makes identification of rockmass behaviours and construction issues difficult. The lack of definition of these parameters may render this rockmass unpredictable during tunnel construction, potentially resulting in a DSC claim (e.g. Figure 4-11). It is surprising that the most dominant rockmass encountered two DSC claims, as the descriptions for this rockmass were considered 100% adequate in terms of the rockmass
properties, behaviours, and construction considerations. However, as the most dominant rockmass along the tunnel alignment, it may also be the most variable.

Approximately half of the respondents reported that the GBR was an effective tool in mitigating differing site conditions claims, with approximately another fifth indicating that the GBR was somewhat effective. This low effectiveness rate could be the culmination of several observed trends observed from the preceding questions: the prevalent use of rockmass classification systems, the low reporting frequency of the in-situ stress condition, the low number of reported anticipated rockmass behaviours, and the low occurrence of excavation means and methods specifications in the GBR. Neglecting these items shifts subsurface ground condition risk to a Contractor, especially on design-bid-build projects.

The last two opinion based questions as to whether excavation means and methods should be specified in a GBR, and recommendations for how GBRs can be improved indicates that the small majority of respondents believe that the GBR should be restricted in its level of interpretation of the expected ground conditions, and furthermore, excavation means and methods should not be specified. This, coupled with the number of anticipated rockmass behaviours reported in this survey, may indicate a potential divide in industry as to how GBRs are applied on projects: those that opt for a more factual interpretation of the expected ground conditions, and those that interpret the ground conditions to obtain the anticipated rockmass behaviours and the excavation means and methods.

4.3.2.5 GBR Industry Survey and Review Conclusion
The GBR survey and review demonstrated that overall a large number of rockmass properties and tools, such as rockmass classification systems, are applied to define the rockmass properties along the tunnel alignment. Beyond this, there appears to be two opposing views as to the level of interpretation of the expected ground conditions that should be included in a GBR: those that rely on the rockmass properties and rockmass classification systems to define the anticipated ground
conditions, and those that interpret the ground conditions to obtain the anticipated rockmass behaviours and subsequently the excavation means and methods. The former level of interpretation promotes uncertainty in determining the rockmass behaviour, as physically different rockmasses can have the same rock quality rating. Once the unique controls on failure are collectively considered, these rockmasses can behave very differently, which has potential implications for the recommended rock support. This will be demonstrated in the following section.

4.4 Rockmass Classification Systems and Rockmass Behaviour Uncertainty
The expected ground conditions are one of the most important aspects of any tunneling project, as they define the excavation means and methods and have a considerable influence on the project schedule and price. Suggested guidelines from the booklet Geotechnical Baseline Reports for Underground Construction state that a GBR should include the “criteria and methodologies used for the design of ground support and ground stabilization systems, including ground loadings” and the “anticipated ground behavior in response to construction operations within each soil and rock unit” (Essex, 2007). While the anticipated rockmass behaviour is linked to the ‘excavation means and methods’ to a degree and must therefore be addressed by the Contractor, it is also a part of the “ground conditions” and should be addressed by the Owner in the GBR. However, as demonstrated in the preceding section, these two guidelines are not always considered in GBRs. In particular, the latter point is often absent, replaced with a list of material properties based on standardized testing, or with empirical classification systems such as RMR, Q, or GSI, with limited or no consideration of individual and unique controls on failure, such as the in-situ stress, brittle versus ductile yield, or the groundwater conditions. Using material properties and rockmass classification systems alone is limited in scope, as it groups physically different rock types together with the same quality rating – these are termed ‘equivalent materials’. These equivalent materials behave differently when all the individual and unique controls on failure are
considered and combined, yet in using rockmass classification systems these materials are all assigned the same rock support recommendation.

To establish the importance of considering all of the separate controls on failure and incorporating rockmass behaviours in a GBR, ‘semi-discontinuum’ Phase² finite element models were used to investigate the impact of five physically different materials with identical rockmass classification ratings on an standardized tunnel and rock support design (RocScience, 2013). The following five ‘equivalent material’ rockmass parameters and external forces were evaluated and applied directly for design using the Q system:

1. Multiple joint sets in a weak sedimentary rock under high stress;
2. Bedding and associated structure in a sedimentary rock;
3. A massive to blocky igneous rockmass under high stress;
4. Foliation and heterogeneity in metamorphic rock;
5. A highly disturbed sedimentary rockmass under low stress with groundwater.

### 4.4.1 Design Approach Using the Q System

As discussed in Chapter 2, the Q system, developed by Barton, Lien, and Lunde (1974) combines the block size, joint shear strength, and active stresses to calculate the rockmass quality with Eq. 4-1:

$$ Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} $$  \hspace{1cm} (4-1)

RQD is the Rock Quality Designation by Deere et al. (1967), $J_n$ is the number of joint sets, $J_r$ is the joint roughness, $J_a$ is the joint alteration, $J_w$ is the joint water reduction factor, and SRF is the stress reduction factor. The five ‘equivalent material’ Q parameters are shown in Table 4-17.
Table 4-17: Case specific Q parameters.

<table>
<thead>
<tr>
<th>Q Parameter</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>55</td>
<td>70</td>
<td>90</td>
<td>55</td>
<td>40</td>
</tr>
<tr>
<td>J_n</td>
<td>9</td>
<td>12</td>
<td>12</td>
<td>6</td>
<td>15</td>
</tr>
<tr>
<td>J_r</td>
<td>2</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>J_a</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>J_w</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.7</td>
</tr>
<tr>
<td>SRF</td>
<td>5</td>
<td>2.5</td>
<td>10</td>
<td>3</td>
<td>0.5</td>
</tr>
<tr>
<td>Q</td>
<td>1.2</td>
<td>1.2</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Case specific rock strength, joint strength and geometry, and groundwater and in-situ stress conditions were quantified based on the parameters presented in Table 4-17.

4.4.1.1 Intact Rock and Rockmass Strength Parameters

To obtain the intact rockmass strength parameters, the Q values were converted to GSI with the following equations by Bieniawski (1989) and Hoek et al. (1995), respectively:

\[
\text{RMR} = 9 \times \ln(Q) + 44
\]

and

\[
\text{GSI} = \text{RMR}_{89} - 5
\]

A Q value of 1.1 – 1.2 equates to a GSI rating of 40.

The intact uniaxial compressive strength and the Hoek-Brown empirical constant \( m_i \) were selected based on the tables ‘Field Estimates of Rock Strengths’ and from the \( m_i \) table, respectively (Hoek, 2007). The residual empirical rockmass constant \( m_b \) was calculated with the Hoek-Brown criterion equations (Hoek, 2007). For the massive to blocky igneous rockmass under high stress, the material was modelled with Generalized Hoek-Brown rockmass strength criterion, and rockmass strength parameters were calculated according to the “Damage Initiation and Spalling Limit” approach developed by Diederichs (2007).

The rockmass deformation modulus was quantified using the Hoek-Diederichs equation, where the intact Young’s Modulus was estimated from the Modulus Ratio (Hoek and Diederichs,
Poison’s ratio values were obtained from Read and Stacy (2009). Equivalent material intact rock strength and stiffness parameters are shown in Table 4-18.

Table 4-18: Case specific equivalent material intact rock strength parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Type</td>
<td>Limestone</td>
<td>Sandstone</td>
<td>Diorite</td>
<td>Amphibolite</td>
<td>Sandstone</td>
</tr>
<tr>
<td>UCS ($\sigma_{ci}$) MPa</td>
<td>75</td>
<td>75</td>
<td>125</td>
<td>175</td>
<td>75</td>
</tr>
<tr>
<td>$m_i$ (Hoek Brown constant)</td>
<td>7</td>
<td>17</td>
<td>25</td>
<td>26</td>
<td>17</td>
</tr>
<tr>
<td>$E_i$ (Young’s Modulus)</td>
<td>41300</td>
<td>52500</td>
<td>40600</td>
<td>78800</td>
<td>20600</td>
</tr>
<tr>
<td>$\nu$ (Poisson’s Ratio)</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

To avoid doubly penalizing the rock, intact rock with a GSI rating of 100 was placed in between the joints. All the materials were modelled as isotropic, elastic-brittle. Material dilation was not considered.

4.4.1.2 Joint Strength and Geometry Parameters

Joint set spacing for Phase 2 was calculated using the Priest and Hudson equation correlating RQD and joint frequency, where $\lambda$ is the apparent joint frequency (1976):

$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1)$$

Random joint sets were ignored for numerical modeling simplicity.

Joint strength was quantified with the Barton and Bandis slip criterion (1990). The joint compressive strength ($JCS$), joint roughness coefficient ($JRC$), and the residual frictional strength ($\phi_r$) were selected based on the $J_s$ and $J_a$.

Case specific joint set geometry and strength parameters are as follows:

1. Three, 0.3 m spaced, “smooth, undulating” joint sets with “slightly altered joint walls [and] non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.” had $JCS$ 47 MPa, $JRC$ 8, and $\phi_r$ 30° (Barton et al., 1974, Pitts and Diederichs, 2011).

2. Three “rough or irregular, planar” joint sets with “silty- or sandy clay coatings, small clay fraction (non-softening)” had $JCS$ 3 MPa, $JRC$ 2.3, and $\phi_r$ 25° (Barton et al., 1974, Pitts and Diederichs, 2011). The sedimentary sequence was composed of 0.1 – 1 m
thick beds with a 0.2 – 2 m spaced vertical cross joints. An inclined out-of-plane cross-set with 5 m spacing comprised the third set.

3. Three, 10 m long joint sets were spaced at 0.9, 1, and 5 m, with persistence varying from 0.8 to 0.9. The “rough or irregular, planar” joints with “unaltered joint walls” had JCS 125 MPa, JRC 20, and $\phi_r$ 35° (Barton et al., 1974, Pitts and Diederichs, 2011).

4. The metamorphic foliation was composed of a 0.09 – 0.11 m vertically spaced cleavage plane with a secondary near orthogonal 0.4 m spaced inclined cleavage plane. Both cleavage planes had JCS 5 MPa, JRC 2.3, and $\phi_r$ 12°, corresponding to “slickenslided, undulating” joint roughness and “softening or low friction clay mineral coatings, i.e. …chlorite, graphite” (Barton et al., 1974, Pitts and Diederichs, 2011).

5. Four 0.4 m spaced joint sets represented the “sugar cube” rockmass (Barton et al., 1974). The “smooth, planar” joints had “silty or sandy clay coatings, small clay-fraction (non-softening)”, corresponding to JCS 10 MPa, JRC 0.9, and $\phi_r$ 25° (Barton et al., 1974, Pitts and Diederichs, 2011).

4.4.1.3 Groundwater and In-situ Stress Conditions

Based on the $J_w$ and SRF parameters, the case specific groundwater and in-situ stress conditions were as follows:

1. A 700 m deep tunnel with in-situ stress ratio $K$ ($\sigma_h / \sigma_v$) of 1 represented the “squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures” SRF value of 5 (Barton et al., 1974).

2. A 20 m deep tunnel represented the “low stress, near surface” stress conditions. A locked in horizontal stress was represented with a $K$ of 1.5 (Barton et al., 1974).

3. The “moderate slabbing after > 1 hour in massive rock” SRF value of 10 was represented with a 1,300 m deep tunnel with $K$ of 2 (Barton and Grimstad, 1994).
4. A 600 m deep tunnel with anisotropic in-situ stress condition (K of 1.7) (Hutchinson and Diederichs, 1996). $\sigma_1$ was oriented 110° to the horizontal.

5. A 300 m deep tunnel with K of 0.75, $\sigma_1$ oriented 90° to the horizontal, and a 25 m water pressure head represented the “high stress” and “medium inflow or water pressure” conditions, respectively (Barton et al., 1974, Hutchinson and Diederichs, 1996).

Figure 4-14 shows the Phase$^2$ model geometry for the five equivalent materials.
Figure 4-14: Structural rockmass model geometry. From 1 to 5: weak limestone in hydrostatic stress conditions, bedded sedimentary structure, massive to blocky diorite rockmass, heterogeneous foliated amphibolite, and highly disturbed sandstone rockmass.
4.4.1.4 Tunnel and Rock Support Design

Two excavation methodologies were selected to determine the impact of each equivalent material on the tunnel and rock support design: a 10 m diameter TBM tunnel and a 10 m span drill and blast arch tunnel. Using the improved longitudinal displacement profiles for convergence-confinement analysis approach developed by Vlachopoulos and Diederichs (2009), in the circular tunnel, rock support was installed behind the cutterhead, at 4 m from the face, while in the arch tunnel, assumed 4 m long drill and blast rounds permitted support to be installed on average 2 m from the face. Good quality blasting was assumed for the arch tunnel analyses (D = 0) (Hoek et al., 2002). An Excavation Support Ratio of ‘1’ was selected for “major road and railway tunnels” (Barton et al., 1974).

In accordance with the poor rock mass quality, tunnel geometry, and the Excavation Support Ratio, permanent rock support consisted of 3 m long bolts spaced at 1.7 m in plane, with 90 mm of fiber-reinforced shotcrete (Figure 4-15) (Barton and Grimstad, 1994).
Figure 4-15: Q rock support is shown with the black dot (from Palmstrom and Broch, 2006, modified from Grimstad and Barton, 1993).

For application in Phase², plastic tieback bolts were selected, as this bolt type best represents a grouted dowel, and a grout bond strength is able to specified (RocScience, 2013). The 19 mm diameter bolts had a 0.1 MN tensile strength, and a 100% grouted bond length with a bond strength of 50 MN/m. The unreinforced elastic liner had a compressive strength of 40 MPa, and a tensile strength of 6 MPa to account for the steel fibers.

4.4.1.5 Phase² Modelling Mechanics

For all the models plane strain conditions were assumed. A graded three noded triangular mesh was applied with the number of elements for the five cases varying from 40,000 to 220,000 elements. This higher number of elements corresponds to the cases which had a tight joint spacing (cases one, four, and five). Three stages were used in all the models: an equilibrium stage (no excavation), an initial excavation stage where a distributed load was applied with the
Field Stress Vector approach in Phase² (this distributed load was installed in accordance with the support installation location in the ground based on the support reaction curve in RocSupport), and a final excavation stage, with no applied load (RocScience, 2013). To reduce model run times due to the computational limitations owing to the tight joint spacing, an equivalent continuum material was placed 1.5 – 2.5 tunnel diameters away from the tunnel boundary (the distance away was dependent on the joint spacing).

4.4.2 Unsupported Case Study Model Results
The numerical modelling results for each of the five cases and two excavation methodologies are presented in Figure 4-16. Actual examples of all these failure modes are presented in Figure 4-17. These results are discussed in this section.
Figure 4-16: Top to bottom: squeezing, structural failure, spalling/slabbing, buckling and shearing, and flowing ground in TBM and drill and blast tunnels. Left: total displacement, right: element and joint yield. $M =$ deformation magnifier. Clear mesh is for cases where significant deformation has occurred, prompting material removal (e.g. scaling).
4.4.2.1 Case 1: Multiple joint sets in a weak sedimentary rock under high stress

Failure in disintegrated or very blocky rockmasses under high stress is typically characterized by large deformations into a tunnel, and shear zones develop around the tunnel in the overstressed ductile material (Marinos, 2012). This is known as squeezing. In these models, the three joint sets with tight 0.3 m spacing entails that the joints make up the ‘intact rock’. Shearing occurs along the joint set planes, and not through what would normally be considered ‘intact rock’ (the rock material in between the joints).

According to Hoek and Marinos, the rockmass strength to in-situ stress ratio of 0.13 for these tunnels predicts ‘very severe squeezing’ to ‘extreme squeezing’ to occur, with 5 – 10%, or greater than 10% strains occurring, respectively (2000). Joint shear failure around the excavation
causes nearly 10% and 20% strain to occur in the circular and arch tunnels, respectively. While an isotropic in-situ stress condition was applied, this very severe squeezing is slightly greater in the horizontal than vertical direction in both tunnel shapes due to the near parallel orientation of a joint set to the vertically oriented stress.

The influence of the excavation methodology on an overstressed rockmass is clearly demonstrated in the contrasting circular and arch tunnel results (Figure 4-16). Due to its shape, the rockmass surrounding the circular tunnel is relatively confined, and therefore, restricted in the amount of total deformation that can occur. In the arch tunnel the tunnel corners create an apparent system stiffness, which, in combination with the confining stress, creates large ‘blocks’ of rock in the tunnel corners which serve as stiff abutments for the arch crown and sidewalls. This is a numerical modelling issue only, and not representative of actual conditions. Additionally the greater induced stress in the arch tunnel excavation results in a greater amount of deformation.

4.4.2.2 Case 2: Bedding and associated structure in a sedimentary rock
Gravity loading in shallow stratified and cross-jointed sedimentary rock tunnels which exceeds the shear strength of the discontinuities and any clamping effect due to the in-situ stress (Diederichs and Kaiser, 1999) causes wedges or blocks to fall or slide out upon excavation.

In both the circular and the arch tunnels, gravity driven structural failure of the bedding planes with the cross-joints acting as release planes above the crown, combined with sliding of a non-persistent joint located left of the left sidewall permitted structural failure to propagate to surface (Figure 4-16). This non-persistent joint demonstrates the importance of structural geometry, as significant additional failure occurs in the sidewalls with the presence of this joint, which otherwise would not have likely occurred had this joint been not so proximate to the tunnels. Structural failure above the tunnel crown is unrestricted as the arch effect is not developed due to insufficient horizontal confinement (Diederichs and Kaiser, 1999).
A similar amount of structural failure occurs in both tunnels; however the arch tunnel shape provides greater unsupported structural stability than the circular tunnel shape. The vertical walls undermine fewer bedding planes than in the circular excavation, which undermines all the bedding planes intersecting the excavation above the springline.

4.4.2.3 Case 3: A massive to blocky igneous rockmass under high stress
Spalling is a hard rock brittle failure mechanism which can occur in the walls of deep tunnels. Extensile cracks form when the induced stress surrounding an excavation exceeds the intact rock crack initiation (CI) threshold (Diederichs, 2007). Progressive overbreak develops when extensile cracks coalesce, forming thin slabs parallel to the excavation boundary (Diederichs, 2007). The overbreak self-stabilizes once the notch geometry has sufficiently changed to provide enough confinement at the notch tip (Martin, 1993).

In both tunnel excavation methodologies, the induced stress concentrations in the crown and invert cause intact material spalling and structural failure. The effects of structure are demonstrated, as stress relaxation allows wedges to form in the sidewalls, and spalling is structurally controlled by the nearly vertically oriented semi-persistent joint set (Figure 4-16). This weaker joint set plane acts as a preferential failure location, halting further intact material failure.

Less rockmass yield occurs in the TBM driven tunnel as the circular tunnel shape concentrates the confining stress ($\sigma_3$) above and below the tunnel, whereas in the arch tunnel, the crown and flat invert deflects this confining stress further away from the tunnel. Accordingly, the circular tunnel shape is best able to mitigate the spalling failure mechanism.

4.4.2.4 Case 4: Foliation and heterogeneity in metamorphic rock
In foliated metamorphic rocks under high stress, the cleavage planes act as preferential weakness planes, permitting shearing and potentially buckling under stress (Schubert and Goricki, 2004).
Similar to the squeezing case, the very tight joint spacing entails that the joint sets comprise the ‘intact rock’, with yield mainly occurring along these planes.

In both excavation methodologies, the perpendicular orientation of $\sigma_1$ to the horizontally inclined cleavage plane causes nearly 0.2 m of shear deformation into the tunnels along the sidewalls, and buckling occurs along this plane in the crown and invert. Very little deformation occurs on the vertically oriented cleavage plane. Buckling is more pronounced in the arch tunnel as the sub-horizontal cleavage plane is oriented nearly parallel to the tunnel crown and invert, allowing this plane to delaminate under the induced stress (Figure 4-16) (Hutchinson and Diederichs, 1996).

4.4.2.5 Case 5: A highly disturbed sedimentary rockmass under low stress

Ravelling occurs in intensely fractured or completely disintegrated rockmasses with minimal frictional or cohesive material and/or discontinuity strength (Schubert and Goricki, 2004, Marinos, 2012). The loss of confinement upon excavation allows for unrestricted progressive failure to occur above the tunnel crown and along the sidewalls (Marinos, 2012). Flowing ground occurs when groundwater is present throughout the disintegrated rockmass material (Marinos, 2012, Palmstrom and Stille 2007).

In both tunnel shapes, the combined effect of the induced stress state, the 25 m groundwater pressure head, the low joint strength, the structural geometry and the tunnel shape causes deformation to propagate uncontrollably upwards along the sidewalls and crown. Similar to the squeezing case, the arch tunnel corners create an apparent system stiffness. This results in less total deformation occurring in the arch tunnel than in the circular tunnel.

4.4.3 Supported Case Study Model Results

The supported numerical modelling results from the five cases are presented in Figure 4-18. Only the better performing excavation methodology (TBM versus drill and blast) is shown. Rock support performance results for all ten supported tunnels (the percent of failed bolts, percent of
liner exceeding capacity, maximum bolt axial load, and max supported displacement) are presented in Figure 4-19. These results are discussed as follows.
Figure 4-18: From top to bottom: selected supported tunnels undergoing squeezing, block failure, spalling/slabbing, buckling and shearing, and flowing ground, showing total displacement and liner bending moment plots. The factor of safety envelopes are 1 and 1.5.
Figure 4-19: Rock support performance for all ten supported tunnels: the percent of failed bolts and liner exceeding capacity, the maximum axial load in the bolts, and the maximum supported displacement.

4.4.3.1 Case 1: Multiple joint sets in a weak sedimentary rock under high stress

For this case the recommended rock support is inadequate for mitigating the squeezing rockmass behaviour in both tunnels, in terms of the rock support type, capacity, and the overall installation timing. In the TBM driven and drill and blast tunnels, approximately 40% and 95% of the bolts fail respectively, while the liner capacity is near completely exceeded in both cases (Figure 4-19). The rock support performance difference between the TBM and drill and blast arch tunnels is related to the tunnel shape – not the installation timing, as the drill and blast arch tunnel rock support performed more poorly.
The arch tunnel shape combined with the apparent system stiffness in the tunnel corners acts to separate the rockmass into four separate units, with each unit acting on one of the four tunnel walls. This is in contrast to the TBM tunnel, where the rockmass acts as one continuous unit around the tunnel. In the drill and blast tunnel, the tunnel shape combined with the liner support capacity is inadequate to prevent failure (Figure 4-19). Large deformations generate significant liner bending moments along the straight walls, with smaller moments along the arched crown. Near zero moments occur in the arch tunnel corners where the apparent system stiffness occurs. Almost all the bolts fail due to the uneven deformation magnitude occurring around the tunnel, putting the bolts in tension. In the TBM tunnel nearly 80% of the liner fails due to large bending moments generated in the sidewalls. However, the circular tunnel shape allows the rockmass to deform as a continuum, thus preventing large differential deformations from occurring, and less bolts fail.

In contrast, the support recommended by Hoek and Marinos for very severe to extreme squeezing conditions includes the use of forepoles, face reinforcement, and yielding support (2000).

4.4.3.2 Case 2: Bedding and associated structure in a sedimentary rock
In this case the recommended support is nearly completely sufficient to mitigate the gravity driven structural failure in both tunnels (Figure 4-19). In the drill and blast tunnel the liner capacity is exceeded in the left tunnel sidewall where the non-persistent joint set intersects this wall. However, this can be easily mitigated with additional bolts and/or shotcrete. The arch tunnel support is shown in Figure 4-18. With regards to the TBM driven tunnel, in actual conditions, the greater time lag between excavation and rock support installation would allow for greater rock fallout to occur due to TBM vibration and time.
4.4.3.3 Case 3: A massive to blocky igneous rockmass under high stress

For this case the type of recommended support is mechanically correct as adding confinement mitigates the amount of extensile cracking which is able to occur. However, the bolt and liner capacity are inadequate. In the circular tunnel, approximately 80% of the bolts failed in tension along failed joints (the majority of which were located in the crown and invert), and approximately 30% of the liner yielded (Figure 4-19). In the drill and blast tunnel as support was installed 2 m closer to the face approximately 60% of the bolts failed (also mostly in the crown and invert) and 40% of the liner yielded. Less bolt failure occurs in the arch tunnel as the arch tunnel has a greater number of bolts located in the relaxed sidewalls. In both tunnels bolt failure was caused by intact material failure (spalling) and/or joint failure. In the TBM tunnel the compressive strength of the liner was able to be utilized, while in the arch tunnel significant liner bending moments were generated in the tunnel corners.

While a large number of hard rock tunnels under high stress were incorporated as case studies for the development of the Q system, the recommended support is not always adequate when the installation timing is considered, as demonstrated here.

4.4.3.4 Case 4: Foliation and heterogeneity in metamorphic rock

In the foliated metamorphic rock under high stress, the buckling and shearing rockmass behaviour renders the recommended support completely inadequate in both tunnels. Buckling and shearing along the cleavage planes (in the form of normal and shear displacement along the joint set planes) cause nearly all the rockbolts to fail in tension, and large bending moments are generated in the shotcrete liner, causing failure. The rockmass and the rock support failed similarly irrespective of the tunnel shape and the support installation timing, with a similar supported deformation magnitude (Figure 4-19). Therefore, the critical design item is not the excavation shape, but to design appropriate rock support (including the rock support type, capacity, installation timing, and sequencing) to mitigate the rockmass failure mechanism.
4.4.3.5 Case 5: A highly disturbed sedimentary rockmass under low stress

In the flowing ground conditions the recommended rock support is largely ineffective at mitigating the failure mechanism. The system behaviour in this rockmass is similar to that of the squeezing rockmass (excluding the greater in-situ stress condition in the squeezing rockmass, and the groundwater pressure in this case), in that both rockmasses contain similar joint structure and the generated apparent system stiffness in the arch tunnel corners. Similar to the squeezing rockmass system behaviour, the rock support type, capacity, and the overall installation timing are inadequate (Figure 4-19). However, as the in-situ stress condition is less (700 m depth in the squeezing rockmass versus 300 m for this rockmass), in the circular tunnel the compressive strength of the liner is utilized, which prevents large deformations from occurring and fewer failed bolts.

4.4.4 Discussion

These five equivalent material, behaviorally different rockmass models demonstrate that in the majority of cases, the recommended support for a poor rock tunnel proves inadequate once the individual mechanistic components affecting the rockmass behaviour are collectively considered. The in-situ stress condition, groundwater pressure, structural characteristics, and the material properties all affect the rockmass behaviour. Additionally, the excavation methodology (tunnel geometry, and the rock support capacity and installation timing) also has a significant effect on the rockmass behaviour. For example, for the squeezing rockmass, there was a significant difference in the amount of scaled material between the TBM and the drill and blast tunnel.

Additionally, the recommended support was sufficient in the blocky sedimentary rockmass, yet inadequate in the squeezing, buckling, and flowing ground rockmasses. Obtaining a tunnel and rock support design based on a rockmass classification quality rating alone is limited in scope, as it neglects consideration of the failure mechanism and the overall system behaviour. This is not only true for the Q system, but for any experience based classification system which directly
relates the evaluation of a limited number of rockmass characteristics and the external forces to a rock support design. From the other perspective, it should be noted that users of rockmass classification systems should be aware of the case histories from which the classification systems were developed to prevent misuse and misapplication.

The SRF parameter may be considered as a means to incorporate rockmass behaviour in the Q system with four categories: weakness zones intersecting the excavation, competent rock with stress problems, squeezing rock, and swelling rock (Barton and Grimstad, 1994). However, selecting a SRF value requires an assumption of the failure mechanism and/or knowledge and certainty of the intact strength, tensile strength, and the in-situ stress condition, which, as the GBR Industry Survey results has shown for this latter parameter, appears uncommon.

Uncertainty in any of these parameters can cause significant variation in the probable rockmass behaviours and resulting rock support recommendation. Furthermore, complex rockmass behaviours such as squeezing, spalling, or swelling, cannot be characterized by just the intact rock strength and the in-situ stress conditions, rather special analysis and design considerations are required for these difficult behaviours to ensure the rockmass is properly supported, as demonstrated here (Palmstrom and Broch, 2006).

4.4.4.1 Implications for the Geotechnical Baseline Reports
The GBR is often the only interpretive document of the expected subsurface ground conditions and platform in which to explain the basis for design and construction on a tunnel project. GBRs which use rockmass classification systems alone to describe the expected ground conditions promote rockmass behavioural uncertainty, as multiple different interpretations are possible for the same rock quality rating. If the Contractor or design-build team misinterprets the Owner supplied rockmass data, this will either lead to underbidding, due to the neglect of critical failure modes, or excessive conservatism and overbidding due to inadequate consideration of the rockmass behaviour and the classification ratings.
4.4.4.2 Implications for the Differing Site Conditions Clause

One of the other primary purposes of the GBR is to administer the DSC clause. During tunnel construction, if an adverse condition is encountered, the subjective rockmass classification system ratings combined with the lack of information about the unique controls on failure in the GBR will create difficulties in assessing the validity of a differing site conditions claim. As demonstrated in this study, the classification systems rating can lead to several significantly different interpretations of the expected ground conditions. An Owner could potentially lose a DSC claim if a Contractor can demonstrate a plausible interpretation of the ground conditions based on the rockmass classification systems ratings.

4.5 Conclusion

The GBR Industry Survey was developed in order to understand the current status and effectiveness of GBRs in the tunneling industry by those that use them: Owners, Engineers, Contractors, and Academics. While there was a limited amount of data, the survey produced some interesting trends.

Along a tunnel alignment popular tools and methods to define the various rockmasses include laboratory testing, in-situ testing, site investigation field estimates, and correlations from empirical methods, such as classification systems. These tools are used to develop a standard rockmass description which includes the intact material properties, application of a rockmass classification system, the geology, alteration, and weathering characteristics, structural characteristics, and the groundwater characteristics. With the exception of the in-situ stress condition, nearly all the different controls on rockmass behaviour are being defined for the various rockmasses along the tunnel alignment, however these rockmass descriptions are not necessarily being translated into rockmass behaviours. Approximately two thirds of the GBRs consulted for this survey reported one to two rockmass behaviours, with the remaining third described four to nine behaviours. This may represent two groups of GBRs: those that are
conservative, requiring the Contractor to interpret the supplied geologic data, and those that are following the ASCE GBR guidelines (Essex, 2007). This trend was verified by an independent study by Heslop and Caruso (2013), who also determined that rockmass behaviours were reported in a similar percentage of projects.

The current trend of defining rockmasses in a GBR with just the intact material properties and rockmass classification system ratings promotes uncertainty in determining the rockmass behaviours. As demonstrated in the second half of this chapter, defining a rockmass based on these two items is limited in scope, as physically different rockmasses can have the same rock quality rating, yet behave very differently once all the individual controls on the rockmass behaviour are collectively considered. Five physically different rockmasses were analyzed with the Q System to determine the effect of a standardized design approach on these rockmasses. These five equivalent materials resulted in squeezing, wedge failure, spalling, buckling and shearing, and flowing ground. The effect of the excavation methodology with the recommended support design was also investigated, with the majority of the rockmass behaviours proving incompatible with regards to rock support type, capacity, and installation timing.

Inadequate rockmass descriptions in a GBR can possibly lead to DSC claims, as observed in the GBR survey. In particular, approximately half of the respondents in the GBR survey indicated that the GBR was somewhat effective, or ineffective, in mitigating differing site conditions claims. This low effectiveness rate could be the conclusion of numerous trends, including the prevalent use of rockmass classification systems, the low reporting frequency of the in-situ stress condition, the low number and variance of described rockmass behaviours and the low occurrence of excavation means and methods prescriptions in the GBR. Neglect of these items shifts subsurface ground conditions risk to a Contractor, particularly on design-bid-build projects.
Regardless of the contracting approach, in a GBR all of the unique controls on the failure mode and the failure mode itself should be included as part of the rockmass description.

Rockmass behaviours should be considered by the Owners as part of the ground conditions, and by the Contractors as part of their excavation means and methods determination. Including the rockmass behaviours in a GBR may also prevent or aid in the resolution of a DSC claim. In this regard, a new methodology for predicting and quantifying rockmass behaviours along a tunnel alignment is presented and discussed in the following chapter.
Chapter 5

Quantifying Rockmass Behavioural Uncertainty for Geotechnical Baseline Reports

5.1 Introduction

The expected ground conditions in GBRs are one of the most important aspects of any tunneling project, however, geologic uncertainty creates difficulty in characterizing the rockmass, assessing the rockmass behaviours, and determining the appropriate excavation methodology. Empirical rockmass classification systems are popular in this regard as these experience based systems classify certain rockmass characteristics to obtain a rock quality rating, which is then used to obtain a rock support design. However, as demonstrated in Chapter 4, the use of empirical rockmass classification systems to describe the expected rockmass behaviours in GBRs promotes rockmass behavioural uncertainty as rockmass classification systems create ‘equivalent materials’: physically different rockmasses with the same rock quality rating, which behave differently once the unique controls on behaviour are considered.

In this chapter a new tool which utilizes geologic uncertainty to predict and quantify rockmass behaviours for GBRs will be presented. This approach predicts a range of possible rockmass behaviours along a tunnel alignment as a function of the uncertainty of three parameters which significantly control rockmass behaviour: the intact strength, the rockmass structure, and the in-situ stress conditions. Once the potential rockmass behaviours have been determined, a simple procedure is used to assess the frequency of occurrence of rockmass behaviours as a function of uncertainty of the parameters within a tunnel domain. This approach relies on

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2 Included in part in the conference papers by van der Pouw Kraan and Diederichs (2014 a, b), and van der Pouw Kraan et al. (2014).
5.2 Review: Discontinuity Uncertainty and Numerical Modelling Approaches

Rockmass behaviour in jointed rockmasses is dominated by the geometrical, physical and mechanical properties of the structural discontinuities (i.e. joints): length, spacing, persistence, aperture, shear strength, and the normal and shear stiffness. The overall rockmass behaviour depends on how these interrelated components interact. Additionally, laboratory testing of joint strength and stiffness has demonstrated complex joint behaviour, such as nonlinear joint closure under increasing normal stress and hysteretic recovery, nonlinear relationships between the shear strength and normal stress, and progressive damage of joints under shear (Bandis et al., 1983, Barton and Choubey, 1977, Cundall and Hart, 1984). However, in numerical modelling of jointed rockmasses, spatial variability across individual joints and across joint sets challenges the selection of representative properties, and numerical modelling tools require simplifying assumptions in their implementation of discontinuities. These simplifying assumptions may lead to rockmass behavioural uncertainty, not only in identifying the failure mode, but also the failure mode severity.

5.2.1 Numerical Modelling of Rockmass Behaviours

All the numerical models generated for this research contain structural discontinuities, or joints. There are a few kinematic and numerical modelling software packages which model discontinuity behaviour around underground excavations, including the kinematic program UNWEDGE, the finite element method program Phase² (both from RocScience, 2014), and the distinct element method program Universal Distinct Element Code (UDEC) from Itasca (2014). UNWEDGE and UDEC are both specialized software programs, in that their modelling capabilities include wedge failure, and discontinuous rockmass behaviours, respectively. In comparison to these two programs, Phase² has greater versatility in its ability to model continuum and ‘semi-
discontinuum’ rockmass behaviours, in that it can manage material and discontinuity heterogeneity and anisotropy and complex boundary conditions (Jing, 2003). All the numerical modelling work was completed with Phase², Version 8.012 to 8.02 (RocScience, 2013, RocScience, 2014).

In modelling discontinuities, the FEM and Phase² contain limitations and assumptions which are not entirely representative of actual joint behaviour. As the FEM is a continuum method, the mesh cannot separate. Consequently, not all the structural mechanics are able to be accurately captured. Phase² excludes joint aperture and dilation, and uses a constant normal and shear joint stiffness (RocScience, 2014). However, for the purposes of this research in predicting potential rockmass behaviours, the flexibility of the FEM and Phase² outweighs these limitations. The following sections discuss how joint stiffness, strength, and geometry are implemented and estimated in the FEM and Phase².

5.2.1.1 Joint Stiffness Implementation in the FEM

One of the significant advances in the development of the FEM for modelling of blocky rockmasses was the development of a joint element. Prior to this, modelling of a rockmass with discontinuities involved decreasing the strength and elastic moduli of the intact material, however the rockmass was still modelled as a continuum (Goodman et al., 1968). The development of a joint element (commonly referred to as the ‘Goodman joint element’) for the FEM allowed for discontinuities to be directly incorporated into the continuum mesh (Jing, 2003). In developing this joint element, Goodman et al. considered the following joint characteristics (1968):

- Joints are planar in nature.
- Joints have high resistance in compression, and even deform under normal loads, but have little or no resistance in tension.
- At low normal stresses the shear strength is governed by the frictional strength of the asperities, as blocks ride over the asperities. At high normal stress the shear strength is
governed by the frictional and cohesive strength of the asperities, with shearing occurring through the asperities.

- Joint shear strength may be represented by either a linear, or bi-linear, Mohr failure envelope.
- Shear displacements can occur before the yield shear stress is reached.

The Goodman joint element has length ‘L’, zero width, and has four nodes in two pairs (two on the top, and two on the bottom) (Figure 5-1) (Goodman et al., 1968). The elastic constitutive model for this element has two constant joint stiffness parameters: the normal (K_n) and shear (K_s) stiffness per unit length.

![Figure 5-1: Goodman’s joint element. ‘n’ and ‘s’ are the normal and shear directions, respectively, and i, j, k, and l are the four nodes (from Jing, 2003).](image)

Goodman et al.’s derivation of the joint element stiffness matrix is as follows (1968):

1. Calculate the relative shear and normal displacement of the top element section to the bottom section (Eq. 5-1):

   \[
   \bar{w} = \begin{bmatrix}
   w_{s_{\text{top}}} - w_{s_{\text{bottom}}} \\
   w_{n_{\text{top}}} - w_{n_{\text{bottom}}}
   \end{bmatrix}
   \]
   \hspace{1cm} (5-1)

2. Relate the applied forces (P) to the joint shear and normal stiffness values (k) and the relative displacements (w):

   \[
   \begin{bmatrix}
   P_s \\
   P_n
   \end{bmatrix} = \begin{bmatrix}
   K_s & 0 \\
   0 & K_n
   \end{bmatrix} \begin{bmatrix}
   w_{s_{\text{top}}} - w_{s_{\text{bottom}}} \\
   w_{n_{\text{top}}} - w_{n_{\text{bottom}}}
   \end{bmatrix}
   \]
   \hspace{1cm} (5-2)

3. The stored energy (\phi) over the element length L is calculated as a function of the applied forces (P) and the relative displacements (w):

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ϕ = \frac{1}{2} \int_{-L/2}^{L/2} (w)^T \begin{bmatrix} K_s & 0 \\ 0 & K_n \end{bmatrix} (w) \, dx \quad (5-3)

4. Substitute the relative displacements (w) in Eq. (5-3) for the displacements occurring at each of the four nodes (u), evaluate $K_n$ and $K_s$ at each of the four nodes (producing the matrix (K)), and integrate over the element length, L, to obtain (Eq. 5-4):

ϕ = \frac{1}{2} L (u)^T (K)(u) \quad (5-4)

Eq. 5-4 is then incorporated into the overall global stiffness matrix, $K_u = f$ (Jing, 2003).

As the Goodman joint element was developed for the FEM, these joint elements cannot be broken, and joint elements undergo similar displacements as their neighbouring elements (Jing, 2003). Phase² employs the Goodman joint element to represent the mechanical behaviour of discontinuities in a rockmass.

5.2.1.2 Estimating Joint Stiffness

As previously discussed in Section 5.2.1.1, Phase² employs a constant normal and shear stiffness for joint elements. Joint normal and shear stiffness can be measured from laboratory testing, or estimated from empirical relationships. In triaxial or direct shear tests the normal or shear displacement is measured in relation to the applied normal or shear stress, respectively.

Barton conducted a series of laboratory tests to analyze joint deformation, in particular, joint stiffness (1972). One result of this work was the development of an empirical relationship relating the joint normal stiffness $K_n$ as a function of the intact Young’s Modulus ($E_i$), the rockmass modulus ($E_{rm}$), and the spacing of the joint set (L) (Eq. 5-5):

$$\frac{E_{rm}}{E_i} = \frac{(K_n)(L)}{(K_n)(L) + E_i} \quad (5-5)$$

A similar equation was developed by Karzulovic and Read (2009) for estimating the joint shear stiffness, which uses the intact shear modulus ($G_i$) and the rockmass shear modulus ($G_{rm}$) and the joint set spacing:

$$K_s = \frac{(G_{rm})(G_i)}{(L)(G_i - G_{rm})} \quad (5-6)$$
In 1975 Kulhawy conducted a literature review of rock and joint deformation behaviour which comprised of a summary of laboratory rock strength and stiffness testing results, and current rock and discontinuity strength and stiffness relationships. From this work Kulhawy proposed an empirical relationship relating the joint normal stiffness to the joint shear stiffness with the Poisson’s ratio ($\nu$):

$$K_s = \frac{K_n}{2(1+\nu)} \quad (5-7)$$

5.2.1.3 Joint Strength
There are four joint strength criterions in Phase²: elastic, Mohr-Coulomb, Barton-Bandis, and Geosynthetic Hyperbolic (RocScience, 2014). Elastic and Geosynthetic Hyperbolic models were not considered in this research, as the former cannot slip, and the latter is applicable for modelling the interface between geosynthetics and soil (RocScience, 2014).

The linear Mohr-Coulomb failure envelope was adopted from soil mechanics for use in rock mechanics (Hoek, 2007). In rock mechanics cohesion is considered a component of joint surface roughness (Hoek, 2007). The joint shear strength ($\tau$) is related to the normal stress ($\sigma_n$) with the joint cohesion ($c$) and friction ($\phi$), as shown in Eq. (5-8):

$$\tau = c + \sigma_n \tan \phi \quad (5-8)$$

The Mohr-Coulomb model is the most widely applied joint failure criterion in practice (Read and Stacey, 2009).

Subsequent laboratory testing by Barton and Choubey demonstrated that the failure envelope for joint surfaces is curved (1977). In direct shear tests at low normal stress values, asperities on the joint surface override each other (increasing $\phi$ and causing dilation), whereas at high values of normal stress, the asperities shear off, and $\phi$ approaches the frictional strength of the intact rock material (Hoek, 2007). The Barton-Bandis shear strength criterion is an empirical nonlinear relationship where the joint shear strength is related to the normal stress with the joint roughness coefficient (JRC), the joint wall compressive strength (JCS), and the residual friction
angle ($\phi_r$), as shown in Eq. (5-9) (Barton, 1973, Barton and Choubey, 1977, Barton and Bandis, 1990):

$$
\tau = \sigma_n \tan(JRC \ast \log_{10} \frac{JCS}{\sigma_n} + \phi_r)
$$

(5-9)

JRC, JCS, and $\phi_r$ can be obtained from laboratory testing and/or in the field with visual estimates or index tests (e.g. JRC roughness profiles, Schmidt Hammer tests) of drill core or structural outcrop mapping (Barton and Choubey, 1977). These parameters are independent of the normal stress (Barton and Choubey, 1977).

5.2.1.4 Joint Geometry

Rockmass discontinuities can follow a certain pattern (e.g. cross-bedding in sedimentary rockmasses, cross jointing in igneous rockmasses), be random, or occur as a combination of these. Two discontinuity implementation options are available in Phase2: joint networks or individual units. Joint network types are: parallel deterministic, parallel statistical, cross jointed, Baecher, Veneziano, and Voronoi (RocScience, 2014). For this work the parallel deterministic and parallel statistical joint networks were employed. The parallel deterministic network requires the joint orientation, spacing, length, and persistence, while the statistical network requires a deterministic input for the joint orientation, and statistical distributions for the joint spacing, length, and persistence (RocScience, 2014). Joint networks are generated through random sampling of the statistical distributions of the defined joint network with a random number generator and an initial seed value (RocScience, 2014). This initial seed value can be changed to alter the appearance or location of joints (RocScience, 2014). Random joints can be individually incorporated by the user.

One advantage of the continuum mesh is that joints can truncate within blocks, which is unlike other commercially available software programs, in which joints are truncated at block boundaries (i.e. UDEC) (Itasca, 2014).
5.2.1.5 Initial Joint Deformation

The ‘initial joint deformation’ option in Phase2 allows the user to define how the joints will initially deform with regards to the applied far field stresses. If this parameter is enabled then the joints will deform in response to both the far field stresses and the induced stresses from an excavation, whereas disabling this option will permit the joints to only deform with regards to the excavation induced stresses (RocScience, 2014). For all the models this option has been disabled as all the joints are assumed to have formed prior to the tunnel excavation and are in equilibrium with the far field stresses.

5.3 Parametric Study of Discontinuities in Phase2

In numerical modelling of rockmass behaviours, care must not only be given to the selection of intact rock and discontinuity parameters, but also the numerical modelling method. To this effect, a parametric study was conducted with Ms. Jennifer Day where the mechanical capabilities of two numerical modelling software programs were compared: that of Phase2 and the distinct element method (DEM) program Universal Distinct Element Code (UDEC) (Itasca, 2014). As Ms. Day completed the UDEC portion of the study, only the Phase2 portion of this study will be discussed. Ms. Day’s contributions to the work can be found in van der Pouw Kraan et al., 2014.

Phase2 requires the selection and definition of several discontinuity parameters, including the failure criterion, strength, stiffness, and geometry. However, discontinuity uncertainty may make selection of representative parameters difficult, and in combination with the FEM and Phase2 modelling limitations, may create rockmass behavioural uncertainty. To determine the effects of discontinuity uncertainty and the joint behaviour assumptions employed in the FEM and Phase2 on the rockmass behaviour predictions for a blocky rockmass, a parametric study was completed. A very blocky to blocky rockmass with poor to good joint surface condition was
selected for this study as it was considered that a wide range of rockmass behaviours were possible (Hoek and Marinos, 2000).

5.3.1 Model Setup

5.3.1.1 Intact Material and Boundary Conditions
The intact material was characterized as an andesite with the properties shown in Table 5-1. The intact strength properties were based on a GSI of 85 to account for micro-defects and flaws in the intact rock material. The material was modelled with the Generalized Hoek Brown failure criterion (Hoek et al., 2002), and was assumed to be elastic-perfectly plastic. Material dilation was not considered.

Table 5-1: Intact Material Properties.

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock type</td>
<td>Andesite</td>
</tr>
<tr>
<td>Unit weight (MN/m³)</td>
<td>0.027</td>
</tr>
<tr>
<td>Young’s Modulus, $E_i$ (MPa)</td>
<td>40,000</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.3</td>
</tr>
<tr>
<td>GSI</td>
<td>85</td>
</tr>
<tr>
<td>Uniaxial Compressive Strength (UCS) (MPa)</td>
<td>100</td>
</tr>
<tr>
<td>$m_i$</td>
<td>22</td>
</tr>
</tbody>
</table>

The intact material and boundary conditions were constant across all the models as joint behaviour was the subject of this study. An isotropic in-situ stress condition was applied at 50 m and 500 m depth. Groundwater pressures and other external loads were not considered.

5.3.1.2 Discontinuity Geometry and Properties
The ‘blocky’ to ‘very blocky’ rockmass on the GSI chart was represented with two long persistent joint sets and two shorter joint sets (Hoek and Marinos, 2000). Discontinuity geometry is shown in Table 5-2 and Figure 5-2.
Table 5-2: Joint set geometry properties.

<table>
<thead>
<tr>
<th>Joint Set Number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing (m)</td>
<td>0.6</td>
<td>0.6</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Length (m)</td>
<td>8</td>
<td>8</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Persistence</td>
<td>0.5</td>
<td>0.5</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Orientation (dip, °)</td>
<td>22</td>
<td>-54</td>
<td>40</td>
<td>-75</td>
</tr>
</tbody>
</table>

Figure 5-2: The blocky to very blocky rockmass joint geometry with a quarter tunnel perimeter.

For this blocky to very blocky rockmass, ‘poor’, ‘fair’, and ‘good’ surface conditions were represented with ‘low’, ‘mean’ and ‘high’ joint strengths for both the Mohr-Coulomb and Barton-Bandis shear strength criterions, as shown in Table 5-3 (Hoek and Marinos, 2000). Failure envelopes are shown in Figure 5-3. (Note: the Mohr-Coulomb tensile strength is for numerical stability only).
Table 5-3: Mohr-Coulomb and Barton-Bandis joint shear strength properties.

<table>
<thead>
<tr>
<th>Shear Strength Criterion</th>
<th>Parameter</th>
<th>High</th>
<th>Mean</th>
<th>Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mohr-Coulomb</td>
<td>Peak cohesion (MPa)</td>
<td>0.8</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Residual cohesion (MPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Peak friction angle (°)</td>
<td>20</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Residual friction angle (°)</td>
<td>16</td>
<td>24</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>Peak tensile strength (MPa)</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Barton-Bandis</td>
<td>Joint wall compressive strength (JRC) (MPa)</td>
<td>70</td>
<td>55</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Joint roughness coefficient (JRC)</td>
<td>15</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Residual friction angle (°)</td>
<td>20</td>
<td>30</td>
<td>40</td>
</tr>
</tbody>
</table>

Figure 5-3: Normal stress-shear stress strength envelopes for Barton-Bandis and Mohr-Coulomb joints, and intact rock strength (GSI = 85) (from van der Pouw Kraan et al., 2014).

The low and high joint strength properties were selected to generate a pivot point, where the low and high joint shear strength envelopes cross (highlighted with the zoomed-in box in
Figure 5-3). This pivot point was generated to examine the effect of the contributing joint shear strength components to the overall joint and rockmass behaviour (e.g. for Mohr Coulomb: c versus $\phi$, and Barton Bandis: JRC and JCS versus $\phi$). The joint properties for both joint shear strength criterions were selected so that this pivot point was approximately located in the same position. The low (50 m) and high (500 m) stress conditions were selected on either side of this pivot point to examine the joint behaviour at low and high stress.

Joint stiffness was varied in terms of the normal joint stiffness and the ratio of normal stiffness to shear stiffness ($K_n:K_s$). ‘High’, ‘mean’ and ‘low’ normal stiffness values were selected and the joint shear stiffness varied according to the stiffness ratio. Joint stiffness properties are shown in Table 5-4.

Table 5-4: Joint stiffness properties.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>High</th>
<th>Mean</th>
<th>Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal joint stiffness, $K_n$ (MPa/m)</td>
<td>80,000</td>
<td>50,000</td>
<td>30,000</td>
</tr>
<tr>
<td>Joint stiffness ratio ($K_n:K_s$)</td>
<td>2:1, 5:1, 10:1, 20:1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Combinations of the above parameters yielded 228 numerical models.

5.3.1.3 Modelling Mechanics

Phase² version 8.02 in plane strain conditions was used for this study (RocScience, 2014). The graded three noded triangular mesh contained approximately 110,000 elements. An increased mesh density was applied around the tunnel vicinity so that the elements were approximately 0.15 m to 0.3 m in length. The external boundary was set at five times the tunnel diameter. Each model had two stages: an equilibrium stage, and an excavation stage.

5.3.2 Phase² Modelling Results Discussion

The following subsections will discuss the encountered rockmass behaviours and the effects of the joint properties on the rockmass behaviour.
5.3.2.1 Encountered Rockmass Behaviours: Definitions
Four different rockmass behaviours were observed in the models: stable, wedge failure, stress driven structural failure, and rockmass squeezing. The former three behaviours were observed under low stress conditions, and the latter occurred under high stress conditions. Stable rockmasses are unaffected by external forces, including gravity, the in-situ stress condition, and/or groundwater conditions. The rockmass can remain unsupported for a prolonged period of time. Wedge, or block failure is a structurally controlled, gravity driven failure mechanism which occurs in a blocky rockmass under low confinement. Individual wedges or blocks can slide or fall, depending on the geometrical and physical characteristics of the rockmass structure. Stress driven structural failure is structural failure driven by induced stress concentrations in blocky rockmasses around the tunnel perimeter. Structural failure causes crown failure, sidewall convergence, and/or invert heave. Rockmass squeezing occurs in overstressed rockmasses, and is characterized by continuous material and structural failure around the tunnel. Large deformations (greater than 2.5% strains) occur (Hoek and Marinos, 2000).

5.3.2.2 Low Stress Rockmass Behaviours
In low stress conditions the contrasting contributions of the various joint strength parameters in the Barton-Bandis and Mohr-Coulomb joint strength criterions, and the effect of joint stiffness, to the overall rockmass behaviour are able to be observed.

In the Barton-Bandis models the rockmass behaviour was relatively consistent, with gravity driven structural failure occurring in the form of either a crown wedge (of varying size), a crown slab, and/or the development of a sidewall wedge (in connection with the crown slab) (Figure 5-4). Increasing or decreasing the JRC, JCS, and \( \phi \), correspondingly increased or decreased the extent of structural failure and the amount of total displacement. There was no discernable difference in rockmass behaviour between the low to high joint strength models, for as seen in Figure 5-4, these joint shear strength failure envelopes left of the pivot point contain
approximately the same range of values. As the rockmass behaviour was gravity driven, increasing or decreasing the joint stiffness (both in terms of magnitude and ratio) correspondingly decreased or increased the amount of joint slip that occurred. Minor material tensile and shear failure occurred along slipped joints and at joint ends which truncated within blocks.

![Figure 5-4: Varying crown failure in low strength and high strength, and low and high stiffness with Barton-Bandis joints. Note the varying displacement. Deformation magnifier = 2.](image)

In contrast to the Barton-Bandis models, the Mohr-Coulomb models exhibited a greater variation of rockmass behaviours, including stable, gravity driven structural failure, and stress driven structural failure. The joint cohesion (when present) appeared to be the most dominant parameter in controlling the rockmass behaviour, for in low stress conditions, this parameter can constitute a significant portion of the overall joint strength. As cohesion increased from 0 – 0.8 MPa in the models, the extent of gravity driven structural failure in the crown progressively decreased, and either stable, or stress driven structural failure occurred. With zero cohesion, similar to the Barton-Bandis models, the joint friction angle and the joint stiffness controlled the extent of structural failure (a crown wedge of varying size and a crown slab) and the total displacement (Figure 5-5). With 0.4 MPa of cohesion, the structural failure extent was limited to
a crown wedge, with the wedge size varying according to the joint friction and the joint stiffness (Figure 5-5). This added ‘glue’ in the rockmass reduced the extent of structural yield (and stress relief) which occurred due to gravity. Increasing the value of cohesion to 0.8 MPa mitigated the effect of gravity on the rockmass, and the rockmass behaviour varied from stable conditions to stress driven structural failure occurring in the crown, lower sidewalls, and invert. With no localized stress relief along joints that would normally fail due to gravity, the induced stress acts on the entire rockmass. Consequently, near uniform deformation occurred around the tunnel. Also, with a high value for joint cohesion, increasing the joint stiffness (both in terms of magnitude and ratio) increased the amount of structural yield which occurred, as a higher joint stiffness accommodates less joint slip as a result of the induced stress condition. Similar to the Barton-Bandis models, in all the models minor material shear and tensile failure occurred.
3.2.3 High Strength Behaviours

Under high stress conditions in all the Barton-Bandis and Mohr-Coulomb models rockmass squeezing converged around the tunnel, with material and structural shear and tensile failure (Figure 5-6). Here, the induced stress exceeds the intact material and joint shear and tensile strengths. Increasing or decreasing the discontinuity strength and stiffness values decreased and increased the amount of deformation which could occur.

Figure 5-5: Top and middle: varying structurally controlled failure magnitude, bottom – stress controlled structural failure, for low, mean, and high strength Mohr-Coulomb joints, with low and high normal stiffness magnitude. Note the varying displacement. Deformation magnifier = 2.
Figure 5-6: Rockmass squeezing for mean strength and stiffness Barton-Bandis and Mohr-Coulomb joints. Deformation magnifier = 2.

5.3.2.4 Quantitative effect of the joint shear strength criterion, joint strength and stiffness
To determine the quantitative effect of joint strength and stiffness in Phase2, the total displacements in the crown were recorded for the low and high values for these parameters, in both the low and high in-situ stress conditions. The series of curves in Figure 5-7 are plotted according to the total displacement occurring in the crown versus the joint stiffness ratio, for four separate model conditions: low stiffness (‘LS’, $K_n = 30,000$ MPa/m), high stiffness (‘HS’, $K_n = 80,000$ MPa/m), and a varying joint strength parameter in each: for Barton-Bandis, JRC, JCS, and $\phi$; and for Mohr-Coulomb, $c$ and $\phi$. 
Figure 5-7: Total crown displacements with respect to the magnitude of joint stiffness and $K_n:K_s$. The total displacements in the crown were observed to be the maximum displacements in each model. ‘LS’ – low stiffness, ‘HS’ – high stiffness.

For the low stress condition in the Barton-Bandis models, it must be kept in consideration that total displacement in gravity driven structural failure modes has no meaning, for once joint slip occurs in a model, kinematically the blocks or wedges should fall to the tunnel invert. This is a limitation of the FEM, in that the continuum mesh prevents blocks from separating and rotating.

In the Mohr-Coulomb low stress models the contrasting effects of cohesion and joint stiffness on the total displacement are able to be observed. With zero cohesion, similar results are
obtained as in the aforementioned Barton-Bandis models. As cohesion increased to 0.4 MPa, the influence of gravity and joint stiffness is reduced. Total displacement is no longer increasing with increasing joint stiffness ratio (i.e. $K_n : K_s$ from 2:1 to 20:1) or decreasing magnitude. With 0.8 MPa of cohesion, regardless of the joint stiffness, nearly zero deformation occurs. This corresponds to the stress driven structural failure observed in the models.

At high stress in both the Barton-Bandis and Mohr-Coulomb models, the quantitative effect of the joint strength and stiffness parameters on the total displacement is shown in Figure 5-7. Amongst the Barton-Bandis low and high stiffness curves, $\phi_r$ had a greater effect than JRC or JCS values on the amount of total displacement. The models with the least displacement had the greatest values of $\phi_r$ (40°). This is not surprising, for as the normal stress increases, the influence of JRC and JCS in the Barton-Bandis joint shear strength equation decrease. In the high stress Mohr-Coulomb models, the effect of the stress dependent joint friction angle is observed, as the low and high joint stiffness models with the highest joint friction incurred the least displacement. The joint stiffness magnitude and ratio contribute to the amount of slip that occurs, with increasing slip in accordance with decreasing magnitude and increasing ratio (i.e. from 2:1 to 20:1).

The quantitative effect of the joint strength and stiffness parameters is further illustrated in Figure 5-8. In this figure total displacements were recorded at eight locations around the tunnel for eight models: Mohr-Coulomb and Barton-Bandis low and high joint strength and stiffness properties with a $K_n : K_s$ ratio of 2:1, for both low and high in-situ stress conditions. In all the models, peak displacement occurs in the crown. At low stress, there is a relatively large difference in the amount of displacement which occurs in the crown and that occurring around the rest of the tunnel, whereas at high stress, this difference is largely reduced, due to the continuum mesh requirements in the FEM.
Figure 5-8: Total displacements around the tunnel for low and high strength Barton-Bandis and Mohr-Coulomb models. $K_n:K_s = 2:1$ for all models (modified from van der Pouw Kraan et al., 2014).

5.3.3 Implications of Discontinuity Uncertainty

Rockmass behaviour is significantly controlled by the discontinuity geometry and properties within a rockmass. However, defining discontinuities is a difficult task, as several parameters required for analysis are not always able to be obtained with laboratory or field testing, causing uncertainty in their determination. As demonstrated in this parametric study, the selection of the joint slip criterion, joint shear strength, and joint stiffness values (including both magnitude and ratio) affects the final rockmass behaviour determination, especially under low stress conditions. Under low stress conditions, the Barton-Bandis and Mohr-Coulomb joint slip models resulted in three mechanically different rockmass behaviours. The Mohr-Coulomb models with zero cohesion, and all the Barton-Bandis models encountered gravity driven structural failure. The
stress independent cohesion term in Mohr-Coulomb constituted a large portion of the joint shear strength under low stress conditions, and essentially ‘glued’ the rockmass together, causing stress driven structural failure. Rockmass squeezing was encountered in all the models under high stress as the induced stress state exceeded the material and joint shear strengths.

The joint stiffness was found to have a larger impact on the overall rockmass behaviour than previously thought. While it did not appear to control the rockmass behaviour type, it had considerable influence on the deformation amount. A previous ‘rule of thumb’ for the joint stiffness ratio is that this ratio varies between $K_n:K_s$ 2:1 to 10:1. For this blocky to very blocky rockmass case study this rule appears valid at low stress, but not at high stress. In high stress conditions, when the induced stress exceeded the joint shear strengths, total displacement increased near linearly between $K_n:K_s$ 2:1 to 20:1 (Figure 5-7).

The rockmass behaviour and failure mode severity have a significant influence on the tunnel design and construction. In the initial tunnel project stages where little information is available and the rockmass behaviours are unknown, Phase2 is a good numerical modelling tool to determine the potential rockmass behaviours, as the effects of discontinuity uncertainty can easily and quickly be computed and analyzed. For example, while the selection of a joint shear strength criterion should represent the actual joint surface conditions, the results from both criteria can be easily compared if desired. However, the simplifying discontinuity implementation assumptions in the FEM and Phase2 should be kept in consideration with respect to the observed rockmass behaviour in the model. Should additional information become available during design or construction, confirmation of the rockmass behaviour should be done with the appropriate numerical modelling software (e.g. UDEC).

As a continuation and expansion of this concept of discontinuity uncertainty, the next portion of this chapter will discuss rockmass behavioural uncertainty as a function of three critical geomechanics parameters: the intact strength, structure, and the in-situ stress condition.
5.4 Predicting Rockmass Behaviours

As discussed and demonstrated throughout Chapters 2, 4, and this chapter, geologic uncertainty creates challenges for prediction of rockmass behaviours. This is demonstrated in Figure 5-9, as geologic uncertainty in the geological model (shown in yellow) and in the geotechnical model (shown in green) affects the rockmass behaviour model (shown in red). The effect of uncertainty in the geotechnical model was demonstrated in the preceding section with the example of rockmass discontinuities. Geologic uncertainty propagates through the entire project process, eventually affecting the project risk assessment, and the accepted tolerated level of risk.
Figure 5-9: Geologic uncertainty propagates through the entire design process, from selection of initial design parameters to assessment of project risk. Of particular interest here is the Behaviour Model uncertainty (middle left) (modified from Langford and Diederichs, 2013a).
Popular tools and methods to qualify and/or quantify the effects of geologic uncertainty throughout the project development process include site investigation, rockmass classification and characterization systems, parametric studies, and the observational approach. As previously discussed, rockmass classification systems are popular in this respect as they are based on previous experience, which provides a measure of confidence in the recommended support design. However, as shown in Chapter 4, rockmass classification systems produce rockmass behavioural uncertainty, as equivalent materials behave differently once the unique controls on the rockmass behaviour are collectively considered.

Using the concept of tunnel domains, a new tool for determining rockmass behaviour for GBRs will be presented. This new approach allows for the entire range of rockmass behaviour to be assessed as a function of uncertainty of three parameters which control the rockmass behaviour: the intact strength, the rockmass structure, and the in-situ stress condition.

5.4.1 Rockmass Behaviour Model Setup

To demonstrate this new methodology to determine rockmass behaviours, a case study of a 10 m span drill and blast tunnel in a hydrothermally altered diorite in the Andes will be evaluated. Varying levels of hydrothermal alteration and tectonic disturbance within this rockmass are represented with three intact rock strengths and six structural conditions as represented with GSI (Hoek and Marinos, 2000). The in-situ stress state was characterized with three tunnel depths and five in-situ stress ratios (K). These parameters form the basis for this analysis and are presented in Table 5-5.

Table 5-5: Critical Geomechanics Parameters for the models.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>40, 75, 150</td>
</tr>
<tr>
<td>GSI</td>
<td>20, 35, 50, 65, 75, 85</td>
</tr>
<tr>
<td>Tunnel Depth (m)</td>
<td>75, 600, 1,500</td>
</tr>
<tr>
<td>K (σH/σV)</td>
<td>0.75, 1, 1.5, 2, 2.5</td>
</tr>
</tbody>
</table>
Combinations of the above parameters produced 210 rockmass models (2 sets of models were not included for simplicity).

5.4.1.1 Intact Rock Parameters

Intact rock strength was based on a GSI value of 85 to account for micro-defects and flaws in the rock material (Hoek and Marinos, 2000). Hoek-Brown intact material strength properties for a diorite were initially developed for this analysis. These properties were correlated to the Mohr-Coulomb failure criterion in order to more directly control the tensile strength and dilative behavior of the rock material.

The Hoek-Brown disturbance factor, D, is typically used to account for blasting or other mechanical excavation damage to a rockmass (Hoek et al., 2002). Here, this parameter was applied to calculate the residual intact strength properties. A D value of 1.5 reduced the intact peak strength by approximately half (Figure 5-10). Intact peak and residual rock strength and stiffness properties are presented in Table 5-6.
Figure 5-10: Hoek-Brown peak and residual failure envelopes with Mohr-Coulomb best-fit lines.
Table 5-6: Intact Material Strength Parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>40</td>
<td>75</td>
<td>150</td>
</tr>
<tr>
<td>$m_i$</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>$E_i$ (Young’s Modulus) (MPa)</td>
<td>13,000</td>
<td>24,400</td>
<td>48,800</td>
</tr>
<tr>
<td>$\nu$ (Poisson’s Ratio)</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>$\phi_{\text{peak}}$ (°)</td>
<td>38</td>
<td>42</td>
<td>47</td>
</tr>
<tr>
<td>$\phi_{\text{residual}}$ (°)</td>
<td>24</td>
<td>28</td>
<td>33</td>
</tr>
<tr>
<td>$c_{\text{peak}}$ (MPa)</td>
<td>10</td>
<td>14</td>
<td>19</td>
</tr>
<tr>
<td>$c_{\text{residual}}$ (MPa)</td>
<td>6</td>
<td>8</td>
<td>12</td>
</tr>
<tr>
<td>$\sigma_{1\text{, peak}}$ (MPa)</td>
<td>5</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>$\sigma_{1\text{, residual}}$ (MPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Material dilation angles for isotropic and anisotropic in-situ stress conditions were calculated using the following equations by Walton and Diederichs (2014):

$$\psi - \phi = 0.5 \left( \frac{\text{Strength}}{\sigma_1} \right) - 0.1$$  \hspace{1cm} (5-10)

And

$$\psi - \phi = 0.5 \left( \frac{\text{Strength}}{3\times\sigma_1-\sigma_3} \right) - 0.1$$ \hspace{1cm} (5-11)

Where $\psi$ is the dilation angle, $\phi$ is the peak friction angle, strength is the rockmass strength, and $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses, respectively.

5.4.1.2 Joint Geometry

The GSI chart was used to determine the joint geometry and strength parameters, as shown in Figure 5-11. Joint geometry for the GSI range presented in Table 5-5 was based on a cross jointed pattern commonly found in igneous rockmasses, and is shown in Figure 5-11 and Figure 5-12. As shown in Table 5-7, GSI 75 and 85 used a parallel statistical joint network (the mean is presented with the standard deviation shown in brackets). As the rockmass became more blocky, the parallel deterministic joint network was applied (i.e. only the mean spacing was considered for these rockmasses).
Figure 5-11: The blue dots indicate the joint geometry and joint surface locations used for the models (modified after Marinos et al., 2005).
Figure 5-12: GSI 20 to 85 structural geometry.

Table 5-7: Joint geometry parameters.

<table>
<thead>
<tr>
<th>GSI</th>
<th>Structure</th>
<th>No. of Joint sets</th>
<th>Spacing (m)</th>
<th>Length (m)</th>
<th>Persistence</th>
<th>Inclination (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>Intact or massive</td>
<td>2</td>
<td>2 (0.1)</td>
<td>60 (1)</td>
<td>0.4 (-)</td>
<td>45, -45</td>
</tr>
<tr>
<td>75</td>
<td>Blocky</td>
<td>2</td>
<td>1 (0.1)</td>
<td>60 (1)</td>
<td>0.8 (0.1)</td>
<td>55, -45</td>
</tr>
<tr>
<td>65</td>
<td>Blocky</td>
<td>3</td>
<td>0.8</td>
<td>10</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>30</td>
<td>0.5</td>
<td>-54</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>40, -10</td>
</tr>
<tr>
<td>50</td>
<td>Blocky – very blocky</td>
<td>4</td>
<td>0.6</td>
<td>8</td>
<td>0.5</td>
<td>22, -54</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>10</td>
<td>0.8</td>
<td>40, -75</td>
</tr>
<tr>
<td>35</td>
<td>Blocky/disturbed/seamy</td>
<td>5</td>
<td>0.3</td>
<td>10</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.4</td>
<td>10</td>
<td>0.7</td>
<td>-60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>10</td>
<td>0.9</td>
<td>10, -31</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>54, -80</td>
</tr>
<tr>
<td>20</td>
<td>Blocky/disturbed/seamy –</td>
<td>5</td>
<td>0.2</td>
<td>4</td>
<td>0.6</td>
<td>22, 50, 80,</td>
</tr>
<tr>
<td></td>
<td>disintegrated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-30, -54</td>
</tr>
</tbody>
</table>
5.4.1.3 Joint Strength Parameters

Corresponding with the GSI values presented in Table 5-5, joint strength properties ranged according to the joint surface conditions on the GSI chart (Marinos and Hoek, 2002). Joint strength was quantified with the Mohr-Coulomb failure criterion, as this criterion can accommodate a wide variety of joint surface conditions. Since no laboratory data was available, upper (GSI 75) and lower (GSI 20) bound joint strength properties were estimated from Read and Stacey (2009), and then distributed. The GSI 85 joint strength parameters do not follow this pattern as the rockmass is considered intact. Peak and residual joint strength parameters are presented in Table 5-8.

Table 5-8: Joint strength parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>GSI 85</th>
<th>GSI 75</th>
<th>GSI 65</th>
<th>GSI 50</th>
<th>GSI 35</th>
<th>GSI 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_{\text{peak}}$ (°)</td>
<td>54</td>
<td>48</td>
<td>44</td>
<td>40</td>
<td>36</td>
<td>28</td>
</tr>
<tr>
<td>$\phi_{\text{residual}}$ (°)</td>
<td>40</td>
<td>40</td>
<td>37</td>
<td>34</td>
<td>31</td>
<td>25</td>
</tr>
<tr>
<td>$c_{\text{peak}}$ (MPa)</td>
<td>1.2</td>
<td>0.8</td>
<td>0.7</td>
<td>0.6</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>$c_{\text{residual}}$ (MPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$\sigma_{t_{\text{peak}}}$ (MPa)</td>
<td>0.12</td>
<td>0.08</td>
<td>0.07</td>
<td>0.06</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>$\sigma_{t_{\text{residual}}}$ (MPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

5.4.1.4 Model Mechanics

Numerical models were developed with Phase\textsuperscript{2}, version 8.02 (RocScience, 2014). Plane strain conditions were assumed. The graded three noded triangular mesh contained 20,000 to 530,000 elements. In the upper GSI models, the element lengths were approximately 0.1 to 0.3 m long. To reduce model run times for models with tight joint spacing (i.e. GSI 20 and 35), an equivalent continuum material was placed approximately 3 tunnel diameters away from the tunnel boundary. Groundwater conditions were not considered a control on failure (with the exception of swelling), but rather, a modifier of failure (e.g. flowing ground from ravelling) (Marinos, 2012).
5.4.2 Rockmass Behavior Model Results

The most dominant rockmass behaviour was identified in each rockmass behaviour model, resulting in ten unique rockmass behaviors. Rockmass behaviour definitions are provided below, with the legend and 3D Rockmass Behaviour Map presented in Figure 5-13 and Figure 5-14. In Figure 5-13, each behavior is identified by a color and letter, and plotted on 3 axes: the vertical in-situ stress normalized by the intact rock strength, the GSI value, and the in-situ stress ratio, K.

The rockmass behaviours are defined as follows:

- **(A) Stable:** rockmasses unaffected by external forces such as gravity, the in-situ stress, and/or groundwater. The rockmass can remain unsupported for a prolonged period of time (Bieniawski, 1989, Stille and Palmstrom, 2008).

- **(R) Ravelling:** progressive gravity driven failure under low confinement conditions of a blocky/disturbed/seamy to disintegrated rockmass (Hoek et al., 2002). The poor to very poor block surface condition prevents block interlocking and self-stabilization from occurring above the tunnel crown (Schubert and Goricki, 2004, Marinos 2012).

- **(C) Chimneying:** structurally controlled, gravity driven failure, rapidly progressing vertically upward under low confinement, of a blocky/disturbed/seamy rockmass with fair to poor block surface condition (Hoek et al., 2002). The progressive overbreak can eventually self-stabilize due to either block interlocking, sufficient confinement, or a better quality rockmass is encountered (Marinos, 2012).

- **(W) Wedge:** for blocky rockmasses, structurally controlled gravity driven failure of individual blocks or wedges under low confinement. Blocks or wedges slide or fall depending on the physical and geometrical characteristics of the rockmass structure (Marinos, 2012).
• **(T) Stress triggered structural failure:** localized material or structural yield around the tunnel excavation caused by the induced stress allows for the subsequent release of blocks or wedges under gravity loading.

• **(D) Stress driven structural failure:** in blocky rockmasses, structural failure caused by the in-situ stress condition causes wedges and blocks in the crown, sidewalls, and invert to converge and heave into the tunnel. Stress driven structural failure may be accompanied by minor material shear and/or tensile failure.

• **(F) Structurally controlled rock fracture:** Structurally controlled rock fracture occurs in very blocky to massive, hard rockmasses where the stress induced fracturing is controlled by the rockmass structure. Failed discontinuities act as stress relief planes and prevent further stress driven fracturing from occurring across the structure (i.e. further into the rockmass). Large slabs can develop if the stress driven fracturing encounters an unfavorably oriented discontinuity with respect to the tunnel excavation. In massive rockmasses, structurally controlled rock fracturing is also directionally controlled by the in-situ stress condition. Rockmass behaviours include structurally controlled slabbing, spalling, or rock or strain burst (Diederichs, 2014)

• **(U) Rock fracture:** in blocky to intact hard rock rockmasses, stress driven fracturing and structural failure fractures the rockmass into small fragments. Failure in massive rockmasses is typically directionally controlled by the in-situ stress condition (both in terms of magnitude and ratio), whereas in blocky rockmasses, failure surrounds the entire tunnel excavation. Rockmass behaviours include slabbing, spalling, and rock or strain bursting.

• **(H) Structurally controlled shearing:** in very blocky to blocky rockmasses, stress driven material shear failure is structurally controlled by the rockmass discontinuities.
Similar to structurally controlled rock fracture, failed structures restrict the intact material failure. Failure typically surrounds the entire tunnel.

- **(Q) Rockmass shearing and squeezing**: occurs in overstressed rockmasses where material and structural shear and tensile failure surround the tunnel excavation.

  Rockmass shearing is typically characterized by medium strains (1 – 2.5%), whereas squeezing typically encounters strains > 2.5% (Marinos, 2012, Hoek and Marinos, 2000). Rockmass squeezing is a dilative behaviour, with no perceptible volume increase (Aydan et al., 1993).
Figure 5-13: The 10 unique rockmass behaviours as a result of the three geomechanics parameters.
Figure 5-14: The 3D Rockmass Behaviour Map: representation of rockmass behaviours based on the three geomechanics parameters.
5.4.3 Discussion

Plotting all the identified rockmass behaviours as a function of the three individual controls on failure allows for the effects of uncertainty in the intact rock strength, rockmass structure, and the in-situ stress condition to be collectively considered and demonstrated as a range of probable rockmass behaviours. In this 3D Rockmass Behaviour Map, uncertainty surrounding the expected ground conditions is reduced as this analysis reveals the entire range of conditions possible for the range of geomechanics parameters considered.

This presentation of rockmass behaviours can lead to a greater understanding of the rockmass behaviour along a tunnel alignment, in terms of how each unique parameter affects the rockmass behaviour, the collective influence of these three parameters on the resulting rockmass behaviours, and how rockmass behaviours relate to each other. Examples include:

- In-situ stress magnitude: there is a clear switch in behaviour for the lower GSI rockmasses (i.e. GSI 20 and 35) between 75 m to 600 m, from gravity driven, structurally controlled behaviours to stress driven behaviours (i.e. ravelling and chimneying to rockmass squeezing).

- In-situ stress anisotropy: rockmass behaviours are transformed from stable, structurally or materially controlled, to those where failure surrounds the entire tunnel. Behaviours which occur under highly anisotropic stress conditions (i.e. 600 m depth, K = 2.5) are also observed to occur under greater isotropic in-situ stress conditions (i.e. 1,500 m depth, K = 1).

- Rockmass structure: as GSI increases (decreasing structural density), the rockmass behaviour acts less like a continuum, and increasingly localized failure occurs. For example, in high stress conditions (i.e. 600 and 1,500 m), in GSI 65, the rockmass behaviours typically surround the tunnel, whereas within GSI 75 failure is more localized.
- Intact material strength: increasing material strength with regards to the in-situ stress ratio results in increasingly brittle rockmass behaviours. Also, with decreasing structural density (i.e. GSI 20 to 85), a transition from ductile to brittle rockmass behaviours is observed, from rockmass squeezing to rock fracture and rockmass shearing/squeezing.

When observed in this summarized format, several failure modes span multiple combinations of intact strength, structure, or in-situ stress conditions. Infrequently occurring failure modes (for example, rockmass behaviours occurring once or twice per structural condition or intact material strength) may actually represent a viable or significant portion of the possible rockmass behaviours. These results may not have been discernable in a deterministic analysis, or analyses with a limited number of variable input parameters.

A trend within the GBR Industry Survey results was the prevalent number of GBRs which contained only one to two rockmass behaviours. While there were a limited number of responses, and it is possible that these projects were located within relatively geologically homogenous regions, most of these respondents defined the numerous rockmass characteristics for the most and least favorable, and most dominant rockmass along the tunnel alignment. Subsequently, this number of rockmass behaviours for a tunnel project seems low. It appears that while the intact and rockmass characteristics, and to a lesser extent the in-situ stress and groundwater conditions are being defined in GBRs, this is not necessarily translating into the determination of rockmass behaviours. This new tool may help in this regard, as a parametric range of input parameters were considered to develop the rockmass behaviour models. Additionally, uncertainty in rockmass behaviour transitions (how rockmass behaviours transition from one to the next along an alignment) is reduced with the rockmass behaviour relationships shown in this map.

Rockmass behaviour is a function of both the Owner supplied ground conditions and the Contractor responsible excavation means and methods. GBRs which contain rockmass
 behavioural uncertainty due to either application of rockmass classification systems and/or contain a limited number of rockmass behaviours may result in Contractor DSC claims, as the ground conditions may not be properly and fully defined.

The following section will discuss how this rockmass behaviour determination procedure can be subsequently applied to quantify the rockmass behavioral uncertainty along a tunnel alignment.

5.5 Quantifying Rockmass Behavioural Uncertainty
The expected rockmass behaviours are one of the most important aspects of any tunneling project. However, unlike the intact material strength or joint spacing parameters where the uncertainty of these parameters can be quantified, the frequency of occurrence of rockmass behaviours along a tunnel alignment is often a ‘best guess’, typically based on geology, structure, and the in-situ stress and groundwater conditions.

In this section the 3D rockmass behaviour map will be applied to a case study to quantify the rockmass behavioural uncertainty along a tunnel alignment. Using the concept of tunnel domains, the quantified uncertainty of the geomechanics parameters (the intact strength, structure, and the in-situ stress condition) per tunnel domain is directly applied to calculate the frequency of occurrence of rockmass behaviours. As the entire range of rockmass behaviours is calculated as a function of these three unique controls on failure, this leads to a greater understanding and certainty of the expected ground conditions.

5.5.1 Case Study
The previously applied 3 km long, 10 m span water conveyance tunnel located in a hydrothermally altered diorite in the Andes will be further used to demonstrate this new tool. The tunnel alignment was divided into nine domains based on the intact strength (UCS), rockmass structure (GSI), vertical in-situ stress ($\sigma_v$), and the in-situ stress ratio (K), as shown in Figure 5-15 and Table 5-9. Along the alignment, the ground conditions vary from a disintegrated rockmass
with low strength under low isotropic stress conditions, to an intact diorite rockmass with high strength under high anisotropic stress conditions (Hoek and Marinos, 2000).

Figure 5-15: The 3 km long tunnel alignment was divided into nine domains based on the three geomechanics parameters.
Table 5-9: Parameter uncertainty within each tunnel domain.

<table>
<thead>
<tr>
<th>Domain No.</th>
<th>Lithology</th>
<th>UCS (MPa)</th>
<th>GSI</th>
<th>Tunnel Depth (m)</th>
<th>$\sigma_3$ (MPa)</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Highly altered diorite (very blocky, highly weathered joint surfaces)</td>
<td>50 – 75</td>
<td>25 – 35</td>
<td>0 – 50</td>
<td>0 – 1</td>
<td>0.9 – 1.2</td>
</tr>
<tr>
<td>2</td>
<td>Highly altered diorite (very blocky – blocky secondary rock, moderately altered joint surfaces)</td>
<td>50 – 75</td>
<td>35 – 45</td>
<td>50 - 300</td>
<td>1 – 8</td>
<td>1.0 – 1.4</td>
</tr>
<tr>
<td>3</td>
<td>Moderately altered diorite (blocky to very blocky, slightly to moderately altered joint surfaces)</td>
<td>50 – 100</td>
<td>50 – 70</td>
<td>300 – 700</td>
<td>8 – 19</td>
<td>1.2 – 1.6</td>
</tr>
<tr>
<td>4</td>
<td>Moderately altered diorite (blocky to very blocky, poor to fair joint surfaces)</td>
<td>50 – 100</td>
<td>40 – 65</td>
<td>700 – 1000</td>
<td>19 – 27</td>
<td>1.4 – 1.8</td>
</tr>
<tr>
<td>5</td>
<td>Intrusion – stockwork quartz veins and siliciclastic breccia (very blocky to blocky)</td>
<td>125 – 175</td>
<td>60 – 70</td>
<td>900 – 1200</td>
<td>24 – 32</td>
<td>0.8 – 1.5</td>
</tr>
<tr>
<td>6</td>
<td>Fault zone, ~ 65 m wide (disintegrated structure, poor joint surface condition)</td>
<td>25 – 50</td>
<td>15 – 25</td>
<td>700 – 1000</td>
<td>19 – 27</td>
<td>0.8 – 1.0</td>
</tr>
<tr>
<td>7</td>
<td>Slightly altered diorite (blocky, slightly weathered joint surfaces)</td>
<td>125 – 175</td>
<td>65 – 80</td>
<td>500 – 900</td>
<td>14 – 24</td>
<td>1.4 – 1.9</td>
</tr>
<tr>
<td>8</td>
<td>Unaltered diorite (massive structure, fresh joint surfaces)</td>
<td>150 - 200</td>
<td>75 – 85</td>
<td>900 – 1600</td>
<td>24 – 43</td>
<td>1.8 – 2.6</td>
</tr>
<tr>
<td>9</td>
<td>Slightly altered diorite (intact/massive secondary rock, slightly weathered joints)</td>
<td>75 - 100</td>
<td>60 – 70</td>
<td>&lt;50 - 500</td>
<td>1 - 14</td>
<td>1.0 – 1.5</td>
</tr>
</tbody>
</table>

5.5.1.1 Probabilistic Analysis Procedure

The probabilistic analysis procedure is as follows:

1. The tunnel alignment was divided into relatively homogenous domains based on the intact rock strength, rockmass structure, and the in-situ stress conditions.

2. The uncertainty for each parameter (intact strength, structure, and the in-situ stress condition: vertical stress and ratio) within each domain was quantified with a mean and standard deviation with an assumed normal or uniform distribution (see note below).
3. For each parameter, 1,000 random samples were generated from the probability distributions.
4. The 1,000 random samples of the intact strength were normalized by the vertical stress (to obtain the same coordinate system as on the 3D rockmass behaviour map).
5. The 1,000 random samples of the intact strength, structure, and in-situ stress were combined (i.e. three side-by-side columns in Excel) to create 1,000 samples consisting of an intact strength value/vertical stress, a structure rating, and an in-situ stress ratio.
6. All of the rockmass behaviours in the 3D behaviour map contain an intact strength, structure, and in-situ stress coordinated. To determine the most likely rockmass behaviour for each sample, the least distance between each sample and each rockmass behaviour coordinate on the 3D map was calculated. The rockmass behaviour associated with closest coordinate on the 3D map was assigned to that sample.
7. The frequency of occurrence of the rockmass behaviours which occurred within the 1,000 samples was calculated.
8. This process (steps 2 to 7) was repeated for each tunnel domain.

In quantifying the uncertainty for each parameter, due to the large parameter ranges in Table 5-9, it was assumed that each parameter range lies within two standard deviations of the mean. The intact strength, structure, and in-situ stress ratio were sampled using a normal distribution, and the vertical in-situ stress was sampled with a uniform distribution. All of the parameters were assumed to be independent, for simplicity (Hozo et al., 2005).

5.5.2 Probabilistic model results
Incorporating the intact strength, structure, and in-situ stress condition uncertainty in the probabilistic analysis allows for the combined effect of the uncertainty in these parameters to be analyzed and evaluated per domain. As shown in Table 5-10, one to a maximum of four rockmass behaviours were observed within each tunnel domain, with the majority of domains
containing three or more behaviours. This rockmass behaviour mode switching within each domain demonstrates the relative influence of each parameter, and which parameter significantly controls the rockmass behaviour per domain. For example, all the rockmass behaviours in domain 1 are structurally related, whereas those in domain 4 are stress driven. The structurally controlled gravity driven failure observed in domain 1 transforms to a combination of structurally controlled and stress driven behaviour as the overburden increases. The combined transitional effect of structure and stress is observed between domains 7 and 8, as domain 7 contains rockmass behaviours controlled by structure, whereas once the in-situ stress increases in domain 8, the influence of structure is reduced and rock fracture significantly occurs.

**Table 5-10: Quantified rockmass behaviours per tunnel domain. Numbers shown are percentages.**

<table>
<thead>
<tr>
<th>Domain No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ravelling</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Chimneying</td>
<td>94</td>
<td>40</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Wedge</td>
<td>-</td>
<td>8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>45</td>
</tr>
<tr>
<td>Stress triggered structural failure</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>71</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>Stress driven structural failure</td>
<td>-</td>
<td>7</td>
<td>45</td>
<td>54</td>
<td>19</td>
<td>-</td>
<td>23</td>
<td>-</td>
<td>55</td>
</tr>
<tr>
<td>Structurally controlled rock fracture</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6</td>
<td>-</td>
<td>6</td>
<td>41</td>
<td>-</td>
</tr>
<tr>
<td>Rock fracture</td>
<td>-</td>
<td>-</td>
<td>40</td>
<td>19</td>
<td>75</td>
<td>-</td>
<td>-</td>
<td>36</td>
<td>-</td>
</tr>
<tr>
<td>Structurally controlled shearing</td>
<td>-</td>
<td>-</td>
<td>15</td>
<td>19</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13</td>
<td>-</td>
</tr>
<tr>
<td>Rockmass shearing and squeezing</td>
<td>-</td>
<td>45</td>
<td>-</td>
<td>8</td>
<td>-</td>
<td>100</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The most frequently occurring rockmass behaviours along this tunnel alignment were stress driven structural failure, rock fracture, rockmass shearing and squeezing, and chimneying. This variation of stress to gravity driven, brittle to ductile rockmass behaviours demonstrates the result of spatial variability in the rockmass quality and the external forces along a tunnel alignment. The stable and wedge rockmass behaviours were not, or marginally encountered, respectively. This is to be expected as the calculated rockmass behaviours per domain depend on the rockmass parameter inputs.
5.6 Discussion

Utilizing the geologic uncertainty along a tunnel alignment to predict and quantify the expected rockmass behaviours leads to a greater certainty of the expected ground conditions, as the range of expected rockmass behaviours is generated as a function of the three unique controls on failure. This 3D rockmass behaviour map and procedure for quantifying the frequency of occurrence of rockmass behaviours along a tunnel alignment has several applications and benefits on tunneling projects, as discussed below.

5.6.1 Implications for Tunnel Design

The 3D rockmass behaviour map highlights the individual and combined influences of the intact strength, structure, and/or the in-situ stress on a rockmass. During the initial project stages when limited information is available, the 3D behaviour map as presented herein could be used as a guide to determine which rockmass behaviours are possible along a tunnel alignment. However, the excavation geometry and the cross jointed structure applied to develop this chart should be kept in consideration, as these factors can affect the resulting rockmass behaviour (the suite of intact rock strengths and in-situ stress conditions make these two parameters more widely applicable to other tunneling projects). Furthermore, the rockmass behaviours shown here are based on the author’s interpretation of the rockmass behaviours presented in the numerical models. Other practitioners may have an alternative interpretation of these models.

This entire analysis procedure (from the generation of the 3D map to the quantified rockmass behaviours) does not require any additional information than what is already typically collected on tunnel project site investigations: the intact rock strength, structural characteristics, and to a lesser extent, the in-situ stress condition. Where the in-situ stress condition is unknown, a parametric range of in-situ stress conditions can be considered, as done here. Additionally, as demonstrated in the GBR Industry Survey, RMR and Q were amongst the most prevalent rockmass classification systems applied on tunnel projects. In this regard, the GSI axis could be
replaced with parameters from the RMR or Q system, such as joint spacing and joint condition, or \( J_n, J_r, \) and \( J_a, \) respectively.

The expected rockmass behaviors along the tunnel alignment were analyzed with the finite element software program Phase\(^2\) (RocScience, 2014). Those discontinuous rockmass behaviours predicted to occur along a significant portion of the tunnel alignment should be further analyzed with the appropriate modelling software to fully characterize the rockmass behaviour. For example, this could include PFC for brittle failure or UDEC for structurally controlled behaviours (Itasca, 2014).

Reliability based design approaches can be applied to assess the probability of failure for brittle spalling or squeezing rockmass behaviours (Langford and Diederichs 2014, Langford and Diederichs 2013b). Then, the predicted rockmass behaviors, their quantified likelihood of occurrence, and probability of failure can be incorporated into a risk analysis where the impact on the project can be quantitatively assessed (Eskesen et al., 2004).

From the Contractor or design-build team perspective, quantified rockmass behaviours can be assessed for expected tunnel lengths. The rock support design (including the support types and capacity), material quantities, advance rates, and tunnel costing and scheduling could be more accurately determined. For construction purposes, similar rockmass behaviours (e.g. ravelling and chimneying, or rock fracture and structurally controlled rock fracture) can be assigned the same rockmass support class to minimize switching of rock support classes during construction.

5.6.2 Implications for GBRs

This new tool should be included in a GBR as the predicted and quantified rockmass behaviours reduce uncertainty in the expected ground conditions along a tunnel alignment. By identifying the rockmass behaviours as a function of uncertainty of the three critical controls on failure (the intact strength, structure, and the in-situ stress condition), the entire range of expected rockmass behaviours for these parameters is obtained. This greater understanding and definition of the
expected ground conditions is of benefit to all project parties, in terms of equitable allocation of subsurface risk in the GBR, the tunnel design, the excavation means and methods selection, costing and scheduling estimates, risk assessments, and may reduce the geotechnical basis for a Type 1 DSC claim.

As previously discussed in Chapter 4, the use of rockmass classification systems in GBRs promotes rockmass behavioural uncertainty and in doing so, allocates subsurface ground conditions risk to a Contractor or design-build team. In comparison, presenting the entire expected range of expected rockmass behaviours in a GBR with the 3D behaviour map shifts risk away from a Contractor due to reduced uncertainty in the expected ground conditions. This may affect the Contractor’s bid price in two ways: the Contractor’s construction contingency is reduced, due to improved definition of the expected ground conditions, and replaced with the increased costs associated with the evaluation of a potentially greater range of expected ground conditions.

Rockmass behaviours are a function of the excavation methodology which is strictly considered to be the Contractor’s responsibility on both design-bid-build and design-build projects. This should not prevent a design Engineer from completing this analysis, as a tunnel excavation shape and approximate geometry (e.g. span) are typically required for the Engineer’s design purposes. As demonstrated here, the tunnel’s excavation shape and span were the only construction consideration used to predict the rockmass behaviours. When the rockmass behaviours are quantified, for those rockmass behaviours which frequently occur along the tunnel alignment, further modelling should be completed to fully characterize the rockmass behaviour with the excavation methodology.

5.6.3 Implications for the Contracting Methodology
On design-bid-build projects this analysis should be completed by the design Engineer for the Owner to reduce the uncertainty regarding the expected ground conditions in the GBR. As
previously discussed in Chapter 3, Contractor’s bids are based on the contents of the GDR and
the GBR. It is suggested that for the most frequently occurring rockmass behaviours along the
tunnel alignment, that if time is available, that the Contractor perform additional analyses on
these behaviours to determine an appropriate excavation methodology. If otherwise (i.e. the
Contractor does not perform additional analyses), at the very least the improved definition of the
expected ground conditions may aid the Contractor to select an appropriate excavation
methodology to mitigate the rockmass behaviours, and not just rely on their previous build
history. This may result in improved construction planning (e.g. the excavation means and
methods, quantities, advance rates), a more technically sound bid, accurate project bid price, and
a reduced construction contingency. Owners could then obtain a range of comprehensive bids for
analysis during the tender evaluation stage. The certainty of the final project price is increased as
the improved understanding of the anticipated rockmass behaviours diminishes the chance of a
Type 1 DSC.

On design-bid-build projects, it is suggested that this analysis be completed during
procurement as a combined effort between the Owner (and/or their design Engineer) and the
design-build teams, and become incorporated into the design-build team’s bid. As the Owner
completes some or all of the site investigation works they are knowledgeable of the ground
conditions, and the design-build team can incorporate the tunnel design and construction aspects.
In working together, all parties understand how the rockmass behaviours were generated
(including the assumptions used). In accordance with the ASCE GBR guidelines for design-build
procurement, the GBR (containing this analysis) from the winning design-build team is then
incorporated into the contract documents (Essex, 2007).

For both project delivery methods, it is proposed that unit-price contracts be used for the
tunnel excavation due to the strong likelihood of rockmass behaviour mode switching within each
tunnel domain. The excavation cost (per meter, for example) can be determined for each
quantified rockmass behaviour as part of the contract negotiations, as well as excavation costs, if the actual encountered frequency exceeds what was predicted. During construction, once a tunnel domain has been completely excavated, the actual frequency of occurrence of each rockmass behaviour encountered can be compared to what was predicted. If the actual frequency of a rockmass behaviour is greater than what was expected, the preexisting contract provisions turn a potential DSC into a contract administration item. If the encountered frequency for a rockmass behaviour was less than expected, this means that another rockmass behaviour was encountered more than predicted, and the Contractor is still compensated, as previously described.

5.6.4 Tunnel Construction and DSC Implications
During construction, it may be possible with visual inspections, field measurements, or probe drilling to assess which types of rockmass behavior could feasibly be expected ahead of the face, based on the intact strength, rockmass structure, and the in-situ stress condition.

The number of predicted rockmass behaviours per tunnel domain ranged from one to a maximum of four. Rockmass behaviour transitions and combinations of these behaviours (e.g. the results of failure mode switching) may prevent these occurrences during excavation from turning into a Type 1 DSC, if this information is included in the GBR.

While this analysis considered the effects of geologic uncertainty in predicting and quantifying the rockmass behaviours, it is possible that a Type 1 (or Type 2) DSC could still occur due to geologic uncertainty. For example, exceedance in the expected intact rock strength, in-situ stress conditions, or a complete change in the actual rockmass behaviours, etc., may force the Contractor or design-builder to change their excavation methodology. However, the improved definitions of the expected ground conditions in the GBR may aid in the evaluation of a potential DSC as a product of either the Owner supplied ground conditions (and if so, then also in evaluating the material difference), or the Contractor or design-builder excavation means and methods. This may prevent costly disputes and lengthy schedule delays.
5.7 Conclusion

Geologic uncertainty challenges all aspects of a tunnel project, from site characterization through to construction. Within this chapter a new tool was presented for GBRs which utilized the uncertainty within three geomechanics parameters which significantly control rockmass behaviour (i.e. the intact strength, structure, and the in-situ stress condition) to predict and quantify the frequency of occurrence of rockmass behaviours along a tunnel alignment.

The 3D Rockmass Behaviour Map was generated through a parametric study using the aforementioned three geomechanics parameters. This presentation of anticipated rockmass behaviours leads to a greater understanding of the rockmass behaviour along a tunnel alignment, in terms of how each unique parameter affects the rockmass behaviour, the collective influence of these three parameters on the resulting rockmass behaviours, and how rockmass behaviours are related. The 3D rockmass behaviour map was subsequently applied to a tunnel case study to predict the expected frequency of occurrence of rockmass behaviours along the alignment.

Rockmass behavioural uncertainty was quantified as a function of uncertainty of the three aforementioned geomechanics parameters per tunnel domain. This may be beneficial to a project, in terms of the determination of the rock support design, advance rates, and for tunnel costing and scheduling purposes. Additionally, risk due to the anticipated rockmass behaviours and mitigation measures can be more accurately established.

Predicted and quantified rockmass behaviours should be included in a GBR as the main benefits from this tool: an increased understanding of the expected ground conditions for all project parties and improved excavation means and methods selection, improve subsurface risk allocation in the GBR, proactively mitigate risk due to tunneling, reduce the possibility of a Type 1 DSC claim, and increase certainty in the final project price.
Chapter 6

Conclusions

6.1 Thesis Summary
This research was conducted to determine the challenges currently facing GBRs in the tunneling industry and to investigate how the description of the expected subsurface ground conditions could be improved. This work was divided into five main sections:

- A literature review of rock engineering design tools and engineering contracts determined how geologic uncertainty and the contractual setting affects the description of the expected ground conditions and subsurface risk allocation in a GBR;
- The GBR Industry Survey determined how GBRs are currently being applied in industry and their perceived effectiveness;
- A demonstration of how rockmass classification systems promote rockmass behavioural uncertainty;
- A study of discontinuity uncertainty and the implications of numerical modelling of discontinuities in a continuum software program;
- Development of a new tool which utilizes geologic uncertainty to predict and quantify the rockmass behaviours along a tunnel alignment.

The following sections briefly summarize the main findings and propose recommendations for future work.

6.2 Rockmass Behavioural Uncertainty in Geotechnical Baseline Reports

6.2.1 Summary
The GBR Industry Survey produced a few trends about the current application and effectiveness of GBRs in the tunneling industry. The main findings were:
• For the various rockmasses along the tunnel alignment, the majority of respondents defined the rockmass according to the intact strength, structural characteristics, groundwater conditions, and through the application of rockmass classification systems.

• The in-situ stress condition was the least defined parameter amongst all the projects.

• Rockmass classification systems are a popular tool in GBRs, with RQD, RMR, and Q as the most popular.

• The most prevalent reasons for selecting a classification system include prior experience, to obtain the preliminary support requirements, the relevance of the input parameters to the project, and to determine the expected rockmass behaviours.

• A relatively significant number of GBRs only reported one to two rockmass behaviours.

• While there was limited data, nearly half of the respondents indicated that the GBR was ineffective at mitigating differing site conditions claims.

That nearly half of the respondents indicated that the GBR was ineffective at mitigating a differing site conditions claim may be the culmination of several observed trends: the prevalent use of rockmass classification systems, the low reporting frequency of the in-situ stress condition, the low number of rockmass behaviours, and the low occurrence of excavations means and methods prescriptions. In particular, the use of intact rock properties and rockmass classification systems to define the various rockmasses along a tunnel alignment leads to rockmass behavioural uncertainty. Rockmass classification systems generate ‘equivalent materials’, which have the same rock quality rating, yet behave differently once the unique controls on failure are collectively considered. These five models produced five different rockmass behaviours: squeezing, block failure, spalling, buckling, and flowing ground. In the majority of cases the recommended rock support was mechanically incorrect for the rockmass behaviour.

The GBRs is typically the only interpretive document of the expected subsurface ground conditions and platform in which to explain the basis for design and construction issues. GBRs
which use intact and rockmass properties and rockmass classification systems alone to describe the expected ground conditions promote rockmass behavioural uncertainty, as multiple interpretations are possible for the same rock quality rating. Incorrect interpretation of the expected ground conditions may lead to Contractor over or under bidding during the tender stage, due to conservatism or neglect of rockmass behaviours, or differing site conditions claims during construction.

6.2.2 Future Work Recommendations
The GBR Industry Survey produced some interesting results about the current status quo and perceived effectiveness of GBRs in the tunneling industry. However, due to the low response rate many questions could not be thoroughly analyzed, relationships between certain key questions could not be developed, and the majority of the interpretation was restricted to defining simple data trends. Several key themes in the latter half of the survey were unable to be definitively developed and concluded, including those regarding tunnel domains, excavation means and methods, differing site conditions claims, and the effectiveness of the GBR. For more conclusive results this survey should be more widely redistributed (in particular, to tunneling Contractors, as they were not represented in the survey results), as the resulting information may be beneficial to the North American tunneling industry to improve the predictions and definitions of the expected ground conditions with respect to the project’s contractual framework.

6.3 Quantifying Rockmass Behavioural Uncertainty for Geotechnical Baseline Reports

6.3.1 Summary
Rockmass behaviour in jointed rockmasses is significantly controlled by the discontinuities. The first half of this chapter evaluated the effects of discontinuity uncertainty and the numerical modelling assumptions used in the FEM and Phase2 in predicting rockmass behaviours. Evaluation of the joint shear strength and stiffness versus the in-situ stress condition produced
three mechanically different rockmass behaviours at low stress, and one behaviour at high stress. The implementation differences between the Mohr-Coulomb and Barton-Bandis joint shear strength models had the greatest effect on the rockmass behaviour at low stress, with the Mohr-Coulomb cohesion term as the most dominant factor in controlling the rockmass behaviour. Numerical modelling of rockmass behaviours at low stress should be performed with at least two joint shear strength criterions to compare differences in the resulting rockmass behaviours.

The second half of this chapter introduced a new tool which utilized geologic uncertainty to predict and quantify rockmass behaviours along a tunnel alignment. The 3D Rockmass Behaviour Map was generated through a parametric analysis of the three critical geomechanics parameters which control rockmass behaviour: the intact strength, the rockmass structure, and the in-situ stress condition. By plotting the rockmass behaviours as a function of the controls on failure, for tunnel projects the uncertainty surrounding the subsurface ground conditions is reduced as this analysis reveals the range of possible rockmass behaviours. Subsequently, rockmass behavioural uncertainty within tunnel domains can be quantified by applying probabilistic ranges of the intact strength, rockmass structure, and in-situ stress condition along a tunnel alignment to this map. The quantified rockmass behaviours per tunnel domain highlight rockmass behaviour mode switching due to geologic uncertainty. The most dominant rockmass behaviours occurring along the alignment should be further analyzed with appropriate numerical modelling software.

This 3D Rockmass Behaviour Map and the quantified rockmass behavioural uncertainty should be included in GBRs as this tool reduces uncertainty in the anticipated rockmass behaviours along a tunnel alignment and provides a greater understanding of the expected ground conditions. This could be of benefit to all project parties, with regards to including improved excavation means and methods selection, reduced construction contingencies, reducing the
potential for a Type 1 differing site condition claims, and greater certainty in the final project price and schedule.

6.3.2 Future Work Recommendations
The 3D Rockmass Behaviour Map was developed using the Mohr-Coulomb joint shear strength criterion and a cross jointed discontinuity pattern. Based on the results from the parametric study on discontinuities in Phase², the rockmass behaviours presented in the 3D Map should be verified with the Barton-Bandis joint shear strength criterion. The cross jointed discontinuity pattern applied in the models may generate an inherent structural bias in the models. As rockmass behaviour is very dependent on the structural geometry in the upper GSI range, other typical joint patterns (e.g. cross bedding in sedimentary rockmasses) should be investigated to determine whether there is a difference in behaviour.

Additionally, the effects of structure location within the models should be investigated for the high GSI value (e.g. GSI 75 and 85) models, at low and high stress. For example, at 600 m, the cross-jointed joint pattern in the GSI 85 models produced a wedge in the crown with low in-situ stress ratios, and structurally controlled rock fracture at higher in-situ stress ratios. However, randomization of the structural network may alter the locations of these rockmass behaviours on the map, or may produce alternate rockmass behaviours.

The effect of groundwater and time dependency was not considered in this analysis. If these conditions are potentially anticipated on a project, their effect on the rockmass behaviour should be analyzed with the appropriate tool.

6.4 Application to the Tunneling Industry
Geologic uncertainty challenges all aspects of a tunnel project, from site characterization through to construction. This thesis has examined how geologic uncertainty and the contractual setting affect the interpretation and presentation of the expected ground conditions in a GBR.
As tunnel projects become increasingly complex, empirical rockmass classification systems are no longer able to adequately capture the effects of geologic uncertainty and the collective impact of the individual controls on rockmass behaviour.

In this thesis a new tool was developed which utilizes geologic uncertainty and the capabilities of numerical modelling methods to predict rockmass behaviours. Incorporating the effects of geologic uncertainty into a rockmass behaviour prediction analysis allows for the range of possible rockmass behaviours along a tunnel alignment to be determined and quantified. It is hoped that the development of this tool may assist to improve the interpretations of the expected ground conditions and allocation of subsurface risk in GBRs, and possibly improve the certainty of the final project price and schedule.
References


Langford, J.C., and Diederichs, M.S. 2013a. CEMI Tunnel Geo-Risk Figure. Reliability for Underground Works 2013 Short Course. 47th US Rock Mechanics/Geomechanics Symposium, San Francisco, USA.


Palmström, A. 2009. Combining the RMR, Q, and RMI classification systems. Available online: www.rockmass.net


Appendix A
Geotechnical Baseline Report Industry Survey
Basic Information

1. Please select the number of years you have worked in the tunneling industry as a consultant, contractor, owner/client, academia, or other:
   
   Number of years:
   
   Consultant
   Contractor
   Owner/Client
   Academia
   Other

2. How many Geotechnical Baseline Reports have you worked on, or with?

3. Please select one or more projects from your experience which you feel are representative of your experience with Geotechnical Baseline Reports (GBRs), and you would be willing to describe in this survey, up to a maximum of 5 projects. For the number of projects you wish to describe, please enter this number below. For each project, the remaining survey questions ask about the project, and how the ground conditions along the tunnel alignment were defined in the GBR.

   Number of projects:
Basic Tunnel Project Information

Please enter the following information for the project.

4. Project Number:
   - [ ] 1
   - [ ] 2
   - [ ] 3
   - [ ] 4
   - [ ] 5

5. Please indicate your role on the project:
   - [ ] Consultant
   - [ ] Contractor
   - [ ] Owner / Client
   - [ ] Academic
   - Other (please specify)

6. Tunnel project environment:
   - [ ] Urban
   - [ ] Rural
   - [ ] Alpine
   - Other (please specify)
7. Tunnel project purpose:
- Water conveyance
- Mining
- Wastewater
- Transit
- Rail
- Road
- Highway
- Utility

Other (please specify)

8. Please select from the following all the tunneling methods used on the project:
- Earth pressure balance TBM
- Slurry TBM
- Compressed air TBM
- Single shield TBM
- Double shield TBM
- Open/marior beam TBM
- Drill and blast
- Road headers, excavators, rippers
- NATM/SEI
- Cut-and-cover

Other (please specify)
9. Please indicate the following tunnel geometry:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of tunnel bores</td>
<td></td>
</tr>
<tr>
<td>(single, double, multi)</td>
<td></td>
</tr>
<tr>
<td>Tunnel length (m, or ft)</td>
<td></td>
</tr>
<tr>
<td>Tunnel span (m, or ft)</td>
<td></td>
</tr>
<tr>
<td>Tunnel cover or depth (vertical - m, or ft)</td>
<td></td>
</tr>
<tr>
<td>Approximate horizontal in-situ stress ratio</td>
<td></td>
</tr>
<tr>
<td>Topography present (yes/no)</td>
<td></td>
</tr>
<tr>
<td>Minimum cover (any direction - m, or ft)</td>
<td></td>
</tr>
</tbody>
</table>

10. Please select the ground type:

- Soil
- Soft Rock
- Hard Rock
- Mixed

11. Please indicate the following groundwater information along the tunnel alignment. The text in brackets after each question is a suggested answer format.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Was the alignment below the groundwater table (yes/no)</td>
<td></td>
</tr>
<tr>
<td>Average groundwater pressure head (m, or ft)</td>
<td></td>
</tr>
<tr>
<td>Average hydraulic conductivity (m/s, ft/s)</td>
<td></td>
</tr>
<tr>
<td>Localized groundwater inflows (yes/no)</td>
<td></td>
</tr>
<tr>
<td>Average localized groundwater inflow (L/s)</td>
<td></td>
</tr>
<tr>
<td>Was the groundwater flowing through the groundmass or discrete fractures</td>
<td></td>
</tr>
</tbody>
</table>
### Tunnel Project Contract Information

**12. Please select the contracting method:**
- [ ] Design-bid-build
- [ ] Design-build
- [ ] Design-build-finance
- [ ] Design-build-finance-operate/maintain
- [ ] Public-private-partnership
- [ ] Turnkey / Engineer-procure-construct

Other (please specify):

**13. Please indicate the payment method (select all that apply):**
- [ ] Lump sum / Fixed price
- [ ] Unit price / Bill of quantities
- [ ] Time and materials
- [ ] Guarantee / Maximum price

Other (please specify):

**14. Was a Geotechnical Baseline Report (GBR) used on the project?**
- [ ] Yes
- [ ] No
15. Was another form of interpretive report available on the project (e.g. a Geotechnical Reference Report)?
   - Yes
   - No

16. Was a Geotechnical Data Report used on the project?
   - Yes
   - No
The next 3 questions ask about the GBR contents - intact and/or rockmass properties, and about rockmass classification systems.

17. How were the soil and/or rock, and rockmass properties specified (select all that apply):
   - From laboratory testing (e.g. Atterburg limits, UCS)
   - With in-situ testing
   - From site investigation field estimates (e.g. Schmidt hammer)
   - With correlations from empirical methods (e.g. classification systems)
   - With a strength criterion (e.g. Mohr-Coulomb, Hoek-Brown)
   - Prior knowledge (e.g. previous engineering or construction experience)
   - Other (please specify)

18. If rockmass classification systems were used on the project, please rank the classification system preference, or select ‘Not used’

<table>
<thead>
<tr>
<th>Classification System</th>
<th>Preference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Quality Designation (RQD)</td>
<td></td>
</tr>
<tr>
<td>Rock Mass Rating (RMR)</td>
<td></td>
</tr>
<tr>
<td>Q</td>
<td></td>
</tr>
<tr>
<td>Q’</td>
<td></td>
</tr>
<tr>
<td>Q TBM</td>
<td></td>
</tr>
<tr>
<td>Geologic Strength Index (GSI)</td>
<td></td>
</tr>
<tr>
<td>Terzaghi’s Rock Mass Classification</td>
<td></td>
</tr>
<tr>
<td>Rock Mass Index (RM)</td>
<td></td>
</tr>
<tr>
<td>Rock Structure Rating (RSR)</td>
<td></td>
</tr>
<tr>
<td>Mining Rock Mass Rating (MRMR)</td>
<td></td>
</tr>
<tr>
<td>Mathews/Potvin Stability Graph Method</td>
<td></td>
</tr>
</tbody>
</table>
19. For the classification system ranked as [1] in the previous question, please select the contributing factors, and rank the importance of the following factors to the classifications/characterizations systems selection:

<table>
<thead>
<tr>
<th>Contributing factor</th>
<th>Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ease of use</td>
<td></td>
</tr>
<tr>
<td>Prior experience / familiarity with the system</td>
<td></td>
</tr>
<tr>
<td>As a checklist to ensure the relevant information has been collected</td>
<td></td>
</tr>
<tr>
<td>The system’s input parameters were most relevant to the project</td>
<td></td>
</tr>
<tr>
<td>Based on the system’s correlation capabilities to obtain rock, or rockmass properties</td>
<td></td>
</tr>
<tr>
<td>To determine the expected rockmass behaviours</td>
<td></td>
</tr>
<tr>
<td>To determine the preliminary support requirements</td>
<td></td>
</tr>
<tr>
<td>To determine the final support requirements</td>
<td></td>
</tr>
</tbody>
</table>
Tunnel Alignment Rockmass Descriptions

The next 4 questions ask you to describe the most and least favorable, and the most dominant rockmass which occurred along the tunnel alignment.

The first question asks which properties were used to describe these 3 rockmasses.

The next 3 questions ask you to describe these 3 rockmasses as they occurred along the tunnel alignment. The drop-down answer box has the answers 'very high', 'high', 'medium', 'low', and 'very low' rankings, which can be correlated to the very high, high, medium, low, and very low parameter values within the preferred rockmass classification system previously selected, where possible.

For example, in the RMR system, an answer of 'medium' rock competency, would approximately correlate to a UCS of approximately 50 - 100 MPa.

For in-situ stress ratios, please consider stress ratio's less than 1 as 'very low', whereas a stress ratio greater than 3 would be considered 'very high'.

Additionally, 'not applicable' or 'unknown' can also be selected.

20. Which properties were included in the GBR for the most and least favorable, and most dominant rockmass (please select all that apply):

- Intact rock properties (e.g. UCS)
- Rockmass classification systems (e.g. Q, RMR)
- Geology, alteration, and/or weathering characteristics
- Joint characteristics (e.g. number of sets, orientation, surface condition)
- In-situ stress (e.g. magnitude, ratio)
- Groundwater characteristics (e.g. rockmass permeability, or localized inflows)
- Expected deformation magnitude

Other (please specify)
21. Along the tunnel alignment, how would you picture the most favorable rockmass?
Select very high, high, etc, from the drop-down menu for each parameter.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock strength</td>
<td></td>
</tr>
<tr>
<td>Rock stiffness</td>
<td></td>
</tr>
<tr>
<td>Joint surface quality</td>
<td></td>
</tr>
<tr>
<td>Degree of rockmass 'blockness'</td>
<td></td>
</tr>
<tr>
<td>Rockmass permeability</td>
<td></td>
</tr>
<tr>
<td>Groundwater inflows</td>
<td></td>
</tr>
<tr>
<td>In-situ stress magnitude</td>
<td></td>
</tr>
<tr>
<td>In-situ stress ratio</td>
<td></td>
</tr>
<tr>
<td>Tunnel face scab</td>
<td></td>
</tr>
<tr>
<td>heterogeneity</td>
<td></td>
</tr>
</tbody>
</table>

22. Along the tunnel alignment, how would you picture the least favorable rockmass?

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock strength</td>
<td></td>
</tr>
<tr>
<td>Rock stiffness</td>
<td></td>
</tr>
<tr>
<td>Joint surface quality</td>
<td></td>
</tr>
<tr>
<td>Degree of rockmass 'blockness'</td>
<td></td>
</tr>
<tr>
<td>Rockmass permeability</td>
<td></td>
</tr>
<tr>
<td>Groundwater inflows</td>
<td></td>
</tr>
<tr>
<td>In-situ stress magnitude</td>
<td></td>
</tr>
<tr>
<td>In-situ stress ratio</td>
<td></td>
</tr>
<tr>
<td>Tunnel face scab</td>
<td></td>
</tr>
<tr>
<td>heterogeneity</td>
<td></td>
</tr>
</tbody>
</table>

23. Along the tunnel alignment, how would you picture the most dominant rockmass?

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock strength</td>
<td></td>
</tr>
<tr>
<td>Rock stiffness</td>
<td></td>
</tr>
<tr>
<td>Joint surface quality</td>
<td></td>
</tr>
<tr>
<td>Degree of rockmass 'blockness'</td>
<td></td>
</tr>
<tr>
<td>Rockmass permeability</td>
<td></td>
</tr>
<tr>
<td>Groundwater inflows</td>
<td></td>
</tr>
<tr>
<td>In-situ stress magnitude</td>
<td></td>
</tr>
<tr>
<td>In-situ stress ratio</td>
<td></td>
</tr>
<tr>
<td>Tunnel face scab</td>
<td></td>
</tr>
<tr>
<td>heterogeneity</td>
<td></td>
</tr>
</tbody>
</table>
**Anticipated Rockmass Behaviors**

Rockmass parameters are different than rockmass behaviours. The next set of questions ask how the most and least favorable, and most dominant rockmasses were described in the GBR.

### 24. Were the most and least favorable, and dominant rockmasses described in the GBR?

<table>
<thead>
<tr>
<th></th>
<th>Yes</th>
<th>No</th>
<th>Partly described</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most favorable rockmass</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Least favorable rockmass</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Most dominant rockmass</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 25. In the GBR, were anticipated rockmass behaviours, or failure modes, described for the most favorable, least favorable, and dominant rockmasses along the tunnel alignment?

<table>
<thead>
<tr>
<th></th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most favorable rockmass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Least favorable rockmass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Most dominant rockmass</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 26. Which rockmass behaviours were described in the GBR (please select all that apply):

- [ ] Wedge failure (gravity driven structural failure)
- [ ] Chimney failure (gravity failure in blocky ground, which eventually self stabilizes above the crown)
- [ ] Raveling (gravity failure in blocky or disturbed rockmasses, which is unable to self-stabilize)
- [ ] Stress induced structural failure (failure along structure caused by in-situ stress)
- [ ] Spalling or slabling (tensile failure through intact rock)
- [ ] Shear failure (shear failure through intact rock)
- [ ] Structurally-controlled spalling (spalling through intact rock is controlled by structure)
- [ ] Structurally controlled shearing (shearing through intact rock is controlled by structure)
- [ ] Squeezing (rock deformation into the tunnel with no volume increase)
- [ ] Swelling (rock deformation into he tunnel with volume increase)

Other (please specify)
<table>
<thead>
<tr>
<th>Question</th>
<th>Options</th>
</tr>
</thead>
<tbody>
<tr>
<td>27. Was the tunnel alignment divided into domains for means and methods selection?</td>
<td>Yes, No</td>
</tr>
</tbody>
</table>
28. If domains were based on geological properties, which properties were dominant in deciding the domains (please select all that apply):

- [ ] Primary Geology
- [ ] Alteration (e.g. hydrothermal, etc.)
- [ ] Weathering
- [ ] Structure
- [ ] Tectonic disturbance
- Other (please specify)

29. If domains were based on geotechnical properties, which properties were dominant in deciding the domains (please select all that apply):

- [ ] Intact laboratory testing results (e.g. UCS)
- [ ] Rockmass classification systems ratings
- [ ] In-situ stress magnitude
- [ ] In-situ stress ratio
- [ ] Degree of jointing / structure
- Other (please specify)

30. Were the tunnel domains divided by, or modified by the following hydrogeological characteristics?

- [ ] Hydraulic permeability
- [ ] Local groundwater inflows
- [ ] Rockmass storativity
- [ ] Groundwater chemistry
- Other (please specify)

31. Were excavation means and methods described, or recommended in the GBR?

- [ ] Yes
- [ ] No
32. If excavation means and methods were described or recommended, please select all that apply from the following statements:

- [ ] Pre-excavation means and methods were recommended or specified (e.g. jet grouting)
- [ ] A ‘design philosophy’ was specified for the entire tunnel length (e.g. TBM versus Drill and Blast)
- [ ] A prescriptive specification was prescribed for the entire tunnel length (e.g. number of headings, rock support requirements)
- [ ] A prescriptive specification was prescribed for individual domains, rockmass classes, or anticipated behaviours (e.g. number of headings, rock support requirements)
- [ ] A performance specification was specified for the entire tunnel length
- [ ] A performance specification was specified for individual domains, rockmass classes, or anticipated behaviours
- [ ] Not applicable

Other (please specify)
Differing Site Conditions Claims

33. For the most favorable, least favorable, and most dominant rockmasses, please rate the suitability of the description of the rockmass properties, behaviour, and anticipated construction issues in the GBR for construction purposes:

<table>
<thead>
<tr>
<th>Rockmass properties</th>
<th>Rockmass behaviour / failure mode description</th>
<th>Anticipated Construction Issues</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most favorable rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
<tr>
<td>Least favorable rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
<tr>
<td>Dominant rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
<tr>
<td>Other (please specify)</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
</tbody>
</table>

34. Did a differing site conditions claim(s) occur on the project? For the choice below, an unforeseen ground condition is defined as a ground condition which is not defined in the GBR. An unforeseeable ground condition is one where all the data was available, and one with a reasonable knowledge of geology and geotechnical engineering could not have predicted the ground condition.

- Type 1 - unforeseen ground condition
  - Yes: [ ]
  - No: [ ]
- Type 2 - unforeseeable ground condition
  - [ ]

35. How effective was the GBR in describing the most and least favorable, and dominant rockmasses along the tunnel alignment?

<table>
<thead>
<tr>
<th>Effective - a differing site condition did not occur</th>
<th>Ineffective - a differing site condition occurred</th>
<th>Unknown</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most favorable rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
<tr>
<td>Least favorable rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
<tr>
<td>Most dominant rockmass</td>
<td>[ ]</td>
<td>[ ]</td>
</tr>
</tbody>
</table>

36. If 1 or more unforeseen site conditions occurred, please select why (select all that apply) (Source: Baynes, 2010):

- [ ] An inadequate site investigation resulted in an insufficient understanding of the ground conditions
- [ ] Unreasonable rock or rockmass properties (design values) were selected for design
- [ ] An unreasonable, or incorrect analytical model was selected
- [ ] The ground condition was contractually unforeseen

Other (please specify) [ ]
37. Of the following dispute resolution methods, if they were applied to the project, please indicate their effectiveness:

<table>
<thead>
<tr>
<th>Method</th>
<th>Effective</th>
<th>Neutral</th>
<th>Ineffective</th>
<th>Not used</th>
<th>Unknown</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dispute Review Board</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mediation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arbitration</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Litigation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other (please specify)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

38. Was the GBR an effective tool for mitigating differing site condition claims on the project?

<table>
<thead>
<tr>
<th>Condition</th>
<th>Effective</th>
<th>Somewhat effective</th>
<th>Not effective</th>
<th>Unknown</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 - unforeseen ground condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 2 - unforeseeable ground condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Please explain why, or why not:

39. Should excavation means and methods be recommended, or specified, in a GBR for the following project delivery methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design-bid-build</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design-build</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

40. Based on the answers provided and your experience, do you have any suggestions or recommendations for improving GBRs (e.g. what kind of information is included, how information is presented)?
Appendix B

Queen’s University GREB – GBR Industry Survey Approval Letter
May 30, 2014

Ms. Michelle van der Pouw Kraan
Master’s Student
Department of Geological Sciences and Geological Engineering
Queen's University
36 Union Street
Kingston, ON, K7L 3N6

GREB Ref #: GGSGE-003-14; Romeo # 6012753
Title: "GGSGE-003-14 Geotechnical Baseline Report Industry Survey"

Dear Ms. van der Pouw Kraan:

The General Research Ethics Board (GREB), by means of a delegated board review, has cleared your proposal entitled "GGSGE-003-14 Geotechnical Baseline Report Industry Survey" for ethical compliance with the Tri-Council Guidelines (TCPS) and Queen's ethics policies. In accordance with the Tri-Council Guidelines (article D.1.6) and Senate Terms of Reference (article G), your project has been cleared for one year. At the end of each year, the GREB will ask if your project has been completed and if not, what changes have occurred or will occur in the next year.

You are reminded of your obligation to advise the GREB, with a copy to your unit REB, of any adverse event(s) that occur during this one year period (access this form at https://eservices.queensu.ca/romeo_researcher/ and click Events - GREB Adverse Event Report). An adverse event includes, but is not limited to, a complaint, a change or unexpected event that alters the level of risk for the researcher or participants or situation that requires a substantial change in approach to a participant(s). You are also advised that all adverse events must be reported to the GREB within 48 hours.

You are also reminded that all changes that might affect human participants must be cleared by the GREB. For example you must report changes to the level of risk, applicant characteristics, and implementation of new procedures. To make an amendment, access the application at https://eservices.queensu.ca/romeo_researcher/ and click Events - GREB Amendment to Approved Study Form. These changes will automatically be sent to the Ethics Coordinator, Gail Irving, at the Office of Research Services or irvingg@queensu.ca for further review and clearance by the GREB or GREB Chair.

On behalf of the General Research Ethics Board, I wish you continued success in your research.

Yours sincerely,

Joan Stevenson, Ph.D.
Chair
General Research Ethics Board

c: Dr. Mark Diederichs, Faculty Supervisor