AMEC Earth & Environmental
A division of AMEC Americas Limited
3456 Opie Crescent, Prince George, BC
Canada V2N 2P9
Tel +1 (250) 564-3243
Fax +1 (250) 562-7045
www.amec.com

WEST QUESNEL LAND STABILITY STUDY
VOLUME 1 OF 2
REPORTS AND FIGURES

DRAFT

Submitted To:

City of Quesnel
410 Kinchant Street
Quesnel, BC
V2J 7J5

By

AMEC Earth & Environmental
A division of AMEC Americas Limited

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EXECUTIVE SUMMARY

AMEC Earth & Environmental (a division of AMEC Americas Limited) has been commissioned by the City of Quesnel since 2000 to study a large, deep-seated landslide in the West Quesnel area. The objectives of the study to date have been to identify the nature and extents of the landslide, identify hydrogeological factors affecting the landslide, and design a remedial program in order to reduce slide movements. Geotechnical work to support the study has included:

- A background study of geological literature, construction records, and previous site specific and regional engineering work.
- Analysis of geological information and instrumentation measurements collected from drill holes throughout the area.
- Analysis of data collected through ground, aerial, and satellite based survey methods.
- Development and analysis of a numerical models.

The geotechnical work since 2000 has been carried out in phases. The current report reflects a summary of the landslide investigations to date, the results of enhanced geotechnical and hydrogeological field investigations carried out in 2005-2006, and interpreted hydrogeological conditions to provide a basis on which to test potential mitigation options. Analysis of instrumentation installed to date has also provided potential options to measure the success of future mitigation strategies.

The major findings of this report show:

- The landslide movements include a combination of discrete shearing along an underlying deep-seated slide surface and plastic deformation throughout the slide mass. The combined movements are expressed at the surface by a spreading pattern with horizontal movement directions varying up to about 90° in plan view.
- The position of the landslide is confirmed to lie approximately within previously reported limits, with the exception of a possible southern extension.
- Correlations between precipitation, groundwater pressure, and surface movement rates indicate a probable connection between long term precipitation patterns, relatively high groundwater pressures and ground movements. However, the concurrent data set supporting this is limited, and continued collection of precipitation, groundwater, and movement data is critical to further the understanding, development, implementation and monitoring of the success of future mitigation options.
- If as postulated, the rate of sliding is linked to high groundwater pressures, it is believed that lowering of groundwater pressures through dewatering could be a suitable mitigation option.
- The hydrogeology of the landslide area is extremely complex and dominated by low permeability materials, with only intermittent, poorly connected higher permeability zones such that effective large-scale dewatering will be a challenging and long term undertaking.
- While it may not be possible to completely halt landslide related movements, information shows that significant movement rate changes are likely influenced by relatively small changes in groundwater pressure. The target of remedial efforts will be to slow the rate of landslide movement to a rate which is manageable and sustainable for current development levels.

- A trial dewatering program is needed in order to design a large-scale dewatering solution. The trial program should target various geological materials with different dewatering methods to evaluate the effectiveness of each.

Recommendations for further work are described in this report and include:

- Continued monitoring of: GPS hubs (minimum 3 times per year), vibrating wire piezometers (monthly data logger downloads in 2007 and quarterly thereafter), standpipe piezometers (at the same interval as datalogger data retrieval), and slope inclinometers (quarterly).
- Continued collection of other data supporting the land stability study, such as climate data (monitor the weather pattern in the West Quesnel area at the newly installed weather station).
- An annual engineering review of instrumentation and other sources of data to provide ongoing updates to potential correlations in precipitation and ground movements.
- Repair or re-installation of VWPs which become damaged or unserviceable.
- Tracking of infrastructure damage in the study area using a GIS database.
- A leak detection program for the city water system in the study area.
- Completion of the water balance data gathering and modeling that is currently underway.
- Completion of a trial dewatering program. The purpose of the trial dewatering program will be to find: the magnitude and rate of change in groundwater pressure with dewatering, the size of the zone of influence for each drain/well, which method of dewatering is most effective, and which geological unit(s) should be targeted in dewatering.
- Decommissioning of an infiltration pit at Uplands Park at the same time as trial dewatering begins.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

In July 2000, the City of Quesnel (City) commissioned AMEC Earth & Environmental (AMEC) to carry out a land stability study in the West Quesnel area. The study was prompted by a review of preliminary geotechnical assessments from various sources indicating that ground movements were potentially responsible for the relatively high numbers of gas line and utility ruptures, and damage to structures and roads in the area. Initial AMEC geotechnical investigations carried out in 2000-2001 confirmed the presence of a very slow moving, deep-seated landslide underlying the West Quesnel area (herein also referred to as the West Quesnel Slide). Through 2001 and 2002, additional geotechnical assessment, drilling, groundwater instrumentation installation, testing and analysis was carried out to better understand the nature of the landslide movement and to identify preliminary options for mitigation. The results of this initial work were documented in two AMEC reports to the City of Quesnel (Dewer & Polysou, August 2001, and Dewer & Polysou, October 2002). In 2003 the City of Quesnel engaged AMEC to undertake a limited trial dewatering (well pumping) test in order to try and determine the potential for use of dewatering wells for control of groundwater pressures and related ground movement. The results of the trial dewatering test were reported to the City of Quesnel in an AMEC report dated May 2004 (Green & Polysou).

Subsequent to 2004, AMEC was engaged by the City of Quesnel to carry out additional detailed and comprehensive geotechnical and hydrogeological investigations of the West Quesnel area through 2005 and 2006. This report presents a summary of these investigations and ongoing activities in the area by AMEC, and provides an update to the understanding of the geology, groundwater and slide mechanics based on the information collected since AMEC's 2002 and 2004 reporting. Following the presentation of the updated understanding of the study area, the slide movements, groundwater patterns and potential controlling mechanisms are discussed. The discussion of potential controls includes comments on the establishment of a basis on which to measure success.

The report concludes with a presentation of a proposed trial dewatering program. The purpose of the trial dewatering program is to try to achieve and monitor specified groundwater drawdown elevations using various dewatering measures. This will provide information to evaluate and optimize the design and installation of a long term pumping well and/or horizontal drainage system that would act to reduce groundwater pressures in critical areas of the sliding mass in an effort to stop or limit landslide related ground movements to an acceptable level.

An inset included on Figure 1, attached, shows the general location of the land stability study area.

2.0 BACKGROUND

A brief summary of key background information that supports the current project understanding follows. Note that this report is intended to be a continuation of previous AMEC reports to the City. For a detailed review of previously presented background information the reader is referred to AMEC's 2002 and 2004 reports.

2.1 GEOLOGY OF THE QUESNEL AREA

Understanding complex landslide movements in natural slopes requires an interpretation of the origin and history of the underlying geological materials. This interpretation provides the basis for establishing the geological model that includes estimates of: the position and history of the slide surface; the potential range and nature of ground strength parameters; kinematic controls on sliding; past and present probable landslide triggers; geological controls on groundwater flow patterns; and, the potential nature of the driving forces. While seemingly academic, the analysis and presentation of this information is critical to understanding potential control options for the West Quesnel area.

For the purposes of the current work, a desktop study of published geological references was carried out. In some cases, this work included reviewing previously referenced papers that provided additional insight based on updated site information. Published reference papers included those by Lay (1940), Rouse and Matthews (1979), Eyles and Clague (1987), Clague (1988), and Hora and Hancock (1995). A summary of the review is provided below in Section 3.1.

2.2 PREVIOUS CONCLUSIONS AND RECENT WORK

Between September 2000 and September 2002, AMEC carried out an initial (two phase) geotechnical assessment of the West Quesnel area, which included various background reviews, site reconnaissance, deep drilling and borehole instrument monitoring. The results of the initial study provided conclusive evidence that a large, very slow moving, deep-seated landslide was present and active at the site, and that the landslide had associated ground movements of up to 100 mm/year.

While the presence of a landslide was confirmed by ground and borehole instrumentation, analysis of potential options to control the sliding required additional studies and drilling to:

- refine the assumed distribution of geological units across the area;
- define the aerial extent of the slide area; and,
- provide quantitative estimates of the influence of groundwater pressures.

Given the potential size of the landslide and high proportion of development on the surface of the slide mass, it was determined that controlling the slide would be best accomplished by lowering groundwater pressures instead of alternatives that involved large-scale demolition, grading, and reconstruction. In order to work towards a greater understanding and long term option for mitigation of landsliding in West Quesnel, key activities recommended by AMEC's 2002 report included:

- ongoing surface (GPS) movement hub measurements
- ongoing groundwater and slope inclinometer measurements
- additional geotechnical drilling and instrumentation to provide a broader and more comprehensive understanding of the area's geological and hydrogeological conditions
- enforcement of interim development restrictions and review of building conditions
- implementation of a program of utility line monitoring and leak detection
- a preliminary trial dewatering well test;

In 2003 the City authorized continued groundwater and slope inclinometer measurements and a limited trial dewatering test. The trial dewatering test was conducted in two wells between late 2003 and the spring of 2004, and is described in AMEC's May 2004 report. Unfortunately the trial dewatering test had only limited success and it was identified that additional comprehensive geotechnical and hydrogeological studies would be required before proceeding further with options for dewatering the landslide area.

In 2004 through to the spring of 2006 the City, Urban Systems Limited (City's civil engineering consultant) and AMEC worked together to develop a multi-disciplinary and comprehensive work plan for the West Quesnel area. This work also included representations to senior government in order to attract funding support necessary to carry out the magnitude of engineering study and remedial works required. Over this period additional comprehensive geotechnical assessment work was authorized by the City of Quesnel as funding permitted, the scope of which is described in significant detail in various AMEC work plans and progress reports to the City of Quesnel dated 5 July 2004, 13 January 2005 and 23 February 2006. Major components of the geotechnical assessment carried out over this period included:

- ongoing slope inclinometer and groundwater instrumentation measurement
- plotting and analysis of ground movement (GPS) hub data provided by Terasen
- tracking of reports of infrastructure and building structure damage
- satellite based ground movement analysis (InSAR) in 2004
- geophysical ground survey (electrical resistivity tomography) in 2004
- deep drilling and coring at 10 sites during 2005 and 2006
- installation and monitoring of additional groundwater instrumentation (vibrating wire piezometers and dataloggers) in 2005 and 2006
- Installation and monitoring of additional deep ground movement instrumentation (slope inclinometers) in 2005 and 2006
- logging and testing of drill core samples in 2006
- installation of additional GPS hubs in 2006
- survey tie in of all drill and instrument locations in 2006
- acquisition and plotting of additional GPS hub data in 2006
- updated topographic modeling (LIDAR survey) in 2006
- development of a physical/kinematic movement model
- development of a hydrogeological model
- slope stability analysis
- data plotting and reporting

Note that additional activities to support the land stability study related to acquisition new surface topography (LiDAR), development of surface water drainage improvements options (storm drainage), surface and utility water flow (instrumentation & measurements), local precipitation measurement and surface hydrology modeling were carried out by the City's civil engineering consultant, Urban Systems Ltd., of Kamloops, BC.

3.0 DATA COLLECTION AND ANALYSIS

The data presented in this report represents information reviewed and collected since 2002 through:

- additional desktop reviews of geology reports;
- a LiDAR survey;
- an INSAR deformation assessment;
- a geophysical survey;
- analysis of Environment Canada climate data for the area;
- monitoring of GPS ground surface movement hubs;
- drilling of boreholes;
- installation and monitoring of instrumentation;
- laboratory testing

The purpose, methodology, and results of each of these activities are given below. At the end of each section, a summary of the key findings from the respective data is presented.

3.1 DESKTOP STUDY OF GEOLOGY REPORTS

The following is a presentation of the interpreted geological history of the Quesnel region, and where appropriate, the area of the West Quesnel Slide. To establish links between the regional geological review and the West Quesnel site, the authors found it was necessary to discuss general results of some work that follows this discussion. It was felt that reversing the presentation of data would reduce the clarity of the discussion.

Without question, the dominant terrain feature in the Quesnel region is the Fraser River. Processes largely associated with flow in this river system throughout its history have resulted in the creation of most of the characteristic geological features of the area. As a result of the relationship, the background geology must consider a review of the history of the Fraser River.

Geological reports suggest that the ancestral Fraser River Valley has flowed in an ancestral valley at Quesnel within several kilometers of its present course dating back to at least the Eocene Epoch of the Tertiary Period (between 37 and 58 million years ago). The existing ancestral valley is incised in the regional basement-level Cache Creek Group phyllite and ribbon chert rocks that range in age from Mississippian to Tertiary (about 200 to 350 million years old) below the elevation of the present-day river (Lay, 1940). Readers can note for time reference that the start of the uplift of the two main western mountain chains (Rocky and Coast Mountains) began in the late Jurassic Period, about 140 million years ago. Cache Creek Group phyllite was encountered in the ends of only a few holes at significant depths during drilling in the study area, and is identified with the symbol **(MTcc)** in this report. Note that a white clay or shale unit appears to mark the upper portion of the Cache Creek Group unit, as noted by Rouse and Matthews (1979). The white shale likely indicates alteration of the chert and phyllite.

Evidence shows that the Fraser River was initially divided into two ancient river systems with the divide located along the present Fraser River course somewhere between Macalister, BC and the confluence of the Fraser and Chilcotin Rivers. To the north and south of the divide, the ancient rivers flowed relatively close to the present-day Fraser River. Remnants of the northerly flowing system are visible today as the wide, relatively flat, northward sloping terrain a few kilometers to the east of the present-day river located along a route connecting the present-day McLeese, Cuisson, Eveline, Dragon and Ten-Mile Lakes. This indicates the river at Quesnel was northerly flowing (ultimately into the Peace River via the Herrick Valley) and about 200 to 230 m higher than the present-day level (Lay 1940). The flow of this early river system at Quesnel likely resulted in the deposition of fluvial sands and gravels over the valley floor and sidewalls as the channel flowed through the ancient valley; however, such ancient fluvial sands and gravels were not encountered (in drill holes) above the Cache Creek Group in the West Quesnel study area.

Lay (1940) points out that the portion of the ancient river system south from the confluence of the Chilcotin and Fraser Rivers was southward flowing and roughly parallel to the current route. However, as western Canadian mountain building progressed, the landscape was continually altered by tectonic movements and erosion. In particular, stream erosion and faulting along a tributary to the southward flowing system progressed northward along the present day path of the Fraser to Macalister. This erosion ultimately led to the capture of the northern system by the lower elevation southern system and reversed the flow of the river at Quesnel. The stream capture likely occurred in the late Cretaceous to Early Eocene Epoch, prior to the main volcanism of the Eocene Epoch. The most significant feature of the stream capture at this point in time is that it led to rapid down-cutting of the river channel and its tributary channels throughout the region including at Quesnel. Based on the location of the previously northerly-flowing abandoned channel, the river appears to have moved to the west, eroding down and into the west sidewall of the ancient valley slopes. Evidence of the downward erosion of the main river channel is visible in the tributary stream profiles, such as Baker Creek, where the upper reaches of the stream valleys are relatively gentle except at the approaches of the Fraser Valley where it is incised in a deep gorge. Based on the profile of the lower Baker Creek area, it is likely that the Fraser Valley sidewalls were steeply incised into the Cache Creek Group rocks in the West Quesnel area.

Lay (1940) proposes that the main Eocene Epoch followed this period of southern stream capture and rapid down-cutting. This time was marked by significant volcanism, particularly explosive volcanism in the later stages. It is suggested that at some stage, Eocene lavas were placed as valley infill deposits in the Fraser Valley, acting to dam the river and create lakes in several areas including one at Quesnel. The lavas are described by Lay (1940) as approaching the composition of andesite. This unit is defined in this report as the Tertiary Kamloops Volcanics with the symbol **(TKV)**.

The lake environment created by the Kamloops Volcanics would have existed through the initial stages of the early portion of the Oligocene Epoch (30 to 37 million years ago). Parent materials for deposits associated with this period consist of volcanic derived sediment, lava and ash that entered the valley directly from the atmosphere or over land (in the case of pyroclastic debris, lahars, and lava) or by way of surface water flow (eroded sediments carried by regional streams and rivers). Sediments on the steep valley wall margins of the lakes were eroded by streams and deposited into the basin by streams, and/or they could have been altered by mud flows originating above or below the lake margin. The term tuff is applied to such volcanically derived sediments that ultimately form weak rocks. The significance of these tuff deposits is critical, as they contain a significant portion of smectitic clay minerals that are characteristically very weak.

In addition, this suggests that the materials were modified by processes that could have resulted in unfavorable, mixed (weakened) soil structures. Rouse and Matthews (1979) refer to these deposits as the lower portion of the Australian Creek Group Tertiary sediments. These are encountered throughout the West Quesnel Slide area above the Cache Creek Group rocks. Based on the results of the slope inclinometers, the West Quesnel Slide surface is typically contained in the Australian Creek Group tuff. These volcanically derived sediments, or tuffs, are identified in this report with the symbol (**TAC**).

Rouse and Matthews (1979) showed that the upper portion of the Australian Creek Group consists of sand and gravel mixtures that are described as sandstones and conglomerates that were also placed in the early Oligocene Epoch. The orientations of the gravel particles (imbricate structure) suggests that the Fraser River resumed a southward flow during the placement of these materials. These Tertiary sediments are characteristically strong, and are predominantly formed of eroded volcanic rocks with significant clay infill. Occasionally the rocks are described as including Cache Creek group type clasts. The inclusion of Cache Creek group rocks indicates that the regional systems were actively eroding both the volcanic deposits noted above, and at some locations, the underlying Cache Creek rocks were also being eroded. While these deposits are noted to occur in the Baker Creek area in published reports, the upper Australian Group rocks were not identified in the drill holes advanced in the area. This may be explained by the events of the next time period.

Following placement of the Australian Creek sediments, there is an erosional contact noted in the regional geological mapping. This indicates that a significant period of time passed where active erosion of the landscape removed material at a regional level. This time period is suggested to extend between the early Oligocene and the Miocene Epoch (24 to 30 million years ago). The materials in the Fraser Valley were likely gently folded during this period along the axis of the river valley, as indicated by present-day bedding orientations of the Australian Creek materials, including measured dips up to about 20°. The folding was likely a result of continued tectonic deformation in Western North America resulting in deformation of the landmass between the two main mountain ranges. This degree of folding would have further increased the slope of the underlying contact of the Australian Group sediments and the Cache Creek Group Rocks, and could have resulted in sliding in the weak clay materials as the river continued flowing in the bottom of the valley.

Following this gap in time, additional deposits termed the Fraser Bend Formation, were placed. These deposits remain relatively flat to date, and thus it is assumed that major tectonic deformations were complete in the region prior to their placement. The Fraser Bend Formation is a fluvial deposit of coarse to fine-grained, stratified sediments that were likely laid down by an aggrading Fraser River. This deposit becomes finer-grained with increasing elevation, and is capped with silts, clays and finally diatomite. The diatomite contains fossils that place it in the late Middle Miocene age, or about 11 million years old. The diatomite deposits are believed to have accumulated in river-made lakes such as oxbows and other river generated shallow lacustrine environments (Rouse and Mathews, 1979). This deposit represents a significant marker in the geological sequence as its location reflects an elevation of the Fraser River system at a specific period in time. Note that the diatomite has been labeled the Crownite Formation, after the occurrence of the Crownite Mine located at about 700 metres above sea level (m asl), near the valley crest above West Quesnel. This location is about 230 m above the present elevation of the Fraser River. The Fraser Bend Formation and Crownite Formation were not encountered during drilling in the study area, likely because these formations have been eroded from the site, and now only exist above or adjacent to the study area.

The placement of the Crownite formation was followed by a period of subsequent volcanism through the late Miocene Epoch (about 5 to 11 million years ago). This volcanic period was characterized by the placement of flood basalts in the region, and was not believed to represent a particularly explosive volcanic period (Lay, 1940). Deposits in the area indicate that minor damming of the Fraser occurred, likely on the order of a few metres only. The lava deposits associated with this period are clearly visible on the Fraser River valley crests south and west of Quesnel. It is believed that subsequent erosion of this deposit is evidenced by the presence of large basalt boulders throughout the lower elevations of West Quesnel.

Following the placement of flood basalts, the river system continued to be active in the Quesnel area, leading to ongoing erosion and fluvial processes through the Pliocene Epoch (about 5 to 1.6 million years ago), which marks the end of the Tertiary Period. Following the Tertiary Period is the Quaternary Period which we are currently in. This Period is divided into two Epochs, the Pleistocene (1.6 million to 10,000 years ago), and the Holocene (10,000 years ago to present). The Pleistocene period was dominated by continental scale glaciation, involving several episodes of ice advance and retreat. The terrain forming processes involved with this time period resulted in most of the final landforms and surface deposits visible in the region today.

Continental and mountain glaciation occurred throughout North America after colder climate conditions set in about 1 million years ago. Several glacial advances occurred throughout this period, including at least two significant ice advances in the Quesnel area from the Cariboo and Coastal Mountains. Several significant geomorphic (terrain shaping) events occurred as a result of episodic glaciation:

- deposition of silts and clays in lakes dammed by the advancing glaciers,
- mechanical erosion and placement of till under the following ice sheets,
- compression and deformation of the land mass from the weight of up to 2km of ice,
- placement of silts and clays in glacial lakes dammed by retreating glaciers,
- erosion and fluvial deposition from meltwater runoff, and,
- erosion and mass-wasting due to rapid draining of temporary ice-dammed lakes.

Of significance to the West Quesnel land stability study is the likelihood that the already underlying weak Tertiary volcanic soils underwent significant mechanical deformation associated with compression/relaxation and shearing while the terrain was over-ridden by the 2 km thick glaciers. In addition, further mechanical damage (weakening) of the underlying soils likely occurred due stress changes or even landsliding as lateral support was removed (river erosion) or weakened (formation of temporary glacial lakes) along the now remnant valley walls during a rapid river level reduction of over 230 m. It is interpreted that erosion of the valley wall at the West Quesnel slide likely resulted in landsliding into the deepening Fraser valley adjacent to the area that was also being incised to the north by Baker Creek. Variable deposits of glaciofluvial (**F^G**) gravel and sand, and glaciolacustrine (**L^G**) silt and clay were encountered overlying the Australian Creek Group in the study area.

The West Quesnel Slide substantially predates the recent residential development of the area. This is suggested by the presence of significant valley-scale subdued scarp features and irregular topography that would have required substantial time to develop and erode to the present configuration. In addition, there is a lack of historic records that indicate major slide movements in the last several tens of years that would account for the present slide features. The slide appears to be very old and the surface has likely been subject to erosion and deposition over an extended period of time as discussed further below.

In terms of the natural history of the slide, the geological review suggests that the initial trigger for the West Quesnel Slide could have been erosion at the toe of the slope by the ancestral Fraser River and/or deformations in response to tectonic movements during late Oligocene or Miocene time, or toe erosion by the ancestral Fraser River during the retreat of glaciation during the Pleistocene Epoch. As discussed in detail further in the report, granular deposits at the toe of the slope, provide evidence of several potential episodes of fluvial activity near the toe (area between BH8 and BH13) as shown by the following sequence:

- erosion of the underlying Australian Creek sediments expressed sub-surface in geophysical survey profiles,
- deposition of a significant thickness of granular soils onto the eroded surface, expressed in geophysical survey profiles and drill hole intersections, and
- subsequent erosion of the same granular soils, expressed on the surface near the toe of the slope as a series of arcuate scarps and terrace features.

The timing of the events that created this sequence is not readily available given the current data, nor is it considered significant to the outcome of this study.

What is significant about fluvial activity at the toe is whether or not the slide was moving into an active river system. Landslides that daylight into active rivers continually lose support at the toe and, therefore, tend to destabilize themselves as ongoing movement occurs. Conversely, landslides that fail onto relatively “dry” ground act to stabilize themselves (ultimately stopping altogether) by increasing the natural toe support with continued movement. If the West Quesnel Slide was initially triggered and moved into a fluvial system, there is a potential that the movements were more rapid during this time as toe material was removed. If the toe was ultimately drained and located on dry ground, as is apparent in the current proposed arrangement, then movements would have continued until relative equilibrium was reached.

As discussed elsewhere in the report, the present ground slope is close to the angle of residual friction of the soils along the failure surface. This condition is likely the product of ongoing movement over an extended period of time, rather than a result of the slope coincidentally being deposited at this angle. Over time, erosion has removed much of the headscarp that might once have been present, leaving only a low scarp.

Ongoing movements at the present rates of movement (i.e., up to 100 mm/year, or say an average of 50 mm/year) would be sufficient to move a particle of soil 25% of the length of the slide (about 400 m) in about 8000 years, or roughly the time since the last glaciation. Since the slide may be much older, it appears likely that the slide experiences episodic movements related to relative imbalances in the ground conditions that define true (i.e. natural) equilibrium of the slide mass. As the active geometric changes related to fluvial erosion are largely complete, changes to equilibrium conditions of the slide are likely dominated by fluctuating groundwater conditions. This offers the prospect that relatively small changes in groundwater pressures may be sufficient to decrease movement rates to a suitable level.

3.2 LIDAR SURVEY

A 3-Dimensional LiDAR (Light Detection and Ranging) model of the ground surface of the West Quesnel area was provided to AMEC by Urban Systems in 2006. LiDAR is a high precision, aerial based survey system that captures a high density of survey points that are used to establish a very accurate (±1 m) surface profile of the surveyed area. The method allows existing infrastructure (such as buildings or roads) and vegetation to be filtered out, a process that provides what is referred to as a bare earth model of the ground surface. A 3-D solid rendering of the bare earth model from a chosen look direction is provided on Figure 1. This surface profile highlights topographic features that may appear hidden on aerial photography, such as the slide scarp, or bulging at the toe of the slide. The surface information from the bare-earth model was also used to develop a Digital Elevation Model (DEM) of the site that was in turn used to produce elevation contours, also shown in Figure 1, and surface profiles of cross sections, presented later in this report.

The bare earth model and the contours shown on Figure 1 were reviewed to examine potential indications of slide extents. The bare earth model and contour plan both indicate a significant scarp in the upper reaches of the West Quesnel area. The arcuate shape and position of the scarp indicates the likely upper extents of the West Quesnel Slide. Additional examination of the bare earth model shows (where development does not mask it) that the topography in the known slide area is irregular, with undulating to hummocky terrain features parallel to the slope contours, a characteristic of surface deformations in complex, rotational-translational landslide terrain. The contours do not indicate a significant pattern with respect to the position of the toe of the slide as much of this portion of the slide is subdued, masked by significantly developed areas or appears to have been undercut by past river erosion.

However; the bare earth model does show large scale patterns based on the relative roughness of the ground and gentle topographic changes that appear to define the limits of the slide. It is suggested that the scarp and general surface texture features of the slide are well represented in the LiDAR survey, but the toe of the slide is less identifiable. The upper and lateral limits of the West Quesnel Slide, as observed in the LiDAR model, are shown on Figure 1.

Note that the southern end of the lower slide extent is not shown on the LiDAR bare earth view. It is important to note that in this area, landslide terrain features were identified in the model indicate an extension of the slide area from the previously defined zone. Future analysis of this area will include reviewing newly installed surface movement hub data to correlate with the observations from this review.

Additional observations noted in the LiDAR review are the presence of slide terrain along Baker Creek. Specific slides, where noted are mapped with separate scarp symbols that define the slide extents on Figure 1.

Key findings from the LiDAR survey are summarized as follows:

- The bare-earth and contour plans provide well defined positions of the upper slide scarp and portions of the lateral slide extents.
- The topographic expression in the slide area indicates there has been significant deformation of the sliding mass.
- The slide area may extend to the south from the currently identified boundaries.
- Individual, small slides are visible along the north and northeastern flanks of the area, extending into Baker Creek.

3.3 REVIEW OF INSAR DEFORMATION ASSESSMENT

In 2004, Synthetic Aperture Radar Interferometry (InSAR) was undertaken in an attempt to define the boundaries of the main slide mass and identify any zones of differential movement within the slide. InSAR is a satellite-based remote sensing technique that can determine partial vectors of deformation over wide areas through comparisons between satellite radar images of a location taken at different times.

The data collection and processing was carried out by Atlantis Scientific Inc. (now Vexcel Corporation) of Nepean, ON. The results were then interpreted and discussed by AMEC in a November 2004 report (Froese and Polysou, 2004) to the City of Quesnel, which is included in Appendix A. The October 2004 Atlantis Scientific report is included with the AMEC report.

Previous records of ground movement collected through Terasen GPS monitoring hubs and AMEC-installed slope inclinometers showed that relatively large slope movements occurred prior to the fall of 2002; therefore, it was decided that the ideal period for InSAR deformation assessment would be between September 1998 and the fall of 2002. Based on the good quality of InSAR data available for October 24, 1998 and July 31, 1999, the timeframe bounded by these dates was selected for study.

Once data was collected and processed, deformation maps were produced, showing movement data overlying an orthophoto of the slide area. Two types of movement maps were produced: slant range maps and a horizontal flow line map. The slant range maps, as shown on Figure 1 of Appendix A, give information about the style of movement as well as an indication of areas which may have experienced movement larger than about 6 cm during the time period or may have been altered for other reasons such as site re-grading. The horizontal flow line map, as shown on Figure 2 of Appendix A, gives a reasonable depiction of the overall landslide boundaries. It must be understood, however, that horizontal flow line data assumes movement parallel to the slope in the maximum downslope gradient direction, which is likely not totally correct and may lead to error, particularly where local topography is not roughly parallel to the overall slide surface and slide movement. For example, subtle changes in topography along the slope may lead to zones of apparent deformation that are not real.

The results of the InSAR deformation assessment agreed well with GPS monitoring hub readings during the selected timeframe, with movement from 28 to 70 mm (error ±25 mm) recorded by GPS hubs and movement up to 80 mm detected by InSAR. Movements in the northern area of the slide were not quantified due to error produced by the angle of slide movement versus the satellite look direction; however, the northern boundary of the slide was defined based on the difference between flow directions seen in this area, as shown on the previously referenced Figure 1 of Appendix A. The eastern-southeastern boundary of the slide was also reasonably defined, showing the inferred toe of the slide. While the eastern boundary error could be as much as ±50 m due to data averaging during InSAR processing, the inferred boundary agrees with GPS readings and previous AMEC field mapping of observed ground and structural distortion. The southern and western boundaries of the slide were not delineated as the data for these zones was not of a high enough quality to make inferences. While Figure 1 shows differential movements captured by the study, too little data was used to allow conclusions to be drawn, and Coherent Target Monitoring was suggested if differential movement is to be further examined in the future. While InSAR is not meant to replace existing forms of deformation monitoring, it was very helpful as a supplement to existing data.

The key findings of the InSAR results are summarized as follows:

- General ground movements in the area were confirmed throughout the known slide area.
- The position of the eastern toe of the landslide agrees with the current known position.
- The work indicates that ground surface movements are not all in a single direction (note the apparent division of movement direction in the northern portion of the slide)

3.4 GEOPHYSICAL SURVEY

In 2004, a total of nine (9) lines totaling 7.5 km (plan length) were surveyed using electrical resistivity tomography (ERT) geophysics methods in order to obtain cross-sectional and plan subsurface information for the slide area. The primary purpose of the ERT survey was to attempt to locate the interface between upper unconsolidated Quaternary sediments and lower partially-consolidated Tertiary deposits. A secondary purpose was to attempt to locate the interface between the Tertiary deposits and underlying consolidated bedrock.

The survey was carried out by Surface Search Inc. of Calgary, AB. The results of the survey are presented in the January 2005 Surface Search report (Tarrant, 2005) included in Appendix B. Drawing 04-860-00 in Appendix B shows the locations of the survey lines.

ERT surveys involve the use of a ground resistivity system made up of: a resistivity meter, an electrode switch-box, multi-core cables, and electrode ground stakes. The electrodes are pushed into the ground at evenly spaced intervals and connected to the multi-core cables, which are in turn connected to the switch-box and resistivity meter. Current is then sent through the electrodes, and measurements between each pair of adjacent electrodes are taken in order to give the resistivity (in ohm-ms) of the volume of ground between each electrode pair. The automatic switch-box coupled to computer control allows large numbers of resistivity measurements to be made using different electrodes and measurement electrodes. The varying ray paths provide a much more detailed view of electrical properties of the subsurface materials than previous electrical geophysical methods, thus allowing more complex views of the subsurface conditions to be interpreted.

In the ERT survey, different geologic materials will yield different resistivities, allowing for interpretation of stratigraphy based on resistivity profiles; for example, granular soils such as gravel will typically give a higher resistivity reading than fine-grained soils such as clay. Refer to Table 1 in the Surface Search report for a list of normal resistivity ranges for common geologic materials.

Survey readings were taken to a maximum depth of 80m, with stakes pushed into the ground at 10m intervals along the survey lines. The data was then processed, which involved the removal of apparent error and smoothing to increase data quality. Vertical and horizontal resolution of ±5 m was achieved. Existing borehole information was used to aid in the interpretation of geological materials. The 30 ohm-m boundary was interpreted as being the likely contact between unconsolidated Quaternary sediments and underlying partly consolidated Tertiary deposits: values less than 30 ohm-m were interpreted as Tertiary sand, silt and clay, while values greater than 30 ohm-m was interpreted as Quaternary sand, silt and clay. In a few locations, the 30 ohm-m boundary corresponded with the depth of maximum ground movement given by slope inclinometer readings, indicating that in these locations the slip surface of the slide coincides with the Quaternary-Tertiary boundary. Values greater than 500 ohm-m were interpreted as likely sand and/or gravel deposits. Attempts to locate the interface between the Tertiary deposits and underlying bedrock were largely unsuccessful. It is important to note that vertical resolution decreases with depth, so thin layers of different materials might have been missed by the survey. There are many sources of error that could contribute to inaccurate survey results, such as errors in electrode spacing during the survey, errors in data processing and variations in electrical properties outside expected ranges.

Drawings 04-860-01 to 04-860-10 in Appendix B show the survey results along cross sections for each survey line, while Drawings 04-860-01A to 04-860-10A show the survey results along cross sections for each survey line along with the interpreted interface between the Quaternary and Tertiary units. Also produced were Drawings 04-860-0R1 to 04-860-0R4 of Appendix B, which show pseudo-plan plots of the ERT data at depths of 10 m, 20 m, 40 m, and 60 m. These drawings clearly show the interpreted zones of sand and/or gravel deposits. The conclusion drawn from the ERT survey was that while the survey produced useful supplemental information, the results could be refined to produce a more realistic model.

When the geophysics survey was conducted, only four boreholes had been cored within the slide boundaries. Since then, boreholes have been drilled at nine further locations within the slide boundaries. Using the stratigraphic information from the new borehole logs, a review of the geophysical model was conducted. In general, the estimated boundary between Quaternary and Tertiary materials found by the geophysics survey agreed with the boundary found during borehole logging to within ±5 m. Also, the geophysics survey showed readings indicative of granular materials in most locations at which significant thicknesses of granular soils were encountered in the field. Figure 2 shows a plan and profile view of the thickest section of a large granular deposit located near the toe of the slide.

There were, however, two borehole logs (BH11 and BH13) which differed significantly from the geophysics results. At these locations, the geophysical model indicated that the Quaternary-Tertiary boundary was much deeper than the borehole logs showed. These boreholes were located were near the eastern margin and toe of the slide. It is possible that the large amount of granular material in this area caused the geophysics profiles to become skewed; the overall shape of the granular deposit located near the toe of the slide is believed to be correct, though depths may vary. Groundwater content and chemistry can also affect the results of an ERT survey; these factors may have contributed to inaccuracy in this area. Based on the new borehole logs, the geophysical model could be calibrated to be more accurate in this area. Still, the good results of the geophysics-borehole log comparison throughout the rest of the slide area allowed the geophysical model to be very useful in refining the overall geological model of the area, with the exception of the east boundary.

The key findings from the geophysical work can be summarized as follows:

- The division between the Quaternary and Tertiary deposits is represented by the 30 ohm-m boundary. This information can be used to extend subsurface geological mapping beyond the drill holes.
- A significant granular deposit was encountered in the lower eastern portion of the slide, as shown in the plan sections noted above. The shape suggests that it could be an erosional, gravel filled depression in the Tertiary materials, a likely target for potential dewatering.

3.5 CLIMATE DATA

Monthly total precipitation data was obtained from Environment Canada for the closest recording station, the Quesnel Airport. The 2005 through 2007 precipitation data is preliminary and still requires official verification. A plot of the recorded total monthly precipitation from 1975 through 2007 along with historical "monthly normal" data is shown as Figure 3. The historical normal precipitation is based on a 30 year moving mean, where an average over the preceding 30 years was calculated for each month to derive what could be considered as a "normal" amount of precipitation for that month.

Figure 4 presents plots of the cumulative difference and the cumulative precipitations between the actual total precipitations experienced each month and the calculated normal total monthly precipitation, starting in 1975. This plot shows the trend of the recorded precipitation, and gives an indication of overall increasing or decreasing (i.e. wetter or drier) trends compared to what could be expected to be normal. Generally, horizontal trends in the plot indicate relatively normal precipitation, rising trends indicate extended wetter periods and falling trends indicate extended drier conditions than normal. Note that the method of defining "normal" varies from the method used to establish Climatic Normals, which are based on 30 year averages over fixed periods (the present normals are based on 1970 to 2000).

The cumulative difference precipitation plot indicates that the period from 1975 through 1978 was slightly wetter than normal. This was followed by a drier period from 1978 through 1980, a wetter period up to about 1980 and slightly wetter than normal conditions from 1980 to 1986. From 1986 to 1989 a significant drying trend occurred. Between 1989 and 1996 precipitation conditions were generally close to normal; however, starting in 1996 a longer term of wetter than normal precipitation occurred all the way up till early 2002. From early 2002 to mid 2003 a drier period was experienced, followed by only slightly wetter but probably normal amounts of precipitation to the end of 2004. Since 2004 there was a close to normal trend to 2005 followed by a drying trend from 2005 to the time of this report (March 2007).

Figure C1 of Appendix C is a plot of monthly recorded total precipitation versus monthly total precipitation as given by 30-year normal climate data from Environment Canada (at the Quesnel Airport) from 2001 to present. This figure gives a clear representation of which months have had more or less total precipitation than the 30-year normal.

The key findings from the climate data can be summarized as follows:

- The comparison of cumulative long-term precipitation patterns shows that extended wet and dry periods exist in the regional climate.
- As sliding was previously reported at high rates between 1997 and 1998, there is a potential that the wet and dry trends correlate to changes in ground movement rates.

FIGURE 3: Monthly Precipitation for 1976 to 2007

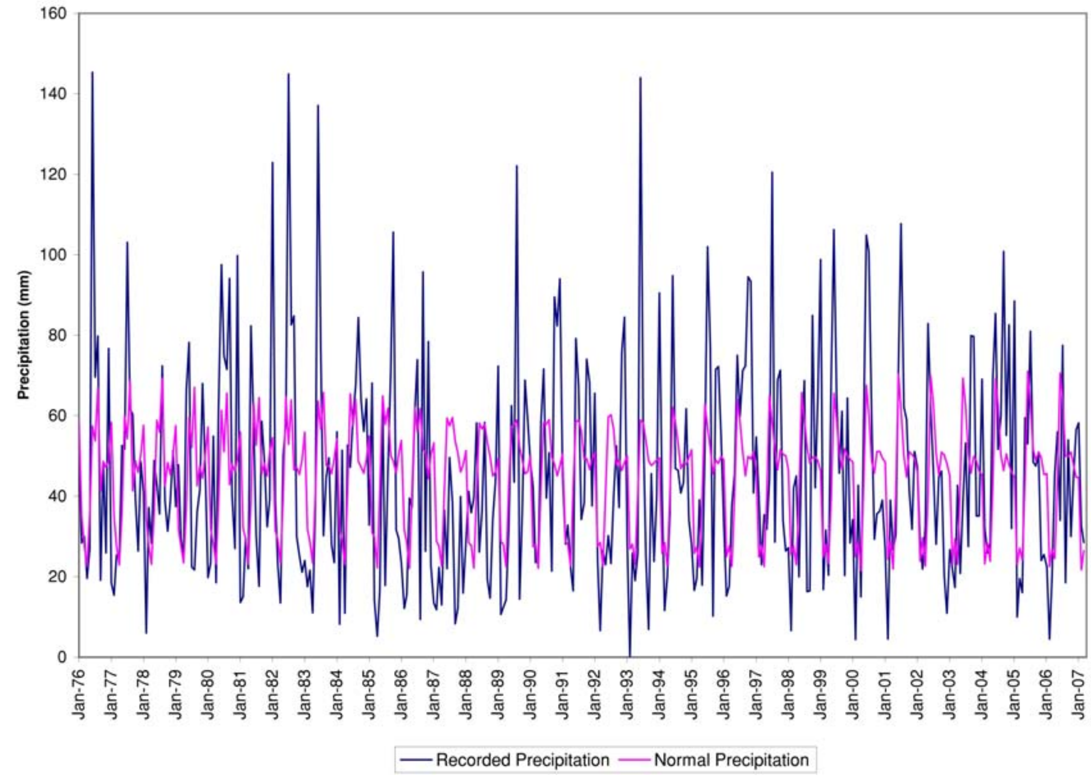
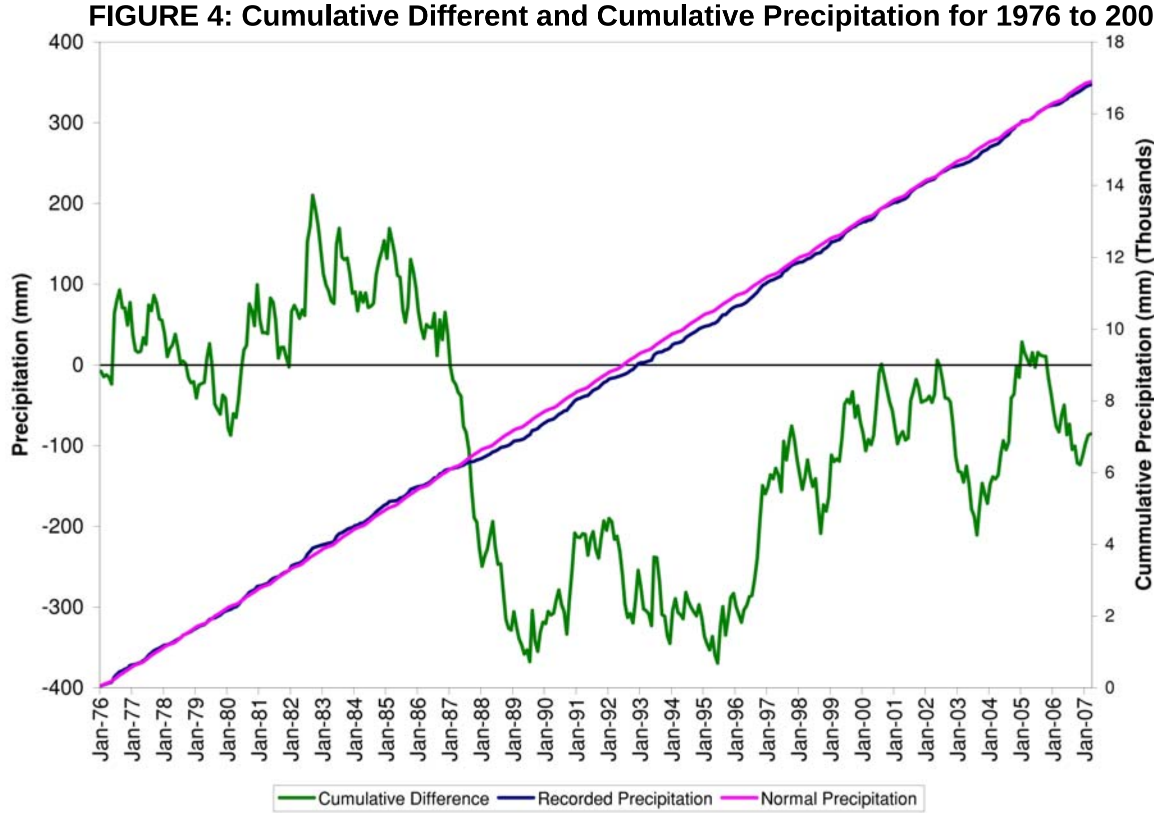


FIGURE 4: Cumulative Different and Cumulative Precipitation for 1976 to 2007



3.6 GROUND SURFACE MOVEMENT HUBS

A ground surface movement hub is a landmark station which is installed and then monitored via surveying at selected intervals; this is done in order to determine the amount and direction of movement of the hub, and therefore the ground surface, with time. The use of a global positioning system (GPS) allows surveying of hubs to be completed quickly and accurately, as a GPS unit can be set up at each hub location to determine the coordinates of the hub at that time. The coordinates can then be compared to those determined during earlier surveys in order to determine the amount and direction of ground surface movement between readings, as well as the cumulative movement since hub installation. Analysis of hub monitoring results provides important information regarding landslide behaviour by providing insight into the movement (both magnitude and direction) of the ground surface. The location of the movement hubs are shown on Figure 5, attached.

In September 1998, an initial set of twenty-nine (29) GPS ground surface movement hubs (GPS98-#) were installed in West Quesnel by Terasen Gas (formerly BC Gas). The hubs were lengths of rebar inserted 2 to 3 m deep into the ground and topped with a brass survey pin. The survey data collected semi-annually to annually for these hubs was provided to AMEC, along with data for the eleven (11) additional hubs installed in December 2001 (GPS01-#). The most recent survey, conducted in November 2006, was undertaken by McElhanney Consulting Services Ltd. of Prince George, BC under contract to AMEC. Twenty-one (21) new hubs (GPS06-#) were also installed under the direction of AMEC and the City of Quesnel at that time. Figure C2 of Appendix C shows the cumulative net lateral displacement of all movement hubs over time since installation. Table C1 of Appendix C summarizes the movement hub data in or near the slide study area collected to date.

As Figure C2 of Appendix C illustrates, the lateral displacement rate of the hubs varies with time defined by the reading intervals. Figure 6 below gives a graphical representation of the lateral displacement rates by monitoring interval in mm per year, excluding hubs which are considered to be outside of the slide boundary. From September 1998 to November 2006, the overall average rate of movement was approximately 33 mm/year, with a maximum rate of about 76 mm/year and a minimum rate of 0 mm/year. Since September 1998, cumulative net lateral displacements as large as 421 mm have occurred at some hub locations. It is important to note that the pattern of average lateral displacement rates between September 1998 and November 2006 as shown on Figure 6 corresponds closely to the pattern observed during that time period for the cumulative difference in precipitation on Figure 4 of Section 3.4. This correlation indicates a strong connection between precipitation and the rate of movement of the ground surface.

While monitoring of GPS ground surface movement hubs gives information regarding the cumulative amount and rate of ground surface movement, it also provides insight into the behaviour of the slide in terms of movement direction. Figure 9 shows vectors depicting the detected horizontal ground movement since installation for each hub. An indication of cumulative vertical movement detected at each hub location since installation is also provided. The movement vectors indicate that the slide is not moving in a single direction, instead, it is spreading out from the scarp to the toe of the slide. Long-term calculated movement directions of the hubs on the slide range from an azimuth of 043 degrees in the north to 134 degrees in the south, a range of 91 degrees. In addition to showing a general spreading style of surface movement, the movement vectors also show a smaller, separate slide movement direction along the north boundary of the installed monitoring points. The vertical components of the movement vector give an indication of areas in which the ground is dropping due to the spreading of material at the top and centre of the slide, and areas in which the ground is rising due to the mounding of material at the toe and sides of the slide.

In order to further develop an understanding of the movement of the slide surface, the horizontal and vertical rates of displacement with time from December 2001 to November 2006 were plotted and contoured on Figures 7 and 8, respectively. These figures show that clear spatial movement rate patterns exist. Horizontal rates are highest at the centre of the slide near the bottom of the slope, with rates decreasing toward the top and sides of the slide. Vertically, the ground is dropping most quickly along the centre of the slide and rising most rapidly along the toe at the northeast and southeast "corners" of the slide.

The key findings from the analysis of the GPS hub movement data can be summarized as follows:

- Consistent movement directions over time confirm the presence of surface movements. This agrees with the findings of the INSAR work.
- Changes in average movement rates over time are consistent with changes in cumulative precipitation patterns.
- Movement vectors indicate that the overall slide surface is spreading.
- Movement rates have clear patterns in both horizontal and vertical directions.
- Analysis of "moving" vs. "non-moving" GPS hubs confirms the position of the slide boundaries identified by other methods.

FIGURE 6: GPS Movement Hub Lateral Displacement Rate by Monitoring Interval

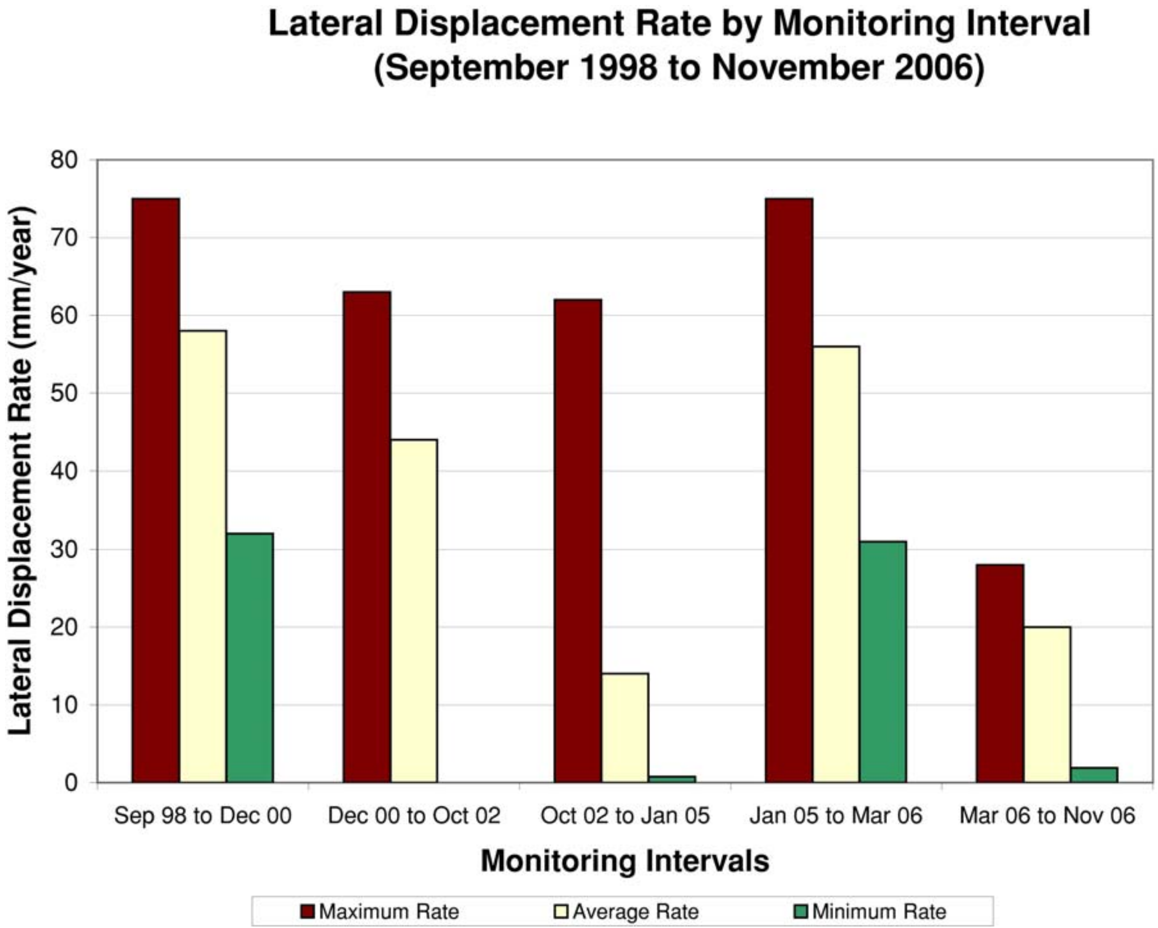


FIGURE 7: LATERAL DISPLACEMENT RATES FROM DEC 2001 TO NOV 2006

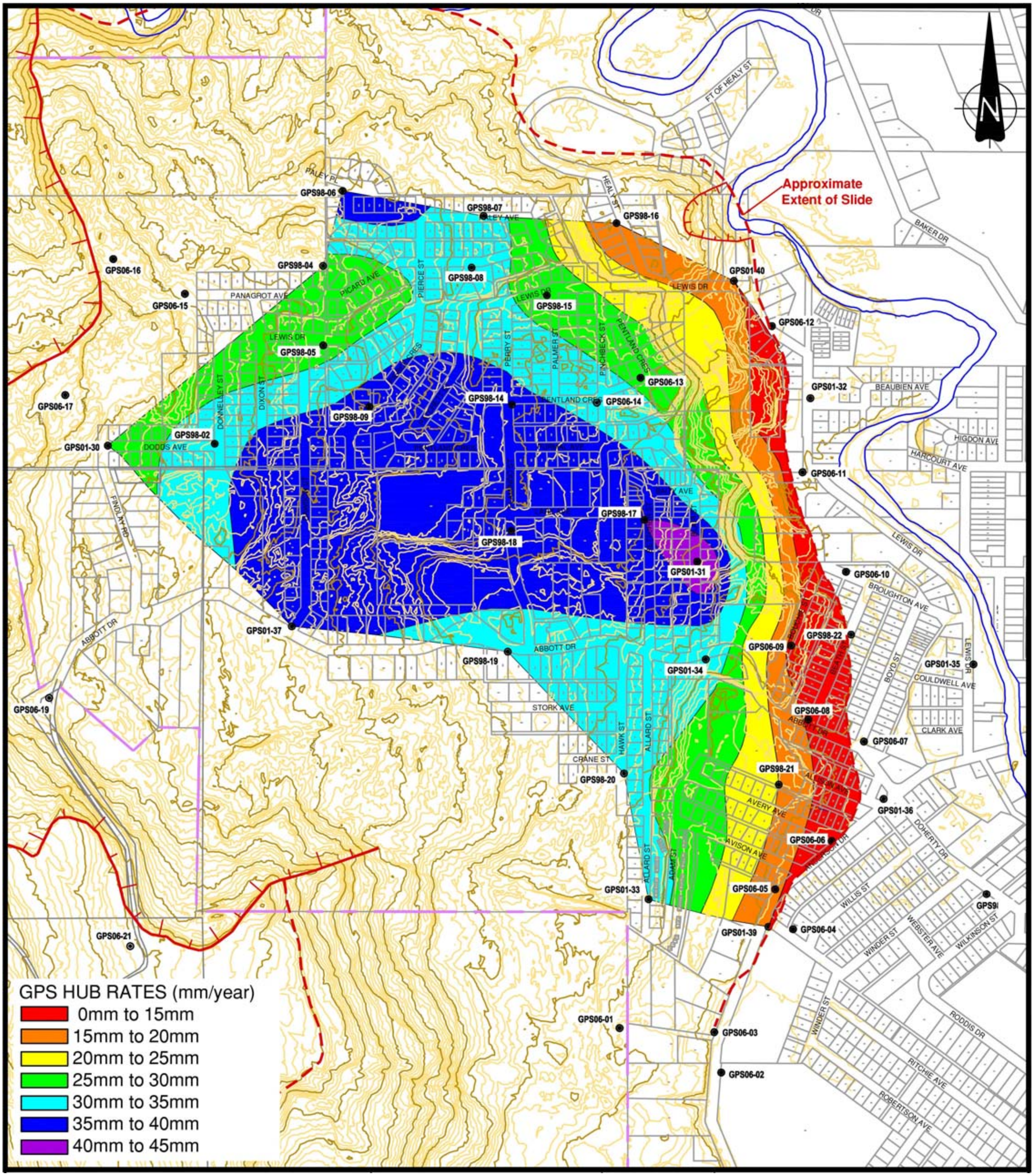
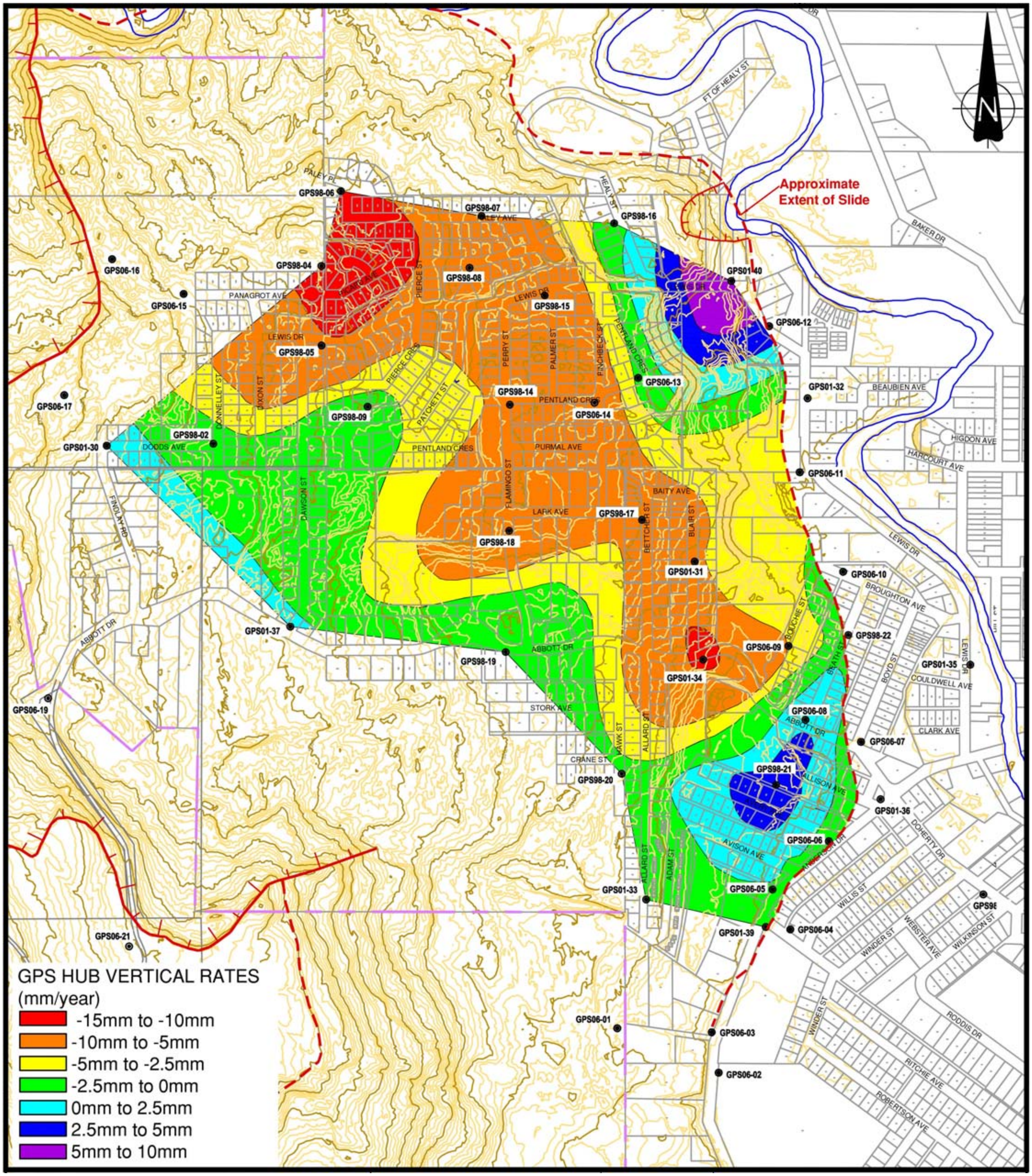


FIGURE 8: VERTICAL DISPLACEMENT RATES FROM DEC 2001 TO NOV 2006



3.7 2005/2006 DRILLING

3.7.1 Drilling Program

Thirteen (13) boreholes (SI8 to SI13, and BH7 to BH13) were completed at the site between March and July 2005. The boreholes were drilled to depths of 30.5 to 80 m below the surface using air-rotary, and diamond coring methods. The purposes of the boreholes were to investigate soil conditions, provide soil samples for testing, and allow installation of monitoring instrumentation. Boreholes SI8 to SI13 were completed using a large truck-mounted air-rotary drill rig for the installation of slope inclinometer instrumentation. Boreholes BH7 to BH13 were completed using a skid-mounted diamond coring rig to provide continuous core samples of the soil and for the installation of groundwater monitoring instrumentation. In addition, a truck-mounted air-rotary (ODEX) drill rig was used to drill and install casing through the upper sand and gravel units of boreholes BH8 and BH12, as poor performance of the diamond coring method through these coarse materials proved to be time consuming. The 2005 borehole locations are shown on Figure 1.

Eight (8) additional boreholes (SI14, SI15, BH8A, BH9, BH14 to BH16, and BH16A) were completed at the site between July and October 2006. The boreholes were completed to depths between 14.3 and 222.3 m depth. The purposes of the boreholes were to replace damaged or faulty instrumentation, investigate soil conditions, provide soil samples for testing, and allow for the installation of additional monitoring instrumentation. Boreholes BH8A and BH9 were completed by a truck-mounted drill rig using air-rotary and wet-rotary methods to replace faulty instrumentation near BH8 (VWP 8B was installed in BH8 within a section of drill casing that was lost during removal in 2005), and to replace damaged instrumentation at BH9 (2 of 3 VWP[1] cables were sheared during installation in 2005). Boreholes SI14, BH14 and BH15 were completed with a truck-mounted drill rig using air-rotary and diamond coring methods to obtain soil samples, and for the installation of slope inclinometer and groundwater monitoring instrumentation. Boreholes 16 and 16A were completed with a skid-mounted drill rig using diamond coring methods to obtain samples and for the installation of groundwater monitoring installation; BH16A was drilled approximately 9 m from BH16 as difficulties encountered during drilling did not allow for BH16 to be completed to the target depth. The 2006 borehole locations are shown on Figure 1.

Location of underground utilities was completed by the City of Quesnel and CMH Underground Utilities Ltd. of Prince George, BC. Drilling was carried out by Cariboo Water Wells Ltd. and Geotech Drilling Services Ltd. of Prince George, BC, using air-rotary, wet-rotary, and diamond drilling methods. Final borehole locations were surveyed by McElhanney Consulting Services Ltd.

Borehole logs are included in Appendix D; also included in Appendix D is Table D1, which gives a list of all holes drilled and explanations for logs that are not included, and a page explaining some of the terms used on the logs. Laboratory index test results are included on the borehole logs, or are included in Appendix E. Table 1 below provides a summary of the borehole details. Photo logs of the core recovered from the 2005/2006 drilling are included in Appendix D.

[1] VWP = Vibrating Wire Piezometer to measure groundwater pressure

Table 1: 2005/2006 Borehole Summary

Borehole	Location UTM (NAD 83)		Ground Elevation (m)	Drill Rig Type	Depth of Hole (m)	Notes
	Northing	Easting				
2005 Drilling Program						
SI8	5870035	532164	525.9	TH-60	71.0	Install: Inclinometer casing
SI9	5870114	531882	534.9	TH-60	79.0	Install: Inclinometer casing
SI10	5869649	531587	545.1	TH-60	80.0	Install: Inclinometer casing
SI11	5870355	532041	530.9	TH-60	71.0	Install: Inclinometer casing
SI12	5870278	531622	556.2	TH-60	73.0	Install: Inclinometer casing
SI13	5869938	532468	482.1	TH-60	40.0	Install: Inclinometer casing
BH7	5870226	531799	541.4	HC2000	78.9	Install: 2 VWP
BH8	5870043	532161	526.1	HC2000	65.4	Install: 3 VWP*
BH9	-	-	534.7	HC2000	69.3	2 VWP lost during installation - abandoned
BH10	5869655	531592	544.8	HC2000	59.7	Install: 3 VWP
BH11	5870360	532037	530.9	HC2000	70.2	Install: 3 VWP
BH12	5870272	531620	556.4	HC2000	70.2	Install: 4 VWP
BH13	5869938	532468	482.1	HC2000	30.5	Install: 2 VWP
2006 Drilling Program						
SI14	5869683	531277	572.3	B-80	100.3	Install: Inclinometer casing
SI15	5870278	531402	566.5	GT-180	100	Install: Inclinometer casing
BH8A**	5870043	532163	526.2	B-80	51.2	Re-install: 1 VWP
BH9**	5870120	531883	534.7	B-80	14.3	Re-install: 3 VWP
BH14	5869693	531276	572.0	B-80	100.7	Install: 3 VWP
BH15	5870278	531417	565.2	B-80	100.6	Install: 3 VWP
BH16	5869544	530634	660.7	HC2000	222.3	Install: 3 VWP
BH16A**	5869552	530634	660.3	HC2000	115.8	Install: 1 VWP

* Bottom 10' of casing left in hole during removal, resulting in installation of VWP 8B within casing.
** No log included for this hole as log was incorporated into original log (i.e. BH8A reconciled with BH8); listed here only because of slightly different location and/or installation.

The drilling encountered three major stratigraphic groups which have been classified according to the rock and soil unit described in Section 3.1. These groups consist of Pre-Tertiary Rocks, Tertiary Sediments and Volcanics, and Quaternary Sediments. Geological descriptions are included below of each group identified in the drill core and recorded on the drill logs. Note that descriptions and intersections from past drilling are also included in the discussion below.

3.7.2 Pre-Tertiary Rocks

Phyllite, (Cache Creek Group), MTcc

Phyllite is a foliated rock derived from medium-grade regional metamorphism of sedimentary rocks such as shale. Dark grey to black phyllite was encountered at the bottoms of boreholes BH15 and BH16 at elevations of 471 m and 543 m respectively. The rock was highly weathered with some siliceous lenses and quartz veins. The rock is typically very weak (R1) to medium strong (R3) and is highly fractured with joints spaced less than 100mm (RQD = 0). The unit contains sections up to 30 mm thick of low plasticity clays (Plasticity Index = 8.2) likely derived from the physical and chemical weathering of the phyllite. Relatively coarser grained layers in the layered phyllite sequence generally show a wavy or deformed bedding structure as a result of low grade metamorphism or soft sediment deformation of the parent sediments prior to lithification. Fractured surfaces and foliation planes generally have a greasy feel and may contain graphite. Lineations trending perpendicular to dip direction (described as crenulations) were observed on many fractured surfaces. Fractures and foliations varied in their orientation; however, they were typically 30° to 40° from the core axis (the holes were nominally vertical).

Siliceous White Shale, (Cache Creek Group), MTCC
Shale and siltstone are siliciclastic sedimentary rocks composed of silt and clay sized particles. White siliceous clays and siltstones were observed in drill holes BH15, BH4 and BH6. The unit consists of an upper gravelly zone which grades into well bedded shale and siltstone. The upper portion was primarily highly weathered chert which breaks into angular, gravel fragments. With depth, the formation becomes clay rich with siliceous seams and interbeds of chert. The base of the unit is characterized by white shale with cross-beds of siltstone dipping between 20° to 70° from the core axis.

3.7.3 Tertiary Sediments and Volcanics

Andesite, (Kamloops Volcanics), TKV
Andesite is a volcanic rock of intermediate chemical composition, similar to that of intrustive rock diorite (note that the grain size is much smaller for andesite). Dark grey andesite was observed locally at BH16 at a depth of 107 m to 110 m (551 to 554 m asl). The andesite was porphyritic with phenocrysts up to 5mm diameter in a dark greyish-blue, fine grained matrix. Slow initial cooling of the lava melt allowed the growth of the larger phenocrysts followed by rapid cooling which produced the fine-grained matrix. The rock was weak (R2) and highly weathered. When fractured, the joints display a plumose structure. Stratigraphically, the unit is an inclusion in the Tertiary tuff units and represents a relatively minor rock type. This rock may also be associated with volcanic activity during the placement of Australian Creek Group sediments, and therefore might also be classified as **TAC**.

Tuff, (Australian Creek Group), TAC

Tuff is a volcanic rock formed from the lithification of pyroclastic sediments such as volcanic ash. The volcanic tuffs encountered in West Quesnel typically consisted of bluish green, very stiff clays and silts with varying amounts of sand and gravel sized clasts with trace lignite (low grade coal) lenses. This rock type was observed at all borehole locations. The unit was derived from ash, lapilli and bomb sized pyroclastic fragments extruded during past volcanic eruption(s). The material was generally massive; however, in some areas it has been reworked by water or sedimentation-gravity processes. Localized intervals have bedding structures including fining/coarsening upward sequences and rare cross bedding.

The fine-grained clay and silt tuffs were very stiff, medium to high plasticity and massive, with a bluish-green colour oxidizing to olive brown. Fractured zones were common with polished, grooved, planar surfaces (slickensides) representing localized areas of brittle shear, while other softer sections may be subject to ductile shearing.

The clay tuffs were interbedded with coarser sections (up to 50 m thick) consisting of sand and gravel tuffs with a clay matrix. Clasts were composed of two main rock types: sub-rounded, very weak, clay-rich sand and gravel clasts of volcanic lapilli; and volcanic bombs aerodynamically rounded during flight. Other dominant rock types include sub-angular, pearly to black, resistant quartz (chert) gravel clasts. Cementation was weak as individual clasts could be easily removed from the matrix, with some minor zones that were highly cemented with calcite. The unit had relatively few fractures.

3.7.4 Quaternary Sediments

A blanket of Quaternary sediments overlies the Tertiary bedrock and was encountered at all boreholes at the West Quesnel site. These sediments are derived from the most recent Fraser glaciation (~20,000 year BP) and/or processes that have occurred since glacial retreat.

Till or colluvium has been deposited and consists of unsorted and non-stratified gravels and cobbles in a silt and clay matrix. Cobbles are of various lithologies but are commonly vesicular basalt, potentially originating from the Chilcotin Group upper Miocene flood basalts that cap the rock units west of the site.

Laminated silts and clays occur locally and suggest a glaciolacustrine (**L^G**) depositional environment. The varved or rhythmite bedding occurs as a result of seasonal changes and are expressed as couplets of silt and clay laminae less than 10mm thick. The coarser silt laminae were deposited when sediment-laden rivers entered the lacustrine environment at freshet, while the clay interbeds were deposited during the lower energy environments that prevailed throughout the rest of the year. Other sediments include minor amounts of well sorted glaciofluvial sands and gravels, and massive overbank silts (**F^G**).

3.7.5 Groundwater Conditions Encountered during Drilling

Groundwater conditions encountered during drilling were variable, typically dependent on drilling method, topographic position, and geological unit at each location. Observations of water encountered during drilling were only available where dry drilling methods were used, such as air rotary drilling, or where artesian conditions were observed in “wet” holes. Wet rotary drilling involves keeping the hole filled with fluid, beginning from the surface, and is not suitable to provide observations of water bearing zones unless they are under pressure and cause artesian flow. Locations at low elevations, such as at BH3, encountered high, or even artesian water pressures particularly when advanced to significant depths. Where the holes were advanced with air-rotary drilling methods through gravel deposits, such as those at SI8 and SI12, significant volumes of water were observed through water bearing zones. Where dry methods were advanced through fine grained, low permeability deposits, such as through the Tertiary sediments, water production from the holes was very low and discontinuous in nature.

The results of the groundwater observations made during drilling suggest that the gravels in the area carry significant and readily available volumes of groundwater. The Tertiary sediments have high groundwater pressures, as evidenced by occasional artesian flow, but groundwater is likely not readily produced in this formation.

3.7.6 Key Findings from Drilling

The key findings from the completion and analysis of the drill holes can be summarized as follows:

- The general stratigraphy of Quaternary sediments over Tertiary rock and dense soils was confirmed throughout the project area.
- The Tertiary deposits include weak clay soils with moderate to high plasticity produced by alteration of volcanic deposits. These are a key aspect to the overall stability and are discussed further in the report.
- The drill holes allowed for the installation of additional instrumentation.
- The stratigraphy encountered in the drilling generally agreed with the findings of the geophysical surveys, with the minor exceptions noted in Section 3.2.
- Groundwater production is expected to be high in the gravel deposits and low in the Tertiary Sediments.

3.8 2005/2006 INSTRUMENTATION

Instrumentation installed in 2005 and 2006 by AMEC included slope inclinometers, vibrating wire piezometers, and dataloggers. The number of installations, purpose for installation, results, and analysis of data is discussed below for each type of instrumentation.

To provide a comprehensive summary, AMEC has included installation detail drawings for all instrumentation installed throughout the project area in Appendix F.

3.8.1 Slope Inclinometers

A slope inclinometer (SI) is a geotechnical instrument which is installed and monitored in order to determine sub-surface deformation of the ground over time. The important components of an SI system are the casing, the survey probe, the readout, and the software. Once a depth or zone of suspected movement is identified or estimated, a hole is drilled deeper than the suspected zone so that the bottom of the system will be in stationary ground. Grooved inclinometer casing is then grouted in the hole. After installation, the SI is surveyed to establish an initial absolute profile of the casing. Future surveys of the casing profile are compared to the initial to examine any potential movements related to deformation of the casing. Any changes in the inclinometer casing profile indicate changes in the profile of the ground surrounding the casing; therefore, profiles from later surveys can be compared to those from earlier surveys in order to determine the rate and cumulative movement of the ground at different depths. In this way, SIs allow for the determination of the type, depth and direction (where discrete movements are observed) of ground movements at select intervals along the casing. As SIs must be installed across, and deform with, shear surface boundaries to measure ground movements, they necessarily have a limited useful lifespan that is related to the total slide movements. With continued deformations along discrete shear zones (typically at a slide shear surface), the casing ultimately deforms such that the survey probe cannot pass a shear surface intersection and are no longer useful beyond such a time.

AMEC has installed a total of fifteen (15) SIs within the slide, including SI1 through SI7, which were installed in the fall of 2000, and SI8 through SI15, which were installed in the spring of 2005. Table G1 of Appendix G summarizes the SI installation details, including the depth to which each SI was installed. Since installation, SI2 through SI7 and SI10 have become blocked and non-operational due to slide movement.

Plots of the SI readings to date are attached as Figures G1 to G32 in Appendix G. A second set of plots has been included for SI7, as the portion of the borehole above 68 m depth is still being monitored; however, these plots do not show the movements that are likely occurring below 68 m depth.

The SI readings are generally presented in incremental and cumulative displacement plots, with a 150 mm and 100 mm horizontal displacement scale versus depth. The incremental plots show the individual changes in inclination of the casing at each reading depth. Consistent sharp sideways spikes in the incremental data plots generally indicate a discrete shear plane (slide boundary). More general ground deformations, such as those expected within a moving slide mass, are visible as more gradual profile changes in the cumulative displacement plots.

The cumulative displacement plots add together the incremental changes starting from the bottom of the installation in order to show the overall apparent lateral displacement profile of the SI casing relative to its initial location. Consistent sharp kinks or bends in the cumulative displacement profiles typically indicate slide shearing surfaces.

Two channels (directions) of measurement are presented: A and B. The A channel is oriented to be approximately “down slope” whereas the B channel is approximately “across slope”, or 90º to the A channel. Positive movements on the cumulative plots are down slope (east) on Channel A and across the slope (south) for Channel B.

Analysis of SI data indicates that small movements of the slip surface have been detected over the past year, up to a maximum of approximately 30 mm in SI13. Based on analysis, the interpreted depth of the main slide surface has been determined for all operational SIs, with the exception of SI8, SI11, SI14 and SI15. It has been concluded that SI8 and SI11 are not deep enough to intersect the slide surface; SI14 and SI15, located near the top of the slide, have only been in service for a short period of time and have not yet shown enough movement to allow slide surface interpretation.

Table 2 below presents a summary of the interpreted casing displacement of each SI based on the data gathered to date. The SIs highlighted in green are currently operating; those shown in grey are no longer useable. Figure 9 shows the locations of the SIs, along with their displacement vectors at the deepest slide plane as of March 2007 or when blocked. A comparison of the life spans of SIs that have been blocked showed an average operational life of approximately 22 months, though one lasted at least as long as 48 months.

Movement by discrete shear along the slide surface is evident in the SI displacement plots included in Appendix G; this type of movement is only one component of the deformation that is occurring throughout the slide mass. A comparison of displacement amounts and rates is made across the discrete basal shear plane only, as all SI locations showed no spatial pattern in the amount or rate of movement measured by surface hubs across the slide. The ground surface undergoes more deformation than the slide surface as it is shifted by a combination of discrete shear movement along the slide surface and deformation of the slide mass above the slide surface.

Table 2: Slope Inclinometer Casing Displacement Summary

Installation No.	Operating Depth (m)	Slide Surface Depth[1] (m)	Operating Depth (m)	Maximum Casing Displacement (mm)[2]		Casing Displacement (mm)[3]	
				Vector (mm)	Azimuth (º)	Vector (mm)	Azimuth (º)
SI1	43.3	**41** *27.5*	43.3	51	100	**20** *51*	**108** *100*
SI2	-	**60** *41*	-	86	143	**41** *86*	**128** *143*
SI3	-	**39**	-	64	97	**64**	**97**
SI4	-	**51**	-	60	136	**60**	**136**
SI5	-	**37**	-	111	91	**111**	**91**
SI6	-	**28**	-	107	162	**107**	**162**
SI7	64.6	**68** *27*	64.6	113[4]	112[4]	**43** *113[4]*	**97** *112[4]*
SI8	71.3	TBA	71.3	14	52	-	-
SI9	79.2	**66**	79.2	43	94	**43**	**94**
SI10	-	**21**	-	11	78	**11**	**78**
SI11	70.7	TBA	70.7	57	42	-	-
SI12	72.5	**63**	72.5	149	81	**45**	**59**
SI13	39.6	**14**	39.6	34	82	**34**	**82**
SI14	99.4	TBA	99.4	19	74	-	-
SI15	90.8	TBA	90.8	13	53	-	-

1 Depths in **bold** represent the deeper slide surface, depths in *italics* represent secondary slide surfaces.
2 Maximum total lateral displacement measured at depth, from initial reading to present or until blocked.
3 Total lateral displacement measured about 1 m above the slide surface, from the initial reading or until blocked.
4 Displacements above 27 m depth relative to below 68 m only are reported after May 2002.

Cumulative deformation profiles in SI3, 5, and 11 show an irregular wavy pattern with depth. This is a frequently observed condition for SIs in many locations. Two explanations are offered for these profiles: either they represent compression buckling of the instrument related to ground deformations, or they represent casing movements associated with incomplete grouting of the casing at the time of installation. As these instruments are located in areas of neutral or net downward surface movements, compression due to ground movements is a likely cause. The compression profile could easily be extended below the basal shear surface of the slide due to the nature of the relatively stiff instrument being installed in relatively soft, weak grout. Furthermore, incomplete grouting or grout leakage resulting in voids in the grout does not usually result in casing deformation patterns such as those observed.

Key findings from the analysis of the SI information can be summarized as follows:

- Given the current information, the discrete shear displacements measured at the base of the landslide are not directly comparable with the rate of deformation of the surface movement hubs.
- The horizontal movement directions at the slide surface appear to indicate spreading type movements, similar to that seen in the surface hubs.
- General deformation throughout the slide mass above the discrete basal shear zone appears to be occurring.
- There are discrete shears in the slide mass above a main basal shear zone in some instances.
- The SIs can be expected to have an average life span of approximately 22 months, depending on local rates of movement.

3.8.2 Piezometers & Dataloggers

AMEC has installed a combination of conventional standpipe piezometers (miniature well) and vibrating wire piezometers (VWP) at West Quesnel. Conventional standpipe piezometers generally require high water volume movements to reflect true groundwater conditions, and in low permeability ground environments can show significant time lag between observed and actual groundwater pressures. VWPs require relatively low entry volumes of water to reflect groundwater pressure changes and are generally more reactive in low permeability ground conditions and were subsequently preferred for use in West Quesnel.

A vibrating wire piezometer (VWP) is a small electronic instrument attached to a communication wire that is used to monitor pore water pressure at a pre-determined location in a drill hole. A VWP is installed in a drill hole and sealed in place with cement-bentonite grout. When it is excited by a small voltage, a thin wire stretched across the VWP produces a characteristic frequency that is a function of the groundwater pressure acting on the tip. In this way, groundwater levels can be monitored with time. In order to simplify the monitoring of VWPs and allow for more frequent readings, electronic dataloggers were used. These devices are located at the surface and are connected to the VWPs at all times. A datalogger, in the simplest form, is a recording device that acquires and stores information from one or more sensors at pre-determined reading intervals. The dataloggers being use in West Quesnel are VWP mini-loggers or digital recorders; they consist of a power supply, a microprocessor, sensors, and a flash memory for storing data and instructions.

AMEC has installed thirty-three (33) VWPs throughout the study area; this is in addition to fifteen (15) standpipe piezometers that are monitored at regular intervals by measuring the depth to water level in each standpipe. Table H1 of Appendix H summarizes the standpipe and vibrating wire piezometer installation details for the study area. Figure 1 shows the locations of the groundwater measurement installations. Single channel VWP mini-loggers were installed at BH3 and BH4 and programmed to record daily pore water pressure readings. This data was downloaded during monitoring visits and subsequently converted to equivalent groundwater levels.

The most recent installations (BH7 through BH16) have multi-channel dataloggers in weather-proof housings locked inside tamper-proof cases above the drill holes. For this project, the dataloggers are used in tandem with a vibrating wire interface that has a number of VWPs attached to it. The interface is used for signal conditioning and signal amplification from the VWPs, and it allows for up to four sensors to be attached to one datalogger. Between two and four VWPs are installed at various elevations in the drill hole located below the datalogger for each installation in the study area. Photos of the dataloggers and field installation are included in Appendix H. Programs for the dataloggers are provided by the distributor; however, these programs had to be modified by AMEC and the dataloggers are currently functioning with reading intervals set at between 1 and 12 hours. The results of the monitoring to date, including graphs displaying water level versus time for the various installations (including standpipe piezometers), are shown on Figures H1 to H19 of Appendix H. Note that due to apparent equipment malfunction at BH8, the groundwater levels used for the purposes of this report are taken from manual readings prior to the installations of the dataloggers. Figure H20 of Appendix H summarizes the monitoring intervals at which data was collected from the instrumentation.

On a large scale (10s of metres), measured groundwater levels recorded in the VWP instruments remained relatively constant and similar to previous years' levels. The levels typically indicate that the phreatic surface (elevation to which water would rise in an open well) was near the ground surface in most cases. Where several VWP instruments are located at similar locations, whether nested in a single hole or included in offset holes at a single location, the instruments at various depths within the Tertiary unit show similar total head conditions. This shows that the groundwater is moving laterally through this unit.

On a micro-scale, measured groundwater levels recorded in the longer term VWP instruments appear to follow trends in the cumulative difference precipitation pattern. This pattern was observed by reviewing the long-term VWP installations at BH3 and BH4. Note that the 2005/2006 VWP instrumentation was not activated and operational until the late fall of 2006, and does not provide suitable long-term data to be included for micro-scale review yet. The results of the correlation are provided in Figure 10. As Figure 10 shows, the groundwater pressure level plots of the VWPs have shapes with some apparent correlation with the cumulative precipitation difference pattern. Specifically, review based on the limited dataset of concurrent slide movement, precipitation, and porewater pressure response indicates the following patterns:

1. Consistent rises in groundwater pressures that **lag** periods of increasing cumulative precipitation difference (wetter than normal precipitation periods).
2. Consistent reduction of groundwater pressures that **lag** periods of decreasing cumulative precipitation difference where no steady periods exist in the cumulative trend between rise and fall (drier than normal precipitation periods).
3. Consistent reduction of groundwater pressures that **occur during** periods of constant cumulative precipitation difference (normal precipitation periods).

The latter point above potentially suggests that groundwater drains from a "steady system" and that the pressure change is controlled instead by the rate of change of cumulative total precipitation difference. A deviation from these trends was observed at BH4, and is discussed below.

For the purposes of discussion, the surface hub movements trends presented in Section 3.6, above have been added to Figure 10 to establish potential correlations between the long term precipitation pattern, VWP readings, and slide movement rates. A correlation between these three factors appears to exist. Specifically, it was observed that higher movement rates were recorded during periods where total head (groundwater pressure) values exceeded 500 m elevation in the VWP installed in BH3. Note that data from BH3 has a range of between 498.5 and 501 m.

Note that the VWP pressure profile at BH4 shows a significant upward rise and fall in pressure between January 2003 and January 2004. This trend is a general departure from the drying precipitation pattern and likely indicates water pressure increases in this area from an alternative source such as increased irrigation (unlikely given duration of pressure increase), broken underground services, or other such sources that could introduce undetected water over an extended period of time.

The potential correlations observed in Figure 10 and discussed above should be examined as more data is collected from the site from a newly established local climate station, the expanded network of VWP instruments and the GPS hubs.

Significant findings from the groundwater monitoring program to date can be summarized as follows:

- Groundwater pressures are high throughout the area. Typical groundwater pressures at depth indicate a phreatic surface at or near the ground surface. This is a very important result, particularly considering the weak soil/rock present at depth in the Tertiary formations.
- VWP installations that are secured in the soil profile tend to show relatively steady macro-scale long-term levels with little variation, while standpipe piezometers that are open to the atmosphere appear to fluctuate more significantly with time. The VW data quality is considered more accurate than the standpipe data.
- Review of the long-term micro-scale VWP records at BH3 and BH4 appears to show a correlation between long-term climate records, VWP pressures, and slide movement rates and shows potential pressure changes in response to non-climatic sources.
- Site specific correlations from VWP data at BH3 indicate higher movement rates where total head values exceed 500 m elevation.

3.9 LABORATORY TESTING

Note that some of the information provided in this section is taken from the 2002 report. It is repeated here to present a full picture of the testing that has been conducted to date, as some of the materials encountered and reported in past work have been reclassified.

3.9.1 Grain Size Analysis

Laboratory testing completed on drill core prior to the 2005/2006 drill programs included washed sieve testing with hydrometer grain size analyses. The purpose of washed sieve testing with hydrometer grain size analysis was to determine the approximate proportions of different grain sizes in the tuff unit. In general, the results of washed sieve testing were as follows:

- Gravel: 0 to 41%, with a typical value of approximately 20%
- Sand: 7 to 72%, with a typical value of approximately 50%
- Fines (silt and clay): 7 to 93% with a typical value of approximately 30%

The results of hydrometer grain size analysis showed that clay was present in proportions of 18 to 36% of the fine component of the samples tested, while silt was present in proportions of 48 to 79% of the fine component. It should be noted that these test results were for relatively large volumes of core and that substantial variations between fine bedding layers would be averaged out.

The strength properties and behaviour of a soil with even a seemingly small amount of fines content can be dictated by the properties of the fine material. The significant portion of fine material present in the samples that were tested is indicative that overall soil behaviour will be very much controlled by the strength properties of the silt and clay components; therefore, the results of Atterberg limit index testing are a key component in determining soil characteristics at this site. While silt is present in greater amounts than clay, there is generally enough clay content for clay mineralogy to significantly affect the overall strength of the tuff unit, which indicates that X-ray diffraction (XRD) results are also very useful in determination of soil characteristics. Both Atterberg limit index testing and XRD testing are dicussed below.

It should be noted that the mode of deposition of the volcanic ash that gave rise to the smectites discussed above was by sedimentation, either through water or air. This frequently results in very widespread thin layers of weak material. The weak material can potentially form a sliding surface, even though it is very thin. Most of the available testing procedures such as grain size analysis and Atterberg Limits tend to "average" out the results over a larger volume of sample than is typically represented by the thin layers.

3.9.2 Atterberg Limit Index Testing

Atterberg limit index testing is a method of determining the consistency of a fine-grained soil based on its moisture content. Two limits are tested for: the liquid limit and the plastic limit. The liquid limit of a soil can be generally considered as a measure of the moisture content above which a soil begins to behave as a liquid; the plastic limit of a soil is a measure of the moisture content below which a soil behaves as a solid, and the range between is where the soil behaves in a plastic manner. The moisture contents for the respective limits are a function of mineralogy and porewater chemistry. A classification system based on the plasticity index (liquid limit-plastic limit) and the liquid limit indicates the plasticity of the soil. A high plasticity soil will be more likely to undergo permanent (plastic) deformation under loading than a low plasticity soil and tends to have lower shear strength. The natural moisture content of a soil gives an indication of whether it is near the plastic or liquid limit in situ, in short whether it is likely to exhibit liquid (flow) behaviour or plastic (brittle) behaviour if disturbed or loaded.

A total of forty-three (43) intervals were sampled from the 2005/2006 drill core and submitted for Atterberg limit index testing and natural moisture content determination. This was in addition to twenty-four (24) intervals from previous drill core which were submitted for Atterberg limit testing and natural moisture content determination in the past. The results of both sets of testing are presented in Table E1 of Appendix E.

Sampling locations were chosen based on one or more of the following sampling rationales:

1) Fractured and slickensided clay tuff zones were sampled, as they showed direct evidence of shear displacement. Slickensides are parallel lineations indicative of shear movement and are generally found on smooth, polished fracture surfaces produced by relative motion across discrete shear planes. Atterberg values at these locations may be representative of the plasticity properties along a weak slide surface, assuming slide movement is occurring on a discrete plane.

2) Soft or highly plastic clay tuff zones were also identified and sampled. The clay tuff unit has a primarily very stiff to hard consistency with minor soft and moist areas. The softer locations were sampled in order to gather data along potential slide surfaces that may be deforming in a ductile manner as opposed to those on discrete shear surfaces.

3) In areas where holes were drilled adjacent to areas with displacement measured by existing slope inclinometers, samples were collected for Atterberg and moisture testing close to zones of indicated displacement. This was to identify soil and rock plasticity properties along or near the slide boundary.

4) Other sampling intervals were selected to collect Atterberg and moisture data from typical or 'average' soils and rock to allow for correlation to areas of suspected or measured displacement.

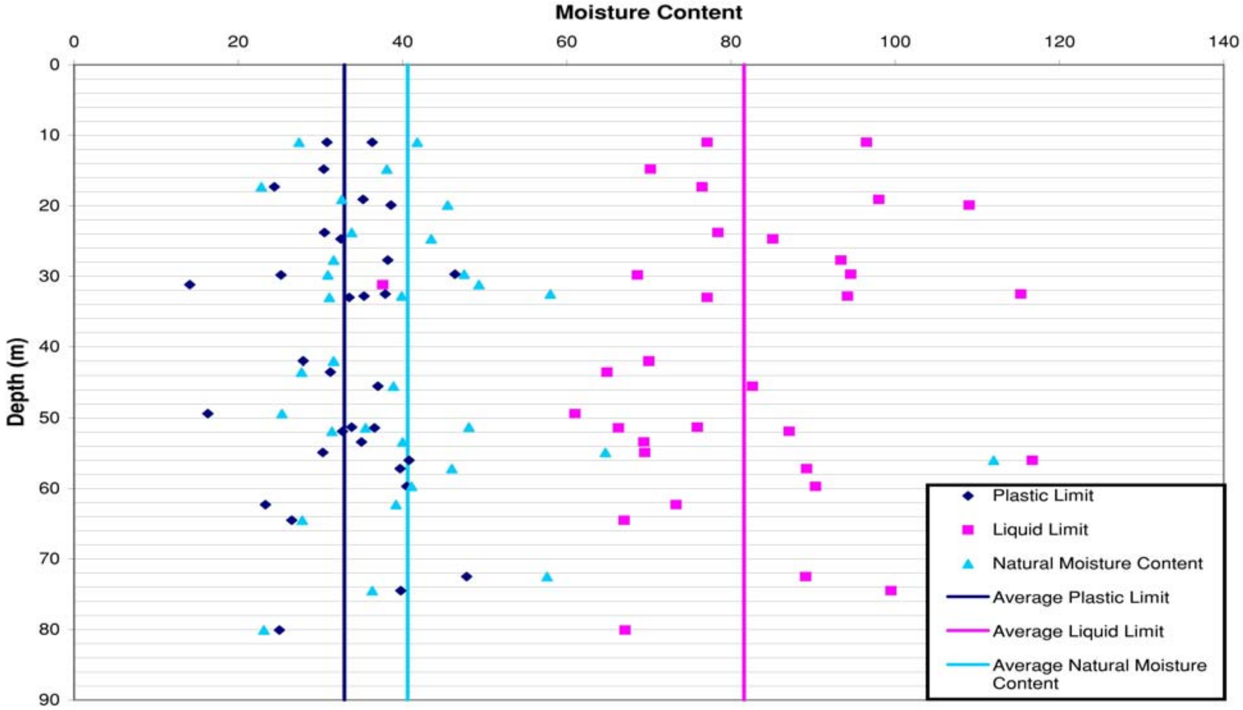
Slickensides have generally been interpreted as an indication of shear movement in the tuff deposits. It should be noted; however, that certain consolidation and other influences can also give rise to slickensides.

The results of Atterberg limit index testing showed a range of different soil plasticities for the samples tested. In an attempt to find correlations between plasticity and other properties, plasticity charts were created showing samples grouped by borehole, soil type, and elevation in metres above sea level. Refer to Figures E1 to E3 in Appendix E for these charts. The chart showing samples grouped by borehole does not show a clear correlation between plasticity and borehole, except that nearly all of the samples taken from the middle or toe of the slide area yielded high plasticity results. The chart showing samples grouped by soil type shows that shale has low plasticity, overburden till and lacustrine soils generally have medium plasticity, and clay tuff has high plasticity. Tuff samples taken from near the interpreted slide surface yielded particularly high plasticity values. The chart showing samples grouped by elevation shows that soils are generally high plasticity from 400 to 600 m asl, with medium to high plasticity values found at elevations greater than 600 m asl.

Figure 11 shows the natural moisture content, plastic limit, and liquid limit results for the tuff unit plotted by depth. Approximately 66% of the natural moisture contents are between the average liquid limit and plastic limit for the samples tested, while approximately 31% are below the average plastic limit. The plot shows that natural moisture content, plastic limit, and liquid limit remain generally similar with depth, as no clear trends are seen in the profile. It should be noted that a significant amount of time passed between drilling for sample collection and natural moisture content testing. This likely led to inaccuracy in natural moisture content results; the moisture contents were probably higher in situ than the test results show. In general, the natural moisture contents of the majority of samples tested fall between the plastic and liquid limits, and moisture content does not vary based on depth. Based on these findings, we expect that the tuff unit along and above the slide surface is behaving as a plastic material and is therefore undergoing plastic deformation.

Following Atterberg limit index testing, samples with high plastic limits were considered good potential candidates for mineralogy testing such as X-ray diffraction testing in order to explore the connection between plasticity and mineralogy; this testing is discussed below.

FIGURE 11: Plasticity Properties of Tuff Unit



3.9.3 X-Ray Diffraction Testing

X-ray diffraction (XRD) is a method of analyzing mineral structure by passing x-rays through a material and examining the angle and intensity of the diffracted rays when they strike the material at varying angles. By performing XRD on a fine-grained soil sample, the proportions of various minerals that make up the soil can be determined. This is particularly useful for a clay-rich soil. There are many different clay minerals, including kaolinite, chlorite, illite, and montmorillonite (or smectite). These different clay minerals have different properties from one another, and can influence the overall strength properties of a soil even when found in minor amounts. Most importantly in this particular case, soils with a significant clay content of certain minerals such as montmorillonite/smectite will generally have high plasticity and very low shear strength due to the clay mineral properties; therefore, the mineralogical breakdown of a soil found through XRD can yield important information about the strength properties of the soil.

AMEC collected five (5) samples from the 2005 and 2006 drill core and submitted them to the Department of Earth and Ocean Sciences at The University of British Columbia for XRD analysis. This was in addition to 2 (two) samples from previous drill core which were submitted for X-ray diffraction in the past. The results of the past testing showed smectite mineral contents within the clay fraction of 31% and 50% for the two samples tested. The Tertiary unit (clay tuff) samples with the highest plasticity values were selected for XRD from the 2006 core in order to look for correlations between high plasticity and clay mineralogy; several of the samples selected were from zones in which slickensides were noted, indicating proximity to the slip surface. The result was that the abundance of clay minerals from the montmorillonite/smectite group inhibited testing and did not allow for proper quantitative analysis; this provides qualitative evidence that the high plasticity samples from the Tertiary sediments (tuff) have a relatively high montmorillonite/smectite content. Refer to Appendix I for the UBC laboratory report.

Based on the XRD results, it is concluded that montmorillonite/smectite is generally the dominant clay mineral present in the tuff unit; therefore, montmorillonite/smectite properties of low strength and high plasticity likely dominate the behaviour of the tuff soil near and along the slide surface. Read (1998) found a very similar result when examining the lithology and clay mineralogy of similar soil found at a slide near Williams Lake, BC. In studying the stratigraphy of the site, Read reported a very similar soil profile to that which has been found at the West Quesnel slide location. A Tertiary (Eocene) sediment layer near the slide surface which was similar to the tuff layer at the West Quesnel location was sampled for XRD testing. The XRD results showed that montmorillonite was the overall dominant clay mineral in the tuff sediments. Read concluded that "[t]he widespread occurrence of montmorillonite provides a mineralogical explanation for the high plastic and liquid limits obtained for the samples tested…" The mineralogical similarity of the Tertiary units at these two sites indicates a consistency in the regional geology of Central British Columbia. Other work by AMEC has indicated that the Australian Creek Group may extend as far as Williams Lake and so this similarity is logical.

3.9.4 Direct Shear Testing

A direct shear strength test was completed on a sample from drill core taken prior to the 2005/2006 drill programs. The purpose of the direct shear test was to determine the shear strength of the soil along the slip surface of the slide. The direct shear test showed a residual friction angle of approximately 15º for the soil, which is considered up to 10º too large for high plastic smectitic silt and clay soil of the type likely found along the slip surface.

An ideal direct shear test would be carried out on a sample of soil that bridged the slide surface. Due to problems recovering disturbed soils while drilling through an existing slide plane, direct shear tests are typically carried out on relatively undisturbed soils, as in this case. Without a pre-existing shear surface, one is created for the test once the sample is loaded. Residual strength is by definition the strength of a soil that has undergone large strains, and in the case of strengths of a slide surface, uni-directional strain. Under natural conditions the strength of the contact soils at the shear surface is reduced by preferential particle arrangement, size reduction, and potential alteration. A standard direct shear-box test is not believed to be capable of reproducing similar slide surface conditions, resulting in higher measured strengths.

Several other methods for establishing the strength of the Tertiary sediments were reviewed, and as a result, it was concluded that the weakest plane of material (at the slip surface) was likely not acquired for the direct shear test. The results of the direct shear test were deemed ineligible for use in analysis. While this test was not considered successful, it is described here simply to acknowledge that it was completed and to highlight the reason for its exclusion from analysis.

Key findings from laboratory testing can be summarized as follows:

- The clay component of the tuff unit is large enough to dominate the behaviour of the unit.
- The majority of the tuff unit has a natural moisture content that causes it to behave as a plastic material; therefore, the tuff unit is undergoing plastic deformation.
- The high montmorillonite/smectite clay content of the samples tested indicates that the tuff unit likely has low strength.

- Overall, the mineralogy and geological conditions are compatible with very low shear strengths along the slide surfaces.

3.10 SHEAR STRENGTH

In order to determine the correct strength parameters for use in modeling of the tuff unit, an in-depth literature review was undertaken.

A Mohr-Coulomb strength criteria is often used for modeling soil strength and is appropriate for the West Quesnel slide. The frictional component of the shear strength is described as the angle of friction, which in simple terms represents the angle at which a block would slide down an inclined surface with no water pressures present.

For materials that are undisturbed, a “peak” shear strength may be appropriate. For materials that have undergone large movements of a few meters or more, a “residual” angle of friction (ϕ'_r) is usually appropriate. The residual angle of friction is typically much lower than the peak angle. As the material moves from peak to residual strength, the cohesion typically is also reduced to near zero.

There are several factors which contribute to the conclusion that large strain has been placed on the tuff unit in the study area, including the following:

- Glacial activity, including the pressure of glacial ice, isostatic rebound during glacial retreat, and lateral strain due to valley wall erosion have all acted to produce deformations in the soil.
- Significant strains are indicated by the presence of polished, planar surfaces found within the soil during drilling. As indicated above, this is not proof of large movements, but it is a likely indication.
- The LiDAR imagery indicates appreciable movements have occurred in the past.
- The slide is already moving with up to 100 mm of movement per year indicating that there has not been an opportunity for healing to occur that may result in higher operational friction angles.

Kenney (1967) conducted detailed direct shear testing and mineralogical/compositional analysis to determine if there was any relationship between soil mineralogy and shear strength. Kenney determined that for a soil containing greater than 20% montmorillonite in a fresh water environment, ϕ'_r typically ranged from 5º to 8º. Pure montmorillonite has yielded ϕ'_r values as low as 4º. Similar work by AMEC and others has indicated that low (<8º) residual friction angles may also occur in illite-kaolinite clays. A literature review including: Coduto (1999); Mesri, Peck and Terzaghi (1996); Lambe and Whitman (1961); Stark, Choi and McCone (2005); Tiwari and Marui (2005); Kakou, Shimizu and Nishimura (2001); and Stark and Eid (1994) confirmed a typical ϕ'_r range of approximately 7º to 13º for a soil with similar mineralogy to the tuff sediments present at the West Quesnel slide, with 5º being a reasonable worst-case value.

4.0 GEOLOGICAL & HYDROGEOLOGICAL MODEL

The geological and hydrogeological modeling presented in this report is a summary of the understanding of the overall structure of the ground, groundwater flow patterns, material properties and the nature of the slide movements in terms of type and apparent controls of movements. The model includes:

- The geometry of the underlying geological materials (stratigraphic relationships).
- Properties of the materials, including strength and groundwater pressures.
- Regional inputs for groundwater flow (Climate and Infiltration Model).
- Geometry of the slide surface and characteristics of the slide movements.

This section presents the details of the geological model and further combined analysis of the data with respect to the components of the geological model.

4.1 STRATIGRAPHIC RELATIONSHIPS

The geometry and distribution of the geological materials within, and adjacent to, the West Quesnel Slide are shown graphically on three representative sections, Sections A, B, and C, through the slide area. The plan and profile locations of the sections are shown on Figure 12, attached.

Simplification of material distributions is required for the continuum style numerical modeling that is used to examine slope stability and groundwater flow in the area. For the purposes of the numerical modeling portion of the study, four (4) main material types are used:

- **MTCC**, Cache Creek Group bedrock
- **TAC**, Australian Creek Group tuff
- **L^{G}**, Quaternary glaciolacustrine sediments
- **F^{G}**, Quaternary glaciofluvial sediments

4.2 MATERIAL PROPERTIES

4.2.1 Soil Strength

Soil strengths used in modeling were determined based on several sources of information including observations from the drill core, laboratory testing, correlations presented in published literature and ultimately through a numerical back-analysis of the existing stability conditions. Note that soil strength discussions are limited to the tuff, as the slide surface is within the tuff over most of its length; the strengths of other units used in modeling were chosen based on typical values for similar materials, as shown in Table 3, below and are not expected to have a major influence on the results.

The index and mineralogy tests were used to establish correlations with published references of residual shear strength in high plastic clays or similar Tertiary sediments. This procedure provided potential bounds on the residual strength of the soil between 5° and 13°. The final step in determination of the soil strength for the purposes of numerical modeling was to complete a numerical back-analysis of the slide, considering that the current unstable conditions represent a Factor of Safety against sliding not higher than 1.0. The results of the back-analyses indicated residual strengths near the lower end of the predicted range of strength, as shown in Table 3, below. The back-analysed strength would vary with pore pressure – the pore pressures used in the analysis were based on the measured values as discussed below.

Table 3: Soil Strength Parameters

MATERIAL TYPE	UNIT WEIGHT γ (kN/m^3)	EFFECTIVE STRENGTH PARAMETERS	
		RESIDUAL FRICTION (ϕ_r')	COHESION (c')
MTCC	20	20º	0
TAC – Tuff	20	6º See Note	0
L^G	15	25º	0
F^G	20	34º	0

Note: The results of 3 separate back analyses provided 6°, as presented in Section 5.1

4.2.2 Groundwater Flow

Groundwater pressure data collected from VWPs was used to establish known initial conditions in three 2-dimensional numerical groundwater models. The purpose of the numerical groundwater models was to define the characteristics of a non-uniform groundwater pressure distribution throughout the entire stratigraphic section where only select data was available. Seep/W, a component of GeoStudio 2004, Version 6.20 produced by GEO-SLOPE International, Ltd., of Calgary, AB, was used to construct the groundwater models for this study. The model locations were chosen to be coincident with the sections chosen for slope stability analysis.

Numerical groundwater models are based on the relationships between:

- hydrogeological material properties,
- groundwater pressure information taken from discrete points within the section, and,
- assumed or known conditions that exist at the boundaries of the section.

The model is computer generated, and consists of an artificial framework of interconnected points (nodes) that are created based on user-defined patterns, density and distribution throughout a stratigraphic sequence. The type and nature of the models (steady-state, finite element methods) used in this study were such that the results could be directly incorporated into a slope stability analysis. A slope stability analysis that includes a non-simplified groundwater pressure distribution is believed to provide a more realistic estimate of stability conditions.

Groundwater pressures based on readings taken on March 9, 2007 were used to construct the models, with the exception of those at BH8. Due to the potential instrument malfunction at BH8, groundwater pressures taken on Nov 8, 2006 were used. Stratigraphic relationships used in the models were taken from the coincident sections shown on Figure 12.

The plan view in Figure 13 presents a graphic version of the VWP groundwater pressure data used in the models. The figure includes descriptions of the observed groundwater conditions at each instrument, or group of instruments where they are nested at each site. The groundwater pressure measured by each VWP instrument is provided graphically by showing the elevation to which water would theoretically rise above the instrument. The relative differences between the elevations (i.e. pressures) shown on the graphic logs for each multiple instrumented site provides an indication of the groundwater flow pattern (flow from relatively higher to lower pressure). It is observed that gentle downward gradients exist across the site for the most part. However, as there is often little difference between instrument readings in the lower geological units, there is evidence of near lateral gradients in the underlying tertiary sediments and Cache Creek group rocks.

For the purposes of model construction, the following data was used:

1. Hydraulic conductivity of the Tertiary sediments was estimated in the 2003 pumping well tests in the range of 10^{-6} to 10^{-10} m/s. Based on that range, 1.5×10^{-8} m/s was selected as a reasonable value for this material.
2. Each groundwater pressure measured at the VWP instruments was entered as a fixed total head condition at a node coincident with the instrument position in the section. The total head elevation datum was taken as 0 m asl.
3. A fixed total head condition equivalent to the average water surface elevation of 472 m was entered along nodes coincident with Baker Creek near the toe of the slope.

For the purposes of model construction, the following assumptions were used:

1. Hydraulic conductivity of glaciofluvial, glaciolacustrine and Cache Creek Group rocks were not measured directly. Values used represent published material properties from the software manufacturer available for coarse grained soils (glaciofluvial), finer grained soils (glaciolacustrine) and open coarse soils (Cache Creek Group rocks). The values used should be treated as order of magnitude estimates only as no in-situ measurements were available. A thorough review of core recoveries, condition, and observed or typical performance in typical materials were used to assess the validity of the values used. A summary of the basic properties used are provided in Table 4.
2. Glaciofluvial soils are assumed to be isotropic. Note that this unit typically comprises generally coarse grained layered materials that are believed to have discontinuous layers and potential mixed conditions. For simplification, this unit also includes occasional inclusions of till, colluvium or other structureless finer-grained units that are believed to be discontinuous in nature.
3. Glaciolacustrine soils are assumed to be anisotropic as they typically have higher permeability parallel to bedding compared to across bedding. The anisotropic condition is typically such that the horizontal permeability is estimated as being 10 times higher than the vertical.
4. Australian Creek Group tuff is massive and believed to have a closely spaced, random fracture network that controls flow. While there may be short range anisotropy, on the large scale it is thought to be appropriate to model the Group as an isotropic low permeability system.
5. The upper part of the Cache Creek Group is thought to have a relatively high hydraulic conductivity due to its high degree of fracturing, and was modeled as isotropic. Groundwater, especially on a regional scale, will preferentially flow through this unit over lower permeability units such as the tuff. Flow will occur equally in the horizontal and vertical directions. Flow in this unit may be controlled by larger fault-controlled flow systems in the region.
6. A no-flow boundary was used for the base of the model.
7. Estimated constant head boundary conditions were developed at the sides of the model. The conditions were based on observations from BH16 (near west side) and the position of Baker Creek (near east side). Approximations were needed where the east sides of the model did not approach the background VWP.
8. Unit influx was established across the top of the model, based on the average infiltration presented in Section 4.3, below.

The results of the steady-state groundwater model are presented on Figure 13. They show that there appears to be a strong lateral gradient through the site. The gradient is largely controlled by water moving through the underlying Cache Creek group rocks since the permeability is relatively high. The large flow through the Cache Creek rocks is considered feasible as the Cache Creek Group rocks form a valley in the area up to about 15 km wide. Outside of this underlying bedrock valley, the Cache Creek rocks are generally expected to be located near surface, allowing for recharge to the groundwater system from precipitation. In this scenario, precipitation enters the system of Cache Creek rocks in the upland region to the west of the study area, well above 700 m elevation. If the rocks remain as disturbed as they are seen in BH16, it is conceivable that the groundwater pressures observed at the site are in response to precipitation recharge through this regional unit. This inflow of groundwater is an estimate only, and is the most uncertain piece of information in the groundwater modeling, however it supports the general pattern of high groundwater pressures and lateral groundwater gradients.

Table 4: Hydrogeological Parameters

MATERIAL TYPE	SATURATED HYDRAULIC CONDUCTIVITY K (m/s)	2-D ANISOTROPY ($K_{horizontal}$:$K_{vertical}$)
MTCC	4.3×10^{-6}	1 (Isotropic)
TAC – Tuff	1.5×10^{-8}	1 (Isotropic)
L^G	2.5×10^{-7}	10 (Anisotropic)
F^G	2.2×10^{-5}	1 (Isotropic)

4.3 CLIMATE MODEL

Based on the analysis of 30-year average calculations for precipitation in the study area, long term annual precipitation was found to be approximately 500 mm/year. In order to simulate the climate for modeling purposes, an estimated infiltration along the ground surface of 25% of the long term annual precipitation (assumed 125 mm/year) was used in the form of a unit flux of about 4×10^{-9} m^3/s/m. This was used as a baseline throughout modeling, but it is an estimate only and may require change based on future water balance work. Estimating infiltration of precipitation to the groundwater table is quite complicated as effects such as runoff due to paved structures, evaporation from forested areas, and additional infiltration due to human activity (such as lawn watering) are difficult to quantify.

4.4 GEOMETRY OF SLIDE SURFACE & TYPE OF GROUND DEFORMATIONS

A 3D spatial model of the slide surface was constructed using information from several sources. Examination of the LiDAR data indicated the location of the upper slide scarp, while INSAR mapping was used to help identify the location of the toe of the slide. Discrete zones of deformation in the slope inclinometer data, and identification of highly fractured and/or disturbed zones within the drill core were used to determine the depth of the slide surface at known locations across the slide mass. Linear interpolation between known locations provided a simplified model of the slide surface. At locations where slope inclinometers are determined to have been installed above the slide surface (too shallow and therefore do not cross the zone of shearing), the depths of the slide surface have been inferred. Final visual smoothing was applied to the calculated surface during construction of the cross section models. Figure 12 shows the location of the slide surface in the three cross sections.

Examination of the information in profile indicates that the slide surface has a very gentle, semi-circular shape, with a large nearly linear portion through the body of the slide that has a slope of approximately 5°. Note that this is very close to the residual shear strength estimated for this material. A close relationship between the average slope of the slide surface and the effective material strength indicates that even without water pressure (a parameter that acts to reduce the material strength), the slope would have a very low maximum factor of safety against sliding. Close similarities between the slide surface slope and the angle of residual friction have been found on many mature slides and is a further indication that the value of residual friction chosen is reasonable.

As noted above, the slope inclinometers in the West Quesnel Slide indicate that the overall slide mass is moving along a discrete basal shear plane. The ground movement is classified as a complex, rotational-translational type landslide. The GPS movement hub data indicates that the slide mass may be moving at a different rate on the surface than on the discrete basal shear plane. For this to occur, deformation within the slide mass must occur. This assumption is confirmed by the parallel-to-contour, hummocky terrain visible in the LiDAR model throughout the apparent slide area. It is postulated that the slide mass deformations are relatively plastic type movements, and are related to the spreading style of surface movements that are observed.

For spreading movement to occur, surface points would tend to diverge from each other, tending to create “tension” within the slide mass. Consequently, the slide mass should either have significant gaps or tension features such as grabens as one portion moves away from another (a brittle response), or the terrain should be generally subsiding to “fill the gap” as one portion pulls away from another (plastic response). The vertical response of the surface movement hubs (average vector dip between 10° and 11°) suggests that a significant component of subsidence is occurring in the centre portion of the slide from north to south. As there is significant subsidence as well as visible surface cracking in the area, the slide is believed to behaving in a plastic manner at depth where confining pressures and plasticity act to “fill the gaps” and in a brittle manner near surface where confining pressures are low and near surface material stiffness is relatively high. It is observed though that while surface conditions are the sum of both forms of deformation, they appear to represent the general direction and ratio of movements occurring on the discrete basal shear plane.

5.0 NUMERICAL MODELLING AND ANALYSIS OF SLIDE ACTIVITY

For the purposes of stability modeling, AMEC used drained, Mohr-Coulomb residual soil shear strength models in a Morgenstern-Price limit-equilibrium method. The stability model requires a groundwater pressure distribution that was estimated using a finite element groundwater pressure model that was compared to the piezometer data discussed above. All modeling was carried out using the analytical programs Slope/W and Seep/W, both components of GeoStudio 2004, Version 6.20 produced by GEO-SLOPE International, Ltd., of Calgary, AB.

Note that limit-equilibrium slope stability methods are based on examination of the relative balance of driving forces to resisting forces in a given slope expressed as the ratio of resisting forces to driving forces, a term defined as the Factor of Safety against sliding (FOS). In this definition it is clear that a slope would slide where the ratio (Factor of Safety) is equal to 1 or lower, but the factor of safety is related to the static forces only and does not indicate slide movement rate or amount of movement. An examination of the rate of movement based on limit-equilibrium methods is typically based on empirical relationships where movement is assumed to be significantly reduced, or even halted following an increase of 20 to 30% in the FOS from existing at failure conditions. Other strain-based analytical procedures are available; however, the necessary increase in complexity of the analytical method must be supported by equally complex and comprehensive data. As the slide area is very large and the geological and groundwater conditions are very complex, a suitable result from strain-based stability analyses might require significantly more investigation and instrumentation than is economically feasible. The authors present an alternative to this approach below.

The numerical modeling process was carried out based on three (3) selected sections through the slide. The sections were constructed based on the information provided in Section 4. The sequence of numerical modeling for each section was as follows:

1. Establish steady-state groundwater conditions to provide groundwater pressure distribution (using known pressures at VWPs) as discussed and presented in Section 4.2.2.
2. Perform a back-analysis of stability conditions (assuming FOS = 1.0) to confirm and refine the strength estimates for the tuff.
3. Review potential effects of pumping and/or horizontal drainage options in a transient groundwater model (using initial conditions established in Step 1).
4. Examine the potential changes in stability conditions (limit equilibrium methods) in terms of increase to the FOS based on the numerical dewatering trials.

Steps 1 and 2 are shown on Figures 12 and 13. Note that the position of the phreatic surface is relatively high in each section, and that the results of the stability modeling using consistent soil strength parameters across each section provide a FOS of 1.0. Preliminary trials were carried out for Steps 3 and 4 to examine the influence of dewatering. The results of these preliminary analyses indicated that significant periods of time would be required to measure changes in the groundwater pressures, particularly in the low permeability tertiary sediments.

It is significant at this point to return to the discussions of the limitation presented above in this section, and to the findings of Section 3.8.2, as presented in Figure 10, where it was shown that measured changes in water pressures on the order of metres resulted in apparent significant changes in rate of movement. This shows that measuring the effectiveness of dewatering using traditional limit-equilibrium slope stability modeling is limited. Note that if movement rates were even reduced to 1 mm per year, the limit equilibrium analysis would theoretically still indicate a FOS of 1.0, however achieving a long-term movement rate of 1 mm/year would be considered a very favorable outcome of any potential remedial solution in this case. Based on these findings, AMEC suggests that future analysis and ongoing monitoring should be focused on the combined datasets of precipitation patterns, surface movement rates, and measured groundwater pressures, and that potential dewatering solutions should be focused on achieving groundwater pressure reductions across the slide mass. Based on observations from Figure 10, a reduction in groundwater pressures that maintains a total head condition in BH4 below 500 m could provide that result. Additional review of data from the recent instrument installations would be required to establish similar threshold values. From the BH4 example, this could be a reduction on the order of single metres.

6.0 DISCUSSION AND RECOMMENDATIONS

The main objectives of the geotechnical work completed to date have been to;

1. Identify the nature and potential extents of the slide;
2. Identify the hydrogeological factors affecting the slide; and,
3. Use the knowledge from 1 and 2 to design remedial measures that act to reduce slide movements to a rate that is manageable and sustainable for current development in the area.

Points 1 and 2 listed above are considered complete. The findings of this study confirm that the West Quesnel Slide is comprised of a spreading slide mass that is moving along a discrete basal shear plane through weakly lithified pre-glacial volcaniclastic sedimentary rocks. Recent work has identified that the boundaries of the West Quesnel Slide are as described in past reports, with a potential extension to the southwest. The geological and groundwater characteristics have been defined over a broader area and in more detail than was previously available. Groundwater has been confirmed as a key destabilizing influence but has also been identified as likely the most effective (albeit challenging) target for mitigation.

To satisfy the outcome of point 3 above it should be noted that it is unlikely that the slide movements could be completely stopped; instead, a successful outcome would be defined as one that reduces the ground deformations to rates that do not have significant impacts on buildings and services. The focus of the work now is to continue monitoring, and to investigate potential groundwater control methods.

Based on the results of the observed groundwater conditions and movement rates to date, dewatering solutions capable of reducing pressures on the orders of meters across the site are judged to have the most potential for effectively slowing the average rate of movement to acceptable levels. Given the complexity of the site hydrogeology and prevalence of low conductivity geological units, a large scale and long term dewatering solution will likely be required, and a trial dewatering program needs to be tested prior to development and implementation of a full program. A proposed trial dewatering program is described below in Section 6.2.

The slide movement rate is sensitive to small changes in groundwater pressures that appear in part to be a function of long term cumulative precipitation. There is potential evidence that the groundwater pressures in the area are also controlled by non-climatic sources of water such as leaking water, sewer, or other such underground services. As the datasets grow, the correlation between groundwater pressure and cumulative precipitation (rate of change or total) are critical to forming the basis of evaluating ongoing movements, and might even be suitable in the long term to predict slide movements and remediation requirements. The recommendations for ongoing monitoring are provided below in Section 6.1.

6.1 RECOMMENDATIONS FOR ONGOING MONITORING OF SLIDE ACTIVITY

It is recommended that the City of Quesnel completes the following activities associated with the ongoing monitoring and analysis of slide movements in West Quesnel.

1. Continue with ongoing monitoring of all functional geotechnical and survey instrumentation that is installed at the site as shown in Table 5, below. The ongoing data collection should be supported by an annual engineering review and update of the data collected, as well as an update to the measured trends in groundwater pressures, surface hub movement rates, and cumulative precipitation trends.

Table 5: Recommended Ongoing Instrument Monitoring Schedules

INSTRUMENTATION TYPE	RECOMMENDED MINIMUM READING INTERVAL
GPS Movement Hubs (1998, 2001, and 2006 series)	Three times per year. (ideally in early spring, mid-summer and early fall)
VWP (all)	Visit monthly in 2007 and quarterly post-2007 to review power and retrieve data. Dataloggers provide daily information
Standpipes (all)	Maintain same schedule as VWP datalogger data retrieval periods. Data is not considered suitable to evaluate correlation to climate patterns/movement rates.
Slope Inclinometers (functional)	Quarterly. Attempt to time two sets of readings with GPS Movement Hub measurements.

2. Monitor the weather pattern in the West Quesnel area at the newly installed weather station. In the coming years, the patterns observed in the regional precipitation measurements used for this analysis (taken by Environment Canada at the Quesnel Airport) should be compared to the patterns observed in the data collected from a newly installed weather station located in West Quesnel. As the historic analysis of patterns is based on the weather data from the airport station, both sets of cumulative data should be reported annually until clear and consistent trends can be shown between the airport data and the West Quesnel data, or until additional evidence suggests another approach.

3. Establish a GIS database to track slide damage in the West Quesnel Area. Reported residential or commercial property or structure damage should be reviewed by AMEC or the City of Quesnel and records of site visits, photographs and summary reports should be kept in the database for future reference and analysis. Records of damage, inspection, and repair of City of Quesnel utility or other infrastructure should also be tracked in the database.

4. Commence a regular leak detection program for city water systems. The program should include a leak detection survey and a summary of inputs and outputs from City services.

6.2 TRIAL DEWATERING PROGRAM

A trial dewatering program is proposed for commencement in the early summer of 2007 and running to the spring of 2008 to investigate potential options for groundwater pressure reductions in the landslide's Tertiary sediments by:

1. Pumping water from vertical wells installed and sealed in glaciofluvial deposits that partly overly the Tertiary sediments.
2. Pumping water from vertical wells installed and sealed in Tertiary sediments.
3. Installing a horizontal drain into glaciofluvial deposits that overly the Tertiary sediments.
4. Installing a horizontal drain into the Tertiary sediments.

In summary, the program would include the drilling of the two vertical pumping wells and two horizontal drill holes as noted above, as well as additional monitoring instruments (VWPs) in an adjacent hole (or multiple holes) to track potential changes in the surrounding areas for a period of at least eight continuous months. The monitoring instrumentation would ultimately be incorporated into the overall geotechnical instrumentation of the site and would be installed in a similar fashion to the 2005/2006 VWPs described in this report. Proposed locations, including alternate sites for the trial dewatering program are shown on Figure 14, attached. Arrangements of test equipment, instrumentation, and drill hole and drain lengths are also included on Figure 14. Locations were selected based on topographic and geological requirements as well as constructability and site access. Discharge of the collected water will be directed and carried in impermeable liners, storm drainage or hoses to suitable discharge points to reduce reentry of water into the ground and disturbance to normal surface flow patterns.

In addition to the dewatering tests at the 4 locations above, AMEC also proposes to incorporate the decommissioning of an existing sump in Uplands Park (near BH9) into the dewatering program as well as renewed pump testing at the previous trial wells (PW1 and PW2).

It was reported to AMEC that a significant stormwater infiltration pit is located at Uplands Park. AMEC understands that the City of Quesnel plans to decommission the infiltration pit (via pump installation and diversion of water into the storm drainage system). As this work may provide additional effective dewatering, it is recommended that the work schedule be planned such that the decommissioning work coincides with the beginning of the trial dewatering program. Redirection of the collected water could be monitored by flow meter or other means, and potential effects on groundwater pressures could be tracked via the nearby VWPs in BH9, and/or in additional instruments installed directly at the pit site.

The existing trial pumping wells, PW1 and PW2, were initially installed and pumped without level controls on the water elevation in the pumping wells, allowing recovery between pumping each time. As a result, the drawdown created by the initial pump testing was not consistent over a long period. AMEC proposes to that pumps be installed in these locations again, and that these include level controls to maintain maximum drawdown for the duration of the trial dewatering program.

The results of the trial dewatering program are intended to provide answers, or ranges of values, for the following:

- What was the magnitude and rate of change in the adjacent or underlying groundwater pressures at each test site?
- How large was the zone of influence at each site?
- Was one method more effective than the other in terms of the size of the zone of influence, rate, or magnitude of potential change?
- Was one geological unit better than the other in terms of affecting changes?

The effectiveness of the trial dewatering will be measured in terms of observed downward changes in the groundwater pressure recorded at each test site that differ from those expected due to regional climate patterns. Note that if a significant drying trend were to occur during this period, it may be difficult to establish a direct correlation between dewatering and pressure change. The long duration (minimum eight months) of the test is expected to provide suitable time to observe measurable differences. Some additional VWP instrumentation may be proposed at the detailed planning stage to provide suitable records of the potential drawdown cone created by the continuous, long-term drawdown conditions at the test locations.

A detailed work plan for the trial dewatering program will be provided separate to this report. The plan will include technical program details, a proposed schedule, and cost estimate.

A joint surface hydrology, pond and water balance assessment is concurrently underway with Urban Systems Ltd. and the City of Quesnel. The results this assessment will provide additional refinements to the hydrogeological model, and may provide specific targets or measurements of success for dewatering options. The water balance assessment results will be incorporated with trial dewatering results to develop full scale dewatering options.

7.0 CLOSURE

This report has been prepared for the exclusive use of the City of Quesnel and their representatives for specific application to the area described within this report. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. AMEC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report was prepared in accordance with generally accepted geotechnical engineering principles and practice. No other warranty, expressed or implied, is made.

Please do not hesitate to contact the undersigned at (250) 564-3243 should you have any questions or require further information.

AMEC Earth & Environmental
A division of AMEC Americas Limited

Reviewed by:

Kathryn Black, EIT
Geotechnical Engineer

Drum Cavers, M.Eng., P.Eng., P.Geo.
Principal Engineer

Shane Kelly, M.Eng., P.Eng.
Senior Geotechnical Engineer

Nick C. Polysou, P.Eng.
Associate Geotechnical Engineer
Regional Manager, Central BC

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Appendix A
Atlantis Scientific INSAR Report

Appendix B
Surface Search Geophysical Survey Report

Appendix C
Climate and GPS Surface Movement Hub Data

Appendix D
Borehole Logs and Core Photos

Appendix E
Laboratory Testing Results

Appendix F
Instrumentation Detail Drawings and Photos

Appendix G
Slope Inclinometer Data

Appendix H
Vibrating Wire Piezometer and Standpipe Piezometer Data

Appendix I
X-Ray Diffraction Report