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WEST QUESNEL LAND STABILITY STUDY
QUESNEL, BRITISH COLUMBIA

Submitted to:

The City of Quesnel
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Quesnel, BC
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Submitted by:

AMEC Earth & Environmental Limited
Prince George, British Columbia

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EXECUTIVE SUMMARY

AMEC Earth & Environmental Limited has carried out a geotechnical assessment of land stability for a developed area located west of the Fraser River in the City of Quesnel, British Columbia. This report documents AMEC's geotechnical assessment activities carried out between September 2000 and September 2002. The following is a summary of the key conclusions and recommendations arising from the assessment:

1. A large and slowly moving ancient landslide (West Quesnel Landslide) underlies the study area. The western extent (upper main scarp) is thought to be at or near the height of land at the western edge of the study area. The toe of the slide is believed to be in the vicinity of Anderson Drive in the southern portion of the study area, and near Baker Creek in the northern portion. The northern boundary of the slide is essentially along Baker Creek. The southern boundary of the slide was not well defined and the slide probably continues or merges with similar ancient slide terrain to the south of the study area. The depth of the slip surface appeared to range from 28 to 68 m below the ground surface were intersected in slope indicator installations.

2. Sliding movement was recorded in slope inclinometers and GPS surface movement hubs. The movement was generally towards the east or southeast, at rate of approximately 66 mm/year (2.6 inches/year) in the Uplands area and 27 mm/year (1.1 inches/year) in the toe area. North of Lewis Drive there was a small northerly component to the movement, essentially towards Baker Creek.

3. Sliding appeared to be along a weak layer within pre-glacial Tertiary sediments that underlie more recent post-glacial soil deposits, including those associated with the Fraser River and Baker Creek. These Tertiary sediments appeared to be pre-sheared and may themselves have been deposited by ancient pre-glacial landslide activity.

4. Relatively high (in some places artesian) groundwater levels were evident in the slide area. Within the Uplands area, piezometers indicated a phreatic surface (equivalent groundwater level) along the slip surface that was at or near the ground surface.

5. It is expected that the displacements of the West Quesnel Slide will continue at rates similar to those experienced during the monitoring period. The actual rates of movement likely depend primarily on weather conditions (particularly precipitation). The slide is expected to continue to move very slowly (i.e. at rates of under 100 mm/year), unless there are significant and sustained changes in precipitation patterns (e.g., high precipitation for at least several weeks or months), or significant changes are introduced to surface water infiltration characteristics or the surface geometry of the area. Future movements of the slide will further deform and stress structures, roads, municipal and utility services within the slide area. It is judged unlikely that extremely rapid movements (i.e. movement rates exceeding 1.5 m/hour) will occur.

6. The most practical and cost effective way to improve the stability of the area and reducing (or even stopping) the slide movements is likely to be via extensive surface and sub-surface drainage measures, which are recommended in the body of this report. Further field trials are recommended in order to determine if these methods would be successful in application.

7. As it is judged that surface drainage measures on their own would be insufficient to reduce slide movements to a manageable level, AMEC also recommends that City of Quesnel pursue the option of subsurface drainage of the West Quesnel area via a system of pumped dewatering wells. Prior to implementation of a full dewatering well system, a pilot test well program should be carried out to confirm the effectiveness of such an option and to obtain the necessary hydrogeologic parameters for design and operation of a full dewatering system.

8. As the City of Quesnel considers and/or implements the various surface and subsurface drainage recommendations:

   - Monitoring of the slide area should be continued.

   - The City of Quesnel should continue to enforce appropriate development restrictions in the study area.

   - Significant changes to slope geometry, vegetation cover, and or surface runoff conditions should be reviewed by a geotechnical engineer prior to implementation.

   - Gas service lines should be regularly inspected for leaks.

   - Services such as water lines and sewers should be monitored for leaks since the introduction of additional water into the slide from these sources would be detrimental.

   - Additional survey, stream, environmental, and geotechnical assessments should be undertaken to design and construct a stabilizing rip-rap berm along the west bank of Baker Creek at the Baker/Healy Slide area.

   - Buildings identified or suspected as being in structural distress should be reviewed by both structural and geotechnical engineers to determine suitability or requirements for continued occupation.

9. Additional geotechnical investigation (drilling, coring, slope inclinometer installation) between the postulated toe of the slide area and the Fraser River, and north of the line of slope inclinometers (generally north of Lark Avenue) would be useful in further confirming the characteristics of the subsurface geology and slide extent.

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1.0 INTRODUCTION

The City of Quesnel has retained AMEC Earth & Environmental Limited (AMEC) to conduct a land stability study in the West Quesnel Area. The scope of service for the West Quesnel Land Stability Study was divided in 3 Stages:

Stage 1: Project planning, reconnaissance, data gathering, slope inclinometer installation and monitoring. The aim of this work was to determine whether or not deep-seated sliding was present, via a series of deep slope inclinometers installed in a line across the study area.

Stage 2: Detailed geotechnical drilling, sampling, piezometer installation and monitoring.

Stage 3: Analysis, remedial recommendations and reporting

Stage 1 of the West Quesnel Land Stability Study (WQLSS) was detailed in the AMEC report of 13 August, 2001 to the City of Quesnel. This current report summarizes AMEC's work to date for the West Quesnel Land Stability Study and forms the final report for Stage 2 and Stage 3 of the assignment.

Figure 1 shows the general study area location. Photo 1 also shows the general study area.

2.0 BACKGROUND

Since 1997, the presence of large scale landslide features in a suburban area of West Quesnel has been postulated by various geotechnical assessments. These assessments were largely based on airphoto interpretation, surface reconnaissance, and a review of reported utility breaks. Subsequently, lateral movements of up to approximately 250 mm were reported at GPS monitoring hubs installed in West Quesnel by BC Gas. Reviews of past work had been unable to conclusively determine whether or not the observed and postulated movements were due to seasonal frost heaving, shallow localized ground movements, larger scale deep-seated land sliding, settlement, measurement inaccuracies or a combination thereof. Detailed subsurface conditions in West Quesnel were unknown.

Given the lack of detailed subsurface information and uncertainty as to whether or not a deep-seated slide existed, AMEC was retained to provide subsurface geotechnical monitoring and assessment of the area.

3.0 METHODOLOGY

3.1 OFFICE REVIEW/BACKGROUND RESEARCH

For this assignment, the following documents have been reviewed:

1. Geotechnical Consultant Reports: Appendix A (References) lists the geotechnical consultant reports reviewed by AMEC. Reports reviewed included documents prepared by AMEC (and its predecessor companies), C.O. Brawner Engineering Ltd., GeoNorth Engineering Ltd., Golder Associates Ltd., Dr. N.R. Morgenstern and Dr. D.M. Cruden of the University of Alberta, R.E Graham Engineering Limited and Thurber Engineering Limited.

2. Geological Background Documents: Appendix A lists the various geological background reports, maps and documents reviewed for this work.

3. Ministry of Transportation (MoT) Files: Files from the MoT's Prince George geotechnical engineering office including available weather data in MoT fileswere reviewed.

4. Ministry of Water, Land and Air Protection (MWLAP) Files: Water well records posted on the internet by the Water Management Division of MWLAP were reviewed. The water well locations were shown on 1:5 000 scale groundwater location maps provided by MWLAP.

5. BC Gas Utility Data: AMEC has reviewed BC Gas GPS monitoring hub data which has been collected since September of 1998. In addition, BC Gas has provided AMEC with their line break data for the study area.

6. City of Quesnel Data: The City of Quesnel provided AMEC with records of utility line breaks and reports from local landowners regarding suspected damage to structures within the study area.

7. Airphotos: Various historical airphotos of the study area were obtained and reviewed. The specific airphotos reviewed are listed in Appendix A.

3.2 RECONNAISSANCE VISITS

On 27 September, 2000, Mr. Nick Polysou, P.Eng., Mr. Drum Cavers, M.Eng., P.Eng., P.Geo., and Mr. Doug Dewar, P.Eng., of AMEC, accompanied by Mr. Jack Marsh of the City of Quesnel, conducted a site reconnaissance visit to west Quesnel to review general site conditions and determine potential slope inclinometer (SI) locations. Following the site reconnaissance visit, seven potential SI locations were selected.

Mr. Nick Polysou, P.Eng., Doug Dewar P.Eng. and Shiloh Jorgensen E.I.T. of AMEC conducted an additional visit on 10 and 11 July, 2002, to review ground features, soil outcrops, and areas of reported damage to local structures and utility services.

3.3 SLOPE INCLINOMETER INSTALLATION

A slope inclinometer consists of a plastic pipe or casing installed in the ground that will deform in response to subsequent ground movements. Periodic electronic surveying of the casing allows one to detect the depth and rate of casing (and presumably sub-surface ground) movements. From 2 to 27 October, 2001, 7 boreholes were drilled to depths ranging from 43 to 158 m, using a compressed air-rotary water well drill supplied by Cariboo Water Wells of Prince George, BC. Mr. Doug Dewar of AMEC was present to conduct a preliminary log of soil conditions from returned drill cuttings and to supervise the installation of a slope inclinometer (SI) casing in each of the 7 boreholes. Photos 2 and 3 show the boreholes being drilled for SI-4 and SI-7. Photo 4 shows SI casing being installed into SI-7. Photo 5 shows the grout pump and grouting supplies used to fill the annulus between the casing and the borehole wall. The SI installation details are summarized in Table 1 below.

The locations of the SI installations were surveyed by Exton, Dodge and Galibois Land Surveying Inc. using a dual frequency GPS system that had a reported 10 mm horizontal and vertical accuracy. Figure 2 shows the slope inclinometer locations.

Table 1: Slope Inclinometer (SI) Installation Summary

	SI-1	SI-2	SI-3	SI-4	SI-5	SI-6	SI-7
Street	Avery Lane	Avery Lane	Abbott Drive	Voyager School	Abbott Drive	Dixon Street	Pierce Crescent
Date of Completion	25 Oct., 2000	23 Oct., 2000	27 Oct., 2000	13 Oct., 2000	5 Oct., 2000	16 Oct., 2000	19 Oct., 2000
A_o Bearing (°) (True North)	120°	120°	120°	110°	110°	105°	100°
Depth of Installation * (feet/m)	143 feet 43.6 m	245 feet 74.6 m	336 feet 102.4 m	520 feet 158 m	475 feet 144.8 m	507 feet 154.5 m	412 feet 125.6 m
SI pipe stick up above ground surface (mm)	720 mm	800 mm	820 mm	726 mm	779 mm	840 mm	660 mm
Elevation of top of SI casing (m A.S.L.)**	480.267 m	485.862 m	503.43 m	537.581 m	517.666 m	557.723 m	542.217 m
Elevation of ground surface (m A.S.L.)**	479.547 m	485.062 m	502.61 m	536.855 m	516.887 m	556.883 m	541.557 m
Northing**	5869298.105	5869342.656	5869598.068	5869878.155	5869633.237	5869933.443	5870231.729
Easting**	532568.712	532489.826	532273.0510	531829.52	531919.020	531509.906	531798.610

* The SI installation depth is deeper than the depth it is read (refer to Table 2) to avoid having the SI probe touch the bottom of the hole. The depth is measured from the top of the casing.

** World Geodetic System 94 datum

During the installation of BH-5, grouting problems were experienced during installation of the casing. Due to sloughing of the hole, it was not possible to lower the grout pipe below 73 m. It appears that after grouting of the hole, the grout may have drained past the obstruction, resulting in a zone where the casing was not grouted between approximately 35 and 90 m depth. This became apparent during subsequent monitoring and is discussed further below.

3.4 SLOPE INCLINOMETER MONITORING

Once the drill hole backfill grout had set (approximately 10 to 20 days after completion of SI installation), an initial reading of the profile of the installed SI casing was taken with an electronic SI probe. Subsequent readings were taken and compared to the initial reading to determine if any casing and hence ground deformation had occurred. Two mutually perpendicular movement directions were monitored in the SI casings, defined as A and B. The A direction was oriented parallel to the approximate direction of expected movement, in this case down slope toward the Fraser River. The B direction was oriented generally across slope, approximately perpendicular to the direction of expected down slope movement.

The casing profile was measured from the bottom upwards at increments of 2 ft, using an electronic SI probe that rides in grooves in the SI casing. At each depth increment, the inclination of the casing in the A and B directions was recorded. At each depth increment, comparing the change in inclination of the casing from the original reading allowed the movement of the casing to be calculated, assuming that the bottom of the casing was in stable ground. By adding up the incremental movements from the bottom of the casing, a complete profile of the change in inclination of the casing was calculated. Where a discrete slip surface cuts across the casing, movement along the slip surface will result in a change in the profile (inclination) of the inclinometer casing, allowing the location of the slip surface to be determined. In addition, plotting the relative horizontal displacement of a point on the casing above the slip surface to one below the slip surface versus time for a series of measurements allows a rate of movement to be determined.

To facilitate the handling of the large amounts of data from the field, the field measurements were recorded on an electronic DataMate readout unit (Photo 6) and transferred to a computer for processing. GTILT PLUS software (Mitre Software Corporation) was used to reduce the data, calculate and plot the SI casing movement.

The SI monitoring schedule including a casing spiral survey is shown in Table 2 below:

Table 2: SI Monitoring Schedule

	SI-1	SI-2	SI-3	SI-4	SI-5	SI-6	SI-7
Location	Avery Lane	Avery Lane	Abbott Drive	Voyager School	Abbott Drive	Dixon Street	Pierce Crescent
Depth SI Read to (m)	43 m	74 m	102 m	154 m	144 m	153 m	125 m
Spiral Survey	11 Jun. 2001	11 Jun. 2001	12 Jun. 2001	11 Jun. 2001	12 Jun. 2001	11 Jun. 2001	12 Jun. 2001
Initial Reading Date	21 Nov. 2000	21 Nov. 2000	21 Nov. 2000	03 Nov. 2000	28 Oct. 2000	22 Nov. 2000	22 Nov. 2000
Reading 1	07 Dec. 2000	07 Dec. 2000	06 Dec. 2000	24 Nov. 2000	24 Nov. 2000	06 Dec. 2000	07 Dec. 2000
Reading 2	12 Jan. 2001	11 Jan. 2001	12 Jan. 2001	06 Dec. 2000	06 Dec. 2000	11 Jan. 2001	12 Jan. 2001
Reading 3	05 Mar. 2001	05 Mar. 2001	06 Mar. 2001	11 Jan. 2001	12 Jan. 2001	05 Mar. 2001	06 Mar. 2001
Reading 4	02 Apr. 2001	02 Apr. 2001	03 Apr. 2001	05 Mar. 2001	06 Mar. 2001	02 Apr. 2001	03 Apr. 2001
Reading 5	28 Apr. 2001	28 Apr. 2001	28 Apr. 2001	02 Apr. 2001	03 Apr. 2001	28 Apr. 2001	29 Apr. 2001
Reading 6	18 Jun. 2001	13 Jun. 2001	14 Jun. 2001	28 Apr. 2001	29 Apr. 2001	13 Jun. 2001	14 Jun. 2001
Reading 7	04 Oct. 2001	04 Oct. 2001	04 Oct. 2001	14 Jun. 2001	14 Jun. 2001	26 Sep. 2001	04 Oct. 2001
Reading 8	20 Nov. 2001	19 Nov. 2001	19 Nov. 2001	04 Oct. 2001	04 Oct. 2001	19 Nov. 2001	20 Nov. 2001
Reading 9	06 May 2002	06 May 2002	06 May 2002	19 Nov. 2001	19 Nov. 2001	NA*	06 May 2002
Reading 10				06 May 2002	06 May 2002		

* The SI casing was blocked

Over large depths, the true orientation of the A and B SI casing directions may rotate due to the accumulation of small amounts spiral in the grooves originally machined in each SI casing section. In a deep casing, such as those installed on this project, the amount of accumulated spiral may be sufficient to appreciably rotate the apparent movement directions. The spiral rotation of the grooves was measured using a special spiral survey tool supplied by Slope Indicator Canada Limited. Photos 7 and 8 show the spiral probe being prepared to be lowered down SI-3. The calculated slope inclinometer results provided in this report were corrected for the measured spiral error.

The SI-6 installation was damaged in September, 2001. A private vehicle crashed into the SI installation destroying the casing protector and shearing the SI pipe at approximately 0.5 m below the ground surface. Mr. Doug Dewar, of AMEC, supervised the repair of the SI installation on 21 September 2001. The damaged section of SI casing was replaced as close to original condition and depth as possible.

SI Monitoring results are presented and discussed in Section 6.0 this report.

3.5 BOREHOLE DRILLING AND SAMPLING

From 2 October to 18 December, 2001, 6 geotechnical boreholes were drilled using a track mounted coring drill rig supplied by Geotech Drilling of Prince George, BC. Mr. Doug Dewar, P.Eng. and Mr. Homan Arabshahi, P.Eng. of AMEC were present during the drilling of the boreholes to conduct a preliminary log of soil conditions and to supervise the installation of the 2 vibrating wire piezometers and 4 standpipe piezometers in the boreholes (detailed in Section 3.6 below). Photos 9, 10 and 11 show the drill rig set up at the BH-2A, BH-3A, BH-4A locations respectively. Figure 2 shows the borehole locations. The borehole details are summarized in Table 3.

Table 3: Borehole Details

	BH-2A	BH-3A	BH-4A	BH-6A
Street	Avery Lane	Abbott Drive	Voyager School	Dixon Street
Date of Completion	02 November, 2001	15 November, 2001	06 December, 2001	18 December, 2001
Depth of Borehole (m)	80.1	95.1	153.3	45.9
Piezometer installations	1 Standpipe	1 Vibrating Wire Piezometer	1 Standpipe 1 Vibrating Wire Piezometer	2 Standpipes
Elevation of ground surface (m A.S.L.)*	484.638	503.188	536.418	556.888
Northing*	5869340.1	5869597.3	5869868.1	5869940.6
Easting*	532480.4	532264.2	531829.5	531514.1

* Estimated using a hand tape/compass or level and the adjacent SI as a reference point. World Geodetic System 94 datum

The boreholes were drilled using the following methods:

- Air Rotary Drilling: Air rotary drilling was typically used in the upper 25 m of the boreholes. Standard Penetration Tests (SPT) and disturbed soils samples were taken at selected depths during air rotary drilling.

- Coring: Typically below approximately 25 m, continuous core sampling was undertaken using a triple barrel HQ (89 mm outside diameter, 62 mm inside diameter) core barrel. Samples were extracted from the core barrel and stored in 10 foot run lengths in core boxes. At selected depths, a plastic core liner was used in order to improve sample integrity and to reduce moisture content changes. Additionally, at selected depths a macro punch (4 foot long, 2 inch diameter sampler) was advanced in front of the core bit to attempt to collect disturbed samples in zones of poor recovery.

Preliminary borehole logs were prepared during the drilling by AMEC's field representatives. Samples were separated and preserved in the field for in-situ moisture content determination. Photo 12 shows a core box with soil samples collected from BH-3A. Following the drilling, the core was logged in more detail by Mr. Doug Dewar and Homan Arabshahi at AMEC's soil laboratory in Prince George, BC. Samples were selected for Atterberg Limit index tests, wet sieve (grain size) analysis, hydrometer (grain size) analysis, x-ray diffraction testing and direct shear testing.

3.6 PIEZOMETER INSTALLATION AND MONITORING

Following completion of the boreholes described in Section 3.5 above, piezometers (standpipe or electronic vibrating wire devices for measuring groundwater levels or pressures) were installed at depths corresponding to the slip surfaces detected in the adjacent SI installations. Standpipe piezometers were also installed in a gravel layer in BH-4A and in bedrock encountered in BH-6A. Table 5 summarizes the piezometer installations.

Table 4: Standpipe and Vibrating Wire Piezometer Installation						
BH	**BH-2A**	**BH-3A**	**BH-4A**	**BH-4A**	**BH-6A1**	**BH-6A2**
Location	Avery Lane	Abbott Drive	Voyager School		Dixon Street	
Installation Type	Standpipe	Vibrating Wire Piezometer	Standpipe	Vibrating Wire Piezometer	Standpipe	
Date of Completion	2 November, 2001	15 November, 2001	6 December, 2001		18 December, 2001	
Bentonite Seal intervals* (m)	41.9 to 53.0 60.2 to 62.4	33.8 to 36.8 40.1 to 43.2	35.6 to 36.6 39.9 to 40.6	47.3 to 47.8 50.8 to 51.8	42.9 to 44.0 45.2 to 45.8	25.9 to 26.4 27.8 to 28.2
Pea Gravel Collection Zone Interval* (m)	53.0 to 60.2	36.8 to 40.1	36.6 to 39.9	47.8 to 50.8	44.0 to 45.2	26.4 to 27.8
Screen /Tip Depth*(m)	54.2 to 60.2	38.0	39.6 to 39.9	49.0	44.0 to 44.3	26.7 to 27.1
Type of Casing Protector	Stick-up	Stick-up	Stick-up	Stick-up	Flush-Mount	
Casing Stick-up (mm)	800	N/A	860	N/A	N/A	

* Depths are measured from ground surface. Screen depth is for standpipes, depth to tip is for vibrating wire piezometers.

Vibrating wire (VW) piezometers were sealed in the boreholes and monitored continuously with an automated data logger to determine groundwater pressures over time. The VW piezometer consists of a diaphragm connected to a tensioned steel wire. The change in water pressure on the diaphragm changes the tension of the wire. When the wire is electrically "plucked", the vibration frequency of the wire varies according to the tension of the wire, which in turn depends on the pore water pressure acting on the diaphragm. During reading a frequency signal is transmitted to a VW minilogger at the ground surface. Electronic VW miniloggers at the surface were programmed to take water pressure readings commencing from the date of installation. Photo 13 shows the data being downloaded from a VM minilogger at BH-3A.

Standpipe piezometers consisted of either 19, 25 or 31 mm diameter solid PVC pipe which was screened/slotted in the collection zone (refer to Table 4). Water levels were read manually using a water well level tape lowered into the standpipe piezometer from the ground surface. The standpipe piezometers were monitored during routine visits to Quesnel by AMEC staff. The standpipe monitoring schedule is shown in Table 5.

Table 5: Standpipe Monitoring Dates

BH-2A	BH-4A	BH-6A1	BH-6A2
Avery Lane	Voyager School	Dixon Street	Dixon Street
9 Nov. 2001	12 Dec. 2001	19 Dec. 2002	19 Dec. 2002
15 Nov. 2001	13 Dec. 2001	28 Jan. 2002	28 Jan. 2002
20 Nov. 2001	14 Dec. 2001	20 Mar. 2002	20 Mar. 2002
23 Nov. 2001	18 Dec. 2001	11 Apr. 2002	11 Apr. 2002
29 Nov. 2001	28 Jan. 2002	25 Jun. 2002	25 Jun. 2002
04 Dec. 2001	28 Mar. 2002	17 Aug. 2002	17 Aug. 2002
12 Dec. 2001	11 Apr. 2002		
18 Dec. 2001	06 May. 2002		
28 Jan. 2002	08 Jun. 2002		
28 Mar, 2002	25 Jun. 2002		
11 Apr. 2002	17 Aug. 2002		
08 Jun. 2002			
25 Jun. 2002			
17 Aug. 2002			

During April of 2002, AMEC repaired the standpipe piezometer in BH-4A. The surface casing protector had been run over by a snowplow, shearing the standpipe at the ground surface. Surface drainage appeared to have flowed into the sheared standpipe.

During May of 2002, AMEC repaired the standpipe piezometer installation in BH-6A. It appeared that the flush mount casing protector had been removed during snow clearing operations and water had ponded in the hole and flowed into the tops of the standpipes. AMEC bailed the hole and replaced the flush mount casing protector.

The results of the piezometer monitoring are presented in Section 6.6 of this report.

3.7 DIGITAL ELEVATION SURVEY

As part of the assignment, McElhanney Consulting Services Limited (MCSL) was retained to produce a digital elevation survey to serve as a base map for the study area. The digital elevation model was prepared using existing 1997 airphotos (30BCC97136, no.142, 143 and 144) with a ground control survey utilizing the existing slope inclinometer installations as tie-in points. The accuracy of the digital elevation survey was reported to be approximately 2 m. The digital elevation survey was also "draped" over an orthophoto of the study area. The digital elevation model was used by AMEC to provide various plans and cross sections of the study area used in this report.

3.8 MOVEMENT HUBS

BC Gas provided AMEC with survey data from ongoing GPS monitoring of movement hubs which were initially installed in the study area in September of 1998. Data was also provided from additional movement hubs installed during December of 2001. Figure 2 shows the locations of the BC Gas movement hubs.

3.9 PUBLIC MEETINGS

In the course of conducting the study, AMEC made presentations at two public information sessions for residents of the West Quesnel area:

21 September, 2000: AMEC provided a presentation to the residents of West Quesnel describing the proposed scope of work, methodology and project schedule for the West Quesnel Land Stability Study.

18 May, 2001: AMEC provided a presentation to the residents of West Quesnel outlining the preliminary results of the Stage 1 assessment, primarily the results of the slope inclinometer monitoring, movement hub monitoring and an update on planned Stage 2 assessment activities.

4.0 PREVIOUS WORK

The following section contains brief summaries of known previous work conducted within and adjacent to the study area.

4.1 MINISTRY OF TRANSPORTATION LANDSLIDE STUDY

Evans and Crook (1973) provided a landslide inventory and reconnaissance level review of landslide features in and around Quesnel based on an airphoto analysis and site reconnaissance visits. The purpose of the work was to provide a landslide inventory and hazard zonation for Ministry of Transportation staff reviewing potential subdivision applications in the Quesnel area. The landslide inventory identified the following slides in or adjacent to the study area:

1) Subdivision Slides and a West Quesnel Slide (now collectively referred to as the Ruric Springs Slide in more recent work by MoT and AMEC): The slides are a deep-seated movement and extend from a terrace near the upper portions of the western Fraser River Valley to the Fraser River.
2) Baker Creek Slides: Relatively small slides on the outside bend of Baker Creek impacting two houses on Twan Avenue.
3) Plateau Slide: A large retrogressive earth flow located south of the Uplands area of West Quesnel (refer to Figures 3 and 5), which appeared to be a retrogressive earth slide/earth flow. Deposits from the Plateau Slide are located on the southeast portion of the study area.
4) Garbage Dump Slide: A deep seated landslide involving the west wall of the Fraser River Valley, in the vicinity of Wells Pit Road, several kilometers to the north of the study area (now the subject of more recent work by MoT and AMEC).

Evans and Crook did not document any slide features within the study area nor any large instabilities within the Baker Creek valley.

4.2 MINISTRY OF TRANSPORTATION DRILLING RECORDS

AMEC reviewed borehole logs available in the Ministry of Transportation (MoT) files. Boreholes were drilled east of the study area for the foundations of the Baker Creek Bridge and the west abutment of the Fraser River Bridge. Table 6 below summarizes the borehole logs, which are also included in Appendix D. The locations of the boreholes are also shown in Figure 2.

Table 6: MoT Boreholes					
Borehole Location	Number	Lithology			Water Table Depth (m)
		To (m)	From (m)	Soil Description	
West Abutment Baker Creek Bridge, Anderson Drive	80-1	0	5.2	Loose to compact gravel with some boulders (fluvial deposits)	2.4
		5.2 :	21.5	Dense silty or gravelly sand (fluvial deposits)	
East Abutment, Baker Creek Bridge, Anderson Drive	80-2	0	2.7	Poorly graded sand or gravel with some boulders (fluvial deposits)	2.1
		2.7:	9.8	Loose to compact gravel with some boulders (fluvial deposits)	
		9.8	21.6	Dense silty or gravelly sand (fluvial deposits)	
West Abutment, Fraser Bridge, Anderson Drive	TH-1 (1969)	0	6.1	Loose silty sand and compact gravel (fluvial deposits)	Not Reported
		6.1:	31.1	Very stiff, high plastic silt and/or clay (probably Australian Creek or Fraser Bend Formation Tertiary Sediments) with some low plastic clay layers below 24.4 m.	

4.3 R.E. GRAHAM ENGINEERING/THURBER ENGINEERING

An R.E. Graham Engineering (R.E. Graham) letter of August 3, 1993 advised the City of Quesnel of the potential for a flow-debris slide potentially affecting developed areas in West Quesnel. The information in the R.E. Graham Engineering letter originated from Thurber Engineering Ltd (Thurber), who provided a review and second opinion to R.E. Graham on the stability conditions for a proposed subdivision in the northern half of DL. 906 (Cariboo District). DL 906 is located in the southern portion of AMEC's current study area. Thurber concluded that there was a hazard of further slope movements which could deposit slide debris (colluvium) in the proposed subdivision property and other areas in West Quesnel.

Thurber provided a figure showing the interpreted source and deposition areas of the flow-slides, which includes the Plateau Slide. The work was based on office studies. Test pit information from R.E. Graham indicated that the original ground surface in the study area had been covered by colluvium.

4.4 AGRA EARTH & ENVIRONMENTAL LIMITED

In 1994, AGRA Earth & Environmental (AGRA) prepared a report for the City of Quesnel, titled City of Quesnel, Slope Stability Study. The purpose of this study was to provide the City of Quesnel with an indication of the potential for landslides in specified areas of West Quesnel. The conclusions of this study indicated that the majority of the currently developed areas of West Quesnel (within the study area) were judged generally suitable for development from a geotechnical point-of-view within acceptable risk levels. No large scale landslide features were identified in the currently developed areas. Relict earthflows and other instabilities were identified to the south, north and west of the developed areas at higher elevations. The areas of instability generally corresponded with the areas interpreted as unstable by Thurber. Other landslide features, such as the Plateau Slide, were identified but were not specifically discussed in the report as they were outside of the specified study area.

In 1998, AGRA was hired to review the West Quesnel area for surficial indicators of ground movement. The review was requested by the City of Quesnel, based on information provided by Golder Associates Limited (refer to Section 4.6) below. The results of AGRA's report were inconclusive with respect to the presence of a large deep seated landslide in the developed portion of West Quesnel.

AGRA (1998) noted areas of localized instabilities west and south of the upper Flamingo pond. AGRA stated:

> *"Shallow translational and rotational sides were observed on the typically 27 to 35 degree slopes north of the ponds and south of a school yard. Hand dug pits and visual observations of exposed soils indicated that the soil was a compact to dense till comprised of silty sand, trace angular gravel and rock fragments. It is believed that recent movements could have been triggered by the increased infiltration behind the crest of the slopes due to the clearing of vegetation for the school playground and sports field.*
>
> *Bedrock was observed on the northwestern tip of the feature interpreted by Golder to be the main scarp of the Baker B Slide. No structural information could be obtained as the rock on the surface was broken and disturbed. The area where bedrock was observed was clearly visible on the 1949 airphotos. There was a possible overgrown rotational slide southeast of the pond where the slope was shallower. The possible slide was estimated to be a maximum of 50 m in length and about 50 m wide."*

Note that a more detailed review conducted by AGRA during a subsequent field visit indicated that the bedrock observed may have been a glacial erratic or boulder sized piece of slide debris originating from the Plateau Basalt, which outcrops well to the west of the study area.

4.5 GEONORTH ENGINEERING

In a letter of 14 August, 1998, GeoNorth Engineering Limited (GeoNorth), detailed a geotechnical assessment conducted at the Voyageur School to determine the cause of a large crack that had occurred between the original school building and an addition. In test pits, GeoNorth encountered between 0.8 to 2.0 m of loose uncompacted silt fill adjacent to the south side addition footings and no fill adjacent to the north side addition footings. Geonorth concluded (p.2), that "it is apparent that the damage to the school addition has been caused by settlement of loose fill under the foundations".

4.6 GOLDER ASSOCIATES

The City of Quesnel provided AGRA with a Geotechnical Assessment Report prepared by Golder Associates Ltd. (Golder), dated July, 1997. The report indicated that Golder was retained by BC Gas Utility Ltd. to assess whether ground conditions in the area may have caused or been a contributing factor to a gas leak. The Golder report concluded that a large portion of West Quesnel was located on a large ancient deep-seated landslide, and that this landslide had undergone recent movements. Golder identified creep of this large ancient landslide feature as the probable cause of a ruptured gas pipeline which subsequently caused an explosion. The evidence used to reach this conclusion included the results of an aerial photograph interpretation, reported compression in gas distribution lines found at 5 locations, and visual observations of exposed soil at/or below the toe of the postulated landslide. The postulated large landslide area was called the "Baker A Slide". Within the "Baker A Slide" a smaller slide area, the Baker B Slide, was identified. The main scarp of the "Baker B Slide" was located just upslope from the Upper and Lower Flamingo Ponds.

In a letter of 8 February, 2000, Golder reported the results of a monitoring program for 16 GPS movement hubs in the West Quesnel Area. Golder noted that significant ground displacements of about 75 mm had occurred over a 15 month period in the Uplands area of West Quesnel. Golder stated (p.5) that *"it is our professional opinion that the magnitude of recorded lateral displacements is much greater than the potential uncertainty resulting from the accuracy or precision tolerances introduced by the method of survey, such that the results of the monitoring have established that significant ground movement has occurred over the 15 month monitoring period".*

4.7 MORGENSTERN AND CRUDEN

Dr. Norbert Morgenstern, P.Eng. (Alberta) and Dr. Dave Cruden, P.Geo. (Alberta) of the University of Alberta were retained by the City of Quesnel to provide a review of previous reporting, geological background and provide a brief site inspection of the study area.

Morgenstern and Cruden (1999) provided a summary that stated (p. 15 and 16):

1) "A landslide has taken place within the outlines of Golder's Baker A Slide during the post-glaciation excavation of the Fraser Valley.
2) The landslide has a relatively shallow surface of rupture that does not extend into the alluvium of the current Fraser River floodplain.
3) No significant overall movements of the Baker A landslide have taken place over the lifetime of the current vegetation. However, small local movements during transient piezometric highs cannot be discounted. Nevertheless, none are evident and this indicates generally good drainage within the slide.
4) The Tertiary sediments underlying the Baker A Slide are weak and could move if groundwater levels rise in the slope. Nevertheless, there are no indications of deep-seated movements evident within the Baker A Slide.
5) The existence of the Baker B slide is not necessary to explain the reported geomorphic features within its outlines.
6) A shallow landslide or flow feature has been identified within the slope to the west of the explosion site. It is of local extent and likely seated in the recent alluvial deposits."

In summary, Morgenstern and Cruden did not identify any evidence of deep-seated sliding in the area. Based on the review, Morgenstern and Cruden (1999) recommended that the City of Quesnel consider more detailed assessments of the shallow landslides using either test pits or boreholes. Additionally, it was recommended that the City of Quesnel should consider efforts to reduce surface infiltration and avoid the use of the Flamingo Street Ponds for storm water detention without additional study.

4.8 C. O. BRAWNER ENGINEERING

C.O. Brawner Engineering (Brawner) reviewed Golder's GPS Movement hub data (detailed in their letter of 8 February, 2000) and conducted a brief field review of the West Quesnel area. Brawner's work is detailed in a letter of 7 February, 2000. In the letter (p.3), Brawner concluded that:

> *"A large old postglacial landslide has reactivated in the area of Abbott Heights and Uplands in West Quesnel. Movement over the last 15 months, in the order of 2.5 to 4 inches (60 - 100 mm), has occurred. The movement appears to be gradually increasing. They may increase further during the spring melt. Tension and compression stresses which develop due to the movement will be greatest near the boundary of the slide or over elevation irregularities in the failure zone. At these locations structures (houses, the school, apartment buildings and all underground services) will suffer distress."*

Brawner recommended a detailed literature review and geotechnical assessment of the slide area.

4.9 AMEC GROUND GAS STUDY

AMEC was retained by the City of Quesnel to conduct an airphoto review of the general Adam Street, Avery Avenue and Avison Avenue area to determine the possible source of ground gas (mainly methane) noted by local residents. Observations made during the airphoto review indicated that areas of ponded water and marshes were covered with fill during development of the area. The ponds and marshes were located south of the current Adam Street Pond (refer to Figure 3).

AMEC conducted a detailed shallow drilling program in the Avery and Avison Avenue and Adam Street area to identify and characterize the potential ground gas source. 20 boreholes were drilled to depths of up to 7.6 m. Soils encountered in the boreholes appeared to be up to 2 m of fill covering a sequence of fluvial overbank silt and clay, organic silt, sands and peat. The fill appeared to have been placed during the development of the subdivision between 1969 and 1976. Standpipes installed in environmental monitoring wells indicated that the water table in the study area was typically 0.5 to 2.8 m below the ground surface.

5.0 GEOMORPHIC SETTING

5.1 TOPOGRAPHY

The West Quesnel Land Stability study area is located within and along the west side of the Fraser River Valley in Quesnel, BC. Figure 3 shows the study area. The study area is bounded by:

- The Fraser River and Baker Creek to the east.
- Highlands of the upper Fraser River Valley to the west, at elevations typically of 600 m.
- Baker Creek (and its valley) to the north.
- A continuation of the Fraser River Valley slope to the south (point of land at elevations of 580 m to 610 m). A landslide area referred to as the “Plateau Slide” is located south of the point of land.

Adjacent to the study area, the Fraser River channel flows from north to south and is at an elevation of approximately 475 m. Baker Creek flows generally from west to east across the study area, also at an approximate elevation of 475 m

AMEC has divided the study area into six general sub areas (Figure 3):

<u>Baker Creek Valley</u>: The Baker Creek valley is located in the northern portion of the study area. The Baker Creek Valley is approximately 120 m deep at the northwest corner of the study area and approximately 30 m deep along the northeast corner of the study area. Elevations in the valley range from 480 to 640 m. Slopes within the valley typically ranged from 15° to 35° with some locally steeper sections of slope which were near vertical. The crest of the Baker Creek Valley is characterized by many circular scarps associated with past and present landsliding activity. Figure 3 shows some of the more evident landslide features on the southern side of the Baker Creek Valley. A slide, referred to in this report as the Baker/Healy Slide, was located near the mouth of the Baker Creek Valley as shown in Figure 3.

<u>Fraser River Valley Slope</u>: The western portion of the study area typically had slopes ranging from 10° to 20° which dipped to the east or northeast, with some isolated steeper sections (up to 30°). Elevations within the Fraser River Valley Slope ranged from 500 to 700 m. There appeared to be some subtle circular scarps associated with landsliding activity within the area. Photo 14 shows the Fraser River Valley Slope in the southern portion of the study area.

<u>Uplands</u>: The Uplands area (essentially a continuation of the mid to lower portions of the adjacent Fraser River Valley Slope) typically consists of undulating terrain with overall slopes of 2° to 5°. Some steeper sections of slope (25° to 35°) occur in the area north of Lewis Drive and around the Upper and Lower Flamingo Road Ponds (see below for description). Elevations within the Uplands area ranged from 500 to 600 m. Photos 14 and 15 show the Uplands area.

<u>Erosional Slope</u>: A steeper terrace-like erosional slope feature (which may have been modified by landslide activity) separates the Uplands area from the Fraser River and Baker Creek Floodplain (detailed below). The Erosional Slope appeared to be created by either a previous Baker Creek or Fraser River channel, and ranged in height from 10 m (south end of study area)

to 30 m (north end of study area). Slopes typically ranged from 15° to 35°. Photo 16 shows the Erosional Slope. Elevations ranged from 490 to 520 m.

Floodplain (of the Fraser River and Baker Creek): The Floodplain area was a flat to undulating section of ground near or somewhat above the elevation of the Baker Creek and/or Fraser River (480 to 470 m), extending essentially to the base of the Erosional Slope. Slopes in the Floodplain area typically did not exceed 3° with the exception a local section of steeper ground west of Anderson Drive where slopes typically ranged from 3° to 10°. This local feature (possibly a remnant erosional or thrust feature) extended east across Anderson Drive in the vicinity of Baker Elementary School, and merged with the base of the Erosional Slope to the south of the study area. This local slope was approximately 2 to 6 m high. Photos 16, 17 and 18 show the Floodplain area.

Plateau Slide: A deposition zone for the Plateau Slide was located in the extreme southern portion of the study area. Slopes within the Plateau Slide area typically ranged from 5° to 10° with some flatter benched sections. Within the study area the colluvium (defined as slide debris) of the Plateau Slide typically ranged from 480 to 500 m in elevation.

There did not appear to be any permanent streams flowing directly through the study area. In the southern portion of the study area there was an ephemeral stream which ran from a logging cut block to a low wet area south of Adam Street (refer to Figure 3). There were several areas of ponded water in the study area as described below and shown on Figure 3:

- Abbott Drive Ponds: Some small ponds were located in the upper western portion of the Uplands near where Abbott Drive turns south and climbs the Fraser River Valley Slope to the south. The largest pond was approximately 25 m in diameter at an elevation of approximately 580 m.

- Flamingo Street Ponds: Two ponds were located in a low area which is crossed by Flamingo Street. The Upper Flamingo Pond was located about 120 m northwest of the intersection of Abbott Drive and Flamingo Street. The upper pond was approximately 30 m by 45 m. Lower Flamingo Pond was south of the upper pond about 40 m south of Flamingo Street. The dimensions of the lower pond were approximately 15 m by 45 m. The ponds were at an elevation of approximately 515 m. Photos 19 and 20 show the upper and lower Flamingo Street Ponds.

- Adam Street Pond: There is a low wet area approximately 100 m long and up to 40 m wide at the toe of the Erosional Slope, north of Adam Street and south of Abbott Drive on the floodplain. The pond was at approximately 470 m elevation.

5.2 SURFICIAL GEOLOGY

The surficial geology of the Quesnel area is very complex and has not been defined in detail by previous work. Tipper (1971) provides only a generalized glacial geomorphology and Pleistocene history of the area. Quaternary (post-glaciation) sediments are nearly continuous and usually conceal Tertiary (pre-glaciation) sediments (predominantly soil-like materials that are usually considered bedrock, Rouse and Mathews, 1979). They were often deposited as thick sequences of valley infill in buried Quaternary valleys of the ancestral Fraser River.

Clague (1988) has attempted to describe the Quaternary stratigraphic record in the Quesnel area. His work describes up to 13 sedimentary units consisting of various glacial tills, fluvial-deltaic sands, colluvium (thought to be the result of landsliding and subaqueous debris flow deposits into Pleistocene glacial lakes), fluvial or glaciofluvial gravels, glaciolacustrine clays, sands and silts, and more recent fluvial sands and gravels associated with the Fraser River. Colluvium associated with modern day landsliding was also described.

Within the study area, the general surficial geology appears to be as follows:

Baker Creek Valley: Soils in the Baker Creek Valley were typically colluvium derived from failing fluvial, glaciolacustrine, glaciofluvial and Tertiary sediments. Exposures of fluvial deposits (deposited by previous channels of Baker Creek following the last glaciation) were evident in the downstream portions of Baker Creek Valley.

Fraser River Valley Slope: Soils on the Fraser River Valley Slope appeared to be either till or colluvium derived from landslides within the local soils and/or bedrock. There were some areas of exposed bedrock and/or bedrock covered by a veneer of weathered bedrock around the abandoned diatomite mine.

Uplands: Soils in the Uplands area appeared to consist of highly variable till, colluvium and glaciolacustrine deposits. The soils appeared to have been disturbed by landsliding activity. Soils interpreted as till or till-like may in fact be colluvium. In the northeast corner of the Uplands area (Lewis Drive and Healy Street area) there were some fluvial sand and gravel deposits. These deposits appear to have been deposited by a previous, higher elevation channel of Baker Creek.

Erosional Slope: Soils exposed in the Erosional Slope area were typically colluvium or till-like. In the northern portion of the study area adjacent to the Baker Creek Valley (in the Healy Street area) soils appeared to be fluvial gravels and sands.

Floodplain (of the Fraser River and Baker Creek): Soils on the floodplain typically consisted of fluvial deposits of sand and/or sand and gravel interbedded with fluvial overbank/floodplain deposits of silty sand, silt and silt and clay.

Plateau Slide: Soil in the Plateau Slide area consisted of colluvium which appeared to be derived primarily from sediments higher on the Fraser River Valley Slope.

5.3 BEDROCK GEOLOGY

The bedrock geology in the Quesnel area is also highly variable, not well exposed and not well documented in any consistent manner. It is generally understood that the Quesnel area is underlain by pre-Tertiary sheared siliceous sediments, ribbon cherts, and phyllites of the late Paleozoic Cache Creek Group, Eocene volcanic deposits and Tertiary sediments from the Australian Creek Formation (Rouse and Mathews,1979). Rouse and Mathews (1979), describe a complex sequence of Tertiary sediments that was deposited in a former broad valley of the ancestral Fraser River which was cut and/or faulted into pre-Tertiary bedrock. Table 7 describes these deposits from youngest to oldest.

Table 7: Bedrock Geology		
Formation Name	**Geologic Age (Millions of Years before present)**	**Lithology**
Plateau Basalts	Tertiary, upper Miocene (7 to 10)	Lava flow found at higher elevations along the crest of the valley cut by the modern Fraser River
Crownite Formation	Tertiary, mid Miocene (11 to 13)	Diatomite deposit with ironstone and silt and clay interbeds
Fraser Bend Formation (Tertiary Sediments)	Tertiary, mid Miocene (13 to 23)	Flat-lying gravels, sands, silts, and clays with minor lignites
Australian Creek Formation (Tertiary Sediments)	Tertiary, early Oligocene (23 to 37)	Lignite, clay, silts, sand and gravels
Tertiary Kamloops Group Equivalent (Eocene Volcanics)	Tertiary, Eocene (39 to 50)	Lavas, pyroclastics, and breccias. Ash beds were reported to be rare.
Cache Creek Group	Late Paloezoic (>225)	Sheared siliceous sediments (including ribbon cherts) and phyllites with local granitic rocks (Tipper 1959)

There were no significant bedrock exposures observed directly within the study area, with the exception of Crownite Formation diatomite, clay and ironstone at the abandoned diatomite mine (on the upper Fraser River Valley Slope) and possible exposures of the Australian Creek Formation observed in the Baker Creek Valley. Rouse and Mathews (1979) reported that "outliers of the Australian Creek Formation on Baker Creek west of Quesnel rest on an irregular surface eroded into sheared and altered cherts and phyllites of the Cache Creek Group" (p.428). It is interesting to note that a typical geologic cross section contained in Rouse and Mathews (Fig. 2. p. 429, 1979) depicts a deep-seated landslide disruption (undefined age) of the geology along the western side of the Fraser River Valley in the vicinity of the study area.

Note that while the Tertiary sediments have been technically classified as bedrock (based on their geological age), the majority of Tertiary sediments would be classified as soil from a geotechnical point-of-view as they have the geotechnical properties of soil.

6.0 OBSERVATIONS AND DISCUSSION

6.1 AIRPHOTO REVIEW

6.1.1 Development in the Study Area

AMEC has reviewed numerous historic airphotos (refer to Appendix A) of the West Quesnel area dated from 1949 to 1997. During the past 53 years there has been significant development in the study area. The following items describe the observed changes from 1949 to 1997. Figure 4 provides annotated images of selected historical airphotos which should be referred to in conjunction with the following items.

1949 (Airphoto BC949 no. 93)

1. The Fraser River Valley Slope, Baker Creek Valley and Erosional Slope were generally undeveloped and treed. The exceptions were Anderson Drive and some access trails. One of the access trails appeared to be a precursor to Baker Drive. Additionally there was an east-west cleared right-of way (possibly a water line) extending from Anderson Drive to Baker Creek.

2. The Uplands area was mainly treed with some cleared areas near the Flamingo Ponds. It appeared the Uplands area was being used for limited agricultural purposes.

3. The floodplain in the study area east of Anderson Drive was cleared and appeared to be used for agricultural purposes. The floodplain west of Anderson Drive had been cleared at some locations for agricultural purposes, but appeared to be mainly treed.

1969 (Airphoto BC5328 no.226)

1. Overall there had been considerable municipal development in the lower elevation portions of the study area. Abbott Drive had been constructed through the study area. Additionally, Lewis Drive (on the floodplain), Baker Drive and other secondary roads (including the Beath Street area subdivision) had been constructed.

2. The Fraser River Valley Slope and the Baker Creek Valley were treed. Some exploration trails and land clearing appear to have been conducted in the future location of the diatomite mine.

3. In the Uplands area, a residential subdivision had been constructed in the area surrounding the Flamingo Ponds and in the Allard Street area.

4. On the floodplain, the majority of the land previously used for agricultural purposes had been converted into a residential subdivision. West of Anderson Drive, the majority of the area had also been cleared and a residential subdivision constructed.

1976: Airphoto BC 5709 no. 241

1. Municipal development had continued to expand into the Uplands area. Lewis Drive and many additional streets had been extended into the Uplands area.

2. The diatomite mine had been constructed on the Upper Fraser River Valley Slope.

3. The Baker Creek Valley remained treed. It appeared that Healy Street had been extended onto the floodplain of Baker Creek, but no houses had been built.

4. In the uplands area, a subdivision had been constructed in the Lewis Drive, Perry Street and Paley Avenue area. Road right-of-ways had been cleared in the Pierce Crescent Area. Additional houses had also been constructed along Abbott Drive. Voyageur Elementary School had been constructed.

5. Adam Street had been constructed along the southern portion of the Erosional Slope. Several apartment buildings and houses had been constructed on the Erosional Slope.

6. On the floodplain, the subdivision east of Anderson Drive had been expanded to the south. Baker Elementary and Correlieu Senior Secondary Schools had been constructed.

1985: Airphoto BC85014 No. 203

1. On the Fraser River Valley Slope, it appeared that the diatomite mine had been abandoned, although the mine buildings were still present.

2. The roads and services appear to have been constructed for a subdivision at the end of an extension to Healy Street on the floodplain of Baker Creek.

3. In Uplands, the land had been cleared, roads built and some houses constructed for the subdivision in the Dawson, Dixon and Bettcher Street areas.

4. The subdivision at the end of Adam Street and Avery/Avison Avenue had been completed (refer to discussion on West Quesnel Ground Gas Study in Section 4.9). During the development it appeared that a pond/marsh area had been filled over in the Avery and Avison Avenue, and Adam Street area. Other than a few additional houses being constructed, there appeared to be little additional development in the floodplain area.

1997: Airphoto BCC97136 No. 73 (refer to Figure 3)

1. On the Fraser River Valley Slope, it appeared that the diatomite mine buildings had been demolished or removed.

2. The roads and services appeared to have been abandoned at the Healy Street extension onto the floodplain of Baker Creek.

3. Houses had been constructed in the subdivision in the Dixon/Dawson Street area.

4. Trees on the southern portion of the Uplands area had been logged.

The review of historical airphotos indicated that there has been significant municipal development within the study area over the past 53 years. During this time:

1. There has been extensive re-contouring and fill placement in developed portions of the study area, particularly in the Uplands area.

2. Timber has been harvested from most of the Uplands and floodplain areas.

3. The entire floodplain area has been developed and approximately two thirds of the Uplands area has been developed. Development has included the installation of sanitary sewers, storm sewers and water mains. During this time, local wells and/or septic disposal fields would likely have been abandoned in favor of municipal services.

Based on the airphoto observations, the loss of evapotranspiration from trees, possible leaking services, lawn watering and the loss of any pumping wells in the study area would be expected to cause a rise in the local groundwater table. In addition, significant modifications to surface drainage patterns would have also taken place.

6.1.2 Terrain Features

During the airphoto interpretation, features indicative of landslides within the study area were identified. Figure 5 shows the slide features delimited on a 1976 airphoto of the study area (BC 5709 no. 241). The 1976 airphoto was chosen because subsequent photos depict municipal development that obscures potential slide features. The slide features delimited on the airphoto include:

1) Slides on the south side of Baker Creek.

   a) Two slides in the northwest portion of the study area appeared to have been triggered by toe erosion along outside bends of Baker Creek. The slides were approximately 400 m wide, 120 m high, 200 m long and estimated to be 5 to 10 m deep. No evidence of significant movement of these slides was identified on the airphotos taken between 1949 to 1997.

   b) A slide in the northwest portion of the study area adjacent and downstream of the slides discussed (Point a) above. The slide was approximately 450 m wide, 100 m high and 600 m long. The slide appeared to be originally caused by toe erosion when Baker Creek was at a higher level. A weak layer at approximately 20 to 30 m depth and/or excessive groundwater seepage could also be contributing factors. Baker Creek did not appear to be actively eroding the toe of the slide from 1949 to 1997. No evidence of significant movement of these slides was identified on the airphotos taken between 1949 to 1997. Note that small movements or creep would not be detectable from the airphoto interpretation.

   c) An active slide, which will be referred to as the Baker/Healy Slide was located in the northeast corner of the study area. The slide appeared to be approximately 100 m wide, 30 m high, 100 m long and approximately 20 m deep. The slide appeared to be triggered by toe erosion by Baker Creek. Historic airphotos (Airphoto BC5070 no.168) indicated the slide initiated between 1949 and 1963. Since 1963, there appears to have been significant retrogression and enlargement of the slide.

2) Features of an apparent ancient landslide moving from west to east that underlies the study area are shown in Figure 5. This slide will be referred to as the West Quesnel Slide and is the main topic of this report. The main scarp of the slide appeared to extend south from the crest of the Baker Creek Valley through the Uplands area and onto the mid to lower portions of the adjacent Fraser River Valley Slope to the south. The northern flank of the slide was generally bounded by Baker Creek, its valley and a series of smaller slides that had moved north into the Baker Creek Valley (see previous points). The southern flank of the slide was poorly defined and appeared to merge with slide terrain south of the study area.

   The toe of the slide appeared to extend along the base of the Erosional Slope from the Baker/Healy Slide to Bouchie Street. From Bouchie Street, the toe of the slide appeared to extend through the subdivision in the Beath Street area and along Anderson Drive to southeastern corner of the study area.

   There appeared to be 2 old large earthflow scarps located above and southwest of the main scarp of the West Quesnel Slide (refer to Figure 5). The earthflows appeared to have deposited debris onto the West Quesnel Slide in the southwest portion of the Uplands.

3) The Plateau Slide was located in the southeastern portion of the study area (as shown in Figure 3 and Figure 5) essentially adjacent to the inferred south flank of the West Quesnel Slide. The main scarps of the Plateau Slide were located on the Fraser River Valley Slopes to the south of the study area (as shown in Figure 5). The Plateau Slide appeared to be a series of retrogressive earth slides that have slid, broken up and flowed onto the Fraser River floodplain. Based on a review of the historic airphotos it appears that there have been some ongoing movements evident in the scarps of the slide since 1949, although there were no indications of any significant earthflows during the past 53 years.

4) During the airphoto interpretation, a possible slide scarp identified near the crest of the Fraser Valley Slope, extending south from the diatomite mine (refer to Figure 5). The scarp could not be delimited to the north due to modifications to the terrain as a result of the diatomite mine. It is possible that the slide scarp extends into or joins the main scarp of the West Quesnel Slide. Structures indicative of ground movement including a possible graben were evident below the southern portion of the scarp (refer to Figure 5).

5) Within the general Uplands and Erosional Slope area, the terrain appeared to be ridged to hummocky in many areas with areas of ponded water (including the Abbott Drive Pond, Flamingo Street Ponds, Adam Street Pond, two ponds in the Dawson Road area and a pond south of Lewis Drive on a bench in the Erosional Slope). The ridged to hummocky terrain is consistent with a ground surface that has been modified by landslide activity. The ponds noted in the study area could be “sag ponds” where water collects in low areas formed by downdropped slide blocks.

6.2 FIELD CONDITIONS

6.2.1 Geology

During the field assessment, AMEC conducted a review of soil and bedrock outcrops at selected locations within the study area. The soil/bedrock outcrops examined are summarized in Table 8 below. The outcrop locations are shown in Figure 6.

Table 8: Soil and Bedrock Outcrops Observed		
Outcrop	**Location**	**Soil/Bedrock Description***
OC1	Area northwest of Dodds Avenue	2.5 m cut into fissured silt and fine-grained sand (either glaciolacustrine or eolian in origin) overlying a till-like soil consisting of sand, some gravel, a trace of silt, a trace of cobbles and a trace of boulders. Many of the cobbles and boulders appeared to be derived from higher elevations outside the study area outcrops of plateau basalts indicating the soil was colluvium.
OC2	Area northwest of Dodds Avenue	Approximately 1 m high cut into fissured glaciolacustrine silt with a trace of clay.
OC3	Area northwest of Dodds Avenue	2.5 m cut into fissured silt and fine-grained sand (either glaciolacustrine or eolian in origin) overlying a till-like soil consisting of sand, some gravel, a trace of silt, a trace of cobbles and a trace of trace boulders. As per OC1, the coarse portion of the soil appeared to be plateau basalts indicating it was colluvium.
OC4	Downstream flank of Baker Healy/Slide	Exposure of fluvioglacial sand and gravel approximately 1 to 2 m high overlying 2 to 3 m of glaciolacustrine silt and clay sand. The glaciolacustrine deposits appeared to be locally sheared.
OC5	Baker/Healy Slide	Baker/Healy Slide which was approximately 100 m wide and 30 m high. The slide consisted of a main scarp approximately 4 to 6 m high with colluvium below sloped at approximately 26°. At stream level, near vertical erosional banks up to 4 m high were encountered. The sliding surface was estimated to be approximately 5 to 20 m below the ground surface along a possible weak layer in Tertiary Sediments. Soils exposed in the main scarp were typically fluvial sands and gravels. In the downstream south portion of the scarp, a sequence of bedded silty sands, lignite, white clays with fossils and high plastic brown clays were exposed (possible Tertiary sediments) below a thin veneer of sand and gravel. Along the toe of the slide and at stream level, Tertiary sediments were also exposed. The Tertiary sediments appeared to be sheared at some locations. The white clay encountered in the scarp had a liquid limit of 62%, a plastic limit of 40% and would be classified as a high plasticity silt (ML). Note that the white clay was not diatomite.
OC6	Gravel Pit located off of Lewis Drive east of Healy Street	The gravel pit was located east of Healy Street. Soils exposed in the gravel pit were sand and gravel with some cobbles. Significant historic gravel extraction had occurred in the area. It appeared that the pit had been abandoned. Based on AMEC's site observations at the Baker/Healy Slide (OC5), it appeared that either sand or Tertiary sediments may underlie base of the pit, although none were exposed at the time of the field assessment.
OC7	Cut into Erosional Slope west of Sikh Temple on Lewis Drive	A till-like soil consisting of silt with some sand some gravel and a trace of cobbles was observed in a cut located behind the building into the Erosional slope. It appeared that the soil was colluvium.
OC8	Cut into Erosional Slope west of Bouchie Street	The soil in the area appeared to be a clay or silt with some boulders (composed of plateau basalts). The soil appeared to be either till or colluvium. The boulders could have been placed during construction of the Renaissance Apartments above the cut.
OC9	Cut slope on floodplain south of Abbott Drive	Soil exposed in the cut slope was sand and gravel. The cut slope was approximately 3 to 4 m high and cut at 35° to 40°.
OC10	Field North of Allison Avenue	Soils exposed in the gravel pit located near the western end of Allison Avenue were primarily fluvial sands and gravels. Interbedded within the sands and gravels was a lense of structureless high plastic clay, with some silt, a trace of sand, a trace of gravel and a trace of organics. The clay appeared to be deposited directly on coarse, clean sand and gravel, and did not appear to be extensive over the area. The origin of the clay is not clear; however, it would have likely been deposited during flood events or during temporary damming of the post glacial stream channel associated with sand and gravel deposits..
OC11	West of Bettcher Street	There was a small cut slope outcrop of what appeared to be weathered Tertiary silt and clay bedding with some organic layers.
OC12	Diatomite Mine	Soils observed near the base of the abandoned pit were diatomite bedded with ironstone and high-plastic silt and clay.
OC13	Slope north of Lewis Drive between Healy Street and Pierce Crescent.	There was a steep slope running between Lewis Drive and Piece Crescent. The slope was estimated to have an overall angle of 40° with heights typically ranging from 4 to 7 m. Exposed soils were compact sands and gravels.

- Photos of each outcrop with the exception of OC13 are included in Appendix I.

6.2.2 Ground Movement and Possible Slide Features

During the field assessment, AMEC reviewed ground and building conditions in West Quesnel for features indicative of ground movement possibly associated with landsliding. These features included slide scarps, service line breaks, pavement distortions or cracking, concrete sidewalk and curb cracks and deformation to houses. Photos of features possibly indicating ground movement and other possible slide features are included in Appendix J.

AMEC reviewed selected areas for the presence of tension cracks or slide scarps. Particular attention was paid to areas in close vicinity to anticipated slide boundaries where differential movements would be expected to be greatest. Within the study area, AMEC could not locate any obvious open tension cracks or slide scarps with indications of recent displacements attributable to landsliding. Numerous slide scarps were evident along the crest of the Baker Creek Valley slope including those at the Baker/Healy Slide (described above). Two possible overgrown slide scarps were observed northwest of Dodds Avenue as shown in Figure 6. The lower elevation (eastern) scarp extended for approximately 300 m and was a maximum of 8 m high at a typical slope of 45° (refer to Appendix J, Photos J1 and J2). The higher elevation (western) scarp appeared to be up to 20 m high at slopes ranging from 15° to 30°. Other possible scarps are shown on Figure 5.

Generally, the majority of pavement cracking observed within the study could be caused by factors other than ground movement. In some areas AMEC noted cracks or distortions in the pavement which may be a result of ground movement, specifically:

- Between 776 and 785 Avery Avenue (refer to Photo J3).
- At the intersection of Adam Street and Allard Street (refer to Photo J4).
- Between 753 and 747 Allison Avenue.
- On Abbott Drive:
    - At the intersection with Beath Street.
    - Near the intersection of Boyd Street (refer to Photo J5).
    - In the areas around 1350 and 1378 Abbott Drive (refer to Photo J6).
- North of 1340 and 1350 Pentland Crescent.
- Near the intersection of Finlay Road and Dodds Avenue, and on Dodds Avenue.
- Along Lewis Drive as it crosses the Erosional slope east of Healy Street.

Major sewer, sanitary sewer of water main line breaks have been reported by the City of Quesnel in the following areas:

- At the intersection of Adam Street and Allard Street in the Erosional slope area
- Near the intersection of Wade Avenue and Anderson Drive
- In the service lanes south and north of Avison Avenue
- In the service lane north of Avery Avenue
- On Abbott Drive:
    - At the intersection with Allison Avenue
    - At the intersection with Beath Street.
    - At the intersection with Boyd Street.
- On Lewis Drive near its intersection with Pierce Crescent

There have also been approximately 40 gas line service breaks reported by BC Gas. The locations of observed pavement distress, and reported water line and gas utility line breaks are shown on Figure 6.

Where possible, AMEC attempted to identify and review building and house locations where structural deformation due to ground movement was suspected or reported. The locations reviewed are described in Table 9, and identified on Figure 6.

Table 9: Building Deformation*			
Address	**Resident/ Owner**	**Date Reviewed**	**Description**
Floodplain Area			
Baker Elementary School	School District 57	July 11, 2002	Failures and line breaks in sprinkler system at Baker Elementary School field have been reported. No deformations were reported at the adjacent Correllieu Secondary School Field. No deformations were evident in the Baker School Building.
Building at the Corner of Wade Street and Anderson Drive		July 11, 2002	Indications of deformation (repaired foundations cracks, deformation of foundations and deck support columns) were observed along the south, west and north sides of the building. It appeared that the southern foundation wall had been repaired recently. The east foundation wall of the building appeared to be bulging inwards.
Building at the Corner of Avison Avenue and Anderson Drive	Mr. Dhillon	July 11, 2002	Deformations were evident in the pavement and walls on the north and south sides of the building. A review of the interior indicated that the basement floor was deformed. Bulges and cracks were evident in many locations in the floor. Doorways and windows appeared to be deformed. The owner reported numerous breaks in the sewer line into the building (north side of building). A water line break had recently occurred in the service lane north of the building.
775, 781 and 793 Avison Avenue		July 11, 2002	There were minor indications of recent deformations on these properties including tilted retaining walls, distortion of the concrete driveways and foundation cracks.
767 Avison Avenue		July 11, 2002	Deformed roof line. A review of the interior of the house indicated that the house foundation had numerous cracks.
Avery Manor, Avery Avenue		July 11, 2002	There was a distorted retaining wall (approximately 1 m high) on the east side of the building.
776 Avery Avenue		July 11, 2002	There was a displaced retaining wall located on the west side of the property. The pavement at the toe of the retaining wall was bulging. The property owner reported that portions of the retaining wall failed in 1995 and had to be rebuilt. Additionally, the south eastern corner of a garage (located in the rear of the property) was experiencing settlements and had to be re-leveled. Cracks were evident in the foundation of the garage.
770 Avery Avenue		July 11, 2002	Deformed roof line evident, particularly at back of house (south side).
785 Avery Avenue		July 11, 2002	The house was severely deformed. It appeared that the eastern portion of the house was settling. The slope on the western side of the house appeared to be pushing on the house. A retaining wall in the driveway (west side of property) was deformed.
772 Allison Avenue		July 11, 2002	Reported settlement of an addition on the east side of the house.
752 Abbott Drive		July 11, 2002	Distortion, heaving and cracks were evident in foundation walls of the building.
House at the Corner of Boyd and Abbot			The eastern portion of the house appeared to be distorted. Foundation cracks were evident.
835 Abbott Drive		July 11, 2002	Sheared windows and evident settlement on right side of the house.
212 Beath Street	Casey, Ann	July 11, 2002	Addition on back side of the house appeared to have settled. The exterior north wall of house appeared to be buckled.
213 Beath Street	Closson, Rose	July 11, 2002	One large crack was evident on outside foundation of the house.
Erosional slope			
206/210 Bouchie Street		July 11, 2002	Deformation evident at and around wall separating two apartment units. Windows and doors were deformed in both units. Resident of complex reported that one unit had been abandoned due to structural damage.
166 Bouchie Street		July 11, 2002	Possible deformation evident in large retaining wall constructed into service lane behind (east of) residence.
Renaissance Apartments Abbott Drive		July 14, 2002	There appeared to be some deformations evident in siding on east wall of building.
Uplands			
580 Dawson Street	Bernier, John	July 11, 2002	Two cracks on outside back foundation wall of the house. Damage in the house due to settlement of south side of house, included: upstairs bathroom floor was sloped, walls were cracked, severe cracking in the basement walls and floor.
1340 Pentland Street		July 11, 2002	Two cracks in front foundation of the house.
1350 Pentland Street		July 11, 2002	Two cracks in front foundation of the house.
510 Perry Street	Iwancinski, Wally	July 11, 2002	Two cracks evident in north and south foundation walls of the house.
534 Perry Street	Atkins, Bud	November, 2000	Cracks evident in house foundation and garage foundation
621 Pierce Crescent	Mueller, Roy	July 11, 2002	Bulged brick foundation of the garage wall.
581 Pierce Crescent	Twan, Joanne	July 11, 2002	Up to a 2 cm crack in basement floor. Four cracks evident on the outside foundation walls, one on each side of the house.
1331 Paley Avenue	Twan, Joanne	Not reviewed	Owner reported a failing retaining wall and distortions in the house. It is understood that the house and retaining wall (in the backyard, to the north) has been repaired two times in the past 5 to 10 years.
Voyageur Elementary School	School District 57	July 11, 2002	There was a large crack between an extension and the original building at Voyageur School. The crack appears to have been progressively widening over the years. At the time of the field visit the roof was being repaired. It appeared that the crack was being caused by differential horizontal and vertical movement. Up to 150 mm of separation was evident between the walls on the south side of the building during the field visit.

* Representative Photos J7 through J33 are included in Appendix J.

In addition to the pond areas, AMEC reviewed two wet areas reported by the City of Quesnel and local residents:

- In the cut slope in the eastern portion of the parking lot south of Flamingo Place (451 Flamingo Street), there was a wet area (Photo J34). Local residents had reported that there was a spring at the base of the cut slope.

- The City of Quesnel reported that a manhole and drainage system had to be installed at the corner of Pierce Crescent and Patchett Street to reduce or prevent infiltration of water into the basement of the residence at 421 Patchett Street. The City of Quesnel reported that the water table was near the ground surface prior to installing the drainage system.

6.3 SUBSURFACE CONDITIONS

6.3.1 Slope Inclinometer Drill Holes (Water Well Rig)

During the drilling of the boreholes for the SI installations, AMEC compiled borehole logs based on the drill cuttings returned to the surface during the air rotary drilling. Appendix D contains the preliminary logs for the boreholes. Note that these borehole logs should be considered approximate due to the following limitations:

1. Samples collected from the drill cuttings were disturbed and may have segregated during travel to the surface.
2. Bedding and other features would be destroyed or disturbed during drilling.
3. Water with a detergent additive was injected into the boreholes to assist with drilling in most of the boreholes, particularly at depth. The water many have further disturbed the drill cuttings.
4. The depth from where the cuttings originated was estimated based on the time for the cuttings to travel to the surface. Depths where there was a significant change in soil or rock type could be more accurately estimated due to changes in the drill behavior observed at surface.
5. Thin beds may be missed due to the cuttings being mixed with adjacent beds during travel to the surface.

In spite of the limitations of logging the disturbed cuttings on air rotary rigs, experience with the use of this method as shown that reasonable logs may be obtained. Table 10 shows the assumed general soil and bedrock conditions based on observations made during the drilling for the SI installations:

Table 10: Soil and Bedrock Units Encountered in Slope Inclinometer Drill Holes							
Soil/Bedrock Unit	**Depth Below Ground Surface (m)**						
	BH-1	**BH-2**	**BH-3**	**BH-4**	**BH-5**	**BH-6**	**BH-7**
Possible Fill: Silt with some sand or sand with some gravel				0.0 to 7.5	0.0 to 1.5	0.0 to 19.7	
Fluvial deposits: Sand or sand and gravel	0.0 to 5.2	0.0 to 8.0	0.0 to 12.3				
Fluvial deposits(overbank): Silt and clay, typically medium to high plastic	5.2 to 7.5						
Glaciolacustrine deposits :Silt and clay							0.0 to 29.4
Till-like deposits: (referred to as till or colluvium on drill logs) Soil consisting of silt with variable amounts of sand, gravel, clay and cobbles/boulders		8.0 to 16.0		7.5 to 33.5	3.0 to 35.0		
Tertiary sediments (Possible Australian Creek Formation): typically consisting of silt with variable amounts of clay, gravel, sand and trace lignite. The deposit appeared to have some gravelly seams or zones (up to 10 m thick in some locations). The silt and clay was typically medium to high plastic.	7.5 to 44.2	16.0 to 74.7	12.3 to 109.8	33.5 to 126.7	35.0 to 152.4	19.7 to 49.2	29.4 to 128.0
Bedrock: Layered siltstone, sandstone and conglomerate with some non-lithified sand and gravel layers				126.7 to 158.5		49.2 to 154.5	

6.3.2 Geotechnical Drill Holes (Core Drilling)

Table 11 provides a brief summary of the drill logs provided in Appendix D. Photo 21 shows the core samples collected during the drilling program laid out on the floor of AMEC's soil laboratory.

Table 11: Soil and Bedrock Units Encountered in Geotechnical Drill Holes				
Soil/Bedrock Unit	**Depth Below Ground Surface (m)**			
	BH-2A	**BH-3A**	**BH-4A**	**BH-6A**
Possible Fill: Soft to firm sandy silt (BH-4A) or loose sand and gravel (BH-6A) .			0.0 to 3.4	0.0 to 0.9
Fluvial deposits: Silty fine-grained sand, compact to dense.	0.0 to 4.3			
Fluvial deposits: Sandy Gravel, dense.	4.3 to 8.2			
Glaciolacustrine deposits: Very stiff to hard clayey silt, or sandy silt of low plasticity and compact silty sand. Photos 22 and 23 show representative samples of varved glaciolacustrine silt and clay.		0.0 to 6.9 19.2 to 22.1 25.1 to 27.5		0.9 to 6.4 10.7 to 15.5
Colluvium (or Till): Typically silt with variable amounts of sand, gravel and clay. The colluvium was typically hard and low to medium plastic. Photo 23 shows a colluvium/till layer.	8.2 to 41.2	6.9 to 19.2 22.1 to 25.1	3.3 to 17.1	6.4 to 10.7 15.5 to 27.0
Tertiary Sediments (Possible Australian Creek Formation Bedrock): Typically consisting of hard silt with variable amounts of clay, gravel, sand and a trace of lignite in some layers (Photos 24 and 25 show the variability of the sediments). The sediments appeared to have seams or zones (up to 12 m thick in some locations) of silty sand and gravel or clean gravel (Photo 26). Some zones of sand and gravel appeared to be lithified and were classified as breccia (refer to Photo 27). The silt and clay within the Tertiary sediments was typically high plastic. Slickenslides were evident at many locations within the core samples (refer to Photo 28). The Tertiary sediments appeared to oxidize from olive green/steel grey to brown overtime (refer to Photos 29 through 31). Additionally, some core samples (particularly near slip zones) swelled up to 230% after recovery.	41.2 to 80.1	27.5 to 95.1	17.1 to 153.3	27.0 to 44.2
Bedrock*: Siliceous sands (not lithified), volcanic ash, siltstone and chert were encountered underlying the Tertiary sediments in BH-4A and 6A (refer to Photo 32). The sediments were typically white to light grey with one horizon of brown ash encountered in BH-4A (refer to borehole logs and Photos 33 and 34).			124.6 to 153.3	44.2 to 45.9

* Note that given the limited information available on the local bedrock geology, AMEC has simply referred to the unit described here as bedrock. It is probable that the unit would be referred to as the "Cache Creek Group" following Rouse and Mathews (1979). It did not appear that Eocene Volcanics were present in the geologic section within the study area.

During drilling, artesian water pressures were encountered in BH-3A and BH-4A as detailed below:

1. In BH-3A, when drilling between approximately 53.0 to 56.0 m below the ground surface, the water level in the drill rods rose to 5.5 m above the ground surface (top of available drill rods). Flow was estimated at a rate of approximately 0.5 l/minute at the ground surface.

2. In BH-4A, when drilling between the 66.4 to 71.0 m depth, the water level rose to 1.5 m above ground surface. The flow rate was not measured but was estimated to be considerably less than 0.5 l/minute at the ground surface .

Based on the information summarized in Table 11 above, AMEC has prepared a generalized geologic cross section through the study area, as shown in Figure 7. Figure 7 also includes information from the piezometer measurements and SI data.

6.4 LABORATORY TESTING

6.4.1 General

Table 12 summarizes the results of natural moisture content Atterberg Limit index testing conducted on selected samples from BH-2A, BH-3A, BH-4A and BH-6A. Atterberg Limit test results are included in plots in Appendix E.

Table 12: Atterberg Limit Test Results

Soil/Bedrock Unit	Hole	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Moisture Content (%)	Soil Class
Glaciolacustrine deposits	6A	13.7	38	24	14	27	CL
Colluvium or Till	2A	30.4	37	21	16	14	CL
	3A	23.6	26	18	8	17	CL
Tertiary Sediments	2A	48.1	65	45	20	35**	MH
	2A	55.7	108	51	57	36**	MH
	2A	58.8	98	33	65	37	CH
	2A	69.4	83	49	34	31**	MH
	2A	77.0	76	49	27	33**	MH
	3A	35.2	35	20	15	13**	CL
	3A	41.0	70	49	21	43	MH
	3A	48.0	59	43	16	31**	MH
	3A	55.7	66	48	18	35	MH
	3A	67.8	75	49	26	34	MH
	3A	81.4	85	45	40	38**	MH
	3A	89.2	93	52	41	31	MH
	4A	46.8	54	29	25	27	CH-MH
	4A	49.4	58	29	29	33	CH
	4A	66.4	105	49	56	14	MH
	4A	84.7	92	42	50	12	MH
	4A	107.6	68	41	27	16	MH
	6A	27	73	39	34	31	MH
	6A	32.8	60	39	21	27	MH
Bedrock	4A	130.5	113	47	66	14	MH
	4A	137.4	26	21	5	14	CL-ML

ML= low plasticity silt, MH= high plasticity clay
CL= low plasticity clay, CH= high plasticity silt
* In-situ moisture content
** approximated from closest moisture content value

Moisture content determinations are included in the drill logs included in Appendix D. Typically moisture contents ranged from:

- Fluvial deposits: 7 to 22%
- Colluvium, Till-Like or Glaciolacustrine deposits: 7 to 49%, typically around 15 to 27%
- Tertiary sediments: 7 to 43%, typically 30 to 43% in silt and clay and 7% to 20% in sand and gravel
- Bedrock: 8 to 15% in siltstone/ash units

Hydrometer grain size analysis was performed on several samples, as detailed in Table 13 below:

Table 13: Hydrometer Grain Size Analysis

Soil/Bedrock Unit	Hole	Depth (m)	Sand (>75μm)	Silt (75μm to 2μm)	Clay (<2μm)
Tertiary sediments	2A	58. 2	8%	62%	30%
	3A	42.2	3%	79%	18%
	4A	47.5	0%	64%	36%
		49.7	17%	48%	35%

Washed sieve testing was performed on selected samples within the Tertiary sediments to determine the relative proportions of gravel, sand and fines (silt and clay). The washed sieve tests were concentrated on sandy/gravelly seams within the Tertiary sediments. The results are summarized below:

- 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%

6.4.2 X-Ray Diffraction Testing

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 (a smectite clay mineral) in a fresh water environment (i.e. no salt in the pore water) residual angles of friction ($\phi'r$) typically ranged from 5° to 8°. Other work by AMEC and others has indicated that low (<8°) residual friciotn angles may also occur in illite-kaolinite clays.

AMEC conducted two X-ray diffraction tests on disturbed samples taken from at or near the slip surface zones in BH-4A and BH-6A. The results of the x-ray diffraction testing are summarized in Table 14 below:

Table 14: X-ray Diffraction Test Results

ID	Sample Fraction *	% of total (by weight)	Non-Clay Minerals			Clay Minerals				Total
			Quartz	Plagioclase Feldspar	Potassium Feldspar	Kaolinite	Chlorite	Ililite	Smectite	
BH-4A 49.7 m	Bulk	95	62	1	1	13	0	7	16	100
	Clay	5	5	0	0	45	0	19	31	100
	Total	100	58	1	1	15	0	8	17	100
BH-6A 27.1 m	Bulk	50	56	1	3	6	2	11	21	100
	Clay	50	3	1	2	15	5	24	50	100
	Total	100	30	1	2	11	4	16	36	100

* The bulk sample had grain sizes from 75 μm to 3 μm, the clay fraction had grain sizes below 3 μm. Note that the classification is different than that of the Unified Soil Classification System, in which clay sized particles are defined as being less than 2 μm size.

The results indicate that the sample from BH-4A contained mainly silt size particles. Approximately 39% of the sample consisted of clay minerals, of which 17% of the sample was smectite clay minerals. BH-6A appeared to have an equal distribution of silt and clay size particles (50% each). Approximately 66% of the sample consisted of clay minerals, of which 36% of the sample was smectite clay minerals. Based on the mineralogy of the samples, it appeared that φ'_r could be as low as 5°.

6.4.3 Direct Shear Testing

Due to poor recovery across the potential slip zones, only one direct shear test was attempted on a sample from BH-2A at approximately 58.8 m (at or near the lower slip surface, refer to Section 6.5 below) depth. Chart 1 (in Appendix B) provides a plot of the testing results. Table 15 summarizes the results of the direct shear testing:

Table 15: Direct Shear Testing Results

Normal Load (kPa)	Peak Shear Stress (kPa)	Residual Shear Stress (kPa)
122	285	50
400	620	200
795	680	260
1199	705	270

Chart 1 (Appendix B) shows the results of the direct shear test with the shear stress plotted versus the normal stress. Typically, direct shear tests yield a linear series of points (either for peak strength φ'_p, or residual strength, φ'_r). The results from the shear test appear to be anomalous as the points (particularly for φ'_p) are not linear. Based on an approximation that the sample had no cohesion during the direct shear test, φ'_r was estimated to be approximately 15° using linear regression. The observed peak strength (φ'_p) appeared to decrease as the normal stress on the sample was increased from 65° at 122 kPa to 30° at 1199 kPa (assuming no cohesion). Given the non-linear shape of the curve for $\varphi'_{p,}$ linear regression was not attempted.

Based on the results of the direct shear test, it appears that the silt and clay sample from BH-2A was either heavily overconsolidated or may have been cemented. Another explanation is that the weakest plane of material (at the slip surface) may not have been acquired for the test. The results of the direct shear test appear to be too high for a high plastic smetitic silt and clay sample, particularly given the low angle (less than 4°) of the West Quesnel Slide.

6.5 SLOPE INCLINOMETERS

6.5.1 Spiral Survey

Table 16 summarizes the calculated spiral at the bottom of the SI casings. The complete data set includes the variation of spiral with depth down each casing. The spiral survey results in Table 16 show the clockwise rotation between the top and bottom of the SI casing.

Table 16: Results of Spiral Survey of SI Installations

SI	SI-1	SI-2	SI-3	SI-4	SI-5	SI-6	SI-7
Clockwise spiral at bottom of casing	9.9°	16.7°	24.1°	30.0°	29.6°	39.7°	17.8°

6.5.2 Calculated SI Data

The results of the SI readings to date, corrected for measured casing spiral are attached in Appendix F. These plots include absolute profiles, cumulative displacement plots, incremental displacement plots and velocity plots. These terms are defined below.

Absolute profile plots show the actual profile of the casing after installation. The scale of the absolute profile plot varies with SI installation depending on the orientation of the hole. The absolute profile of the casing is relatively unimportant for most applications. In particular, whether or not the casing is initially perfectly vertical and straight is generally not a major factor in obtaining movement data since the relative change from the initial orientation is measured. However, if the initial casing profile is sloped at over 3% from vertical, minor systematic errors may be introduced into the calculated displacements. Additionally, the absolute orientation of the casing can be useful in particular circumstances. For example, if the casing is undergoing vertical compression, then the bends in the absolute casing profile will tend to be increased over time.

Incremental plots show the individual changes in casing inclination relative to the initial inclination of the casing at each measurement depth. The incremental plots have a 50 mm horizontal scale. These plots are useful for examining the depth and characteristics of movements.

Cumulative plots show the overall lateral displacement of the SI casing profile since installation. Downslope movements are shown as positive displacements in the A direction. Cross slope movements are shown in the B direction with positive movements towards the right when looking downslope. For the inclinometers installed in the study area, downslope movements in the A direction are to the east. Cross slope movements to the right (positive movements) are towards the south. Cumulative plots have a 100 mm horizontal scale.

Velocity plots show the displacement versus time at one given depth relative to another depth. Velocity plots are presented for the A and B directions at selected intervals.

6.5.3 Discussion

The discussion below describes observations and conclusions regarding individual SI installations and should be read in conjunction with the SI plots in Appendix F.

6.5.3.1 SI-1, Avery Lane

Absolute position: SI-1 was relatively straight and vertical with relative deviations between the bottom and top of the hole of less than 50 mm in the A direction (downslope) and 400 mm in the B direction (across slope to the north).

Incremental deflection: The incremental deflections for SI-1 are characteristic of settlement above the 28 m depth. The deflections at 28 and 41 m depth indicated movements along thin slip surfaces less than the length of the SI probe (0.6 m).

Cumulative deflection: Two 5 mm lateral shifts were evident at 28 m and 41 m depth. The deflections at 28 and 41 m depth were first evident April 2001. There appeared to be surface deflections of approximately 10 mm at the top of the casing. There was some bulging of the SI casing towards the east (downslope) and north above 28 m depth. This may be due to vertical settlement of the casing, possibly combined with grout loss in a gravel layer between 16 and 21.3 m depth (grout loss is not known for certain to have occurred in this gravel layer, but could explain the observed deformations).

Velocity Plots: The velocity plots in the A direction in SI-1 indicated that there were accelerations in the rate of movement between March 2001 to June 2001 and again during October 2001 to November 2001. Otherwise the rate of movement appeared small.

6.5.3.2 SI-2, Avery Lane

Absolute position: At the bottom of SI-2 there was 4800 mm of downslope deflection (A direction) and 2400 mm of across slope (B direction, north) relative to top of the hole.

Incremental deflection: Two potential slip surfaces were indicated as spikes in the data at approximately 42 m and 60 m depth. The deflections at 42 and 60 m depth indicated movements along thin slip surfaces less than the length of the SI probe (0.6 m).

Cumulative deflection: Clear translational movement surfaces are evident at 42 m and 60 m depths (each with approximately 20 mm of translational movement). The total deformation over the period of monitoring was approximately 58 mm to the east (downslope) in the A direction and 8 mm to the south in the B direction at the ground surface. Very slight backward rotation was evident above 42 m depth in the A direction.

Velocity Plots: The velocity plots in the A direction of SI-2 indicated that there was an acceleration in the rate of movement between March 2001 to June 2001 and again during October 2001 to May 2002. From June 2001 to October 2001, the rate of movement appeared to be much slower.

Note that only one reading was taken November 2001 to May 2002; therefore, the rate of movement on the plot has been averaged over the winter and early spring of 2002. The actual rates of movement during the winter may have been relatively slow with an acceleration in March to May of 2002.

6.5.3.3 SI-3, Abbott Drive and Bettcher Street

Absolute position: Relative to the top of the casing, the bottom of the casing in SI-3 was approximately 3000 mm downslope (A direction) and 3000 mm across slope to the north (B direction).

Incremental deflection: The incremental deflection plot showed consistent movement along a slip surface at approximately 39 m depth. The thickness of the slip surface appeared to be less than the length of the slope inclinometer probe (0.6 m). The was also a small incremental deflection evident on the 12 January, 2001 set of readings at 53 to 54 m which was not evident on later readings. The small deflection could be due to dirt in the casing and is not regarded as a true slope movement since the deflection was not evident in later data sets.

Cumulative deflection: There was a translational slip surface evident at approximately 39 m depth. Deflections along the slip surface over the period of monitoring were approximately 62 mm to the east (downslope) in the A direction and 3 mm to the north in the B direction.

Velocity Plots: The velocity plots in the A direction in SI-3 indicated that there was a relatively constant rate of movement from November 2000 to March 2001. There were accelerations in the rate of movement between March 2001 to June 2001. During the summer, from June 2001 to October 2001, the rate of movement appeared to be much slower. As previously discussed, the rate of movement from November 2001 to May 2002 is averaged over the winter and spring. There may have been an acceleration from March to May 2002, which would have been evident with more frequent SI measurements over that time period.

6.5.3.4 SI-4, Voyageur School

Absolute position: Relative to the top of the casing, the bottom of SI-4 was approximately 250 mm uphill (A direction) and was approximately 3700 mm south (B direction).

Incremental deflection: The incremental deflection plot showed a slip surface at 51 m depth. The thickness of the slip surface appeared to be less than the length of the slope inclinometer probe (0.6 m).

Cumulative deflection: There was clear evidence of a slip surface at 51 m with movements of 60 mm in the A direction (downslope to the east) and 8 mm in the B direction (north toward Baker Creek).

Velocity Plots: The velocity plots in the A direction in SI-4 indicated there was an acceleration in the rate of movement between March 2001 to June 2001. During the summer, from June 2001 to October 2001, the rate of movement appeared to be much slower. As previously discussed, the rate of movement from November 2001 to May 2002 was averaged over the winter and spring. There may have been an acceleration from March to May 2002, which would have been evident with additional SI readings.

6.5.3.5 SI-5, Abbott Drive, West of Flamingo Street

Absolute position: The bottom of SI-5 was approximately 1100 mm downslope (A direction) and 2400 mm north (B direction) of the top of the casing.

Incremental deflection: The casing is likely ungrouted between at least 35 m and approximately 90 m (refer to Section 3.3) below ground. The sinusoidal incremental deflections indicated that the casing was relatively free to move within the hole and may be experiencing a slight axial compression. At 37 m depth there appeared to be a slip surface evident after June 2001. Prior to June 2001, it is probable that the slip surface was masked by the sinusoidal compressive deflections in the casing. It is possible that additional lateral deflections could become evident over time within the zone from 35 to 90 m.

Cumulative deflection: The maximum deflections of the casing corresponded to the approximate dimensions of the annular space around the casing (63 mm) below 37 m. At a depth of 37 m, translational movement has occurred. Prior to April 2001, it appeared that the sinusoidal compressional movements had masked the slip surface. Translational movements at 37 m depth were 54 mm in the A direction (east) and 8 mm in the B direction (north) (not including any allowance for the annular space in the hole). It is likely that the total translational movement is greater than that shown, by up to the total annular space within the hole or approximately 63 mm.

Velocity Plots: The velocity plots in the A direction in SI-5 indicated there was an acceleration in the rate of movement between May 2001 to June 2001, although the rate of movement may not represent actual slip zone movements due to the compression in the SI casing. During the summer, from June 2001 to October 2001, the rate of movement appeared to be much slower. As previously discussed, the rate of movement from November 2001 to May 2002 was averaged over the winter and early spring. There may have been an acceleration from March to May 2002, which would have been evident with additional SI readings.

Note that 2 sets of plots are included for SI-5 in Appendix F. The first set of plots is for the entire monitoring period. The second set of the plots starts from April 2001 when the slip surface at 37 m became evident.

6.5.3.6 SI-6, Dixon Street

Absolute position: Relative to the top of the casing, the bottom of casing SI-6 was approximately 20 000 mm downhill (A direction) and 11 000 mm south (B direction). The casing had an increasing curvature with depth with an inclination of approximately 16° near the bottom of the hole (a “J” shaped casing).

Incremental deflection: A slip surface at 28 m was evident on the incremental plots.

Cumulative deflection: There was a slip surface at approximately 28 m depth in SI-6. There was approximately 43 mm of downslope (to the east) deflection in the A direction and 3 mm of deflection toward the north in the B direction from 22 November, 2000 to 19 November, 2001. There was also an apparent but gradual systematic shift evident on the plot below 28 m depth. The shift may be caused by small systematic errors in the readings due to an effect related to the almost “J” shaped profile of the original casing installation.

SI-6 appeared to have been blocked by shear deformations at 27.1 m depth between 19 November, 2001 and 06 May, 2002.

Velocity Plots: The velocity plots in the A direction in SI-6 indicated there was an acceleration in the rate of movement between January 2001 to April 2001. During the summer, from June 2001 to November 2001, the rate of movement appeared to be much slower. Data was not available for the period between November 2001 and May 2002 because the SI was blocked.

6.5.3.7 SI-7, Pierce Crescent

Absolute position: Relative to the top of the casing, the bottom of SI-7 was found to be approximately 12 500 mm upslope (A direction) and 15 000 mm north (B direction). The casing had an increasing curvature with depth with an inclination of approximately 9° near the bottom of the hole (referred to as an almost “J” shaped casing).

Incremental deflection: Slip surfaces were evident at 27 m and 68 m depth on the incremental plots.

Cumulative deflection: The total movement in the A direction (down slope) was approximately 13 mm to the east at 27 m depth and 40 mm to the east at 68 m depth. In the B direction (across slope), the movement at 27 m was 4 mm to the south and the movement at 68 m was 17 mm to the north. The total movement evident at the ground surface over the period of monitoring was approximately 77 mm down slope (to the east) in the A direction and 25 mm to the north (towards Baker Creek) in the B direction. Below 68 m depth, there appeared to be an apparent but gradual systematic shift on the plot. The shift may be caused by small systematic errors in the readings due to an effect related to the almost “J” shape profile of the original casing installation.

Velocity Plots: The velocity plots in the A direction in SI-7 indicated there was an acceleration in the rate of movement between April 2001 to June 2001. During the summer, from June 2001 to October 2001, the rate of movement appeared to be much slower. There did not appear to be any acceleration in movement during the early spring (March to May) of 2002.

6.5.3.8 Summary

Table 17 provides a summary of SI casing deformations observed in the SI's from to May 6, 2002. The cumulative profile plots are projected onto the cross section shown in Figure 7. The vectors for the observed displacements are shown in Figure 8.

Table 17: Interpretation of SI Displacements to 6 May, 2002

SI	Location	Zone/Depth of displacement	Slip Surfaces	Interpretations
SI-1	Avery Lane	41 m	41 m and 28 m	▪ There were two lateral shifts of 5 mm at 41 m depth and 5 mm at 28 m depth in the A direction. ▪ The depth of the lower movement (41 m) corresponded approximately to the elevation of the shallower movement in SI-2. Given the depth of the slip surface in SI-2, the base of SI-1 could be above a deeper slip surface. ▪ There was possible settlement in the casing between 12 and 28 m depth. ▪ The slip surface appeared to be within the Tertiary sediments (silt and clay)
SI-2	Avery Lane	Above 60 m	42 m and 60 m	▪ There were well-defined slip surfaces at 60 and 42 m depth, each with approximately 20 mm of displacement in the A direction towards the Fraser River. ▪ The slip surface appeared to be within the Tertiary sediments (silt and clay).
SI-3	Abbott and Bettcher	Above 39 m	39 m	▪ There was a well-defined slip surface at 39 m depth with up to 62 mm of displacement in the A direction (down slope) towards the Fraser River. ▪ The slip surface was located a short distance above a gravel seam within Tertiary sediments consisting of sand interbedded with clay and silt.
SI-4	Voyageur School	Above 51 m	51 m	▪ There was a well-defined slip surface at 51 m depth. ▪ Displacements above the slip surface appeared to be towards the Fraser River with up to 60 mm of movement in the A direction. ▪ The slip surface was within Tertiary sediments (silt and clay).
SI-5	Abbott Drive	Above 93 m	37 m	▪ The casing was undergoing apparent compression movements from 35 to 90 m depth. The casing movements in this area may be a result of grouting problems; in particular, there may be no grout within this part of the hole. ▪ Apparent translational movement was occurring at a depth of approximately 37 m. ▪ The movement direction was east towards the Fraser River with approximately 54 mm of movement in the A direction. ▪ The depth of movement corresponded to the top of the Tertiary sediments (silt and clay). There may be additional movements deeper in the hole between 35 and 90 m that have not been detected to date.
SI-6	Dixon Street	Above 28 m	28 m	▪ There was a well-defined slip surface at 28 depth which has resulted in the SI pipe being blocked between November 19, 2001 and May 6, 2002. Prior to the pipe being blocked there was about 43 mm of downslope displacement. ▪ Movements below 28 m appear result of the deviation of the casing from vertical and do not appear to represent movement. ▪ The slip surface was within Tertiary sediments (silt and clay).
SI-7	Lewis and Pierce Street	Above 68 m	27 m and 68 m	▪ There were well-defined slip surfaces at 68 and 27 m depth. ▪ Displacements of the lower slip surface at 68 m depth appeared to be toward the Fraser River (east) with a small component towards the north. ▪ Displacements of the upper slip surface at 27 m depth appeared to be toward the Fraser River (east) with a small component towards the south. ▪ The upper slip surface appeared to be near the top of the Tertiary sediments (silt and clay) and the lower slip surface was within the Tertiary sediments (silt and clay).

Note that the displacements shown in Figure 8 for SI-6 and SI-7 are for the displacements of the SI's judged to be not caused by systematic shifts due to the almost "J" shape in the casing installation.

Given the gradual shifts observed in the cumulative plots, there is a possibility that SI-6 and SI-7 terminated above the bottom of the deepest sliding movement. However, a review of Figure 8 indicates that the corrected movements along the slip surfaces in SI-6 and SI-7 are comparable to the movements observed in the other SI installations.

It is also possible that the base of the SI casing in SI-1 is above the deepest probable slip surface, given the deeper depth of the slip surface observed in SI-2.

A review of the velocity plots for all SI installations indicates an expected general acceleration of movements in the spring of 2001 and possibly during the spring of 2002 (although there are not enough readings during the winter or spring of 2002 to confirm this). The accelerations were most evident in S-3, SI-4, SI-5 and SI-6 velocity plots. The rate of movement observed in SI-7 appeared to increase approximately 1 month after the other SI installations in the Uplands area. The increased rates of movement observed in the SI's are probably due to increased seasonal groundwater levels.

6.6 PIEZOMETERS AND GENERAL GROUNDWATER CONDITIONS

Chart 2 through 5 (Appendix B) show the water levels (phreatic surfaces) recorded in the standpipe and vibrating wire piezometers in BH-2A, 3A, 4A and 6A.

Peak water levels originating at or near the inferred slip surface during the monitoring program were:

BH-2A: 0.7 m above ground surface on 17 August, 2002, in the standpipe screened from 54.2 to 60.2 m below ground surface. Note that the water level has been steadily rising since the standpipe was bailed following initial installation.

BH-3A: 0.3 m below ground surface on 19 December, 2001, in the vibrating wire piezometer installed at 38.0 m depth.

BH-4A: 14.0 m below ground surface on 18 December, 2001, in the vibrating wire piezometer installed at 49.0 m depth.

BH-6A: 4.9 m below ground surface on 1 April, 2002, in the standpipe screened from 26.4 to 27.8 m below ground surface.

Note that the standpipes in BH-4A and 6A were damaged during snow removal activities. AMEC has repaired both standpipes. It also appeared that surface water may have infiltrated into the standpipes from surface runoff following the damage. The suspect temporarily elevated water level readings are noted in Charts 4 and 5 (Appendix B).

In BH-4A, the water level was approximately 4 to 8 m below the ground surface in the standpipe screened at 39.6 to 39.9 m depth in a gravel layer above the Tertiary Sediments.

In BH-6A a standpipe installed in a sand seam in the bedrock screened from 44.0 to 45.2 m depth indicated that the water table was a maximum of 41 m below the ground surface. The standpipe was dry during the last reading on 17 August, 2002.

6.7 PRECIPITATION

Monthly total precipitation data was obtained from Environment Canada for the Quesnel Airport. Plots of the recorded data from 1975 to 2002 are shown in Charts 6 and 7 along with historical normal data and the cumulative difference. The historical normal data is based on a 30 year moving mean, where an average over the preceding 30 years was calculated for each month to determine the expected normal amount of precipitation for that month. The cumulative difference is the sum of the difference between the recorded monthly precipitation data and the calculated historical normal. Plotting the cumulative difference is useful in showing the trend of the recorded precipitation over time, whether it is increasing or decreasing (i.e. wetter or drier trends) compared to what would be historically expected. Sustained periods of higher than normal precipitation would be expected to have a destabilizing influence on landslides. Note that this method differs from standard climatic methods that currently use the period from 1970 to 2000 to define average values.

The precipitation data indicates that the period from 1975 through the spring of 1988 was close to or only slightly drier than average. From the spring of 1988 through the end of 1996, conditions tended to be much wetter than normal, with 1991 being close to average. From 1997 to the summer of 2000, conditions have been average or drier than normal. From the summer of 2001 to the summer of 2002, conditions appeared to close to average.

6.8 WATER WELLS

A summary of the available well data gathered from the BC. Ministry of water, Land and Air Protection (MWLAP) water well records and MWLAP water well location maps for the study area is presented in Table 18. Appendix H contains the detailed water well data information sheets. It should be noted that most of the well records are based on driller's comments and measurements and may not be entirely precise with regard to soil/bedrock descriptions and well yield measurements.

Table 18: Summary Of MWLAP Water Well Database Records						
Well Tag Number	**District Lot**	**Owner**	**Well Type**	**Well Depth (m)**	**Bedrock Depth (m)**	**Static Water Level (m) (Below ground surface)**
6116	704	Cariboo High School	Drilled	13	-	6
61064	704	Real Gamache	Drilled	77	-	6
61066	704	Gilles Menard	Drilled	14	-	8
61238	704	Peter Kaptis	Drilled	50	-	49
15269	1226	City of Quesnel	Drilled	15	-	3
47975	1227	Glen and Joyce Nordin	Drilled	108	-	NR
61070	1227	Dave Giesbrecht	Drilled	45	38*	12
6123	1229	Mobely	Hand Dug	6	-	NR
6128	1229	Art Chesley	Hand Dug	7	-	6
6132	1229	Ronald Priest	Hand Dug	9	-	8
6150	1229	Fred Skov	Hand Dug	9	-	8
6151	1229	Jonas Waldnar	Hand Dug	4	-	2
6152	1229	Robert Olson	Hand Dug	9	-	NR
6171	1229	John Retan	Hand Dug	5	-	1
6108	1229	Cormier	Hand Dug	5	-	NR
16558	1229	Paul Benoit	Drilled	22	-	NR
20949	1229	M.J. Findlay	Drilled	110	21	58
Not in Records**		1585 Abbott	Hand Dug	5	-	NR
			Hand Dug	12	-	NR

NR = Not Reported
** The abandoned wells were reported by the resident during the field work on 11 July, 2002

Note that none of these wells could be located during the AMEC's field work and it appears that they have been abandoned in favor of municipal services. While the water levels in the abandoned wells could not be verified during field work, it appears that the majority of historic wells in the West Quesnel area encountered relatively shallow water tables within 30 feet of the ground surface, as indicated by high-percentage of hand dug wells.

The well data is for open holes and may not be comparable to the standpipes and electrical piezometers that were sealed at specific locations within the boreholes. In an open well, if the water level rises to an elevation that is not saturated, flow out of the well into the unsaturated or lower pressure area occurs. Also, in low permeability formations, there may not be sufficient water available to fill the well within a reasonable time. For both these reasons, piezometric elevations derived from well data may contain significant variations from actual conditions and may tend to be low more often than high.

6.9 MOVEMENT HUB DATA

Figure 8 shows the location of the BC Gas GPS movement hubs and the associated horizontal displacement vectors over three periods:

1. From September 1998 to May 2002.
2. From December 2000 to May 2002 (this period roughly corresponds to the period AMEC has been monitoring the SI's).
3. From December 2001 to May 2002 (this period roughly corresponds to the first period of monitoring for the additional hubs installed during December of 2001 and the last interval of SI monitoring.

Appendix E contains the BC Gas Movement Hub data used to prepare the vector data shown on Figure 8. Table 19 below summarizes the movement hub data gathered to date.

Table 19: BC Gas Movement Hub Data to May 2002

Movement hub	Date Installed	Horizontal Displacement	
		Magnitude	Vector*
2	September 1998	229 mm	104°
4	September 1998	180 mm	180°
5	September 1998	179 mm	102°
6	September 1998	192 mm	52°
7	September 1998	168 mm	61°
8	September 1998	179 mm	69°
9	September 1998	229 mm	95°
14	September 1998	218 mm	97°
15	September 1998	173 mm	78°
16	September 1998	107 mm	83°
17	September 1998	247 mm	100°
18	September 1998	222 mm	105°
19	September 1998	232 mm	107°
20	September 1998	219 mm	104°
21**	September 1998	88 mm	116°
22	September 1998	68 mm	109°
30	December 2001	44 mm	108°
31	December 2001	38 mm	61°
32	December 2001	15 mm	148°
33	December 2001	47 mm	125°
34	December 2001	40 mm	138°
35	December 2001	50 mm	152°
36	December 2001	35 mm	100°
37	December 2001	35 mm	117°
39	December 2001	45 mm	119°
40	December 2001	29 mm	149°

*Taken from true north
**Destroyed after Dec. 2001

Within the Uplands area and the Erosional Slope, the movement hubs north of and along Lewis Drive have vectors to the east (towards the Fraser River) with a small northerly component (towards Baker Creek). Movement hubs in the Uplands area south of Lewis Drive show vectors to the east (towards the Fraser River) with a small southerly component. Total displacements over the 3 years and 9 months of monitoring have ranged from 168 to 247 mm, which corresponds to an average annual displacement of 45 to 66 mm/year. One exception is the 107 mm of displacement for movement hub BCG98-16 in the Healy Street area that corresponds to an annual rate of movement of 29 mm/year.

Two additional hubs (BCG01-30 and BCG01-37) were installed in the Uplands area prior to the last the monitoring period. During the last monitoring period, displacements in the Uplands area typically ranged from 19 to 56 mm. Displacements for BCG01-30 and BCG01-37 were 50 mm and 35 mm respectively, indicating that the hubs were located within the slide mass.

Typically in the Floodplain area (on or below the toe of the inferred slide area) displacements of movement hubs appeared to be less. Movement hubs that appeared to be within the toe area of the slide included BCG98-21, BCG98-22, BCG01-21, BCG01-22, BCG01-36 and BCG01-39 with 19 to 45 mm of displacement to the east or southeast during monitoring period from

December 2001 to May 2002. For BCG98-21, 88 mm of displacement was reported from September 1998 to December 2001, indicating an average rate of movement approximately 27 mm/year. BCG98-22 had reported movements of 68 mm from September 1998 to May 2002 indicating an average rate of movement of approximately 18 mm/year.

The vectors for movement hub BCG98-23 were generally within the reported +/- 10 mm accuracies of the GPS instruments. Therefore, movement hub BCG98-23 appeared to be outside of the toe area of the slide area.

Movement hubs BCG01-32, 35 and 40 have reported horizontal displacements ranging from 19 to 50 mm along a vector towards the south to southeast. The vector towards the south or southeast (refer to Figure 8) does not appear to be consistent with the direction of movement of the slide observed in the remaining movement hubs. Since the magnitude of the vectors exceeds the reported accuracies (+/- 10 mm) for the GPS instruments, further monitoring will be required to determine if there are any trends in the data or if these are anomalous readings.

The azimuths of the displacement for the vectors of the GPS Data from September 1998 to May 2001 appear to correspond to the general trend of the azimuths for AMEC's SI data over the same general monitoring period. Table 20 compares SI data to Movement Hub data.

Table 20: Comparison of SI Displacement Data to Movement Hub Data

SI	SI Displacement (Nov 00 to May 02)		GPS Data (Dec 00 to May 02)		
	Magnitude	Vector*	Hub	Magnitude	Vector*
SI-1	10 mm	103°	98-21***	47 mm	180°
SI-2	58 mm	128°	98-21***	47 mm	180°
SI-3	62 mm	117°	98-17	85 mm	101°
			98-20	85 mm	114°
SI-4	60 mm	102°	98-09	64 mm	110°
			98-14	76 mm	104°
			98-18	90 mm	116°
SI-5	55 mm	102°	98-19	107 mm	130°
SI-6	43 mm**	119°	98-02	90 mm	105°
SI-7	75 mm	106°	98-04	53 mm	68°
			98-05	48 mm	101°
			98-08	67 mm	82°
			98-09	64 mm	110°

* Vector angles are measured from true north
**Value is for displacements from Nov 00 to Nov 01
***Value is for displacements to May 01, Hub 98-21 was destroyed between May 01 and Dec 01

With the exception of SI-2, and SI-7, the nearby GPS hubs consistently read somewhat higher than the SI's. This could be attributable to a number of factors including seasonal frost movements near surface, slide movements below some of the SI's, instrument detection tolerance (25mm reported for GPS) and simply the difference slight difference in locations of the monitoring points. In order to further investigate the relationship between the observed movements of the SIs and the GPS movement hubs, several velocity plots comparing the SI movement over time to that of the nearest relevant GPS movement hubs were developed (Charts 8 through 13, Appendix B). These charts indicate some parallel trends (within the detection tolerance of the instruments) suggesting that both sets of instruments are monitoring the same general deeper-seated movements. Additional measurements over time are required before any more definitive conclusions can be drawn. However, it is probable that SI-1 does not fully penetrate the depth of ground movement.

7.0 ANALYSIS AND CONCLUSIONS

7.1 GROUND MOVEMENT

7.1.1 Summary

Based on the information and discussions presented above, it is concluded that a large, deep-seated, active ancient landslide underlies the study area. The slide appeared to be approximately 1500 m long (west to east) and up to 1400 m wide (north to south). The toe of the slide was most evident in the area of Anderson Drive (as indicated by site observations of deformed buildings in the Floodplain area), but movements may extend farther east, closer to the river. The extent of the main scarp in the western portion of the study area was not as evident, as there were no indications of recent ground disruptions observed during the field work. The location of the main scarp of the slide appeared to be at or near the height of land to the west of the developed portion of the study area based on terrain features evident during the airphoto interpretation and the field review. The northern flank of the slide appeared to be bounded by Baker Creek. and the southern flank of the slide appeared to be on the Fraser River Valley Slope south of the study area. However, it is likely that the slide continues or merges with similar ancient slide terrain to the south (e.g. Plateau Slide). Figure 9 shows the estimated extent of the slide.

The recorded rate of movement over the period of monitoring was approximately 66 mm/year (2.6 inches/year) in the Uplands and Erosional Slope areas and 27 mm/year (1.1 inches/year) in the toe area on the Fraser River floodplain. The direction of movement was generally towards the east. North of Lewis Drive, the surficial movement hub vectors had a small northern component. South of Lewis Drive, the surficial movement hub vectors had a small southern component. It should be noted that the movement rates have been monitored over a period that has been slightly drier than long term average conditions and that the length of monitoring is very short. Some large slides undergo significant variations in movement rates. Such changes cannot be completely ruled out for this slide.

A comparison of the movement hub data and SI-7 data indicated that there may be separate overlapping slip surfaces (or slide blocks) in the area north of Lewis Drive. The upper portion (above 27 m depth) was moving in towards the east with a small component towards the north. The lower portion (27 to 68 m depth) appeared to be moving towards the east with a small southern component with vectors similar to the surficial and deep movements observed in the area south of Lewis Drive.

The slide appeared to be a translational movement with a length to depth ratio of approximately 30:1. The depth of the slip surface appeared to range from 28 to 68 m below the ground surface. The average inclination of the inferred slip surface was approximately 4°, as shown in Figure 7.

The sliding appeared to be along a weak layer within pre-glacial Tertiary sediments, that underlie more recent post-glacial soil deposits, including those associated with the Fraser River and Baker Creek. These Tertiary sediments appeared to be pre-sheared and may themselves have been deposited by ancient pre-glacial landslide activity. Laboratory testing conducted on samples of Tertiary sediments taken from at or near the slip surface indicated a high plastic silt and clay with a significant (greater than 20%) smectite clay mineral content and appreciable contentes of illite and kaolinite. Based on the clay mineralogy, residual angles of friction ($\acute{\varphi}_r$) of soil along the slip surface could be at least as low as 5°.

Relatively high groundwater levels were evident in slide area. Within the Uplands area, piezometers indicated a phreatic surface (equivalent groundwater level) along the slip surface that was at or near the ground surface. The exception was BH-4A, where the observed phreatic surface along the slip surface was approximately 15 m below the ground surface. Artesian groundwater pressures were present in the BH-2A standpipe and in zones approximately 15 m below the slip surface in BH-3A and 4A. Given the pattern of artesian groundwater conditions observed, the recharge area for the groundwater within the study area likely lies in the highlands west of the study area. Consequently, activities west of the study area (i.e. logging) will affect groundwater recharge rates and subsequent groundwater levels within the study area.

It is possible that ancient sliding (pre-sheared Tertiary sediments) extends beneath the floodplain to the Fraser River. It is also possible that there have been some compression related ground deformations beyond the observed toe of the slide due to thrusting at the toe of the slide. These deformations are expected to be much smaller in magnitude than the reported deformations within the area interpreted as being within the body of the slide. Future monitoring of BC Gas movement hubs within the Floodplain area will provide additional information.

In accordance with the initial scope of work for the West Quesnel Land Stability Study, AMEC installed a line of inclinometers through the approximate center of the suspected area of greatest slide movement. Based on the information and ground movements obtained during this study, additional geotechnical investigation (drilling, coring, slope inclinometer installation and piezometer installation) between the postulated toe of the slide area and the Fraser River, and north of the line of slope inclinometers (generally north of Lark Avenue) would be useful in further confirming the characteristics of the subsurface geology and slide extent.

7.1.2 Future Slide Movements

Without any remediation, it is expected that the displacements of the West Quesnel Slide will most likely continue at rates similar to those reported during the SI and movement hub monitoring, averaging approximately 60 to 70 mm/year. More rapid movements and/or increased movement rates triggered by periods of higher precipitation or possibly major changes in water infiltration characteristics (such as undetetected or major service line breaks) could occur. The actual rates of movement experienced would depend primarily on weather conditions (particularly precipitation) and on other activities that materially affect water infiltration into the ground or slope geometry. Locally higher rates of movement could also be induced by seismic motions and accelerations.

Based on the historical behavior of the study area, it is judged unlikely that extremely rapid movements (i.e. movement rates exceeding 1.5 m/hour) will occur. The slide is expected to continue to move very slowly (i.e., typically at rates of under 100 to 200 mm/year), unless there are significant and sustained changes in precipitation patterns, or significant changes to surface water infiltration characteristics or the surface geometry of the area are introduced. Further monitoring through periods of higher precipitation will assist in refining these movement rate estimates and might result in changes to the foregoing predictions.

Future movements of the slide will further deform and stress structures, roads, municipal and utility services within and close to the slide area delimited in Figure 9. As noted above, the potential for movements beyond what is presently delineated as the toe area cannot be completely ruled out at this point.

7.2 SLOPE STABILITY ANALYSIS

AMEC conducted computer aided slope stability analyses with SLOPE/W software (GEO-SLOPE International Ltd., Version 5.0, 2001) using the Morgenstern-Price limit equilibrium method. The analyses used the inferred groundwater, slip surface, and cross section geometry shown in Figure 7. The following material properties were assumed for the soils within the slide mass and along the slip surface:

Fluvial sediments: $\varphi' = 30°$, $c'=0$, unit weight= 19 kN/m^3
Glaciolacustrine deposits, till or colluvium: $\varphi'_r = 17°$, $c'=0$, unit weight= 19 kN/m^3
Tertiary sediments: $\varphi'_r = 6°$, $c'=0$, unit weight= 19 kN/m^3

An initial slope stability analysis for the present slide condition (Run 1 – back analysis of unstable condition) yielded a factor of safety against sliding (F) approximately equal to 1.03 (effectively unity). This suggested that the assumed φ'_r (residual angle of friction) of 6° within the Tertiary sediments (where the majority of the slip surface was located) was reasonable.

Further analyses (Runs 2 through 5) were conducted to determine the impact of lowering the groundwater pressure acting on the failure surface on overall stability of the slide (i.e. increasing F). The groundwater (phreatic) level was successively lowered in 5 m increments during the analyses.

Table 21: Summary of Slope Stability Analyses

Condition	Run #	F
Current conditions as shown in Figure 7, slip surface within Tertiary sediments ($\varphi'_r = 6°$)	1	1.03
Lowering the groundwater level 5 m	2	1.17
Lowering the groundwater level 10 m	3	1.30
Lowering the groundwater level 15 m	4	1.43
Lowering the groundwater level 20 m	5	1.55
Constructing a 12.5 high, 250 m wide (west to east) granular stabilization berm on the floodplain along and against the Erosional Slope. Berm properties of $\varphi' = 35°$, $c'=0$ and unit weight= 19 kN/m3	6	1.64
The potential for a failure to daylight above the top of the stabilization berm	7	1.11

Additional analyses (Runs 6 and 7) were conducted to demonstrate the magnitude of ground geometry changes (e.g. a toe berm constructed on the floodplain) required to achieve similar affects, even though such radical ground geometry changes would not be practical for a built up area. The same would be true for analysis of options (i.e. excavation) that reduced the overall slope of the ground surface.

7.3 REMEDIAL OPTIONS

7.3.1 Earthworks

Among the options usually considered for remediating slide areas, one of the common options are changes to the ground topography. This involves either excavation of upper areas to reduce the slope height and angle (and hence reduce forces driving the slide), or building up (supporting) the lower or toe area of the slope (providing more resisting forces against slide movements). During the stability analyses, the option of constructing a stabilizing berm at the toe of the slide was modeled. A 12.5 m high by 250 m wide berm at the toe of the slope increased the overall factor of safety from 1.00 (Run 1) to 1.64 (Run 6). Given the translational nature of the movement and the ratio of length to depth (30:1) of the slide mass, there is a possibility that a new slip surface could daylight above the top of the toe berm (a thrust condition). The slope stability analysis (Run 7) for the possible thrust condition indicated that the F was 1.11.

A stabilizing berm is not considered to be a suitable or feasible remedial measure for the West Quesnel Slide since:

- There was a potential for development of a thrust surface day lighting above the berm (F=1.11). Such thrust surfaces have been observed on other slides with similar geometries. Typically, an acceptable F would be greater than 1.35. Given the translational nature of the slide, achieving a higher value of F would require a far more massive berm, extending a thick blanket of material up into the Uplands area.

- The toe berm would require significant quantities of material (up to 3 000 000 m^3, not including any blanket that would have to be extended up the slope). Considering the cost of mining, transporting and placing granular material at approximately \$20.00/m^3, the cost of earthworks could exceed \$60,000,000.00, not including site preparation and resource location costs.

- There may not be a suitable granular source with the required quantities of granular material located within a reasonable distance of the study area.

- Any buildings, municipal structures, utilities, roads and services within the footprint of the berm would have to be removed and replaced following construction of the berm. During berm construction, temporary services would have to be provided to the Uplands area.

- In the northern portion of the study area the toe berm would require the relocation of Baker Creek.

The other option of reducing the driving forces contributing to the slide mass by excavation in the upper areas of the slide was very briefly considered. However, given the geometry of the translational movement and the very low angle of the terrain, massive excavation over a very large area would be required to be effective. This excavation would destroy much of the development and infrastructure that is the subject of this study. Therefore, this and other options that change the overall ground geometry of the slide area were not considered further as they would be considered too onerous and impractical to implement.

A possible exception is the area of active instability along Baker Creek, which is referred to in this report as the Baker/Healy Slide. At this location, Baker Creek is actively eroding the toe of the slope below the old gravel extraction area, which may coincide with the toe of the larger landslide feature. While eliminating the erosion caused by Baker Creek at this location would not have a significant effect on stabilizing the overall slide mass, a properly designed and installed rip-rap berm would reduce the local destabilizing affect and would assist in reducing the tendency for future deterioration of stability in this area. Additional, stream survey, environmental and steam engineering assessment would be needed to properly design and install such a berm.

7.3.2 Dewatering

Perhaps the single most significant parameter affecting the stability of landslides of the nature of that observed in West Quesnel is the groundwater pressure acting on the failure surface. Decreasing the groundwater levels in a slide mass has three beneficial effects:

1) There would be an increase in the effective stress and hence frictional resistance forces acting along the slip surface.

2) The weight of the slide mass would be slightly reduced and hence the driving forces for the slide would be reduced if dewatering can remove groundwater held in open cracks and fissures within the slide mass.

3) Work on similar slides suggests that the movement rates are highly dependent on groundwater pressures. Relatively small reductions in water pressures and water availability may significantly reduce slide movement rates even at relatively small overall increases in factor of safety.

Runs 4 through 5 of the stability analyses indicated that the F of the slide increased from 1.03 to 1.43 if groundwater levels acting along the slip surface were reduced by 15 m, or increased to 1.55 if groundwater levels were reduced by 20 m.

There are two typical ways to reduce the groundwater levels within a slide mass:

1. Reducing recharge of the groundwater table by reducing infiltration of surface water (surface drainage and control measures).

2. Lowering the groundwater table using subsurface drainage measures such as horizontal drains, drainage adits, and dewatering wells.

These remedial measures are discussed further below.

7.3.2.1 Surface Drainage Measures

There are a variety of natural and man made sources of water infiltration in the slide area. There are also a wide variety of measures that can be considered to reduce infiltration. Such remedial surface drainage measures include:

1. Restricting or eliminating lawn watering and other forms of artificial irrigation.

2. Eliminating swimming pools or ornamental ponds.

3. Eliminating in ground septic disposal.

4. Capturing surface runoff from impermeable surfaces (building roofs, driveways, parking areas, streets etc.) and diverting it into a contained storm drainage system.

5. Eliminating and/or providing artificial drainage for low areas where surface water naturally accumulates. Natural springs and ponds could be drained, or alternatively drained and reconstructed with an impermeable synthetic liner to reduce the potential for infiltration into the ground.

6. Regular monitoring, repair and/or replacement of municipal services that leak (storm drains, water and sewer services).

7. Encouraging vegetation growth that reduces surface permeability and naturally takes up near surface water.

8. Controlling upslope activities (i.e. clearing/logging, earthworks, road construction) that could increase water infiltration?

Measures similar to those above have been implemented in other developed areas that have significant slope stability issues. Examples include the West Bench area in Penticton, and the Aberdeen Hills Subdivision in Kamloops, BC. Klohn Crippen (1997) that Aberdeen Hills' residents added an equivalent of 1000 mm in precipitation over grassed areas while watering their lawns (the annual precipitation for the Kamloops area was approximately 300 mm/year). Klohn Crippen recommended that irrigation (mainly lawn watering) controls be implemented to reduce the impact of irrigation on groundwater levels. Note that Kamloops has significantly less annual precipitation than Quesnel (270 mm of precipitation as compared to 540 mm based on the 1961 to 1990 Canadian Climatic Normals) and a warmer/dryer climate; therefore, the residents' perceived requirement for lawn watering in Quesnel may be significantly less than Kamloops.

7.3.2.2 Subsurface Drainage Measures

Horizontal Drains

The installation of horizontal drains (gently sloping drill holes lined with 50 to 150 mm diameter perforated pipe) has been a technique effectively used to stabilize many landslides. The main advantage of horizontal drains is that drainage is usually driven by gravity; therefore, pumps and other equipment requiring maintenance are not required. In reasonable ground conditions, using conventional equipment, horizontal drains have been drilled as much 100 to 200 m into a slope. However, in order to be effective, slide topography and sub-surface geometry must be such that a significant portion of the slide surface (or relatively permeable water bearing seams) can be reached by the horizontal drains.

The West Quesnel Slide has an estimated length of 1500 m, an average slip surface depth of approximately 50 m, and an overall slope of approximately 3.8°. Given the relatively flat lying geometry and translational nature of the slide there are very limited opportunities to effectively penetrate the slide with free draining horizontal drains. The only potential locations for drain installation would be along the toe of the Erosional Slope feature, which would be some distance above the slip surface area where drainage would be most effective. The horizontal drains would also have to be approximately 1200 to 1500 m long which may not be possible or would be extremely expensive with current drilling technologies (upwards of $140.00 per meter to drill). Drains of such length are also very susceptible to disruption by ground movements that occur before the full effect of drainage is established to the point which stabilization can occur.

Drainage Adits

Another method of sub-surface drainage for slide areas that can be considered is via tunneling drainage adits into or under the slide mass. Additional drainage is achieved by radial drilling from the adits into the slide mass to extend the effective zone of influence of the adits. As with horizontal drains, the main advantage with this method is that water would drain by gravity. However, construction of drainage adits within or below slide areas is highly specialized, risky and very expensive work. Given the variable ground conditions encountered during the West Quesnel drilling program (lenses of saturated sand and gravel and pre-sheared material), extensive artificial ground support (tunnel lining) would likely be required. Drainage adits would also suffer from the same geometrical limitations as horizontal drains. To further pursue the feasibility of such an option extensive additional sub-surface geotechnical investigation and analysis would be required. Even if determined to be feasible, it is judged unlikely that such an option would be cost effective.

Dewatering Wells

Given the geometry of the West Quesnel area, conventional water wells drilled vertically into the slide mass to at least the depth of the slip surface (generally greater than 50 m) may be the most suitable subsurface drainage measure. Given the high (and even artesian) groundwater pressures encountered during the geotechnical drill investigation program, there is a good possibility that an appropriate network of deep pumped wells can be employed to lower groundwater levels to a point where stability of the slide mass can be greatly improved (or at least movement rates reduced). The main advantage of water wells is that they use a relatively simple and proven technology with locally available equipment. Disadvantages of water wells include:

- The system would have ongoing operational (electricity) and maintenance costs. Pumps would require regular maintenance and may have to be occasionally replaced. Wells may also have to be periodically serviced or re-developed. Wells may need to be replaced or redrilled if they are sheared off.

- The water wells would have to discharge into a contained surface discharge (e.g. municipal storm sewer) system. Numerous service connections would be required. The environmental quality of the discharge water would also need to be considered in relation to the method chosen for disposal.

- To be effective, the well system would have to be fairly extensive, covering a wide area of the slide area. Until the entire system was in place, the first few installed wells would be particularly vulnerable to disruption from ongoing slide movements. Consequently the final system would have to be installed and brought online in as short a time frame as possible.

- One of the key aspects to the successful implementation of a well system is the lateral influence of the wells. Specifically, if the zones containing the groundwater are not laterally continuous, then wells might not be successful since the radial influence of a single well would be very limited and significant quantities of water could not be produced from the slide. A detailed assessment including a ‘pilot’ test well program would be required to determine the hydrogeological properties of the ground within and below slide mass, and to confirm the potential effectiveness of pumped wells in lowering the groundwater levels. This assessment would provide additional information necessary to determine the functionality, number, spacing, size, screen and pump details of the final dewatering well system.

- A further possibility that has been used in some instances is to put the production zone of the well under a vacuum. While this has been primarily used for horizontal drain systems, it can also be applied to wells, with some increase in the complexity of the well installation. Vacuum methods have been very successful in rapidly reducing slide movements early in groundwater control programs and in inducing drainage in otherwise marginal permeability conditions.

To further assess the suitability of water wells as a stabilization measure, AMEC has included recommendations for a pilot test well program in Section 8.2 of this report.

8.0 RECOMMENDATIONS

Given the desirability of effectively stabilizing the West Quesnel area, AMEC has developed a series of recommendations aimed at reducing (or possibly even stopping) the slope movements to manageable levels. These recommendations include a variety of surface and subsurface drainage measures, land use controls, and supporting additional investigation and monitoring programs. These measures on their own will not instantly stabilize the area, but should be considered as part of a long term program to reduce slide movements to a manageable level. In addition, other recommendations for learning more about and managing the slope stability issue in West Quesnel are provided.

8.1 SURFACE DRAINAGE CONTROL

The following recommendations are given for surface drainage control in the West Quesnel area:

1. Low lying or areas of ponded water within the study area should be drained and re-graded to reduce the potential for water ponding and infiltration. This includes draining the Adam Street Pond, Abbott Drive Ponds and the Flamingo Street Ponds. Alternatively, the ponds could be reconstructed with an impermeable geosynthetic liner and linked to a contained discharge system. It is likely that elimination of surface ponding would be carried out in conjunction with an overall review of storm water management for West Quesnel. Note that merely filling in the ponds with uncompacted fill would not be sufficient to reduce infiltration in these low areas.

2. The City of Quesnel should conduct a detailed and regular review of all utility services (sanitary sewers, storm sewers and water mains) in the study area to determine if any leaks or breaks are present. Leaks or breaks should be repaired as discovered. Where possible, flexible utility pipe and/or connections should be used.

3. The City of Quesnel should consider adopting and implementing interim controls for surface water drainage in the West Quesnel study area. Such controls should include:

    - Restricting or eliminating lawn watering and other forms of artificial irrigation.
    - Prompt reporting and repair of all utility line breaks.
    - Eliminating swimming pools or ornamental ponds.
    - Eliminating in ground septic disposal, if any remain.
    - Capturing surface runoff from impermeable surfaces (building roofs, driveways, parking areas, and diverting it into a contained storm drainage system.
    - Controlling clearing and/or vegetation removal.

Note, depending on the implementation and observed effectiveness of the active sub-surface dewatering measures recommended below, some of the above controls could be relaxed or modified over time.

4. The City of Quesnel should consider carrying out an updated and comprehensive overall review of storm water management and design for West Quesnel. Additional storm water facilities to discharge intercepted surface runoff (roof, driveways, parking, streets, drained low areas) should be identified and considered. Future modifications to the current drainage and/or services in the study area should be reviewed by a geotechnical engineer.

5. In conjunction with other agencies and jurisdictions, The City of Quesnel should consider development of and implementation of a water management plan for the lands upslope (west) of the study area. The plan should be particularly focused at those activities that have the potential to increase infiltration of water into the ground (i.e. clearing, logging, earthworks, road construction, irrigation etc.).

8.2 SUBSURFACE DRAINAGE (DEWATERING WELLS)

AMEC recommends that City of Quesnel pursue the option of subsurface drainage of the West Quesnel area via a system of pumped dewatering wells. Prior to implementation of a full dewatering well system, it is recommended that pilot test well program be carried out to confirm the effectiveness of such an option and to obtain the necessary hydrogeologic parameters for design and operation of the full system. The pilot test well program would consist of the following four stages:

Stage 1, Design and Installation of Pilot Test and Observation Wells

The locations of two pilot pumping wells and four observation wells would be determined in consultation with the City of Quesnel. Ideally, one pumping well would be located in the cleared vacant lot west of Bettcher Street, between Abbott Drive and Flamingo Street (refer to Figure 9). The other pumping well would be located near the upper Flamingo Street Pond (refer to Figure 9). The locations of the four adjacent observation wells would be determined following the final sitting of the pumping wells, but would be typically located within 25 to 75 m of the pumping wells.

The two pilot pumping wells and four observation wells would be drilled, developed and screened. The pilot pumping and observation wells are expected to be approximately 60 m deep. Groundwater level monitoring instrumentation would be installed in the observation wells. In addition, groundwater samples would be collected and analyzed for water quality determination.

Stage 2, Short Term Pump Test:

Stage 2 would consist of a short term pump test lasting approximately 6 days (3 days for monitoring pumping and 3 days for monitoring recovery). Data from the pump test would be analyzed by AMEC to determine the parameters for a long-term pumping test (detailed in Stage 3 below).

<u>Stage 3, Installation of Permanent Pumps and Long-Term Monitoring</u>:

During Stage 3, permanent pumps would be installed and operated in the pilot pumping wells. These wells would be operated and monitored over a period of approximately 4 to 6 months to determine the long-term draw down effectiveness of the pilot wells. Water levels would be monitored during Stages 2 and 3 using the existing vibrating wire piezometers and groundwater instrumentation installed in the observation wells. Local movement rates would be monitored via the slope indicators.

<u>Stage 4, Design of Dewatering System</u>:

Following the completion of Stage 3, AMEC would review the information gathered during Stages 2 and 3 and prepare a detailed design for an overall water well dewatering system for the West Quesnel Slide. Recommendations for future work would be included if required. A cost estimate would be included with the final design.

A cost estimate for the proposed pilot well production tests is included in a separate document to the City of Quesnel.

8.3 ONGOING MONITORING

As the City of Quesnel considers and/or implements the recommendations described above, it is recommended that monitoring of the slide area be continued in order to further understand the characteristics of the slide area. This data will be particularly useful as baseline information against which to judge the effectiveness of future dewatering measures. Specifically:

1. The SI installations should be monitored a minimum of 3 times a year, once in the fall and twice in the spring, with shorter intervals if sudden increases in rates of movement are detected. An allowance should be made for replacement of installations that become dysfunctional over time, or for installation of additional slope inclinometers.

2. Existing groundwater instrumentation should also be monitored. Standpipes can be monitored at the same time as the SI installations. Vibrating wire installations can be left operational with data downloading during the same or other field visits to the area.

3. GPS monitoring of movement hubs should continue, with measurements taken at least 3 times a year, once in the fall and twice in the spring, with shorter intervals if sudden increases in rates of movement are detected. Movement data should continue to be reviewed by a geotechnical engineer in conjunction with a review of the SI and groundwater instrumentation.

4. The City of Quesnel should continue to keep a record of any water line, storm sewer or sanitary sewer line breaks. Additionally, the City of Quesnel should continue to document reports of building distortions and other phenomena potentially related to slope movements within the study area.

5. Although remedial repairs have been carried out, a crack/displacement monitoring program should be implemented for the Voyageur Elementary School.

A cost estimate for Items 1 through 3 will be included in a separate document to the City of Quesnel. Item 4 would be conducted by the City of Quesnel. Item 5 would have to be performed in conjunction with School District 57.

8.4 OTHER RECOMMENDATIONS

AMEC recommends that the City of Quesnel continue to enforce appropriate development restrictions in the study area until remedial measures have been completed and monitoring indicates that slide movement has stopped or slowed to an acceptable rate of movement. At that time, development restrictions could be reviewed.

Significant changes to slope geometry, vegetation cover, and or surface runoff conditions that could affect the study area should be reviewed by a geotechnical engineer prior to implementation.

Gas service lines should be regularly inspected for leaks.

Additional survey, stream, environmental, and geotechnical assessment should be undertaken to design and construct a stabilizing rip-rap berm along the west bank of Baker Creek at the Baker/Healy Slide area.

Buildings identified or suspected as being in structural distress should be reviewed by both structural and geotechnical engineers to determine suitability or requirements for continued occupation.

In order to more fully characterize and monitor the slide movements in the northern portion of the study area (north of Lark Avenue) and in the toe area of the slide, AMEC recommends some additional subsurface investigation beyond the area investigated by the initial line of slope inclinometers. Table 22 summarizes the approximate location and depth of proposed locations for additional geotechnical drilling, sampling, and slope inclinometer installation.

Table 22: Additional Drilling / Slope Inclinometer Locations

Location	Depth
Intersection of Lark Avenue and Pinchbeck Street	100 m
Intersection of Pentland Crescent and Pinchbeck Street	100 m
Intersection of Pierce Crescent and Paley Avenue	100 m
Near Firehall, at intersection of Allison Avenue and Abbott Drive	100 m
Along Beath Street, near 213 Beath Street	100 m
Along Lewis Drive, near the Sikh Temple	100 m

A cost estimate for the proposed additional SI installations is included in a separate document to the City of Quesnel.

9.0 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 Limited **Reviewed by:**

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DD/NP/rmm

APPENDIX A

REFERENCES: **GEOTECHNICAL CONSULTANT REPORTS**
GEOLOGICAL BACKGROUND
AERIAL PHOTOGRAPHY

REFERENCES

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AERIAL PHOTOGRAPHY

Airphotos Reviewed			
Airphoto	Year	Scale	Colour
30BCC97136 no. 73 and 74, 142 to 144	1997	1:20 000	Colour
BCB91026 no.126 and 127	1991	1:15 000	Black and White
BCB85014 no. 203 to 204	1985	1:15 000	Black and White
BC5709 no. 241 and 242	1976	1:15 000	Black and White
BC5328 no. 225 and 226	1969	1:30 000	Black and White
BC5070 no. 168	1963	1:30 000	Black and White
BC949 no. 93 and 94	1949	1:50 000	Black and White

APPENDIX B

FIGURES AND CHARTS

APPENDIX C

PHOTOS

APPENDIX D

DRILL LOGS

APPENDIX E

LABORATORY TESTING DATA

APPENDIX F

SI DATA

APPENDIX G

BC GAS GPS MOVEMENT HUBS

APPENDIX H

WATER WELL DATA – GOVERNMENT RECORDS

APPENDIX I

SITE OBSERVATIONS- OUTCROPS

APPENDIX J

SITE OBSERVATIONS- GROUND MOVEMENT

APPENDIX K

SLOPE STABILITY ANALYSES

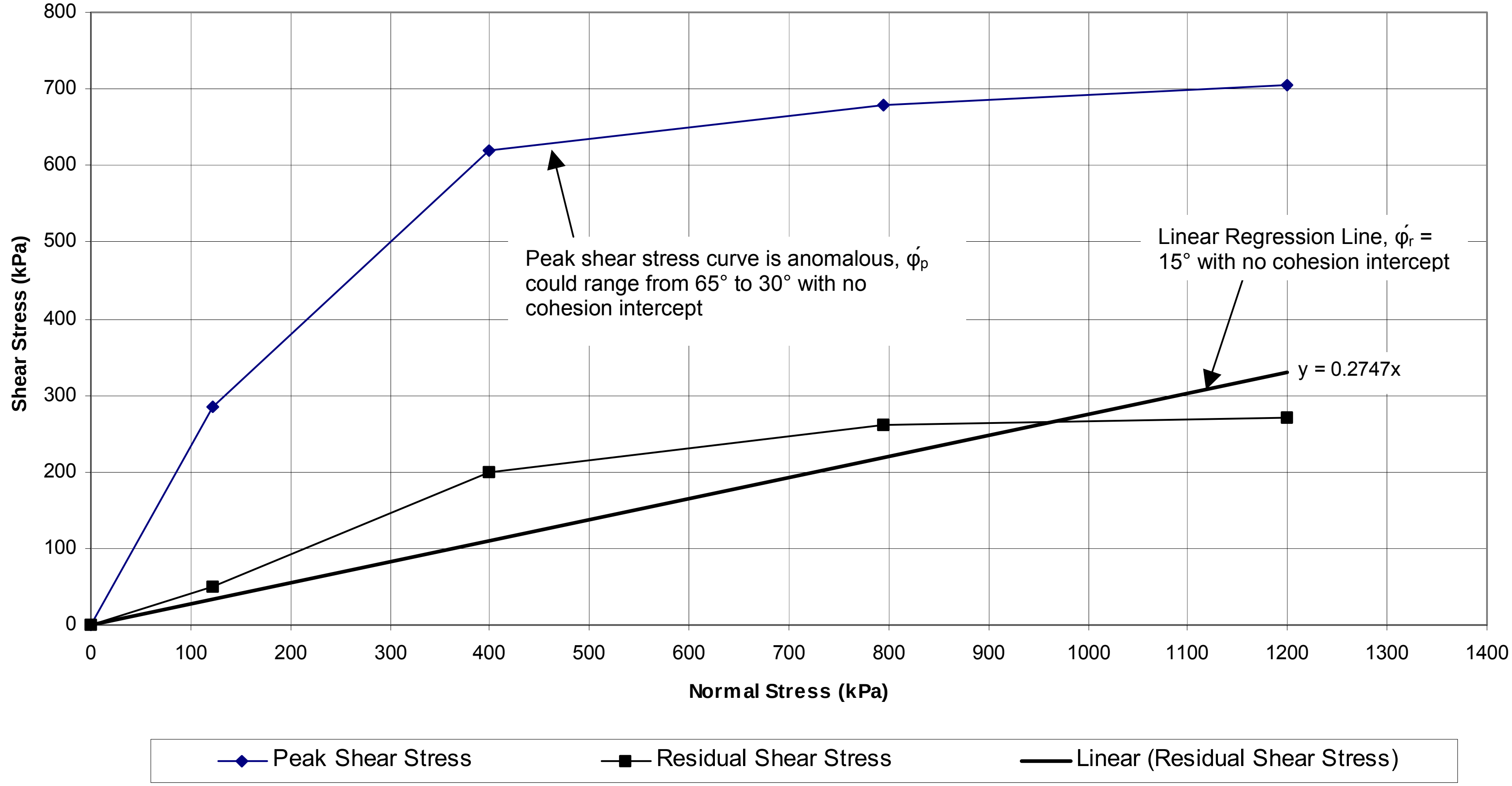

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Chart 1: Direct Shear Test Results

City of Quesnel

West Quesnel Land Stability Study
Quesnel, BC

DATE:	SCALE:	DRAWN BY:	PROJECT No: KX03903
Sept. 2002	NTS	SMJ	

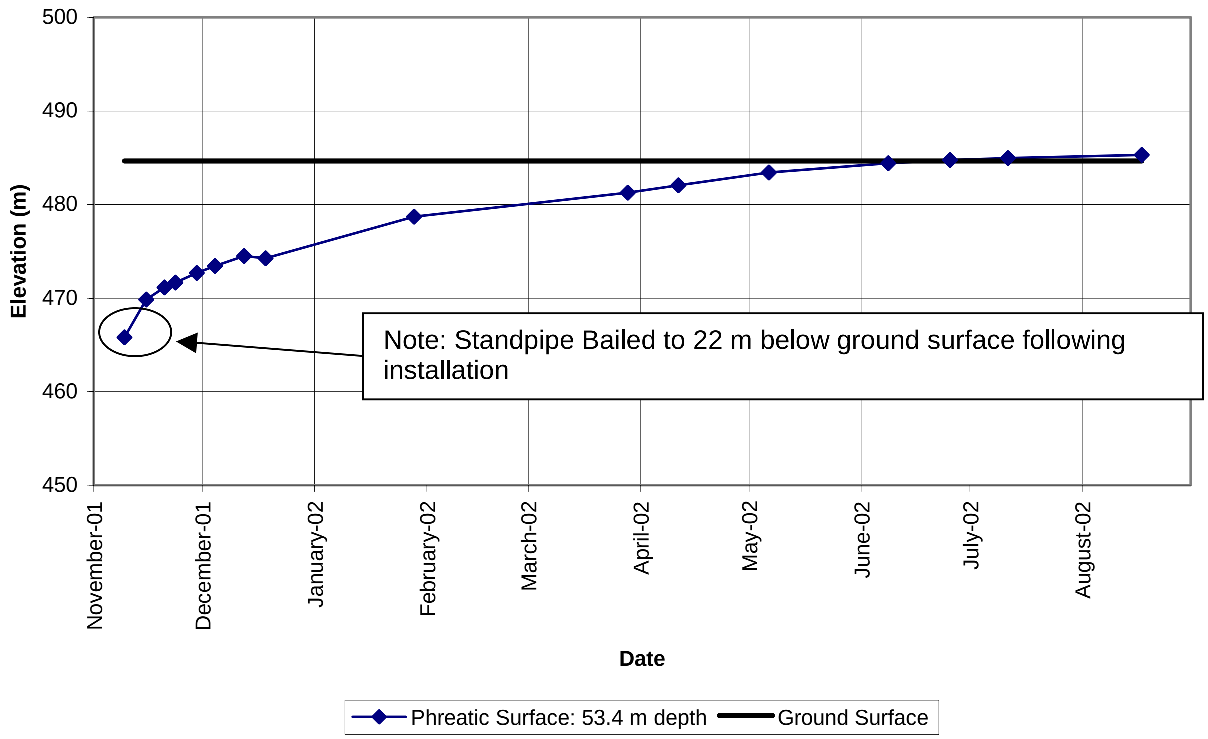

Chart 2: Piezometric data from BH-2A.

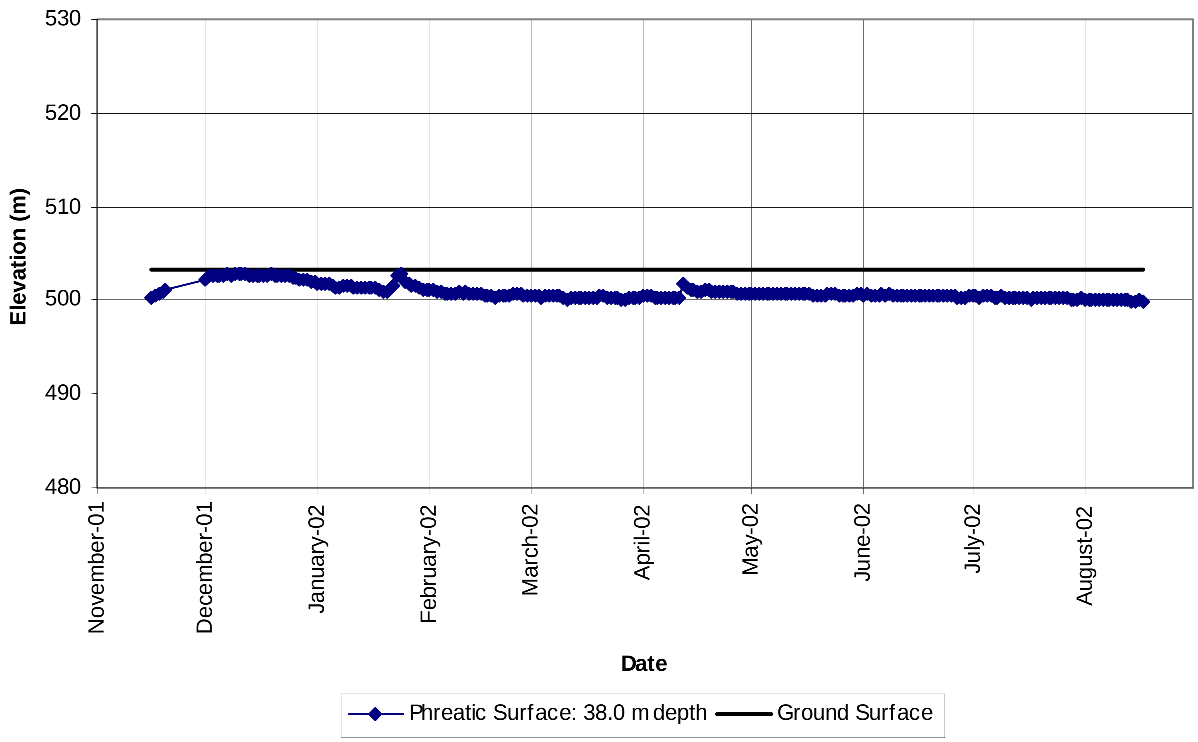

Chart 3: Piezometric data from BH-3A.

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City of Quesnel

West Quesnel Land Stability Study
Quesnel, BC

Charts 2 and 3	DATE: Sept 2002	SCALE: NTS	DRAWN BY: SMJ	PROJECT No: KX03904

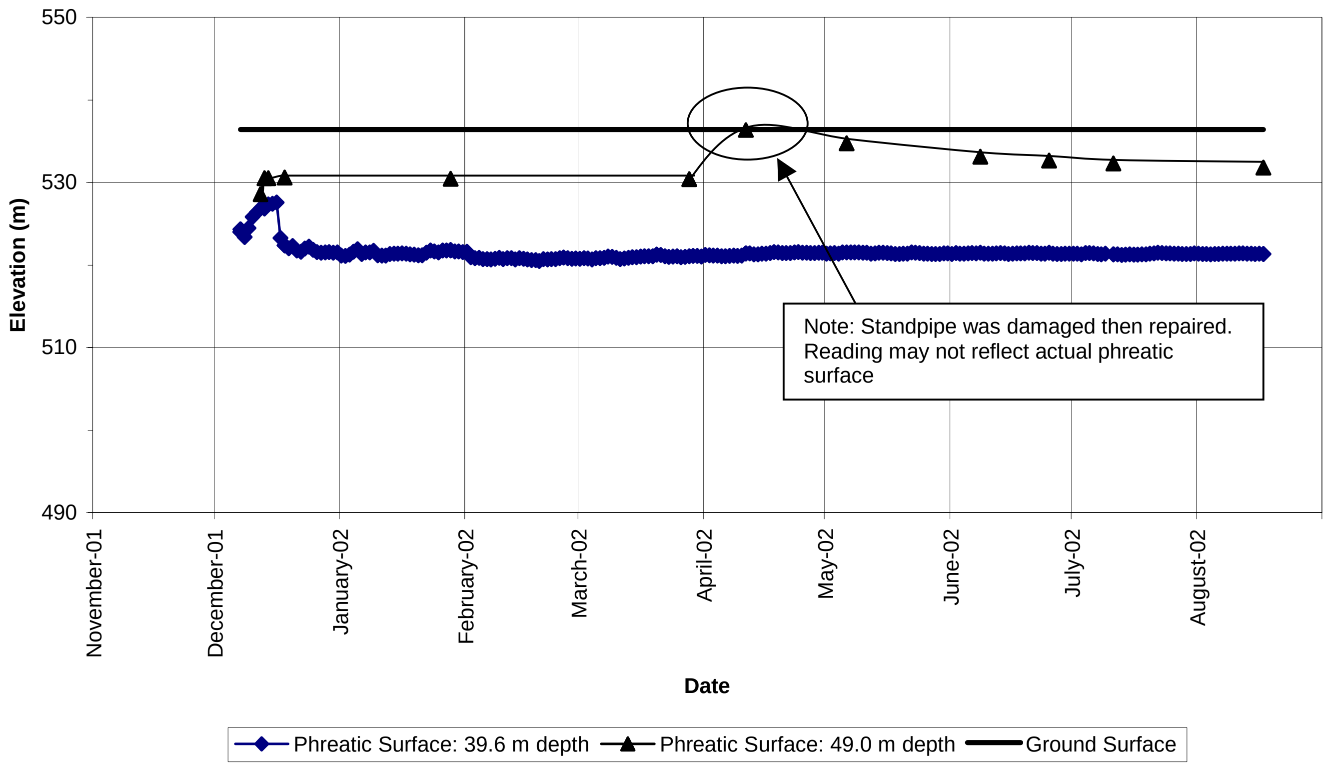

Chart 4: Piezometric data from BH-4A.

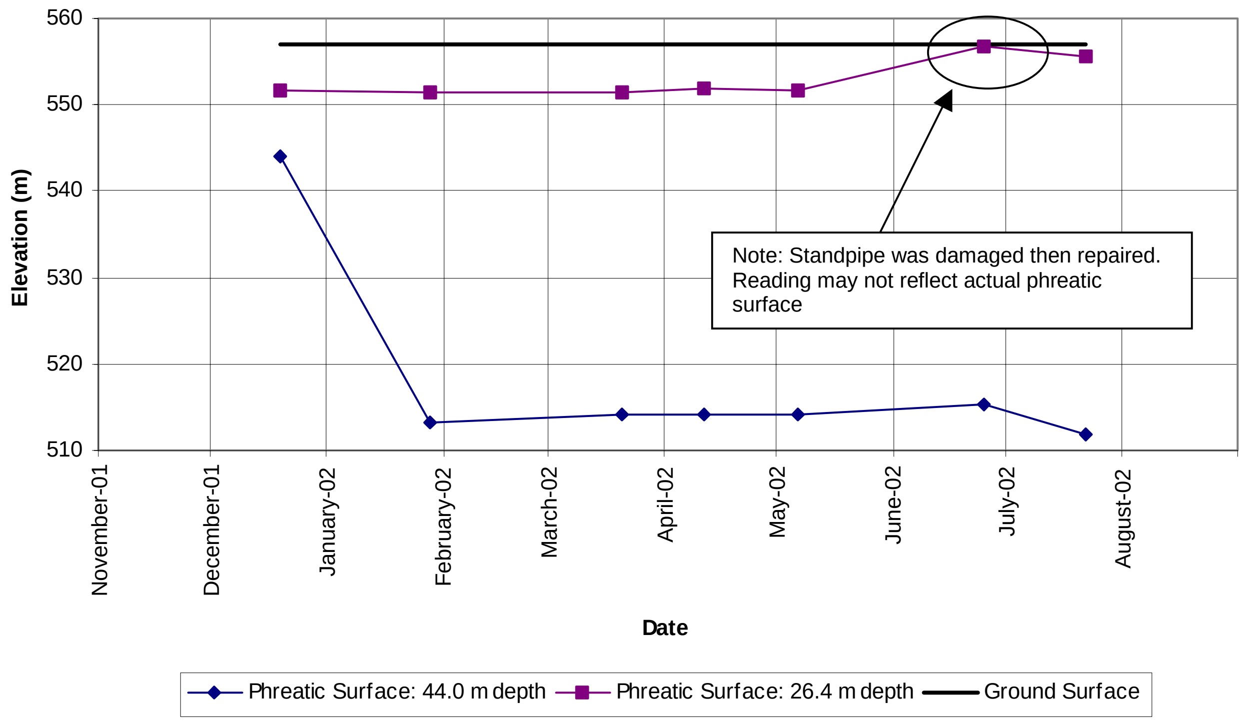

Chart 5: Piezometric data from BH-6A.

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West Quesnel Land Stability Study
Quesnel, BC

Charts 4 and 5	DATE: Sept 2002	SCALE: NTS	DRAWN BY: SMJ	PROJECT No: KX03904

Monthly Precipitation for 1975 to 2002

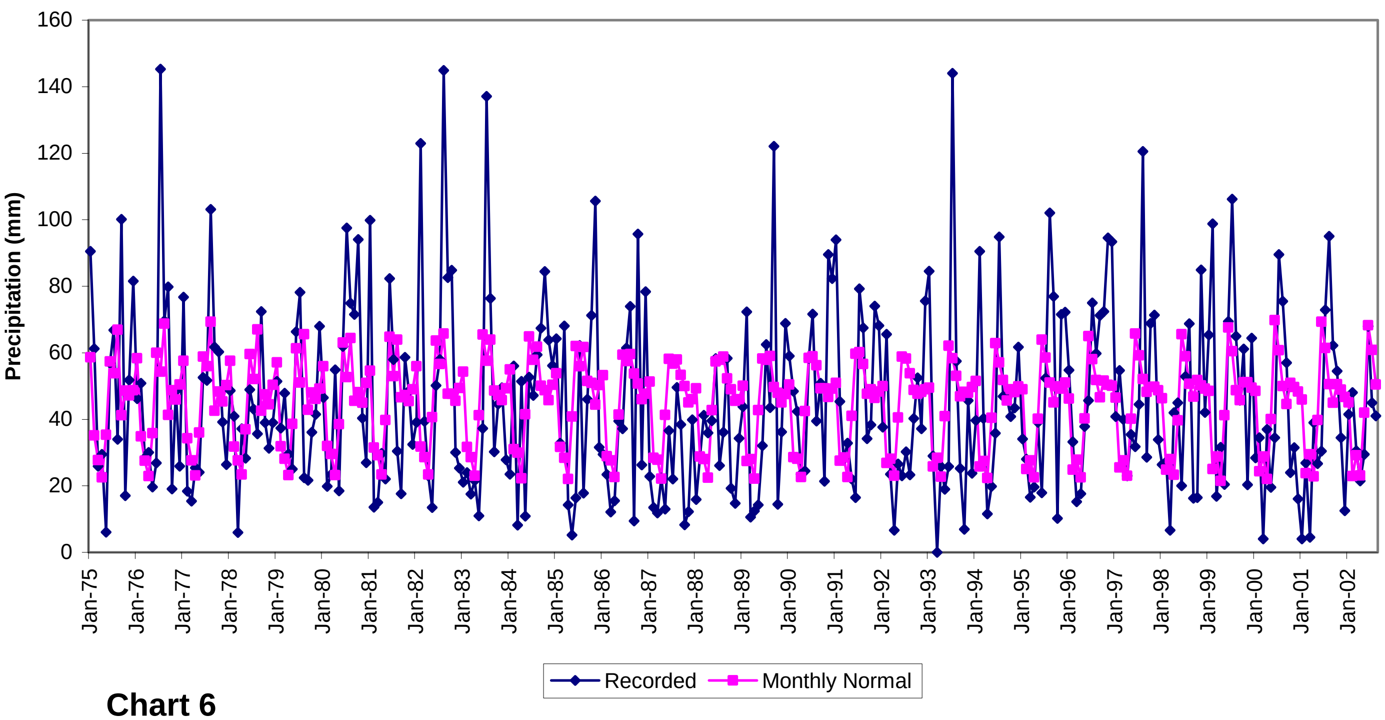

Chart 6

Cummulative Difference for 1975 to 2002

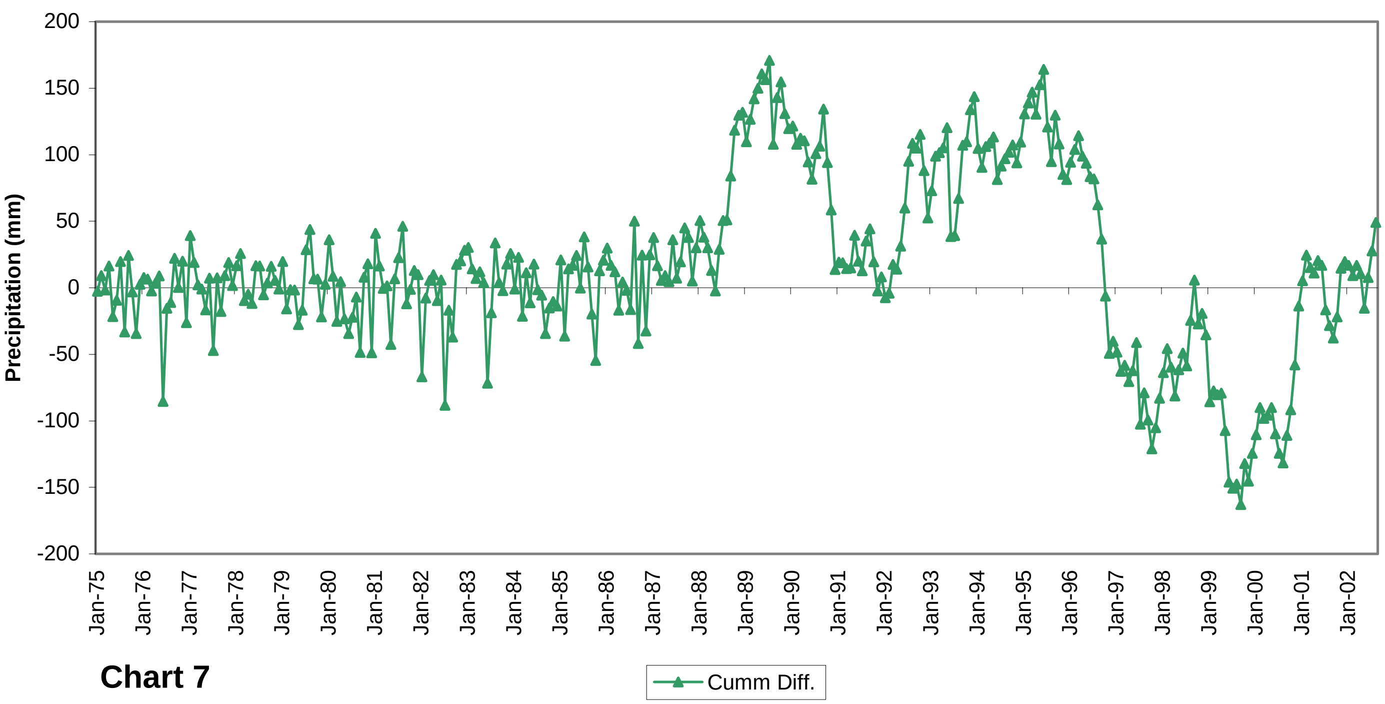

Chart 7

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West Quesnel Stability Study
Quesnel, BC

Charts 6 and 7

DATE:	SCALE:	DRAWN BY:	PROJECT No: KX03904
July 2002	NTS	SMJ	

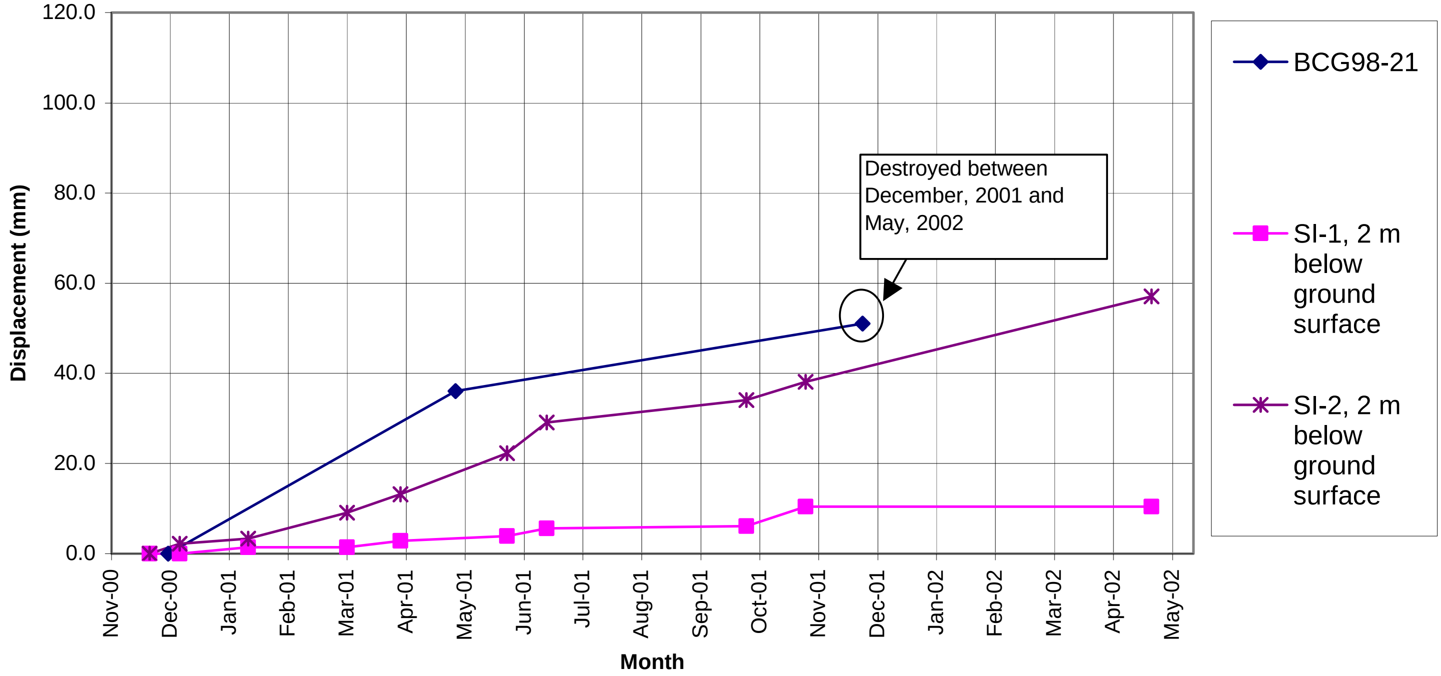

Chart 8: Velocity plots for BC Gas movement hub 98-21, SI-1 and SI-2

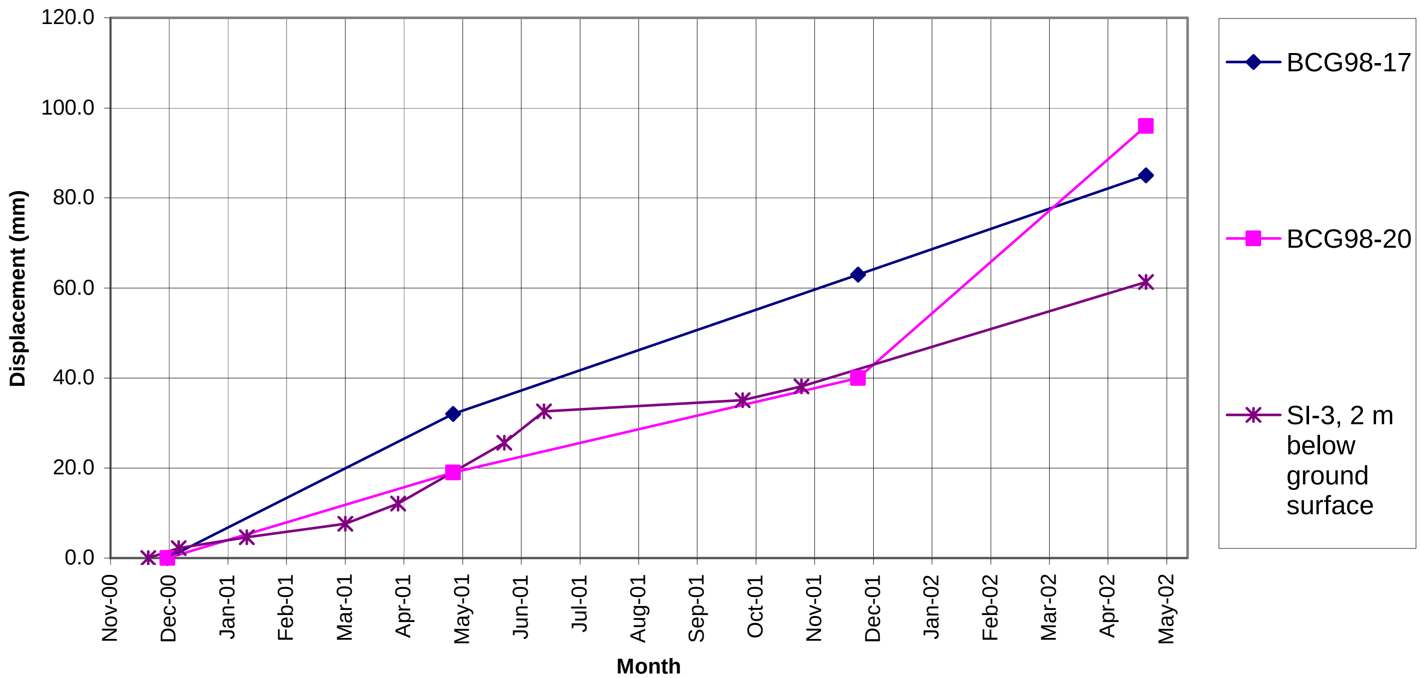

Chart 9: Velocity plots for BC Gas movement hubs 98-17, 98-20 and SI-3

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West Quesnel Land Stability Study
Quesnel, BC

Charts 8 and 9

DATE:	SCALE:	DRAWN BY:	PROJECT No: KX03904
Sept 2002	NTS	SMJ	

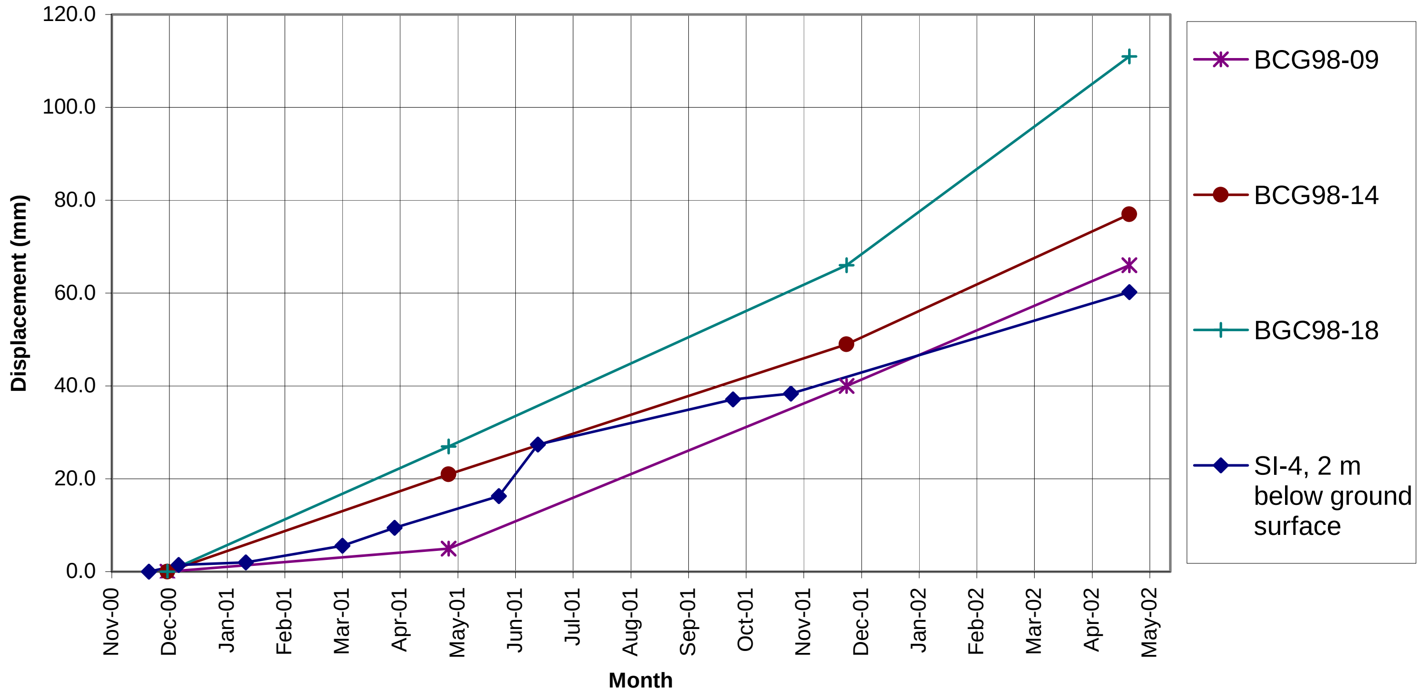

Chart 10: Velocity plots for BC Gas movement hubs 98-09, 98-14, 98-18 and SI-4

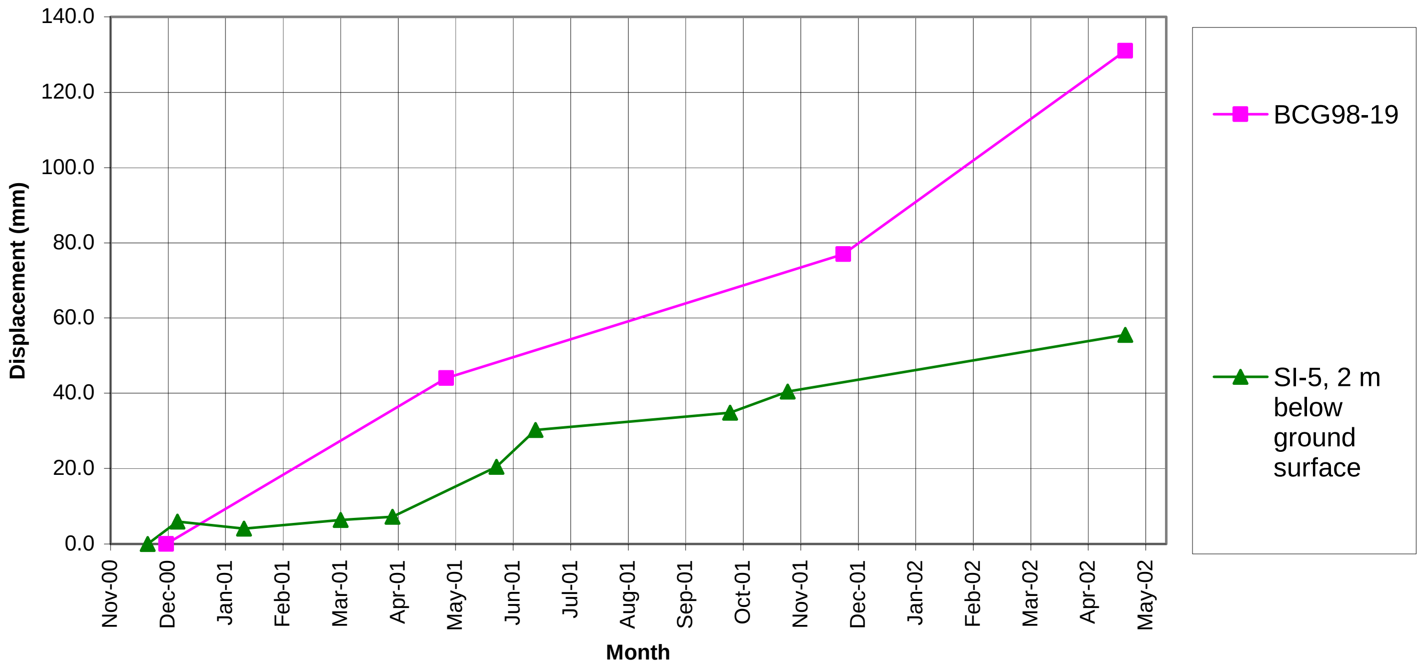

Chart 11: Velocity plots for BC Gas movement hub 98-19 and SI-5

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West Quesnel Land Stability Study
Quesnel, BC

Charts 10 and 11	DATE: Sept 2002	SCALE: NTS	DRAWN BY: SMJ	PROJECT No: KX03904

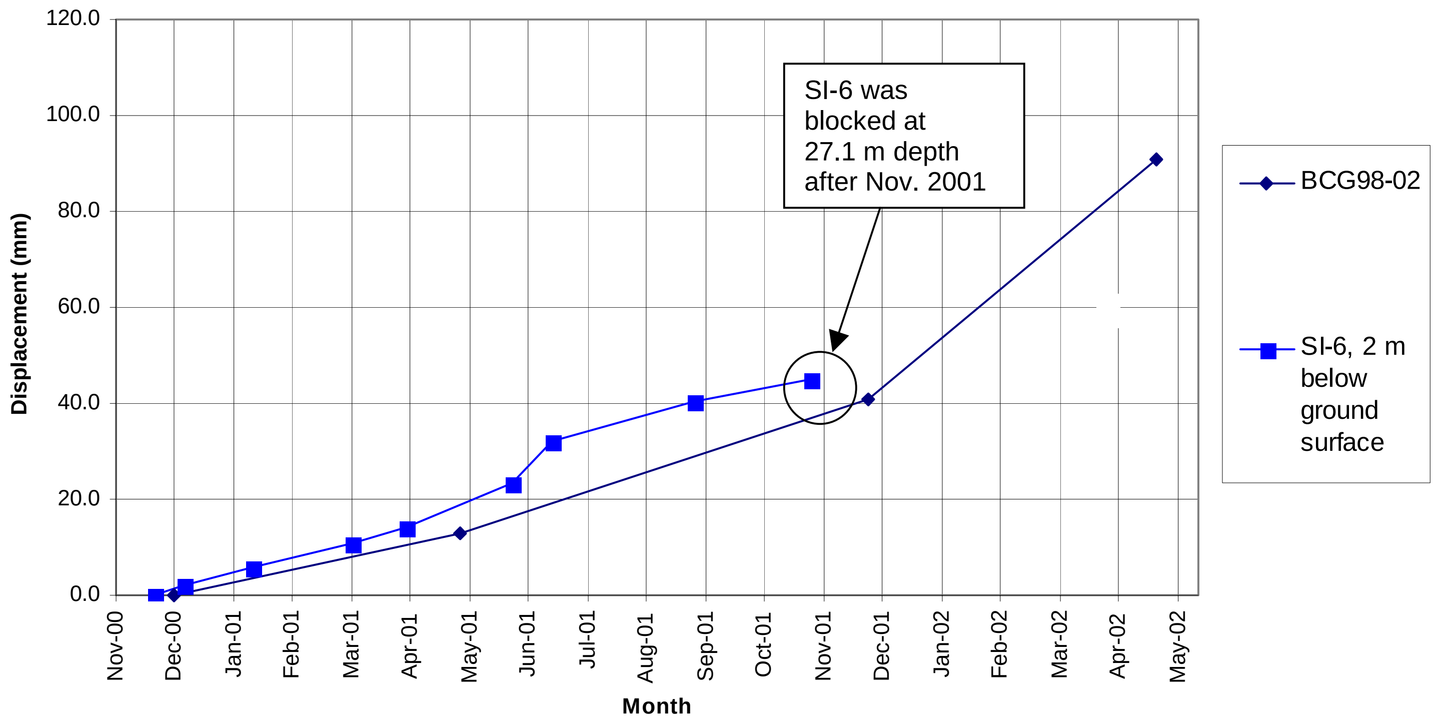

Chart 12: Velocity plots for BC Gas movement hub 98-02 and SI-6

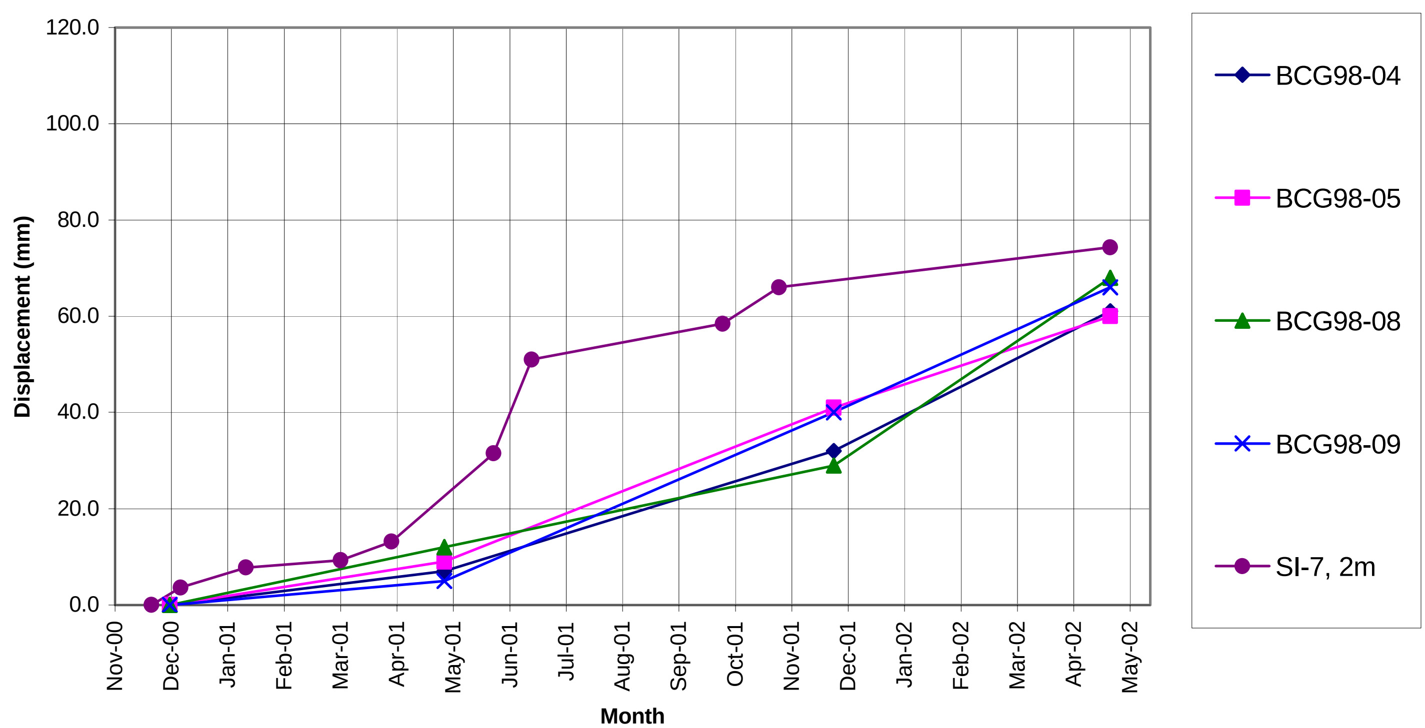

Chart 13: Velocity plots for BC Gas Movement Hubs 98-04, 98-05, 98-08, 98-09 and SI-7

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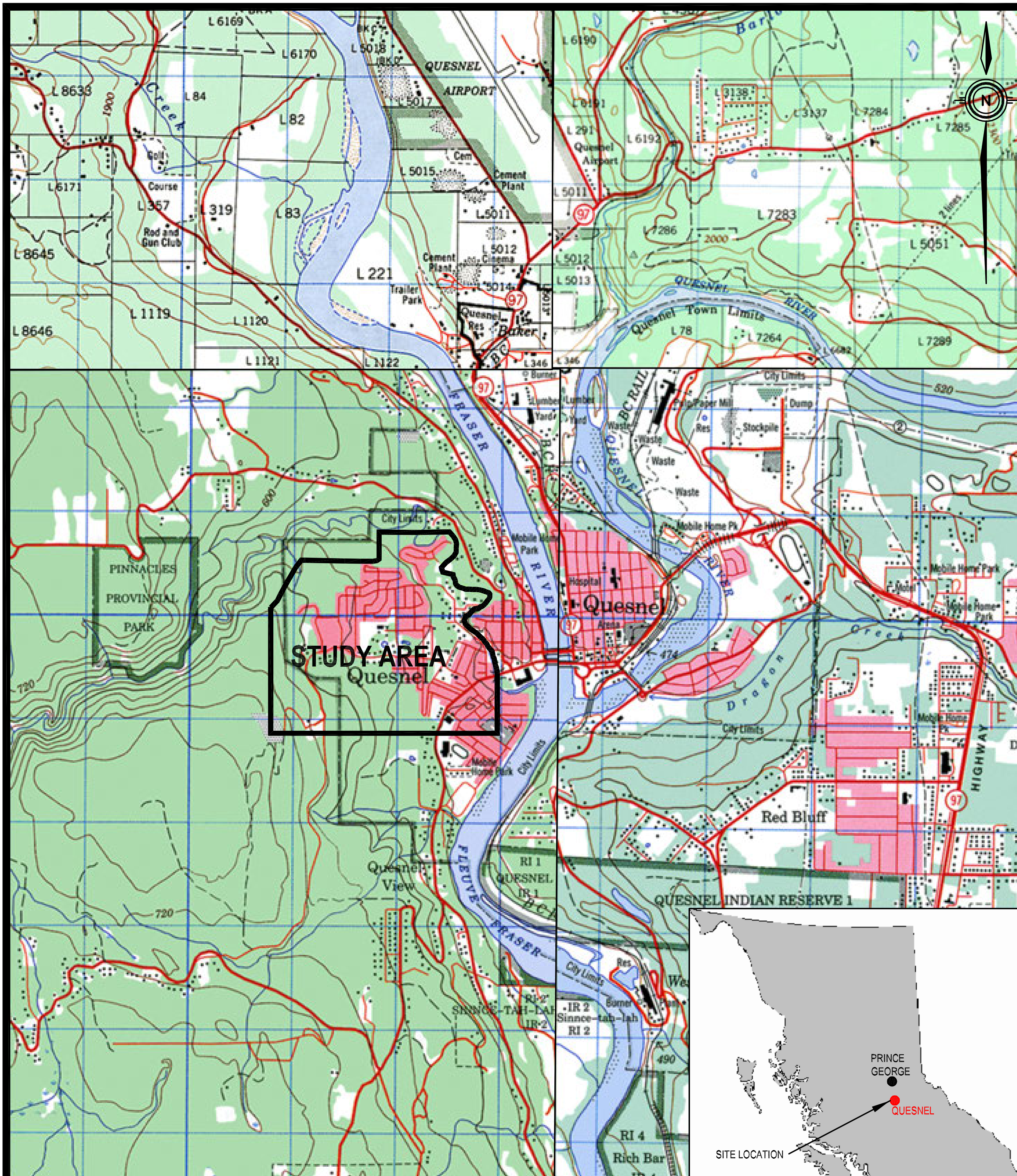
City of Quesnel

West Quesnel Land Stability Study
Quesnel, BC

Charts 12 and 13

DATE:	SCALE:	DRAWN BY:	PROJECT No: KX03904
Sept 2002	NTS	SMJ	



Note: Spectrum Digital NTS map 93B15, 93B16, 93G1 and 93G2 Scale 1:50 000

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CLIENT: CITY OF QUESNEL

TITLE: STUDY AREA LOCATION

PROJECT: WEST QUESNEL LAND STABILITY STUDY
QUESNEL, BC

DATE:	SEPTEMBER 2002
JOB #:	KX03904
DRAWN BY: SANDE	PROJECT MGR: DDE
CAD FILE:	DRAFTING\KX03904\FIGURE1.DWG
FIGURE #:	FIGURE 1

NOTE: ORIGINAL BASE MAP PRODUCED FROM DATA PROVIDED BY MCELHANNEY CONSULTING AND SERVICES LTD. AND CADASTERAL MAPPING PROVIDED BY BC GAS UTILITY LTD. CONTOURS AT 2.5m INTERVALS.
DIGITAL IMAGE IS SCANNED FROM BCC97136 NO.73 (1997)

LEGEND:

- BCG98-23 — 1998 BC GAS GPS SURVEY HUB
- BCG01-36 — 2001 BC GAS GPS SURVEY HUB
- AMEC SLOPE INCLINOMETER INSTALLATIONS (2000)
- BH-2A — AMEC BOREHOLES (2001)
- MOT-TH1 — MOT BOREHOLES FROM PREVIOUS REPORT
- TAG06132 — WATERWELLS

SCALE: 1:4000 — 0, 100, 200, 300m

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610 Richard Road
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V2K 4L3
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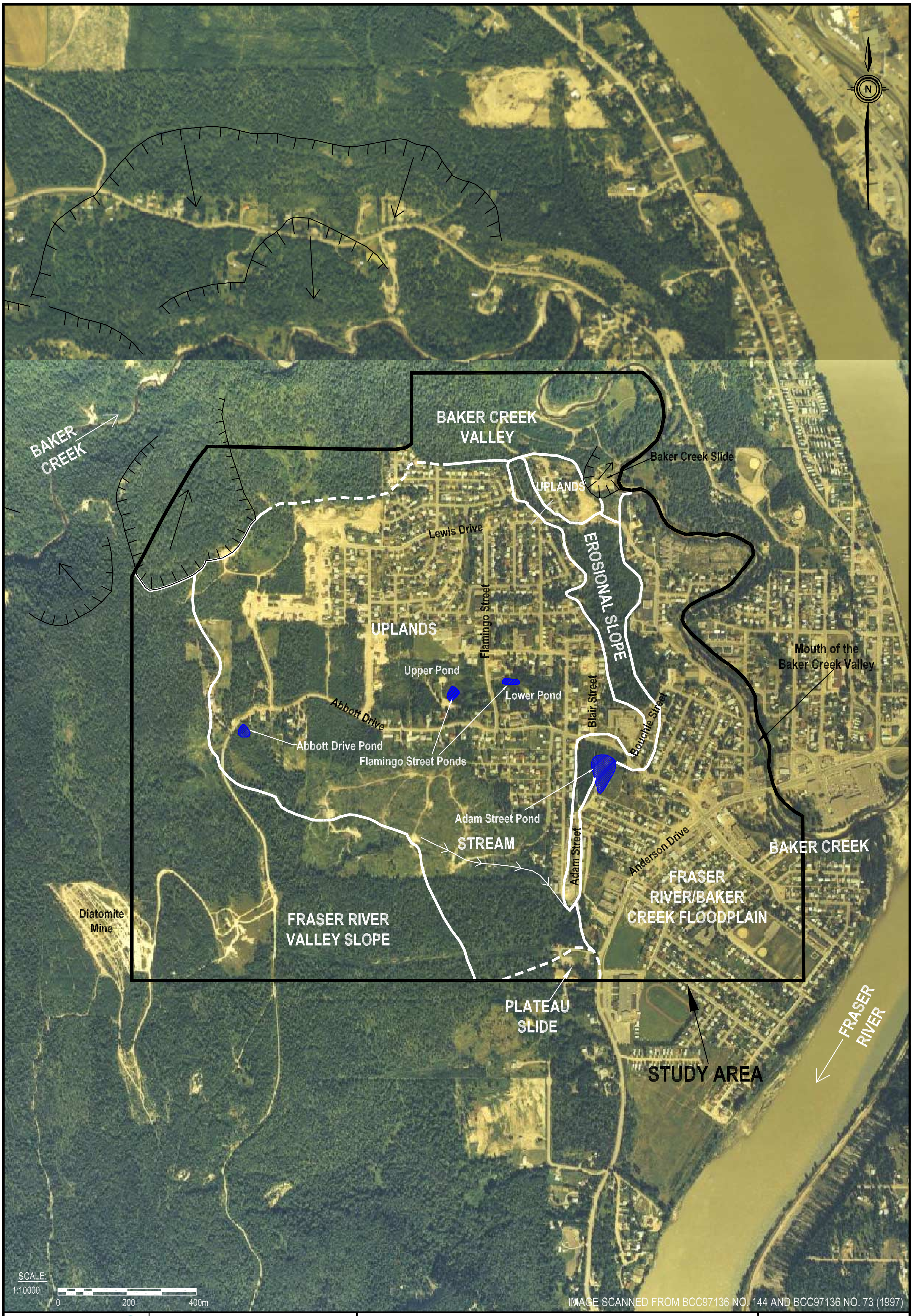
Fax. 562-7045

CLIENT: **CITY OF QUESNEL**

TITLE: **STUDY AREA LAYOUT**

PROJECT: **WEST QUESNEL LAND STABILITY STUDY QUESNEL, BC**

DATE:	SEPTEMBER 2002
JOB #:	KX03904
DRAWN BY:	SANDE
PROJECT MGR:	DDE
CAD FILE:	DRAFTING\KX03904\FIGURE2.DWG
FIGURE #:	FIGURE 2



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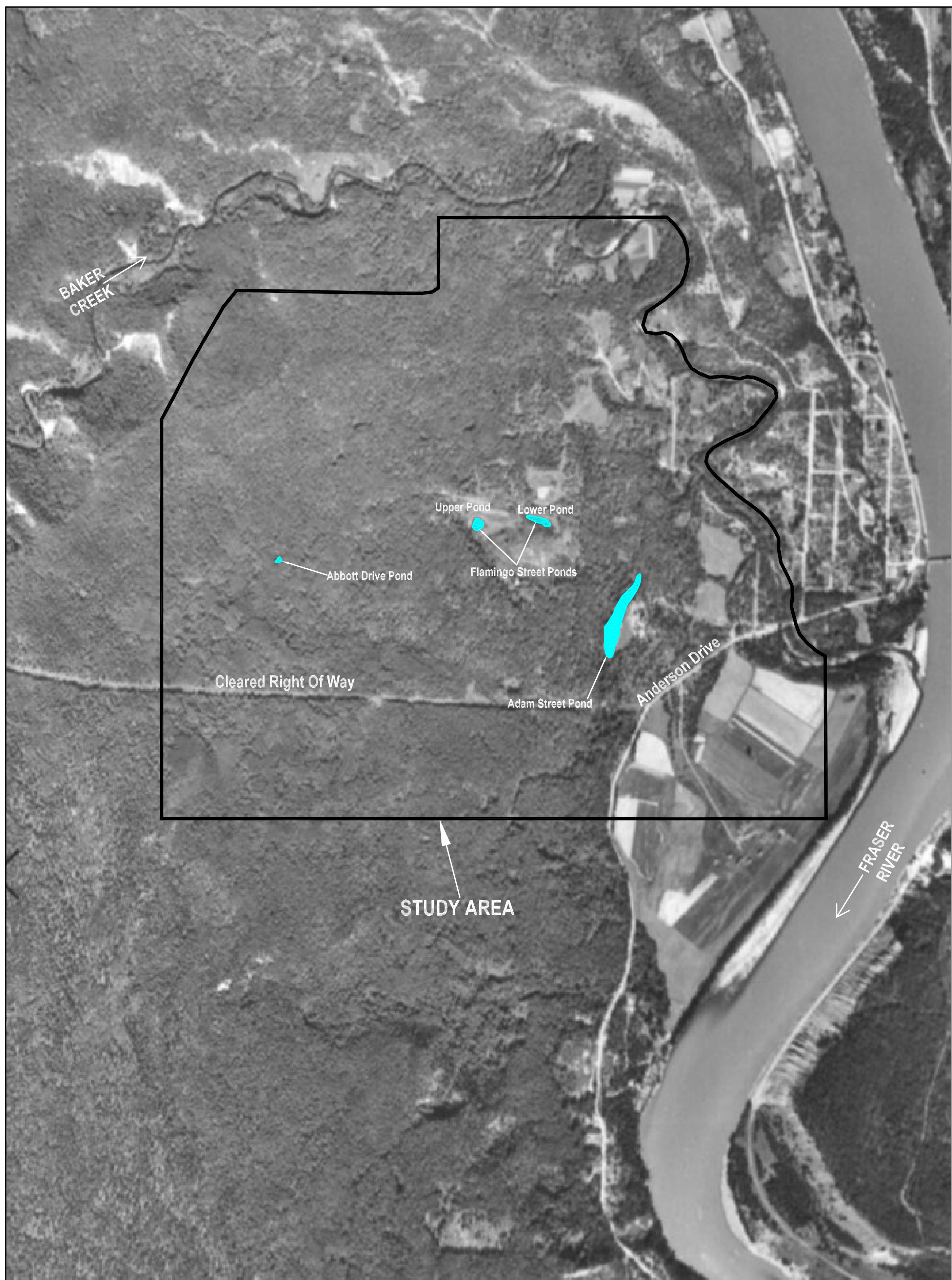
Prince George, B.C.
V2K 4L3
Tel. 564-3243
Fax. 562-7045

CLIENT:
CITY OF QUESNEL

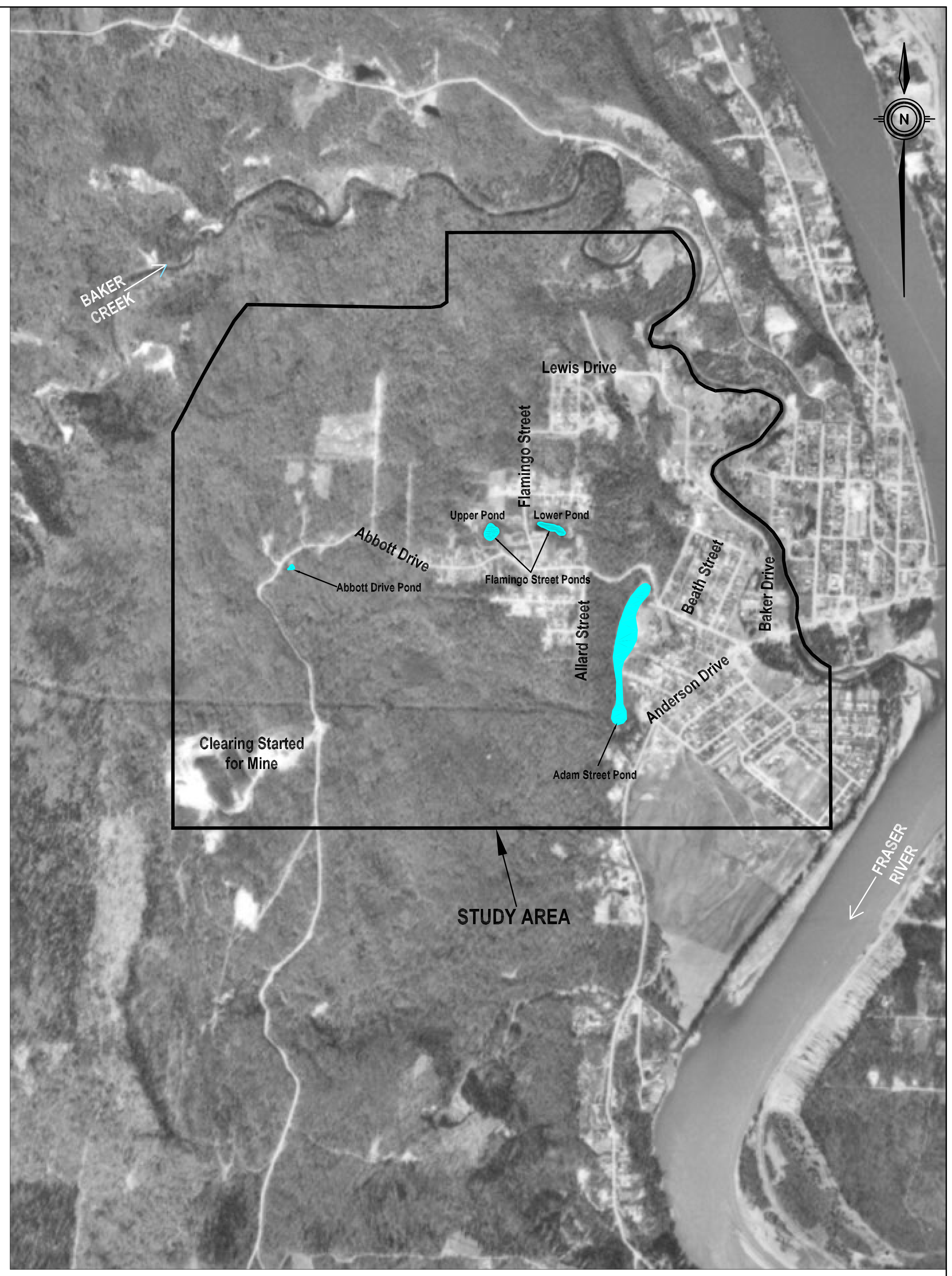
TITLE:
TOPOGRAPHIC FEATURES WITHIN STUDY AREA

PROJECT:
WEST QUESNEL LAND STABILITY STUDY
QUESNEL, BC

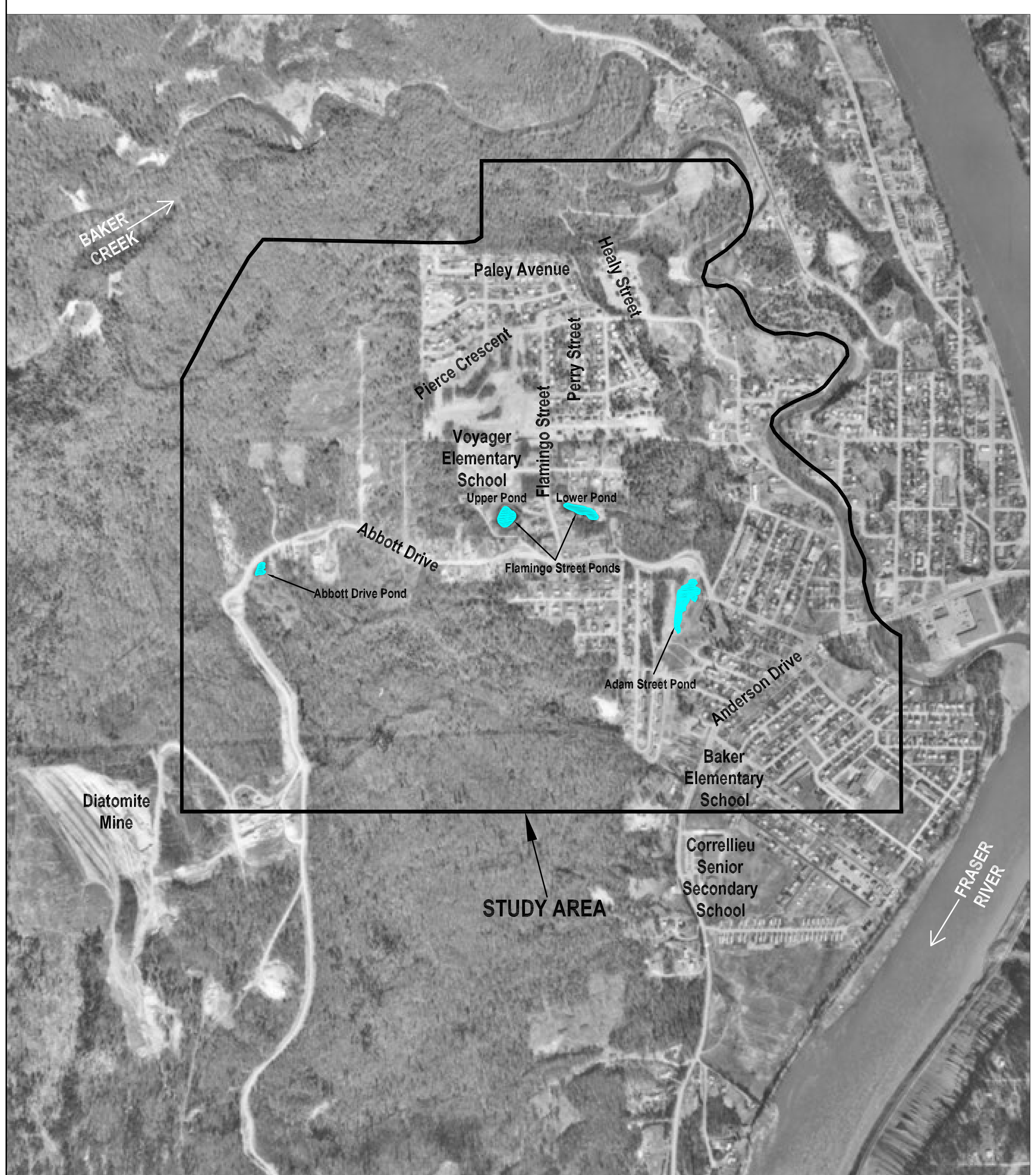
DATE: SEPTEMBER 2002
JOB #: KX03904
DRAWN BY: SANDE
PROJECT MGR: DDE
CADFILE: DRAFTING\KX03904\FIGURE3.DWG
FIGURE #: FIGURE 3



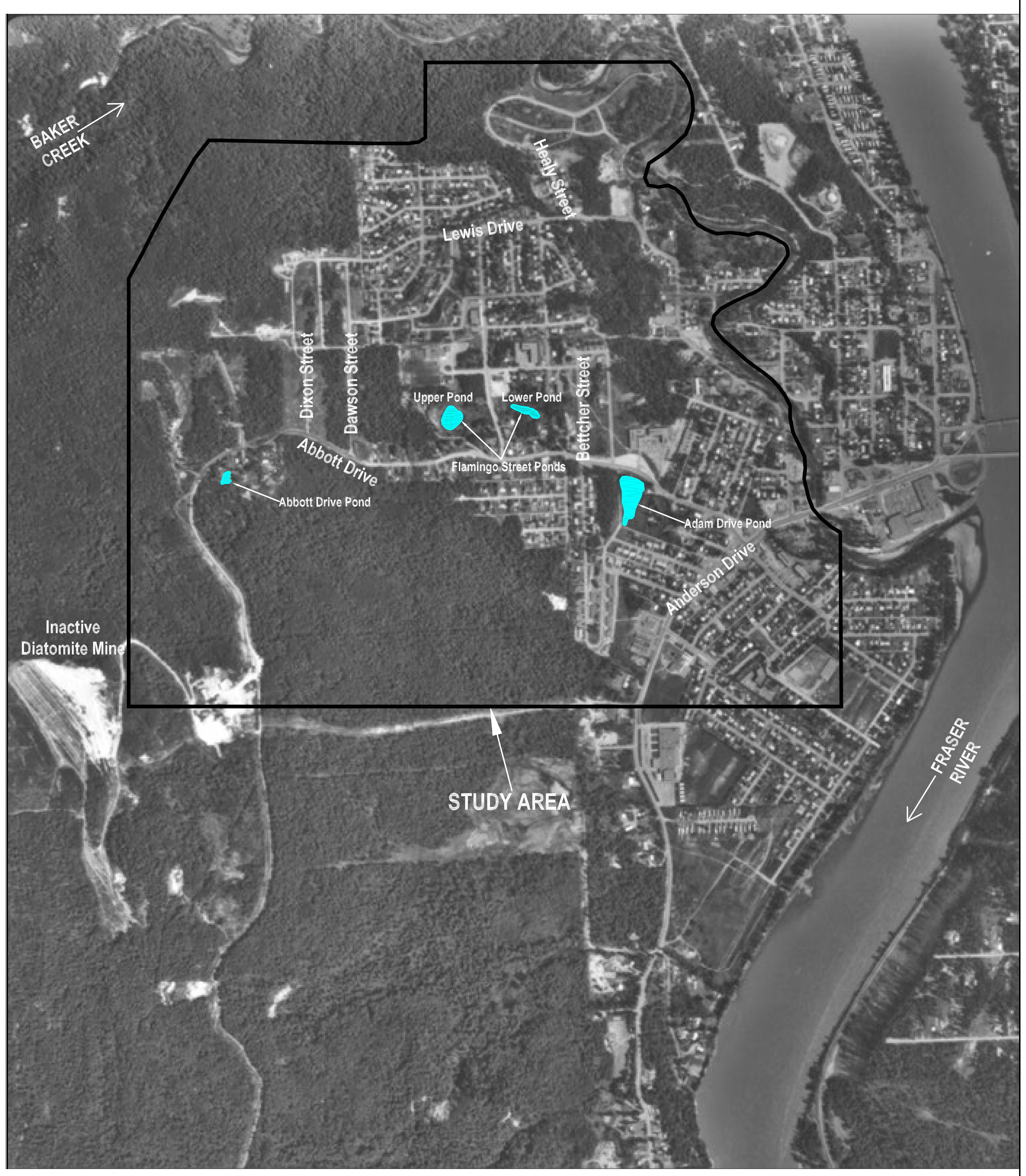
1949 Air Photo BC940 no. 93



1969 Air Photo BC5328 no. 226



1976 Air Photo BC5709 no. 241



1985 Air Photo BC85014 no. 203

SCALE:
1:15 000
0 300 500 700m

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Tel: 564-3243
Fax: 562-7045

CLIENT:
CITY OF QUESNEL

TITLE:
MUNICIPAL DEVELOPMENT SINCE 1949

PROJECT:
WEST QUESNEL LAND STABILITY STUDY
QUESNEL, BC

DATE: SEPTEMBER 2002	
JOB #: KX03904	
DRAWN BY: SANDE	PROJECT MANAGER: DDE
CAD FILE: DRAFTING\KX03904\FIGURE4.DWG	
FIGURE #: FIGURE 4	

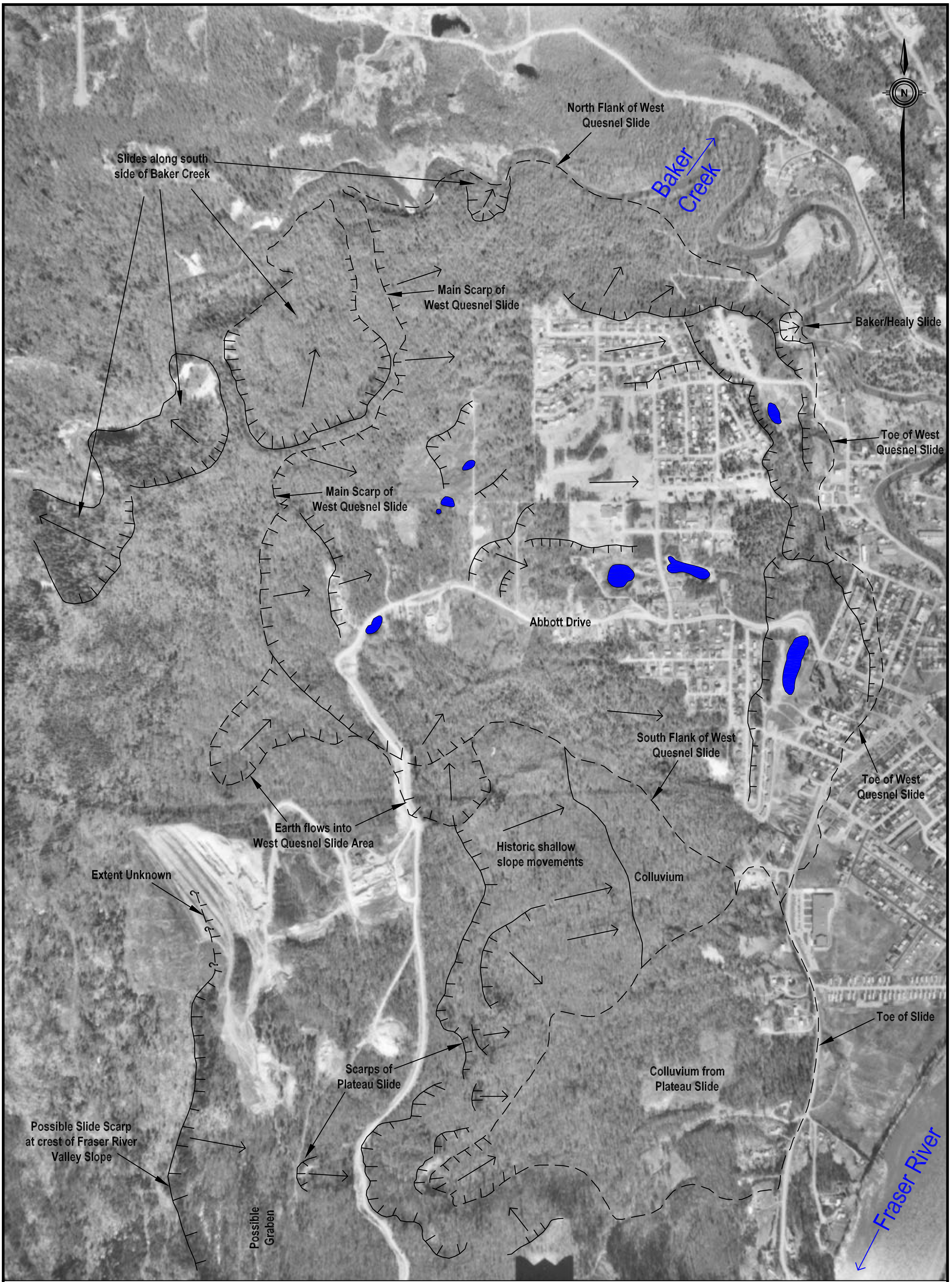


Image scanned from airphoto BC5709 No. 241 (1976)

SCALE:
1:10000
0 200 400m

Legend
Wet area or ponded water
Scarp
Approximate scarp
Slide Boundary (Flank or Toe)

Note: Only slides within the study area have been delineated. Other instabilities exist North of Baker Creek within the extent of the figure presented above.

All boundaries of the West Quesnel Slide are approximate and may vary from those shown

AMEC Earth & Environmental Limited 610 Richard Road Prince George, B.C. V2K 4L3 Tel. 564-3243 Fax. 562-7045	TITLE: **LANDSLIDE FEATURES**	DATE: **SEPTEMBER 2002**	
		JOB #: **KX03904**	
CLIENT: **CITY OF QUESNEL**	PROJECT: **WEST QUESNEL LAND STABILITY STUDY QUESNEL, BC**	DRAWN BY: **SANDE**	PROJECT MGR: **DDE**
		CADFILE: **DRAFTING\KX03904\FIGURE5.DWG**	
		FIGURE #: **FIGURE 5**	

NOTE: ORIGINAL BASE MAP PRODUCED FROM DATA PROVIDED BY MCELHANNEY CONSULTING AND SERVICES LTD. AND CADASTERAL MAPPING PROVIDED BY BC GAS UTILITY LTD. CONTOURS AT 2.5m INTERVALS.
DIGITAL IMAGE IS SCANNED FROM BCC97136 NO.73 (1997).

LEGEND:

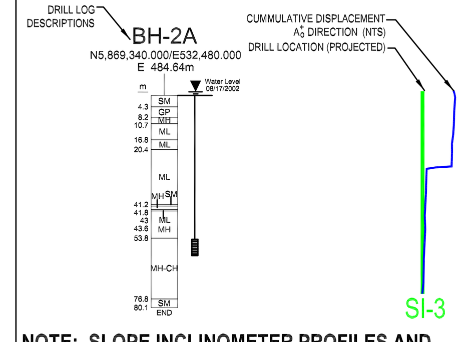
- Reported City Service Line Break
- OC1 Out Crop Location
- Observed/Reported Cracks in Pavement and Houses
- Reported BC Gas line breaks

SCALE:
0 100 200 300m

amec	AMEC Earth & Environmental Limited 610 Richard Road Prince George, B.C. V2K 4L3 Tel. 564-3243 Fax. 562-7045	TITLE: SITE OBSERVATIONS	DATE: AUGUST 2002 JOB #: KX03904
CLIENT: CITY OF QUESNEL		PROJECT: WEST QUESNEL LAND STABILITY STUDY QUESNEL, BC	DRAWN BY: SANDE PROJECT MGR: DDE CAD FILE: DRAFTING\KX03904\FIGURE6.DWG FIGURE #: FIGURE 6

LEGEND:

- FLUVIAL SEDIMENTS
- GLACIOLACUSTRINE DEPOSITS, TILL OR COLLUVIUM
- TERTIARY SEDIMENTS
- BEDROCK



NOTE: SLOPE INCLINOMETER PROFILES AND BOREHOLE LOGS ARE PROJECTED ONTO THE CROSS SECTION. ALL SLOPE INCLINOMETER AND BOREHOLE LOCATIONS ARE WITHIN 200m OF THE CROSS SECTION

SCALE:
(APPROXIMATE)
1:2000
0 25 50 75 100 125m

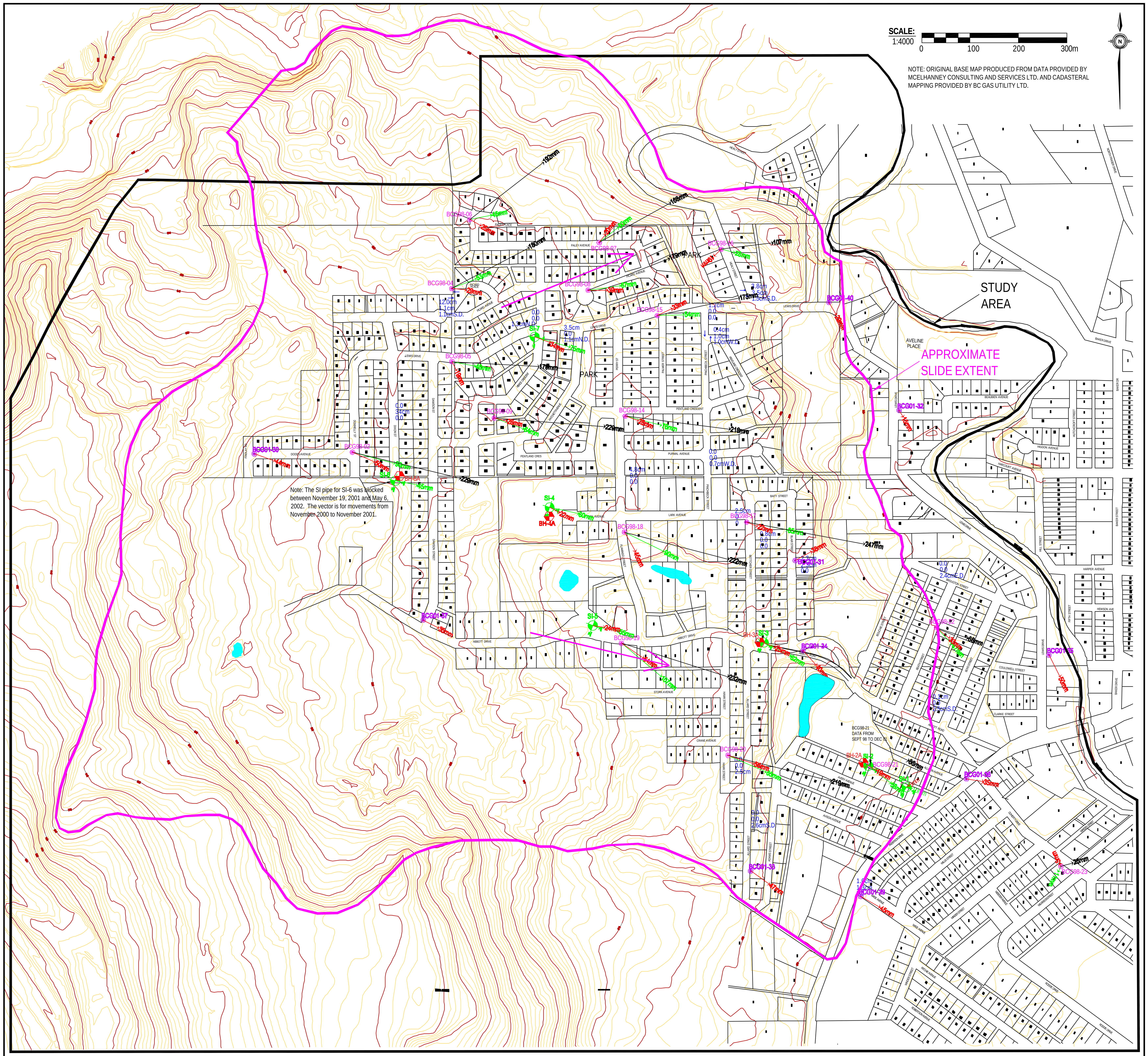
Client:
CITY OF QUESNEL

AMEC Earth & Environmental Limited
610 Richard Road
Prince George, B.C.
V2K 4L3
Tel. 564-3243
Fax. 562-7045

Title:
CROSS SECTION

Project:
WEST QUESNEL LAND STABILITY STUDY

Drawn By: S.SANDE	Project No.: KX03904
Approved By: DLD/NP	Date: SEPTEMBER 2002
CADD File: DRAFTING\KX03904\FINAL\FIGURE7.DWG	
Sheet No.: FIGURE 7	



LEGEND:

- 45mm TOTAL VECTOR - SEPTEMBER 1998 TO MAY 2002
- 45mm INCREMENTAL VECTOR - DECEMBER 2000 TO MAY 2002
- 45mm INCREMENTAL VECTOR - DECEMBER 2001 TO MAY 2002
- BCG98-23 1998 BC GAS GPS SURVEY HUB
- BCG01-36 2001 BC GAS GPS SURVEY HUB
- AMEC SLOPE INCLINOMETER

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Prince George, B.C.
V2K 4L3
Tel. 564-3243
Fax. 562-7045

CLIENT: CITY OF QUESNEL

TITLE: SI AND BC GAS GPS MOVEMENT HUBS DISPLACEMENT VECTORS

PROJECT: WEST QUESNEL LAND STABILITY STUDY QUESNEL, BC

DATE:	SEPTEMBER 2002
JOB #:	KX03904
DRAWN BY:	SANDE
PROJECT MGR:	DDE
CAD FILE:	DRAFTING\KX03904\FIGURE8.DWG
FIGURE #:	FIGURE 8

LEGEND:

AMEC SLOPE INCLINOMETER INSTALLATIONS (2000)

BH-2A AMEC BOREHOLES (2001)

PROPOSED PRODUCTION WATER WELL LOCATION

NOTE: ORIGINAL BASE MAP PRODUCED FROM DATA PROVIDED BY MCELHANNEY CONSULTING AND SERVICES LTD. AND CADASTERAL MAPPING PROVIDED BY BC GAS UTILITY LTD. CONTOURS AT 2.5m INTERVALS. DIGITAL IMAGE IS SCANNED FROM BCC97136 NO.73 (1997)

SCALE: 1:4000

0 100 200 300m

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V2K 4L3
Tel. 564-3243
Fax. 562-7045

CLIENT:

CITY OF QUESNEL

TITLE: **SLIDE EXTENT**

PROJECT: **WEST QUESNEL LAND STABILITY STUDY QUESNEL, BC**

DATE:	AUGUST 2002
JOB #:	KX03904
DRAWN BY: SANDE	PROJECT MGR: DDE
CAD FILE:	DRAFTING\KX03904\FIGURE9.DWG
FIGURE #:	FIGURE 9