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April 16, 1999

Geotechnical and Materials Engineering
BC Ministry of Transportation and Highways
Thompson Okanagan Region
1165 Battle Street, Rear Top Floor
Kamloops, British Columbia
V2C 2N4

Mr. Terry Harbicht, P.Eng.
Senior Geotechnical Engineer

Dear Mr. Harbicht:

Final Report on Geotechnical Stability Investigation
Clarke-Bromley Landslides, Princeton, BC

We are pleased to submit two bound copies and one unbound copy of our final report on the geotechnical stability investigation for the Clarke and Bromley landslides near Princeton, British Columbia. As per your instructions of April 14, 1999, we have also submitted one copy of the report to the South Okanagan District office in Penticton, British Columbia.

Yours truly,

KC GEOTECHNICAL CONSULTANTS LTD.

Neil K. Singh, P.Eng.
Project Manager

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1. INTRODUCTION

This report presents the results of KC Geotechnical's geotechnical stability investigation for the Clarke and Bromley landslides about 3 km west of Princeton, BC. Problems at the site consist of two large scale landslides which are affecting Highway 3, Stevenson Road and two private properties. The slides are essentially back-to-back with Highway 3 located on a narrow ridge between the headscarps (Figure 1). Slide retrogression at Clarke slide, located northwest of the highway, has removed one lane of Highway 3 and damaged private property. Retrogression at the Bromley slide headscarp has disrupted a proposed development, is threatening services to an existing development, and ultimately is threatening Highway 3 from the southeast. Evidence of landslides is also found at the toe of the Bromley slope adjacent to the Similkameen River. In a large knob area, the ground east of Stevenson Road is failing, with debris protruding into the river. Elsewhere on the lower Bromley slope, evidence of slope instability includes tension cracks, curved and bent trees and cracking damage to private property.

KC Geotechnical Consultants Ltd. was retained by the BC Ministry of Transportation and Highways (MoTH) to carry out a geotechnical investigation of the slides. The objectives of the investigation were:

- Define the perimeter, groundwater conditions and failure surfaces of the actively moving landslides;
- Perform stability analyses to determine the most probable causes of failure, contributing factors and proposed remedial measures; and
- Present recommended design solutions to the stabilization of the landslides, and other mitigative measures which may be undertaken.

The work was authorized by Mr. Terry Harbicht, P.Eng., Senior Geotechnical Engineer, under Consulting Services Contract No. 208CS0203 dated November 4, 1998. During the course of the work MoTH and KC Geotechnical Consultants agreed to increase the scope of a proposed drilling program and add two objectives. The added objectives are:

- To review proposed temporary remedial measures to place limited fill and restore the guardrail at the Clarke slide; and,
- To provide a contingency plan in the event of failure which would block Highway 3.

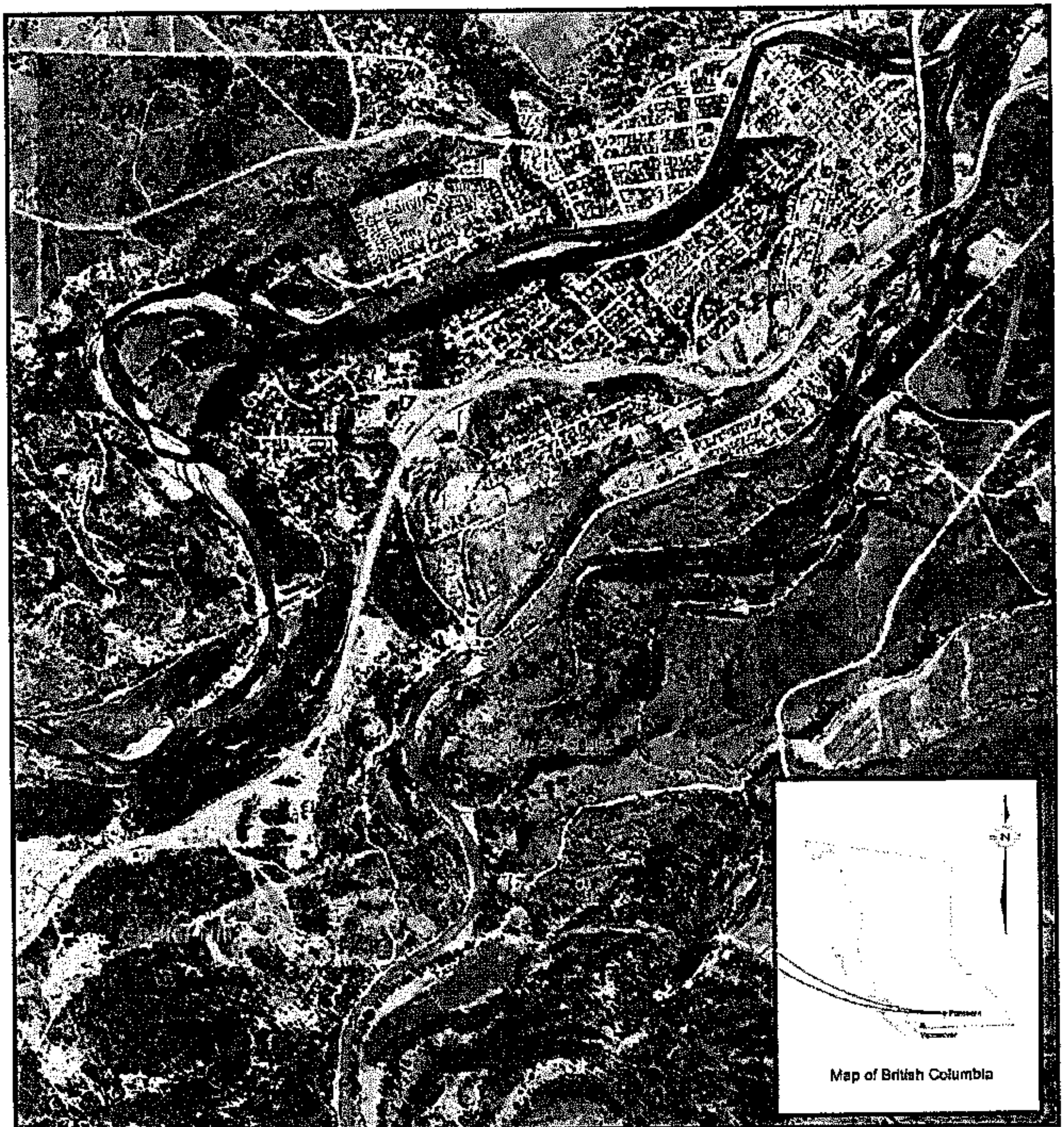
The additional scope was authorized under Amending Agreement No. 1 to the above Consulting Services Contract, dated November 23, 1998.

During the work, KC Geotechnical Consultants issued a progress report dated October 21, 1998. The progress report reviewed results of the geologic mapping and test pitting, presents our understanding of the slides at the time, and made recommendations for the proposed drilling program.

On November 17, 1998, MoTH faxed a sketch showing the proposed guard rail replacement details to KC Geotechnical Consultants. These details were reviewed and a letter concurring with the proposed work was sent to MoTH on November 20, 1998. The guard rail replacement work was completed on November 28, 1998.

The drilling portion of the investigation was undertaken in January, 1999 followed by laboratory testing of core samples. On February 12, 1999, KC Geotechnical issued a data report including test pit logs, drill hole logs and laboratory test results to date. This is KC Geotechnical's final report on the geotechnical investigation. A letter report concerning the contingency plan will be issued separately.

As a mutual protection to our client, the public, and ourselves, all reports and drawings are submitted for the confidential information of our client for a specific project and authorization for use and/or publication of data from or regarding our reports is reserved pending our written approval.



MAP SOURCE: AERIAL PHOTOGRAPH 30BCC96044 No.140, 1996

SCALE APPROX. 1:16,000

April 16, 1999

KLOHN-CRIPPEN		DATE
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DRAWN	DSK	Mar 22/99
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RECOMMENDED		
APPROVED		

KC GEOTECHNICAL

CLIENT: BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

PROJECT: GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY SLIDES

TITLE: **AREA PLAN**

April 16, 1999 | PG 8545 01 | Figure 1

2. GEOTECHNICAL INVESTIGATIONS

2.1 Background Information

Before the field investigations, KC Geotechnical Consultants reviewed topographic mapping, drill hole logs and instrumentation records provided by MoTH. One of the topographic maps shows the Clarke slide at 1:500 scale with a 0.5 m contour spacing, as surveyed by MoTH in September, 1996. A second map, at 1:1,000 scale with a 2 m contour spacing, shows both the Clarke and Bromley slides, surveyed by MoTH in September, 1998. Drill hole logs for TH96-01, -02, and TH97-01, -02 and -03 were reviewed along with inclinometer records for TH96-01 and -02 and piezometer records for TH97-01 and -02. Preliminary sections were plotted and used in the field investigations.

Six sets of aerial photographs of the site dating from 1947, 1959, 1967, 1972, 1985 and 1996 were obtained to assist in the assessment of the history of the slides (UBC Photo Library, 1998). BC Geological Survey Map 1987-19 was obtained to assess the regional bedrock geology. In Princeton, the Tory family, who own property on the Bromley slide, provided copies of their photographic record of the slides and the log of their water well which was drilled to approximately 50 m depth below their property. They also provided water level records and the depth of two well cave-ins which have occurred.

A potentially similar landslide has occurred about 1 km north of the Clarke - Bromley landslides, just east of the Mohawk Station. A memorandum summarizing results of a geotechnical investigation at that slide, labelled PS2, was sent to KC Geotechnical by Golder Associates in Kamloops, British Columbia. A copy of the memorandum is presented in Appendix VI.

Information provided to KC Geotechnical by MoTH has been assumed to be correct. Cursory checks for obvious errors or omissions has been conducted but formal and thorough reviews of client supplied data have not been conducted. Similarly, reports by others referenced in this report have been assumed to be correct. KC Geotechnical has conducted cursory checks for obvious errors, but has not conducted formal reviews of reports by others. Information related to KC Geotechnical through verbal communications should be independently verified through checks of written or photographic records.

2.2 Surficial Mapping and Test Pitting

The field investigations were completed on September 30 to October 3, 1998. Surficial mapping was completed using the 1:1,000 map as a base. Slope stability features were mapped including headscarp locations, tension cracks, slide runouts and seepage locations. Some surficial geology was mapped in slide headscarps and road cuts. Results of the mapping are presented in Drawing D-1001.

A total of six test pits, TP98-01 to TP98-06, were dug on October 3, 1998 with an Hitachi UH07 track-mounted excavator. The excavator was supplied and operated by a local contractor, Ross Ferguson. The test pit locations are shown on Drawing D-1002. At the Bromley slide, three pits were excavated at different elevations at the southwest edge of the scarp. Unfortunately, test pits could not be excavated within the slide mass due to poor access. At the Clarke slide, two test pits were excavated within the slide mass and one was excavated southwest of the slide. Test pit logs are presented in Appendix I.

2.3 Drill Program

Five holes, DH99-01 to DH99-05, were drilled from January 6 to January 23, 1999 by the drilling subcontractor, Layne-Christensen Canada Ltd. The drill rig was a CT 350 Canterra top-drive rotary. The drill hole locations are shown on Drawing D-1002. At four of the holes, PQ core (85 mm diameter) was retrieved in overburden, with some near-surface triconing carried out to set surface casing. At DH99-01, HQ core (63.5 mm diameter) was also retrieved in sedimentary bedrock. One hole, DH99-05, was triconed for its entire length. Drill hole logs are presented in Appendix I.

Inclinometers were installed in the two holes at the Bromley slide, DH99-01 and DH99-02. The purpose of the inclinometers was to determine the depth(s) of shearing in the Bromley slide; inclinometers were not previously installed at this slide. At the Clarke slide, standpipe piezometers were installed in DH99-03 and DH99-04 to assess piezometric levels within the slide mass which were not previously known. Finally, an inclinometer was installed in DH99-05 as a replacement for inclinometers at TH96-01 and TH96-02 which cannot now be read due to Clarke slide movement.

2.4 Laboratory Testing

Laboratory tests were carried out on selected soil and bedrock samples. Index properties and shear tests on soils were completed at KC Geotechnical's laboratory in Richmond, BC. Shear tests on bedrock were carried out at Golder and Associates' laboratory in Burnaby, BC. Clay minerology tests were completed by Geotex Consultants, Vancouver, BC and Pacific Soil Analysis Inc., Richmond, BC. All laboratory test results are presented in Appendix II.

During the surficial mapping and test pitting program in October 1998, disturbed grab samples were retrieved. Moisture content and grain size tests were carried out on selected samples.

During the drill program in January 1999, heavy wall shelby tube samples (72 mm diameter), were retrieved in fine grained soil for possible shear strength testing. One direct shear test and two triaxial tests were carried out on selected samples which are known to represent weak layers in the stratigraphy. Soil unit weights and moisture contents were also recorded during the shear tests.

Two HQ core samples (63.5 mm) in bedrock, one in coal and one in a dense clay-rich seam, were wrapped for testing. Direct shear tests were carried out on each of these samples. After the direct shear tests, samples of the clay rich seam were submitted for

X-ray diffraction tests to assess clay minerology, and for cation exchange capacity and exchangeable cation tests for sodium, potassium, calcium and magnesium. Results of the tests are presented in Appendix II.

3. GEOLOGY AND GEOTECHNICAL CONDITIONS

3.1 Surficial Geology

The ridge containing Highway 3 is mantled by glacial deposits with a maximum thickness of at least 40 m (Drawing D-1003). The main capping layer has been mapped as glacial till ranging from 5 m to about 22 m in thickness. The glacial till consists mainly of clay and sand (CI) with a gravel content ranging from some gravel to gravelly (Appendix V, Photos 5 to 7). East of the highway at the Bromley slide, the glacial till is overlain by 5 to 15 m of coarser gravel, sand and gravel and gravelly sand with some silt (GW-GM, SW and SM). This material could be glaciofluvial or a coarser glacial till, and it may contain some local fill associated with Highway 3 construction and a proposed development (Appendix V, Photo 13).

The glacial till is underlain by interlaminated, glaciolacustrine clayey silt and silty clay. The generally fine grained texture in the till probably results from glacial erosion of this deposit. The interlaminated silt and clay layer has a consistent thickness of about 4 m beneath the Clarke slide as observed in drill holes and test pits (DH99-03 and TP98-05) (Appendix V, Photo 8). At the Bromley slide the layer thickness reduces to about 1.4 m, as observed in DH99-02. Drill hole intersections and outcrops (Appendix V, Photo 17) indicate that the layer dips southeast at about 2 to 3 degrees below horizontal. Cores retrieved from this material exhibited polished subhorizontal partings between clay laminae.

The silt and clay are underlain by massive, fine sand which could be glaciolacustrine or glaciofluvial in origin. Finally, a thick glaciofluvial sandy gravel layer mantles the bedrock. This gravel deposit was found underlying the sand and mantling the bedrock in road cuts southwest of the Clarke slide.

Two additional overburden units were identified in the lower Bromley slide near the Similkameen River. Both units underlie the thick sandy gravel and overlie bedrock. The first is a bedrock-like unit composed of weathered coal and shale with local sandstone layers. This unit outcrops on failing slopes below Stevenson Road, and was intersected in DH99-01 at 6.7 to 13.50 m depth. In the drill hole, the unit is underlain by 3.4 m of sand overburden, which indicates that it is a pendant of rock which was transported to its current location, possibly by colluvial or glacial transport processes. The second unit is a dense silty sand with gravel and cobble layers, which was intersected in DH99-01 at 13.50 to 16.90 m depth, and in Tory's Well at 19.5 to 29.5 m depth (description = gravel with some sand and binder, see Tory well log, Appendix I). The material is interpreted as glacial till. The overburden units are labelled numerically and their descriptions are summarized in Table 3-1.

Table 3-1 Summary of Overburden Units

Number	Description	Interpreted Origin
1	Gravel, Sand and Gravel, Gravelly Sand	Glacial Till or Glaciofluvial, and Fill
2	Clay and Sand	Glacial Till
3	Interlaminated Clay and Silt	Glaciolacustrine
4	Sand	Glaciolacustrine or Glaciofluvial
5	Sandy Gravel	Glaciofluvial
6	Disturbed Coal and Shale	Colluvial or Glacial Transport
7	Silty Sand	Glacial Till

This overburden stratigraphy is limited in aerial extent. An indication of the variability is seen in test pits TP98-04 and TP98-06, which were excavated at approximately the same elevation, about 85 m apart. TP98-04 encountered glacial till composed of clay and sand, while TP98-06 encountered clean glaciofluvial gravel. These data may indicate that the fine grained glaciolacustrine deposits and associated fine grained glacial till are limited to a relatively small area which was submerged by a glacial lake.

3.2 Bedrock Geology

The bedrock comprises sedimentary rocks of the Power Plant and Summers Creek formations. Descriptions of these units are presented in Geological Survey Map 87-19 (BC Geological Survey, 1987) as:

- Power Plant Shale: Grey shale, carbonaceous shale and coal; minor bentonite and thin sandstone beds; and
- Summers Creek Sandstone: White to ochre weathering sandstone, minor siltstone and shale, rare carbonaceous shale and coal.

KC Geotechnical's geologic section through the slides, Drawing D-1003, reflects the inferred location of the contact between these units shown on Map 87-19. The dip of 10 degrees below horizontal shown in section was observed in bedrock core retrieved from DH99-01. Map 87-19 shows outcrops with dips ranging from 6 degrees, south of the Bromley slide, to 31 degrees, north of the slide, so the 10 degree dip is within this range. The south-southeast bedding dip directions on the map are also approximately perpendicular to the section (i.e. true dips). During the surficial mapping, the only bedrock identified near the slides was a Summers Creek Sandstone outcrop about 400 m southwest of the Clarke slide at about El. 683.5 m.

Both the Power Plant Shale and Summers Creek Sandstone are known to contain bentonite clay seams. Bentonite is a clay with a high content of montmorillonite (Terzhagi et al., 1996). The bedrock units may also contain other clay seams composed of illite or kaolinite. The closest bentonite showing on Map 87-19 is a 1.5 m thick layer located in the Summers Creek Sandstone on the right bank of the Similkameen River, about 0.8 km downstream from the Bromley slide. In the Power Plant Shale, a 3 m to 4.3 m thick bentonite seam outcrops about 1.2 km southeast of the Bromley slide.

3.3 Piezometric Surfaces

Piezometric surfaces at the Clarke and Bromley slides were estimated based on available information including standpipe piezometer data, water levels in open drill holes and mapped seeps. Two inferred piezometric surfaces are plotted in section on Drawing D-1003. The upper piezometric surface probably results from perching of groundwater in the low permeability clay and sand and laminated silty clay and clayey silt. The clay and silt has a relatively low estimated coefficient of permeability of 1×10^{-6} cm/s based on a falling head slug test in DH99-03 - standpipe P2. It is also possible that shear surfaces in these materials contribute to the perching effect. The piezometric surface shown on the drawing is estimated, from left to right, by a seep at about El. 700 m in the Bromley slide and piezometric levels in TH97-01 - P2, DH99-03 - P2 and DH99-04 - P1.

In the Clarke slide area, evidence for a perched water table is corroborated by groundwater found in sheared, capped, inclinometer casings. Table 3-2 summarizes water levels recorded in the casings. Increasing water elevations may have resulted from periodic rainfall and above freezing temperatures which occurred during January, 1999. Both inclinometers responded to slug tests.

Table 3-2 Groundwater Levels in Inclinometer Casings

Date	Inclinometer	Depth to Water (m)	Water Elevation (m)
Jan. 7, 1999	TH96-01	3.68	706.06
Jan. 14, 1999		3.62	706.12
Jan. 21, 1999		3.45	706.29
Jan. 7, 1999	TH96-02	5.23	715.40
Jan. 14, 1999		5.215	715.42
Jan. 21, 1999		5.16	715.58

Note: Drilling commenced in the Clarke slide area on January 18, 1999.

Dry conditions at TH97-02 - P2 may result from a coarser, better drained, surface layer at that drill hole location, which is not within the Clarke slide mass. In addition, the standpipe may not penetrate deep enough to intersect the perched water table at that location.

Deep piezometers TH97-01 - P1, TH97-02 - P1 and DH99-03 - P2 are set in sand or sandy gravel and are either dry or essentially dry. In the sandy gravel, a coefficient of permeability of 1×10^{-4} cm/s was estimated from a constant head slug test in DH99-03 - P2. Therefore, the sand and sandy gravel layers are probably unsaturated.

The lower piezometric surface is estimated based on the Similkameen River level, an open hole water level in DH99-01, and the water level in the Tory's well. These data are indicative of a regional water table rising from the Similkameen River.

3.4 Precipitation Records

High precipitation is often cited as a cause of landslides. To address this issue, we have collected and plotted rainfall and precipitation records from the Princeton airport weather station. Three plots, monthly rainfall from 1990 to 1997, monthly precipitation from 1990 to 1997, and total annual rainfall/precipitation from 1961 to 1996, are presented in Appendix IV.

3.5 Stability Analysis

3.5.1 Methodology

Stability analyses were used to derive a limit equilibrium factor of safety (FOS) for failure surfaces at the slides. The analyses were carried out using the computer program SLOPE/W, Version 4.2, by Geo-Slope International Ltd., Calgary, Alberta. The analysis method was Morgenstern-Price which satisfies both force and moment equilibrium, with a constant interslice force function. The program is capable of circular and block searches to determine the failure surface with the lowest factor of safety. Specific failure surfaces can also be entered for analysis.

3.5.2 Design Stability Criteria

In most transportation situations, highway slope designs generally require a nonseismic FOS in the range of 1.25 to 1.50 (Transportation Research Board, 1996). Lower factors of safety may be used if the engineer is confident of the accuracy of the input data and if good construction control may be relied upon. KC Geotechnical Consultants Ltd. considers an FOS of 1.25 to be the recommended minimum after remedial works are complete.

3.5.3 Material Properties

The material properties used in the stability analyses were derived from three sources: laboratory tests, published data and a back analysis of a possible deep-seated slide. The material properties used, and their sources, are summarized in Table 3-3.

Table 3-3 Material Properties

Unit No.	Material	Unit Weight	Effective Shear Strength			Source of Shear Strength Parameters
		δbulk (kN/m³)	Cohesion c' (kPa)	Peak φ' (degrees)	Residual φ' (degrees)	
1	Gravel, Sand and Gravel, Gravelly Sand	20	0	37	-	Published data
2	Clay and Sand	22	0	31.5	26.5	Lab tests
3	Interlaminated Clay and Sand	19.5	0	25.5	13.5	Lab tests
4	Sand	17	0	33	-	Published data
5	Sandy Gravel	22	0	38	-	Published data
6	Disturbed Coal and Shale	19	0	18	18	Lab tests
7	Silty Sand	22	0	38	-	Published data
8	Power Plant Shale	19	0	24	-	Lab tests
9	Summers Creek Sandstone	23	0	33	-	Published data
8 & 9	Clay Seams	19/23	0	24	18	Lab tests
8 & 9	Clay Seams	19/23	0	-	11	Back analysis

<u>Laboratory Shear Tests</u>

Laboratory shear tests were carried out on soil samples from Unit 2 - Clay and Sand and Unit 3 - Interlaminated Clay and Silt. These tests are summarized in Table 3-4 and are presented in Appendix II.

Table 3-4 Summary of Laboratory Shear Tests

Unit	Slide	Drill Hole	Depth (m)	Type of Test	Results
Unit 2 - Clay and Sand	Clarke	DH99-03	9.39-9.81	CU Triaxial	Residual φ'
Unit 2 - Clay and Sand	Clarke	DH99-03	10.39-11.37	CU Triaxial	Peak φ'
Unit 3 - Interlaminated Clay and Silt	Bromley	DH99-02	21.4	Direct Shear	Peak and Residual φ'
Unit 8 - Coal	Bromley	DH99-01	22.95-23.12	Direct Shear	Peak and Residual φ'
Unit 8 - Clay Seam	Bromley	DH99-01	25.65-26.16 (25.84-25.88)	Direct Shear	Peak and Residual φ'

Previous investigations by MoTH indicate that these units are very stiff to hard, with high blow counts typically ranging from N = 25 to 50. These consistencies were confirmed by high torvane test values in PQ core retrieved from DH99-02 and DH99-03 (see Appendix I - drill hole logs). Nevertheless, both units contain prexisting planes of weakness. For Unit 2 - Clay and Sand the planes are randomly oriented joint surfaces within the massive glacial till. During the triaxial test at DH99-03, 9.39-9.81 m, failure occurred along a preexisting joint, and the result was interpreted as the residual friction angle value. The peak value was obtained during a second triaxial test which did not encounter a preexisting joint plane. For Unit 3 - Interlaminated Clay and Silt, polished subhorizontal partings were found between the laminae. The direct shear test was carried out along a parting, and peak and residual friction angles were obtained.

For the Bromley slide, failure in weak bedrock or clay seams is also possible. The deep-seated PS2 slide, about 1 km north, is seated in Summers Creek Sandstone with failure possibly occurring along clay seams (Appendix VI). To test bedrock strengths, direct shear tests were carried out on HQ core samples of coal and a 40 mm thick clay seam (Table 3-4). For the coal, a preexisting planar, rough joint surface oriented at 71 degrees from the core axis was tested. For the clay seam, the shear direction was parallel to the contacts with the surrounding coal, which were oriented at 72 degrees to the core axis.

Published Data

Published data were used to estimate the shear strengths of coarse granular soils, which generally have a higher friction angle and lower friction angle variability than fine grained soils. Terzhagi et al. (1996, p. 60 and 151), Department of the Navy (1982, p. 7.1-149, Fig. 7), Craig (1983, p. 122) and Hunt (1984, p. 198) contain correlations between granular soil density and friction angle which were used to estimate the friction angles. The choice of friction angle was also partly based on engineering judgment with the knowledge that there are significant consequences associated with ongoing movement or deformation of a major highway.

The friction angles were estimated for Unit 1 - Gravel, Sand and Gravel, Gravelly Sand, Unit 4 - Sand, Unit 5 - Gravelly Sand, and Unit 7 - Silty Sand. Previous investigations by MoTH indicate that Units 2, 4 and 5 are very dense, with high blow counts and refusal typical. However, blow count data in gravelly soils should not be considered reliable, therefore, the friction angles selected are based on relative densities of 50% to 75% in those soil textures. The friction angle for Unit 4 - Sand is relatively low because it is poorly graded. The friction angle for Unit 7 - Silty Sand is relatively high because it is gravelly and very dense. PQ core retrieved in DH99-01 indicates that the material is a glacial till.

The friction angles for the Power Plant Shale and the Summers Creek Sandstone are based partly on lab testing and partly on published data in Goodman (1980). The lab tested peak friction angle in coal is consistent with published values for shale, therefore, this value is used for the Power Plant Shale. The friction angle for the Summers Creek Sandstone is based on published values. For both formations, it is probable that weaker clay seams ultimately control slope stability.

Back Analysis

In general, soil and rock shear strength parameters obtained from lab tests and published data produced realistic results when entered in the slope stability analysis program. However, analysis of a deep-seated failure at the Bromley slide produced FOS values significantly greater than one using lab test parameters. Field evidence such as tension cracks and curved trees indicates that deep-seated movement is occurring. Therefore, it was decided to back analyze a deep-seated failure surface to further assess the shear strength of the clay seams in rock. Results of the back analysis will be discussed in Section 6.

4. CLARKE SLIDE

4.1 Historical Record

The history of Highway 3 and the Clarke slide was reconstructed with the aid of MoTH personnel in the South Okanagan Highways District, and the air photo record. This history is intended to provide a historical backdrop to the existing site conditions, but may not be fully comprehensive as it is based on limited data. In particular, verbal communications should be independently verified where possible. The following highway record is based on personal communication with Mr. Eric Madsen, Paving Technician and Mr. Rusty Hewitt, District Technician:

- 1967-1968: Highway 3 was paved, including a third lane, at the Clarke slide location.
- 1986-1987: New paving fill placed (50 mm paving removed, replaced with 100 mm).
- Spring 1996: Third lane failed due to movement on the Clarke slide.
- Throughout 1996: Extensive maintenance including fill and patching required to maintain the third lane.
- Spring 1997: Third lane failed again due to Clarke slide movement; decision made to leave lane closed due to hazard posed by landslide.
- 1998 to present: Some slumping of fill placed at highway shoulder, third lane remained closed.

A local heavy equipment operator worked at the Clarke property. The following are his reports on the slide activity (R. Ferguson, personal communication, 1998):

- Prior to 1996, movement near the house and garage occurred which didn't affect Highway 3. Pit run gravel was placed locally as fill to repair the damage; and

- In the fall of 1996, the house was removed from the property just before the lower slope failed.

The air photo record also provides a historical perspective of slide movement. The following interpretations are relevant at the Clarke slide:

- 1947: Highway 3 is gravel based, and some fill was placed to construct the road where it passes the Clarke slide area, however, slope movement is not evident on the photos. An access road had been constructed to Tulameen valley bottom which later served as the driveway to the Clarke property;
- 1959: Headscarp and slide block may be present, however, scale is too small for positive identification. The highway is clearly narrower at the Clarke slide location, possibly due to removal by slide movement;
- 1967: Major headscarp and slide block are evident. The slide removed a portion of the highway to make it narrower. Highway is still gravel based;
- 1985: Fill had been placed at slide headscarp to construct the paved highway. There is no clear evidence of slope movement; and
- Mid 1996: New fill and a patched third lane are evident in response to failure of the third lane in the spring. Buildings including a house and garage are evident on the Clarke property which were not visible on the 1985 photos.

The historical record indicates that significant movement on the Clarke slide has occurred at least three times, before 1967, in the spring and fall of 1996, and in the spring of 1997. Topographic surveys from September, 1996 and September, 1998 show evidence of this movement including the slumped block below the headscarp and numerous scarps and tension cracks (Drawings D-1001 and D-1004). Smaller magnitude movements may have occurred before 1967.

4.2 Geotechnical Considerations

The soil stratigraphy of the Clarke slide is interpreted as Unit 2 - Clay and Sand overlying Unit 3 - Interlaminated Clay and Silt (Drawing D-1003). This subsurface assessment is based on data from two test holes by MoTH, and two test pits and two drill holes by KC Geotechnical (Drawing D-1002). Unit 2 varies from about 5 m to 25 m thick and forms the foundation for Highway 3. Unit 3 is about 4 m thick and essentially flat lying in the Clarke slide area. Sand and sandy gravel, Units 4 and 5, are found at depth.

The main concern and geotechnical consideration at the site is the obvious indicators of ongoing slope movement and failure. Ground evidence of failure includes two headscarps, one at the highway and one at the Clarke's garage, which define upper and lower slide masses (Drawing D-1001). The upper slide mass contains numerous tension cracks, horst and graben topography, and jack-strawed and back tilted trees which are indicative of relatively slow moving slope failure (Appendix V, Photos 2 to 4). The lower slide mass contains completely disturbed soil with numerous small scarps which appear to have failed more rapidly (Appendix V, Photos 9 and 10).

MoTH has provided deflection and time-displacement plots from inclinometers which are critical to the understanding of the slide movement (Appendix III). Deflection plots indicate that the depth of shear is at about 11 m below the ground surface in the middle of the slide (TH96-02), and only 6.5 m below ground at the toe (TH96-01) (Drawing D-1003). Essentially no movement is recorded below those depths in the inclinometer data. These data corroborate the laboratory test data which indicate that Unit 3 - Interlaminated Clay and Silt is a weak layer in the stratigraphy upon which the sliding is occurring. Time-displacement plots show that about 10 mm movement occurred between installation in September 1996 and January 1997, after which an additional 35 mm movement was recorded in two months to mid March 1997. The inclinometers then sheared past the point where they could be monitored. Data from a replacement inclinometer, DH99-05, are also presented in Appendix III. The deflection plot shows shearing at about 5.8 m depth, and the time displacement plot shows that about 15 mm of movement occurred between March 12 and March 30, 1999.

4.3 Stability Analysis

Results of the stability analysis are presented in Drawings D-1004 and B-1005. Drawing D-1004 is a plan and stability section showing the site conditions in September, 1996, before the spring 1997 slide movements. Drawing B-1005 presents a stability analysis section based on September, 1998 topography. The failed ground condition indicates that Unit 2 - Clay and Sand and Unit 3 - Interlaminated Clay and Silt are at their residual friction angles. Soil pore water pressures used in the stability analysis result from the perched piezometric surface discussed in Section 3.3.

Two probable sliding mechanisms were detected during assessment of potential failure surfaces with the lowest factors of safety. The first probable mechanism is translational sliding along Unit 3 - Interlaminated Clay and Silt, represented by failure surfaces B-B through E-E in Drawings D-1004 and B-1005. The second probable mechanism is a shallow seated earth flow at the toe of the translational slide, represented by failure surface A-A. These two mechanisms are considered reasonable representations of the failure surfaces that have produced the Upper Slide Mass and Lower Slide Mass, respectively, shown in Drawing D-1001.

According to the Transportation Research Board (1996), translational slide masses displace along a planar or undulating surface of rupture and slide out over the original ground surface which correspond with the Upper Slide Mass. As sliding continues the mass may break up, particularly if its velocity or water content increases. The disrupted mass may then flow, becoming an earth flow rather than a slide which is consistent with our interpretation of the Lower Slide Mass. This description fits the Clarke slide, although it is not clear that the translational slide triggered the earth flow. An important aspect of translational slides is that movement will continue indefinitely if the surface of separation is sufficiently inclined.

The stability analyses show that failure surfaces B-B through D-D have FOS values near or below one, which is consistent with field evidence that the slope is failing. Scarps and tension cracks mapped in MoTH's plan, and included on Drawing D-1004, appear to confirm failure surface C-C as having the lowest factor of safety.

Failure surface A-A is shallow seated because it is seated mainly in granular materials, and it is unstable at FOS = 0.99. Oversteep fill slopes northeast of the stability sections in the carport and house area have also proven to be unstable and have failed as a shallow earth flow.

Potential failure surface E-E, shown on Drawing B-1005, indicates that Highway 3, without its third lane, has a FOS of 1.19, which, although currently stable, is below our recommended minimum FOS of 1.25. Failure of fill placed at the highway edge, Drawing D-1001, may have been caused by removal of toe support when downslope movement of the slide mass occurred along failure surface D-D (Appendix V, Photo 1).

4.4 Probable Cause of Failure and Contributing Factors

4.4.1 General

Causes of failure and contributing factors are listed separately for the upper translational slide and the lower shallow seated slide or flow.

4.4.2 Translational Slide

The glacial soil stratigraphy which includes Unit 2 - Clay and Sand and Unit 3 - Interlaminated Clay and Silt is a contributing factor which set the stage for the failure. These layers have relatively low shear strengths and are vulnerable to sliding. The weak units have been exposed, or 'daylighted', as a result of post-glacial erosion by the Tulameen River.

There is little evidence to evaluate factors which caused or contributed to the slide movement which is visible on the 1967 air photos. The effect of precipitation is unclear as there are no anomalous years in the annual precipitation records from 1961 to 1967 (Appendix IV, Figure IV-1). Man-made factors which may have contributed are the construction of the access road at the toe of the slide and placement of fill at the potential head of the slide. The access road would have destabilized the slope by removing soil from the toe, and the fill would have increased the driving forces on a wedge of soil near the headscarp.

The causes and contributing factors for the spring 1996 and spring 1997 movements are easier to analyze because the record of events is more recent, and there is excellent topographic control. The timing of both events in the spring indicates that increased pore water pressures due to snowmelt, possibly combined with rainfall, are the most probable causes of both failures. The rainfall/precipitation records indicate that extraordinary snowfalls occurred in November 1995 and November/December 1996 (Appendix IV, Figure IV-2). The resulting snowmelts in the following springs probably caused higher than average increases in pore water pressure which triggered the slides.

The following three factors may have contributed to the failures:

- Elevated pore water pressures due to surface runoff from Highway 3;
- Cuts for borrow material at the toe of the slide; and
- Loading of the head of the slide by highway fill.

These factors are described and evaluated in Drawing D-1004. The plan shows the location where at least some highway runoff probably entered the headscarp, just beyond the end of the asphalt drainage curb at the south edge of the headscarp. Highway runoff to the scarp was also observed during the 1999 investigations, and probably continues to elevate the piezometric surface in the Clarke slide (Appendix V, Photos 11 and 12).

The effect of the other two factors has been evaluated quantitatively using the equilibrium stability program SLOPE/W on Sections A-A to E-E, Drawing D-1004. FOS values were first assessed using topography from the September, 1996 survey, which includes a 5 m high cut at the garage location. The stability was then reevaluated with the slope reconstructed to its original condition before the cut. The effect of the cut was to decrease the FOS by values ranging from about 0.22 (1.21 to 0.99) at failure surface B-B, to 0.05 (1.16 to 1.11) at failure surface E-E. These data show that the garage cut was significantly detrimental to stability of the translational slide mass.

In contrast, placement of highway fill to construct the third lane had little influence on the stability of the slope. Failure surfaces B-B to D-D were not affected, and failure surface E-E experienced only a slight reduction in FOS from 1.13 to 1.11. These data again indicate that the slide headscarp is located downslope from the highway, and that the fill is sloughing into a graben left behind after movement occurs.

Finally, the fills placed throughout the area, especially at the house, carport and garage, may have temporarily buttressed the toe of the translational slide. The shallow seated lower earth flow in the fall of 1996 removed this toe support and may have contributed to the spring 1997 failure. The extent of the failure is illustrated by the contrast in ground conditions at the garage, carport and house area between September 1996, Drawing D-1004, and during the surficial mapping in 1998, Drawing D-1001.

4.4.3 Shallow Seated Earth Flow

The direct cause of the shallow seated earth flow was probably pore water pressure increases during rain events in the fall of 1996. Rainfall data indicate that September and

October, 1996, were wetter than average months. The main contributing factor is the placement of oversteep fill in the area where Unit 3 - Interlaminated Clay and Silt, daylights on the slope. Fill slopes up to 3 m high at angles of 0.6H:1V (60°) were constructed. Movement and bulging of the toe of the translational slide in the spring of 1996 may have also contributed by disturbing the fill. If disturbed, the fill would have lower frictional resistance and increased pore water pressures due to water accumulation in tension cracks.

4.5 Design Remedial Solutions

4.5.1 General

Remedial solutions were considered to stabilize the Clarke slide and allow for reconstruction of the third lane at Highway 3. Conceptual designs for two remedial alternatives, concrete piles and a replacement toe berm, are presented. Gravity drain holes were also considered as a remedial measure, however, their effectiveness would be limited by the low permeability of the units to be drained. Lowering the highway grade to reduce driving forces was considered, but was not pursued because the stability was only slightly improved and the problem of retrogressive translational sliding would not be addressed. The objective of the remedial measures is to increase the static FOS to the minimum of 1.25 indicated by the Transportation Research Board (1996). Smaller scale mitigative measures to improve stability are also discussed. Estimated costs for the two main remedial alternatives are presented in Section 6.

4.5.2 Concrete Piles

Large diameter drilled shafts, or piles, can stabilize landslides. The piles are drilled through the failure surface to form a wall which arrests the sliding movement. A conceptual design for the piles, Drawing D-1006, was developed based on recent experience in similar ground near Fort St. John, BC (Heavy Construction News, 1999). The conceptual pile design consists of a steel pipe, 9.5 mm thick, 1,500 mm diameter, and 22 m long, filled with concrete. The concrete will be exposure Class C-1 concrete with a minimum 28 day compressive strength of 35 MPa (Canadian Portland Cement Association, 1991), and will be reinforced with a cage of forty-five 25 mm diameter steel re-bar for its entire length. To install each pile an oversize hole is augered to the target depth, and the casing is installed and cleaned out. The re-bar is then lowered down the casing and the concrete tremied down the pipe. Two rows of 50 piles will be installed at 2 m spacing, for a total of 100 piles and a total wall length of 100 m.

The shear strength of the pile wall is estimated based on the strength of the concrete. The shear strength is estimated as 706 kPa, or 1.25 MN force per pile (Canadian Portland Cement Association, 1985). SLOPE/W analysis shows that if shear through one pile is assumed, the minimum FOS is 1.31 on potential failure surface A-A, Drawing D-1006. A similar result is obtained by comparing the total driving force to the total resisting force provided by the soil and concrete piles. The shear strengths of the steel casing and reinforcement are ignored in this analysis. The conceptual design meets the minimum design criterion of FOS greater than 1.25 against shear. The design does not quantitatively address bending moments which will be addressed in later design stages. However, we have included typical steel reinforcement to provide bending resistance based on our previous construction experience with reinforced piles (Klohn Leonoff, 1995). This alternative also includes reconstruction of the third lane and resloping to lower the driving forces and load the toe, as shown on Drawing D-1006.

4.5.3 Replacement Toe Berm

The second remedial alternative is a replacement toe berm (Drawing D-1007). The first stage of this concept is to excavate clay and silt down to at least El. 702.5 m at the toe of the translational slide mass, and replace it with free draining granular fill. The replacement fill will provide greater frictional resistance to sliding, and will serve to drain the toe area. The toe must be excavated in slots to reduce the likelihood of slide movement during the work (Drawing D-1007). Each slot should be a maximum of 10 m

5. BROMLEY SLIDE

5.1 General

Evidence gathered during the course of our geotechnical investigations in 1998 - 1999 indicate that the Bromley slide contains at least three slide masses which contain distinct failure mechanisms. The upper slope above about El. 700 m is similar to the Clarke slide with apparent translational sliding on Unit 3 - Clay and Silt, with a perched water table. The lower slope, below about El. 680 m contains evidence of a deep-seated, slow moving slide. This evidence includes tilted and curved trees and a large tension crack. Finally, an active landslide has been identified adjacent to Stevenson Road, just below the Tory residence. This section addresses the three slide masses separately as the Upper Bromley, Lower Bromley and Stevenson Road slides.

5.2 Historical Record

A history of events at the Bromley slide has been developed based on discussions with local residents and the City of Princeton, a review of consultants' reports, and a review of the air photo record. This history is intended to provide a historical backdrop to the existing site conditions, but may not be fully comprehensive as it is based on limited data. In particular, verbal communications should be independently verified where possible.

Two properties are within or immediately adjacent to the slide area (Drawing D-1001):

- DL 277, which has been partially developed as the Westridge subdivision and the Bromley Pub; and
- The Tory property at the base of the slope, which is accessed via Stevenson Road.

In 1992, the Tory's reported ponded water in their back yard which they had not previously observed since moving to the residence in 1979. The City of Princeton hired Pacific Hydrology Consultants Ltd. of Vancouver, BC to review the site. Pacific

Hydrology's report to the City, stated that, "There are convincing signs of downslope movement of blocks of ground starting near the northeast corner of the Tory property. These consist of narrow linear depressions along the slope." (Pacific Hydrology, 1992). At this time, developments including the Bromley Pub and the Westridge subdivision were proposed at the crest of the slope adjacent to Highway 3. Pacific Hydrology recommended that a stability assessment be carried out prior to these developments. In the fall of 1992, a slope stability assessment report was produced for Chara Holdings Ltd., owners of DL 277, by Wilson Associates Geotechnical Inc., Vernon, BC. That report concluded that DL 277 was stable provided 6 m setbacks were observed at the top and toe of slopes greater than 40%, runoff was controlled, and sewage effluent disposed of in a safe area at an adjoining site (Wilson Associates, 1992).

Development of the Bromley Pub and Westridge subdivision commenced in 1993. The history of this development was assessed based on discussions with employees of the City of Princeton and water works plans provided by the City (Corny Froese, personal communication). According to City plans, a 250 mm diameter PVC water main has been constructed to connect the Westridge subdivision and Bromley Pub to the existing Wilson subdivision in the east. The water line runs parallel to Highway 3 at the base of the fill on the south side of the highway. Between the Bromley pub and the Westridge subdivision, a gas line runs parallel to the water line. In 1995 or 1996, an above-ground storage reservoir was completed to service the Westridge subdivision, and the water main was connected to the reservoir.

The Bromley Pub was completed in 1993 or 1994, after the construction of the water main and before completion of the storage tank. To provide a temporary water supply to the pub, the City used a 1 hp pump which operated continuously to fill the water main. When the water supply was not being used, it was bled off at a hydrant located in front of the pub (Drawing D-1001). The City of Princeton estimates the bleed off rate was about 5 imperial gallons per minute (K. Gibson, personal communication, 1999). The water

((Na+K)/(Ca+Mg+Na+K)=36%). Table 5-1 indicates that the friction angle would be expected to decrease with increased sodium montmorillonite content.

5.4.2 Stability Analysis

A SLOPE/W stability analysis was undertaken based on the September 1998 topography, Drawing B-1011. The depth to bedrock is based on drill hole data at DH99-01 and the Tory's well (Drawing D-1003). In the Highway 3 area, the depth to bedrock is not known, but it is lower than the bottom of DH99-02 at El. 671.8 m. A continuous clay seam is assumed to dip down slope subparallel to the bedding. Near the toe of the slope, the failure surface is assumed to pass through the cave-in location in the Tory's well and the clay seam cored in DH99-01, which contained slickenslides. The toe of the slide 'daylights' in the Similkameen River. At the time of this report limited inclinometer data were available from DH99-01, which monitors both the Lower Bromley and Stevenson Road slides. A time-displacement plot for 25.0 to 26.5 depth indicates that 1 mm of movement occurred along the clay seam between March 2 and March 24, 1999 (Appendix III). However, these data are not conclusive and additional monitoring will be required to confirm the presence of a deep-seated slide. Soil pore water pressures in the stability analysis result from the lower, regional piezometric surface discussed in Section 3.3.

The shear strength of the clay seam is critical to the stability of the slope. Analyses based on laboratory test results, which gave a friction angle of 18° in the clay seam, result in a minimum FOS of about 1.39 (potential failure surfaces L-L, N-N and P-P, Drawing B-1011). However, field evidence indicates that failure has occurred. To reassess the shear strength of the clay seam, a back analysis was carried out assuming a FOS of about 1.00. The back-analysed friction angle is 11 degrees, which is similar to the estimated friction angle for a calcium montmorillonite composition (12 degrees).

flow pattern. Water use at the Tory residence may locally influence the piezometric surface at the toe of the slide, however, the net effect is probably neutral because their water supply is derived from groundwater pumping. Loading of the head of the slide would again increase the driving forces, although the net destabilizing effect would be less because the slide mass is much larger.

5.4.4 Design Remedial Solution - Toe Berm

A remedial solution for the Lower Bromley slide should not be undertaken without additional monitoring and a clearer understanding of the failure mechanism. However, if a deep-seated slide is clearly identified, one possible remedial measure would be to construct a large toe berm, as shown in Drawing D-1012. A similar berm was constructed to stabilize the PS2 slide near the Mohawk Station (Appendix VI). A stability analysis of this solution is not appropriate based on the limited data which are presently available. KC Geotechnical's opinion of the cost of the berm shown is included in Section 6.

5.5 Stevenson Road Slide

5.5.1 Geotechnical Considerations

The Stevenson Road slide contains unique stratigraphy of Unit 5 - Sandy Gravel, overlying Unit 6 - Disturbed Coal and Shale, which in turn overlies Unit 7 - Silty Sand (Drawing D-1003). The disturbed coal and shale are a block of bedrock which was deposited over the silty sand by colluvial or glacial processes. Bedding planes within this block were seen to be essentially subhorizontal. This subsurface assessment is based on data from one drill hole and field observations by KC Geotechnical, and the drill log from the Tory's well (Drawings D-1001 and D-1002). The disturbed coal and shale are about 7 m thick in DH99-01, but apparently were not encountered at the Tory's well.

During the surficial mapping, active slide debris with numerous scarps was identified on the slope between Stevenson Road and the Similkameen River, and tension cracks and a small scarp were mapped above the road (Drawing D-1001). Surface exposures of Unit 6 were highly weathered (Appendix V, Photo 22). At the Similkameen River, the slide mass is clearly protruding out from the shoreline for a distance of about 60 m, and fluvial erosion is occurring at the bank.

MoTH has provided deflection and time-displacement plots from an inclinometer installed in DH99-01 (Appendix III). Deflection plots indicate shear is occurring at about 13 m depth below the ground surface (Drawing D-1003). The material at this depth is within Unit 6 and is described as, interbedded shale and sandstone with coal laminae. The time-displacement plot shows that about 36 mm movement occurred between March 2, 1999 and April 7, 1999, and the movement has occurred at a constant rate of about 1 mm per day.

5.5.2 Stability Analysis

Results of the stability analysis are presented in Drawing B-1011. The surficial mapping and inclinometer data indicate that Unit 6 - Disturbed Coal and Shale is at its residual friction angle of 18 degrees. Soil pore water pressures in the stability analysis result from the lower, regional piezometric surface discussed in Section 3.3.

The failure surfaces shown on the drawing distinguish the Stevenson Road slide from the larger Lower Bromley slide. Failure surface R-R, with an FOS of 0.80, represents relatively shallow failure with scarps located below the road. Failure surface S-S, with an FOS of 0.98, represents a deeper seated, translational slide which passes through the known shear depth of 13.5 m, from the inclinometer data. The headscarp of the slide is about 4 m above the road, where tension cracks were mapped. Both failure surfaces 'daylight' just above river level. The length of the slide is probably controlled by the size of the block of disturbed coal and shale, which appears to be about 60 m based on the length of slope which protrudes from the shoreline. Based on evidence of movement and from the geotechnical investigations described in this report, these failure surfaces are considered reasonable representations of the failure mechanism controlling movement in the Stevenson Road area.

5.5.3 Probable Causes of Failure and Contributing Factors

The inclinometer time-displacement plots indicate that movement accelerated in early March of this year, which probably coincided with a period of snowmelt, possibly combined with rainfall. Therefore, the cause of the sliding is probably pore water pressure increases associated with snowmelt and rainfall. The weak nature of the disturbed coal and shale, combined with its location near the river bank, established the conditions which made sliding likely. One contributing factor is continuous fluvial erosion by the Similkameen River, which removes toe support from the slide mass. A second factor which contributes to continued movement is the lack of vegetation on the failed slope. Vegetation would improve stability by intercepting and transpiring a portion of the precipitation which falls on the slope. Finally, it is possible that runoff from the road is being diverted to the slide mass below the running surface, and pore pressures are being elevated as a result.

5.5.4 Design Remedial Solution - Drain Holes and Anchors

The design remedial solution was undertaken assuming that relocation of the Stevenson Road is not a viable alternative. If MoTH acquires the private property west of the road, the simplest and most cost effective solution would be to relocate the road away from the slide area.

A remedial solution of horizontal drain holes combined with tensioned threadbar anchors is proposed to stabilize the Stevenson Road slide (Drawing D-1012). Horizontal drain holes are considered because Unit 6 - Disturbed Coal and Shale has relatively high permeability, which was indicated by the fractured condition of surface exposures and fluid loss in this unit during drilling at DH99-01. The estimated drained piezometric surface is shown on the conceptual design drawing. The drains will consist of 50 mm slotted PVC pipe, and they will discharge into the Similkameen River.

The anchors will comprise 25 mm diameter threadbars with an estimated yield strength of 211 kN, which will be tensioned to 67% of yield, or 141 kN. The bars will be installed at least 4 m into bedrock, including a 3 m bond length, and grouted to the surface.

The combined remedial solution of drain holes and anchors increases the minimum FOS to 1.44 on failure surface B-B in the stability section, Drawing D-1012. The FOS for the slide mass as a whole may be slightly lower, but it will still be sufficient to meet the Transportation Research Board FOS criterion of 1.25. Estimated costs for the proposed drain holes and anchors are presented in Section 6.

5.5.5 Other Mitigative Measures

A mitigative measure which would improve stability is seeding or planting the stabilized slide mass. Once reestablished, vegetation would intercept precipitation and mitigate the risk of sliding due to pore water pressure increases. A second measure would be to inslope the road to reduce the likelihood of diversion of runoff to the slide mass.

6. OPINION OF PROBABLE COST FOR PROPOSED REMEDIAL MEASURES

6.1 Level of Accuracy and Reliability

KC Geotechnical has prepared an opinion of probable cost for proposed remedial measures in the Clarke and Bromley slide areas. This opinion is to be considered an order-of-magnitude estimate suitable for preliminary budgetary purposes. This opinion must not be relied upon as the actual cost, as it is based on conceptual designs and many factors that can change with time. The estimates are based on KC Geotechnical's professional experience with similar remedial measures, and the site information available to date. KC Geotechnical does not guarantee or warranty this opinion of probable cost.

New cost estimates will be required following the preparation of detailed design drawings. To improve the accuracy of the cost estimates in the future, issues including availability of contractors and equipment, staging of work, finalization of design measures and quantities, future labour and equipment costs, and future material costs will have to be taken into consideration.

6.2 Assumptions

The following assumptions were used in preparation of our opinion of probable cost:

- The conceptual designs as presented in this report form the basis of the cost estimates. Refinements to the designs, such as modifying the steel reinforcement to the concrete piles or changing the length and number of piles or drains may significantly change the cost basis.
- We have assumed that the work would be conducted by local, BC based contractors, using typical 1998 rates for equipment, labour, and materials.
- Nominal estimates for mobilization and demobilization have been included in the estimates, but no costs have been included for room and board, and vehicle expenses for the contractor.

- Engineering inspection and monitoring costs are not included in these estimates.
- Conceptual design is based on site information available to date.
- Material price estimates are partially based on non-binding quotes from suppliers of anchors, concrete and steel, located in the Lower Mainland of BC.
- Drilling and excavation equipment estimates are partially based on non-binding quotes from drilling companies and construction companies located in the Lower Mainland of BC.
- We have assumed that all drilled holes must be cased at all times.
- No costs for royalties for sand and gravel borrow material have been included.
- No costs for disposal of spoil material have been included.
- The haul distance to the nearest gravel pit has been estimated as 1 km to 2.5 km, dependent on the specific work area.
- No costs for acquisition of private property or removal of existing buildings have been included.
- Applicable taxes have not been included in these estimates.
- A 25% contingency has been included in the estimates.

6.3 Estimated Costs

A summary of the order-of-magnitude cost estimates for all proposed remedial measures is presented in Table 6-1, with costs rounded to the nearest $5,000. The total estimated cost for all proposed works will depend on which conceptual alternatives are selected. Additional breakdown of the cost estimates is presented for each alternative in Tables 6-2 to 6-7.

Table 6-1 Summary Opinions of Probable Costs

Remedial Alternative	Opinion of Probable Cost (to nearest $5,000)	Contingency (25% to nearest $5,000)	Total Estimated Cost
Clarke Slide - Alternative 1 - Concrete Piles	$2,445,000	$610,000	$3,055,000
Clarke Slide - Alternative 2 - Replacement Toe Berm	$380,000	$95,000	$475,000
Upper Bromley Slide - Alternative 1 - Concrete Piles	$2,710,000	$680,000	$3,390,000
Upper Bromley Slide - Alternative 2 - Drain Holes	$270,000	$70,000	$340,000
Lower Bromley Slide - Toe Berm	$825,000	$205,000	$1,030,000
Stevenson Road Slide - Drain Holes and Anchors	$215,000	$55,000	$270,000

Table 6-2 Clarke Slide - Alternative 1 - Concrete Piles

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$25,000	$25,000
2	Excavate Slopes	m^3	3,375	$6.00	$20,250
3	Place Granular Fill at toe of slope	m^3	9,900	$7.00	$69,300
4	Supply 100 steel pipe piles, each 1500 mm dia., 0.375"thick, 22 m long, c/w 25 mm bar reinforcement.	m	2,200	$582.00	$1,280,400
5	Install Steel Pipe Piles	m	2,200	$125.00	$275,000
6	Supply Concrete for Pile Backfill	m^3	3,875	$125.00	$484,375
7	Tremie Pour Concrete, including Pile Cleanout	m^3	3,875	$75.00	$290,625
8	Pile Cap over Caissons (not required)	LS	1	0.0	$0
				Subtotal	$2,444,950
	Contingency (25%)	LS	1	$611,237.50	$611,238
				TOTAL	$3,056,188

Table 6-3 Clarke Slide - Alternative 2 - Replacement Toe Berm

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$50,000	$50,000
2	Excavate replacement toe berm area	m^3	6,600	$10.00	$66,000
3	Place sand and gravel fill in toe berm area	m^3	6,600	$12.00	$79,200
4	Excavate slopes.	M^3	7,050	$6.00	$42,300
5	Place random fill at select locations	m^3	4,700	$7.00	$32,900
6	Place geogrid reinforced fill	m^3	9,500	$8.00	$76,000
7	Supply/install geogrid for fill	LS	1	$25,000	$25,000
8	Place geogrid/geotextile wrap	m^2	140	$10.00	$1,400
9	Supply/install geogrid/geotextile face wrap	m^2	300	$12.00	$3,600
10	Hydroseed face	m^2	140	$20.00	$2,800
				Subtotal	$379,200
	Contingency (25%)	LS	1	$94,800	$94,800
				TOTAL	$474,000

Table 6-4 Upper Bromley Slide - Alternative 1 - Concrete Piles

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$25,000	$25,000
2	Excavation	m^3	2,925	$6.00	$17,550
3	Supply 120 steel pipe piles, each 1500 mm dia., 0.375"thick, 21 m long, c/w 25 mm bar reinforcement.	m	2,520	$582.00	$1,466,640
4	Install Steel Pipe Piles	m	2,520	$125.00	$315,000
5	Supply Concrete for Pile Backfill	m^3	4,435	$125.00	$554,375
6	Tremie Pour Concrete, including Pile Cleanout	m^3	4,435	$75.00	$332,625
7	Pile Cap over Caissons (not required)	LS	1	0.0	$0
				Subtotal	$2,711,190
	Contingency (25%)	LS	1	$677,797.50	$677,798
				TOTAL	$3,388,988

Table 6-5 Upper Bromley Slide - Alternative 2 - Drain Holes

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$25,000	$25,000
2	Drill Drain Holes & Install PVC	m	1,215	$200	$243,000
				Subtotal	$268,000
	Contingency (25%)	LS	1	$67,000	$67,000
				TOTAL	$335,000

Note Unit cost for drilling may be reduced if casing of holes not required.

Table 6-6 Lower Bromley Slide - Toe Berm

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$25,000	$25,000
2	Supply and place granular fill	m^3	114,275	$7.00	$799,925
				Subtotal	$824,925
	Contingency (25%)	LS	1	$129,668.75	$206,231
				TOTAL	$1,031,156

Table 6-7 Stevenson Road Slide - Drain Holes and Anchors

Item No.	Description of Work	Unit	Estimated Quantity	Estimated Unit Cost ($CAN)	Total Estimated Cost (CAN$)
1	Develop access to work area. Clear and grub as required.	LS	1	$15,000	$15,000
2	Drill Drain Holes & Install PVC	m	210	$200	$42,000
2	Drill Holes, Supply and Install Anchors	ea.	40	$4,000	$160,000
				Subtotal	$217,000
	Contingency (25%)	LS	1	$54,250	$54,250
				TOTAL	$271,250

Note Unit cost for drilling may be reduced if casing of holes not required.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Clarke Slide

Conclusions

Geotechnical investigations at the Clarke slide have identified the soil conditions, the geometry of the two slide masses, and the causes of sliding. The soil conditions include a glaciolacustrine interlaminated clay and silt with a perched piezometric surface, which is a weak layer upon which translational sliding has occurred. Shallow seated failure is occurring at the toe of the translational slide mass. Increased pore pressures during spring snowmelt probably cause slide movements. Contributing factors include elevated pore water pressures due to surface runoff from Highway 3, cuts for borrow material at the toe of the slide, and loading of the head of the slide by highway fill. Oversteep fill placement was the main contributing factor at the shallow slide.

Two remedial solutions are proposed to stabilize the translational slide: a concrete pile wall or a replacement toe berm. Another proposed mitigative measure is to install a catch basin to intercept runoff from Highway 3.

Recommendations

Construction of the replacement toe berm is recommended based on its lower estimated cost, $475,000, compared to the concrete pile wall, $3,055,000. However, the replacement toe berm carries a significant construction risk which should be addressed by carefully controlling the work. One critical aspect is that the interlaminated clay and silt at the slide toe must be completely excavated and replaced without additional sliding occurring during excavation. Secondly, the fill and geogrid for the toe berm must be carefully placed and compacted. A minimum of four inclinometers should be installed, two in the replacement toe and two in the berm, to monitor slope movements during and after construction.

In the period before construction, monitoring should continue at the inclinometer installed in DH99-05, and at standpipe piezometers installed in DH99-03 and DH99-04. Bimonthly readings, with biweekly readings from mid February to mid April, are recommended. The catch basin should be installed to mitigate the detrimental effects of runoff from the highway.

7.2 Bromley Slide

7.2.1 Upper Bromley Slide

Conclusions

Geotechnical investigations indicate that conditions at the Bromley slide are similar to those at the Clarke slide. The soil conditions include the same glaciolacustrine interlaminated clay and silt layer with a perched piezometric surface, on which translational sliding is occurring. Increased pore pressures during spring snowmelt probably cause slide movements. Contributing factors include elevated pore water pressures due to recent ground clearing and development, and loading of the head of the slide by fill for development fill. A possible contributing factor is movement on a deep-seated slide in bedrock.

Two remedial solutions are proposed to stabilize the translational slide: a concrete pile wall and gravity drain holes. Both remedial measures would be required to meet the Transportation Research Board criterion for the minimum stability of slopes. Other proposed mitigative measures include restoration of vegetation in the slide scarp area, abandonment of the septic field at the Bromley Pub, removal of fill at the headscarp area, and periodic testing of the Westridge water main.

Recommendations

The proposed remedial measures of combined concrete piles and drain holes are recommended to meet the stability criterion. However, these measures are costly (combined estimated total = $3,730,000), and there is a risk that movement on a deep-seated slide could damage the piles and drains after construction. Therefore, the stabilizing measures should not be implemented until the possible deep-seated slide is further investigated and, if required, stabilization measures are complete.

In the interim, implementation of less costly mitigative measures is recommended. Revegetation may include seeding and planting of indigenous tree species in the headscarp and pub area. The removal of fill is a stabilizing measure which should be undertaken carefully to avoid damage to existing utilities and/or inadvertently triggering new movement. Further development in the cul-de-sac area should not be allowed. Transfer of Bromley Pub sewerage to the City sewage system is a measure which the City of Princeton should consider.

The inclinometer installed at DH99-02 should be carefully monitored. Bimonthly readings, with biweekly readings from mid-February to mid-April, are recommended. Significant movement at the depth of interlaminated clay and silt, about 20 m to 22 m, would indicate retrogressive failure of the translational slide, and an increased risk of failure at the highway. In this case, immediate implementation of remedial measures should be considered.

7.2.2 Lower Bromley Slide

Conclusions

The Lower Bromley slide is a possible deep-seated failure founded on bentonite seams in the Summers Creek Sandstone and Power Plant Shale. Surface evidence for failure includes disturbed vegetation, a major tension crack, and structural cracks at a residence. Subsurface evidence consists of two cave-ins at identical depths in the Tory's well, which correlate with extraordinary precipitation and indicate deep-seated movement.

Direct shear tests of a slickenslided bentonite seam produced a residual friction angle of 18 degrees. However, back analysis and a shear strength - minerology correlation indicate a friction angle of about 11 degrees. If the latter assessment is more accurate and a continuous bentonite seam exists, then a deep-seated slide with a headscarp coincident with Upper Bromley slide is likely.

Recommendations

Developing a more complete understanding of the Lower Bromley slide is critical to stabilization of the Bromley slope, which includes the Upper Bromley and Stevenson Road slides. Inclinometers installed at DH99-01 and DH99-02 should be monitored carefully for signs of deep-seated movement. Bimonthly readings, with biweekly readings from mid February to mid April, are recommended. At DH99-01, deep-seated movement could occur at any depth below the bedrock contact at 16.90 m. Movement at the bentonite seam, 25.84 to 25.88 m, should be considered evidence of a deep-seated failure. At DH99-02, movement at greater than 22 m depth should also be considered evidence that the deep-seated failure is present. Because the drill hole did not penetrate to bedrock, the sliding surface could be below the drill hole and movement would be caused by failure at the headscarp or internal shear within the slide mass. Deep-seated movement would indicate retrogressive failure of the translational slide in bedrock, and an increased risk of failure at the highway. Under these circumstances, remedial measures such as a toe berm should be considered. The rate of movement at both inclinometers should be used to assess the schedule under which the remedial measures are implemented.

Additional geotechnical investigations are recommended to better understand the slide mechanism and allow design of remedial measures. Specifically, two holes should be drilled at mid slope near the major tension crack. The drill holes should penetrate at least 20 m into rock, and core samples of bentonite seams should be retrieved for direct shear tests. One hole should be completed with two standpipe piezometers in bedrock, and the other should be completed with an inclinometer. A third drill hole should be put down near the present location of DH99-02. This hole should penetrate at least 10 m into bedrock, and core samples should again be retrieved for testing. The hole should be completed with two standpipe piezometers.

7.2.3 Stevenson Road Slide

Conclusions

The Stevenson Road slide is a local failure which extends for about 60 m along the bank of the Similkameen River. The location of the slide probably correlates with a block of relatively weak coal and shale which was deposited by colluvial or glacial processes. Translational sliding is occurring at the base of this block, as indicated by inclinometer data, and shallow seated failures are occurring as indicated by scarps below the road. Increased pore pressures during snowmelt and rainfall probably cause movement, as was observed in February - March of this year. A contributing factor is the fluvial erosion removal of debris from the toe of the slide by the Similkameen River. A combination of horizontal drain holes and anchors is recommended to stabilize the slide, based on the assumption that the road cannot be relocated to the west.

Recommendations

Implementation of the proposed drain holes and anchors is recommended to meet the stability criterion. However, the measures are costly (estimated total = $270,000), and there is a risk that movement on a deep-seated slide could damage the drains and anchors after construction. Therefore, the stabilizing measures should not be implemented until the possible deep-seated slide is further investigated and, if required, stabilization measures are complete.

7.3 Closure

The Clarke and Bromley landslides are complex and considerable effort and capital will be required to implement stabilization measures. Careful construction monitoring is recommended if the concrete piles or replacement toe berm are undertaken at the Clarke slide. At the Lower Bromley slide, monitoring and additional geotechnical investigations are recommended to gain a better understanding of the slide mechanisms. Major stabilization works at the Upper Bromley and Stevenson Road slides should not be undertaken until the extent of the Lower Bromley slide mass is defined, and the slide is stabilized. Finally, the high combined cost of stabilization of the Clarke and Lower and Upper Bromley slides may warrant investigation of an alternative route for Highway 3.

KC GEOTECHNICAL CONSULTANTS LTD.

Peter N. Buchanan

PROFESSIONAL PROVINCE OF P.N. BUCHANAN BRITISH COLUMBIA GEOSCIENTIST

Peter N. Buchanan, P.Geo.
Project Engineer

Neil K. Singh, P.Eng.
Project Manager

PNB/NKS/HRS:tm

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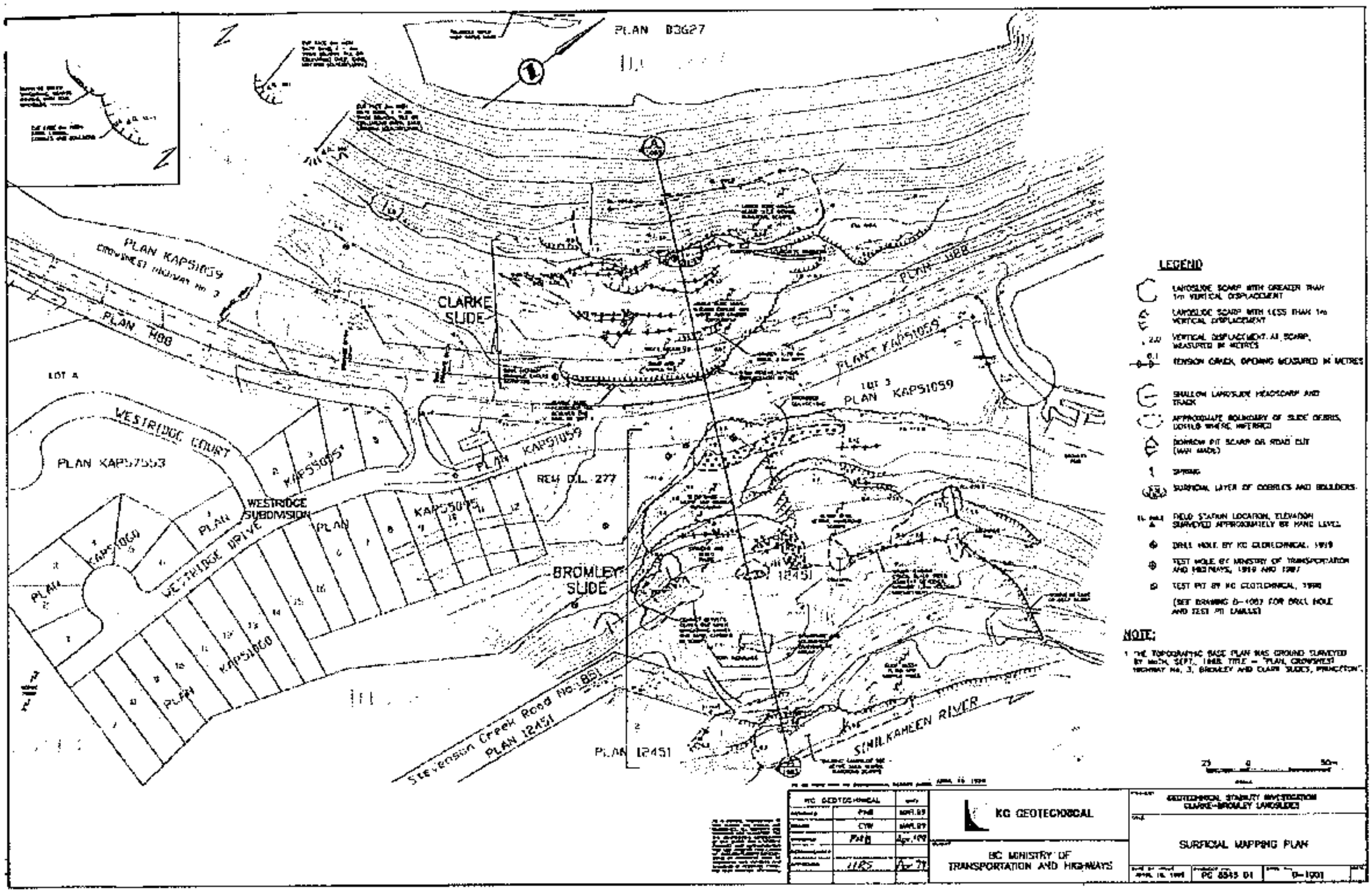
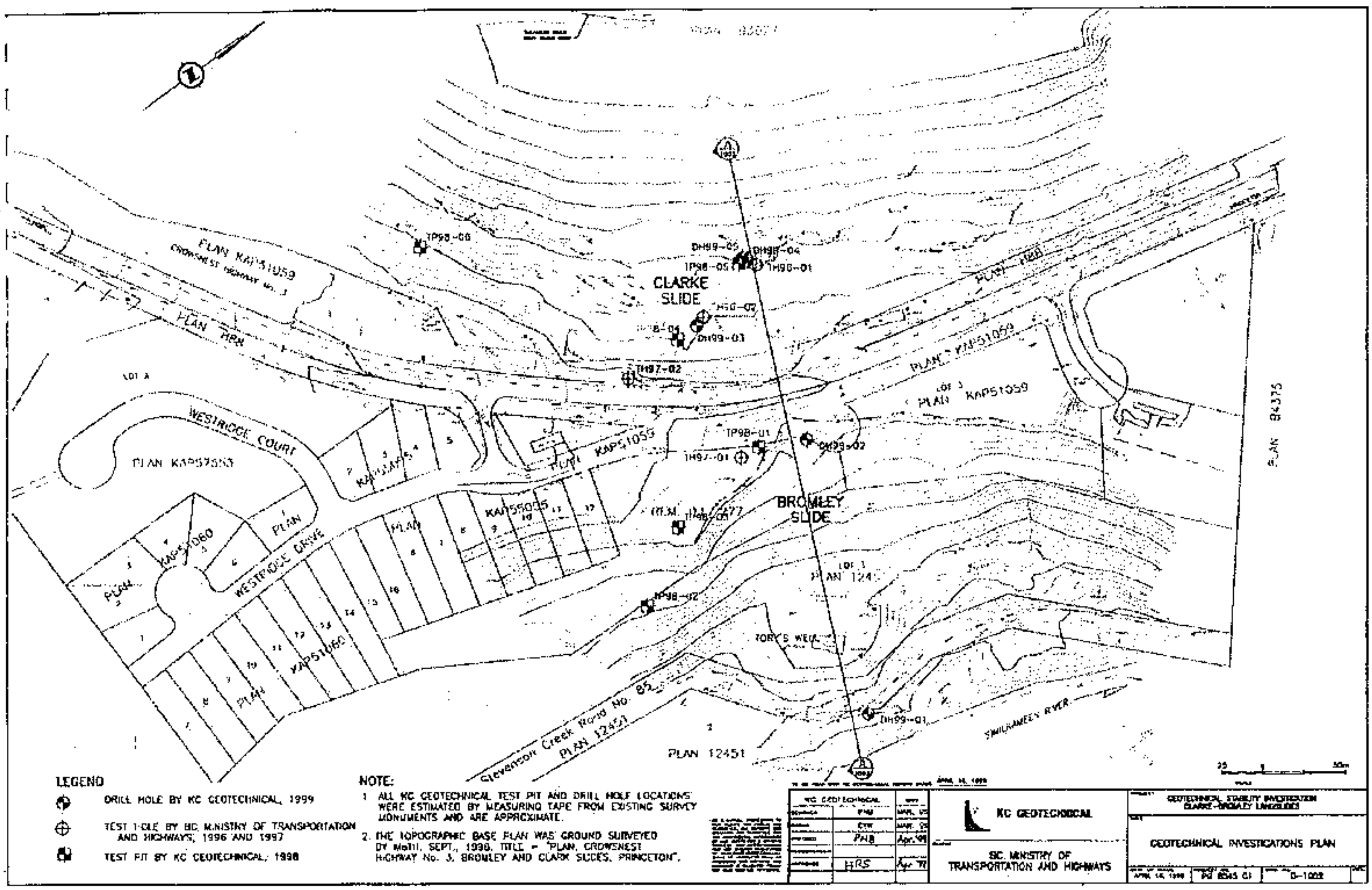
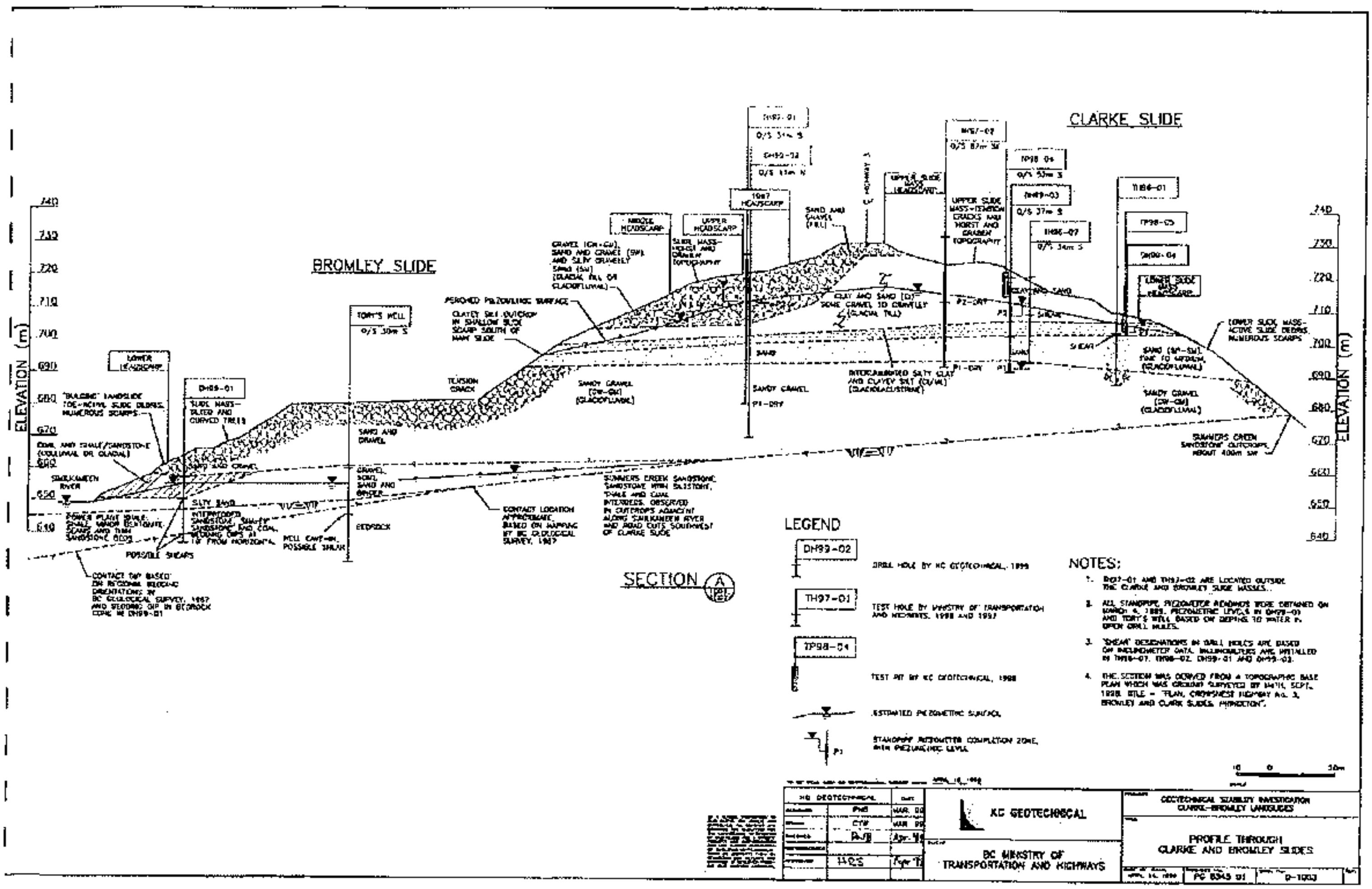
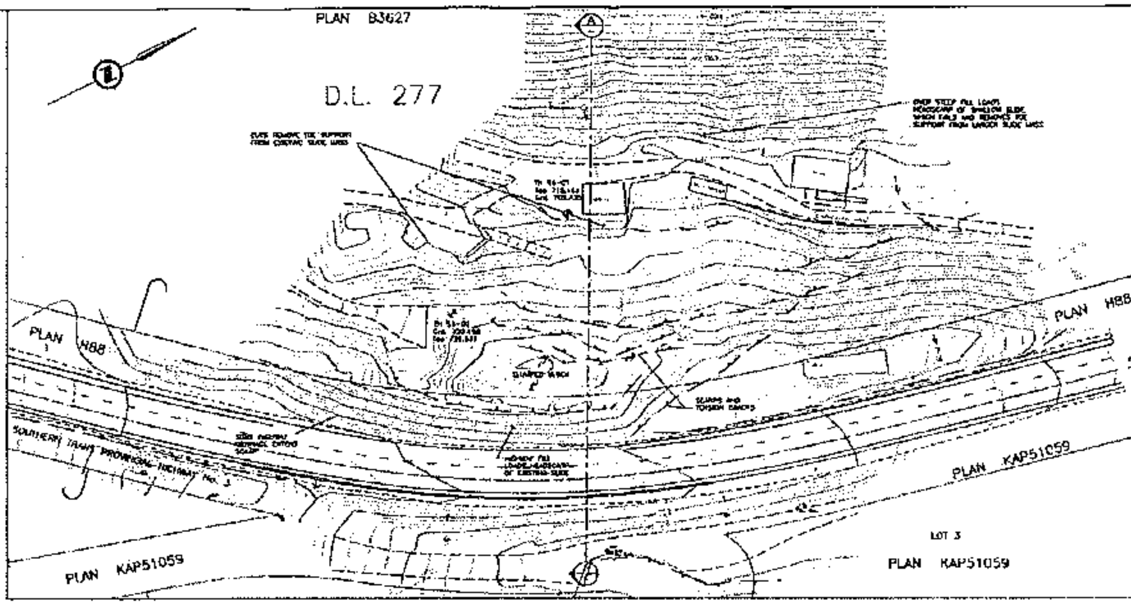


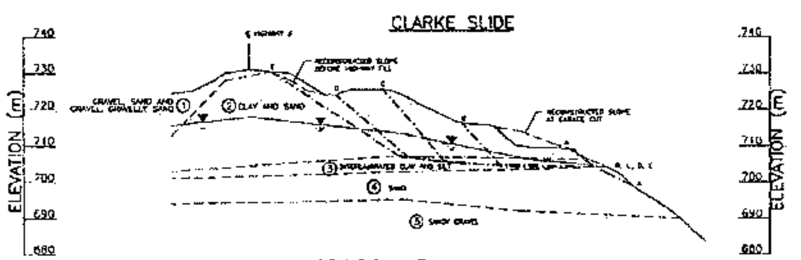
PLAN B3627
PLAN KAP51059
CROWSNEST HIGHWAY No. 3
PLAN H00
CLARKE SLIDE
LOT A
LOT 3 PLAN KAP51059
PLAN KAP57553
WESTRIDGE COURT
WESTRIDGE SUBDIVISION
WESTRIDGE DRIVE
REM D.L. 277
KAP51060
KAP55095
BROMLEY SLIDE
12451
Stevenson Creek Road No. 861
PLAN 12451
SIMILKAMEEN RIVER
LEGEND
LANDSLIDE SCARP WITH GREATER THAN 1m VERTICAL DISPLACEMENT
LANDSLIDE SCARP WITH LESS THAN 1m VERTICAL DISPLACEMENT
VERTICAL DISPLACEMENT AT SCARP, MEASURED IN METRES
TENSION CRACK, OPENING MEASURED IN METRES
SHALLOW LANDSLIDE HEADSCARP AND TRACK
APPROXIMATE BOUNDARY OF SLIDE DEBRIS, DOTTED WHERE INFERRED
BORROW PIT SCARP OR ROAD CUT (MAN MADE)
SPRING
SURFICIAL LAYER OF COBBLES AND BOULDERS.
FIELD STATION LOCATION, ELEVATION SURVEYED APPROXIMATELY BY HAND LEVEL
DRILL HOLE BY KC GEOTECHNICAL, 1999
TEST HOLE BY MINISTRY OF TRANSPORTATION AND HIGHWAYS, 1919 AND 1987
TEST PIT BY KC GEOTECHNICAL, 1999
(SEE DRAWING D-1002 FOR DRILL HOLE AND TEST PIT LOGS)
NOTE:
1 THE TOPOGRAPHIC BASE PLAN WAS GROUND SURVEYED BY MoTH, SEPT., 1988. TITLE – "PLAN, CROWSNEST HIGHWAY No. 3, BROMLEY AND CLARK SLIDES, PRINCETON".
25 0 50m
KC GEOTECHNICAL
KC GEOTECHNICAL
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY LANDSLIDES
SURFICIAL MAPPING PLAN
PC 6545 01
D-1001


CLARKE SLIDE
BROMLEY SLIDE
TP98-00
DH99-05
DH99-04
TP98-05
TH98-01
DH99-03
TH97-02
TP98-01
TH97-01
DH99-02
TP98-02
TH99-01
PLAN KAP51059
CROWSNEST HIGHWAY No. 3
PLAN KAP51056
WESTRIDGE COURT
WESTRIDGE DRIVE
PLAN KAP57553
KAP55035
KAP51060
PLAN 12451
Stevenson Creek Road No. 85
SIMILKAMEEN RIVER
TORY'S WELL
PLAN 94375
LEGEND
DRILL HOLE BY KC GEOTECHNICAL, 1999
TEST HOLE BY BC MINISTRY OF TRANSPORTATION AND HIGHWAYS, 1996 AND 1997
TEST PIT BY KC GEOTECHNICAL, 1998
NOTE:
1. ALL KC GEOTECHNICAL TEST PIT AND DRILL HOLE LOCATIONS WERE ESTIMATED BY MEASURING TAPE FROM EXISTING SURVEY MONUMENTS AND ARE APPROXIMATE.
2. THE TOPOGRAPHIC BASE PLAN WAS GROUND SURVEYED BY MoTH, SEPT., 1998, TITLE = "PLAN, CROWSNEST HIGHWAY No. 3, BROMLEY AND CLARK SLIDES, PRINCETON".
KC GEOTECHNICAL
PHB
HRS
KC GEOTECHNICAL
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL INVESTIGATIONS PLAN
D-1002


CLARKE SLIDE
BROMLEY SLIDE
ELEVATION (m)
740
730
720
710
700
690
680
670
660
650
640
SECTION A
LEGEND
DH99-02
TH97-01
TP98-04
NOTES:
KC GEOTECHNICAL
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS
PROFILE THROUGH CLARKE AND BROMLEY SLIDES
D-1003



CLARKE SLIDE



SECTION A

POTENTIAL FAILURE SURFACE	SEPTEMBER 1994 TOPOGRAPHY	RECONSTRUCTED SLOPE AT GARAGE CUT	RECONSTRUCTED SLOPE BEFORE HIGHWAY FILL	SHEAR STRENGTH CONDITIONS
AA	[illegible]	[illegible]	[illegible]	RESIDUAL FRICTION ANGLES FOR ② AND ③ PEAK FRICTION ANGLE FOR ④
BB	[illegible]	1.27	0.99	
CC	[illegible]	1.07	0.98	
DD	1.02	1.06	1.02	
EE	1.11	1.14	1.15	

LEGEND

A———A POTENTIAL FAILURE SURFACE

③ UNIT NUMBER

▼ PIEZOMETRIC SURFACE #1 (SEE NOTE 2)

NOTES:

1. THE TOPOGRAPHIC BASE PLAN WAS GROUND SURVEYED BY MoTH, SEPT., 1994. TITLE = "PLAN, BROMLEY SLIDE AT PRINCETON". THE TOPOGRAPHY PREDATES ONE OF THE SLIDES WHICH REMOVED THE PASSING LANE AT HIGHWAY 3
2. PIEZOMETRIC SURFACE #1 REFLECTS A PERCHED WATER TABLE ABOVE UNIT ③.
3. SEE DRAWING B-1003 FOR MATERIAL PROPERTIES AND STABILITY ANALYSIS METHOD.

10 0 20m

BC GEOTECHNICAL		
	PHB	MAR. 95
	CTP	MAR. 95
	PHB	Apr. 95
	HRS	Apr. 95

KC GEOTECHNICAL

BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

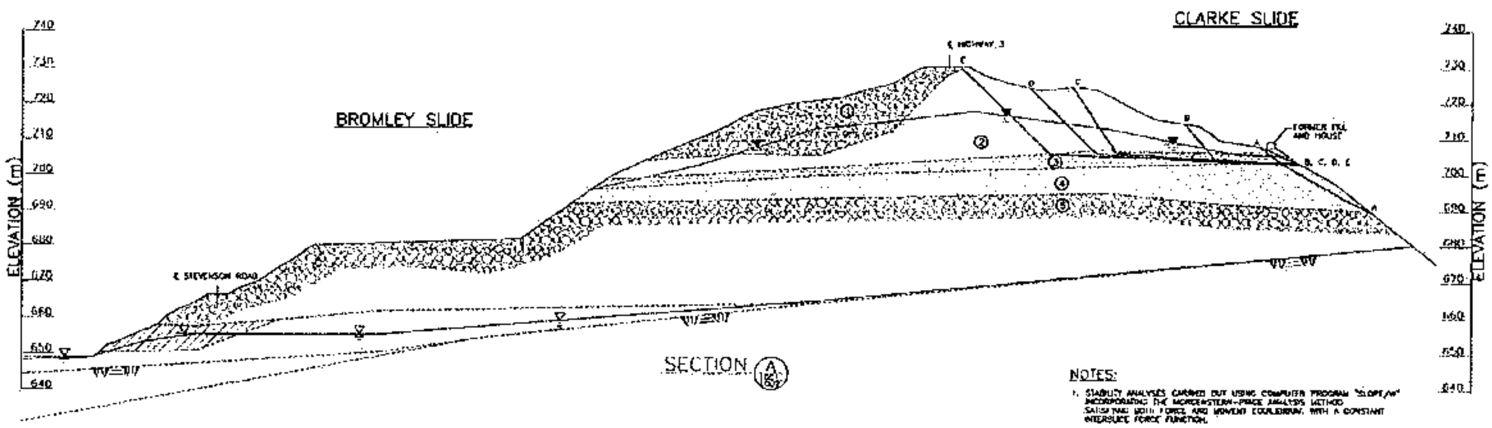
GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY LANDSLIDES

CLARKE SLIDE
STABILITY ANALYSIS
SHEET 1 OF 2

APRIL 18, 1995 | PG 8545 01 | B-1004

MATERIAL PROPERTIES						
UNIT NO.	MATERIAL	UNIT WEIGHT γ BULK (kN/m³)	EFFECTIVE SHEAR STRENGTH c' (kPa)	PEAK ø' (DEGREES)	RESIDUAL ø' (DEGREES)	PIEZOMETRIC SURFACE No
1	GRAVEL, SAND AND GRAVEL, GRAVELLY SAND	20	0	37	-	1
2	CLAY AND SAND	22	0	31.5	28.5	1
3	INTERLAMINATED CLAY AND SILT	19.5	0	25.5	13.5	1
4	SAND	17	0	33	-	-
5	SANDY GRAVEL	22	0	38	-	-

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN-PRICE METHOD)	SHEAR STRENGTH CONDITIONS
AA	1.01	RESIDUAL FRICTION ANGLES FOR 2 AND 3 PEAK FRICTION ANGLE FOR 4
BB	0.89	RESIDUAL FRICTION ANGLES FOR 2 AND 3
CC	0.95	RESIDUAL FRICTION ANGLES FOR 2 AND 3
DD	1.03	RESIDUAL FRICTION ANGLES FOR 2 AND 3
EE	1.19	RESIDUAL FRICTION ANGLES FOR 2 AND 3



SECTION A

LEGEND

A ——— A POTENTIAL FAILURE SURFACE

1 UNIT NUMBER

PIEZOMETRIC SURFACE #1 (SEE NOTE 2)

PIEZOMETRIC SURFACE #2 (SEE NOTE 2)

NOTES:

1. STABILITY ANALYSES CARRIED OUT USING COMPUTER PROGRAM "SLOPE/W" INCORPORATING THE MORGENSTERN-PRICE ANALYSIS METHOD SATISFYING BOTH FORCE AND MOMENT EQUILIBRIUM, WITH A CONSTANT INTERSLICE FORCE FUNCTION.
2. PIEZOMETRIC SURFACE No. 1 REFLECTS A PERCHED WATER TABLE ABOVE UNIT 3. PIEZOMETRIC SURFACE No. 2 IS A REGIONAL WATER TABLE RISING FROM THE SIMILKAMEEN RIVER. UNITS 4 AND 5 ARE ASSUMED TO BE UNSATURATED.
3. THE SECTION WAS DERIVED FROM A TOPOGRAPHIC BASE PLAN WHICH WAS GROUND SURVEYED BY MoTH, SEPT., 1998, TITLE = "PLAN, CROWSNEST HIGHWAY No. 3, BROMLEY AND CLARKE SLIDES, PRINCETON"

10 0 20m

APRIL 15, 1999

KC GEOTECHNICAL	
PHB	MAR. 99
CYW	MAR. 99
PHB	Apr. 99
HRS	Apr. 99

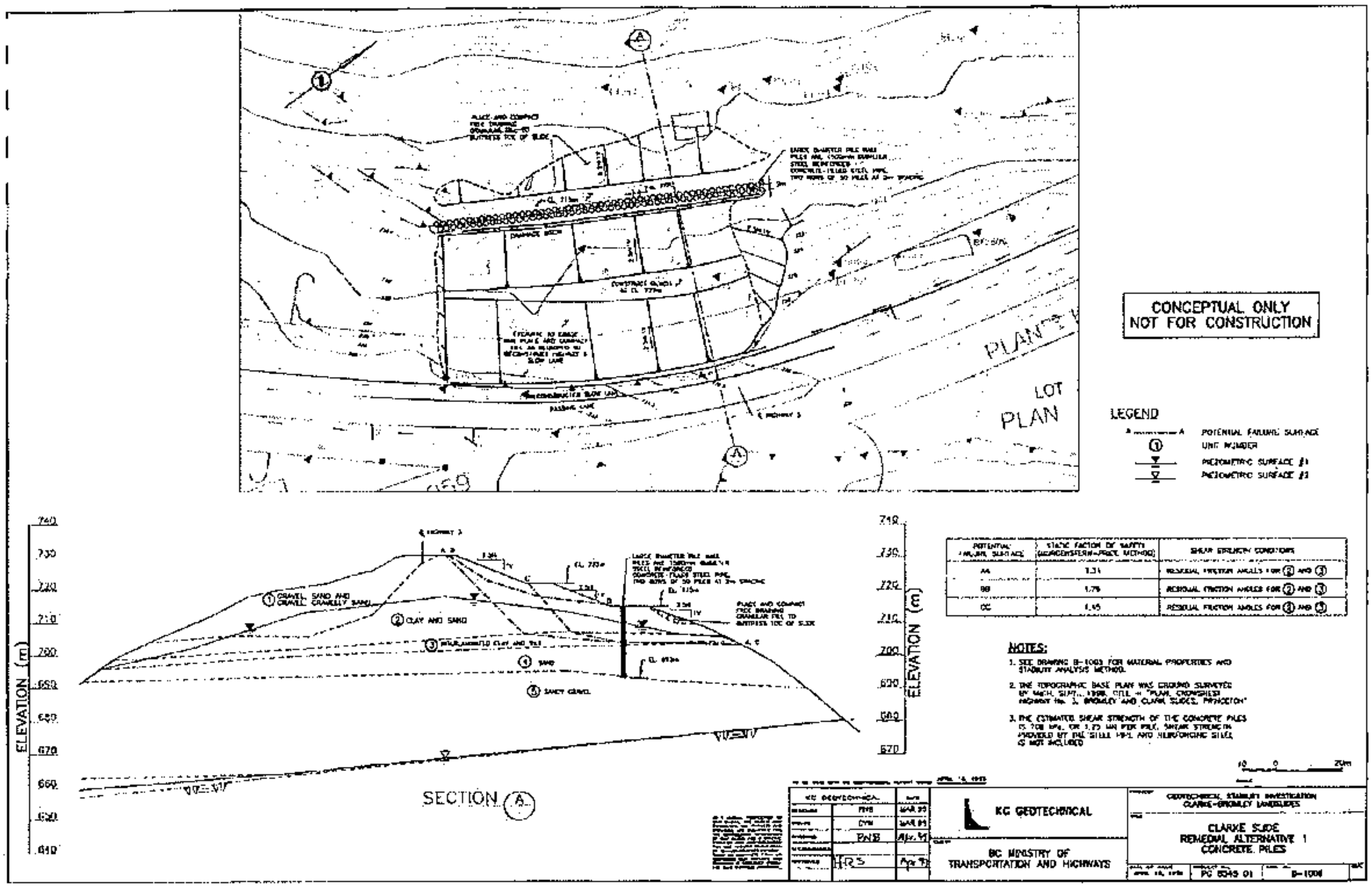
KC GEOTECHNICAL

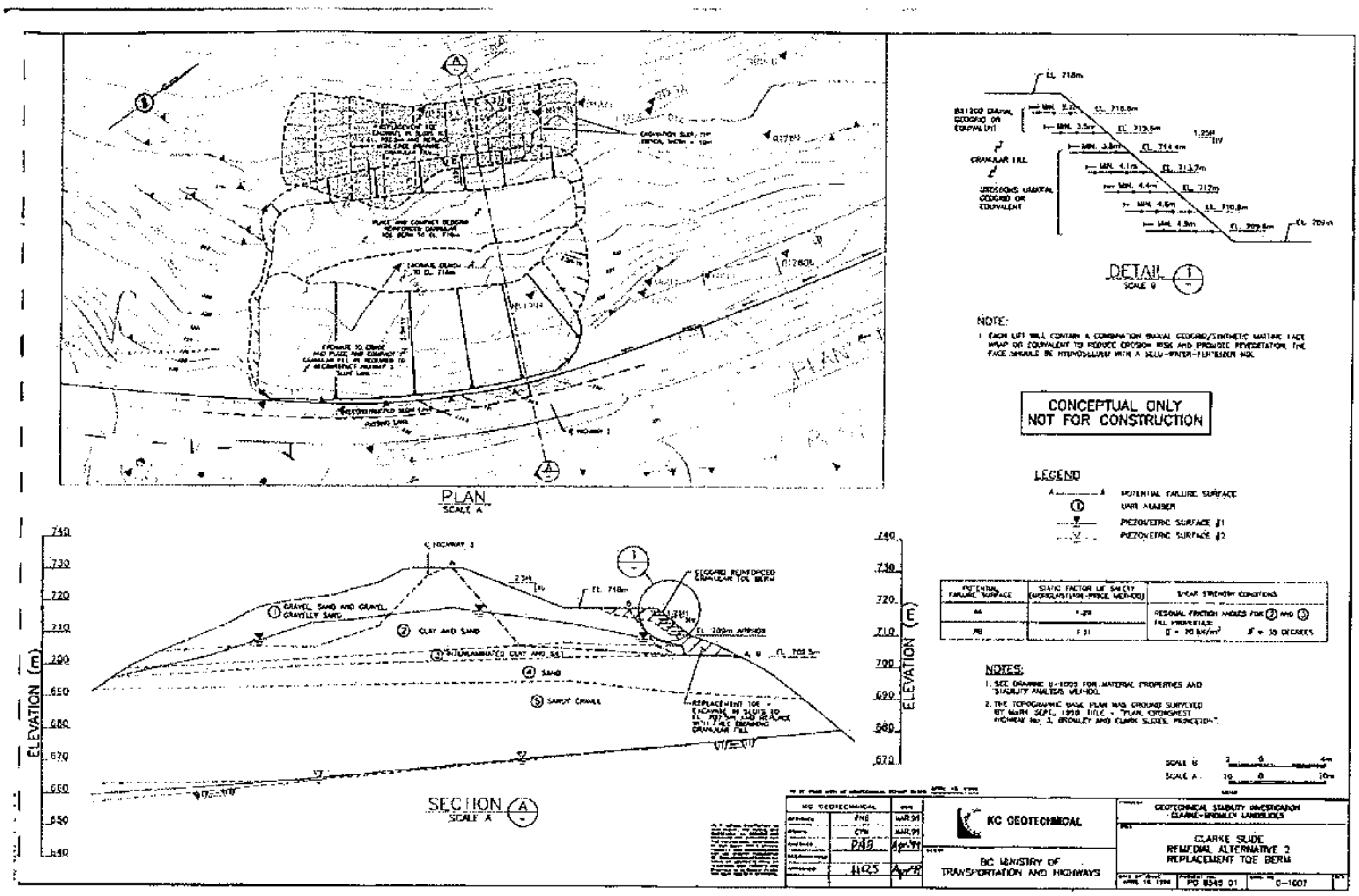
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY LANDSLIDES

CLARKE SLIDE
STABILITY ANALYSIS
SHEET 2 OF 2

APRIL 15, 1999 | PG 8543 01 | B-1005

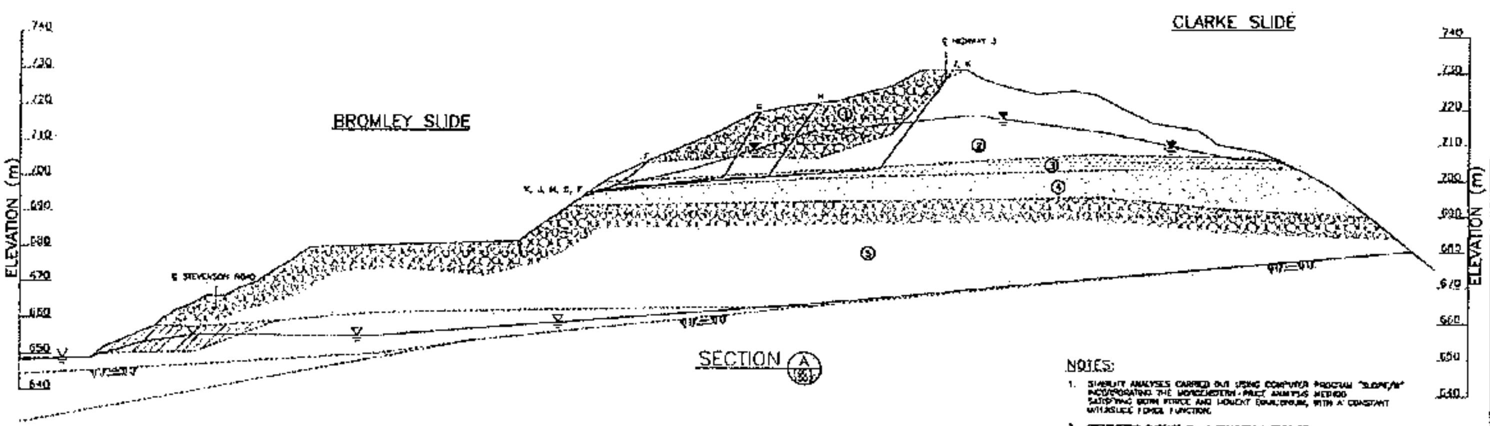

CONCEPTUAL ONLY
NOT FOR CONSTRUCTION
PLAN
LEGEND
POTENTIAL FAILURE SURFACE
UNIT NUMBER
SECTION A
ELEVATION (m)
GRAVEL, SAND AND GRAVEL
CLAY AND SAND
SAND
NOTES:
1. SEE DRAWING B-1003 FOR MATERIAL PROPERTIES AND STABILITY ANALYSIS METHOD.
KC GEOTECHNICAL
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS
CLARKE SLIDE
REMEDIAL ALTERNATIVE 1
CONCRETE PILES



PLAN
SCALE A
SECTION A
SCALE A
DETAIL 1
SCALE B
ELEVATION (m)
GRAVEL, SAND AND GRAVEL, GRAVELLY SAND
CLAY AND SAND
SAND
SANDY GRAVEL
EL. 718m
EL. 709m
NOTE:
CONCEPTUAL ONLY
NOT FOR CONSTRUCTION
LEGEND
POTENTIAL FAILURE SURFACE
PIEZOMETRIC SURFACE #1
PIEZOMETRIC SURFACE #2
SHEAR STRENGTH CONDITIONS
NOTES:
SCALE B
SCALE A
KC GEOTECHNICAL
BC MINISTRY OF
TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL STABILITY INVESTIGATION
CLARKE SLIDE
REMEDIAL ALTERNATIVE 2
REPLACEMENT TOE BERM
PO 8545 01

MATERIAL PROPERTIES						
UNIT NO	MATERIAL	UNIT WEIGHT γ BULK (kN/m³)	EFFECTIVE SHEAR STRENGTH c' (kPa)	PEAK φ' (DEGREES)	RESIDUAL φ' (DEGREES)	PIEZOMETRIC SURFACE NO
1	GRAVEL, SAND AND GRAVEL, GRAVELLY SAND	20	0	37	-	1
2	CLAY AND SAND	22	0	31.5	28.5	2
3	INTERLAMINATED CLAY AND SILT	19.5	0	25.5	13.5	1
4	SAND	17	0	33	-	-
5	SANDY GRAVEL	22	0	38	-	-

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN-PRICE METHOD)	SHEAR STRENGTH CONDITIONS
FF	1.01	PEAK FRICTION ANGLES FOR 2 AND 3
GG	0.87	RESIDUAL FRICTION ANGLES FOR 2 AND 3
HH	0.92	RESIDUAL FRICTION ANGLES FOR 2 AND 3
JJ	0.93	RESIDUAL FRICTION ANGLES FOR 2 AND 3
KK	1.07	PEAK FRICTION ANGLE FOR 2 BELOW HIGHWAY. RESIDUAL FRICTION ANGLE FOR 3 BELOW HIGHWAY. RESIDUAL FRICTION ANGLES FOR 2 AND 3 IN EXISTING SCARP AREA



NOTES:

1. STABILITY ANALYSES CARRIED OUT USING COMPUTER PROGRAM "SLOPE/W" INCORPORATING THE MORGENSTERN-PRICE ANALYSIS METHOD SATISFYING BOTH FORCE AND MOMENT EQUILIBRIUM, WITH A CONSTANT INTERSLICE FORCE FUNCTION.
2. PIEZOMETRIC SURFACE No. 1 REFLECTS A PERCHED WATER TABLE ABOVE UNIT 3. PIEZOMETRIC SURFACE No. 2 IS A REGIONAL WATER TABLE RISING FROM THE SIMILKAMEEN RIVER. UNITS 4 AND 5 ARE ASSUMED TO BE UNSATURATED.
3. THE SECTION IS DERIVED FROM A TOPOGRAPHIC BASE PLAN WHICH WAS GROUND SURVEYED BY MoTH, SEPT., 1998. TITLE = "PLAN, CROWSNEST HIGHWAY No. 3, BROMLEY AND CLARK SLIDES, PRINCETON".

LEGEND

- A —— A POTENTIAL FAILURE SURFACE
- 1 UNIT NUMBER
- PIEZOMETRIC SURFACE #1 (SEE NOTE 2)
- PIEZOMETRIC SURFACE #2 (SEE NOTE 2)

10 0 20m

KC GEOTECHNICAL		
	PWB	MAR. 99
	CYR	MAR. 99
	PWB	Apr. 99
	HRS	Apr. 99

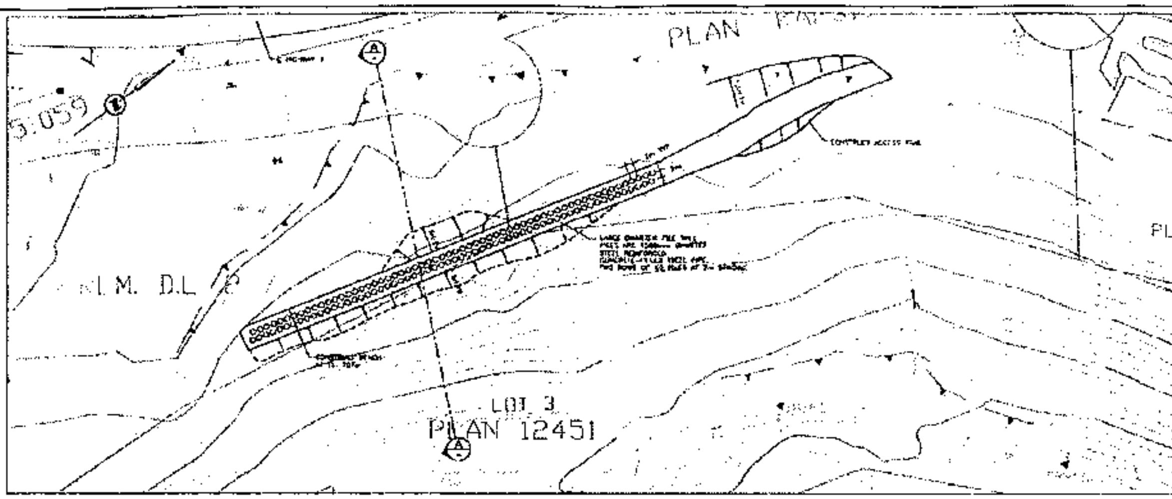
KC GEOTECHNICAL

BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY LANDSLIDES

UPPER BROMLEY SLIDE STABILITY ANALYSIS

APRIL 15, 1999 | PO 8345 01 | B-1008



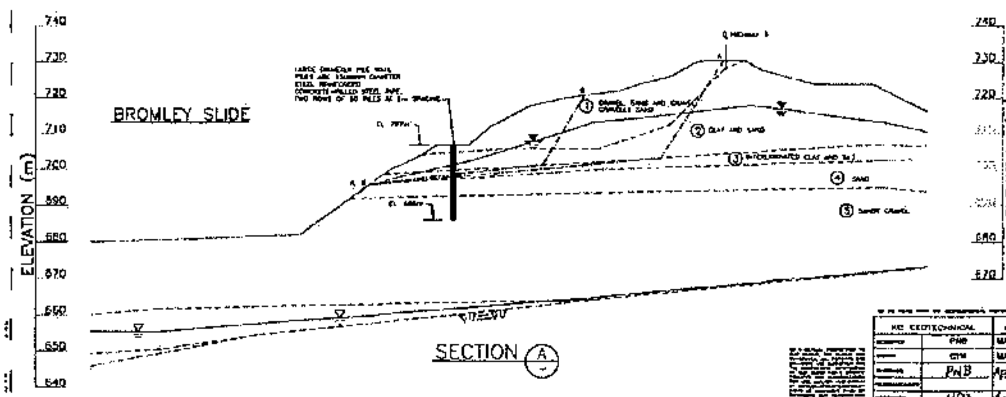
CONCEPTUAL ONLY
NOT FOR CONSTRUCTION

LEGEND

A — — — A POTENTIAL FAILURE SURFACE
① UNIT NUMBER
PIEZOMETRIC SURFACE #1
PIEZOMETRIC SURFACE #2

NOTES:

1. SEE DRAWING D-1008 FOR MATERIAL PROPERTIES AND STABILITY ANALYSIS METHOD.
2. THE TOPOGRAPHIC BASE PLAN WAS GROUND SURVEYED BY MoTH, SEPT., 1998. TITLE – "PLAN, CROWSNEST HIGHWAY No. 3. BROMLEY AND CLARK SLIDES, PRINCETON".
3. THE ESTIMATED SHEAR STRENGTH OF THE CONCRETE PILES IS 706 kPa, OR 1.25 MN PER PILE. SHEAR STRENGTH PROVIDED BY THE STEEL PIPE AND REINFORCING STEEL IS NOT INCLUDED.



CONCRETE PILES

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN-PRICE METHOD)	SHEAR STRENGTH CONDITIONS
AA	1.18	PEAK FRICTION ANGLE FOR ② BELOW HIGHWAY. RESIDUAL FRICTION ANGLE FOR ③ BELOW HIGHWAY. RESIDUAL FRICTION ANGLES FOR ② AND ③ IN EXISTING SCARP AREA
BB	1.26	RESIDUAL FRICTION ANGLES FOR ② AND ③

COMBINED CONCRETE PILES AND DRAIN HOLES
(SEE DRAWING D-1010 FOR DRAIN HOLE DETAILS)

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN-PRICE METHOD)	SHEAR STRENGTH CONDITIONS
AA	1.58	PEAK FRICTION ANGLE FOR ② BELOW HIGHWAY. RESIDUAL FRICTION ANGLE FOR ③ BELOW HIGHWAY. RESIDUAL FRICTION ANGLES FOR ② AND ③ IN EXISTING SCARP AREA
BB	1.33	RESIDUAL FRICTION ANGLES FOR ② AND ③

10 0 20m

KC GEOTECHNICAL		
	PHB	MAR.99
	GTM	MAR.99
	PHB	Apr.99
	HRS	Apr 99

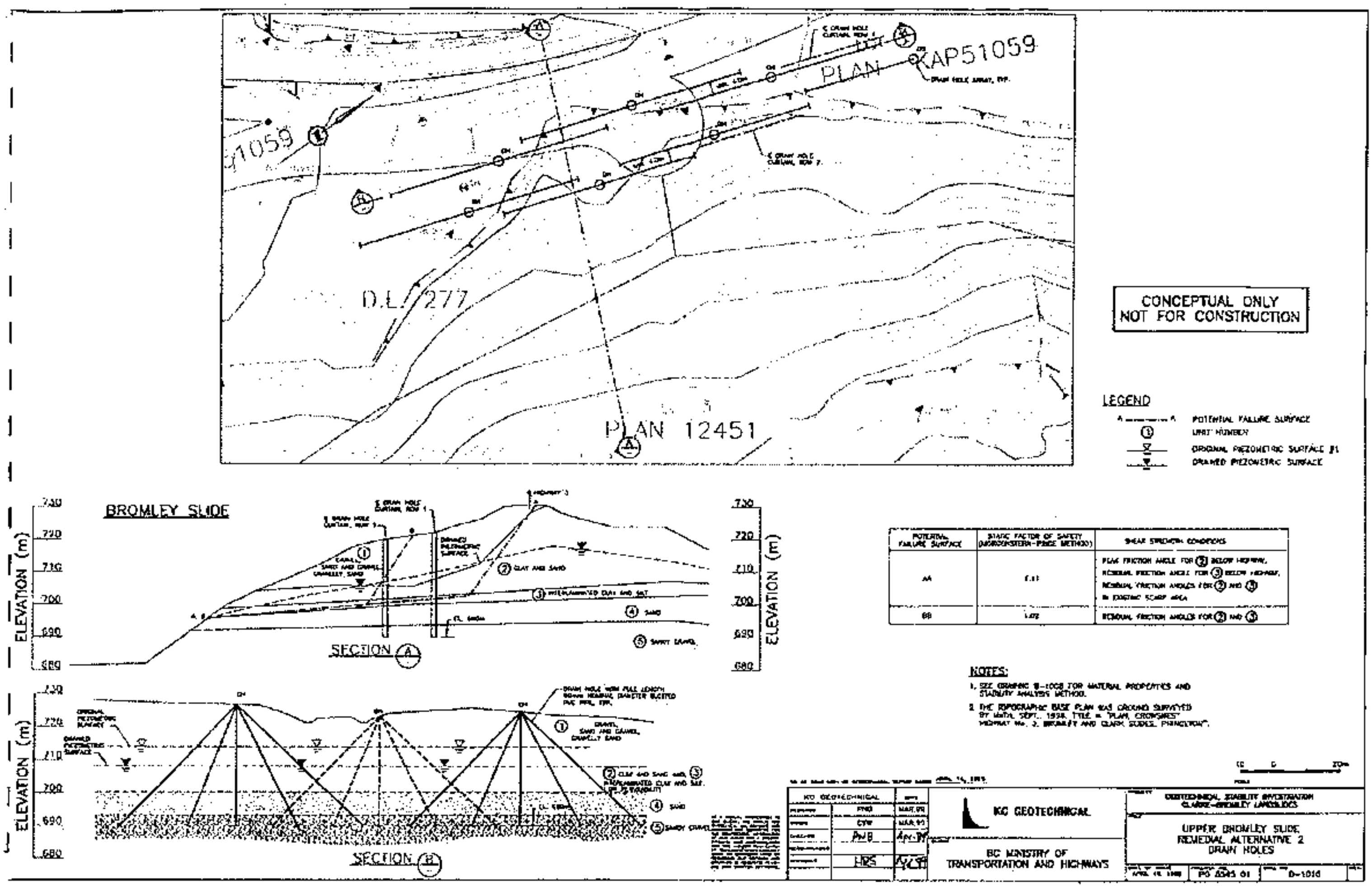
KC GEOTECHNICAL

BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

GEOTECHNICAL STABILITY INVESTIGATION CLARK-BROMLEY LANDSLIDES

UPPER BROMLEY SLIDE
REMEDIAL ALTERNATIVE 1
CONCRETE PILES

APRIL 16, 1999 | PO 6543 01 | D-1009



CONCEPTUAL ONLY
NOT FOR CONSTRUCTION
PLAN KAP51059
D.L. 277
PLAN 12451
LEGEND
POTENTIAL FAILURE SURFACE
UNIT NUMBER
ORIGINAL PIEZOMETRIC SURFACE
DRAINED PIEZOMETRIC SURFACE
BROMLEY SLIDE
ELEVATION (m)
730
720
710
700
690
680
CLAY AND SAND
INTERLAMINATED CLAY AND SILT
SAND
SANDY GRAVEL
SECTION A
SECTION B
NOTES:
1. SEE DRAWING B-1008 FOR MATERIAL PROPERTIES AND STABILITY ANALYSIS METHOD.
2. THE TOPOGRAPHIC BASE PLAN WAS GROUND SURVEYED BY MoTH, SEPT., 1998, TITLE = "PLAN, CROWSNEST HIGHWAY No. 3, BROMLEY AND CLARK SLIDES, PRINCETON".
KC GEOTECHNICAL
BC MINISTRY OF
TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL STABILITY INVESTIGATION
CLARKE-BROMLEY LANDSLIDES
UPPER BROMLEY SLIDE
REMEDIAL ALTERNATIVE 2
DRAIN HOLES
PO 4545 01
D-1010

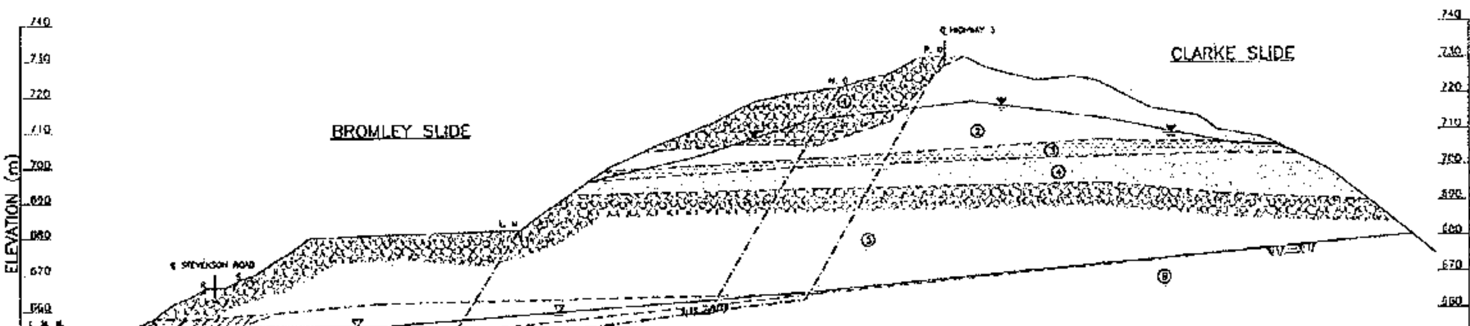
MATERIAL PROPERTIES						
UNIT NO.	MATERIAL	UNIT WEIGHT Γ BULK (kN/m³)	EFFECTIVE SHEAR STRENGTH c' (kPa)	PEAK ϕ' (DEGREES)	RESIDUAL ϕ' (DEGREES)	PIEZOMETRIC SURFACE No
1	GRAVEL, SAND AND GRAVEL, GRAVELLY SAND	20	0	37	–	1
2	CLAY AND SAND	22	0	31.5	20.5	1
3	INTERLAMINATED CLAY AND SILT	19.5	0	25.5	13.5	1
4	SAND	17	0	35	–	–
5	SANDY GRAVEL	22	0	38	–	–
6	DISTURBED COAL AND SHALE	19	0	18	18	2
7	SILTY SAND	22	0	38	–	2
8	POWER PLANT SHALE	19	0	21	–	2
9	SUMMERS CREEK SANDSTONE	23	0	35	–	2
8 AND 9	CLAY SEAMS – SHEAR STRENGTH TESTS	19/23	0	21	16	2
8 AND 9	CLAY SEAMS – BACK ANALYSIS	19/23	0	–	11	2

LOWER BROMLEY SLIDE

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN–PRICE METHOD)	SHEAR STRENGTH CONDITIONS
LL	1.55	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – TESTED
MM	1.12	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – BACK ANALYSED
NN	1.39	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – TESTED
OO	0.99	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – BACK ANALYSED
PP	1.45	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – TESTED
QQ	1.02	RESIDUAL FRICTION ANGLES FOR CLAY SEAMS IN 8 AND 9 – BACK ANALYSED

STEVENSON ROAD

POTENTIAL FAILURE SURFACE	STATIC FACTOR OF SAFETY (MORGENSTERN–PRICE METHOD)	SHEAR STRENGTH CONDITIONS
RR	0.80	6 COAL AND SHALE HAVE BEEN DISTURBED BY COLLUVIAL OR GLACIAL PROCESSES, RESIDUAL FRICTION ANGLE USED.
SS	0.38	



SECTION A

LEGEND

POTENTIAL FAILURE SURFACE
UNIT NUMBER
PIEZOMETRIC SURFACE #1 (SEE NOTE 2)
PIEZOMETRIC SURFACE #2 (SEE NOTE 2)

NOTES:

1. STABILITY ANALYSES CARRIED OUT USING COMPUTER PROGRAM "SLOPE/W" INCORPORATING THE MORGENSTERN-PRICE ANALYSIS METHOD SATISFYING BOTH FORCE AND MOMENT EQUILIBRIUM, WITH A CONSTANT INTERSLICE FORCE FUNCTION.
2. PIEZOMETRIC SURFACE No. 1 REFLECTS A PERCHED WATER TABLE ABOVE UNIT 3. PIEZOMETRIC SURFACE No. 2 IS A REGIONAL WATER TABLE RISING FROM THE SIMILKAMEEN RIVER. UNITS 4 AND 5 ARE ASSUMED TO BE UNSATURATED.
3. THE SECTION WAS DERIVED FROM A TOPOGRAPHIC BASE PLAN WHICH WAS GROUND SURVEYED BY MoTH, SEPT. 1998. TITLE – "PLAN, CHOPAKA HIGHWAY No. 3, BROMLEY AND CLARKE SLIDES, PRINCETON".

10 0 20m

KC GEOTECHNICAL		
	PHB	MAR. 99
	CTW	MAR. 99
	PHB	Apr. 99
	HPS	Apr. 99

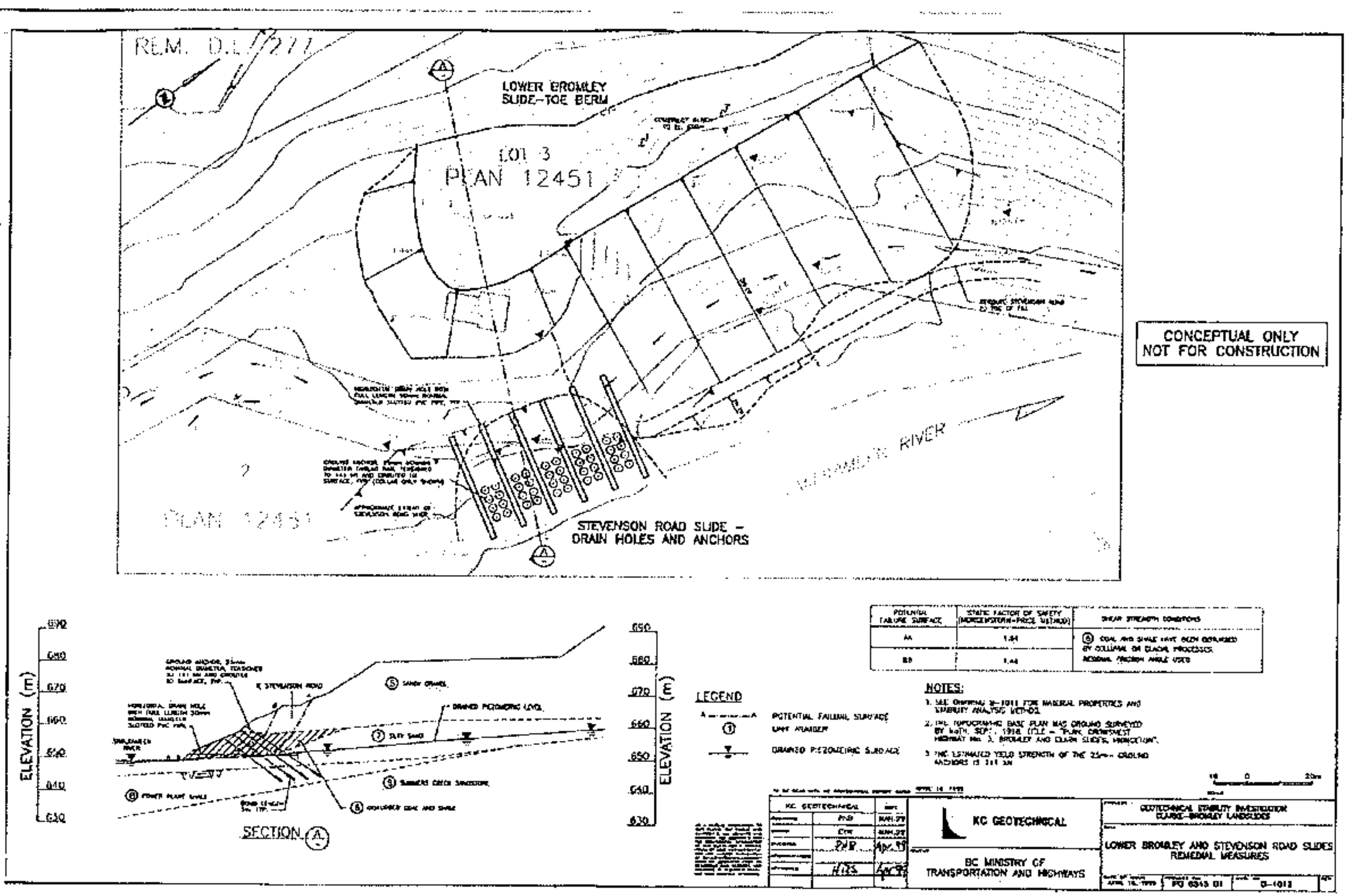
KC GEOTECHNICAL

BC MINISTRY OF TRANSPORTATION AND HIGHWAYS

GEOTECHNICAL STABILITY INVESTIGATION CLARKE–BROMLEY LANDSLIDES

LOWER BROMLEY AND STEVENSON ROAD SLIDES STABILITY ANALYSIS

APRIL 16, 1999 | PO 8545 01 | B-1011



LOWER BROMLEY SLIDE-TOE BERM
LOT 3
PLAN 12451
ELK RIVER
STEVENSON ROAD SLIDE - DRAIN HOLES AND ANCHORS
CONCEPTUAL ONLY NOT FOR CONSTRUCTION
ELEVATION (m)
SECTION A
LEGEND
POTENTIAL FAILURE SURFACE
DRAINED PIEZOMETRIC SURFACE
NOTES:
KC GEOTECHNICAL
BC MINISTRY OF TRANSPORTATION AND HIGHWAYS
GEOTECHNICAL STABILITY INVESTIGATION CLARKE-BROMLEY LANDSLIDES
LOWER BROMLEY AND STEVENSON ROAD SLIDES REMEDIAL MEASURES