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FINAL REPORT ON

GEOTECHNICAL
SEISMIC VULNERABILITY ASSESSMENT
OF FRASER RIVER ESCARPMENT
MAPLE RIDGE, B.C.

Submitted to:
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1.0 INTRODUCTION

This report presents the results of a seismic vulnerability assessment of the Fraser River Escarpment, located south of River Road between Haney and Port Hammond in the District of Maple Ridge, B.C. The scope of work, which was described in our proposal to the District dated August 8, 2002, is outlined below in Section 1.3.

1.1 Description of Study Area

River Road is located north of the crest of steep bluffs which form the north bank of the Fraser River over a length of about 1.7 km from just east of Carshill Street to just west of Fraserview Street. These bluffs are bounded to the east and west by major landslide features known as the Haney Slide (of 1880) and Port Hammond Slide, respectively, as shown on Figure 1. The bluffs slope down into the river at an overall slope angle that is generally in the range of 22 to 26 degrees. The overall height of the bluffs decreases from a maximum of about 55 m (including a submerged depth of about 20 m at low river level) at the east end of the bluffs, to about 40 m (including a submerged depth of about 16 m at low river level) near the west end of the bluffs just east of the Port Hammond slide. This is believed to be one of the deepest sections of river downstream of Hope.

The Canadian Pacific Railway (CP Rail) runs along a bench about 15 m wide, which is located midway up the overall slope, approximately 7 to 8 metres above low river level (elevation zero, Geodetic).

River Road is classified as a primary road within the District of Maple Ridge road network. River Road is located as close as 50 m from the crest of the bluffs near Carshill Street, just west of the Haney Slide, with the set-back increasing in a westerly direction to about 280 m just east of the major Port Hammond Slide (west of Fraserview Street). East of Carshill Street, River Road drops down along the backscarp of the Haney Slide and crosses the landslide failure mass in a southeasterly direction.

We understand from utility mapping information provided by the District of Maple Ridge that there are a number of sanitary sewer mains in the study area, as indicated on Figure 2 and described below:

- a 450 to 500 mm diameter concrete sanitary forcemain, located along River Road between Best Street and the pump station on the north side of the Haney Bypass at 225th Street;

- a 525 mm diameter asbestos-cement sanitary gravity main, traversing between Laity and 212th Streets south of 117th Avenue, running south along 212th Street to River Road, and west along River Road from 212th Street to Best Street;

- a 1050 mm diameter concrete sanitary gravity main belonging to the GVRD, located along River Road from Best Street to west of Steeves Street, after which it traverses southwest across the Maple Ridge Golf Course and then west along Lorne Avenue; and

- a 375 to 450 mm diameter concrete gravity main, running south along 223rd Street to River Road and southeast to the 225th Street Pump Station.

Numerous secondary roads and houses are located between River Road and the crest of the bluffs, with some of the houses located within 30 m of the crest, as shown on Figure 2. In addition, housing developments have been constructed within the Haney Slide failure mass north of River Road and on the Port Hammond slide failure mass at the south end of Best Street.

1.2 Landslide Hazards Along Fraser River Bluffs

The Fraser River Escarpment has been historically subjected to landslide activity. The major slides at Haney (between about the 220th and 222nd Street alignments, referred to as the Haney Slide,) and at Port Hammond (at the east side of the Maple Ridge Golf Course, referred to as the Port Hammond Slide) both extended about 250 to 300 metres back from the crest of the bluffs. Two smaller slides, both extending about 60 m back from the crest of the bluffs, have occurred at Port Hammond just west of the major slide (the Minor Port Hammond Slide), and southeast of the foot of Fir Street (the Fir Street Slide). The crest of the existing slope is broken by these slide areas, and by several ravines that daylight at the slope face, as can be seen on Figure 1. Surficial sloughing and shallow slides along the slopes above and below the CPR bench have also occurred periodically.

Previous studies carried out for the BC Ministry of Environment (MoE) by Golder Associates (Golder) between 1978 and 1986 (Golder 1979, 1983, 1986) indicated that the stability of the slopes forming the Fraser River Escarpment is marginal under static conditions. Limited analyses of the impact of seismic accelerations on the stability of the bluffs were carried out during the 1978/79 study, but design ground motions used at that time were much lower than those used today.

A regional overview assessment of seismic vulnerability for the entire District of Maple Ridge was carried out by Golder in 2001/02, the results of which are documented in a report submitted to the District (Golder 2002). In that report, the steep bluffs along the north bank of the Fraser River, south of River Road, were identified as being at moderate to high risk of failure due to seismic accelerations, with impacted areas possibly extending back to, or just beyond, River Road.

1.3 Purpose and Scope of This Study

This study was limited in scope and had specific objectives, as outlined in our proposal. The purpose of this study was:

- to compile and review available information, including piezometer monitoring data that has been collected by the District since the early 1980's, and assess whether there have been any changes in the static stability of the bluffs since our previous studies between 1978 and 1986,

- to provide a preliminary assessment of the vulnerability of properties and infrastructure within the vicinity of the Fraser River Escarpment, including River Road and a number of sanitary sewer mains, to damage resulting from potential failures of the escarpment slopes which could occur during or following a major seismic event, and

- to determine requirements for more detailed investigation and analyses.

While seismic ground motion intensities are identified in the report for various risk levels, discussions of levels of "risk" to facilities and residents associated with various seismic hazards are limited to qualitative descriptions of likelihood of occurrence ("high" or "low") along with identification of *potential* consequences. A comprehensive Risk Assessment Study, involving much more detailed investigation, laboratory testing and analysis, would be required to quantify the probabilities associated with the *potential* consequences of the seismic hazards that have been identified in this study.

An assessment of the vulnerability of specific properties within the study area is also beyond the scope of this study, requiring detailed site-specific investigations and analyses. Furthermore, no analyses were carried out in this study to assess the impacts on stability of secondary features such as the backscarps of existing slides and ravine slopes.

2.0 REVIEW OF AVAILABLE INFORMATION

The following sources of information were reviewed during this study:

- Surficial geology maps by the Geological Survey of Canada,

- Topographical plans, legal mapping and pipeline system mapping provided by the District of Maple Ridge (2001),

- Reports by Golder for MoE (August 1979, July 1983, March 1986), accompanying this report as Appendices I, II and III, respectively.

- Piezometer data collected by the District of Maple Ridge (1983-86, 1993-99, 2003),

- Fraser River survey data collected by the Water Management Branch of the BC Ministry of Environment (1978, 1981, 1985, 1986, 1992, 1997),

- Fraser River discharge data collected by the Water Survey of Canada (1950-2002),

- Data collected by Golder for CP Rail.

- Cone Penetration Testing (CPT) data from the Haney Slide area collected by the University of British Columbia (UBC) (Davies, 1985),

- Publications on the behaviour of Haney Clay by UBC (Campanella & Vaid, 1974 and Vaid & Campanella, 1977),

- Report by Cook Pickering & Doyle Ltd. (CP&D) on the Haney Slide area (November 1977),

2.1 Surficial Geology

The upland urban area of Maple Ridge is generally underlain by glacio-marine silty clay to fine sand of the Fort Langley Formation, including extensive deposits of silty clay known locally as Haney Clay. West of the Haney Bypass and for some distance south of River Road, the Fort Langley sediments are overlain by Sumas Drift deposits typically consisting of raised proglacial deltaic sands and gravels.

2.2 Field Investigations

No additional field investigations have been carried out specifically for this present study. During the 1978/79 study for MoE, five boreholes (BH101, BH103 to BH106) were put down at locations that range from 45 to 90 metres behind the slope crest (as shown on Figures 1 and 2), which supplemented information available from boreholes put down during a 1976 investigation for C.P. Rail. These boreholes provided stratigraphic information along a 2.13 km length of river bank. Standpipe piezometers were also installed in all five of the 1978 boreholes.

Additional subsurface information was available from a series of boreholes drilled within the Haney Slide by CP&D. CPT soundings were carried out in the Haney Slide area by UBC personnel, including one sounding (CPT-UBC5) that was carried out at the top of the bluffs directly west of the Haney Slide (location indicated on Figures 1 and 2).

2.2.1 General Stratigraphy

A stratigraphic profile parallel to the bluffs, as well as cross-sections perpendicular to the bluffs at six different locations, were provided in our 1979 report (Appendix I).

In general, the bluffs are comprised of firm to stiff silty clay (Haney Clay) interlayered with fine sand to silty sand. The thickness of the interbedded sandy layers varies from lenses that are 1 to 5 mm thick, to layers that are several metres in thickness. The degree of interlayering varies with depth and from east to west along the bluffs, but in general, the greatest amount of sand is encountered within the upper 17 to 19 m, particularly at the east end of the bluffs near the Haney Slide and at the west end of the bluffs near the Port Hammond slide. Very dense silty sand and gravel was encountered at 82 m depth at BH101 (Golder 1979) at the east end of the bluffs near the Haney Slide, but was not encountered in any of the other borings located further west.

Based on available Standard Penetration Test (SPT) and CPT data, the relative density of the sand to silty sand layers ranges from loose to dense but is generally compact. Davies (1985) reported that dense gravelly layers were encountered at depths of between 3 and 4 metres during attempts to carry out CPT soundings along Cliff Avenue north of the Haney Slide.

Based on available field vane shear test and CPT data, the sensitivity of the Haney Clay (the ratio of peak undrained strength to fully remoulded undrained strength) appears to typically range from about 3 to 6, indicating a moderate sensitivity to strength loss during undrained shearing. However, sensitivities as high as 10 have been measured.

2.3 Laboratory Testing

Detailed descriptions of the properties of the soils encountered during the 1978 Golder investigation are provided in Appendix I of our 1979 report to MoE.

During this study, a database of index properties for Haney Clay was compiled from available results of moisture content and Atterberg limits determinations. The Atterberg limits results indicate that Haney Clay can be classified as a silty clay with medium to high plasticity. Typical ranges (mean +/- one standard deviation) as well as mean values for the various index properties are listed in Table 1. It should be noted that some samples from above 13 m elevation had moisture contents and liquid limits that were significantly higher than the typical ranges. Based on the average moisture content in Table 1, a unit weight for Haney Clay of 18 kN/m^3 was selected for our analyses.

Table 1
Summary of Index Properties for Haney Clay

	Typical Range (Mean +/- 1 Std Dev)	**Mean**
Natural Water Content (w)	30 – 46 percent	38 percent
Liquid Limit (LL)	39 – 61 percent	50 percent
Plastic Limit (PL)	18 – 28 percent	23 percent
Plasticity Index (I_p = LL-PL)	20 - 36	28
Liquidity Index (w-PL)/(LL-PL)	0.33 – 0.85	0.59

The drained shear strength parameters for Haney Clay have been measured in consolidated undrained triaxial compression tests in a number of different studies, including advanced tests carried out by the University of British Columbia (UBC) using block samples of Haney clay obtained from the area of the old brick factory (located close to the intersection of 225^{th} Street and River Road). The available shear strength parameters (effective friction angle – ϕ’ and effective cohesion - c’) from the various studies are listed in Table 2.

Based on the data in Table 2, a range of ϕ’ from 28 to 32 degrees (best estimate of 30 degrees) and a range of c’ from 0 to 10 kPa (best estimate of 5 kPa) were used to represent the strength of the silty clay layers in our stability analyses.

Table 2
Summary of Drained Shear Strength Parameters for Haney Clay

Source	Index Properties		Drained Strength Parameters		Test Description
	LL	I_p	ϕ' (deg)	c' (kPa)	
Campanella & Gupta (1969)	45	20	31.5	7	Triaxial
Campanella & Vaid (1974)	44	18	31.5 29.0 31.5	0 0 0	NC-I Triaxial, NC-K_o Triaxial NC-K_o Plane Strain
Vaid & Campanella (1977)	44	18	31.0	0	NC-I Triaxial
Cook Pickering & Doyle (1977)	53	28	28.0	5	Triaxial
Golder Associates (1979)	100 60	67 35	28.0 32.0	0 7	Triaxial

Note: NC = normally consolidated, I = isotropically consolidated, K_o = consolidated with vertical stress greater than horizontal stress

2.4 Piezometer Monitoring

In 1982, tri-level nested standpipe piezometers were installed by Golder at 10 locations behind the slope crest, and twin-level nested standpipe piezometers were installed at 5 locations along the C.P. Railway bench (locations indicated on Figures 1 and 2). Limited stratigraphic information was acquired during drilling for the piezometer installations in 1982. The standpipe piezometer filter zones were typically sealed within or intersected sandy layers.

Regular monitoring of the groundwater levels within the 1982 piezometers was carried out by staff of the District of Maple Ridge following installation in December 1982 until April 1983, and then on a monthly to semi-monthly basis from April 1983 to April 1984, followed by periodic measurements since then.

The piezometer data acquired between 1982 and 1986, along with precipitation data over the same period, led to the following conclusions in our 1986 report:

- The piezometric pressures in the area underlying the uplands and slopes are considerably lower than the hydrostatic pressure, indicating a strong downward gradient. Piezometric level fluctuations of up to 1.0 m in the deeper piezometers and up to 2.5 m in the shallower piezometers have been observed, which appear to be in response to seasonal fluctuations in precipitation.

- The piezometric pressures beneath the C.P. Rail bench increase hydrostatically with depth, and fluctuations of up to 2.3 m have been observed. The piezometric levels appear to be controlled by C.P. Rail drainage measures, river levels and precipitation.

- Since there have been several periods of heavy precipitation and snow melt during the monitoring period, the range of piezometric levels in the future (due to future precipitation patterns) is unlikely to vary significantly from that recorded during that period.

Additional piezometer monitoring data from January 1992 to May 1999 and from January to March, 2003 was provided by the District of Maple Ridge. This includes data from six of the ten 1982 piezometer locations behind the crest of the bluffs, as well as two of the five 1978 piezometers. No data after 1985 was available from the piezometers along the CP Rail bench.

2.4.1 Data Review

The available piezometer monitoring data from 1983 to 2003 was compiled and, based on this data as well as piezometer tip soundings that were carried out by District staff on February 27, 2003, the apparent status of each of the piezometers was assessed and is documented in Table IV-1 in Appendix IV. No field performance tests to confirm the continuing functionality of the piezometers were carried out as part of this study.

Plots of groundwater elevation with time for each of the piezometers for which the long-term monitoring data were available are provided in Appendix IV. The elevations are based on reported ground surface elevations at each piezometer location, which should be considered as approximate since it does not appear that surveying was carried out. The reported piezometer tip elevations and sounding elevations are also plotted on these piezometer data plots, and are included in Table IV-2. Representative “average” groundwater elevations for each of the piezometers that were used for our analyses during this study, are also included in Table IV-2. Readings that are clearly anomalous or that suggest that the piezometers were dry or had become flooded by surface water were not included in determining these “average” values.

The increase in groundwater pressure with depth between nested piezometers at different locations north of the crest of the bluffs is, on average, only about 50% of hydrostratic conditions (typical range from 30% to 70%). Below the CP Rail bench, the increase in groundwater pressure with depth is about 80% to 90% of hydrostatic conditions.

2.4.2 Assessment of Groundwater Conditions

The available data indicates that there has not been any significant changes in the background groundwater pressure regime at the locations of the actively monitored piezometers over the course of the 20-year monitoring period. Based on this information, there appears to be little justification for ongoing monitoring at frequent intervals. However, it is suggested that occasional further monitoring of functional instrumentation be considered following record wet weather over an extended period of time (eg. if monthly precipitation for two consecutive months is at least 90 percent of the maximum on record). As a guideline, weather conditions which would trigger monitoring include those causing flooding impacts in nearby low-lying areas or causing an unusual number of landslides along slopes throughout the District of Maple Ridge. The District could consult with a qualified geotechnical engineer to determine if additional monitoring is required in the event of any unusually wet season.

While the groundwater levels within the shallow piezometers (located above 20 m elevation) appear to fluctuate to varying degrees, the groundwater levels in the deeper piezometers appear to be relatively stable (ignoring data that suggests that the piezometers periodically dry out, which could also be due to conductivity problems with the water level meter). This suggests that the water pressure within the deeper granular strata is not affected significantly by variations in precipitation.

It was not possible to assess the long-term variation in groundwater pressures at locations south of the crest of the bluffs, since no monitoring data was available after 1985 for the piezometers along the C.P. Rail bench.

2.5 Topographical Data and River Surveys

Recent topographical information for the project area above river level was provided by the District of Maple Ridge. The topographical contours are included on Figure 1.

Information on the topography of the river banks south of the CP Rail bench was available from periodic soundings of the river channel which were performed by MoE along a series of 29 cross-sections located from east of the Haney Bypass to west of the Port Hammond slide. In order to assess the potential for erosion of the lower slopes of the bluffs, surveys were carried out along some or all of the cross-sections at the following times:

- April 1978,
- July 1981,
- August 1985,
- July 1986,
- October 1992,
- July 1997, and finally
- September 1997.

We understand from speaking with MoE staff that the erosion monitoring program was terminated following the September 1997 survey.

The July 1997 river survey data along the cross-sections was used to establish approximate river bank/bed contours between the survey lines using contouring software, which are plotted on Figure 1 (contours below 2 m elevation).

A report prepared for Golder by Northwest Hydraulic Consultants Ltd. (NHC), dated February 1986, suggested that erosion of the north bank of the river in the order of up to 10 to 15 m could occur during or shortly after a major flood event (e.g. a return period in excess of 10 years). The available monitoring data at the time that their report was written was limited to the three surveys between 1978 and 1985, but the largest freshet during the monitoring period only had a return period of 3 years. In their report, NHC recommended that repeat soundings be taken along the same survey lines if any river flows in excess of the 10-year return period event were to occur.

Available Fraser River flow data from the Water Survey of Canada Station at Hope (No. 08MF005) indicates that the annual peak flows at Hope in 1999 and 1997 equaled or exceeded 10-year return period events (based on flow data from 1950 to 2002). A histogram showing the annual peak flows compared to the 10-year return period peak flow is included in Appendix V.

The 1997 river surveys would have captured the erosion effects from the peak river flow during the 1997 freshet, which exceeded the 10-year return period event. A comparison of the slope profiles generated from the 1997 data (July and September soundings) with previous surveys for seven representative sections (Sections 5, 7, 9, 12, 15, 18, and 21) did not reveal any evidence of significant erosion of the river bank in general following the 1997 freshet, as the 1997 profiles were within the range of variation of previous surveys. For each of the representative sections, the profiles of the north bank of the river which were generated using the river survey data between 1978 and 1997, are included in Appendix V.

Considering the 20 year record, it appears that erosion of the submerged river banks may not be as much of a concern as was initially anticipated. Nevertheless, it would be prudent to survey the river in this area following extreme freshet events, in order to determine if significant erosion is caused by river flows significantly greater than that which has occurred during the monitoring period considered in this study. While major river bank erosion would tend to reduce the static factor of safety of the escarpment slopes, it is expected that localized failures of the river banks below the CP Rail tracks would occur before any movement of the escarpment slopes would be triggered. Regular monitoring of track conditions carried out by CP Rail as part of their normal operations will assist in identifying localized bank failures which could be indicators of erosion.

The District of Maple Ridge could consider establishing a Memorandum of Understanding with CP Rail, the Ministry of Environment and any other concerned parties, which would:

- set out protocols for communication between the concerned parties in the event that evidence of slope instability and/or river bank erosion is observed by the District of Maple Ridge or CP Rail, and

- establish procedures and possible cost-sharing arrangements to have a new river survey carried out along previous MoE survey lines and to re-evaluate the slope stability, in the event of a major river flow event that is believed to have caused bank erosion.

2.6 Historical Landslide Activity

A description of historical landslide activity along the Fraser River bluffs was provided in our 1979 report (Appendix I). Minor surficial failures, typically caused by localized flow slides due to seepage zones exiting from sandy layers on the slope face, have frequently occurred along the upper slope of the bluffs above the CP Rail bench. Shallow failures of the river bank south of the CP Rail line have also occurred periodically. However, it is the four major landslides that have occurred along the bluffs that are of particular significance to the assessment of future major instability in the area, and so a description of the four major slide features is also provided below.

2.6.1 Fir Street Slide and Minor Port Hammond Slide

The Fir Street and Minor Port Hammond Slides both extended about 60 m back from the crest of the bluffs and involved a length of at least 150 m. These slides are believed to be at least 50 to 75 years old (Golder, 1979).

Borings drilled near the backscarps of these slides in 1978 revealed that the stratigraphy at both locations consisted primarily of silty clay. Occasional sand seams up to 3 m in thickness were encountered near the Fir Street Slide.

2.6.2 Haney Slide

The Haney Slide occurred on January 30, 1880, and was well-documented by newspaper articles due to its size and destructive nature. The slide occurred suddenly, and the material that moved into the river caused a wave up to 20 m high that inundated the south shore of the river, killing one man, destroying boats, bridges and buildings along the river, and causing river levels to rise 6 m. Approximately 2/3 of the width of the Fraser River was blocked by the slide debris.

Observations of the slide area made on February 3, 1880, 4 days after the slide, were reported in a newspaper article (Victoria Daily Colonist, Feb. 5, 1880) which described the backscarp of the failure as consisting of 12 feet (3.7 m) of yellow-red earth, overlying 20 feet (6.1 m) of blue clay, underlain by gravel through which "coursing" water was observed. Large tension cracks were observed up to 150 feet back from the crest of the backscarp. The northern extent of the collapse zone as of February 3, 1880 was not indicated.

Meteorological data indicates that 635 mm (25 in.) of snow fell between January 6 and 12, 1880, when the temperatures remained below freezing (Golder 1979). This was followed by several days of higher temperatures and rain. The total precipitation during January 1880 was 45% higher than modern averages. According to historical records, no earthquakes large enough to have impacted the site had occurred at the time of, or within several years prior to, the Haney Slide.

Subsurface investigations by Cook Pickering & Doyle (1977) and UBC (1984) within the Haney Slide area north of the CP Rail line encountered a great deal of variability in the stratigraphy of the failure mass at different test hole locations. At a number of drill hole locations in the Cook Pickering & Doyle investigation, sand to sandy gravel was encountered to depths in excess of 15 m below the surface of the slide debris. Silty sand to sand up to 17 m thick was encountered behind the west scarp of the slide, south of River Road. Gravelly soil was encountered at depths of 3 to 4 m behind the north scarp of the slide south of Cliff Avenue during the CPT investigation by UBC.

In the UBC study (Davies, 1985), zones of highly disturbed silty clay were identified from changes in measured cone parameters at certain depths at a number of the CPT locations within the slide area, which were interpreted to be shear zones from the slide. The interpreted shear zones were located at depths that varied between 9 and 13 metres below the present ground surface, and were up to 1.1 metres thick. The relatively shallow depth of this shear zone suggests that the Haney Slide must have been retrogressive in nature for it to have extended on the order of 250 m back from the crest of the bluffs.

The triggering mechanism for the Haney Slide is not known for certain, but was likely caused by an increase in groundwater pressures due to infiltration of rain and meltwater from the relatively thick snow cover. The ability for the granular soils to drain to the face of the bluffs or to nearby ravine slopes may have been impeded by frozen ground behind slope faces. This combined with the relative high permeability of the upper sandy/gravelly soils in the slide area could have allowed groundwater levels to rise quickly. Furthermore, the overall stability of the bluffs in the Haney Slide area prior to January 1880 may have been marginal due to erosion of the submerged slopes by river flows, which seems likely given the location of the slide area on the outside of a bend in the river.

The reason that the Haney Slide retrogressed so far from the crest of the bluffs is unknown. The presence of thick sequences of granular soils behind the backscarps of the Haney Slide and in the slide debris suggests that the retrogression distance is linked to the lateral extent and thickness of the granular soils, which were probably saturated at the time that the Haney Slide was triggered. The thickness of granular soils behind the smaller Fir Street and Minor Port Hammond slides is much more limited.

2.6.3 Port Hammond Slide

The major Port Hammond Slide extended approximately 300 m back from the crest of the bluffs, and based on the existing terrain, also appears to have been a retrogressive failure. The age of the Port Hammond Slide is unknown, but is assumed to have occurred at least 200 years ago (Golder, 1979). An extensive thickness of primarily silty sand was encountered to a depth of about 25 m in a boring located immediately east of the Port Hammond Slide. Like the Haney Slide, the presence of thick granular deposits may have contributed to the great extent of the Port Hammond Slide. The triggering mechanism for this major slide is not known, but extreme weather conditions and/or a major earthquake cannot be ruled out.

2.7 Previous Slope Stability Assessments

2.7.1 Fraser River Bluffs

The analyses of the stability of the Fraser River bluffs, which were carried out during our studies for MoE in 1979 and 1986, were based on two-dimensional limit equilibrium solutions that considered deep-seated failures which would extend at least 30 m back from the crest of the bluffs and could therefore affect existing structures. In the 1979 analyses, five different cross-sections located between the Haney and Port Hammond slides were analyzed to assess the potential variability in stability across the study area. The 1979 analyses considered both static stability and seismic stability using a design horizontal ground acceleration of 0.08g. In the 1986 analyses, the static stability of the most critical section from the 1979 analyses was reviewed using the improved data on piezometric levels within the slopes, which were acquired following the 1982 work. No seismic analyses were carried out during the 1986 study. Our stability analyses in 1979 and 1986 did not consider the surficial stability of the slopes, including the potential for seepage-induced sloughing failures along the upper slope face.

The various stability analyses included sensitivity analyses to determine the effects of variations in soil strength parameters, piezometric levels, river levels, and potential erosion of the lower slopes, on the calculated factor of safety. The main conclusions from these studies were as follows:

- The most critical factor of safety against a major deep-seated failure under static conditions is between 1.2 and 1.4, depending on the assumed strength parameters and piezometric pressures.

- A loss of material at the toe of the slope of about 15 horizontal metres would reduce the factor of safety by about 10%, to levels of concern.

- A factor of safety of 1.3 under static conditions would drop to 1.0 as a result of a horizontal ground acceleration of 0.08g in a seismic event.

The stability of the back-scarp of a major failure was analyzed and factors of safety of about 1.0 were calculated, assuming a worst-case scenario in which the original slide material would provide no support to the exposed back-scarp. This indicates that there is a potential for retrogression of deep-seated bank failures. However, the potential extent of such retrogressive failures could not be assessed analytically due to limited stratigraphic information and uncertainties about the run-out behaviour of the slide material. Therefore, it could only be assumed that such failures could retrogress to a similar extent as the Haney and Port Hammond slides (i.e. about 300 m back from the existing slope crest).

2.7.2 Haney Slide Area

A stability evaluation for the housing development that was constructed on the Haney Slide debris was carried out by Cook Pickering & Doyle in 1977. The conclusions in their report were that the overall long-term static stability of the slide debris was adequate for residential construction, provided that cuts and fills were kept to a minimum, that the back-scarp area of the 1880 slide was left untouched, and that no “sudden removal of a substantial portion of the old slide debris” south of the CP Rail line were to occur.

However, the CP&D study did not consider the effects of seismic accelerations on the stability of the slide debris or the backscarps.

3.0 SEISMIC HAZARDS

3.1 Seismic Risk

It is the current standard of practice to consider seismic events corresponding to a 10% probability of exceedence within a 50 year design life (i.e. an annual risk level of 0.21% or a 475-year return period) as the Design Basis Earthquake (DBE) for structures covered by the BC Building Code. Seismic events corresponding to a 40% probability of exceedence within a 50 year period (i.e. an annual risk level of 1.0% or a 100-year return period) are considered as the Operating Basis Earthquake (OBE). The use of seismic hazard predictions for these risk levels is considered reasonable since they are commensurate with: (a) the National Building Code of Canada (NBCC) and BC Building Code; (b) the approaches adopted by the Greater Vancouver Regional District (GVRD) and by several other utilities in the Western United States in establishing their performance goals.

Predictions of peak horizontal ground acceleration (PHGA) and peak horizontal ground velocity (PHGV) on "firm ground" in the study area were obtained for these different risk levels from the Pacific Geoscience Centre (PGC), and are included in Table 3. Their predictions of "firm ground" motion are based on the location of the project site relative to inferred seismic sources and attenuation relationships which have been incorporated into their seismicity model. "Firm ground", as defined in these models, generally includes bedrock or very dense pleistocene soils, such as glacial drift.

Table 3
"Firm Ground" Motion Predictions for Study Area

Annual Seismic Risk	**Peak Horizontal "Firm Ground" Motions** (ref: PGC, 2003)	
	PHGA (g)	**PHGV (m/s)**
1/100	0.083	0.071
1/200	0.123	0.110
1/475	0.201	0.190

Based on the inferred significant depth to "firm ground" in the project area (in excess of 80 m), it is expected that the intensity of ground shaking would amplify during upward propagation through the overlying sediments. For this study, in the absence of a detailed ground response analysis, it was assumed that the ground surface motions would be 50% greater than the "firm ground" motions given in Table 3. This assumed amplification factor is consistent with published relationships by Idriss (1990) and with recommendations by Task Force for Earthquake Design in the Fraser River Delta (1991).

3.2 Geotechnical Seismic Hazards

The primary geotechnical hazards associated with a seismic event within the study area are:

- direct impacts of ground motions on structures and facilities;
- permanent vertical and horizontal ground displacements, and in particular differential displacements, due to ground deformations;
- liquefaction of saturated granular soils, possibly causing lateral ground deformations, post-liquefaction subsidence, loss of bearing capacity, and lateral flow slides;
- slope failures;
- landslide-induced waves and temporary increases in river level up-stream due to channel blockage, possibly leading to flooding of low-lying areas depending on river levels prior to landslide event.

3.2.1 Slope Deformation and Failure

If the horizontal ground accelerations are high enough to reduce the factor of safety of the slope to 1.0 or less, horizontal ground displacements toward the free-face of the slope will tend to accumulate during seismic shaking. Vertical slumping of the top of the slope also tends to be associated with the horizontal displacements.

The ground displacements cause internal deformations (strain) to occur within the soil. If the resulting accumulation of strain is greater than the strain required to mobilize the maximum strength of the soil, a "post-yield" reduction in strength (called strain-softening) can occur in some soils under certain loading conditions. Such soils continue to lose strength with increasing strain until some residual strength is achieved. Sensitive fine-grained soils, such as Haney Clay, can have residual strengths that are a small fraction of their peak strength if sheared rapidly under undrained conditions.

If the soil that is supporting the slope is strained sufficiently so that its strength is reduced to the point where the calculated factor of safety of the slope drops below unity, without seismic shaking, large slope deformations or landslides could occur during or following the earthquake. Conversely, if soil strains are insufficient to reduce the calculated factor of safety below unity, only limited ground deformations would be expected.

As far as we are aware, investigation of the behaviour of the predominant soils in the study area under seismic shaking has not been carried out. Consequently, there is considerable uncertainty as to magnitude and effects of seismic ground deformations.

3.2.2 Liquefaction

During seismic shaking, pore water pressures are modified as the volume of the soil tries to change in response to horizontal shearing. In loose, saturated, granular soils and normally consolidated fine-grained soils, the pore water pressure tends to increase, which leads to a reduction in effective stress within the soil and a corresponding reduction in soil strength and stiffness (referred to as cyclic softening). Strictly speaking, liquefaction occurs if the pore pressure, which tends to accumulate with continued shaking, becomes high enough to reduce the effective stress in the soil to essentially zero, causing the soil to behave as a viscous fluid. The term "liquefaction" used hereafter will refer to the state in which significant cyclic softening has occurred, irrespective of whether a state of essentially zero effective stress has been achieved or not.

Soil that has "liquefied" is prone to large deformations when subjected to external forces such as gravity or seismic loads. Lateral spreading and/or flow slides can occur when liquefied soils are located under or near a slope. The shear strength of the liquefied soil under these deformation scenarios tends to be significantly lower than the peak strength of the soil under drained conditions.

Once shaking has stopped, the pore water will tend to drain as the soil consolidates to a denser state, which causes ground subsidence.

3.3 Risks Associated with Seismic Hazards

Within the study area, the geotechnical hazards resulting from a seismic event have the *potential* to cause one or more of the following consequences:

- damage to the local road network, including River Road which is classified as a Primary Road in the District of Maple Ridge road network and would be expected to provide emergency access to the area;
- damage to, or rupture of, buried utilities;
- damage to, or destruction of, houses and residential properties;
- houses could become unsafe for occupancy, temporarily or permanently (without costly mitigative measures or repairs);
- damage to CP Rail line and long-term disruption of this major transportation corridor;
- impedance to Fraser River flows and disruption of the waterway;
- injury or loss of life.

There is a probability associated with each of the above consequences occurring. This probability is the product of:

- the probability of an earthquake occurring with a certain intensity and duration of ground motion within a given time period (e.g. the DBE has an annual probability of 0.21%), and
- the probability that that particular earthquake will cause a particular type of hazard (such as a landslide) with a particular magnitude (surficial slumping vs. major retrogressive collapse), and
- the probability that any of the potential hazards will impact a particular building or property at a particular location, and will produce a particular consequence (e.g. a slope failure could damage a building, but may not cause personal injury).

Thus, while there is the potential for catastrophic consequences in the event of a major earthquake, the level of risk to an individual or facility may not be greater than many other hazards to which the public is routinely exposed. A comprehensive risk assessment would be required to quantify these risks, which is beyond the scope of this study.

4.0 SLOPE STABILITY ASSESSMENT

4.1 Methodology

Three different cross-sections (Sections A-A, B-B, C-C) perpendicular to the river bluffs were generated for this study using the updated topographical information described in Section 2.5. The locations of these sections, which are indicated on Figure 1, are very similar to Sections 12, 15 and 18, which were analyzed during our 1978/79 slope stability study (Golder, 1979). Sections A-A, B-B and C-C were selected because of the availability of stratigraphic information and piezometer data at these locations. Static, seismic and post-seismic conditions were considered in our analyses. As for previous studies, a low river level, corresponding to 0 m elevation, was assumed for all our analyses, which represents a worst-case scenario from a slope stability perspective.

All stability analyses were carried out using the computer program SLOPE/W, which uses limit equilibrium solutions to compute the factor of safety (FoS) for many different potential failure surfaces. The FoS is the ratio of the total moments tending to cause failure divided by the total moments resisting failure. A FoS of 1.0 would indicate an unstable slope. The Morgenstern-Price method of analysis (Morgenstern & Price, 1965), which considers both moment and force equilibrium, was used in this study.

4.1.1 Static Stability Analyses

The static stability of each of the three cross-sections was analyzed using drained shear strength parameters (effective friction angle – ϕ' and effective cohesion - c'), and using a groundwater pressure regime that is based on interpolation between different piezometer locations where measurements of groundwater levels were available.

Representative "average" groundwater elevations for each of the piezometers used in our analyses are included in Table IV-1 in Appendix IV. The elevation of the phreatic surface at each of the piezometer locations was inferred by using calculated vertical gradients between the shallow and intermediate piezometers to extrapolate the groundwater pressure at the top of the fine-grained layer underlying any surficial granular layers, and assuming hydrostatic conditions within surficial granular layers. The shape of the phreatic surface between piezometer locations was assumed based on our best judgement.

The minimum FoS under static conditions for potential deep-seated failure surfaces that exit the slope below the CP Rail bench and extend at least 10 m back from the crest of the bluffs were computed using "best-estimate" soil strengths, as well as high strength and low strength estimates corresponding to expected upper and lower bounds. These critical failure surfaces, along with best-estimate soil parameters and assumed phreatic surface, for Sections A-A, B-B and C-C are presented on Figures 3a, 3b and 3c, respectively.

4.1.2 Seismic Slope Stability Analyses

The main objective of the seismic analyses was to evaluate the variation in estimated ground displacement with distance from the crest of the bluffs due to the peak horizontal ground acceleration associated with the 100-year and 475-year events. Estimates of horizontal ground displacement (d) were calculated using the following equation derived by Newmark (1965):

$$d = \frac{V^2}{2gk_y}\left(1 - \frac{k_y}{A}\right)\frac{A}{k_y} \qquad (1)$$

where: V = peak horizontal ground velocity,
g = acceleration due to gravity,
k_y = seismic coefficient required to cause yielding of the slope (i.e. required to reduce the FoS to 1.0),
A = seismic coefficient corresponding to peak horizontal ground acceleration in a given seismic event

In the seismic stability analyses, which were carried out for each of Sections A-A, B-B and C-C, we applied seismic coefficients (k) corresponding to different horizontal ground accelerations, and for each level of shaking we searched for the potential failure surface having FoS = 1.0 which extended the maximum distance back from the crest of slope (i.e. the maximum horizontal extent of slope yielding). The same best-estimate drained shear strength parameters and piezometric conditions that were used in the static analyses were used in the seismic analyses.

Using the relationship between k_y and maximum distance of yielding behind the slope crest, profiles of horizontal ground displacement versus distance behind the slope crest were generated for each of the cross-sections using Equation 1.

4.1.3 Post-Seismic Stability of Failure Backscarps

Assuming an initial slope failure is caused by the seismic shaking, the post-seismic stability of an assumed backscarp configuration (without seismic forces applied) was also analyzed for each of Sections A-A, B-B and C-C. The backscarp configurations were generated by assuming that seismic shaking would cause an initial failure extending 50 m back from the existing crest of slope, and that the slide mass would slump down enough to expose a minimum backscarp toe elevation of 0 m. Pre-failure piezometric conditions were maintained for our analyses to check the critical short-term stability immediately after initial collapse, before drainage occurs and the piezometric conditions adjust to the new slope geometry.

4.2 Results of Slope Stability Analyses

4.2.1 Static Conditions

The minimum FoS for deep-seated failures that would extend at least 10 m back from the crest of the bluffs are summarized in Table 4. Using best-estimate strength parameters, the minimum FoS is about 1.2 to 1.3, which indicates that the slopes are only marginally stable under static conditions. If the FoS were to drop below 1.1 (due to increased groundwater levels for example), some slope deformation could be anticipated. The static FoS for failures extending further back from the crest of slope increases with increasing distance from the crest.

These results are in general agreement with the static FoS determined during our 1979 study, which were based on more adverse piezometric conditions and deep-seated failures extending at least 30 m back from the crest of slope. The monitoring carried out since the previous studies has provided greater confidence regarding the variations in groundwater levels and river scour.

Table 4
Computed Minimum Factors of Safety under Static Conditions

Cross-Section	Minimum Factor of Safety		
	Lower Bound Strength Estimates	Best-Estimate Strengths	Upper Bound Strength Estimates
A-A	1.10	1.19	1.34
B-B	1.09	1.27	1.44
C-C	1.01	1.19	1.37

4.2.2 Seismic Conditions

The maximum horizontal extent of slope yielding (distance behind the slope of crest) that were computed for different levels of horizontal ground acceleration for each of the three cross-sections are listed in Table 5. Estimates of horizontal ground displacement for the 475-year and 100-year seismic events are plotted against distance behind the crest of slope on Figures 4a and 4b, respectively.

Table 5
Maximum Horizontal Extents of Slope Yielding due to Seismic Acceleration

Horizontal Ground Acceleration	Maximum Distance (m) of Yielding Behind Slope Crest		
	Section A-A	Section B-B	Section C-C
0.06g	10	No Yielding (FoS > 1.0)	9
0.075g	24	13	20
0.10g	39	34	32
0.125g	50	45	45
0.15g	70	70	60
0.20g	97	118	108
0.25g	127	164	152
0.30g	160	212	203

The threshold displacement at which straining would reduce the soil strength sufficiently to cause a general slope failure requires further field and laboratory investigation, and cannot be determined from the available information. Given the relatively low static safety factors, we believe that horizontal displacements of 50 mm or more may be sufficient to present a high risk of slope failures. However, given the lack of data on the strain-softening characteristics of the Haney Clay, the potential for instability should not be ruled out for horizontal displacements as low as 25 mm without more detailed study.

Therefore, based on the displacement vs. distance from crest profiles on Figure 4a, it is conceivable that slope instability could be triggered by the **475-year** seismic event within distances on the order of 80 m back from the crest of the bluffs; within distances of about 50 m from the crest of the bluffs the risk of instability could be qualitatively described as "high". Based on the displacement vs distance profiles on Figure 4b, there is a "low" risk of a deep-seated failure occurring due to the **100-year** seismic event, although surficial slides could be triggered.

4.2.3 Retrogression Potential

If the seismic shaking were to induce a slope failure extending to a distance in the order of 50 m back from the existing crest of the bluffs, saturated conditions within the face of the backscarp could be expected below a depth of about 5 m or less, based on the piezometric conditions interpreted from the piezometer data.

Under these conditions, factors of safety of 1.0 or less were calculated for potential rotational failure surfaces extending to distances of 25 to 30 metres back from the exposed back-scarp of the initial failure zone (for backscarp heights of 26 to 33 metres). This should be considered to be a minimum retrogression distance in the event of a major landslide. Additional instability could be expected until the accumulation of material in front of the retrogressing backscarp is adequate to buttress the slope.

Where saturated granular layers are exposed in the backscarp, sloughing of the granular soil due to groundwater outflow (called seepage erosion), along with spalling of undermined fine-grained soils, could also lead to retrogression of portions of the slope located above exposed granular layers.

If significant thickness of soil were to liquefy during the earthquake, flowslides could occur which could lead to rapid losses of extensive areas of ground behind the initial backscarp. The liquefaction susceptibility of the soils in the study area is discussed further in Section 5.

The maximum extent of retrogression that can be expected following a major seismic event will depend on the ability for the slide material to accumulate in front of the retrogressing backscarp. Where the backscarp is composed entirely of clayey soils, landslides are expected to comprise slumping material that is unlikely to travel large distances horizontally and therefore will expose less backscarp and provide more buttressing. However, where saturated granular layers exist, the landslide mass would be more fluid and would tend to flow further from the backscarp rather than accumulate next to it. This is one postulated mechanism for the large extents of the major Haney and Port Hammond Slides. Therefore, the extent of retrogression is expected to be highly dependent on the thickness and lateral extent of the saturated granular layers within the study area. Since this has not yet been well defined, the maximum extent of retrogression under seismic conditions cannot be realistically assessed in this study. In the absence of better stratigraphic information and seismic performance data, retrogression back to a distance of 300 m behind the crest of the bluffs (similar to the major Haney and Port Hammond Slides) remains a possibility.

5.0 LIQUEFACTION SUSCEPTIBILITY

The available data within the study area was not adequate to carry out a rigourous assessment of liquefaction susceptibility or to delineate liquefiable zones. However, a limited assessment was carried out in this study using the available data. The methodology and results of this assessment are described below.

5.1 Methodology

5.1.1 Granular Soils

The ratio of the horizontal cyclic shear stress, τ_{cy}, induced within the soil by the ground shaking, to the initial vertical effective stress, σ'_{vo}, which acts normal to the horizontal plane of shearing, is called the Cyclic Stress Ratio (CSR = τ_{cy}/σ'_{vo}). The liquefaction susceptibility of a soil in a seismic event is commonly evaluated by comparing the liquefaction resistance of the soil to the CSR applied during the earthquake. The liquefaction resistance of the soil is quantified using the Cyclic Resistance Ratio (CRR), which is the CSR that is required to cause liquefaction in a given seismic event. If the CRR of the soil is higher than the CSR generated by the earthquake, liquefaction does not occur.

The CSR profiles associated with the 475-year earthquake ($PHGA_{surface}$ = 0.30g) were calculated using the following equation, based on the Seed and Idriss (1971) simplified approach:

$$CSR = \tau_{avg}/\sigma'_{vo} = 0.65(PHGA_{surface}/g)(\sigma_{vo}/\sigma'_{vo})\cdot r_d \qquad (2)$$

where σ_{vo} and σ'_{vo} are the total and effective overburden pressures, respectively, and r_d is a stress reduction factor to account for soil flexibility. The mean r_d versus depth profile developed by Seed and Idriss (1971) was used to generate the CSR profile for this study.

The CPT data from CPT-UBC5 (approximate location shown on Figure 1) was used to evaluate the typical liquefaction resistance of the granular soils in the study area. Plots of the various cone parameters for CPT-UBC5 are included in Appendix VI. The relationship between CRR for magnitude 7.5 earthquakes ($CRR_{7.5}$) and normalized cone tip resistance in clean sands, $(q_{c1N})_{cs}$, established by the 1996 NCEER Workshop was used to derive CRR values from the CPT-UBC5 data. The measured tip resistances were corrected for apparent fines content and the calculated $CRR_{7.5}$ values were corrected to an earthquake magnitude of 7.0 (applicable to the 475-year event) and specific overburden stress levels, according to the procedures established by the 1996 NCEER Workshop.

5.1.2 Fine-Grained Soils

The “Chinese” criteria for silts and clays (Marcuson et al., 1990) were used to evaluate the liquefaction susceptibility of the Haney Clay. According to this set of criteria, fine-grained soils are susceptible to liquefaction if all of the following 3 conditions are met:

- < 15% finer than 0.005 mm,
- liquid limit < 35%, and
- water content > 90% of liquid limit

5.2 Results of Liquefaction Assessment

Profiles of applied CSR for the 475-year seismic event and CRR for the granular soils at CPT-UBC5 are compared on Figure 5. The computed CRR for the soil is less than the estimated CSR due to the 475-year seismic event over a 5 m thickness of compact sand to silty sand between 5.5 and 13 metres depth. Within the potentially liquefiable zones, relative densities of 35 to 50 percent and equivalent $(N1)_{60}$ values of 10 to 18 blows/0.3 m were interpreted from the CPT data.

SPT N values recorded within the silty sand between 10.5 and 16 metres depth at nearby BH101 ranged from 11 to 17 blows/0.3 m. Roughly half of the SPT N values measured at other borehole locations within the study area are less than 18 blows/0.3 m. This suggests that the potential for liquefaction of the granular layers is not limited to the location of CPT-UBC5 alone.

Based on the measured index properties of the Haney Clay, it would not be classified as potentially liquefiable, according to the “Chinese” criteria.

6.0 POTENTIAL IMPACTS OF SEISMIC HAZARDS ON EXISTING PROPERTIES AND INFRASTRUCTURE

It should be noted that the following preliminary assessments are based on very limited subsurface information and analyses, and are subject to change based on consideration of new subsurface information. In particular, ground subsidence and differential settlements due to liquefaction of saturated granular zones may be possible and could result in damage to structures and utilities, although the risk of major damage due to these effects within the study area is considered to be low compared to the overall slope hazards

6.1 Properties and Infrastructure Within 100 Metres of Crest of Bluffs

Based on the results of this study, properties and infrastructure located within a distance of at least 100 m behind the crest of the bluffs should be considered as being at *high risk* of being seriously damaged due to slope deformation or failure *in the event of the Design Basis Earthquake* (annual probability of occurrence of 1/475). This area includes most of River Road between 216th Street and River Bend, including a portion of the 500 mm diameter concrete sanitary forcemain, as well as numerous houses and properties, and the CP Rail lines.

Secondary impacts from one or more major landslides into the Fraser River include damage to low-lying properties and infrastructure due to wave impact and/or flooding, navigational impedance along the river, and environmental impacts.

6.2 Properties and Infrastructure Beyond 300 Metres from Crest of Bluffs

Based on the limits of retrogression of the major historical landslides in the areas, properties and infrastructure located beyond a distance of roughly 300 m behind the crest of the bluffs are presently considered to be at *low risk* of being damaged *due to slope movements,* except in the vicinity of the backscarps of the Haney Slide and Port Hammond Slide. Near these features, higher risk areas may extend beyond 300 m from the crest of the bluffs. The potential for instability of the backscarps of these major slide features requires site-specific investigation and analysis, and was not assessed in this present study.

6.3 Properties and Infrastructure Between 100 and 300 Metres from Crest of Bluffs

Between a distance of about 100 and 300 m (or greater in the vicinity of the backscarps of the Haney and Port Hammond Slides) behind the crest of the bluffs, the level of risk of damage to properties and infrastructure is difficult to characterize based on the information available at the time of this study, but it is expected to vary between *high* and *low, in the event of the Design Basis Earthquake*. The risk of major damage to properties and infrastructure within this area in the event of the DBE will depend on:

- the proximity to local features such as the backscarps of the Port Hammond Slide and Haney Slide and the ravine between Wood Street and Anderson Place,
- the extents of initial landslides triggered by the earthquake, and
- retrogression distances, which will by highly dependent on the thickness, horizontal extents and density of saturated granular layers, as described in Section 4.2.3.

Some ground deformations, both horizontal and vertical, should be anticipated within this region due to slope movements and/or liquefaction of saturated granular zones. These ground deformations could cause damage to some structures and utilities, although the risk of major damage due to these effects within the study area is considered to be low.

6.4 Properties and Infrastructure Within Existing Slide Areas

No analyses of the stability of the Haney and Port Hammond Slide areas under seismic conditions were carried out in this study, and we are not aware of any seismic impact studies that have been carried out previously for these areas. Housing developments have been constructed on the Haney Slide debris and on the Port Hammond Slide debris at the foot of Best Street.

There is a possibility that seismic shaking during the 475-year earthquake could re-mobilize the slide mass along residual shear planes. While the re-mobilized mass may not travel a large distance, differential ground deformations could result which could damage buildings and utilities, as well as the CP Rail lines. Differential movements between the Haney Slide mass and the adjacent escarpment would pose a particular threat of damage to the 500 mm diameter concrete sanitary forcemain that is located along River Road and crosses the backscarp of the Haney Slide area between Carshill Street and River Bend.

There is also a possibility that slope failures could occur within the backscarps of the slide areas due to the seismic shaking, and the resulting landslides could impact buildings. If the failure zones involved saturated granular soils, such landslides could become flow slides which tend to be highly mobile and impact larger areas.

7.0 IMPACTS ON FUTURE DEVELOPMENT

7.1 Existing Development/Subdivision Policies

We understand that, in 1993, the District of Maple Ridge established development/subdivision policies for the region surrounding the Fraser River Escarpment (Policy Numbers 6.04 and 6.05), based on the results of the stability studies carried out by Golder between 1978 and 1986.

7.1.1 Policy No. 6.05

Policy No. 6.05 prohibits development/subdivision of land within 100 m of the crest of the Fraser River bluffs until such time as there has been a commitment by the Provincial and/or Federal Governments to install river erosion protection. We are not aware of any such commitment or actual implementation of erosion control measures along the Fraser River Escarpment by the Provincial or Federal Governments.

Furthermore, this policy requires that any proposed development within this area would require reports to be prepared by a qualified geotechnical engineer providing:

- recommendations for the design and construction of the development such that there would be no detrimental impacts to the stability of the adjacent slopes, and

- ensurance that sufficient construction inspection was provided to ensure that the construction is adequate to meet the geotechnical stability requirements.

Additional design and construction requirements are provided in Policy 6.05, including:

- drainage requirements minimizing discharge into the groundwater system or onto slopes,

- prohibition of fill placement or vegetation removal that could be detrimental to stability, and

- prohibition of all structures, slabs, pavements or impoundments (eg. swimming pools) within 10 m of the slope crest.

7.1.2 Policy No. 6.04

Policy No. 6.04 is applicable to the development/subdivision of land beyond 100 m from the crest of the Fraser River bluffs and extending as far north as 124th Avenue between 207th and 224th Streets. This policy provides surface drainage requirements and various restrictions on sources of groundwater recharge (including septic fields, landscape ponding and swimming pools) in order to reduce the potential for increases in groundwater levels that could cause slope instability.

7.2 Comments on Existing Policies

The District requested that we review and comment on the existing policies in light of the results of this preliminary seismic vulnerability study.

The requirements for surface drainage measures and restrictions on sources of groundwater recharge, which are specified in Policy No. 6.04, are prudent for the entire area covered by both Policy Numbers 6.04 & 6.05. Such measures will reduce the potential for slope instability triggered by rising groundwater levels, and could help to reduce the potential extent of instability triggered by a major earthquake.

Based on the preliminary assessment carried out to date, and the potential seismic impacts described in Section 6, we suggest that the following restrictions to new development/subdivision within the Fraser River Escarpment area be considered:

- **Within 100 metres of the crest of the Escarpment, from the east crest of the Port Hammond Slide backscarp to the west crest of the Haney Slide backscarp** (proposed 100 m Setback Zone shown on Figure 6), and within a distance (that has not yet been determined) from the crest of ravine slopes and the backscarps of existing slide areas:

  In order to avoid increasing the risk to human life above that which presently exists, subdivision of land which allows an increase in the population density in this area should not be permitted, unless the potential risks to the public can be quantified by more detailed investigation and analysis, and the calculated risk levels are considered to be acceptable.

  Development which involves replacement of existing structures with comparable new structures could be considered without increasing the risk to human life; however, this may increase the financial risk. Since the weight of single family residential structures is small compared to the overall potential failure mass, a larger structure in itself would not be expected to reduce the stability significantly, and might even enhance stability if more surface water is collected and diverted away from the slopes.

- **Within 300 metres from the crest of the Escarpment and outside of the 100 m Setback Zone,** and within a distance (that has not yet been determined) behind the crest of ravine slopes and the backscarps of existing slide areas, and within the existing slide areas:

New development be subject to: i) approval by the District based on the input of a qualified geotechnical engineer who must carry out subsurface investigations and stability analyses that are adequate to assess the potential retrogressive impact on the property in question of a failure of the bluffs (on or off the property in question), as well as any more proximate features such as ravine slopes or slide backscarps, on the new development; and ii) the requirements of Policy No. 6.04.

This area is identified on Figure 6, as being between the proposed 300 m Setback Line and the 100 m Setback Zone. It should be noted that the indicated setback from the crest of ravine slopes and backscarps of existing slide areas is a first approximation and is not based on any analysis.

- **Beyond 300 metres from the crest of the escarpment**, or a distance (that has not yet been determined) from the crest of ravine slopes and the backscarps of existing slide areas, whichever extends further from the crest of the escarpment:

  New development/subdivision be allowed to proceed, subject to the requirements of Policy No. 6.04.

The District may wish to delay establishing geographic boundaries for the various levels of development restrictions, if a more comprehensive study is being considered by the District to better define specific setback requirements.

8.0 RECOMMENDATIONS FOR FURTHER WORK

This study was carried out using available stratigraphic information and in-situ and laboratory test data. The number of available test holes within the study area is very limited and is not considered to be adequate to assess variations in hazard level across the study area, particularly given the present uncertainties about retrogression potential. It is also not adequate to carry out assessments for specific features within the study area.

Additional geotechnical investigation, laboratory testing, and possibly more sophisticated analyses, would be required to provide assessments of the following:

- the magnitudes and extents of ground deformations associated with instability of existing slide backscarps and ravine slopes;
- the potential retrogression extents of slope failures triggered by the earthquake; and
- the potential extents of liquefiable deposits and estimates of settlement and lateral displacements due to liquefaction.

In order to quantify the risks (in terms of probabilities and/or cost of losses) associated with specific consequences of the seismic hazards, and to compare these risk levels with other recognized risks (such as automobile travel), a comprehensive Risk Assessment Study would be required. Such a study would also aid District staff in assessing requirements for mitigation/response to the seismic hazards.

9.0 CLOSURE

We trust that this report provides sufficient information for your current requirements. Should you have any questions, or require further input, please do not hesitate to contact us.

Your very truly,
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The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. ***No other party may use or rely on this report or any portion thereof without Golder's express written consent. Golder will consent to any reasonable request by the Client to approve the use of this report by other parties as Approved Users.*** The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, and only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. ***In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use by any party of portions of the report without reference to the entire report.***

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs, techniques and equipment choice, scheduling and sequence of operations would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (CONTINUED)

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgement, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions

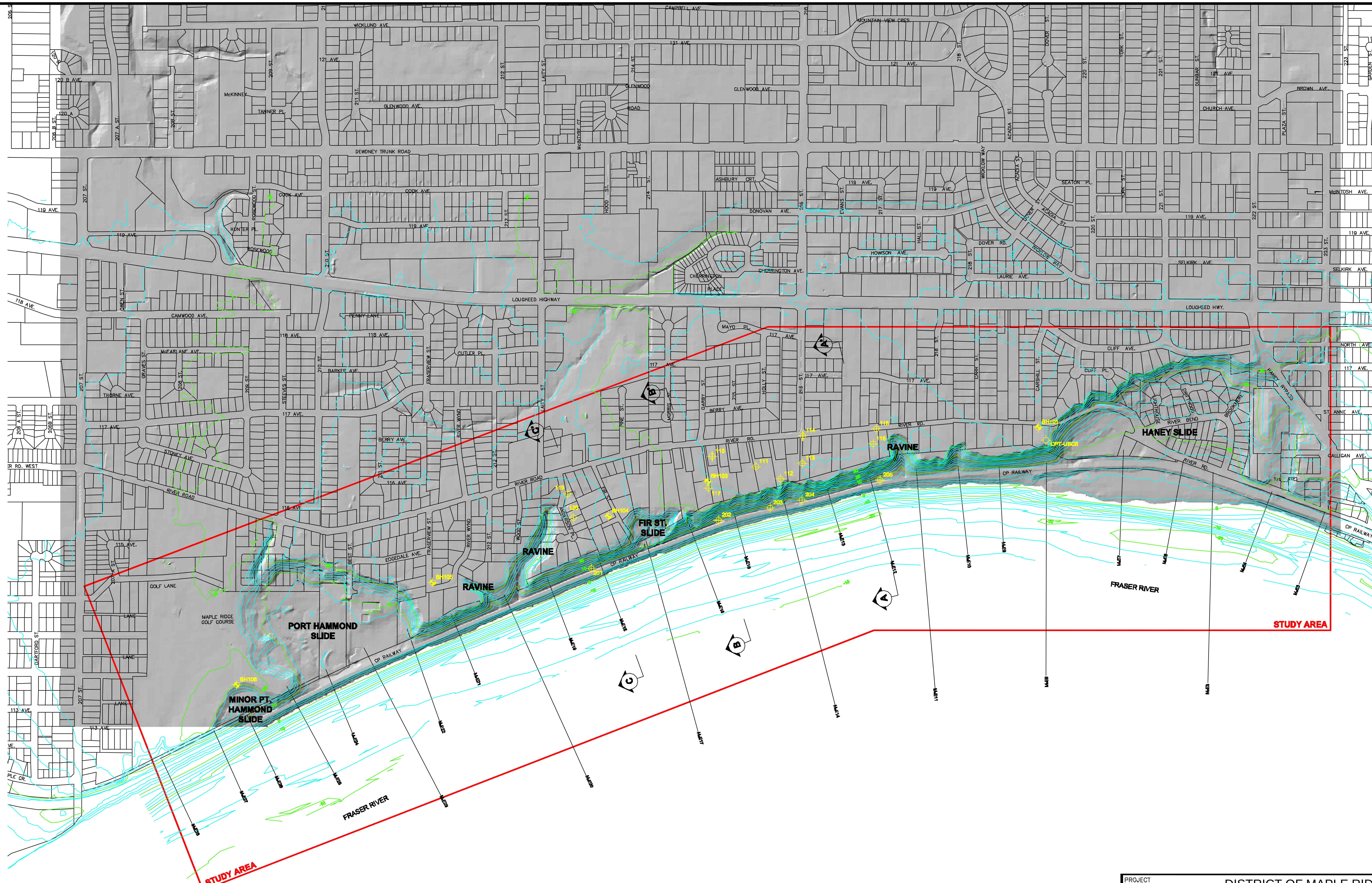
Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect certain conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between sampling points may differ from those that actually exist.

Groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their measurement. Groundwater conditions may vary between reported locations and can be affected by annual, seasonal and special meteorological conditions or tidal fluctuations. Groundwater conditions may also be altered by construction activity on or in the vicinity of the project site.

Sample Disposal: All contaminated samples and materials shall remain the property and responsibility of the Client for proper disposal. Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense.

Follow-Up and Construction Services: All details of the design and proposed construction may not be known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction is necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities.



REVISION DATE: 03/06/20 9:57am By: jkownacki CADD FILE: N:\Bur-Graphics\Projects\2003\1411\03-1411-018\Drafting\cad\P03-1411-018-03.dwg

LEGEND

- Boring by Golder (1978)
- Standpipe Piezometer Installations by Golder (1982)
- Cone Penetration Test by University of British Columbia (1984)
- River Survey Lines by BC Ministry of Environment (1978-1997)

NOTES

- All Test Hole Locations are Approximate.
- All Elevations in Meters, referenced to Geodetic Datum
- Elevation Contours above 0m based on District of Maple Ridge Plans (2000)
- Contours below 2m Generated from River Survey Data along MOE Survey Lines
- Neither the Corporation of the District of Maple Ridge nor Golder Associates Ltd. make any guarantee regarding the accuracy or present status of the information shown on this figure.

REFERENCE

Map Projection: UTM NAD83 Zone 10
Sources: District of Maple Ridge, Ministry of Environment Lands & Parks

0 50 100 150 200 250
Scale in Meters

PROJECT

DISTRICT OF MAPLE RIDGE
SEISMIC VULNERABILITY STUDY
FRASER RIVER ESCARPMENT, MAPLE RIDGE, B.C.

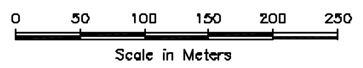
TITLE

TOPOGRAPHIC RELIEF WITHIN STUDY AREA

Golder Associates

PROJECT No.	03-1411-018	FILE No. P03-1411-018-03
DESIGN		SCALE AS SHOWN REV. -
CADD	JK 20JUN03	
CHECK		
REVIEW		

FIGURE 1



REVISION DATE: 03/06/20 8:57am By: jkownacki CADD FILE: N:\Bur-Gr...

LEGEND

- Boring by Golder (1978)
- Standpipe Piezometer Installations by Golder (1982)
- Cone Penetration Test by University of British Columbia (1984)
- River Survey Lines by BC Ministry of Environment (1978-1997)
- 1050 C — Ø1050mm Concrete Sanitary Gravity Main (GVS&DD)
- 450C F.M. — Ø450mm Concrete Sanitary Force Main
- 525 AC — Ø525mm Asbestos-Cement Sanitary Gravity Main

NOTES

- All Test Hole Locations are Approximate.
- Neither the Corporation of the District of Maple Ridge nor Golder Associates Ltd. make any guarantee regarding the accuracy or present status of the information shown on this figure.

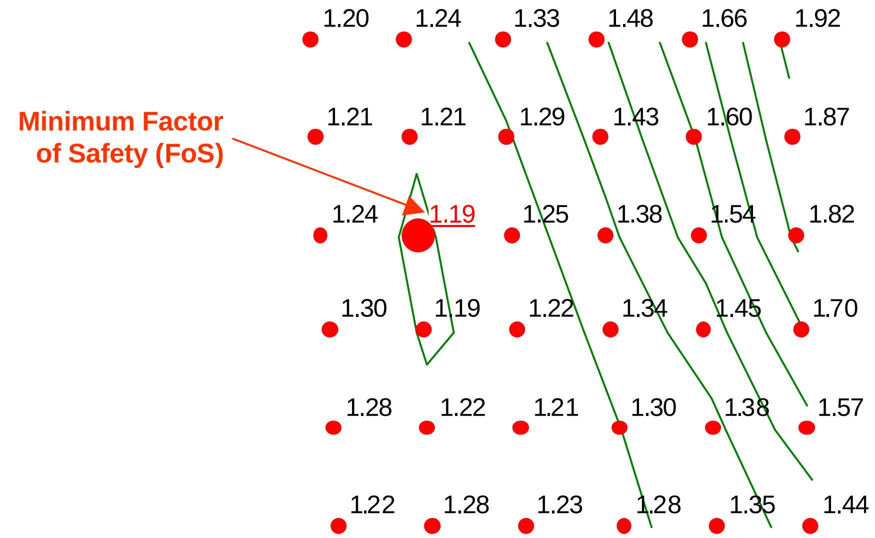
REFERENCE

Map Projection: UTM NAD83 Zone 10
Sources: District of Maple Ridge, Ministry of Environment Lands & Parks

PROJECT	DISTRICT OF MAPLE RIDGE SEISMIC VULNERABILITY STUDY FRASER RIVER ESCARPMENT, MAPLE RIDGE, B.C.
TITLE	**PLAN OF STUDY AREA**

PROJECT No. 03-1411-018			FILE No. P03-1411-018-01
DESIGN			SCALE AS SHOWN REV. –
CADD	JK	20JUN03	**FIGURE 2**
CHECK			
REVIEW			

Grid of Potential Failure Circle Centres



Best-Estimate Soil Parameters

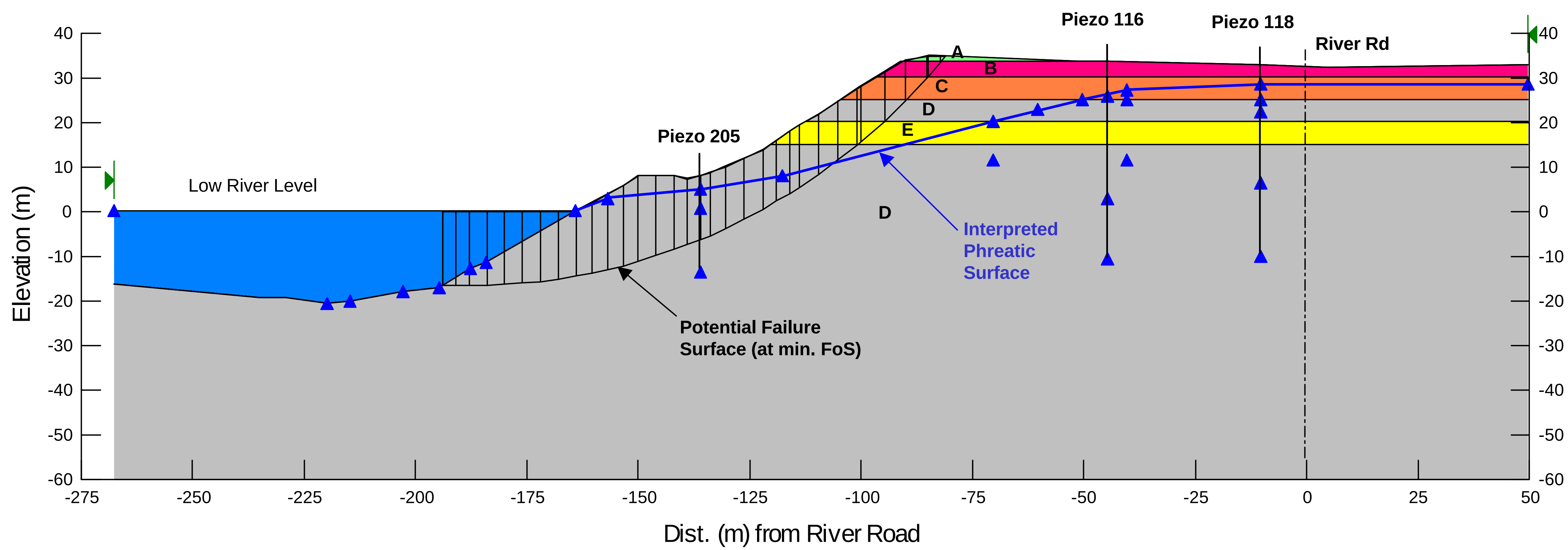
Soil A: Silty Clay Crust, c' = 10 kPa, ϕ' = 28º, γ = 16 kN/m³

Soil B: Sand & Gravel, c' = 0 kPa, ϕ' = 40º, γ = 19 kN/m³ (γ_{sat} = 21 kN/m³)

Soil C: Sand to Silty Sand, c' = 0 kPa, ϕ' = 35º, γ = 19 kN/m³ (γ_{sat} = 20 kN/m³)

Soil D: Silty Clay, c' = 5 kPa, ϕ' = 30º, γ = 18 kN/m³

Soil E: Silty Sand, c' = 0 kPa, ϕ' = 33º, γ = 18 kN/m³ (γ_{sat} = 19 kN/m³)



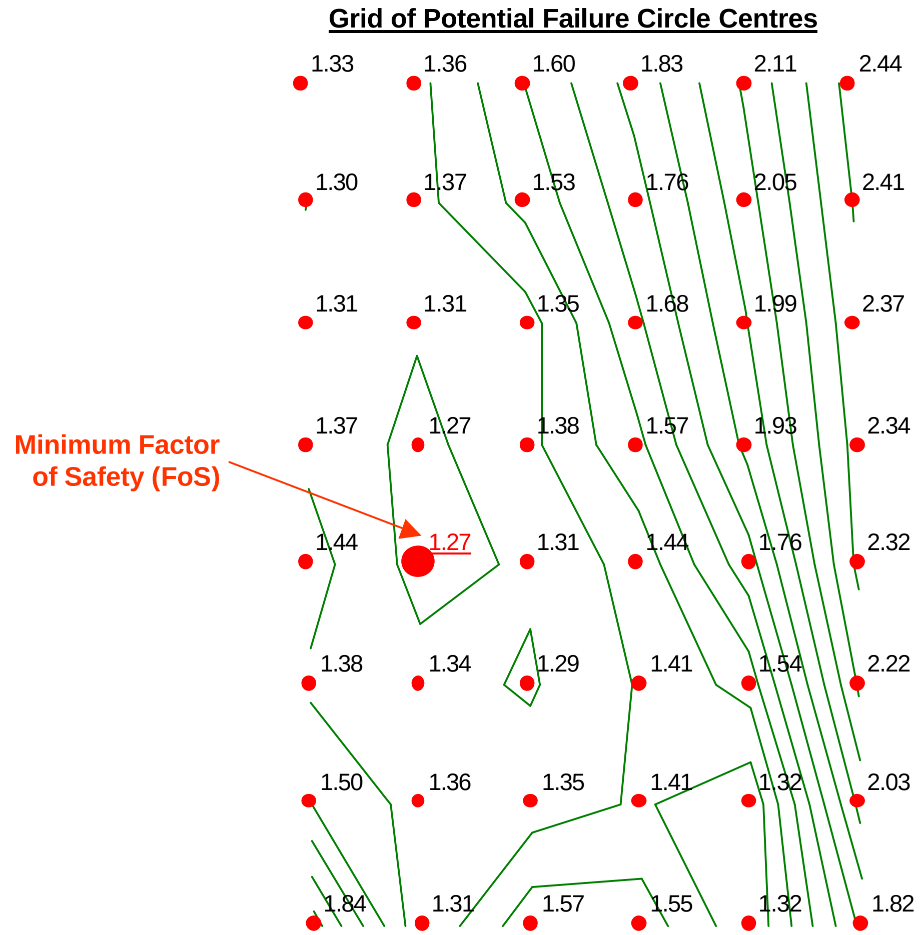
Note: Piezometer numbers referenced to Golder numbering system (for locations see Figures 1 & 2)

Golder Associates

Static Stability Analysis Section A-A

Drawn: CNW	App'd:	Date: June 20, 2003	Figure: **3a**
Project No.: 03-1411-018			Revision No.:

PROJECT NO. 03-1411-018 DRAWN: GGG REVIEWED BY: XX DATE: 06-23-03 FILE LOCATION: FINAL\2003\1411\100\03-1411-018\Figure 3b



Best-Estimate Soil Parameters

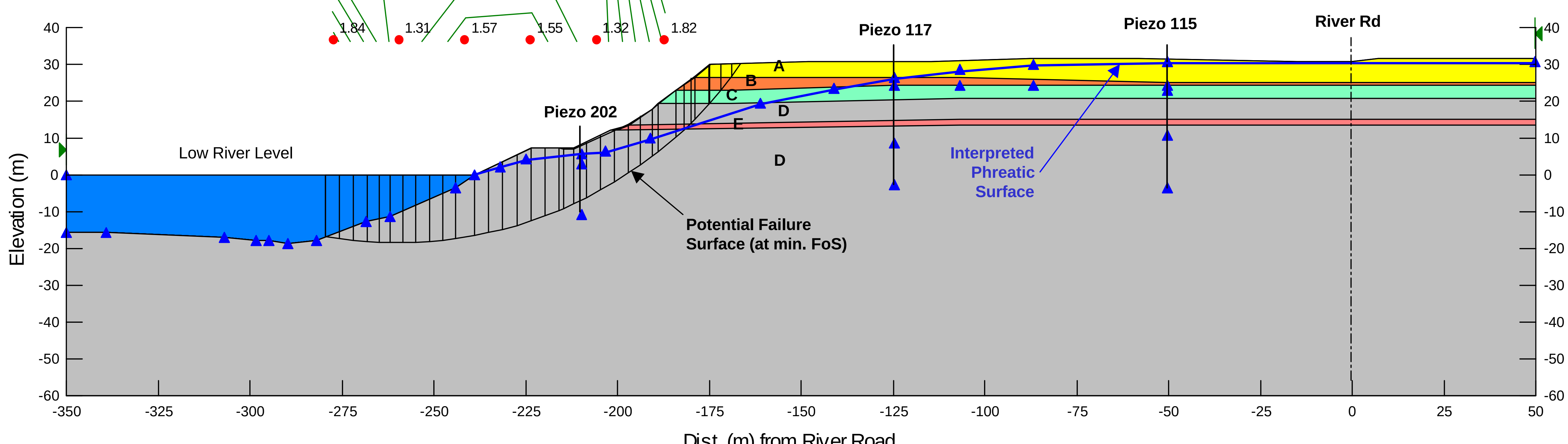
Soil A: Sand with silty clay layers, c' = 5 kPa, ϕ' = 33º, γ = 18 kN/m³ (γ_{sat} = 19 kN/m³)

Soil B: Sand to Silty Sand, c' = 0 kPa, ϕ' = 35º, γ = 19 kN/m³ (γ_{sat} = 20 kN/m³)

Soil C: Silty Clay to Silty Sand, c' = 5 kPa, ϕ' = 32º, γ = 18 kN/m³ (γ_{sat} = 19 kN/m³)

Soil D: Silty Clay, c' = 5 kPa, ϕ' = 30º, γ = 18 kN/m³

Soil E: Compact Silty Sand, c' = 0 kPa, ϕ' = 35º, γ = 19 kN/m³ (γ_{sat} = 20 kN/m³)



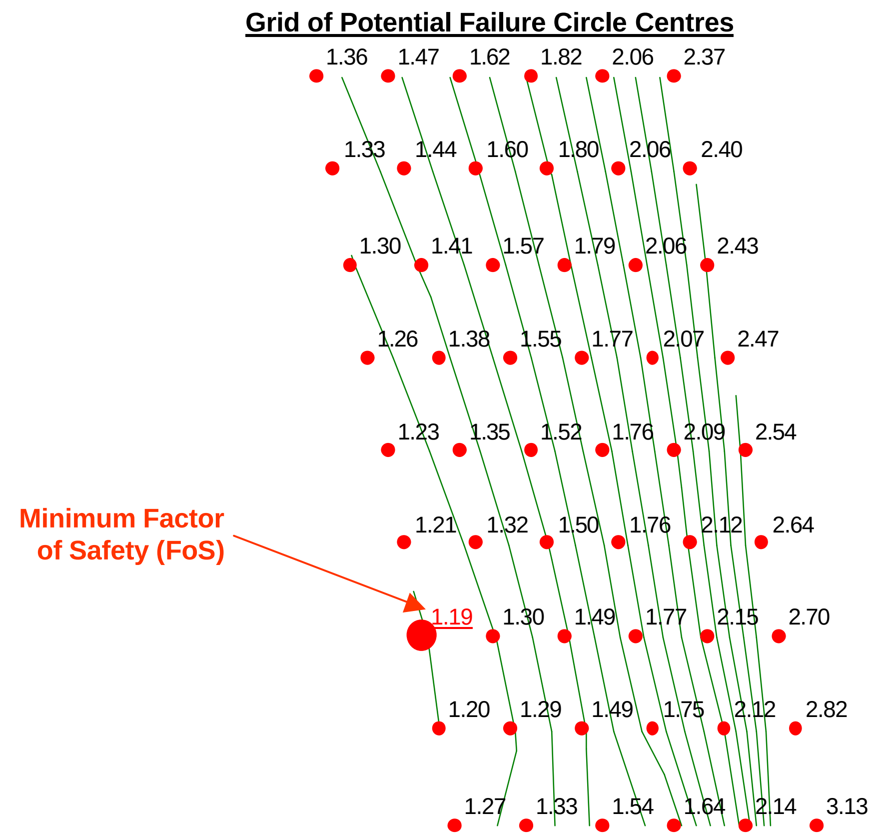
Note: Piezometer numbers referenced to Golder numbering system (for locations see Figures 1 & 2)

Golder Associates

Static Stability Analysis
Section B-B

Drawn: CNW	App'd:	Date: June 20, 2003	Figure: **3b**
Project No.: 03-1411-018			Revision No.:

PROJECT NO. 03-1411-018   DRAWN: GGG   REVIEWED BY: XX   DATE: 06-23-03   FILE LOCATION: FINAL\2003\1411\100\03-1411-018\Final Report\Figure 3c……



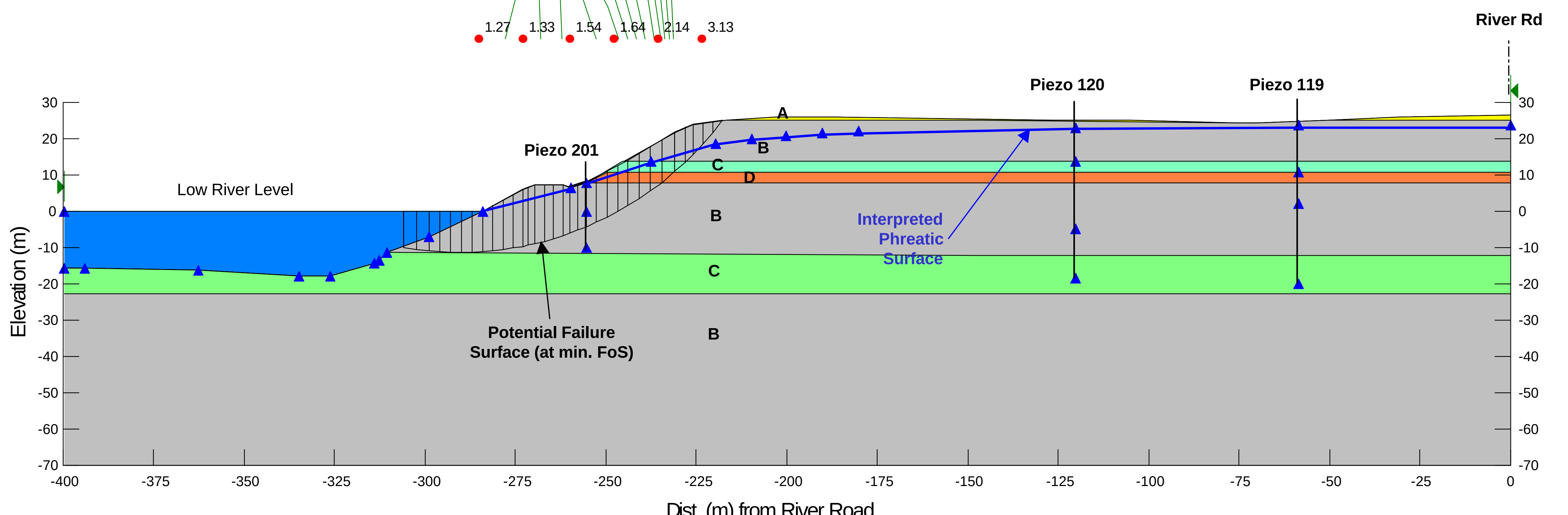
Best-Estimate Soil Parameters

Soil A: Sand, c' = 0 kPa, ϕ' = 35º, γ = 19 kN/m³ (γ_{sat} = 20 kN/m³)

Soil B: Silty Clay, c' = 5 kPa, ϕ' = 30º, γ = 18 kN/m³

Soil C: Silty Clay to Silty Sand, c' = 5 kPa, ϕ' = 33º, γ = 18 kN/m³ (γ_{sat} = 19 kN/m³)

Soil D: Compact/dense Sand to Silty Sand, c' = 0 kPa, ϕ' = 38º, γ = 19 kN/m³ (γ_{sat} = 20.5 kN/m³)

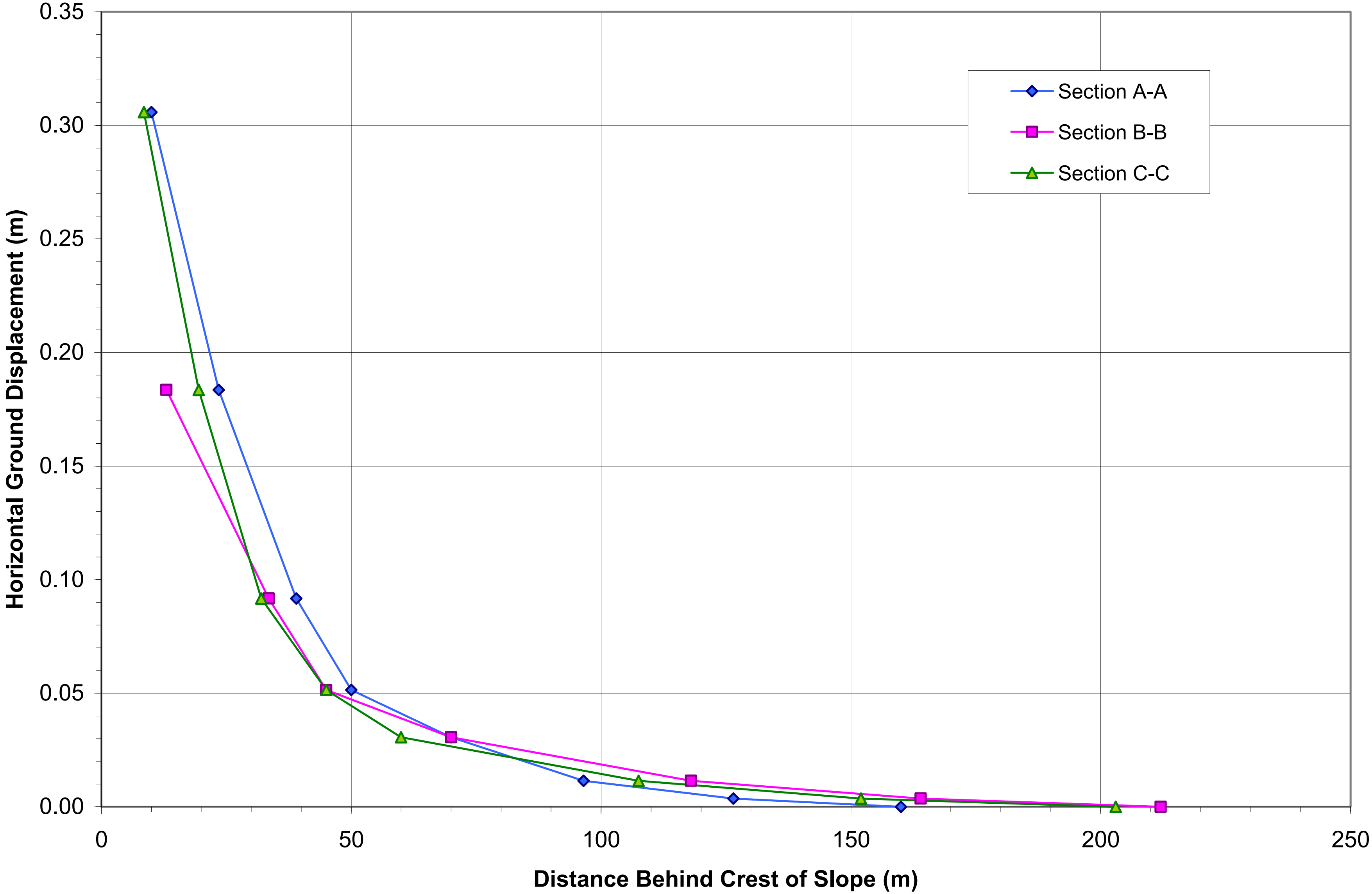


Note: Piezometer numbers referenced to Golder numbering system (for locations see Figures 1 & 2)

Golder Associates

Static Stability Analysis
Section C-C

Drawn: CNW	App'd:	Date: June 20, 2003	Figure: **3c**
Project No.: 03-1411-018			Revision No.:



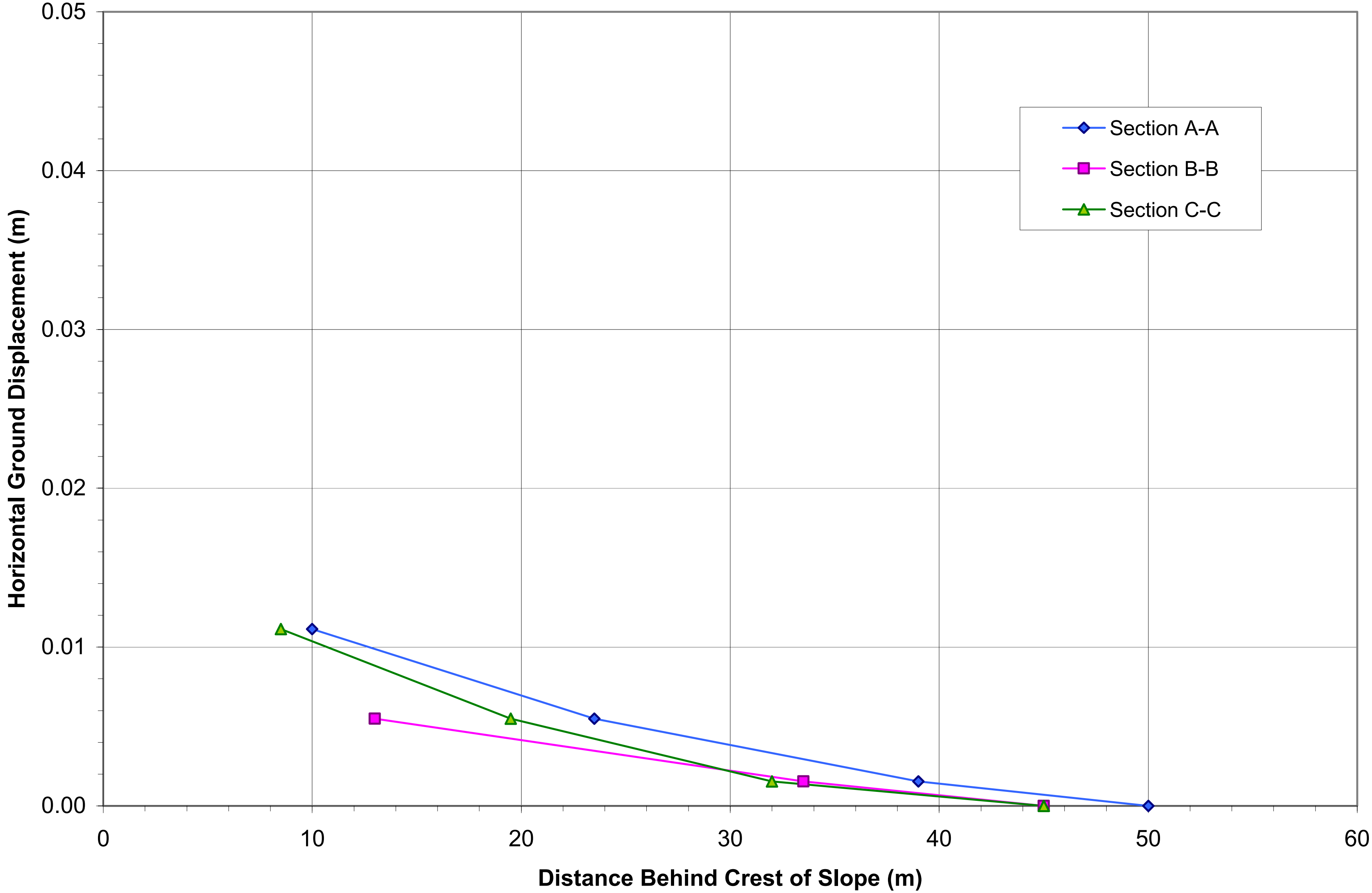
Golder Associates

Horizontal Ground Displacement vs Distance 475-Year Seismic Event

Drawn: CNW	App'd:	Date: June 20, 2003	Figure: **4a**

Project No.: 03-1411-018

Revision No.:



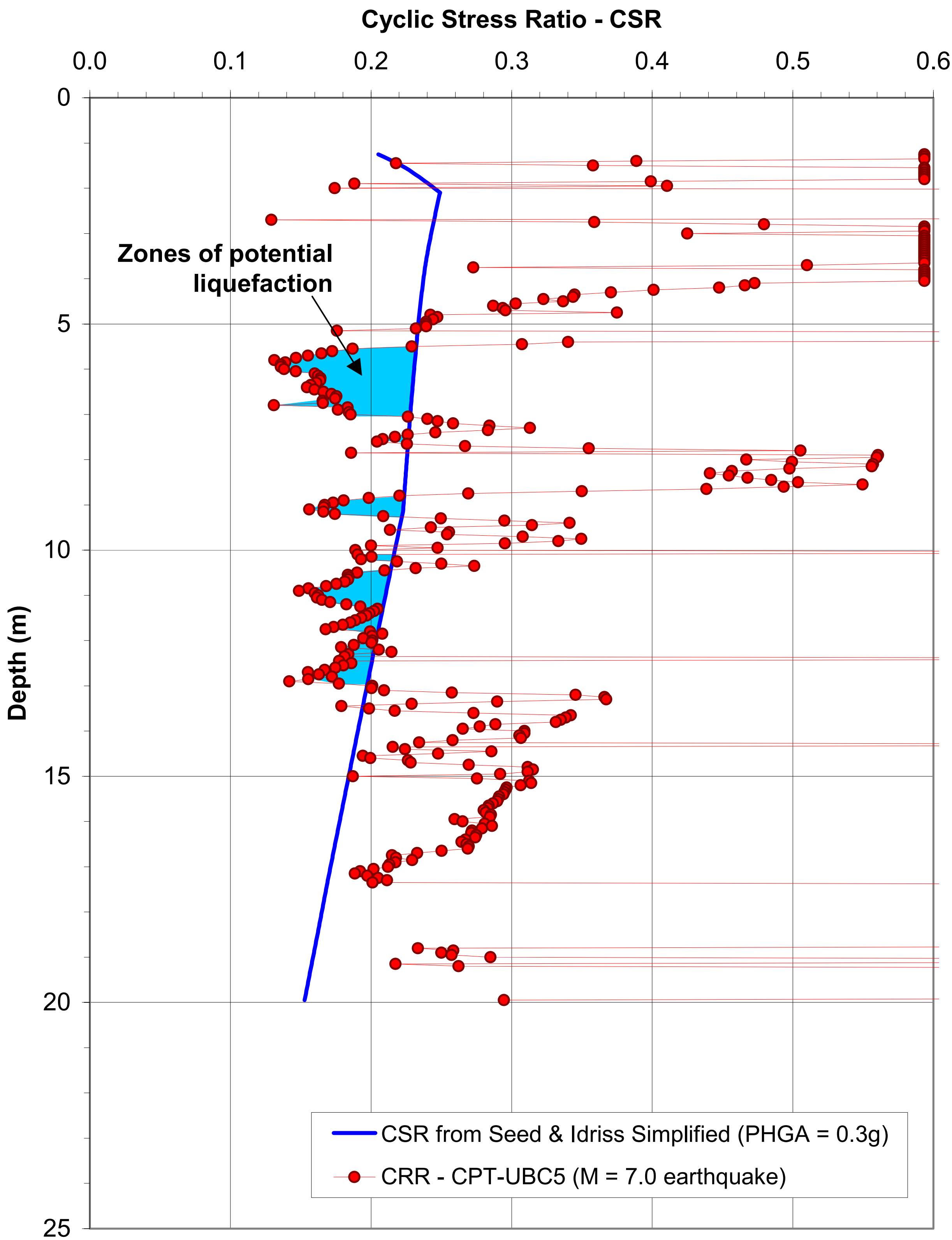
Golder Associates

Horizontal Ground Displacement vs Distance 100-Year Seismic Event

Drawn: CNW	App'd:	Date: June 20, 2003	Figure: **4b**

Project No.: 03-1411-018

Revision No.:

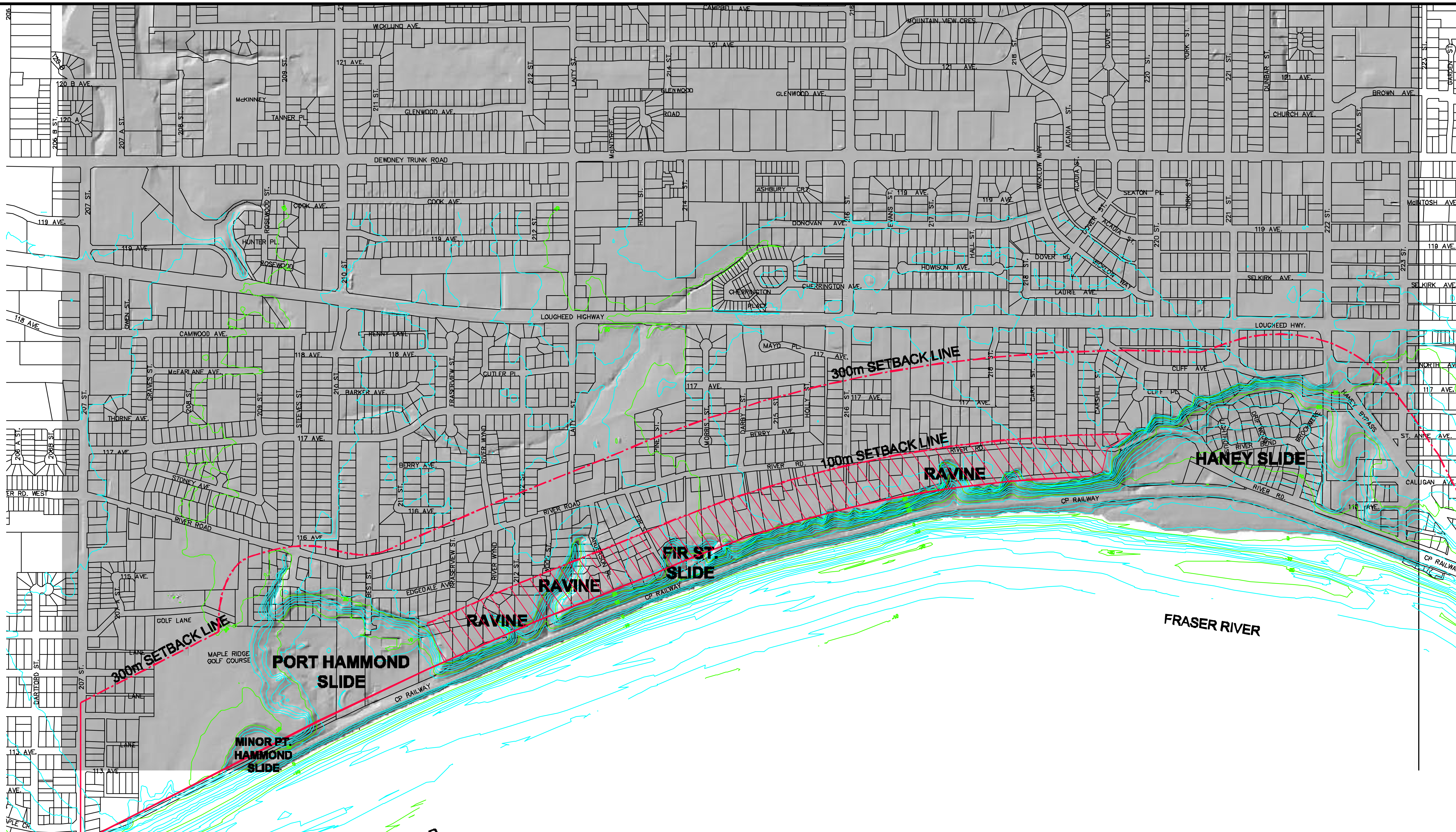


Note: Soil is considered to be susceptible to liquefaction where CRR < CSR

Golder Associates	Liquefaction Susceptibility (CPT-UBC5)	
Drawn: CNW	App'd:	Date: June 20, 2003	Figure: 5

Project No.: 000-0000 Revision No.:

PROJECT NO. 03-1411-018 DRAWN: GGG REVIEWED BY: XX DATE: 06-23-03 FILE LOCATION: FINAL\2003\1411\100\03-1411-018\Final Report\Figure 5.



0 100 200 300 400 500

Scale in Meters

NOTES

- All Elevations in Meters, referenced to Geodetic Datum
- Elevation Contours above 0m based on District of Maple Ridge Plans (2000)
- Contours below 2m Generated from River Survey Data along MOE Survey Lines
- Neither the Corporation of the District of Maple Ridge nor Golder Associates Ltd. make any guarantee regarding the accuracy or present status of the information shown on this figure.

REFERENCE

Map Projection: UTM NAD83 Zone 10
Sources: District of Maple Ridge, Ministry of Environment Lands & Parks

LEGEND

- Crest of Escarpment Line
- 100m Setback Line
- 300m Setback Line
- 100m Setback Zone

PROJECT

DISTRICT OF MAPLE RIDGE
SEISMIC VULNERABILITY STUDY
FRASER RIVER ESCARPMENT, MAPLE RIDGE, B.C.

TITLE

PROPOSED ESCARPMENT CREST AND SETBACK LINES

Golder Associates

PROJECT No.	03-1411-018		FILE No. P03-1411-018-03
DESIGN	CNW	12MAR04	SCALE AS SHOWN REV. -
CADD	JK	12MAR04	
CHECK	CNW	12MAR04	**FIGURE 6**
REVIEW			

APPENDIX I – III

(PLEASE SEE SEPARATE FILES)

APPENDIX IV

PIEZOMETER MONITORING DATA
1983 - 2003

Table IV-1
Status of Piezometers

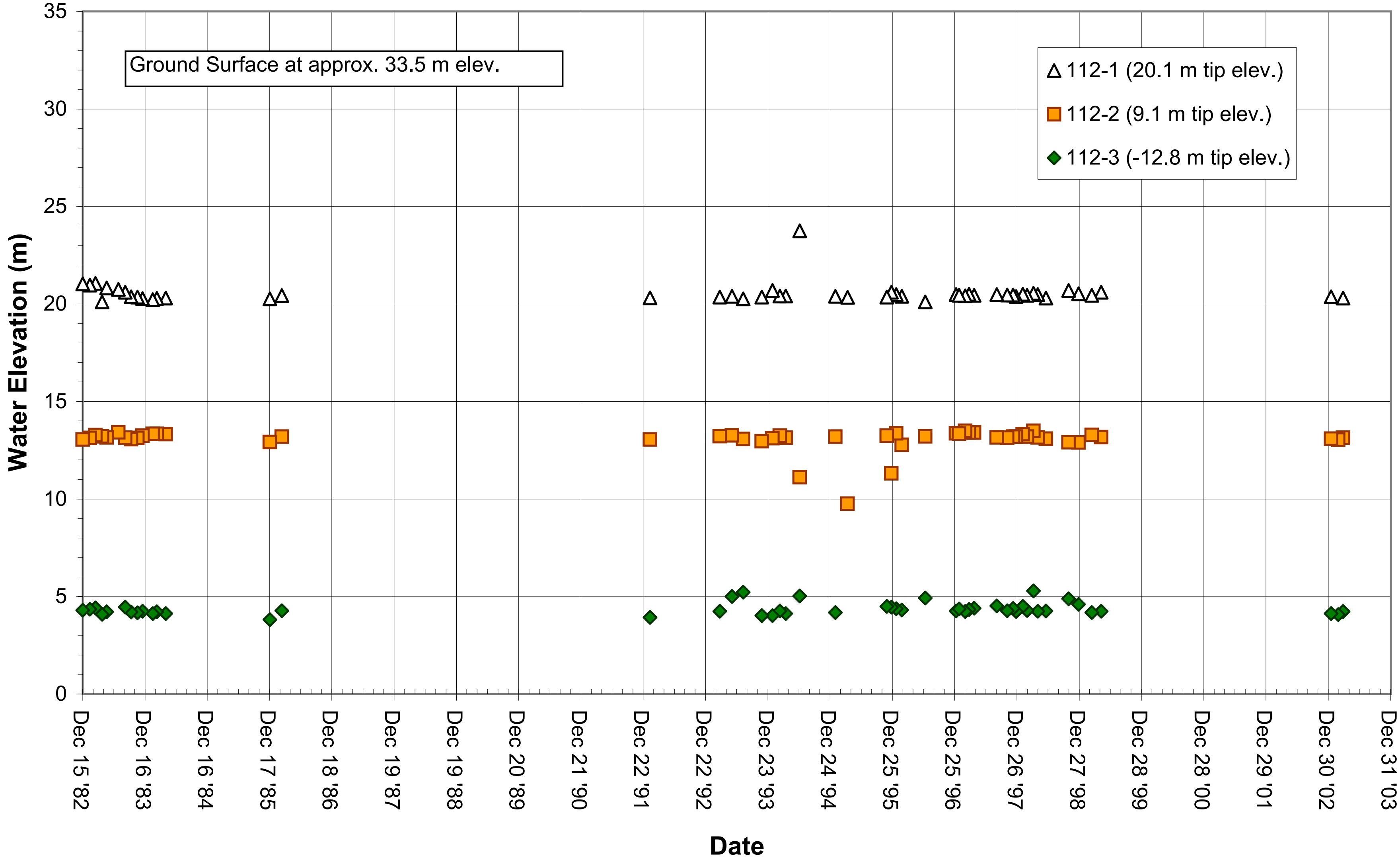
Golder Piezo No.	Street Address	Maple Ridge Piezo No.	Apparent Status
101A,B,C	21934 River Road		No monitoring data since Jan. 1983
103A,B	N/A		No monitoring data since Jan. 1983
104A 104B	11545 Fir Street		Plugged Highly erratic readings
105	11420 River Wynd		Plugged(?)
111-1,2,3	21536 River Road	112	No monitoring data since Feb. 1986
112-1,2,3	21564 River Road	111	Appear to be functioning adequately
113-1,2,3	N/A	109	No monitoring data since Feb. 1986
114-1,2,3	N/A	110	No monitoring data since Feb. 1986
115-1 115-2 115-3	21484 River Road	113	Highly erratic readings Appears to be functioning adequately Flooded in 1986; water levels still dropping
116-1,2,3	21694 River Road (in back yard)		Appear to be functioning adequately
117-1 117-2 117-3	21474 River Road	114	Feb. 2003 sounding 1.7 m above tip depth Flooded prior to 1993; water levels still dropping Flooded in 1984; no monitoring since; buried
118-1,3 118-2	21694 River Road (at road)		Appear to be functioning adequately Flooded between 1999 & 2003 (?)
119-1,2,3	N/A	116	No monitoring data since Feb. 1986
120-1,2,3	11562 Anderson Pl.	115	Appear to be functioning adequately
201-1,2	CP Rail bench		No monitoring data since Dec. 1985
202-1,2	CP Rail bench		No monitoring data since Dec. 1985
203-1,2	CP Rail bench		No monitoring data since Nov. 1983; Destroyed by slide in Jan. 1984
204-1,2	CP Rail bench		No monitoring data since Dec. 1985
205-1,2	CP Rail bench		No monitoring data since Dec. 1985

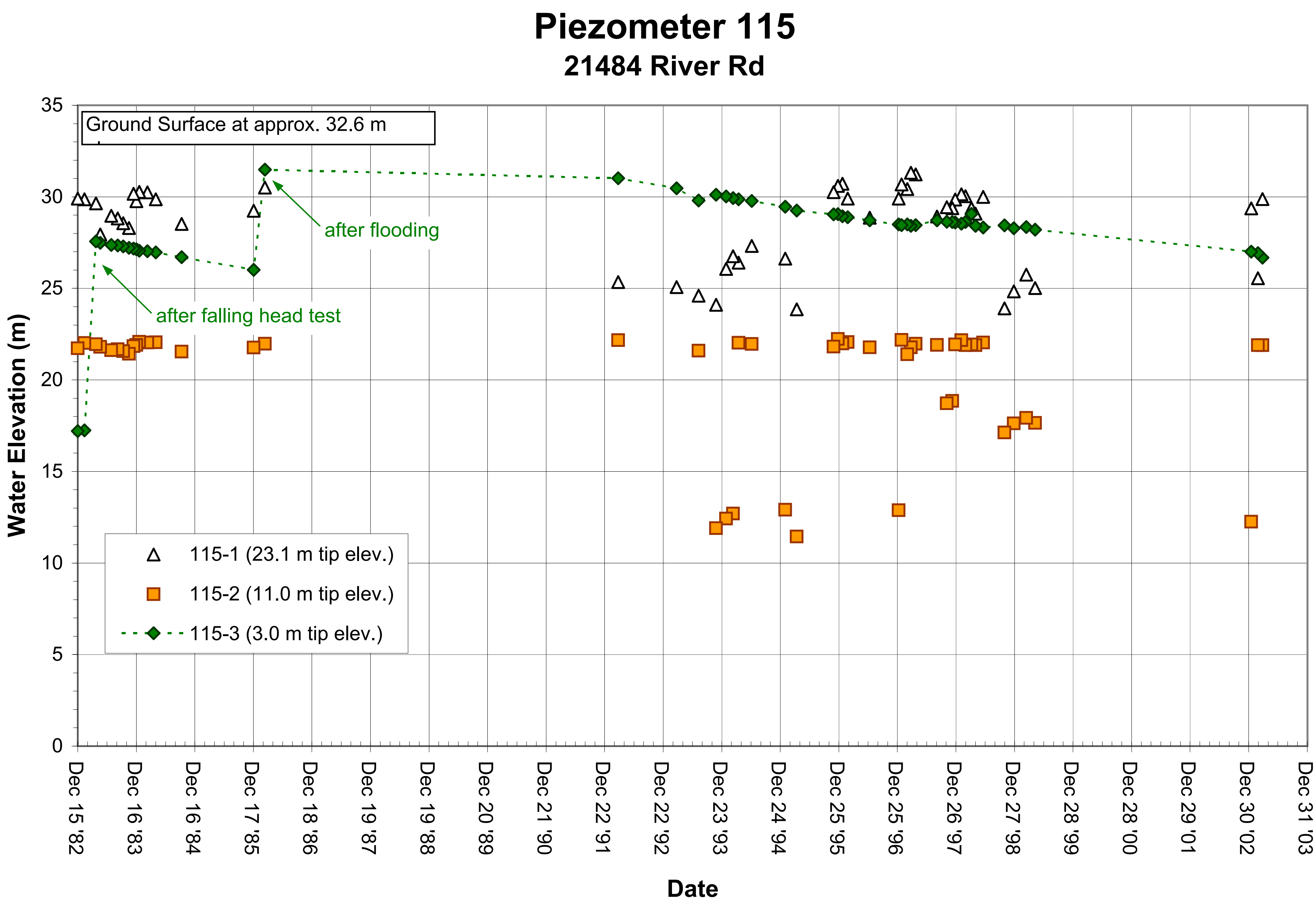
Note: N/A = Not Available

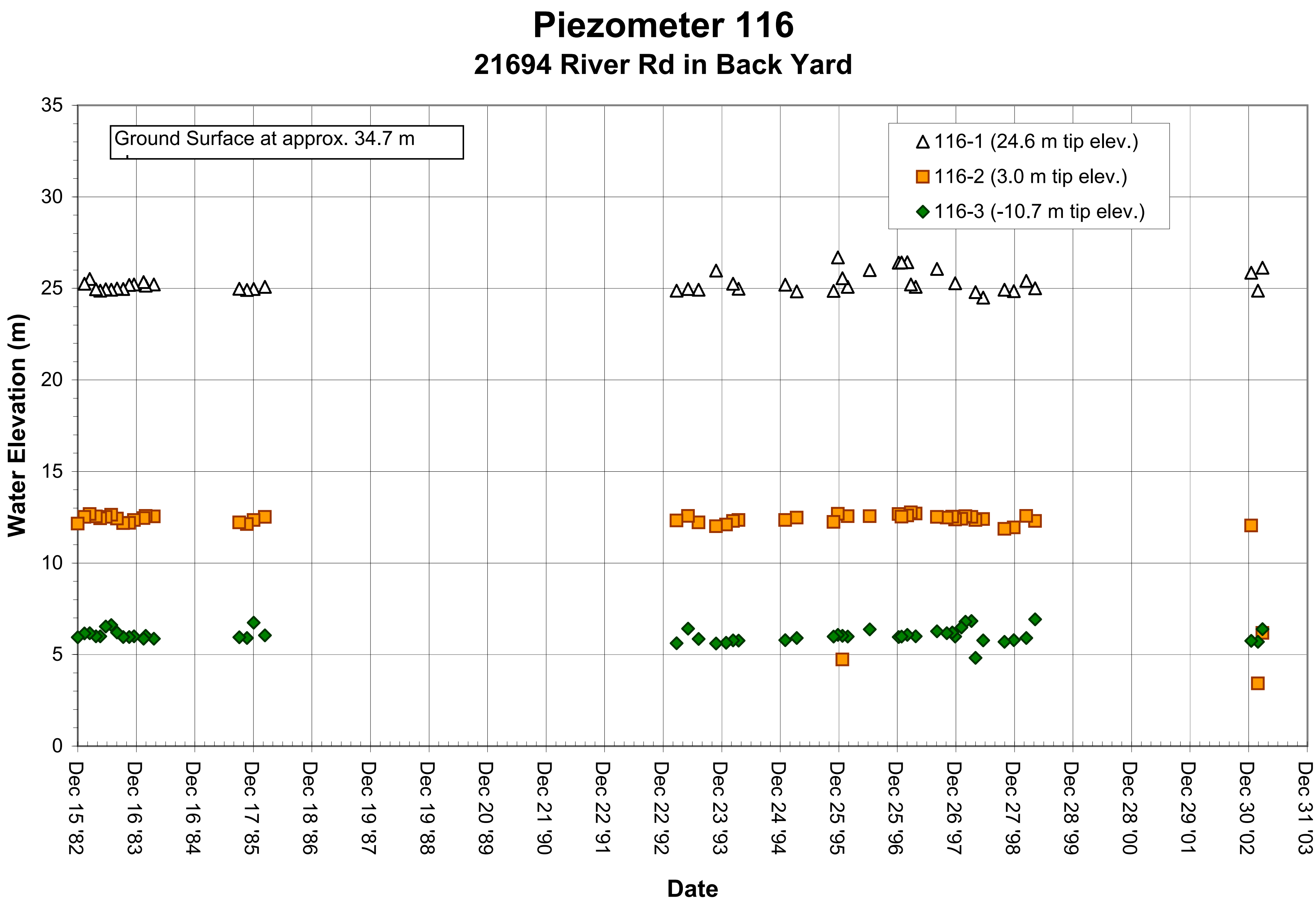
Table IV-2
Piezometer Details

Golder Piezometer No.	Approximate Ground Elevation (m)	Approximate Tip Elevation (m)	Sounding Elevation (m) Feb. 27 '03	"Average" Groundwater Elevation (m)
112-1	33.5	20.1	20.5	20.4
112-2		9.1	9.7	13.1
112-3		-12.8	-12.5	4.3
115-1	32.6	23.1	23.7	30.0
115-2		11.0	11.4	22.0
115-3		3.0	3.7	17.2 ('83 only)
116-1	34.7	24.6	24.9	25.3
116-2		3.0	3.0	12.5
116-3		-10.7	-10.6	6.0
117-1	32.0	24.4	26.1	27.0 ('83&'03)
117-2		8.5	11.9	14.5 ('83 only)
117-3		-3.1	Buried	7.0 ('83 only)
118-1	34.4	22.2	22.5	27.0
118-2		6.4	7.0	17.8
118-3		-10.1	-9.8	6.0
120-1	26.0	13.5	13.8	20.0
120-2		-5.1	-4.7	14.8
120-3		-18.8	-18.4	13.0
201-1	8.0	0.1		6.8 ('83 only)
201-2		-10.3		5.1 ('83 only)
202-1	8.0	3.0		5.2 ('83 only)
202-2		-10.9		2.7 ('83 only)
205-1	8.0	0.8		4.5 ('83 only)
205-2		-13.5		3.5 ('83 only)

Piezometer 112
21564 River Rd




Piezometer 115
21484 River Rd
Ground Surface at approx. 32.6 m
after flooding
after falling head test
Water Elevation (m)
35
30
25
20
15
10
5
0
115-1 (23.1 m tip elev.)
115-2 (11.0 m tip elev.)
115-3 (3.0 m tip elev.)
Dec 15 '82
Dec 16 '83
Dec 16 '84
Dec 17 '85
Dec 18 '86
Dec 19 '87
Dec 19 '88
Dec 20 '89
Dec 21 '90
Dec 22 '91
Dec 22 '92
Dec 23 '93
Dec 24 '94
Dec 25 '95
Dec 25 '96
Dec 26 '97
Dec 27 '98
Dec 28 '99
Dec 28 '00
Dec 29 '01
Dec 30 '02
Dec 31 '03
Date


Piezometer 116
21694 River Rd in Back Yard
Ground Surface at approx. 34.7 m
116-1 (24.6 m tip elev.)
116-2 (3.0 m tip elev.)
116-3 (-10.7 m tip elev.)
Water Elevation (m)
0
5
10
15
20
25
30
35
Dec 15 '82
Dec 16 '83
Dec 16 '84
Dec 17 '85
Dec 18 '86
Dec 19 '87
Dec 19 '88
Dec 20 '89
Dec 21 '90
Dec 22 '91
Dec 22 '92
Dec 23 '93
Dec 24 '94
Dec 25 '95
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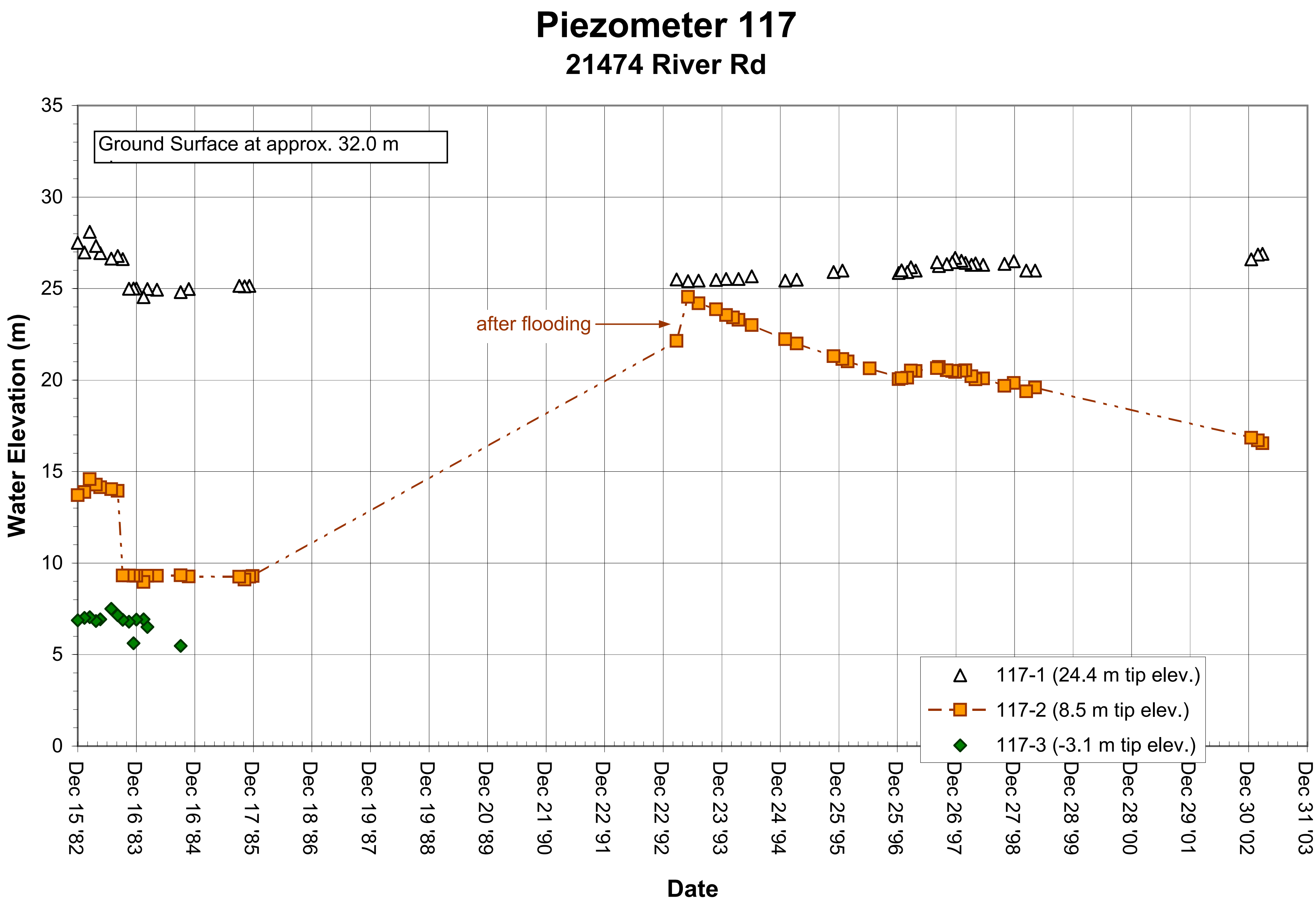
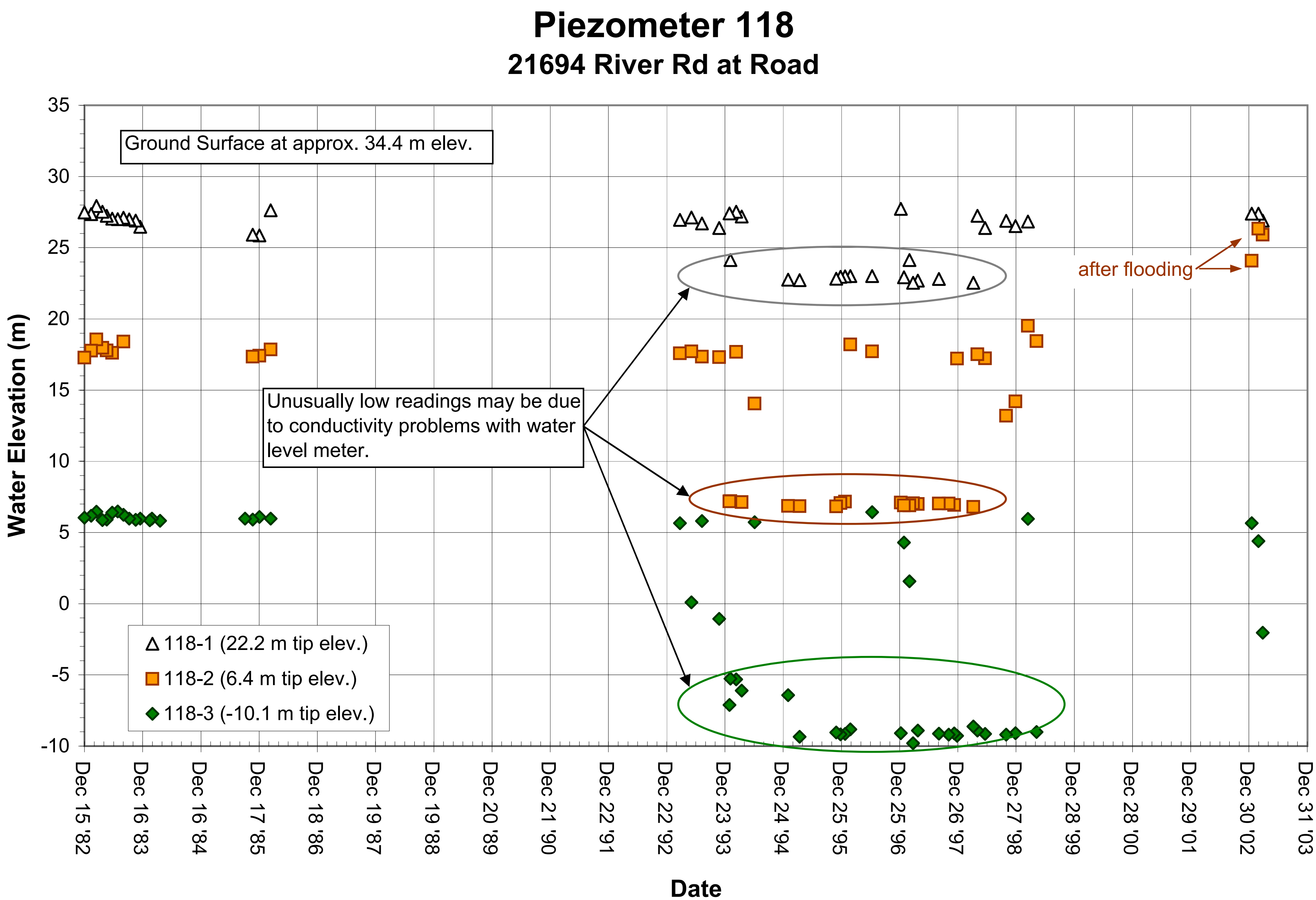
Dec 26 '97
Dec 27 '98
Dec 28 '99
Dec 28 '00
Dec 29 '01
Dec 30 '02
Dec 31 '03
Date



Piezometer 117
21474 River Rd
Ground Surface at approx. 32.0 m
after flooding
Water Elevation (m)
Date
117-1 (24.4 m tip elev.)
117-2 (8.5 m tip elev.)
117-3 (-3.1 m tip elev.)


Piezometer 118
21694 River Rd at Road
Ground Surface at approx. 34.4 m elev.
Unusually low readings may be due to conductivity problems with water level meter.
after flooding
Water Elevation (m)
35
30
25
20
15
10
5
0
-5
-10
118-1 (22.2 m tip elev.)
118-2 (6.4 m tip elev.)
118-3 (-10.1 m tip elev.)
Dec 15 '82
Dec 16 '83
Dec 16 '84
Dec 17 '85
Dec 18 '86
Dec 19 '87
Dec 19 '88
Dec 20 '89
Dec 21 '90
Dec 22 '91
Dec 22 '92
Dec 23 '93
Dec 24 '94
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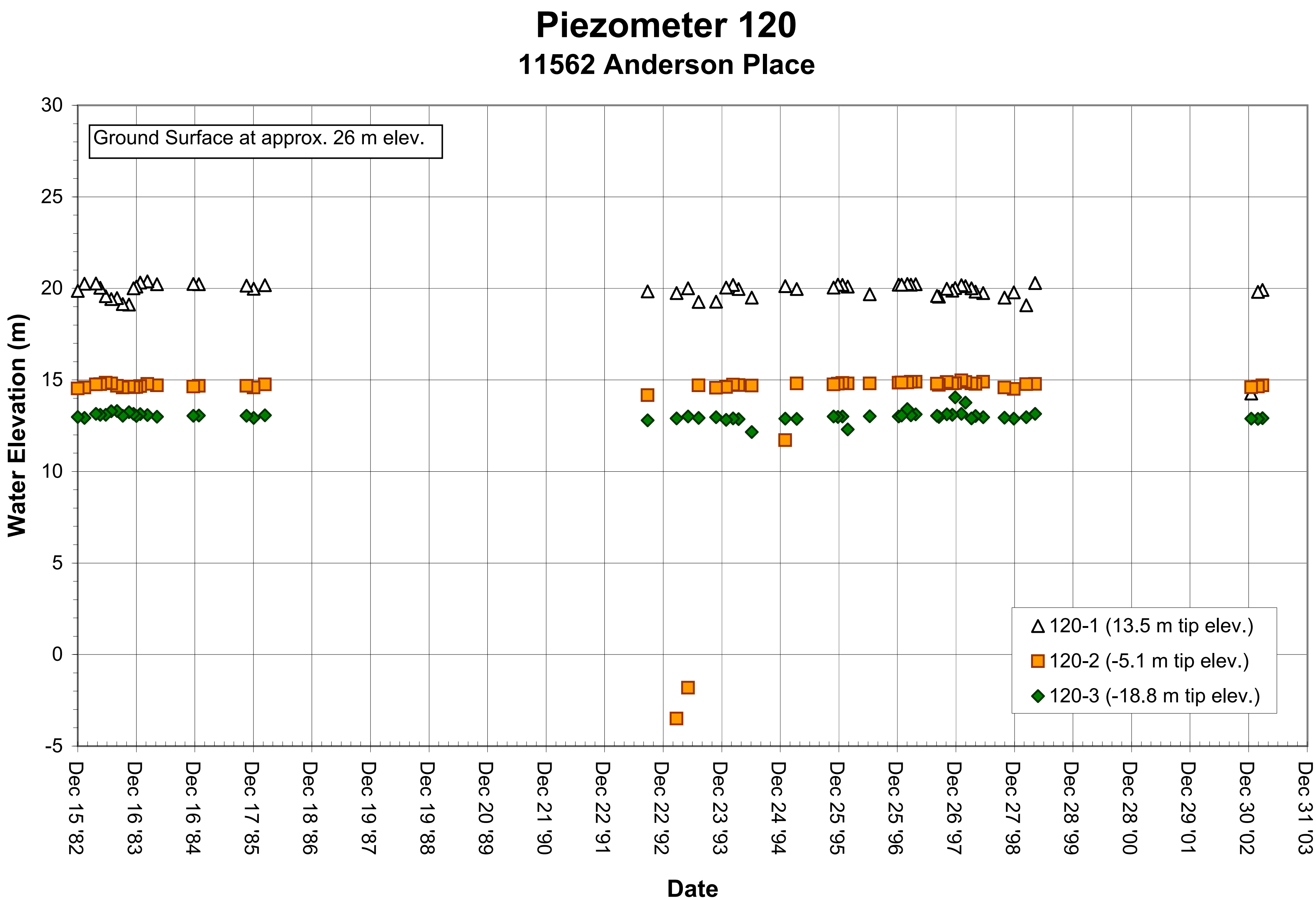
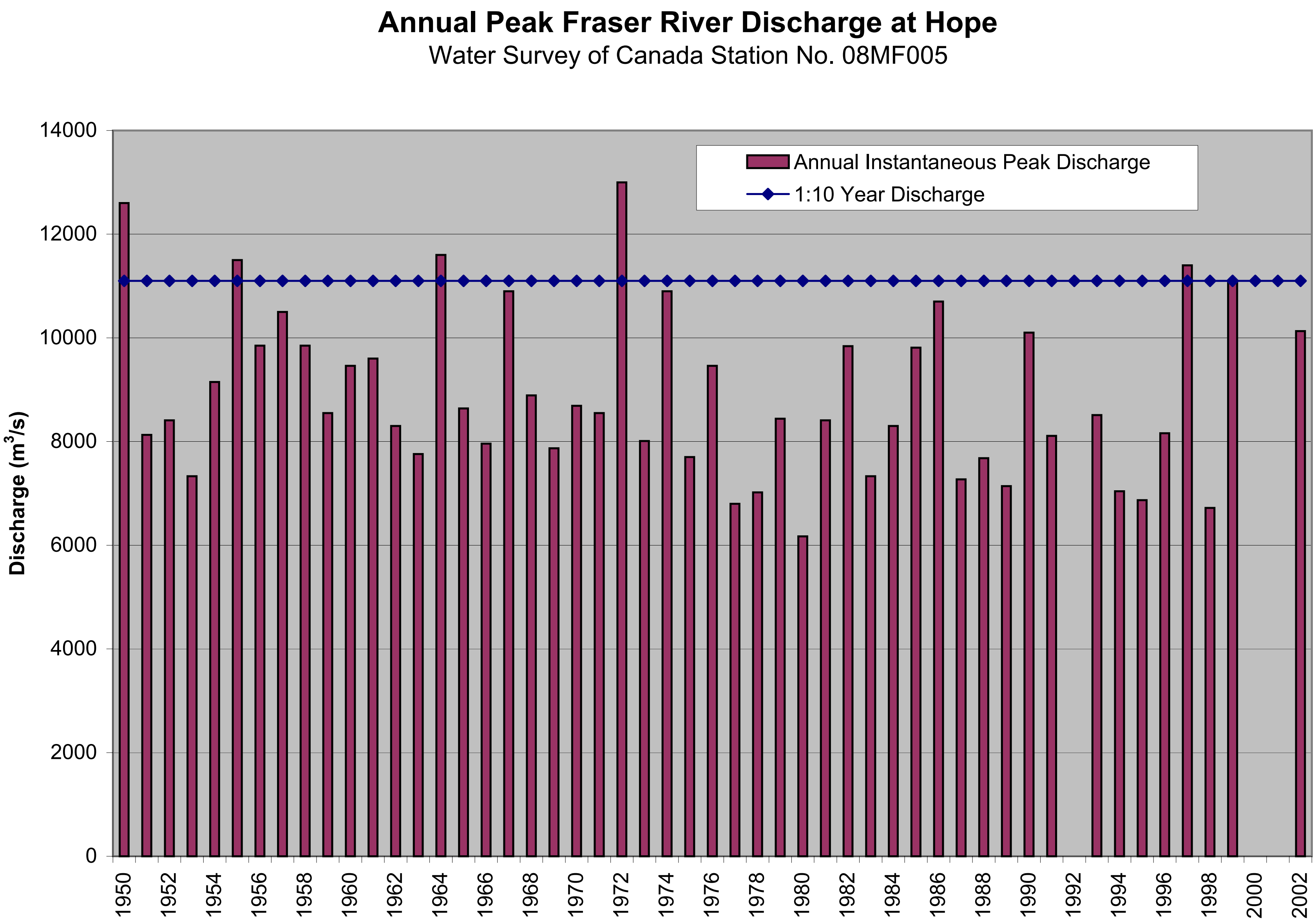
Dec 28 '00
Dec 29 '01
Dec 30 '02
Dec 31 '03
Date

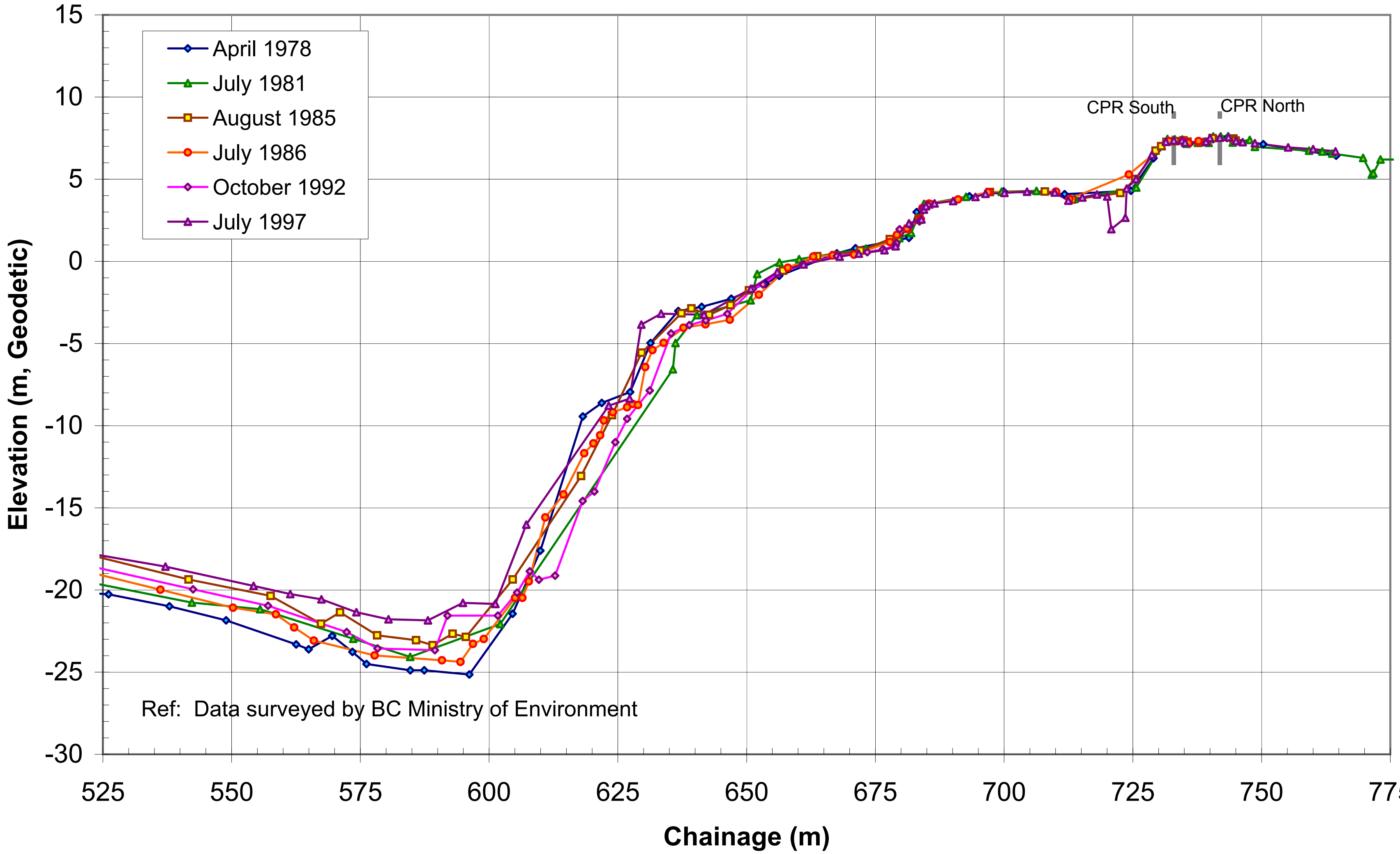

Piezometer 120
11562 Anderson Place
Ground Surface at approx. 26 m elev.
Water Elevation (m)
30
25
20
15
10
5
0
-5
Dec 15 '82
Dec 16 '83
Dec 16 '84
Dec 17 '85
Dec 18 '86
Dec 19 '87
Dec 19 '88
Dec 20 '89
Dec 21 '90
Dec 22 '91
Dec 22 '92
Dec 23 '93
Dec 24 '94
Dec 25 '95
Dec 25 '96
Dec 26 '97
Dec 27 '98
Dec 28 '99
Dec 28 '00
Dec 29 '01
Dec 30 '02
Dec 31 '03
Date
120-1 (13.5 m tip elev.)
120-2 (-5.1 m tip elev.)
120-3 (-18.8 m tip elev.)

APPENDIX V

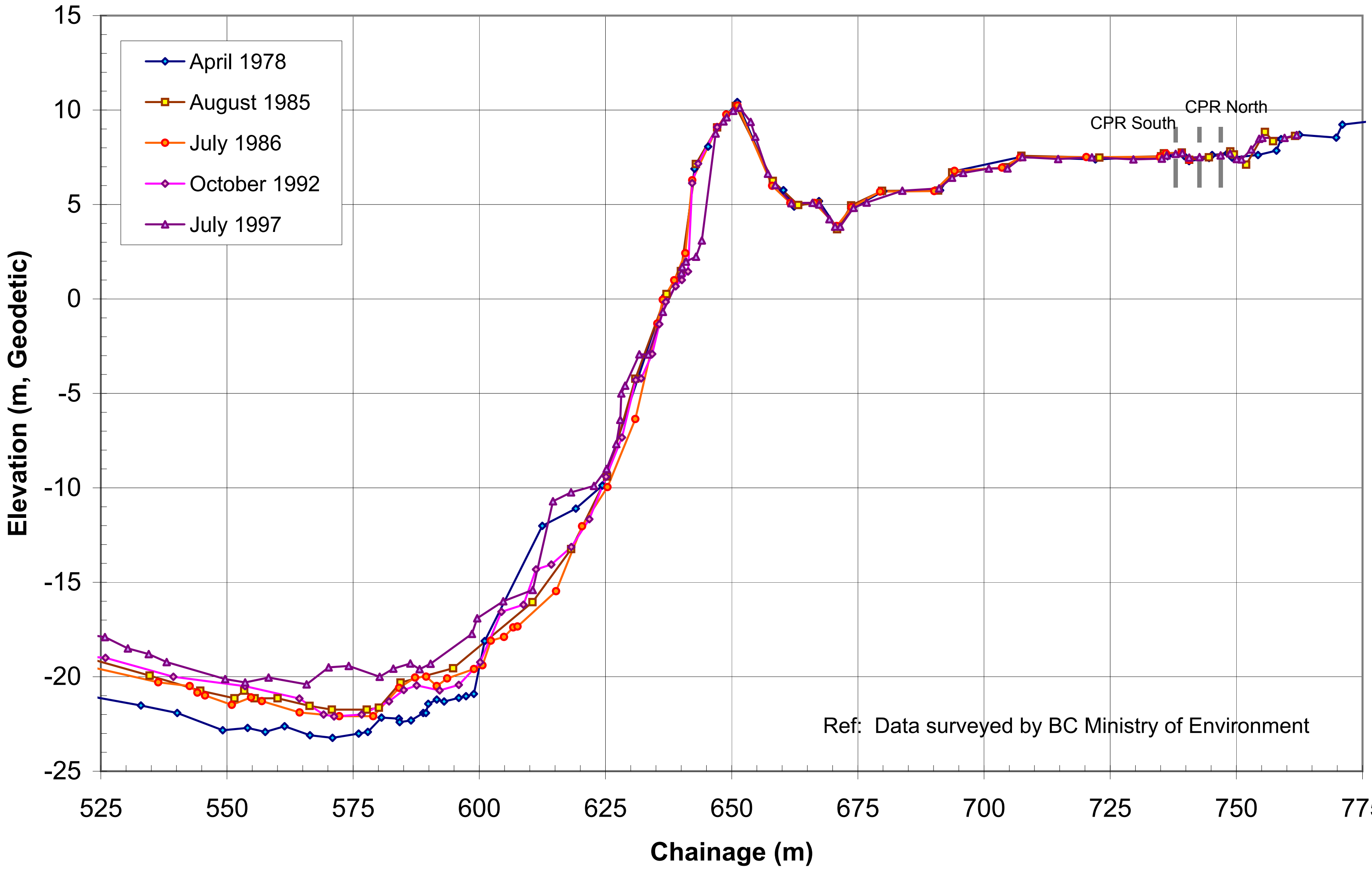
RIVER SURVEY DATA
FROM BC MINISTRY OF ENVIRONMENT, 1978 – 1997
(REPRESENTATIVE SECTIONS)


Annual Peak Fraser River Discharge at Hope
Water Survey of Canada Station No. 08MF005
Annual Instantaneous Peak Discharge
1:10 Year Discharge
Discharge (m3/s)
0
2000
4000
6000
8000
10000
12000
14000
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
1998
2000
2002

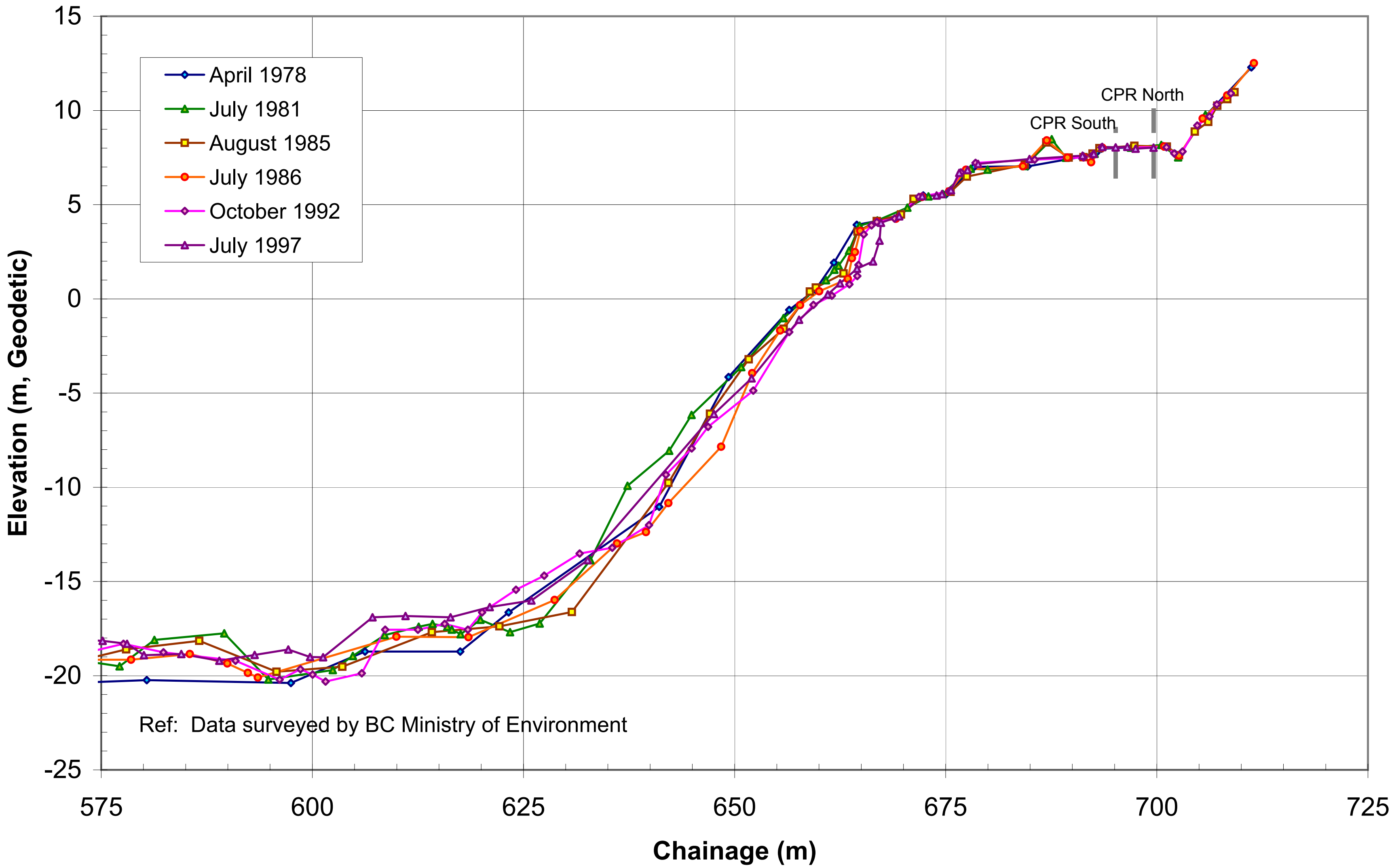
Cross Section 5

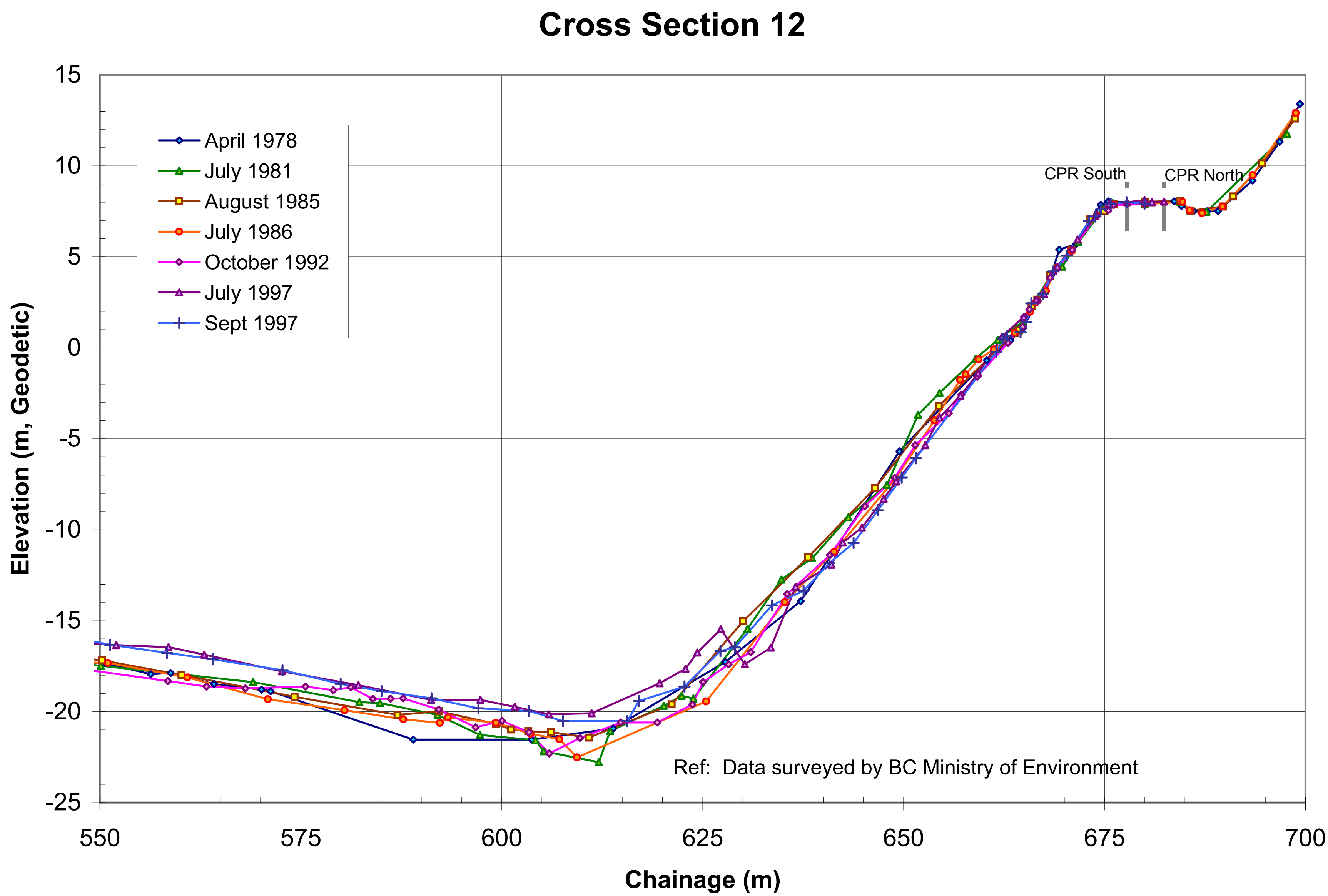


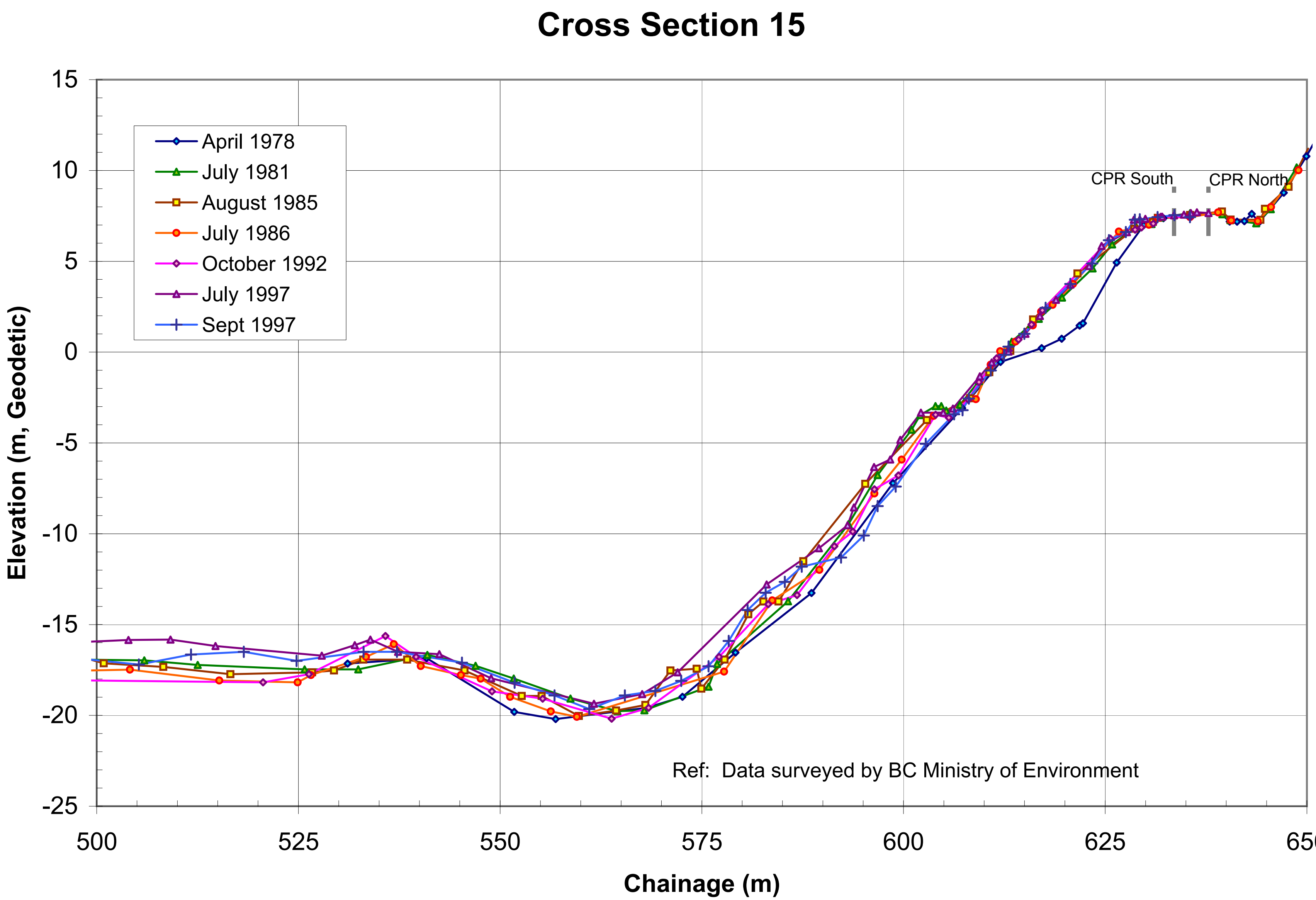
Cross Section 7



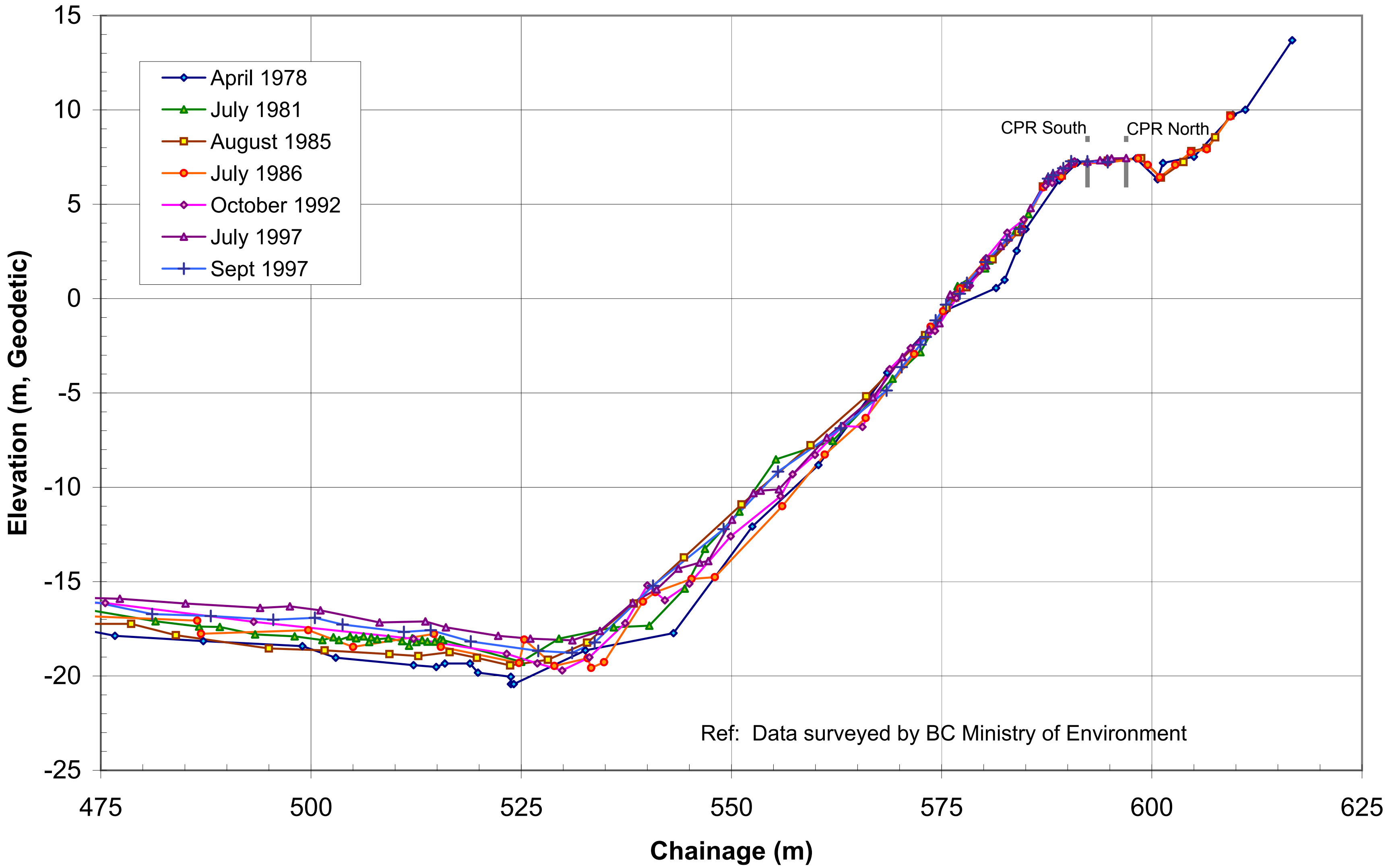
Cross Section 9



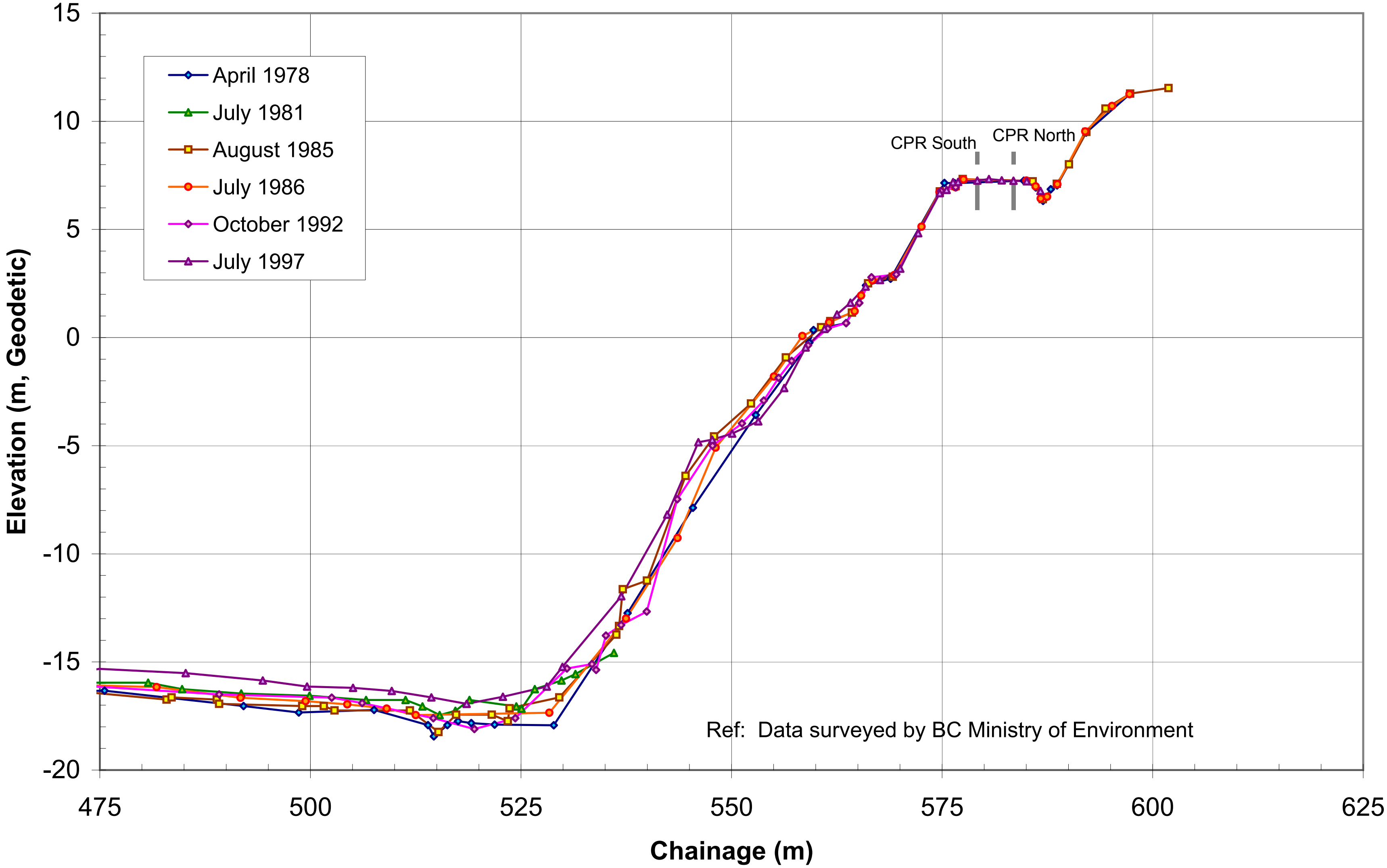

Cross Section 12
Elevation (m, Geodetic)
Chainage (m)
April 1978
July 1981
August 1985
July 1986
October 1992
July 1997
Sept 1997
CPR South
CPR North
Ref: Data surveyed by BC Ministry of Environment
15
10
5
0
-5
-10
-15
-20
-25
550
575
600
625
650
675
700


Cross Section 15
April 1978
July 1981
August 1985
July 1986
October 1992
July 1997
Sept 1997
CPR South
CPR North
Ref: Data surveyed by BC Ministry of Environment
Elevation (m, Geodetic)
Chainage (m)
15
10
5
0
-5
-10
-15
-20
-25
500
525
550
575
600
625
650

Cross Section 18



Cross Section 21



APPENDIX VI

CONE PENETRATION TEST DATA FROM UNIVERSITY OF BRITISH COLUMBIA, 1984 (CPT-UBC5)

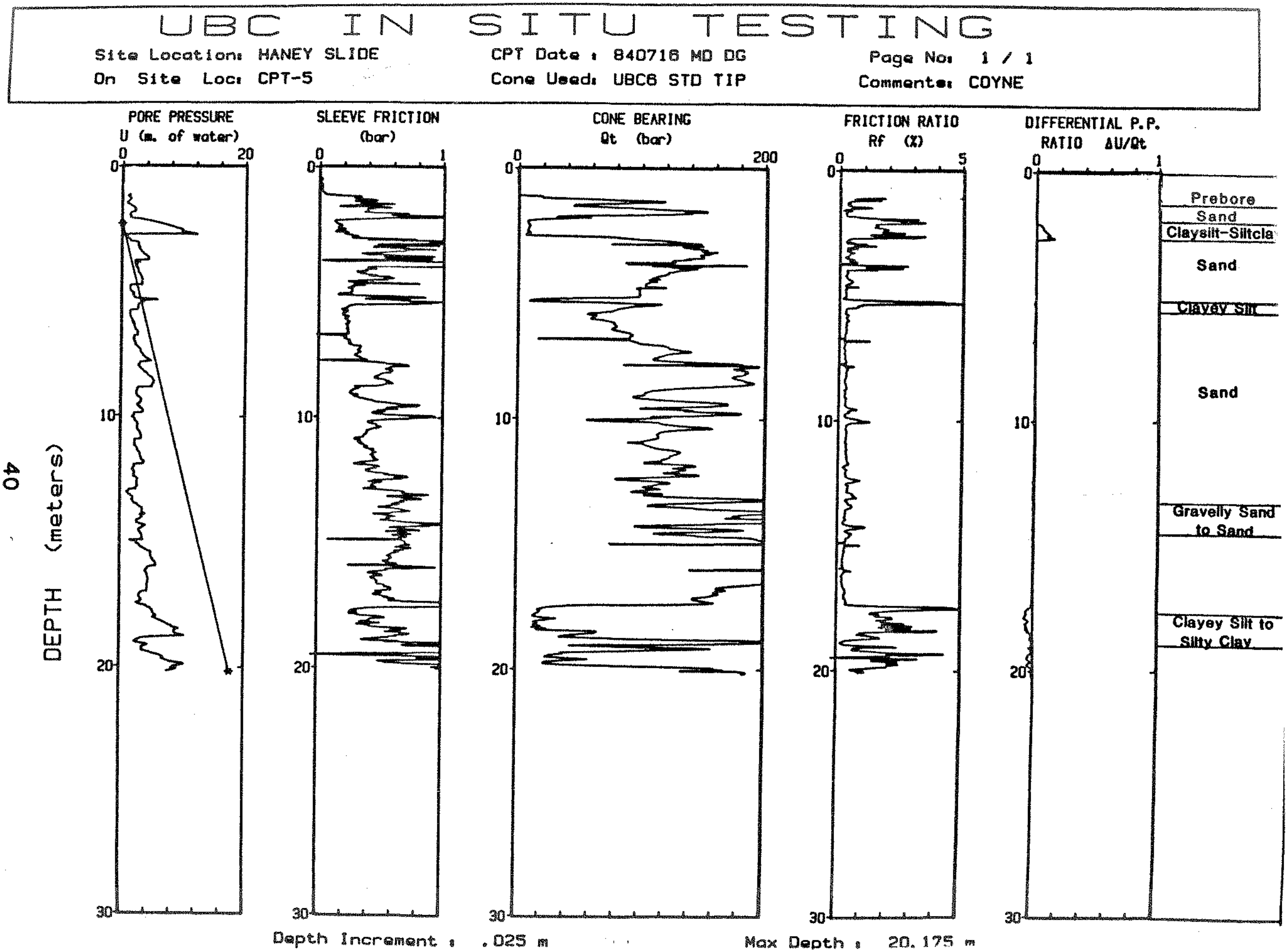

UBC IN SITU TESTING
Site Location: HANEY SLIDE
On Site Loc: CPT-5
CPT Date : 840716 MD DG
Cone Used: UBC6 STD TIP
Page No: 1 / 1
Comments: COYNE
PORE PRESSURE
U (m. of water)
SLEEVE FRICTION
(bar)
CONE BEARING
Qt (bar)
FRICTION RATIO
Rf (%)
DIFFERENTIAL P.P.
RATIO ΔU/Qt
DEPTH (meters)
Prebore
Sand
Claysilt-Siltcla
Sand
Clayey Silt
Sand
Gravelly Sand to Sand
Clayey Silt to Silty Clay
Depth Increment : .025 m
Max Depth : 20.175 m

Fig 3.14

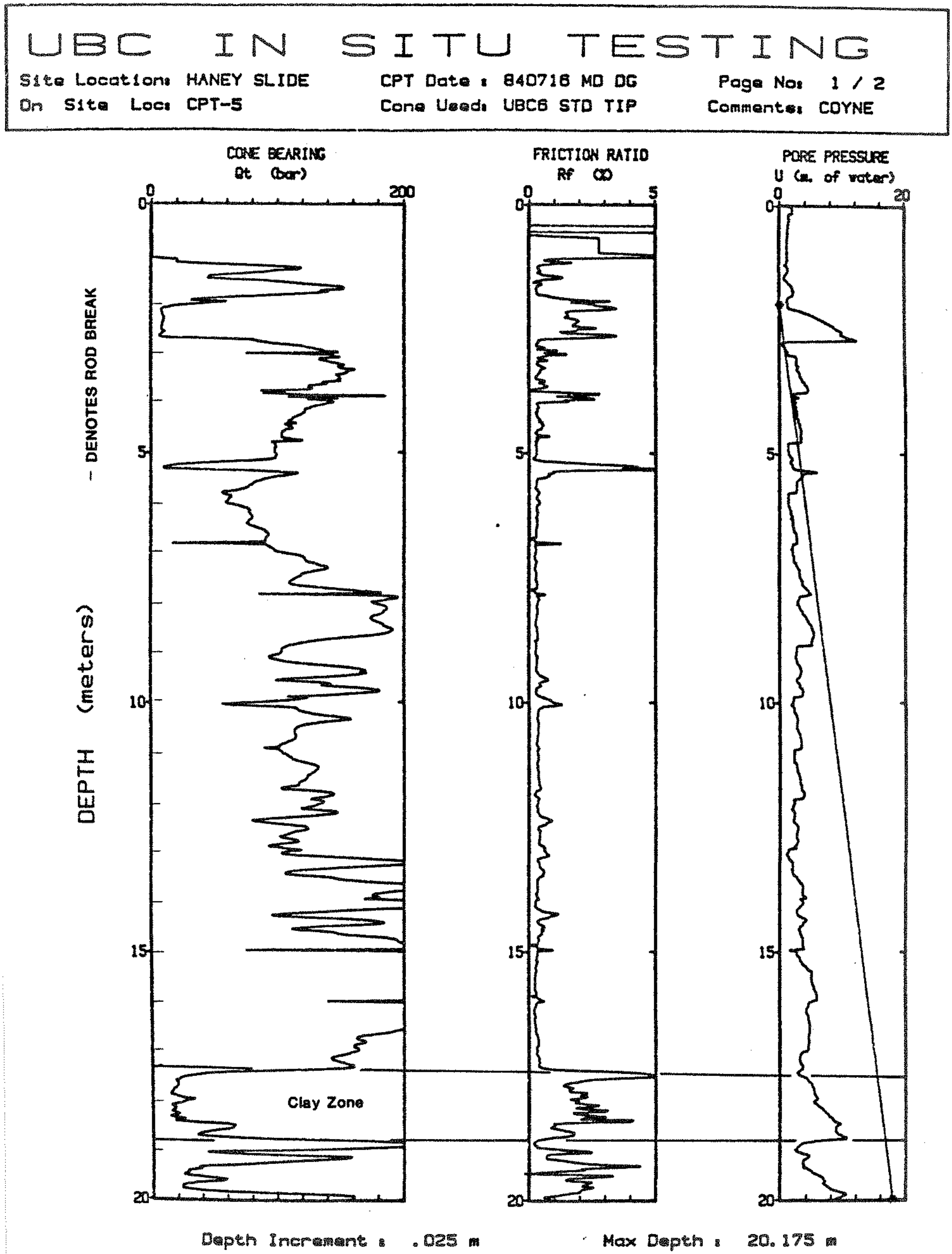

UBC IN SITU TESTING
Site Location: HANEY SLIDE
CPT Date : 840716 MD DG
Page No: 1 / 2
On Site Loc: CPT-5
Cone Used: UBC6 STD TIP
Comments: COYNE
CONE BEARING
Qt (bar)
FRICTION RATIO
Rf (%)
PORE PRESSURE
U (m. of water)
- DENOTES ROD BREAK
DEPTH (meters)
Clay Zone
Depth Increment : .025 m
Max Depth : 20.175 m

Fig. 3.15