APPENDIX III

GOLDER REPORT TO MINISTRY OF ENVIRONMENT
MARCH 1986

LIMITATIONS OF USE

The information contained in this appendix should be reviewed in conjunction with the main body of the report entitled "Geotechnical Seismic Vulnerability Assessment of Fraser River Escarpment, Maple Ridge, B.C.", dated March 23, 2004. Golder Associates Ltd. cannot be responsible for use by any party of this information without reference to the entire report. Unless otherwise stated, the suggestions, recommendations and opinions given herein are intended only for the guidance of the Client at the time of writing of the document and are not applicable to any other project or site location. Furthermore, the extent and detail of the factual information contained herein was intended for consideration of regional subsurface conditions and the potential for landslides occurring along the Fraser River Escarpment. Any person(s) wishing to assess the potential impacts of landslide events on any specific property should rely on their own investigations and their own interpretations of the factual data presented herein.

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT
TO
BRITISH COLUMBIA
MINISTRY OF ENVIRONMENT
ON
FRASER RIVER BANK STABILITY
MAPLE RIDGE, BRITISH COLUMBIA

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March, 1986 862-1906

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1.0 SUMMARY

This report presents the results of a review of the stability of the riverside bluffs between Haney and Port Hammond, British Columbia. Previous assessments of the stability and conditions were given in Golder Associates reports of July 1983 and August, 1979.

During the interval of 1982 to 1985, the piezometers installed as part of the previous investigation have been monitored periodically. Three sets of marine profiles have also now been taken to assist in analyzing potential erosion and scour. These sections were recorded in 1978, 1981 and 1985.

Additional information supplied by U.B.C. and C.P. Rail has been utilized in this review. Engineering reports prepared by D. Bazett and R. Blunden, and by Cook Pickering & Dolye have been reviewed.

The piezometric results show that the ground water pressures have remained relatively constant throughout the monitoring period and have not responded significantly to seasonal meteorological conditions. The piezometric data is therefore positive and leads us to conclude that ground water variations alone will not cause a major deep-seated slide as was earlier considered possible. Storm water control and infiltration minimization is however required to improve slope frontal stability.

Northwest Hydraulic Consultants have reviewed the river survey data. This data is not conclusive as no major freshet flows have been experienced over the study period. It is concluded that no significant erosion or scour has taken place since 1979, but that loss of materials could result during a major flood event. It is also concluded that erosion could potentially reduce the stability to critical levels.

Local shallow failures on the immediate bluff slopes could occur as a result of seepage. These events would only affect the C.P. Rail and river front properties on the bluff. Such failures have been experienced periodically over the study period and in recent history.

We conclude that the potential for river erosion should be addressed and that a comprehensive system of storm water control should be adopted. Two alternative approaches to river erosion are given.

1. Positive river slope protection consisting of placed rip-rap could be provided in the more critical areas.

2. An alternative contingency plan could be developed which would permit rapid response in the event that future monitoring indicated significant scour or erosion.

Provided storm drainage in the municipality is addressed and that river erosion is monitored and/or controlled as required, we believe that restrictions to development may be reduced. Further land use and development should be contingent on:

1. monitoring of ground water and river scour
2. establishment of surface runoff control
3. treatment of scour as required
4. geotechnical control for each new development within 100 m of the bluff crest.

2.0 INTRODUCTION

This report presents the results of a geotechnical/hyrological review of the stability conditions adjacent to the Fraser River in the District of Maple Ridge, British Columbia. The results of previous studies, including a ground water monitoring and testing program, are presented in our reports 782-1179, dated August, 1979 and 822-1071, dated July, 1983.

This review has been carried out as requested in the Ministry of Environment Services Agreement, dated February 13th, 1986. The terms of reference are outlined in Schedule "A" of the Agreement and include a review of piezometer data, river survey results, and other data collected since March, 1983. In the light of recent data, including UBC Cone testing, we have re-assessed the historical slide activity and its relationship to the current and future stability of the study area. The geological and stability scenarios, as discussed in the reports prepared by Bazett-Blunden, Cook Pickering & Doyle, Northwest Hydraulic Consultants and Golder Associates, have been considered. Based on all of the above data, we have re-evaluated the stability of the area and potential remedial measures and costs.

The geological considerations have been reviewed with Dr. J. Armstrong, and the construction techniques and costs have been reviewed with Miller Contracting Ltd. and Geo Products Inc. Discussions were also held with the C.P. Rail and Haney Historical Society to obtain current and former observations.

3.0 REVIEW OF AVAILABLE DATA

The following data has been utilized in this review; as well as that information available previously:

- Piezometer and precipitation data to February, 1986
- River survey data of August, 1985 and river discharge data
- CP Rail records and observations, 1983 to 1985
- UBC Cone testing and analysis in the 1880 Haney slide area
- Reports by D. Bazett and R. Blunden (draft version only) and by Cook Pickering & Doyle Ltd.

3.1 Piezometer Monitoring and Performance

The piezometers installed at the site have been monitored by District of Maple Ridge personnel since March, 1983. The piezometers were monitored at one to two month intervals between March, 1983 and April, 1984 and at less frequent intervals thereafter. Based on previous testing of the piezometers, and the monitoring results, 28 of the 40 1982 piezometers appear to be performing satisfactorily, 2 have been destroyed by a small slide at the C.P. Rail tracks, and at least 6 of the remaining pizometers require only minimal repair.

The data collected from the 1982 piezometers are presented in Appendix B. Suspect data have been omitted from the graphical plots. These data generally confirm the conclusions reached in our previous report which were based on only three months of data. The data shows however that the seasonal fluctuations have been less than anticipated. The data collected since 1982 increases the level of confidence regarding piezometeric levels considerably. The following conclusions can be drawn regarding piezometric levels in the instrumented area:

- The piezometric pressures in the area underlying the uplands and slopes are considerably lower than the hydrostatic pressure. Piezometric level fluctuations of up to 1 and 2.5 metres have been observed in deeper and shallower piezometers, respectively. These fluctuations appear to be in response to seasonal precipitation.

- The piezometric pressures beneath the C.P. Rail bench increase hydrostatically with depth. Fluctuations of up to 2.3 metres have been observed. The piezometric levels are 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 of the past few years it is concluded that the range of piezometric levels is unlikely to vary significantly from that recorded to date.

3.2 River Surveys

River cross sections have been surveyed by the Ministry of Environment in 1978, 1981, and most recently in August, 1985. The survey data has been reviewed by Northwest Hydraulic Consultants Ltd. Their report is presented in Appendix A. The following is a summary of their conclusions:

- Part of the study area is located on the outside of a major bend in the Fraser River where bank erosion would be expected to occur.

- The thalweg is deepest in the study area and rises in the upstream and downstream directions. This is one of the deepest sections of the Fraser River between Hope and the delta.

- Based on review of historical mapping, erosion of the 1880 Haney slide debris has occurred at an estimated rate of 1 m per year over the last 50 years. There is no evidence of significant bank erosion in the study area downstream of the slide during this period, except in the immediate downstream area where C.P. Rail has placed rip rap to minimize erosion.

- Based on the river surveys, no significant erosion is evident in the study area between 1978 and 1985. The largest freshet during this period was only a "3 year event" which occurred in 1982.

- Without providing protection it is considered that erosion in the order of 10 to 15 m could occur during a major flood event if erodible soil exists.

No offshore drilling of the river bed has been carried out. However, available data suggests that erodible granular soils are present in the north bank. It is probable that stiff clay exists at the thalweg level, but it is also possible the thalweg has been deeper in the past and has been infilled with granular soil.

3.3 UBC Study of Haney Slide Area

Cone penetration testing was carried out in the 1880 Haney Slide area in the summer of 1984 by Mr. M.P. Davies under the direction of Dr. P. Robertson. His data, analysis, and discussion provide additional

information on the soil stratigraphy and strength, ground water conditions and possible geometry of the slide.

One cone test was located close to borehole 101 on the bluff just west of the slide. The results confirm that the soil in this area consists of sand and some gravel to a depth of 17 m. Granular soil along the slide back-scarp prevented testing in this area. Within the base area of the slide the soil consists predominantly of clayey silt with inter-bedded sand generally in the upper 5 m but up to 12 m depth in one area. Potential shear zones and planes were identified at depths of 9 to 13 m and also up to 19 m below ground surface.

Predicted shear strengths based on the cone data compare well with results of testing carried out previously by UBC and Golder Associates.

Reference is made to the potential for catastrophic, brittle failure initiated by pore pressure increases in sandy layers within the marine deposits. It is postulated that clay immediately adjacent to the sand layers could be very sensitive due to leaching of salts. If leaching has occurred, it is postulated that movements initiated by high water pressures in the sand layers could result in a rapid decrease in shear strength and subsequent catastrophic failure, and retrogression. No data has been presented, or is known to exist, in support of such leaching.

3.4 C.P. Rail Observations

Discussions with C.P. Rail staff indicate that they have had no significant problems related either to stability or erosion in the study area during the three years since our last report was issued. A minor surficial slide occurred at the location of piezometer 203 in January, 1984 destroying this instrument. It is worth noting that a rise in water pressure was observed prior to the slide.

Prior to 1982, C.P. Rail experienced considerable sloughing and shallow failure of the slopes above the rail grade. River bank erosion was also experienced downstream of the 1880 slide. This erosion periodically prompted placement of rip-rap erosion protection. The rock was placed between sections 12 and 18. It is considered doubtful that the rock fill extended to the base of the channel.

3.5 D. Bazett - R. Blunden Draft Report

The report was prepared in May, 1981 by Messrs. Bazett and Blunden, on behalf of some River Road property owners. At this time we have been unable to obtain a final copy of the report and therefore have reviewed a draft copy of the report which was forwarded to us in 1981.

The basic thrust of the report is that the Fraser River Derby Reach channel was formed as a glacial channel nearly 10,000 years ago and has not been subject to scour or erosion by the Fraser River. The Haney to Port Hammond slopes are stated to have been subject to regression or instability due to:

- surficial slope slumps
- major regressive failures
- massive partial failure during a former glacial setting
- chemical weathering of the exposed slopes

The surficial sloughing is considered due to seepage associated with development, and the major regressive slides with forest clearing and land development. The major slides are considered to be affected by the end effects of the massive partial failure.

The Bazett-Blunden report therefore considers the 1880 Haney slide and the Port Hammond slide to be atypical in relation to the intermediate bluff area. The report concludes that river erosion is not critical and that land instability is a result of man and his works. The report concurs that the stability of the slopes is low, but that the risk of further major slides occurring is low. Further, it is concluded that municipal and residential control of ground water, effluent and runoff will control surficial slides.

The report is based on a geological approach to the area of concern.

3.6 Cook Pickering & Doyle Report

The Cook Pickering & Doyle report was prepared to assist in development of the former Haney slide area property and to provide recommendations for that development. The report concludes that the former slide area is now stable under the current river regime. The report considers that slopes at the toe of 222 Street could become critical if unchecked river scour were allowed.

The report concludes that the most critical failure section is that which includes the backslope above the former 1880 slide.

4.0 RE-EVALUATION OF STABILITY

Based on consideration of the available data discussed above we have reviewed the stability analysis carried out previously.

It should be understood that, as in previous reports, only major deep-seated failures have been considered in this study and review. Such failures, if they occurred, would be expected to extend at least 30 to 60 m from the bluff crest and well below river level. Retrogressive slides would extend up to 300 m from the bluff crest. Smaller surficial failures, as have previously been experienced in the C.P. Rail backslopes and at the rear of adjacent properties, have not been considered.

Under present conditions we estimate that 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 shear strength and piezometric pressures. The factor of safety could be reduced under earthquake loading, by varying the piezometric pressure, or by river erosion.

The geological review carried out with Dr. J. Armstrong has indicated that the historical geological and stability conditions of the

Maple Ridge area can not be conclusively defined based on proven geological knowledge.

A review of meteorological data indicates that the precipitation during the current study period was as adverse as that experienced during 1879 to 1880, and is periodically more severe than average conditions (see Appendix B). Severe total monthly precipitation was experienced during November 1983, January 1984 and October 1985. Severe cold weather was experienced during November 1985. Based on this information, an evaluation of the piezometric data indicates that ground water levels do not vary significantly with variable meteorological conditions.

The data collected over the past three years suggests that piezometric pressure increases which would have a significant impact on the overall stability of the bluffs in the study area are unlikely to occur provided reasonable municipal storm drainage control is provided. Furthermore, because the piezometric pressures are low, the risk of retrogressive failure is reduced.

The 1880 slide resulted in a deep retrogressive movement. Our analyses show that present conditions within the study area are not conducive to a repetition of such a failure. The cause of the adverse 1880 conditions may have been associated with the blockage of a major drainage feature to the north of the bluffs, possibly by development of Laity Street in 1879 and 1880. Unfortunately, full records of these conditions are not available.

The review carried out by Northwest Hydraulic Consultants suggests that significant erosion of the underwater slopes should be considered. Our analysis indicate that erosion resulting in horizontal loss of about

15 m at the toe of the slope would reduce the factor of safety by about 10 per cent, to levels of concern.

Failures of a single event or non retrogressive nature, such as the Fir Street slide or the smaller Port Hammond slide, could be triggered by river scour or erosion without adverse ground water levels.

Neither this study nor previous analyses have considered the existing backslopes of the Haney Slide. Although river erosion is of no consequence to these slopes, they should be considered similar to the waterfront slopes with respect to control of ground water and runoff. The slopes are of fragile stability as evidenced by a slide triggered by fill placement some years ago.

5.0 REMEDIAL MEASURES

The potential contributory factors to slope instabiilty can be broadly categorized into the following:

1. Adverse piezometric levels resulting from surface water infiltration and ground water seepage or blockage of drainage discharge.
2. River scour or erosion resulting in a steeper overall waterfront slope.
3. Seismic activity.

Items 1 and 2 are discussed below. Provided the stability of the slope is maintained at an adequate level, seismic activity alone would not be critical.

5.1 Drainage Improvement

The upland area adjacent to the bluffs is flat and naturally poorly drained. We understand that flooding sometimes occurs in response to heavy precipitation.

The existence of granular soil strata near ground surface aids surface water infiltration. Golder Associates' 1983 report identified a drainage system suitable for control of ground water levels in the upper strata as a desirable remedial treatment. The purpose of this system was to intercept and control ground water near the bluff area. However, as the piezometric records obtained since 1982 have shown only minor response to seasonal variations, we believe that this system is unnecessary provided suitable storm runoff is employed.

The most suitable method of enhancing upland water control would be the adoption of a comprehensive municipal program. The following measures should be included as considerations in such a plan. Although some of the following have been employed, they are included for completeness:

- eliminate all septic fields or other ground discharge systems

- provide a storm drainage system to allow removal of all surface runoff from road surfaces, residential and commerical properties

- peak storm runoff must be adequately taken by the storm system so that flooding does not occur. Peak discharge should not be reduced by any system which employs ground water recharge

- all water from perimeter drains, roof drains, roads and parking lots, swimming pools, etc. should discharge to a positive drainage system. No ground discharge systems should be allowed.

- o no surface water should be directed in a southerly direction to the vicinity of the bluffs
- o no storm sewers or other drains should discharge above the crest of bluff slopes, on the slopes, or into ravines on the slopes
- o if open ditch systems are used, no ponding of water should be allowed
- o all buried services installed should be constructed so as to minimize seepage towards the bluff area. Where free draining bedding or other materials are used, care must be taken to ensure that they will not produce adverse effects.
- o adequate maintenance and inspection of buried services should be employed.

We believe that provided the above and other beneficial measures are undertaken, ground water levels can be controlled.

5.2 Erosion Protection

Protection of a portion of the underwater slopes against river erosion should be considered for adequate safety of the study area. Two alternative approaches could be considered:

1. Erosion protection could be placed in the area identified as most critical.

2. The requirement for erosion protection could be determined based on the results of the monitioring program recommended below.

As discussed in previous reports severe erosion would be expected to occur during large flood events. Such events would not generally coincide with periods of maximum ground water pressures which typically occur in January and February. However, if severe erosion did occur it would be important to implement remedial works quickly so that they were in place prior to piezometric increases. If the second option is chosen or the first option delayed, we recommend that remedial designs be prepared for implementation should the need arise.

In either case further river sections should be taken following each one in ten year Fraser River flood event until the effects of scour are defined.

A conceptual design of the required slope erosion protection was prepared and discussed in our previous report. In order to update the costs of such measures, they were reviewed as part of the present study. It is considered desirable to place the protection between sections 11 and 16 with possible extension to section 9. This portion of the bank toe is considered the most prone to erosion.

A typical cross section of the tentative rip rap treatment is given on Figure 4. The tentative rip rap thickness is shown as 2 m to accommodate construction techniques. The use of filter cloth or a well graded quarry tailings filter is considered desirable but difficult to employ. The design of the erosion protection may be modified during final design to accommodate a two-layer system.

The proposed rip rap consists of a well graded rock ranging in size from 25 mm to 750 mm. The rock would be placed by clam from a scow. If employed, the underlying granular filter layer could consist of quarry tailings, or a synthetic filter if special design and construction techniques were used.

The construction considerations were reviewed with Miller Contracting Ltd. who have recently completed a similar construction project at Annacis Island. The use of filter cloth and artificial protection systems were reviewed with Geo Products Inc.

5.3 Cost Estimate

Cost estimates have not been prepared for the proposed municipal storm runoff improvements. We believe that much of the proposed system is currently in place but that moderate modifications and future controls may be required.

The proposed erosion protection is estimated to require 120 m^3 per metre length of bank. It is estimated that the cost of supply and placement of the rip rap would be in the order of $20.00 cu.m. The cost of protection would therefore be $2400 per metre, or $1,080,000 for the intial 450 metres between sections 11 and 16.

5.4 Future Monitoring

It is recommended that future monitoring be undertaken in order to further define the conditions and potentially achieve further economy in remedial requirements and construction.

The following are suggested:

1. Repair existing piezometers as required.

2. Monitor all piezometers during the following months: November through April monthly, plus August and September

3. Compile an inventory of information from homeowners, municipal records and C.P. Rail relative to drainage installation and flooding.

4. Carry out a river sounding survey following each 1 in 10 year freshet event, until an erosion rate is established or to monitor erosion protection installed.

The above data should be reviewed following each 1 in 10 year freshet event or following each 3 years of piezometric monitoring.

6.0 DEVELOPMENT

Previous restrictions to land use or development were provided which considered a risk to all areas between the Port Hammond and Haney slides and extending south of a line joining the backslopes of these features.

Based on the current studies and review of the more recent information, and provided that certain controls and monitoring are adopted, the probability of retrogressive slides occurring which could extend to the former development control limits is low. If ground water levels and runoff are controlled, and development is carried out so as to enhance stability, the critical area with respect to stability may be reduced.

This would allow land use or development to within approximately 100 m of the bluff crest.

Stability of the remaining 100 m wide band, adjacent to the slope crest would then be controlled by river scour and erosion plus local geotechnical constraints. Two alternatives have been put forward to deal with river erosion. Local geotechnical constraints must be assessed on an individual basis considering the proposed land use.

Individual property or development restrictions should include:

1. A nominal minimum setback of all improvements from the crest of the slope of 10 metres.
2. A restriction to control removal of trees and vegetation from the bluff slopes.
3. A restriction to control discharge of runoff, seepage, or other water to the bluff slopes.
4. A restriction to control fill placement on the bluff slopes, in ravines, or at the crest of the slopes.

We would be pleased to assist you with further discussion or interpretation should you require it.

Yours very truly,

GOLDER ASSOCIATES

R.M. Wilson, P. Eng.

T.P. Fitzell, P. Eng.

RMW/TPF/gg
862-1906

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KEY PLAN

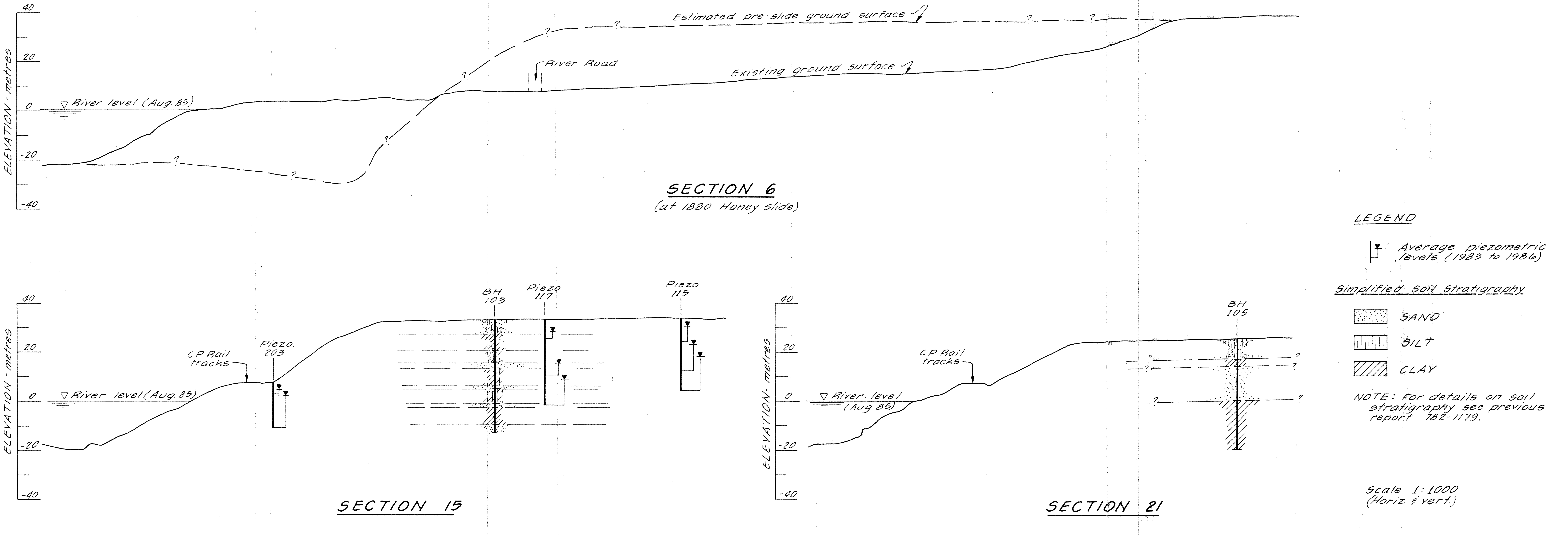
Figure **1**

PROJECT NO. 862/906 DRAWN .... REVIEWED .... DATE ....

SITE PLAN

Figure 2

TYPICAL CROSS SECTIONS | **Figure 3**

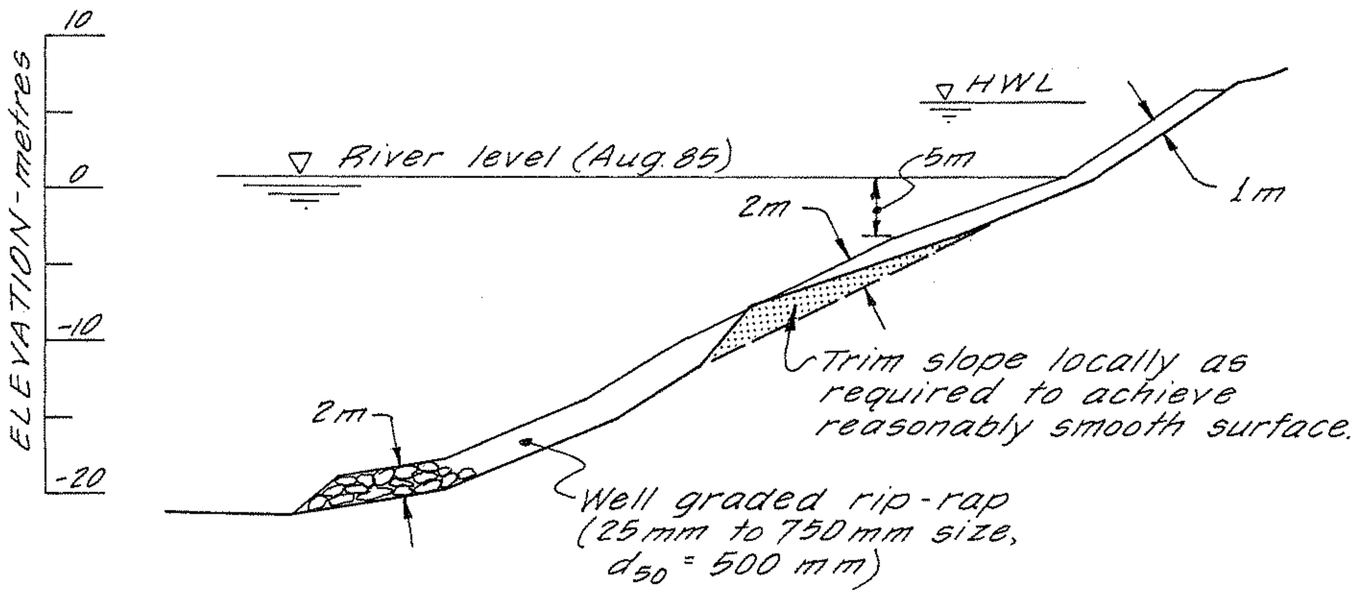


Golder Associates

862 1906

CONCEPTUAL RIP RAP DESIGN - SECTION 12

Figure **4**



SECTION 12 (1985)

Scale - 1:500

PROJECT NO. 862/906 DRAWN N.F. REVIEWED .... DATE ....

APPENDIX A

Northwest Hydraulic Consultants Report

northwest hydraulic consultants ltd.

FRASER RIVER AT HANEY
REVIEW OF CHANNEL STABILITY
AND
PROTECTION AND MONITORING OPTIONS

BRITISH COLUMBIA
MINISTRY OF ENVIRONMENT

FOR

GOLDER ASSOCIATES

VANCOUVER, BRITISH COLUMBIA

by

NORTHWEST HYDRAULIC CONSULTANTS LTD.

NORTH VANCOUVER, BRITISH COLUMBIA

February 1986

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1.0 INTRODUCTION

This report summarizes, from a river/hydraulic engineering viewpoint, results of a February 1986 review and reassessment by Northwest Hydraulic Consultants Ltd. (NHC) of channel stability along the north bank of the Fraser River at Haney, B.C. The review was authorized by Golder Associates, who are reviewing slope stability in the same area as consultants to the B.C. Ministry of Environment (MOE).

The review included the following tasks:

- review and compare results of 1985 channel surveys with surveys from previous years;
- site visit to observe any apparent changes;
- reassessment of the overall situation using the latest available information and giving new thought to alternative feasible actions;
- discussing results with Golder Associates.

To avoid duplication, this report deals primarily with new information, and therefore assumes the reader is familiar with both the river and previous reports (see references). However, for the sake of continuity, conceptual information that is salient to conclusions is re-presented.

2.0 FIELD INFORMATION

2.1 Field Survey

In August 1985, MOE completed a new channel survey which produced cross sections at the same nominal locations as previous surveys in 1981 and 1978. Their 1985 survey was controlled horizontally using more accurate equipment than was used for the two previous surveys, which means the later survey was likely more accurate in terms of the longitudinal positioning of cross sections. (However, this difference in accuracy does not affect our general conclusions which are based on a comparison of the river cross sections.) Vertical accuracy is believed to be the same in all instances. Both plotted and tabulated sections were provided to NHC such that comparisons between all three surveys could be readily made.

We have reviewed the new survey and conclude that compared to 1978 and 1981 surveys, the 1985 survey shows there has been no significant erosion trend in the area.

2.2 Field Inspection

The site was visited by M.H. Okun on 13 February 1986, who inspected the north bank between cross sections 5 and 21. Based on this inspection and comparison with previous photographs and notes, there was no evidence of any new erosion in this reach. In fact, at almost every location, the same pieces of riprap, logs, trees and even garbage were in the same position in 1986 as they were in 1983 (when the last inspection was made). It also appears that C.P. Rail has not added any more riprap in this reach over the past three years. A photographic log is on file with NHC.

3.0 DISCUSSION

From a river engineering viewpoint, the first question that must be reviewed is what would be the expected rate of future right bank erosion along the reach in question? The second question is, assuming that erosion will still occur, what can be done to accommodate it?

In the following discussion, erosion refers to a horizontal process whereas scour refers to a vertical process. However, it is important to realize that river bed scour can induce bank erosion by causing the banks to slump.

3.1 Future Erosion and Scour

The Fraser River is a flowing mass of water and sediments that are transported according to the energy available. For a given set of hydraulic conditions, the river will carry a resultant amount of sediments, either in suspension or along the bed and lower banks. At any point along the river, the amount of water and sediment that are transported is dependent, amongst other things, on the local velocity. As the flow of the Fraser River approaches and moves through the bend at Haney, it accelerates so that local velocities become higher - especially at the outside (north) of the bend - than in adjacent straight reaches. At the same time, secondary currents that result in a helical-type flow pattern are generated; again, this would not be common in adjacent straight reaches. The result of these two phenomena is that the hydraulic forces - i.e. those of the moving water - are stronger in the bend. This increased strength means that the river will attempt to pick up additional sediments in areas of locally higher velocities, and deposit them in areas of locally lower velocities. This is typically seen by the triangular shape of the channel cross section, which is shallow on the inside (lower velocity) and deep at the outside (higher velocity) of the bend.

The repeat sounding surveys have shown no significant erosion has taken place at the bend between 1978 and 1985. However, it should be realized that the magnitude of the freshets during the period of these surveys have been unusually low. For example, the largest freshet between 1978 and 1985 occurred in 1982 and had a return period of only three years. All of the other freshets in this period had return periods of less than 1 1/2 years. Therefore substantially greater changes might have been measured if a 10-year to 20-year flood had occurred during this period.

Based on the available historical map data, it appears that except for the bank formed by the toe of the 1880 slide, the right bank of the Fraser River at the Haney Bend has not moved appreciably over the last 50 years (NHC, 1977). Yet this bank is on the outside of a bend where bank erosion would normally be expected to occur. Part of the answer to this apparent dilemma lies in the occurrence and location of the 1880 slide: it occurred at the point of maximum attack by the river, and the volume of material left in the river acted as a natural protection device against river forces for some distance downstream. The toe of this slide has retreated approximately 1 m per year over the last 50 years (NHC, 1977), so that this sheltering effect will be less significant in the future than it has been in the past. Another part of the answer appears to be that the river, because it was unable to pick up material from the banks, has done so off the bed. Evidence of this is that:

- the maximum depth at this bend is about the largest in the Fraser River between Hope and the delta;

- the thalweg (locus of deepest point) of the river **rises in the downstream direction** about nine metres between cross sections 8 (in the bend) and 28 (where the river is fairly straight); this indicates the strength of the river's erosive forces at this bend.

Seeing what has and is happening then, it is not simple to predict a scenario for future erosion and scour over say the next 50 years. However, the following comments are appropriate:

(1) Based on geotechnical information, the north bank of the river consists of material that is erodible; the layering of clays or silts in between sands and gravels suggests that the erosion process would be slower than the case of sands alone. Nevertheless, it must be assumed to be erodible.

(2) The river is already quite deep and apparently remains deep throughout the year. A very preliminary estimate suggests that the maximum scour that might occur at this bend is not significantly more than observed during the surveys. A rough estimate is that another 3 to 7 metres could occur during more extreme floods; this is a relatively small number for the conditions of this location. On the other hand, it is possible that very little future scour would occur since the river bottom may quickly approach stiff clay which would be resistant to scour.

It is concluded that without providing protection, it must be expected that erosion of the north bank will occur in the future. It is most likely that in the worst case, erosion in the order of up to 10-15 metres could be expected during or shortly after a major flood event and this would happen in the May to August period.

It should be noted that relatively rapid bank erosion in excess of 10 - 15 m has occurred along some previously stable bends on the Fraser River. For example, the large bend between Sumas Mountain and Matsqui experienced rapid erosion between 1982 and 1984 following a long period of apparent stability. This bend has several similarities to the Haney site as the bed is composed predominately of sand and the bank material is composed of silts and clays.

3.2 Options for the Future

Alternatives for protecting the north bank against erosion were presented in NHC's 1983 report. In our opinion, there is no new technology that would be feasible for this situation. Except for cost updating (which has been done by Golder Associates), these alternatives are still available, and the recommended method -

conventional riprapping - would represent a conservative and the most proven approach. However, final selection of protection works, if required, would require a detailed cost and construction feasibility comparison; the latter should be done in consultation with experienced contractors.

Provided erosion in a single event in the order of up to 20 metres would not trigger a slide, then the option of delaying construction of erosion protection works and continuing monitoring is still available. Due to the relative inactivity observed over an eight-year period, it would be acceptable to reduce the future monitoring process (surveying cross sections) to taking place immediately after larger floods - say greater than a 10-year return period.

The timing of the monitoring process is important in that there may be very little time available between discovering that significant erosion has occurred and being able to act accordingly. Therefore it is recommended that a contingency plan - including preparation of detailed construction drawings for erosion protection works - be developed in 1986. With this done, the following monitoring sequence is recommended, effective in 1986:

(1) Monitor the magnitude of flood peak annually. Using the MOE forecast, prepare to mobilize a survey crew for flood peaks predicted in excess of the 10-year event.

(2) If monitoring the flows at Hope indicates a flow beyond the 10-year event, immediately mobilize a survey crew to take soundings at the same locations and in the same manner as was done in 1985. It is expected that such surveys would typically be done between May 15 and August 15, most likely in late June or early July.

(3) As soon as the survey is completed, undertake a preliminary review to establish if new erosion is evident. If so, final survey results should be done as soon as possible.

(4) Assess impact of erosion, and decide on action plan. If construction of erosion works is advised, then implement contingency plan immediately.

Finally, NHC's previous recommendation to obtain selected velocity profiles should be considered.

REFERENCES

(1) Report to British Columbia Ministry of Environment, Water Investigations Branch on Ground Water Monitoring and Stabilization Study Fraser River North Bank, Maple Ridge, British Columbia, 1983.

(2) "River/Hydraulic Aspects of Fraser River Bank Stabilization, Haney, British Columbia", for British Columbia Ministry of Environment through Golder Associates, Northwest Hydraulic Consultants Ltd., March 1983.

(3) "Haney By-Pass, Fraser River Regime Study", letter from NHC to Golder Associates, 15 December 1977.

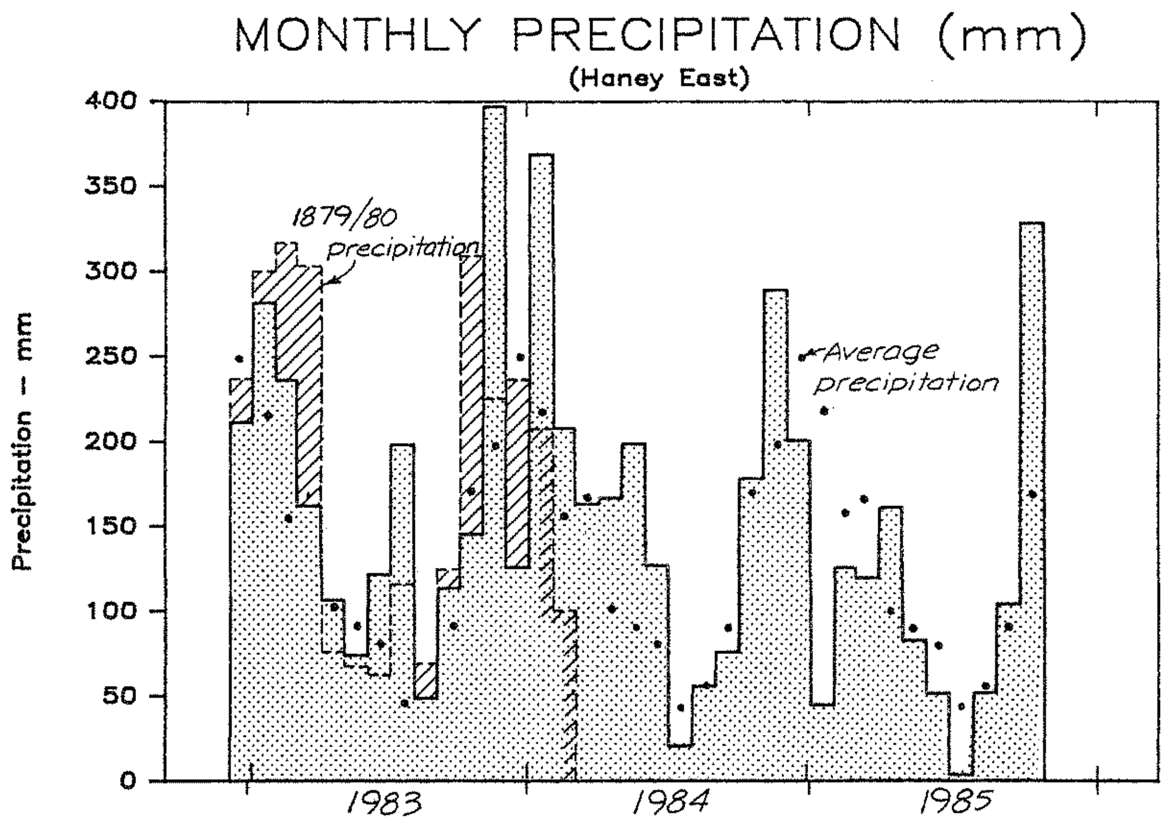
(4) "Fraser River - Haney to Port Hammond, Bank Erosion", letter from NHC to Golder Associates, 23 March 1979.

APPENDIX B

Precipitation and Piezometer Data

SUMMARY OF PRECIPITATION DATA

Figure **B-1**



	Yearly Total	Total (Oct. to Jan.)
1879	2199.1	980.9
1983	2012.2	1037.6
Average	1630.7	839.7

NOTE

1879-80 data obtained by correcting data collected at Langley based on average total precipitation.

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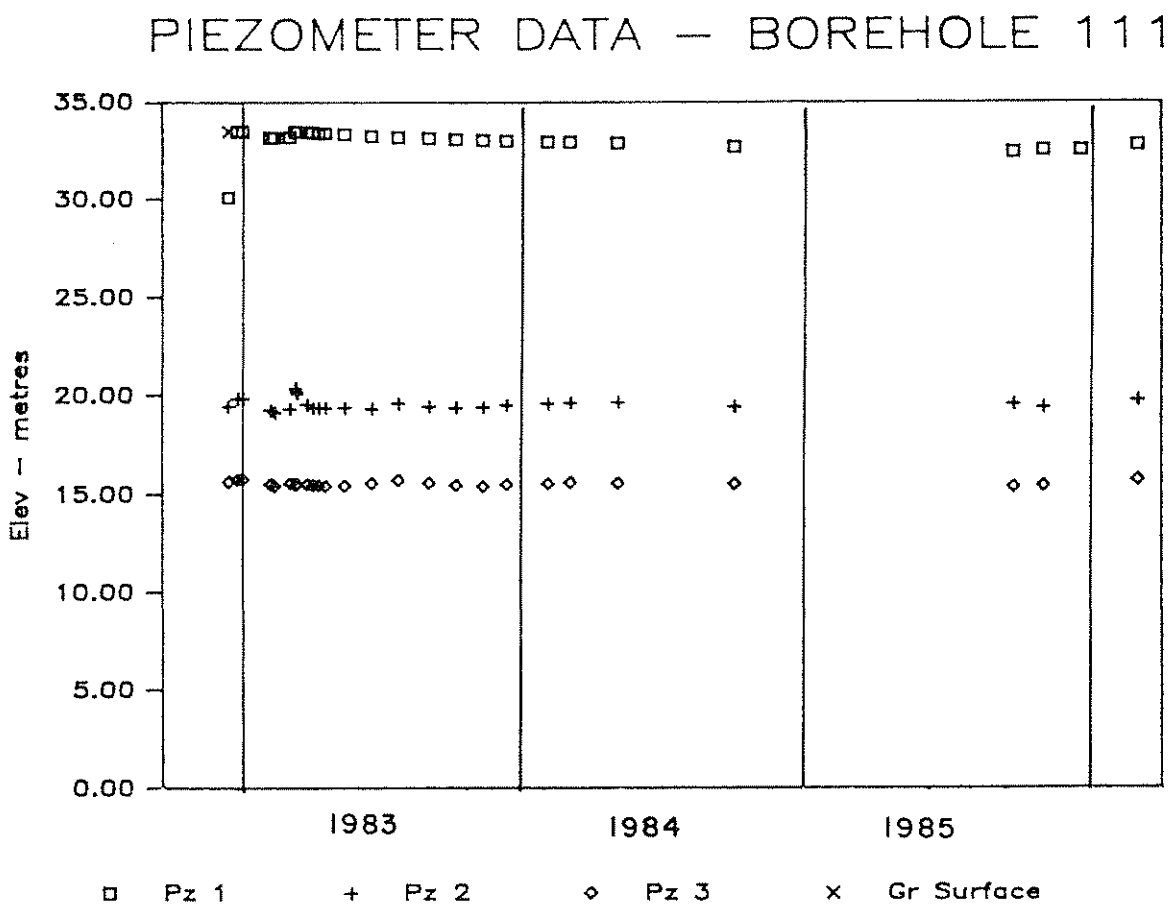
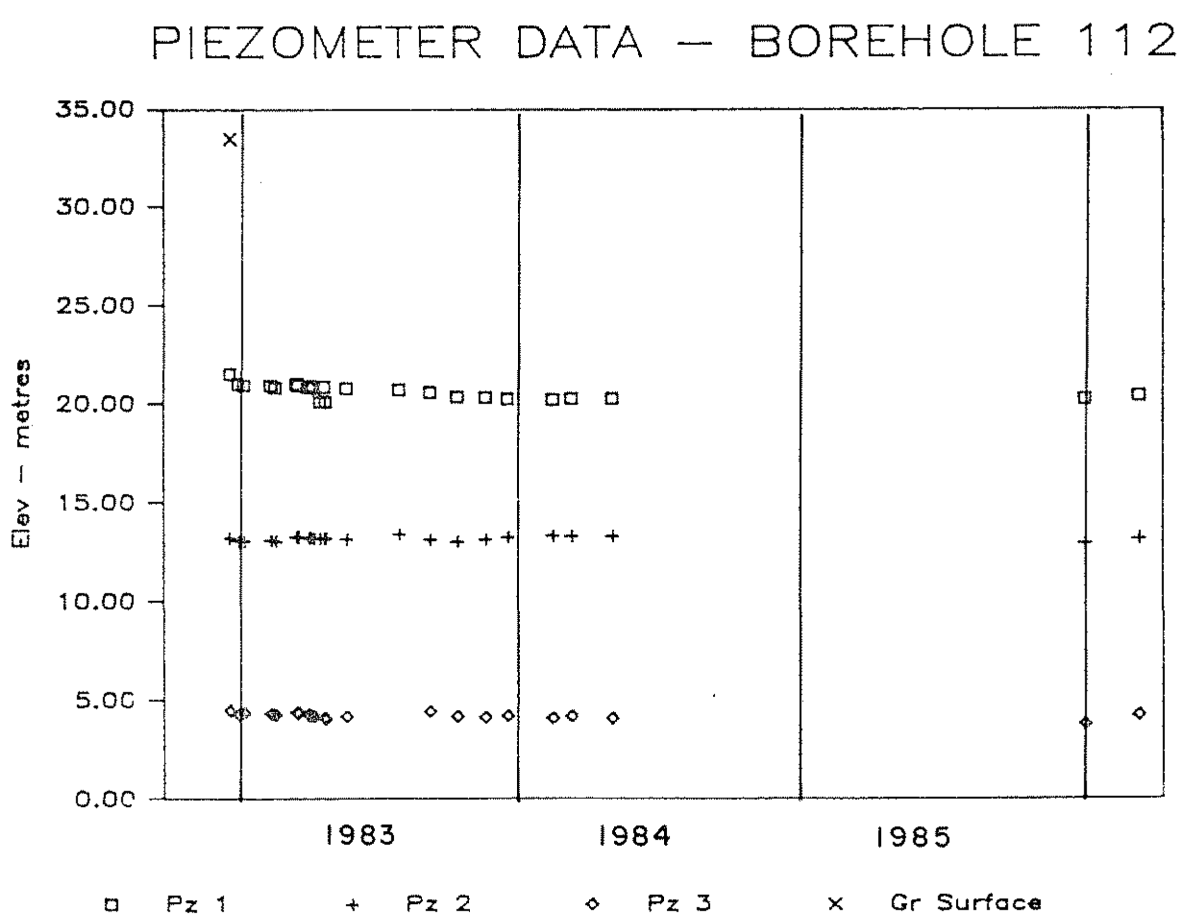
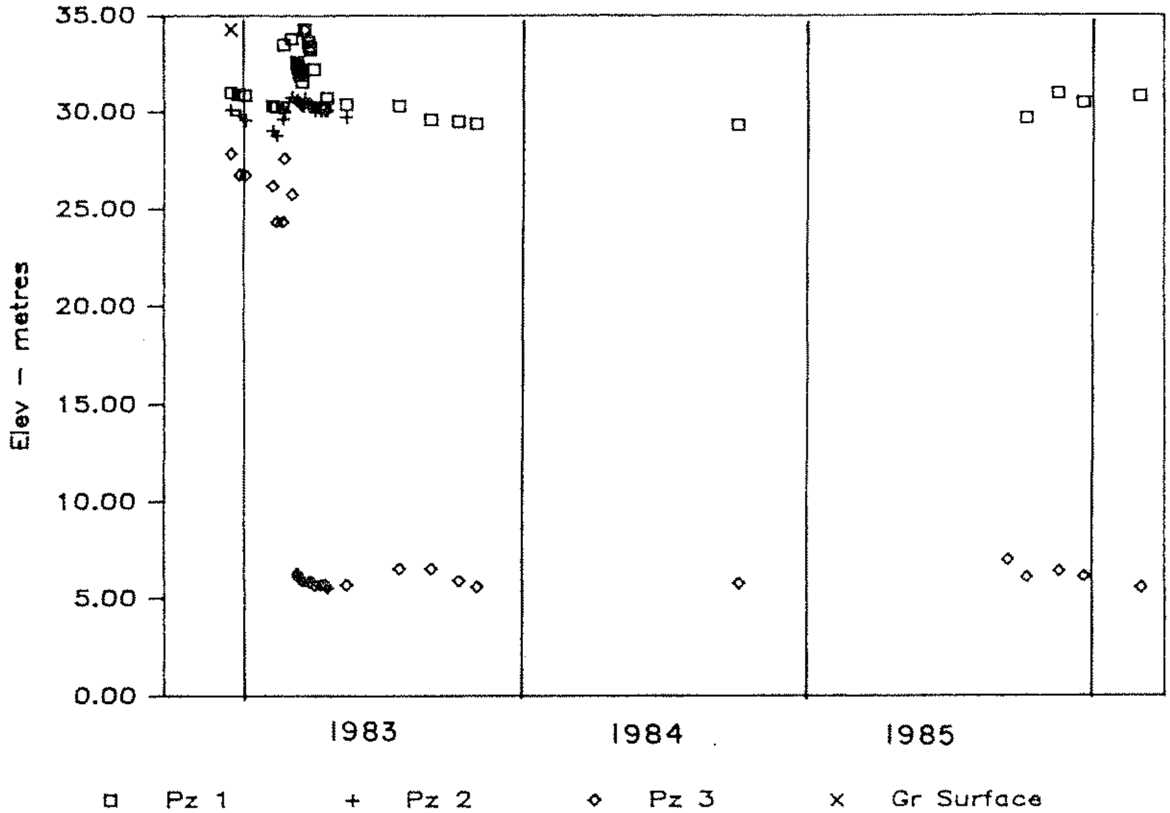


PIEZOMETER DATA — BOREHOLE 111
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev — metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface

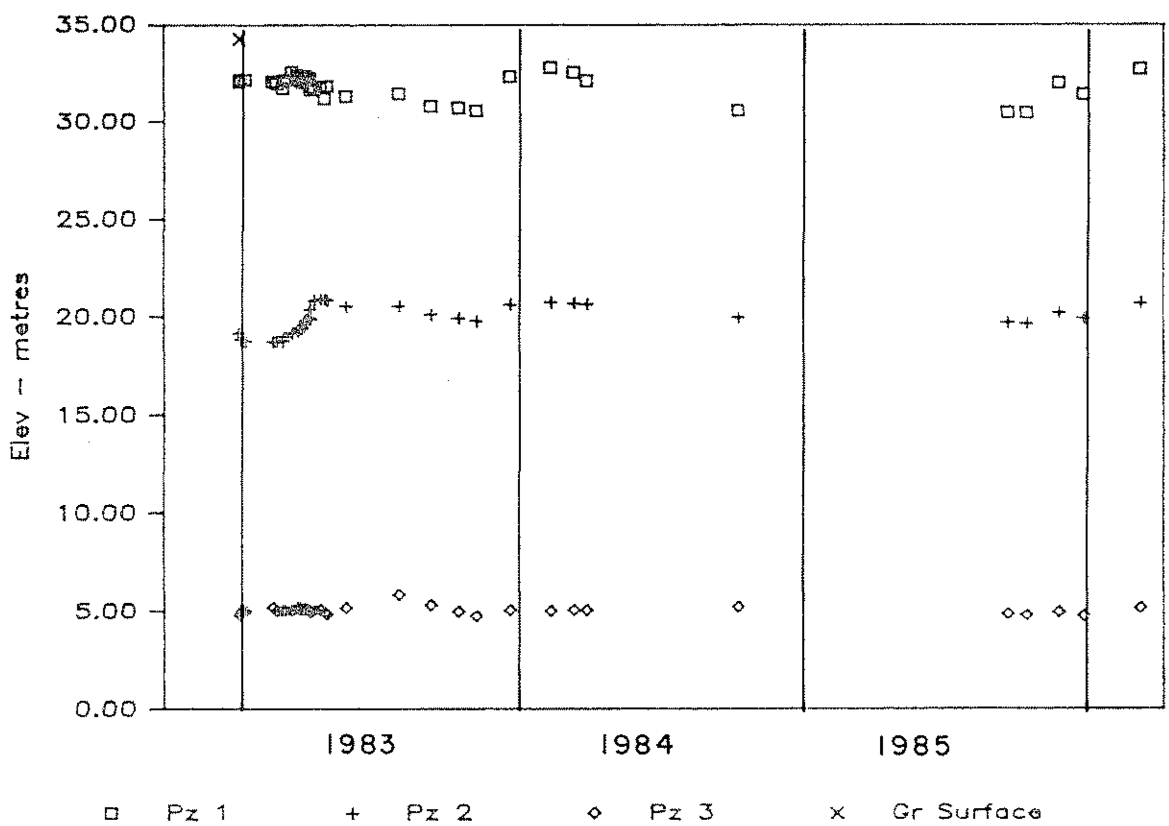


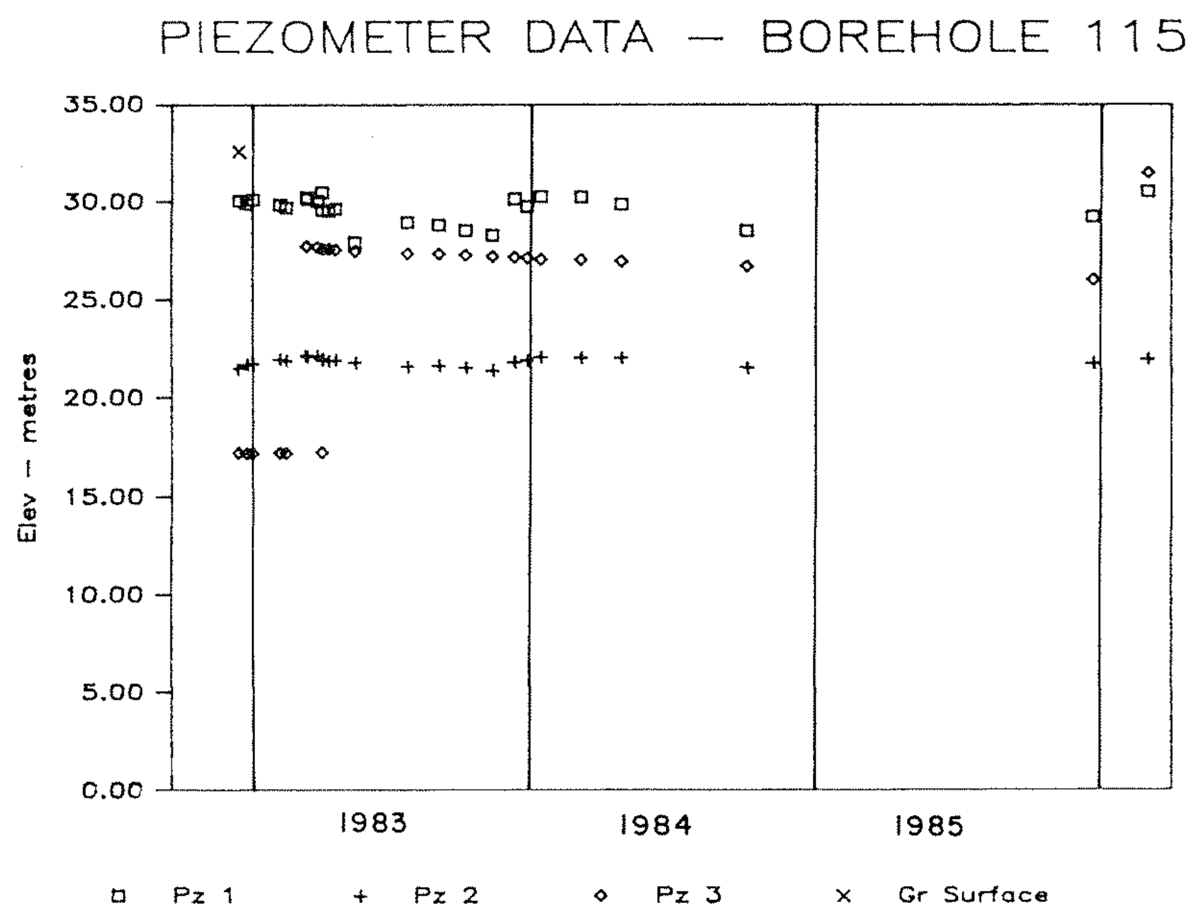
PIEZOMETER DATA — BOREHOLE 112
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev — metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface

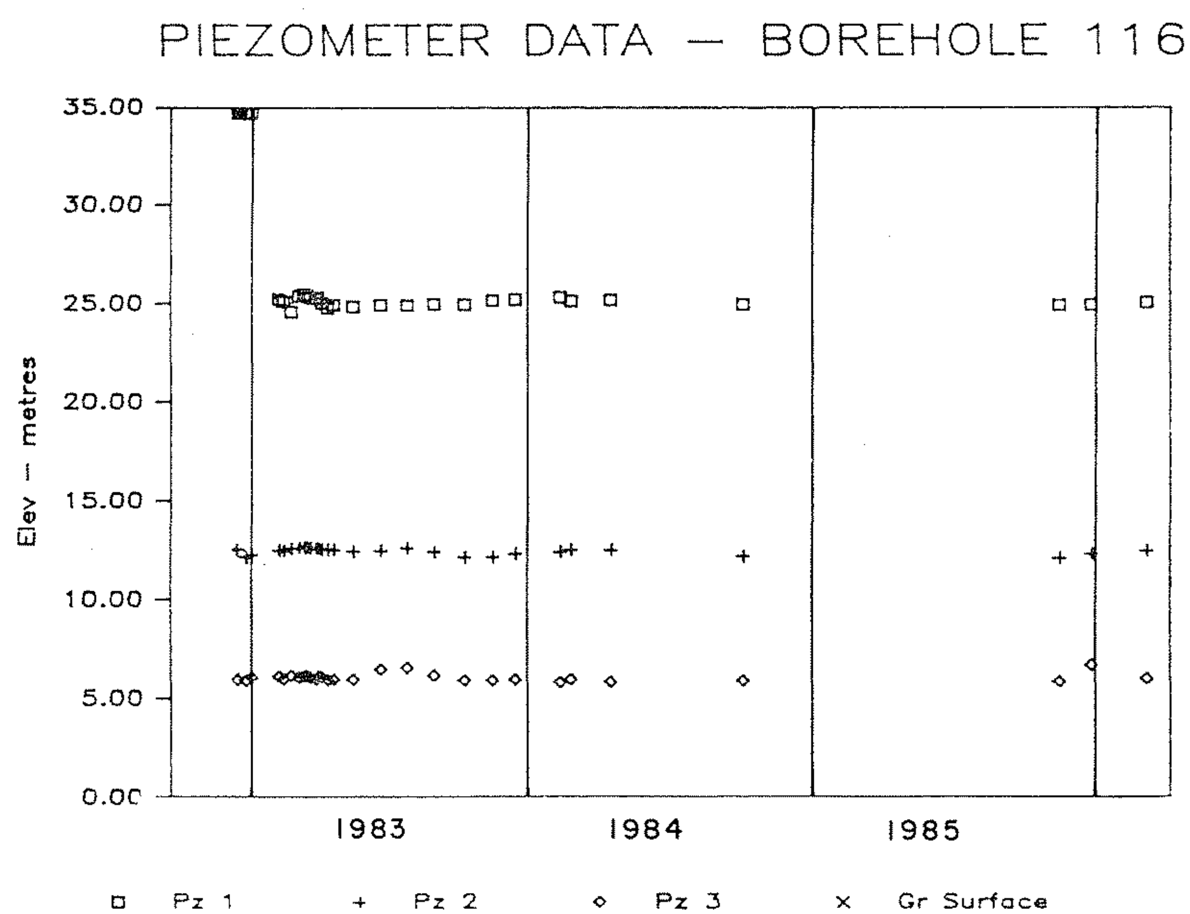
PIEZOMETER DATA – BOREHOLE 113

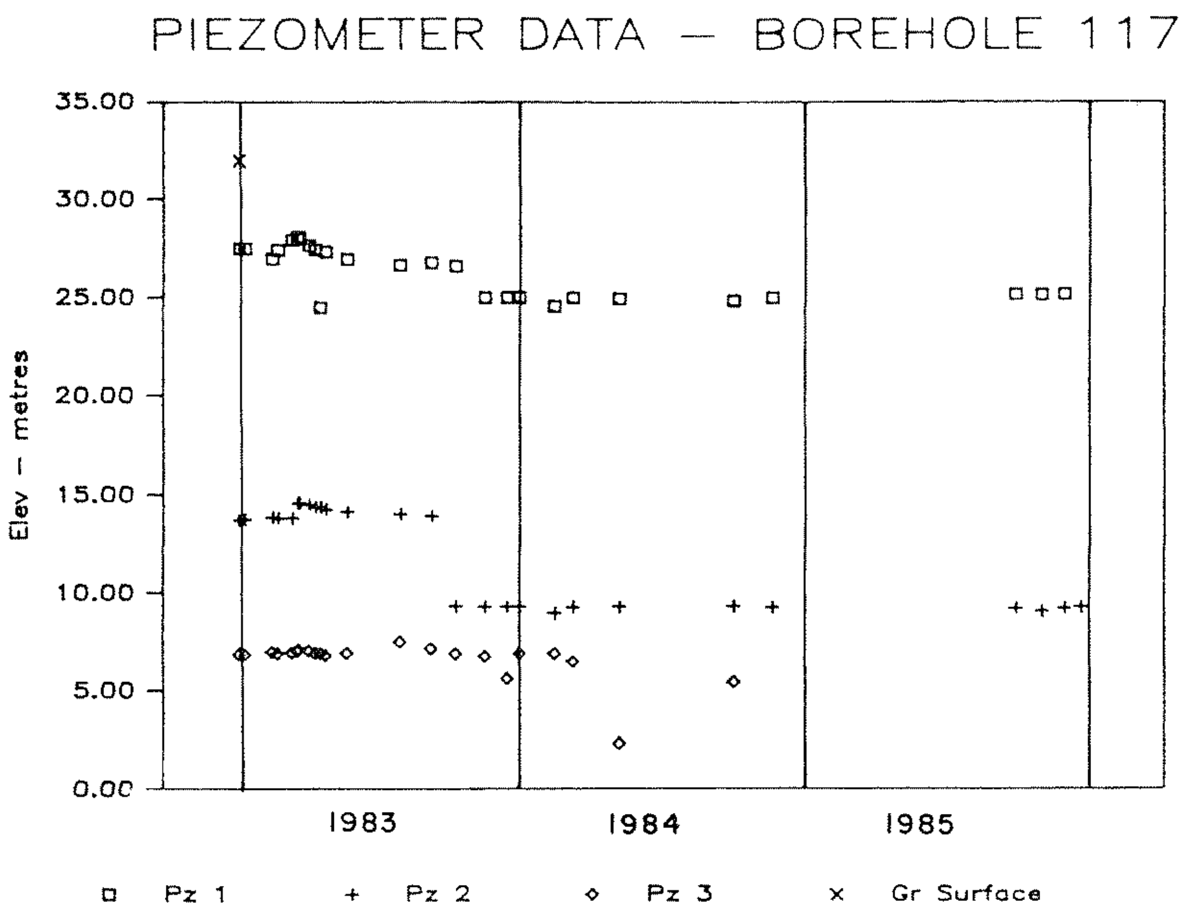


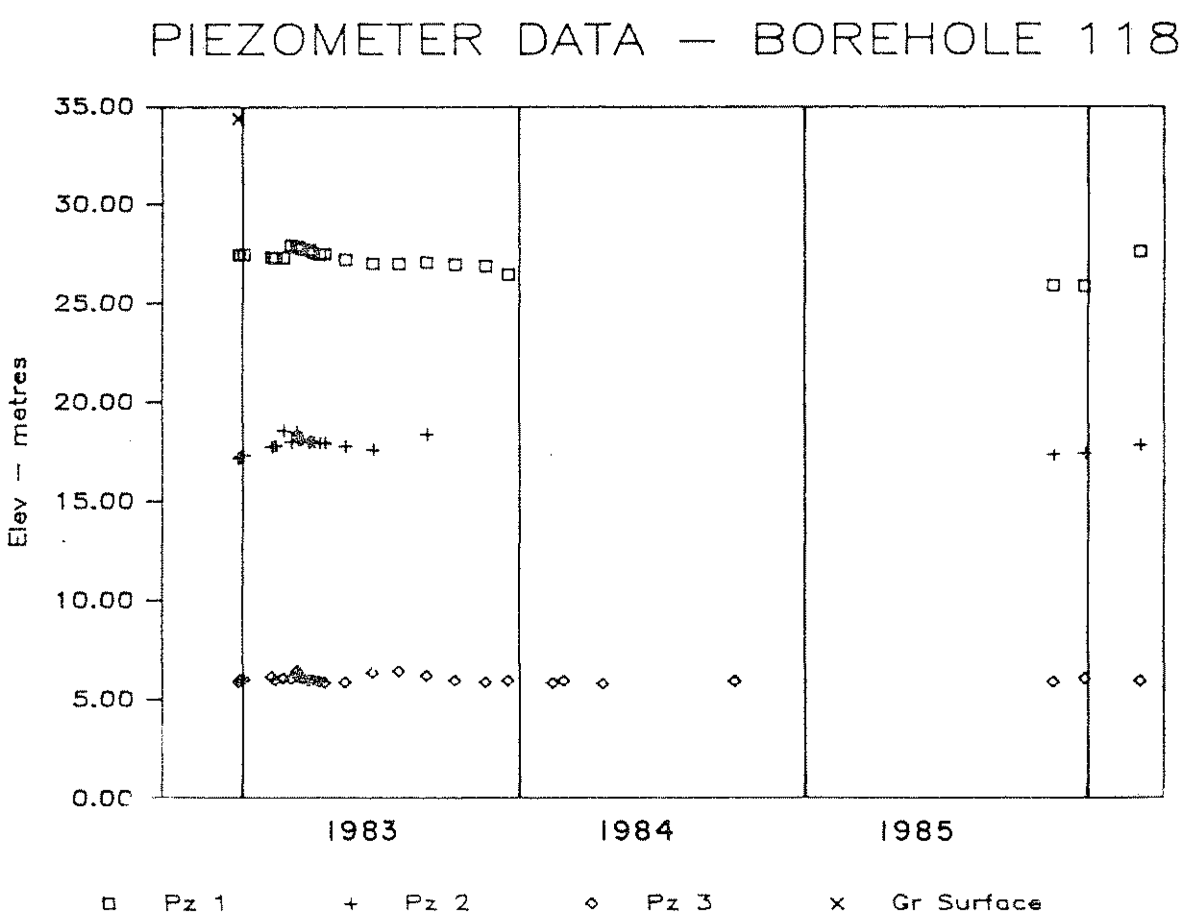
PIEZOMETER DATA – BOREHOLE 114



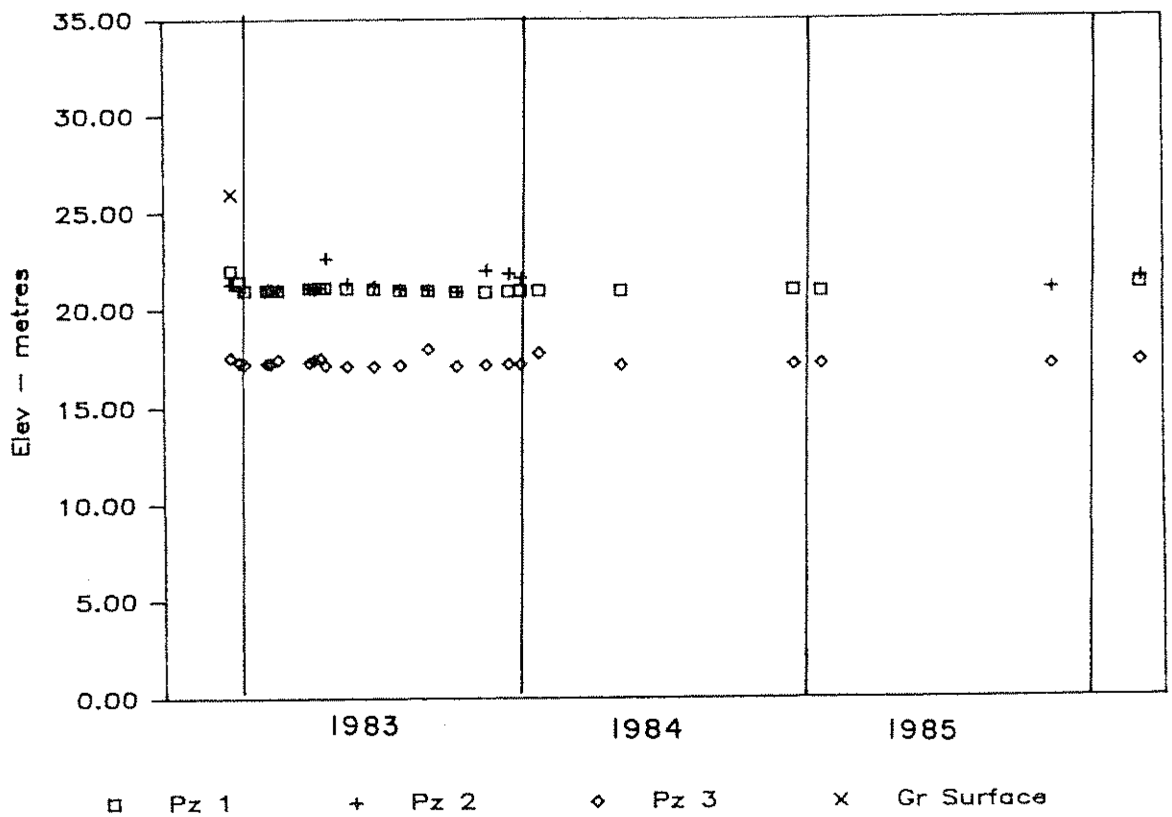

PIEZOMETER DATA — BOREHOLE 115
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev — metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface


PIEZOMETER DATA — BOREHOLE 116
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev — metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface

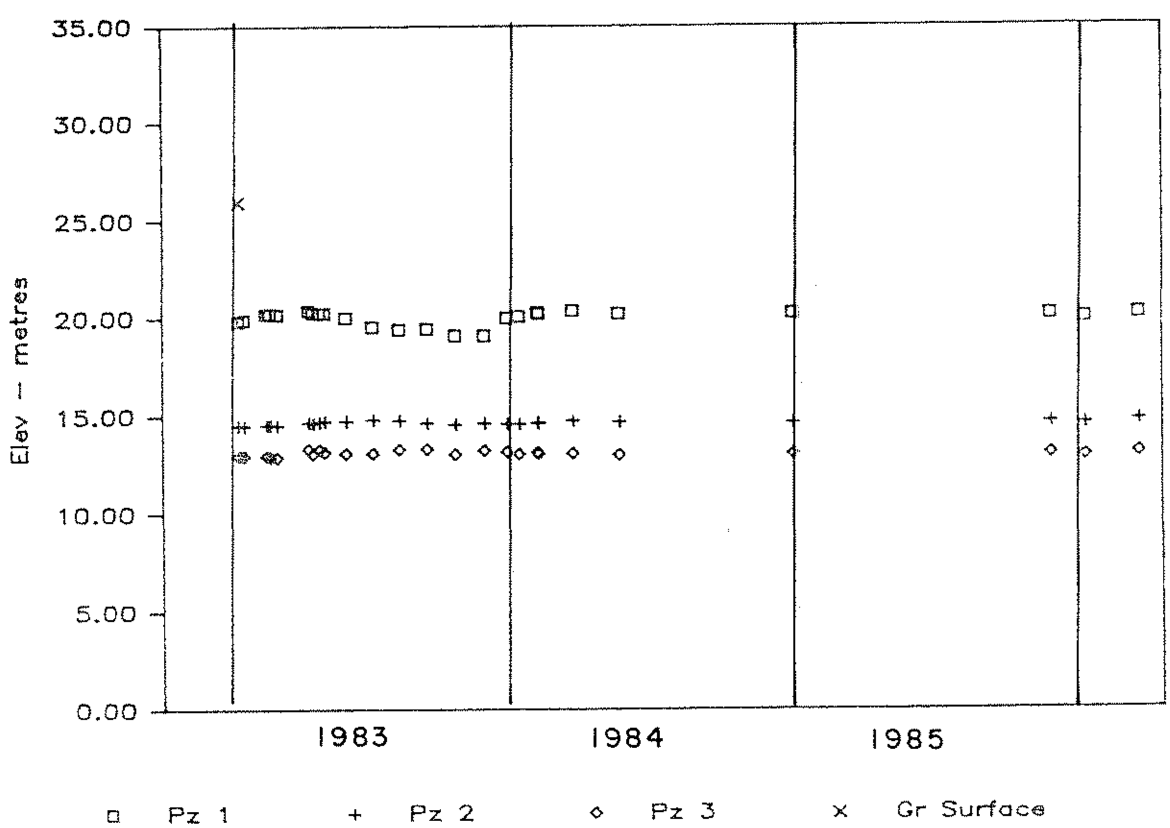

PIEZOMETER DATA — BOREHOLE 117
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev – metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface


PIEZOMETER DATA — BOREHOLE 118
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elev – metres
1983
1984
1985
Pz 1
Pz 2
Pz 3
Gr Surface

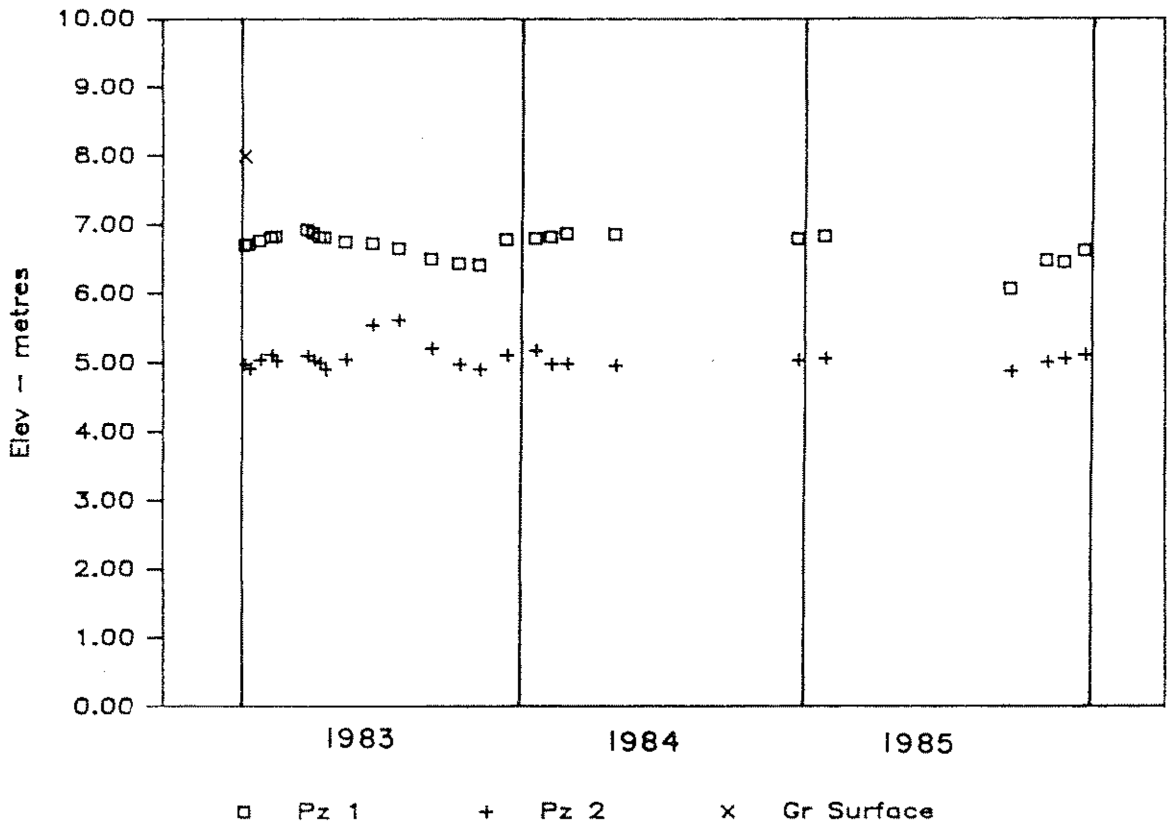
PIEZOMETER DATA — BOREHOLE 119



PIEZOMETER DATA — BOREHOLE 120



PIEZOMETER DATA – BOREHOLE 201



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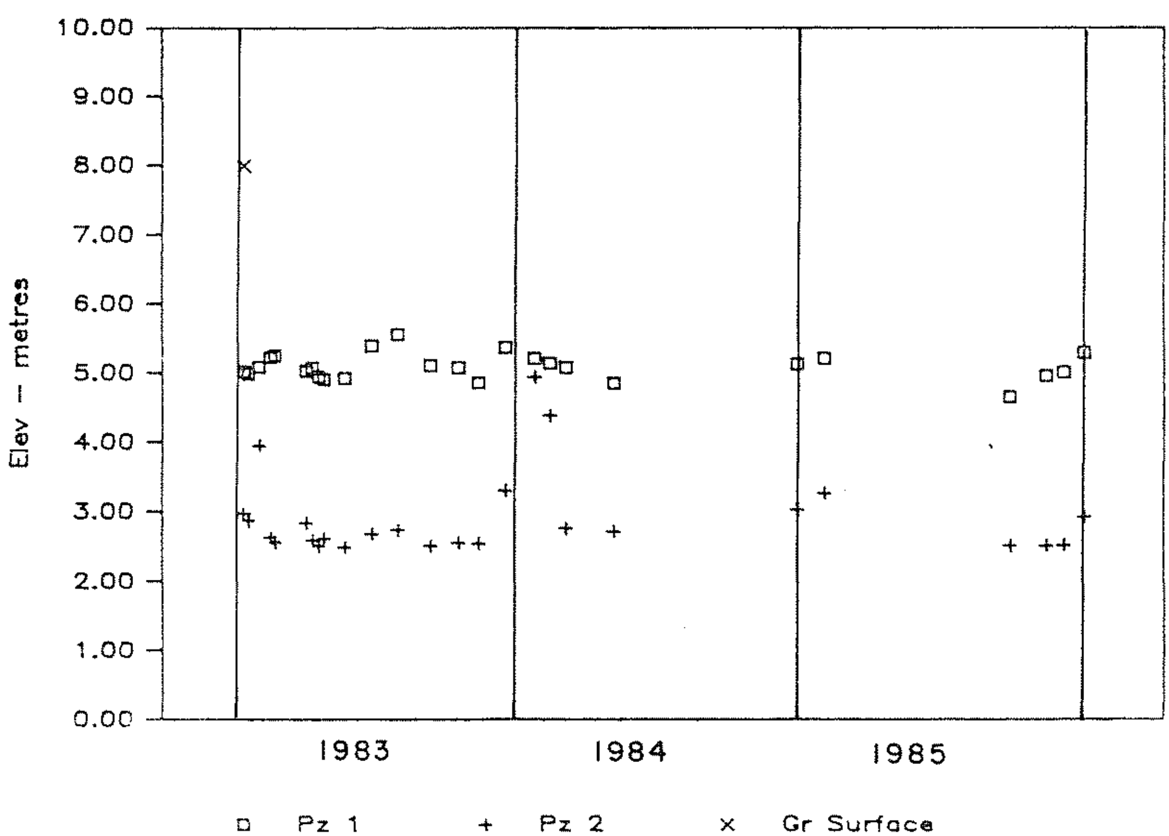
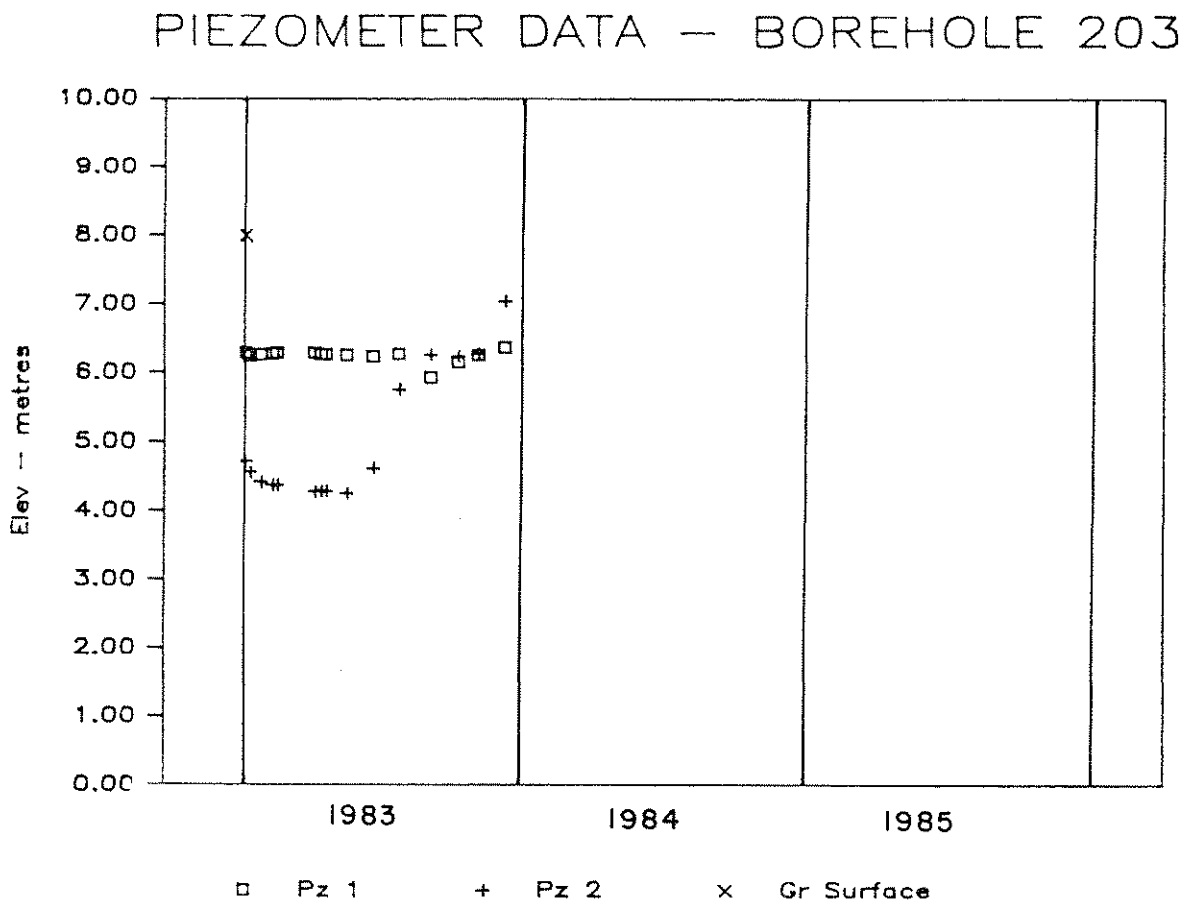
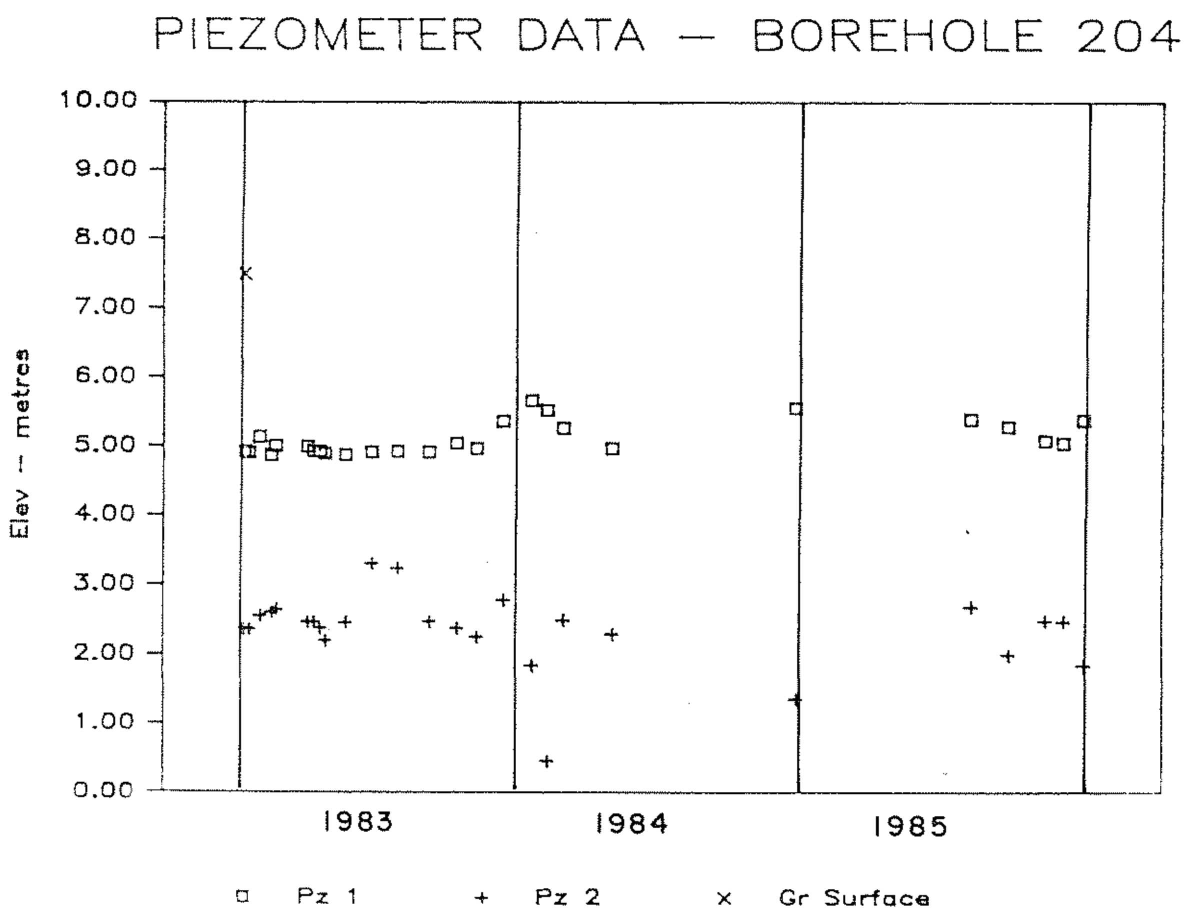



PIEZOMETER DATA — BOREHOLE 203
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Elev -- metres
1983
1984
1985
Pz 1
Pz 2
Gr Surface


PIEZOMETER DATA — BOREHOLE 204
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Elev -- metres
1983
1984
1985
Pz 1
Pz 2
Gr Surface

PIEZOMETER DATA — BOREHOLE 205