APPENDIX I

GOLDER REPORT TO MINISTRY OF ENVIRONMENT
AUGUST 1979

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

COVERING LETTER
TO
MINISTRY OF ENVIRONMENT
WATER INVESTIGATIONS BRANCH
ON
STABILITY STUDY
FRASER RIVER NORTH RIVER BANK
HANEY TO PORT HAMMOND, BRITISH COLUMBIA

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Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

August 14, 1979

Province of British Columbia
Ministry of the Environment
Water Investigations Branch
Parliament Buildings
Victoria, B.C.
V8V 1X5

ATTENTION: Mr. P.M. Brady, Director

Re: Stability Study
Fraser River North River Bank
Haney to Port Hammond

Dear Sir:

We are pleased to present our report on the "Stability Study of the North Bank of the Fraser River from Haney to Port Hammond". The main report is presented under separate cover and contains a description and findings of our site investigation work and engineering analysis and identifies areas with similar stability characteristics.

The discussion of the findings of the study and our conclusions, together with cost estimates for monitoring and remedial measures, are presented in this covering letter. This covering letter should be read together with the text of the main report. The following sections contained in this letter follow the section numbering of the main report.

9.0 DISCUSSION AND CONCLUSIONS

The slopes in the study area are known to be susceptible to landslides. One of the primary causes of slope instability is river erosion at the toe of the slopes. This results in a gradual removal of the submerged slope leading to over-steepening and local failures. However, this process is slow and the large landslides which have occurred in the past have probably been triggered by either:-

a) high piezometric levels in the slope,

or, b) earthquake effects.

High piezometric levels would be most likely to develop after a period of extremely cold weather, when freezing of the sand layers on the upper slope face could occur. Such frozen zones could effectively dam the water within the slope resulting in high piezometric levels. This hypothesis agrees well with the climatic conditions at the time of the Haney slide, i.e. freezing weather, followed by heavy rains. Slides of this type would therefore be most likely in the period between December and February. Earthquake-triggered landslides could occur at any time, independent of weather conditions. However, an earthquake would be more likely to initiate a slide at time of high piezometric levels. If an earthquake occurred at a time of low piezometric levels, a slide may not result.

There is a significant possibility that single slides could retrogress to form much larger slides. The potential for retrogression and the magnitudes of slides which would be formed will depend on the slope stability prior to failure and the slide trigger mechanism. Our best estimates of the plan geometry of future single and retrogressive slides are shown on Figure 2. These geometries are based on previous slide geometries and stability analyses.

Based on our stability analyses and our evaluation of past landslide activity in the area, we have sub-divided the study area into zones of varying degrees of stability (see Figure 2).

9.1 Zone 'A'

The existing slope has very low factors of safety in the range 1.05 to 1.20. High piezometric levels or an earthquake would reduce the factor of safety to less than 1.0, resulting in failure. River erosion is concentrated along this section of the study area. The slope has poor lateral drainage, with no significant drainage gullies or ravines present.

We expect that single failures would extend about 30 m back from the slope crest, and could involve some structures. However, based on the low present factors of safety, it is possible that failures in this area could retrogress a significant distance back from the slope crest, and could affect up to 50 properties. Typical single and retrogressive failures which could be expected are shown on Figure 2.

This section of slope is therefore classed as 'poor'. The slope requires remedial measures in order to ensure that future landslides do not occur. We recommend that the remedial measures should initially involve monitoring of piezometric loads for one year, to define changes in piezometric levels. Following this period, drainage measures should be installed, as discussed in Section 11.2.

9.2 Zones 'B' and 'C'

This section of slope has factors of safety generally in the range 1.2 to 1.4. High piezometric levels or an earthquake would reduce these factors of safety to about 1.0 to 1.2. River erosion is taking place along the bank in Zone 'B', although no erosion is occurring in Zone 'C'. There are a number of drainage gullies or ravines intersecting the slope, which would tend to limit high piezometric levels in the slope.

We would expect that future river erosion, together with high piezometric levels or an earthquake in these areas might result in landslides. The landslides would, however, be less likely to retrogress than in Zone 'A'. The stability of these slopes is therefore classed as 'poor' in Zone 'B' and 'fair' in Zone 'C'. We recommend that river erosion rates and piezometric levels should be monitored in these areas. Based on this data, it should be possible to define any remedial works which are required.

9.3 Zone 'D'

These sections consist of river bank slopes at the toes of the existing large slides and low lying river bank to the east of the Haney Slide. The river bank slopes may be subject to small-scale single failures, due to river erosion at the toe. These failures would generally not tend to retrogress. The stability of the overall slopes in these areas is classed as 'good'.

10.0 MONITORING

We recommend that a program of monitoring should be instituted on the slopes in Zones 'A', 'B' and 'C'. The monitoring should define the rate of river bank erosion in the area, and the range of piezometric levels within the slope.

10.1 River Bank Erosion

This monitoring should be performed to establish the present rate of river bank erosion. It is possible that the erosion rate is extremely slow, so that erosion protection would not be justified considering the design life of the structures at the slope crest.

The following program is recommended to monitor the erosion at the site:-

a) Re-survey the cross-sections annually, at the same locations as was done in 1978. It is preferable to survey at or near spring freshet conditions (e.g. late June, early July). The cross-sections should be carried up to a stable reference point - such as the railway - so that changes in horizontal distance from waterline can be measured.

b) Visit the site annually and take representative photographs to compare with previous years. Any significant changes or local erosion pockets should be noted.

10.2 Piezometric Levels

It will be necessary to install a series of piezometers to establish more accurately the piezometric levels within the slope.

We anticipate that 5 or 6 typical sections would be monitored. The instrumented sections would be concentrated in the poor zones, with at least 2 or 3 sections located in Zone 'A'. Each section would contain at least three instrumented boreholes, containing piezometer sets. The boreholes would be located on the CPR bench, at the slope crest, and about 50 m back from the slope crest. Each borehole would contain 2 or 3 piezometers, so that a typical instrumented section would contain about 6 to 9 piezometers. The boreholes would be cased and logged during drilling to obtain additional stratigraphical information, and to ensure that the piezometers are located within permeable strata.

Water levels in the piezometers should be monitored on a twice-monthly basis during the winter and spring months and monthly during summer and fall. This should enable an assessment to be made of variations in the piezometric levels. The piezometers would be useful in determining the influence of any remedial drainage measures installed at the site. This monitoring system could also, to some extent, be used as a warning system.

11.0 REMEDIAL MEASURES

We recommend that a program of remedial measures should be instituted in Zone 'A', in order to improve the stability of the existing slopes. The actual design of remedial measures is beyond the terms of reference of this report. However, some comments regarding conceptual remedial measures which could be considered are given below.

11.1 Erosion Protection

Erosion protection could be provided by placing rip-rap down the face of the submerged slope. The rip-rap would have to extend at least 30 m out into the river channel, to prevent undercutting. The rip-rap should consist of angular, durable, rock fragments. If we assume that the rip-rap layer would have to be at least 2 m thick, then about 1 million cubic meters of material would be required to protect Zone 'A' alone. An alternative to rip-rap protection would be the provision of groyne structures. These could consist of steel sheet pile or concrete structures projecting into the river.

Either of the above alternatives would prove a very costly undertaking and could also result in modifications to the river patterns to the downstream of the protected bank. A detailed hydraulic study would have to be performed to ensure that extensive erosion downstream would not result. We therefore recommend that no erosion protection should be undertaken until erosion rates have been monitored for several years. As previously noted, it is possible that the river bank erosion rates are sufficiently slow that protection would not be justified.

11.2 Drainage Measures

A less expensive method of increasing the stability of the slopes would be to install drainage measures to maintain piezometric levels below a specified level. We recommend that this type of system should be installed in Zone 'A', following monitoring of piezometric levels for one year. As noted in Section 8.3, the factors of safety of the slopes in this area would then be increased to at least 1.25. The slopes would then be generally stable even under seismic conditions.

An important concern will be to establish a realistic target level for the upper limit of piezometric levels. It will also be important to ensure that the resulting water table drawdown does not result in general settlement of the area adjacent to the slope crest. For these reasons, it would be preferable if the installation of the system could be delayed until the piezometers had been monitored for at least one year.

a) Pumped Wells

A suitable system would consist of a series of wells drilled at about 50 to 100 m spacing, about 50 m back from the slope crest. The actual spacing of such wells would be best determined after the initial monitoring period and a field pump test. The wells would be equipped with submersible pumps to maintain the ground water at a specified level, e.g. 15 m below the ground surface. The wells would probably be connected to a concrete header pipe and discharged into the storm sewer system. A subsidiary pumping station may be required. These wells would be relatively inexpensive to install, but would be expensive to maintain.

b) Gravity Drainage by Tunnelling

A long term alternative would consist of a gravity drainage system. This could consist of a drainage gallery, driven from the CPR bench. The gallery would have to be drilled using a tunnelling shield and would be provided with a permanent liner. A series of vertical sand drains would then be installed from the slope crest, down into the drainage gallery. Seepage in the sand layers would then be intercepted by the vertical drains and would drain into the gallery. The gallery would be connected to a culvert located beneath the CPR tracks, and would drain into the river.

c) Inclined Slope Drains

A less costly long term alternative would consist of a system of inclined slope drains. The slope drains would be driven from the toe of the upper slope and along the C.P. Rail bench. They would extend up to 100 m into the slope at an inclination of about 10 degrees to the horizontal in order to intercept the sand interlayers present over the height of slope. The slope drains would consist of a system of sand drains within a perforated pipe that would be connected to a discharge system along the toe of slope. It will be necessary to install the slope drains below the local depth of frost penetration or provide additional cover to prevent freezing and blockage. The system performance would require continuous and periodic monitoring and installation of additional drains, as found necessary. Such monitoring may consist of observation of piezometric levels and slope movement, if any, together with measurement of slope drain discharge volumes.

12.0 COST ESTIMATES

We have carried out preliminary cost estimates for the initial monitoring program and subsequent remedial measures. Selection of the remedial measures will be subject to the results of the monitoring program, relative cost effectiveness and system reliability. The present cost estimate for the required remedial measures is for purposes of sizing and would be further refined after the initial monitoring program.

12.1 Monitoring - Sections A, B, and C

We estimate that the approximate installation cost for the piezometer monitoring program would be about $8,000 to $10,000 per instrumented section. The total cost of installation would be $40,000 to $60,000. We further estimate the cost of bi-monthly monitoring, record updating and preparation of a final report covering the results and interpretation of the monitoring program would be in the range of $7,000 to $9,000.

12.2 Remedial Measures - Section A

a) Pumped Wells

Costs associated with a system of pumped wells include costs related to initial installation as well as the long term cost of operation and maintenance. We estimate the unit cost of installation of a pumped well to be about $4,000. The total cost of installation will depend on the number of wells required together with the necessary discharge and pumping systems.

It will be necessary to carry out a field pump test to determine the well spacings. The cost of the pump test can be reduced by using the piezometers already installed for the monitoring program as observation wells. The pumped well installed for the field pump test can also be incorporated in the final system of pumped wells. Therefore the additional cost of the pump test will be that required to cover testing and data interpretation only. We estimate the cost of testing and data interpretation to be about $5,000.

The long term cost of operation and maintenance is difficult to estimate. The cost of operation involves estimation of power consumption together with future cost escalation. The long term maintenance costs should include periodic maintenance together with pump replacement at approximately every 5 years. It will also be necessary to replace the wells every 10 years to maintain operations at an acceptable well efficiency.

b) Gravity Drainage by Tunnelling

A long term drainage measure involving a combination of drainage galleries and vertical sand drains would require a relatively large initial cost. We estimate the initial cost of such a system to be in the range of $1.5 million if tunnelling is carried with dewatering and under dry conditions. However, if tunnelling is carried out under pressure the estimated cost of installation would be in the range of $3 million. The long term maintenance cost for this system would be minimal.

c) Inclined Slope Drains

Another alternative of gravity drainage dewatering would be consideration of inclined slope drains that would extend up to 100 m into the slope. We estimate the unit cost of installation of inclined slope drains to be about $2,400. The total initial cost for this system would be in the range of $500,000. Depending on the system long term performance, it may be necessary to install additional slope drains at some future date.

We trust that the information contained in the report and this covering letter is adequate for your requirements at this time. Please contact us if you have further inquiries.

Yours very truly

GOLDER GEOTECHNICAL CONSULTANTS LTD.

Per: R.M. Wilson, P. Eng.

for D. Negussey

RMW/DN/ba

782-1179

PLAN OF STUDY AREA

Figure 2

ZONE	SYMBOL	STABILITY
A	▨	Poor
B	▨	Poor
C	▨	Fair
D	▨	Good

LEGEND

⯐ Location of Borehole

12 Location of Cross-section

NOTE:
Contours on plan in Imperial units.

Reference: Corporation of the District of Maple Ridge Topographic Map, Sheets #14&24, dated January 1977.

Golder Associates

Drawn SF
Reviewed
Date Feb. '79

782-1179

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
MINISTRY OF ENVIRONMENT
WATER INVESTIGATIONS BRANCH
ON
STABILITY STUDY
FRASER RIVER NORTH RIVER BANK
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TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a stability study carried out for the slopes on the north side of the Fraser River between Haney and Port Hammond, B.C. (see Figure 1). The study was performed by Golder Associates, at the request of the Water Investigations Branch.

The scope of this report is as follows:-

a) to assess the soil and ground water conditions in the area.

b) to define the overall stability of the existing slopes, considering the existing development in the area.

Golder Associates were also responsible for a stability study carried out for the section of river bank adjacent to the proposed bypass alignment. This work was performed for Willis Cunliffe Tait & Company Ltd., and covered the river bank slopes to the east of the present study area.

2.0 PRESENT SLOPES

The existing river bank slopes consist of steep bluffs sloping into the Fraser River. The overall slope, including the submerged section, is about 58 m in height at the Haney end of the study area, and decreases to about 35 m at Port Hammond. The depth of the river ranges from about 23 m at Haney to about 15 m at Port Hammond. The overall slope is generally in the range 22 to 26 degrees.

Development has taken place close to the top of the slope, with houses located within 30 m of the upper slope crest. The upper slope is subject to frequent shallow slides. Seepage through this slope has resulted in the formation of drainage gulleys and ravines in the slope.

The Canadian Pacific Railway runs along a bench about 15 m wide, located midway up the overall slope. This bench is generally located about 5 m above low river level. The lower slope, below the bench, is mainly submerged. This slope is subject to river erosion, and rip-rap protection has been provided in several areas.

The slopes are known to be subject to landslide activity. Major slides have occurred at Haney and Port Hammond. The Haney slide took place in 1880, and is well documented by newspaper reports. Both of the slides extended at least 300 m back from the slope crest, and included about 13 to 20 hectares of land. In addition, two smaller slides have occurred in the study area. These slides are located at Port Hammond, to the west of the main slide, and at Fir Street. The small slides generally extended about 40 m back from the slope crest, and included about 1.5 to 2 hectares of land.

A site plan of the study area is shown in Figure 2. The length of river bank under study in this report is located between about Sections 1 to 29. A profile along the river bank is shown in Figure 3, and typical slope sections are shown in Figures 4 to 6.

3.0 INVESTIGATION PROGRAM

3.1 Survey Work

A series of 29 cross-sections was performed by the Water Investigations Branch, in order to define the geometry of the underwater slopes and the lower part of the river bank. These sections were obtained by sounding in the river channel, and were tied into reference points established on the bank.

The geometry of the upper slopes was obtained from topographic maps supplied by the District of Maple Ridge. Using this information, the profiles and sections shown in Figures 3 to 6 were obtained.

3.2 Drilling Program

A field and laboratory testing program was performed to determine the soil and ground water conditions within the slopes.

The field program was performed between November 15 and December 4, 1979. Five boreholes were drilled to depths of up to 78 m using a truck-mounted rotary drill rig. The boreholes were located about 30 to 50 m north of the upper slope crest and spaced along the slopes in the study area (see Figure 2). Piezometers were installed in each of the boreholes, in order to determine ground water levels in the slopes. Water levels are presently being monitored at regular intervals. All of the field work was performed under the supervision of a member of our engineering staff.

Representative samples were obtained at regular intervals of depth in the boreholes and in situ strength testing was also performed in the boreholes. All of the samples were brought to our laboratory for detailed classification. Following completion of the field work, a program of laboratory testing was performed, in order to evaluate the general properties of the soils present in the slopes and to determine soil shear strengths for stability analyses.

The information for Borehole 102 was obtained from boreholes drilled at this location for C.P. Rail. This data was originally published in our report, V76049, dated August, 1976.

4.0 SOIL CONDITIONS

A profile showing the general stratigraphy along the river bank is given in Figure 3. A detailed discussion of the soil conditions and properties is given in Appendix I.

4.1 General

The soil conditions are relatively uniform over the entire study area, consisting of interlayered silty clay and silty sand, extending to depths of over 75 m. These strata are of glacio-marine origin, i.e. they were deposited in a marine environment during an active glacial period.

These strata are underlain at depth by a deposit of compact to dense silty sand and gravel, which is probably of glacial origin. The surface of this stratum dips sharply to the west (see Figure 3).

4.2 Haney Clay

The predominant material in the river bank consists of blue-grey silty clay, known locally as Haney Clay.

The silty clay generally has a firm to stiff consistency with locally soft zones near the ground surface. The long term shear strengths of the Haney Clay are highly dependent on the mineralogy and plasticity of the material. High plasticities and clay contents normally reflect low shear strengths. The Haney Clay generally has a medium plasticity, with clay contents of about 20 per cent. The variations in plasticity and clay content are generally random, with no distinct zones or layers of highly plastic clay.

Laboratory triaxial tests were performed to define the long term shear strengths of the silty clay over the range of material properties identified. Based on these results, it has been possible to define the range in the Haney Clay shear strengths as follows:-

Angle of Internal Friction : 30 to 34 degrees
Cohesion : 0 to 14 kPa

4.3 Sand Layers

The Haney Clay contains frequent layers of silty fine sand or fine to medium sand. The thickness of these layers varies from local zones and lenses 1 to 5 mm in thickness, up to layers greater than 10 m thick. For instance, at borehole 105 the upper 25 m of material consists almost entirely of sand, with only one 3 m layer of clay.

The sands are generally loose to compact, with locally denser zones. These strata generally have shear strengths slightly higher than the Haney Clay. The sand layers are considerably more permeable than the clay strata, and act as relatively free-draining media. The sand layers were observed to exit on the upper slope face in several areas, resulting in local seepage zones. The piezometers were installed within sand layers encountered in the boreholes, in order to determine the water levels present within these layers.

4.4 Sand and Gravel

The sand and gravel is generally dense to very dense, with high shear strengths. However, this stratum is located at considerable depths below the ground surface, so that it will have a minimal effect on the stability of the river bank slopes.

5.0 GROUND WATER CONDITIONS

Ground water levels have been monitored in the piezometers, over the period between December 1978 and March 1979. These levels indicate that the piezometric levels in the sand strata are generally located within 5 to 15 m of the ground surface at the slope crest. Ground water levels close to the ground surface have been recorded locally, e.g. in borehole 103. The water levels were recorded during a period of generally high rainfall. It is likely that the water levels may fall significantly during summer months.

No detailed information is available on the actual shape of the piezometric surface (ground water table) across the slope sections. The piezometric surface is probably located close to the upper slope face, since seepage was noted at several locations. It is likely that the water table beneath the CPR bench is close to the river level.

Several of the piezometers are installed within sand layers which are inferred to exit on the lower slope face below the river level. It is interesting to note that the recorded water levels indicate that significant head differentials may exist between the piezometer locations and the lower slope face.

6.0 HISTORICAL LANDSLIDE ACTIVITY

Three main types of landslide activity have been identified along the study area. These consist of major (retrogressive) slides, minor (single) slides, and shallow surficial slope failures.

6.1 Major (Retrogressive) Landslides

The two major landslides are located at Haney and at Port Hammond (see Figure 2). The typical geometry of these landslides is shown in

Figure 7. The geometry indicates that the failures did not occur as single landslides, i.e. along a single slide plane. Rather the frontal slope failed, followed by failure of the exposed back scarp. This resulted in retrogressive failures which extended some 300 m back from the original slope crest.

The Haney Landslide occurred on January 30, 1880. Newspaper reports describing the landslide are available, together with meteorological records (see Appendix III). The following points are worthy of note:-

1. The slide occurred rapidly, with little warning.
2. The slide resulted in substantial constriction of the Fraser River channel and caused a wave up to 60 ft. high, resulting in extensive damage to boats, wharves, etc., along the river.
3. Between January 6 and 12, 1880, 25 inches of snow fell while temperatures remained below freezing. This was followed by several days of higher temperatures and rain, with a maximum daily precipitation of 6 inches of snow and 1.6 inches of rain. The total precipitation during January 1880 was 45 per cent higher than modern averages.
4. Seismic records indicate that no known earthquake could be linked with the Haney slide.

The toe debris from the slide can be clearly seen in Figure 2. This debris is subject to active river erosion, and has been removed at a rate of about 1 m/year during recent years.

The age of the Port Hammond landslide is not known. Based on the trees present in the slide area and the absence of slide debris at the toe of the failure, it is likely that the slide occurred at least 200 years ago. The retrogressive nature of this slide is confirmed by the scarps at the west side of the slide, which are aligned perpendicular to the river (Photo 6). These represent smaller slides which failed laterally into the main body of the slide, resulting in a lateral expansion of the failure area.

6.2 Minor (Single) Landslides

Two smaller slides have been identified in the study area, located at Fir Street, and west of the major Port Hammond slide (see Figure 2 and Photos 5 and 7). Typical geometries for these slides are shown in Figure 7.

The present topography indicates that these slides occurred mainly as single failures of the slope fronting on the river. The landslides did not retrogress, probably because the mass of land involved in the original failure did not slide completely into the river, but slumped in place. This slumped debris then provided some support for the exposed back scarp, so that retrogression did not occur. The slides, therefore, only extended some 30 to 60 m back from the original slope crest.

The age of these slides is not known but, based on local information, we know that the slides are at least 50 to 75 years old.

6.3 Surficial Slope Failures

Minor surficial failures are frequently observed along the upper slope above the CPR bench. These failures take the form of shallow flow slides of clay and sand materials, and are generally associated with seepage zones exiting in sand layers on the slope face. Shallow back scarps have been formed at the upper slope crest (see Photos 9 and 10).

These failures frequently cause movements close to the railway tracks. The CPR has undertaken a program of maintenance, consisting mainly of drainage to stabilize shallow failures on the slope. Ongoing work is being performed at Mile 103.5 (Section 11) and at Mile 103.9 (Section 16 - Photo 8).

The most active slides in the area consist of surficial failures on the upper slopes. There is no evidence of old backscarps of retrogressive failures associated with a former river channel geometry. It is therefore likely that the minor, single landslides probably occur more frequently than the major, retrogressive slides.

7.0 RIVER BANK EROSION

Northwest Hydraulic Consultants Ltd. have carried out a study of the river bank erosion in this area (see Appendix IV). This study included a review of the cross-section data, a discussion with CPR maintenance staff, and examination of historical aerial photography.

The section of river bank under study is located on the outside of a large meander reach in the river. The deepest point in the river (thalwag), is typically located at the toe of the river bank slope. The evidence indicates that most of the river bank under study is subject to varying degrees of erosive forces. Strong evidence of the erosion potential in this area is provided by the continuing maintenance work which is being performed by CPR on the river bank slope consisting mainly of rip-rap placement. In several areas the rip-rap has slumped into the river, indicating undercutting and erosion (see Photos 3 and 4). River erosion has also resulted in the removal of the slide debris at the Fir Street and Port Hammond slides.

The indications are that the erosion is presently concentrated on the slide debris at the Haney slide. In the remainder of the study area, the erosion is expected to decrease in severity in a downstream direction, and is negligible downstream of the minor Port Hammond slide (Section 27). The only exception is the short area of about 400 m between about Sections 8 to 12. This area is protected by the toe of the Haney slide, so that no erosion is presently occurring in this area.

Erosion of the bank above the low water level is resisted in several areas by the presence of rip-rap, placed by CPR. In addition, log boom piles and dolphins located along the bank probably provide resistance to erosive forces. At the unprotected toe of the Haney slide, the rate of scour has been identified as about 1 m per year between 1938 and 1971. The rate of erosion over the remainder of the study area is expected to be slow, and would not be expected to exceed about 0.1 m per year.

8.0 SLOPE STABILITY

A series of stability analyses have been performed to assess the stability of the slopes in the study area. The slopes considered in this section mainly consist of the existing high slopes. The stability of the lower river bank slopes along the old slide areas is discussed in Section 8.9. A detailed description of the stability analyses is given in Appendix II.

8.1 General

The stability analyses provide factors of safety for the existing slopes. These factors of safety represent the ratio of forces tending to cause failure divided by the forces resisting failure. A factor of safety of 1.0 indicates an unstable slope, and a factor of safety of 1.3 to 1.4 would normally be considered adequate for this type of slope.

Stability analyses were generally carried out for five typical sections along the slope, to obtain information on variations of stability in the study area. The analyses generally consider single slope failures which would extend at least 30 m back from the upper slope crest and could thus affect existing structures. The analyses do not consider local sloughing failures on the upper slope face.

8.2 Present Slope Stability

A series of analyses was performed to assess the present stability of the slopes based on our best estimate of the various parameters.

The soil parameters obtained from the field and testing program were used and a low river level was assumed. The piezometric surface was assumed to be located 10 m below the slope crest and at the ground surface along the upper slope face (see Figure 8). The following factors of safety were obtained:-

Section	Estimated Factor of Safety
9	1.25 to 1.30
12	1.05 to 1.10
15	1.20 to 1.25
18	1.25 to 1.30
21	1.35 to 1.40

The above results indicate that the section of slope between about Sections 11 to 15 has a very low factor of safety. The remaining slopes have a low, but adequate factor of safety. It should be noted that the above factors of safety are based on the present slope geometry. Future river erosion would result in steepening of the lower river bank slope, resulting in a decrease in the stability of the overall slope.

The next step in assessing the stability of slopes was to perform a series of sensitivity analyses. These analyses were performed to assess the sensitivity of the factors of safety to various possible changes in the assumed conditions and parameters.

8.3 Slope Stability - Varying Piezometric Levels

One of the main factors which typically affect the stability of slopes is the piezometric level within the slope, or the degree of slope saturation.

We have assumed that it is possible that the slope water level could rise to the ground surface at the slope crest and along the upper slope face. We have also assumed that the piezometric surface could fall up to 15 m below the ground surface at the slope crest, and as much as 10 m into the upper slope face. Typical factors of safety for high and low piezometric levels are shown below:-

Section	Estimated Factor of Safety		
	High Water Level	Low Water Level	Present Water Level
9	1.00 to 1.05	1.50 to 1.55	1.25 to 1.30
12	0.85 to 0.90	1.25 to 1.30	1.05 to 1.10
15	0.95 to 1.00	1.45 to 1.50	1.20 to 1.25
18	1.00 to 1.05	1.50 to 1.55	1.25 to 1.30
21	1.05 to 1.10	1.60 to 1.70	1.35 to 1.40

The above results show that the stability of the slopes is highly sensitive to the actual piezometric level within the slope. For instance, if the water level rises to the ground surface, i.e. a completely saturated slope, the slopes between about Sections 11 to 15 would become unstable, and the remaining slopes would have a very low factor of safety in the range 1.0 to 1.1. However, if the water level falls about 15 m below the slope crest and 10 m into the slope face, all of the slopes would be stable, with factors of safety of at least 1.25.

It is likely that the development in the area, together with associated roads and drainage, has resulted in a general lowering of the piezometric levels which may develop in the slopes. The development may therefore have resulted in a small increase in the factor of safety of the slopes, with respect to the previous conditions.

8.4 Stability Under Seismic Conditions

A major seismic event could also lead to a reduction in the factors of safety of the slopes. We have assumed a design horizontal acceleration of 0.08 g, together with the static conditions and parameters described in Section 8.2. The results of the analyses are given below:-

	Estimated Factor of Safety	
Section	Seismic	Static
9	1.00 to 1.05	1.25 to 1.30
12	0.85 to 0.90	1.05 to 1.10
15	0.90 to 0.95	1.20 to 1.25
18	0.95 to 1.00	1.25 to 1.30
21	1.05 to 1.08	1.35 to 1.40

The results indicate that the slope between Sections 12 and 15 would be unstable, while the remaining slopes would have very low factors of safety. These analyses may represent conservative assumptions, since they assume coincidental occurrence of low river level, present piezometric levels, and an earthquake.

8.5 Further Sensitivity Analyses

Two further parameters which could have an effect on the stability of the existing slopes are the assumed soil parameters and the river level.

The soil parameters were varied within the range of properties identified in the field and testing program. The parameters were found to generally have a minimal effect on the factors of safety, causing less than 2 per cent change.

A series of analyses was performed to check the influence of river levels on the stability of the slopes. The previous factors of safety were generally obtained assuming a low river level at elevation +0 m. The normal change in river level recorded in this area is about 5 m, with a high river level at about elevation +5 m. The results indicate that this rise in river level would increase the slope factor of safety by about 7 per cent.

8.6 Back Analyses

In order to obtain a check on the accuracy of the above stability analyses, we have performed back analyses of the two single failures identified in the study area. This is relatively simple, since the failure surface geometries for these slides are determined by the present topography, and the previous slope geometries can be inferred from the existing slopes either side of the failures (see Figure 6 - Section 26, and Photos 5 and 7). The slopes must, by definition, have had factors of safety of less than or equal to 1.0 at failure.

The back analysis of the Fir Street slide shows that a completely saturated slope would have resulted in a reduction of the factor of safety to about 0.95 to 1.0. Alternatively, a design earthquake shock would also have resulted in failure. This gives good agreement with the estimated present factors of safety for the adjacent slopes.

The back analysis of the minor Port Hammond slide indicates that substantial piezometric pressures would be required to cause failure, i.e. about 10 m above the slope crest. Alternatively, a completely saturated slope combined with the design earthquake would result in a low factor of safety. It is possible that the failure may have taken place when the river channel was significantly deeper, so that the assumed preslide geometry may be incorrect.

8.7 Retrogressive Failures

The above factors of safety apply to single failures extending about 30 m back from the present slope crest. A major concern will be whether these failures would tend to retrogress to form failures of similar proportions to the Haney and Port Hammond slides. A series of analyses was therefore performed to assess the retrogressive potential of the initial failures.

We have assumed that the material involved in the original slide would provide no support to the exposed failure back scarp. These analyses indicate that the failure backscarps would have low factors of safety, of about 1.0, so that retrogression could be expected. A major uncertainty of this type of analysis is the amount of slide debris which would remain at the toe of the exposed slope. This would be dependent on the speed or violence of the initial failure, which would in turn depend on the trigger mechanism for the slide, i.e. seismic, high piezometric levels, etc. It is likely that slopes with low factors of safety at present would be more liable to future retrogressive failures.

Because of the above problems, it is difficult to perform meaningful back analyses of existing retrogressive slides. Of the observed slides in the study area, it may be noted that two retrogressed and two did not retrogress.

It is also difficult to predict analytically how far such failures could retrogress. It is possible that the soil conditions could change with increasing distance back from the present slope crest. This could limit the retrogression of the slide, as the back scarp is formed in more competent material. Based on our present knowledge, it must be assumed that the failures could retrogress to the same extent as the Haney and Port Hammond slides, i.e. about 300 m back from the existing slope crest (see Figure 2).

8.8 Slope Drainage

Because of the sensitivity of slope stability to the actual piezometric levels, it is important to consider the drainage characteristics of the present slopes. Features such as ravines, drainage gullies and existing slides may intercept the sand layers within the upper slopes. These features would therefore encourage lateral drainage and would reduce the likelihood of high piezometric levels within the adjacent slopes. However, as noted in Section 5.0, significant head differentials exist between piezometer locations and locations where the sand layers exit on the lower slope face. This indicates that existing drainage features within the upper slope would only reduce the water levels in the adjacent slopes a limited distance either side of the drainage feature.

The section of slope between Section 7 to 12 is drained by two ravines and the Haney slide. Similarly, the slope between Sections 16 to 22 is intersected by two ravines and the existing slides. The maximum length of slope between these drainage features typically does not exceed 100 m.

In contrast, the section of slope between Sections 12 to 16 has little drainage relief. This section of slope is about 300 m long and has only small gullies present, which do not extend a significant distance back into the slope. This indicates that the section of slope between Sections 12 to 16 would be most likely to be subject to high piezometric levels.

8.9 Previous Slide Areas

In the areas of the major retrogressive landslides, i.e. the major Port Hammond slide and the Haney slide, the high slopes have been removed by the failures and the existing slopes in these areas consist only of river banks about 6 m high.

The major landslides are now stable against further large scale failures, due to the subdued topography (see Figure 7). However, river erosion may be expected to lead to over-steepening and failure of the local low level river bank. It is our opinion that these failures would generally extend a small distance back from the present river bank crest, so that the initial effects would be limited to the area of the railroad and south to the river. These failures would not be expected to retrogress landward until an ongoing cycle of erosion and over-steepening occurs.

We trust that this report presents the information you require at present. If you need further details, please contact us.

Yours very truly,

GOLDER ASSOCIATES

Per: R.M. Wilson, P. Eng.

N. E. Wedge

for Roger Jinks, P. Eng.

RMW/RJ:rme

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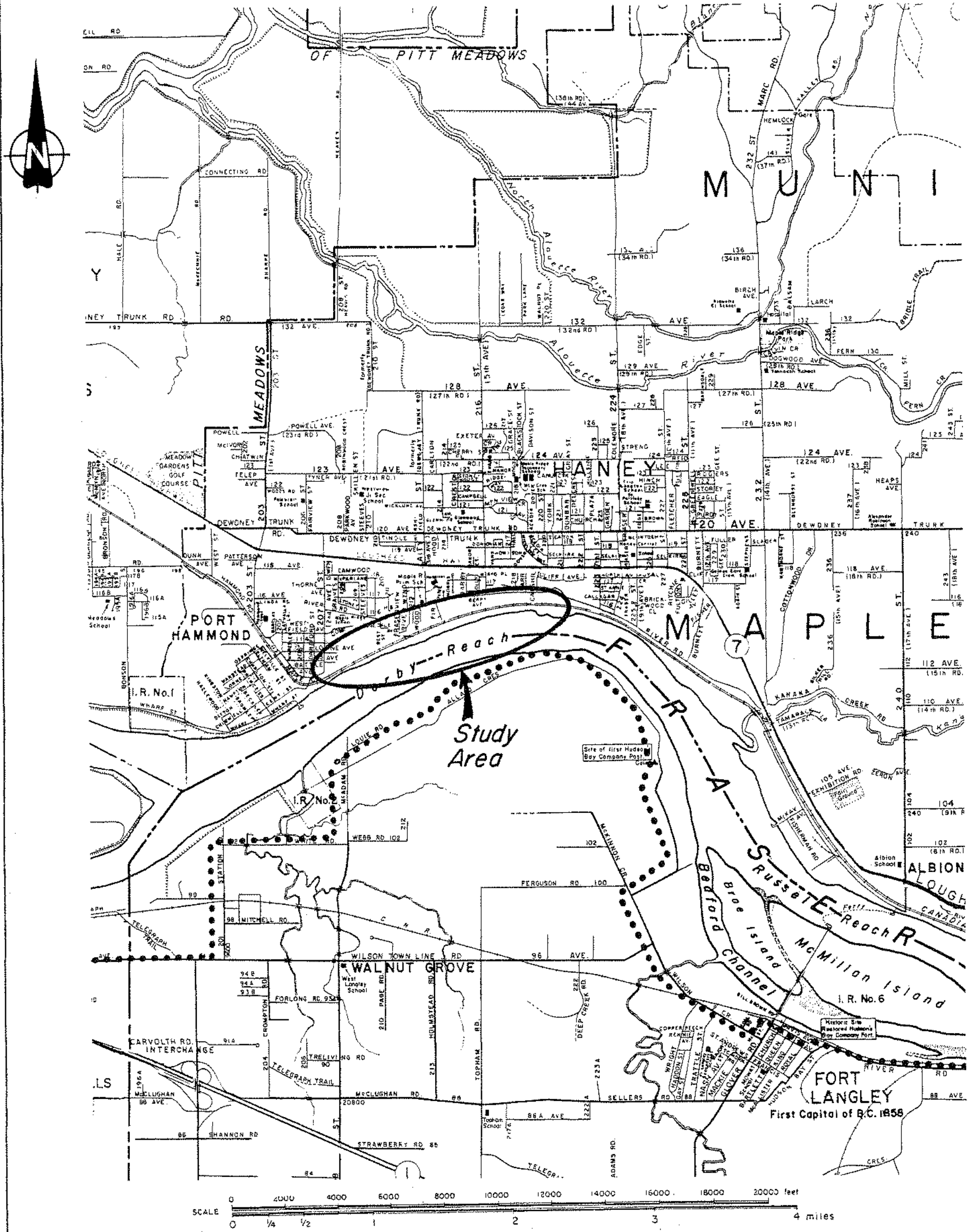
KEY PLAN

Figure 1



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PLAN OF STUDY AREA

Figure 2

LEGEND

Location of Borehole

12 Location of Cross-section

NOTE:
Contours on plan in Imperial units.

Reference: Corporation of the District of Maple Ridge Topographic Map, Sheets #14&24, dated January 1977.

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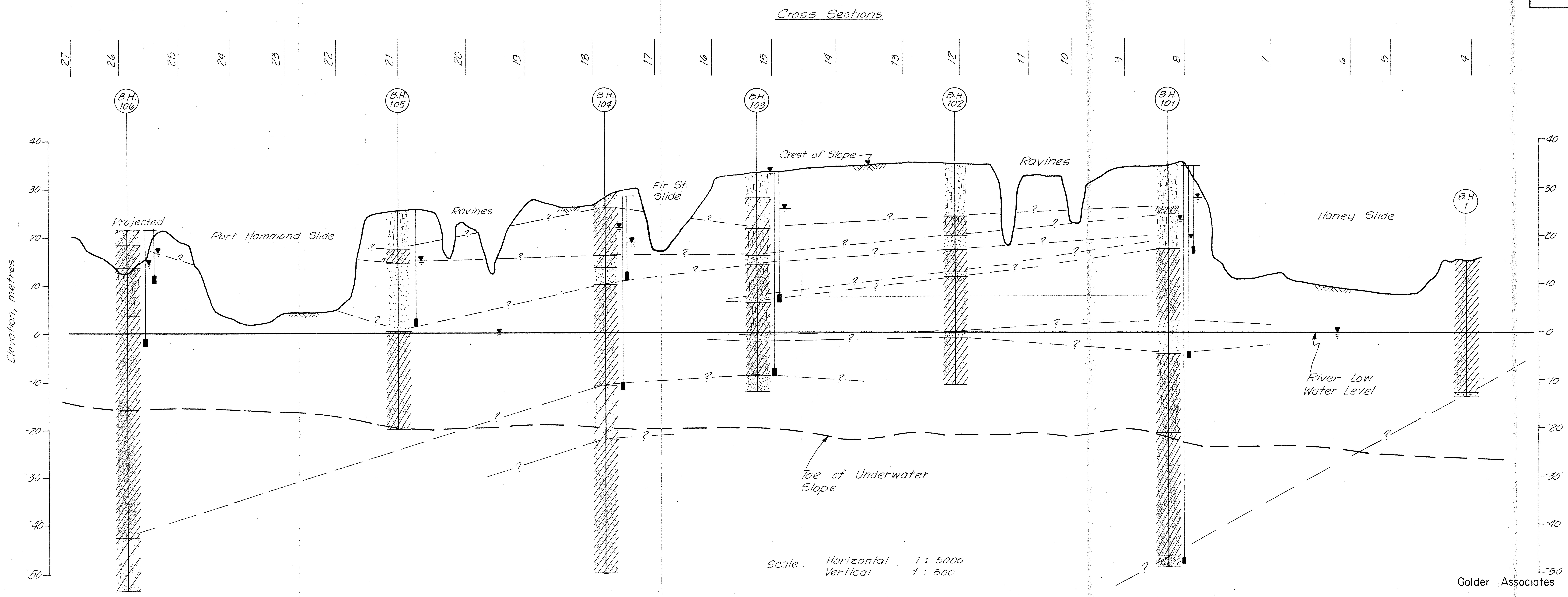
PROFILE ALONG RIVERBANK SHOWING INFERRED STRATIGRAPHY

Figure 3



Scale: Horizontal 1 : 5000
Vertical 1 : 500

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LEGEND

- Borehole in profile
- Location of Piezometers
- Average water level in Piezometers (Dec. 1978 to Mar. 1979)

INFERRED STRATIGRAPHY

- ▨ Loose to dense silty fine to medium SAND.
- ▨ Loose to dense silty fine SAND with layers of silt & silty clay.
- ▨ Interlayered firm to stiff silty CLAY & loose to dense silty fine SAND
- ▨ Firm to stiff silty CLAY, with layers of silty fine sand.
- ▨ Firm to stiff silty CLAY, occ. thin partings of silt & silty fine sand
- ▨ Compact to dense silty SAND and GRAVEL

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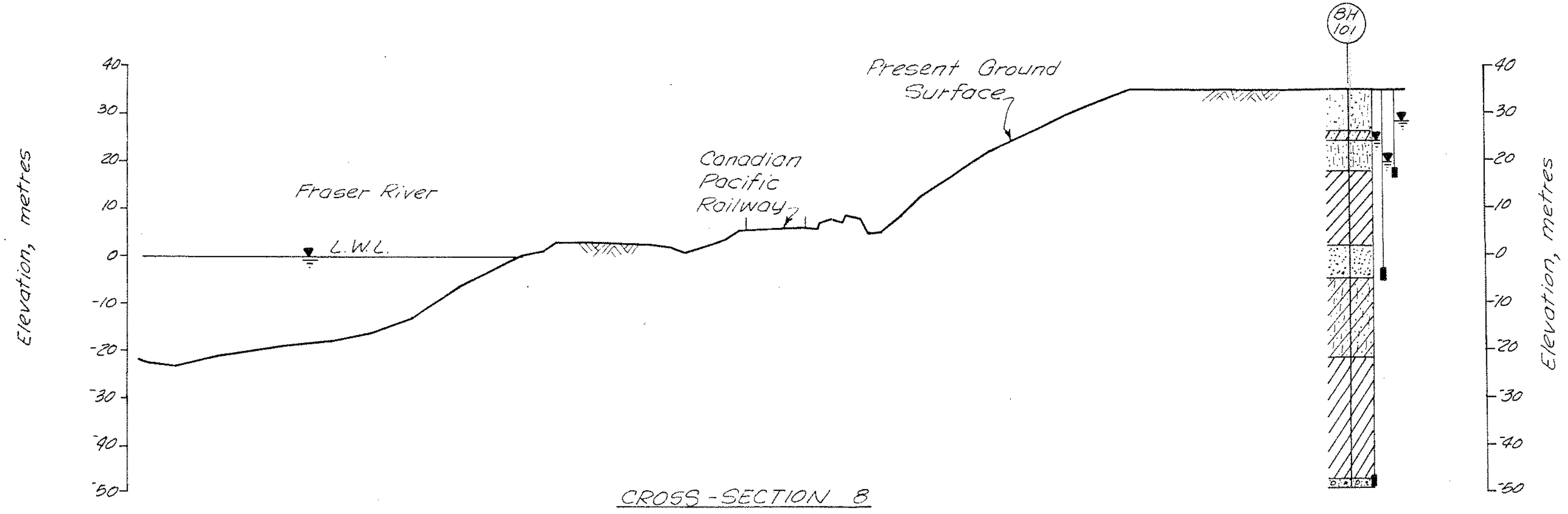
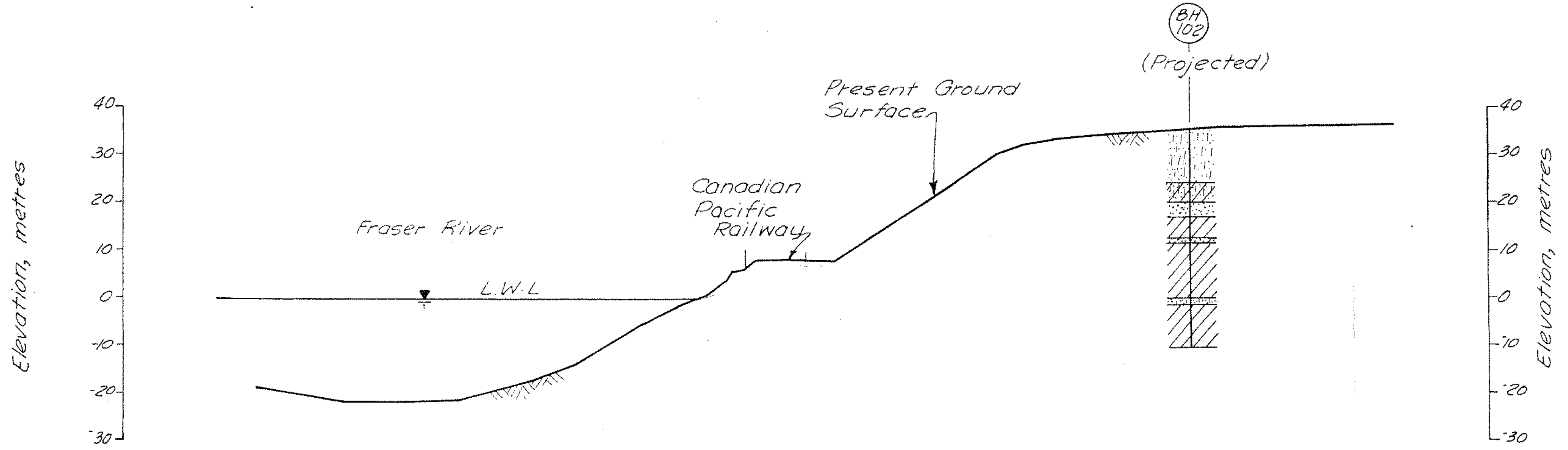
TYPICAL RIVERBANK SECTIONS

Figure **4**



CROSS-SECTION 8



CROSS-SECTION 12

For Legend, See Figure 3

Vertical & Horizontal Scale 1 : 1000

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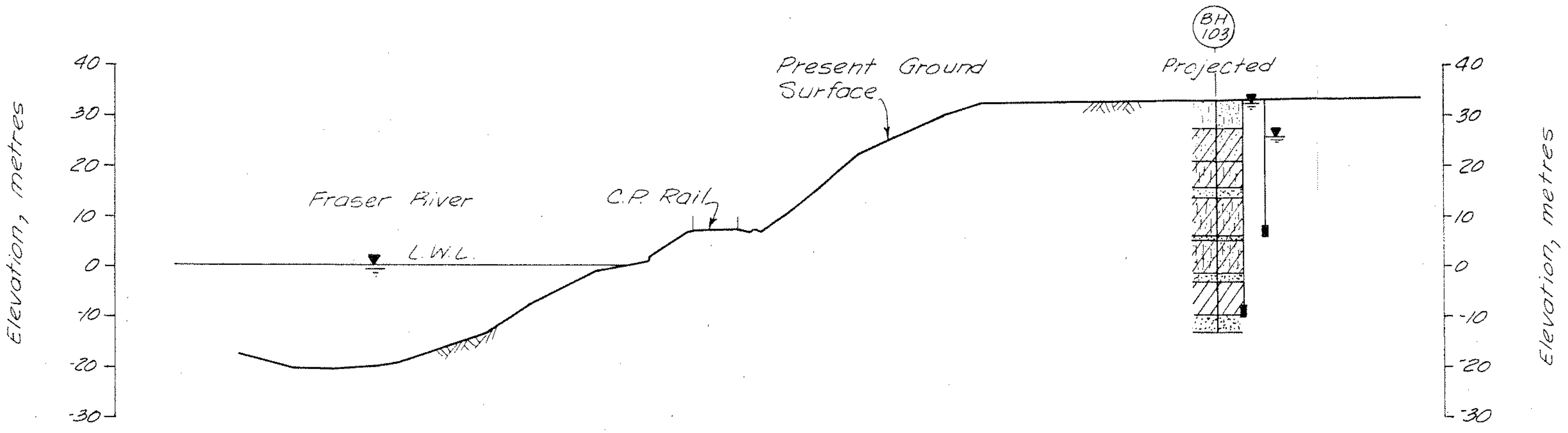
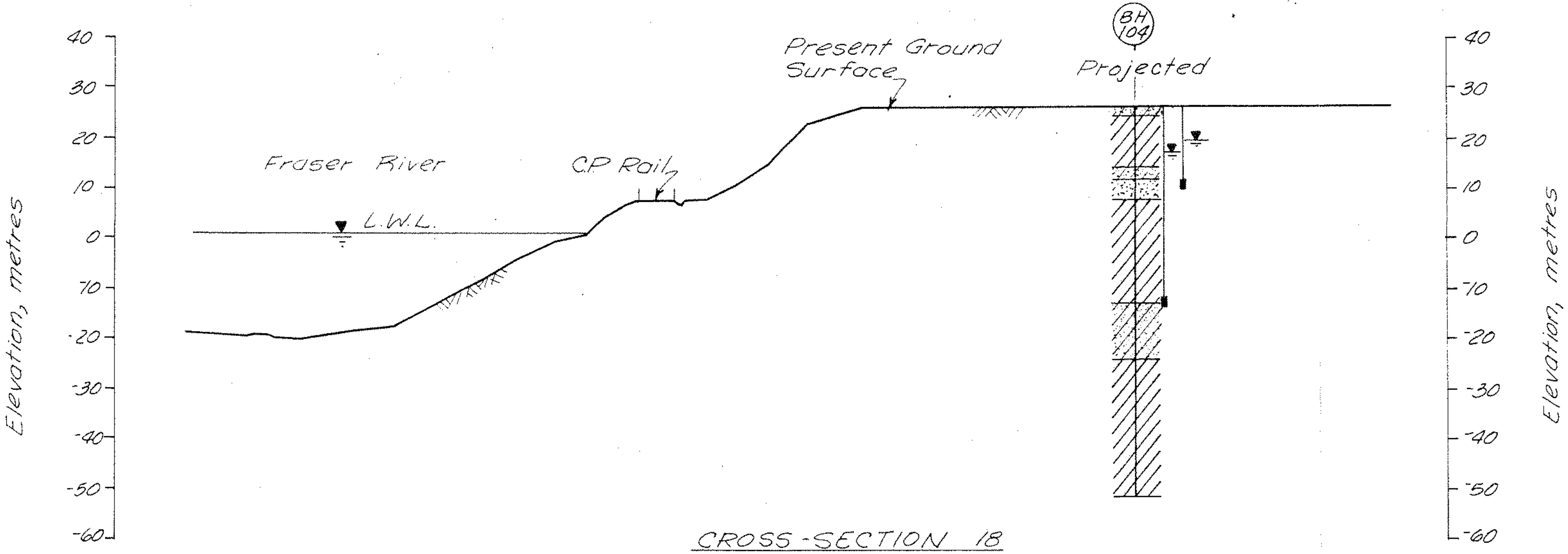
TYPICAL RIVERBANK SECTIONS

Figure **5**



CROSS-SECTION 15



CROSS-SECTION 18

For Legend, See Figure 3.

Vertical & Horizontal Scale: 1:1000

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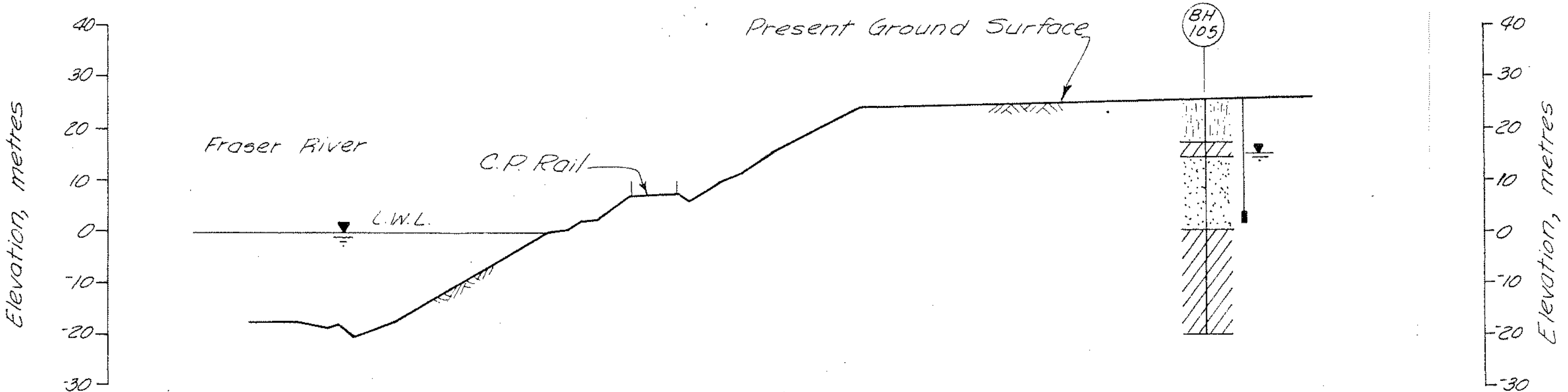
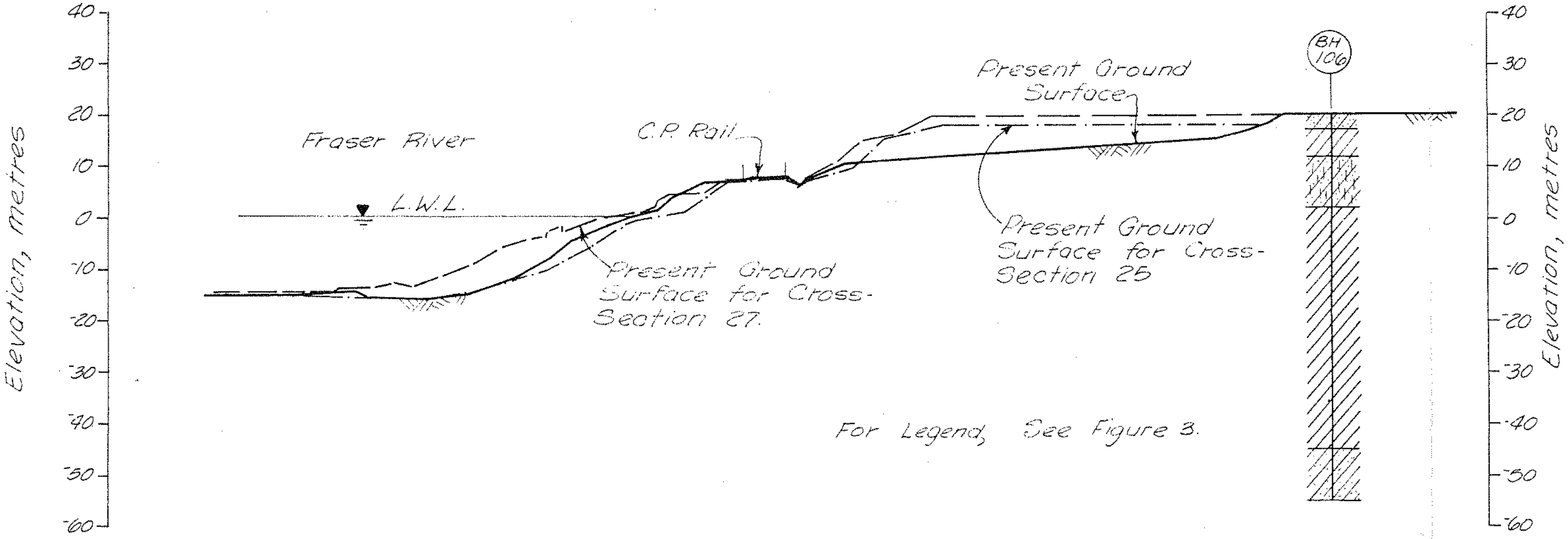
TYPICAL RIVERBANK SECTIONS

Figure **6**



CROSS-SECTION 21



For Legend, See Figure 3.

CROSS-SECTION 26

(MINOR PORT HAMMOND SLIDE)

Vertical & Horizontal Scale 1:1000

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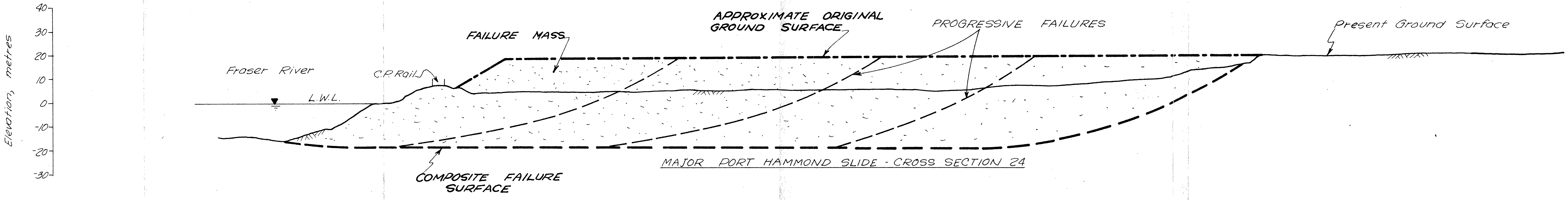
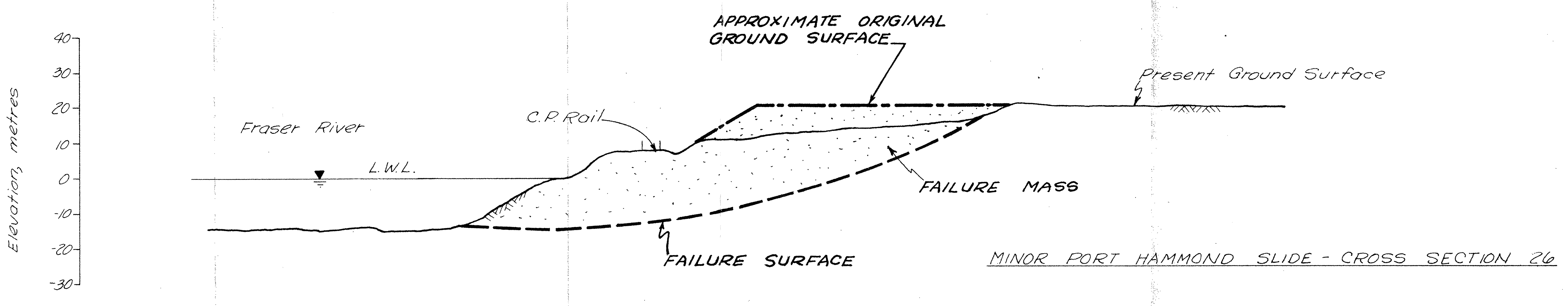
TYPICAL SECTIONS THROUGH LANDSLIDE AREAS

Figure 7



MAJOR PORT HAMMOND SLIDE - CROSS SECTION 24



MINOR PORT HAMMOND SLIDE - CROSS SECTION 26

Vertical and Horizontal Scale 1 : 1000

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APPENDIX I

Detailed Description of Soil Conditions and Records of Boreholes

This appendix contains a detailed discussion of the soil conditions and properties in the study area. A profile showing a general stratigraphy along the river bank is given in Figure 3. Detailed descriptions of the strata encountered in the boreholes are given on the enclosed Record of Borehole sheets, together with the results of laboratory testing.

1.0 GENERAL STATIGRAPHY

The soil conditions are relatively uniform over the entire study area. The area is underlain by at least 75 m of glacio-marine deposits, i.e. materials that were deposited in a marine environment during an active glacial period. The deposits consist of interlayered silty clay and silty sand. The bedding of the material dips to the west at a general grade of about 1.5 per cent (see Figure 3).

The predominant deposit within these materials consists of blue-grey silty clay, known locally as Haney Clay. In many areas, the clay is relatively homogenous. However, it frequently contains significant layers of silty fine sand or fine to medium sand. The thickness of these layers varies from local zones and lenses 1 to 5 mm in thickness, up to layers greater than 10 m thick. For instance, at borehole 105, the upper 25 m of material consists almost entirely of sand with only one 3 m layer of clay.

The glacio-marine strata are underlain at depth by a deposit of compact to dense silty sand and gravel. This deposit is probably of glacial origin. The surface of the sand and gravel dips sharply downward to the west (see Figure 3). This deposit was only identified in borehole 101.

2.0 HANEY CLAY

This material is generally described as a blue-grey silty clay, with frequent thin layers and zones of silty fine sand. The properties identified in the field and laboratory testing are discussed below.

2.1 Plasticity

A series of Atterberg Limit tests were carried out on samples of the Haney Clay, to identify the variations in the mineralogy of the material. A wide variation was found in the values of both liquid and plastic limits. Liquid limit was found to range from about 31 to 92 per cent, an average value being about 45 to 50 per cent. Plasticity indices ranged from 12 per cent to 59 per cent, an average value being about 25 per cent. The results of these tests are plotted on a plasticity chart (see Figure I-1). On the basis of this plot, the material may be classified as a silty clay with medium to high plasticity.

The variations in plasticity were generally found to be random, with no distinct zones or layers of highly plastic clay.

2.2 Undrained Shear Strength

Field and laboratory shear vane tests were performed on the Haney Clay, to determine the undrained shear strength of this material. The results of these tests are plotted against depth in Figure I-2. The test results indicate that the clay generally has an undrained shear strength of about 40 to 60 kPa near the ground surface, indicating a firm to stiff consistency. The undrained shear strengh increases with depth to values in the order of 80 to 100 kPa. Locally, in boreholes 104 and 106, the upper 10 to 15 m of clay was found to have undrained shear strengths of about 15 to 30 kPa, indicating a soft to firm consistency.

2.3 Sensitivity

The sensitivity of the clay normally ranges from 2.5 to 5.0, indicating a clay of medium sensitivity. Locally, zones of clay with high sensitivity, in the range 6 to 8 were noted. These were generally associated with the zones of softer clay identified in the upper 10 to 15 m in boreholes 104 and 106.

2.4 Drained Shear Strength

The most important parameter to be defined in this series of tests was the long term shear strength of the clay. An extensive testing program has been carried out on samples of Haney Clay at the University of British Columbia. The samples tested in this program were mainly obtained from the area of the old brick factory, close to the intersection of 225th Street and River Road. The Haney Clay in this area typically has liquid limits of 45 per cent and plastic limits of about 25 per cent. Results of a series of laboratory triaxial tests on this type of material are given in Reference 1.*

The results of the index property tests indicated that the Haney Clay in the present study area frequently consists of a more plastic clay than the above, with liquid limits and plasticity indices up to 92 and 59 per cent, respectively. A series of laboratory triaxial tests was therefore performed on samples of these higher plasticity clays, in order to define the possible range in drained shear strength parameters. Tests were performed on two samples of material, with liquid limits of about 60 and 92 per cent, respectively. The tests consisted of multistage consolidated

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* Ref. 1 Campanella R.G., Gupta R.C., Soils Mechanics Series No. 6, Department Civil Engineering, University of British Columbia, 1969

undrained triaxial tests, with pore pressure measurements. Three tests were performed on each sample at varying cell pressures. The results of the tests are shown in Figure I-3.

The results of the tests are summarized in the following table:

Source	Index Properties(%)		Drained Shear Strength Parameters	
	Liquid Limit	Plasticity Index	Effective Friction Angle, ' (degrees)	Cohesion C' (kPa)
Campanella and Gupta	45	20	31.5	7
Triaxial test - Borehole 106, Sample 10	60	35	32.0	7
Triaxial test - Borehole 104, Sample 3	92	59	28.0	0

The results of these tests indicate that, over the normal range of index properties identified, the drained shear strength parameters are relatively uniform. Based on the above results, the range of long term shear strength used in the stability analyses was defined as follows:

Angle of internal friction	30 to 34 degrees
Cohesion	0 to 14 kPa

3.0 SAND STRATA

The Haney Clay contains frequent layers of sand, varying in thickness from local zones 1 to 5 m thick, up to layers greater than 10 m thick. The sand strata have variable gradations ranging from sandy silts to silty fine sand, with some zones of fine to medium sand.

The results of standard penetration tests carried out in the boreholes within sand layers are shown on Figure I-4. These indicate that the sand strata are generally loose to compact, with locally denser zones. Standard penetration test 'N' values can be used to estimate drained shear strength parameters for sand strata (Ref. 2*).

Based on this correlation, the following drained shear strength parameters were defined for the sand strata:

Sand Density	Drained Shear Strength Parameters	
	Effective Friction Angle, ' (degrees)	Cohesion C' (kPa)
Loose to compact	32	0
Dense	36	0

4.0 GLACIAL SAND AND GRAVEL

The sand and gravel is generally dense to very dense, with standard penetration test 'N' values in excess of 50 blows per 300 m. This stratum has high drained shear strengths. However, the material is located at considerable depths below the ground surface, so that it will have a negligible effect on the stability of the slopes in the study area.

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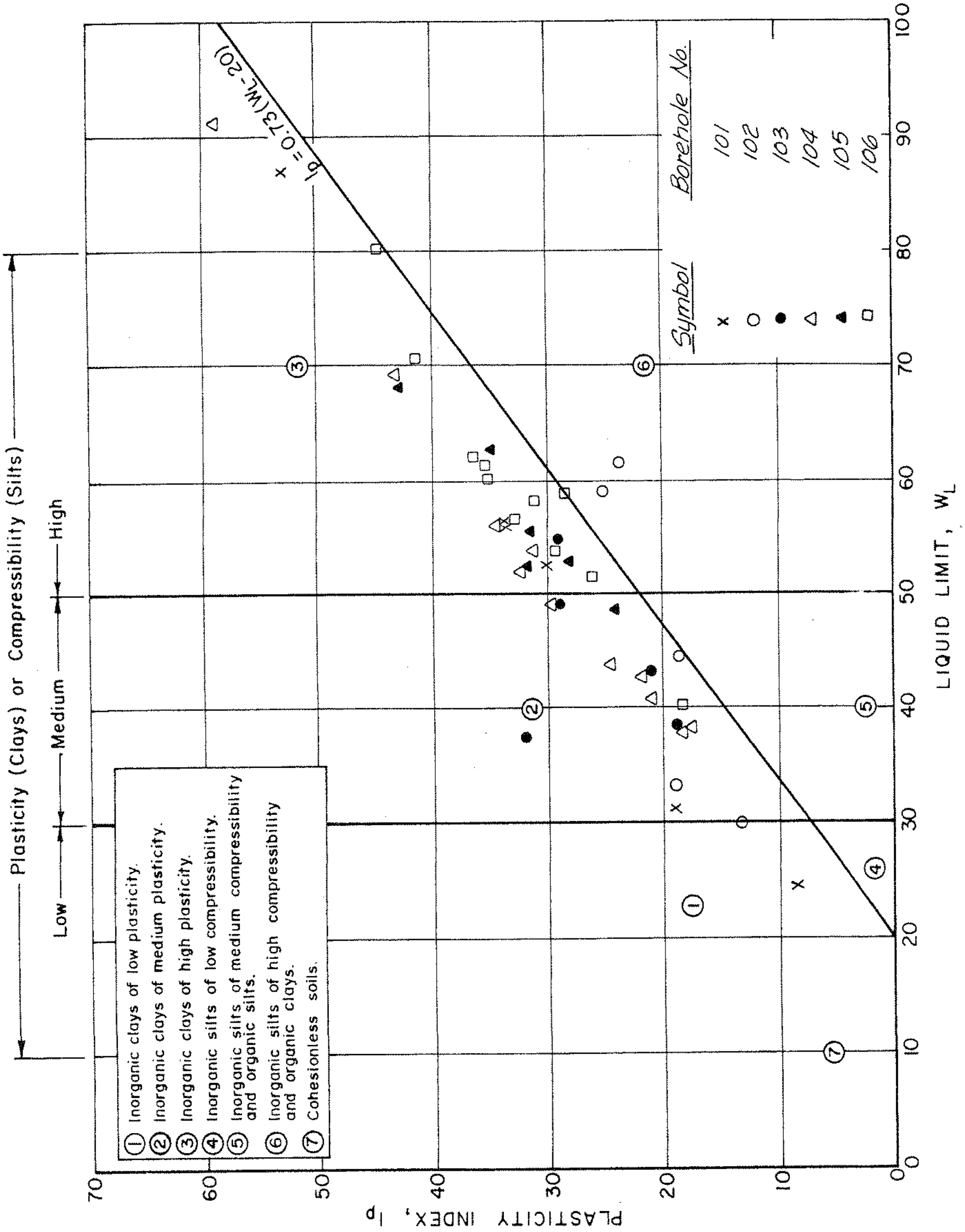
* Ref. 2 Peck, R.B. Hanson, W.E. and Thornburn, T.H.,
Foundation Engineering, p. 310
(New York, John Wiley, 2nd Ed., 1974)

PLASTICITY CHART - HANEY CLAY

Figure I-1

Project No. 782-1179 Drawn SF Reviewed Date Apr. '79



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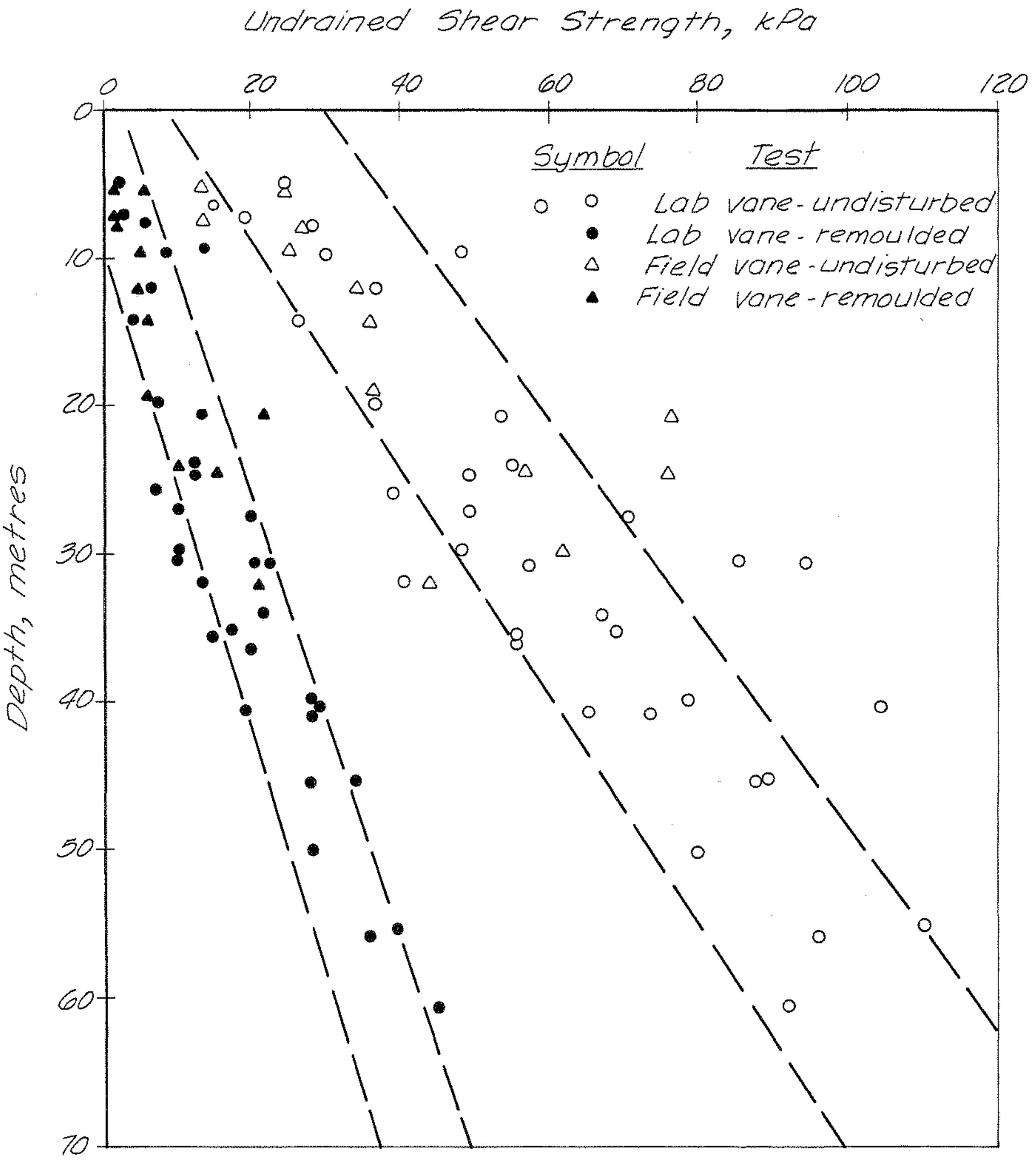
UNDRAINED SHEAR STRENGTHS- HANEY CLAY

Figure I-2

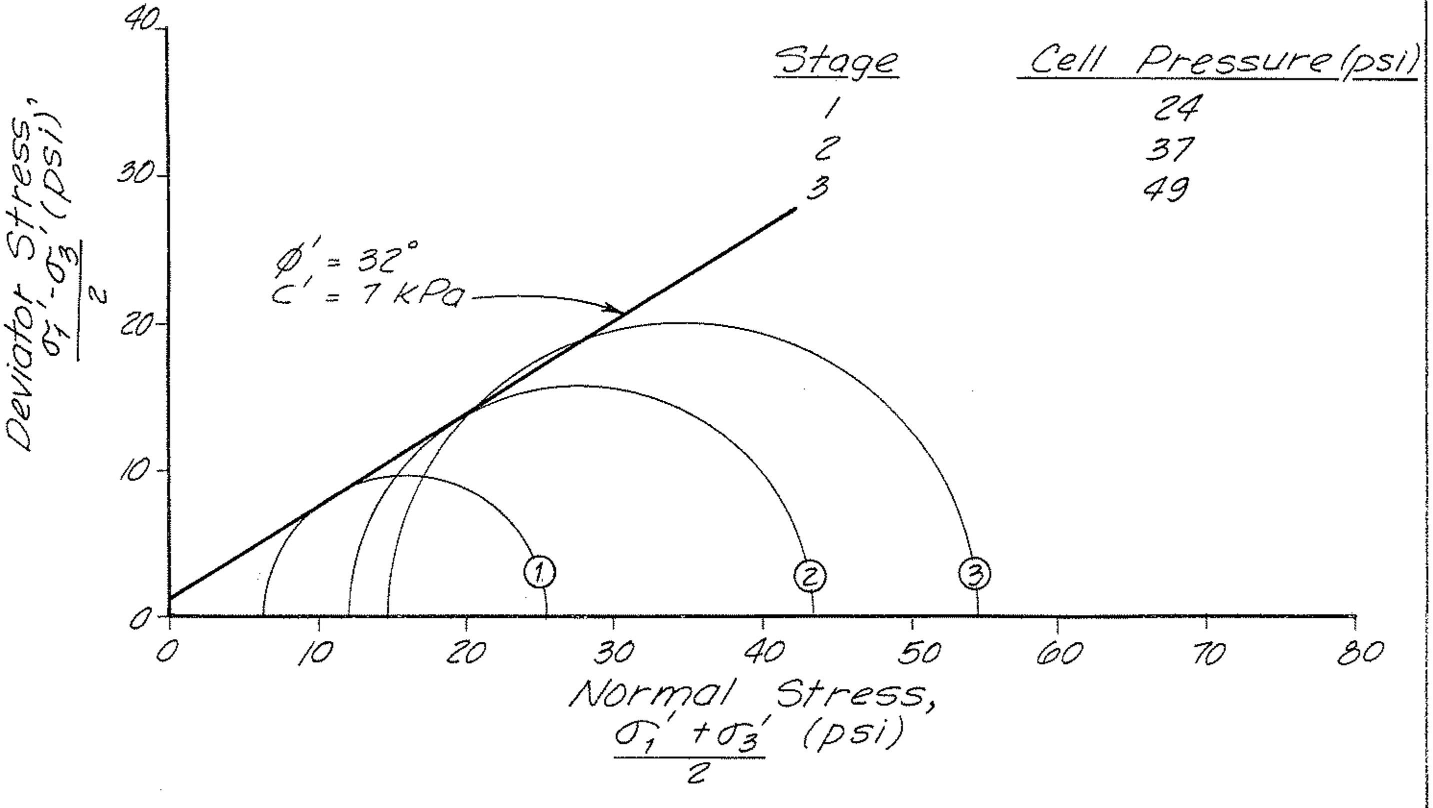


Project No. 782-1179 Drawn SE Reviewed Date Apr. '79

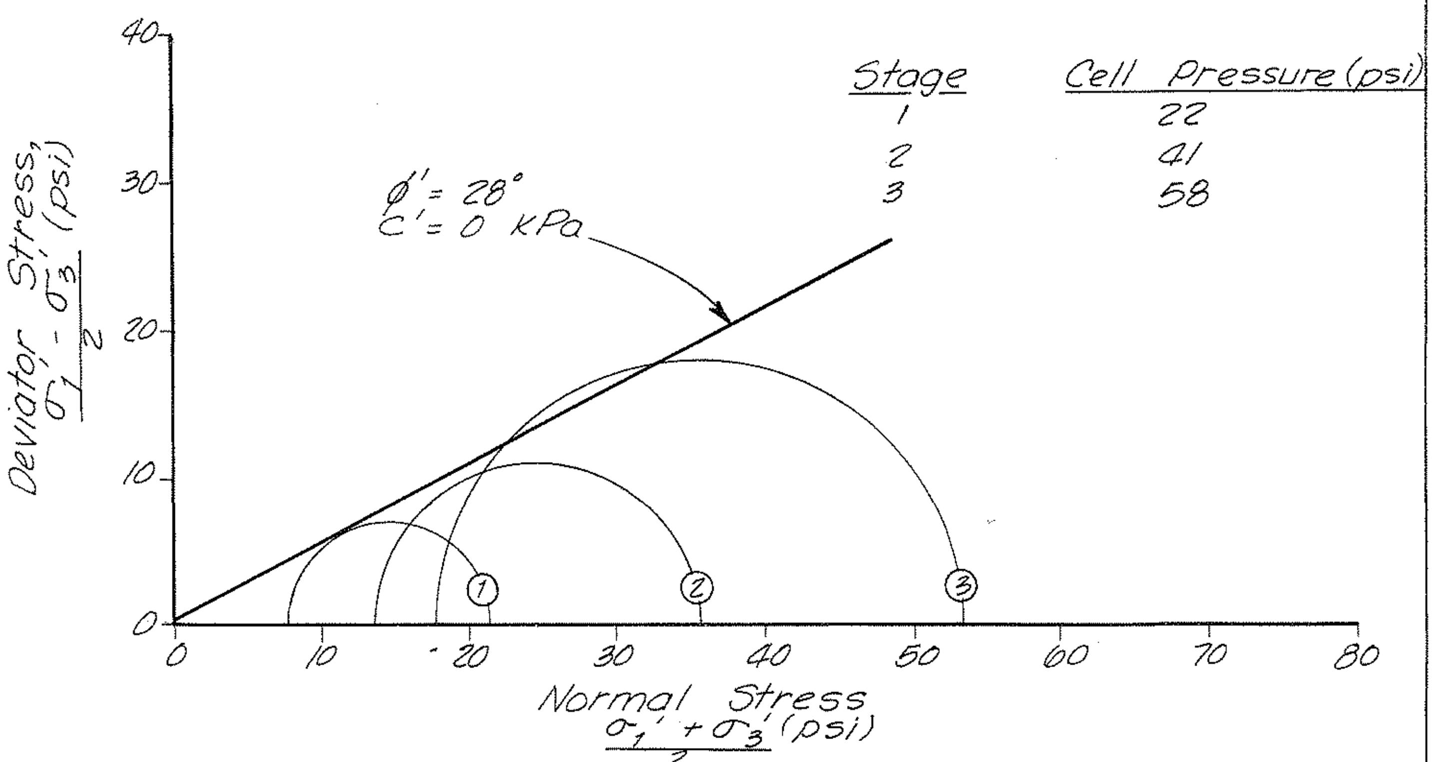
CONSOLIDATED UNDRAINED TRIAXIAL TESTS - HANEY CLAY

Figure I - 3

Test 1 B.H. 106 SA. 10 Depth: 14.1 to 14.7m.
Clay Data: LL = 60% ; PL = 24%



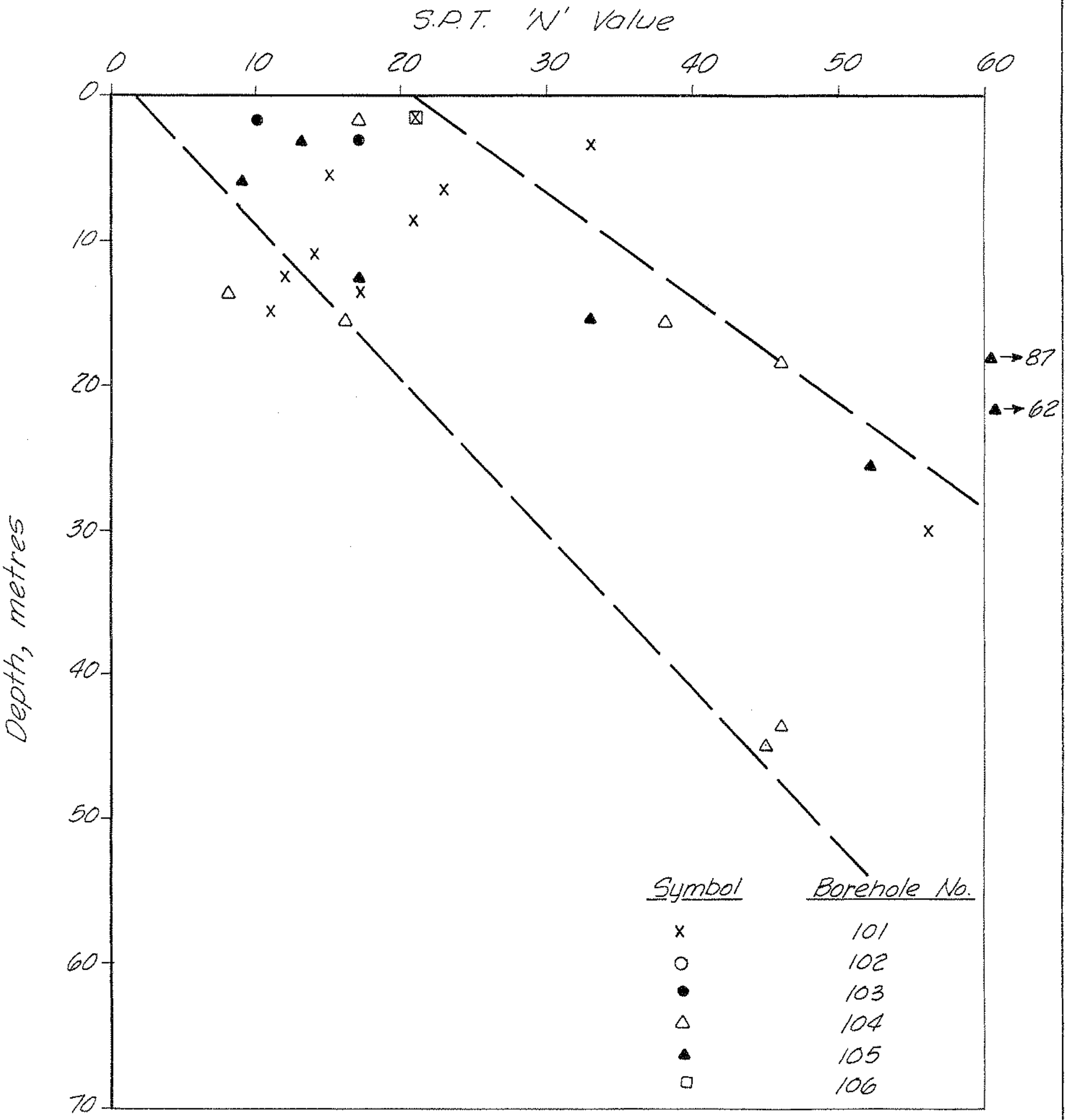
Test 2 B.H. 104 SA. 3 Depth 3.5 to 4.1m.
Clay Data : LL = 92% ; PL =



Project No. 782-1179 Drawn SF Reviewed RJ Date Apr. '79

STANDARD PENETRATION TESTS - SAND STRATA

Figure I-4



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LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, N: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH	sampler advanced by static weight—weight, hammer
PH	sampler advanced by pressure—pressure, hydraulic
PM	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1] Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2] Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

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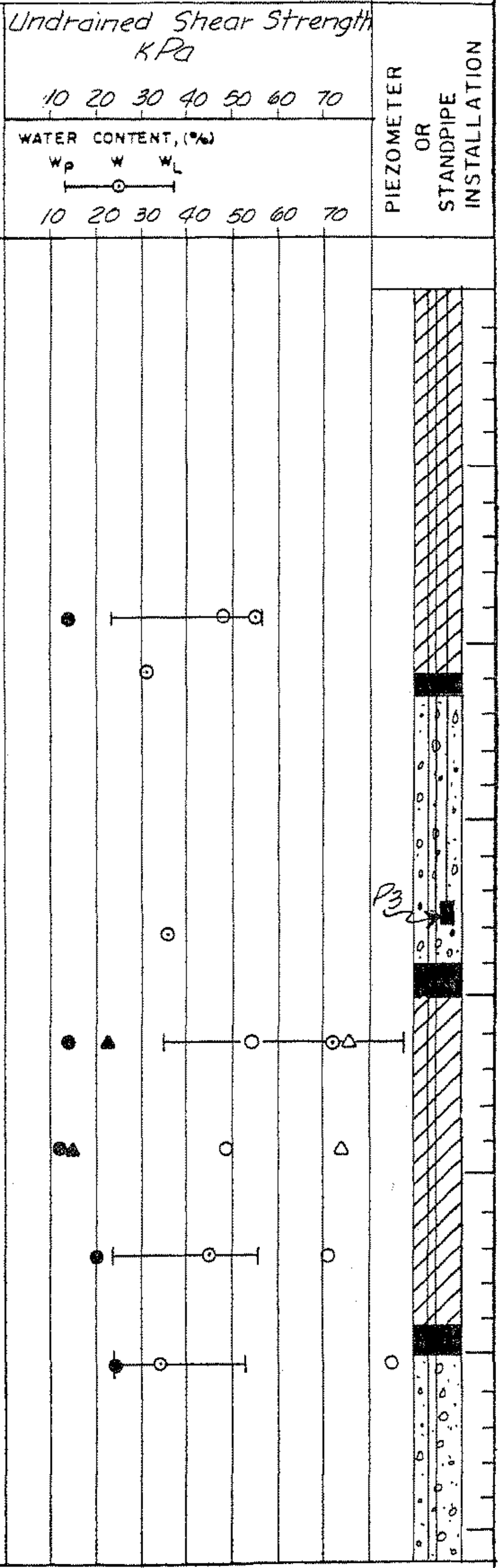
RECORD OF BOREHOLE 101

SHEET 1 OF 3

LOCATION (see Figure 2)
BORING DATE Nov. 27-29, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (m.)	ELEV. (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
0.00	35.1	GROUND SURFACE					
		Loose to compact grey-brown silty fine SAND, occ. layers of firm grey clayey silt and medium to coarse sand and gravel.		1	D.O.	21	
				2	"	33	
				3	"	15	
				4	"	23	
				5	"	21	
8.50	26.6	Interlayered firm grey silty CLAY & loose to compact silty fine SAND		6	TP	Ph	
10.50	24.6	Loose to compact grey silty fine SAND, occ. layers of firm grey silty clay, occ. gravel		7	D.O.	14	
				8	"	12	
				9	"	17	
				10	"	11	
17.00	18.1	Stiff blue-grey sensitive silty CLAY, occ. bands of dark grey clay. Below 29.0m. - occ. zones & partings of silt & silty fine sand.		11	D.O.	7	
				12	TP	Ph	
				13	"	Ph	
				14	TP	Ph	
				15	"	Ph	
32.50	2.6	Dense grey fine to medium silty SAND Cont'd					



VERTICAL SCALE
1:200

DRAWN SF
CHECKED RD

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RECORD OF BOREHOLE 101

SHEET 2 OF 3

LOCATION (see Figure 2)

BORING DATE Nov. 27-29, 1978

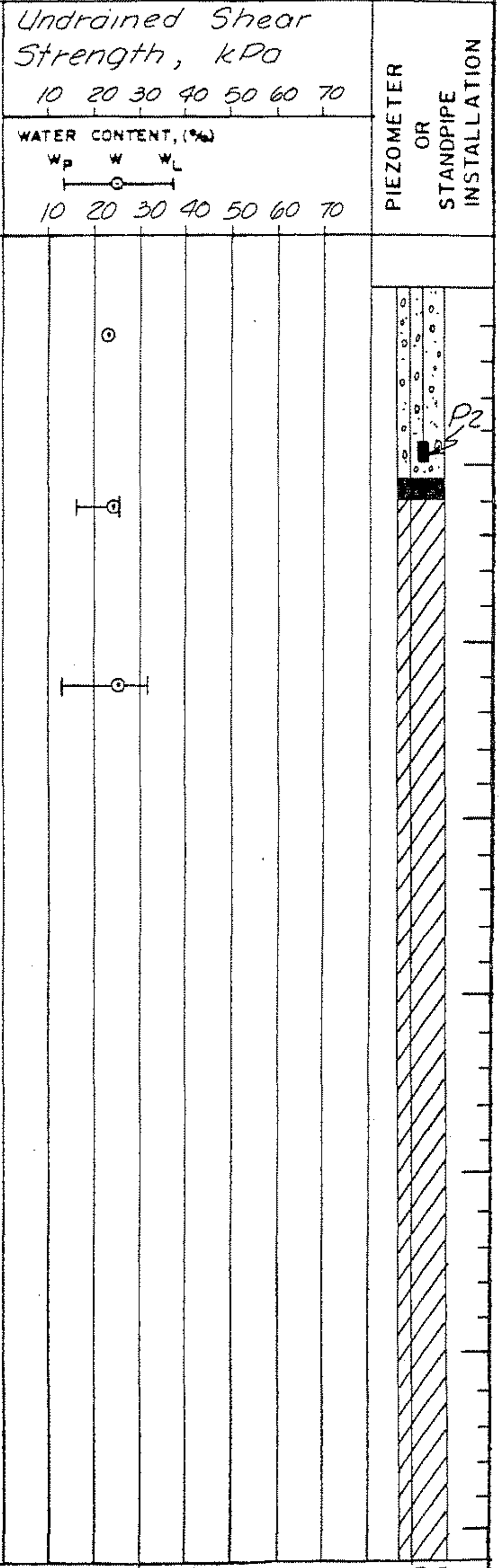
BOREHOLE TYPE Rotary

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

DATUM Ground Surface

DEPTH (metres)	ELEV. (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
		Dense grey fine to medium silty SAND		16	D.O.	56	
39.5	-4.4	Stiff to very stiff blue-grey sensitive silty clay, with frequent layers (up to 500 mm) of sandy silt and silty fine sand.		17	TP	Ph	
				18	TP	Ph	
56.0	-20.9	Stiff to very stiff grey sensitive silty CLAY, occ. sand layers					
		Cont'd					



VERTICAL SCALE 1 : 200

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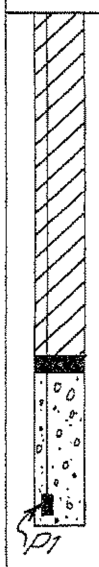
RECORD OF BOREHOLE 101

SHEET 3 OF 3

LOCATION (see Figure 2)
BORING DATE Nov. 27-29, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m.)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING	Undrained Shear Strength, kPa 10 20 30 40 50 60 70 WATER CONTENT, (%) Wp W WL 10 20 30 40 50 60 70	PIEZOMETER OR STANDPIPE INSTALLATION
75		Stiff to very stiff grey sensitive silty CLAY, occ. sand layers.							
80									
82.0	-46.9	Very dense grey silty SAND & GRAVEL							
83.5	-48.4	End of Borehole							P1
85									
90									
95									



LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

Piezometer Installation

- ▨ General Backfill
- ■ Bentonite Seal
- ⁘ 'Pea' Gravel
- ▮ Piezometer

VERTICAL SCALE
1 : 200

DRAWN SF
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Project No. 782/179

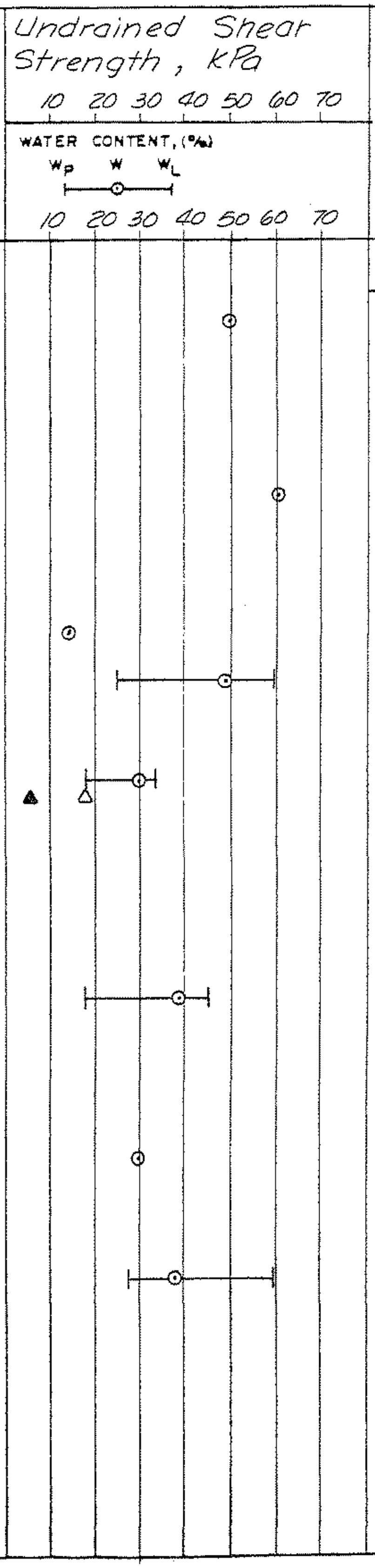
RECORD OF BOREHOLE 102

(derived from BHs 103.5-1, 103.5-2, 103.5-3)

SHEET 1 OF 2

LOCATION (see Figure 2)
BORING DATE
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0.00	37.50	GROUND SURFACE						
1.50	36.0	Stiff brown silty CLAY						
5.15	32.35	Loose brown SAND and GRAVEL						
10.05	27.45	Loose brown silty fine to medium SAND, layers of soft brown clayey silt						
14.60	22.90	Firm grey-blue silty CLAY, seams of fine sand						
17.05	20.45	Loose to compact fine grey silty SAND						
21.35	16.15	Firm to stiff grey-blue silty CLAY						
22.85	14.65	Loose to compact fine grey silty sand						
30.00	7.5	Stiff grey silty CLAY						
		Stiff grey SILTY clay						
		Cont'd						



VERTICAL SCALE
1:200

DRAWN SF
CHECKED RJ

Golder Associates

Project No. 7821179

RECORD OF BOREHOLE 102

SHEET 2 OF 2

LOCATION (see Figure 2)	BORING DATE
BOREHOLE TYPE Rotary	BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M	DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
34.45	+3.05	Loose to compact grey fine silty SAND						
		Stiff grey silty CLAY						
45.75	-8.25	End of Borehole						

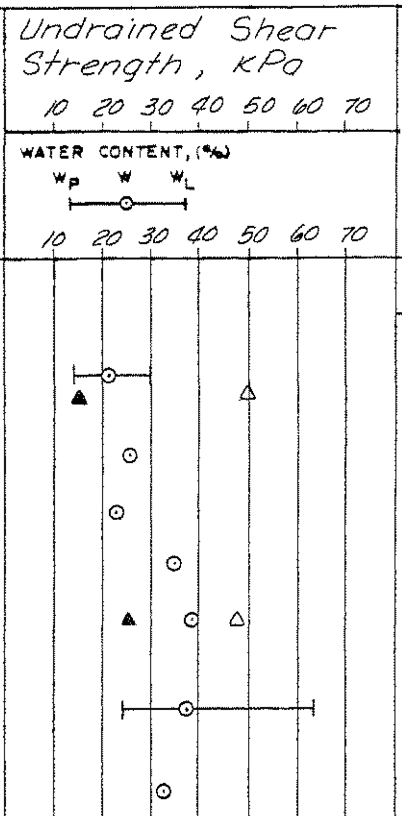
40

45

50

55

60



LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

VERTICAL SCALE

1 : 200

DRAWN SF

CHECKED KT

Golder Associates

Project No. 782179

RECORD OF BOREHOLE 103

SHEET 1 OF 2

LOCATION (see Figure 2)

BORING DATE December 1, 1978

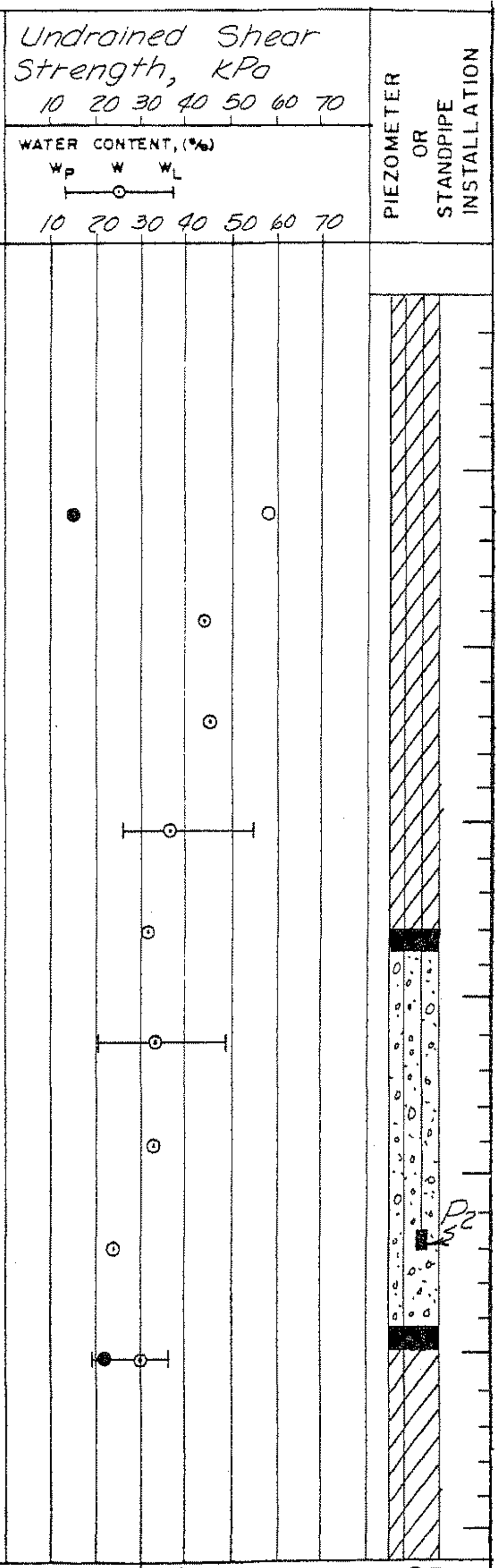
BOREHOLE TYPE Rotary

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
0.00	32.0	GROUND SURFACE					
5.50	26.5	Compact grey-brown mottled laminated fine to medium SAND, layers of firm to stiff desiccated silty clay		1 2	D.O. D.O.	10 17	
11.00	21.0	Interlayered loose grey & brown mottled silty fine to medium SAND, and firm grey silty CLAY		3 4	TP D.O.	Ph 5	
16.50	15.5	Firm grey sensitive silty CLAY, with 25mm. layers & zones of silty fine sand & silt partings		5 6	D.O. TP	3 Ph	
18.00	14.0	Compact fine silty SAND					
26.00	6.0	Firm to stiff grey sensitive silty CLAY; to frequent layers and partings of silty fine sand		7 8 9	D.O. T.P. D.O.	14 Ph 8	
27.00	5.0	Compact fine silty SAND		10	D.O.	8	
34.00	-2.0	Firm to stiff grey sensitive silty clay to clayey SILT, frequent layers (up to 300mm.) of silty fine sand.		11	T.P.	11	
		Compact fine silty sand Cont'd					



VERTICAL SCALE 1:200

DRAWN SF

CHECKED

Golder Associates

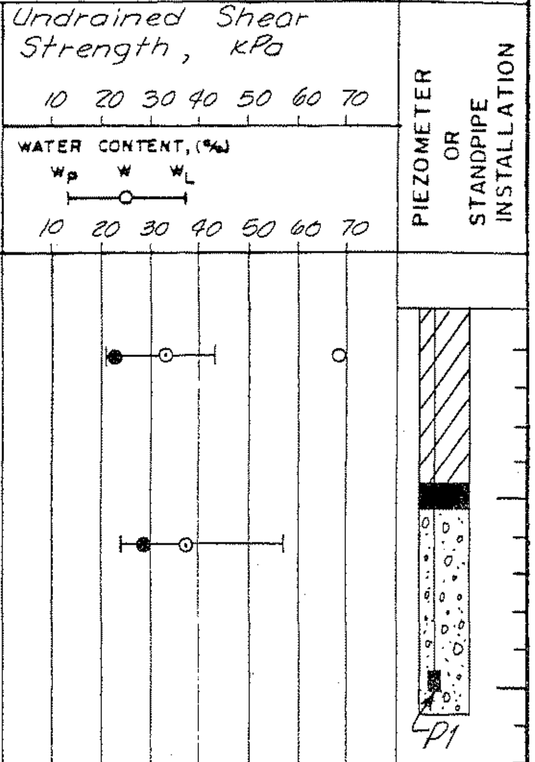
Project No. 7821179

RECORD OF BOREHOLE 103

SHEET 2 OF 2

LOCATION (see Figure 2)
BORING DATE Dec. 1, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
		Very stiff grey sensitive silty CLAY, occ. zones of silt and silty fine sand.		12	T.P.	Ph	
41.00	-9.0	Compact to dense silty fine SAND		13	T.P.	Ph	
45.60	-13.6	End of Borehole		14	T.P.		



LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

Piezometer Installation
▨ General Backfill
■ Bentonite Seal
░ 'Pea' Gravel
▮ Piezometer

VERTICAL SCALE
1:200

DRAWN SF
CHECKED RL

Golder Associates

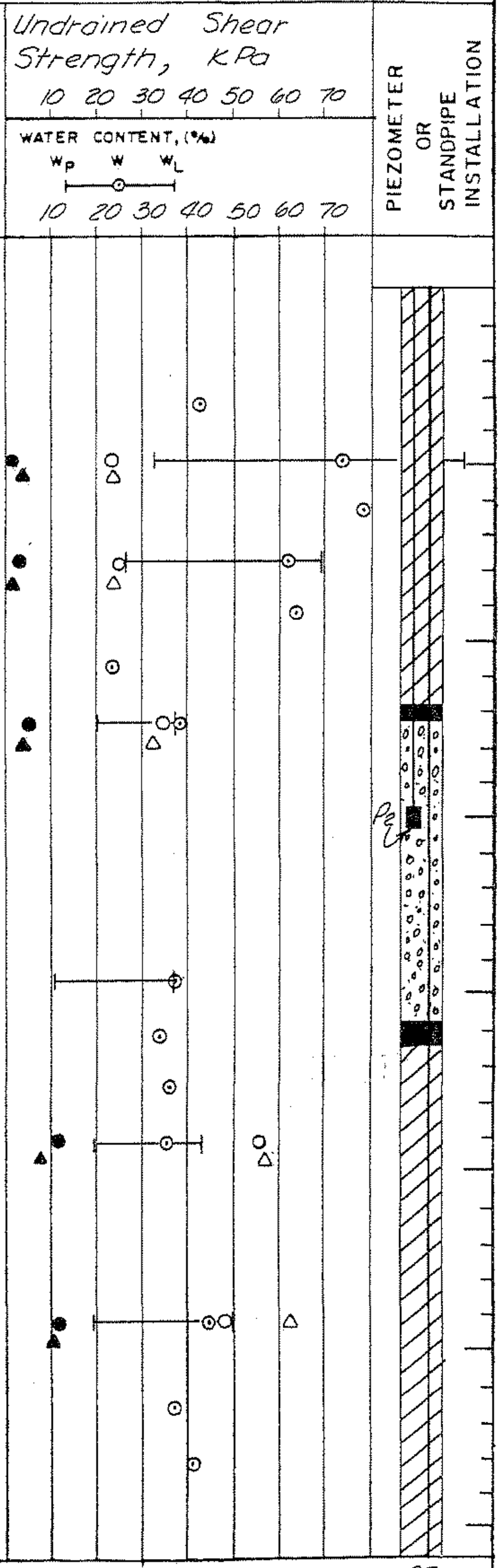
Project No. 7821179

RECORD OF BOREHOLE 104

SHEET 1 OF 3

LOCATION (see Figure 2)
BORING DATE Nov. 15-21, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
0.00	27.0	GROUND SURFACE					
0.55	26.5	Sand & Gravel (FILL)					
2.10	24.9	Compact grey & brown mottled laminated fine to medium SAND.		1	D.O.	17	
4.00	23.0	Firm grey & brown silty CLAY, fine sand partings		2	"	3	
13.50	13.5	Soft to firm blue-grey sensitive silty CLAY		3	T.P.	Ph	
				4	D.O.	Pm	
				5	T.P.	Ph	
				6	D.O.	Pm	
				7	D.O.	Pm	
				8	T.P.	Ph	
16.00	11.0	Interlayered compact grey fine silty SAND and firm blue-grey silty clay		9	D.O.	8	
				10	D.O.	16	
19.20	7.8	Compact to dense layered fine silty SAND and fine to medium SAND		11	T.P.	Ph	
				12	D.O.	38	
				13	D.O.	46	
34.00	-7.0	Firm to stiff blue-grey sensitive silty clay, occ. thin layers of silty fine sand.		14	T.P.	Ph	
				15	D.O.	2	
				16	D.O.	Wr	
				17	T.P.	Ph	
				18	D.O.	Wr	
				19	D.O.	Wr	
				20	T.P.	Ph	
				21	D.O.	Wr	
				22	D.O.	Wr	
		Cont'd					



VERTICAL SCALE
1:200

DRAWN SF
CHECKED

Golder Associates

Project No. 7821179

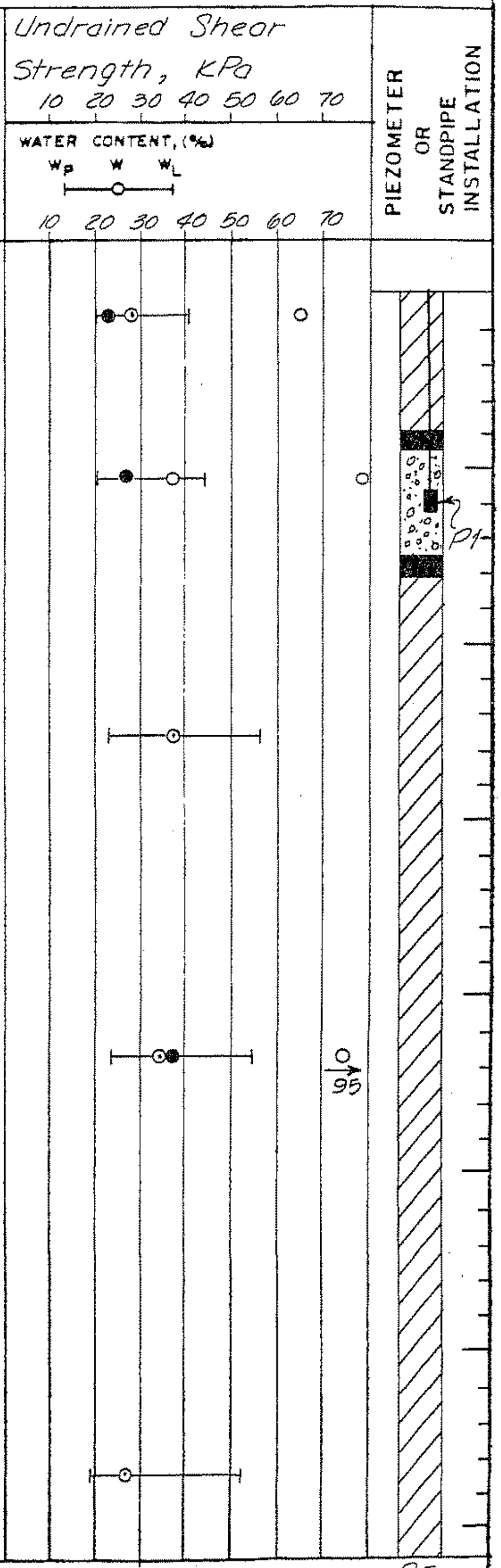
RECORD OF BOREHOLE *104*

SHEET *2* OF *3*

LOCATION (see Figure *2*)
BOREHOLE TYPE Rotary
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

BORING DATE *Nov. 15-21, 1978*
BOREHOLE DIAMETER 114 mm
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
34.00	-7.0						
		Firm to stiff blue-grey sensitive silty CLAY		23	T.P.	Ph	
				24	D.O.	Wr	
39.00	-12.0			25	D.O.	Wr	
40		Interlayered stiff blue-grey silty CLAY and compact to dense grey fine to medium silty sand.		26	T.P.	Ph	
				27	D.O.	17	
				28	D.O.	46	
45				29	D.O.	45	
				30	D.O.	17	
				31	T.P.	Ph	
				32	D.O.	19	
50.00	-23.0						
50				33	D.O.	Wr	
				34	T.O.	Ph	
				35	D.O.	Wr	
55		Stiff blue-grey sensitive silty CLAY; occ. thin layers of silty fine sand					
				36	T.O.	Ph	
60				37	T.O.	Ph	
65							
				38	T.O.	Ph	
70		Cont'd					



VERTICAL SCALE
1:200

DRAWN *SF*
CHECKED *GT*

Golder Associates

Project No. 7821179

RECORD OF BOREHOLE 104

SHEET 3 OF 3

LOCATION (see Figure 2)

BORING DATE Nov. 15-21, 1978

BOREHOLE TYPE Rotary

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING	Undrained Shear Strength, kPa 10 20 30 40 50 60 70 WATER CONTENT, (%) Wp W WL 10 20 30 40 50 60 70	PIEZOMETER OR STANDPIPE INSTALLATION
		Stiff blue-grey sensitive silty clay							▨
75									▨
77.75	-50.75	End of Borehole							
80									
85									

LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

Piezometer Installation

- ▨ General Backfill
- ■ Bentonite Seal
- ⁘ 'Pea' Gravel
- ▮ Piezometer

VERTICAL SCALE 1:200

DRAWN SF

CHECKED BJ

Golder Associates

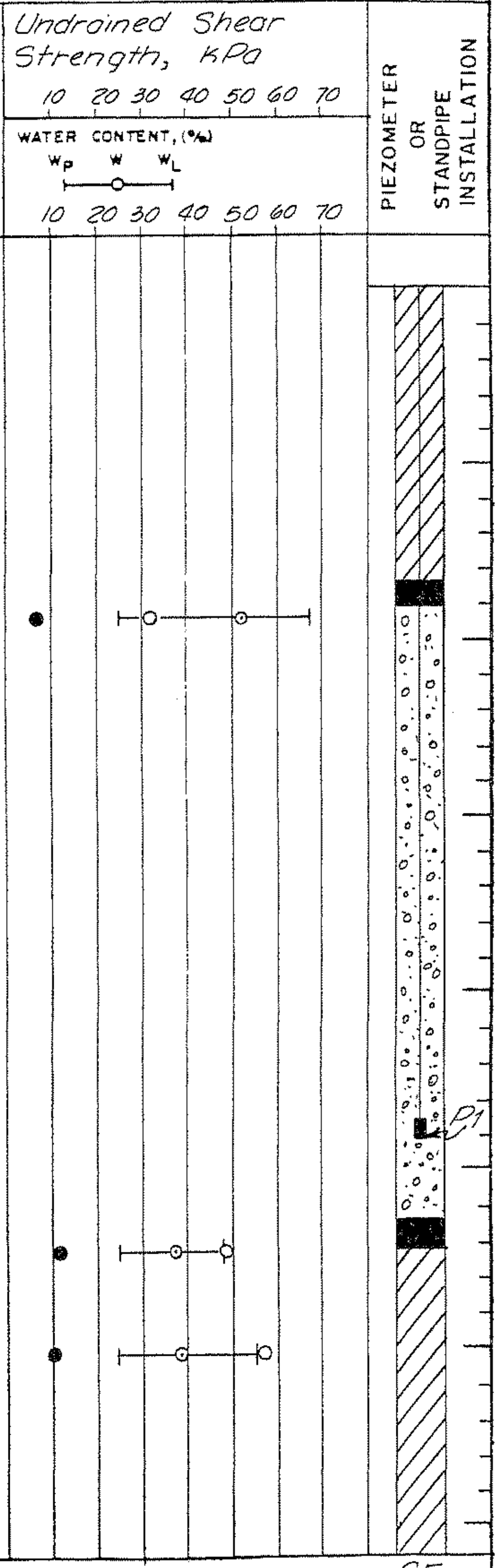
Project No. 782/179

RECORD OF BOREHOLE 105

SHEET 1 OF 2

LOCATION (see Figure 2)
BORING DATE Dec. 2-4, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
0.00	25.2	GROUND SURFACE					
		Loose to compact grey-brown mottled fine to medium silty SAND, layers of firm brown desiccated silty clay		1	D.O.	13	
				2	D.O.	9	
8.00	17.2	Firm to stiff blue-grey sensitive silty clay		3	T.P.	Ph	
11.00	14.2	Compact to dense grey & brown silty medium SAND		4	D.O.	17	
13.00	12.2	Dense to very dense grey fine to medium silty SAND		5	D.O.	33	
				6	D.O.	87	
				7	D.O.	62	
				8	D.O.	52	
25.00	0.2	Stiff grey sensitive silty CLAY, occ. partings of silty fine sand		9	T.P.	Ph	
				10	T.P.	Ph	
		Cont'd					



VERTICAL SCALE
1:200

DRAWN SF
CHECKED RO

Golder Associates

Project No. 7821179

RECORD OF BOREHOLE 105

SHEET 2 OF 2

LOCATION (see Figure 2)

BORING DATE Dec. 2-4, 1978

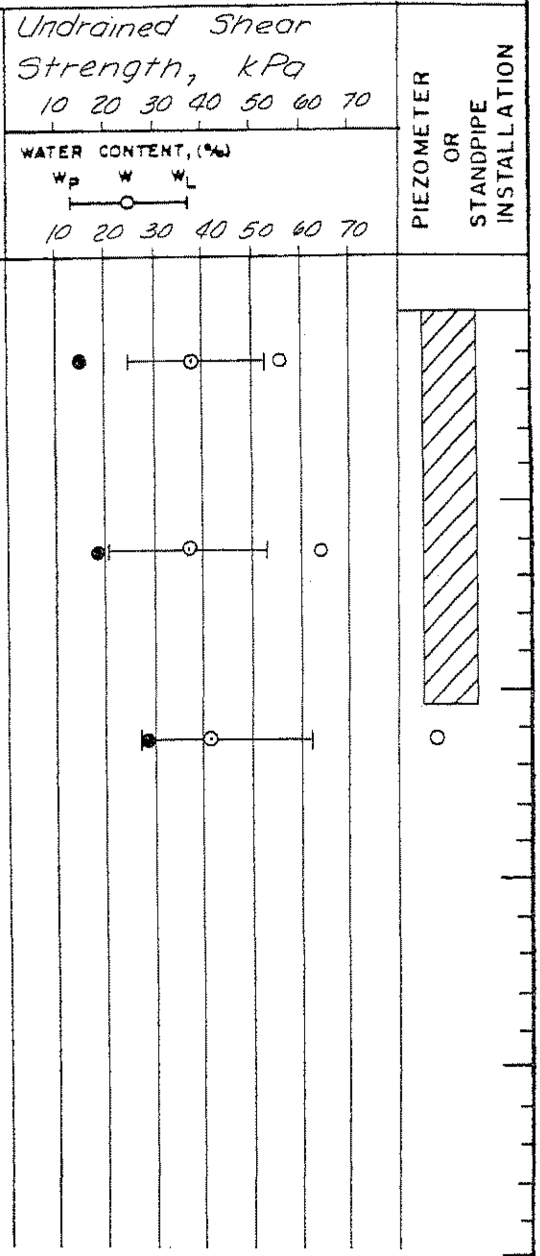
BOREHOLE TYPE Rotary

BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
		Stiff grey sensitive silty CLAY, occ. thin layers and zones of silty fine sand		11	T.P.	Ph	
40				12	T.P.	Ph	
45 — 45.60	-20.4	End of Borehole		13	T.P.	Ph	
50							



LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

Piezometer Installation

- ▨ General Backfill
- ■ Bentonite Seal
- ░ 'Pea' Gravel
- ▮ Piezometer

VERTICAL SCALE 1:200

DRAWN SF

CHECKED

Golder Associates

Project No. 7821179

RECORD OF BOREHOLE 106

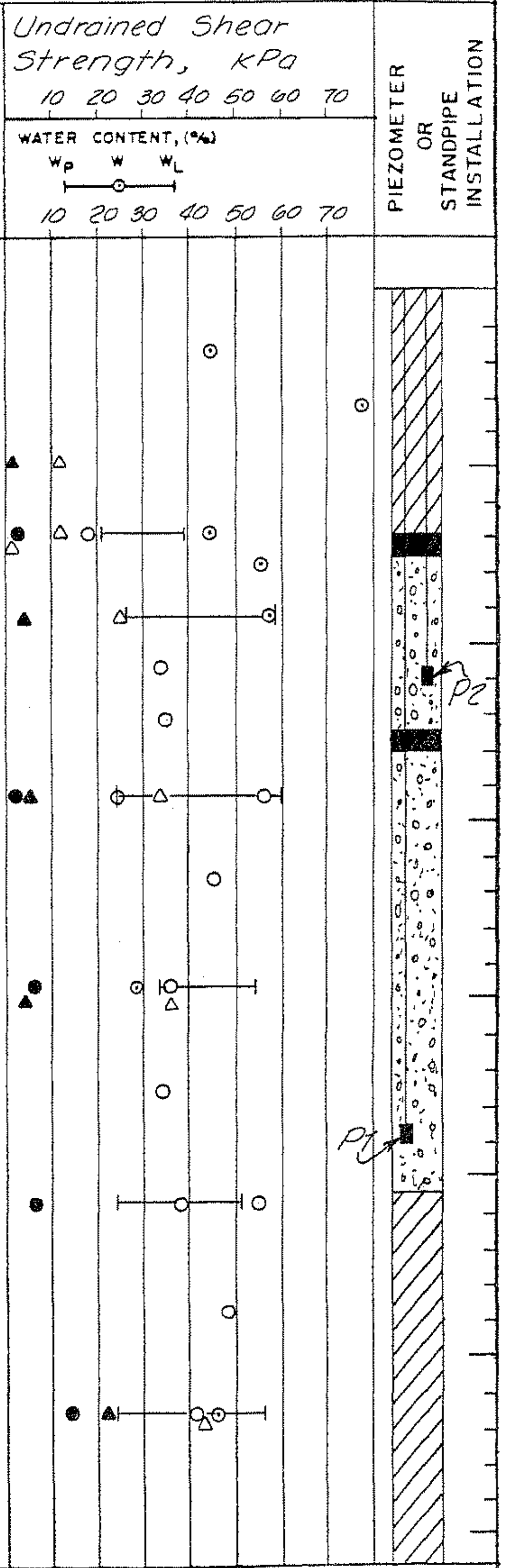
SHEET 1 OF 2

LOCATION (see Figure 2)
BORING DATE Nov. 22-24, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION
0.00	19.8	GROUND SURFACE
1.00	18.8	Topsoil & firm brown silt
2.50	17.3	Firm to stiff brown-grey clayey SILT and compact grey brown fine to medium SAND
8.00	11.8	Soft blue-grey sensitive silty CLAY
18.00	1.8	Soft to firm blue-grey sensitive silty CLAY occ. layers (up to 100mm.) of silty fine sand
		Firm blue-grey sensitive silty CLAY; occ. thin layers of silty fine sand.

SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M
1	D.O.	21
2	D.O.	Wr
3	T.O.	Ph
4	T.P.	Ph
5	T.P.	
6	D.O.	Wr
7	T.P.	Ph
8	D.O.	3
9	D.O.	Wr
10	T.P.	Ph
11	D.O.	Wr
12	T.P.	Ph
13	D.O.	Wr
14	T.P.	Ph
15	D.O.	Wr
16	T.P.	Ph

STRATIGRAPHY PLOT
LAB. TESTING



0 5 10 15 20 25 30 35

Cont'd

VERTICAL SCALE 1:200

DRAWN SF
CHECKED ES

Golder Associates

Project No. 7821179

RECORD OF BOREHOLE 106

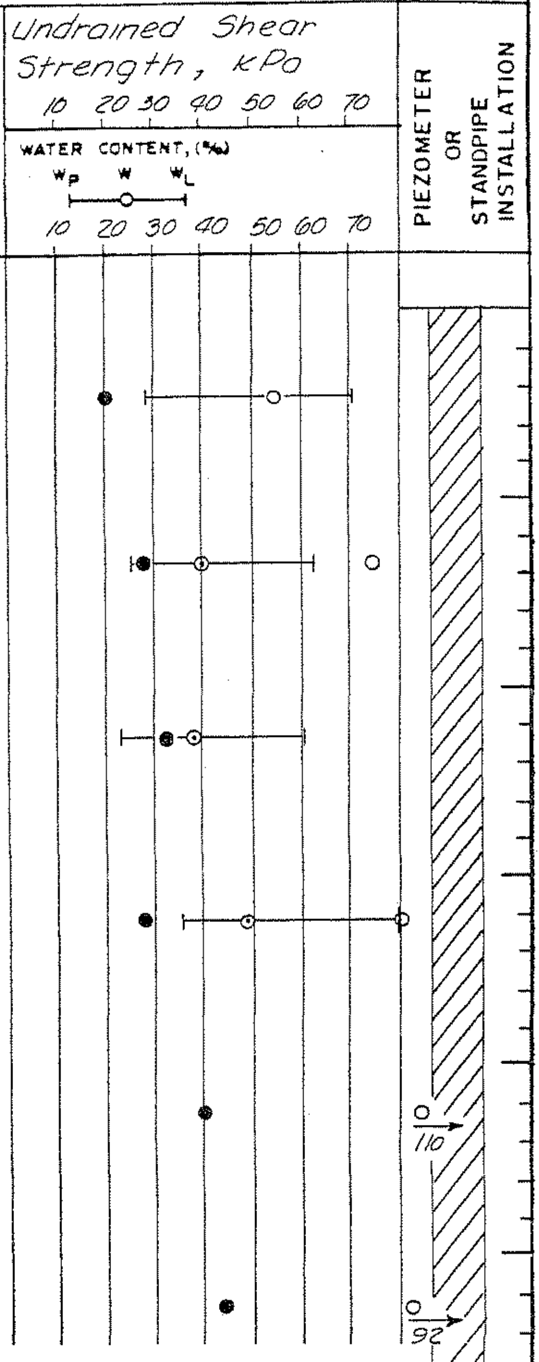
SHEET 2 OF 2

LOCATION (see Figure 2)
BOREHOLE TYPE Rotary
SAMPLER HAMMER WEIGHT 0.6 KN DROP 0.76 M

BORING DATE Nov. 22-24, 1978
BOREHOLE DIAMETER 114 mm
DATUM Ground Surface

DEPTH (metres)	ELEV. (m)	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / 0.3 M	LAB. TESTING
		Stiff blue-grey sensitive silty CLAY, occ. thin layers of silty fine sand		17	T.P.	Ph	
				18	T.P.	Ph	
				19	T.P.	Ph	
				20	T.P.	Ph	
54.00	-34.2	Stiff to very stiff blue-grey silty CLAY, occ. layers of silty fine sand		21	T.P.	Ph	
				22	T.P.	Ph	
65.00	-45.2	Interlayered very stiff blue-grey silty CLAY and dense silty fine sand					
75.00	-55.2	End of Borehole					

40
45
50
55
60
65
75



LEGEND

	Undisturbed	Remoulded
Field Vane	△	▲
Lab Vane	○	●

Piezometer Installation
▨ General Backfill
■ Bentonite Seal
▒ 'Pea' Gravel
▮ Piezometer

VERTICAL SCALE 1:200

DRAWN SF
CHECKED RD

APPENDIX II

Photographs of Site

Photo 1: Aerial oblique view of riverbank in study area. Fir Street slide at centre, Haney slide at far right.

Photo 2: Aerial view looking west along CPR tracks

Photo 3: Rip-rap protection on riverbank slope below CPR tracks.

Photo 4: Rip-rap protection. Note area where rip-rap has slumped into river, indicating ongoing undercutting and erosion in river.

Photo 5: Minor Port Hammond Slide. Note slide backscarp and hummocky slide debris terrain. Area is developed as a golf course.

Photo 6: Major Port Hammond slide, view to the north, ie. into the slide. Major backscarp concealed by tree cover in background. Note on left, backscarps of subsidiary slides which occurred laterally into main body of slide.

Photo 7: Fir Street Slide. Note hummocky slide debris terrain and existing development.

Photo 8: Drainage stabilization of surficial slide on upper slope above CPR, at approx. Section 16 (CPR Mile 103.9). Drainage consists of perforated pipes installed in gravel backfilled trench.

Photos 9 and 10: Surficial slides at crest of upper slopes. Note formation of backscarps.

APPENDIX III

Newspaper Report of Haney Slide (1880)

A RIDGE OF LAND BREAKS FROM ITS MOORINGS

And Becomes a Floating Island.

CURIOUS AND ALARMING EVENT ON THE LOWER FRASER.

The Westminster papers received last evening record an extraordinary occurrence at Maple Ridge on Friday last between 2 and 3 o'clock in the afternoon. A sound resembling the discharge of a heavy cannon was heard, and all those residing on the bank of the river rushed out of their homes to see the cause of the strange commotion. They beheld, not the quiet and tranquil river, upon whose bosom "Jack Frost" was quietly reigning, but a gigantic wave about seventy feet in height, rushing onward like a mighty invader, demolishing and conquering everything in its course. As the terror-stricken citizens cast their eyes up the river, they there beheld the cause of Father Fraser's anger, for his progress was interrupted by a great slide of about twenty acres of Mr. Justice Howison's land; and now as you journey up the river (which was here about half a mile wide) you will behold large trees growing two-thirds of the way across. The wave naturally swept with the greatest force opposite the slide; here, the trees on fifteen acres were mowed down as though they were ferns, and huge firs were stripped of their branches fully twenty feet from their roots. Mr. William Edge, who was clearing land here, was frightfully wounded by the falling trees and flying ice; and when found by his sons, after much searching, he was insensible, and about ten feet from the ground on some trees; he is not expected to recover. Messrs. Ewen & Co.'s fishery was struck and succumbed as though it was a card house, leaving only a few boards on the ground to remind the owners they had had a house there. It next swept against the public wharf, and the slip gave way; onward it rushed, and Mr. Muench's wharf came down with a crash. Two valuable horses belonging to that gentleman were struck by the water and ice, and narrowly escaped. A scow belonging to Mr. Maio was carried about a hundred yards, and as the water receded, it was left, like Noah's Ark, high and dry on a hill about forty feet high. Bridges along the river's bank were demolished, and all boats on the river as far as heard from (9 miles) were destroyed. Great excitement prevails there and those of the inhabitants living on the bank are contemplating moving their houses back. Loss and damage to property not known.

OUR SPECIAL CORRESPONDENT VISITS THE LOCALITY.

New Westminster, Feb. 3, 1880.

Editor Colonist:—Your correspondent learning that the tug Princess Louise, jr., Capt. ~~Meyers,~~ was going to Maple Ridge at 1 p. m. to-day put in his appearance at the dock and in a short time the juvenile steamer was ploughing up the noble Fraser. There were on board Dr. Sievewright, who went upon professional business, Messrs. J. A. Webster, Wise, Ewing, Howison, of the Occident, a few other gentlemen and the writer.

Maple Ridge is situated upon the north bank of the river, about twelve miles above the Royal City, and as we reached the Howison dock the "slide" was quite apparent a short distance ahead. Proceeding to the residence of Mr. George

Newspaper report of Haney Landslide -
Victoria Daily Colonist, Feb. 5, 1880

Howison, that gentleman and his brother, Mr. J. W. Howison, the owner of the "slide," and Mr. Webster and the writer crossed a large field and a clearing, and upon reaching the southeast corner of Mr. J. W. Howison's 110 acres it was quite a sight to see the upheaval. The bank is at that point, and to the west for some distance, 100 feet high; the river is nearly a quarter of a mile wide, and a bend in the river a little upstream causes a swift current to wash the shore where the slide occurred. It is probable that the water thus undermining the bank, together with a ravine a few rods back of the ridge which gave way, accumulated water that washed through the bank. Having given some idea of the location before the mishap, we would now say that about half-past two o'clock last Friday while Mr. Geo. Howison was busy upon his farm he heard a noise like a heavy wind blowing and on looking in the direction of his brother's farm witnessed the falling and disappearance from sight of large fir and other trees, as well as the sinking of the river ridge. Hurrying to the scene the sight presented was one that greatly surprised him, as well as others who soon joined him to gaze at what was once supposed to be about twenty-five acres of the river frontage of a choice farm. But there were the ruins of land and forest, stretched half-way across the river, with, at the outer edge of the sort of island, large and small trees as erect as if they had not moved a hundred or more feet from where ten minutes before they stood apparently as firm as any forest growth. However, they still held position in their native soil, which had moved en masse and left space behind. It is true that the trees are only eight or ten feet above the water, instead of over 100 feet as before the slide. Of the immense quantity of earth that moved forward and now covers up huge trees and fills up a large space of the river we can form no estimate. The bank would seem to be formed of light, yellow-red earth for the first dozen feet, then blue clay for twenty feet, and gravel beneath, through which water is coursing. For 150 feet back from where the bank broke away there are large cracks along the surface, and in some places large trees standing that upon the first storm will tumble over and carry quantities of the remains back with them, thus greatly adding to the injury of Howison's farm. To do that gentleman justice we must say that he takes the "change of front" like a philosopher, and thinks there will be land enough left for all the work he will ever do upon the farm, believing he has a better thing in the Occident Hotel. His present apprehension is for the frontage and neat cottage and superior fruit trees of his brother's farm, which certainly are not out of harm's way and sooner or later will have to yield. The rush of land into the river caused the water to rise nearly twenty feet to the top of the Howison wharf, north side of the river and, also, high at the brick yard of Mr. Heney, a little above on the same side, but the great wave across the river raised over the ten-foot bank, and as Mr. Wm. Edge was working nearly opposite the slide he was washed before the monster wave that rose high (as witnessed by Mr. Geo. Howison and experienced by Mr. E.), and thrown against trees, stumps, etc., and when found was unconscious. We learn from Dr. Sievewright and the gentlemen who crossed over in the tug to the north side that Mr. Edge is seriously bruised inwardly and fear is entertained of his recovery. He has a wife and six children who are in great anxiety respecting his situation.

— Messrs. Wise & Ewen's fish packing house was demolished, trees prostrated and washed inland, and a number of

APPENDIX IV

Report on Riverbank Erosion

- Northwest Hydraulic Consultants Ltd.

northwest hydraulic consultants ltd.

222 brooksbank ave., n.vancouver, b.c. V7J 2C1
tel. (604) 980-6011

edmonton
vancouver

Our File Number

March 23, 1979

Golder Geotechnical Consultants Ltd.,
224 West 8th Avenue,
Vancouver, B.C.
V5Y 1N5

Attention: Mr. R.M. Wilson, P. Eng.

Gentlemen:

Re: Fraser River - Haney to Port Hammond
Bank Erosion

Following your request of 18 December 1978 we have briefly reviewed river data in the subject area with the objective of assessing erosion of the right (viewing downstream) bank. Your specific interest was to confirm that erosion is taking place, and to attempt to quantify any erosion rates and their relative values along the reach shown on the attached Figure. Also, you requested a recommendation on a suitable method of monitoring possible future erosion.

Our investigation included review of cross section data taken in 1978 by the B.C. Water Investigations Branch, a field inspection, a telephone discussion with C.P. Rail's maintenance staff, and review of historical air photography. Knowledge acquired by NHCL on the specific area through a previous study with Golder Associates was also applied to this investigation.

Existing Conditions

The Haney-Port Hammond reach is located in a large meander bend of the Fraser River, Figure 1. It is typical that the outside bank of such a bend is continously susceptible to erosive forces. Evidence from field observations and May 1978 air photography indicates that most of the bank in question is subjected to varying degrees of erosive forces and the shape of sounded cross sections indicates there is no relatively inerodable natural material present beneath the water surface. However most of the bank above the low water surface is resisting lateral movement due to the presence of riprap, which has been placed by C.P. Rail to protect their adjacent railway track.

/2

Additionally, the presence of log booms, piles and dolphins along the bank (see attached photographs) probably provides some further resistance to erosive forces. Accordingly, it is our opinion that the existing erosion rate along this bank is very slow, probably less than an average of 0.1 metres per year.

In general, the maximum erosion potential exists between about points A and B, Figure 1, and the potential decreases to negligible by the Port Hammond area (point C on Figure 1). A good indication of the change in erosion potential with increasing downstream distance along the reach is shown on Figure 2. This figure shows the thalweg (minimum bed) level between survey cross sections 8 and 29 - ie. between the 1880 slide and Port Hammond. (Sections are shown on the Corporation of the District of Maple Ridge Topographic Mapping, supplied by Golder Associates). The thalweg rises from a depth of 23.4 metres at the slide to 14 metres at Port Hammond. It is typical that the maximum depth in a river channel increases and moves from the centre area towards the outside bank as the flow enters into a bend, and then decreases and moves more towards the centre as flow leaves the bend and straightens. Thus Figure 2 shows that the strongest erosion potential is at the 1880 slide area, and that the least (and probably negligible) erosion potential is at Port Hammond, Section 29. The only exception is the short reach of about 400 metres, between the slide and about Section 12 (Photo 9), which is protected to some degree by the toe of the slide.

It is interesting to note the two scour holes located at about sections 14 and 21, Figure 2. These holes are probably created by a large volume of riprap lying below the water surface at these points (see photos 1 and 8).

C.P. Rail Maintenance

Discussions with C.P. Rail maintenance staff in Vancouver revealed that the area has a long history of stability/ erosion problems. The major portion of the existing riprap was placed in 1967-68 by dumping from a barge; another large placement was done in 1978, and more is planned for 1979 and beyond. Recent placements have had more volume dumped per foot (Photo 8), and rock placed earlier shows evidence of sloughing and loss of material. C.P. Rail's maintenance program is strong evidence of the continuing erosion potential in the area.

/3

Historical Erosion

Based on review of historical air photography dating back from 1978 to 1930, no general tendency to measurable erosion could be seen above the water surface. Localized erosion of about 6 metres in 40 years extending over a length of about 100 metres was observed near Section 21.

Monitoring Erosion

The following program is recommended to monitor erosion at the site:

(1) Re-survey, at regular intervals and at the same time of the year, cross sections at the *same* locations as was done in 1978. It is preferrable to survey at or near spring freshet conditions (eg. late June, early July) and a maximum interval of 2 years is suggested. The cross sections should be carried up to a stable reference point - such as the railway - so that changes in horizontal distance from waterline can be measured.

(2) Visit the site annually and take representative photographs to compare with previous years. Any significant changes or local erosion pockets should be noted.

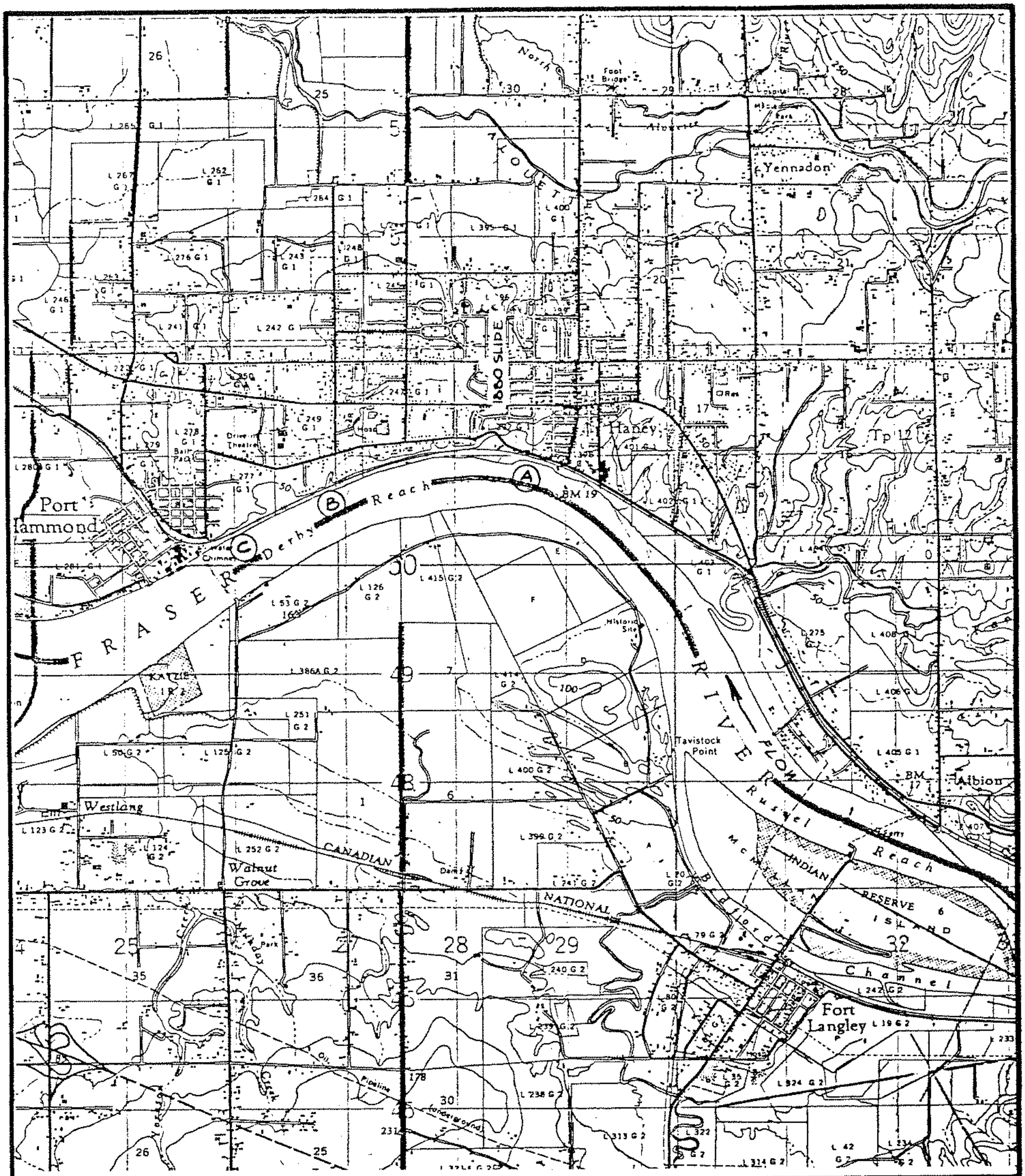
If you have any questions on any aspect of this investigation, please feel free to call us at any time.

Sincerely,

NORTHWEST HYDRAULIC CONSULTANTS LTD.

M.H. Okun, P. Eng.
Branch Manager

MHO/smh
Encl.



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SCALE : 1 : 50,000

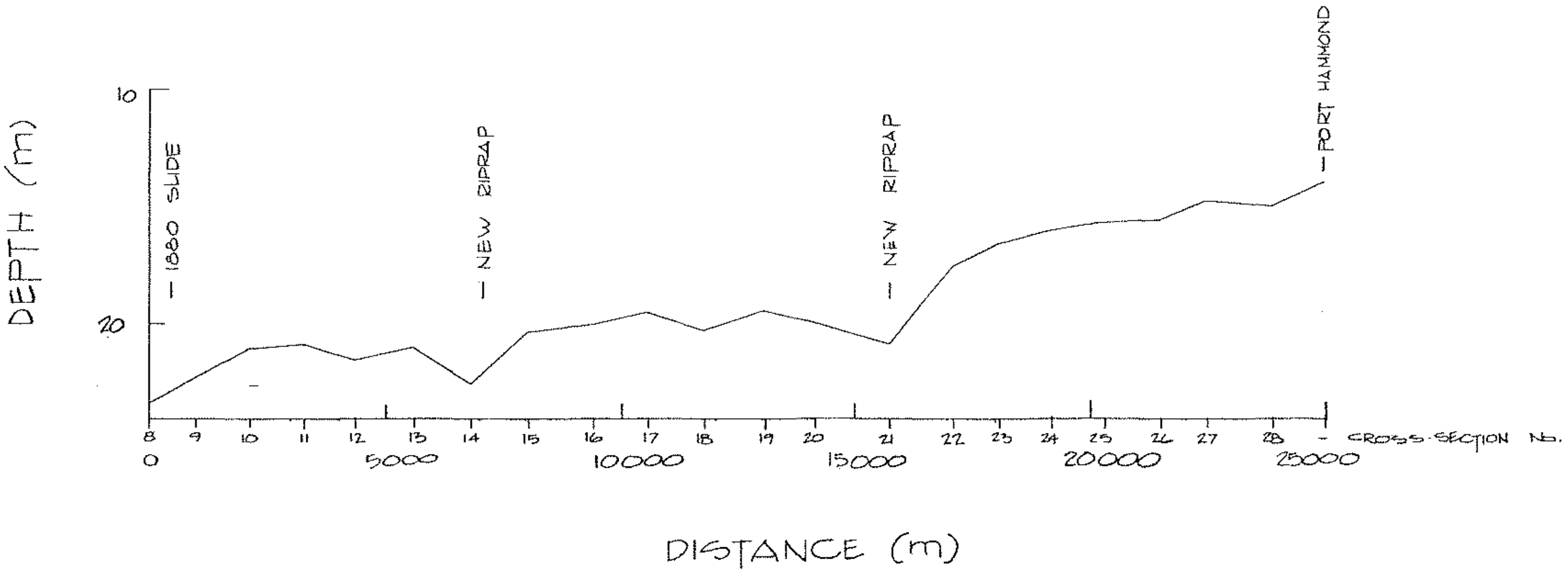
GOLDER ASSOCIATES LTD

FRASER RIVER - PORT HAMMOND AREA

AREA PLAN

northwest hydraulic consultants ltd.

Drawn: V.U. Date: MAR. 79 FIG. 1



NOTES :
CROSS SECTIONS NO. REFERS TO TOPOGRAPHIC MAP,
CORPORATION OF THE DISTRICT OF MAPLE RIDGE,
SHEETS NO 14 AND 24 .

GOLDER ASSOCIATES LTD.
FRASER RIVER - PORT HAMMOND
AREA

THALWEG PROFILE

northwest hydraulic consultants ltd.

Drawn: V D | Date: MARCH 79 | FIG. 2

northwest hydraulic consultants ltd.

FIELD TRIP TO HANEY - PORT HAMMOND

13 January 1979

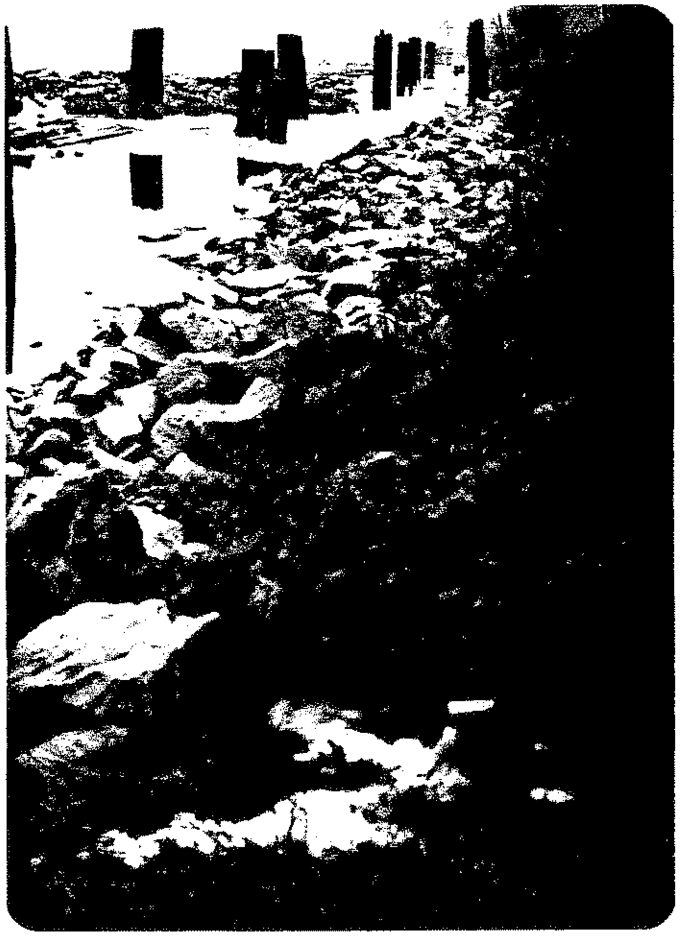


Photo 1. View upstream towards Port Hammond (mill in background). Note riprap which stops well below top of railway embankment. Location approximately Section 21. Also note extent of dolphins and piles along bank.

Photos 2 & 3. View downstream towards Port Hammond location approximately Section 19. Note riprap in foreground is well below railway grade, while a short segment is as high as the railway grade in the center background. Riprap in this area is relatively poorly graded, although the rock is of good quality.

Photos 4 & 5. View upstream approximately from cross section 16. Riprap extends almost to railway grade at this point. This appears to be relatively newly placed rock. Note material deposited over riprap in centre of photo. Again note extent of dolphins and log booming. 1880 Slide is at centre background of photo.

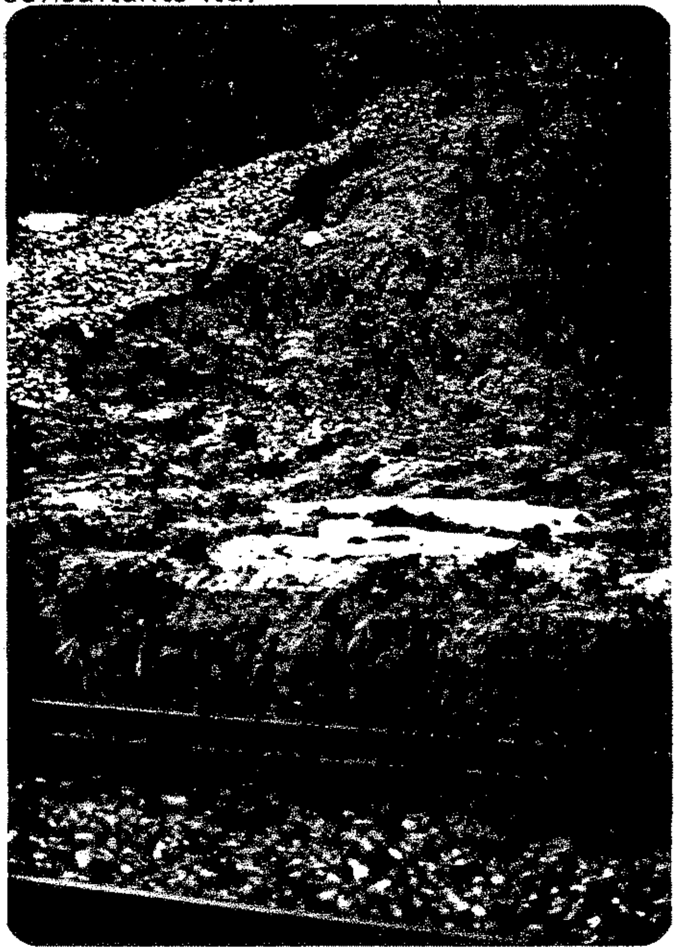


Photo 6. View to recent small slide area just upstream of previous photo and to north of tracks.

Photo 7. Assumed deposition from slide of photo 6. Note culvert exit in embankment at the right centre of photograph.

Photo 8. View upstream from about cross section 14.

Photo 9. View upstream around Section 12. Between here and the 1880 slide, there has obviously been deposition as opposed to erosion. Photo taken from protruding culvert.

Photos 10 & 11. View downstream from the same location as photo 9. Note steeper natural bank and evidence of erosion in this area. There is a considerable amount of launched riprap visible at the waterline. Again note extent of dolphins and log booms.

Photo 12. Evidence of launched riprap from same location as previous 3 photos.

Photo 13. Another view upstream, a short distance upstream of photo 12. Riprap in this area has been placed for some time. Note old piles cut off above low water line at left. Also, looking upstream note intermittent placement of riprap.

Photo 14. View upstream towards 1880 slide from same location as photo 13. Again note older riprap, which is overgrown with local vegetation, and has launched some distance into the river.