APPENDIX II

GOLDER REPORT TO MINISTRY OF ENVIRONMENT
JULY 1983

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
WATER INVESTIGATION BRANCH
ON
GROUND WATER MONITORING AND STABILIZATION STUDY
FRASER RIVER NORTH RIVER BANK

MAPLE RIDGE, BRITISH COLUMBIA

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July, 1983

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TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF FIGURES

SUMMARY

The intention of the present study was to consider in greater detail those factors which affect the overall stability of the Maple Ridge escarpment. The major influence on the present stability is that of ground water or piezometric levels in tnatural strata and scour of the Fraser River at the slope toe. The study was to consider only deep-seated major failures such as those which have previously occurred at Haney in ~~1980~~ 1880, at Fir Street, and two slides located on the Maple Ridge golf course, the larger of which is referred to as the Port Hammond slide. These larger slides are all of a similar nature, except that two have retrogressed during failure to involve larger, secondary land masses. The two slides which retrogressed are the Haney and Port Hammond slides. The Fir Street slide and the minor slide at the golf course did not retrogress. They are, however, similar to the larger slides which have been of concern in the present study.

Local shallow slides occurring on the face of the escarpment are not of potential danger to the developed municipality area and have not been considered in our study.

Present Stability

The present study was primarily centred in the more critical area defined by previous studies. The results of the present study show that differential piezometric levels occur within the natural stratified soils forming the escarpment area. The results of the study also show that the deeper granular strata have lower piezometric levels than were anticipated from the earlier study. Further, the upper granular strata have considerably higher water levels which are also subject to considerable variation during periods of precipitation.

The stability of major land masses, such as the former failure areas, would be affected by both the upper and lower piezometric levels. The actual levels during failure can only be assumed, but based on assumptions which can be reasonably predicted, the stability of the areas can be shown to approach unity.

The stability of the more critical area between Section 11 and 16 is considered to be marginal if adverse ground water levels are assumed.

Remedial Treatment

The stability of the existing slopes can be improved by lowering the piezometric pressures in the upper or lower levels. As the fluctuations in the upper strata are greater, and the drainage methods available to lower these levels are more economic, it is considered that this is the most practical method of initially dealing with the problem. We recommend that water levels in the surficial strata be controlled by installation of a shallow interceptor drain parallel to and within 50 m of the crest of the slopes. The provision of the interceptor drainage will improve the stability of the major land masses to a low but adequate value.

This proposed improvement will remain adequate unless considerable piezometric increases in the lower granular strata are encountered in future years. Such increases are not anticipated; however, continued periodic monitoring is recommended to assure these assumptions are correct and to provide a greater data base than is available to date.

If excessive increases in the deep piezometric levels are noted during the continued monitoring program, then future deep drainage measures would be required. We do not recommend that this additional deep treatment be installed at this time, and believe that it will not be required in the future.

River Erosion

The effects of river erosion on stability of the major land masses is secondary unless allowed to continue until the overall slopes are critically undercut.

We recommend that river erosion protection be provided in the most critical slope area which also corresponds to the area of observed scour.

We believe that this protection is required as the actual stability of the adjacent slopes is marginal under adverse ground water conditions, and provision of the remedial drainage will not increase the factor of safety by a high percentage.

We recommend that continued monitoring be carried out over an extended period to fully appreciate the rate of scour or lack of it.

In conclusion, we believe that the results of the present study have shown that the overall mass stability is slightly better than initially assumed from our earlier study. We recommend a staged remedial treatment and monitoring program. The initial work would involve shallow drainage along the total escarpment length and river erosion protection in the most critical area defined. These works would involve estimated expenditures of $1.2 million and $0.9 million, respectively. Additional works which would provide an additional degree of safety are not presently recommended based on analyses of present data. These would only be considered if future monitoring results indicated a need.

Benefits of Proposed Remedial Action

To improve the stability of the escarment with respect to large mass failures, we recommend the remedial works as discussed. These remedial measures are intended to provide sufficient security to permit continued development of the area. The proposed works, if totally effective, would also be of considerable benefit to reducing local shallow slope surface failures. The erosion protection will also minimize water front slope failures and loss of railroad grade in the most critical area.

1.0 INTRODUCTION

This report deals with the slope stability of the Maple Ridge escarpment adjacent to the Fraser River.

Golder Associates have been retained to carry out a ground water monitoring study in the area adjacent to the Fraser River between Haney and Port Hammond in the District of Maple Ridge, B.C. (see Figure 1). The results of a previous study on the stability of the slopes in this area are presented in our report 782-1179, dated August, 1979.

The previous report identified areas where calculated safety factors were low. The ground water pressures were considered to be crucial to the stability of the slopes. The purpose of the present work was to install instrumentation and monitor the ground water levels in the area. A pump test was carried out to obtain data required for assessment of possible schemes to control ground water pressures in critical slope areas. Northwest Hydraulic Consultants reviewed river bank erosion and have outlined possible erosion control measures.

The site and soil conditions in the study area are outlined in the previous 1979 report, and will not be discussed in detail in this report.

This report summarizes remedial work recommended to control ground water levels and to minimize river erosion. Future monitoring requirements are defined.

The factual data on the installation and monitoring of the piezometers and the pump test are presented in the attached appendices.

2.0 DISCUSSION

2.1 Subsurface Stratigraphy and Ground Water Regime

The subsurface stratigraphy, as determined during the previous study in 1979, consists of interlayered silty clay and sand, with varying amounts of silt. These strata are of glacio-marine origin, and the thickness of these layers is variable, as confirmed during the recent installation of piezometers.

An assessment of the ground water regime has been made following examination of ground water levels recorded in the various stratigraphic zones and observations made of the site respect to seepage, topography and rainfall.

Examination of ground water levels indicates a decrease in hydraulic head with depth, typical of a ground water recharge zone with ground water flowing with a downward vertical component through the soil profile. Recharge to the local shallow ground water system is via infiltration of precipitation. Piezometers installed in the near surface silty sands show a rapid hydraulic response to precipitation events indicating periods of ground water recharge. The recharge of ground water due to local infiltration is most pronounced in the upper 10 m below ground surface. Ground water within the silty sand will tend to flow either horizontally toward the slope face, where it appears as seepage discharge, or seep through the underlying less permeable silty strata. The deeper piezometers, and those completed in the siltier strata, do not show a rapid fluctuation in water level, indicating that they probably receive slow recharge via seepage from the upper strata. The deeper strata are likely to receive recharge through flow from the regional area to the north. These are represented by granular strata generally below about 30 m depth.

2.2 Piezometric Levels and Previous Slides

Back-analysis of the previous slides in the area has resulted in the conculsion that very high ground water pressures would be required to cause slope failure if peak shear strengths are assumed. The presently established ground water profiles would require failure to occur at lower back-analysis strength parameters.

It is possible that very high ground water levels did exist in the slide areas at the time of failure. This could occur if soil conditions adjacent to the slide areas were such that rapid recharge of the soil strata occurred during periods of high precipitation. There is some evidence to suggest that more granular soils exist in the areas adjacent to the previous major slides. The surficial geology (Geological Survey of Canada, Map No. 1484A) indicates raised pro-glacial gravel and sand deposits up to 40 m thick exist to the north of the glacio-marine deposits in the slope area. The sand and gravel deposit is believed to extend all the way to the Fraser River in the area immediately to the west of the Port Hammond slides, and also to the backscarp of the 1880 Haney slide. Borehole 105 (Golder Report 782-1179) is located close to and east of the Major Port Hammond slide and indicates that the soil consists almost entirely of sand or silty sand to a depth of about 25 m. Similarly, coarse gravel was encountered in test pits put down near the golf club house. However, Borehole 106, located at the backscarp of the Minor Port Hammond slide, shows that clay strata are predominant. Available data from studies carried out for the recently constructed Haney By-pass indicates an 8 m thick sand and gravel layer, between approximate elevations 12 m and 20 m, exists to the northeast of the 1880 Haney slide, and that a surficial layer of silty sand and gravel at least 4 m thick underlies the existing ground surface within the actual slide area.

Higher water pressures could also be caused by blockage of drainage from the sandy layers by surficial slides or by freezing on the slope surface.

The former slides could have occurred with lower water pressures if pre-existing shear planes were present and if residual shear strengths only could be mobilized. It has been postulated that pre-existing shear planes may have been generated due to straining when ice masses which existed adjacent to the slopes in this area melted. There is some evidence on airphotographs to support the presence of such shear planes in the major slide areas. There is presently no evidence to support the presence of pre-existing shear planes in the area between the major slides, but it is nevertheless a posibility. Such planes would be extremely difficult to dtect, even with continuous soil sapming or probing.

2.3 Effect of Piezometric Levels on Slope Stability

The following discussion concerns deep seated failures with a backscarp some 40 to 50 m from the slope crest. It does not apply to shallow surficial failures such as occur regularly above the C.P. Rail bench.

The results of stability analyses carried out for typical slope sections are presented in our previous report 782-1179, dated August 1979. At that time, sections 12 to 15 (see Figure 2) were identified as having the lowest safety factors with respect to deep seated failures. Based on the available data in 1979, it was assumed that the phreatic surface was located about 10 m below the surface at the potential slide backscarp and that it followed along the slope face to the C.P. Rail bench. The water pressures below the phreatic surface were assumed to increase hydrostatically with depth. Assuming peak shear strengths with these ground water conditions, the factor of safety for section 12 was calculated to be in the range of 1.05 to 1.10. These results indicated that the assumed peak shear strength parameters were the minimum that existed, since lower shear strengths with the assumed 1979 water pressures would have yielded the impossible result of a safety factor less than unity for an existing slope. It was determined that higher or lower water pressures could have a significant effect on the slope stability.

The recently installed piezometers indicate that the piezometric pressures in the upper escarpment area near the river do not increase hydrostatically with depth, and that they are significantly lower than was assumed to exist in 1979. Assuming peak shear strengths apply, the calculated safety factors under the apparent existing ground water conditions would be higher than those discussed above. The percentage increase, considering the lower piezometric profiles and the peak shear strengths, would be about 10 to 20 per cent higher.

If pre-existing shear planes exist along part of the potential slip surfaces and are considered in conjunction with the recent 1983 ground water data, calculated safety factors would be lower and similar to those calculated using peak shear strengths and the very high ground water conditions.

Limited data is available to allow prediction of expected fluctuations in the piezometric pressures. Piezometric pressure head fluctuations of up to 1.4 m have been observed in the recently installed upper piezometers located within about 10 m of ground surface, but only minor fluctuations have been observed in the deeper piezometers. It is likely that ground water fluctuations within the general site area will be highly dependent upon the proximity of any major granular deposits to the layered soils which exist in the slope area.

We have carried out a sensitivity analysis to examine the effect of possible ground water fluctuations on the stability of the slope at cross-section 12 (see Figure 2). In these analyses, we have assumed peak shear strengths, and have assumed that only modest increases (up to 5 m) above the present piezometric levels would occur in the deeper strata. Three alternative ground water conditions were assumed for the upper strata within 25 m of the ground surface as follows:

(1) Piezometric levels in the upper strata at existing ground surface.

(2) Piezometric levels in the upper strata 5 m below existing ground surface.

(3) Piezometric levels in the upper strata at existing levels, about 9 m below existing ground surface.

Case (1) represents the worst anticipated ground water conditions without any control of surficial drainage, and the calculated safety factor, which is marginally below unity, indicates that slope failure would occur. Case (3) represents the case where ground water levels in the upper strata are controlled at their present levels, possibly using a well dewatering system. The calculated safety factor for Case (3) is acceptable under static conditions (F_S = 1.2), but would become marginal under design earthquake loading. Case (2) represents an intermediate condition where piezometric levels in the upper strata are prevented from rising to within 5 m of ground surface. Under these conditions, the calculated static safety factor is acceptable (F_S = 1.2) with existing piezometric pressures at depth, but becomes marginal if the deeper piezometric levels increase by 5 m.

It is apparent that ground water conditions have a major influence on the stability of the slopes. If piezometric levels in the deeper strata do not increase significantly above their present levels, a shallow ground water control system would be adequate to control the stability of the slopes. However, if a deep aquifer water level increase of greater than 5 m were shown to be possible, the stability would be reduced to levels of concern. A deeper dewatering system would then be required to control water pressures at or below their present levels. A deep dewatering system is not justified based on available data provided the upper water levels are controlled, and provided monitoring shows only moderate deep pressure changes.

Water pressures would be expected to increase if drainage is impeded by freezing or slides on the slope faces, as well as in response to precipitation. Of these factors, precipitation and freezing are considered to be most important since the occasional surficial slides, which originate above the CP Rail bench, are of limited dimensions and would not be expected to significantly affect the overall ground water levels in the slope area.

2.4 Effects of River Bank Erosion on Slope Stability

Northwest Hydraulic Consultants Ltd. have carried out an assessment of river bank erosion and their report is presented in Appendix A. Additional comments are included in their report of March, 1979 (see Appendix IV, Report 782-1179).

Erosion of the river bank will reduce the overall stability of the slopes. The significance of this factor depends on the magnitude and rate of erosion. The 1978 and 1981 river survey data indicates overall erosion of the river bank which would significantly affect the stability of the slopes has not occurred during that period, although some local pockets of erosion were noted just downstream of the 1880 Haney slide. As discussed in NHC's report, peak flows were low between 1978 and 1981, and also rip-rap placed by C.P. Rail may have prevented or reduced erosion in some areas.

Major, and expensive, bank protection measures would be required to ensure erosion of the river bank does not occur, particularly during periods of high peak flow. The risk of significant erosion under such conditions is difficult to assess. However, it is the opinion of NHC that significant erosion could occur in the area between cross-sections 5 and 19 during a major Fraser River flood.

Peak ground water levels are not generally associated with peak river flows, and this somewhat lowers the risk of an immediate slope failure should erosion occur during high river flows.

It is our opinion that ground water control is more critical to the stability of the slopes at present than control of river bank erosion. Furthermore, the risk of slides occurring, even with significant erosion, could be reduced if a dewatering system is in operation.

As the stability of the more critical area between Sections 11 and 16 is considered to be marginal with adverse ground water levels, presently, and would be increased to low acceptable values by ground water control measures, it is considered advisable to also control erosion in this area.

2.5 Remedial Treatment Alternatives

Under the present ground water conditions, the slopes in the critical Zone A (Section 11 to 16) are considered to have an adequate factor of safety against major, deep-seated rotational failure.

Significant fluctuations of the ground water levels in the upper surficial strata are anticipated. Fluctuations in the lower confined strata are expected to be less, but may vary locally depending on the proximity of granular deposits to the north of the slopes. Our analyses indicate that possible ground water fluctuations in the surficial strata could reduce the safety factors below acceptable levels, particularly during seismic loading. The stability of the existing slopes could be maintained at an acceptable level by ground water control and by preventing erosion of the river bank.

The following is a summary of alternative measures and their benefits:

(1) Interceptor Drain Only
- Control of anticipated ground water fluctuations in surficial strata.
- Lower risk of shallow slides during periods of heavy precipitation.
- Effective in reducing the risk of deeper slides and retrogression. However, if lower level ground water levels rise independently, the effectiveness of a drain may not be adequate.

(2) Interceptor Drain plus River Erosion Protection
- as for (1).
- Prevent the loss of toe support, and therefore creation of future stability problems due to river scour.

(3) Interceptor Drain plus Deep Dewatering System
- Control anticipated ground water fluctuations in shallow and deep strata to 30 m depth.
- Lower risk of slides during heavy precipitation or due to regional ground water infiltration.
- Significant improvement in overall stability relative to shallow and deep-seated slides regardless of future ground water levels.
- Lower risk of retrogression of frontal slides.

(4) Interceptor Drain plus Deep Dewatering System plus River Erosion Protection
- as for (3).
- Prevent the loss of toe support, and therefore creation of future stability problems due to river scour.

3.0 RECOMMENDATIONS

The following remedial measures are intended to provide sufficient security to permit continued development of the area. It is assumed that monitoring will continue, and additional stability improvement measures will be undertaken if required in the future.

Since the risk of slope failures which would extend back about 40 m from the crest of the slopes appears to be low at present, we suggest that the main thrust of remedial measures initially be in improving surficial drainage.

If deep dewatering is considered necessary based on future monitoring, we recommend that this be achieved using wells as discussed below. Consideration was given initially to controlling ground water pressures using inclined drains driven from the toe of the slope at the C.P. Rail bench. Such a system may be effective in stabilizing shallow surficial slides, but would be less effective in draining deeper strata at the potential deep failure surface location. For this reason, inclined drains were not considered as viable as pumped wells for improvement of stability with respect to deep-seated failures.

3.1 Improvement of Surficial Drainage

Significant ground water level fluctuations have been observed in the upper strata since monitoring of the new piezometers commenced in December, 1982. As expected, the ground water levels in these upper strata appear to respond quickly to precipitation.

Improvement of surficial drainage is considered to be an essential first step in improving the stability of the slopes with respect to both shallow surficial slides, and deeper rotational failures. The improvement is expected to be greater for shallow slides, but such a system may also be adequate for the control of deeper slides. To this end, the District of Maple Ridge should be encouraged to develop a storm drainage plan which would minimize infiltration of water into the ground. This should include all septic discharge, storm drainage and runoff from adjacent upland areas to the north.

In addition to general storm drainage improvement, we recommend that consideration be given to installation of an interceptor drain parallel to and within 50 m of the slope crest.

A preliminary interceptor drain design is presented on Figure 5. The drain could be installed in stages, but should eventually extend along the entire length between the 1880 Haney slide and Port Hammond, about 1800 m. If a staged installation is considered, we recommend that the initial section be installed between sections 11 and 18, about 850 m, see Figure 2.

The drain could consist of a perforated pipe located as deep as is practical below ground surface (5 to 6 m). The pipe should be surrounded by coarse sand and gravel or drain rock, protected with filter cloth if required.

The pipe size should be determined based on estimated seepage flows with maximum anticipated ground water levels, as well as hydraulic considerations. The design should be such that transfer of water along the drain from one zone to another is minimized. Unless the perforated pipe can be bedded within a clayey layer, it may be necessary to use pipe which is perforated on the top only and/or to have frequent discharge points into a closed storm sewer. We recommend such a storm sewer be designed to accommodate discharge from a possible deep well dewatering system in the future.

For preliminary costing of an interceptor drain, we have assumed that a 300 mm diameter closed pipe with a 150 mm perforated collector pipe would be installed at an average depth of 5 m. The estimated cost of supplying and installing such a drain is summarized below:

	ESTIMATED COST/METRE LENGTH
Trench Excavation and Backfilling	$ 415
Supply and Installation of 300 mm Concrete Storm Sewer, 150 mm Perforated Pipe and Filter Cloth	175
Engineering	60
TOTAL ESTIMATED COST/METRE LENGTH	$ 650

The total estimated cost of installing an interceptor drain between the 1880 Haney slide and Port Hammond is $1.2 million. An interceptor drain could be installed in the critical area initially for about half the total cost.

We recommend that detailed testing be carried out prior to final design to define the soil stratigraphy along the proposed drain alignment. The investigation program should include an assessment of soil permeability and existing water levels for use in design of the drain. Recommendations on installation of the drain should also be included. It would be useful to install some additional piezometers in areas remote from the proposed drain for use as control in monitoring the effectiveness of the drain.

3.2 Future Ground Water Control Using Wells

Deep wells are discussed herein and would improve stability. We do not recommend that such installation proceed at this time, as the need for full slope dewatering has not been confirmed.

The calculated static safety factor, with respect to deep-seated failure, is acceptable with the range of ground water conditions experienced during monitoring. The proposed interceptor drain and general storm drainage improvement will help control anticipated ground water fluctuations in the surficial silty sand stratum. Should future monitoring indicate that a deeper dewatering system is required, we recommend that such a system be designed to lower water levels at least 5 m below the existing levels. This will increase the calculated static safety factor and provide an additional safety margin during seismic loading. A discussion of design concepts for a deep dewatering system is given in Appendix C.

3.3 River Bank Erosion Control

Northwest Hydraulic Consultants have concluded that significant erosion could occur during a major flood in the area between sections 5 and 19. The area identified as being critical with respect to slope stability in our previous study (Zone A, report 782-1179) extends from about Sections 11 to 16, a length of about 450 m. We therefore recommend that consideration be given to placing erosion protection in this critical area at an estimated cost of $0.9 million. This would alleviate the necessity for C.P. Rail to continue placing rip-rap in this zone. The requirement for erosion protection in other areas should be assessed in the future based on the results of the recommended continued monitoring program. A conceptual design for erosion protection of the river bank is presented in the NHC report (Appendix C).

The cost of carrying out all of the remedial measures required to reduce the risk of slope failure under anticipated future conditions may be prohibitively high. If limited funds are available, we recommend that priority be given to control of ground water pressures initially. While erosion protection, in conjunction with ground water control, is considered necessary for comprehensive control of frontal slope failures and will likely be required in the long term, it is our opinion that erosion protection will not by itself prevent slides occurring; nor will it offer any protection against retrogression of frontal slides.

4.0 MONITORING

The ground water pressures and slope geometry are critical to the stability of the slopes. We recommend that monitoring of the piezometers and surveying of the river bed be carried out at regular intervals in the future.

Provision should be made for regular review of piezometer and river survey data so that the stability of the slopes can be assessed and remedial measures undertaken, if required. It is suggested that such review be undertaken in June/July, immediately after the Fraser River freshet, and also in January/February when the highest water levels are expected.

4.1 Piezometric Levels

The piezometers should be monitored at regular intervals throughout the year. Since we are more concerned about the maximum piezometric levels, we suggest that readings be taken twice monthly during January and February. Readings could be taken less frequently at other times during the year, say once every 1 to 2 months. It would also be useful if the piezometers were monitored shortly after any extended periods of high precipitation, particularly if this occurs during the winter months.

Due to the layered stratigraphy and the combined effects of surficial and deeper water pressures, it is not possible to define a single critical water level which could be used as a warning against slope failure. We can, however, make the following general comments regarding interpretation of piezometric levels with respect to deep-seated slides.

(1) The slopes are considered to have an adequate safety factor with respect to deep-seated failure under the present ground water conditions. Water level increases of about 2 m above January 1983 levels are not considered to be critical.

(2) The stability of the slopes would be marginal if water levels in the upper 25 m rose close to ground surface, while those at depth rose higher than about 5 m above their present levels.

(3) The slopes would be unstable if overall ground water levels were at or close to ground surface.

In view of the complexity of the problem, it is suggested that provision be made for immediate review of the stability by a geotechnical engineer if water levels in any of the piezometers show significant increases (several metres) above present levels, particularly where a continuing upward trend has been observed. The concern would be greatest where the intermediate and deep piezometers are involved.

4.2 River Bank Erosion

As recommended in Section 8.0 of Northwest Hydraulic Consultants Ltd.'s report, the river bank cross-sections should be re-surveyed during the next peak flows. Surveys should be carried out in the future at regular intervals (every 2 to 3 years) unless bank protection measures are undertaken. Additional surveys should be carried out immediately after any significant Fraser River flood.

We trust this report provides the information you require at present. If you have any questions concerning the above, please do not hesitate to contact us.

Yours very truly,
GOLDER ASSOCIATES

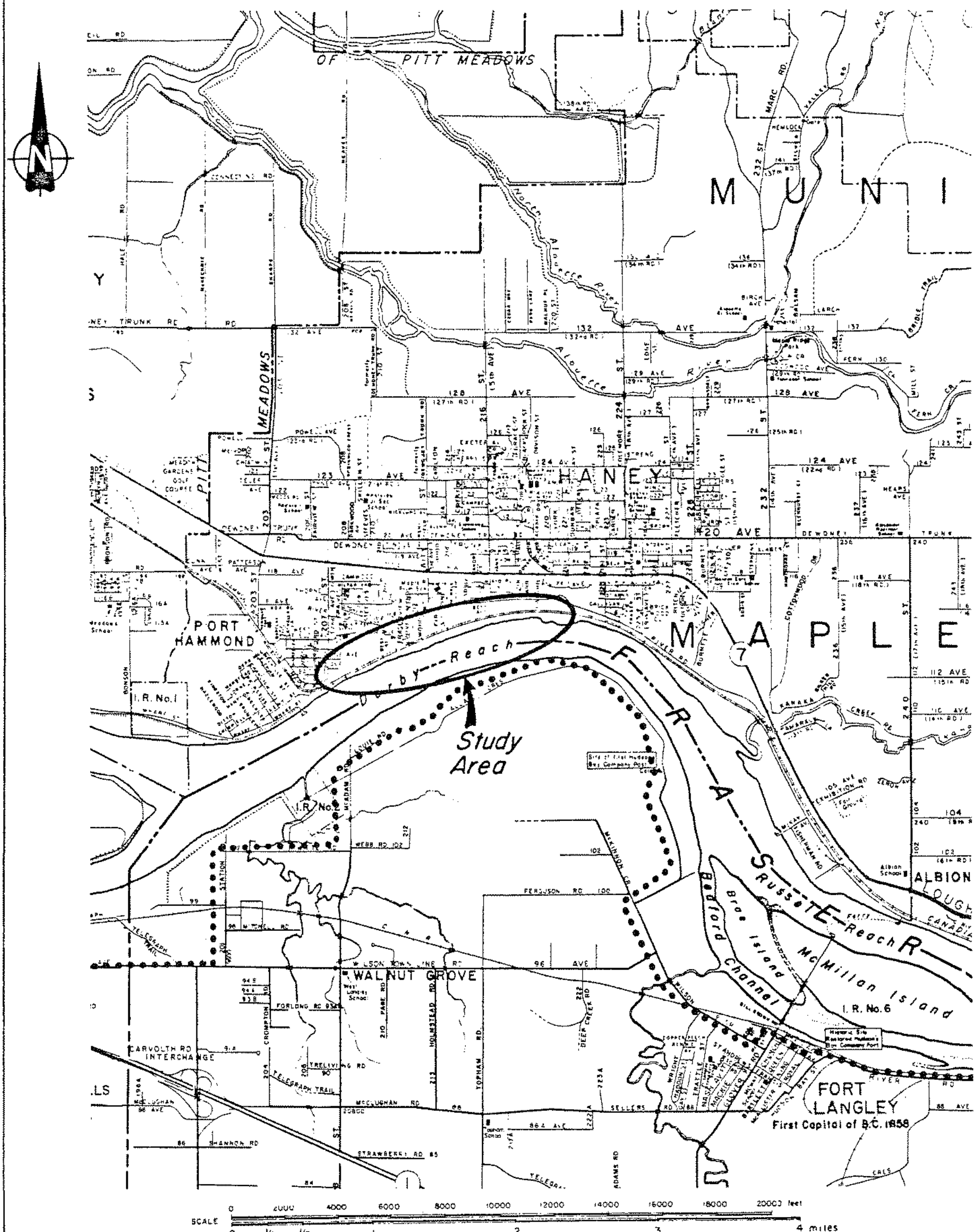
R.M. Wilson, P. Eng.

T.P. Fitzell, P. Eng.

RMW/TPF/sek
822-1071

KEY PLAN

Figure 1



Project No. 822-1071 Drawn SF Reviewed TX Date Mar '83

Golder Associates


SITE PLAN
Figure 2
TEST WELL
101
103
104
105
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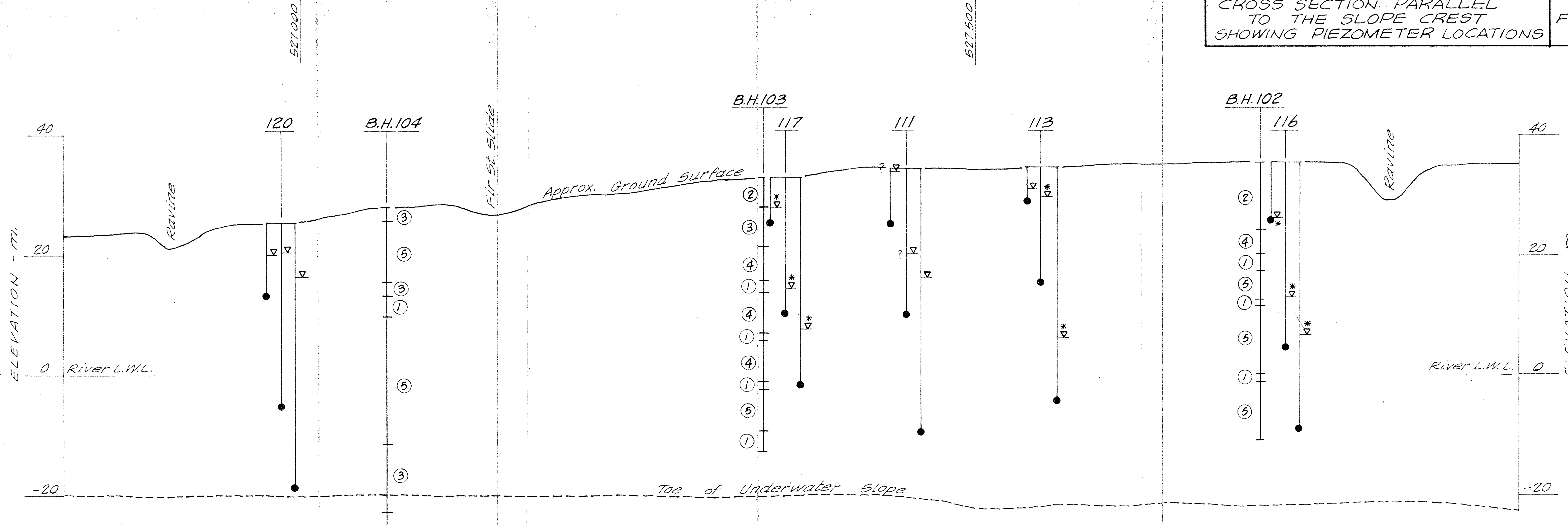
115
116
117
118
119
120
201
202
203
204
205
A
XS-1
XS-2
XS-3
XS-4
XS-5
XS-8
XS-9
XS-10
XS-11
XS-12
XS-13
XS-14
XS-15
XS-16
XS-17
XS-18
XS-19
XS-20
XS-21
XS-22
XS-23
XS-24
XS-25
XS-26
XS-27
FRASER RIVER
MON. NO. 2
MON. NO. 1
WRS MON. 81HAW002
WRS MON. 894
5451 000
5450 500
526 500
527 000
527 500
528 000
528 500
529 000
529 500
LEGEND:
Piezometer installations -
No's. 101, 103-105 - Installed 1978
111 to 120 } Installed 1982
201 to 205 }
100m 0 100 200 300 400 500m
SCALE 1:5000
REF. DWG.: Prov. of B.C.-Water Management Branch
Haney Erosion Mon. Program - Dwg. A5323-1-Jan./82.
Drawn Ing.
Reviewed T.Z.
Date Mar. '83

CROSS SECTION PARALLEL TO THE SLOPE CREST SHOWING PIEZOMETER LOCATIONS

Figure 3



INFERRED STRATIGRAPHY: (From G.A. Report 782-1179, Fig. 3, Aug. '79)

1. Loose to dense silty fine to medium SAND.
2. Loose to dense silty fine SAND with layers of silt and silty clay.
3. Interlayered firm to stiff silty CLAY and loose to dense silty fine SAND.
4. Firm to stiff silty CLAY with layers of silty fine sand.
5. Firm to stiff silty CLAY; occ. thin partings of silt and silty fine sand.

LEGEND:

- B.H.103 — Location of Borehole (1979 program)
- 113 — Location of Piezometer Hole (1982 program)
- ● — Location of Piezometer
- ▽ — Recorded Piezometric water levels Jan. 1983
- * — Indicates piezometers tested with good response.

For location of section see fig. 2

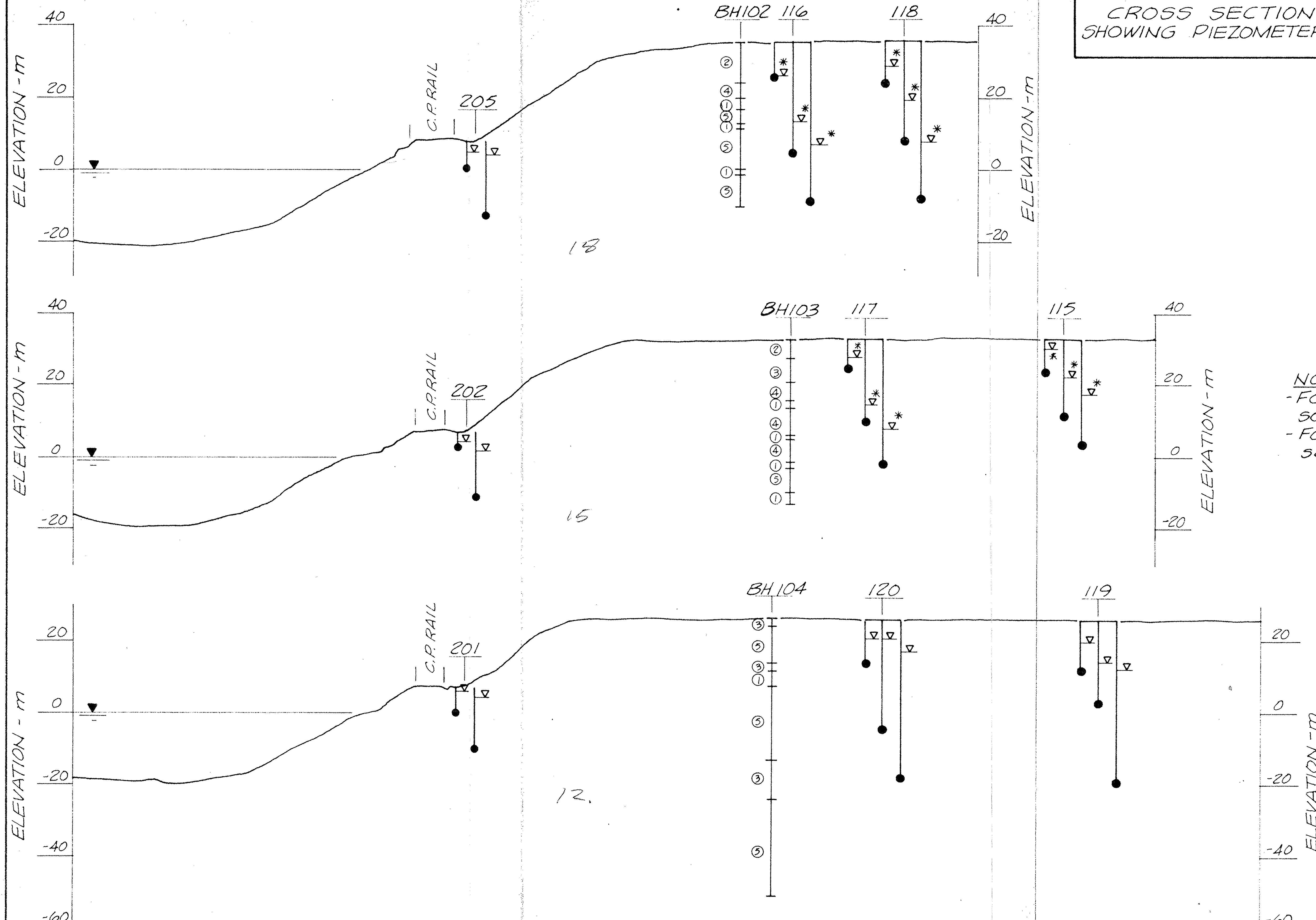
SCALE 1:2500 HOR. 1:500 VERT.

Golder Associates

Drawn Ing
Reviewed TF
Date Mar '83

CROSS SECTIONS 12, 15 & 18 SHOWING PIEZOMETER INSTALLATIONS

Figure 4



NOTE
- For legend and inferred soil stratigraphy see fig. 3.
- For locations of sections see fig. 2.

Scales Hor. & Vert. 1:1000

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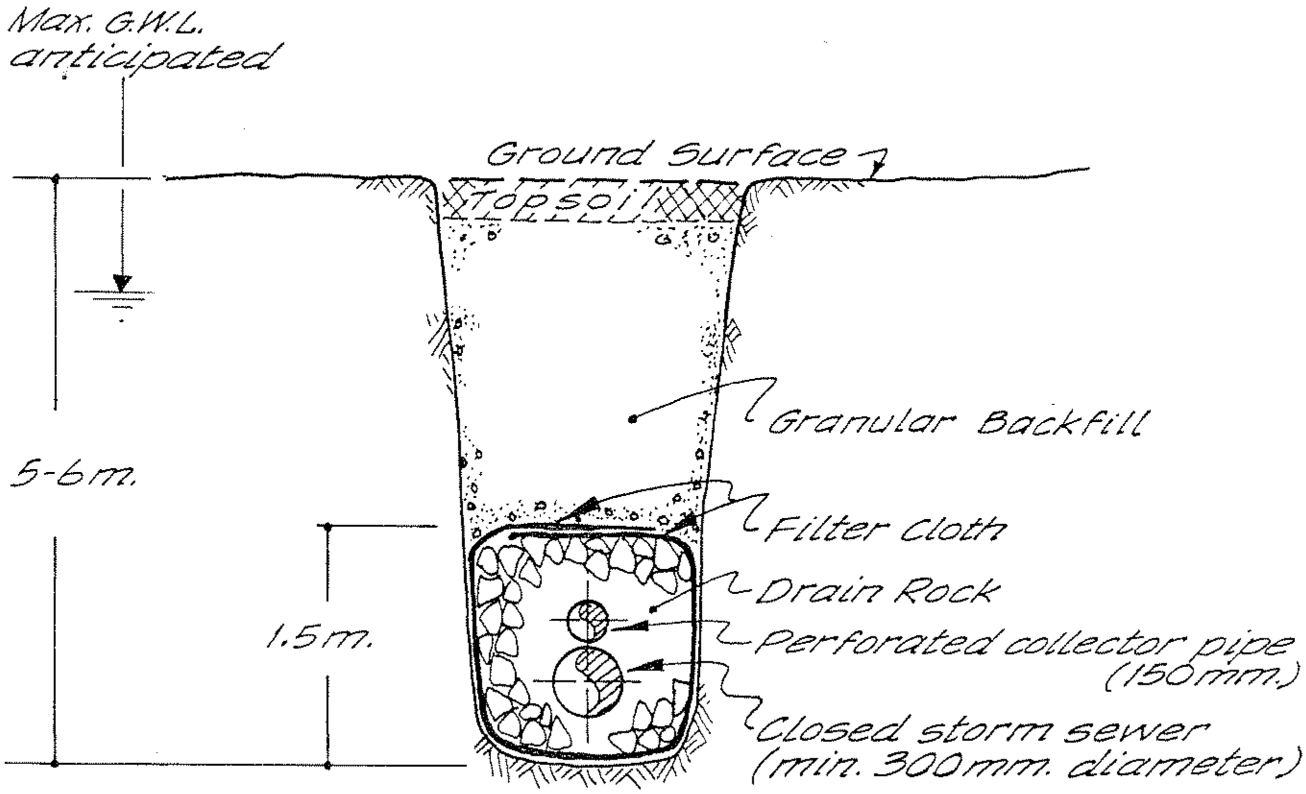
822-1071

SKETCH ILLUSTRATING PRELIMINARY INTERCEPTOR DRAIN DESIGN.

Figure 5



Schematic only - Not to Scale

PROJECT NO. 822-1071 DRAWN ing. REVIEWED TJ DATE Mar. '83

APPENDIX A

NORTHWEST HYDRAULIC CONSULTANTS LTD.

REPORT ON

RIVER/HYDRAULIC ASPECTS OF

FRASER RIVER BANK STABILIZATION

HANEY, BRITISH COLUMBIA

RIVER/HYDRAULIC ASPECTS OF
FRASER RIVER BANK STABILIZATION
HANEY, BRITISH COLUMBIA

for

BRITISH COLUMBIA
MINISTRY OF ENVIRONMENT

through

GOLDER ASSOCIATES
VANCOUVER, BRITISH COLUMBIA

by

NORTHWEST HYDRAULIC CONSULTANTS LTD.
NORTH VANCOUVER, BRITISH COLUMBIA

March 1983

TABLE OF CONTENTS

1.0 INTRODUCTION

Golder Associates have been retained by the B.C. Ministry of the Environment to undertake a slope stabilization study of the north bank of the Fraser River at Haney. Northwest Hydraulic Consultants (NHC) were retained by Golder to investigate the river/hydraulic aspects of this study. The terms of reference for NHC's work are described in our letter of 25 January 1982 to Golder. Briefly, the scope of this work has included:

- comparison of 1978 and 1981 cross section surveys
- site visit during low water
- review of C.P. Rail's recent maintenance work
- summary of observations and conclusions: 1977-83
- review of alternative methods to stabilize the river bank
- conceptual design and rough cost estimate for bank protection at the site
- recommendations for further work

This investigation concentrated on a 3000 metre river reach downstream of the upstream end of the 1880 slide (about 224th street).

2.0 CROSS SECTION SURVEYS: 1978 and 1981

The B.C. Ministry of Environment, Water Management Branch (The Ministry), has surveyed 29 river cross sections in both 1978 and 1981. Their locations are shown on Ministry Drawing Nos. A5323-1 & 2. A portion of these are reproduced on the attached Figure 1. The 1978 survey was done during April when the mean monthly discharge was 2350 m^3/s at Mission; by contrast, the 1981 survey was done during the period 6-30 July when the estimated discharge at Mission ranged from 5800 to 4800 m^3/s.* The cross sections were taken at identical locations in both years and plotted by the Ministry to similar scales, being 1:200 undistorted for all partial cross sections (i.e. those taken basically in the north portion of the channel) and 1:1000 horizontal and 1:500 vertical for complete cross sections. The Ministry's presentation of these plots was most convenient and allowed comparisons to be readily made. In addition to the plots the Ministry also provided, for each full cross section, comparable hydraulic elements including flow area, wetted perimeter, hydraulic radius and (for all cross sections) thalweg or minimum river bed level.

NHC has reviewed and compared these two surveys on a section by section basis. Based on this review, the following general conclusions were reached:

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* As reported by Water Survey of Canada, Station No. 08MH024

(1) The surveys were taken at substantially different river discharges, with the 1981 flow being about twice the 1978 flow. Generally speaking, the higher the flow, the more tendency there is to transport of bed material load. Accordingly, the river bed and lower banks during the 1981 survey were likely quite active compared to that in the 1978 survey. Since material removed from the upper bank during a flood would not likely be replaced at lower flows, comparisons of upper bank movement can be directly made between the two surveys. However, comparisons of the lower bank and bed must be made with caution, since regions that might be scouring in higher flow might also fill in once the flow recedes.

(2) For the relatively small Fraser floods of 1978 through 1981, riprap placed by C.P. Rail seems to have prevented bank erosion in those areas. However, the 1981 survey does show a few local pockets of considerable erosion (up to 10 metres horizontal movement) at the toe area of the north river bank - near the channel thalweg between Ministry cross sections 8 and 14. Figure 1 summarizes the approximate location of both noted erosion and deposition. This latter term refers to both riprap additions by C.P. Rail and to natural siltation. Erosion as noted, if left unchecked, would lead to future riprap slumping and thus more bank erosion.

(3) Based on observed cross sections, riprap appears to have been placed all the way down to the thalweg in some locations. At these places, the riprap has to date successfully arrested erosion.

(4) Although cross section nos. 6 and 7 were not taken in 1981 at the 1880 slide toe, sections immediately upstream and downstream showed evidence of continuing erosion on the underwater slopes.

3.0 SITE INSPECTION

The site was inspected on 16 March 1983 by M.H. Okun, P.Eng. Since the study period did not coincide with the Fraser peak flood period, it was not possible to inspect the site during the 1982 flood.

Generally speaking, it was observed that between the March 1983 inspection and previous inspections of 13 January 1979 and 29 November 1977, there was little evidence of significant erosion above the water line in the study reach. Selected site specific examples of the comparative observations are provided in the attached photos 1 through 10.

Other observations were as follows:

- the extent of both log booms and corresponding dolphins was similar to that on previous visits
- although there was some evidence of occasional slumping of older (pre 1978) riprap near the winter water line, almost all of the visible riprap placed within the last 5-6 years had not been noticeably disturbed (see for example photos 7 and 8)
- downstream of about cross section 20, there is a gently sloping, natural beach area located at the toe

of the relatively steeper slope between the beach and the railway grade; some minor erosion and/or slumping of bank material was evident at the toe of this steeper slope (Photos 11-13).

- while the quality of the rock used in C.P. Rail's riprap is excellent, the gradation of the placed rock is inconsistent. There are alternate areas or pockets of either consistently large or consistently small rock, as seen in Photos 7 or 8.
- since the 1979 visit, there was no visible evidence of noticeable amounts of new rock placed in the study reach; however, C.P. Rail does report some small riprap placement in two small areas (see next section).

4.0 C.P. RAIL MAINTENANCE

C.P. Rail has reported to NHC the following placement of riprap within the study area:

Year	C.P. Milepost	Approx Cross Section Locations	Volume of Rock Placed (cu. yds)	Comments
1978	103.8	CS 15	13,000	8,000 yd^3 below low water 5,000 yd^3 above low water
1978	103.9	CS 16-17	2,000	
1978	104.1	CS 19	2,200	
1980	103.6	CS 12	1,500	
1980	104.5	CS 25-26	600	

The in-place cost for 1978 rock was approximately \$12.50/yd^3; in 1980, the approximate cost was \$17.00/yd^3. 1978 riprap was placed by Fraser River Pile from a barge using rock quarried from Dillingham's Pitt River quarry. Nothing has been placed since 1980.

C.P. Rail does not keep detailed records of riprap placement and condition. Generally, they protect their railroad from severe river erosion by (a) spending money according to local budget and other priorities, and (b) placing riprap after they observe problems.

5.0 SUMMARY OF OBSERVATIONS AND CONCLUSIONS: 1977-83

The general river regime, evidence of historical erosion, causes of erosion, and previous C.P. Rail maintenance have all been discussed in two previous NHC reports.* For the reach of river between Ministry cross section 5 and 29, the following summarizes both previous and current observations and conclusions:

(1) It is typical that the outside bank of a bend in a river - such as the reach in question - is subject to continuous erosive forces. The success of these forces in eroding bank material is stronger during peak flows, and is affected by natural bank material, flow obstructions, degree of curvature and man-made material (e.g. riprap, piles, etc).

(2) The surveyed cross sections in this area are generally triangular with the thalweg located close to the outside (north) bank; this is consistent with what is normally found in such eroding bends, and verifies the natural tendency for erosion of the north bank at Haney. Other than riprap, there is no evidence in the surveyed cross sections to suggest the presence of any natural bank material that might arrest the natural erosion forces.

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*(1) "Haney by-Pass, Fraser River Regime Study", letter from NHC to Golder Associates, 15 December 1977
(2) "Fraser River-Haney to Port Hammond, Bank Erosion", letter from NHC to Golder Associates, 23 March 1979

(3) Based on the curvature and plan form of the bend, it is expected that the reach with the highest erosion potential is between cross sections 5 and 19; between the 1978 and 1981 surveys, erosion was observed between Ministry cross sections 6 and 15. There is no reason to expect a decrease in either the potential or observed erosion in the foreseeable future.

(4) Above the winter water line, the bank has remained relatively stable for over 50 years; this is most likely due to the presence of riprap and timber piles. Below the winter water line, some areas are showing evidence of erosion - especially between the 1880 slide (Ministry cross sections 5-8) and cross section no. 14. Underwater areas that did not show surveyed erosion are most likely benefitting from the presence of riprap.

(5) The placement of riprap has reduced the natural erosion that would otherwise have taken place. The fact that C.P. Rail has continued to place riprap for many years is evidence of the erosion potential of this bank.

For protection of the railway, C.P. Rail's maintenance program has proved adequate. However, if the integrity of the entire right bank (from high water to the thalweg) is critical to slope stability above the bank, an improved protection program is required. In other words, if continued erosion of the right bank - and especially the toe - could trigger another "1880 slide", improved protection is required. The significance of erosion and of protecting against erosion must be determined by assessing all factors related to stability of the slope in question.

Since improved protection, if implemented, will be costly, it is relevant to try and assess from a river viewpoint the urgency of requiring such protection. Basically, the removal of river bank material can be expected to occur sometime during the spring/summer high flow period. Due to low flows between 1978 and 1981, not much erosion occurred. However, during a single large flood (say 25 year return period or more), it is judged that without protection up to 15-20 metres of bank material could be removed, most likely on the lower portion of the bank, and most likely between Ministry cross sections 5 and 19.

6.0 BANK PROTECTION ALTERNATIVES

6.1 General

If bank protection is required, the slope length of bank to be protected generally extends from just below railroad grade to the channel thalweg, a distance of 60-70 metres. Over 3/4 of this length is in flowing water. This is a relatively large area of bank for each lineal metre; accordingly both the technically and economically feasible alternatives for bank protection are quite restricted.

Basically, there are three concepts that could theoretically be applied to arrest potential north bank erosion. These are:

(1) Continuous bank protection: such as riprap. This would cover the entire slope length of bank. If properly constructed, it would be effective. It would most likely, however, require placement of a filter to prevent leaching of the bank fines.

(2) Intermittant Protection - Groynes: such as regularly spaced rock or timber structures tied into the bank and projecting into the flow. These could also be effective but would not only likely have a negative

impact on the log booming industry, but would essentially reduce the channel flow width, thus increasing velocities and transferring the erosion potential downstream or to the opposite bank.

(3) Training Walls: such as closely-spaced timber or other piles driven in a smooth line roughly paralleling the bank at a point about 1/4 of the slope distance north of the thalweg. This option has the same problems as the groyne concept.

Because of the likelihood of transferring the problem elsewhere, the scope of this investigation concentrated on the continuous bank protection alternative.

6.2 Continuous Bank Protection

Considering that much of the construction would have to be done in relatively deep, flowing water, the only alternatives considered were those that have a proven track record under these conditions. These include riprap and articulated blocks.

Articulated blocks consist of small prefabricated concrete blocks that are threaded together with steel cables or rods to form a flexible mat. They can be "rolled" from a barge onto the channel slope in moving water. However, they require a well prepared smooth bank and the use of either a filter fabric or suitable gravel filter to prevent fine bank material from leaching out.

This approach has so far only been economically competitive with riprap in situations where many kilometres of protection are needed - such as on the Mississippi River in the U.S.A. In 1983 dollars the cost of installing articulated blocks at the Haney site would be many times that of riprap. Additionally, due to the proximity of the railroad, there would be a construction access problem for bank preparation.

Riprap at this site has proven to be an effective technique in protecting the railway embankment. With improved design and construction, and with planned maintenance, it could be used effectively to prevent further erosion of the north bank.

7.0 RECOMMENDED PLAN

7.1 Conceptual Design

If it is required, NHC's recommended protection involves the placement of a Class II riprap over a filter* - both placed on the reworked (for smoothness) natural slope below water and on a prepared 2:1 slope above water. Class II riprap is described as follows:

Nominal 20 inch (50 centimetre) diameter or 400 lb (180 kg) weight; local maximum velocity up to 13 ft/second (4 metres/second) Grading Specification.

	100%	smaller than	30 inches (75 cm)	or	1500 lb (680 kg)
at least	20%	larger than	24 inches (60 cm)	or	700 lb (320 kg)
at least	50%	larger than	20 inches (50 cm)	or	400 lb (180 kg)
at least	80%	larger than	12 inches (30 cm)	or	70 lb (32 kg)

Figure 2 illustrates a typical cross section through the recommended protection, using the Ministry's cross section no. 19 as an example.

The Class II rock is available by barge or truck at a nearby quarry next to Pitt River. The installed cost of riprap is estimated to be $20 per cubic yard ($10 per metric ton). Allowing for a minimum riprap thickness of 2.5 feet (0.75 metre) and a 50% increase in required volume (due to possible

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* Both the need for and design of a filter would have to be determined at the detailed design stage; for budgeting purposes, it has been assumed herein that a filter would be needed.

loss of material) for placement in flowing water, the typical volume of rock required is 26.5 yd^3/foot or 67 m^3/metre. Therefore the unit cost of riprap for a section as shown on Figure 2 is roughly $530 per lineal foot ($1740 per lineal metre).

For preliminary budgeting purposes, we have assumed the use of a filter fabric to protect against leaching of bank fines through the riprap. This fabric would be hand placed above water and likely rolled from a barge for underwater placement. Its cost is estimated at $1.75 per square metre delivered at site; the cost of placing the material must be added to this (local experience figures were not found). For a typical section, such as shown on Figure 2, the delivered cost amounts to about $115 per lineal metre ($35 per lineal foot) <u>plus placement</u>.

Thus the total estimated cost per lineal foot for both riprap and filter is about $600 per lineal foot or say $2,000 per lineal metre; this estimate has allowed for an assumed doubling of the filter fabric cost to cover installation.

7.2 Existing Riprap

C.P. Rail already has some riprap placed in this region, and it is expected they would place more rock in the future. However, as previously noted, the placement of this rock is not satisfactory, and there is no evidence of an underlying filter. Furthermore, the extent and nature of the existing riprap is unclear.

Before proceeding with detailed design of bank protection, it is recommended that the Ministry establish by survey the extent and condition of existing protection, and that they discuss with C.P. Rail what their future maintenance plans might be. Any bank protection work done by the Ministry will provide future protection for C.P. Rail.

7.3 Maintenance

Any bank protection will deteriorate with time. Also, it is always difficult to know exactly what the as-built protection looks like in the deeper, flowing water. Therefore, riprap should be inspected annually to look for slumping or any other changes. Additionally, after each significant Fraser flood (say greater than a 10 year event), the river should be sounded and compared to as-built or existing conditions.

8.0 FURTHER WORK

There currently are enough hydraulic data to conservatively design the recommended protection works. However, to allow a more optimal sizing of riprap, it would be useful to have a velocity distribution at 2 or 3 cross sections (this was recommended in our 1979 report) during the peak flow period. Also, some construction savings could be achieved by:

- ascertaining the grain size distribution of material on the bank and beneath the water level - especially at lower levels; this would be required to properly assess the need for a filter, and to prepare a filter design.
- reviewing plans for future log booming in the area; the presence of booms and dolphins will hamper construction of the bank protection.

With regard to monitoring of bank erosion it has been pointed out that the two available cross section surveys were taken at substantially different river flows. The sections should be re-surveyed during the peak flow period of 1983. Velocities should also be measured at that time. This re-survey could be restricted to cross sections 5 through 26. Results of such a survey will provide a more meaningful comparison with the 1981 survey. Also, the peak, Fraser River flow in 1982 was considerably higher than it had been since 1977, so that comparison of results between 1978 and 1981 has unfortunately been limited to a period of very low peak flows.

If bank protection is required, the Ministry might also give consideration to the use of hydraulic model studies to:

(1) More precisely determine the volume and method of placement for rock needed to effect a 0.75 metre thick cover placed in flowing water from a barge; and

(2) More precisely determine the lineal extent of riprap protection required.

It is expected that the savings in construction costs resulting from model tests would be several times the cost of the model studies. If this suggestion is pursued, field measurements of velocity distribution during the anticipated construction period would be required.

PHOTOGRAPHS

PHOTO 1 29 November 1977

Comparative view upstream from downstream end of 1880 slide - from approx. Ministry cross section no. 8. Note the position of trees at centre background is the same in both photos.

PHOTO 2 16 March 1983

16 March 1983

PHOTO 3 29 November 1977

PHOTO 4

Comparative views upstream taken from about Ministry cross section no. 8. Almost no erosion has taken place above the water line.

PHOTO 5 13 January 1979

16 March 1983

PHOTO 6

View upstream near Ministry cross section no. 12. Position of piles relative to bank is similar in each photo.

16 March 1983

PHOTO 7

13 January 1979

PHOTO 8

Comparative view upstream from about Ministry cross section No. 14
Note that most of the riprap is in the same position in both photos.

16 March 1983

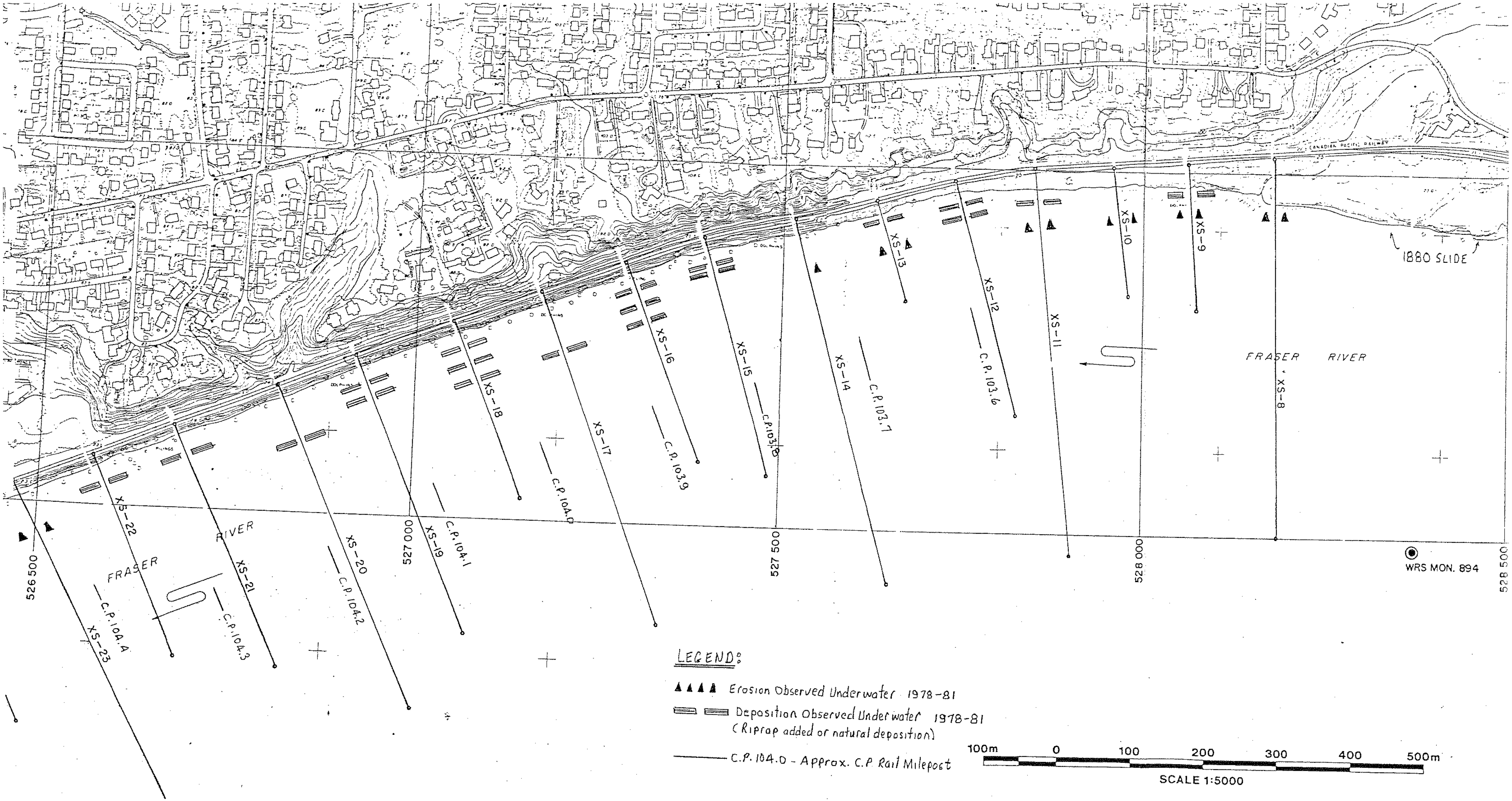
PHOTO 9 13 January 1979 PHOTO 10

View downstream from approximately Ministry cross section no. 19. Cut off timber piles at centre of Photo 9 (in ice) are the same ones seen in centre foreground of Photo 10. Again there is little evidence of erosion above the water line.

PHOTOS 11-13 16 March 1983

View upstream at approximately Ministry cross section no. 23. Above the water line, there is some minor erosion/slumping of the toe on the steep bank at left. Note the beach area typical of this downstream reach.

FIGURES



MAP COPIED FROM MINISTRY DWG. A5323-2

FIGURE 1 PLAN OF CROSS SECTIONS & EROSION/DEPOSITION AREAS

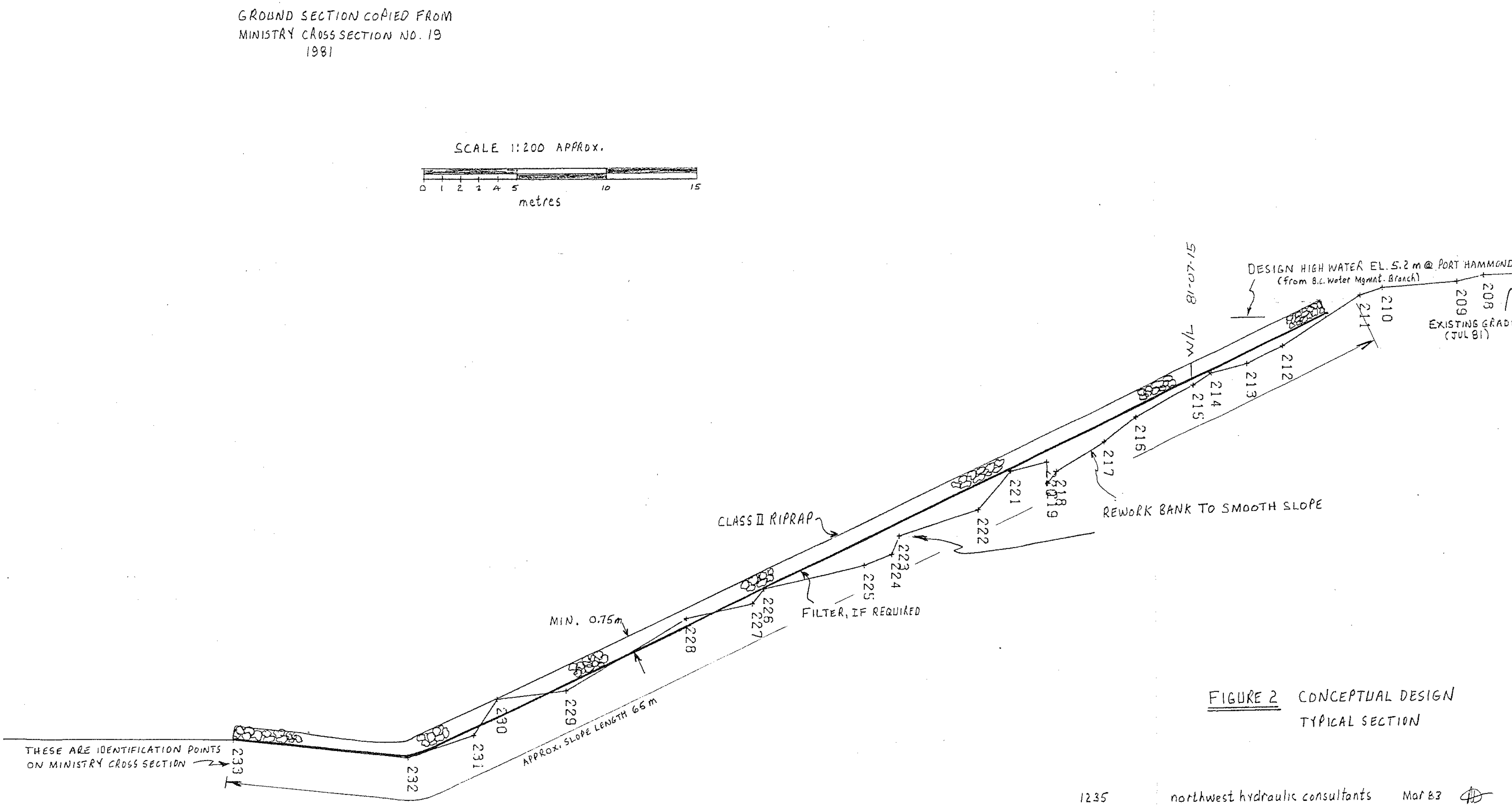


FIGURE 2 CONCEPTUAL DESIGN
TYPICAL SECTION

APPENDIX B

Piezometer Installation and Monitoring

TABLE OF CONTENTS FOR APPENDIX B

PIEZOMETER INSTALLATION AND MONITORING

1.0 PIEZOMETER INSTALLATION AND TESTING

1.1 Installation Procedure

Piezometers were installed between September 2nd and December 15th, 1982, at fifteen locations within the most critical areas identified in the previous stability analyses (see Figure 2). Many of the piezometers were installed on private property where access is limited, resulting in some problems and delays. Three piezometers were installed to depths of up to 46 m at each of the ten locations in the area above the slopes (Piezometers 111 to 120). Two piezometers were installed to depths of up to 22 m at each of the five locations on the C.P. Rail bench (Piezometers 201 to 205). The field work was carried out under the full-time supervision of a member of our staff.

Details on the piezometer installations are presented on Figures B-1 to B-4 (Appendix B).

A light truck-mounted rotary drill rig was used to install the piezometers in the area above the slopes. A bombardier-mounted rotary drill rig was used to install those piezometers on the C.P. Rail bench. The boreholes, which had a nominal diameter of 120 to 160 mm, were generally drilled using open hole techniques and Revert (a biodegradable drilling fluid) where necessary. Where possible, the piezometers were completed within sandy zones at approximate pre-selected depths. The soils encountered in the boreholes were inferred from the drilling conditions. Standard 51 mm OD split spoon samples were taken when suspected sand layers were encountered close to the proposed piezometer depths. These sample locations are shown on Figures B-1 to B-4, along with the inferred sandy layers or zones. The results of gradation tests carried out on selected samples of sand are presented on Figures B-5 and B-5A.

The piezometers consist of 19 mm diameter plastic standpipes with a 25 mm diameter filtered slotted tip. No. 8 sand was used as backfill around the piezometer tips and in the zones between the seals. The seals, which are at least 1 m thick in most cases, were formed using bentonite pellets. The standpipes in the area of River Road were cut off just below ground surface and protected with a locking cover. Those on the C.P. Rail bench protrude about 0.6 m above ground surface.

The piezometer locations have not been accurately surveyed. The approximate locations of the instruments in plan (as shown on Figure 2) were determined relative to existing structures and topographic features shown on the topographic plan. The approximate ground surface elevation (relative to Geodetic Datum) at the piezometer locations was estimated from the available topographic plan (reference Water Management Branch, Drawing No. A5323-1, January, 1982).

1.2 Testing of Piezometers

Falling head tests were carried out in some of the piezometers to identify stabilized piezometric levels and provide estimates of formation hydraulic conductivity. The falling head tests involved the introduction of water into the standpipes, and monitoring the subsequent decay of the water level within the standpipe until approximately 80 per cent of the excess head had dissipated. The data were analyzed according to the Hvorslev (1951) method. Table B-III summarizes the results of the testing.

The calculated hydraulic conductivities are generally in the range of 10^{-6} to 10^{-8} m/sec within the more permeable zones. As indicated on Table B-III, certain piezometers showed very little change following the introduction of water. These piezometers appear to be completed within more silty or clayey strata, with estimated hydraulic conductivities of less than 1×10^{-10} m/sec.

The falling head tests confirm that most of the piezometers have been successfully located within the more permeable strata, and that they should respond well to fluctuations of the piezometric pressure.

2.0 GROUND WATER MONITORING PROGRAM AND RESULTS

The piezometers are being monitored by District of Maple Ridge personnel, with occasional assistance from Golder Associates staff. The data obtained during the ground water monitoring program to date is presented in Appendix B. A sumary of the piezometer data (to February, 1983) is presented on Table B-1, and a plot of piezometric pressure head versus elevation is shown on Figure B-6.

2.1 1978 Piezometer Installations

The piezometers installed in boreholes put down during the 1978/79 stability study were monitored between December 1978 and March 1979, and again between November 1981 and the present time. Water levels were recorded at about weekly intervals during 1982, and selected monthly readings are tabulated on the Piezometer Data sheets.

The available 1978 piezometer data indicates the variation of piezometric pressure with depth is almost hydrostatic (see Figure B-6). The shallowest of these piezometers, which are located 20 to 25 m below ground surface, indicate January 1983 water levels at depths of 3 to 10 m below the ground surface (or between approximate elevation 32 m and 15 m).

The recorded ground water levels in the 1978 piezometers show fluctuations in the range of 0.3 to 1.6 m during the period between January 1982 and January 1983. The 1982 data indicates that ground water levels are at a minimum in October/November, and reach a maximum in January/February. Ground water level fluctuations of up to 7.5 m were recorded between January 1979 and 1983, but the 1979 data is questionable since the piezometric levels may not have stabilized by that time. There is some evidence to suggest that water levels are generally lower now than they were in 1979, but this may also be due to the slow stabilization of piezometric levels.

2.2 1982 Piezometer Installations

The recently installed piezometers have been monitored regularly since December, 1982. Falling head permeability tests have been carried out in many of these instruments to obtain values of hydraulic conductivity and to identify stabilized piezometric levels (see Section 2.2). The majority of the 1982 piezometers appear to have stabilized and to be responding well to ground water fluctuations. The following discussion is weighted toward the data obtained from the 1982 piezometer installations.

The recent data indicates that the piezometric pressure profile is significantly lower than the hydrostatic pressure profile, except in those areas below the CP Rail grade. This is clearly illustrated on Figure B-6. The recorded water pressures in piezometers 201 to 205, which are located on the C.P. Rail bench, are only slightly below the hydrostatic pressure. This is a result of dissipation along near horizontal stratigraphy toward the river.

In the area above the CP Rail bench, the data indicates that water levels in the upper 10 m are in the range of 2 to 9 m below ground surface (approximately elevation 32 to 25 m) and that significant fluctuations occur with precipitation. Measured fluctuations between December 1982 and January 1983 are generally in the range of 0.4 to 1.4 m, with some higher fluctuations probably due to surficial flooding.

Water levels below the toe of the slope at the C.P. Rail bench are generally 2 to 4 m below ground surface, and are probably controlled by the drainage measures installed by C.P. Rail. Measured fluctuations between December, 1982, and January, 1983, are in the range of 0.1 to 0.4 m.

The expected dissipation in water pressure toward the river is illustrated on Figure B-6 where it can be seen that, for a given elevation, water pressures in Piezometer 112 (close to the crest of the slope) are lower than at other installations in the area above the slopes.

2.3 Comparison of 1978 and 1982 Piezometers

The recent piezometers indicate significantly lower piezometric pressures exist at depth than is indicated by the 1978 piezometers. The 1982 piezometers have only been monitored for a relatively short period. However, the falling head test results indicate that most of them are responding to piezometric pressure changes, and that they have stabilized. On the other hand, falling head tests carried out in 1978 Piezometer 103, which is very close to the 1982 Piezometer 117, indicate that the deep piezometer (103A) is plugged. Sounding of the 1978 piezometers in 1982 indicates that many of these piezometers are blocked well above the recorded tip depth (see Piezometer Data Sheets, Appendix B).

The 1978 piezometers were installed in boreholes, drilled using bentonite mud, put down primarily to define and obtain samples of the underlying stratigraphy. The presence of bentonite in the borehole would lower the permeability of the soil strata, and would affect the performance of the piezometers installed in boreholes drilled using this technique.

The water levels in the piezometers installed in 1982 were close to ground surface at the time of installation. Since that time, water levels within the standpipes have dropped and, presently, water levels vary between 2 to 30 m below ground surface with the deepest piezometers recording the lowest water levels. Falling head tests carried out in these piezometers indicate that many of the piezometers are responding to piezometric pressure changes, and that these low readings recorded at depth are in fact stabilized piezometric levels. Based on this testing, those piezometers completed in zones with hydraulic conductivies greater than $1x10^{-10}$ m/sec are providing accurate piezometric levels. Piezometers completed in zones with hydraulic conductivities less than $1x10^{-10}$ m/sec show a very slow response to pressure changes due to the phenomenon of time lag, and readings from these piezometers should be reviewed as more data becomes available. It may be possible to measure the piezometric pressure within the lower permeability zones by the installation of a pneumatic piezometer tip within the existing standpipes. Such piezometers would respond quicker to pressure changes, but have the disadvantage that a special device is required for monitoring.

TABLE B-I

SUMMARY OF DATA FROM 1978 PIEZOMETERS

PIEZOMETER NUMBER	APPROXIMATE TIP ELEVATION (m)	APPROXIMATE ELEVATION OF WATER IN PIEZOMETER JANUARY 1983 (m)	RANGE OF WATER LEVELS JAN. 1982 TO JAN. 1983 (m)	RANGE OF WATER LEVELS JAN. 1979 TO JAN. 1983 (m)	REMARKS
101 A	+17.0	31.5	0.80	3.57*	*Flooded
B	-5.0	27.8	0.80	7.16*	
C	-48.2	21.0	0.60	3.23	
103 A	-13.0	29.3	0.42	5.29	Tested, Very Slow Response
B	+5.0	25.2	1.55	7.59	Tested, Good Response
104 A	-12.8	16.5	0.28	4.15	
B	+10.5	17.4	0.36	3.44	
105 A	+1.5	15.2	0.55	3.55	
106 A	+8.8	14.8	0.46	0.54	
B	-4.0	12.6	0.36	0.57	

TABLE B-II

SUMMARY OF DATA FROM 1982 PIEZOMETERS

PIEZOMETER NUMBER	APPROXIMATE TIP ELEVATION (m)	APPROXIMATE ELEVATION OF WATER IN PIEZOMETER JANUARY 1983 (m)	RANGE OF WATER LEVELS DECEMBER 1982 TO FEBRUARY 1983 (m)	REMARKS
				*Flooded
111 1	+24.2	33.2*	3.40	Tested, Slow Response
2	+9.0	19.3	1.24	Tested, Slow Response
3	-11.0	15.5	0.33	Tested, Good Response
112 1	+20.1	21.0	0.71	
2	+9.1	13.1	0.22	
3	-12.8	4.4	0.21	
113 1	+23.2	30.9	3.54	Tested, Slow Response
2	+9.3	29.6	2.02	Tested, Good Response
3	-10.5	6.3 (Feb)	-	Tested, Good Response
114 1	+25.4	31.9	0.86	Tested, Good Response
2	+8.8	18.6	0.55	Tested, Good Response
3	+0.3	5.0	0.40	Tested, Good Response
115 1	+23.1	29.9	0.78	Tested, Good Response
2	+11.0	22.0	0.46	Tested, Good Response
3	+3.0	17.3	0.06	Tested, Slow Response
116 1	+24.6	25.3	0.40	Tested, Good Response
2	+3.0	12.5	0.52	Tested, Good Response
3	-10.7	6.2	0.26	Tested, Good Response
117 1	+24.4	27.0	1.13	Tested, Good Response
2	+8.5	13.9	0.88	Tested, Good Response
3	-3.1	7.0	0.17	Tested, Good Response
118 1	+22.2	27.4	0.68	Tested, Good Response
2	+6.4	17.8	1.42	Tested, Good Response
3	-10.1	6.2	0.28	Tested, Good Response
119 1	+11.7	20.3	0.40	
2	+2.5	14.6	0.06	
3	-19.7	12.9	0.09	
120 1	+13.5	21.0	1.02	
2	-5.1	21.1	0.24	
3	-18.8	17.3	0.33	
201 1	+0.1	6.8	0.13	
2	-10.3	5.1	0.21	
202 1	+3.0	5.2	0.27	
2	-10.9	2.6	0.40	
203 1	+0.7	6.3	0.05	
2	-12.7	4.4	0.34	
204 1	+1.1	4.8	0.27	
2	-14.4	2.6	0.29	
205 1	+0.8	4.5	0.43	
2	-13.5	3.4	0.27	

NOTE: Elevations were estimated from topographic plan and are approximate.

TABLE B-III

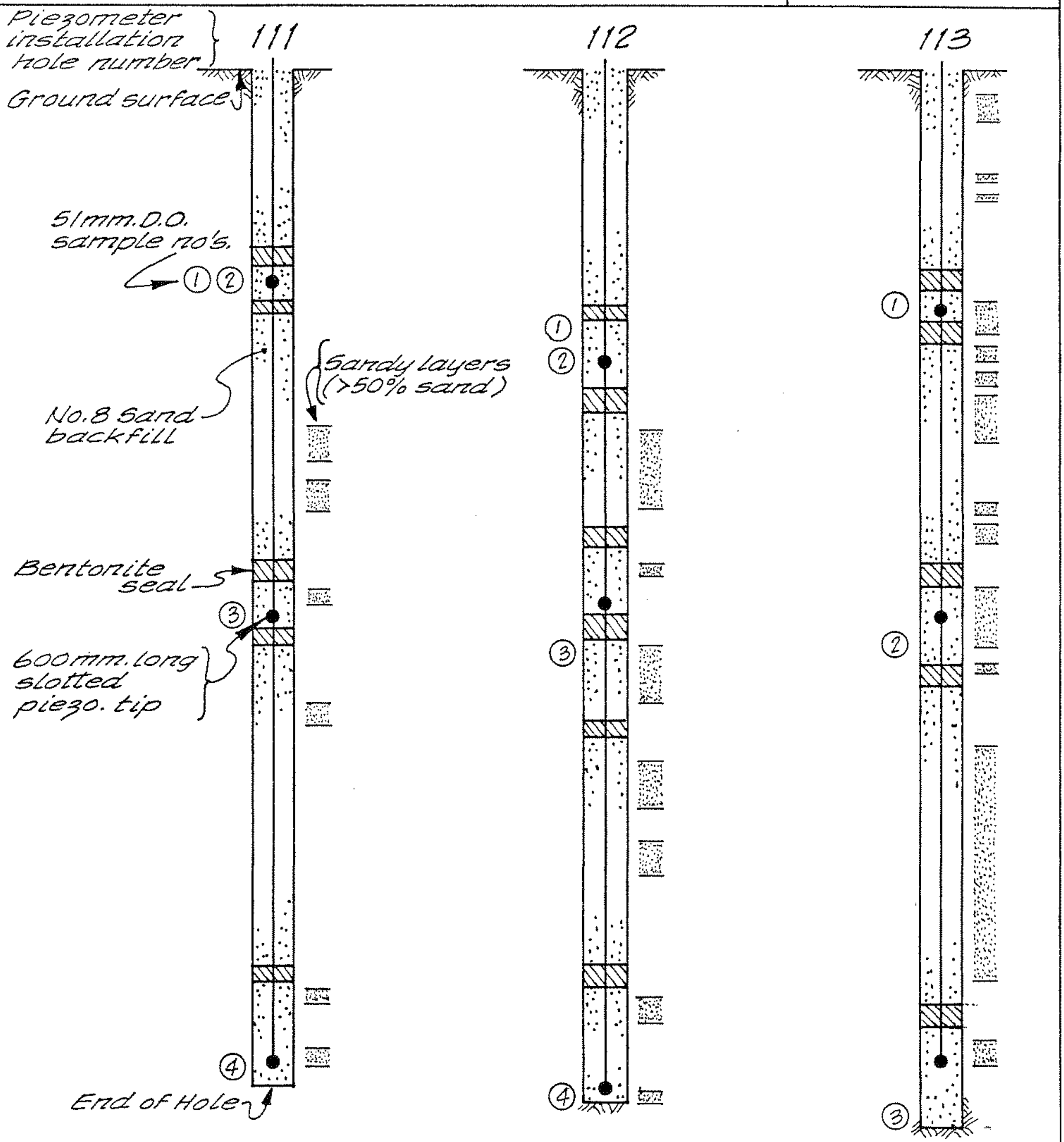
SUMMARY OF FALLING HEAD TEST RESULTS

PIEZOMETER NUMBER	BASIC[a] TIME LAG (sec)	LENGTH OF GRAVEL PACK (m)	CALCULATED HYDRAULIC CONDUCTIVITY (m/sec)	COMMENTS
111 1	-	2.40	-	Negligible Response k <1 x 10^{-10}
2	-	2.13	-	Negligible Response k <1 x 10^{-10}
3	41,400	4.60	1.6×10^{-9}	
113 1	-	1.67	-	Negligible Response k <1 x 10^{-10}
2	185	3.50	4.0×10^{-7}	
114 1	21	3.65	2.4×10^{-6}	
2	1,830	3.20	3.0×10^{-8}	
115 1	19,200	2.60	5.5×10^{-9}	
2	4,740	3.65	1.6×10^{-8}	
3	-	3.04	-	Negligible Response k <1 x 10^{-10}
116 1	210	8.53	2.0×10^{-7}	
2	5,280	3.05	1.7×10^{-8}	
3	2,400	6.40	2.0×10^{-8}	
117 1	152	6.70	2.0×10^{-7}	
2	3,060	11.58	9.6×10^{-9}	
3	4,500	13.10	5.9×10^{-9}	
118 1	165	10.97	1.0×10^{-7}	
2	2,055	10.36	1.6×10^{-8}	
3	18,240	2.13	6.8×10^{-9}	
103 B	1,380	10.0	2.4×10^{-8}	
A	-	5.3	-	Negligible Response k <1 x 10^{-10}

(a) See Hvorslev (1951)

PIEZOMETER INSTALLATION DETAILS

Figure B-1



NOTE: Sandy zones shown were inferred primarily from drilling conditions with only infrequent split spoon sampling at the locations shown.

SCALE 1:250 vertical.

PROJECT NO. 822-1071 DRAWN M.Z. REVIEWED J.W. DATE Mar. '83

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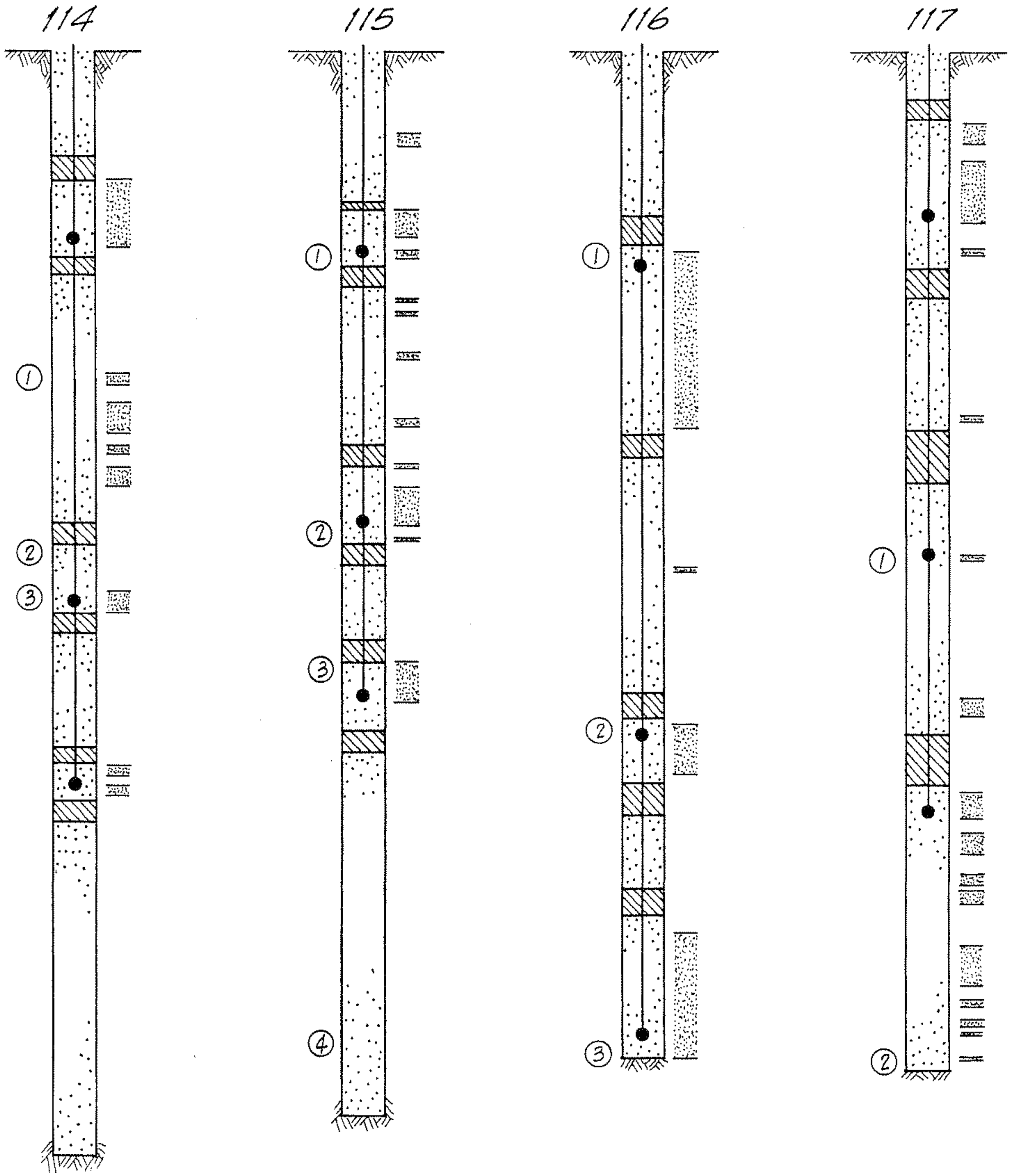
PIEZOMETER INSTALLATION DETAILS

Figure B-2



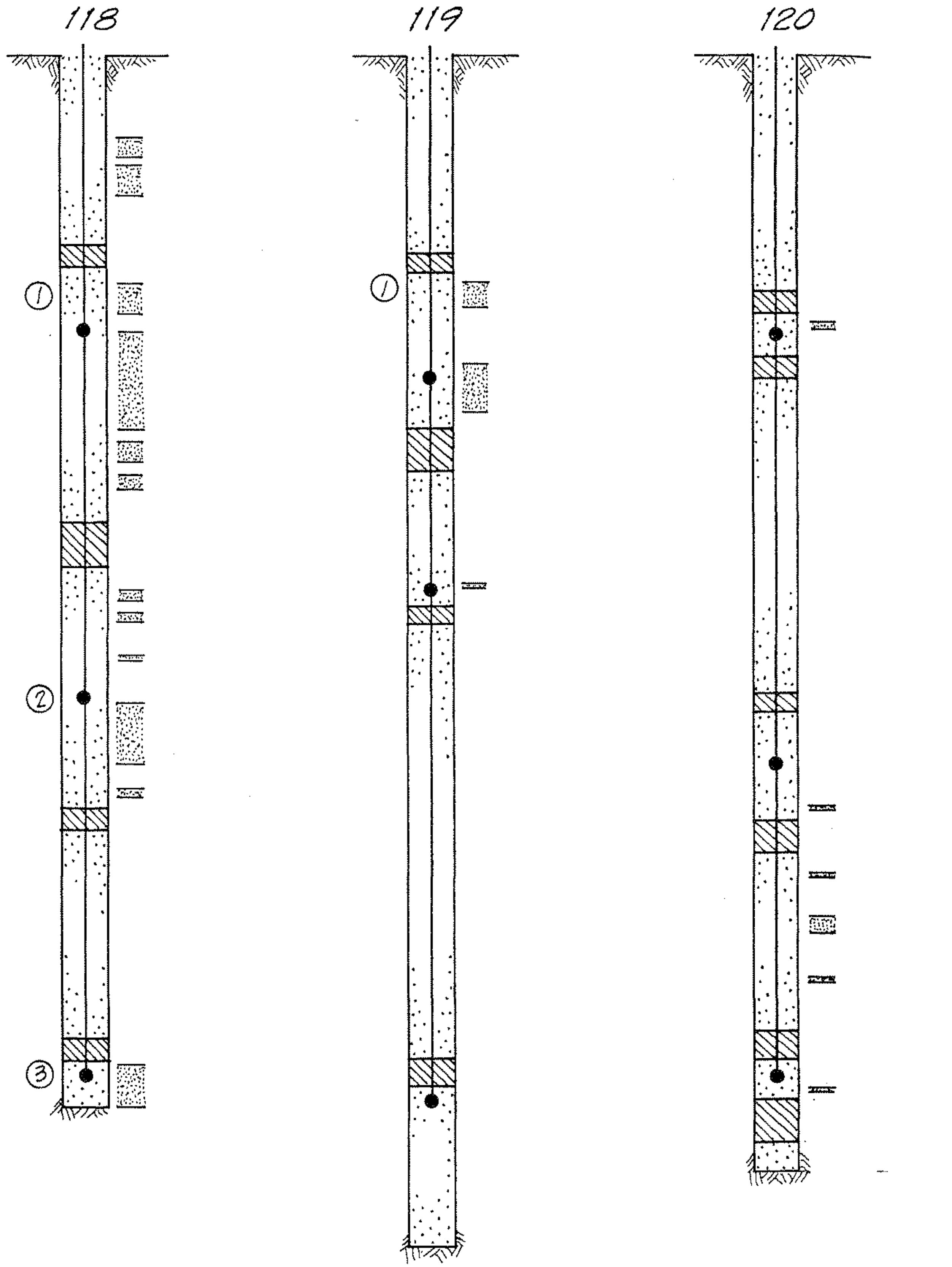
For explanatory notes, see Fig. B-1.

SCALE 1:250 vertical

PROJECT NO. 822-1071 DRAWN Mg REVIEWED T.D. DATE Mar. '83

PIEZOMETER INSTALLATION DETAILS

Figure B-3



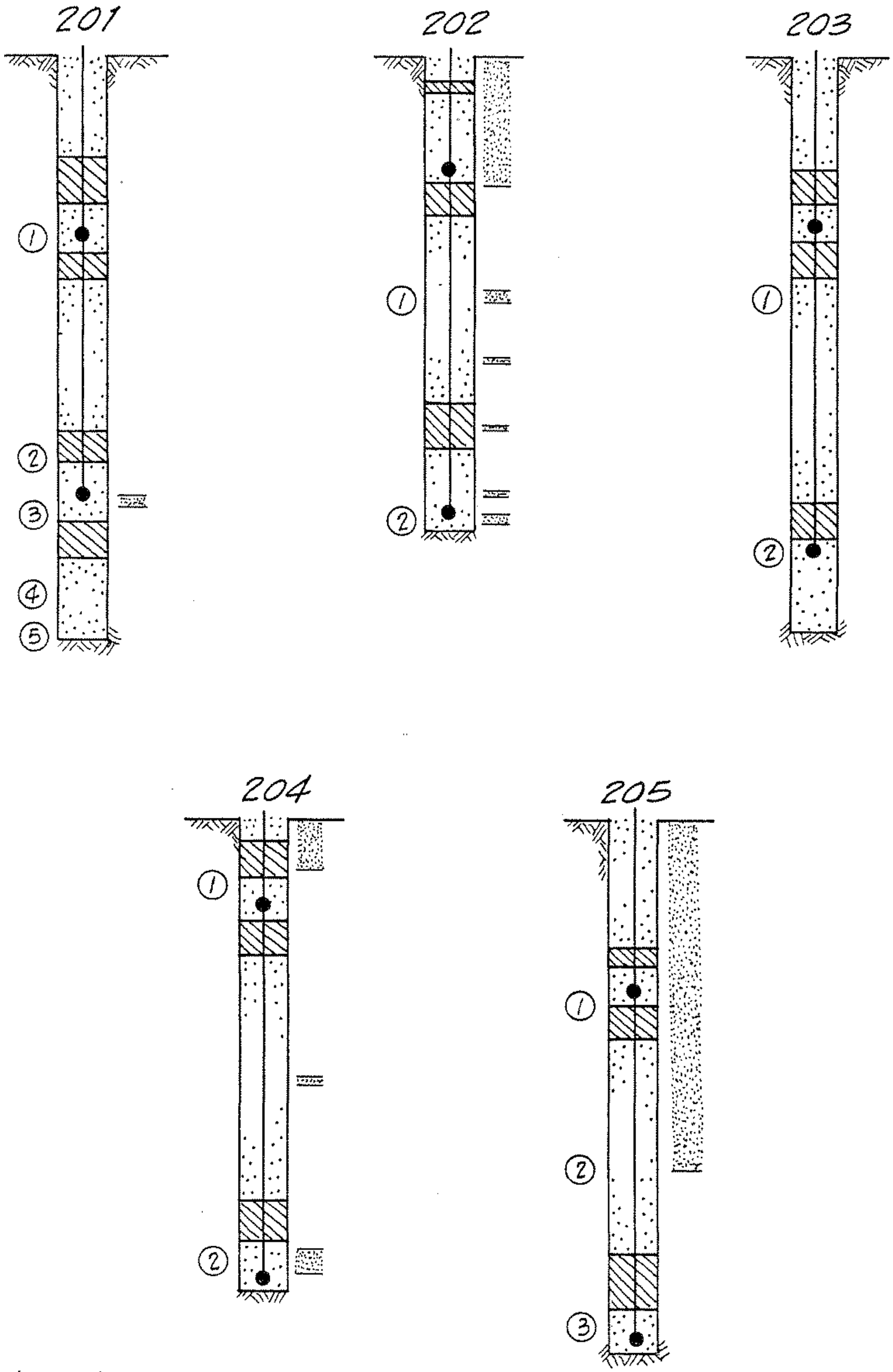
For explanatory notes, see Fig. B-1.

SCALE 1:250 vertical

PROJECT NO. 822-1071 DRAWN Ing. REVIEWED T.A. DATE Mar.'83

PIEZOMETER INSTALLATION DETAILS

Figure B-4



For explanatory notes, see Fig. B-1

SCALE: 1:250 vertical

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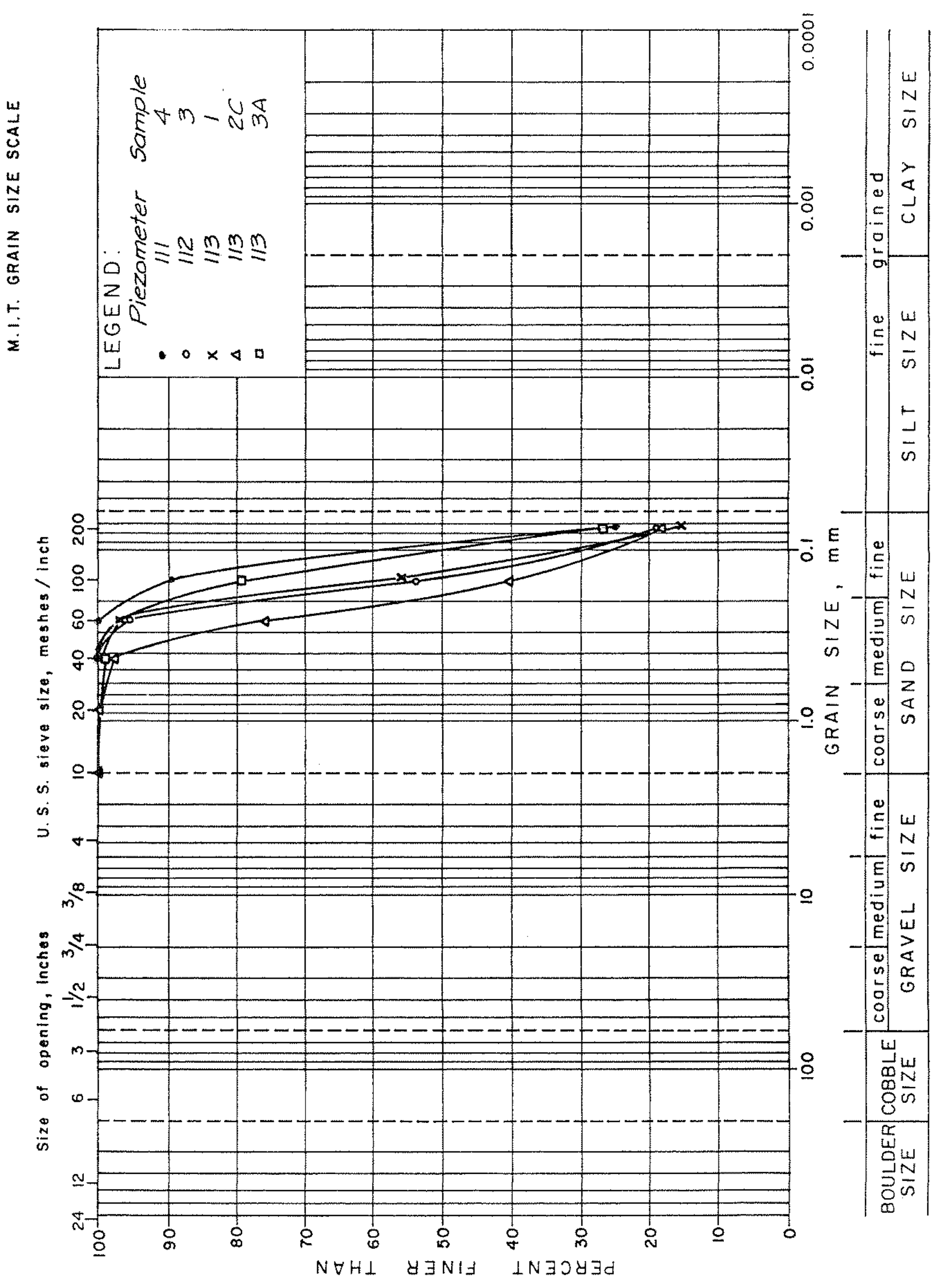
GRAIN SIZE DISTRIBUTION

Figure B-5



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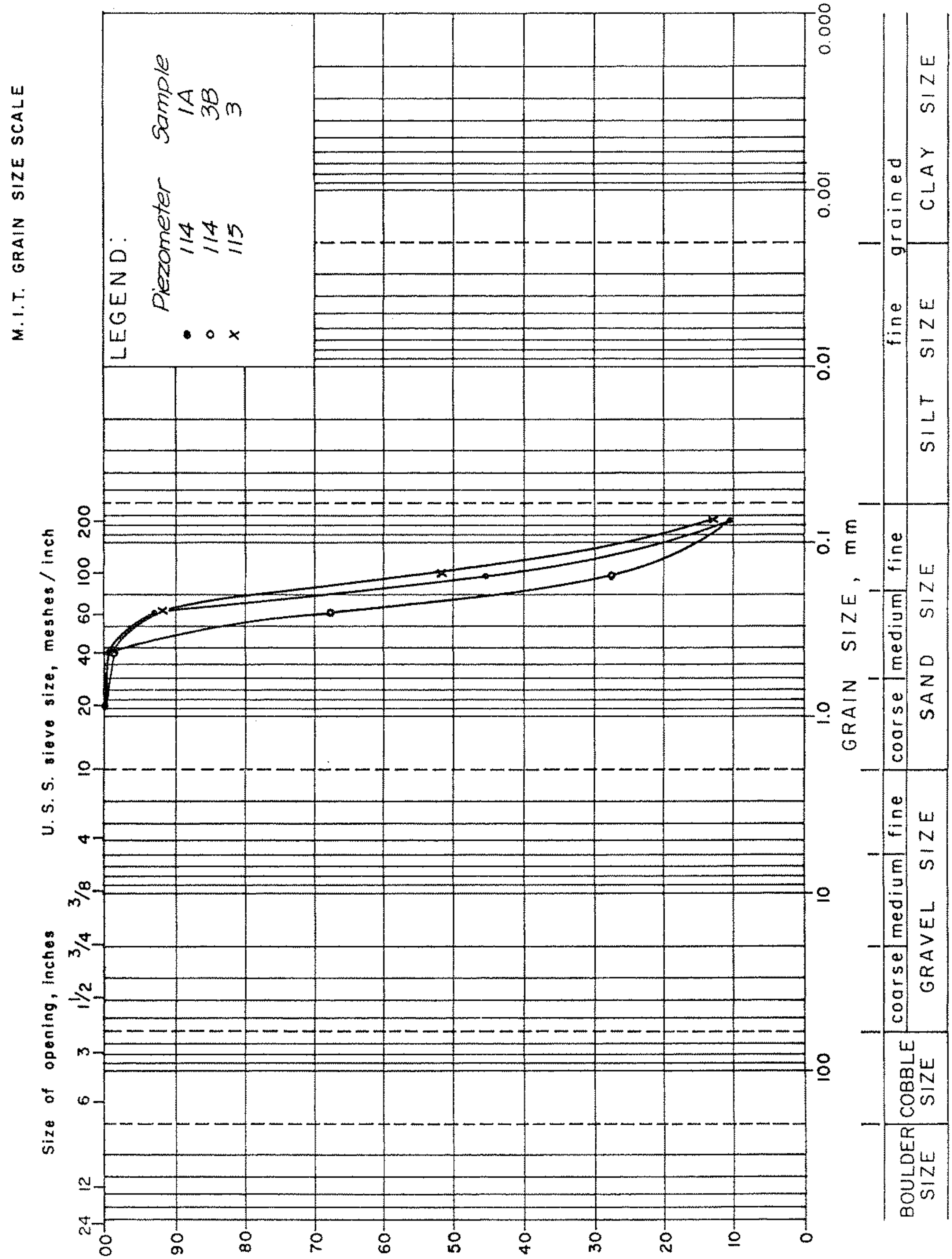
GRAIN SIZE DISTRIBUTION

Figure B-5A



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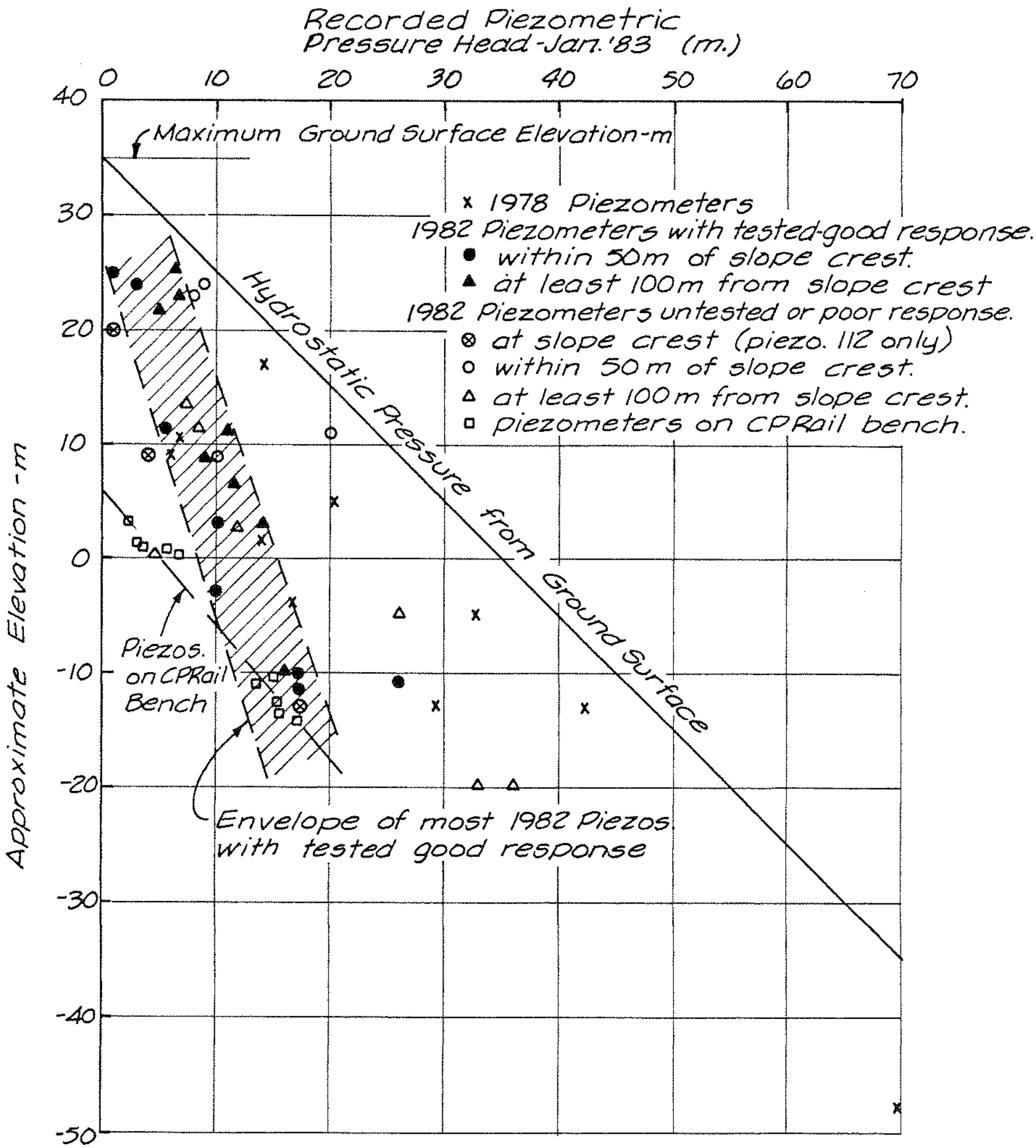
VARIATION OF PIEZOMETRIC PRESSURE WITH DEPTH

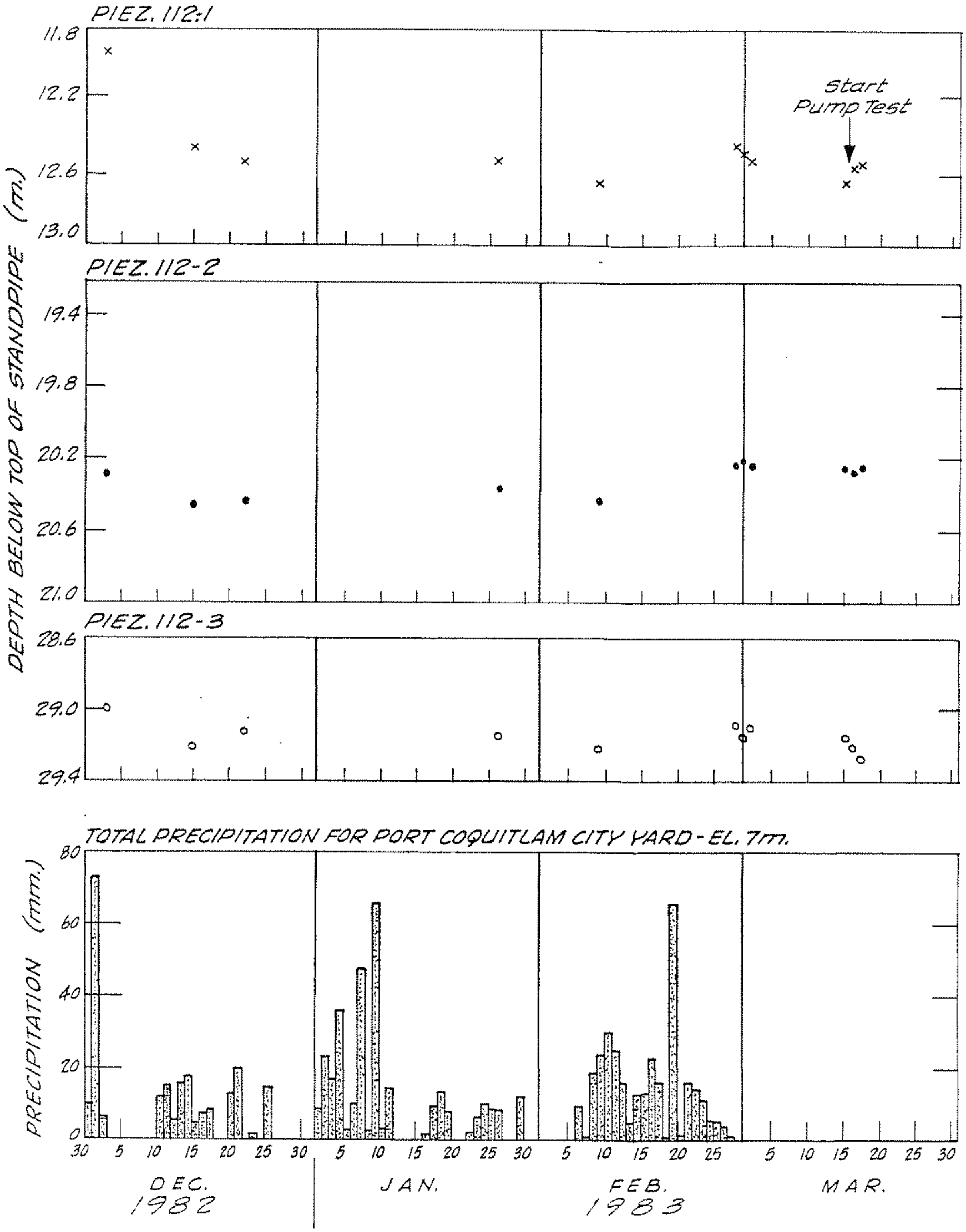
Figure B-6



PROJECT NO. 822-1071 DRAWN R.D. REVIEWED 72 DATE Mar. '83

HYDROGRAPH - PIEZOMETER 112

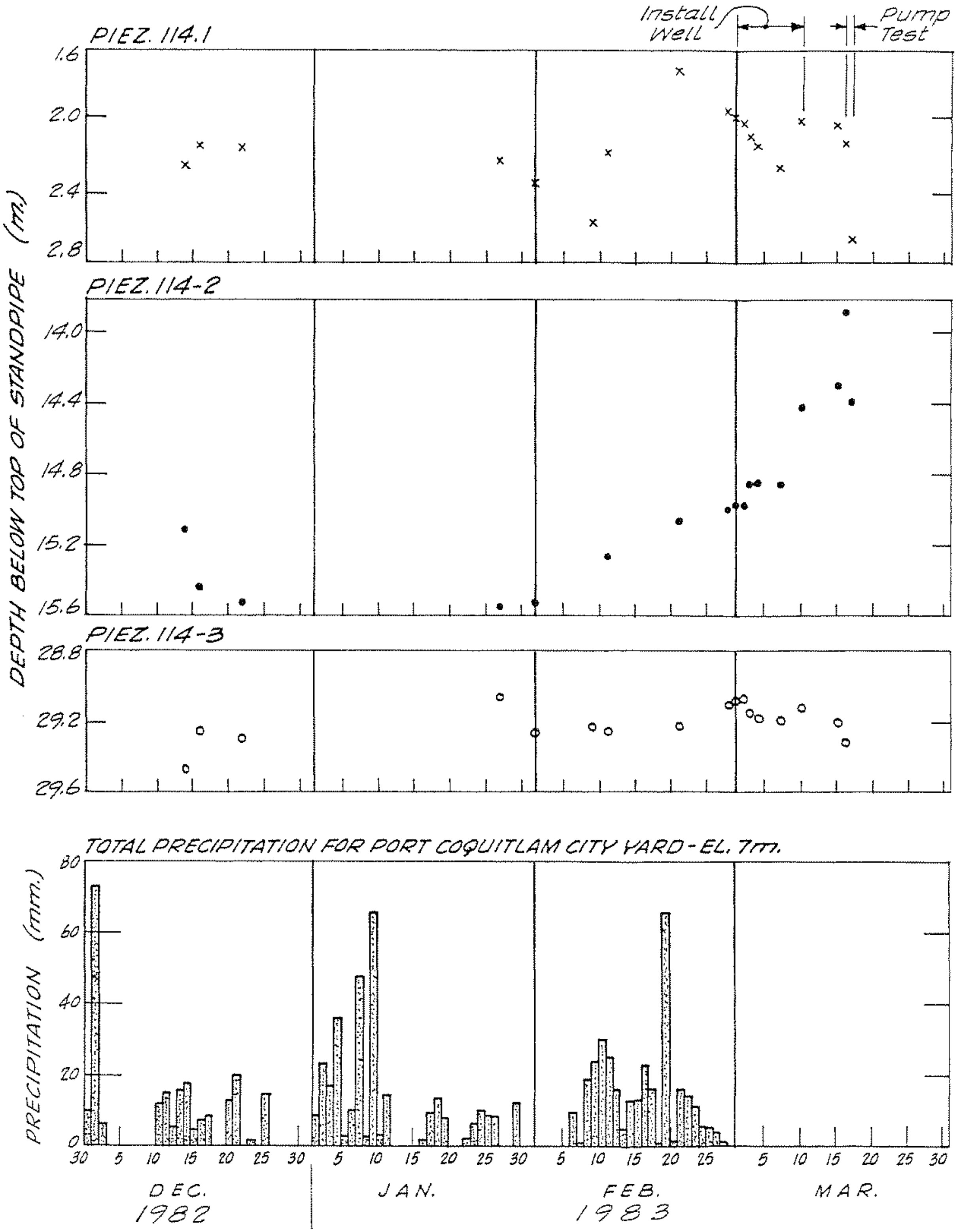
Figure B-7



PROJECT NO. 822-1071 DRAWN Ing. REVIEWED DATE Mar. '83

HYDROGRAPH - PIEZOMETER 114

Figure B-8



PROJECT NO. 822-1071 DRAWN Ing. REVIEWED DATE Mar. '83

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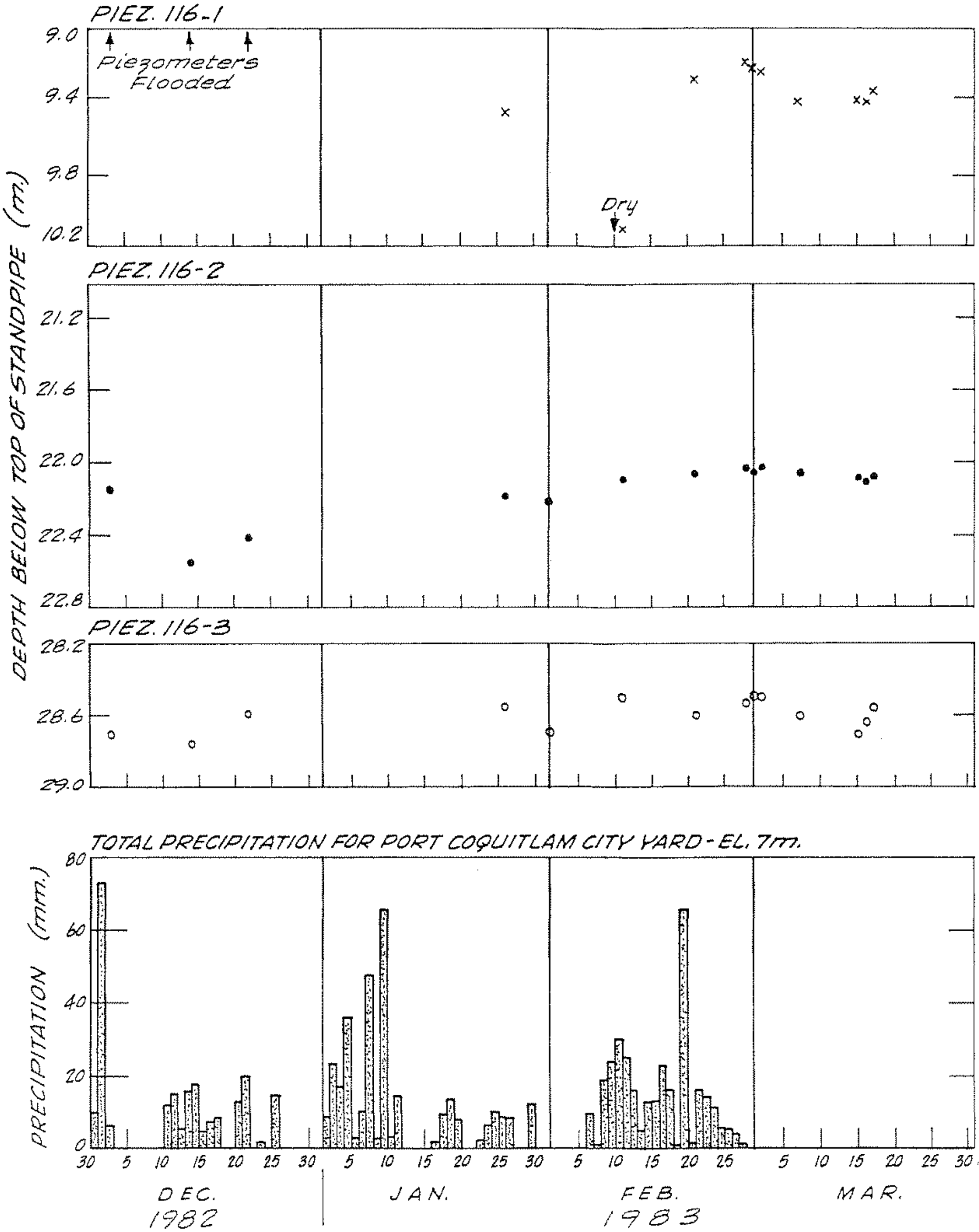
HYDROGRAPH - PIEZOMETER 116

Figure B-9



PROJECT NO. 822-1071 DRAWN Ing. REVIEWED TL DATE Mar. '83

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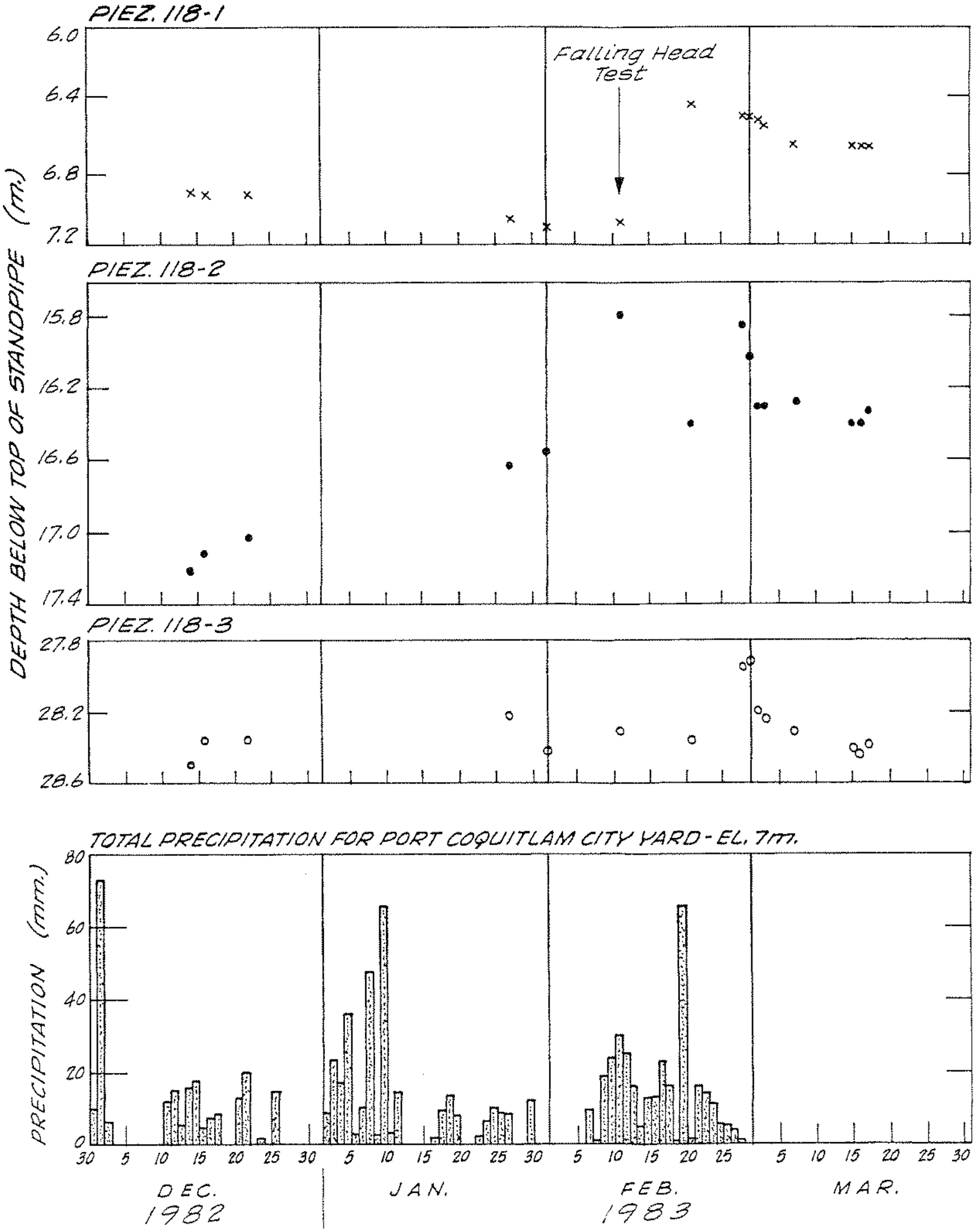
HYDROGRAPH - PIEZOMETER 118

Figure B-10



PROJECT NO. 822-1071 DRAWN Ing. REVIEWED J.A. DATE Mar. '83

APPENDIX C

Pump Test Program

TABLE OF CONTENTS FOR APPENDIX C

PUMP TEST PROGRAM

As part of the present study to assess the effectiveness of slope depressurization techniques, a test well was drilled and pump tested. Analyses of the data has provided useful information on the effectiveness of wells for slope depressurization.

1.0 WELL INSTALLATION

Prior to the drilling of the borehole, the stratigraphy and soil samples from the earlier boreholes were examined to determine the optimum well design. Grain size analyses were run on the sandier samples from several boreholes (see Figures B-5 and B-5A). It was determined that due to the heterogeneous nature of the strata, maximum flow into the well would be achieved by screening as much as possible of the more permeable zones.

The well is located approximately 14 m southwest of piezometer 114. The well was drilled by Norwest Water Well Drilling of Langley, B.C., using a percussion/cable tool method. During the drilling, soil samples were taken every 0.6 m for classification. The borehole penetrated varying thicknesses of clayey silt, sandy silt and fine sands. Little ground water inflow into the borehole was detected during drilling, although a zone of fine sand between 15.8 and 18.6 m appeared to produce more significant ground water inflow.

Upon completion of the drilling, the well screen and riser pipe were installed. In order to enable maximum flow into the well, the more permeable zones were screened with 152 mm nominal diameter, 20 slot PVC well screen. To prevent migration of the siltier material into the well, the annulus between the screen/riser and the drilled hole was backfilled with an f-16 sand pack.

The well design is shown on the hydrogeological log in Appendix C and consists of a tail pipe followed by sections of screen and riser to the surface. The tail pipe acts to collect the sediment that may move into the well during pumping. The well screen assembly and sand pack were installed by the pull-back method. During the installation of the sand pack, a blockage occurred in the annulus approximately 15 m below surface. On trying to free the casing, the well screen assembly was pulled from the hole. The casing was re-advanced to the required depth and the hole cleaned out. The screen assembly was re-set and sand packing repeated. During the second attempt, no problems were encountered. The annulus was backfilled with sand to a depth of 4.2 m as the casing was withdrawn. Upon completion of sand packing, a large cavity was visible approximately 2 to 3 m below ground surface. Ground water was noted flowing into the well from this zone. The cavity was backfilled with crushed rock to allow this water to enter the well during the pump test. Following completion, the water level in the well was recorded as 1.95 m below surface.

The well was developed by bailing for a period of 1 to 2 hrs. Some fine sand from the sand pack and silt was removed during the development. Further development was carried out by C.P.I. Equipment of Langley, B.C., who later pump tested the well. A 5 HP submersible pump was set in the well at 25.9 m below ground surface and used to backwash the well. The technique involved drawing the water level down in the well to approximately 20 m below surface, then turning the pump off and allowing the water in the drop pipe to fall back into the well. Backwashing was carried out for a period of 3 hrs, little improvement in well efficiency was noted during this time and development was thus terminated.

2.0 PUMP TEST PROCEDURE

The 5 HP pump used in development was replaced by a 1/2 HP submersible pump set at 25.9 m below ground surface. A 51 mm diameter drop pipe was used to conduct the water to the surface where it was discharged via 100 mm plastic pipe into the surface ditch draining north parallel to 216th Street. The ditch was seen to be underlain by silty clay and infiltration of discharged water into ground was considered to be minor.

Flow rate measurements during the pump test were carried out by recording the time required to fill a 5 imperial gallon pail. Prior to the start of the test, ground water levels were recorded in neighbouring piezometers. The test commenced on March 16th, 1983, and was continued for a duration of 1360 min. The well was pumped at a near constant rate of 0.52 l/sec for the duration of the test. Following the drawdown phase, the pump was switched off and the water levels allowed to recover. During the test, water levels were measured in the well and surrounding piezometers. Appendix C contains the pump test data. Hydrographs for the well and the piezometers 114-1 and 2 during the pump test are presented as Figures C-1, C-2, and C-3.

3.0 PUMP TEST ANALYSIS AND DISCUSSION

From the monitoring of piezometers adjacent to the well, a downward hydraulic gradient is present at the pump test site with piezometric heads* decreasing with depth. The well is completed within differing hydrostratigraphic zones, and as such the effect of pumping the well will be different within each zone. The hydraulic response to pumping would be first seen in the near surface zones, where piezometric heads are greatest. Deeper piezometers (e.g. piezometer 114-2) are completed within less permeable zones with lower piezometric heads, and therefore the water level within the well would have to be drawn down below these lower piezometric heads to see a hydraulic response.

The conditions described above make analysis of the data by standard analytical techniques difficult. In this respect, only data from the pumping well was analyzed by standard techniques (Cooper and Jacob, 1946 and Boulton, 1963). The analyses are presented as Figures C-4 and C-5 in Appendix C, with a summary presented in the following Table.

*Piezometric head is the sum of the elevation head and the piezometric pressure head.

SUMMARY OF PUMP TEST ANALYSES

OBSERVATION POINT	TRANSMISSIVITY m2/sec	METHOD OF ANALYSIS
Pumping Well	1×10^{-5}	Jacob Drawdown
Pumping Well	6×10^{-6}	Theis Recovery
Pumping Well	Early Time No Analysis Possible	Boulton
	Late Time 9×10^{-6}	Boulton

The values of transmissivity calculated by the various methods are in reasonable agreement. It is assumed that most of the water entering the well was derived from the near-surface silty sand and the fine sand encountered between 15.8 and 18.6 m depth (a total saturated thickness of 7m) resulting in a hydrualic conductivity of 1×10^{-6} m/sec for these zones.

A maximum drawdown response to pumping of 15 m was recorded in the pumping well during the pump test. The water level in the well had not stabilized when the pump was switched off. The response of the well, as shown on Figure C-4, is typical of an unconfined aquifer exhibiting delayed yield. Piezometer 114-1 sustained a maximum drawdown of 0.44 m. The water level was close to apparent stabilization, although continued pumping of the well may have induced continuing drawdown. Piezometer 114-2, completed at an elevation of 8.8 m, did not show a hydraulic response to pumping the well until some 250 min after pumping commenced. The delayed hydraulic response is due to the early time water level in the well being much higher than the static level in the piezometer. When the water level in the well had been drawn down below this static level, the piezometer in this deeper zone showed a hydraulic response. Piezometer 114-2 recorded a maximum drawdown of nearly 0.5 m during the test, and was not approaching stabilization. The response of this piezometer is typical of a quasi-confined situation.

The following table illustrates the response of other piezometers monitored during the pump test.

PIEZOMETER NUMBER	APPROXIMATE TIP ELEVATION (m)	APPROXIMATE DISTANCE FROM WELL (m)	DEPTH TO WATER (m) BEFORE PUMP TESTING 16/3 9.00 hrs	DEPTH TO WATER (m) AFTER PUMP TESTING 17/3 11.00 hrs
112-1	+20.1	115	12.56	12.55
-2	+9.1	115	20.25	20.25
-3	-12.8	115	29.15	29.26
113-1	+23.2	60	0.87	1.06
-2	+9.3	60	3.90	4.02
-3	-10.5	60	28.45	28.43
116-1	+24.6	200	9.40	9.41
-2	+3.0	200	22.10	22.10
-3	-10.7	200	28.64	28.62
118-1	+22.2	210	6.65	6.71
-2	+6.4	210	16.40	16.33
-3	-10.1	210	28.43	28.35

Piezometers 113-1 and 2 recorded a drawdown of 0.19 and 0.12 m, respectively, during the test. Fluctuations recorded in the other piezometers are considered to be due to natural causes and not associated with the pumping of the well.

4.0 PRELIMINARY DESIGN OF DEEP WELL SYSTEM FOR REMEDIAL TREATMENT

Analyses of the pump test data indicates that wells may be a suitable method of reducing the ground water levels underlying the site. The test indicated that both the near surface silty sand and the underlying less permeable silty zones are both drainable.

A preliminary estimate of well spacing has been prepared based on the results of the pump test, but longer term pumping (in the order of a few months) will be required to verify this design. In order to achieve a minimum 5 m drawdown between wells in the siltier zones after 1 year of operation, it is estimated that wells spaced approximately every 100 m will be required. This well spacing would be subject to revision following long term pump testing, or initial permanent well installation. The wells would be drilled to an anticipated depth of between 30 and 35 m, and completed as illustrated on Figure 6. It is anticipated that long term pumping rates for each well will be less than 0.7 l/sec.

If required, it is recommended that the wells be completed at 127 mm nominal diameter, rather than the 152 mm nominal diameter as used in the test well. The 127 mm nominal diameter screen and casing should be internal and external flush to prevent possible bridging of the sand pack during well completion. The well screen should be installed with centralizers to keep the screen aligned in the centre of the hole, and ensure even sand pack distribution. The installed pump should be capable of pumping up to 0.7 l/sec against a total dynamic head of 40 m.

Two well designs are possible:

1) screening the entire thickness of saturated material;
2) screening only the less permeable sandy silts found below approximately 12 m, with a grout seal placed above the top of the screened section to prevent drainage from the silty sands.

Screening the entire saturated thickness will allow some drainage of the near surface silty sands to occur. In the event that the well is not pumping, the water level in such a well will probably be near surface, reflecting the level in the relatively permeable near ground surface silty sand. Under these conditions, the well will act as a vertical drain allowing the transfer of water from the silty sands to the less permeable material at depth due to the prevailing downward hydraulic gradient. This flow of water into the deeper zones will act as a source of recharge and induce a rise in water levels at depth. It is therefore recommended that these wells (screened over the entire saturated thickness) should be continually pumped.

The alternative well design will only allow drainage from the lower less permeable silty strata. Wells completed according to this design could be pumped on an intermittent basis. A combination of an interceptor drain installed in the silty sands and wells to depressurize the less permeable material at depth (Alternative 2) would appear the most feasible ground water control measures.

It should be noted that lowering water pressures within the slope using wells, as discussed above, will take a finite time (probably several months to a year). Unless wells are installed at a much closer spacing, it will not be possible to rapidly lower water pressures by only switching the pumps on once critical water levels have been recorded. Rather, it is anticipated that pumping would be carried out for at least several months during the year (depending on whether alternative 1 or 2 is chosen) to ensure that water levels do not reach critical levels.

The cost of such permanent wells has been estimated based on 1983 costs incurred during the present investigation. Drilling and completion of a well to between 30 and 35 m is estimated to cost $10,000. Provision of a submersible pump, drop pipe, electrical fittings and installation, is estimated at $1,500. Therefore, the installation cost for a single well is estimated as $11,500, excluding engineering and the cost of providing hydro-connection and discharge of the pumped water into the sewer system.

It is difficult to estimate costs that may be incurred during the operation of the wells. Siltation of the well may occur, but removal of the pump and redevelopment of the well should be possible. In the event that remedial measures are unsuccessful, a replacement well could be drilled. There may also be a need to replace worn out pumps after a period of pumping. Life expectancy for a small submersible pump cannot be accurately estimated.

The estimate of operating costs given below is based on the following arbitrary assumptions:

(1) All wells will be redeveloped and sediment removed after first year of operation, and then at 10 year intervals. The 1983 cost is assumed to be $1,000/well.

(2) One third of the wells will be replaced at 20 year intervals at a 1983 cost of $10,000/new well.

(3) Pumps will be replaced every 7 years at a 1983 cost of $1,200/pump.

(4) Power requirements are 3300 Kw.hr/pump/year at a 1983 cost of $0.05/Kw.hr.

YEARLY INFLATION RATE (%)	AVERAGE ANNUAL OPERATING COST PER WELL	
	OVER 25 YEAR PERIOD	OVER 50 YEAR PERIOD
5	$1,300	$ 3,200
10	$2,500	$16,000

If wells are utilized in order to depressurize the slope, it is recommended that a staged drilling and testing program be adopted. It will be necessary to install additional piezometers to monitor ground water levels during testing and operation of the wells. Test wells to be later used as production wells should be drilled at the west and east end of the site, to verify the hydrogeological conditions. At that time, it may be necessary to make changes to the anticipated well field design based on the additional data.

4.1 Effects of Deep Dewatering

Lowering of the ground water pressures below their minimum level will increase the effective stress in the underlying soil strata. This will result in some settlement of the dewatered area, as well as an increase in the shear strength of the soils. The magnitude of such settlements will vary depending on the effectiveness and extent of dewatering, and the thickness and relative depth of the various strata. The settlements are expected to be greatest in the clayey strata which are at least 6 m below ground surface and would be expected to express themselves as a general, fairly uniform lowering of the ground surface. Severe differential settlements would not be expected due to dewatering, but some gentle tilting of the ground surface may occur. Settlements within the sandy strata would occur fairly rapidly, while consolidation settlement of the clayey layers could continue for several years under sustained dewatering.

There is no available consolidation test data for the site area. We have carried out a preliminary assessment of the possible magnitude of such settlements using data from near surface strata at nearby sites. The results of this assessment indicate that total long term settlements in the order of 0.5 m could be expected if the proposed well dewatering system is in operation. Settlements resulting from installation of the proposed interceptor drain are expected to be minor.

If dewatering using wells is considered necessary in the future, we recommend an investigation be carried out to assess the magnitude of such settlements and their effects on structures, roads and services in the area. The investigation should include a series of consolidation tests carried out on samples obtained from boreholes put down to depths of at least 30 to 40 m. This sampling could be carried out in boreholes put down for installation of additional piezometers, as discussed above.

PUMPING TEST DATA

PUMPING WELL PW1

WELL DIAMETER 254 mm

OBSERVATION WELL PW1

DISTANCE FROM PUMP WELL 0.0

DEPTH TO STATIC WATER LEVEL 3.10 m

GROUND ELEVATION (ESTIMATED) 34.2 m

TYPE OF TEST Constant Rate

DATE STARTED
Year/Month/Day/Hours/Minutes 83/03/16/10/30

DATE STOPPED
Year/Month/Day/Hours/Minutes 83/03/17/09/10

PUMPING RATE 0.52 1/sec

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE 1/sec	COMMENTS
09/20	0.00	2.86			Install Pump
09/58		2.89			
10/00/30		3.19			Pump on, fill discharge line
00/40		3.23			
00/50		3.26			
00/57		3.29			
01/03		3.31			
01/14		3.40			
01/30		3.38			
10/12/00		4.60			Pump off
12/30		4.28			
12/45		4.17			Recovery in well
13/30		4.02			
14/00		3.85			
15/00		3.58			
16/00		3.46			
20/00		3.29			
23/30		3.22			
10/26/30		3.16			
10/30/00	0.00	3.10	0.0	0.52	Pump on
/10	0.17	3.33	0.23		
30/25	0.42	3.45	0.35		
30/35	0.58	3.54	0.44		

PUMPING TEST DATA

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
30/50	0.83	3.60	0.50		
31/00	1.00	3.60	0.50		
31/15	1.25	3.70	0.60		
31/30	1.50	3.73	0.63		
32/00	2.00	3.80	0.70	0.51	
32/30	2.50	3.85	0.75		
33/00	3.00	3.90	0.80	0.49	
33/30	3.50	3.96	0.86		
34/00	4.00	4.06	0.96	0.49	
35/00	5.00	4.28	1.18	0.53	
36/00	6.00	4.46	1.36		
37/00	7.00	4.55	1.45	0.52	
38/00	8.00	4.62	1.52	0.52	
39/00	9.00	4.66	1.56	0.49	
40/00	10.00	4.71	1.61	0.52	
45/00	15.00	4.83	1.73	0.52	
50/00	20.00	4.87	1.77	0.51	
55/00	25.00	4.98	1.88	0.52	
11/00/00	30.00	5.08	1.98	0.52	
05/00	35.00	5.21	2.11	0.60	Pump rate
10/00	40.00	5.38	2.28	0.54	increased
15/00	45.00	5.54	2.44	0.51	Pump rate
25/00	55.00	5.81	2.71	0.51	adjusted
31/00	61.00	5.98	2.88	0.52	
42/00	72.00	6.30	3.20	0.52	
50/00	80.00	6.54	3.44	0.52	
12/00/00	90.00	6.86	3.76	0.52	
10/00	100.00	7.22	4.12	0.51	
20/00	110.00	7.57	4.47	0.52	
35/00	115.00	8.11	5.01	0.51	
50/00	130.00	8.58	5.48		
13/01/00	151.00	8.94	5.94	0.52	
15/00	165.00	9.33	6.23		
25/00	175.00	9.60	6.50	0.52	
40/00	190.00	9.96	6.86	0.51	
55/00	205.00	10.28	7.18		
14/10/00	220.00	10.58	7.48	0.52	
14/25/00	235.00	10.90	7.80	0.53	
14/50/00	260.00	11.40	8.30	0.52	
15/05/00	275.00	11.63	8.53	0.52	
15/20/00	290.00	11.85	8.75	0.52	
15/35/00	305.00	12.02	8.92	0.52	

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
15/50/00	320.00	12.26	9.16	0.52	
16/10/00	340.00	12.57	9.47	0.51	
16/32/00	362.00	12.81	9.71	0.52	
16/45/00	375.00	12.97	9.87		
17/00/00	390.00	13.13	10.03	0.51	
18/00/00	450.00	13.57	10.47	0.52	
18/30/00	480.00	13.64	10.54		
19/00/00	510.00	13.76	10.66	0.51	
19/30/00	540.00	14.03	10.93	0.52	
20/00/00	570.00	14.26	11.16	0.51	
20/30/00	600.00	14.35	11.25		Pump rate constant at 0.52 l/sec
21/00/00	630.00	14.49	11.39		
21/30/00	660.00	14.52	11.42		
22/00/00	690.00	14.91	11.81		
22/30/00	720.00	15.32	12.22		
23/00/00	750.00	15.49	12.39		
23/30/00	780.00	15.61	12.51		
00/00/00	810.00	15.73	12.63		
01/30/00	900.00	16.50	13.40		
02/00/00	930.00	16.72	13.62		
02/30/00	960.00	16.99	13.89		
03/00/00	990.00	17.02	13.92		
03/30/00	1020.00	17.09	13.99		
05/00/00	1110.00	17.32	14.22		
05/30/00	1140.00	17.38	14.28		
06/00/00	1170.00	17.40	14.30		
06/30/00	1200.00	17.43	14.33		
07/00/00	1230.00	17.46	14.36		
07/30/00	1260.00	17.59	14.49		
08/30/00	1320.00	17.81	14.71		
09/05/00	1355.00	17.98	14.88		
09/09/00	1359.00	18.00	14.90		
09/10/00	1360.00	18.01	14.91		Pump off start recovery

RECOVERY DATA

PUMPING WELL PW1

OBSERVATION WELL PW1 DISTANCE FROM PUMP WELL 0.0

INITIAL STATIC WATER LEVEL 3.10 m

FINAL PUMPING WATER LEVEL 18.01 m

DURATION OF TEST 1360 min

PUMPING RATE 0.52 l/sec

TIME Hr/min/sec	ELAPSED TIME (min) t'	t/t'	DEPTH TO WATER (m)	RESIDUAL DRAWDOWN (m)
09/10/00	0.00	-	18.01	14.91
09/10/05	0.08	17,000.0	18.87	15.77
09/10/10	0.17	8,000.0	17.67	14.57
09/10/15	0.25	5,440.0	17.50	14.40
09/10/20	0.33	4,121.2	17.39	14.29
09/10/25	0.42	3,238.1	17.20	14.10
09/10/30	0.50	2,720.0	17.03	13.93
09/10/35	0.58	2,344.8	16.92	12.82
09/10/40	0.67	2,029.9	16.27	12.17
09/10/45	0.75	1,813.3	16.63	12.53
09/10/50	0.83	1,638.6	16.47	12.37
09/10/55	0.92	1,478.3	16.30	12.20
09/11/00	1.00	1,360.0	16.15	12.05
09/11/45	1.75	777.1	14.70	11.60
09/12/00	2.00	680.0	14.10	11.00
09/12/10	2.17	626.7	13.70	10.60
09/12/30	2.50	544.0	13.15	10.05
09/12/45	2.75	494.6	12.85	9.75
09/13/23	3.38	402.4	11.60	8.50
09/13/35	3.58	379.9	11.30	8.20
09/13/45	3.75	362.7	11.00	7.90
09/14/00	4.00	340.0	10.55	7.45
09/14/30	4.50	302.2	9.75	6.65
09/15/00	5.00	272.0	9.00	5.90
09/15/15	5.25	259.1	8.63	5.53
09/15/30	5.50	247.3	8.27	5.17
09/15/45	5.75	236.5	7.88	4.78

Where t = total time since start of test

RECOVERY DATA

TIME Hr/min/sec	ELAPSED TIME (min) t'	t/t'	DEPTH TO WATER (m)	RESIDUAL DRAWDOWN (m)
09/16/00	6.00	226.7	7.58	4.48
09/16/30	6.50	209.2	6.91	3.81
09/17/00	7.00	194.3	6.30	3.20
09/17/30	7.50	181.3	5.74	2.64
09/18/00	8.00	170.0	5.25	2.15
09/18/30	8.50	160.0	5.00	1.90
09/19/00	9.00	151.1	4.90	1.80
09/19/30	9.50	143.2	4.81	1.71
09/20/00	10.00	136.0	4.69	1.59
09/20/30	10.50	129.5	4.50	1.40
09/21/00	11.00	123.6	4.37	1.27
09/21/30	11.50	118.3	4.27	1.17
09/22/00	12.00	113.3	4.18	1.08
09/23/00	13.00	104.6	4.03	0.93
09/24/00	14.00	97.1	3.94	0.84
09/26/00	16.00	85.0	3.80	0.70
09/28/00	18.00	75.6	3.72	0.62
09/30/00	20.00	68.0	3.69	0.59
09/35/00	25.00	54.4	3.61	0.51
09/41/00	31.00	43.9	3.57	0.47
09/50/00	40.00	34.0	3.53	0.43
10/00/00	50.00	27.2	3.49	0.39

PUMPING TEST DATA

PUMPING WELL: PW1

WELL DIAMETER: 254 mm

OBSERVATION WELL: BH-114 Grey (1)

DISTANCE FROM PUMP WELL: 14 m

DEPTH TO STATIC WATER LEVEL: 2.20 m

GROUND ELEVATION (ESTIMATED): 34.2 m

TYPE OF TEST: Constant Rate

DATE STARTED
Year/Month/Day/Hours/Minutes: 83/03/16/10/30

DATE STOPPED
Year/Month/Day/Hours/Minutes: 83/03/17/09/10

PUMPING RATE: 0.52 l/sec

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
09/55/00		2.13			
10/25/00		2.20			
10/30/00	0.0				
10/30/30	0.5	2.18	-0.02		
10/31/00	1.0	2.19	-0.01		
10/31/30	1.5	2.19	-0.01		
10/32/00	2.0	2.21	+0.01		
10/32/30	2.5	2.21	0.01		
10/33/00	3.0	2.22	0.02		
10/34/00	4.0	2.22	0.02		
10/35/00	5.0	2.23	0.03		
10/36/00	6.0	2.24	0.04		
10/37/00	7.0	2.25	0.05		
10/38/00	8.0	2.27	0.07		
10/39/00	9.0	2.275	0.075		
10/40/00	10.0	2.285	0.085		
10/45/00	15.0	2.32	0.12		
10/50/00	20.0	2.345	0.145		
10/55/00	25.0	2.37	0.17		
11/00/00	30.0	2.38	0.18		
11/05/00	35.0	2.40	0.20		
11/10/00	40.0	2.415	0.215		
11/15/00	45.0	2.425	0.225		

PUMPING TEST DATA

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
11/30/00	60.0	2.445	0.245		
11/40/00	70.0	2.46	0.26		
11/50/00	80.0	2.48	0.28		
12/00/00	90.0	2.51	0.31		
12/10/00	100.0	2.51	0.31		
12/25/00	115.0	2.53	0.33		
12/35/00	125.0	2.53	0.33		
12/50/00	140.0	2.52	0.32		
13/05/00	155.0	2.53	0.33		
13/20/00	170.0	2.535	0.335		
13/40/00	190.0	2.56	0.36		
14/00/00	210.0	2.55	0.35		
14/20/00	230.0	2.555	0.355		
14/45/00	255.0	2.53	0.33		
15/20/00	290.0	2.55	0.35		
16/00/00	330.0	2.57	0.37		
16/25/00	355.0	2.57	0.37		
17/00/00	390.0	2.55	0.35		
18/00/00	450.0	2.56	0.36		
19/00/00	510.0	2.58	0.38		
19/30/00	540.0	2.58	0.38		
20/00/00	570.0	2.57	0.37		
21/00/00	630.0	2.58	0.38		
22/00/00	690.0	2.59	0.39		
23/00/00	750.0	2.61	0.41		
00/00/00	810.0	2.61	0.41		
02/00/00	930.0	2.60	0.40		
03/00/00	990.0	2.59	0.39		
05/00/00	1110.0	2.60	0.40		
06/00/00	1170.0	2.61	0.41		
07/00/00	1230.0	2.61	0.41		
08/30/00	1320.0	2.63	0.43		
09/04/00	1354.0	2.64	0.44		

RECOVERY DATA

PUMPING WELL PW1

OBSERVATION WELL BH-114 Grey (1) DISTANCE FROM PUMP WELL 14.0 m

INITIAL STATIC WATER LEVEL 2.20 m

FINAL PUMPING WATER LEVEL 2.64 m

DURATION OF TEST 1360 min

PUMPING RATE 0.52 l/sec

TIME Hr/min/sec	ELAPSED TIME (min) t'	t/t'	DEPTH TO WATER (m)	RESIDUAL DRAWDOWN (m)
09/10/30	0.5	2720.0	2.64	0.44
09/11/00	1.0	1360.0	2.635	0.435
09/11/30	1.5	906.7	2.635	0.435
09/12/00	2.0	680.0	2.63	0.43
09/12/30	2.5	544.0	2.63	0.43
09/13/00	3.0	453.3	2.63	0.43
09/14/00	4.0	340.0	2.64	0.44
09/15/00	5.0	272.0	2.65	0.45
09/16/00	6.0	226.7	2.65	0.45
09/17/00	7.0	194.3	2.64	0.44
09/18/00	8.0	170.0	2.635	0.435
09/19/00	9.0	151.11	2.62	0.42
09/20/00	10.0	136.0	2.63	0.43
09/25/00	15.0	90.7	2.57	0.37
09/30/00	20.0	68.0	2.54	0.34
09/35/00	25.0	54.4	2.53	0.33
09/40/00	30.0	45.3	2.49	0.29
09/50/00	40.0	34.0	2.46	0.26
10/00/00	50.0	27.2	2.45	0.25
10/10/00	60.0	22.7	2.44	0.24
10/20/00	70.0	19.4	2.435	0.235
10/30/00	80.0	17.0	2.41	0.21
10/50/00	100.0	13.6	2.39	0.19
11/10/00	120.0	11.3	2.38	0.18
11/30/00	140.0	9.71	2.37	0.17
12/00/00	170.0	8.0	2.34	0.14

PUMPING TEST DATA

PUMPING WELL PW1

WELL DIAMETER 254 mm

OBSERVATION WELL BH-114 Orange (2)

DISTANCE FROM PUMP WELL 14 m

DEPTH TO STATIC WATER LEVEL 13.85 m

GROUND ELEVATION (ESTIMATED) 34.2 m

TYPE OF TEST Constant Rate

DATE STARTED
Year/Month/Day/Hours/Minutes 83/03/16/10/30

DATE STOPPED
Year/Month/Day/Hours/Minutes 83/03/17/09/10

PUMPING RATE 0.52 l/sec

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
10/43/00		13.85			
10/46/00		13.85			
10/51/00		13.85			
10/55/00		13.85			
11/01/00		13.85			
11/06/00		13.85			
11/11/00		13.85			
11/30/00		13.79			
11/40/00		13.86			
11/50/00		13.81			
12/00/00		13.81			
12/10/00		13.86			
12/25/00		13.84			
12/35/00		13.84			
12/50/00		13.88			
13/05/00		13.85			
13/20/00		13.84			
13/40/00		13.85			
14/00/00		13.84			

PUMPING TEST DATA

TIME Hr/min/sec	ELAPSED TIME (min.sec)	DEPTH TO WATER (m)	DRAWDOWN (m)	PUMPING RATE l/sec	COMMENTS
14/20/00	230	13.85	0		
14/45/00	255	13.865	0.015		
15/20/00	290	13.88	0.03		
16/00/00	330	13.89	0.04		
16/25/00	355	13.89	0.04		
17/00/00	390	13.90	0.05		
18/00/00	450	13.95	0.10		
19/00/00	510	13.97	0.12		
19/30/00	540	13.99	0.14		
20/00/00	570	13.99	0.14		
21/00/00	630	14.02	0.17		
22/00/00	690	14.07	0.22		
23/00/00	750	14.09	0.24		
00/00/00	810	14.09	0.24		
02/00/00	930	14.12	0.27		
03/00/00	990	14.19	0.34		
05/00/00	1110	14.21	0.36		
06/00/00	1170	14.25	0.40		
07/00/00	1230	14.29	0.44		
08/30/00	1320	14.30	0.45		

RECOVERY DATA

PUMPING WELL PW1

OBSERVATION WELL BH-114 Orange (2) DISTANCE FROM PUMP WELL 14.0 m

INITIAL STATIC WATER LEVEL 13.85 m

FINAL PUMPING WATER LEVEL 14.32 m

DURATION OF TEST 1360 min

PUMPING RATE 0.52 l/sec

TIME Hr/min/sec	ELAPSED TIME (min) t'	t/t'	DEPTH TO WATER (m)	RESIDUAL DRAWDOWN (m)
09/17/00	7	194.3	14.32	0.47
09/19/00	9	151.1	14.32	0.47
09/20/00	10	136.0	14.32	0.47
09/25/00	15	90.7	14.34	0.49
09/30/00	20	68.0	14.34	0.49
09/35/00	25	54.4	14.35	0.50
09/40/00	30	45.3	14.35	0.50
09/50/00	40	34.0	14.35	0.50
10/00/00	50	27.2	14.37	0.52
10/10/00	60	22.7	14.39	0.54
10/20/00	70	19.4	14.37	0.52
10/30/00	80	17.0	14.37	0.52
10/50/00	100	13.6	14.30	0.45
11/10/00	120	11.3	14.34	0.49
11/30/00	140	9.71	14.33	0.48
12/00/00	170	8.0	14.31	0.46

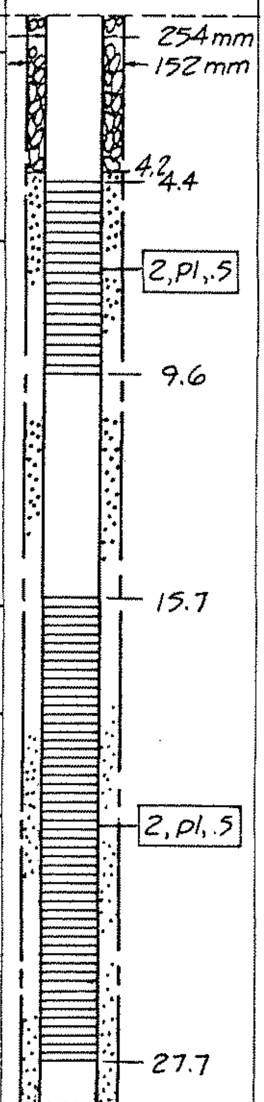
HYDROGEOLOGIC LOG

DRILLHOLE No. PUMPWELL
Sheet 1 of 1

Job No. 822-1071

Project: Slope Monitoring Haney
Type of drilling: Cable Tool
Rig: Bucyrus Erie
Drilling fluid: Water
Coordinates: E ..... N .....
Angle from horizontal: 90°
Bearing: – °Azimuth
Reference elevation: .....
Elevation type: surveyed ☐ altimeter ☐ from map ☐
Purpose of hole: Dewatering Test Well

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling (2) Depth (m)	(2)(4) Water Level (m)	(5) Water Flow (l/s)	(6) Other	After Drilling (2)(7) Water Level (m)	Permeability (8) (2) Depth (m)	Method	Value (cm/s)	Comments
Ground Surface										
Topsoil 1.0	254mm; 152mm					1.95				10 Mar. '83
Fine, SILTY SAND 6.1	4.2; 4.4									
Clayey SILT, occ. trace to some fine sand. 15.8	2,Pl,.5; 9.6; 15.7									
Fine SAND, trace silt. 18.6				MW						
Fine, sandy SILT. 21.3										
Clayey SILT trace to some fine sand. 28.92	2,pl,.5; 27.7									
End of Borehole										



Contractor: Norwest
Date started: 1 Mar. '83
Date finished: 9 Mar. '83
Logged by: D.B.
Checked by: PE
Date: .....

* NOTE: Bracketed numbers refer to notes preceding the logs.

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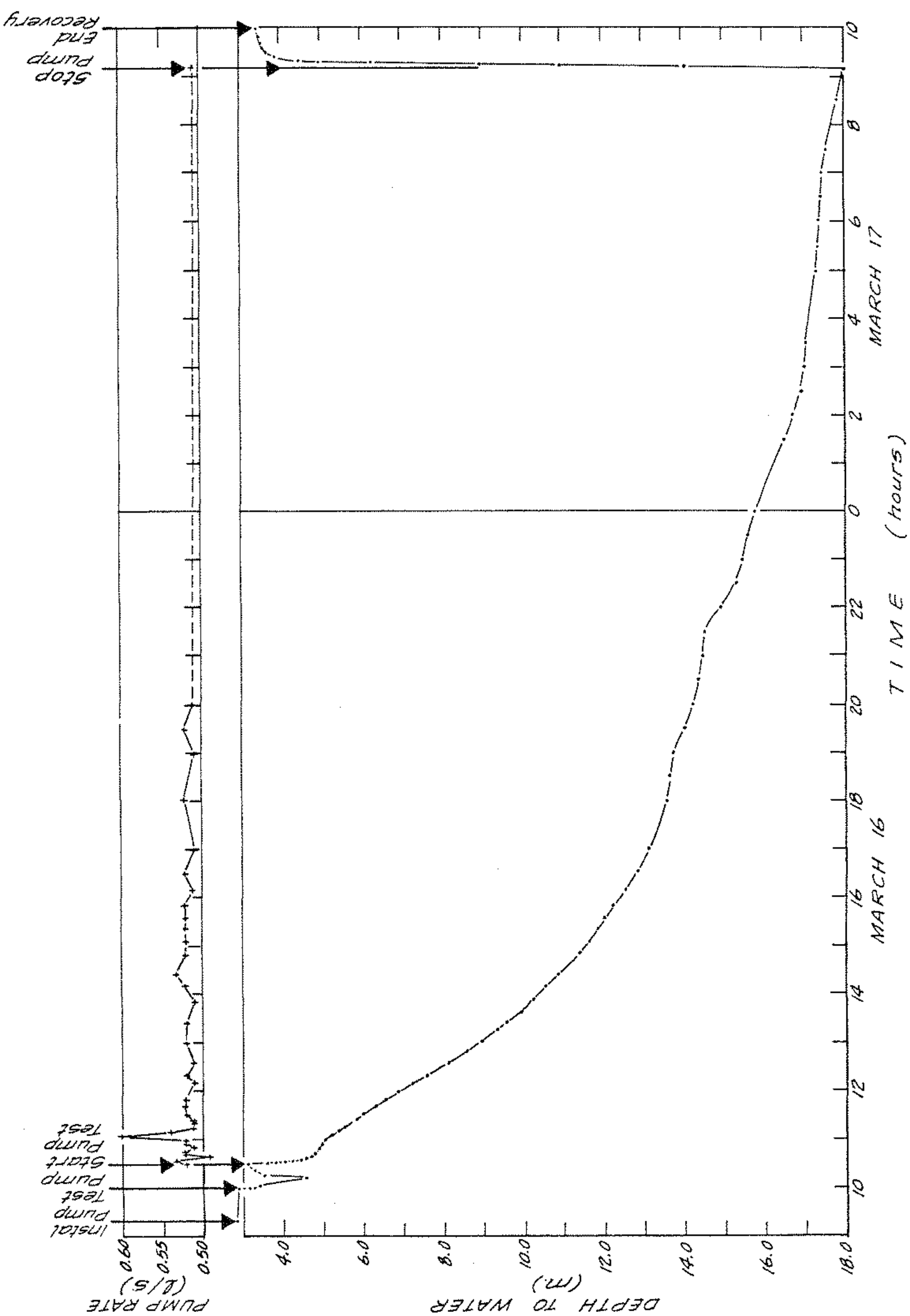
Scale: 1:200

PUMP TEST HYDROGRAPH
PUMP WELL

Figure C-1



PROJECT NO. 822-1071 DRAWN Mg. REVIEWED DATE Mar. '83

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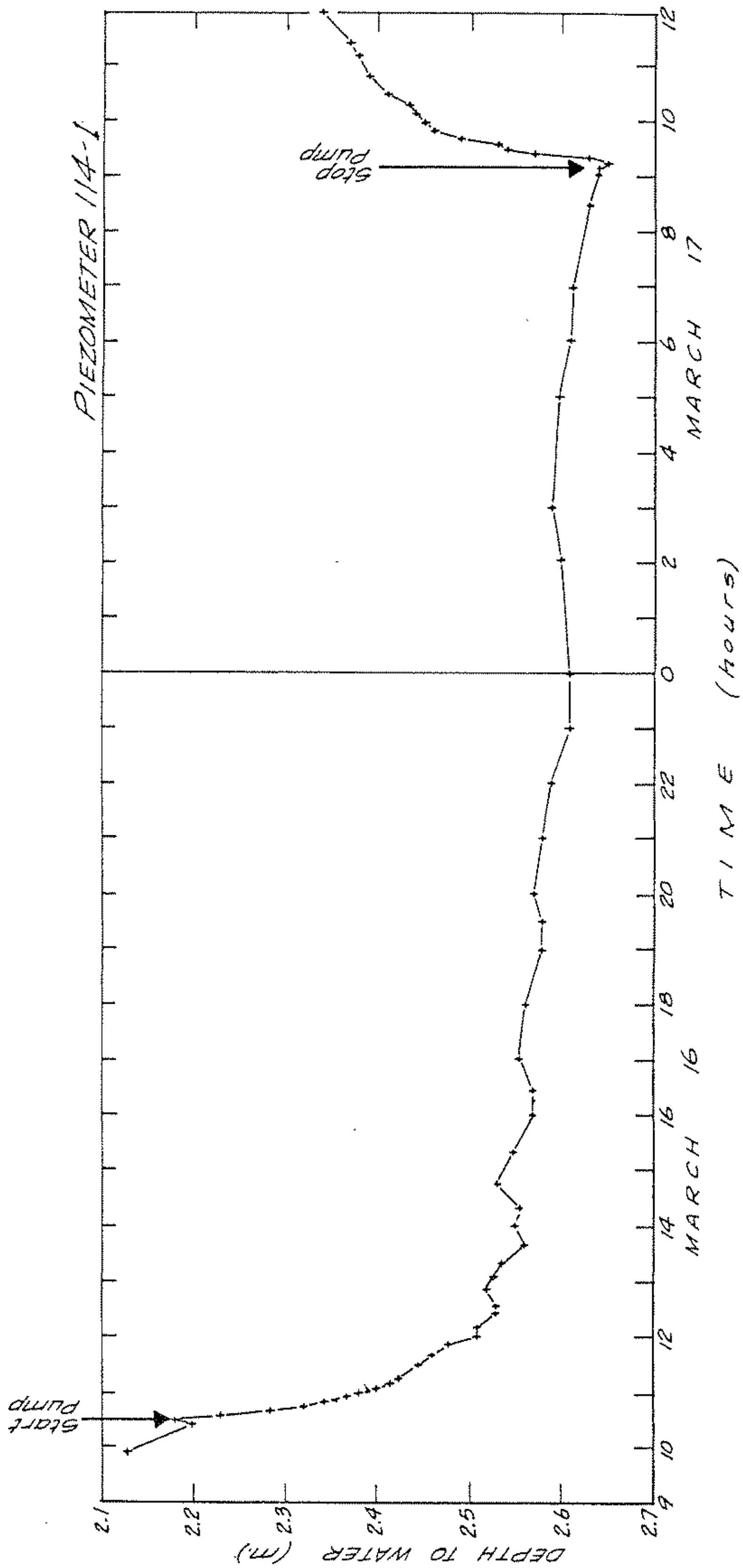
PUMP TEST HYDROGRAPH
PIEZOMETER - BH 114-1

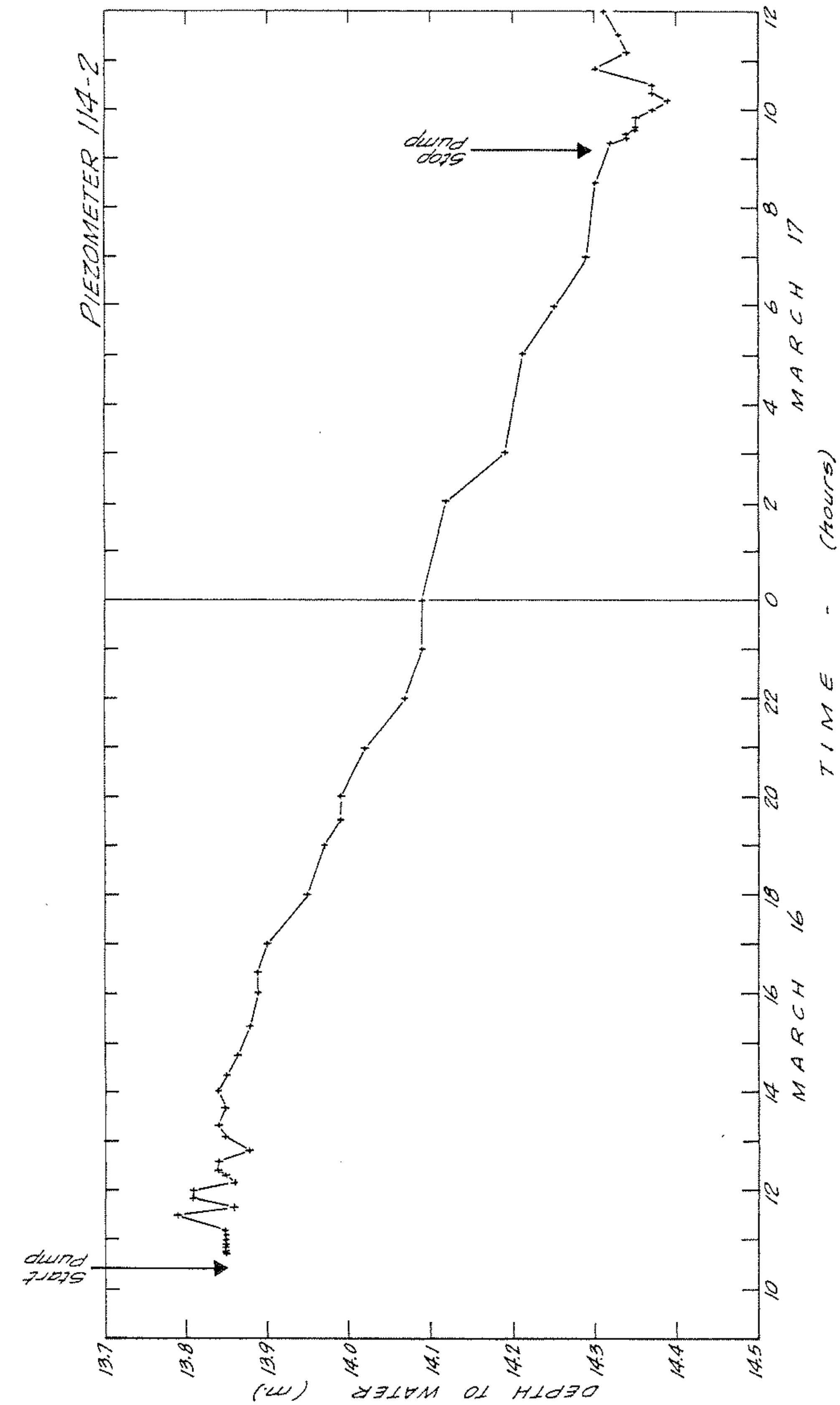
Figure C-2



PROJECT NO. 822-1071 DRAWN Ing. REVIEWED DATE Mar '83

PUMP TEST HYDROGRAPH
PIEZOMETER - BH 114-2

Figure C-3



PROJECT NO 822-1071 DRAWN lng REVIEWED ... DATE Mar '83

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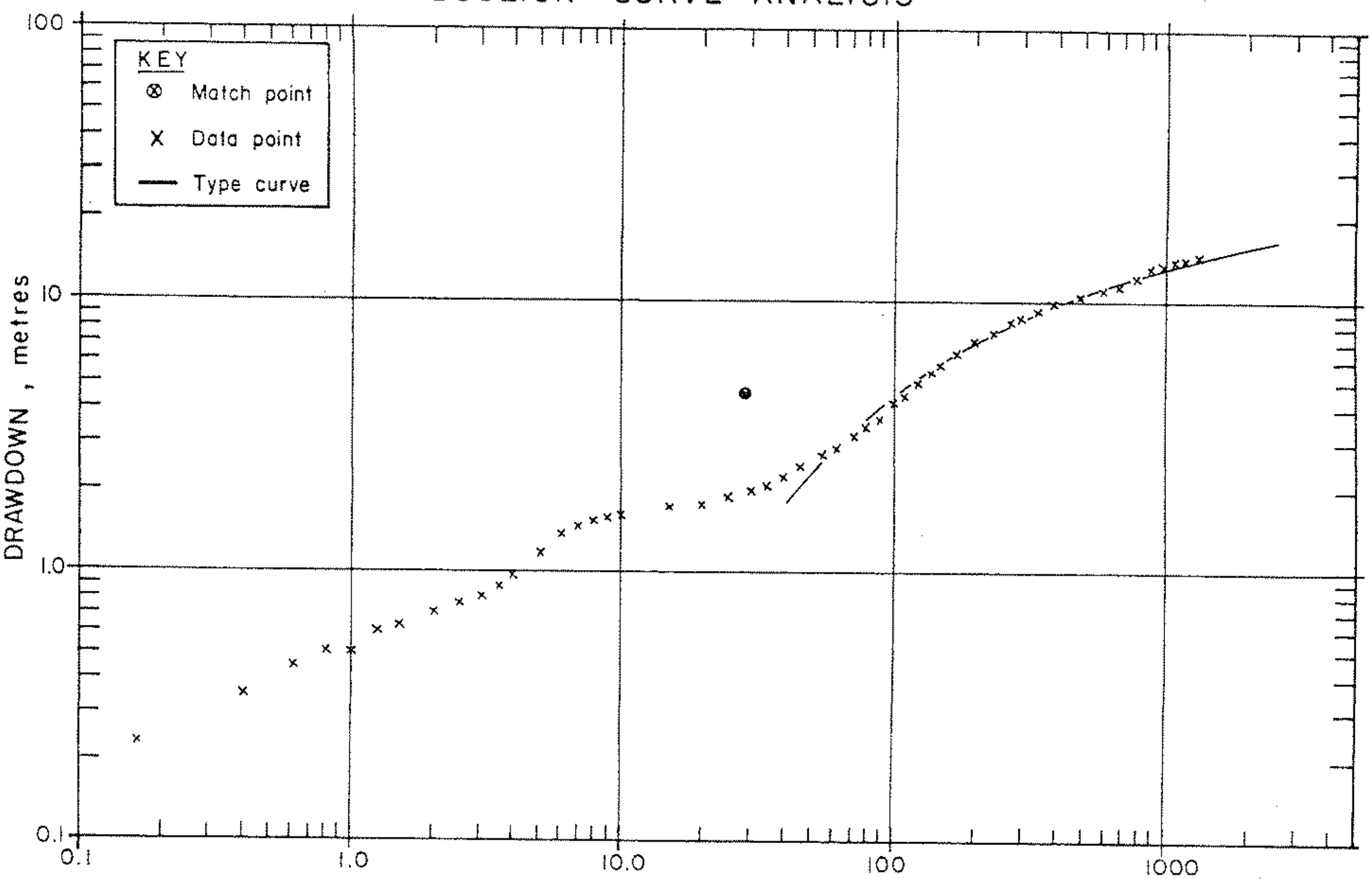
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 1

Well No. PUMPWELL Data observed in PUMPWELL

Figure C-4

HY-10

BOULTON CURVE ANALYSIS



CALCULATIONS:

EARLY TIME (DIFFICULT FIT - ONLY LATE TIME ANALYZED)

$$T = \frac{Q\,10^{-3} W(U_A, r/b)}{4\pi s} = \frac{(\quad)10^{-3}(\quad)}{12.57(\quad)} = \text{..........} \text{ metres}^2/\text{sec.}$$

$$S_A = \frac{240\, T t u_A}{r^2} = \frac{240(\quad)(\quad)(\quad)}{(\quad)^2} = \text{..........}$$

LATE TIME

$$T = \frac{Q\,10^{-3} W(U_Y, r/b)}{4\pi s} = \frac{(0.52)10^{-3}(1)}{12.57(4.7)} = 8.8\times10^{-6} \text{ metres}^2/\text{sec.}$$

$$S_Y = \frac{240\, T t u_Y}{r^2} = \frac{240(\quad)(\quad)(\quad)}{(\quad)^2} = \text{..........}$$

where:

- r = radius from pumped well 0 (metres)
- Q = pumping rate 0.52 (litres/sec.)
- T = transmissivity ...... (metres² /sec.)
- s = drawdown 4.7 (metres)
- t = time since pumping started - (minutes)
- S_A = storage coefficient (confined)
- S = storage coefficient (unconfined)

match point parameters from Boulton delayed yield type curves:

- $W(U_A, r/b)$ ....
- U_A ....
- $W(U_Y, r/b)$ 1.0
- U_Y 1.0
- r/b 3.0

PROJECT NO. 822/071 DRAWN .... REVIEWED I.K. DATE DEC.'81

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HY-I

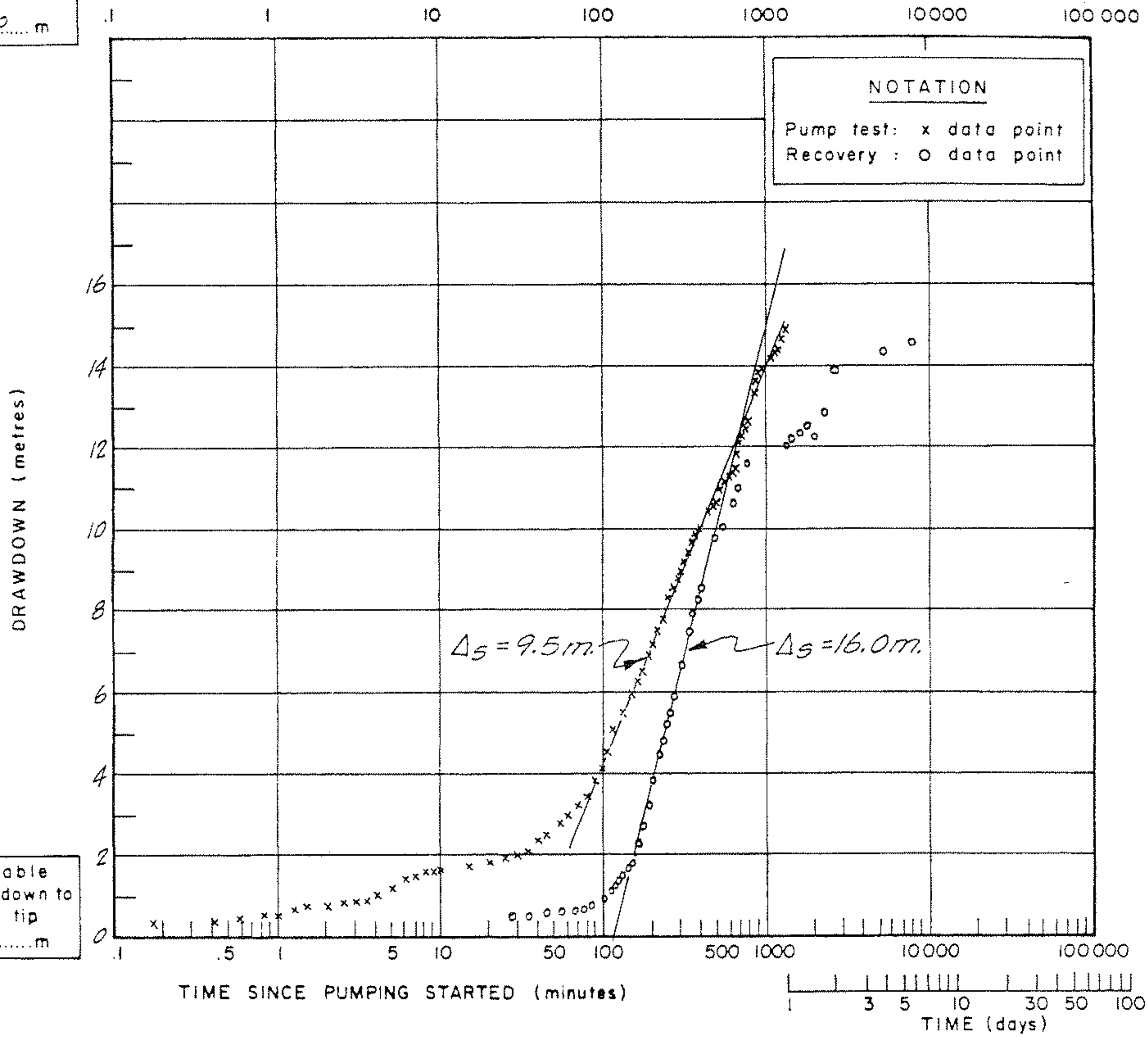
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 1

Well No. PUMPWELL   Data observed in PUMPWELL

Figure C-5

Depth to static water level 3.10 m

t/t' = Ratio of time since pumping started to time since pumping ceased.



Available drawdown to piezo tip ....m

CALCULATIONS

Leg no. 1 $T = \frac{1.83\ Q}{\Delta s \times 10^4}$

Leg no. .... $T = \frac{1.83\ Q}{\Delta s \times 10^4}$

Pumping	Recovery
$\frac{1.83 \times 0.52}{10^{+4} \times 9.5} = 1.0 \times 10^{-5}\ m^2/s$	$\frac{1.83 \times 0.52}{10^{+4} \times 16.0} = 5.95 \times 10^{-6}\ m^2/s$
$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$

$$S = \frac{135\ T.t_0}{r^2} = \frac{135\ (\quad)(\quad)}{(\quad)^2} = \ldots \times 10^{-}$$

$$t_{min} = \frac{.42\ r^2 S}{T} = \frac{.42\ (\quad)^2(\quad)}{(\quad)} = \ldots \text{ minutes}$$

WHERE r = Radius from pumped well 0 (metres)   Δs = Drawdown (metres per log cycle)

Q = Pumping rate 0.52 (litres/sec.)   T = Transmissivity (metres²/sec.)

t_0 = Time intercept for zero drawdown ...... (min.)   S = Storage coefficient (fraction)

t_{min} = Approx. minimum value for which u < 0.01

822-1071   Drawn   Mar '83

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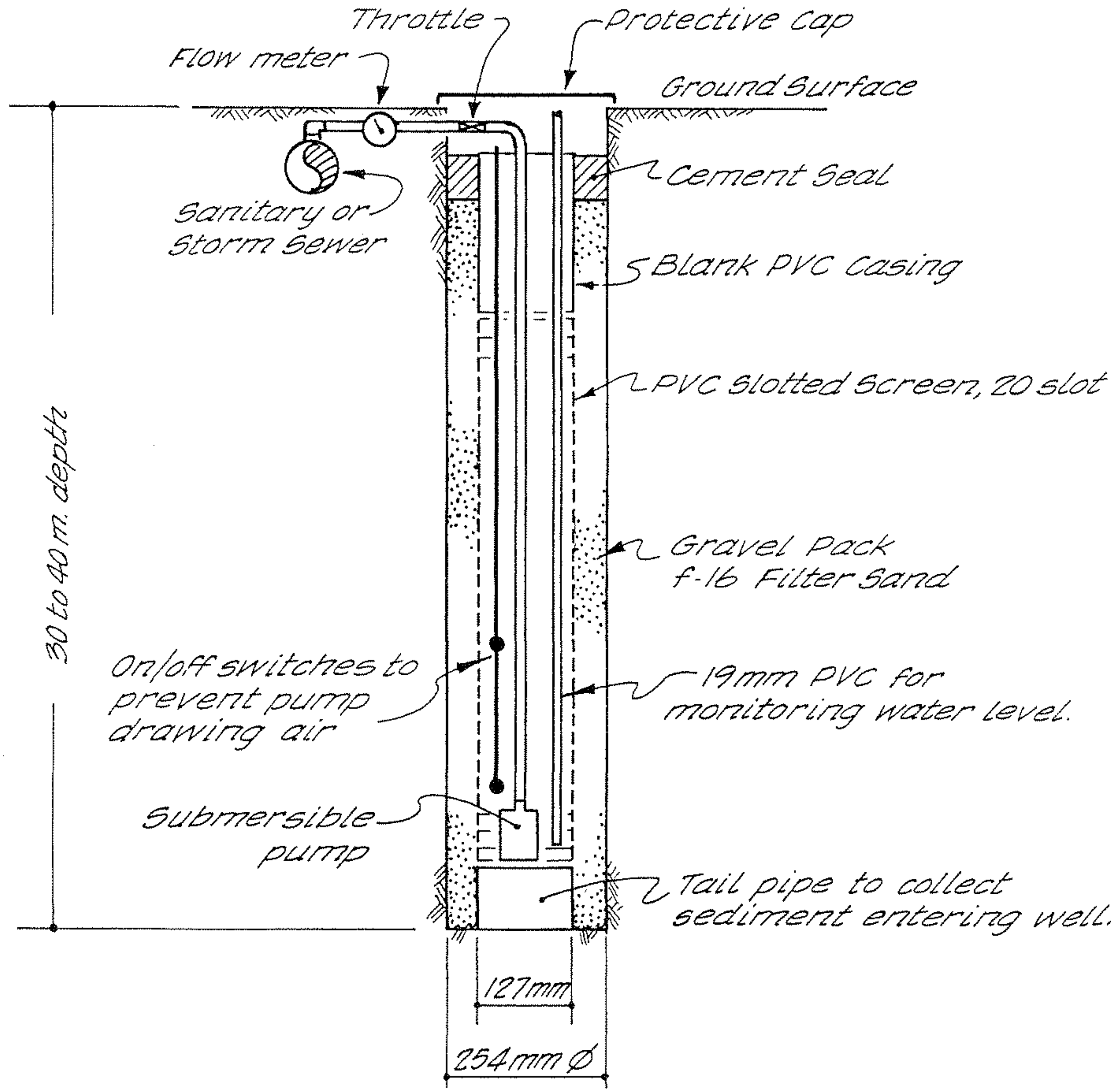
PRELIMINARY DEWATERING WELL DESIGN

Figure C-6



Schematic only - Not to Scale

PROJECT NO. 822-1071 DRAWN lmg REVIEWED T.B. DATE Mar. '83