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Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO B.C. HYDRO ON THE HAT CREEK PROJECT GEOTECHNICAL AND HYDROGEOLOGICAL UPDATE, FALL 1982

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SUMMARY AND CONCLUSIONS

SCOPE OF WORK

This report contains the results of the geotechnical and ground water studies carried out by Golder Associates at Hat Creek during 1982.

The work comprised the following:

- o Assessment of the geology on the east side of the pit in relation to the diversion tunnel investigation results.
- o Assessment of the structural data obtained from previous drilling to establish a zonation of the east side of the pit.
- o Reassessment of the geotechnical basis for the pit slope design.
- o Assembly and reassessment of all the ground water data accumulated since 1978.
- o Execution of a geophysical survey to investigate the depth of the surficial deposits in the northeast of the pit.
- o Ground water exploration by drilling to assess the deep glacial deposits of the northeast buried channel.
- o Reworking of the 1978 estimates of ground water inflow to the 2240 MW Pit and assessment of the inflow to the 800 MW Pit; dewatering designs.

GEOLOGY OF EAST SIDE OF PIT

Data acquired from the tunnel investigation to the east of the pit [Golder Associates, 1982(A)] has permitted a clearer understanding of the geology of the eastern escarpment, but has only assisted in the understanding of the geological relationship between those rocks and the Medicine Creek Formation to the west to a limited extent. However, the rock mass strength of the escarpment indicates that the rocks should not pose a hazard to the proposed 2240 MW Pit which would be excavated at flat slope angles.

Structural data acquired from previous drilling has been analyzed and an attempt has been made to zone the east side of the pit within the limitations of the data.

GEOTECHNICS

A re-appraisal has been made of some of the geotechncial aspects of the project which required further clarification. These included: rock strength, seismic analysis of the waste dumps, pit slope stability and a comparison with the Panama Canal slopes.

A complete re-analysis was carried out on all the triaxial tests performed by Golder Associates on the claystone/siltstone sequence. The trends indicated in the previous reports were demonstrated much more clearly. Two strength envelopes can be drawn: for the brecciated samples $c' = 0$ MPa, $\phi' = 16°$; for the structureless samples $c' = 0.38$ MPa, $\phi' = 20°$. When the proportion of these materials can be assessed in any particular slope within the pit, its stability can be computed more reliably than hitherto.

The stability of the Medicine Creek waste dump has already been analyzed under seismic loading using pseudo-static stability analyses; this report contains the results of similar studies on the Houth Meadows dump. The lowest static factor of safety using conservative assumptions is 2.08. A factor of safety of 1.0 is achieved with a horizontal earthquake acceleration of 0.05 g, assuming liquefaction of foundation silts. Such silts would need to be removed if shown to be present. Analyzing the dump for displacements by the Newmark method, using an acceleration as above, a downstream movement of 0.6 to 1.0 m could be expected. By comparison with the behaviour of El Infiernillo and La Vallita Dams in Mexico under loading imposed by a magnitude 7.6 event, the Houth Meadows retaining embankment should suffer acceptably small displacements for an event of that size.

An independent evaluation of Golder Associates' geotechnical work has been carried our by Professors P. Rowe of Manchester University and N. Morgenstern of the University of Alberta. Both have endorsed the approach taken, have largely agreed with the conclusions and have made recommendations for the future.

The report reiterates the factors upon which the geotechnical design of the slopes is based and recommends a careful flexible approach to excavation. Due attention must be given to the geology, material strength, ground water conditions and rate of excavation.

Analogies are drawn with the experience of the Panama Canal from the benefit of a visit to the slopes there.

GROUND WATER

The piezometric data accumulated over the period 1976-82 has been put onto computer file to facilitate future use. Piezometer hydrographs have been plotted and values for hydraulic conductivity recalculated where it is apparent that stabilization had not yet occurred. Revised piezometric contours have been drawn for bedrock and surficial deposits. Abnormally low piezometric levels in two piezometers close to the burn zone probably indicate negative pore pressures developed on unloading of the area by burning. The piezometric head distribution remains largely unchanged from 1978 but the heads are slightly lower in some cases.

An exploration program in the buried valley in the northeast of the pit area was carried out by geophysical survey and drilling. It was shown that the glacial deposits infilling that valley had hydraulic conductivities in the range of 1.0×10^{-7} to 9.0×10^{-7} m/sec. For this reason, screened wells were not installed for test pumping as planned.

Based on the re-evaluation of the hydrogeological parameters, bedrock inflows to the 2240 MW Pit in Year 35 are anticipated to be in the range of 1.7×10^{-1} to 1.25×10^{-5} m^3/sec; surficial inflows to the same pit would likely be approximately 5.7×10^{-3} m^3 /sec. Inflows from the surficials would be reduced from that calculated in 1978 due to the absence of seepage from the previously proposed diversion canal and the lower recorded permeability from the northeast area. A revised mine dewatering arrangement is presented.

For the 800 MW Scheme, 35-year bedrock inflows are anticipated to be in the range of 3.2×10^{-2} to 2.4×10^{-6} m^3/sec and total surficial flows 3.4×10^{-3} m^3/sec. A mine dewatering plan is presented.

High transient inflows are likely, but would probably be of short duration. They would most likely be associated with faults or closely jointed zones which are difficult to predict.

FURTHER WORK

Most of the further work required for design would be carried out in the early phases of excavation when good exposures would be available. However, it has been recommended by Professor Rowe that detailed testing before design be carried out on large diameter samples; these could be obtained from adits or large diameter auger holes. When the project activities are resumed, it is recommended that consideration be given to this appraoch.

We thank you for the opportunity of carrying out these further studies on the Hat Creek Project. We have pleasure in submitting this final report.

G.E. Rawlings

G.E. Rawlings, P. Eng.

N.A. Skermer, P. Eng.

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1.0 INTRODUCTION

1.1 General

The geotechnical and hydrogeological update program was undertaken to reassess the data on the Hat Creek No. 1 Coal Deposit (Figure 1) prior to the final engineering design of the 2240 MW Scheme.

During the implementation of this program, the Hat Creek Project was delayed due to reduced load growth forecasts. This resulted in the planned program being cut due to budget constraints imposed by B.C. Hydro.

The following work was planned; those sections actually completed are identified:

- Geophysical survey - completed
- Ground water exploration - 50 per cent completed
- Ground water monitoring wells - design largely completed; contract preparation and installation not carried out
- Geological, hydrological and geotechnical reassessment - completed as far as possible within reduced budgets
- Excavation of clay trench (F) - eliminated

After the initial planning of this work for the 2240 MW Scheme, it became necessary to consider the geotechnical, hydrogeological, and, in some cases, environmental aspects of a reduced development, the 800 MW Scheme. Separate budgets were established for some sections of this work and they have been reported on separately (i.e. ash dump seepage; pit drainage; creek diversions; seismic analysis). Other aspects of the 800 MW Scheme are included in this report.

1.2 Objectives of 1982 Field Studies

The objectives of this study as set out at the proposal stage were as follows:

- Reassessment of the dewatering aspects of the pit including a risk analysis of potential inflows
- Assessments of the movements which might develop in the East Pit Slopes and the potential risk to diversion structures sited close to the pit
- Planning for the future location of a trench (F) into the Medicine Creek Formation claystone (subsequently deleted)
- Assembly of the geological and geotechnical data collected to date for slope and embankment stability purposes and consideration of the role of structures in slope stability.

1.3 Methods of Work

Figure 2 shows the various activities planned for the updated program. Those activities which were deleted or reduced in scope are identified.

The program of work actually carried out was as follows:

- Assembly and assessment of all the ground water data accumulated since 1978. This primarily related to the routine piezometric measurements made by BCH but it also included the results of the Construction Water Supply Investigation (Golder Associates, 1982A)
- Execution of a geophysical survey by Geo-Physi-Con to investigate the depth of surficial deposits in the northeast of the pit and to establish, where possible, the location of faults

- Consideration of various designs of monitoring well for staged installation in the early phases of the project
- Letting of contracts for ground water exploration in the north and northeast of the pits (curtailed to 50 per cent)
- Assessment of the geology on the east of the pit, the tunnel investigation results and the results of the ground water field work to determine the long-term stability of the East Pit slopes
- Rework the 1978 estimates of pit inflows
- Assess the structural geological data obtained from current and earlier coring programs in the east side of the pit
- Produce a structural zonation of the pit where applicable
- Consider the implications of the structural data for pit slope stability.

Some of the planned programs suffered due to the curtailment; most were completed satisfactorily, however. It was possible to reorientate some of the work to good advantage. For example, use was made of two external geotechnical consultants, Professor N. Morgenstern of the University of Alberta and Professor P. Rowe of the University of Manchester in the review of the previous geotechnical work. Mr. G. Rawlings was able to extend a visit to Panama for other purposes in order to spend time examining the geotechnical problems of the Panama Canal.

1.4 Work Carried Out Since 1978 Studies

Mining geotechnical work on the project has generally been sparse in the period 1978/82. The main programs carried out at the site during that period were the following:

- Power plant investigation (Klohn Leonoff Consultants)
- Diversion canal and dam investigation (HEDD)

- Tunnel diversion investigation (Golder Associates)
- Construction water supply investigation (Golder Associates)
- Seismicity study and capable fault investigation (Klohn Leonoff Consultants)

Of these studies, the power plant investigation has provided little data of use in this update report. The canal diversion investigations have yielded some peripheral information and the tunnel diversion has provided some important geotechnical results. The construction water supply study reached some interesting conclusions which have been used in this work. The analysis of the seismicity at Hat Creek has interfaced with the design of the engineered structure in and around the pit; this is considered further in the body of the report.

1.5 Acknowledgments

We wish to acknowledge the help of BCH during this study and, in particular, Dr. G. Lange, Mr. W.G. Fothergill, Mr. H. Kim, and Mr. S. Ridley who have provided much assistance.

2.0 GEOLOGY

2.1 General

The geology of pit slopes has not been reassessed to the extent originally anticipated. It was envisaged that the results from a large scale excavation in the claystone would be able to be extrapolated to the other slopes likely to be excavated in the same formation by means of a re-examination of the core and a reassessment of the earlier test results. As the claystone excavation was unable to be carried out, this exercise was not possible. However, a reappraisal was made of the structural data already accumulated from the eastern pit slopes. In addition, the data obtained from the diversion tunnel investigation through the eastern escarpment has been assessed in relation to the proposed slopes.

2.2 Geology of the East Side of the Pit

The report on the Hat Creek Diversion Alternatives (Golder Associates, 1982) describes the geology to the east of the 2240 MW Pit margins as deduced from the 1982 field investigations. The detailed studies did not extend beyond the proposed tunnel portal areas.

The east and northeast areas of the pit were investigated by geophysical survey (Geo-Physi-Con, 1982) to define the extent of the buried glacial channel. It was anticipated that data would also be accumulated on the bedrock materials. Although it proved possible to identify the coal sequence, it did not prove to be possible to locate the eastern limit of the Medicine Creek Formation or position a boundary fault (if it is present). The bedrock surface was well defined.

The relationship between the Medicine Creek Formation as seen in DDH 77-815 and the interbedded claystones, sandstones and conglomerates seen further east in DDH 77-816 and DDH 75-36 is obscure. A boundary fault may not be necessary, a facies variation seems more likely. The presence of a well-rounded basal conglomerate above the andesitic sequence in DDH 77-816 supports the view that the Medicine Creek deposits are overstepping andesitic rocks of Coldwater Formation age or earlier. However, it seems much more likely that a boundary fault must separate these rock types further south in the vicinity of DDH 78-839 and DDH 78-841.

2.3 Geological Structure

The role of geological structure in slope stability of Hat Creek is a much argued question. It is known that it would be an important factor on the bench scale, its role in the overall pit slope stability would depend on the attitude and continuity of the structures. Moreover, the concept of depressurization is heavily dependent on the permeability of the ground which is related to structure.

A reasonable assumption can be made of the structure in the pit slopes for the Hat Creek Coal Formation and the Coldwater Formation because bedding is generally recognizable in those rocks. Bedding cannot generally be recognized in the Medicine Creek Formation although discontinuities are extensively recorded in the core. In order to make an estimate of how the structures in the Medicine Creek claystones might dip, histograms have been drawn for the structural data recorded in cores from holes drilled on the east side of the pit which encountered the claystones.

This data has been assembled on computer file and a program written to provide histograms of various selections of data. The histograms of significance are discussed in Section 2.4

Structures of importance to overall slope stability (i.e., excluding local bench failures) which could constitute failure planes must dip at angles less than the slope angle. The slope angle currently recommended for use in those materials is 20 degrees; it is likely to be increased in practice if it can be demonstrated that steeper angles are feasible. Structures oriented out of the pit slopes with angles between the residual friction angle and the slope angle are of significance. The difficulty in considering such flat angles is that they partly fall into the same range of angles as core breaks produced during drilling. During coring in such weak materials, rotational breaks develop at angles nearly normal to the core axis, these can easily be confused with low angle bedding or joints. For this reason, when manipulating the structural data, where there is a preponderance of readings in the range 80 to 85 degrees or 85 to 90 degrees (angles measured with reference to the core axis) they have been omitted. In addition to providing a misleading picture of the joint angles, a large number of drilling breaks also serves to obscure concentrations of other data at steeper dips (e.g., see plots of DDH 76-801).

Without oriented core, it is not possible to measure, or estimate, dip directions. During the first geotechnical field program in 1976, attempts were made to orient the core using the Christensen-Hugel core barrel and the Craelius Core Orienter. Neither proved to be successful because of the weakness of the rocks.

It is therefore apparent that without good rock exposures to provide control, deductions on the role of structure in the pit slopes is at best vague. If in later years, but prior to opening up the pit, it is decided that further structural data is required in the Medicine Creek claystones, it is recommended that one of the following methods of investigation be considered: a large open trench, a large diameter bucket-augered hole, or an adit. Any of these methods would yield structural data and permit large diameter sampling of materials below the phreatic surface.

2.4 Discussions of the Histograms

The histograms of discontinuity dip values for the east side of the pit are shown on Figures 3 to 17. A summary of the concentrations of data is given on Table 1.

It is apparent that some trends exist. In the southeast of the pit area, very similar concentrations appear in DDH 76-801, DDH 76-821 and DDH 77-841. The results from DDH 78-867 show some features in common. It is likely, therefore, that this could represent a structural block with similar dip or joint orientations throughout. If the major dip concentration of 16 to 20 degrees were related to discontinuities dipping towards the northwest, it could have considerable implications for pit slope instability in that area.

Major concentrations in DDH 76-815 and DDH 77-843 are similar but are steeper and are less likely to be of prime significance.

Concentrations in DDH 77-846 and DDH 78-870 in the east to northeast sector of the pit are evident, but the data is not plentiful. The high incidence of steeply dipping joints in DDH 78-870 may be due to the proximity of the Finney Fault in that area.

Figure 18 shows the areas in the east pit slopes where structural dips are similar and they may be interpreted broadly as structural zones. Further confirmation would be required, however, since dip directions are lacking.

TABLE 1

Summary of Discontinuity Dip Values

Drillhole	Concentrations Measured to Core Axis (Dip values)			Comments
DDH 77-841	70 - 74 (16 - 20)	58 - 62 (28 - 32)		Similar to DDH 76-801, DDH 76-821
DDH 77-843	40 - 44 (46 - 50)	50 - 54 (40 - 36)	60 - 64 (30 - 26)	Could be close to faults which offset Finney Fault. Similar to DDH 76-815
DDH 76-801	70 - 74 (16 - 20)	56 - 64 (26 - 34)		Similar to DDH 77-841, DDH 76-821
DDH 76-821	70 - 74 (16 - 20)	56 - 60 (30 - 34)	40 - 44 (46 - 50)	Similar to DDH 76-801, DDH 77-841
DDH 78-867	76 - 80 (10 - 14)	66 - 70 (20 - 24)	56 - 60 (30 - 34)	Some similarity to DDH 77-841
DDH 76-815	40 -50 (40 - 50)	56 - 60 (30 - 34)	76 - 80 (10 - 14)	Main concentration similar to DDH 77-843
DDH 76-816	50 - 64 (26 - 40)	70 - 74 (16 - 20)		Disregard angled hole
DDH 78-870	6 - 10 (80 - 84)	56 - 64 (26 - 34)	70 - 74 (16 - 20)	Also 20 - 30 (60 - 70). Could be close to Finney Fault considering concentration of steeply diping joints
DDH 77-846	56 - 64 (26 - 34)			Only limited data available.

3.0 GEOTECHNICS

3.1 General

Reappraisal of the mining geotechnical aspects of the Hat Creek Project has been concentrated in areas, namely:

- Rock strength
- Seismic analysis of waste dumps
- Review of pit slope stability
- Comparison with Panama Canal slopes

Although no additional rock strength data has become available, it was considered useful to reappraise the previous interpretations, especially with the benefit of an external consultant's comments, to see if they could yield any further information on the behaviour of the materials.

A detailed analysis of the Medicine Creek Waste Dump stability under seismic loading has been reported on elsewhere and is attached herewith as Appendix A. Due to the much greater importance placed on the Houth Meadows Dump by the 800 MW Scheme (no Medicine Creek dump would be required for this), the effect of seismic loading on that structure has also been analyzed and is included in this report.

Some analysis of the structural data collected from cored holes in the east pit slopes in earlier years has been analyzed (see Section 2). It is considered in this section with regard to its role in slope stability.

The final part of this section of the report brings together these various geotechnical aspects of the 2240 MW pit together with the external consultant's comments and the review of the Panama Canal slopes made by G.E. Rawlings in February 1982 during a field visit, to produce the final pre-design appraisal of the pit slope stability. A brief review of the 800 MW pit slopes has already been made and this is included as Appendix B.

3.2 Rock Strength

3.2.1 Coal

No further testing of the coal was carried out, but a re-analysis of the 1977 data was made by calculating stresses on the failure planes of the specimens. The results tended to show a curvi-linear strength envelope but, within the stress range under consideration, it could be closely approximated by an envelope represented by $\emptyset'$ = 40 degrees and cohesion c' = 15 psi (0.1034 MPa). This data was used in the analysis of the proposed 800 MW pit slopes (see Appendix B).

3.2.2 Claystone/Siltstone

A detailed re-analysis of all the triaxial tests that had been performed on the claystone/siltstone sequence in both the 1976 and 1978 laboratory testing programs (Golder Associates 1977, 1978) was performed. Doubtful test data was discarded. Only tests on which the specimens had failed along clearly identified failure planes were re-examined. Stresses along these failure planes were computed and the results were plotted in the form of shear strength, normal strength envelopes as shown on Figure 19. Both drained and undrained triaxial compression test data was analyzed. Brecciated samples were differentiated from the structureless specimens. The results indicate that the strengths of these materials are independent of the material type, i.e. whether the samples are from the Coldwater Formation or the Medicine Creek Formation. This conclusion was arrived at previously and is covered in the Golder Associates report, 1978. A few samples sheared along planar discontinuities and these stand out from the mass of the results, e.g. in the 1976 program, a sample of the Medicine Creek Formation failed along a smooth slickensided failure plane close to residual shear strength. Similarly, in the 1978 program, two further planar shears occurred.

The main conclusion from the analysis is that two clearly defined strength envelopes can be drawn, one for the structureless 'intact' material, and another for the brecciated material. The strength envelopes can be closely approximated by an angle of friction of $\phi' = 16$ degrees and zero cohesion for the brecciated samples, and $\phi' = 20$ degrees and a cohesion of 55 psi (0.38 MPa) for the structureless samples. Depending upon the distribution of these materials within any one particular pit slope, this data would provide an improved basis for reassessing the stability of that particular section of the pit. Future work on the strength parameters should take into account the considerations of Professor P.W. Rowe as discussed in Section 3.4.

This largely accords with the conclusions of the 1978 study in which the lower bound or envelope shown on Figure 19 was selected on a more judgemental basis. This reanalysis substantiates that work.

3.3 Seismic Analysis - Waste Dumps

The stability of Medicine Creek waste dump was analyzed under earthquake loading using pseudo-static stability analyses. The work was carried out in 1981 and it was reported by letter to B.C. Hydro on November 16th, 1981, see copy attached in Appendix A. It was concluded that for an acceptable factor of safety of 1.5, the seismic coefficient that could be tolerated would be 0.11 which would be comparable to about a magnitude 6.5 earthquake. This was for the maximum volume dump proposed in Medicine Creek as outlined in our 1978 report.

Subsequently, further analyses were carried out on the Houth Meadows waste dump using the same techniques. The analysis was computed using Sarma's method (Geotechnique, 1973). The following results were obtained.

(a) The lowest static factor of safety was 2.08. This assumes a massive slip surface through the base of the waste and that the foundation silts had liquefied for a distance of 1200 m behind the sand and gravel retaining embankment. The other strength parameters are as assumed for the Medicine Creek waste dump except that beneath the sand and gravel retaining embankment the foundation strength parameter was assumed to be $\phi' = 27$ degrees and $c = 0$. These are very conservative assumptions.

(b) A factor of safety of 1.0 was reached at a seismic coefficient of 0.053, i.e. a horizontal earthquake acceleration of about 5 per cent gravity. Such a condition would correspond to something less than a magnitude 6 earthquake. However, the analysis was carried out for the maximum volume waste dump in Houth Meadows, which has a crest elevation of 1005 m. It also assumes liquefaction of the foundation silts. If the foundations did not liquefy, the factor of safety of the embankments would be similar to that calculated for the Medicine Creek dump. Further investigation of the embankment foundations would be needed at the design stage. If it were established that silts and fine sands of a sufficiently low density such that they could liquefy were present, over-excavation of the foundation might be necessary.

Pseudo-static analyses are only an indication of the stability of an embankment under transient loading. Displacements during an earthquake are a more important measure of embankment behaviour. The displacement method of analysis was outlined by Newmark (Geotechnique, 1965). Applying his method to these embankments, assuming a maximum acceleration of 0.5 g and a velocity of 30 inches/sec, we have calculated that the embankment might shift in the downstream direction in the order of 0.6 to 1.0 m. This could be associated with small vertical crest slumping. Clearly, these orders of displacement would be acceptable unless further studies show that the severity of earthquakes assumed at Hat Creek are likely to be much larger in the distant future.

Useful information on the behaviour of high embankments of similar design under earthquake loading is provided by the displacements measured at the El Infiernillo and La Vallita Dams in Mexico during an earthquake on March 14th, 1979. The magnitude of the earthquake was 7.6, the epicentral distance was 87 km to El Infiernillo Dam and 108 km to La Vallita Dam. Both dams are high rockfill structures composed of dumped rockfill outer shells, compacted sand and gravel inner zones, and clay cores. Both structures retain water and are founded on alluvial soils. Due to earthquake shaking and slumping of the crests of the dams, the observed free board losses were 13 cm and 5 cm at El Infiernillo and La Vallita Dams, respectively. At El Infiernillo, the side slopes of the dams were 1.75 horizontal to 1 vertical and at La Vallita, the side slopes were 2.5 horizontal to 1 vertical, the same as proposed for the waste dump retaining embankments at Hat Creek. Extrapolating this behaviour, therefore, it would seem that even if the clay waste behind the retaining embankment were to liquefy completely in an earthquake, the retaining embankments ought to suffer acceptably small displacements for at least a magnitude of 7.5 earthquake. These are tentative conclusions and a more refined analysis should be undertaken during the final design stages of the project.

The 1982 studies by Klohn Leonoff on the presence of capable faults have not indicated any potentially damaging structures (personal communication) at the site. The seismic monitoring station established in 1981 is continuing to record.

3.4 Review of Pit Slope Stability

3.4.1 Planned Program

At the outset of the 1982 field program, Golder Associates were requested by BCH to review the question of pit slope stability particularly with relation to ground water control and the concept of depressurization by excavation, see Volume 1, Section 6 and Volume 6, Appendix 15 of Golder Associates report (1978). We responded in discussions at various meetings held with B.C. Hydro and by letter during the period January to March, 1982.

The basis of our proposals was threefold:

(a) That pit slope stability issues should be reviewed by independent outside soil mechanics consultants, Professor N.R. Morgenstern of the University of Alberta and Professor P.W. Rowe of the University of Manchester, England, both of whom are acknowledged authorities on shear strength and stability aspects of clays and shales.

(b) That the concept of depressurization should be tested by excavating a large excavation into the claystone of the Medicine Creek Formation and measuring the pore water pressure response using prior installed piezometers located beneath the excavation.

(c) That the in situ structure of the Medicine Creek Formation should be examined either in an excavation in the valley bottom, or in a large diameter auger hole of a diameter sufficient to allow access, or via an exploratory adit. Samples of the formation to be taken for shear strength testing. Re-analysis of existing strength test data to be undertaken.

Because of budget restrictions only a limited amount of work was carried out in these areas. A preliminary discussion was held with Professor Morgenstern on January 24th, 1982, in Edmonton; his opinion after a limited briefing was that future efforts might be directed toward obtaining a more detailed picture of the geology of the various formations particularly in the siltstone/claystone sequences. He referenced useful exploration work that was being carried out in the claystones of the Bearpaw Formation in Alberta, see Kaiser, Mackey and Morgenstern (1982). It was intended to continue discussions with Professor Morgenstern in persuing proposed item (c) above. However, as it was decided subsequently not to consider the excavation of test shafts or adits, no further discussions were held and no reports or letters were issued by Professor Morgenstern in connection with our discussion.

However, despite the cut backs in the scale of the work, we were still able to take advantage of Professor Rowe being in Western Canada during May 1982. After a site visit, discussions were held in Vancouver and he subsequently considered further data on the project in the U.K. The opinions of Professor Rowe are presented in Appendix C and are commented on in Section 3.4.2.

A detailed proposal for testing the concept of depressurization was presented verbally to B.C. Hydro on January 28th, 1982, and later by letter, but this proved to be too expensive at this juncture, and the work was postponed. The scheme was later reduced in scale and reintroduced to monitor the behaviour of coal trench "D" that was extended just into the Medicine Creek Formation in the faulted syncline at the west end of the trench. Eventually, only a visual examination of the very limited exposures of the Medicine Creek Formation was able to be made.

3.4.2 Comments of Professor Rowe's Report

Professor P.W. Rowe of Manchester University visited the site in May, 1982; he inspected cores, was shown around the trench excavations and examined the landslide areas. He was accompanied on his visit by N.A. Skermer and G.E. Rawlings of Golder Associates. Golder Associates geotechnical reports dated 1977 and 1978 were subsequently sent to him for review.

Rowe's assessment of our work is described in his letter report of September 3rd, 1982, enclosed as Appendix C. It will be seen that Rowe agrees that the concept of depressurization is correct, but the question at issue is how long that depressurization can be relied upon to assist in slope stability. He points to the presence of thin coal layers or partings within claystone deposits and suggests that these might have a much higher permeability than the surrounding claystone and therefore could lead to a reduction of negative pore pressures and sliding on such layers. The presence and the continuity of these layers ought therefore to be investigated in more detail in any future studies that are undertaken. With respect to the ground water conditions Professor Morgenstern pointed out in January that he would expect different pore pressures at different levels in the slope and that these would be controlled by structure. This is similar to Rowe's statement with respect to the pore pressures in the coal seams.

Rowe agrees that while mass strength of the material would control slope failures, discontinuities along which the water pressures could dominate might in turn affect stability. With regard to the strength of the materials, Rowe feels that for present feasibility purposes, the strength parameters have been adequately defined, at least as far as is practicable with these materials. However, he suggests that for final design and before mining operations start, test techniques could be developed which would suit the particular stress conditions of this site. The best way to do this might be to form a research program initially, and Rowe has outlined the type of studies that he would favour. The testing would be carried out very slowly and, clearly, such work would be better carried out by a university rather than a commercial testing organization. At a future date, we would recommend that consideration be given to the testing along the lines that he has suggested.

3.4.3 Summary of Pit Slope Stability Results to Date

Golder Associates have recommended pit slope angles for both the 2240 MW and 800 MW pit developments. Those angles represent average angles to which we consider the pits could be developed and are primarily for the basis of mine planning. At any particular location, the pit slope might be more or less than the overall angles currently recommended for a certain material. Detailed geological structures may give rise to instability and result in a flattening of the slope; elsewhere, stronger materials may lead to an overall steepening of the slopes. The stability in the pit would be an ongoing consideration as mining develops and the mine plan must be sufficiently flexible to cope with it.

The studies carried out by Golder Associates over the period 1975/82 on the Hat Creek Project, have demonstrated that pit slope stability would depend on the following aspects:

- geology - including rock material, degree of weathering/alteration/softening, structure (especially shearing or brecciation)
- material strength
- ground water conditions
- rate of excavation

The successive geological and geotechnical excavations at Hat Creek have collected sufficient data for the geology to be broadly described; the detail is still elusive because of the structural and lithological complexity. The geology, as known, is adequate for feasibility and early design studies. The detail can only be appreciated from actual excavations or large scale methods of investigation (adit, large diameter auger hole, trench, etc.).

The ranges of material strengths are now adequately bounded. Actual strengths in situ would be dependent on a knowledge of the detailed geology. There is much more to be known about the behaviour of the materials with time, under varying conditions of stress and in different ground water situations. Again, it will only be appropriate to carry out this work when large diameter samples are available and the representativeness of the samples can be appreciated by means of a large excavation. A material strength test research program should eventually be established to form the basis of a pit slope stability monitoring program.

Ground water conditions are paramount. It has been shown conceptually (Golder Associates, 1978) that the stability of the slopes in the short term (at least) would be dependent on the development of negative pore pressures on unloading. Professors Rowe and Morgenstern have substantiated this from their own knowledge of low permeability materials in actual slopes. The degree to which that process is a potent factor in slope stability is dependent on the rate of dissipation of these negative pressures, that is the rate at which the water could be sucked into the system to equalize the negative pressures and eventually to re-establish positive pore pressures. The rate of dissipation would be dependent both on the geology of the slope and the changes of permeability with time. Professor Rowe feels that the finer elements of the stratigraphy could dominate this process.

The rate of excavation is also of importance because rapid excavation and steep slopes would result in the development of high stresses in these slopes. Severe creep, creating shear planes at or near residual shear strength, could occur. Conversely, the material could dilate under the high stresses and develop high negative pore pressures producing greater stability. A program of further testing at the design stage, augmented by an observational approach on cut slopes, would be required.

It is apparent, therefore, that the stabiity of the slopes is not just dependent on the inherent properties of the materials forming the slope, but also on the sequence of excavation and the control of ground and surface waters.

It is agreed by all the experienced geotechnical engineers who have been involved with the Hat Creek Project that the geotechnical problems are of considerable interest and have great significance for the stability of a major excavation. Materials of high plasticity and low permeability, reflecting the high proportion of expansive clay minerals (montmorillonites) are not normally encountered in large open pit mines. Unfortunately, Hat Creek is without parallel in scale and complexity and hence analyses are tenuous. The projects which provide the best examples are the Panama Canal, the Centralia Mine in Washington State, and the foundations of the dams in Saskatchewan and the Mid-West U.S.A.; all have proved to be troublesome.

It must be emphasised that the open pit at Hat Creek is likely to abound in geotechnical problems. They are unlikely to be insuperable providing a planned, flexible approach is adopted in which due cognizance is given to the geotechnical aspects described here.

3.5 Panama Canal Slopes

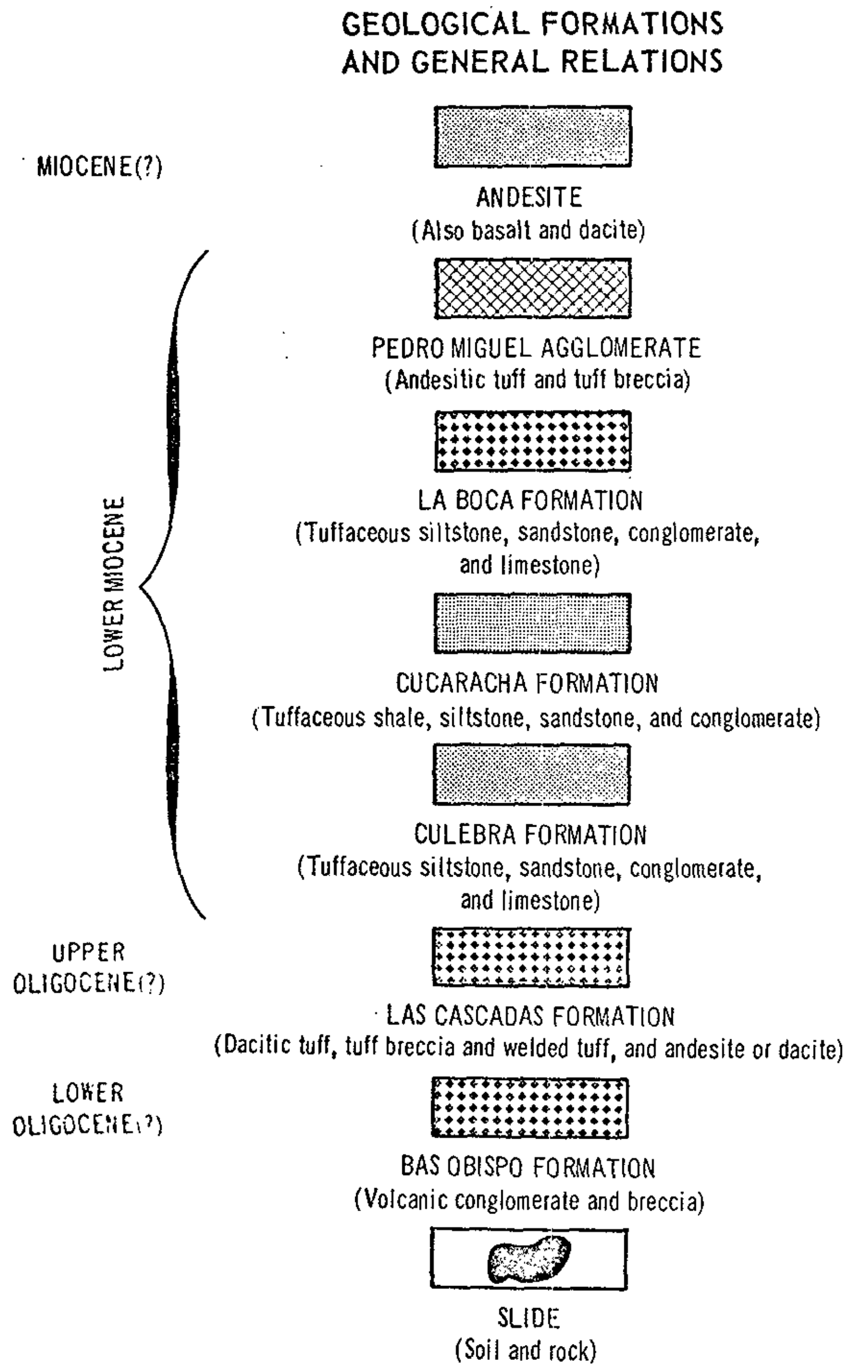
3.5.1 General

It has been noted in previous reports that the Panama Canal is the closest analogy that can be found in the literature to the proposed excavation at Hat Creek (Golder Associates, 1978). The rocks are of comparable strength and permeability; they are bentonitic in part; they are structurally disturbed; and they are overlain by a volcanic sequence. Because of the number of slides that have developed in those excavations both during construction and subsequently, there has been much published on the problem. As Golder Associates had a project in Panama in early 1982, one of the authors of this report (G.E. Rawlings) was able to take advantage of that situation to visit the canal. It was particularly required to compare the materials being engineered on the two projects and to see if there were techniques or approaches being used in Panama which could usefully be employed at Hat Creek.

3.5.2 Material Description

The generalized stratigraphic sequence is shown on Table 2. Slides have developed within most of the sedimentary or tuffaceous sequences, but they are considerably more common in the Cucaracha and Culebra Formations. Detailed stratigraphic sequences for those two formations are shown on Figure 20. The rocks in the canal cuts are now poorly exposed because they weather so quickly; the volcanic rocks remain as resistant bluffs. Cores were generally not available and would probably have been in a poor condition due to drying out. The best exposures were to be found in those areas currently being excavated (usually by scraper). Systematic geological descriptions of the materials are generally lacking. There is difficulty in reconciling the descriptions made by different people over the long history of the canal; Banks (1978) and Banks et al (1975) have tried to piece together the various data.

TABLE 2 - GENERAL STRATIGRAPHIC SEQUENCE OF PANAMA CANAL, GAILLARD CUT



The role of structure has been of particular importance to engineers trying to back-analyze the failures along the canal. Slickensiding is obviously present at some horizons, but appears to be more common closer to bounding faults. Slickensiding can also develop on sampling and care has to be taken in dealing with materials for testing.

The few sequences that were seen in exposures were highly stratified and bedding was the predominant discontinuity. The shales showed strong fissility in distinction to the largely structureless claystones of Hat Creek. According to the Panama Canal Commission (PCC) personnel, these sequences could be regarded as typical of the formations as a whole. The sequences appear to resemble the tuffaceous sequences of the Eastern Escarpment/Medicine Creek area rather than the Hat Creek basin itself.

3.5.3 Ground Water

Studies on the ground water have been made to a limited extent in the past. Piezometric data has been obtained and monitoring on a routine basis has been carried out. This has since been discontinued. Ground water control has been attempted primarily by the use of horizontal drains. Its success was never entirely proven and it has now been discontinued. Lime injection is no longer practiced. Currently, there is no systematic measurement of ground water for analytical purposes.

3.5.4 Slope Stability

(a) <u>Panama Canal</u>

The assessment and control of slope stability is now carried out on a pragmatic basis. Experience is that rapid failures do not occur, and hence monitoring to check on accelerating movements is a satisfactory procedure providing that the surveillance is regular. Simple, cheap methods are preferred. Poor-boy probes and EDM survey are the techniques relied upon.

Drainage is essential to the stability of the slopes (mean annual rainfall is 70 inches). The prevention of ingress of water by means of table drains behind the slopes and French drains across the slopes is preferred.

When accelerating movements are indicated, stabilization is effected by means of unloading by excavation of the upper slopes. Few other short term methods are available. It is not possible to back-analyze these failures with the data which is available. It must be recongnized, however, that many of the most troublesome areas have already been cut back to stable slopes over the long span of remedial works on the canal.

The following comments are made on the geotechnical surveillance of the Panama Canal slopes:

- PCC currently has two separate teams covering the geological and geotechnical aspects of the slopes. This is undesirable and impractical; the two disciplines should be integrated into one operation; the gaps in the data prove this.
- More structural data is needed, especially as it is considered that the failure mode is dependent on structure. Geophysical survey could be used to define the broad structural zones.
- More piezometers are required to define the piezometric pressures in the slopes.
- Terrestrial photogrammetry could be employed for monitoring and geotechnical mapping.

(b) Hat Creek

The movement monitoring techniques used on the Panama Canal could also be employed at Hat Creek. It is likely that a practical pragmatic approach would ultimately be used when the mechanisms of slope stability are fully understood. Quick and cheap methods are likely to be necessary.

Drainage has already been identified as being fundamental to the Hat Creek slopes. However, due to the large difference in precipitation between the two areas, the extent of local drains might be less at Hat Creek.

The experience at Panama of slides being slow-moving is useful data to apply to Hat Creek. This has been suggested by Golder Associates in the past and by Professor Rowe more recently; it is now substantiated in practice by the Panama Canal analogy. However, structural control would differ; there are likely to be more circular failures at Hat Creek. Whether large structurally controlled slides would also occur is an open question; with our present knowledge, it would seem to be unlikely but close surveillance would be required to verify this.

In summary, it can be stated that our knowledge of the Hat Creek geotechnics is at least as good as that on the Panama Canal, and in many respects (ground water for example) vastly better. Our knowledge of the geological structure for geotechnical analysis at Hat Creek requires more clarification which will only be available when large excavations are made. However, structure is not the dominant feature that it is on the Panama Canal.

4.0 GROUND WATER

4.1 Previous Work

The 1977 and 1978 Golder Associates' geotechnical reports on the Hat Creek Project presented details of the overall hydrogeology of the Hat Creek Valley. Generally, three hydrogeological units were recognized, the surficial deposits, the coal, and the Tertiary sedimentary rocks above and below the coal. Recent ground water exploration associated with the drilling of wells for construction water supply purposes (Golder Associates 1982A) has provided additional information on the hydrogeology of the surficial materials in the area of Hat Creek and Marble Canyon.

Three surficial aquifers were encountered in the Marble Canyon area; they were grouped under the Marble Canyon Aquifer System and are believed to be isolated from the pit area. Near the confluence of Hat Creek and Houth Creek a shallow aquifer (Hat Creek Alluvial Aquifer) was identified by the investigation; it is separated by a thick silty clay aquiclude from a deep sand and gravel aquifer (Hat Creek Aquifer). It is believed that the Hat Creek Aquifer was a probable extension of the Buried Valley Aquifer identified in the 1978 report located to the northeast of the pit.

Further ground water studies were undertaken during 1982 in connection with the diversion studies. Where appropriate, these results are included in this report, but they are also covered in the report on the Diversion Study (Golder Associates, October 1982B).

4.1.1 Scope of Work

In order to reconsider the quantity of drainage into the proposed pit and critically review data previously obtained the following hydrogeological work was carried out:

(1) Reassessment of existing piezometric and permeability data

(2) Evaluation of the hydrogeological regime in the area to the northeast of the pit (glacio-fluvial channel)

(3) Re-assessment of pit inflows and dewatering requirements.

4.2 Reassessment of Existing Data

4.2.1 Piezometric Levels

Piezometers to monitor ground water level fluctuations were installed during the hydrogeological field investigation between 1976 and 1978. Regular readings of ground water levels have since been recorded by BCH staff and submitted to Golder Associates for processing. A total of 295 piezometers (3/4-inch diameter standpipes and pneumatics) have been monitored for the past 4 to 5 years with the data being used manually to plot ground water hydrographs. In order to facilitate manipulation of the results, Golder Associates have developed a computer storage and retrieval system and plotting routine to produce hydrographs as required. Appendix D (Volume 2) contains computer plots of these piezometer hydrographs.

Analysis of the ground water hydrographs indicates both long- and short-term changes in ground water regime at the site. Areas of ground water recharge (downward hydraulic gradient) and ground water discharge (upwards hydraulic gradient) can be identified. The ground water flow pattern in and around the pit area can be determined from these plots and has been used in the assessment of pit inflows.

It was noted that many of the standpipe piezometers installed in low permeability bedrock in the earlier drilling programs had not stabilized with in situ ground water pressures due to the phenomena of time lag (Hvorslev, 1951). For the low permeability bedrock at the Hat Creek site (hydraulic conductivity 1×10^{-11} to 1×10^{-12} m/sec), the time required for up to 90 per cent equalization of water levels for 3/4-inch diameter standpipes is estimated to be up to 6 years. Therefore, only now are many of the standpipe piezometers reflecting true piezometric levels. With this in mind, it was necessary to prepare a revised piezometric surface map for bedrock materials and surficial deposits to indicate the direction and gradient of ground water flow in the pit areas.

The bedrock piezometric surface map (Figure 21) is a combination of data from within various lithologic units and, as such, represents only a generalized two-dimensional flow pattern. The map does not indicate the vertical movement of ground water that is occuring within recharge or discharge zones. In general, the map shows little change from that produced for the Golder Associates' 1978 Report, although many of the piezometers have shown head changes in excess of 5 m over the time period. These are primarily piezometers completed in lithologies with permeabilities less than 1×10^{-10} m/sec. The data from the 1982 program for the diversion tunnel (Golder Associates, 1982B) has been used to prepare contours for the east side of the pit. The contour map indicates a steep hydraulic gradient in this area but this may be due to the fact that these piezometers are still stabilizing following installation and do not reflect the true in situ piezometric elevations.

The piezometer in DDH76-150, completed within the siltstone, sandstone, conglomerate unit (Tcl) of the Coldwater Formation in the area of the northwest pit slope is recording an apparently anomolous piezometric elevation of approximately 795 m. Analysis of the hydrograph for this piezometer indicates that near stabilization of the ground water level has been reached. Piezometer RH77-61A-2, 100 m distant from DDH76-150, is indicating a piezometric elevation of approximately 810 m. This piezometer has not yet stabilized and appears to be approaching the piezometric level recorded in DDH76-150. These piezometric elevations are some 60 to 65 m below neighbouring piezometers, RH77-61A-1 and 3 and DDH76-808-1. These piezometers are completed within the same siltstone, sandstone, conglomerate (Tcl) unit but at different horizons and indicate near stabilized piezometric elevations between 859 and 872 m.

Geologically, this area in the northwest of the pit is underlain by a Tertiary rock sequence that dips eastward and is overlain by up to 50 m of overburden. The overburden is predominantly till and is underlain by burn zone material. It is considered possible that burning of the coal at outcrop resulted in unloading of the underlying material; the reduction in normal load is likely to have produced an expansion of the material and an immediate decrease in pore water pressure. Due to the low hydraulic conductivities of the bedrock, these reduced piezometric pressures have not yet equalised to the higher pressures observed in neighbouring piezometers. This could be a practical example of the process which it is anticipated would be crucial to the stability of the slopes (see Section 3.4.2).

Figure 22 presents a contoured map of the piezometric surface of the surficial materials over the mine site, including 1982 data and information from the 1981 construction water supply program. It is seen that the general direction of ground water flow in this area is to the north-northwest or north under a hydraulic gradient of approximately 0.025. This figure shows very little change from the Golder Associates 1978 Report.

An examination of the individual piezometer hydrographs (Appendix D) provides the following conclusions regarding the short-term ground water fluctuations at the Hat Creek site.

(1) Bedrock and overburden piezometers in the Medicine Creek and Trachyte Hills area show seasonal fluctuations of approximately 2 m. Highest ground water levels are recorded in June/July with lowest levels in February to May. There is a rapid rise in ground water levels, indicative of a relatively high permeability in the early summer (period of ground water recharge) probably associated with the snow melt followed by a slow decline thoughout the summer and autumn and winter, due to release of ground water from storage.

(2) Shallow piezometers in bedrock in the north central area of the pit near Houth Creek show a similar seasonal variation, with a late winter minimum and spring/summer maximum. Indirect recharge from Hat Creek during high flows and recharge associated with snow melt is probably responsible for the rise in ground levels. Permeability testing in these piezometers indicates a relatively high k of between 1×10^{-6} to 1×10^{-8} m/sec.

(3) It was earlier reported that a seasonal fluctuation of up to 3 m was seen in many piezometers. Closer examination of the individual hydrographs reveals that standpipe piezometers completed in low permeability lithologies (claystones, siltstones, and coals, $k < 1 \times 10^{-10}$ m/sec) have not shown any significant fluctuations over the 5-year monitoring period. Several pneumatic piezometers, completed within similar lithologies have indicated a seasonal fluctuation in piezometric levels of between 0.5 and 3 m. However, with a reading resolution of ± 0.5 m, their senstivity to detect seasonal changes is limited. Professor Rowe considers it possible that the seasonal response observed may be due to presence of small permeable fractures within the bedrock (see Appendix C).

(4) A number of piezometers located either in the Houth Meadows area, or north of the proposed pit, showed a significant decline in ground water levels during late summer 1981 (see Table 3). During autumn 1981, some piezometers showed only partial recovery, while others continued to decline and as of spring 1982 none of these piezometers had recovered to spring 1981 levels.

The decline in ground water levels in this area is considered to be due to the pumping of Well PW1 (screened within the Hat Creek Aquifer) during July 1981 as part of the construction water supply program. The pump test data and the response of the piezometers in Hat Creek and Houth Meadows provides valuable hydrogeological information for the understanding of the ground water regime in the surficial and bedrock materials to the north of the proposed pit.

TABLE 3

Summary of Piezometric Data
Houth Meadows/Hat Creek

Piezometer No.	Piezometric Level June 1981 (m)	Piezometric Level December 1981 (m)	Decline in Water Level (m)	Lithology
DDH76-813-1	840.8	831.0*	9.8	Sand and gravel
DDH77-829-1	846.7	842.1	4.6	Limestone
DDH77-831-1	867.9	866.5	1.4	Limestone
DDH77-834-1	840.7	836.9	3.8	Limestone
DDH77-835-1	840.4	836.3	4.1	Limestone
DDH77-835-1	841.1	837.5	3.6	Silty sand

* Recorded in August 1981.

Figure 23 shows the wells drilled in the northeast area, while Figures 24 and 28 show hydrogeological cross-sections along the Hat Creek valley in the area of the northern pit rim and Houth Meadows. Piezometric information both before and after pumping of Well PW1, where available, is included on these sections. Figure 29 shows contours on the base of the surficial deposits in the same area drawn on the basis of the drilling and geophysical survey. It is seen that during the pre-pumping period, the piezometric level in the Hat Creek aquifer was above ground surface resulting in flowing conditions in Wells OW4 and PW1. Well PW1 was pumped for a period of eight days at a rate of 420 U.S. gpm (26.5 l/s) and resulted in a drawdown of approximately 30 m in the well and a drawdown of 19.5 m in OW4 at a radius of 21.5 m. Stabilization of water levels within Wells PW1 and OW4 did not occur during the pump test. Following the cessation of pumping, both wells recovered slowly taking up to two weeks to recover to 90 per cent of original static water levels. Piezometer DDH 76-813-1 installed within sands and gravels approximately 350 m from PW1 indicated a piezometric level in the middle of August 1981, 9.8 m lower than in June 1981 (see Table 3). Previous years' monitoring (1977 to 1980) had not indicated any significant fluctuations at this time of year. Since pumping of Well PW1 ceased on July 28th, the August reading in piezometer DDH 76-83-1 reflects a partially recovered piezometric level. It is determined from distance/drawdown calculations that the maximum drawdown induced in this piezometer due to the pumping of PW1 could have been 14 m.

Piezometer DDH 76-814-2 located approximately 750 m from Well PW1 (see Figures 23 and 24) showed little change from June to August, 1981. It is estimated from distance/drawdown calculations that if this piezometer had been completed in material in direct hydraulic connection to the well, a drawdown response of approximately 8 m would have been recorded. It is considered that a direct hydraulic connection between the sands and gravels present in borehole DDH 76-814 and the Hat Creek aquifer does not exist. A relatively impermeable layer of silty sand and clay detected in borehole DDH 76-813 (see Figure 24) likely restricts movement of ground water between the two zones.

Piezometer DDH 76-814-1 is located in the siltstone/sandstone Coldwater Formation (Tcl). The water level in this piezometer has declined since installation in 1977, declining approximately 0.5 m from June to August 1981. The decline from June to August 1981 is not considered due to the pumping of Well PW1, but due to stabilization of the piezometer water level with the in situ ground water pressures following installation (see ground water hydrographs in Appendix D).

During late 1981/early 1982, the water level in piezometer DDH 76-813-1 recovered to approximately 80 per cent of the pre-pumping level. At this time, Well OW4 was flowing at between 20 and 30 U.S. gpm (1.3 to 1.9 l/s) at the surface due to a drawdown of approximately 3 m at the well head. Well PW1 was not flowing at the surface due to the installation of above ground casing. It is calculated that the natural overflow of Well OW4 would result in a drawdown of approximately 2.8 m at a radius of 350 m, based on the transmissivity obtained from the early pump test data of Well PW1. This would therefore account for the less than full recovery of the water level in piezometer DDH 76-813-1. Basing the same calculation on the transmissivity obtained from the later pump test data from Well PW1, a minimal drawdown would have been induced by the overflow of Well OW4. In this case, the less than full recovery of the water level could be due to removal of ground water from storage. The slow rate of recovery of both Well PW1 and OW4 following pump testing indicates a limited recharge to this aquifer.

Figure 25 presents a hydrogeological section approximately west to east from Houth Meadows to Hat Creek. It is seen that the pump testing influenced piezometric levels within the surficials and limestone bedrock in this area. The earliest available water level readings following pump testing are for December 1981, some 5 months after pumping had ceased, see Table 3. These water levels probably represent partially recovered piezometric levels. The response of piezometers within both the silty sand and limestone bedrock indicates a hydraulic connection between these zones and the Hat Creek aquifer. Piezometric levels within the silty sand and limestone have not fully recovered to the pre-pump test levels reflecting the slow rate of recharge to both units and continual overflow from Well OW4. It is recommended that further testing and more widespread monitoring should be carried out when a permanent pump is installed in Well PW1 to assess the regional drawdown within the Hat Creek Aquifer and limestone underlying Houth Meadows.

4.2.2 Permeability Data

As a result of the non-equalization of water levels in standpipe piezometers completed in low permeability materials, it was necessary to re-evaluate some of the permeability tests detailed in the 1978 report. It was found that earlier analyses had assumed stabilized ground water levels which have not been substantiated by the subsequent monitoring. The currently recorded levels are in many cases lower than the projected equilibrium levels. Following a check of all permeability tests previously carried out, it was seen that a total of 17 of the rising and falling head permeability tests required re-analysis. For this re-analysis, the currently recorded ground water level was taken where it was considered that it represented the stabilized ground water level. Where it was considered that stabilization had not been reached, then the projected stabilized ground water level was used in the analysis. The remainder of the permeability data collected during previous programs does not require re-analysis and is considered representative of the lithologies tested. The revised estimates of hydraulic conductivities for the bedrock materials are used to assess ground water inflow into the proposed pits (see Sections 4.4 and 4.5).

The re-analysis included data from piezometers located in various lithologic units, including:

i) Medicine Creek Formation - (upper siltstone/claystone - Tcu)

ii) Hat Creek Coal Formation - (A-zone siltstone and coal - Tcc)

iii) Coldwater Formation - (lower siltstone-sandstone-conglomerate - Tcl)

iv) Coldwater Formation - (conglomerate - Tco_1)

The data was analysed according to the Hvorslev method. The results of the current re-analysis, together with the previously calculated values of hydraulic conductivity are presented in Table 4.

In general, the reassessment of the permeability test data resulted in a calcuated hydraulic conductivity one to two orders of magnitude lower than the 1978 estimates. Table 5 presents a summary of all falling head tests carried out on bedrock units at the Hat Creek site, including those obtained from the re-analysis. In general, there does not appear to be a significant change in the overall median value of hydraulic conductivity for each lithologic unit compared with the 1978 report.

4.3 Data Acquired from 1982 Program

4.3.1 Glacio-fluvial Channel

A north-south trending glacio-fluvial channel was identified in the 1978 report as being present, underlying the northeast area of the proposed pit. A drawing presented in the 1978 report (Drawing 2 "Contours on the Base of the Surficial Deposits") indicated a deep but wide 'low' in the top of the bedrock surface, extending north-south with the valley deepening northward. The thickness of surficial material within this zone was proved to be at least 180 m (borehole DDH 78-870). The material was identified as interbedded gravel, sand and silt. Tills were often shown to be present at the base of the buried valley. Falling head permeability tests carried out in DDH 78-870 indicated the hydraulic conductivity of the material to range between 1.2×10^{-8} m/sec and 5.7×10^{-9} m/sec.

TABLE 4

Summary of Hydraulic Conductivity Re-analysis

Piezometer No.	Lithologic Unit	Hydraulic Conductivity (m/sec) 1978 Report	Recalculation
Medicine Creek Formation:			
DDH78-870-1	Clayey Siltstone (Tcu)	1.0×10^{-12}	9.3×10^{-13}
DDH78-867-1	Clayey Siltstone (Tcu)	1.9×10^{-11}	8.8×10^{-13}
DDH77-843-1	Sandstone (Tcu)	1.8×10^{-11}	1.6×10^{-12}
DDH77-846-1	Sanstone (Tcu)	1.1×10^{-10}	4.1×10^{-11}
DDH76-815-1	Siltstone (Tcu)	1.5×10^{-10}	4.0×10^{-11}
Hat Creek Coal Formation:			
DDH77-236-1	Sandstone A-Coal (Tcc)	1.4×10^{-11}	2.8×10^{-12}
DDH77-256-1	Claystone A-Coal (Tcc)	8.8×10^{-12}	4.8×10^{-12}
Coldwater Formation:			
DDH76-150-1	Sandstone Siltstone (Tcl)	4.5×10^{-9}	8.9×10^{-13}
DDH77-240-1	Sandstone Siltstone (Tcl)	No analysis	1.0×10^{-12}
DDH78-865-2	Siltstone, Sandstone Conglomerate (Tcl)	3.7×10^{-10}	4.6×10^{-12}
DDH78-865-3	Sandstone, Siltstone (Tcl)	1.3×10^{-10}	4.9×10^{-12}
DDH77-842-1	Conglomerate (Tco_1)	1.4×10^{-10}	2.6×10^{-12}
DDH77-851-1	Conglomerate (Tco_1)	3.8×10^{-11}	1.5×10^{-11}
DDH77-849-2	Sandstone (Tcs)	3.4×10^{-11}	4.4×10^{-12}
DDH77-849-1	Conglomerate (Tco_2)	8.5×10^{-12}	7.4×10^{-13}
DDH78-868-1	Sandstone Siltstone	3.7×10^{-10}	1.3×10^{-12}
Cache Creek Formation:			
DDH78-858-1	Sandstone	8.7×10^{-11}	1.0×10^{-11}

TABLE 5

Summary of Results of Falling Head Tests on Bedrock Units

Lithologic Unit	Number of Tests	Hydraulic Conductivity Range (m/sec) From	To	Median Value
Medicine Creek Formation:				
Upper Siltstone Claystone (Tcu)	17	8.8×10^{-13}	1.0×10^{-6}	4.0×10^{-11}
Hat Creek Formation:				
A Zone Siltstone and Coal (Tcc)	5	2.8×10^{-12}	2.6×10^{-10}	3.0×10^{-11}
B Zone Coal (Tcc)	3	2.0×10^{-7}	5.0×10^{-7}	4.0×10^{-7}
C Zone Siltstone and Coal (Tcc)	13	3.0×10^{-11}	3.0×10^{-8}	1.4×10^{-10}
D Zone Coal (Tcc)	12	5.0×10^{-11}	1.0×10^{-6}	6.0×10^{-8}
Coldwater Formation:				
Lower Siltstone-Sandstone Conglomerate (Tcl)	15	9.0×10^{-13}	1.0×10^{-7}	3.0×10^{-11}
Conglomerate (Tco_1)	4	2.6×10^{-12}	1.3×10^{-10}	5.0×10^{-11}
Cache Creek Formation:				
Limestone	7	1.2×10^{-9}	1.0×10^{-4}	3.0×10^{-8}
Greenstone	5	4.0×10^{-10}	5.0×10^{-7}	1.8×10^{-7}
Tertiary Basalt	5	2.3×10^{-11}	1.8×10^{-6}	7.0×10^{-9}

Subsequent well drilling for the Construction Camp Water Supply in 1981 identified a sandy gravel aquifer (Hat Creek Aquifer) up to 45 m thick in boreholes PW1 and OW4 in Hat Creek valley to the north of the proposed pit. This aquifer was encountered at approximately elevation 770 m but was not fully penetrated. The boreholes were both screened in this aquifer and pump tested at rates of between 113 to 420 U.S. gpm (7.1 and 26.45 l/s). Following the pump testing of Wells PW1 and OW4, it was suggested that further investigation be carried out on the ground water regime in this area to ascertain whether there could be any adverse ground water impact on the open pit (Golder Associates 1982A) as a result of the presence of that aquifer.

The 1982 ground water investigation program was thus designed to provide a more definitive understanding of hydrogeological conditions to the north and northeast of the proposed pit. A staged program consisting of geophysical surveys, well drilling and analysis of ground water hydrographs (see Section 4.2.1) was undertaken to define both the geometry of this aquifer and the potential ground water inflow into the pit. A geophysical survey, conducted by Geo-Physi-Con was run on the north and east sides of the proposed pit to define the extent of the glaciofluvial channel and to aid in the location of boreholes designed to intersect a maximum thickness of overburden. The results of the geophysical survey have already been presented separately to BCH (Geo-Physi-Con, 1982).

The drilling program following the Geo-Physi-Con geophysical survey was planned so that if drilling identified significant ground water flows, well screens could be installed in the boreholes and the completed wells pump tested. The drilling of two boreholes was carried out by Drillwell Enterprises Ltd. of Duncan, B.C., under the supervision of Golder Associates during June 1982. The boreholes were located close to the northeast pit rim as shown on Figure 23 and were drilled by the air-rotary method. Both boreholes were started in 254-mm (10-inch) diameter, then reduced to 203-mm (8-inch) diameter at 42.8 m (140 ft). Soil samples or rock cuttings were collected every 3 m (10 ft) for description.

Borehole RH 82-102 was drilled to a total depth of 201.3 m (660 ft) and encountered bedrock at 190.8 m (626 ft). Borehole RH 82-103 was drilled to 189.0 m (620 ft) with bedrock encountered at 175.6 m (576 ft). In both boreholes, bedrock was identified as a clayey siltstone. The surficial deposits consisted of silty sand and gravel with layers of silt and clay. Immediately overlying bedrock in both boreholes was a dense clayey silty sand and gravel till. The two boreholes are both located in the vicinity of borehole DDH 78-870 drilled in an earlier program. The log from this borehole indicates a similar sequence to that proved in the two boreholes drilled this year. Figure 28 shows a hydrogeological cross section in this area of the pit.

As no significant ground water flows or highly permeable zones were encountered during drilling, the boreholes were completed with standpipe piezometers rather than well screens; no deposits worthy of screening and pump testing were found. One piezometer was installed in RH 82-102 and two in RH 82-103. The piezometers consisted of a 1.2-m (4-ft) long, 25-mm (1-inch) diameter slotted PVC tip attached to a 19-mm (3/4-inch) diameter PVC standpipe. Gravel was used as a filter around the tips and as backfill between piezometer locations. Bentonite seals were used to isolate the piezometers in different zones in the boreholes.

Hydraulic conductivity testing was carried out in piezometers RH 82-102-1 and RH 82-103-1 in September 1982. No testing was carried out in RH 82-103-2 since this piezometer was blocked 1.6 m below the surface. The cause of the blockage is unknown. Prior to testing, piezometers were monitored periodically in order to determine stabilized piezometric levels. The testing involved pouring a slug of water down the standpipes and monitoring the decay of water levels until approximately 80 per cent of the excess head had dissipated. The data was analysed according to the method described in Hvorslev (1951).

Analysis of the data indicates the hydraulic conductivity of the tested material, predominantly silty sand and gravel, to be between 9 x 10^{-7} m/sec and 10 x 10^{-7} m/sec. Table 6 summarizes the details of the testing.

The logs of the boreholes and details of the piezometer installations are included in Appendix E.

The results of the geophysical and hydrogeological investigations carried out in the north and northeast areas of the pit now provide a more detailed understanding of geology and ground water regime in this area. It is seen that the glaciofluvial channel deepens northwards in this area of the pit and then appears to swing north-northwest to coalesce with a second bedrock channel in the area of the confluence of Houth Creek and Hat Creek (see Figure 29). In the area of the northeast pit rim, drilling has proved up to 190 m of overburden. Drilling in the Hat Creek Valley, north of the pit, did not fully penetrate the overburden; a thickness of at least 116.7 m (PW1) being proved. The geophysical survey indicated an overburden thickness of at least 120 m in the area of Well PW1.

The overburden infilling the two channels is seen to be of differing composition. In the area of the northeast pit slope, overburden is principally a silty sand and gravel with occasional lenses of silty clay. Overlying bedrock, boreholes DDH 78-870, RH 82-102 and RH 82-103 identified a till layer (clayey silty sand and gravel) between 4.9 and 15.8 m thick. The surficial silty sand and gravel can be traced northwards from the pit and appears to pinch out and grade into a silty clay with thin fine sand laminae. The silty clay with fine sand is underlain by coarse sand and gravel (Hat Creek Aquifer) composed predominantly of limestone fragments. It is considered that the coarser material found at depth in the Hat Creek Valley is probably derived from erosion of the limestone area to the west of the pit, whereas the silty sand and gravel materials present in the northeast pit slope probably are the result of erosion of weaker volcanic outcrops to the east of the pit.

TABLE 6

Summary of Hydraulic Conductivity Testing, 1982 Program

Piezometer No.	Time Lag (secs)	Length of Gravel Pack (m)	Diameter of Gravel Pack (m)	Hydraulic Conductivity (m/sec)
RH82-102-1	65	2.3	0.203	9×10^{-7}
RH82-103-1	215	3.6	0.203	2×10^{-7}

From the evidence available, it appears that the Hat Creek sand and gravel aquifer is of limited extent and pinches out as it approaches Houth Meadows and the northern edge of the pit. The aquifer is overlain in Hat Creek by a thick layer (up to 60 m) of impermeable silty clay. Recharge to the Hat Creek aquifer is limited and appears to be principally derived from downward leakage from the silty sand and upward leakage from the limestone in the Houth Meadows area. The pump test has significantly impacted the piezometric level within this aquifer since monitoring indicates that recovery to pre-pumping levels has yet to occur.

Surficial materials that will be exposed in the pit slopes to the north and northeast of the pit are principally silty sands and gravels, interbedded clays, silts and fine sands and basal tills (see Figures 26 and 27). Evidence from the two boreholes drilled in the 1982 program (RH 82-102 and RH 82-103) indicates that the overburden materials in the northeast pit are not likely to yield large quantities of ground water. Hydraulic conductivities determined from falling head tests are between 9×10^{-7} m/sec and 1×10^{-7} m/sec for silty sand and gravel. Recharge to these materials is probably via infiltration of precipitation on the eastern slopes of the valley and seepage from the underlying bedrock.

An unusual feature has been noticed in the two boreholes drilled in the northeast pit area: a flow of air or gas intermittently emanates from the casing annulus around the piezometers. An analysis (B. Dutt, personal communication) shows that it is composed primarily of CO_2 with some minor constituents; there is no indication of methane. It is believed that it results from flow through air-permeable material from a zone of higher external pressure either higher or lower on the hillside to the exploratory well. It could be expected to have a pattern of diurual variation. It indicates a permeable pathway through the glacial deposits.

4.4 2240 MW Pit

4.4.1 Pit Inflows

The previously detailed hydrogeological work has provided the framework for the critical reassessment of the quantity of ground water draining into the proposed pit. Ground water inflows would be derived from the bedrock materials exposed in the pit slopes and the overlying surficial materials.

4.4.2 Bedrock Inflows

The re-evaluation of the permeability data for the bedrock units indicates that the median hydraulic conductivities of the bedrock that would be exposed in the pit slopes would range between 4 x 10^{-7} m/sec and 3 x 10^{-11} m/sec. Based on these values, it is calculated that ground water inflow for the final 35-year pit might range between 1.7 x 10^{-1} m^3/sec and 1.25 x 10^{-5} m^3/sec. The figure for bedrock inflow of 1.7 x 10^{-3} m^3/sec presented in the 1978 report was considered the average anticipated inflow for the 35-year pit. It is considered that the bedrock inflows would attain the upper bound inflow quantities on a long-term basis, but for design purposes it would be advisable to select a system to handle this quantity of ground water inflow.

During the life of the pit, bedrock inflows would increase to the maximum quantity indicated above. Table 7 presents ranges of anticipated bedrock inflows for interim pits based on the range of bedrock hydraulic conductivity previously stated.

TABLE 7

Anticipated Bedrock Ground Water Inflow - 2240 MW Pit

Year	Inflow m^3/s	
	Maximum	Minimum
5	1.8 x 10^{-2}	1.35 x 10^{-6}
15	6.0 x 10^{-2}	4.55 x 10^{-6}
25	1.3 x 10^{-1}	1.00 x 10^{-5}
35	1.7 x 10^{-1}	1.25 x 10^{-5}

4.4.3 Surficial Inflows

In the northeast area of the pit, overburden exposed in the pit slopes would be principally granular material including sand, gravel and some silty clay. A till layer up to 15 m thick is present at the base of the surficials. The saturated thickness of the overlying units (silty sand and gravel) is estimated as between 35 and 40 m. From in situ permeability testing, hydraulic conductivity of these units range between 9 x 10^{-7} to 6 x 10^{-9} m/sec. The ground water table presently slopes gently to the northwest under a gradient of 0.025, but a reversal in flow direction would occur when the pit is opened up with ground water flow directed toward the pit. It is calculated that the ground water inflow to the ultimate pit might reach a maximum of 1.8 x 10^{-3} m^3/sec after 35 years from the surficial sediments to the north and northeast of the pit. Steady state inflows associated with the interim pits are shown on Table 8. These calculations are based on a hydraulic conductivity value of 1×10^{-6} m/sec and represent maximum anticipated flows.

TABLE 8

Anticipated Ground Water Inflows From Surficial Sediments to North and Northeast - 2240 MW Pit

Year	Inflow m^3/s
5	4.0×10^{-5}
15	5.8×10^{-4}
25	1.8×10^{-3}
35	1.8×10^{-3}

The replacement of the Hat Creek Diversion Canal by a pipeline or a lined tunnel would reduce the estimated ground water flow derived from surficial sediments to the east of the pit from the 1978 estimate. It is estimated that ground water inflows into the 35-year pit would be reduc- from 9.6 x 10^{-3} m^3/sec as reported in 1978 to 5 x 10^{-3} m^3/sec. Table 9 summarizes the calculated steady-state inflows associated with the interim pits. It is anticipated that steady state inflows to the pit would not increase after 25 years since the configuration of the pit within the surficials to the east of the pit shows little change after this time.

TABLE 9

Anticipated Ground Water Inflows From Surficial Materials To East of Pit - 2240 MW Scheme

Year	Inflow m^3/s
5	1.2×10^{-5}
15	1.0×10^{-3}
25	5.0×10^{-3}
35	5.0×10^{-3}

To the north of the pit, the shallow Hat Creek alluvial aquifer would be cut by the pit. This aquifer would initially drain into the pit at an estimated 1×10^{-3} m^3/sec, but this quantity would decline with time as the aquifer drains northward. The deep Hat Creek aquifer is not anticipated to provide ground water inflow into the pit due to its limited extent. Inflow to the south of the pit would show no change from the value given in the 1978 Report (4.5×10^{-3} m^3/sec) and is not considered to vary with time. To the west of the proposed pit, ground water inflow would show only small increases with time from an estimated 1×10^{-3} m^3/sec after 5 years to 5×10^{-3} m^3/sec after 35 years. Total surficial ground water inflow to the pit of 5.7×10^{-3} m^3/sec is anticipated for the 35-year development.

A summary of the average anticipated ground water inflow for the 2240 MW Pit is shown on Figure 30. This figure was initially presented in the 1978 report but has been altered to depict the better understanding of the surficial sediments to the north and northeast of the proposed pit.

It must be emphasized that in spite of considerable information obtained at the Hat Creek site, there are still uncertainties regarding the hydrogeological characteristics of the bedrock and overburden materials; this is only to be expected at this stage with such complex geology. In this light, a range of anticipated ground water inflows rather than average quanitities has been presented for bedrock inflows. It is anticipated that short-lived ground water inflows in excess of average quantities are likely to occur. These short-lived inflows are likely to be the result of the opening up of faults, more permeable zones within a coal sequence or the presence of small permeable gravel pockets within the overburden not detected during drilling and permeability testing. These short-lived higher inflows might result in a greater quantity of ground water entering the pit instantaneously than anticipated in the dewatering design. In order to counteract these occasional higher inflows, consideration must be given to sizing the facilities to deal with these eventualities. However, both sedimentation and leachate ponds are capable of staged increases in size. It is recommended that the high end of the ranges of anticipated flows be used and the the flows be monitored in the early years to check against the design storages. The pit itself could act as a sump where very short duration excess flows could be stored, thus avoiding oversizing pumps unnecessarily.

4.4.4 Dewatering

Previous sections of this report have identified areas of the Hat Creek Coal Project where additional hydrogeological data has been obtained during the 1982 investigation. In this section, the recalculated average anticipated ground water inflows detailed in Sections 4.4.2 and 4.4.3 are used to determine the dewatering requirements for the pit.

In general, the quantity of ground water requiring pumping is only slightly lower than that presented in the 1978 report. A flow chart indicating dewatering requirements and mine seepage for the 2240 MW Pit is shown on Figure 30. If this system were adopted, the estimated steady state pumping from wells around the final pit perimeter (2240 MW Scheme) would be 1.16×10^{-2} m^3/sec. A limited proportion of ground water would escape the dewatering system and enter the pit. This quantity is estimated as 7.4×10^{-3} m^3/sec and would require handling by in pit drains and sumps.

The changes made to the 1978 dewatering design are primarily related to the handling of ground water inflows from surficial sediments to the north and northeast of the pit. The hydrogeological drilling program carried out in this area identified sediments of relatively low permeability that were unsuitable for screening and pump testing. Therefore, the use of wells to control ground water inflows from this area is considered unsuitable and inflows would require handling via in-pit sumps. Inflows derived from the Hat Creek alluvial aquifer from the north would also be allowed to seep into the pit as wells are considered unsuitable due to the limited depth of this aquifer and the fact that inflows from this direction would decline with time due to natural drainage northward of the aquifer. A contingency for some wells should nonetheless be made in the costing.

To the east of the pit, the replacement of the Hat Creek Diversion Canal by a pipeline during the operational phases for both schemes would reduce the ground water inflows presented in the 1978 report by approximately 50 per cent. No contribution would be provided by the recommended diversion arrangement. The 1978 report indicates that the anticipated ground water inflow could be handled by a series of wells around the pit perimeter. In the light of the hydrogeological investigations to the northeast of the pit, wells might not now be suitable as a means of dewatering. Since no additional hydrogeological information is available in this area, the wells presented in Figure 30 are tentative. Further exploratory well drilling could be carried out during early construction to the east of the pit to examine, in more detail, the suitability of the overburden materials for screening and well completion. Assuming a well dewatering system were feasible, an allowance has been made for eight 200-mm diameter wells spaced at approximately 200 m centres.

For the areas to the south and west of the pit, anticipated ground water inflow and necessary dewatering requirements remain unchanged from the 1978 report. The data has been re-evaluated but in the absence of more detail, there is no justification for revising the earlier predictions.

4.5 800 MW Pit

4.5.1 Pit Inflows

Ground water inflows into the proposed 800 MW pit have been calculated based on similar hydrogeological parameters to the inflow calculation for the 2240 MW pit. Overall, inflows to the 800 MW pit are reduced due to shallower depths of pit and less exposure of surficial materials to the east and west of the pit.

Ground water inflows to the ultimate pit derived from bedrock are anticipated to range between 3.2×10^{-2} m^3/sec and 2.4×10^{-6} m^3/sec. Average ground water inflows from bedrock are calculated as 1.7×10^{-4} m^3/sec. It is considered prudent to base a dewatering design on the higher figure since most of the material exposed in the pit faces would be coal with a relatively higher hydraulic conductivity than the siltstones and claystones. Ground water inflows derived from bedrock have also been calculated based on interim pit developments. This is presented in Table 10.

TABLE 10

Anticipated Bedrock Ground Water Inflows - 800 MW Pit

Year	Maximum	Minimum
	m^3/sec	
5	3.0×10^{-3}	2.7×10^{-7}
15	8.4×10^{-3}	6.3×10^{-7}
25	1.6×10^{-2}	1.2×10^{-6}
35	3.2×10^{-2}	2.4×10^{-6}

Ground water inflow from the surficial sediments is also reduced due to the pit configuration. The shallow Hat Creek alluvial aquifer is expected to provide 1.0×10^{-3} m^3/s inflow to the pit. This inflow would decline with time as the aquifer drains naturally northward. Inflow from the surficial sediments to the north and northeast of the pit would be limited since most of the saturated materials exposed are anticipated to be tills and interbedded clay, silts and fine sands. Ground water inflow from this area is calculated to be a maximum of 5.6 x 10^{-5} m^3/sec into the ultimate pit. Inflow from the eastern pit slopes is calculated to be a maximum of 2 x 10^{-4} m^3/s into the ultimate pit.

To the south of the pit, the anticipated ground water inflow would not be reduced from the estimate provided for the 2240 MW pit (see Section 4.4.3). This is calculated as 4.5 x 10^{-3} m^3/sec. However, in the smaller scheme, the diversion dam for the pipeline intake would be much closer to the pit and seepage control wells have been allowed for at the downstream toe of this structure.

On the western pit slopes, the slide zone would not be intercepted by the pit excavation and inflow would be principally derived from the surficial materials. It is calculated that inflows from this source would reach a maximum of 7 x 10^{-4} m^3/s after 35 years.

Total ground water inflow from surficial materials surrounding the 800 MW pit is, therefore, estimated to be 3.4 x 10^{-3} m/sec by the 35 year pit development.

4.5.2 Dewatering

The dewatering requirements for the 800 MW pit are considerably reduced from the 2240 MW pit scheme. Much lower quantities of ground water inflow are anticipated as presented on the mine seepage and dewatering flow chart (see Figure 31). Wells would only be required for the surficial materials south of the pit and for the bedrock within the pit area due to the low inflow quantities. It is estimated that the total quantity of water pumped from surficials would be 3.1 x 10^{-3} m^3/sec with total seepage into the pit from the surficial materials of 3.4 x 10^{-3} m^3/sec. Ground water inflow from bedrock seepage would require handling either via wells, drains or sumps. It would be necessary to intercept as much ground water as possible before it enters the pit to maximize the quantity of water which would not require treatment.

5.0 FURTHER WORK REQUIRED

At the time of writing (December 1982), the Hat Creek Project has been placed on hold and no further programs are planned in the foreseeable future. However, the Hat Creek coal deposits are a major resource and there can be no doubt that they will eventually be exploited by one means or another.

There are many geotechnical areas which require the collection of more data and further analysis with possible consequent implications for design. However, it is now felt that such further work would be inappropriate until a major excavation is undertaken from which in situ data could be obtained on the low permeability claystones and siltstones. This could be part of the initial excavations for the project for it would surely be costly. However, there would be considerable interest in being able to carry out some developmental work aspects such as slope monitoring, depressurization and slope drainage prior to the initiation of the project.

Professor Rowe has recommended (see Appendix C) that large diameter samples should be obtained on which detailed sophisticated laboratory testing could be carried out. These samples would also be best obtained from major excavations but consideration could be given to obtaining them from adits or large diameter auger holes prior to the commencement of the project.

When the project is revived, these aspects should be considered along with the proposed schedules.

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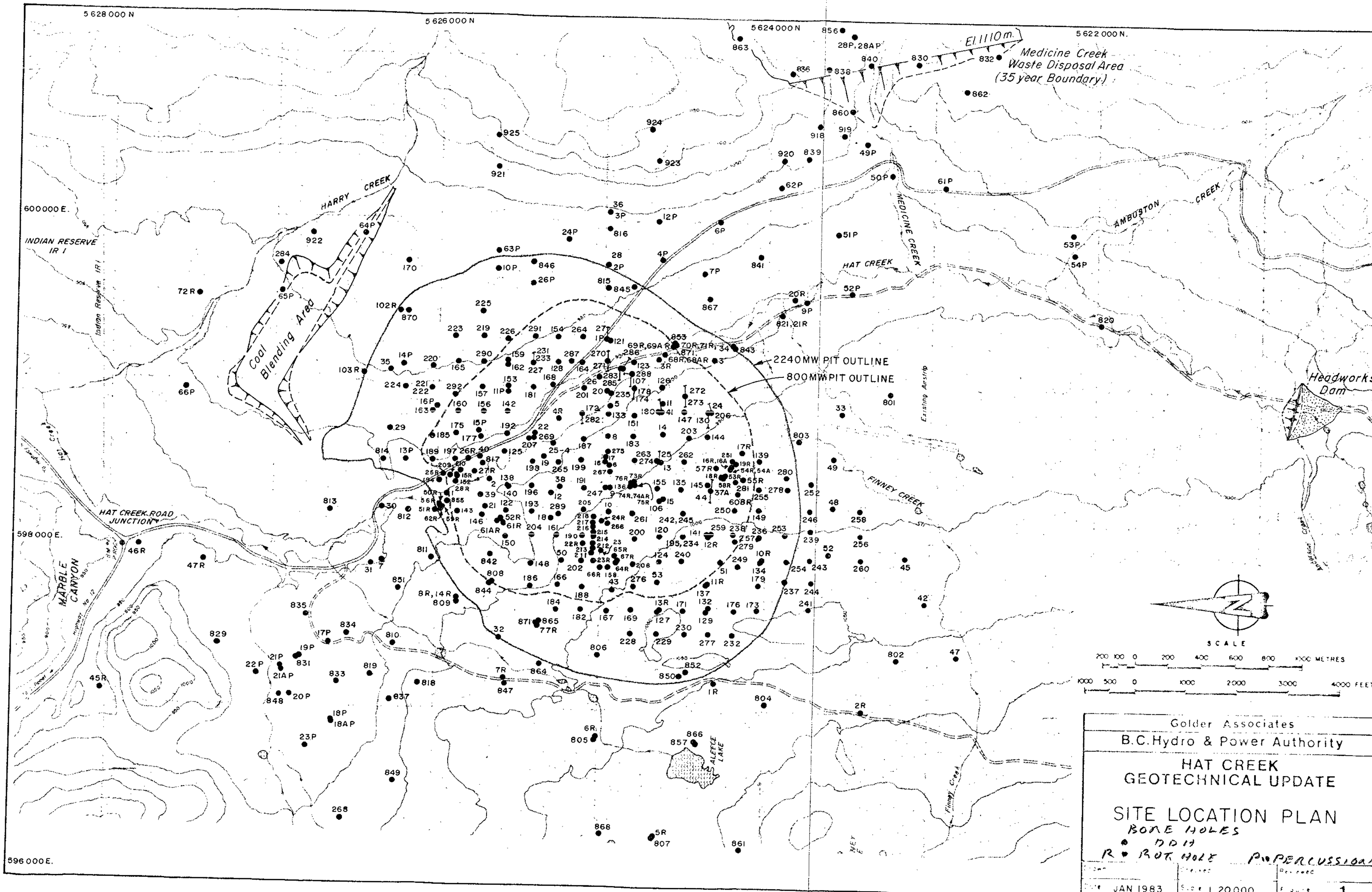

5 628 000 N
5 626 000 N
5 624 000 N
5 622 000 N.
600 000 E.
598 000 E.
596 000 E.
El 1110m.
Medicine Creek
Waste Disposal Area
(35 year Boundary)
HARRY CREEK
INDIAN RESERVE
IR 1
Indian Reserve IR1
Coal Blending Area
MEDICINE CREEK
HAT CREEK
AMBUSTON CREEK
2240 MW PIT OUTLINE
800 MW PIT OUTLINE
Headworks
Dam
FINNEY CREEK
Existing Airstrip
HAT CREEK ROAD
JUNCTION
MARBLE
CANYON
ALEECE LAKE
Finney Creek
SCALE
200 100 0 200 400 600 800 1000 METRES
1000 500 0 1000 2000 3000 4000 FEET
Golder Associates
B.C.Hydro & Power Authority
HAT CREEK
GEOTECHNICAL UPDATE
SITE LOCATION PLAN
BORE HOLES
• DDH
R• ROT HOLE
P• PERCUSSION
JAN 1983
1 20000
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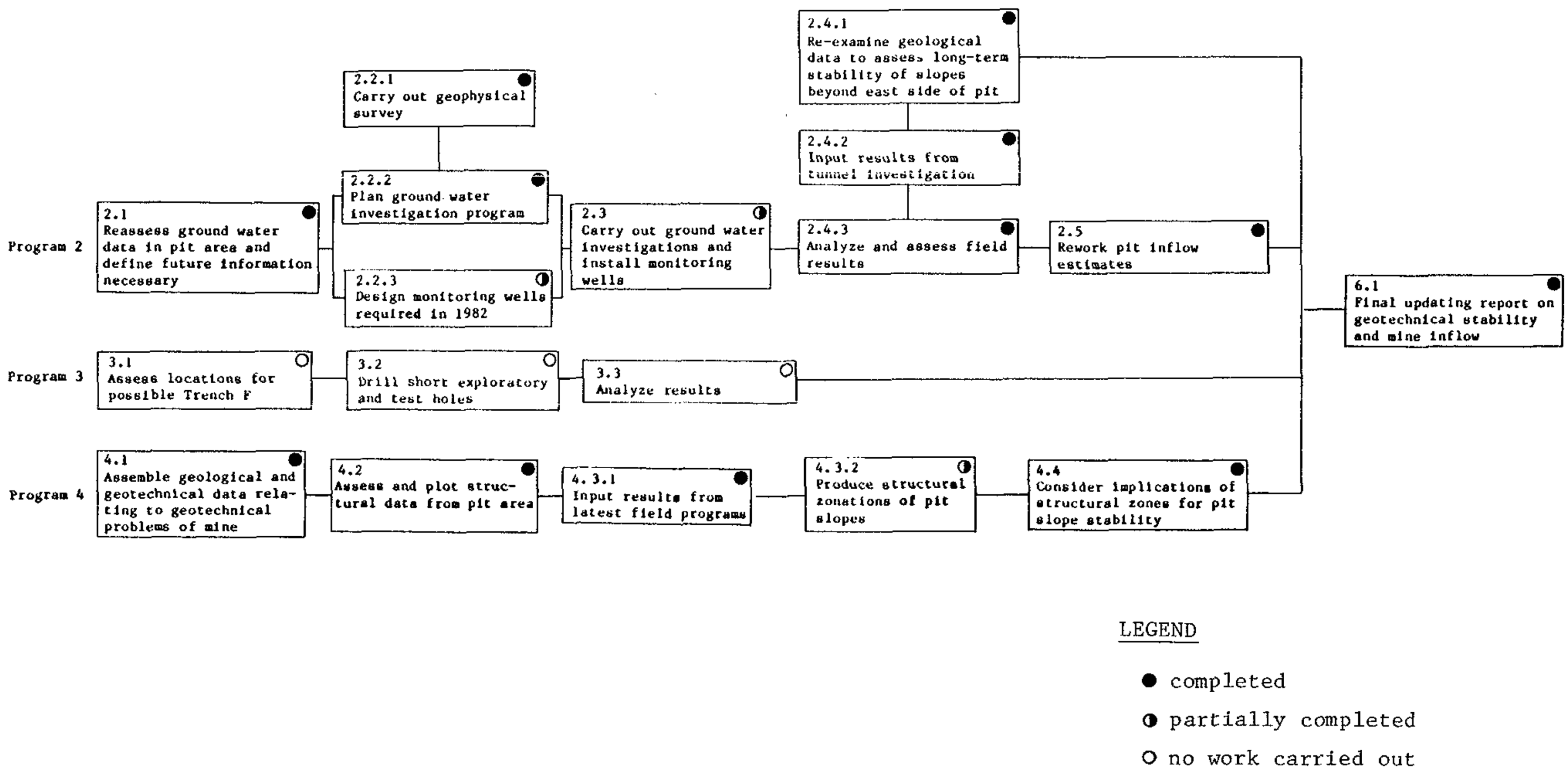
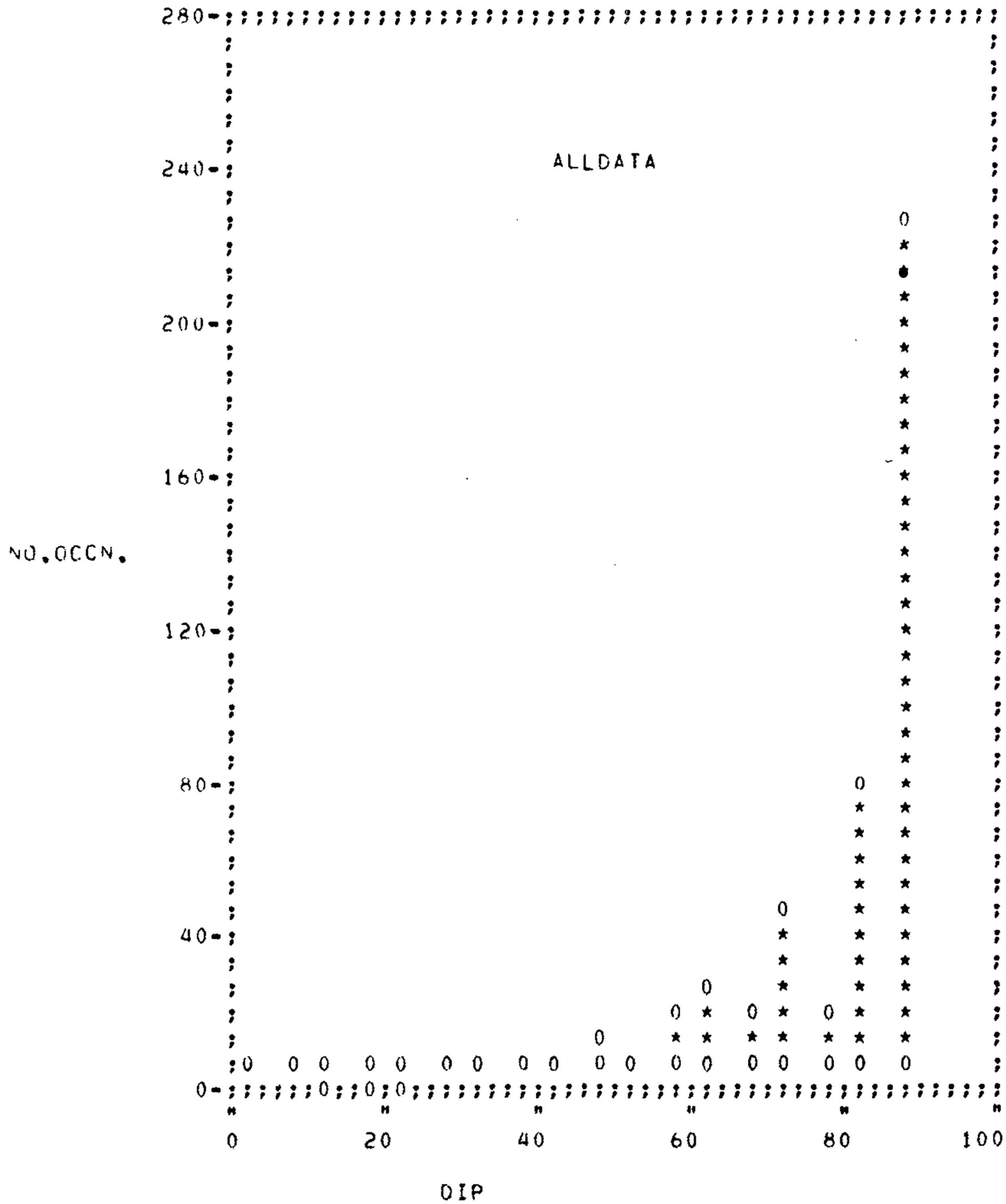
FIGURE 2

ACTIVITIES PLANNED FOR 2240 MW SCHEME



HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 76-801

Figure 3



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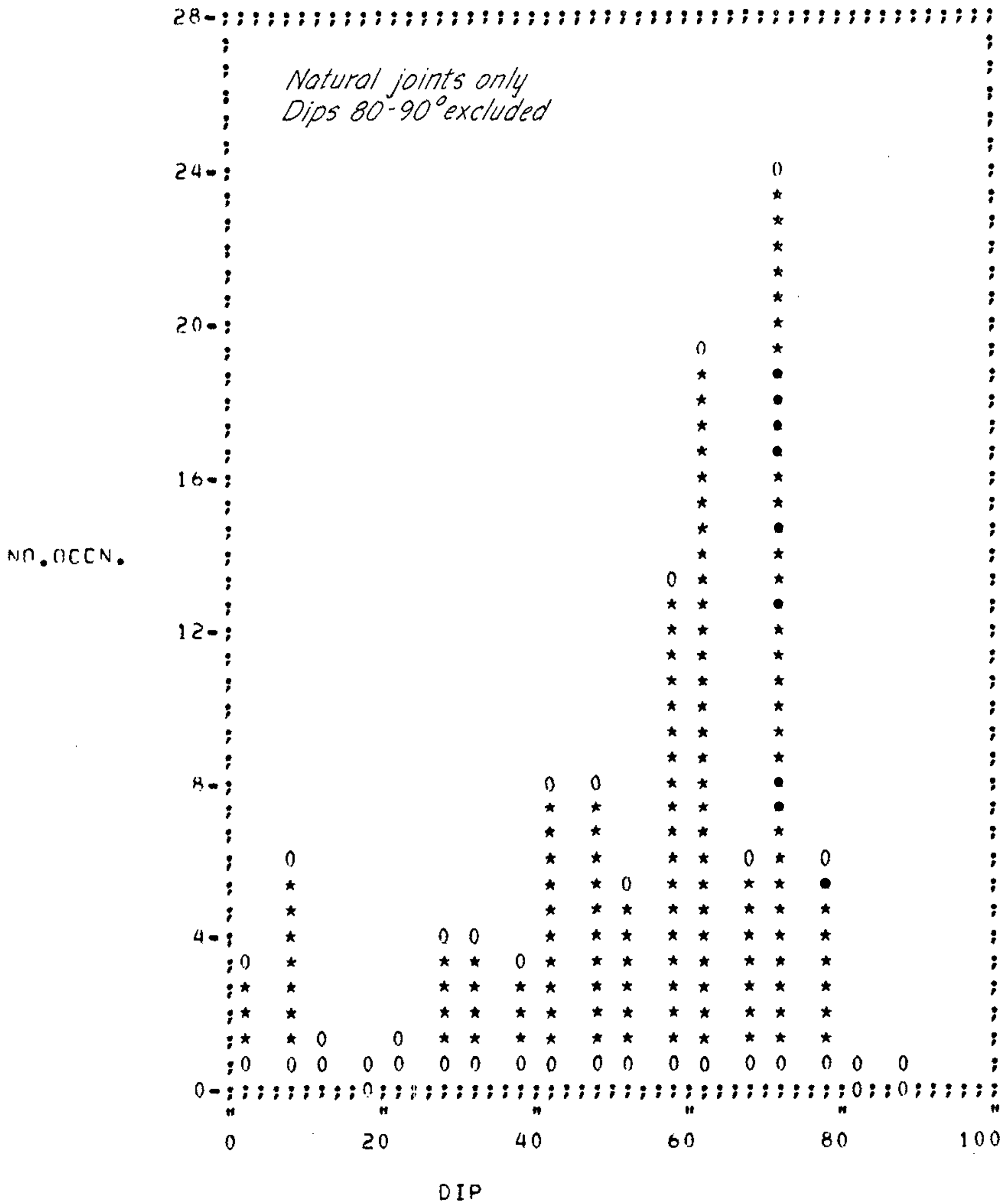
HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 76-801

Figure 4



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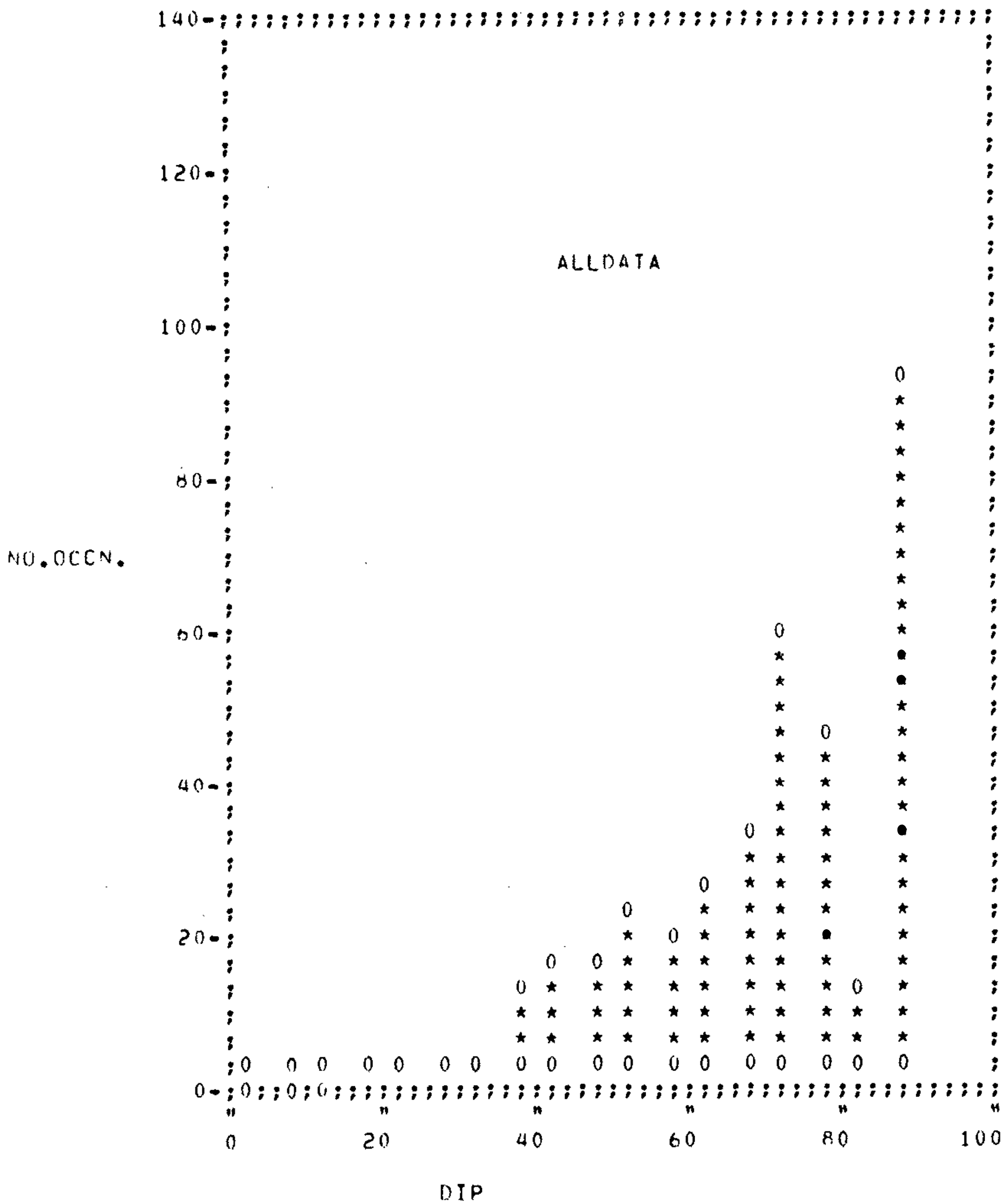
HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 76-815

Figure 5



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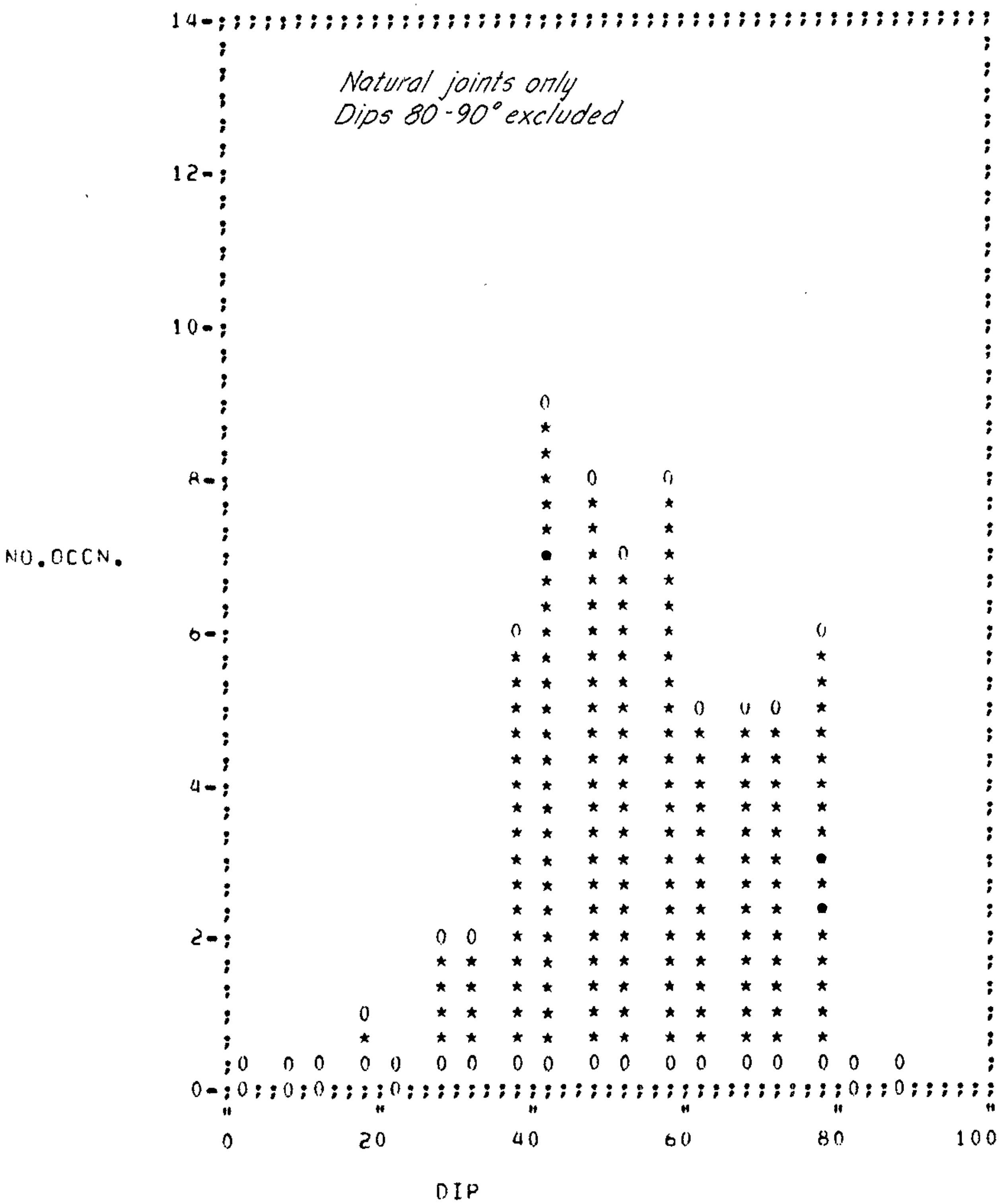
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT
DDH 76-815

Figure 6



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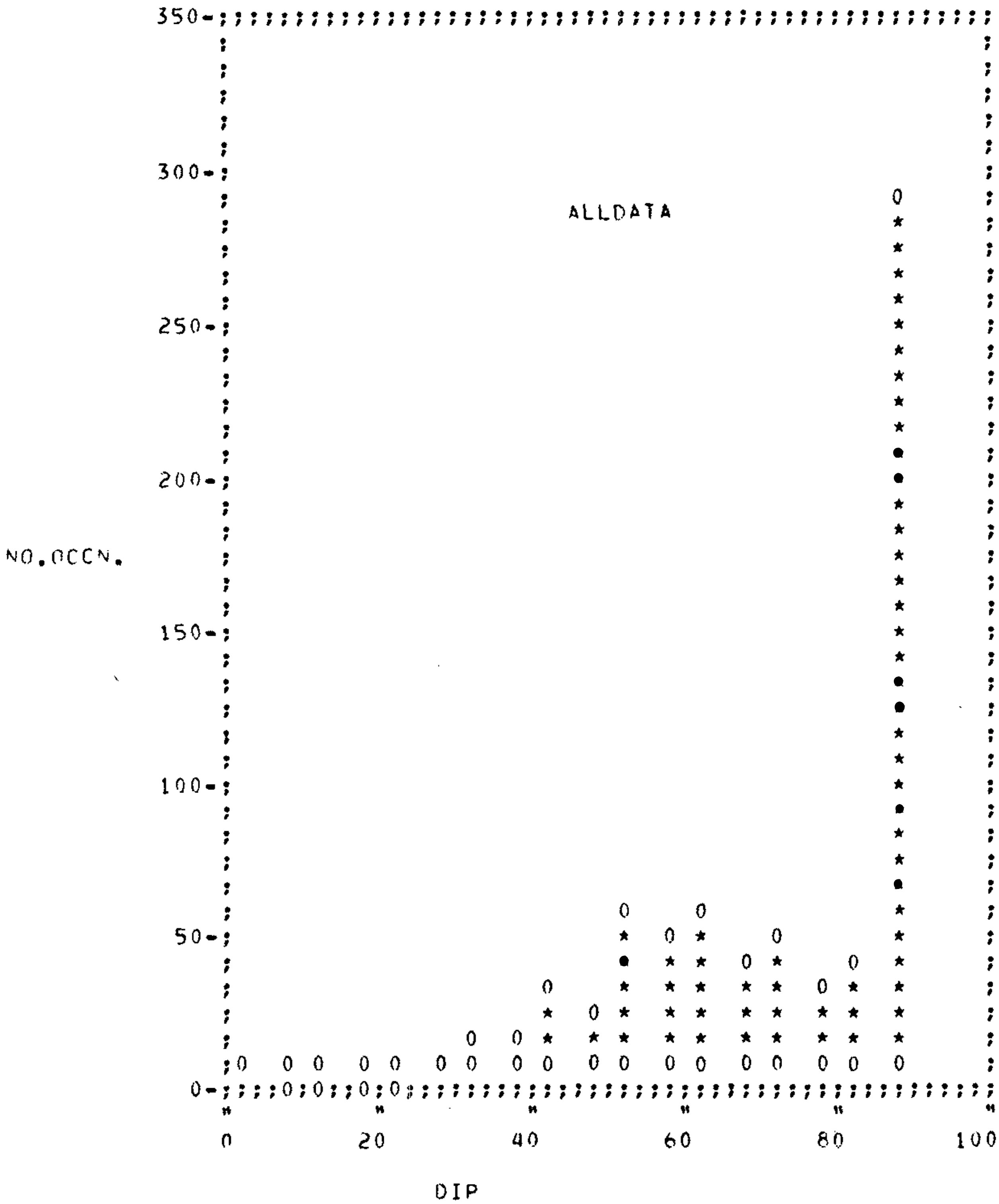
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT
DDH 76-816

Figure 7



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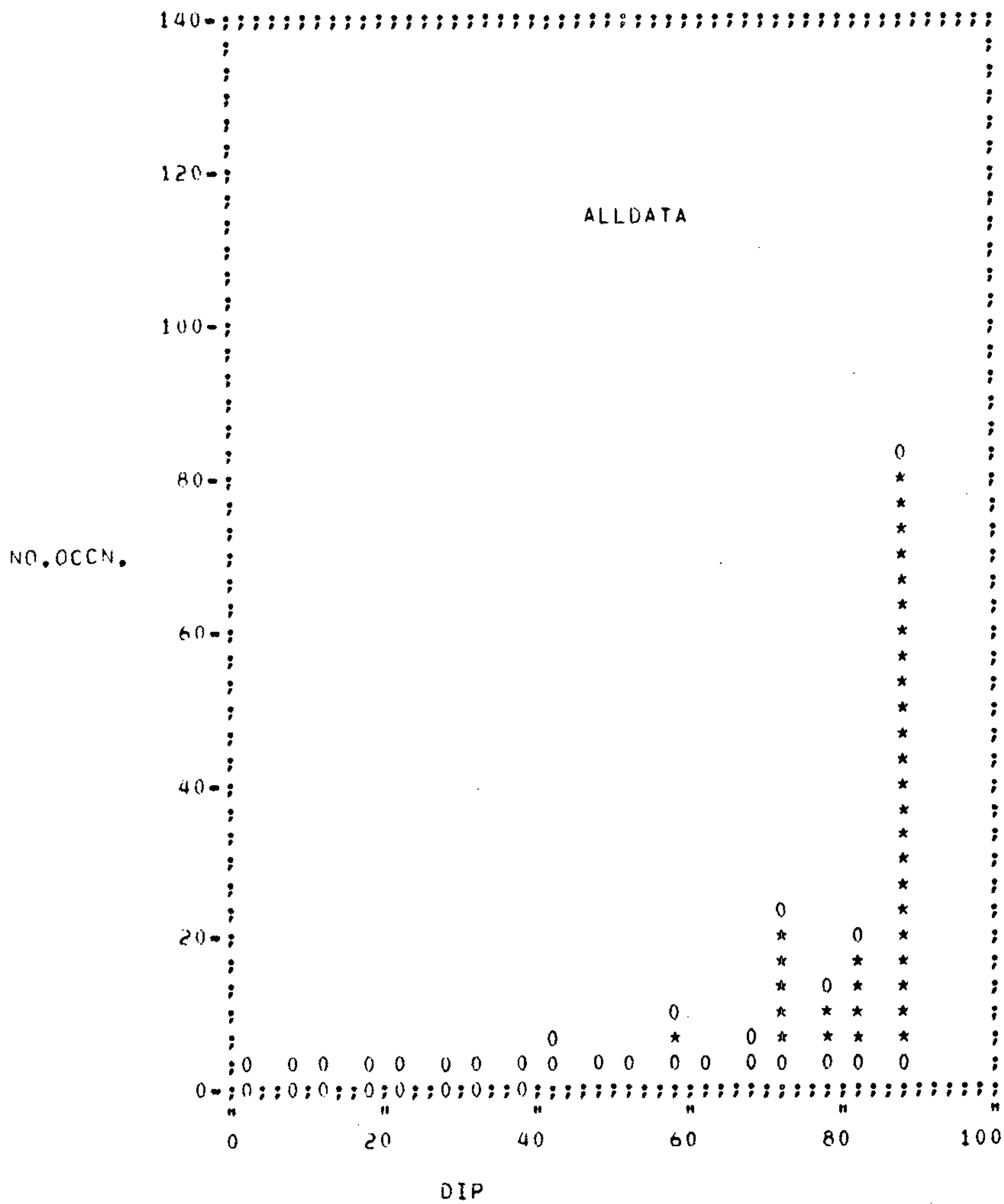
HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 76-821

Figure **8**



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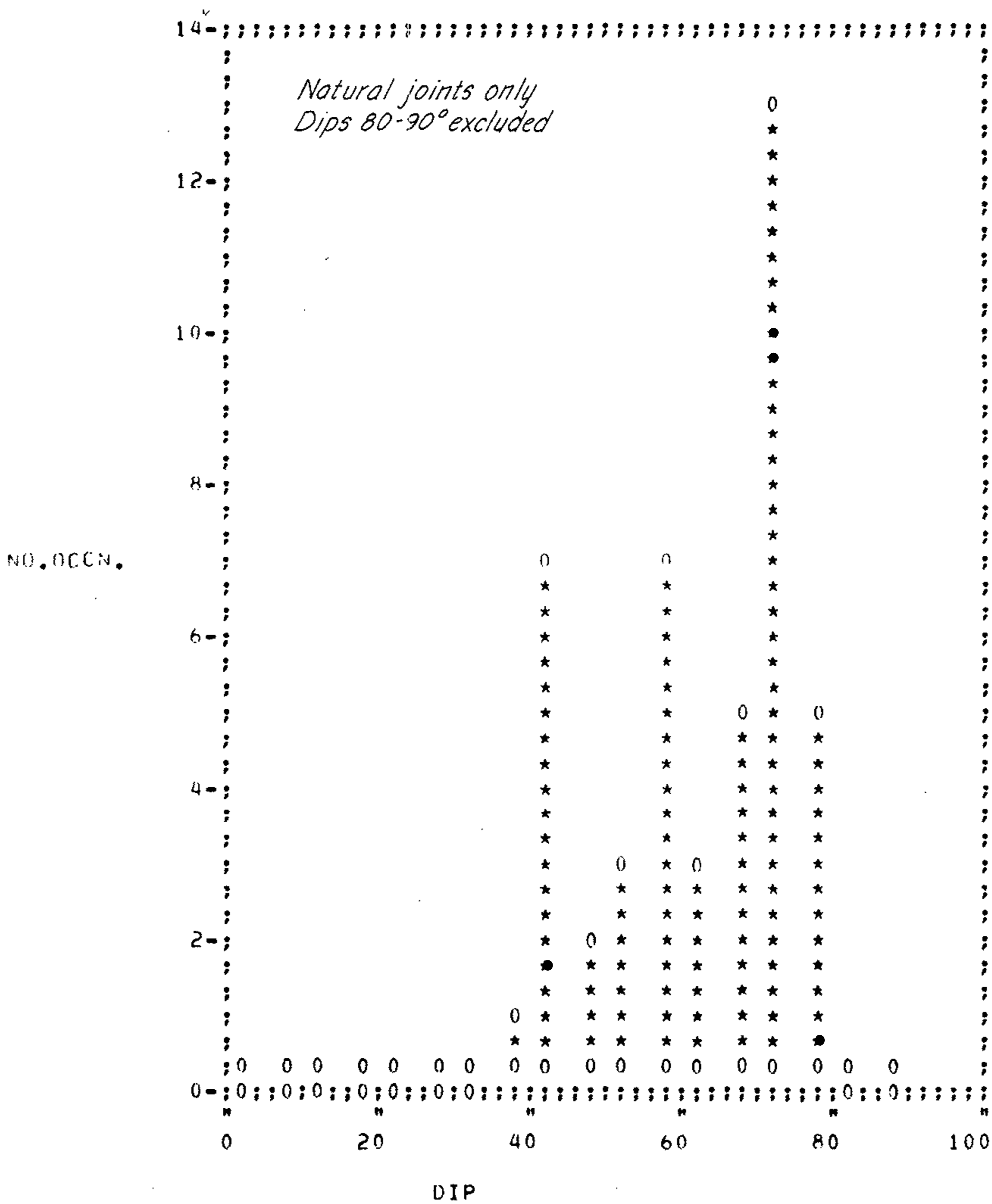
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 76-821

Figure 9



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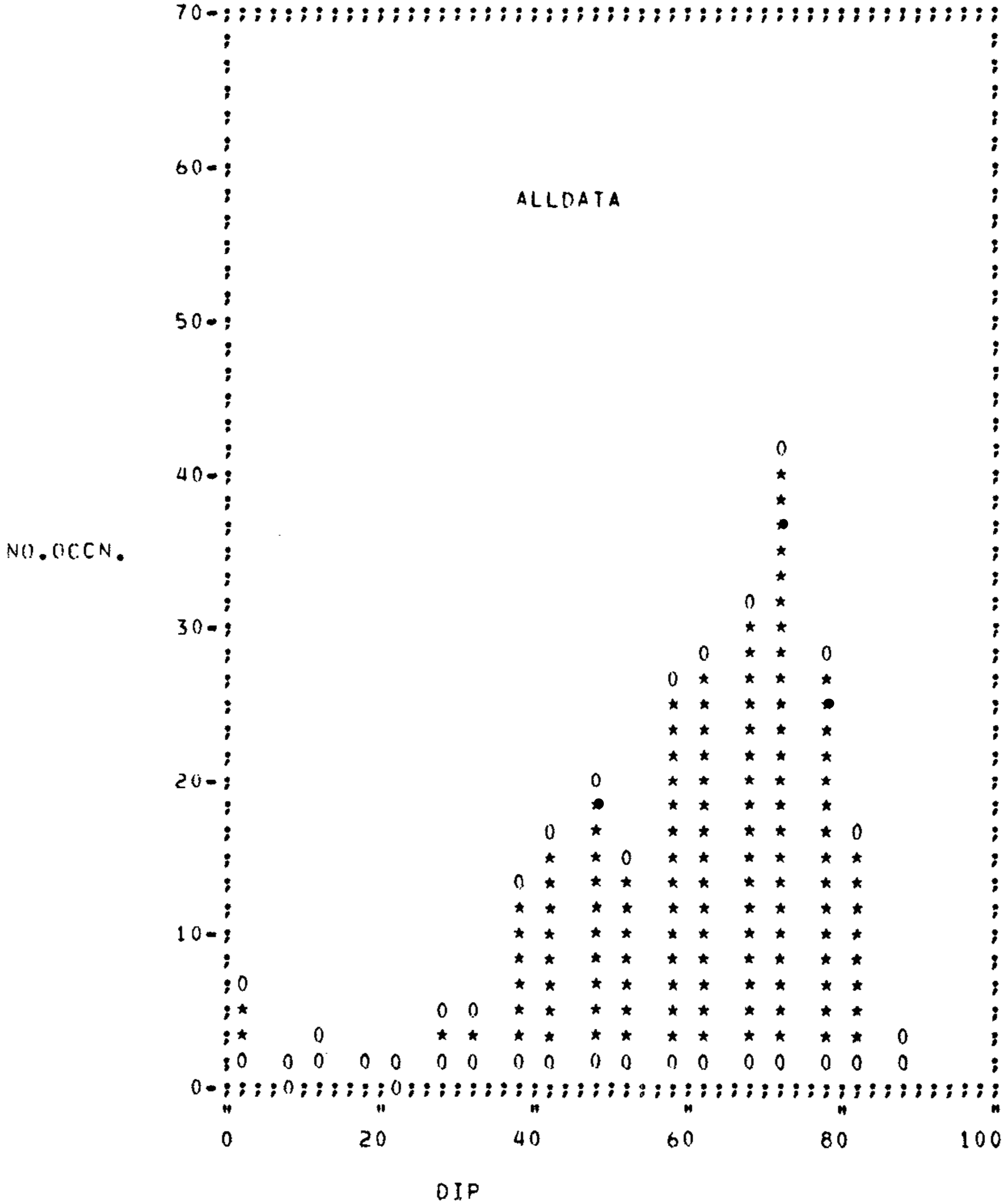
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 77-841

Figure **10**



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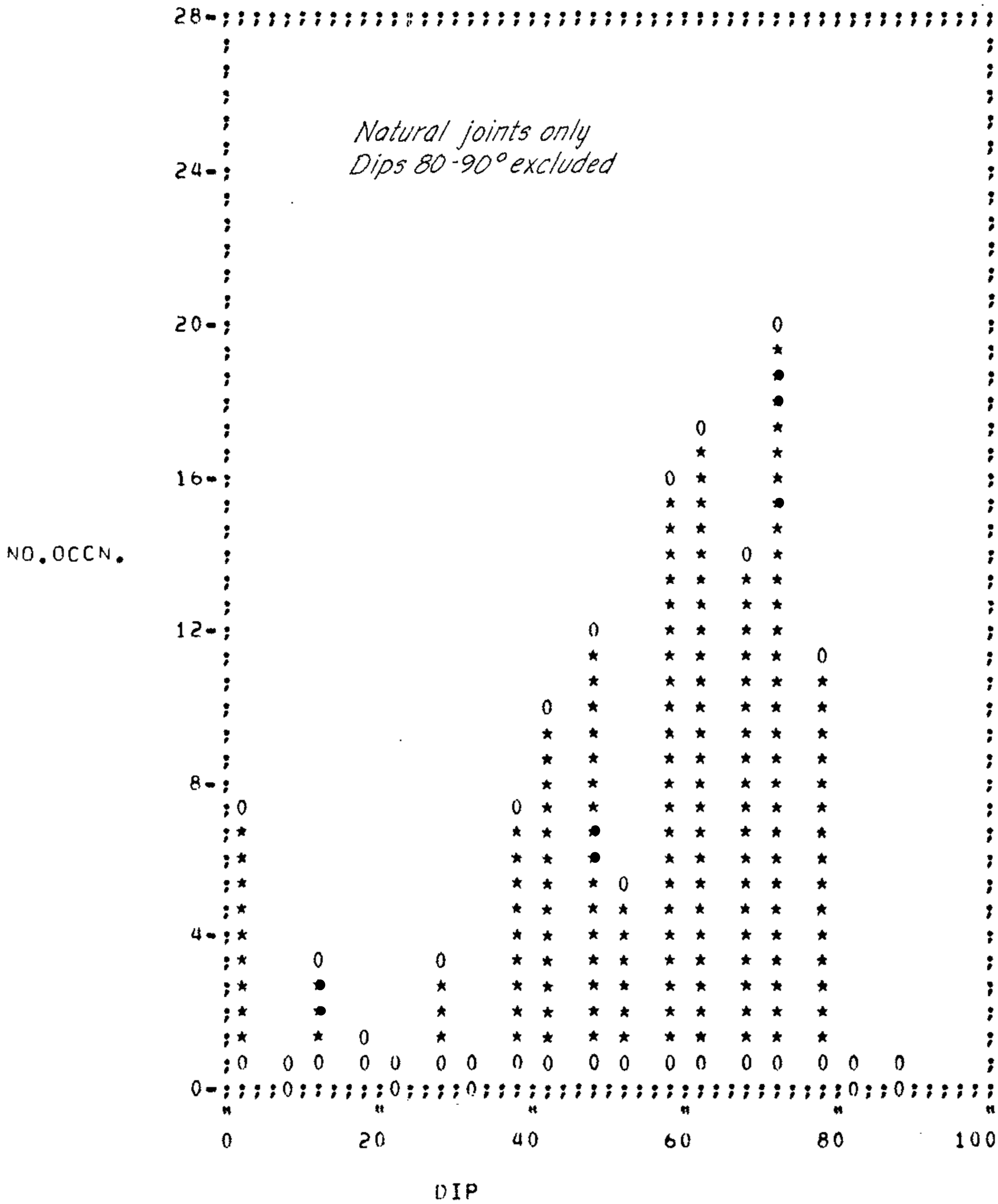
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 77-841

Figure **11**



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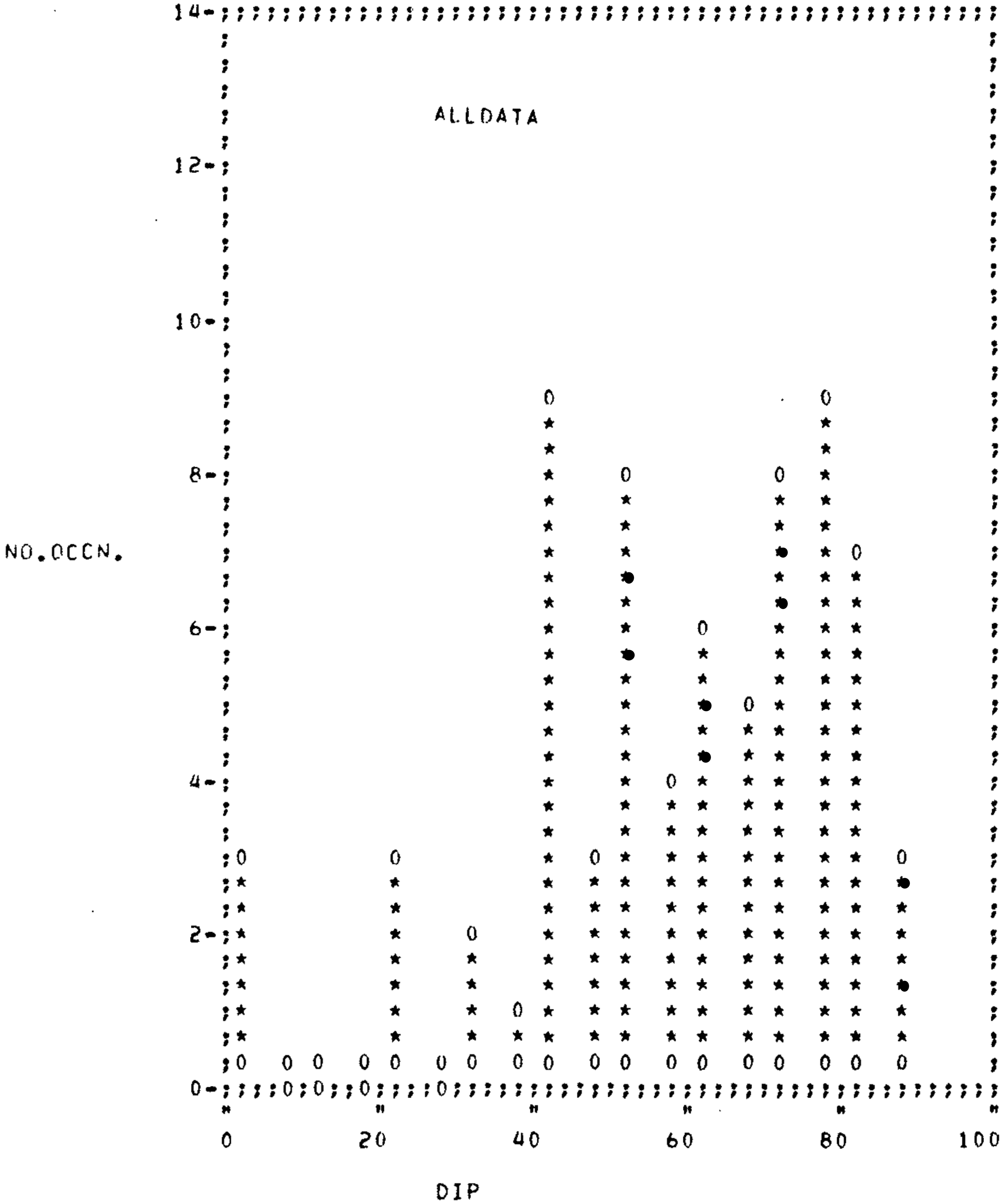
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 77-843

Figure **12**



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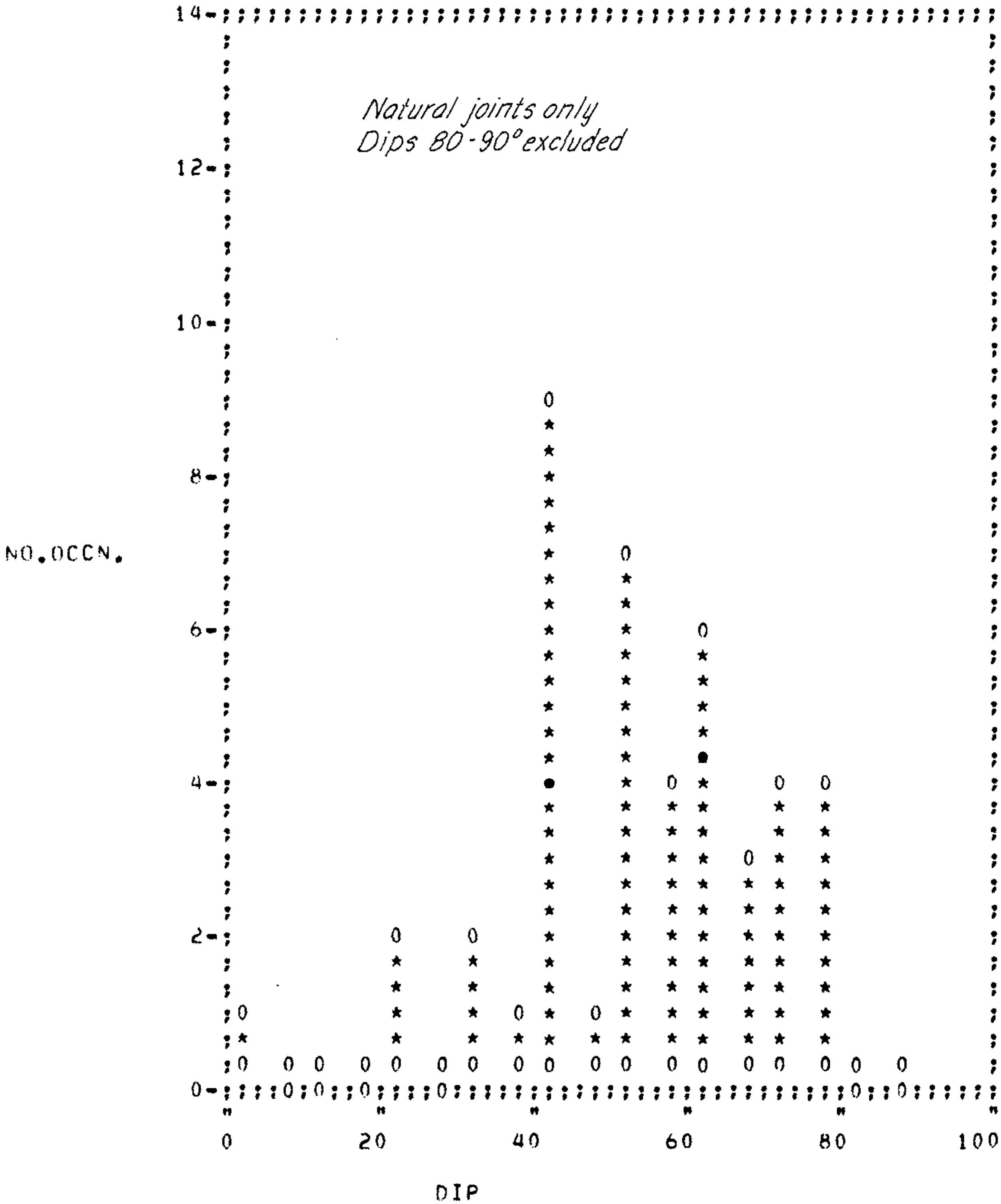
HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 77-843

Figure **13**



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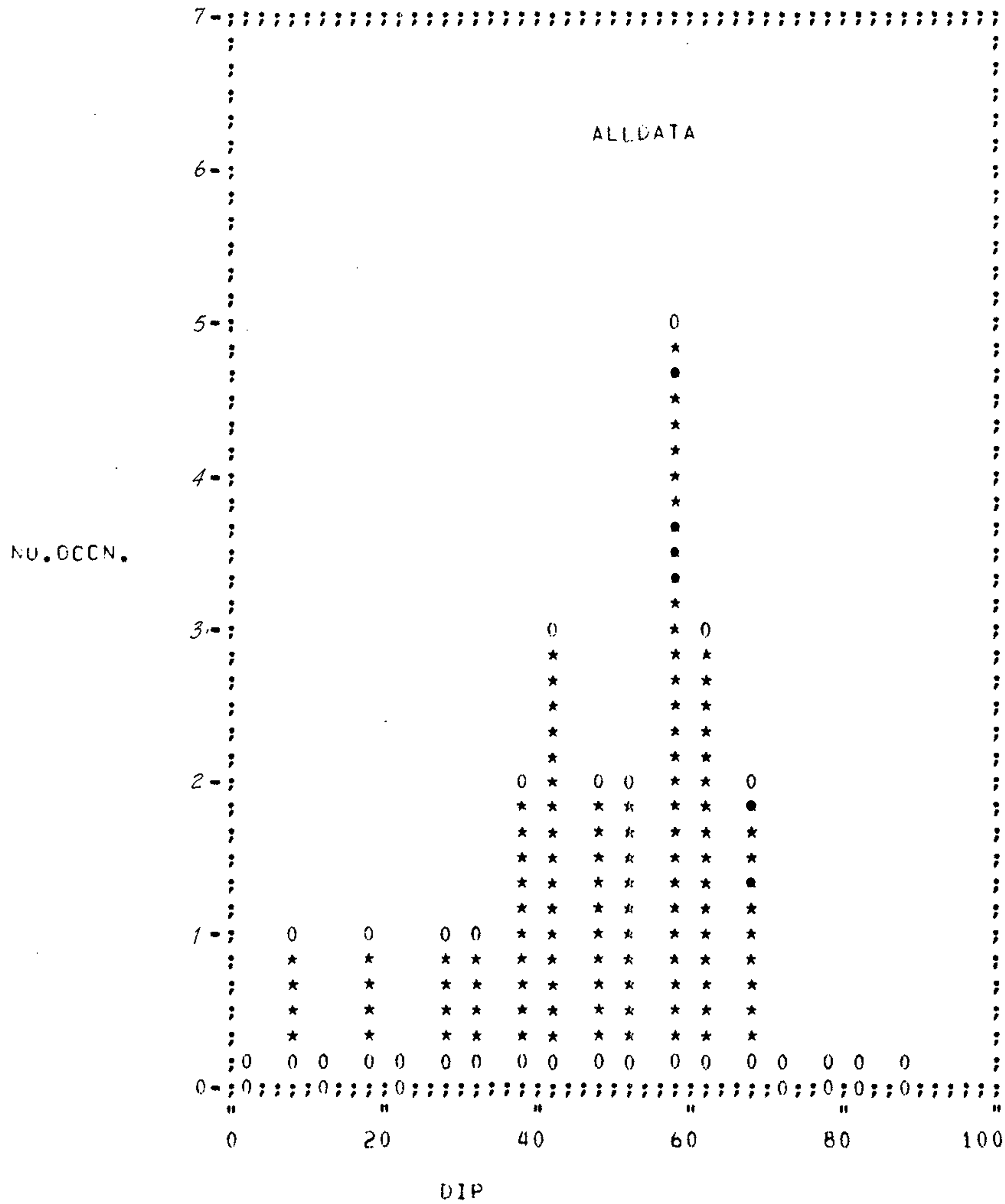
HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 77-846

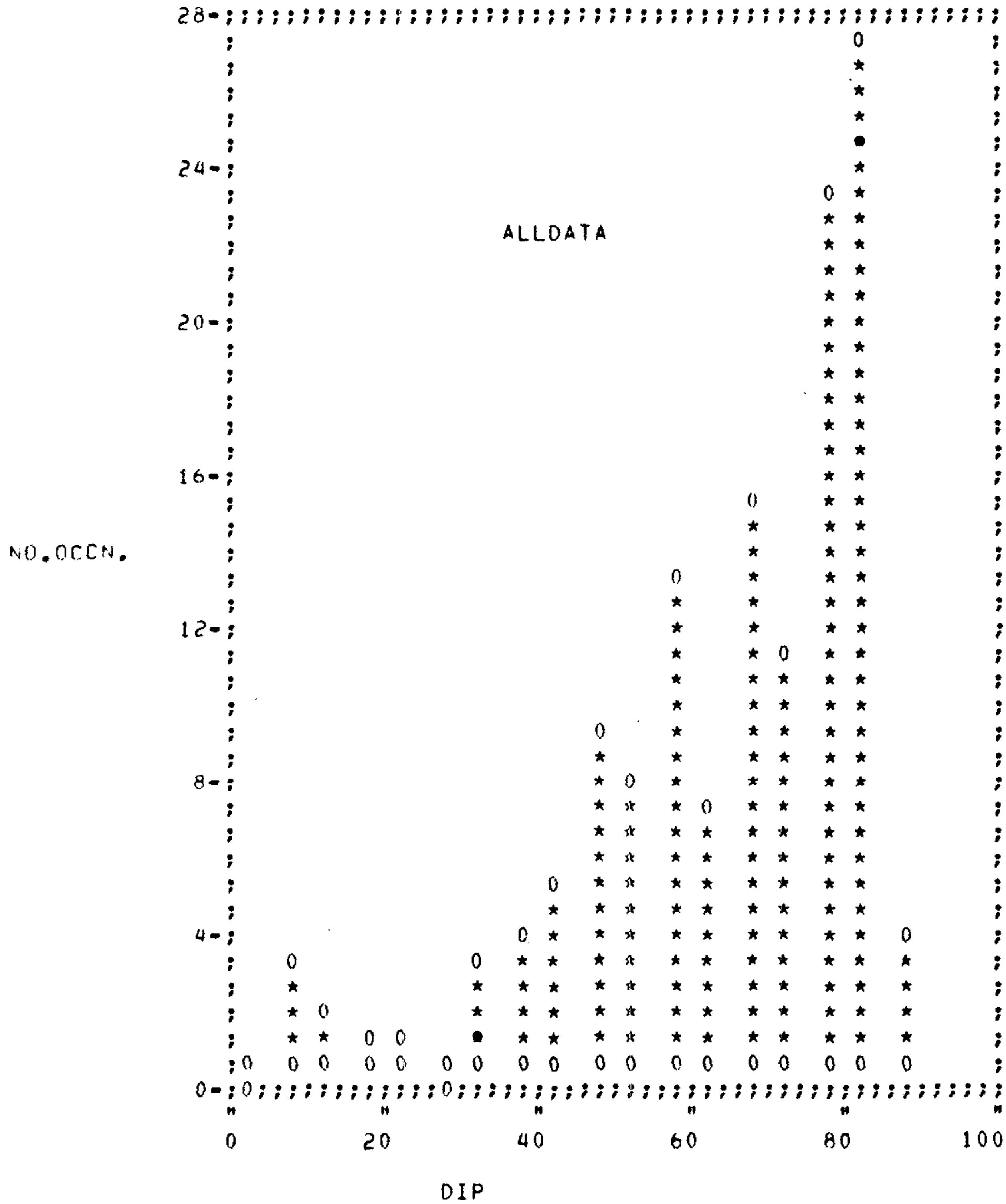
Figure **14**



PROJECT NO. 822-1524B DRAWN G.R. REVIEWED ...... DATE Nov. '82

HISTOGRAM OF DISCONTINUITY
DIP VALUES, EAST SIDE OF PIT
DDH 78-867

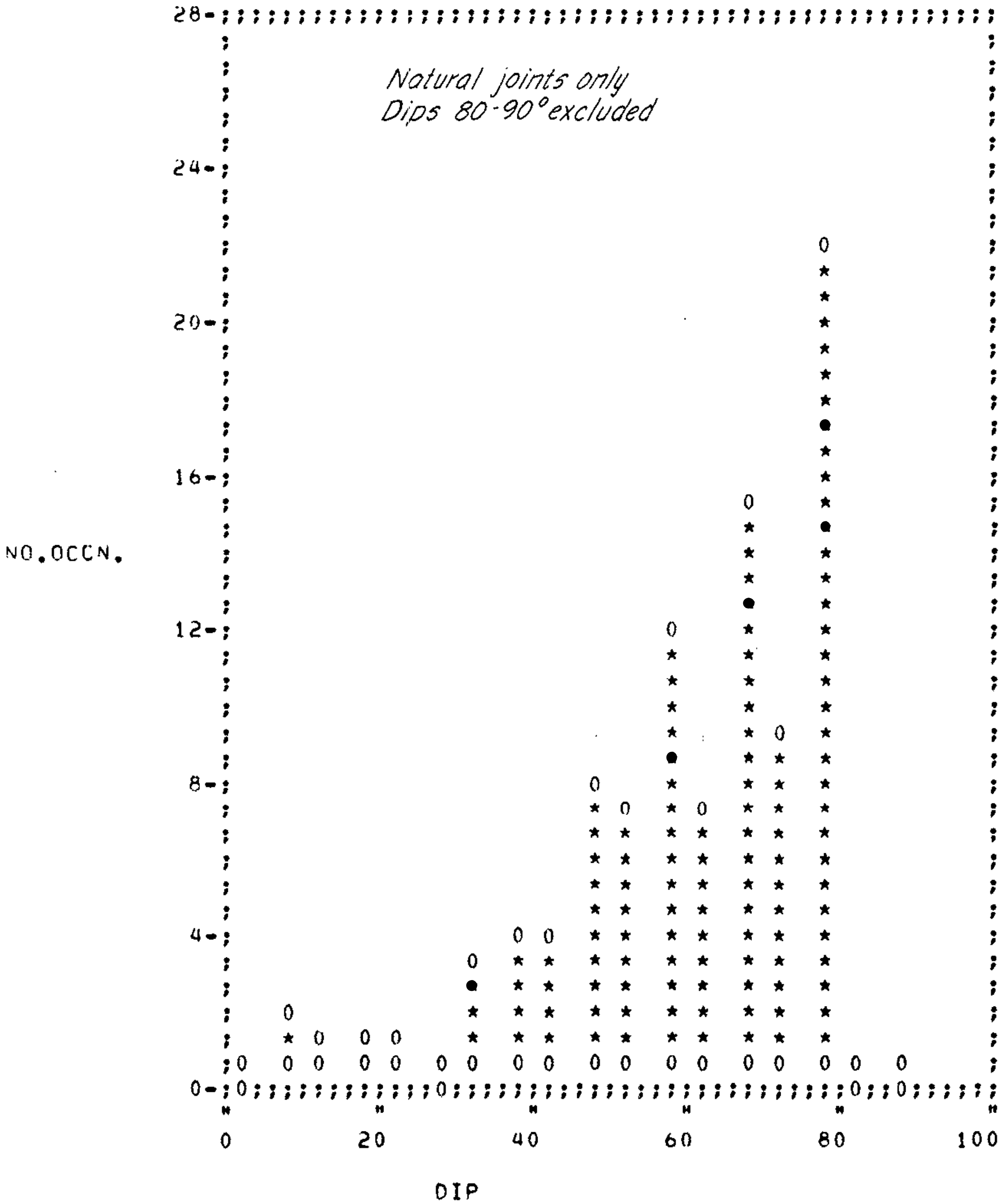
Figure **15**



PROJECT NO. 822-1524B DRAWN G.R. REVIEWED DATE Nov. '82

HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 78-867

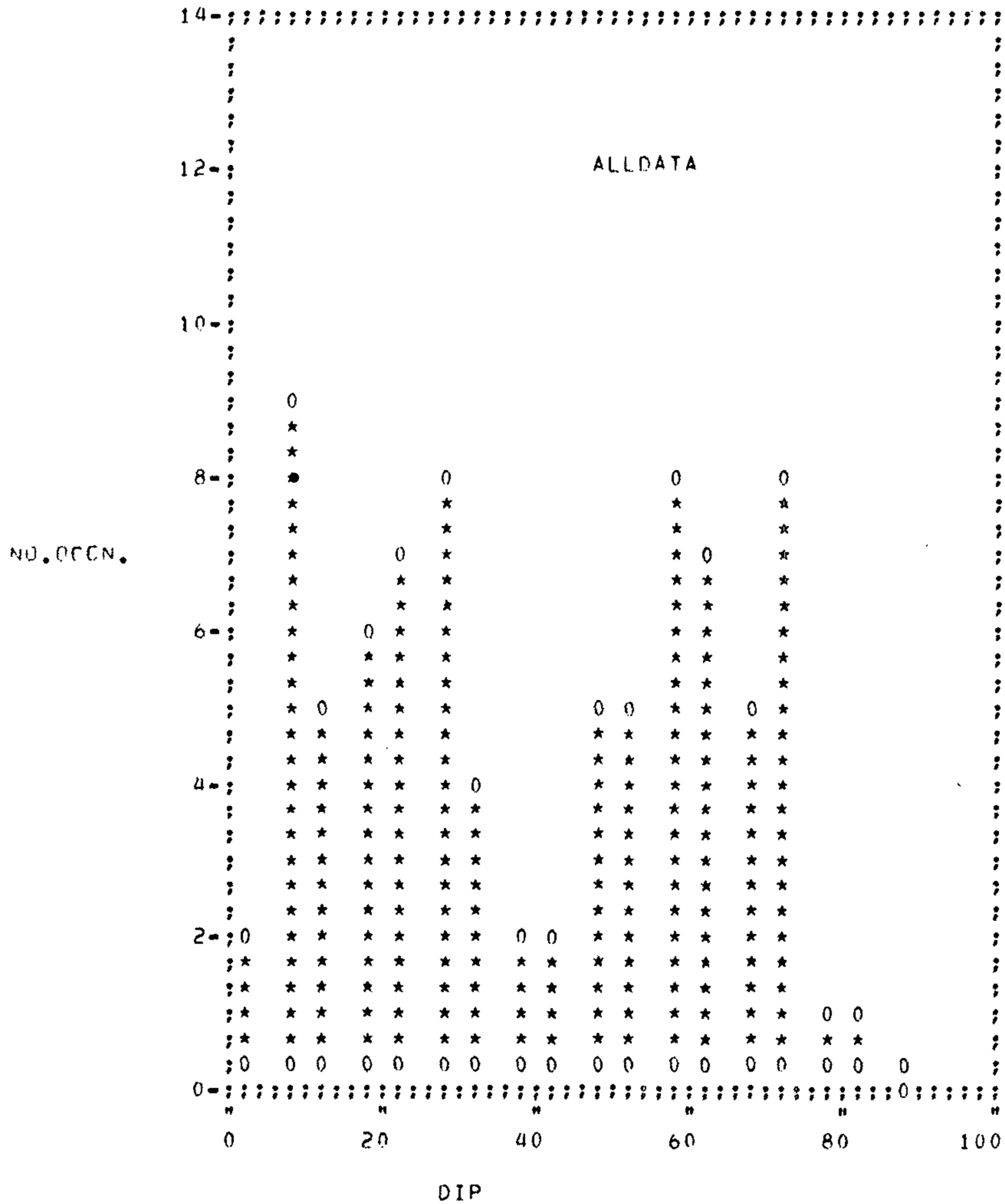
Figure **16**



PROJECT NO. 822-1524B DRAWN G.R. REVIEWED .... DATE Nov. '82

HISTOGRAM OF DISCONTINUITY DIP VALUES, EAST SIDE OF PIT DDH 77-870

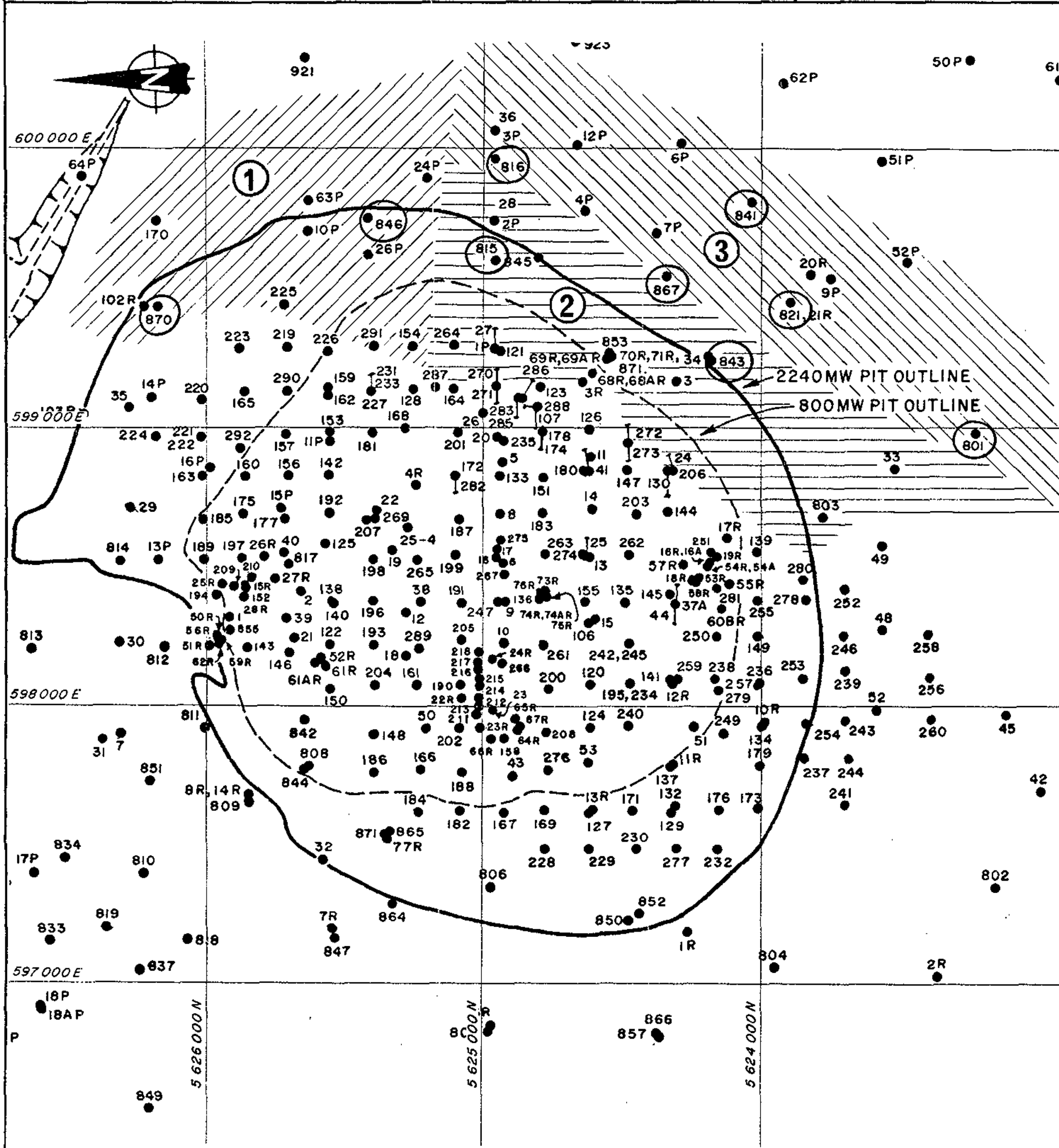
Figure **17**



PROJECT NO. 822-1524B DRAWN G.R. REVIEWED ...... DATE Nov. '82

POTENTIAL STRUCTURAL ZONES ON EAST PIT SLOPE

Figure **18**



Scale 1:20 000

(1) *Structural Zones*

(801) *Boreholes referenced in table 1.*

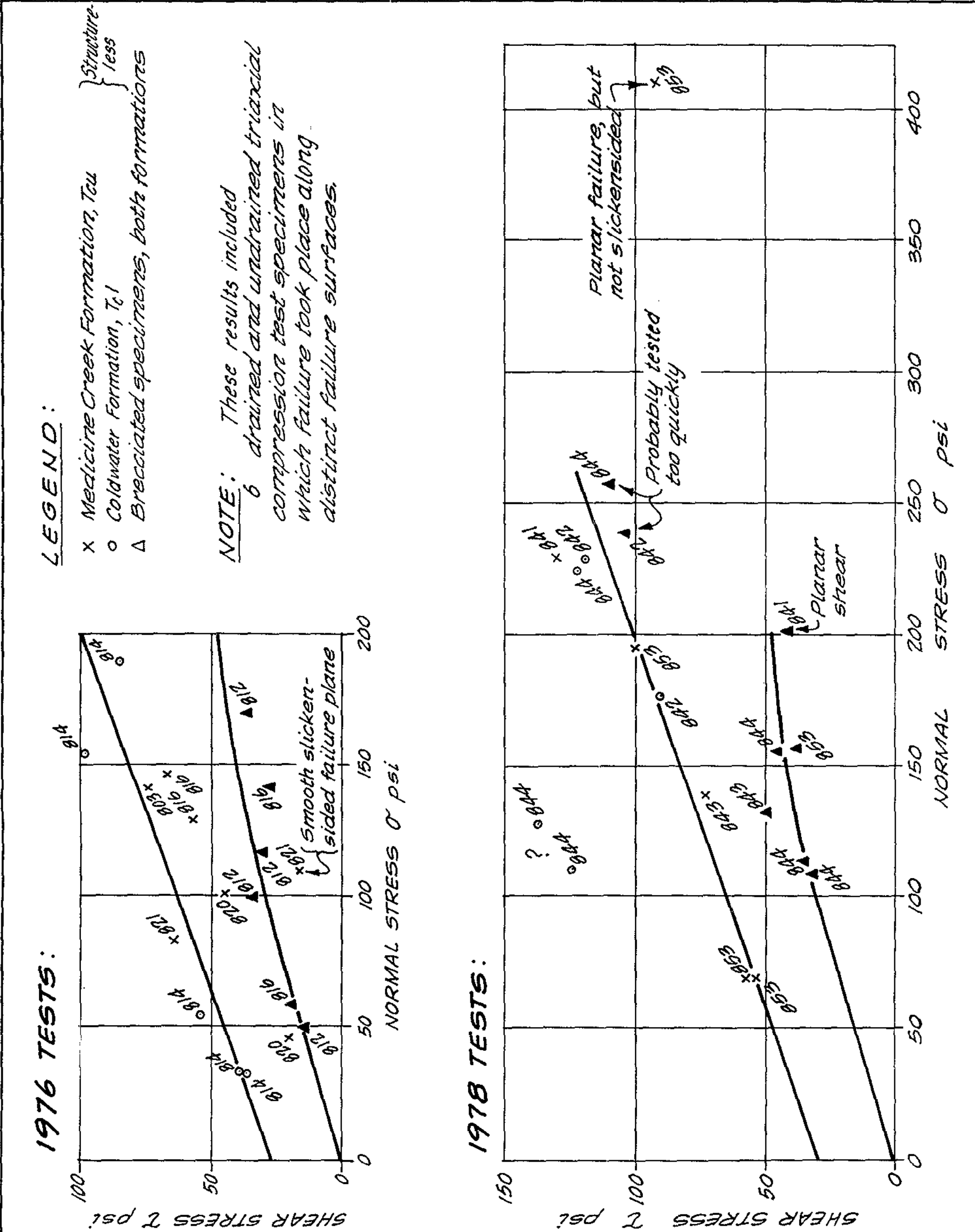
SCALE

0 500 1000 1500 METRES

PROJECT NO. 822-1523B DRAWN G.R. REVIEWED DATE Nov. '82

TRIAXIAL TESTS ON CLAYSTONE/SILTSTONE

Figure **19**



PROJECT NO. 822-1523B DRAWN lng. REVIEWED .... DATE Oct. '82

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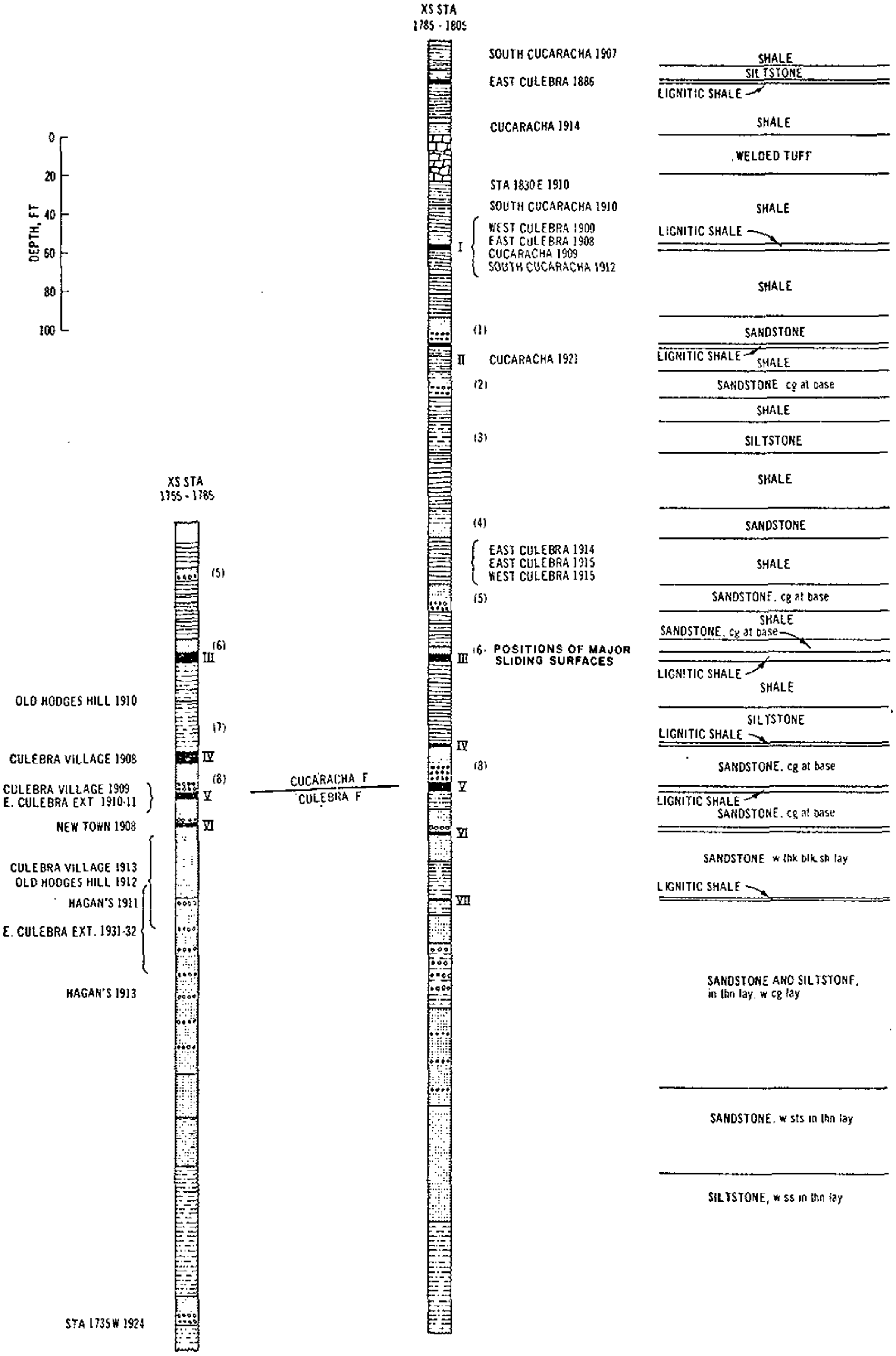
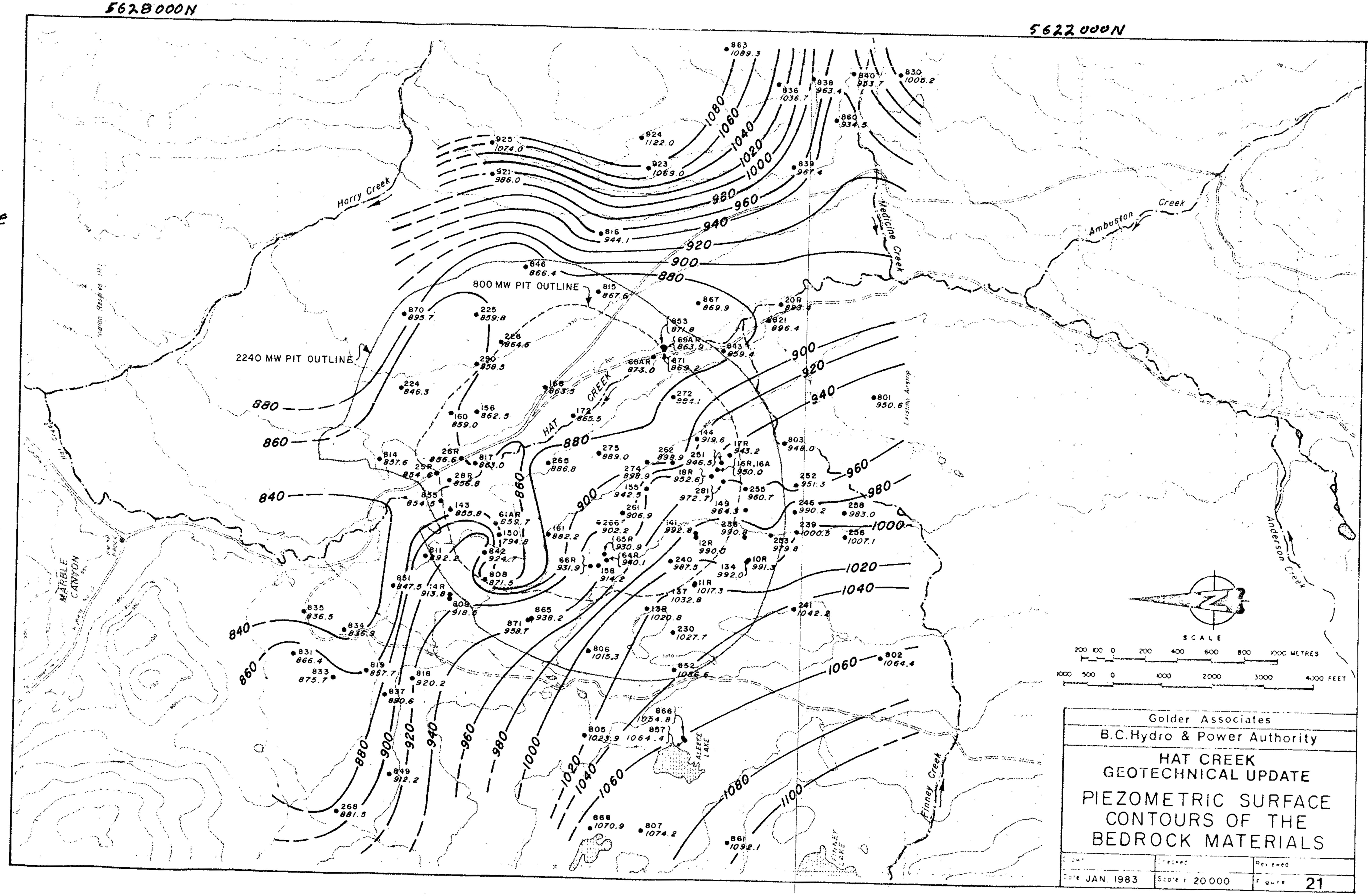
COMPOSITE STRATIGRAPHIC COLUMNS FOR CUCARACHA AND CULEBRA FORMATIONS IN CULEBRA REACH, PANAMA CANAL

Figure 20



PROJECT NO. 822-1524B DRAWN ... REVIEWED ... DATE Nov. '82



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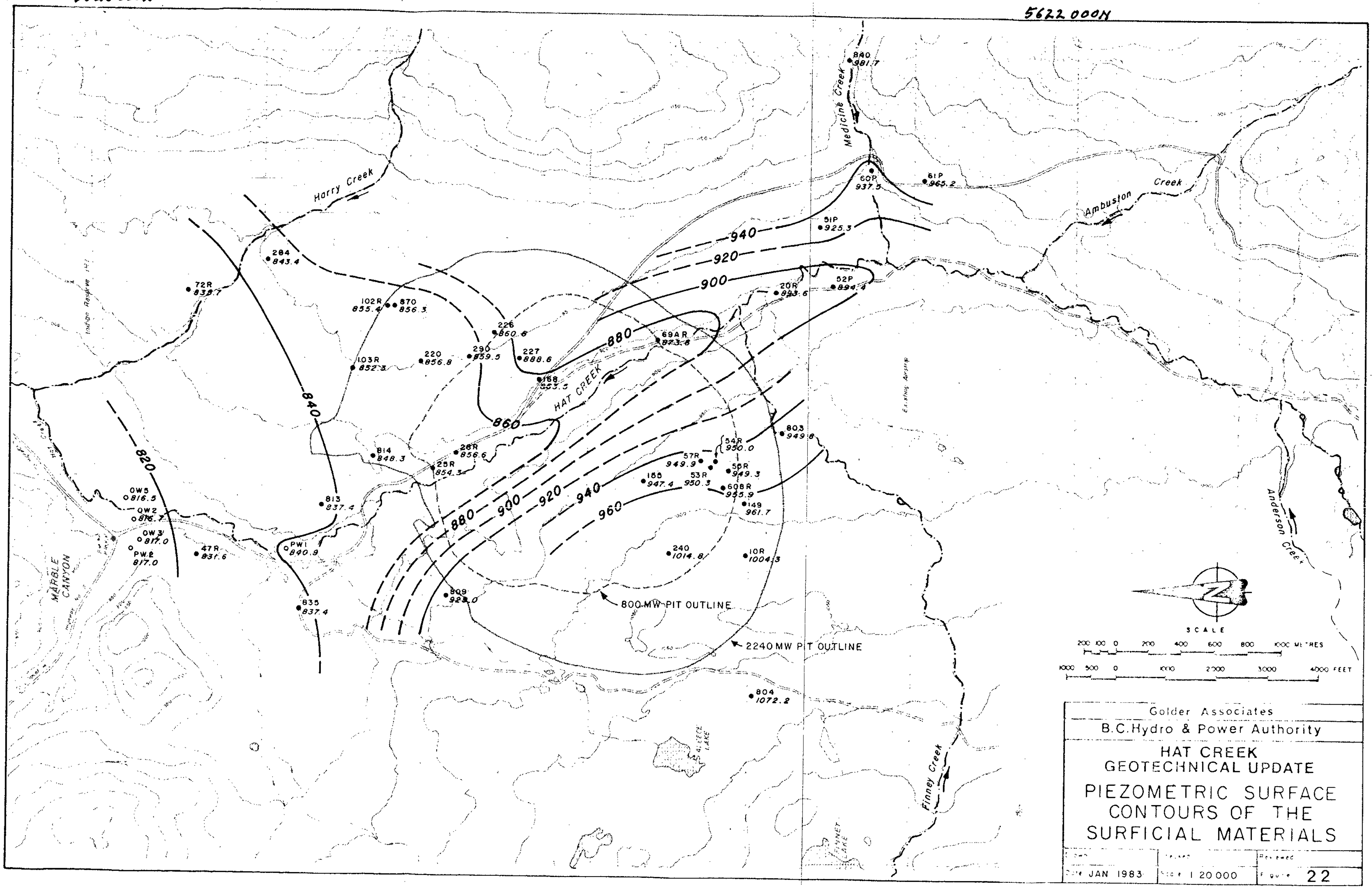
B.C.Hydro & Power Authority

HAT CREEK GEOTECHNICAL UPDATE

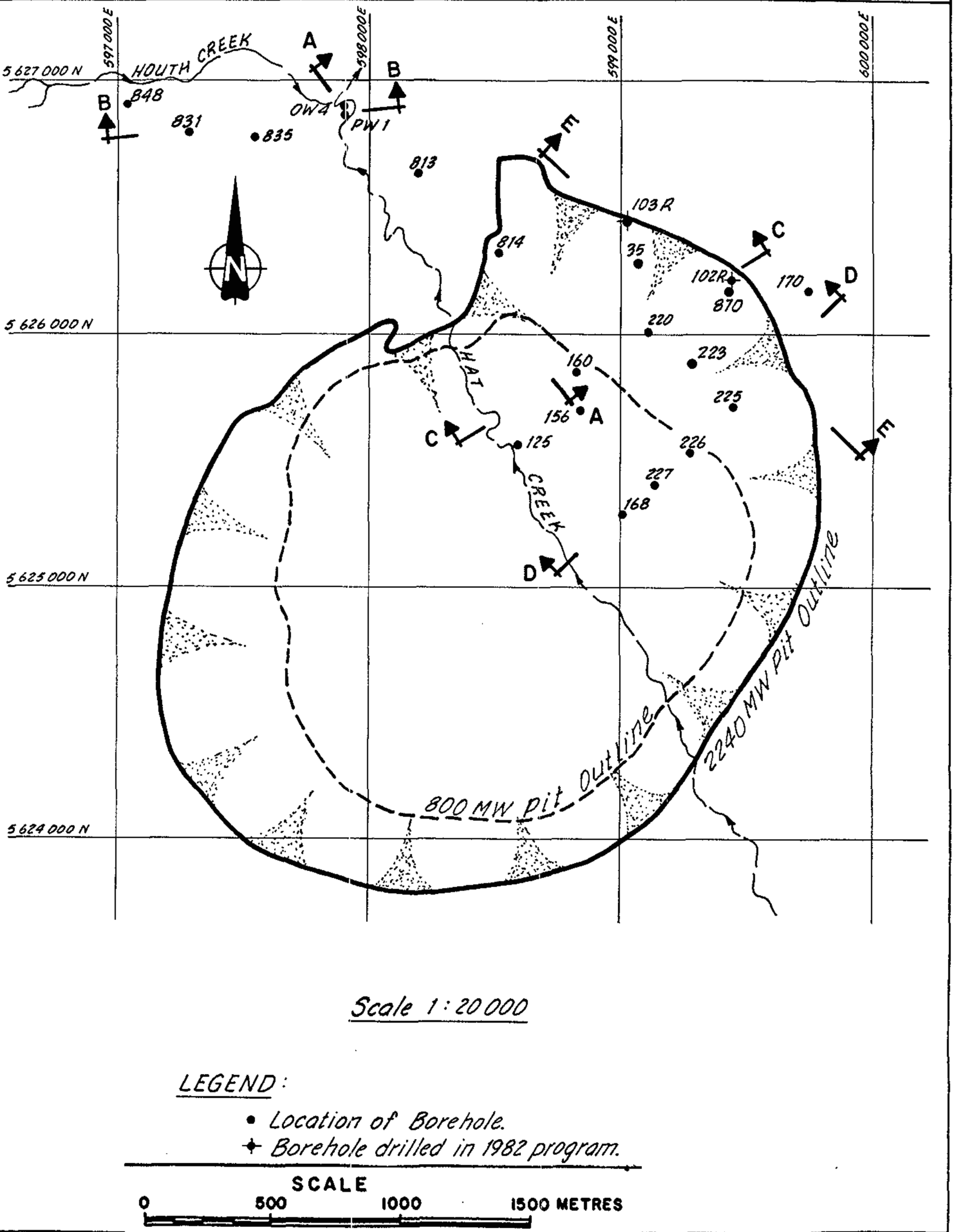
PIEZOMETRIC SURFACE CONTOURS OF THE BEDROCK MATERIALS

Date JAN. 1983 | Scale 1:20000 | Figure 21


5628000N
5622000N
600000E
598000E
Harry Creek
Medicine Creek
Ambuston Creek
HAT CREEK
Anderson Creek
Finney Creek
MARBLE CANYON
800 MW PIT OUTLINE
2240 MW PIT OUTLINE
820
840
860
880
900
920
940
960
SCALE
200 100 0 200 400 600 800 1000 METRES
1000 500 0 1000 2000 3000 4000 FEET
Golder Associates
B.C.Hydro & Power Authority
HAT CREEK
GEOTECHNICAL UPDATE
PIEZOMETRIC SURFACE
CONTOURS OF THE
SURFICIAL MATERIALS
Date JAN 1983
Scale 1:20 000
Figure 22

LOCATION OF HYDROGEOLOGICAL SECTIONS

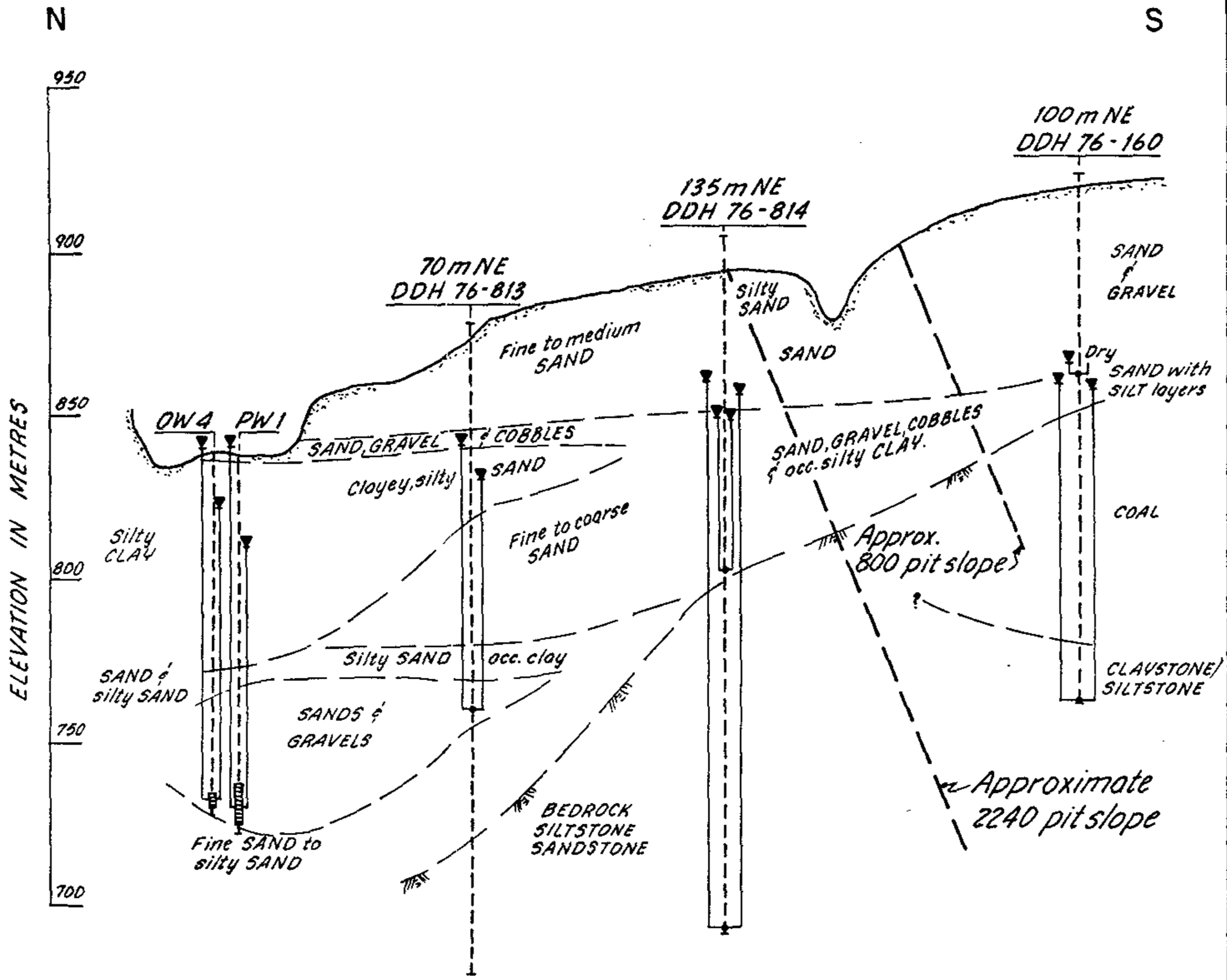
Figure 23



PROJECT NO. 822-1524B DRAWN BAD REVIEWED .... DATE Sept. '82

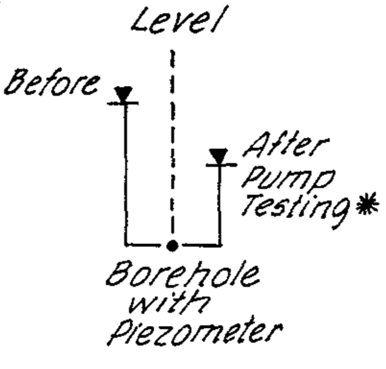
HYDROGEOLOGICAL SECTION A-A
HAT CREEK VALLEY

Figure **24**



Vertical Scale 1:2000
Horizontal Scale 1:10000

LEGEND:

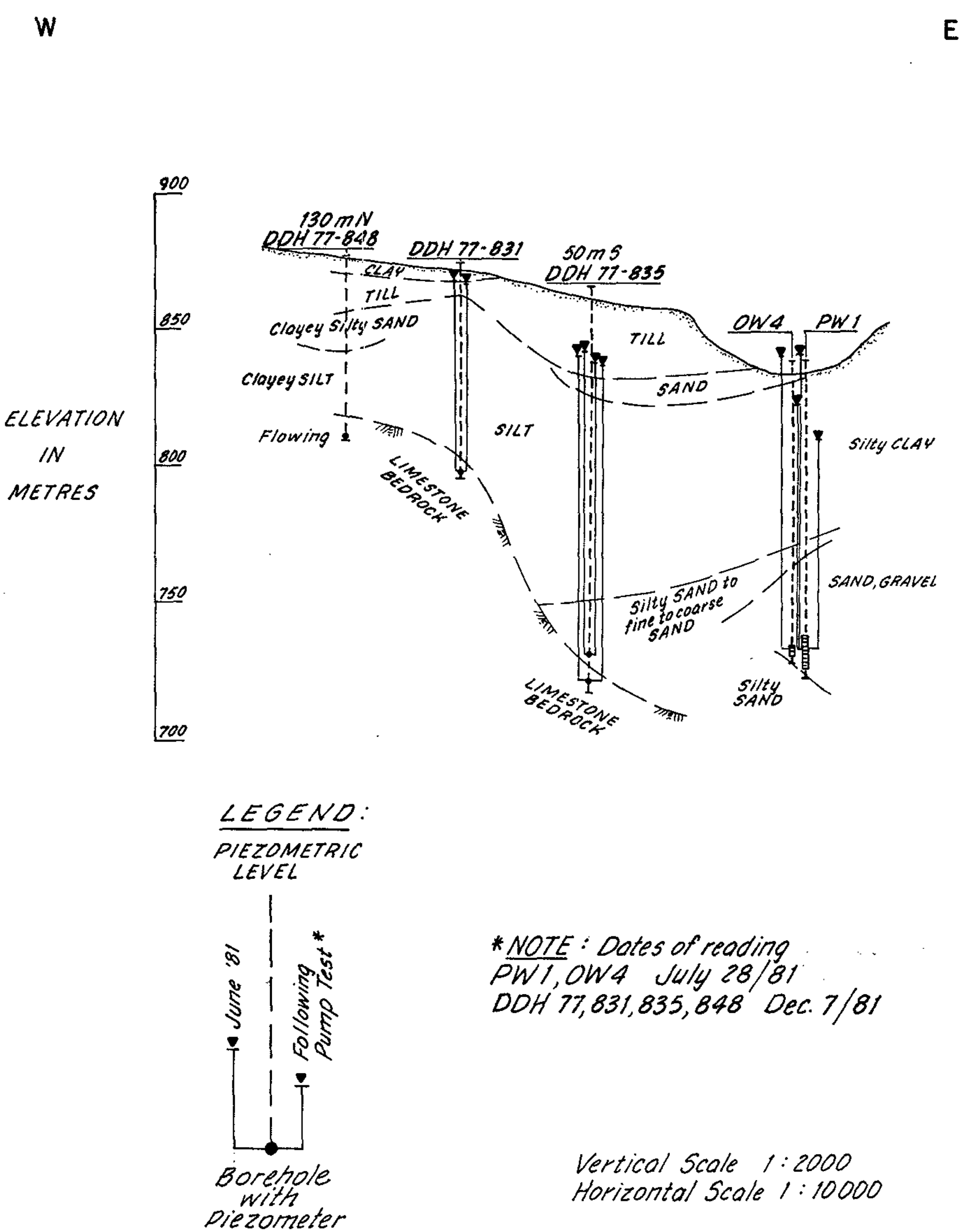


* NOTE: Dates of reading
OW4, PW1 July 28/81
DDH 76-813, 814, 160 Aug. 10/81

PROJECT NO. 822-1524B DRAWN BAD REVIEWED ...... DATE Sept. '82

HYDROGEOLOGICAL SECTION B-B
HOUTH MEADOWS - HAT CREEK

Figure **25**



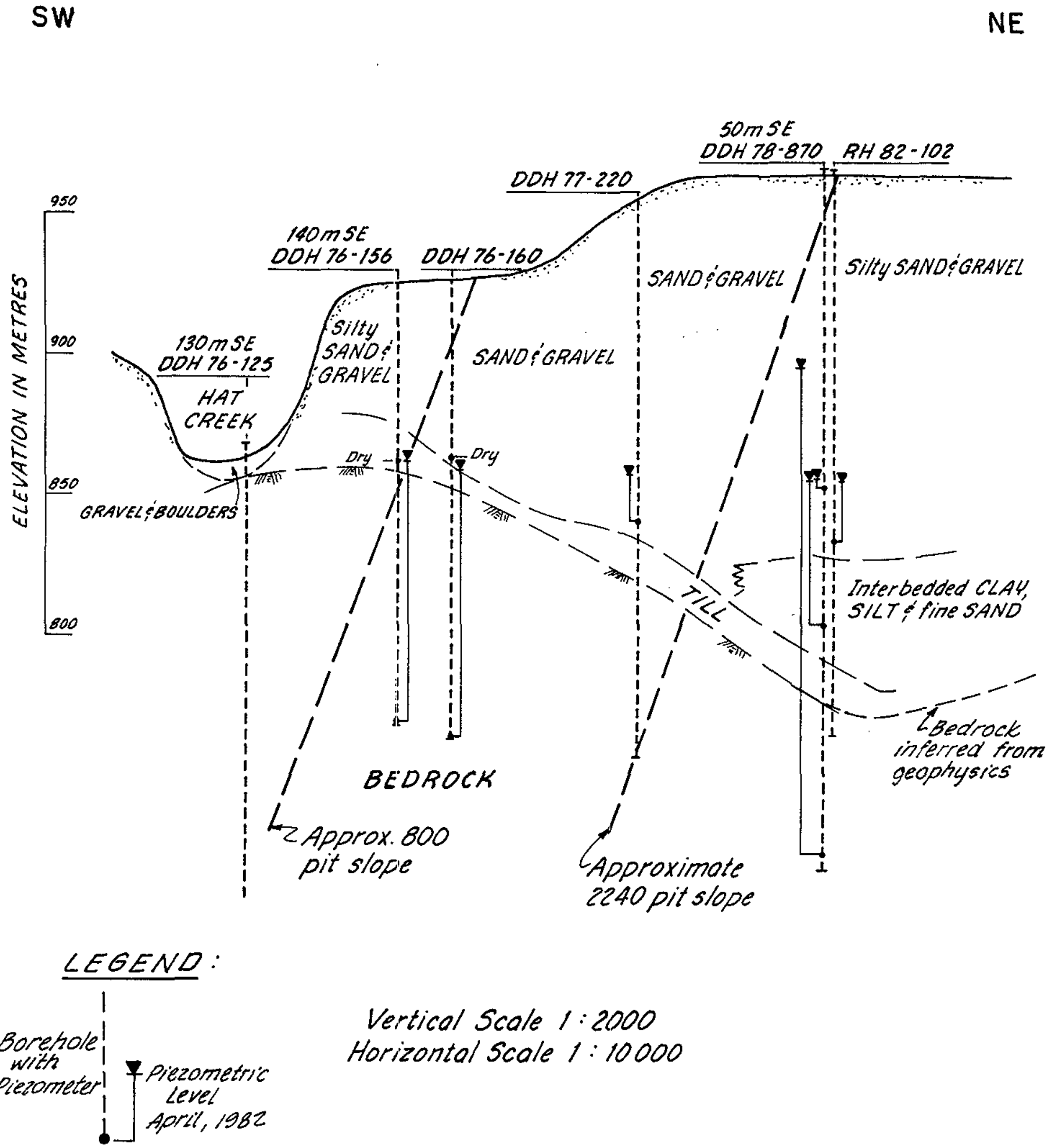
*NOTE : Dates of reading
PW1, OW4 July 28/81
DDH 77,831,835,848 Dec. 7/81

Vertical Scale 1 : 2000
Horizontal Scale 1 : 10000

PROJECT NO. 822-1524B DRAWN BRD REVIEWED .... DATE Sept. '82

HYDROGEOLOGICAL
SECTION C-C NE PIT SLOPE

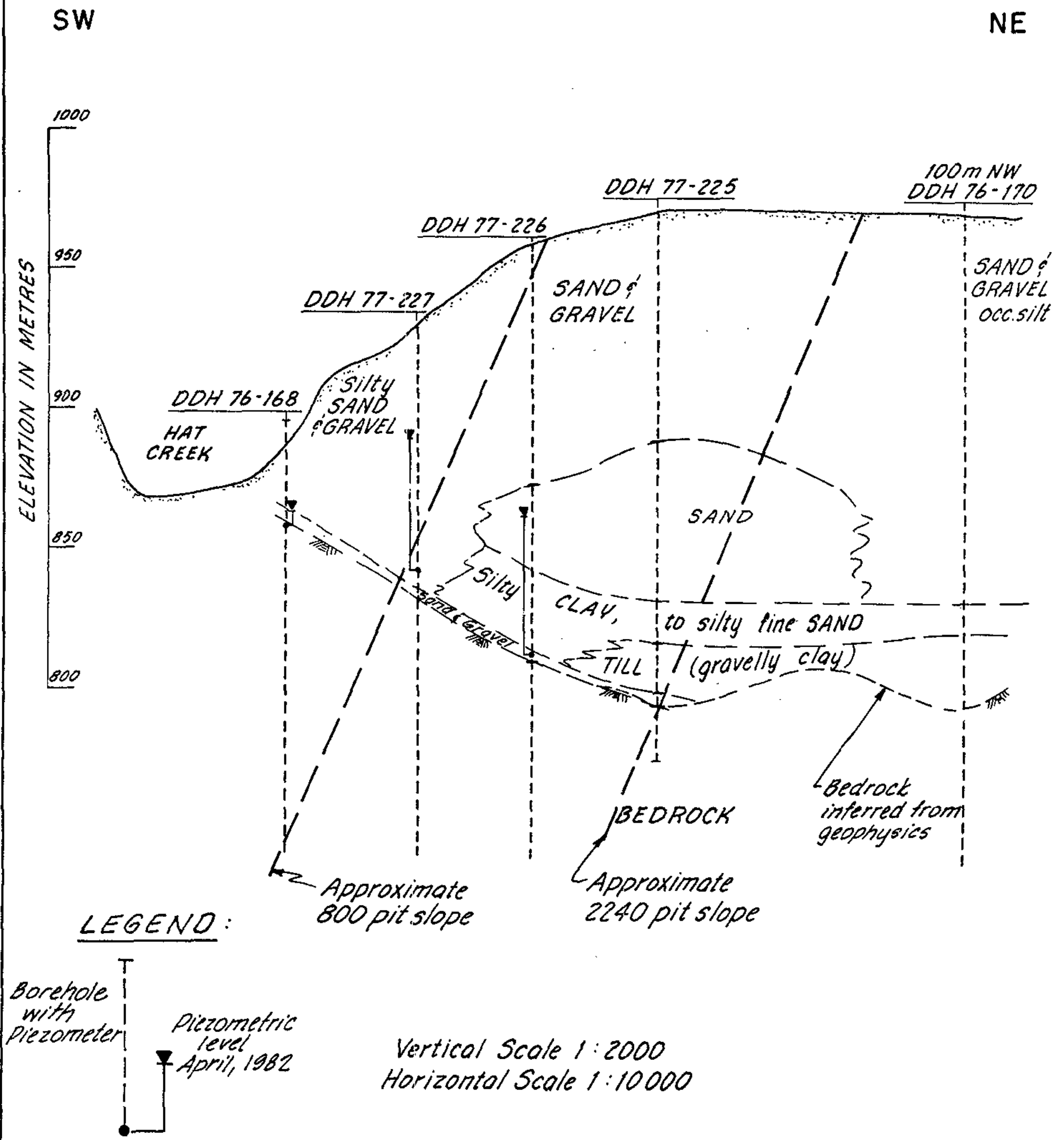
Figure **26**



PROJECT NO. 822-1524B DRAWN BAD REVIEWED DATE Sept. 82

HYDROGEOLOGICAL
SECTION D-D NE PIT SLOPE

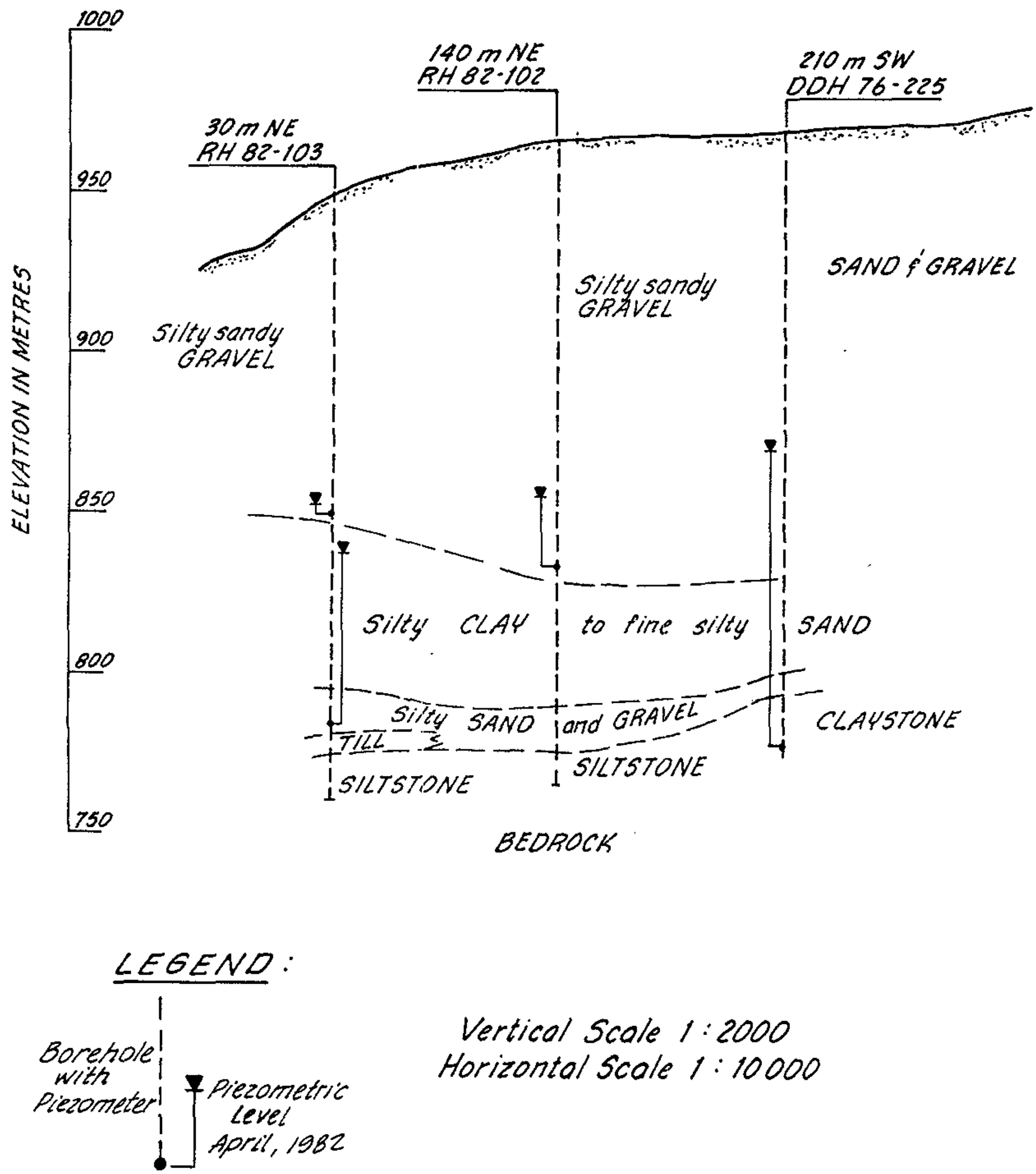
Figure 27



PROJECT NO. 822-1524B DRAWN BAD REVIEWED .... DATE Sept. '82

HYDROGEOLOGICAL
SECTION E-E NE PIT SLOPE

Figure **28**

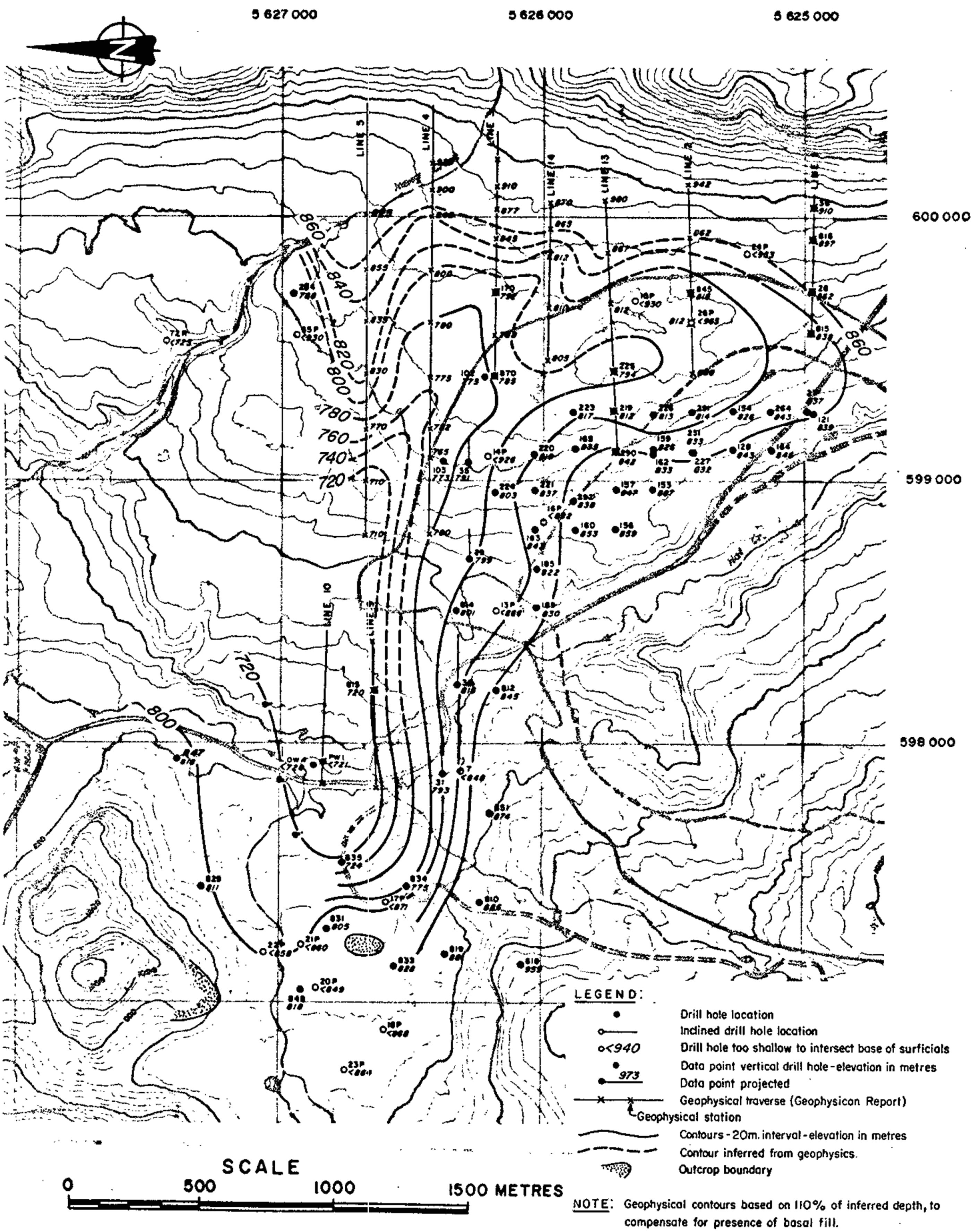


Vertical Scale 1 : 2000
Horizontal Scale 1 : 10 000

PROJECT NO. 822-1524 B DRAWN BAD REVIEWED ...... DATE Nov. '82

CONTOURS ON THE BASE OF THE SURFICIALS - NORTHERN PIT RIM.

Figure 29

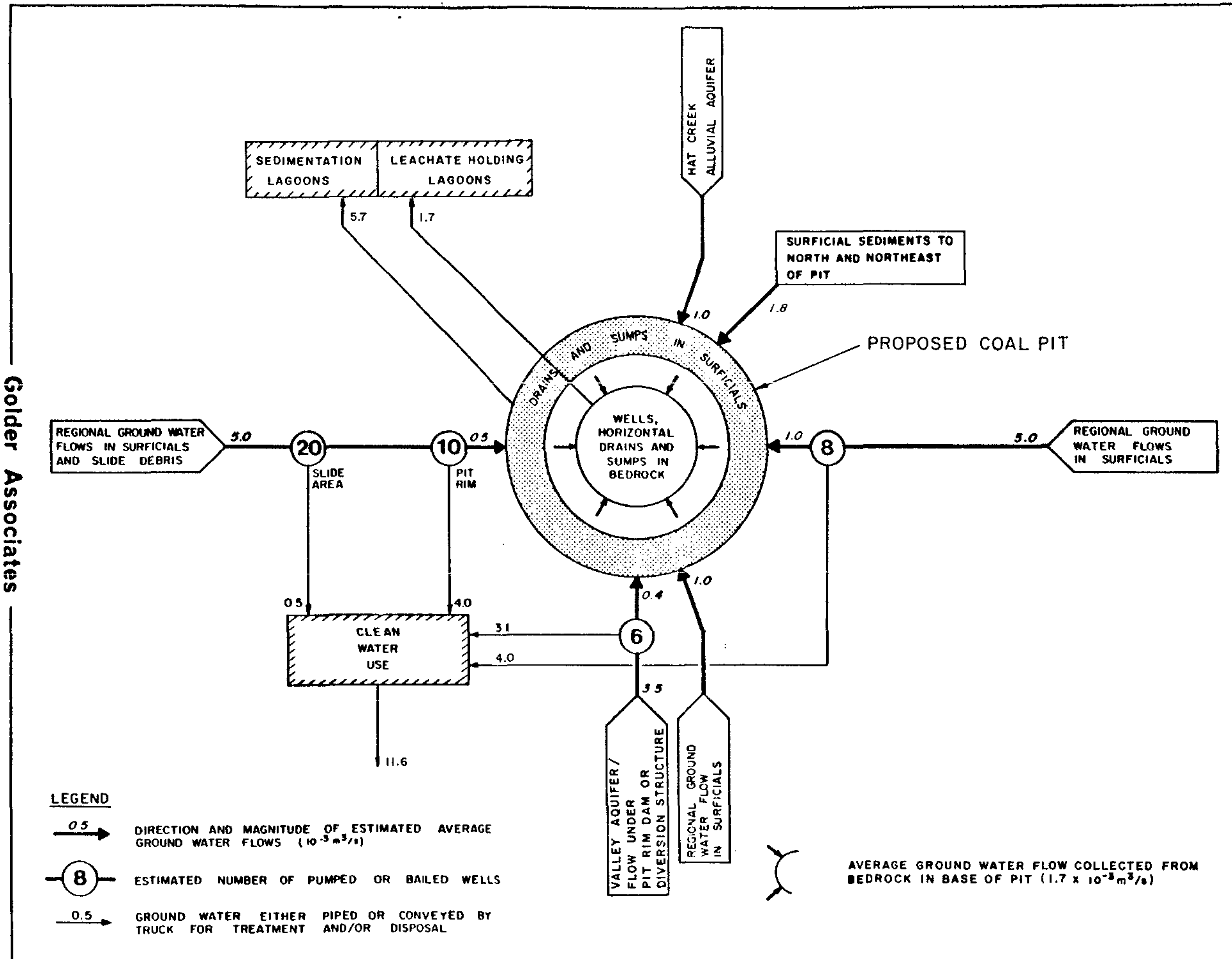


PROJECT NO. 822-1522AB DRAWN G.A. REVIEWED .... DATE Nov. '82

Golder Associates

SCALE 1:20 000

Project No .... Drawn .... Reviewed GL ..... Date ....

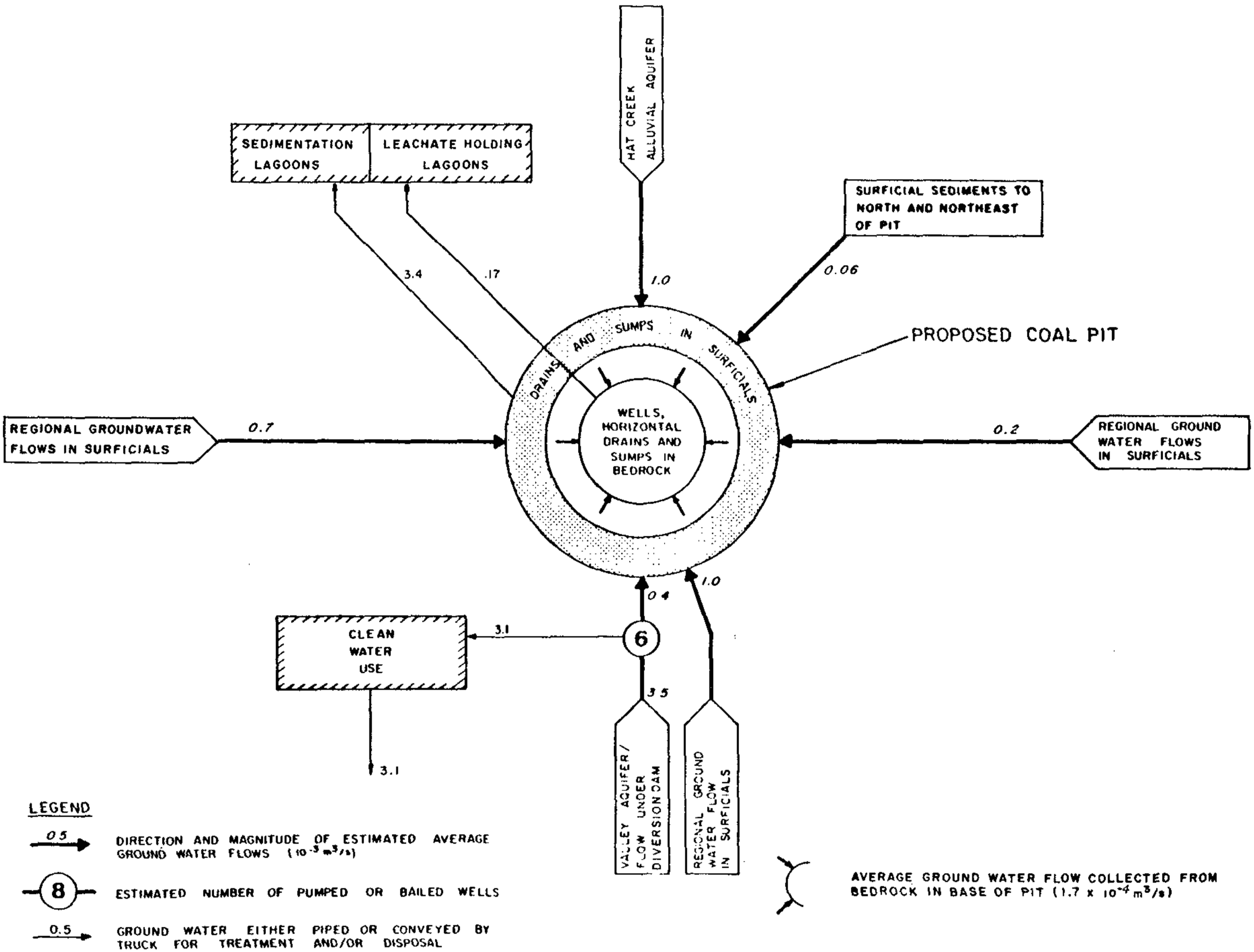


MINE SEEPAGE AND DEWATERING FLOW CHART
2240 MW SCHEME

Figure 30

Golder Associates

Project No .... Drawn ..... Reviewed ..... Date ....



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MINE SEEPAGE AND DEWATERING FLOW CHART
800 MW SCHEME

Figure 31

CONFIDENTIAL

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT
TO
B.C. HYDRO
ON THE
HAT CREEK PROJECT
GEOTECHNICAL AND HYDROGEOLOGICAL UPDATE, FALL 1982
CENTRAL BRITISH COLUMBIA

VOLUME II - APPENDICES

CLS 2752, 2755, 2756

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GEOLOGICAL BRANCH
ASSESSMENT REPORT

December, 1982 00 145 822-1524

6 of 7.

LIST OF APPENDICES

APPENDIX A

STABILITY OF MEDICINE CREEK WASTE DUMP
UNDER SEISMIC LOADING
(Letter from N.A. Skermer to B.C. Hydro)

Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

COPY

E/81/2049

November 16th, 1981

Dr. G.F. Lange, P. Eng.
Director of Mining
Thermal Division
B.C. Hydro and Power Authority
P.O. Box 12121
555 West Hastings Street
Vancouver, B.C.
V6B 4T6

Re: Hat Creek Project
Seismic Design Criteria

Dear Sir:

This letter summarizes our findings on pseudo-static stability analyses of major waste dump slides under earthquake loadings as suggested in our letter of October 8th, 1981, and later verbally authorized by you. The object of the analyses was to provide Klohn Leonoff with an indication of the sensitivity with respect to horizontal earthquake accelerations.

We analyzed the Medicine Creek waste dump on what is believed to be a critical section through the south abutment, see Figure 1 attached. The maximum volume dump was analyzed, although I realize that your present mining plan calls for a smaller (and therefore less critical) dump in Medicine Creek. Our analysis is for a retaining embankment crest elevation of 1200 m. The section analyzed is shown on Figure 2. The slip surface passes through the clay waste at the base of the dump and emerges downhill through the foundations of the retaining embankment. The input strength parameters for the clay waste are as given on Figure 11 of our 1978 report Volume 1, and the strength of the foundations been approximated by $\emptyset'=30$ degrees, $C'=0$. These latter parameters represent post-peak strength conditions, although I would like to confirm the results for tests carried out at higher normal stresses than we tested previously. We shall be writing to you in that connection in a separate letter. The analysis was computed using Sarma's method (Geotechnique, 1973, No. 3). The results are as follows:

(a) The lowest static factor of safety was 2.45. A massive slip involving the whole of the waste was more critical than a smaller slip.

(b) A factor of safety of 1.0 was reached at a seismic coefficient of 0.13, i.e. a horizontal earthquake acceleration of 13 per cent gravity.

As stated by Terzaghi and later confirmed by Seed in his Rankine lecture (Geotechnique, 1979, No. 3), pseudo-static analyses are applicable for soil conditions where large pore water pressures do not build up and substantial strength losses do not occur during earthquake shaking. Provided that the waste retaining embankments are properly drained, these conditions will be satisfied at Hat Creek: strength loss has been allowed for in the foundation rocks by inputing post-peak strength parameters. Seed concludes that given these conditions computed displacements, for most earthquakes producing embankment crest accelerations less than 0.75 g, will be acceptable. Seed further concludes that to ensure acceptably small embankment displacements, one can design for a factor of safety of 1.15, and the following design criteria:

Earthquake Magnitude	Seismic Design Coefficient
6.5	0.10
8.25	0.15

Referring to Figure 2, you will se that for an FS=1.15, the seismic design coefficient that can be tolerated is 0.11, which is comparable to an earthquake magnitude just in excess of 6.5.

The question that should be directed to Klohn Loenoff, therefore, is whether the maximum credible earthquake at Hat Creek is much in excess of 6.5. If it is not, the analysis need be carried no further; if it is we should perhaps carry out a more detailed displacement analysis.

By copy of this letter we are requesting Klohn Leonoff's best estimate of the maximum credible earthquake.

Yours very truly,

GOLDER ASSOCIATES

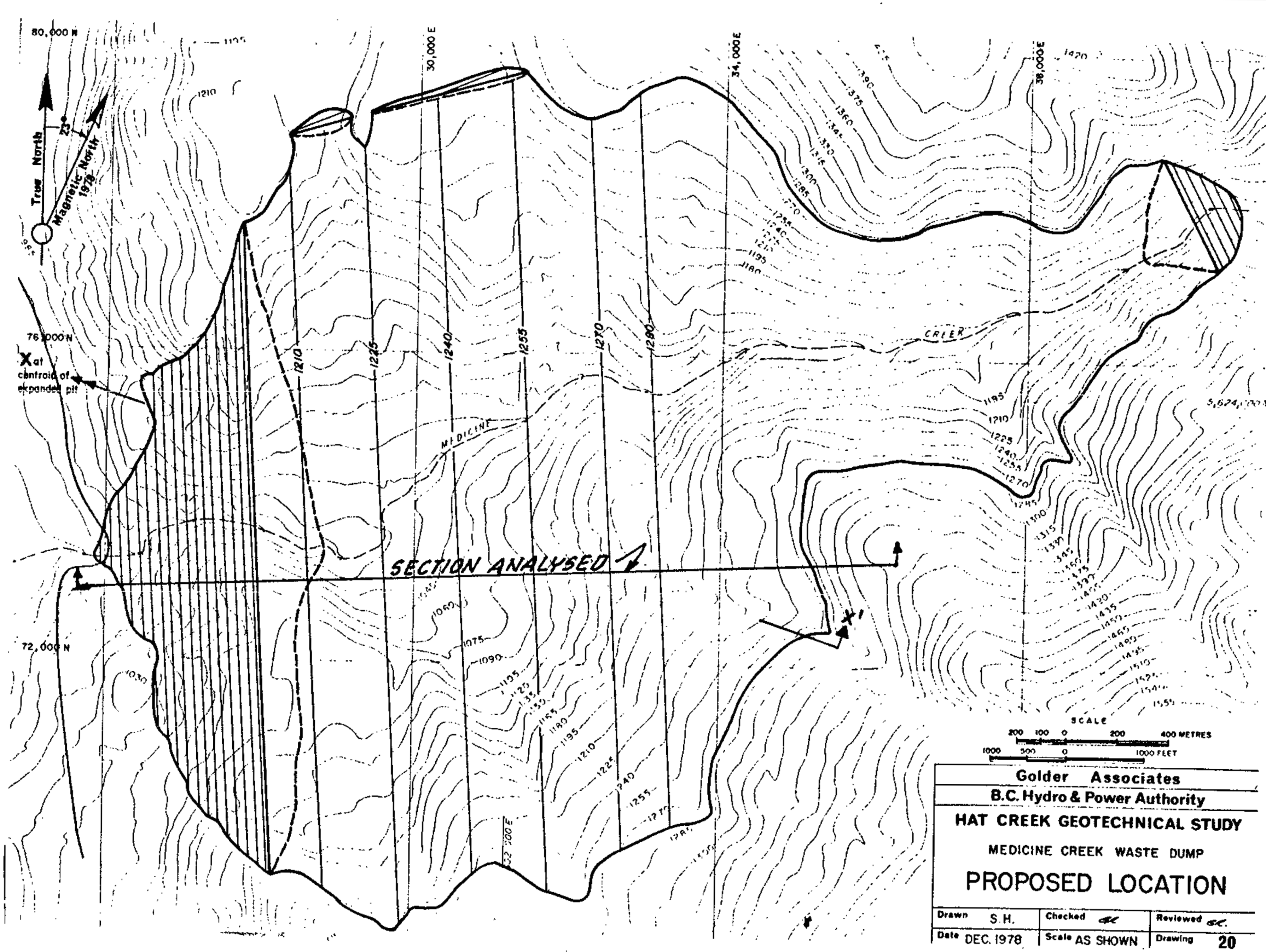
G.E. Rawlings

for. N.A. Skermer, P. Eng.

NAS/sek
812-1543

cc: Mr. R.G. Charlwood
Klohn Leonoff Ltd.

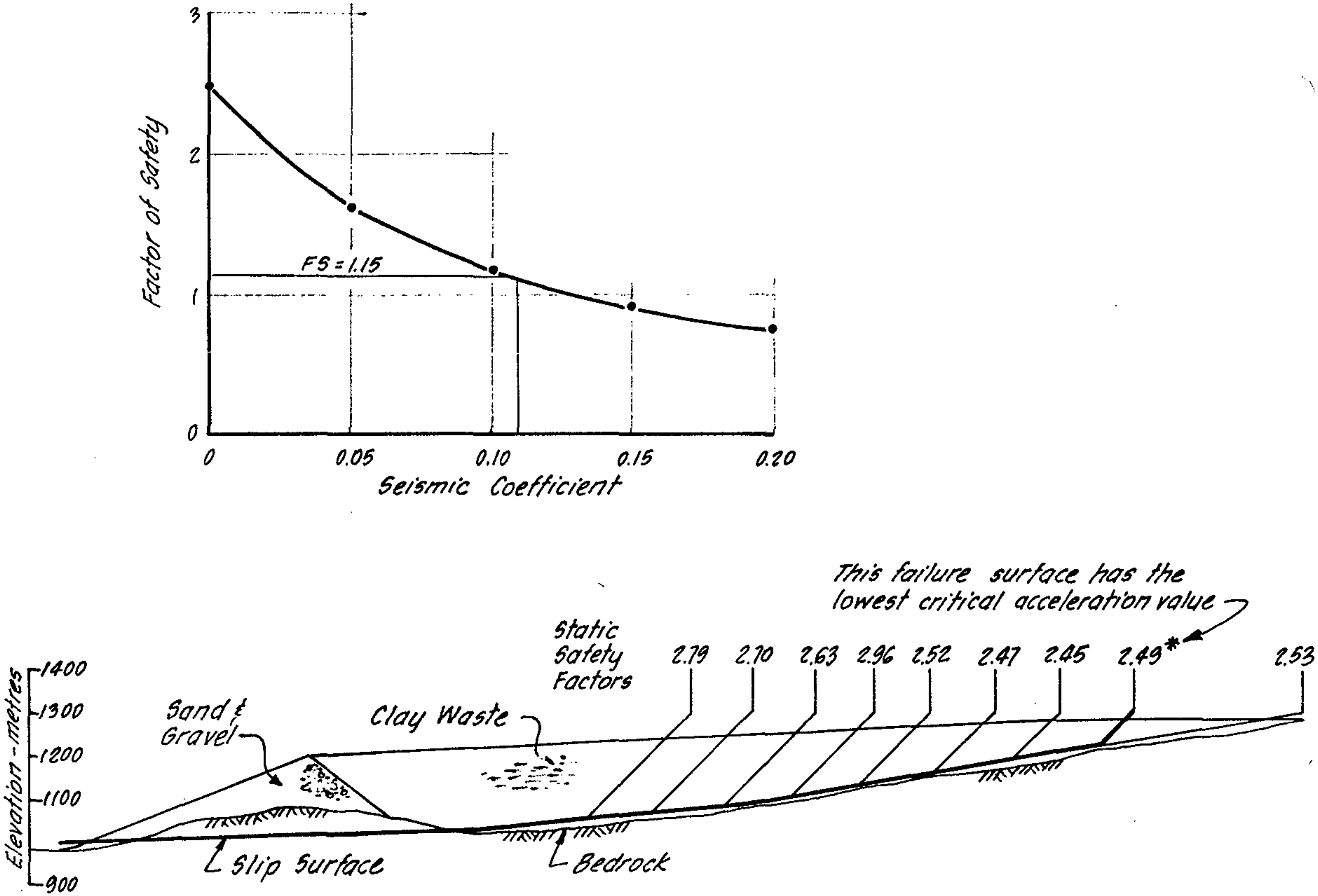
PROJECT NO. 812-1543 DRAWN ... REVIEWED MAS DATE NOV. '81



Golder Associates

Figure 1

PROJECT NO. 812-1543 DRAWN dh REVIEWED NAS DATE Nov. '81



Golder Associates

SEISMIC ANALYSIS

Figure 2

APPENDIX B

800 MW PIT SLOPES

(Letter from N.A. Skermer to B.C. Hydro)

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

COPY

E/82/2209

July 30th, 1982

B.C. Hydro
P.O. Box 12121
555 West Hastings Street
Vancouver, B.C.
V6B 4T6

ATTENTION: Dr. G.F. Lange
Director of Mining

Re: Hat Creek Project
800 MW Pit

Dear Sir:

Further to our meetings of July 12th to 16th, 1982, we summarize below our recommendations for pit slope design criteria and mine dewatering for the proposed alternative 800 MW Pit. The final pit outline of the proposed 800 MW is shown attached.

Pit Slope Design Criteria

We have re-examined in some detail the strength test data that we used to arrive at the design slope angles given in our report - "Preliminary Engineering Work - Geotechnical Study 1977 to 1978" (Golder Associates, December 1978). The slope angles recommended then were as follows:

Surficial deposits (other than slide debris)	25°
Slide debris	16°
Coal	25°
Medicine Creek & Coldwater - Tcu and Tcl	20°

Basically we feel that these criteria remain valid for both the 2000 MW Pit and the proposed 800 MW Pit, except that slopes which lie entirely in coal and are reasonably well drained, could probably be designed at 30 degrees. This condition would apply along the south wall where the coal plunges to the south. This improvement is partially as a result of our re-evaluation of the strength of the coal (although further tests would need to be done on the coal for final design). It also results from the smaller slope height. The 800 MW Pit bottom is at elevation 730 m compared to 640 m for the original 200 MW Scheme.

Application of the above criteria to the 2000 MW Pit resulted in the recommended pit slope angles shown on Figure 12 of the above mentioned report. Many of the final pit slopes were in Coldwater and Medicine Creek Formations. The resulting slope angle was 20 degrees.

The 800 MW Pit is different. The final slopes are in general in coal. A substantial thickness of coal would overlie the siltstones and claystones, and it would offer much improved toe support to the slopes. The slopes would intercept little of the Upper Siltstone (Medicine Creek) Formation. Little or no slide debris would be intercepted. As a result, we are recommending that the overall pit slope angles could be designed for a uniform 25 degree angle.

In a more refined design, it would be necessary to flatten the slope on the northwest sector to 20 degrees, since in that corner of the pit the final slope be in Lower Claystone/Siltstone. On the other hand on the south wall of the pit the angle could be increased to 30 degrees. Hence an average for preliminary feasibility purposes of 25 degrees all round is probably valid.

The same dynamic slope criteria, shown on Figure 16 of our 1978 report, would apply to the proposed 800 MW Pit.

The 800 MW Pit should be excavated in such a manner that it would not preclude excavation of the 2000 MW Pit utilizing the concept of depressurization by excavation.

Dewatering Wells

For the 2000 MW Scheme reported in 1978, an allowance was made for 61 wells in the surficials around the pit perimeter and in the slide areas, 20 wells in the coal and 10 additional observation wells.

The 800 MW Pit would not intercept the slide zones and therefore 20 wells allowed for in those areas could be eliminated. Since the 800 MW Pit is smaller, we would allow for only 15 wells in the coal. The 10 observation wells should be retained.

We would suggest therefore that you allow for a total of 66 wells in the 800 MW Pit.

Yours very truly,

GOLDER ASSOCIATES

for N.A. Skermer, P. Eng.

NAS/bjh
822-1549

Encl.

APPENDIX C

REPORT BY PROFESSOR P.W. ROWE

on

HAT CREEK PROJECT - GEOTECHNICAL REASSESSMENT

PROFESSOR P. W. ROWE
D.SC., F.I.C.E.

COPY

STYPERSON HOUSE,
ADLINGTON,
MACCLESFIELD,
CHESHIRE,
SK10 4JX.
TEL: 0625 72170.

YOUR REF. E/82/964

4 Sept. 1982

Messrs Golder Associates, (Western Canada) Ltd,
224 West 8th Avenue,
Vancouver.
British Columbia.
V5Y 1N5

Dear Sirs,

Hat Creek Project.

I have pleasure in enclosing my report on aspects of this site investigation and I do hope that you may find something in it constructive towards any future work you may be called upon to undertake. Also on instruction from Mr. Skermer I enclose an account of fees and expenses and trust you find these in order.

I enjoyed the visit to Vancouver immensely and regard it as a privilege to have had the opportunity to discuss aspects of this major project with you. I understand that progress is likely to be deferred during the recession but it may be that this period will be seen as a chance for you to extend your detailed research prior to the commencement of mining operations.

Yours faithfully,

Peter Rowe

PROFESSOR P. W. ROWE
D.Sc., F.I.C.E.

STYPERSON HOUSE,
ADLINGTON,
MACCLESFIELD,
CHESHIRE,
SK10 4JX.
TEL: 0625 72170.

YOUR REF. E/82/964

COPY

3 September, 1982

Messrs Golder Associates, (Western Canada) Ltd,
224 West 8th Avenue,
Vancouver,
British Columbia.
V5Y 1N5

Dear Sirs,

Hat Creek Project. Geotechnical Reassessment.

During the period 18 - 21 May, 1982, I visited the site with Mr Rawlings and Mr Skermer. Subsequently I have had the opportunity to read the 10 volumes of your reports dated 1977 and 1978, and I have the pleasure to comment on the subjects of ground water pressure control and on shear strength properties. It will be appreciated that whereas most if not all of my remarks, which compliment your reports, will be matters well known to you, they are made to enable me to provide the broad basis of my opinion. Reference to a report e.g. Report 1977, Vol.1, page 34 is denoted by R 77.1.34.

Ground Water Control.

1. This is the dominant factor. Apart from affecting pore water pressure directly it also controls stability, and progressive failure or degree of softening and therefore it affects the shear strength parameters to be relied upon.

2. Depressurisation is a certainty as an immediate response but how long can it be held ? This, and effective drainage, depends on mass permeability or c_v, which in turn depends on the presence or otherwise of relatively permeable horizons, however thin or discontinuous in places on plan they may be.

3. During the site visit I examined some of the cores and noted the presence of thin carbonaceous or coal layers within the

formations labelled Tcl and Tcs. I agree that Tcl assumes great importance (R78.1.17). The excellent detail shown in the logs confirms the presence of these coaly horizons. R77.1.34 indicates that coal (in the major deposits) has a permeability some 10^4 times larger than the Coldwater Sediments and if this applies only approximately to the coaly layers they would dominate ground water movement and pressure. The sandstones which would normally be expected to have a similar effect appear to be of rather low permeability but I am not certain that this is so at all horizons.

4. Looking at the ground water levels as recorded on the cross sections the results appear rather as if the water were free to move about and adopt simple overall seepage gradients but I have not had the time to sort out which were piezometers and which standpipes. I accept (R78.1.51) that only 0.5 to 2% of the total flow moves through the clastic sediments but of course we are concerned with rates of pressure change in respect of stability rather than rates of flow as may affect the handling of any drainage output. Re R78.1.49c the changes of 3 m head in 6 months due to seasonal effects seems to me to be very significant, bearing in mind the small gradient changes imposed. After depressurisation, assuming firstly that this could be applied rapidly, the gradient and the gradient change would be large between the claystone and any permeable layers and much larger absolute changes in pressure would occur. Alternatively, as excavation takes place gradually, little depressurisation may hardly occur at all.

5. Put another way, one would be surprised to find any seasonal variation whatsoever within the middle of a massive deposit of bentonitic claystone/siltstone with $k = 10^{-10}$ m/s or lower. Certainly there is no fluctuation in a puddle clay core of a dam, $k = 10^{-10}$ m/s, during a six month drawdown in a drought, and the core thickness is small compared to the Tcl formation. So there must be permeable passages somwhere.

6. Thus while I agree that slope failures would be controlled by the strength of the mass rather than by particular discontinuities, the latter may well affect the distribution of water pressures which in turn dominate stability.

7. R78.1.xvi shows that, in contrast to laboratory specimen c_v values of 1 m^2/yr your judgement is that the mass value is unlikely to exceed 100 m^2/yr. Possibly the result (R78.6.A13-12 of c_v = 500 m^2/yr (range 134 - 2495 m^2/yr) was disregarded due to high test stress gradients and questions of hydraulic fracture. Of course, k does tend to increase with decrease in effective stress and whereas this will occur with excavation it is only likely to be significant near the surface of the excavation. But unless piezometers were located so as to include the coaly layers and analysed in relation to the geological structure of these layers the interpretation could be affected.

8. These matters affect how slow is " slow " (R78.1.68ii) at Hat Creek. There are no coaly layers in London Clay (but see below) nor at the Panama Canal. If the equivalent mass c_v were 500 to 1000 m^2/yr, only up to 1 order of magnitude greater, the expected field performance would be very different see R78.1.Fig.24. In view of R78.1.70 line 5 this matter might become critical and I believe it would be worthy of further study before R78.1.72 para. 3 could be relied on.

9. Relief drains connected to permeable layers have a much wider influence than in the case of uniform ground. Conversely, permeable layers untapped by drains or natural outlets cause greater uplift and instability below excavated formations than in the case of uniform ground. Consequently the existence and performance of any such permeable horizons is dominant on the mining operations, almost irrespective of the rate of flow within any such layers.

10. If relief wells are eventually used, presumably of small diameter acting as bleeder points, they would preferably be drilled from the base of open trenches taken well below the main formation at any one stage in order to lower the terminal pressure as far as is necessary.

11. I see a need therefore to include a field study of pore pressure drawdown and distribution in the coaly horizons compared to the claystone during any further field pumping tests. It may be necessary to enlarge the hole with an expanding bit when the coaly horizon is identified to ensure good response.

Shear Strength Parameters.

12. The most reliable values on any particular site for any particular ground water regime and rate of construction are those obtained by back analysis of field slips with field water pressure measurement in the slip surface at the time of slip. This has always been difficult to achieve. Consequently an ongoing laboratory test research programme, both before and during mine operations, is essential for the control and interpretation of field events. It could only cost a very small fraction of the remedial measures to any one slide.

13. Good progress has been made already on shear testing within the practical limits of sampling to date, certainly to have identified objectives and to bracket the likely limiting slopes for given ground water conditions. The next step would be to develop testing techniques to suit the stress state and paths for this site for samples represenative of the geological structures at a time well before mining operations commence and to continue during construction when a greater variety and size of block samples could become available from the base and sides of the excavations. At that stage it will be most important that the testing techniques be already developed and agreed so that relevant data can be fed back to site as rapidly as possible.

14. In broad outline the techniques I would favour are as follows:-

Sample size. Up to 300 mm in view of the multiple fine fracture pattern spacing in the claystone. At present the available 135 mm size would be useful for development work. If a trial excavation were made in the Upper Siltstone/Claystone Tcu formation as proposed in Section 7 of the final report Vol.1 1978, large block samples could be taken then also.

Stress State. The samples suffer triaxial unloading and one should reload to the insitu vertical total stress, estimated lateral stress and measured water pressure. Equipment capable of applying cell pressures and pore pressures of several 1000 kN/m^2 to specimens up to 300 mm is available at Manchester.

Stress Path. Both "triaxial" and plane strain can be applied but the triaxial path with decreasing minor principal stress is most likely to allow simulation of the strain softening process on samples of different orientation of geological structure.

This should include stepped changes in back pore pressure to simulate loss of suction. (With a block sample the specimen can be orientated so as to apply the principal stresses in the direction within the slope.)

Pore Pressures should be measured in the centre as well as the base, keeping the testing rate of strain low enough to ensure equilibrium. This uniformity of pore pressure can be achieved more readily using lubricated end platens.

Creep is likely to be accelerated by ground water level fluctuations which effect could be included but would take up a great deal of testing time. One tends to think of creep as a continuing strain at constant stress, but in this case the field movement with time is dominated by changes in pore pressure and decrease in the structural interlock. However described, the laboratory tests will take time (because of the specimen size, the low permeability and need to keep pore pressures uniform, for each back pressure step.) Hence the specimen membranes would need a mercury jacket to ensure completely against water migration. A single large specimen might be under test for 6 months to 1 year, quite apart from the conventional creep observations under static pore pressure, drained, or under undrained conditions where pore pressure changes are part of the creep observations.

Critical State tests on small remoulded specimens.

Residual Strength using the ring shear (which for London Clay gives about 2° lower than the repeated shear box movement) .

15. Having measured c', Ø' at peak, critical state, and residual, one can adopt some intermediate stage such as critical state or the " fully softened " condition (an unfortunate choice of description) but the question in the case of a fresh cut is how long will it stand ? Strain at peak causes dilatancy and suction increments, which delays the progressive action. The above tests would give the basic data. One also needs, inter alia, the geological structure and permeability distribution of the slope. For example, in London Clay there are pronounced silt beds in places; elsewhere as in the brown fissured clay there are profuse silt and sand intrusions within fissures, and in large masses there are no permeable features. These dominate the rate of softening. I know some areas of London Clay which have softened and slipped in one day in cut, and other regions where

the clay has stood for years.

16. The basic steps I envisage are as follows:-

a) Finite element trial analyses to guide the distribution and build up of strains below the slope, in relation to the geology and the proposed programme of cut etc.
b) Initial estimate, and later observations, of boundary water pressures in relation to the proposed drainage system.
c) Field data to estimate the time for loss of developed suctions, having regard to the permeability structure of the strata.
d) Using the above, design undrained loading paths to measure the suction induced in undisturbed representative specimens following the expected total stress paths and initial ground water levels through peak to critical, and repeat with stepped induced losses of suction to simulate data from (a,b,c,) above.
e) Feed results of (d) into (a) and reiterate testing as necessary.

17. In principle, this would lead to distributions of c',Ø' and water pressure (and hence strength) in both space and time. In view of the variations in the geology one would have to start by identifying the likely critical areas.

18. If, during excavation, samples up to 1 m^2 x $\frac{1}{2}$ m could be taken one could study the rate of progressive softening in a large centrifuge, as we have at Manchester, using artificial drains to accelerate the process in addition to the scale.

Surficial Pre-existing Slip Surfaces.

19. In addition to the important drainage measures you have proposed there will be the new cut through the slip leaving the new toe of the slip over part of the circumference of operations. One might consider cutting the slip material back and replacing with granular backfill, working in steps and stages so as not to induce slip in the process, and cutting out the existing slip plane at its lowest level.

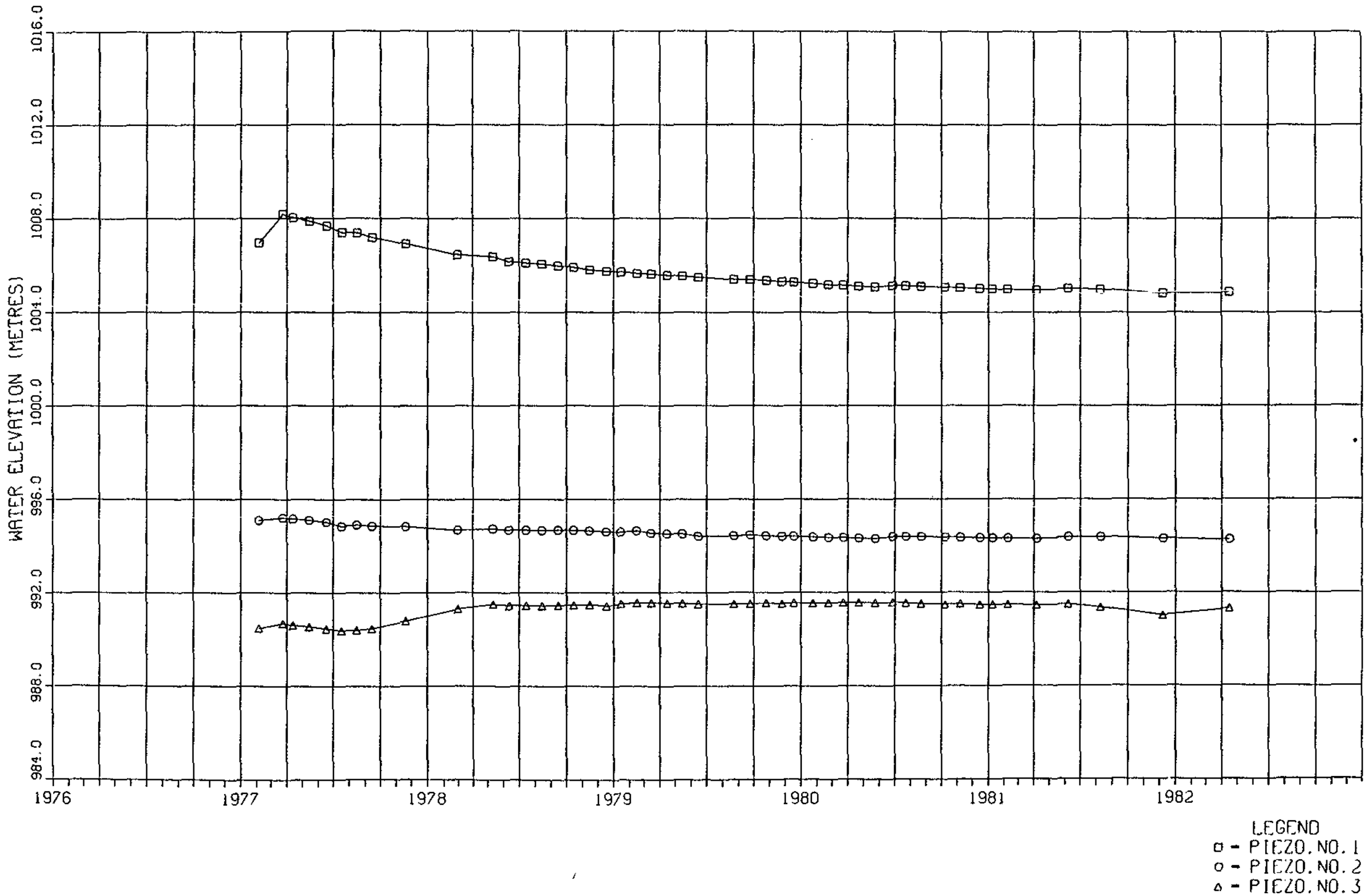
I am well aware that this must be a project of world wide interest and importance both in coal mining and in geotechnical engineering. If there should ever be any small contribution we could make at Manchester you may be sure of an active response.

Yours faithfully, P. W. Rowe.

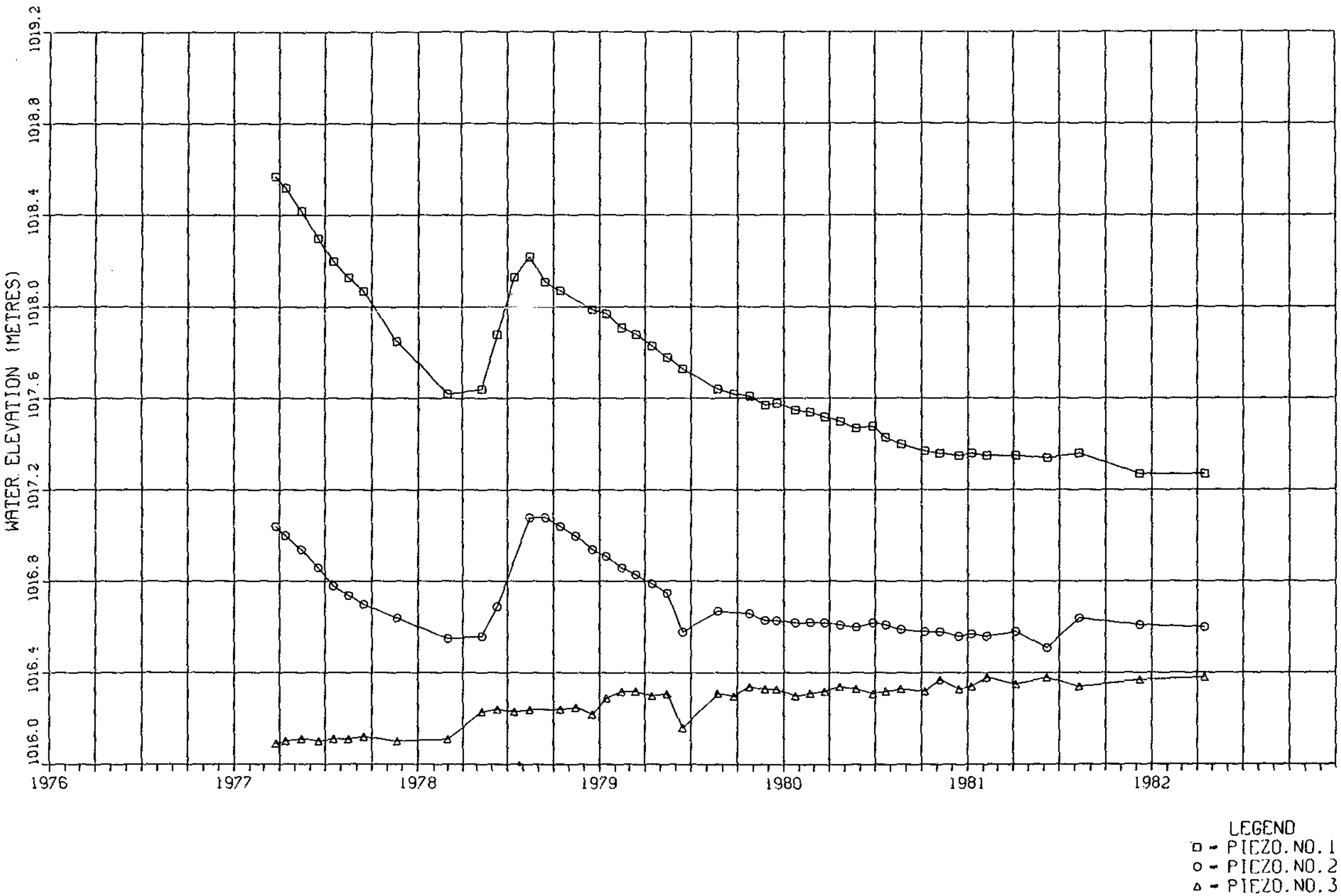
APPENDIX D

COMPUTER PLOTTED PIEZOMETER HYDROGRAPHS

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-10

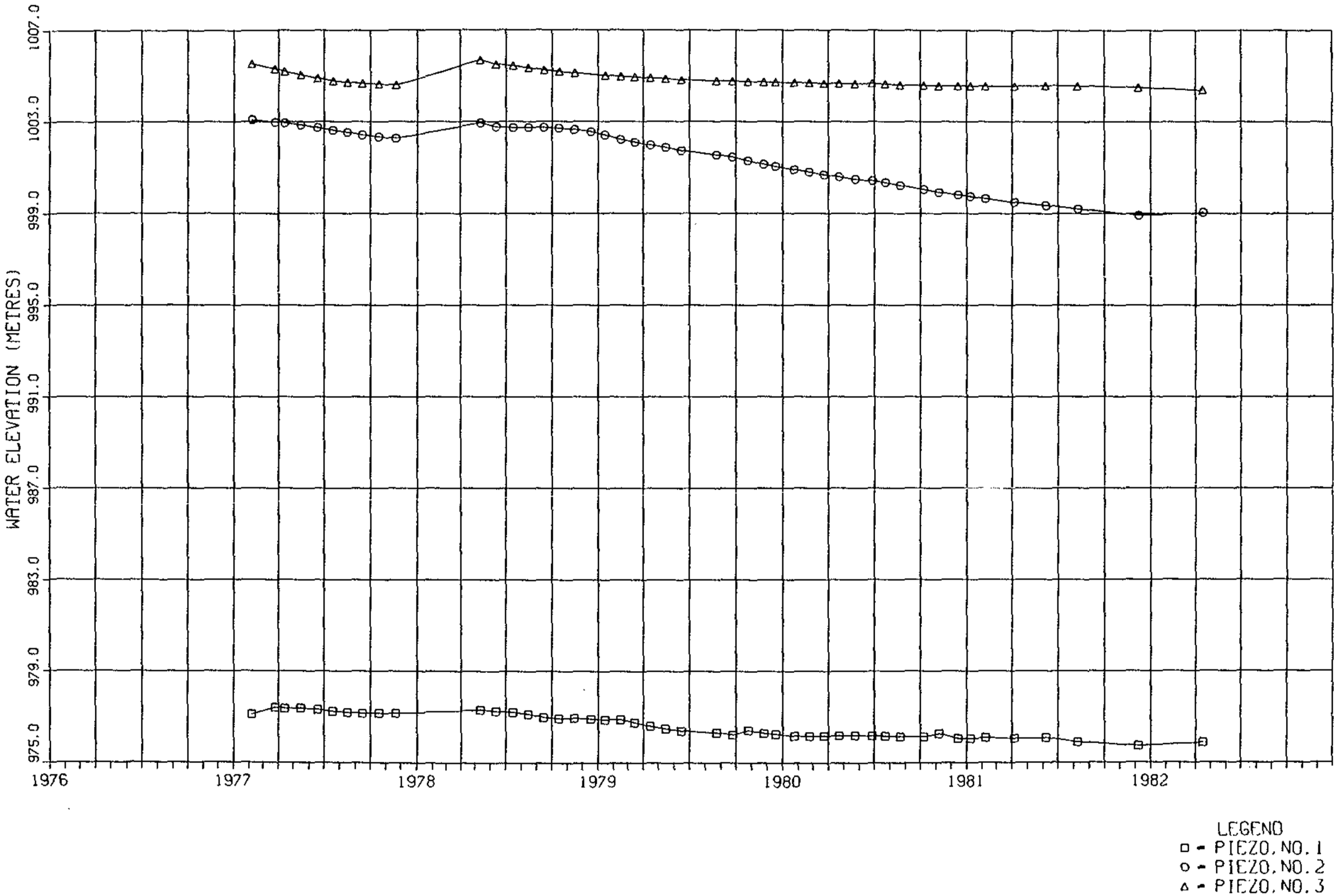


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-11



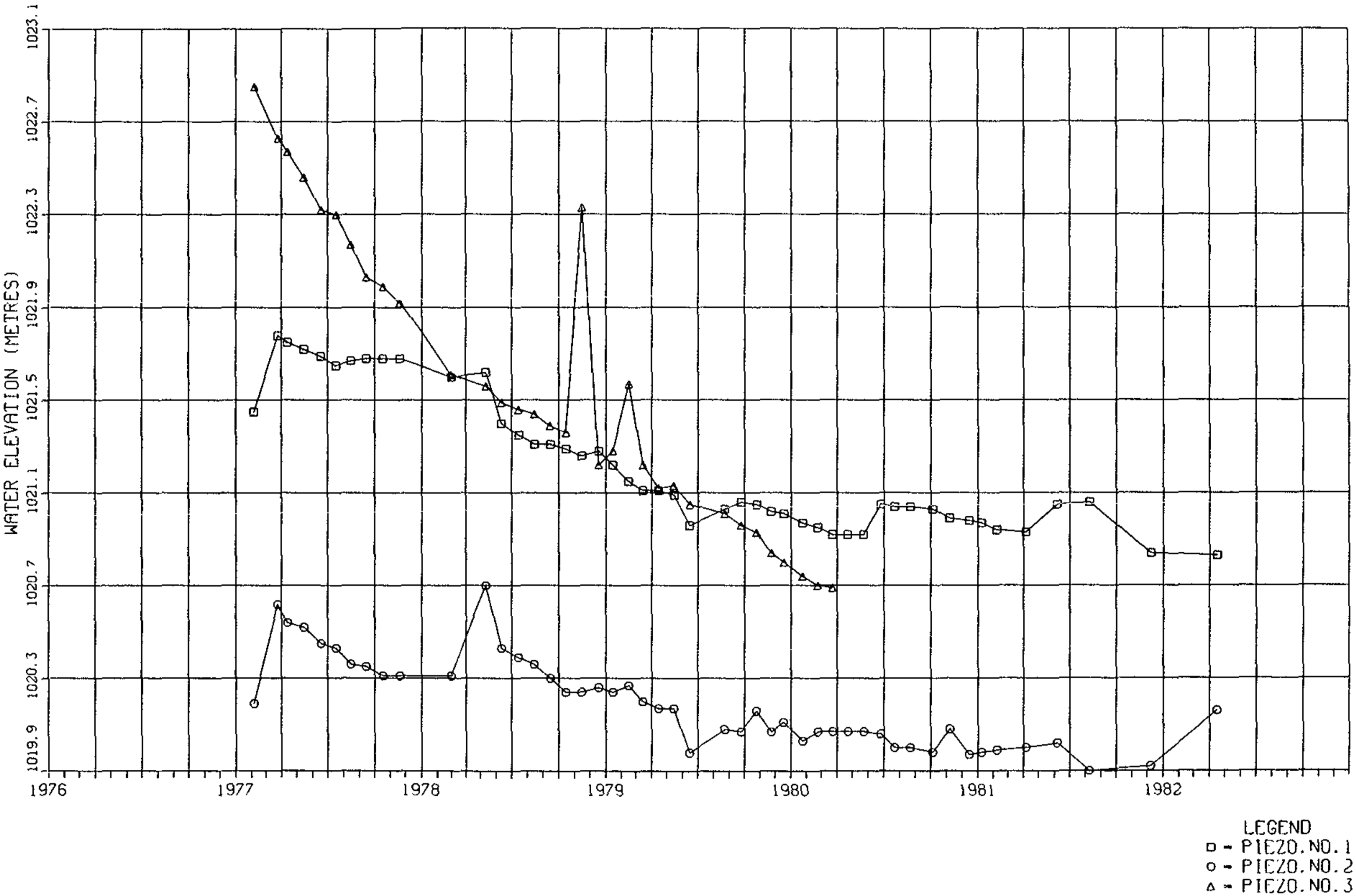
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-12

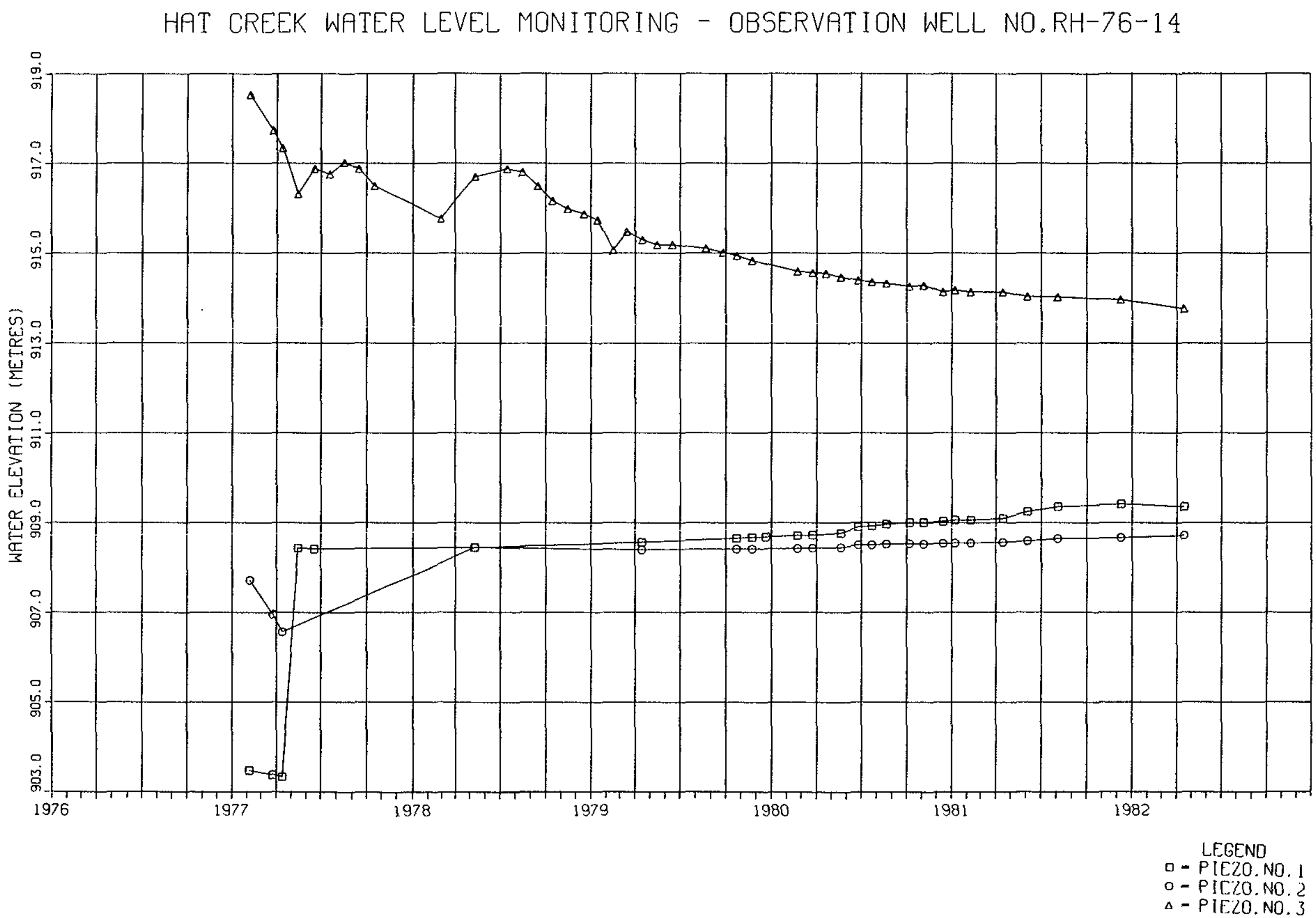


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-13

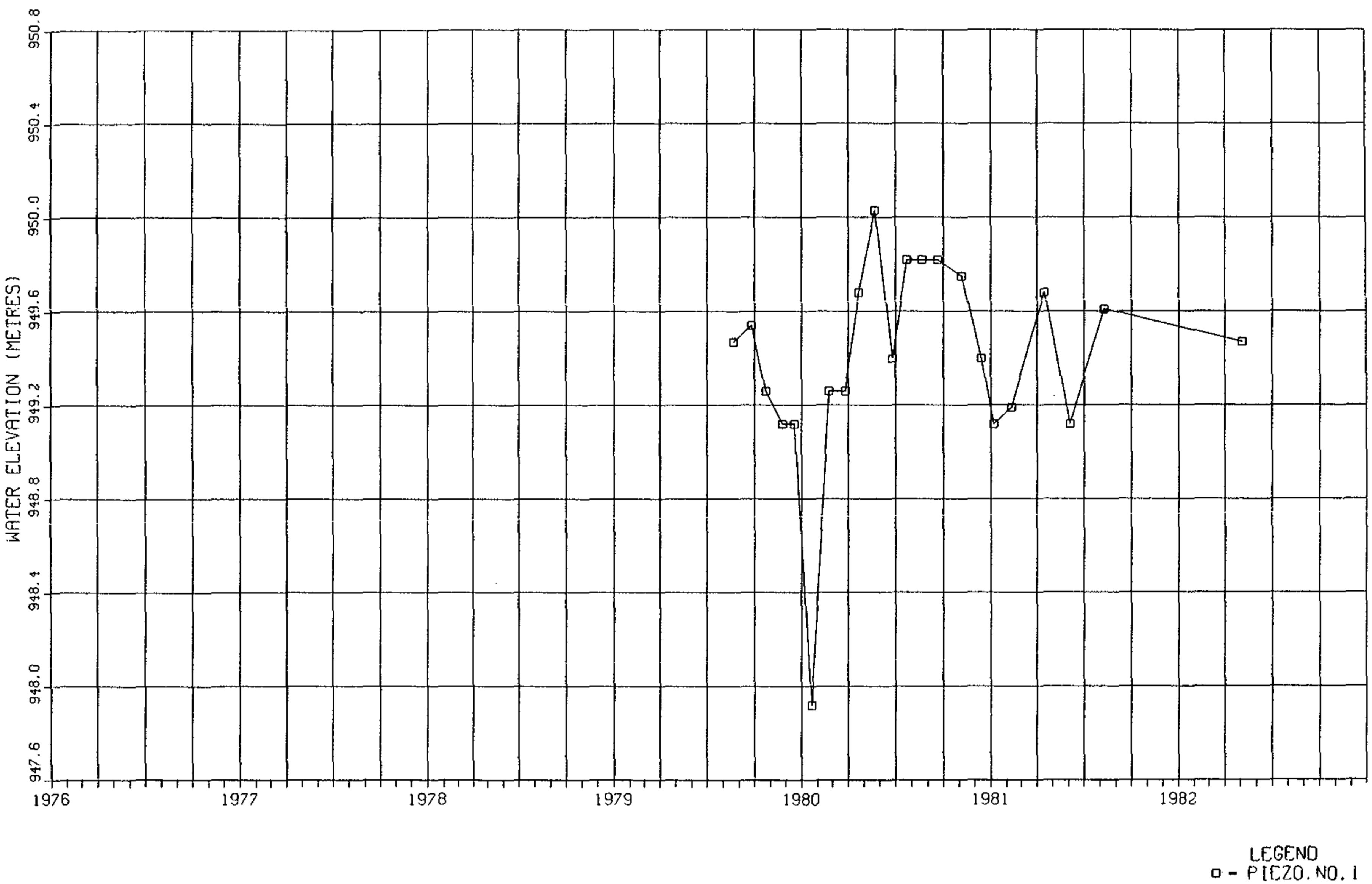


Golder Associates



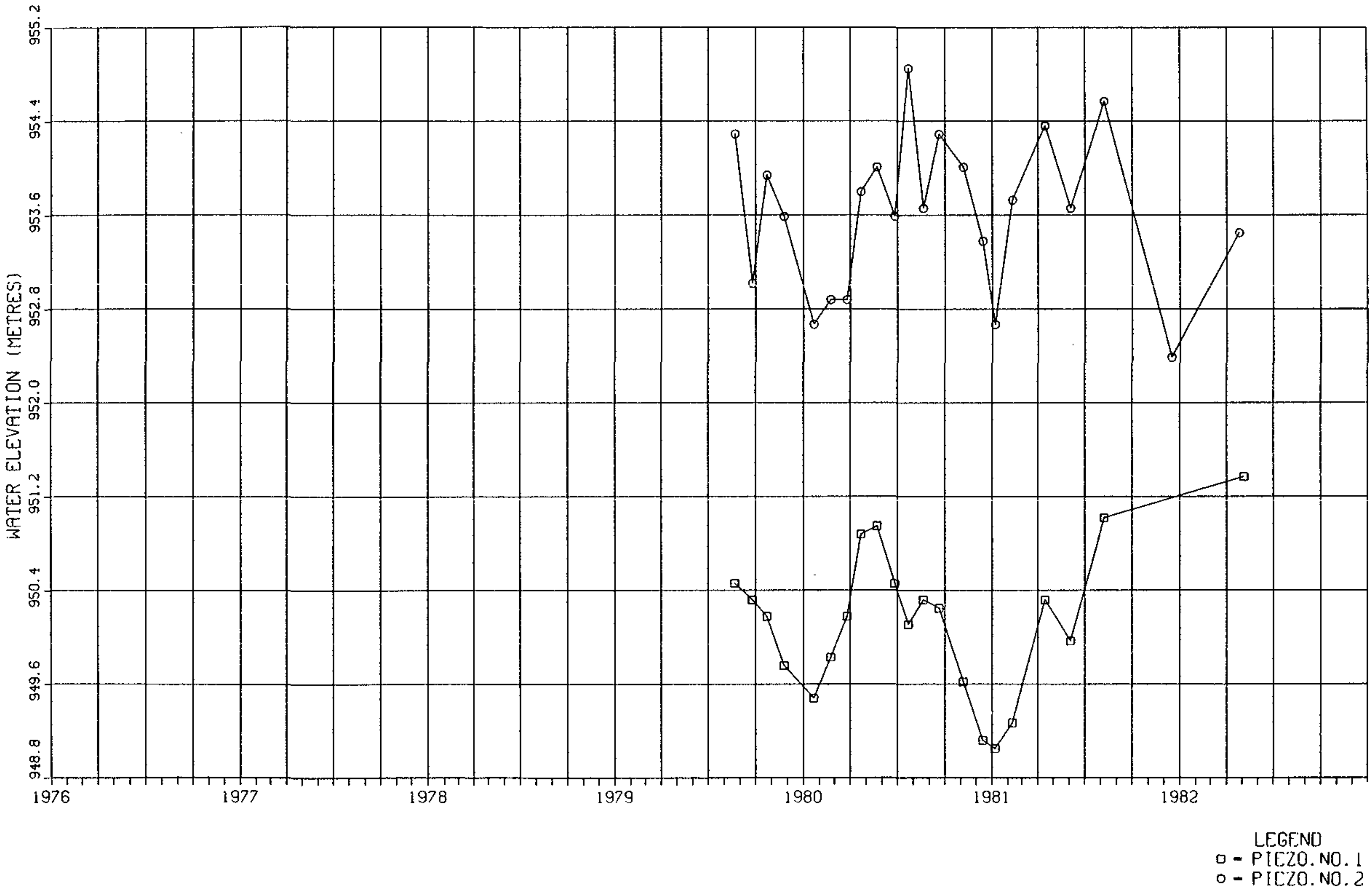
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-16

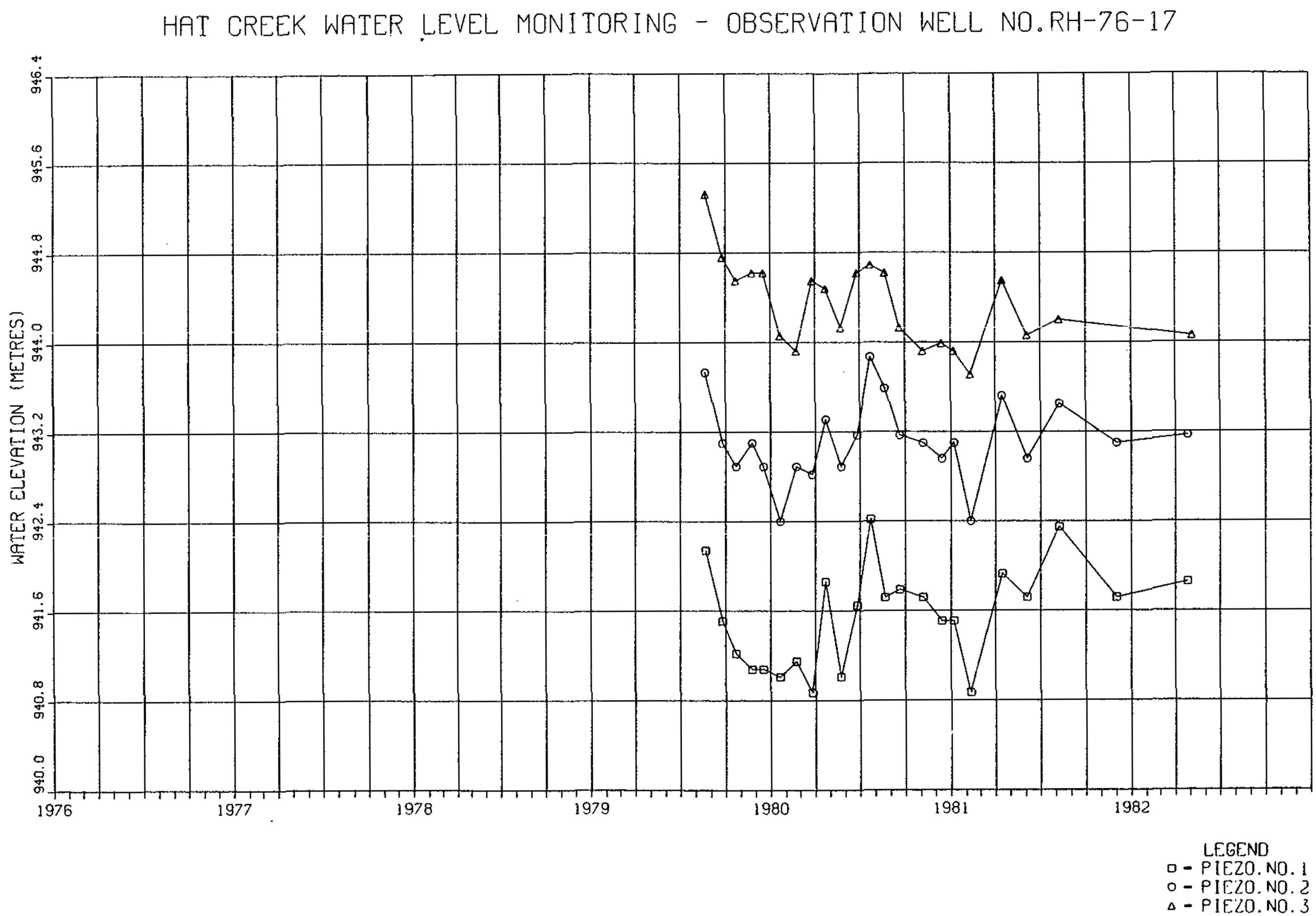


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-16A



Golder Associates



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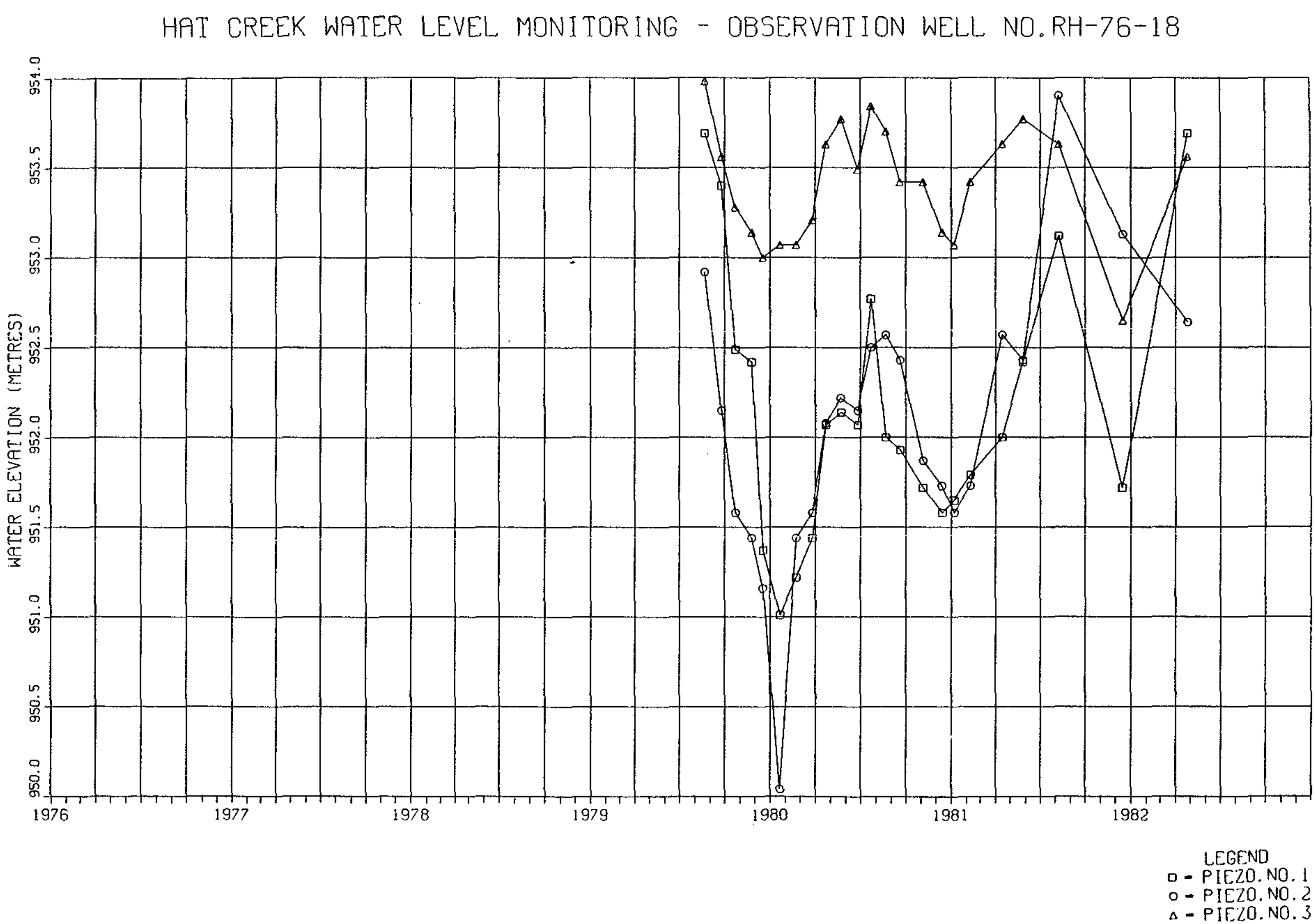
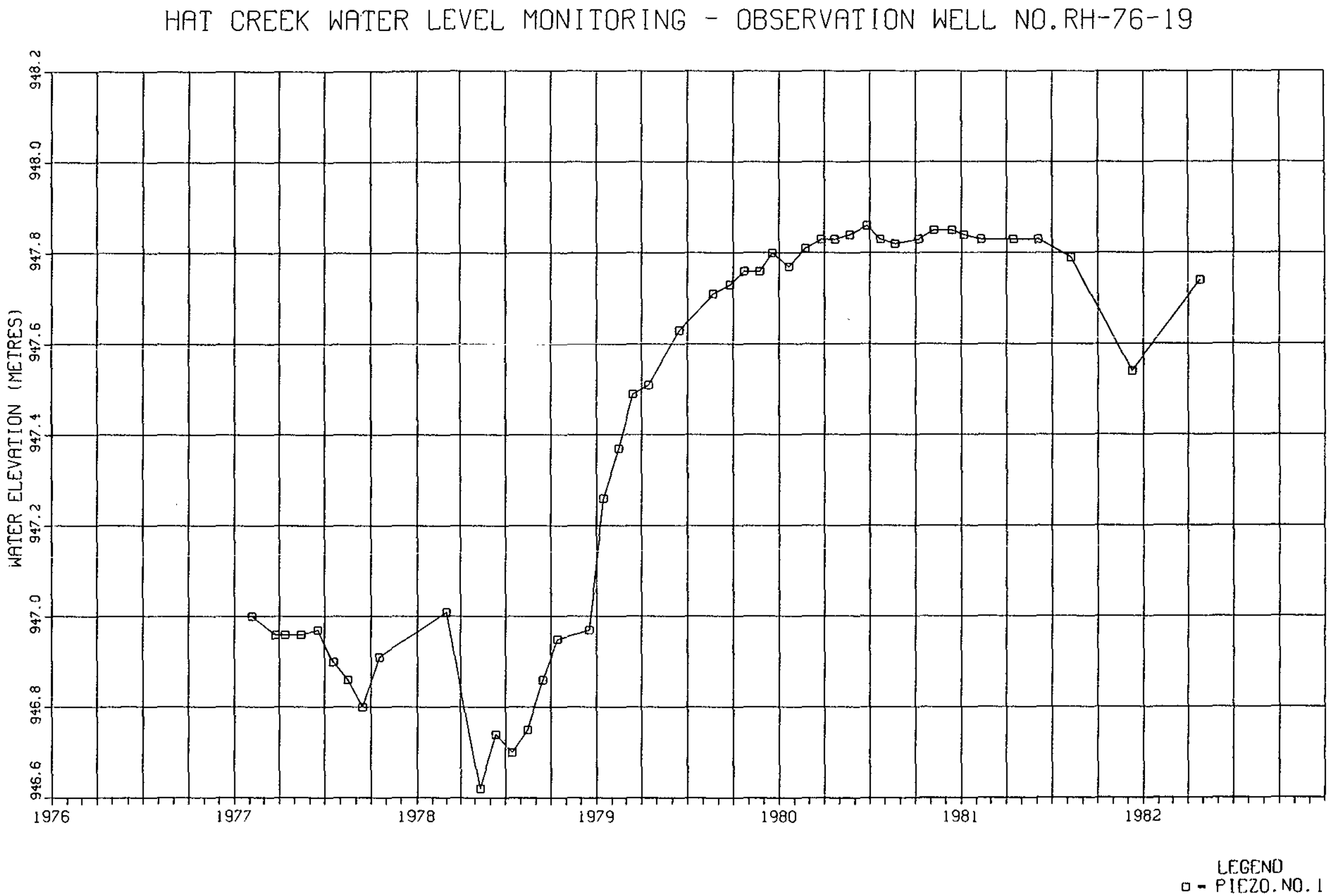
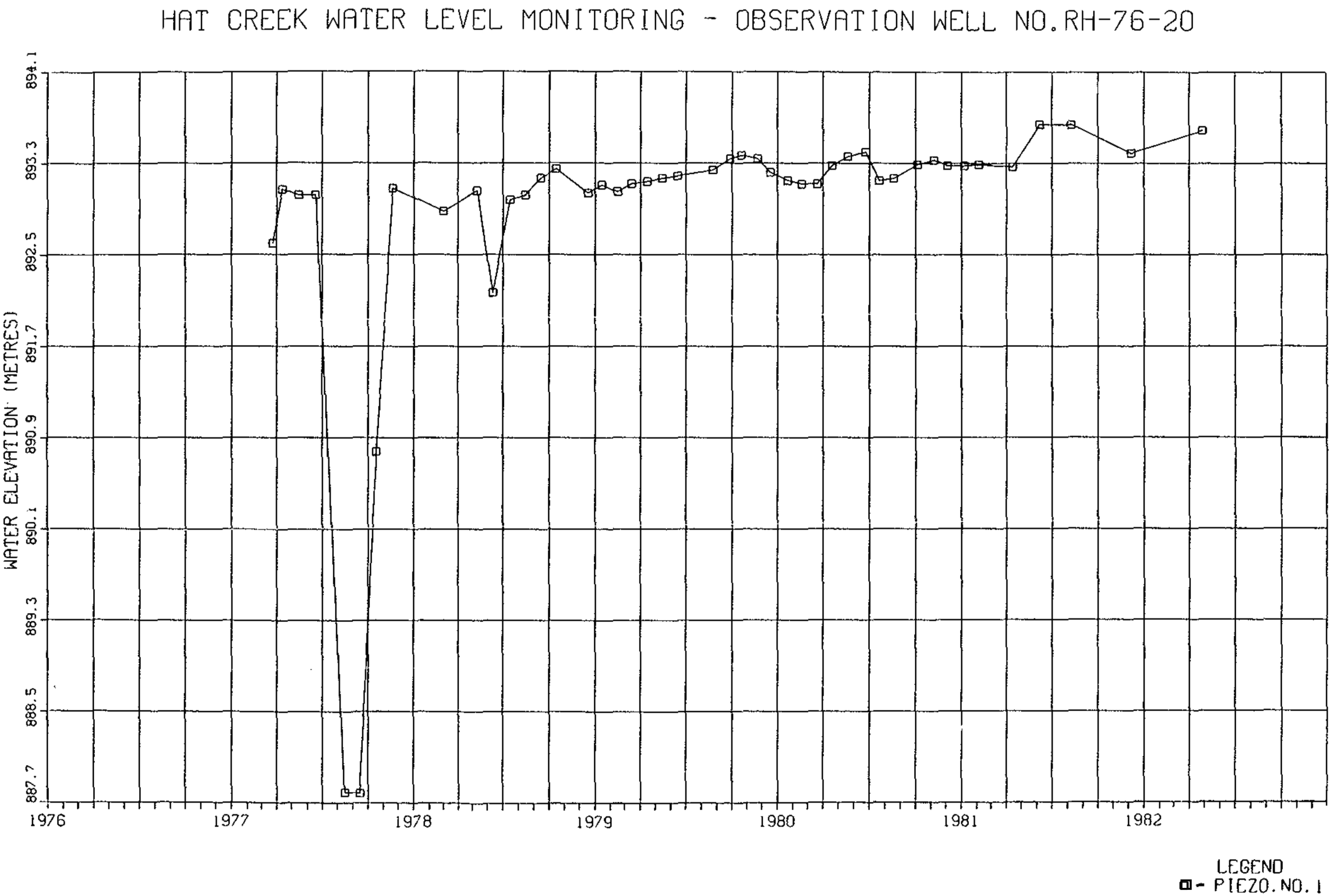


Golder Associates

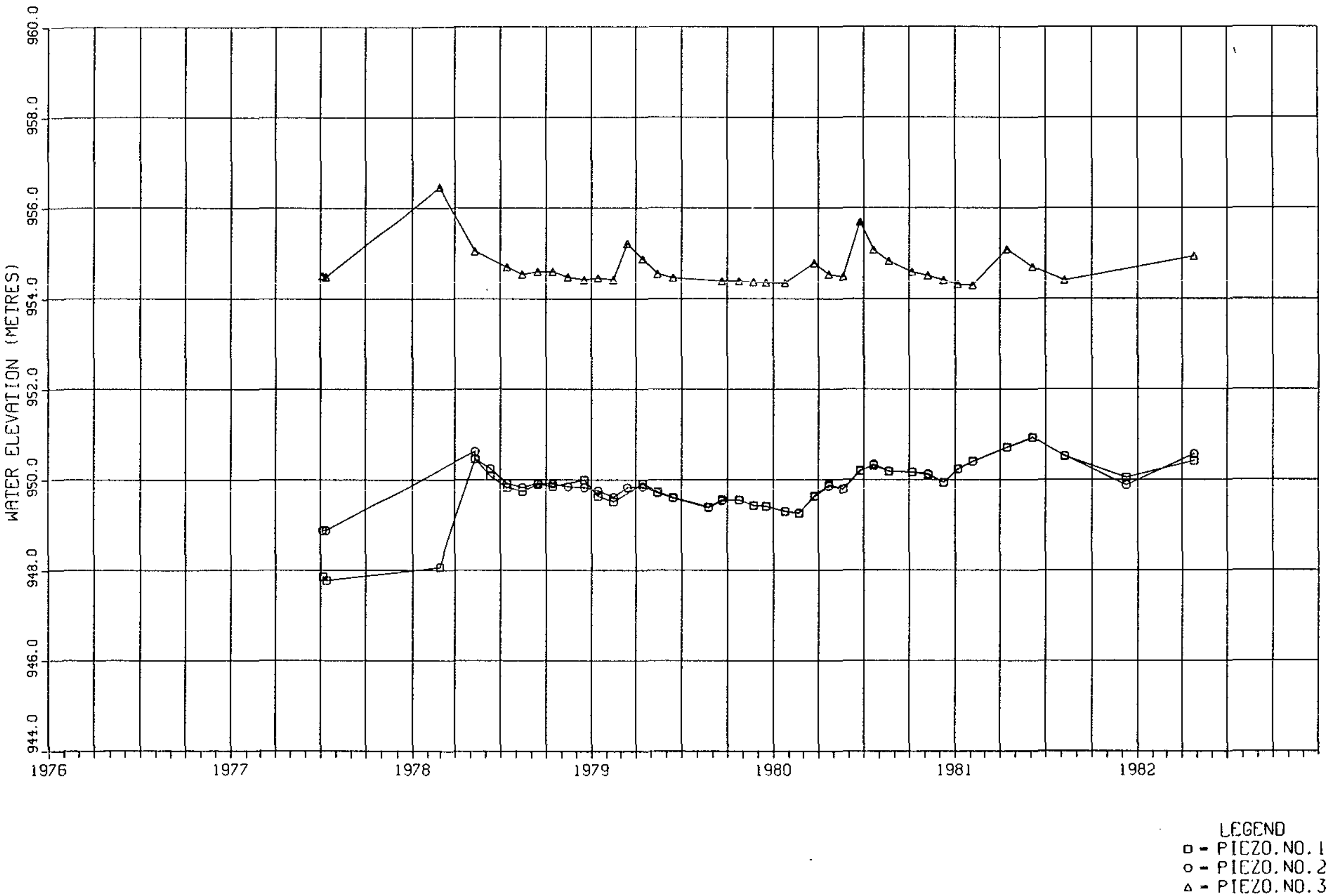


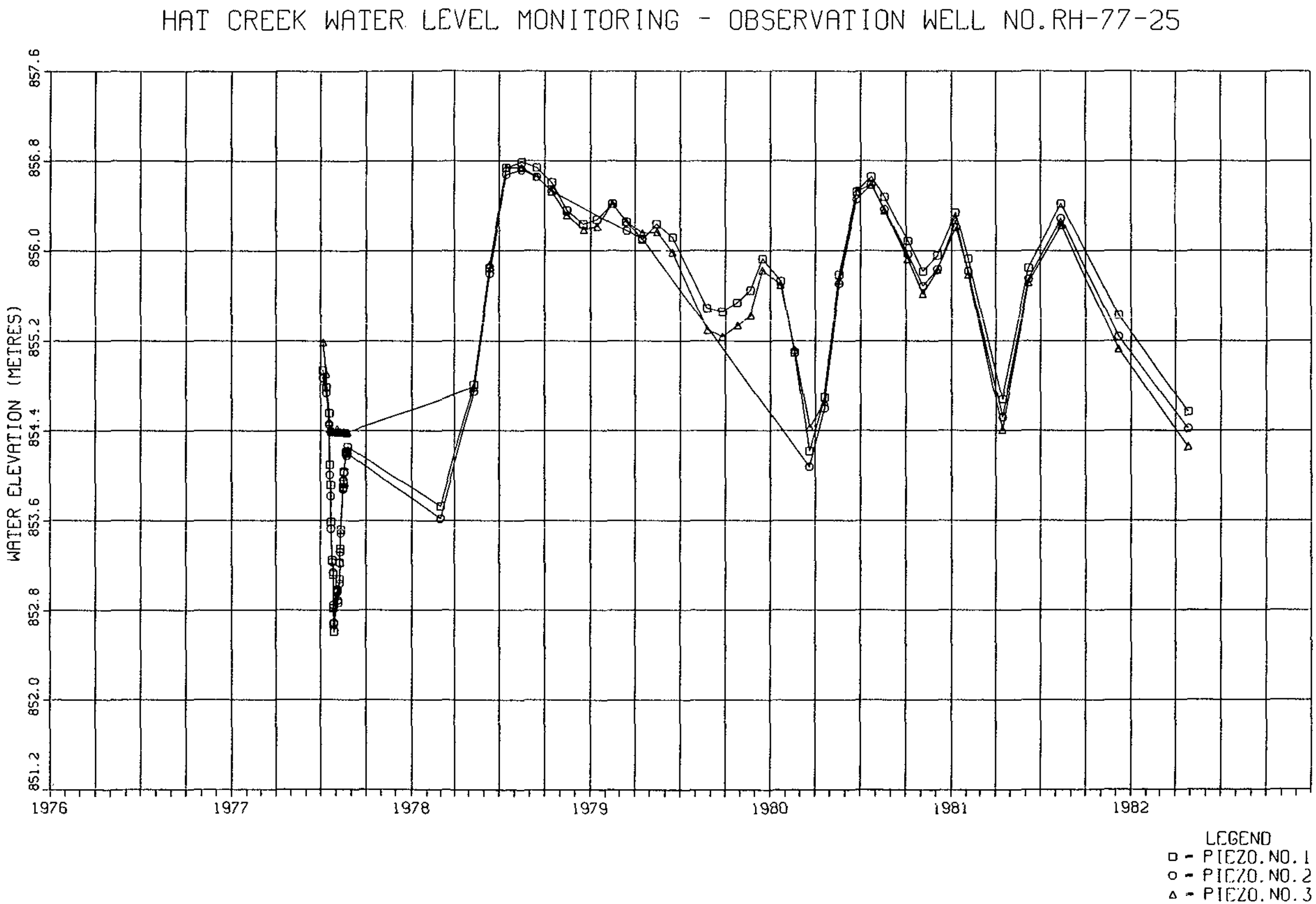
Golder Associates



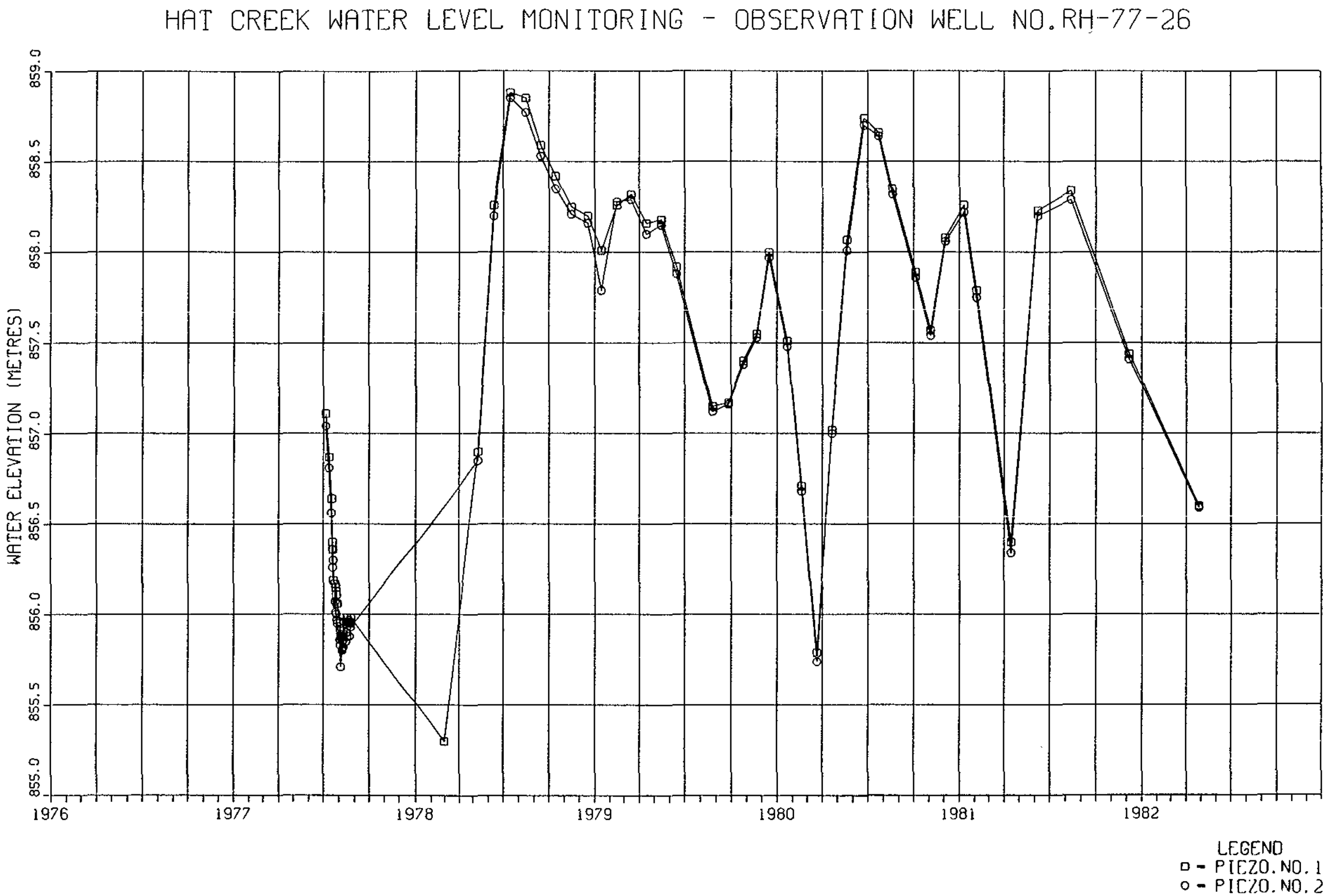
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-76-20
WATER ELEVATION (METRES)
894.1
893.3
892.5
891.7
890.9
890.1
889.3
888.5
887.7
1976
1977
1978
1979
1980
1981
1982
LEGEND
- PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-22





Golder Associates



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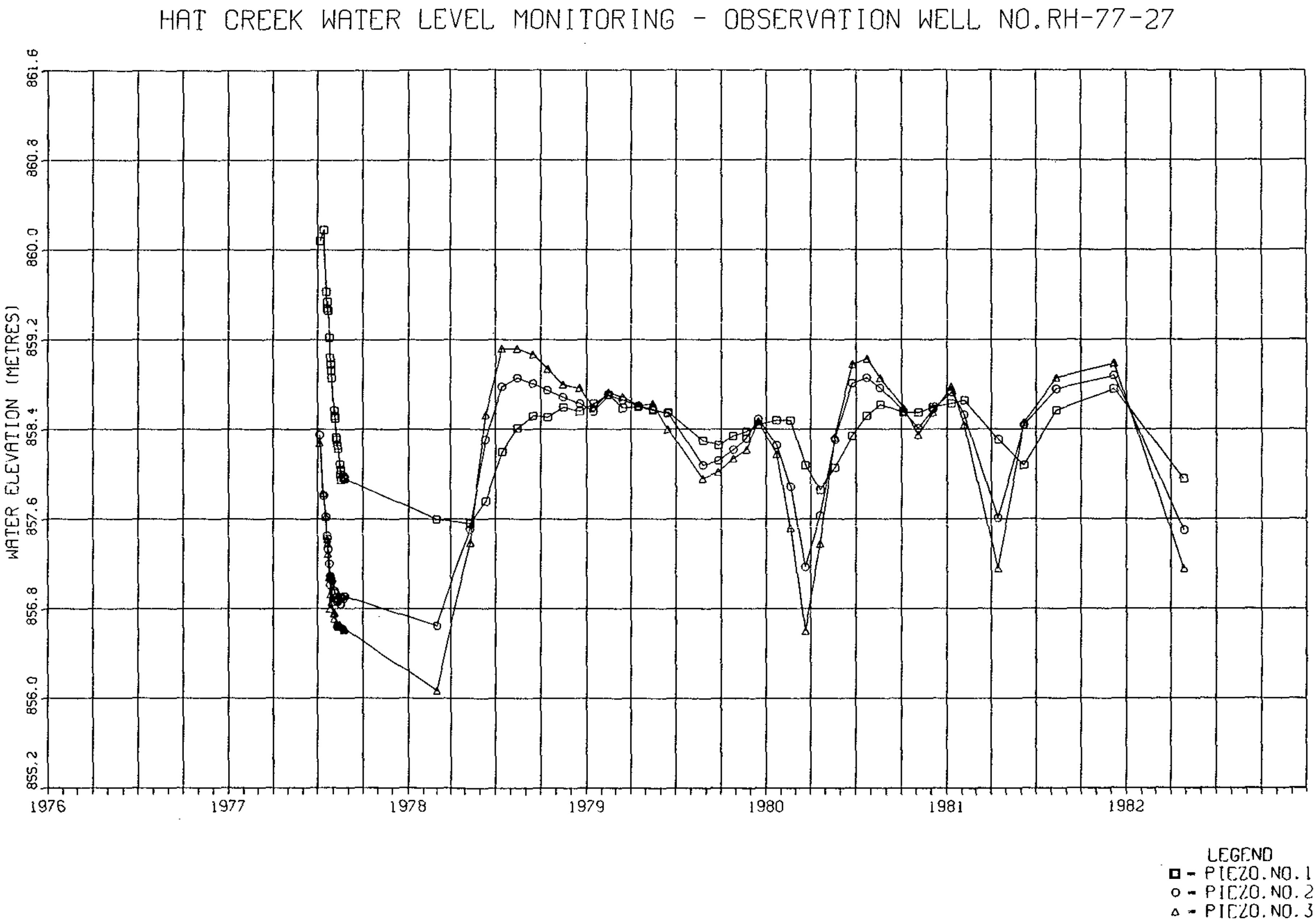
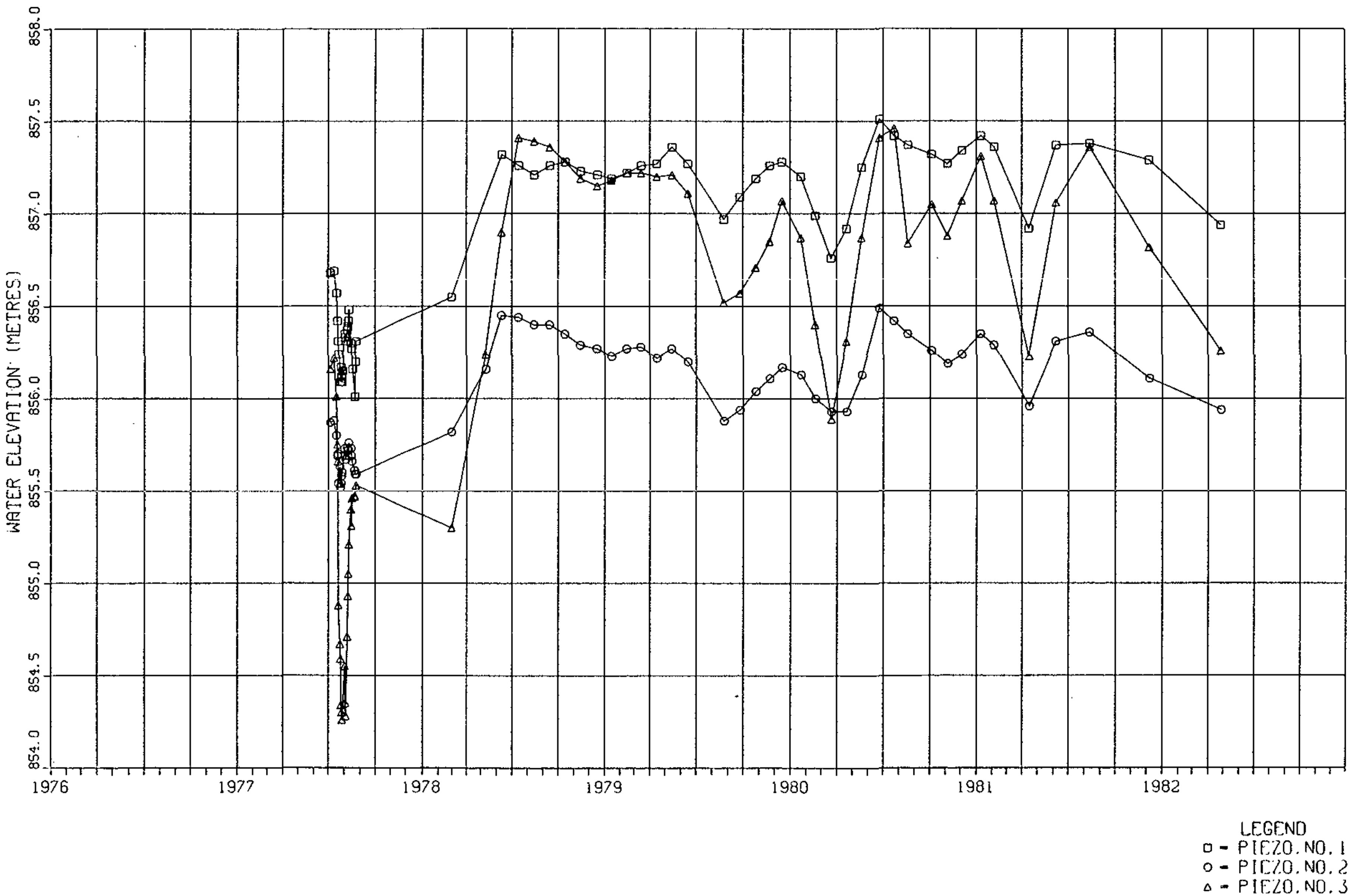


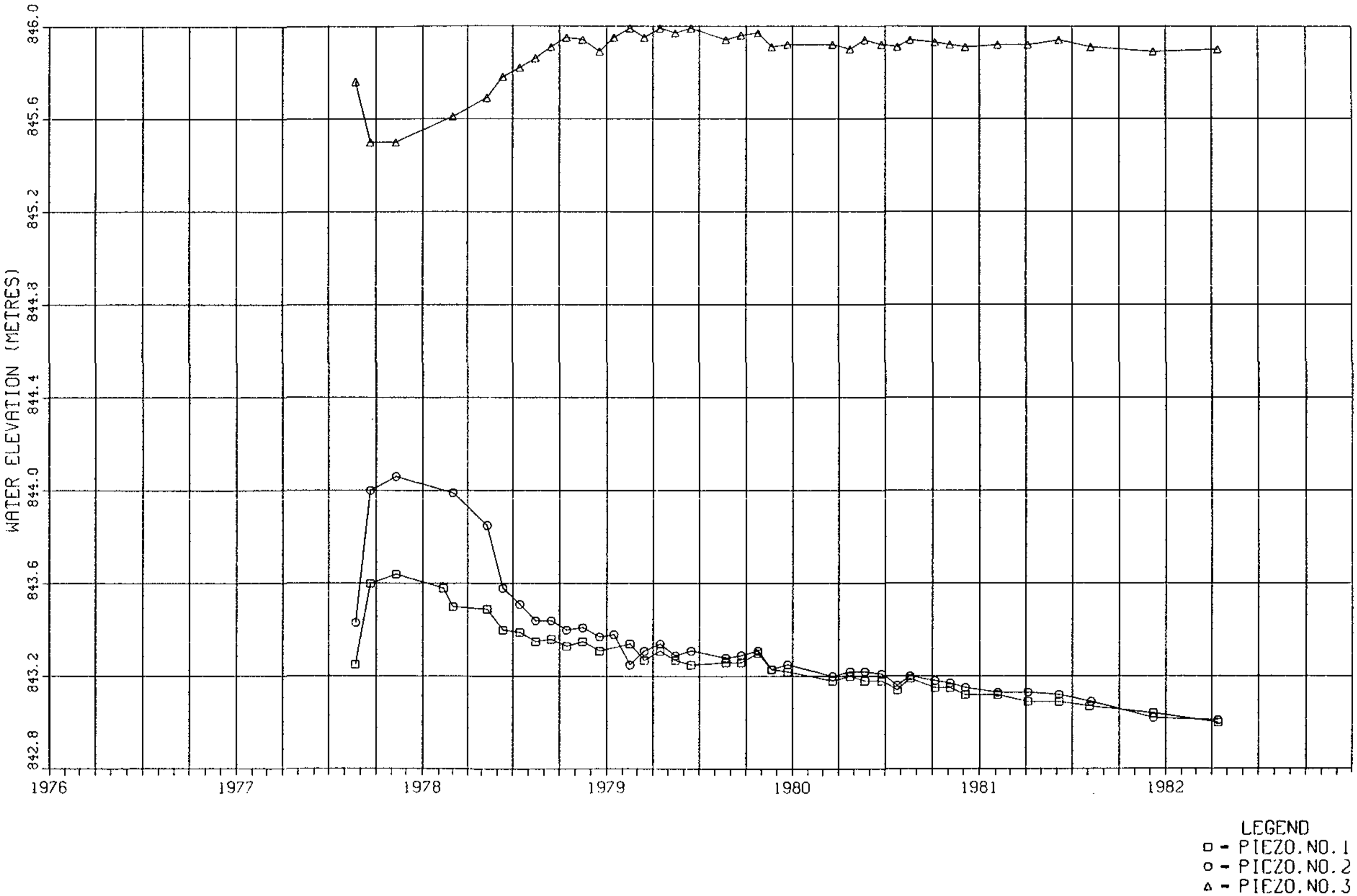
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-28



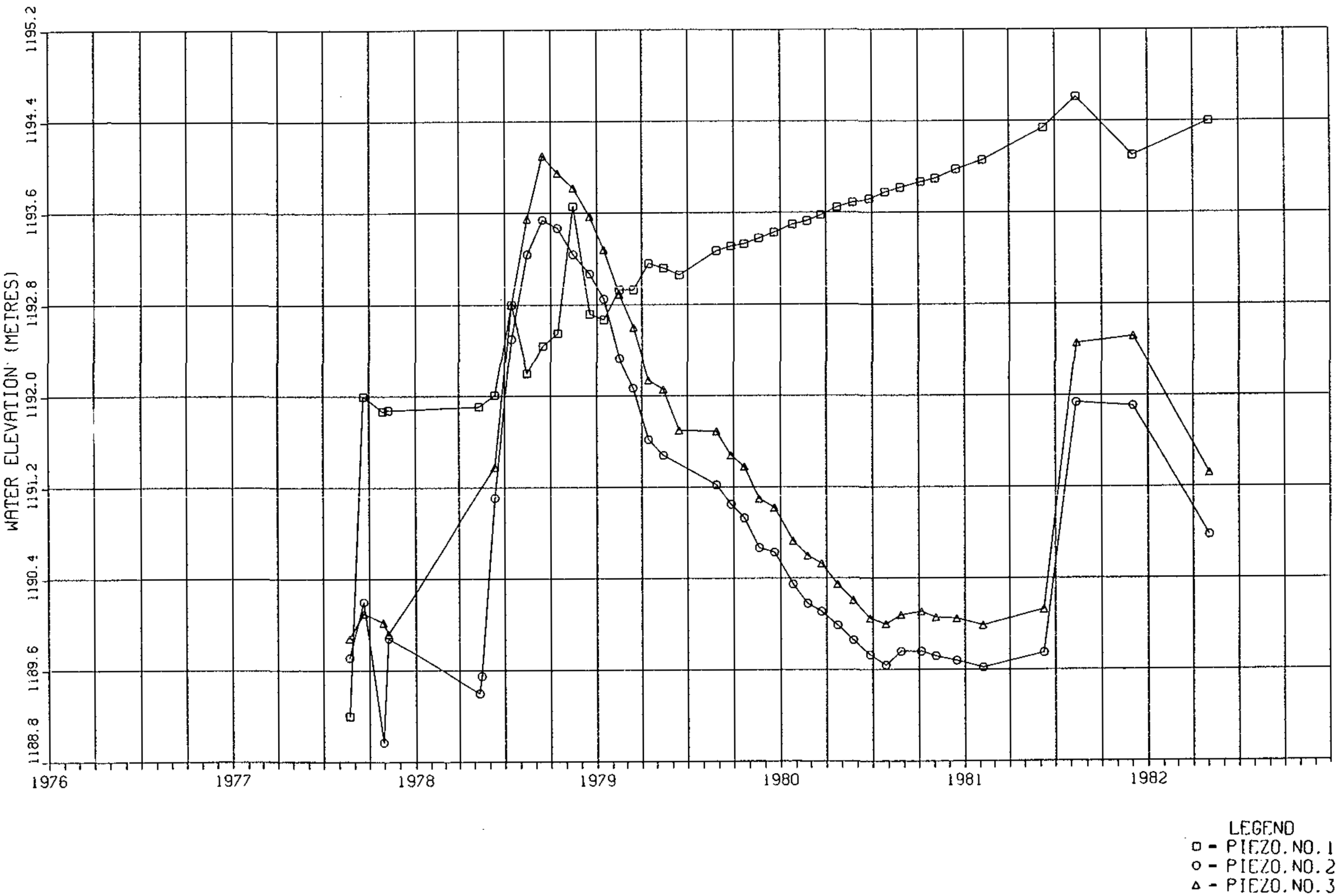
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-45

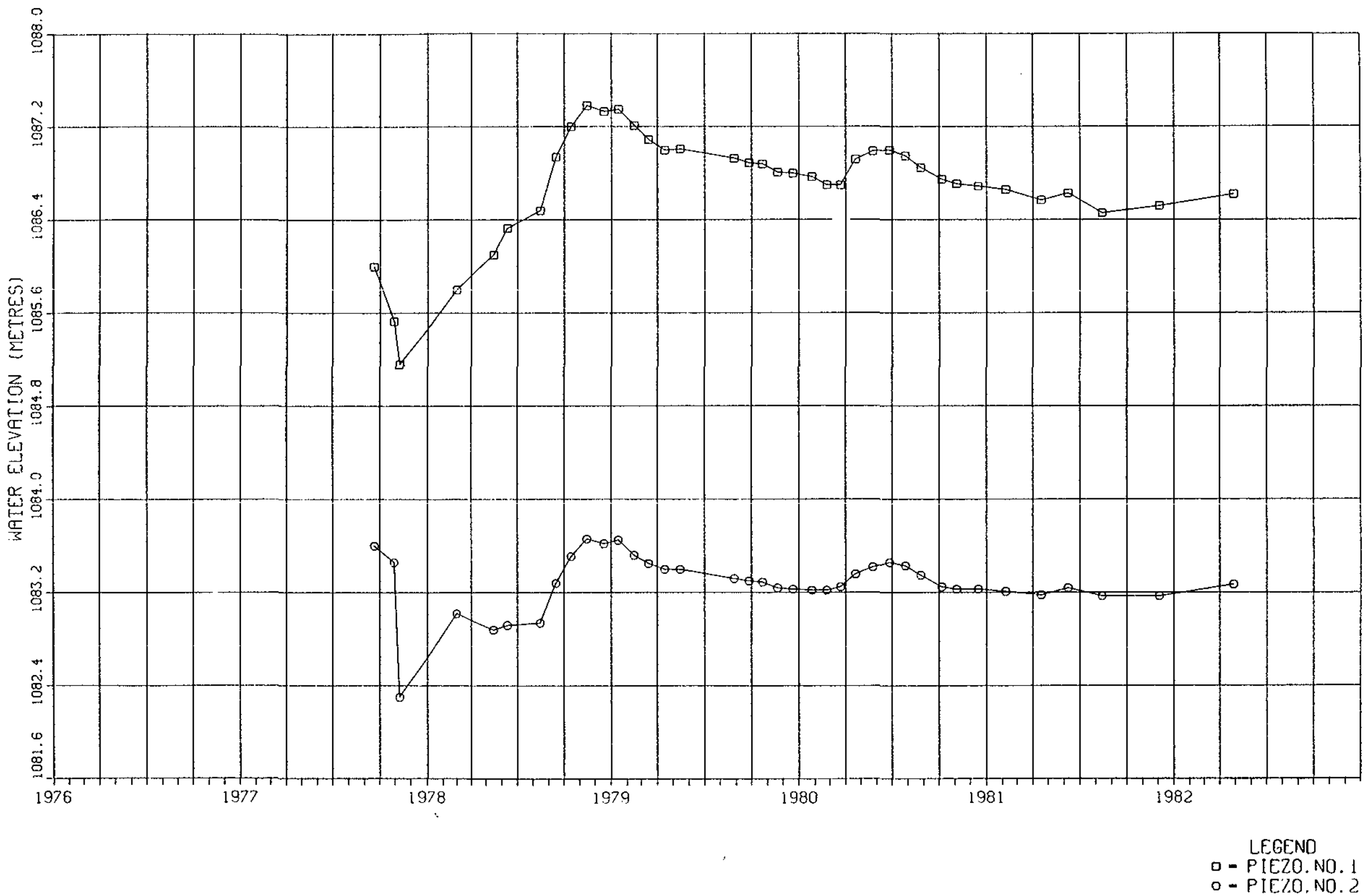


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-48

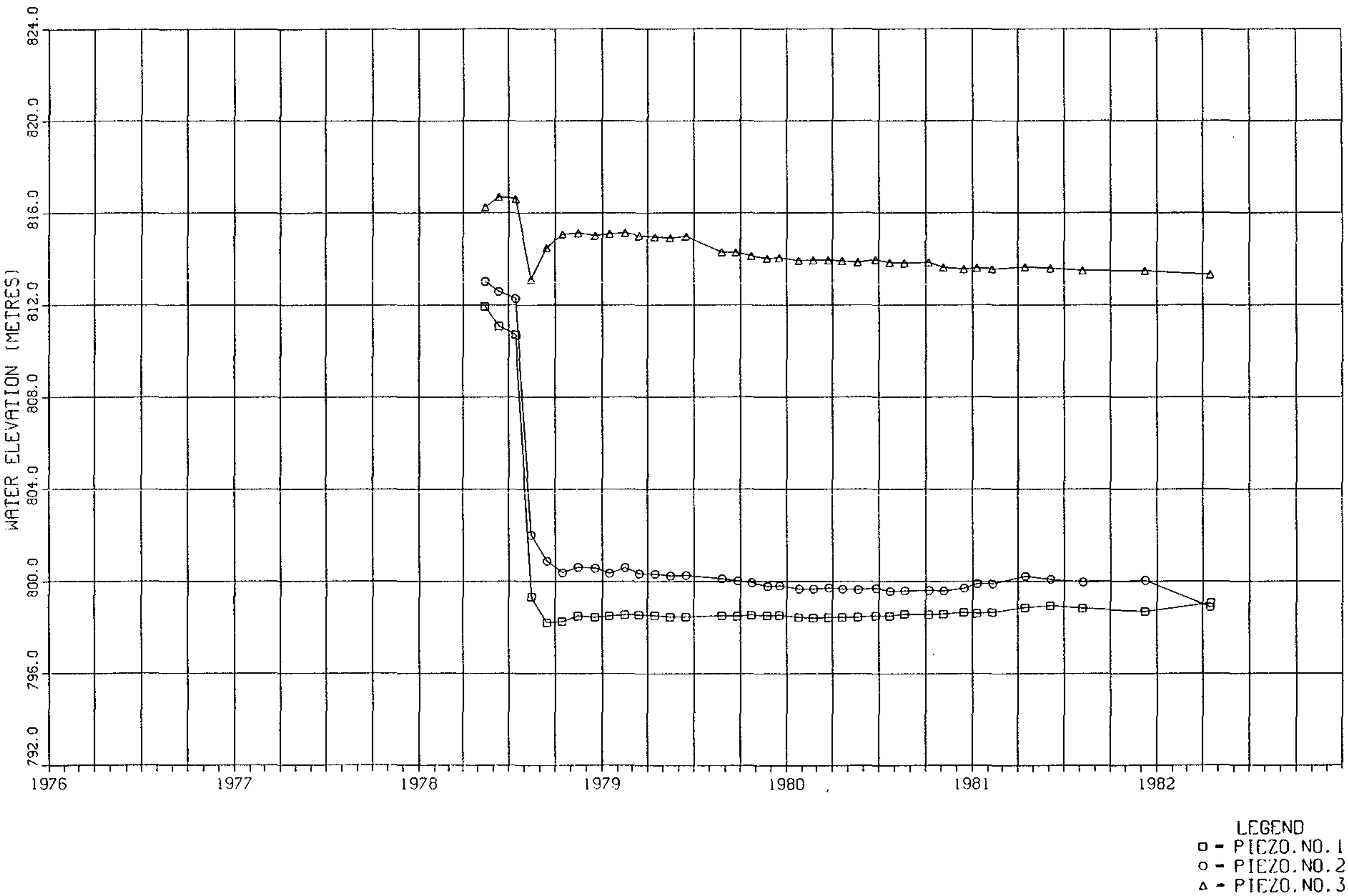


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-49

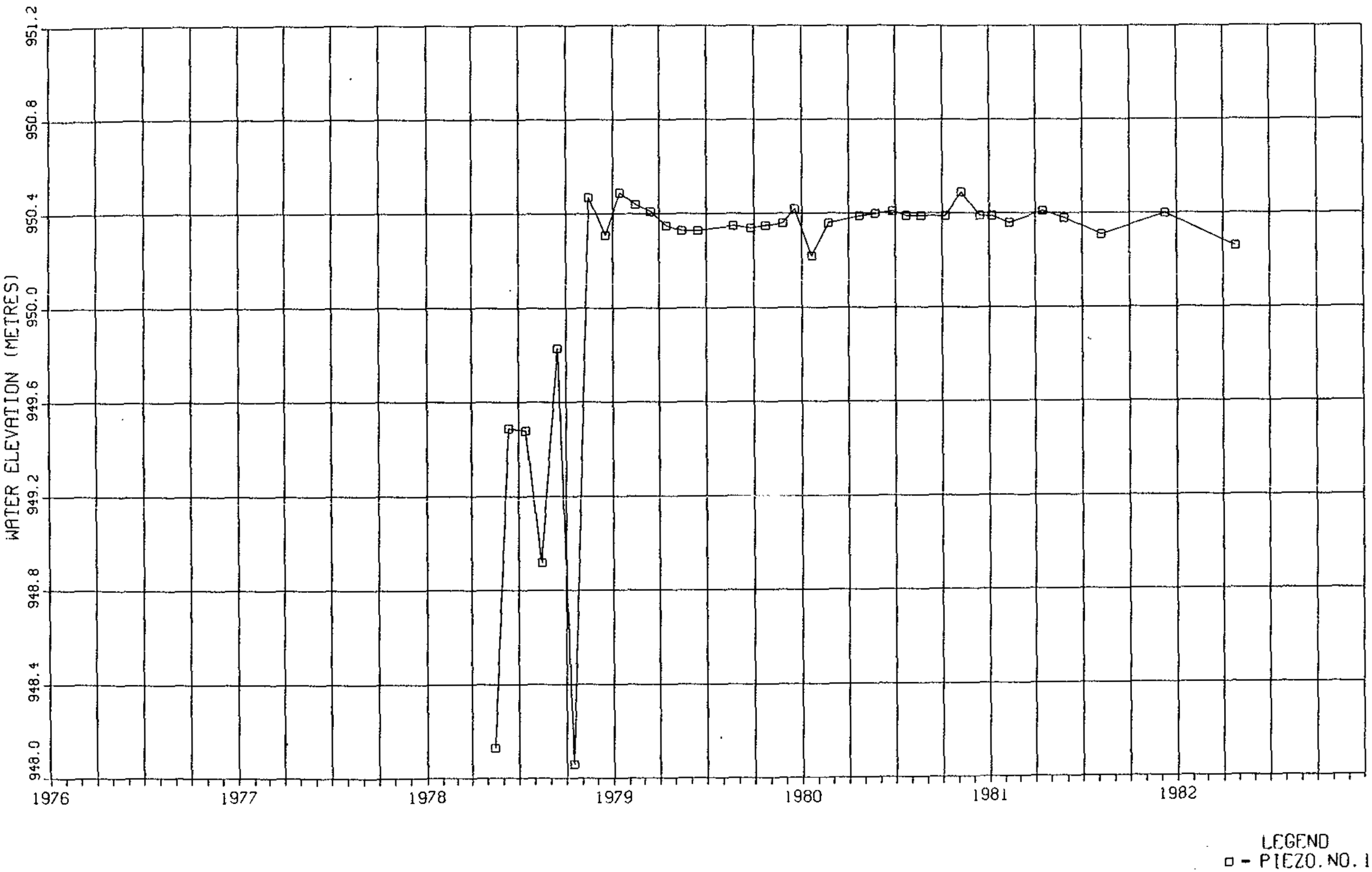


Golder Associates

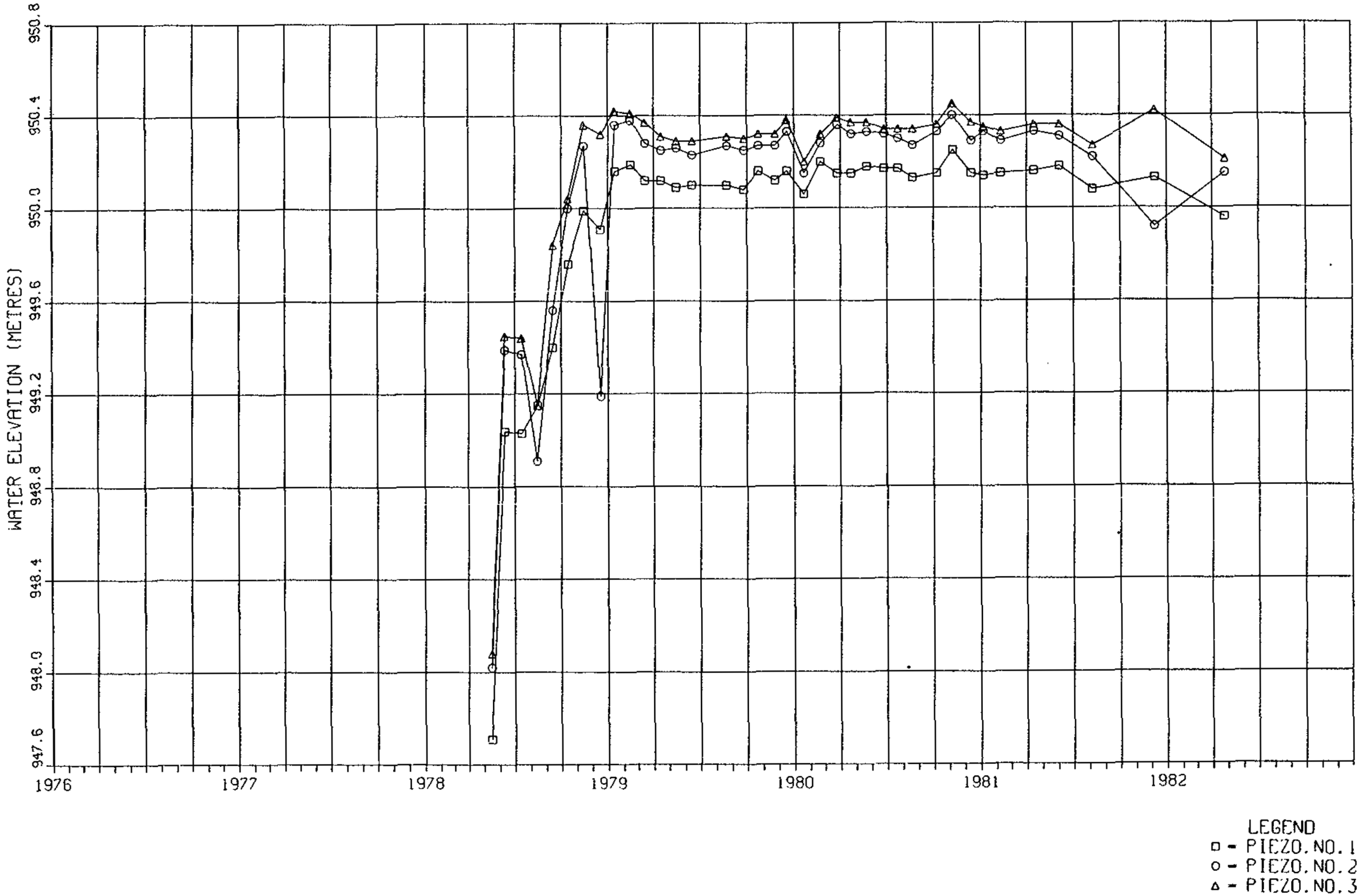
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-51



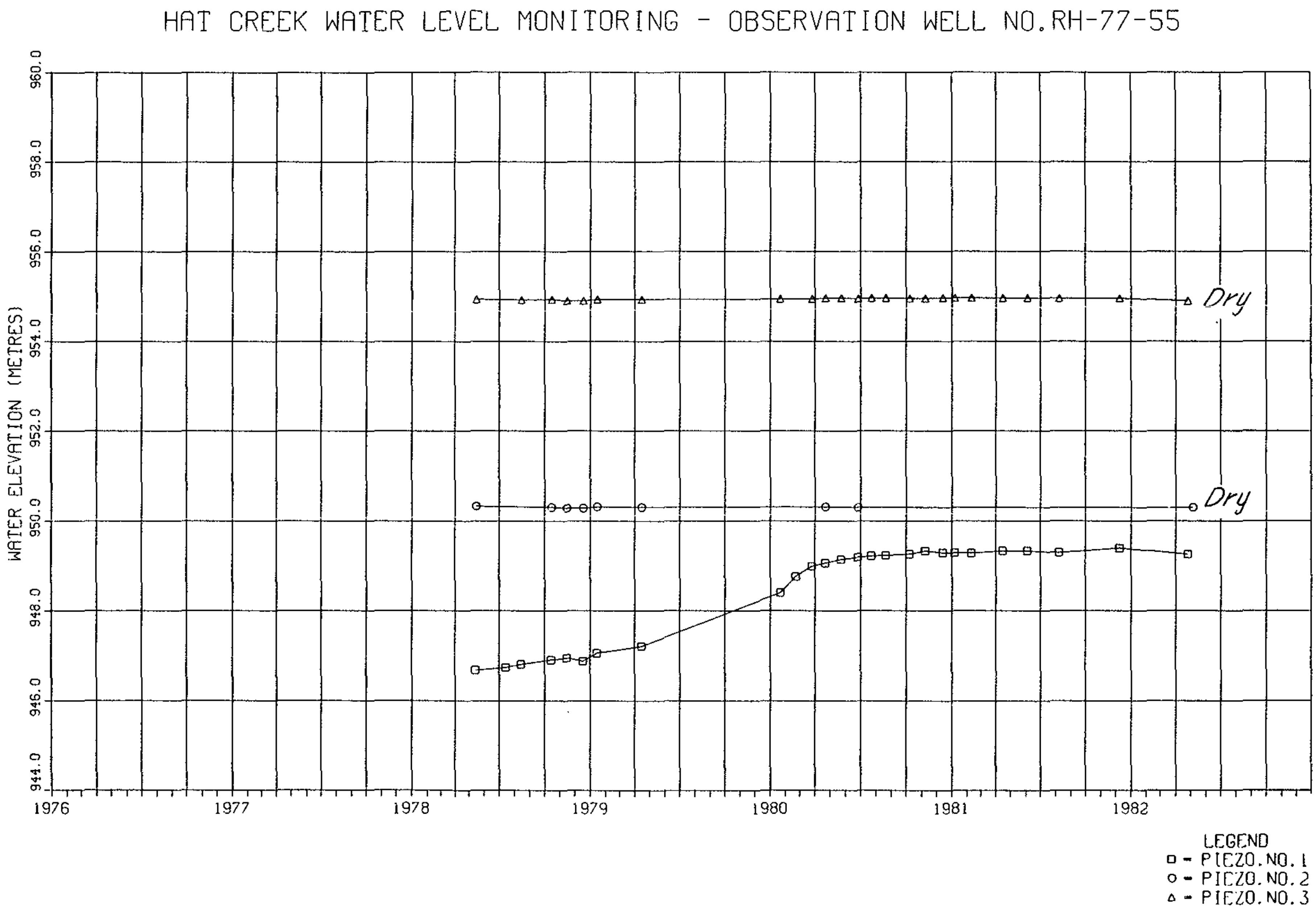
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-53



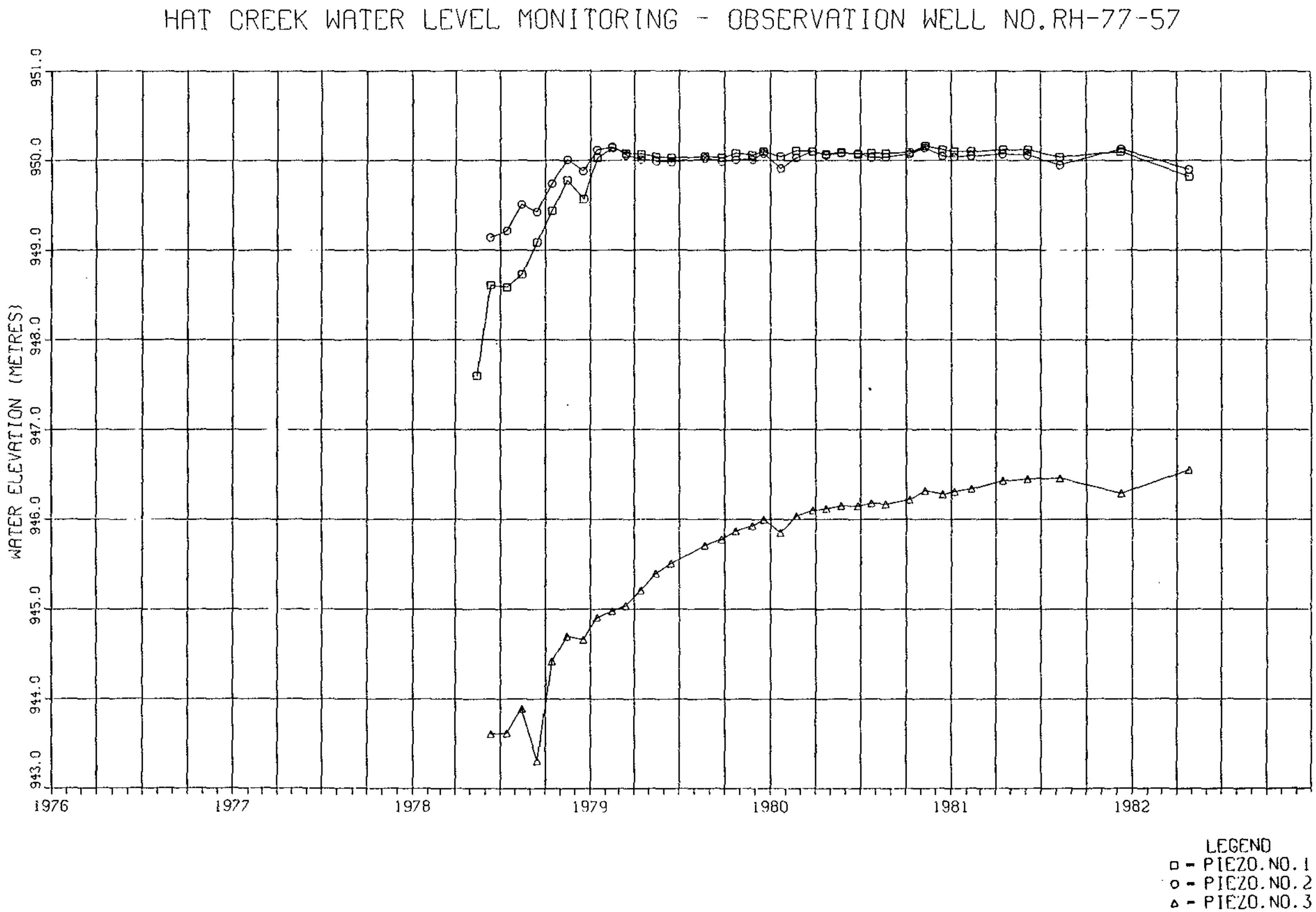
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-54A



Golder Associates

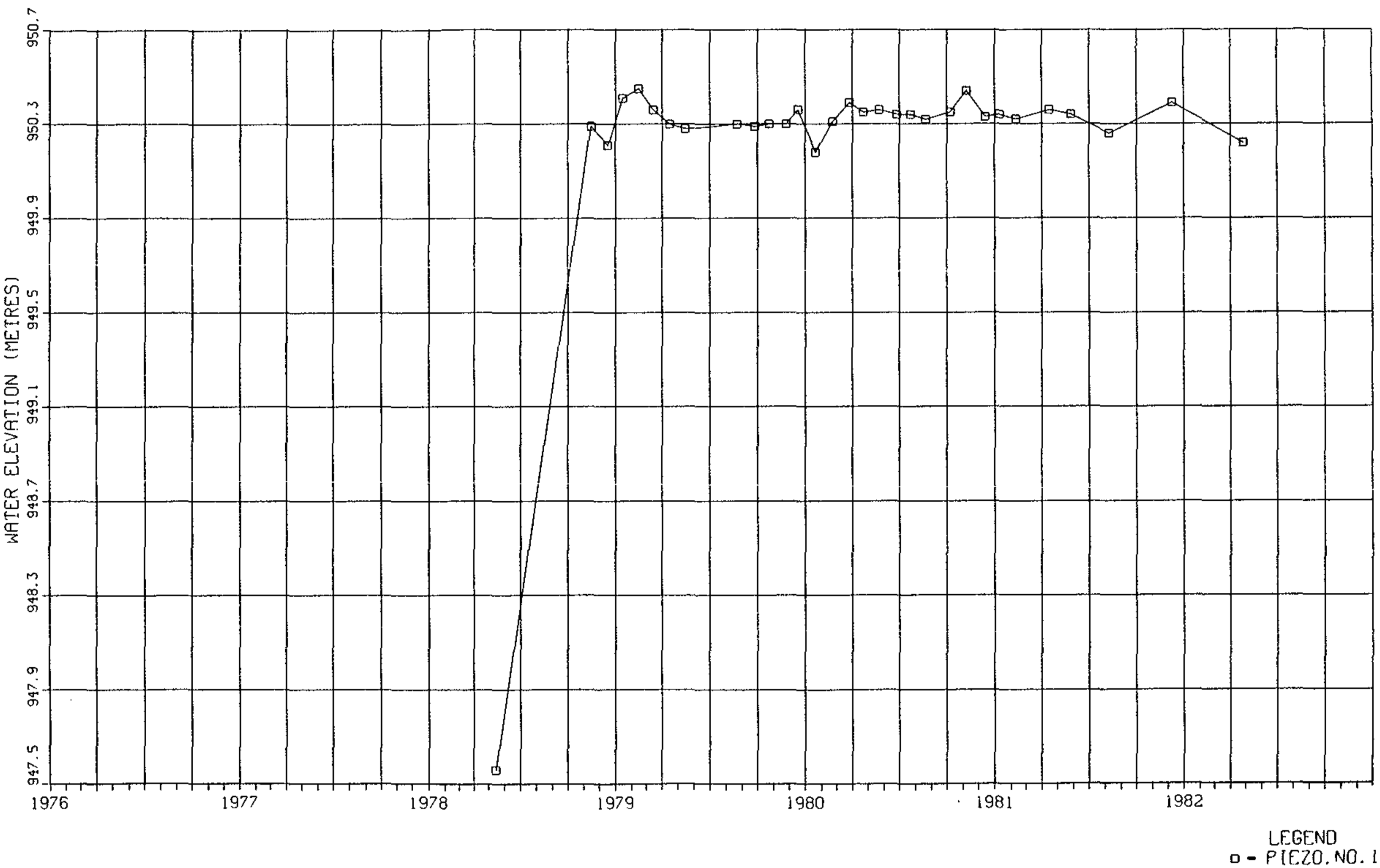


Golder Associates



Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-58



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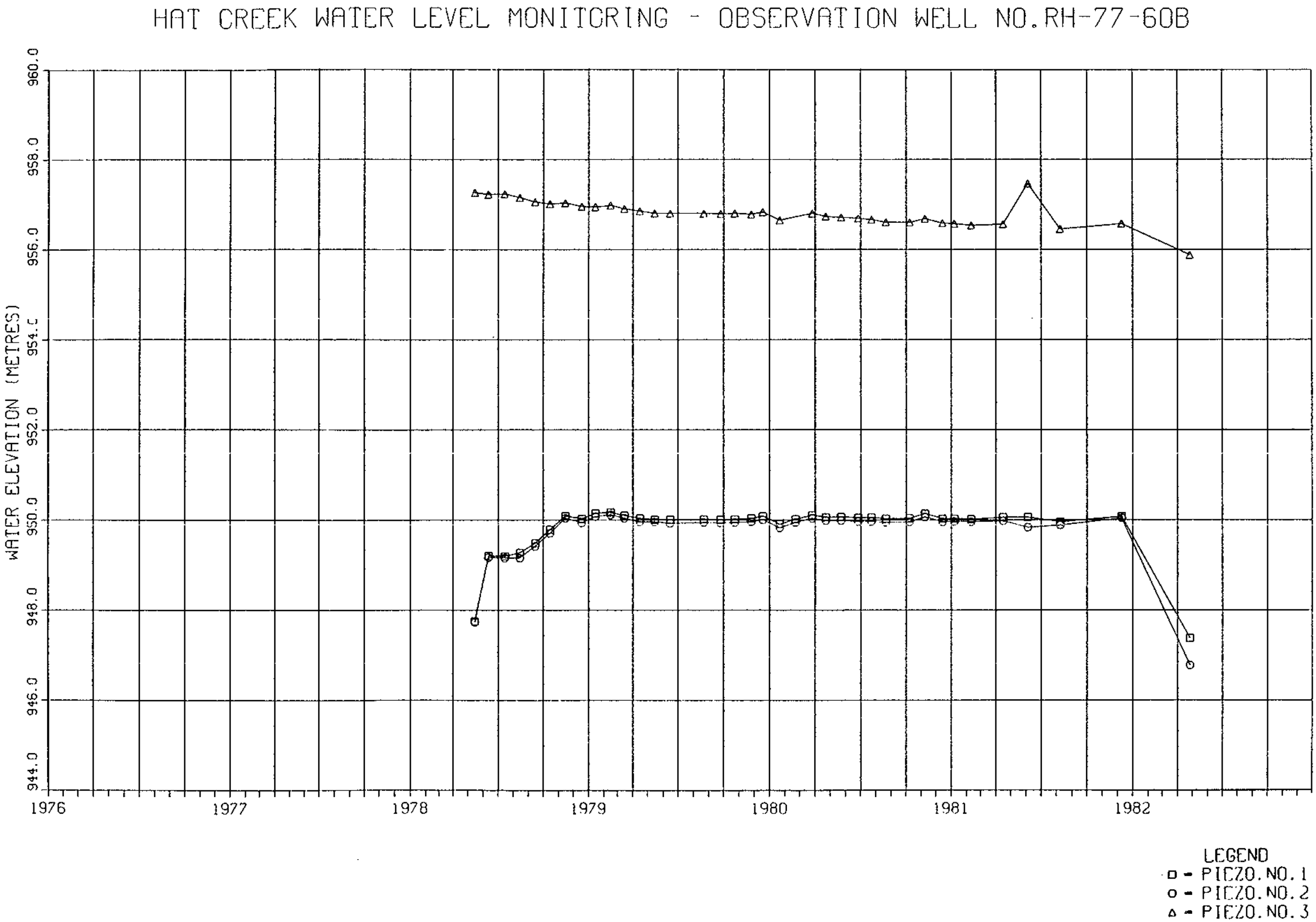

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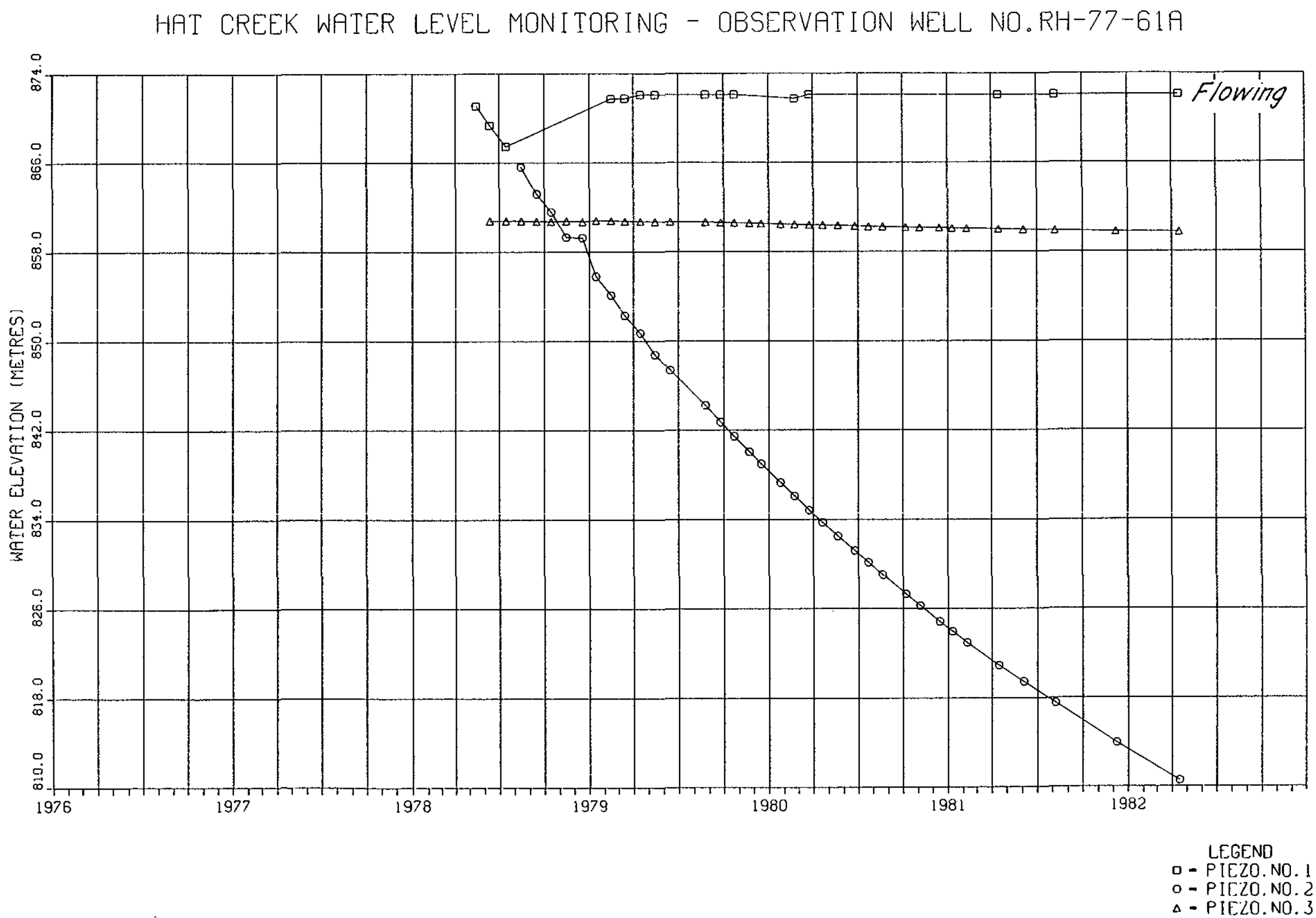

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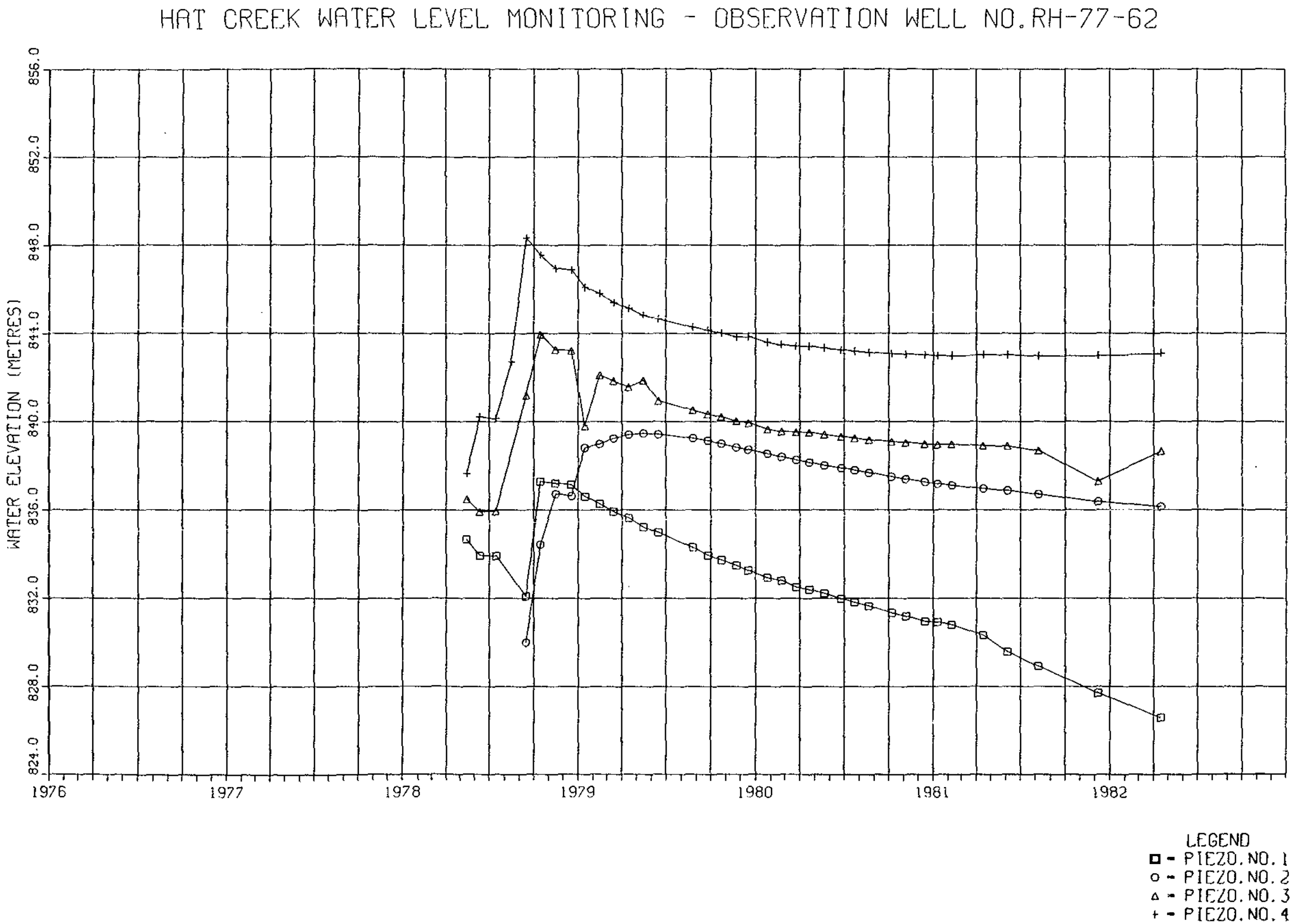

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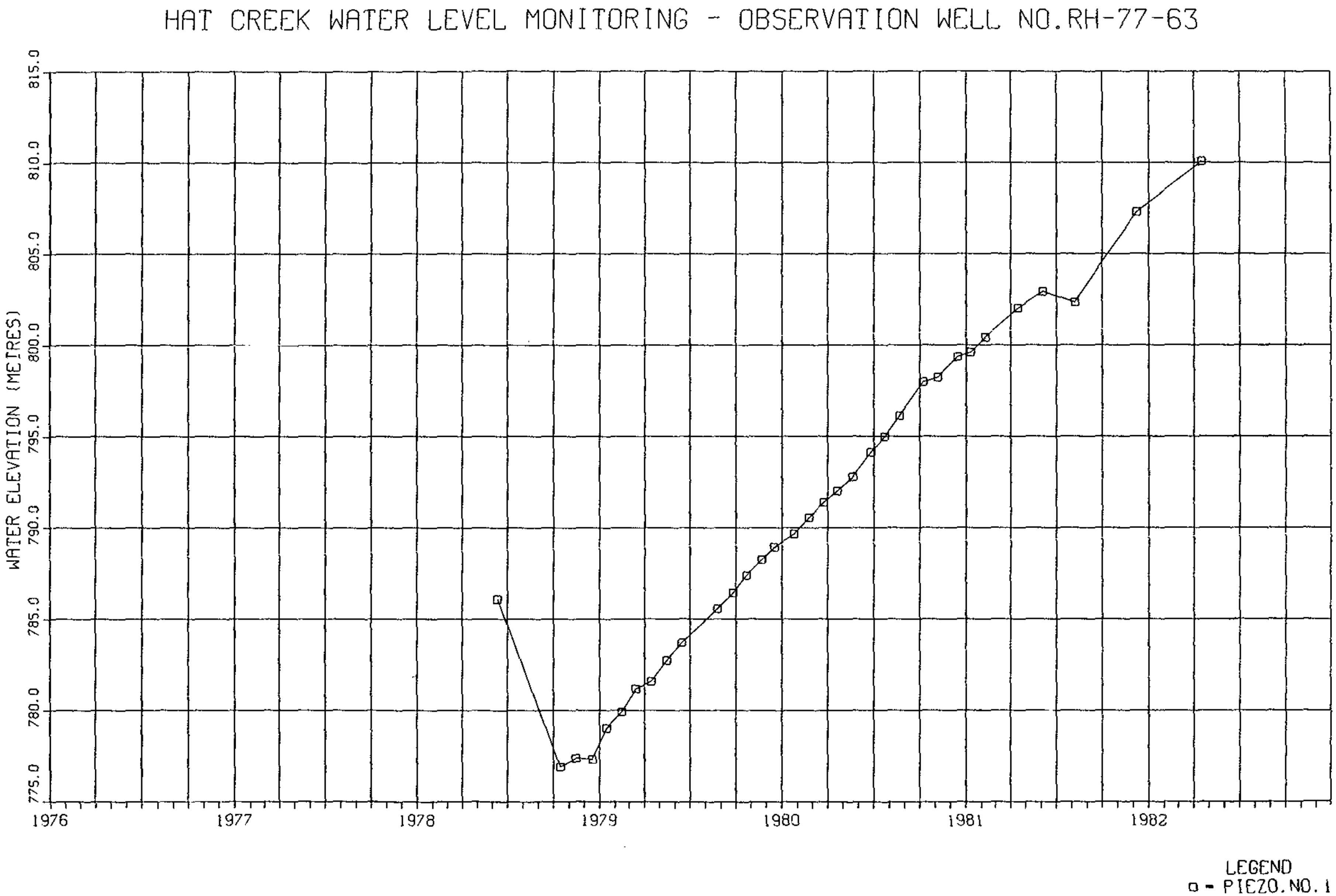
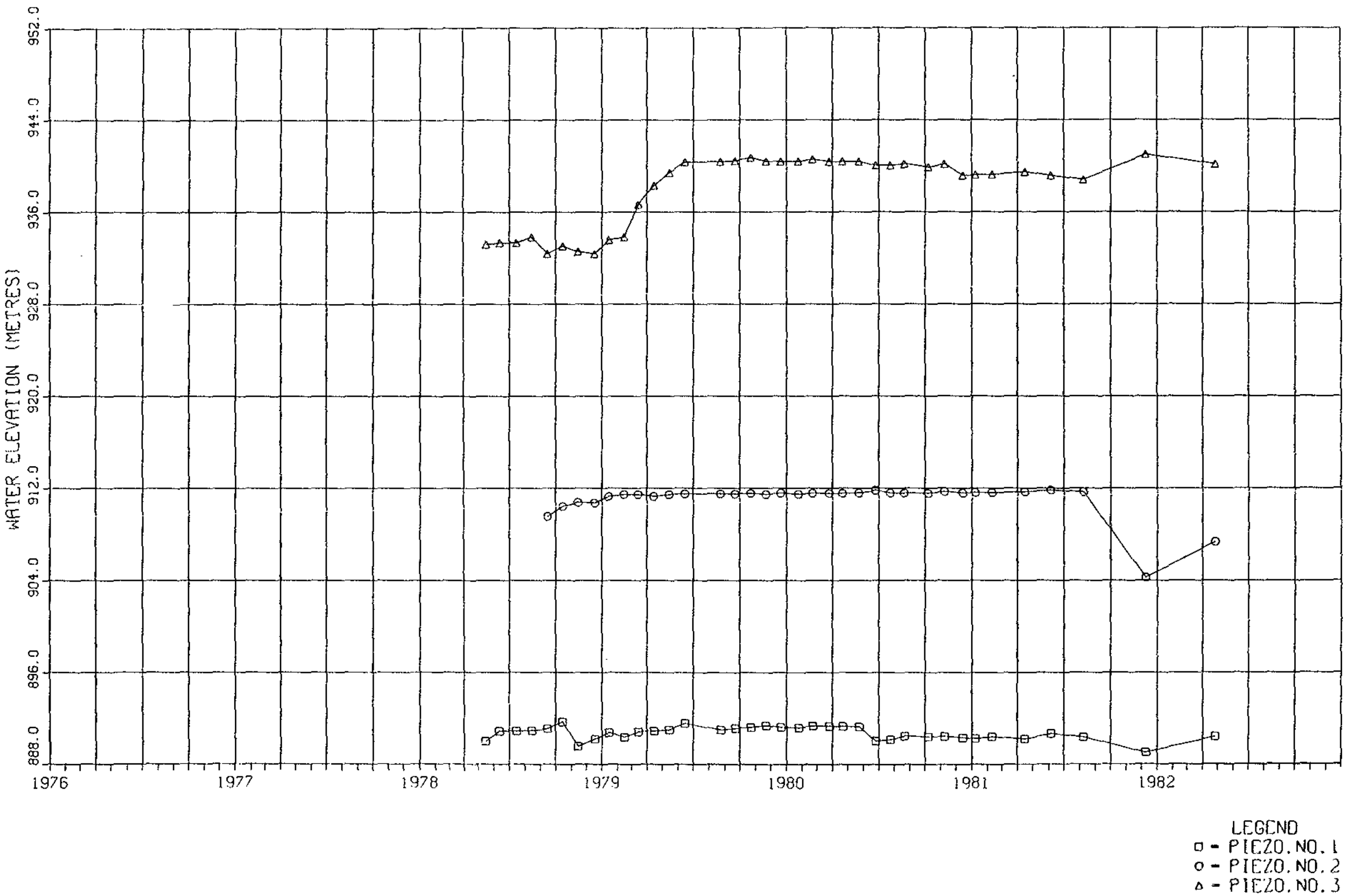
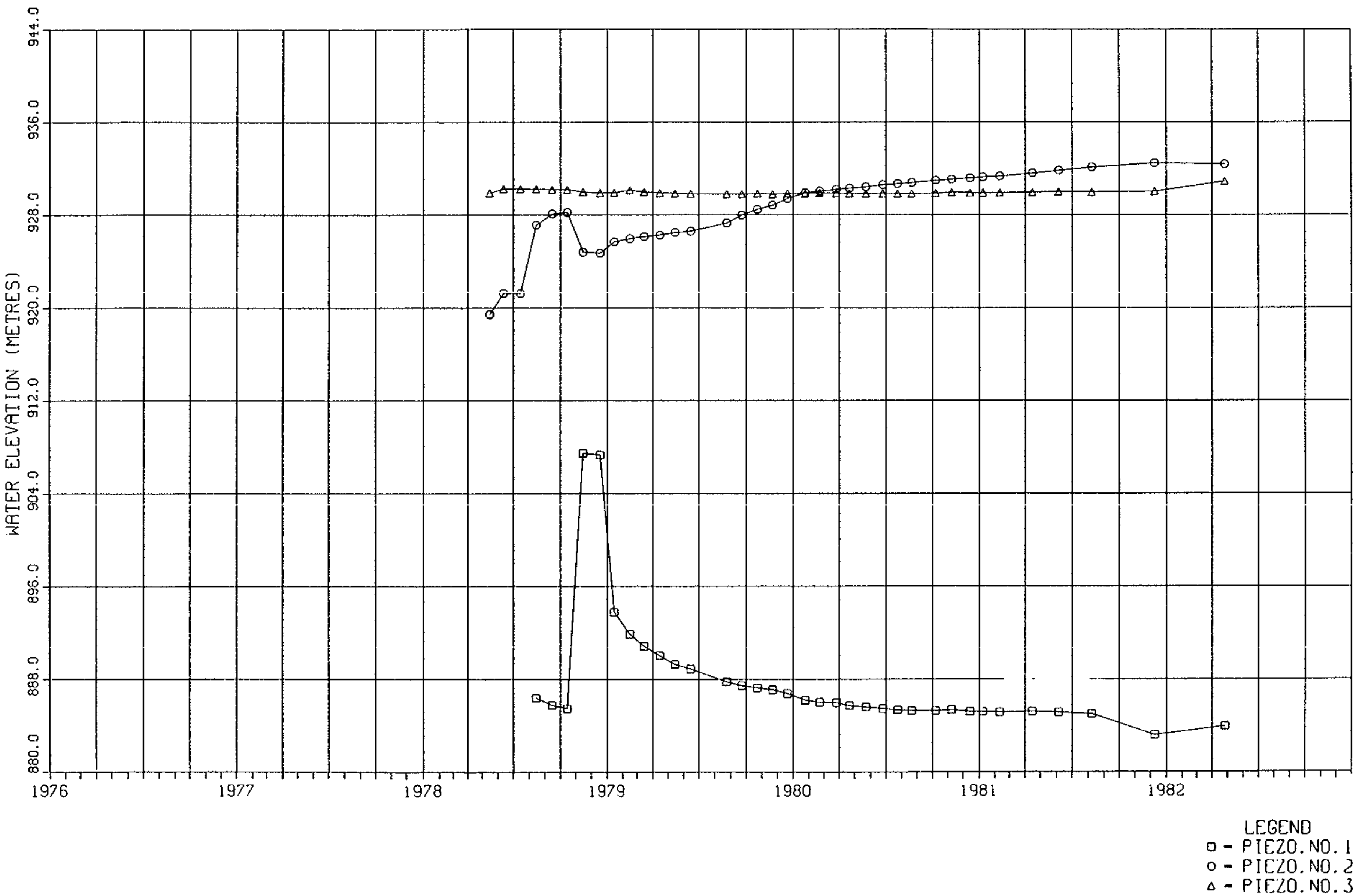

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-77-63
WATER ELEVATION (METRES)
815.0
810.0
805.0
800.0
795.0
790.0
785.0
780.0
775.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

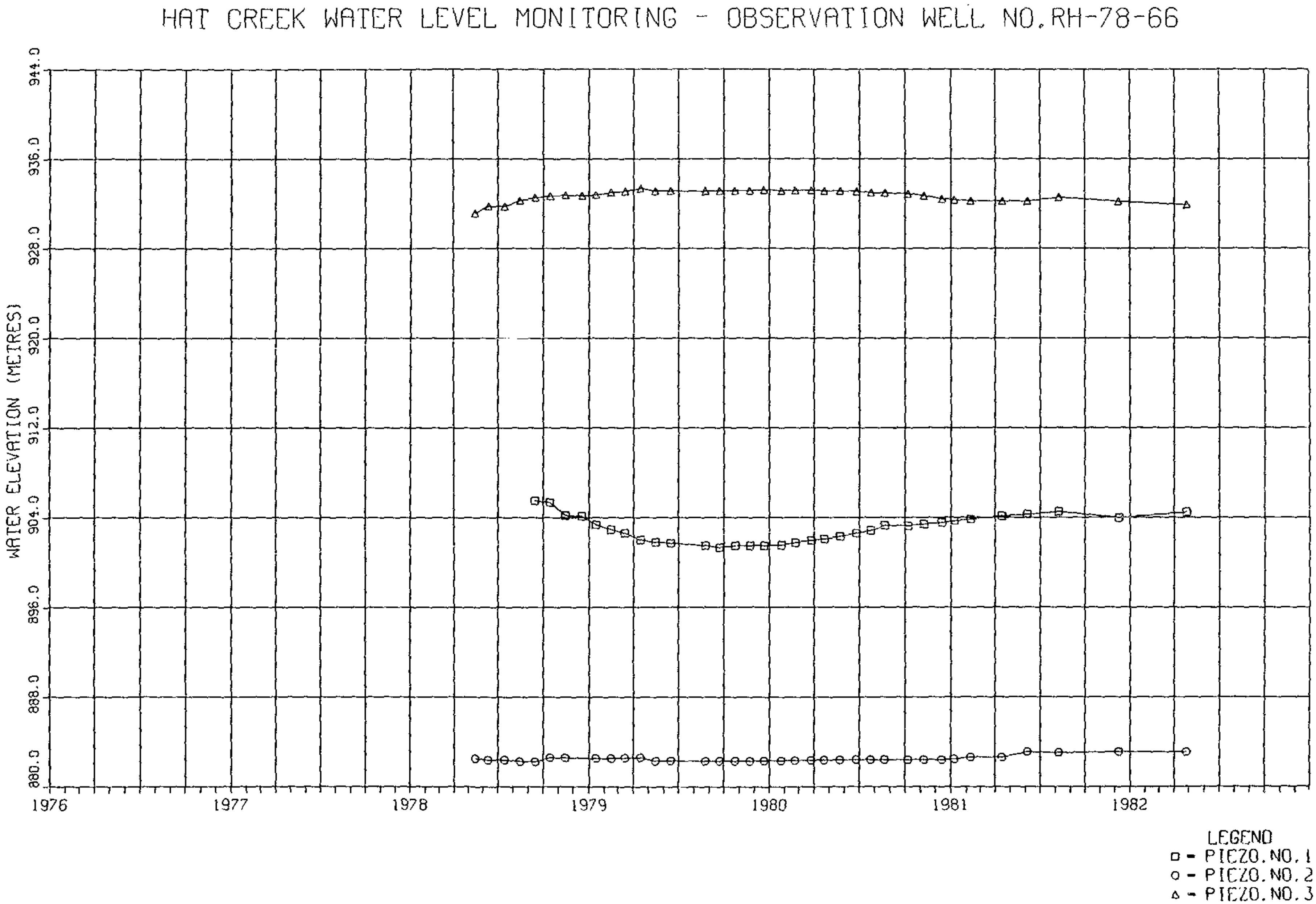
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-64



Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-65





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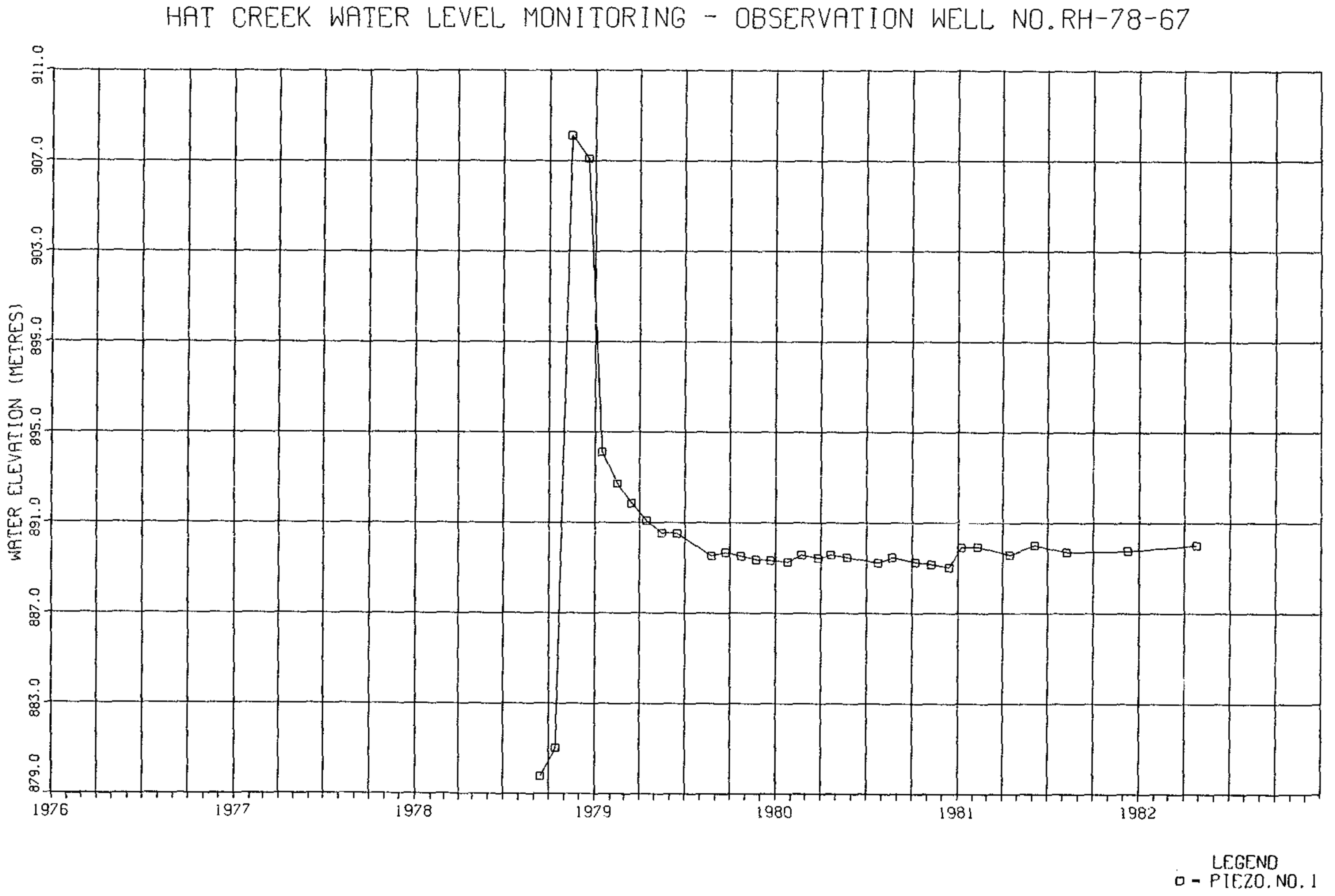

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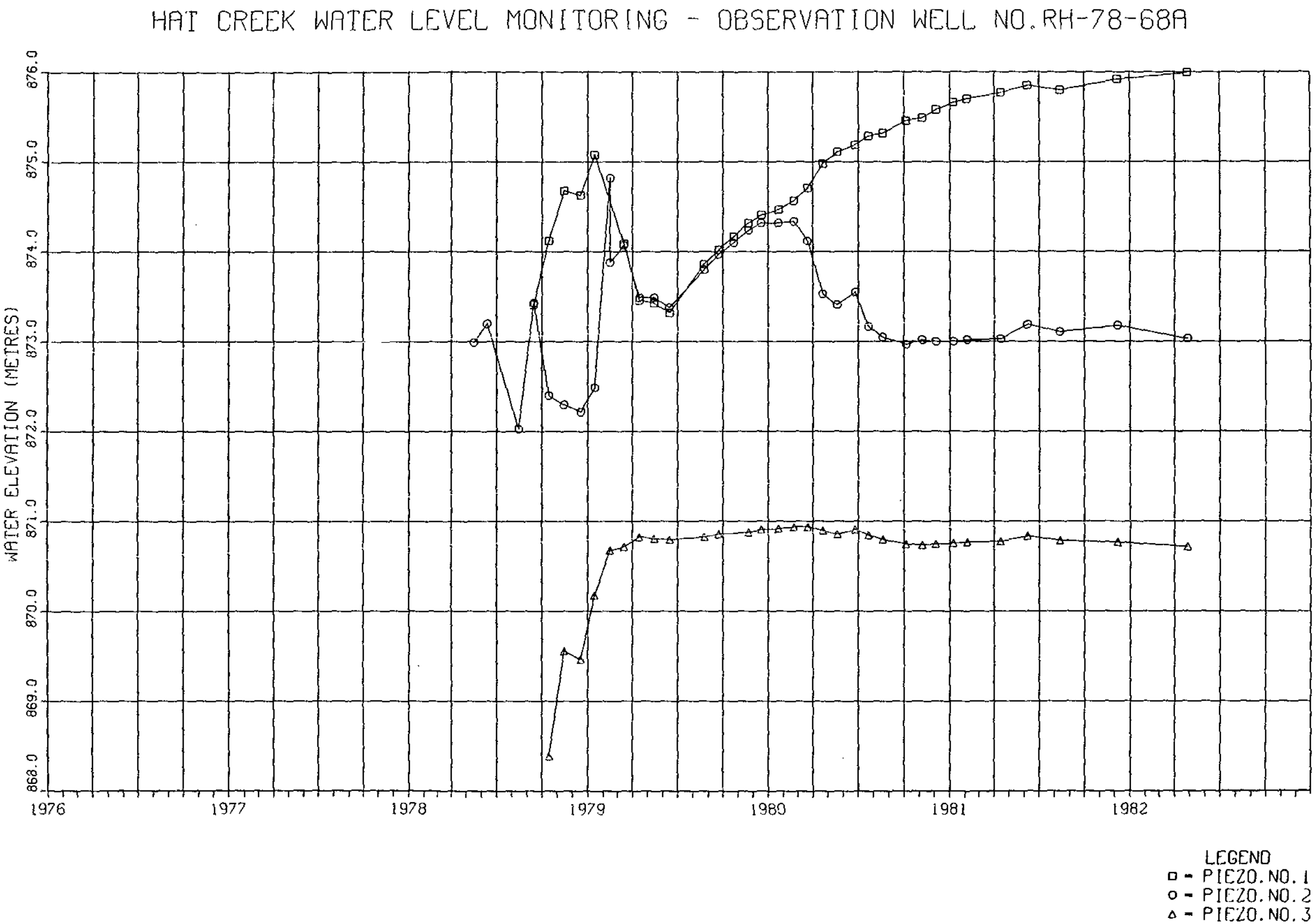

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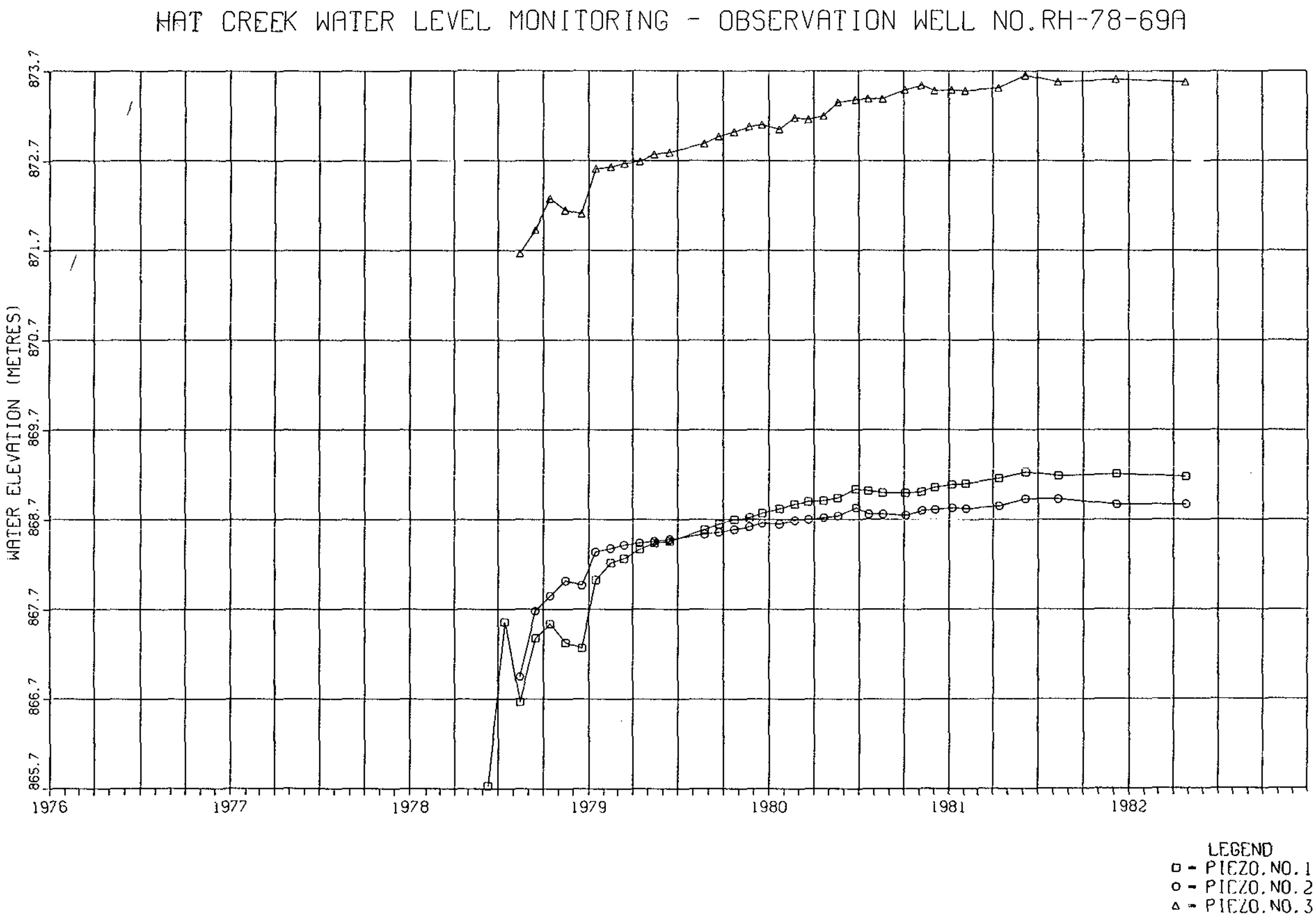
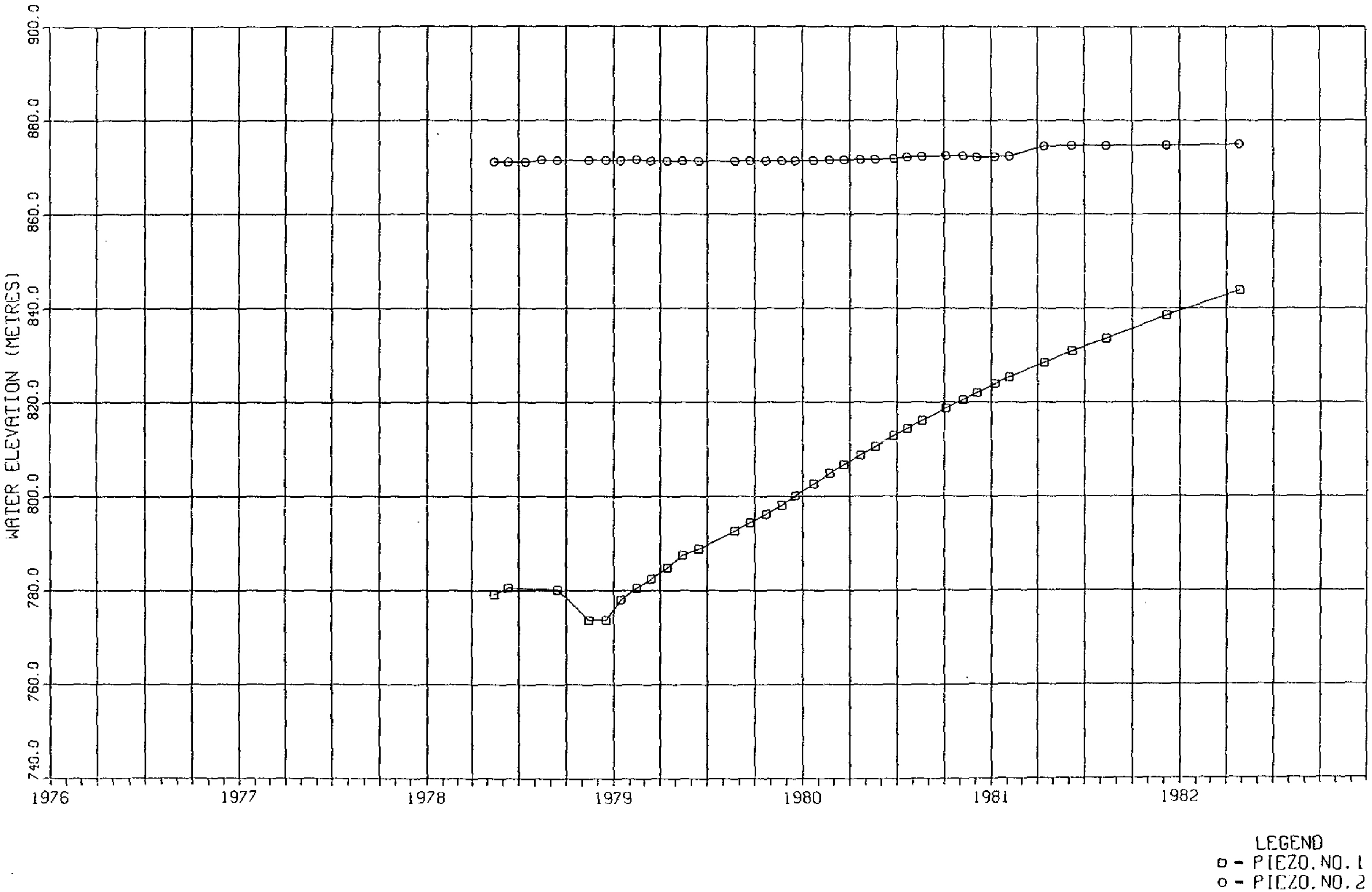

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-69A
WATER ELEVATION (METRES)
873.7
872.7
871.7
870.7
869.7
868.7
867.7
866.7
865.7
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1
○ - PIEZO.NO.2
△ - PIEZO.NO.3

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-70



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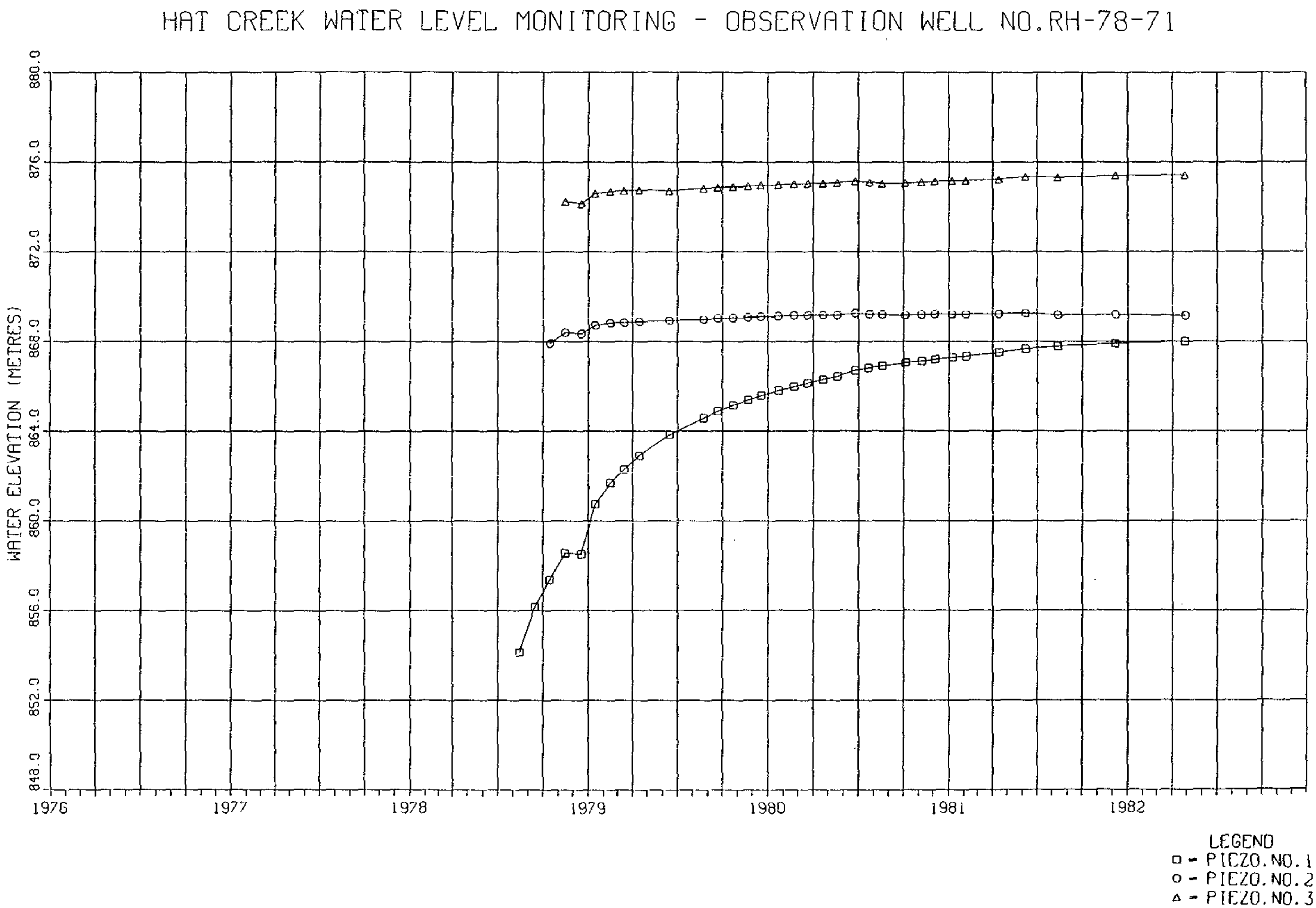
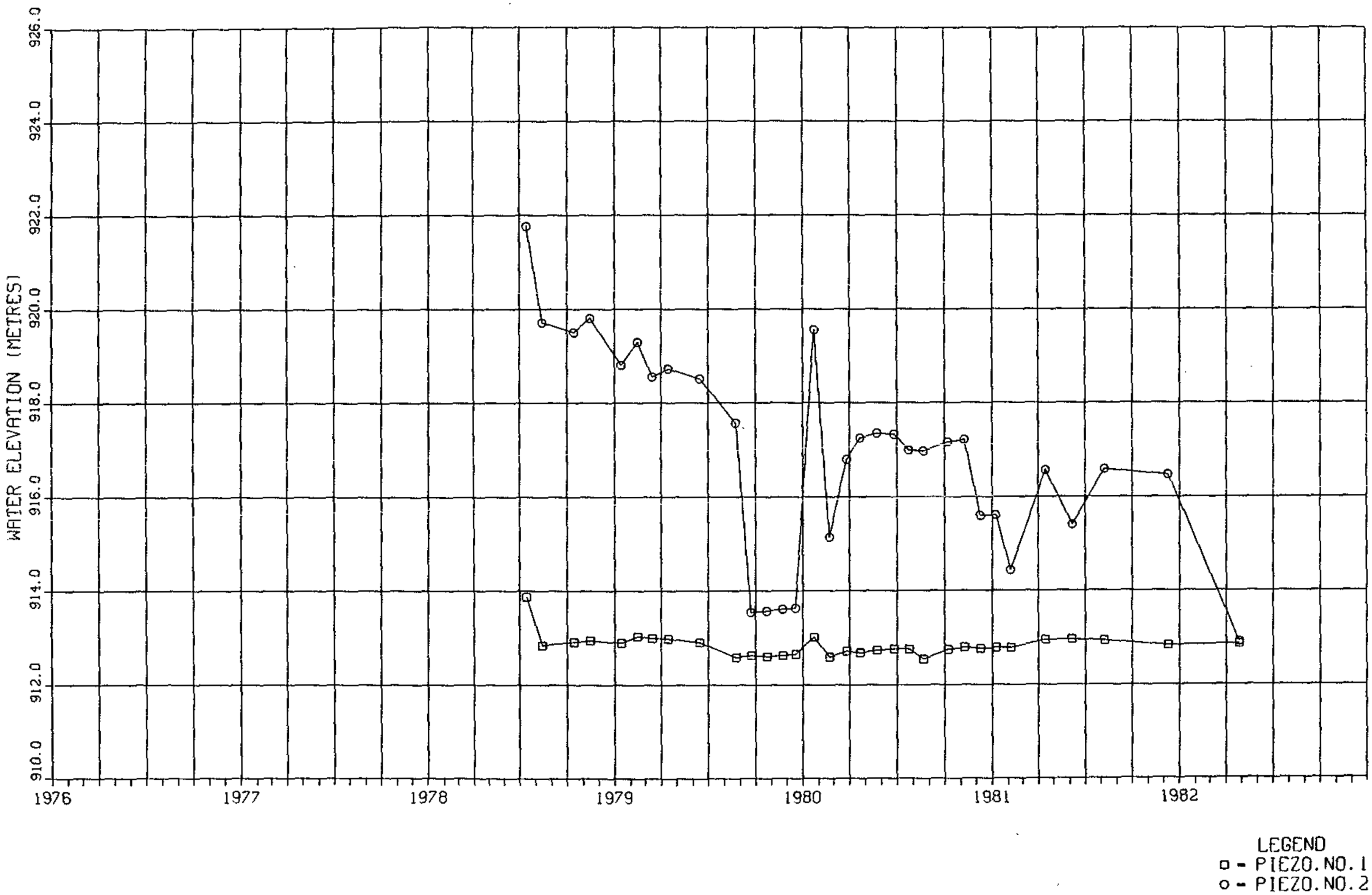
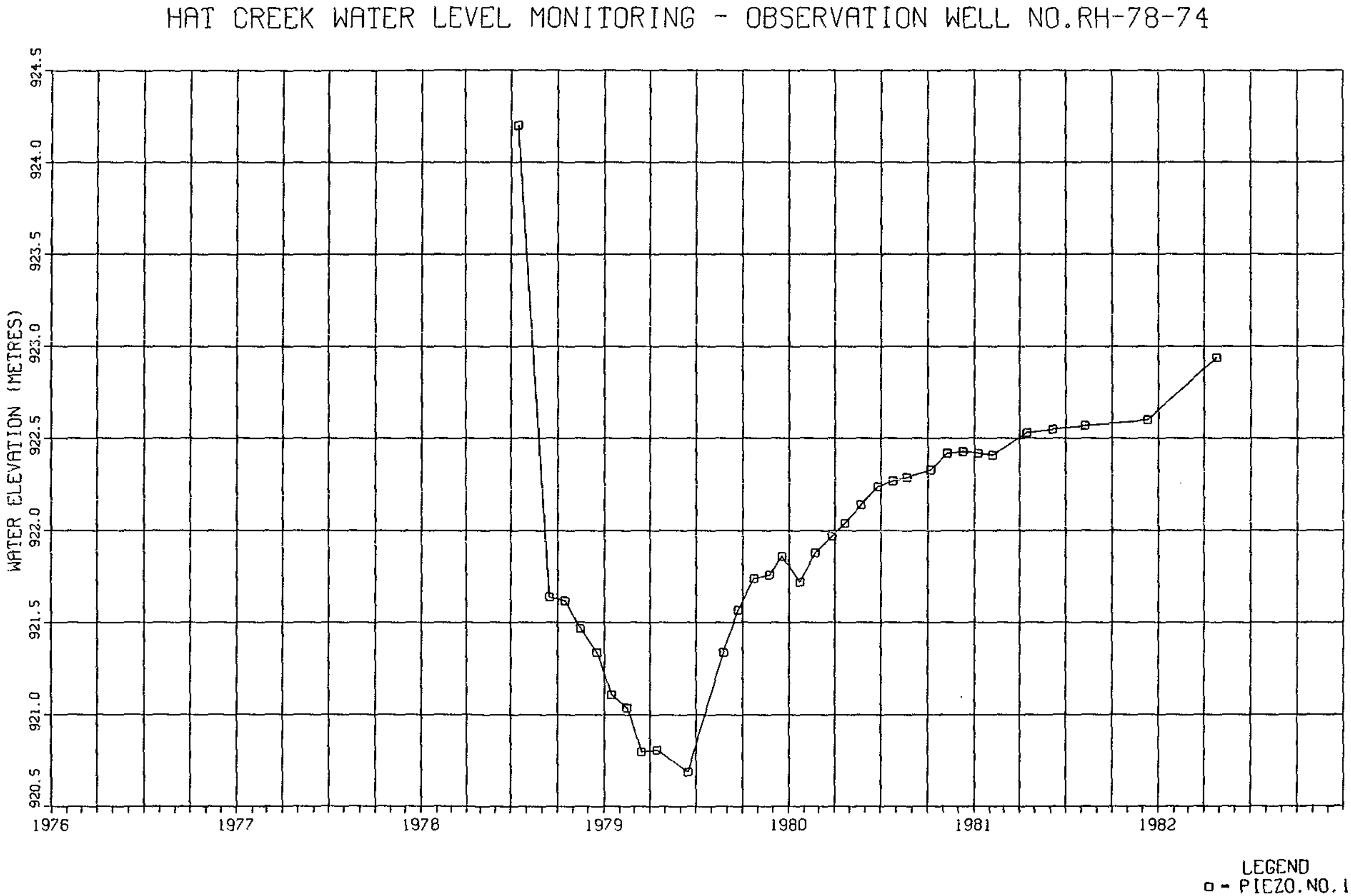


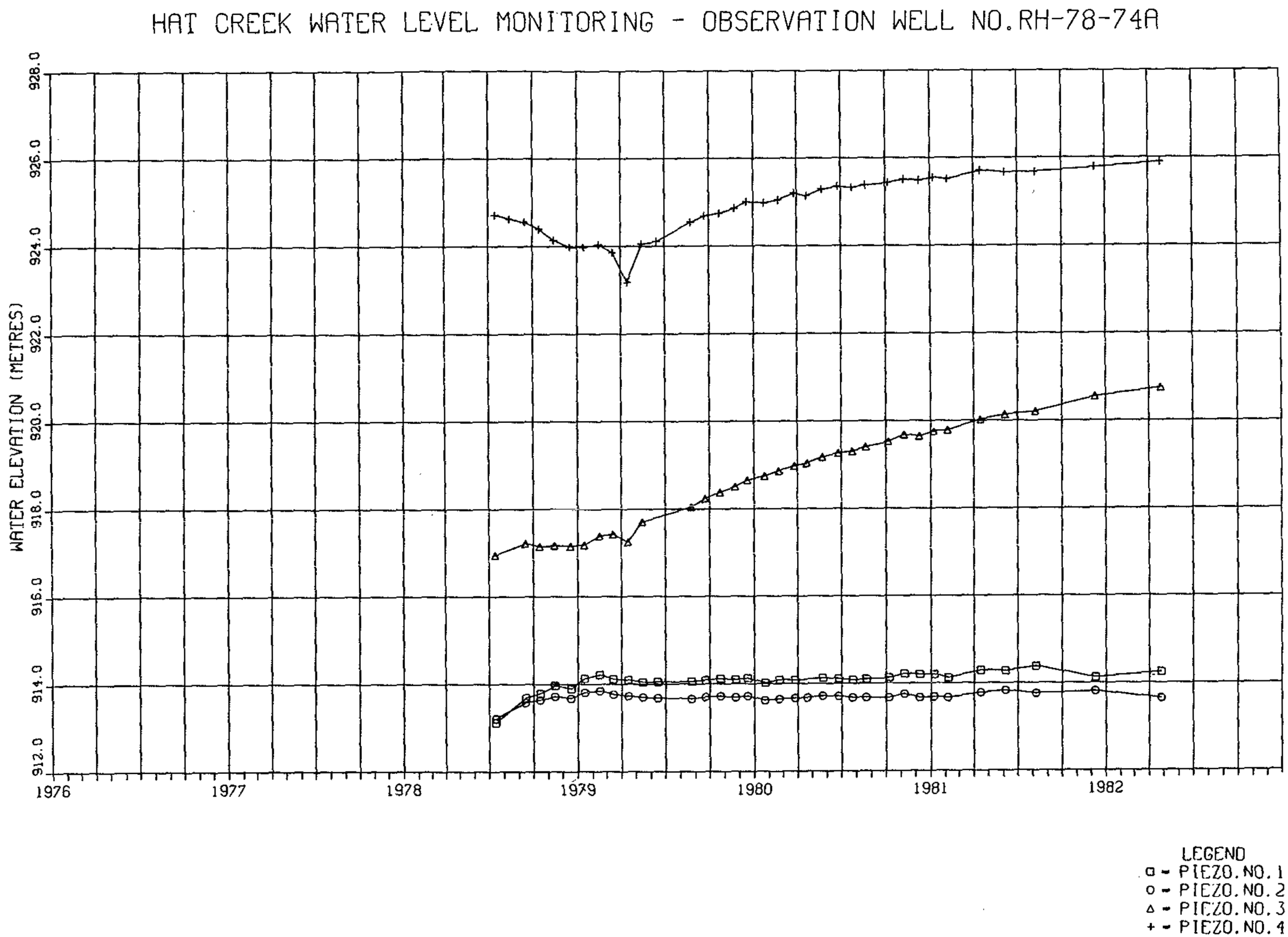
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-73

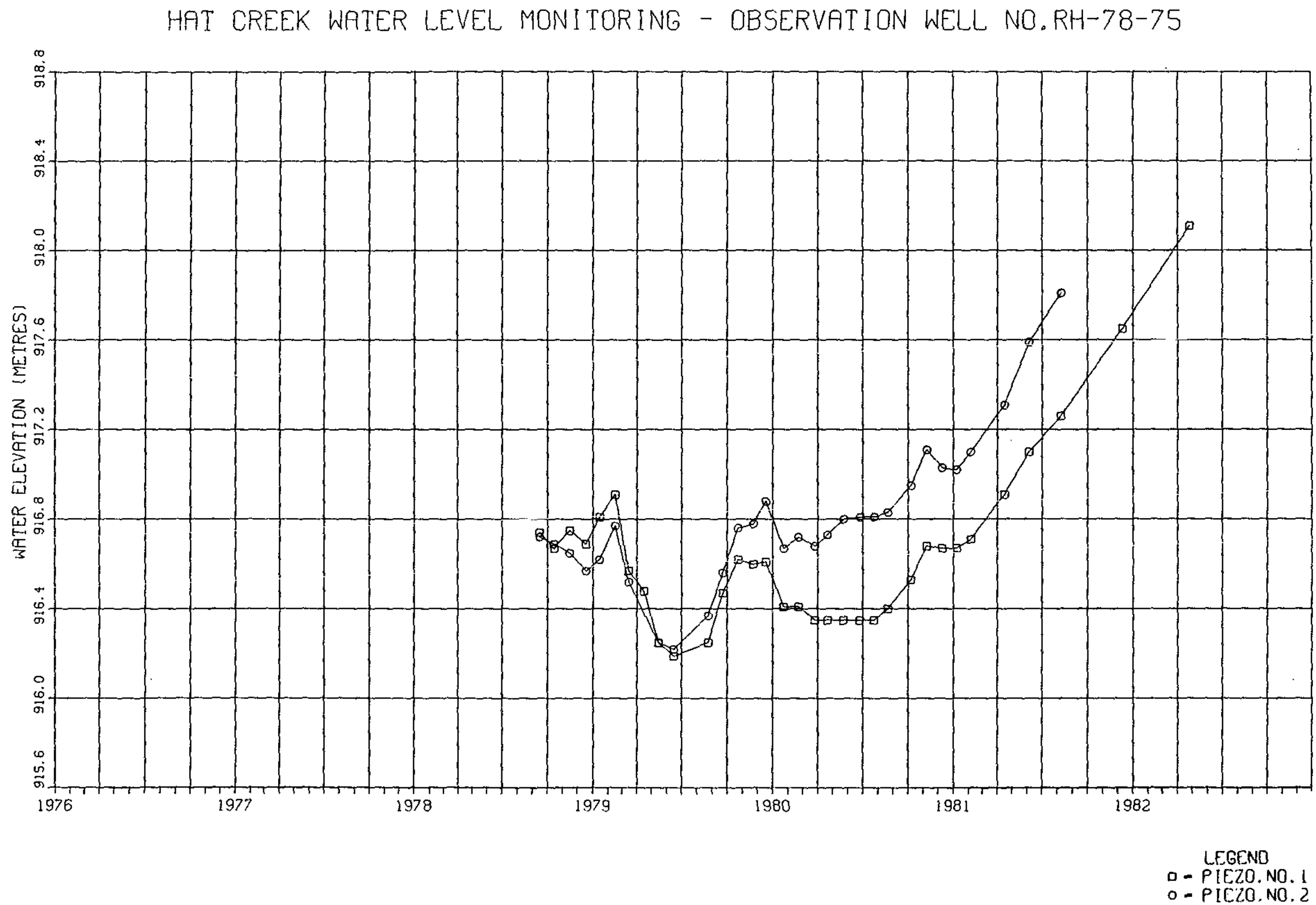


Golder Associates

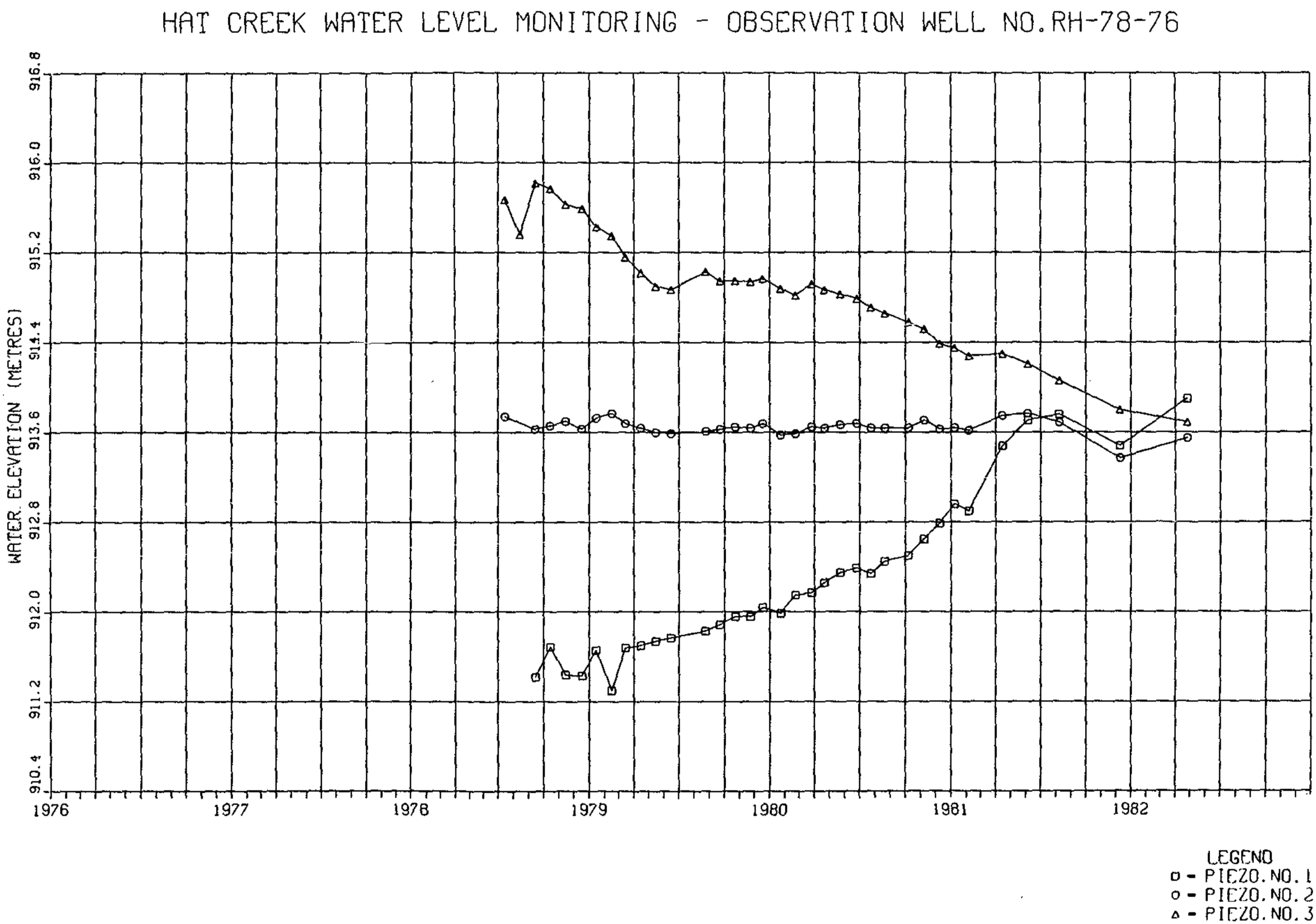

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-74
WATER ELEVATION (METRES)
924.5
924.0
923.5
923.0
922.5
922.0
921.5
921.0
920.5
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1



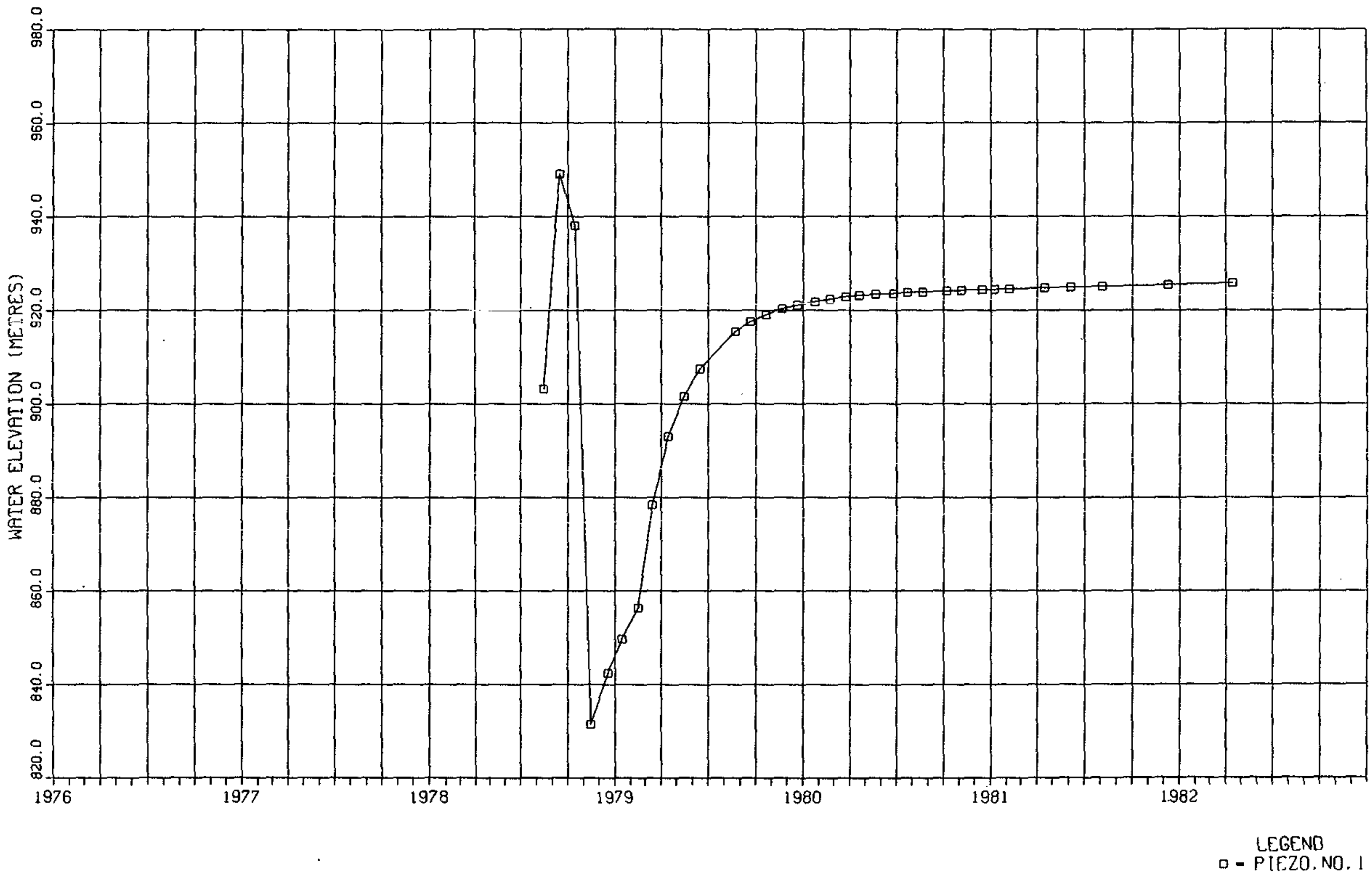
Golder Associates

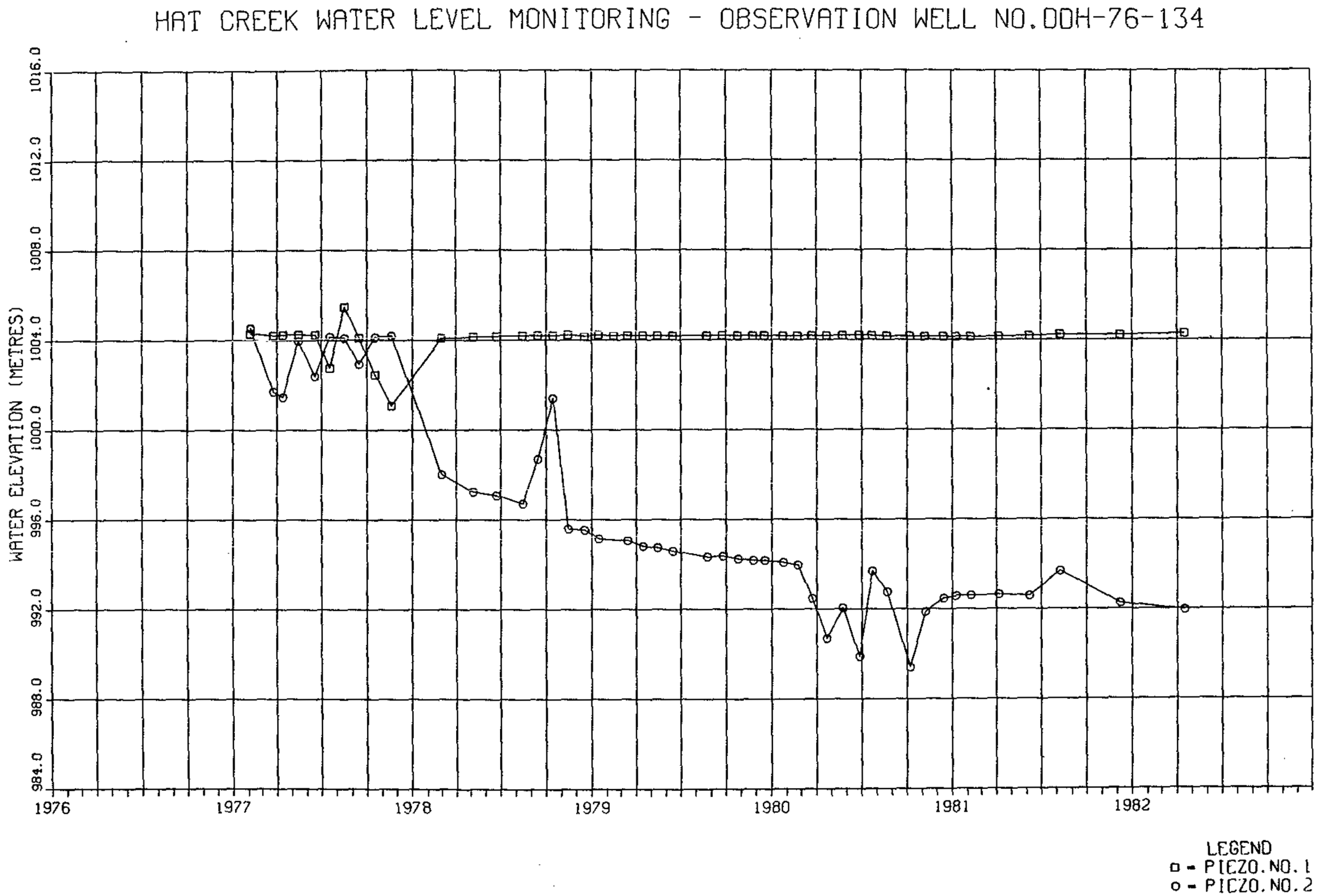


Golder Associates


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-76
WATER ELEVATION (METRES)
916.8
916.0
915.2
914.4
913.6
912.8
912.0
911.2
910.4
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO. NO. 1
○ - PIEZO. NO. 2
△ - PIEZO. NO. 3

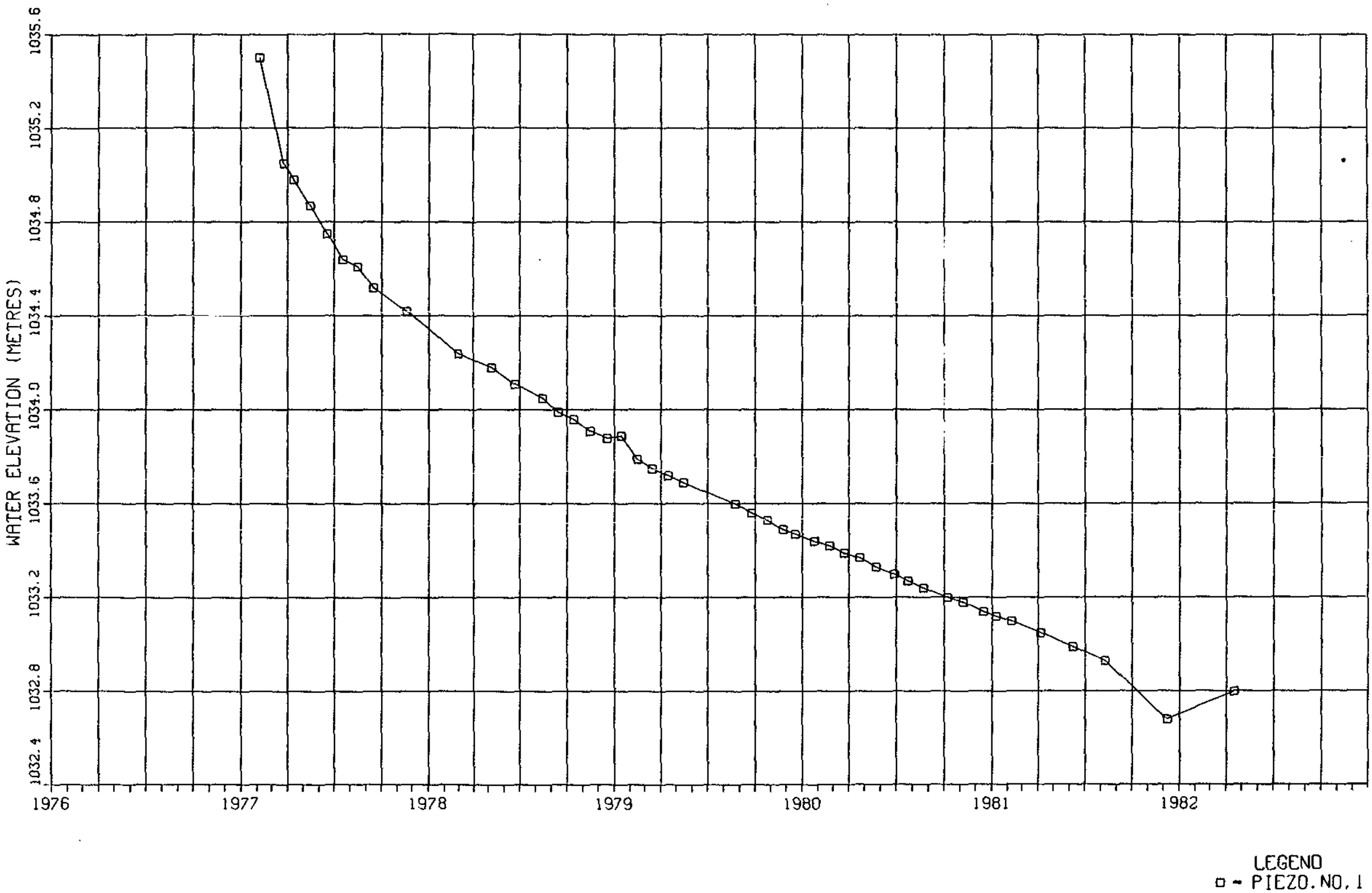
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.RH-78-77





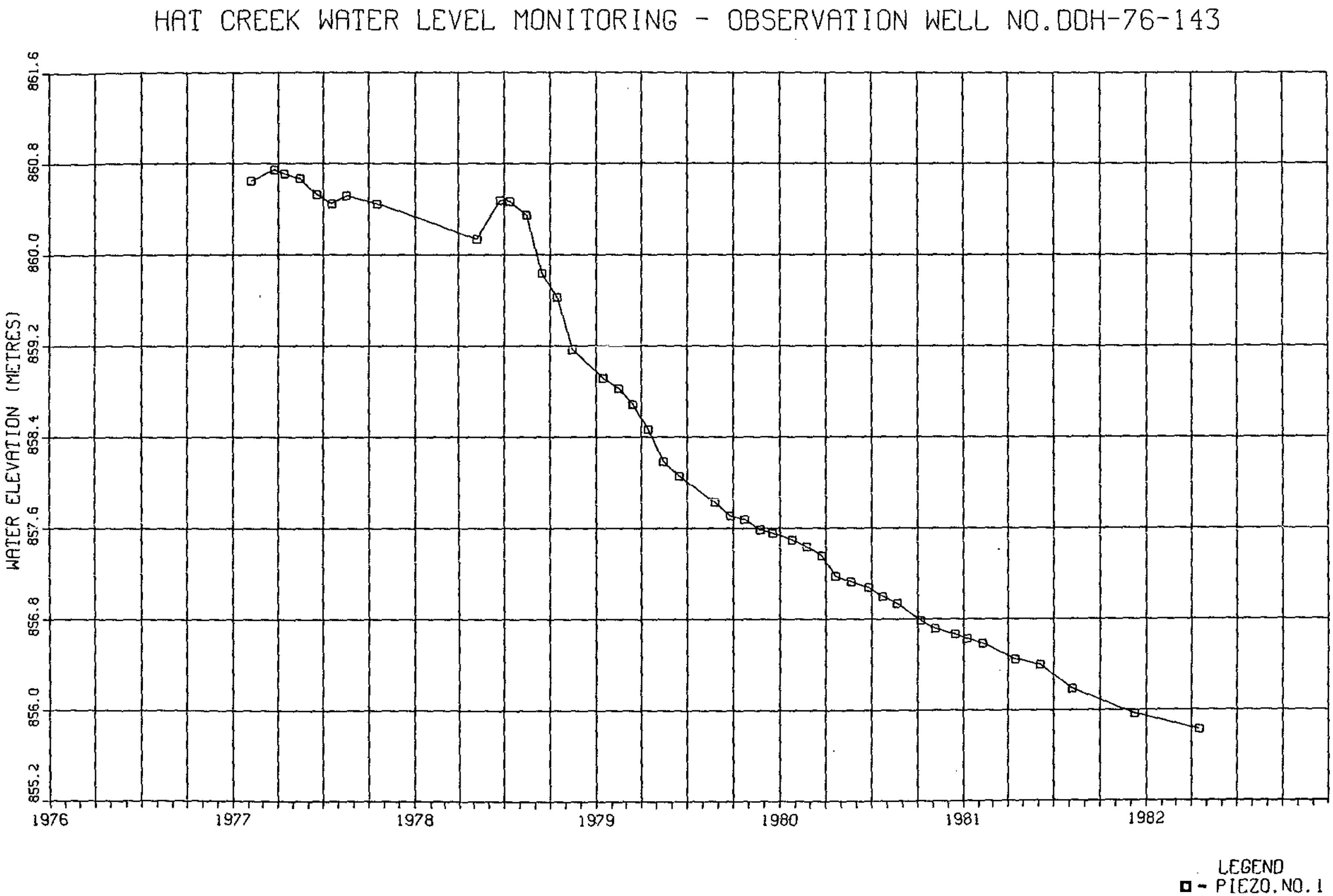
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-137

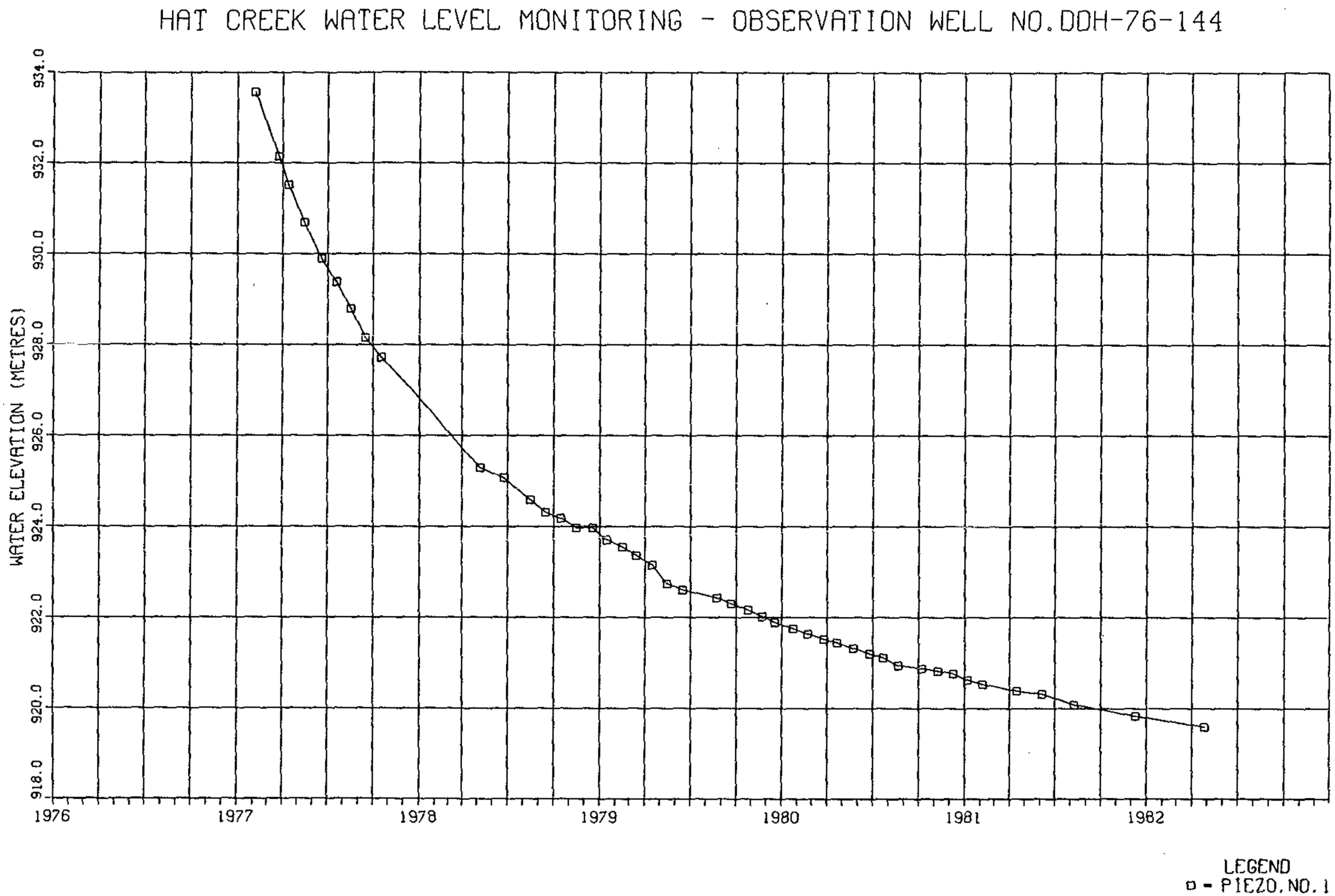


Golder Associates

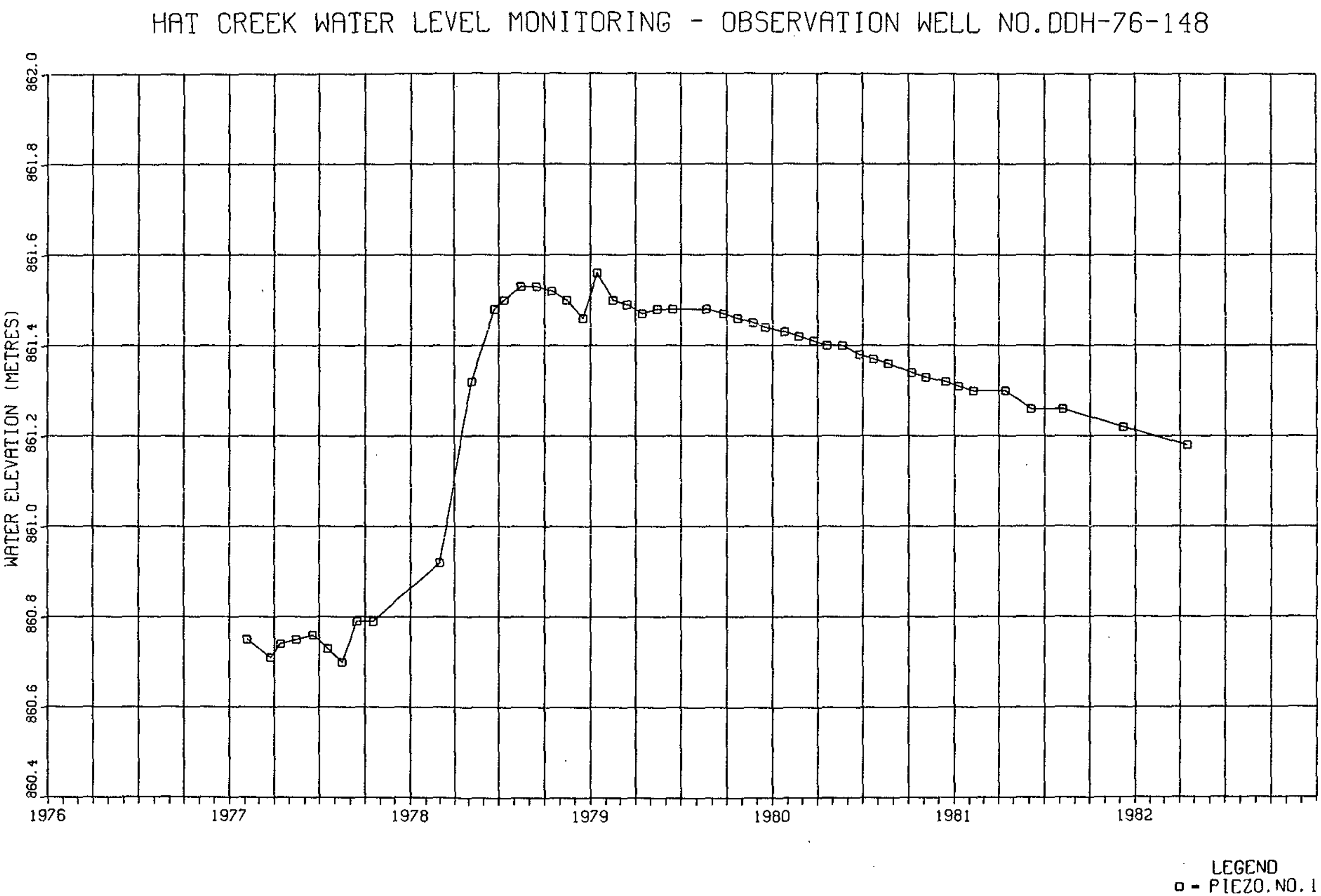
Golder Associates



Golder Associates



Golder Associates



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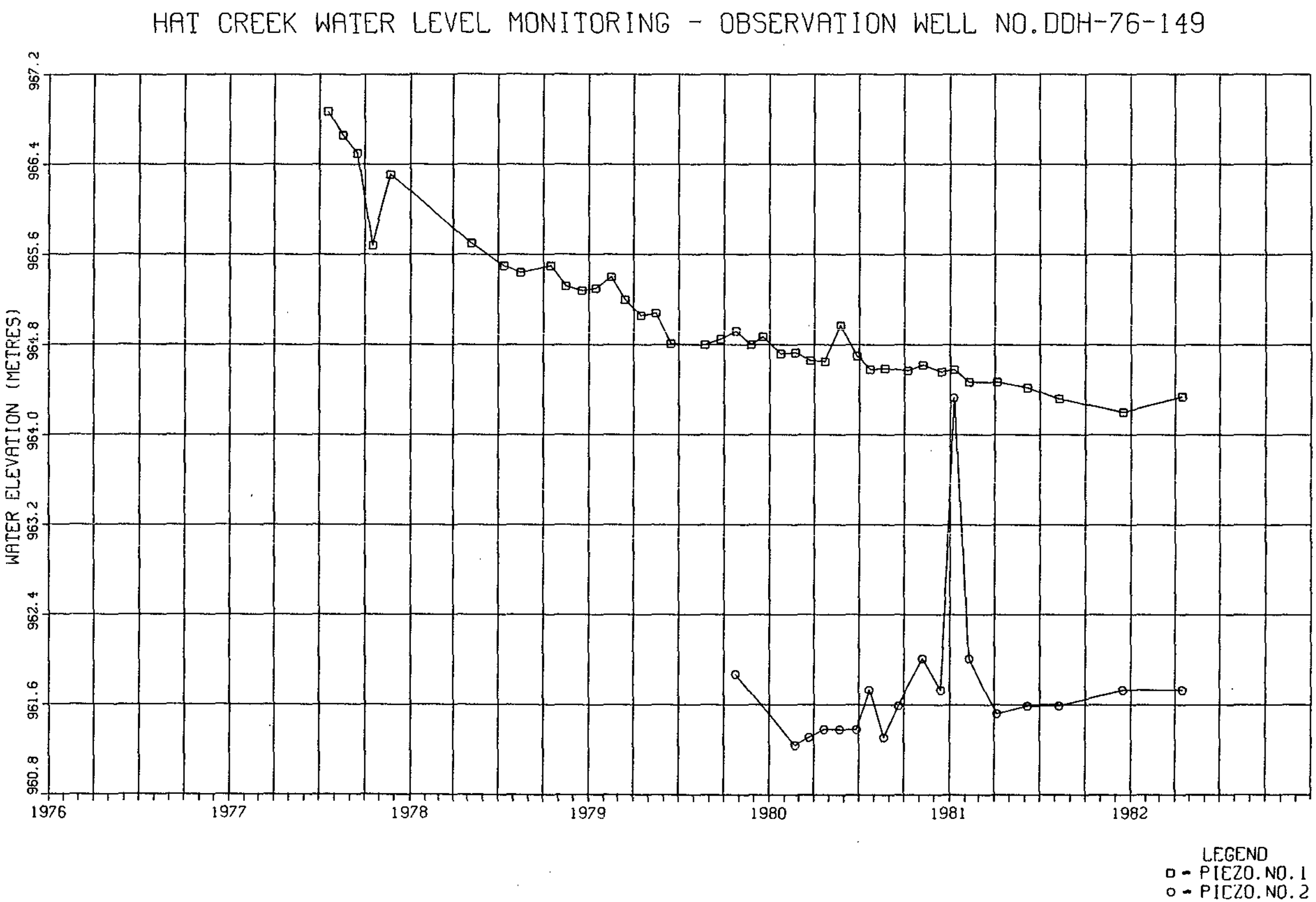
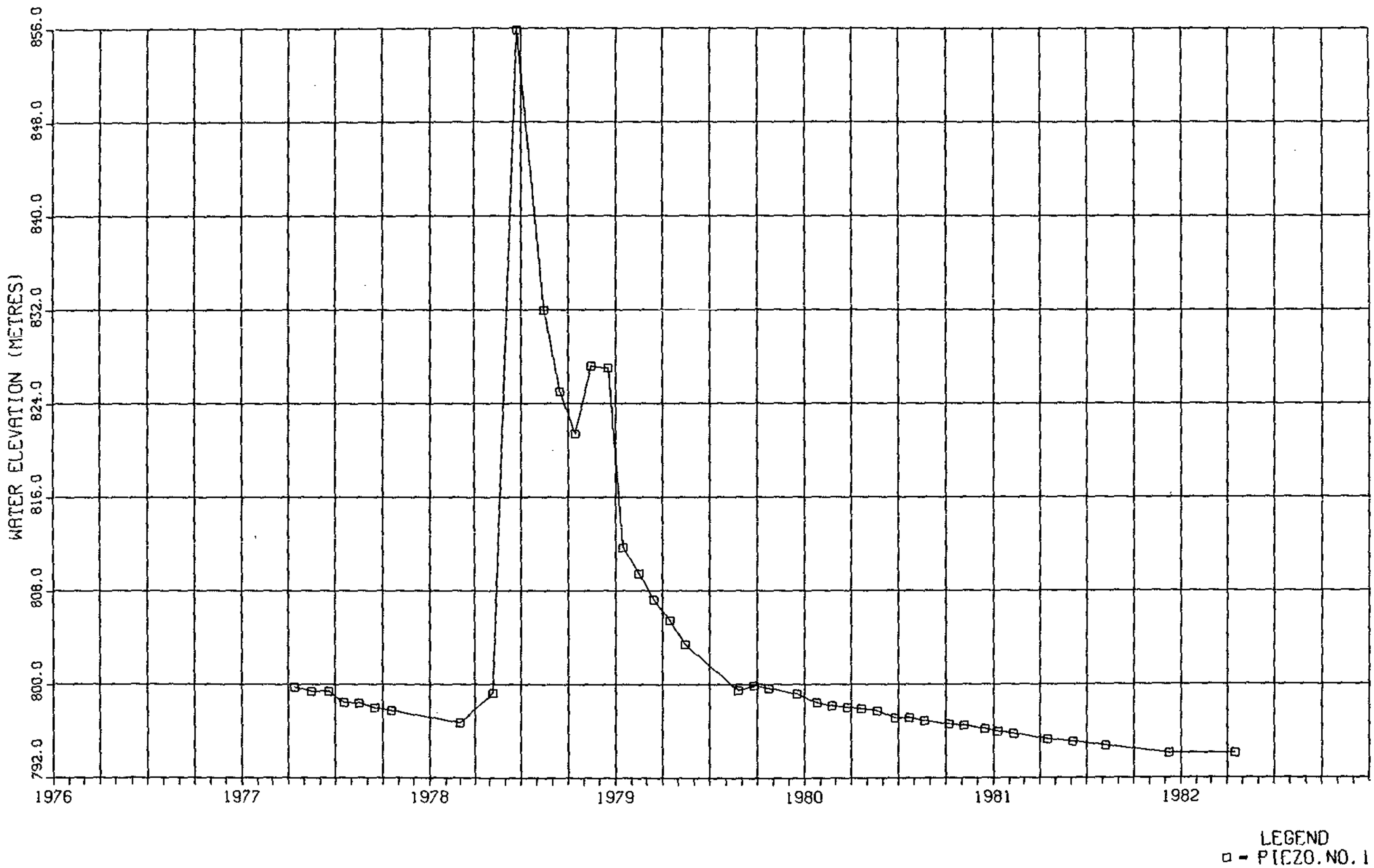
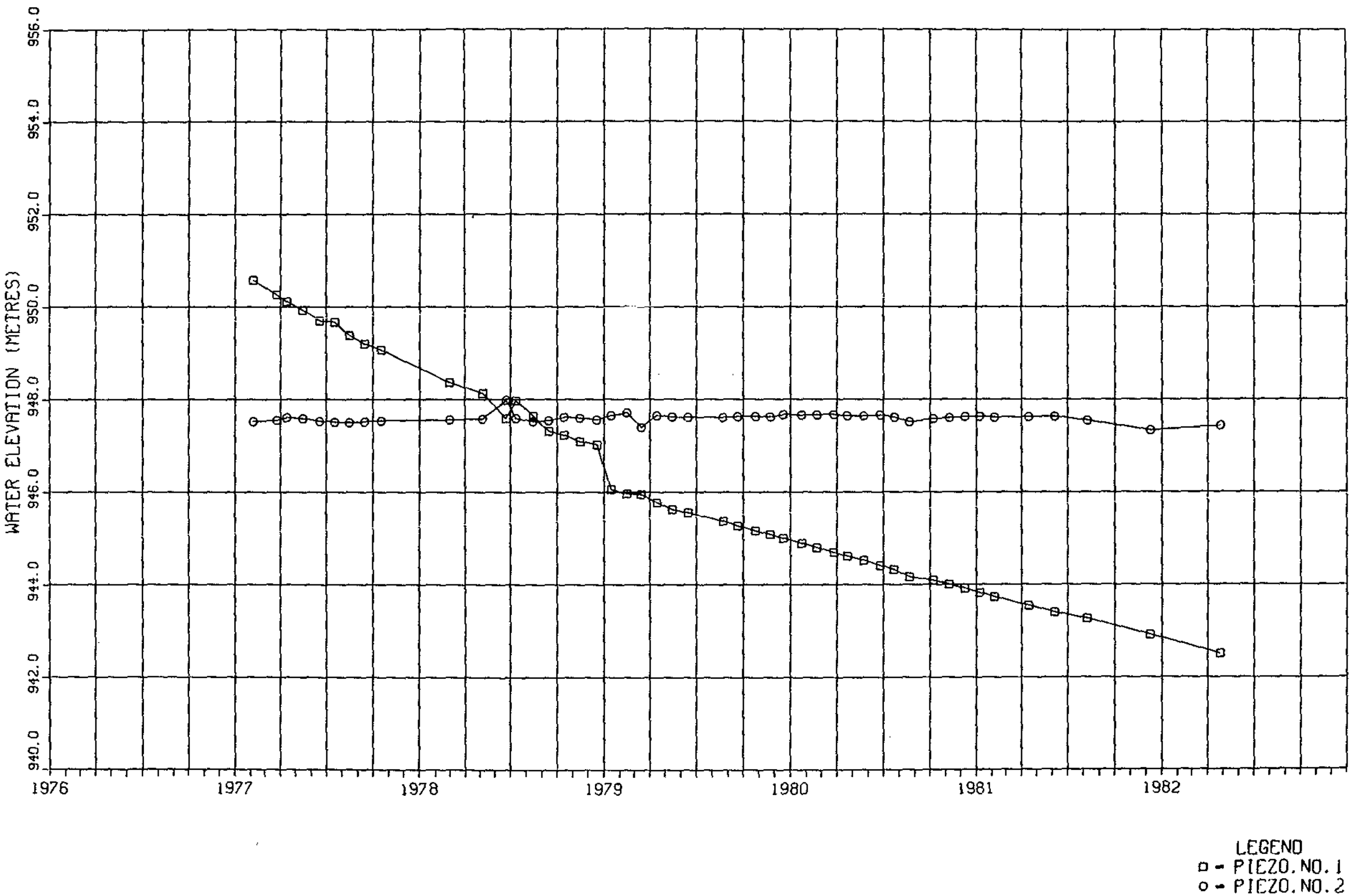


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-150

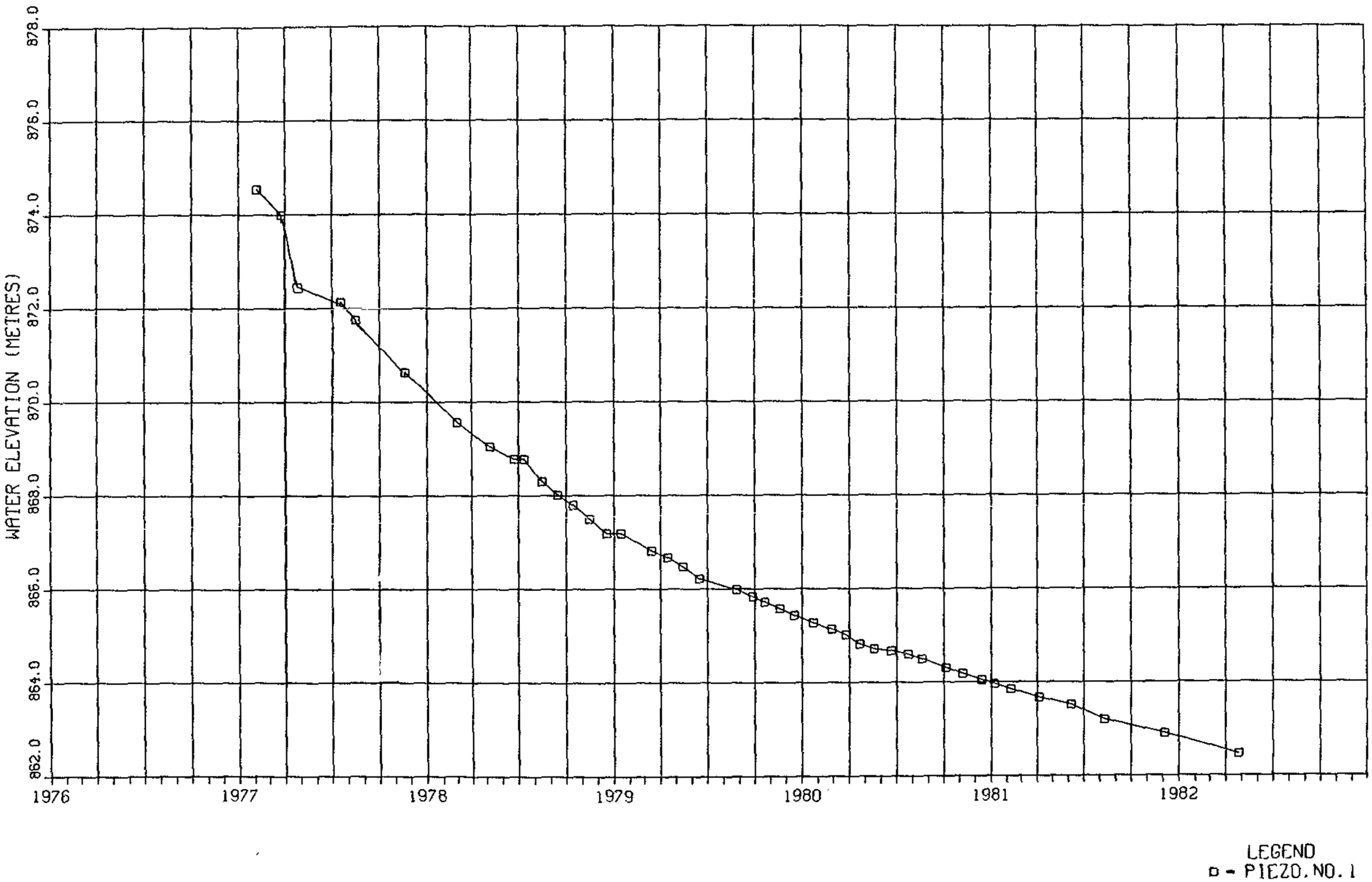


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-155

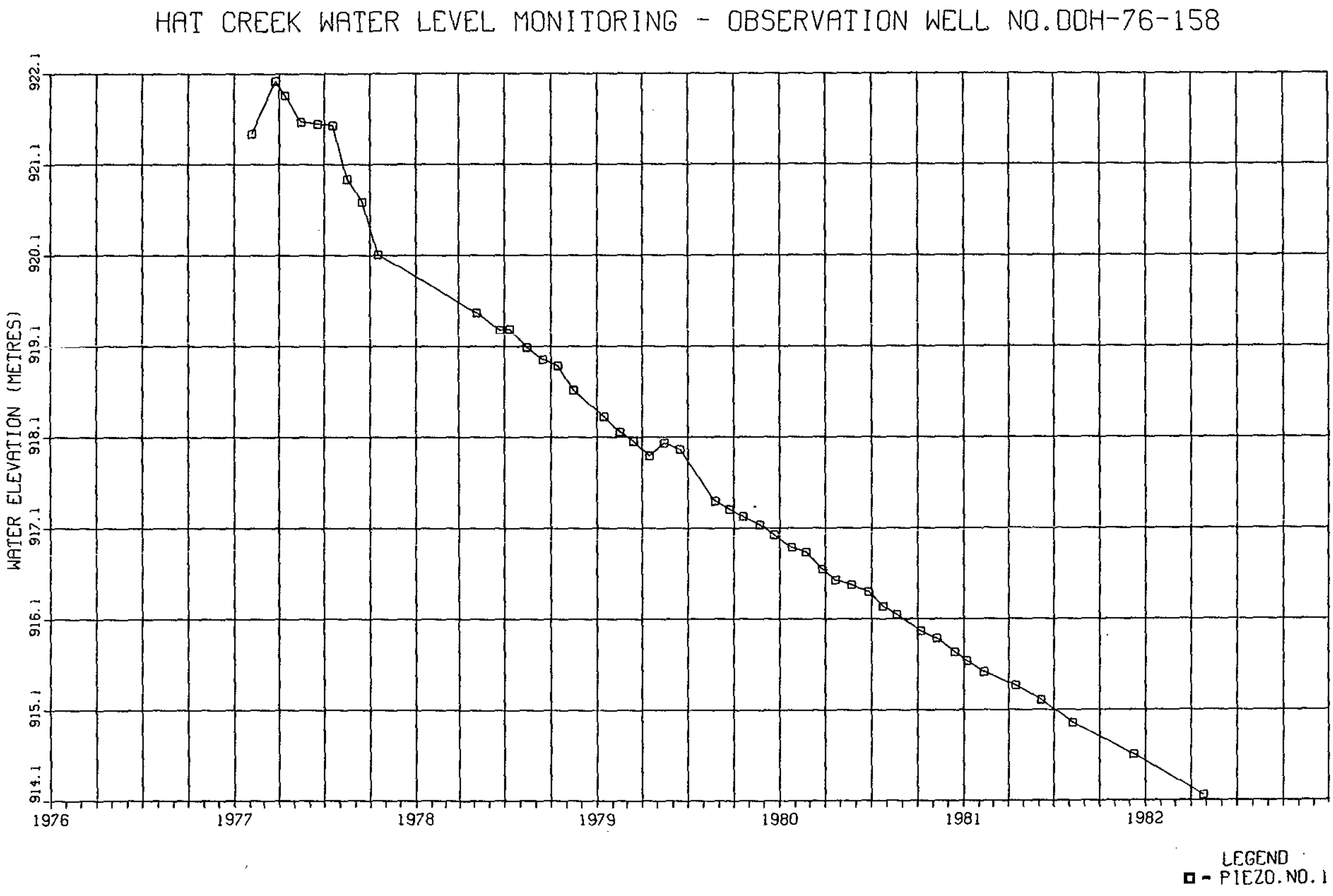


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-156

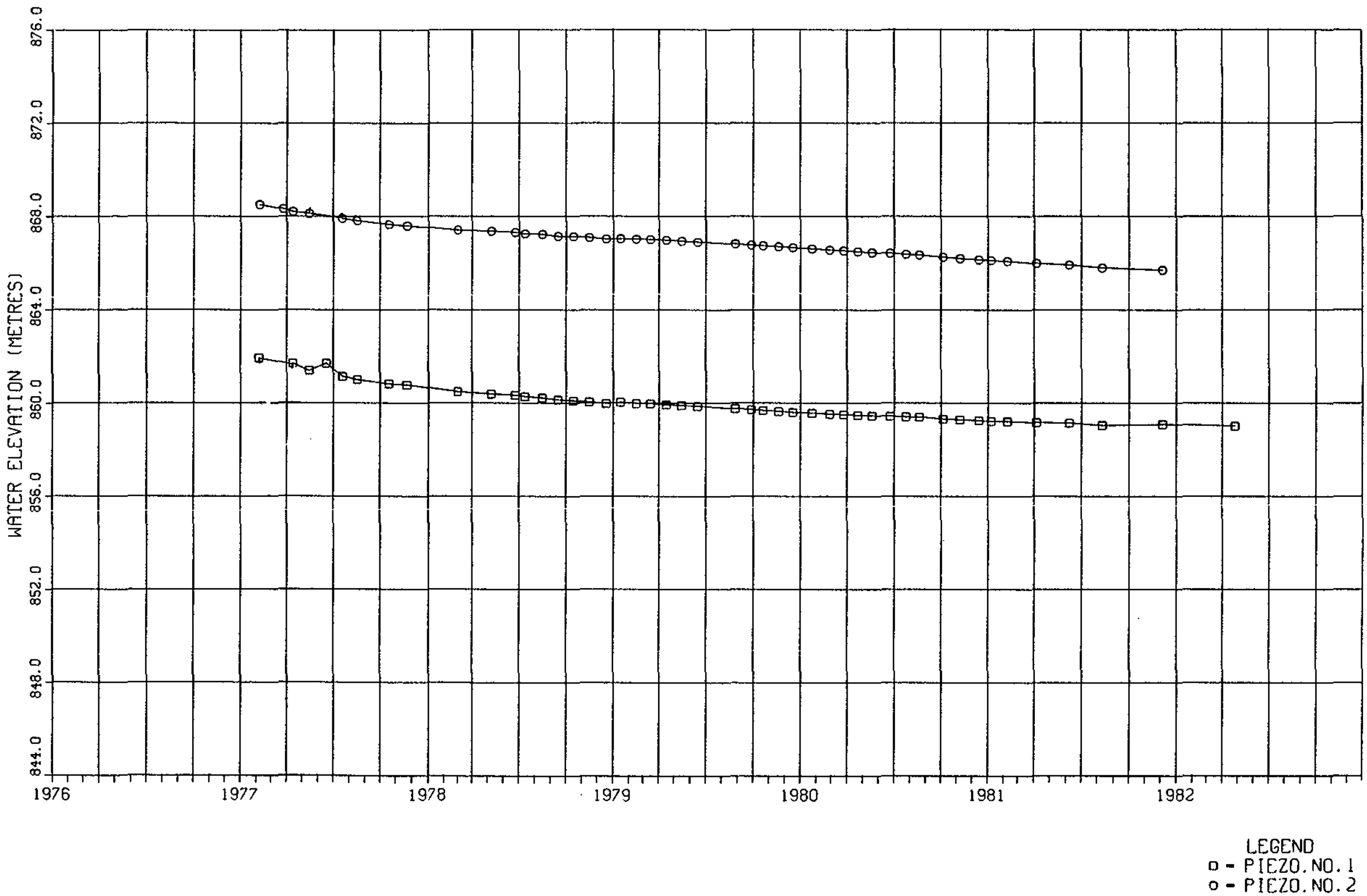


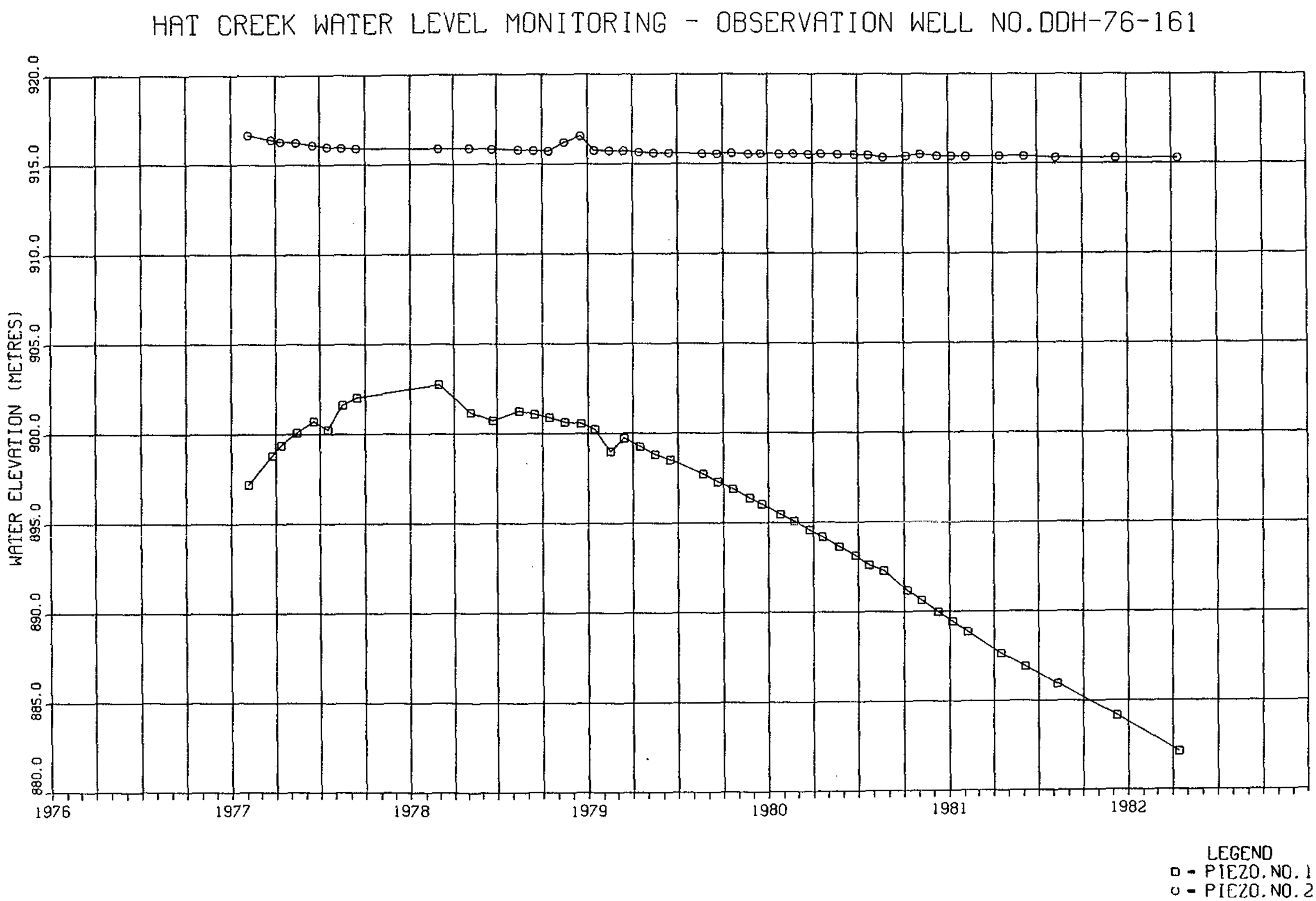
Golder Associates



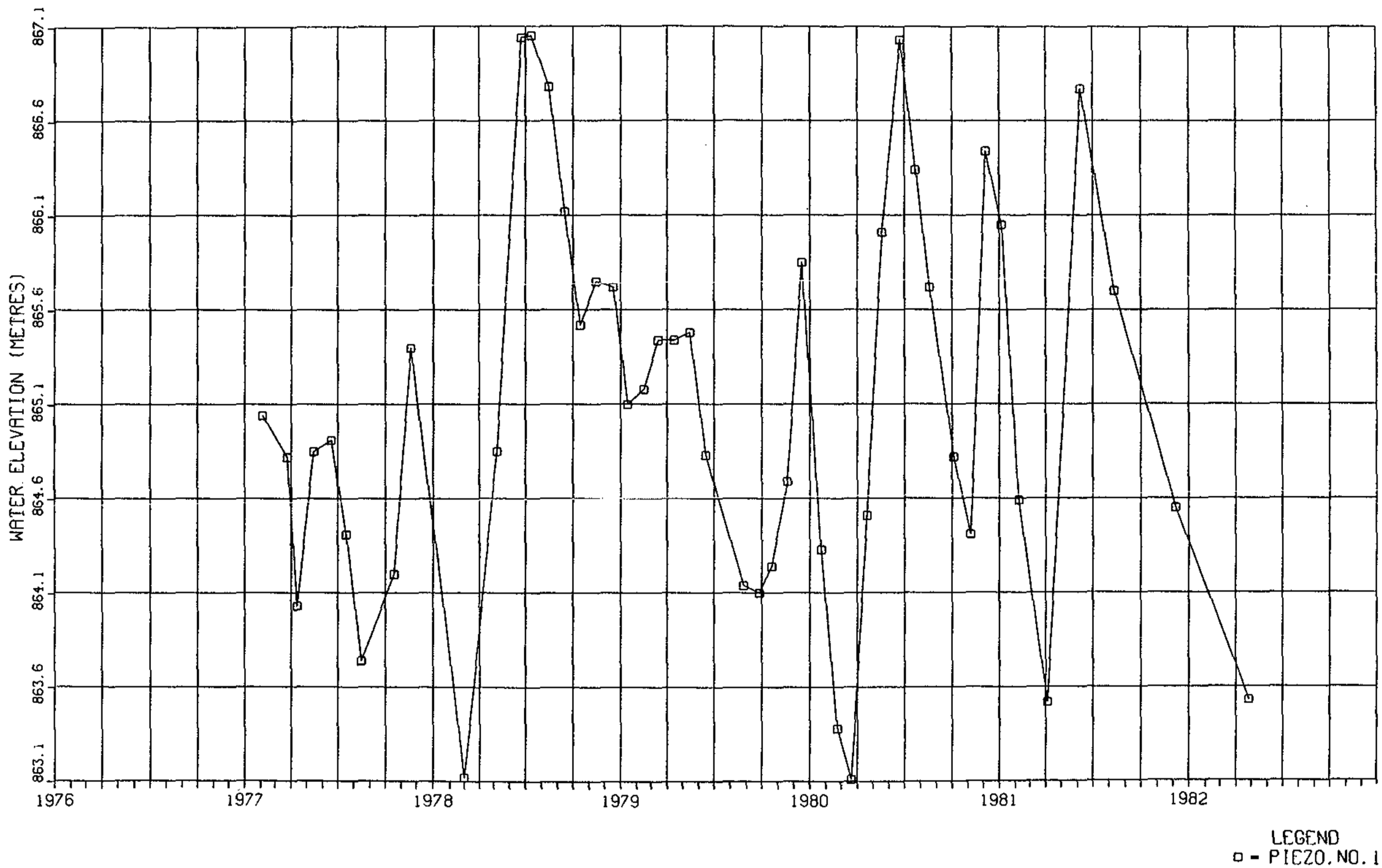
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-160



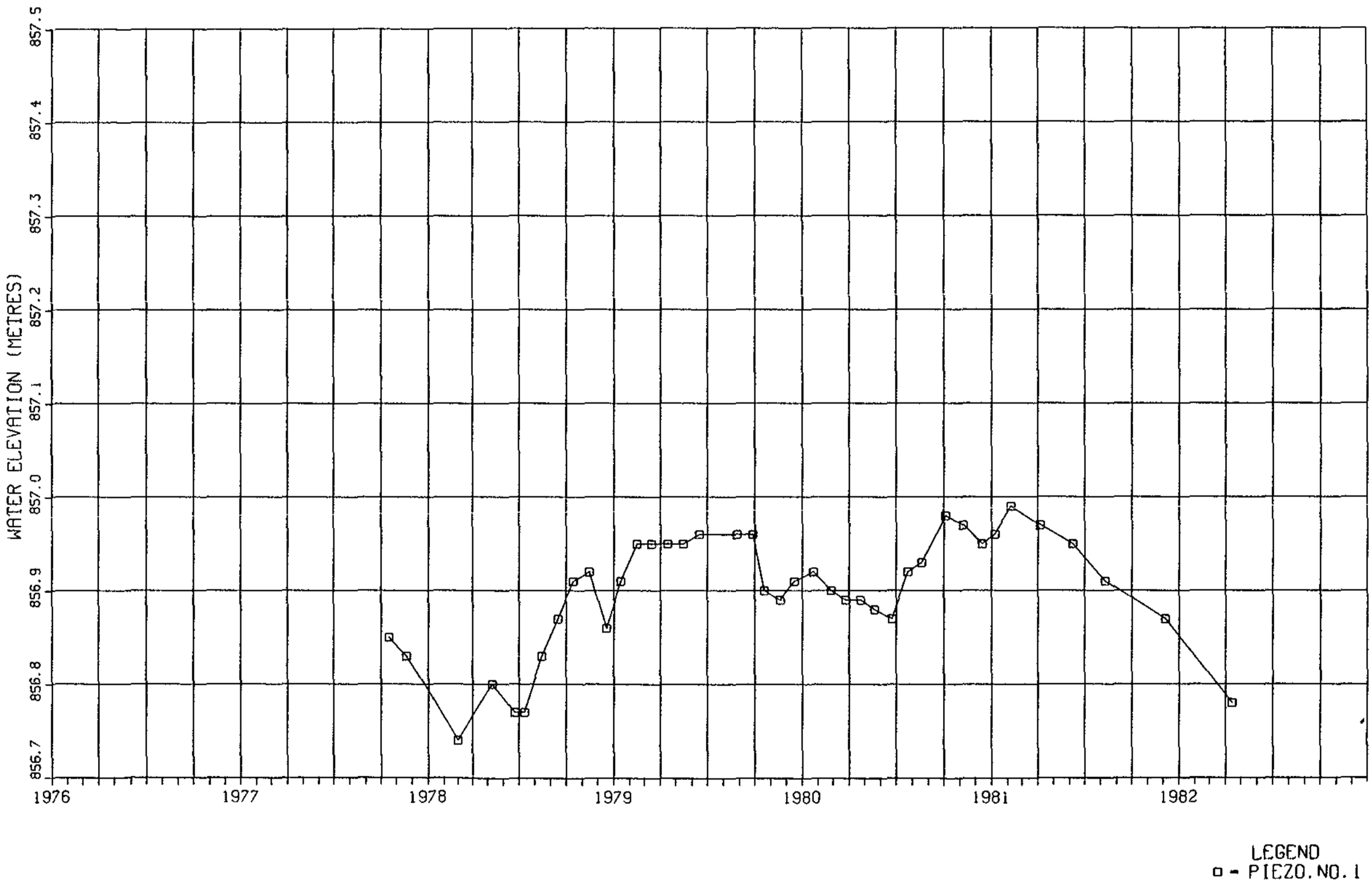

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-161
WATER ELEVATION (METRES)
920.0
915.0
910.0
905.0
900.0
895.0
890.0
885.0
880.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1
○ - PIEZO.NO.2

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-168

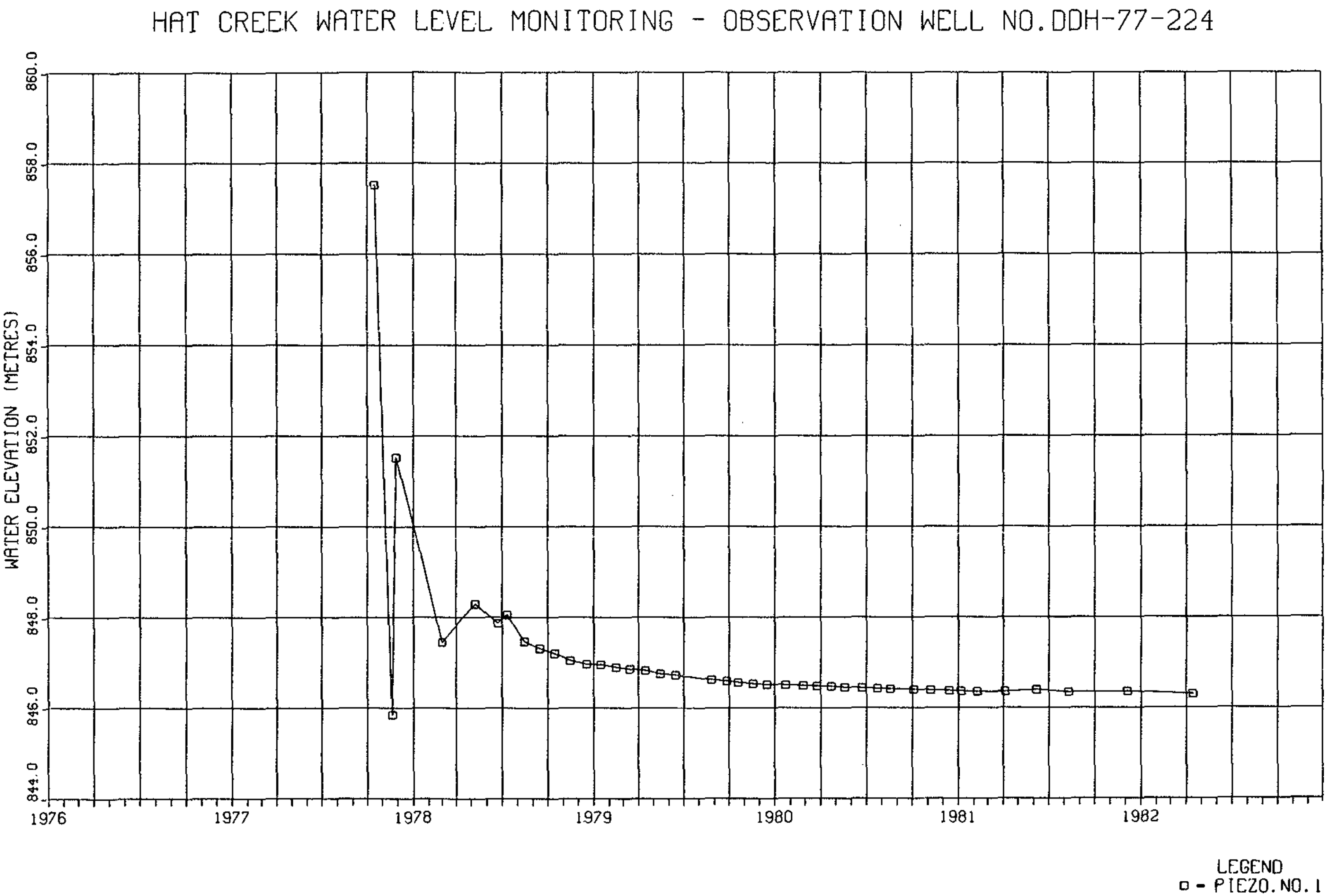


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-220

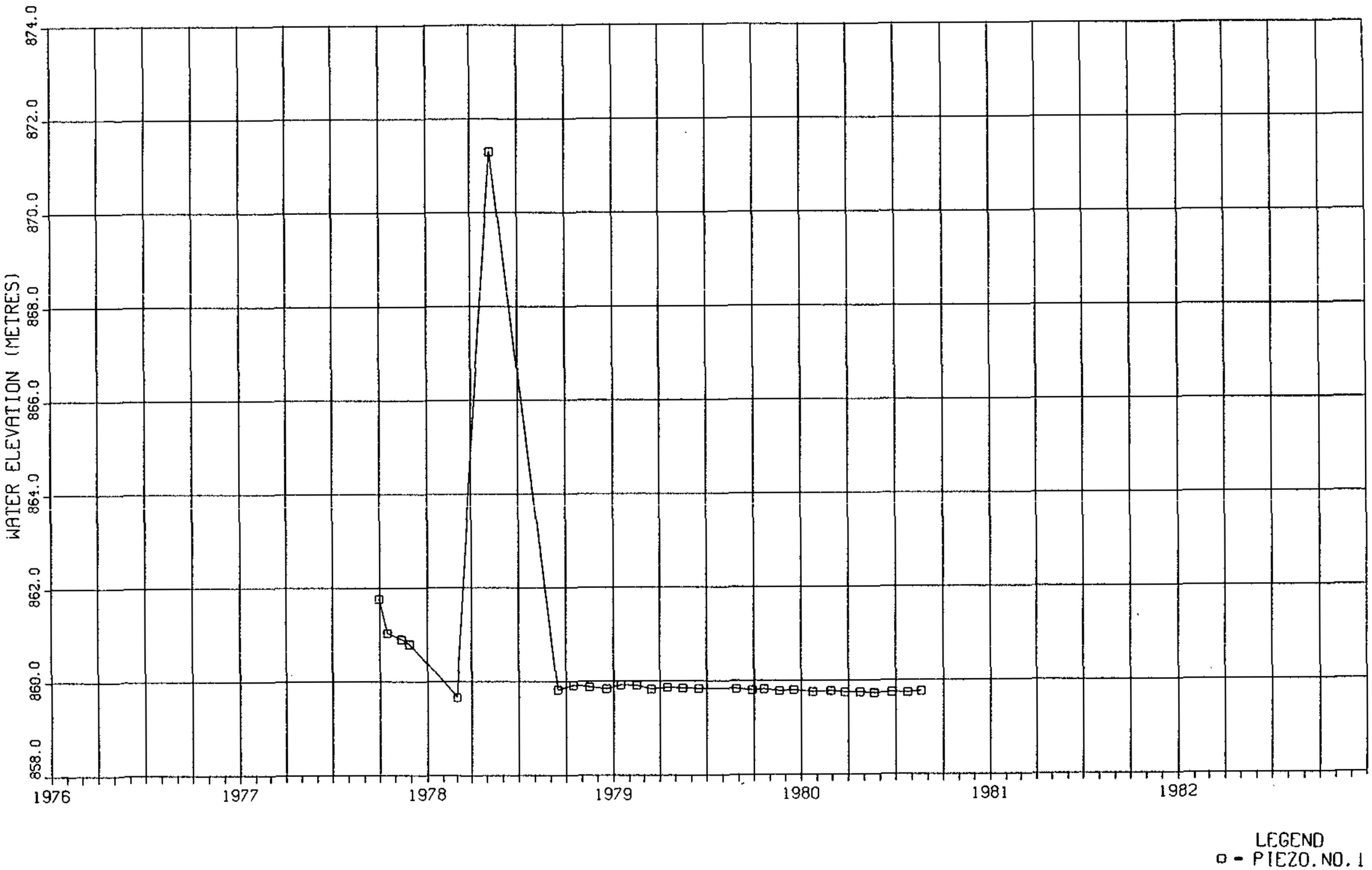


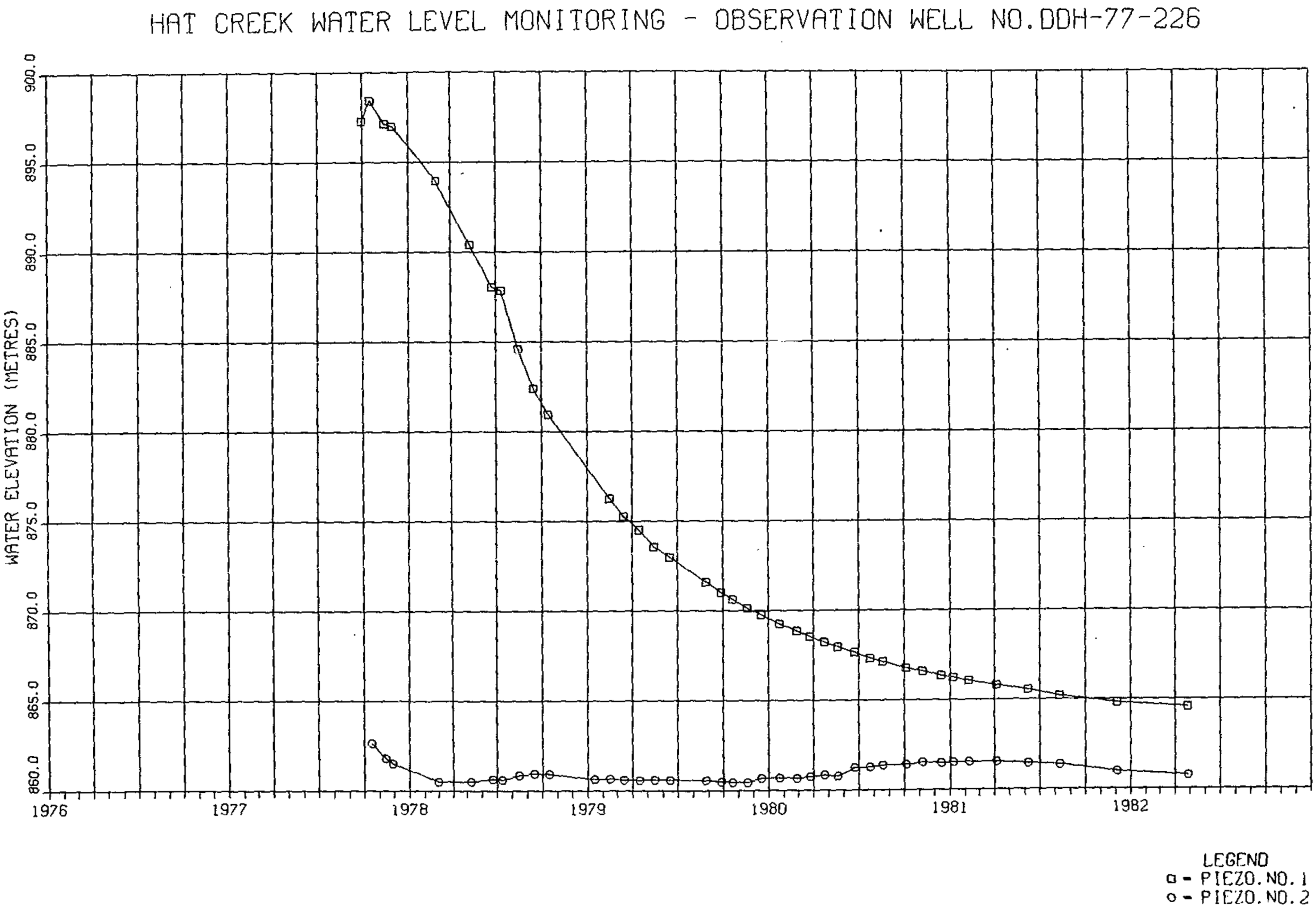
Golder Associates



Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-225

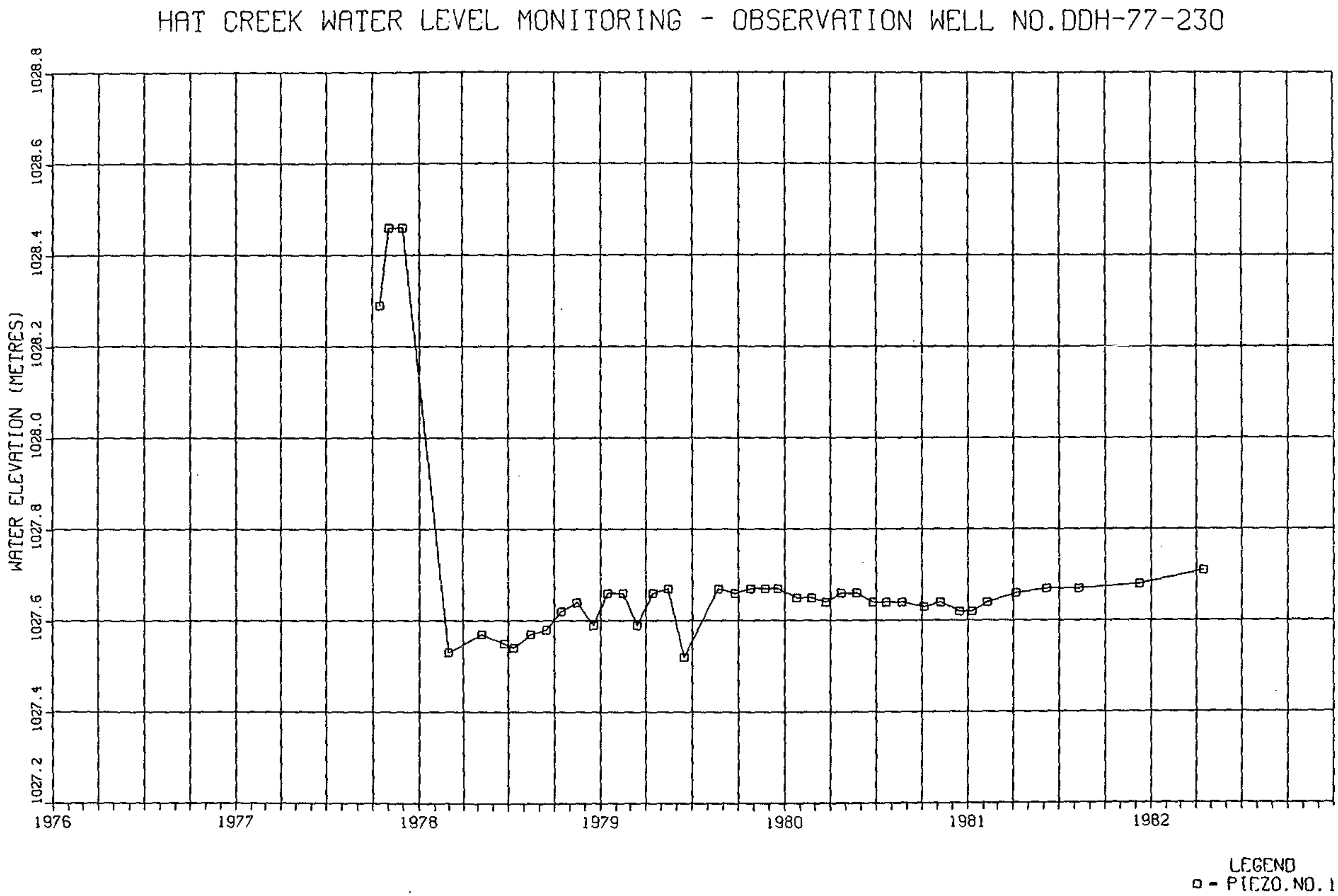




Golder Associates

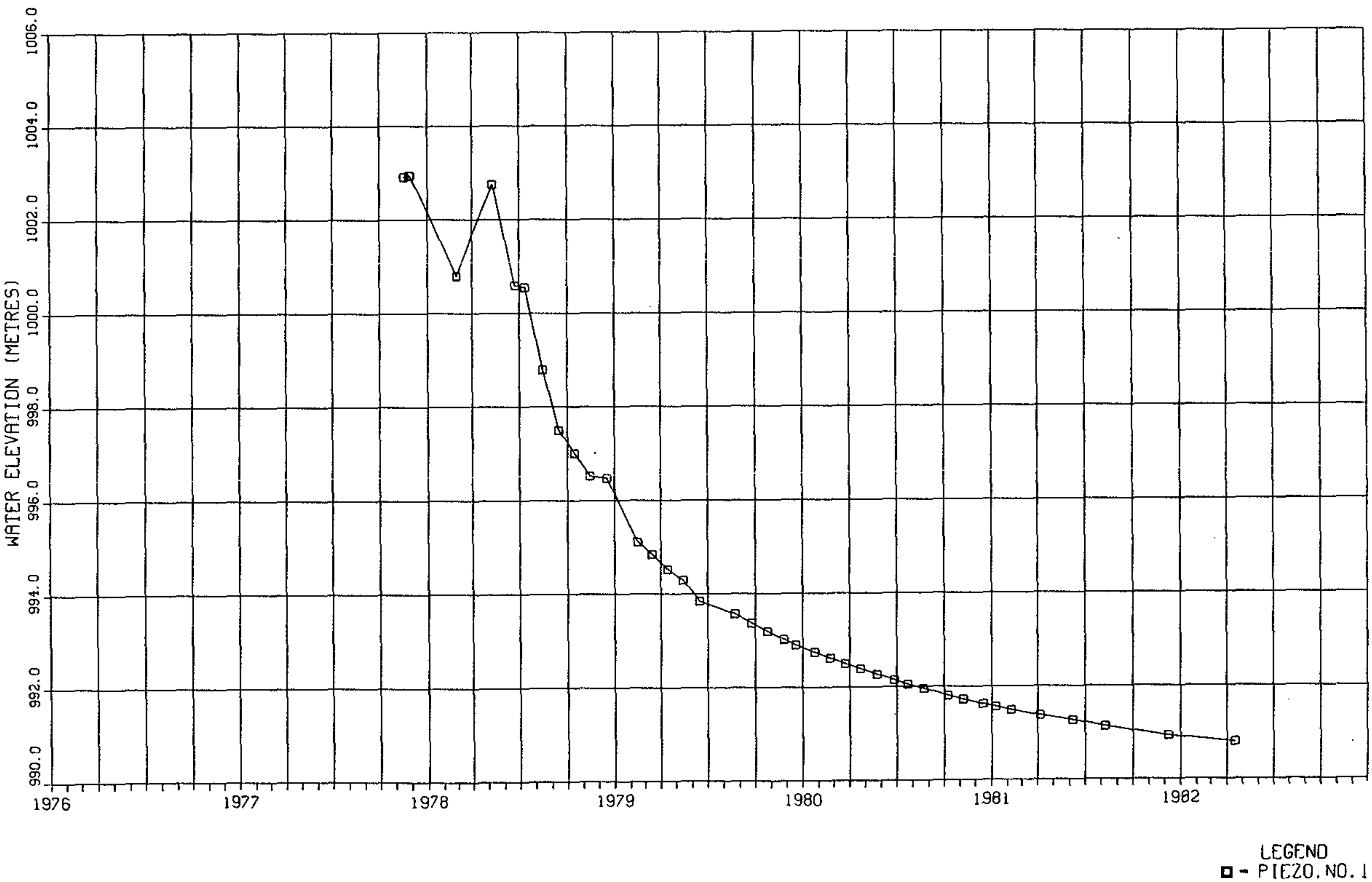
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-227

Golder Associates



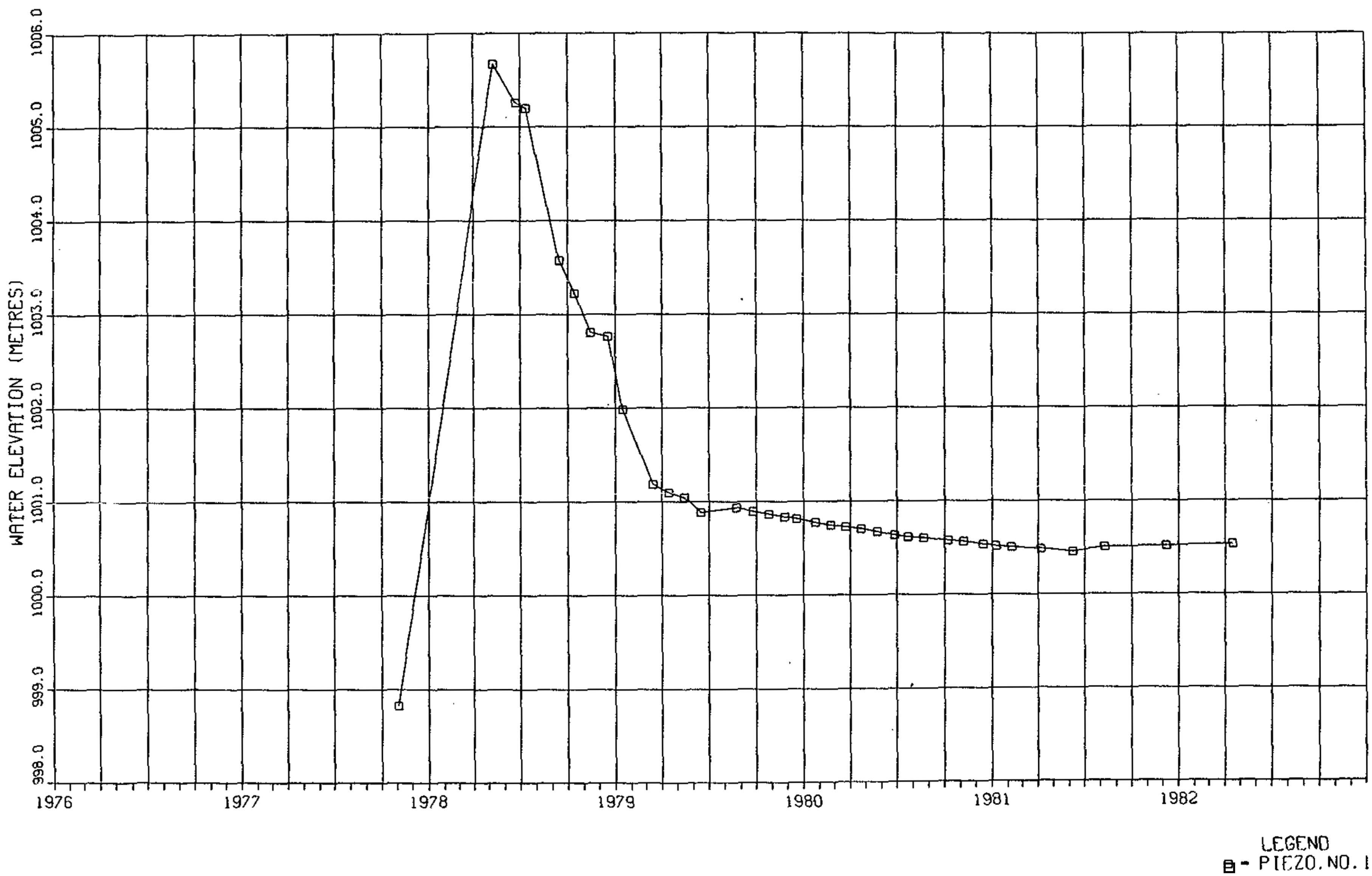
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-230
WATER ELEVATION (METRES)
1028.8
1028.6
1028.4
1028.2
1028.0
1027.8
1027.6
1027.4
1027.2
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-236

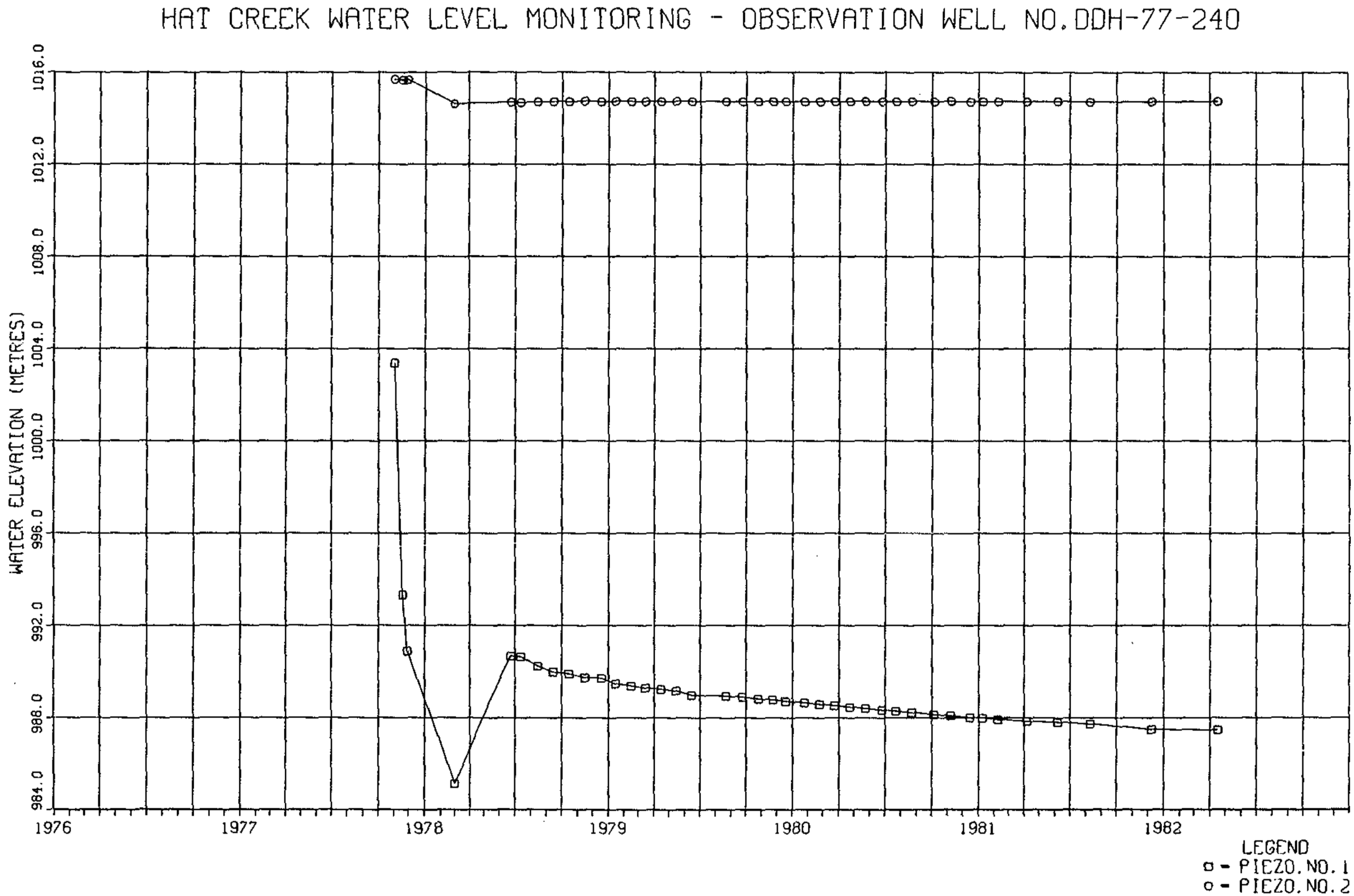


Golder Associates

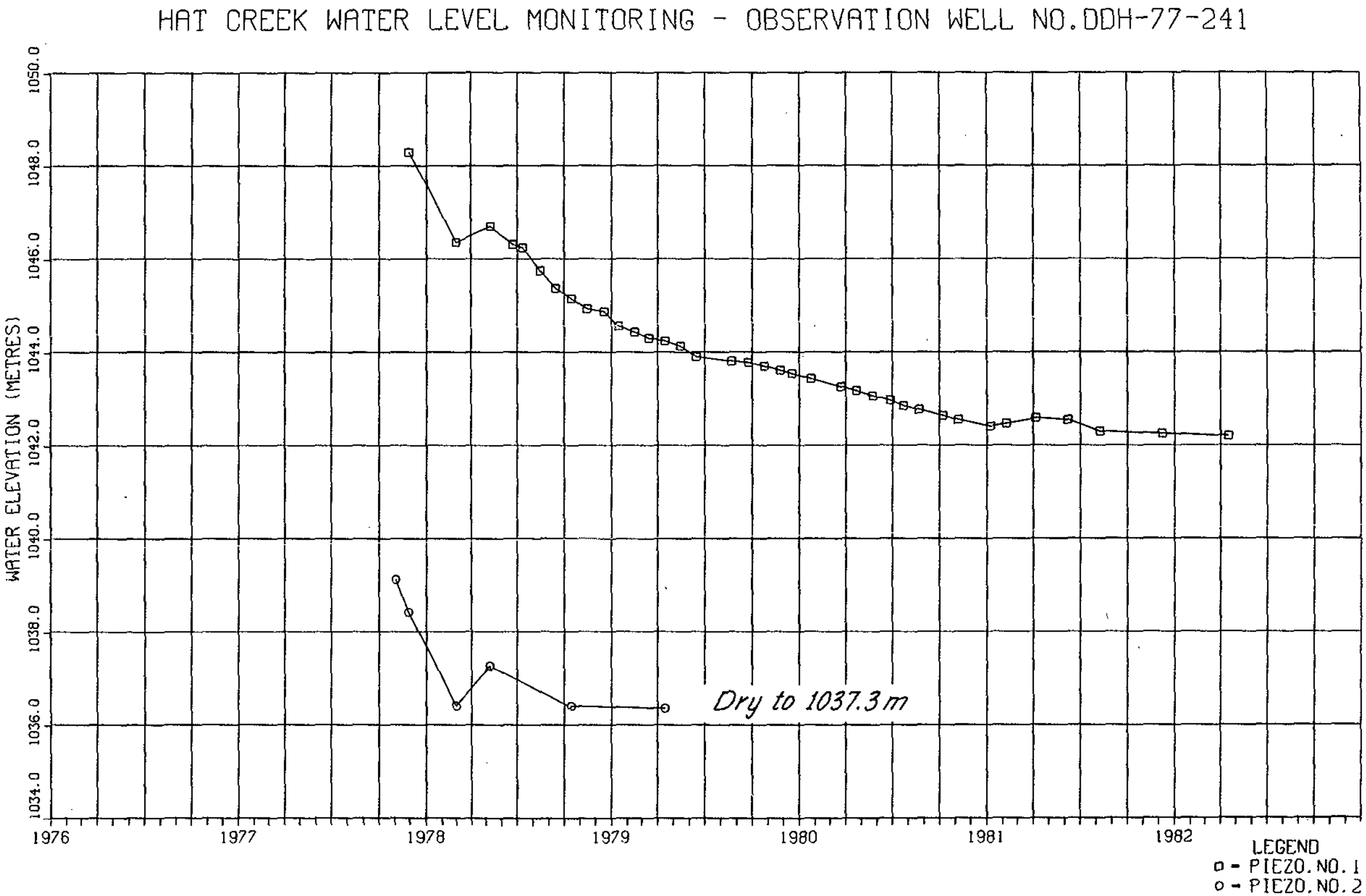
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-239



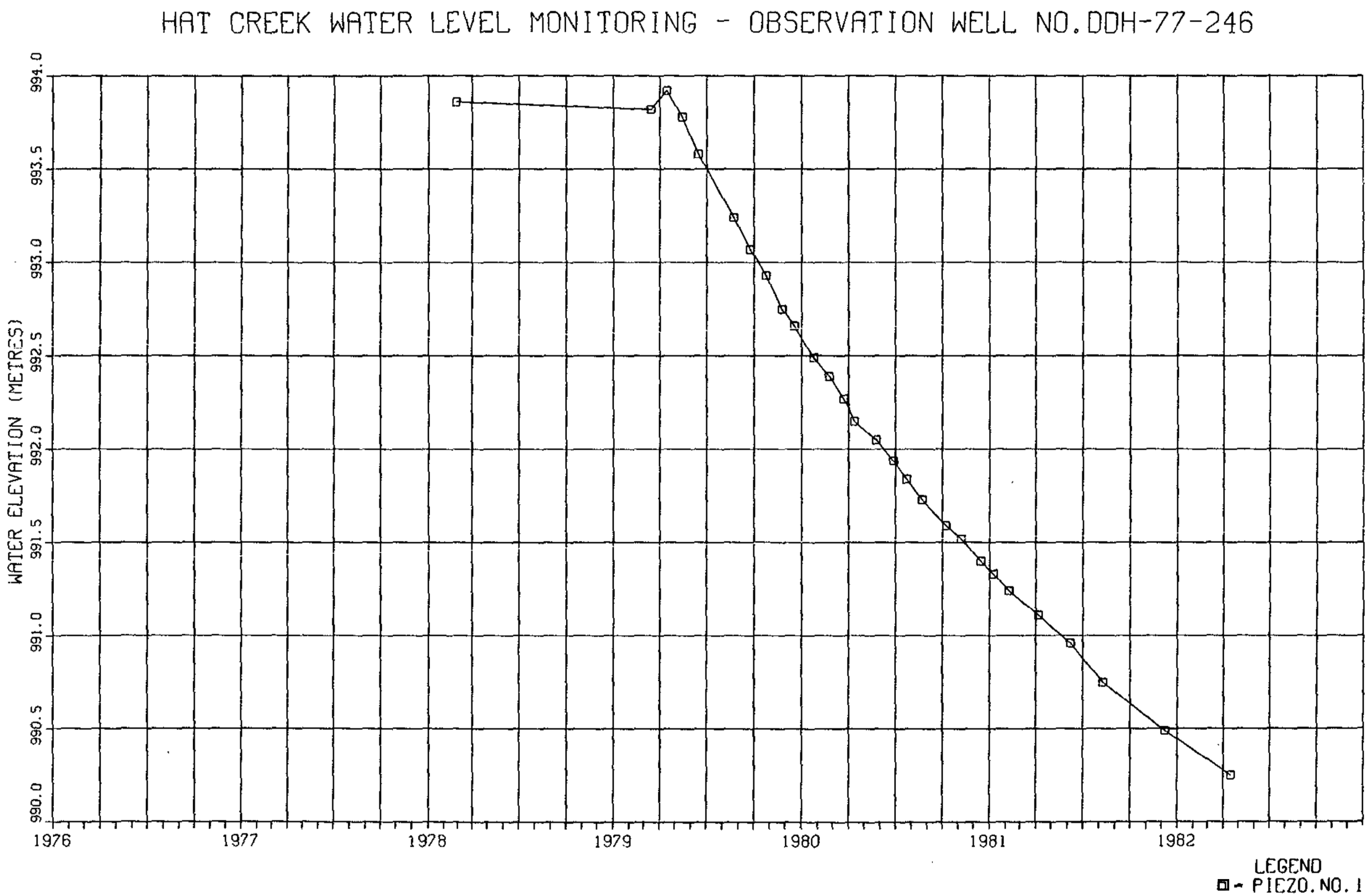
Golder Associates



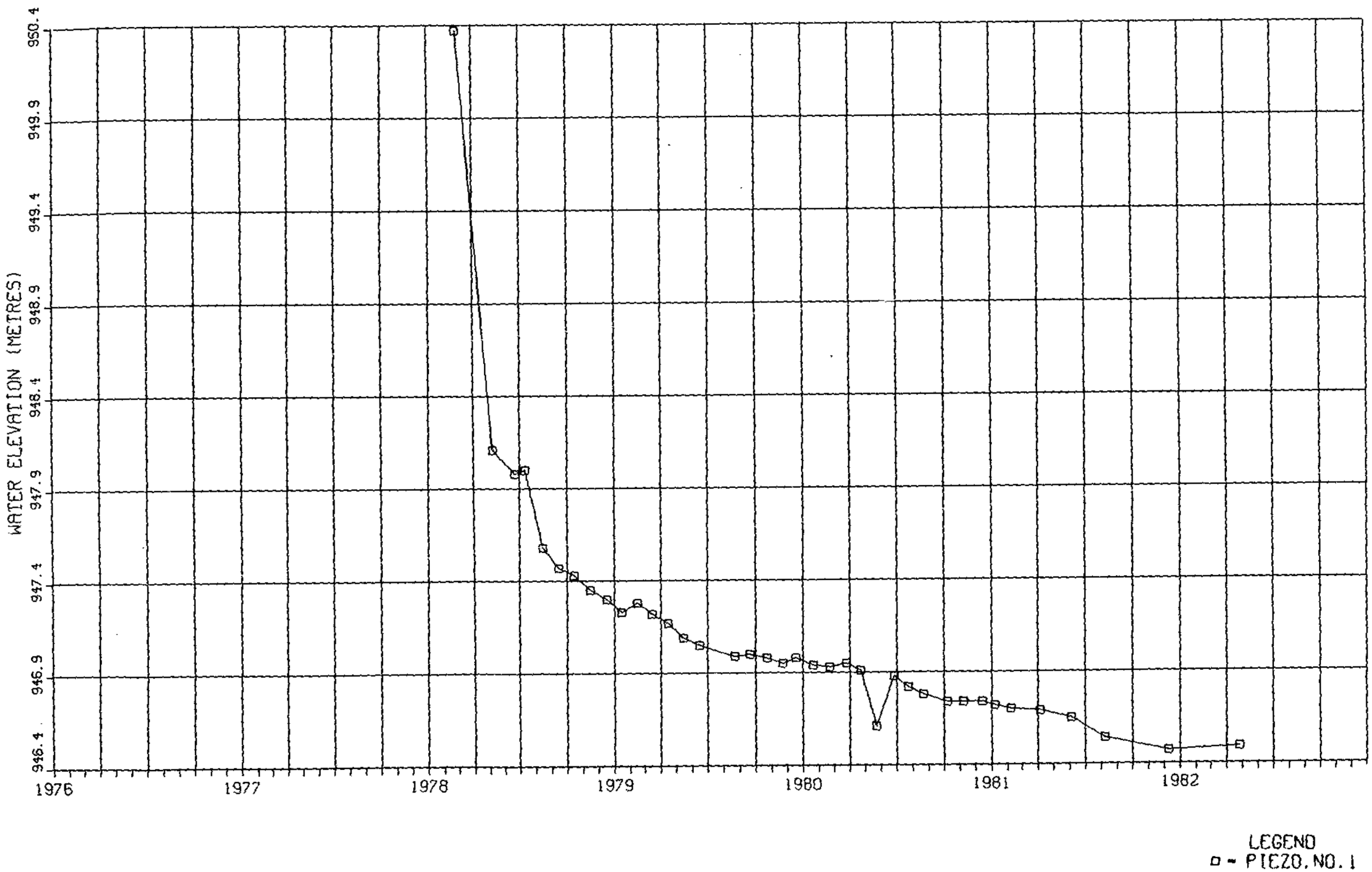
Golder Associates



HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-241
WATER ELEVATION (METRES)
1050.0
1048.0
1046.0
1044.0
1042.0
1040.0
1038.0
1036.0
1034.0
1976
1977
1978
1979
1980
1981
1982
Dry to 1037.3 m
LEGEND
□ - PIEZO. NO. 1
○ - PIEZO. NO. 2

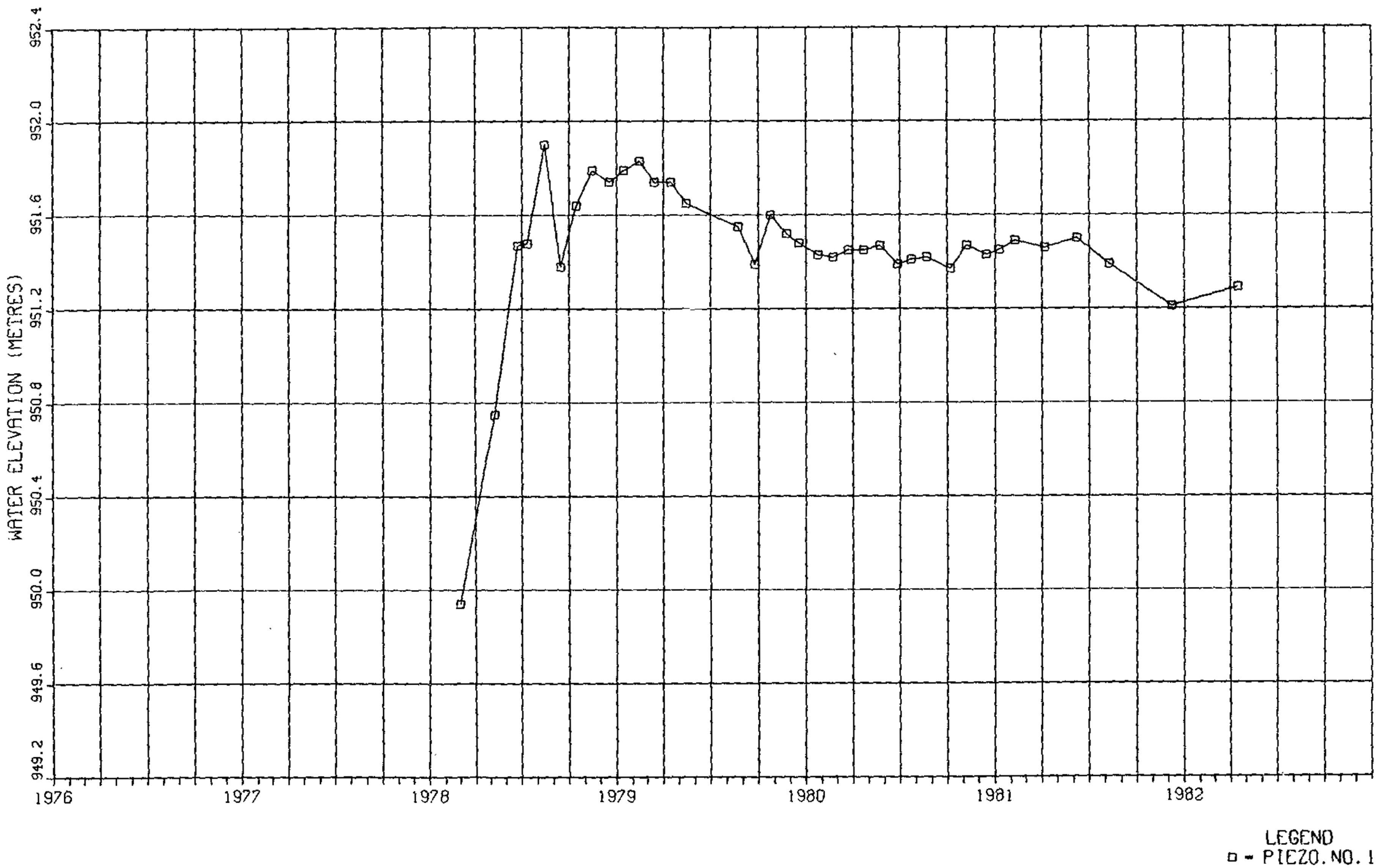

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-246
WATER ELEVATION (METRES)
994.0
993.5
993.0
992.5
992.0
991.5
991.0
990.5
990.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-251

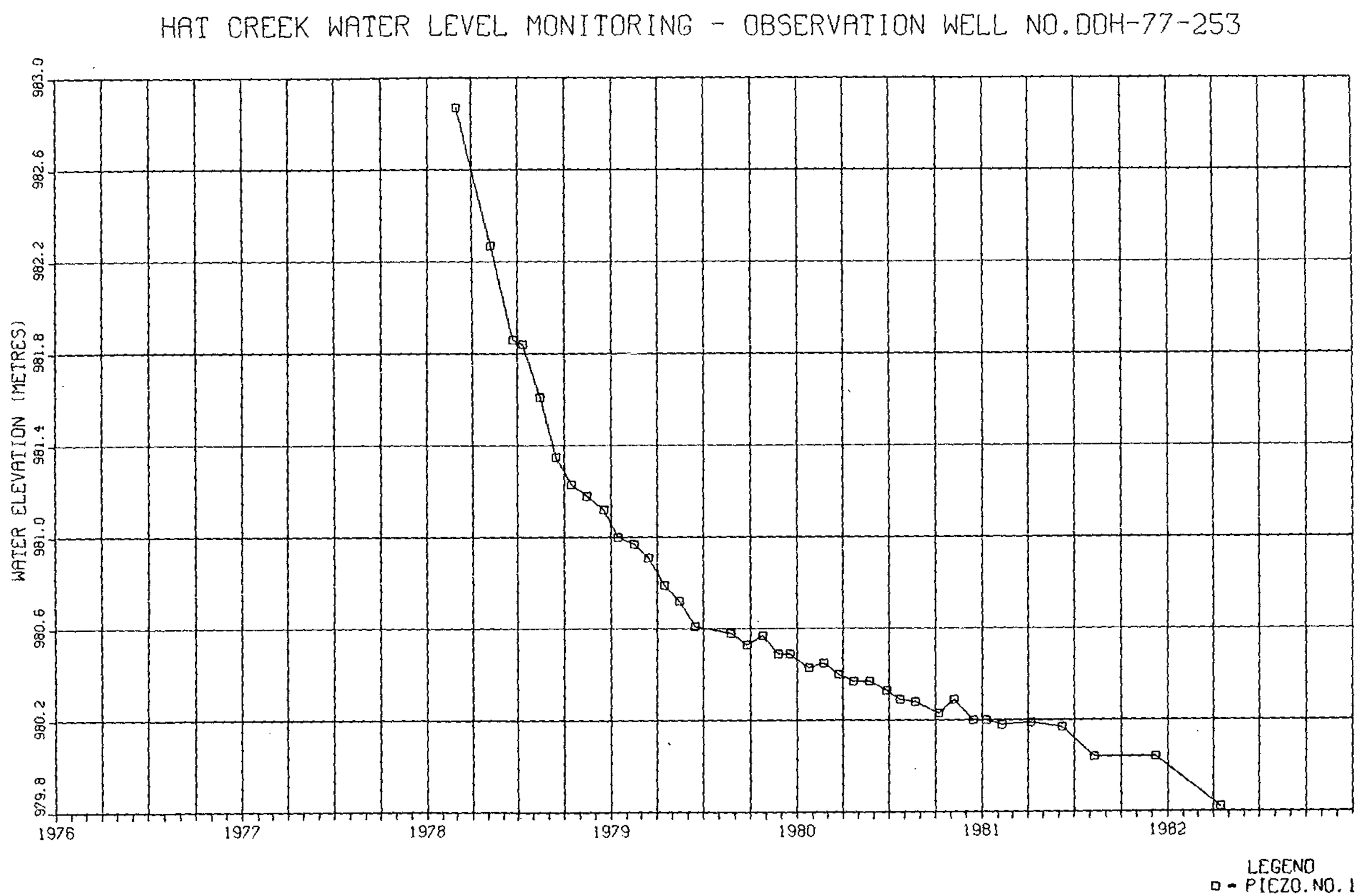


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-252



Golder Associates

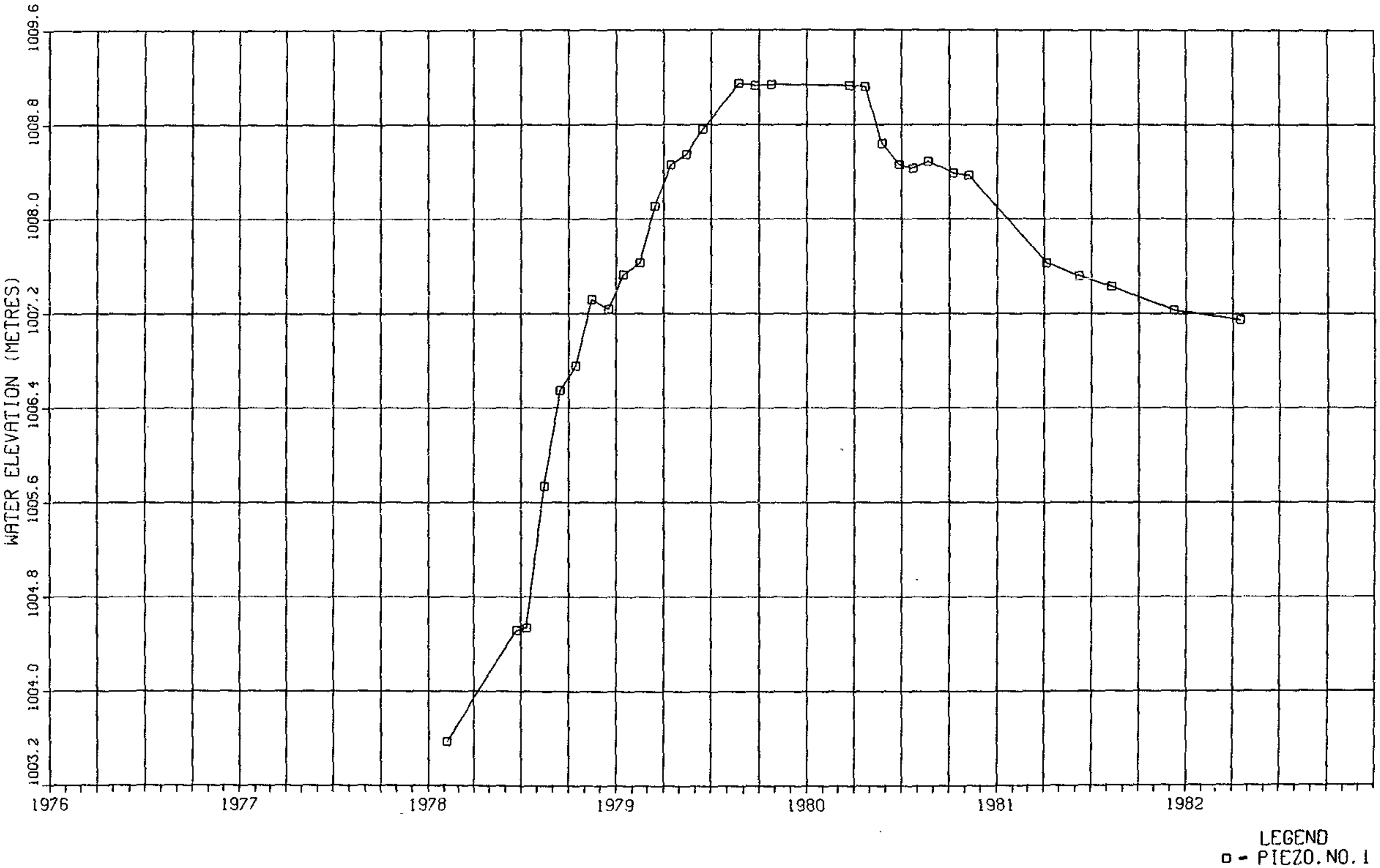


Golder Associates

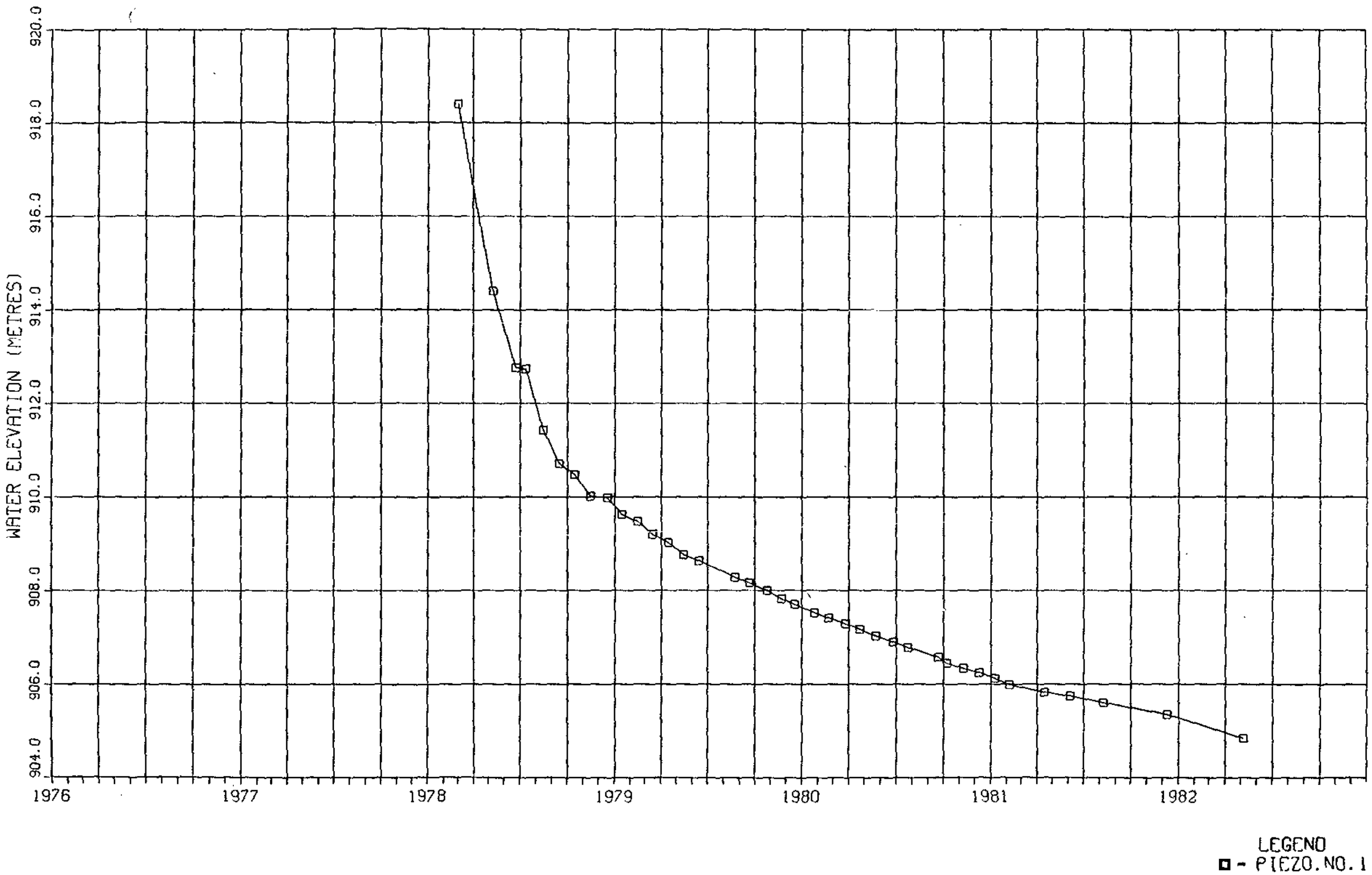
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-255

Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-256



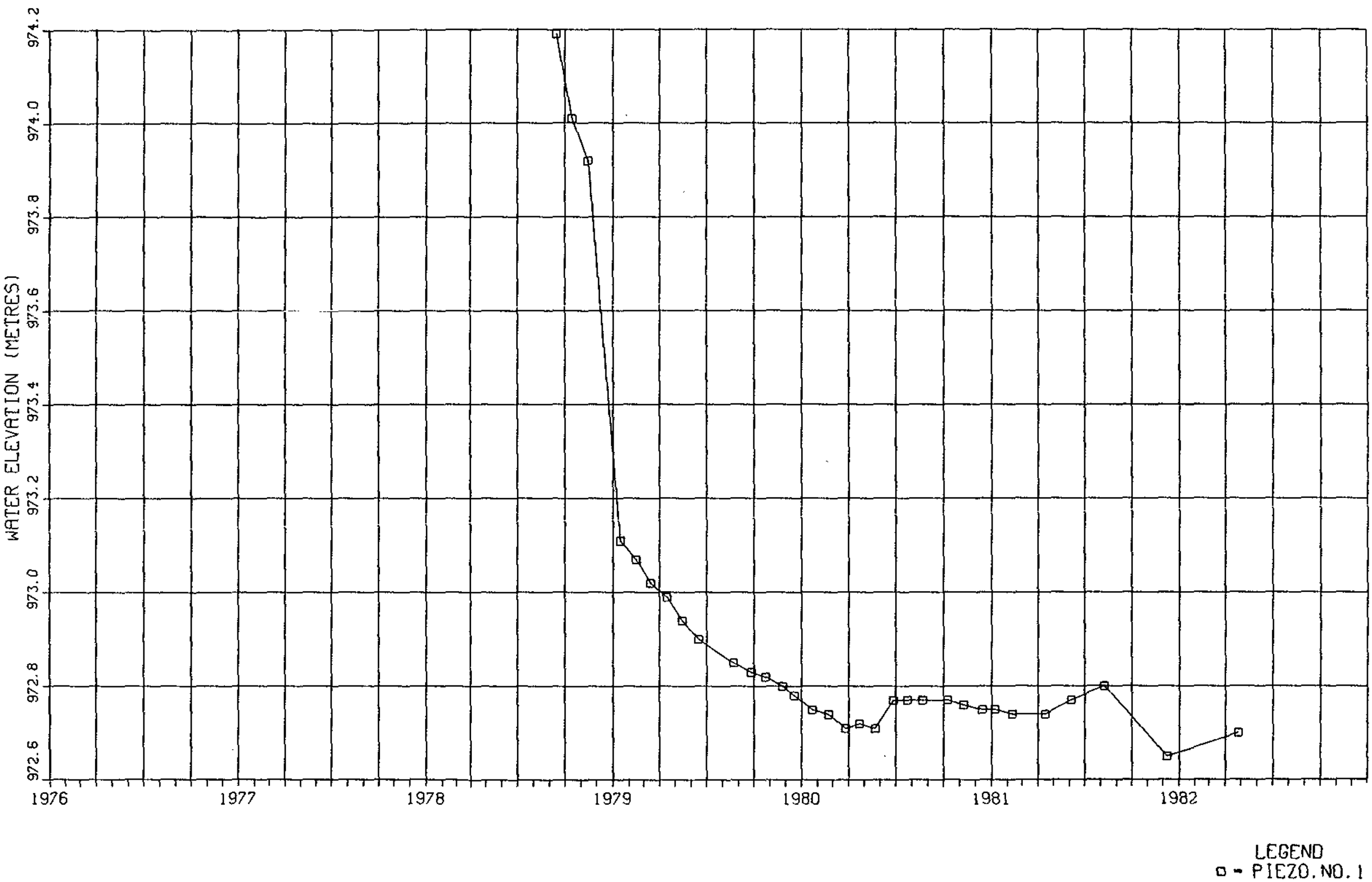
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-261



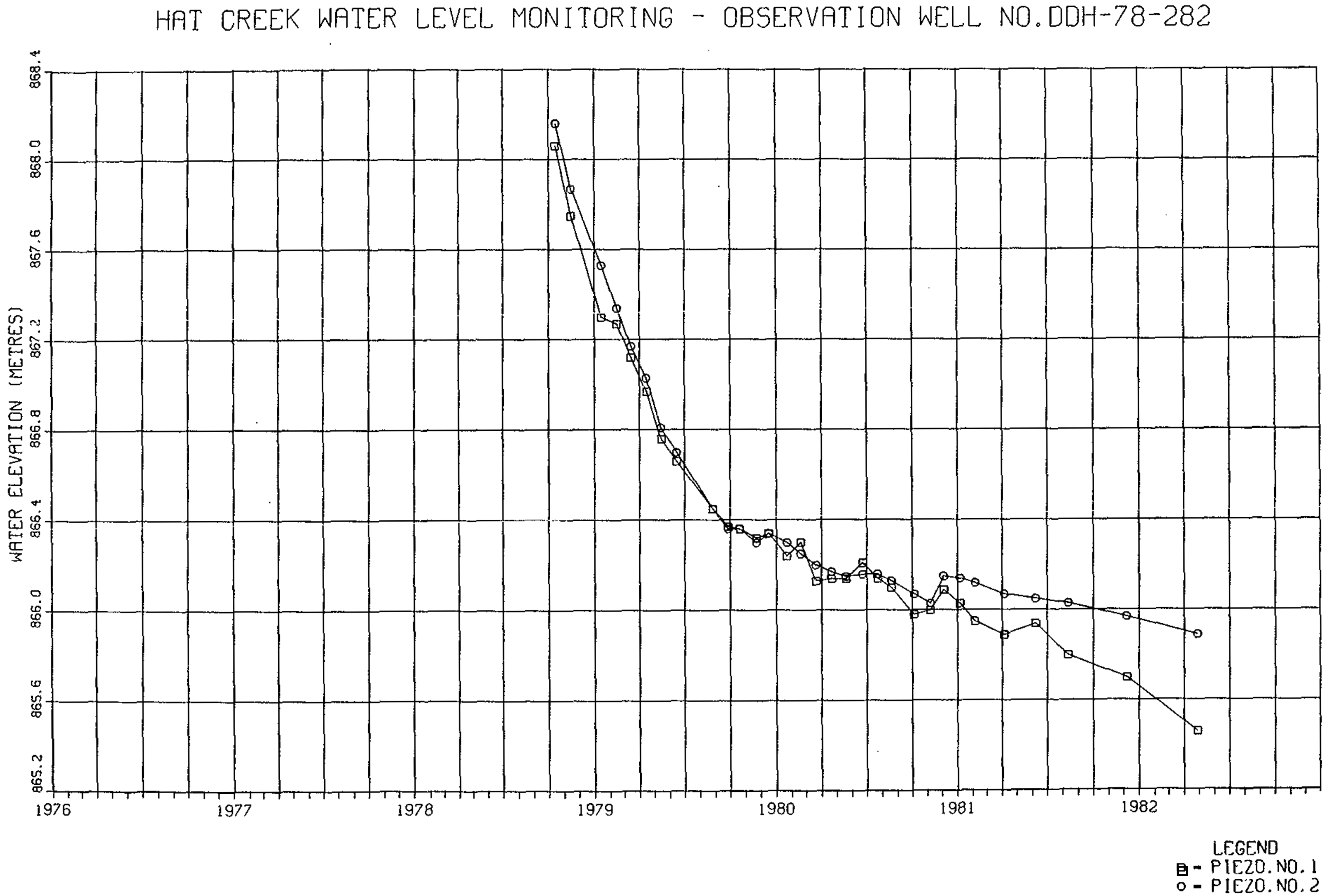
Golder Associates

Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-281



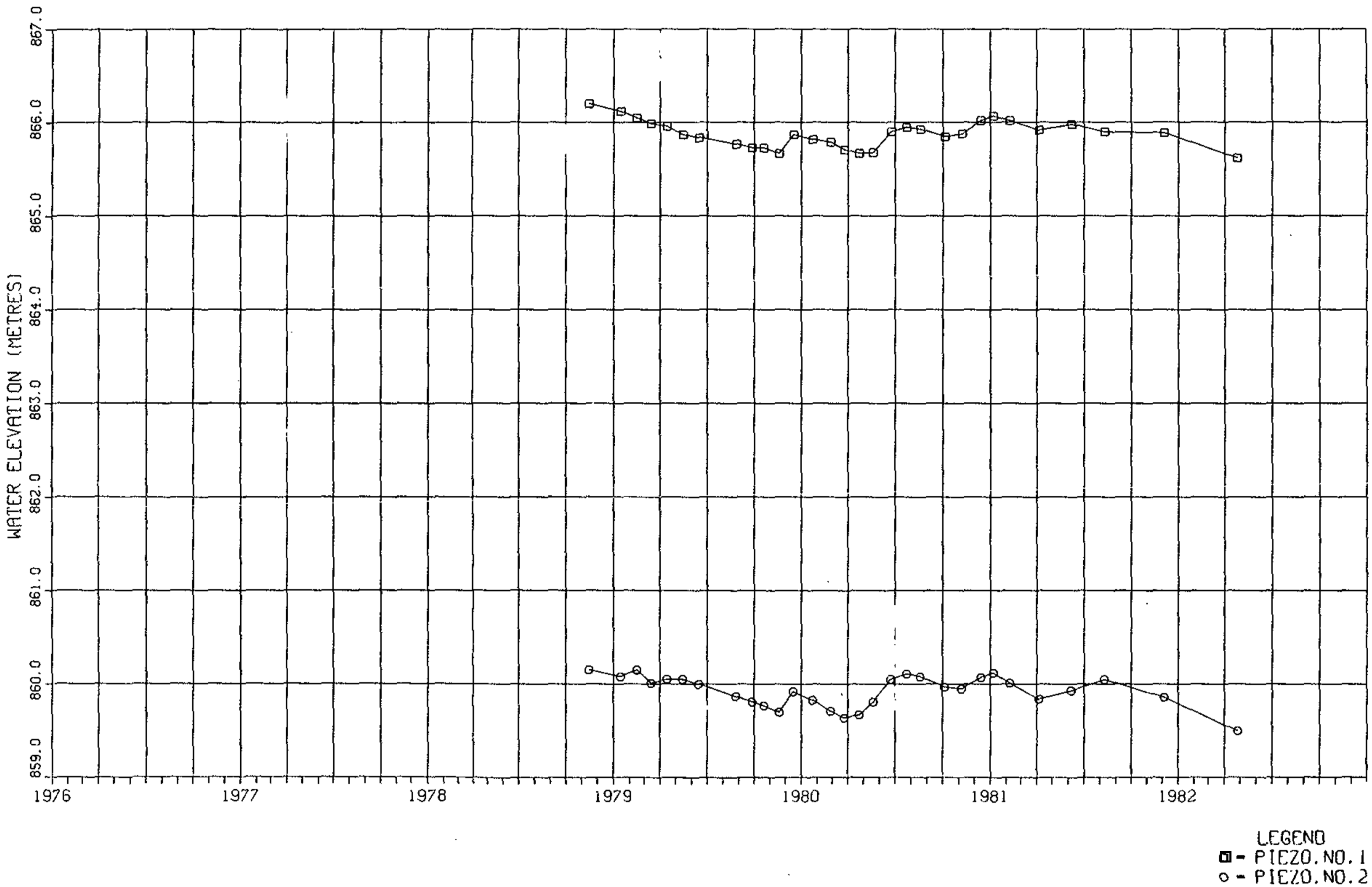
Golder Associates



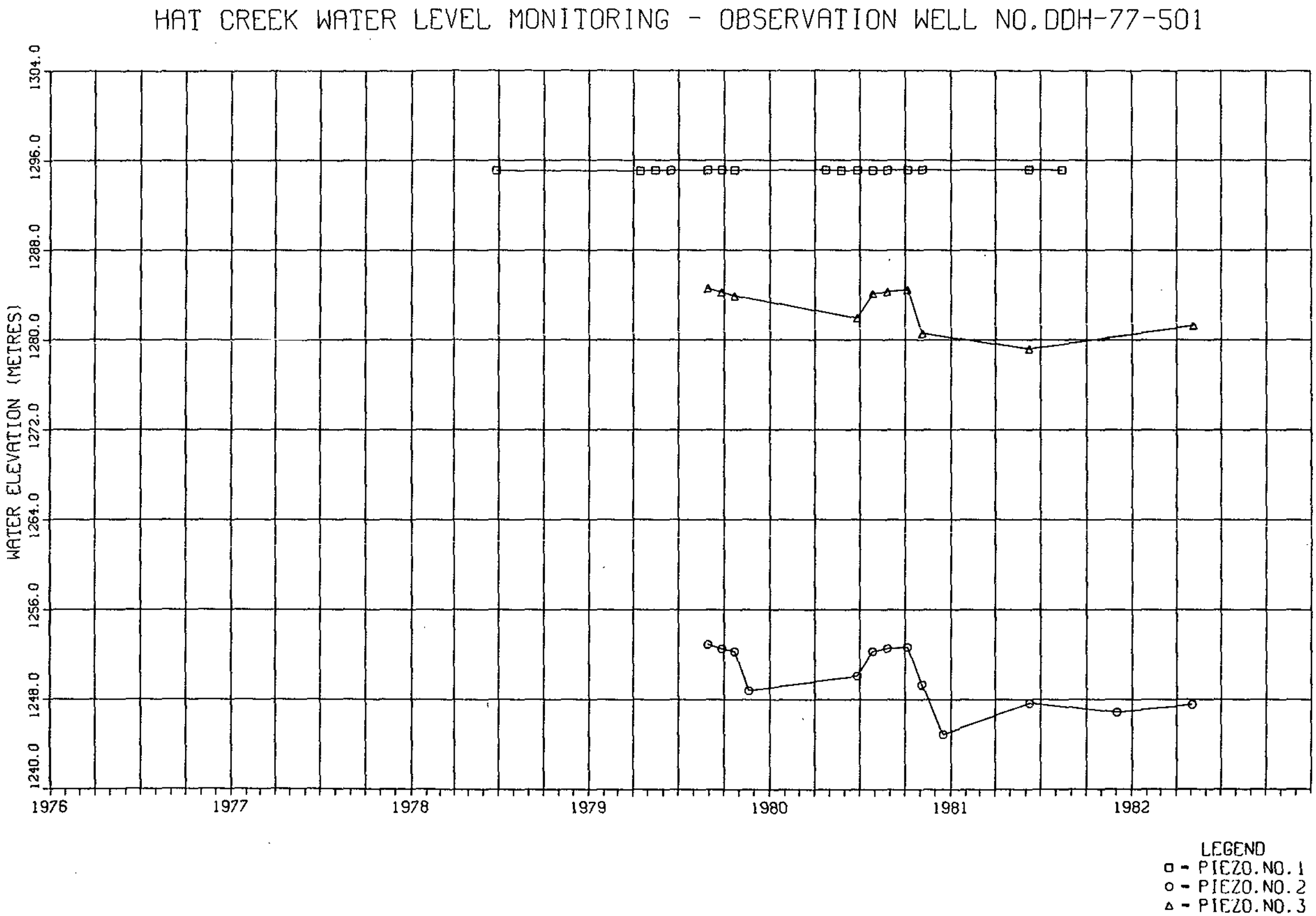
Golder Associates

Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-290

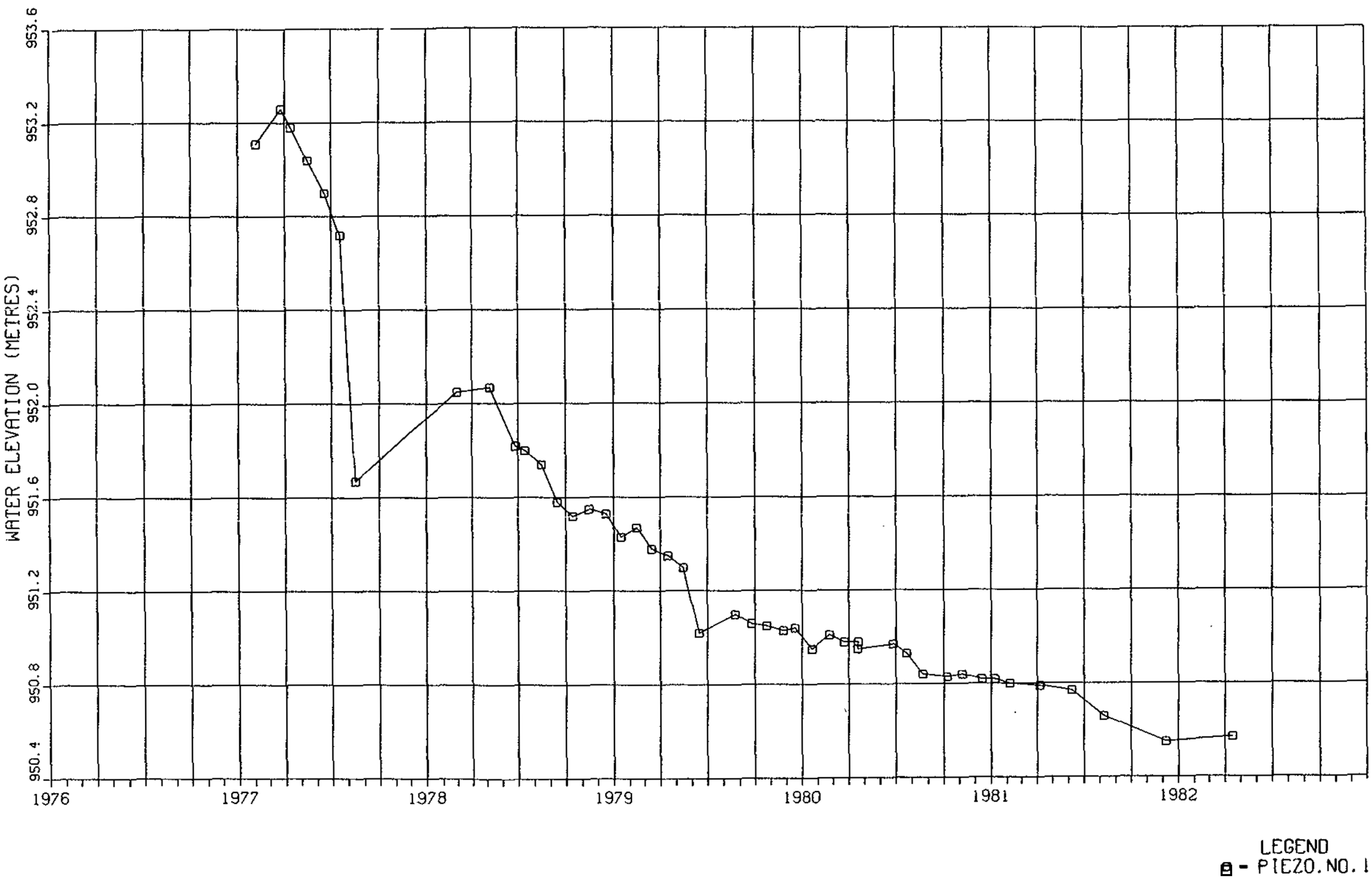


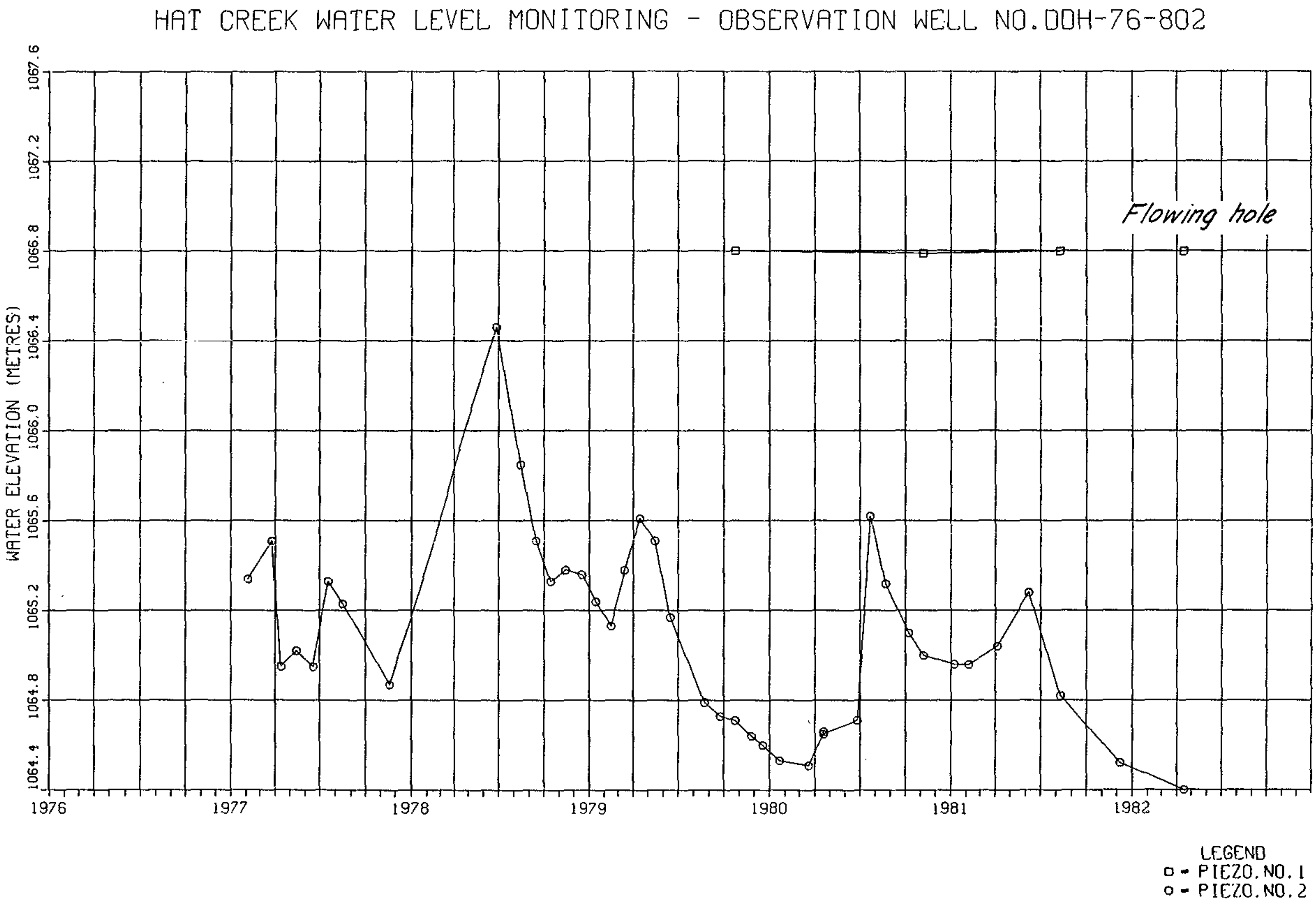
Golder Associates



Golder Associates

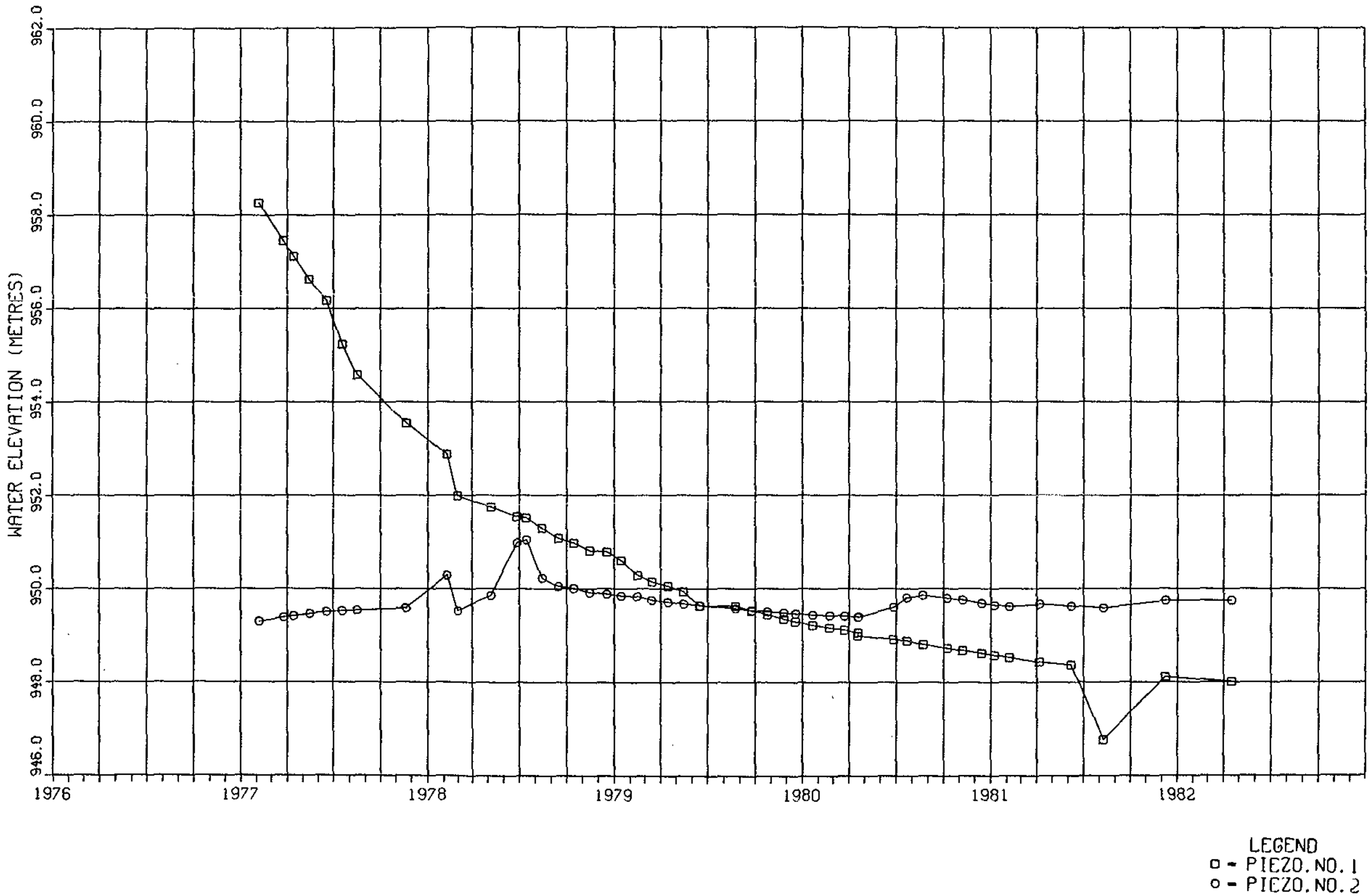
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-801



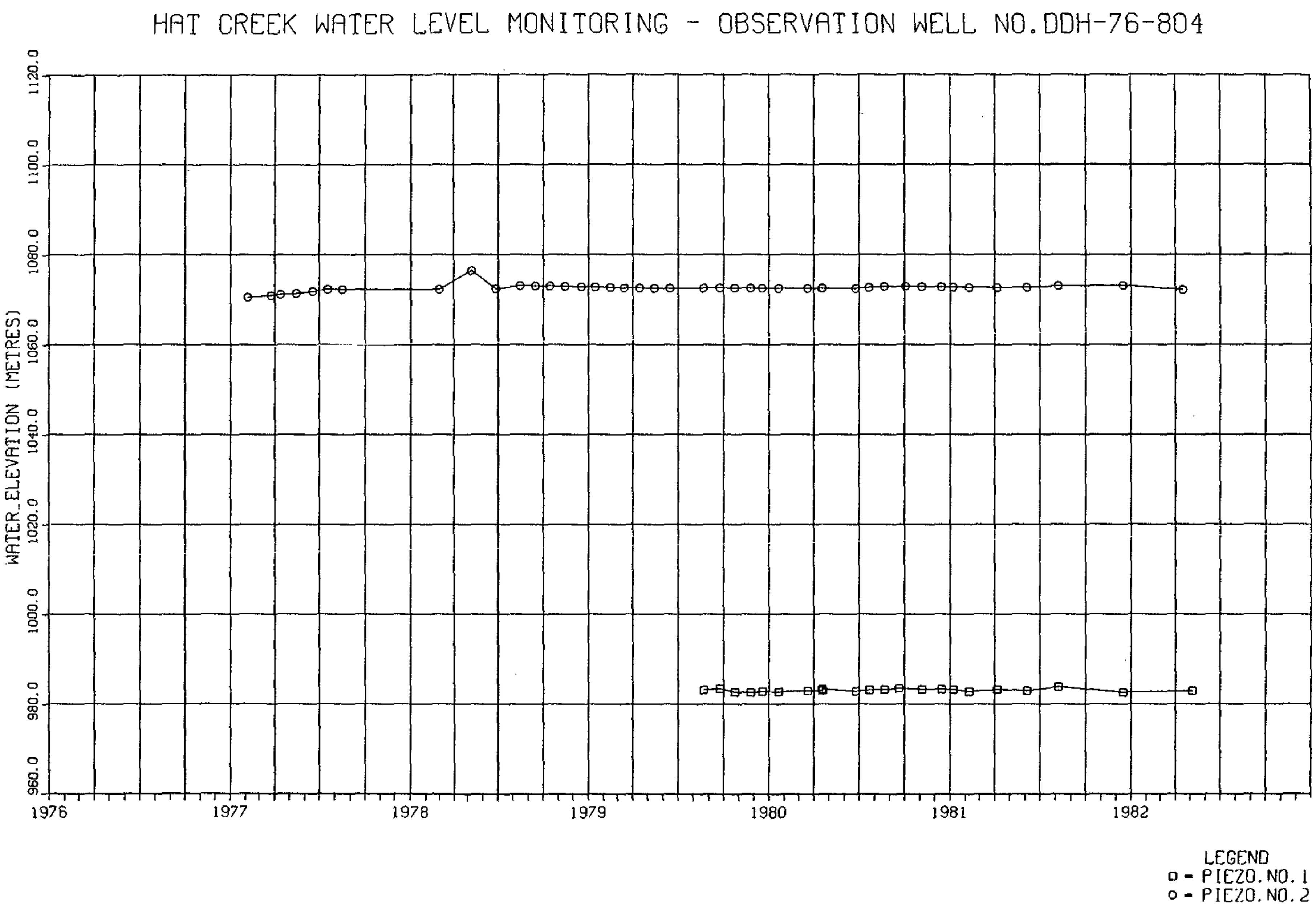


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-802
WATER ELEVATION (METRES)
1067.6
1067.2
1066.8
1066.4
1066.0
1065.6
1065.2
1064.8
1064.4
1976
1977
1978
1979
1980
1981
1982
Flowing hole
LEGEND
□ - PIEZO.NO.1
○ - PIEZO.NO.2

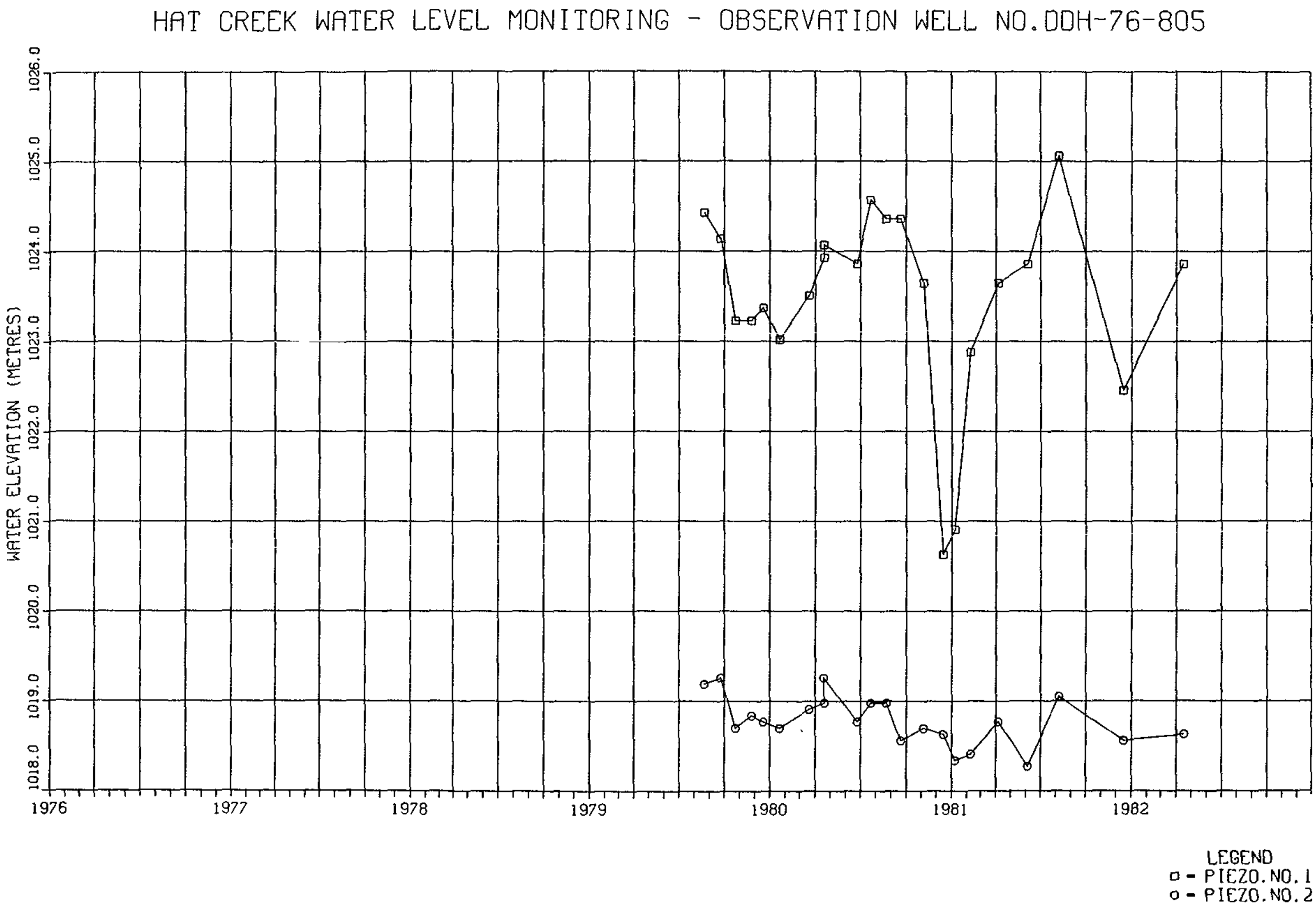
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-803



Golder Associates

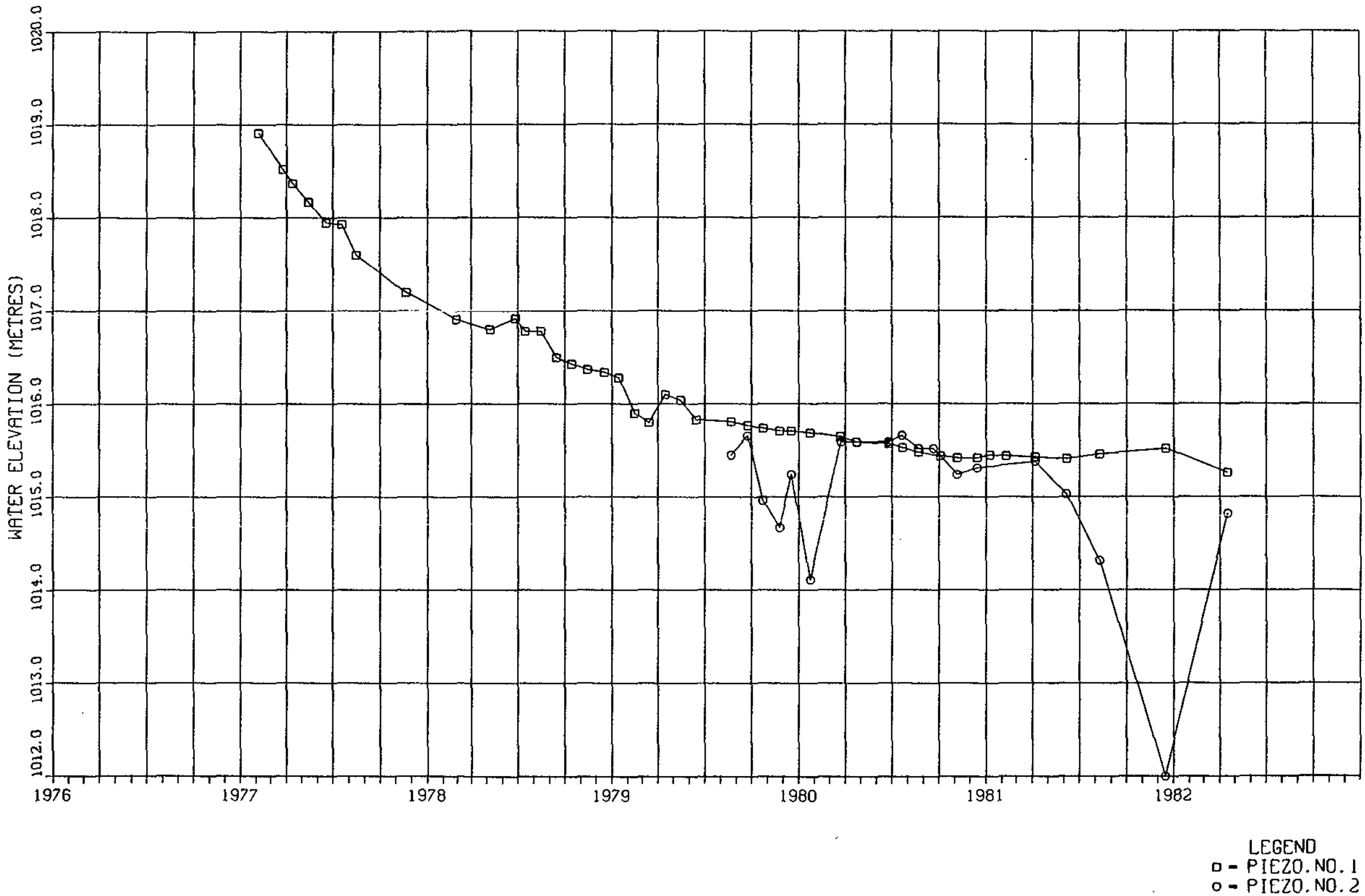


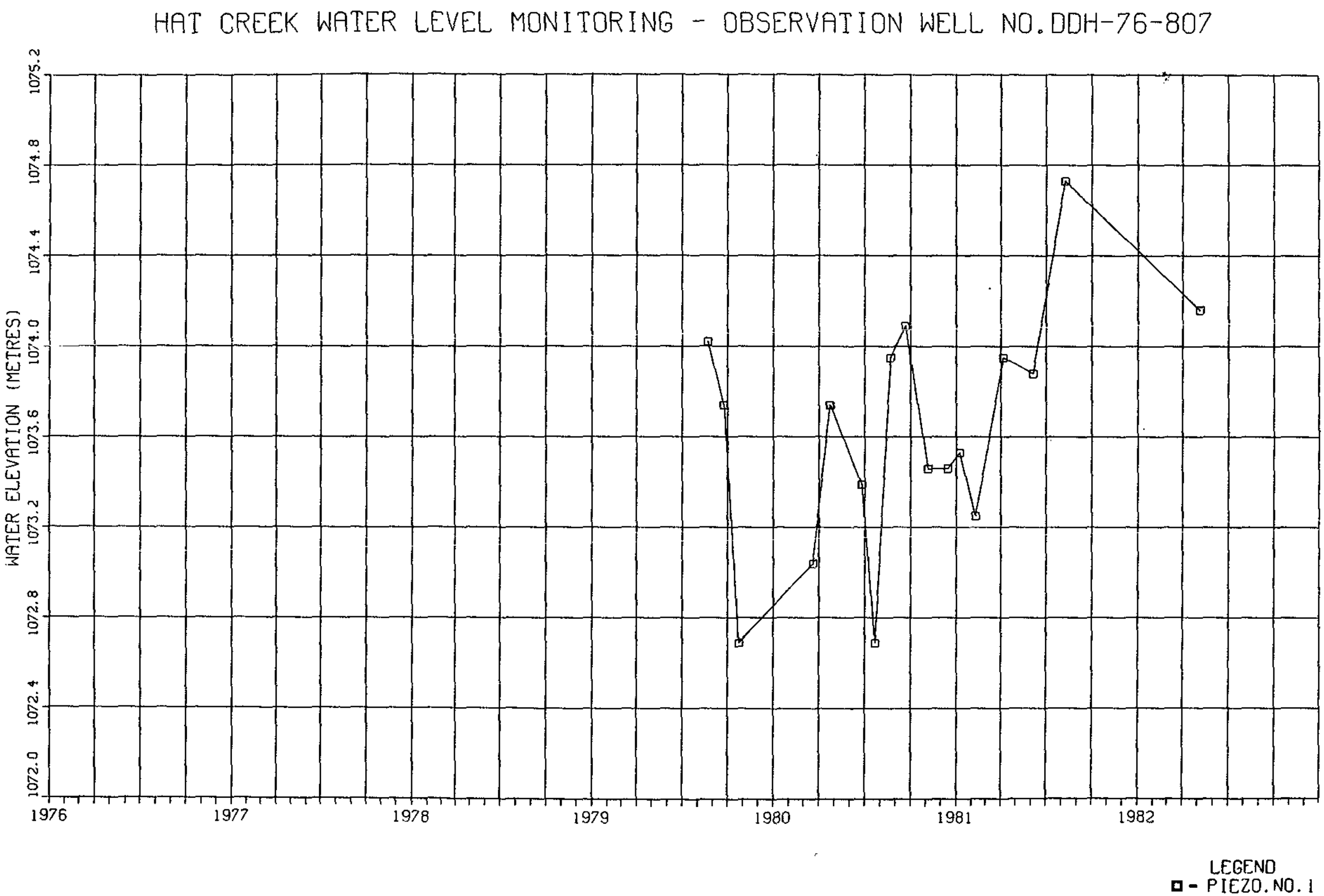
Golder Associates



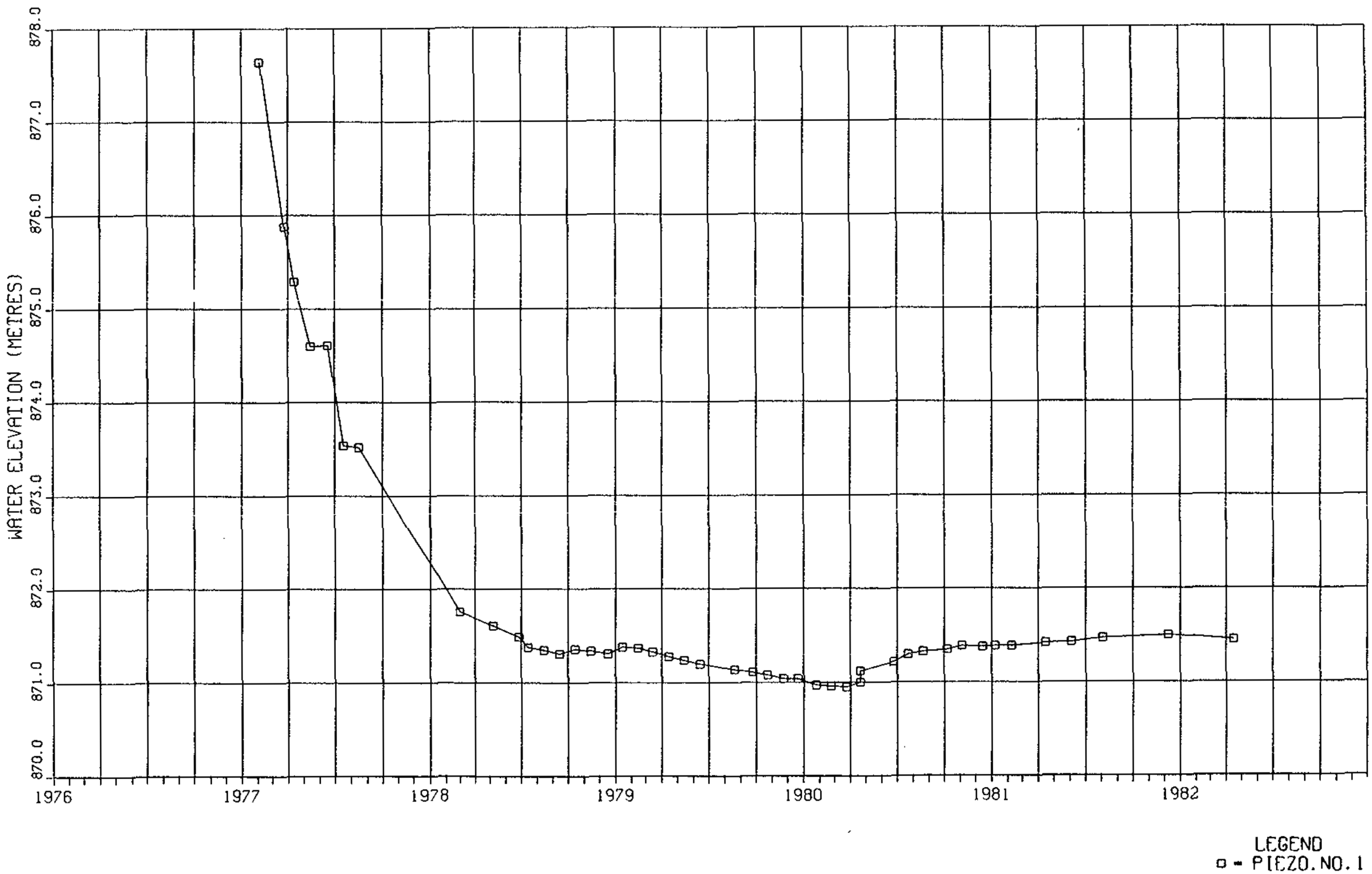
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-806



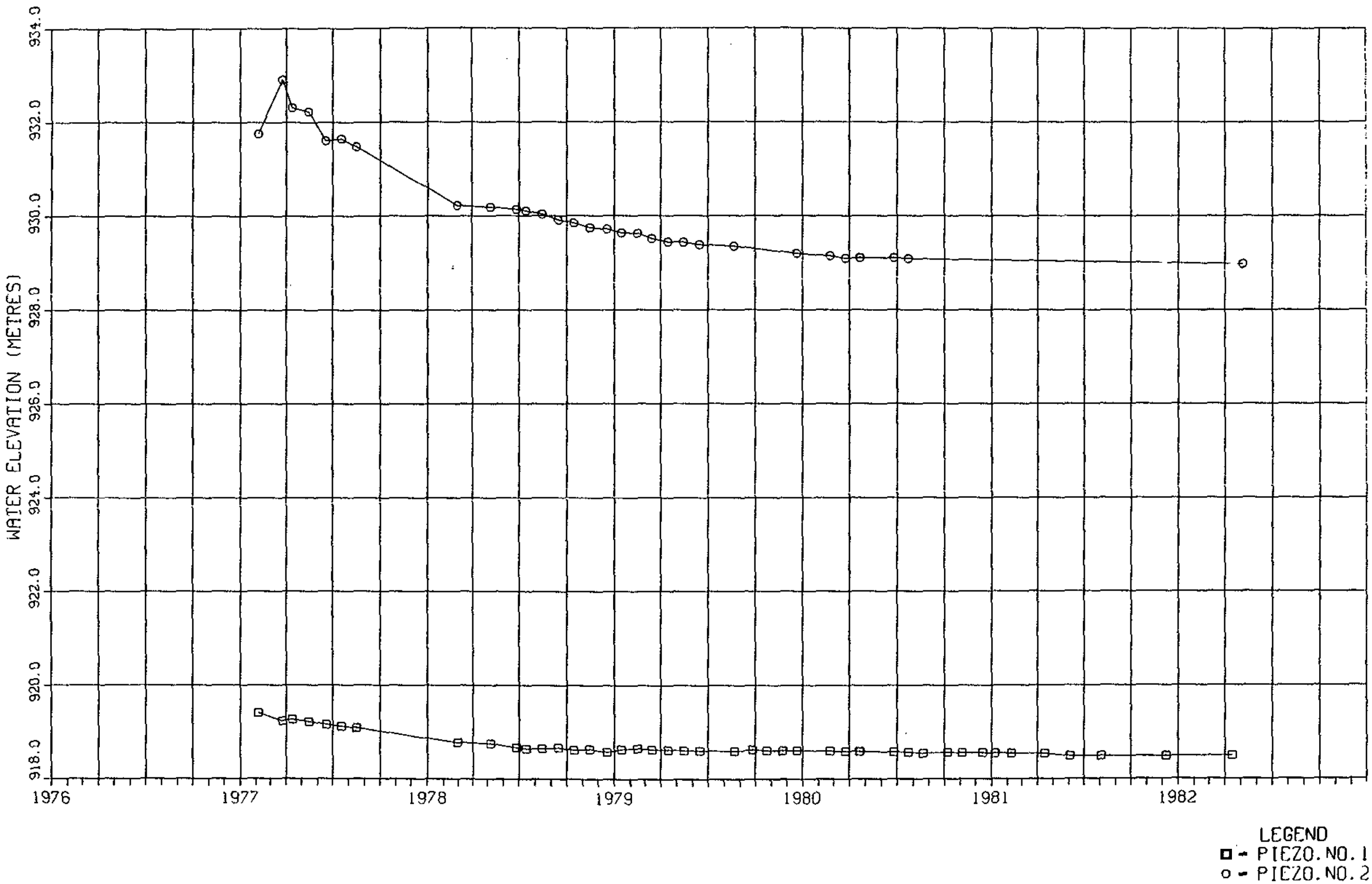

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-807
WATER ELEVATION (METRES)
1075.2
1074.8
1074.4
1074.0
1073.6
1073.2
1072.8
1072.4
1072.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
- PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-808



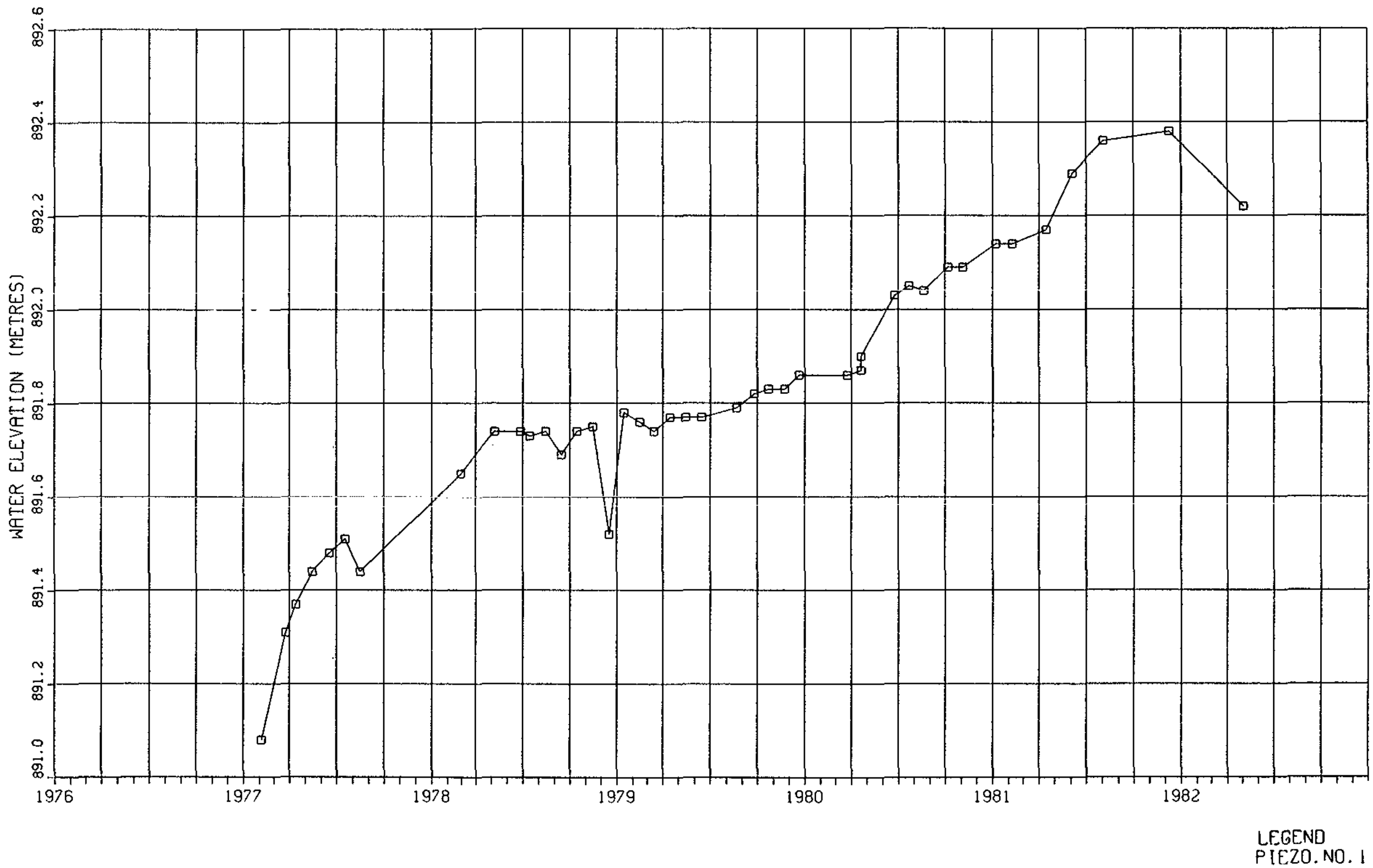
Golder Associates

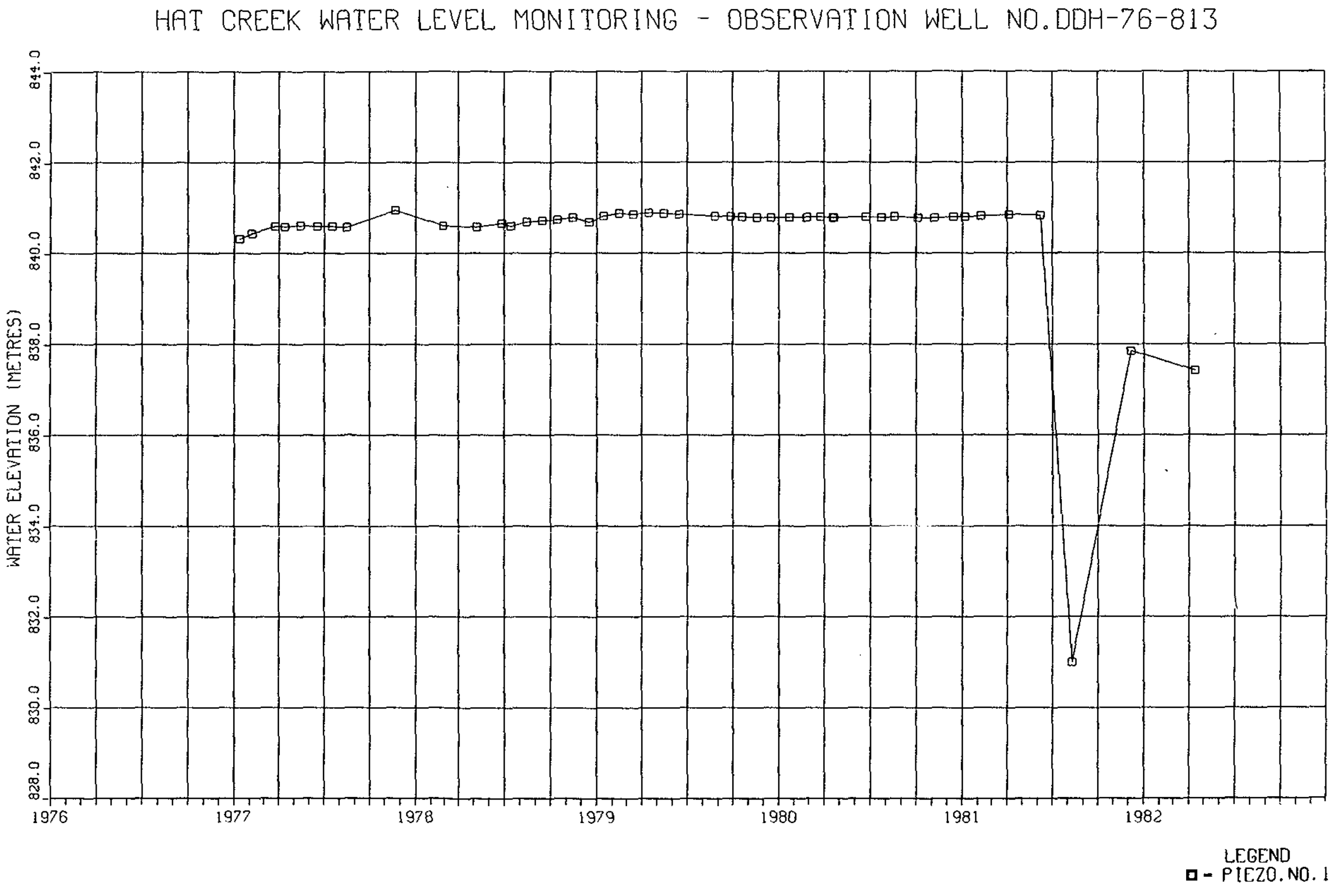
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-809



Golder Associates

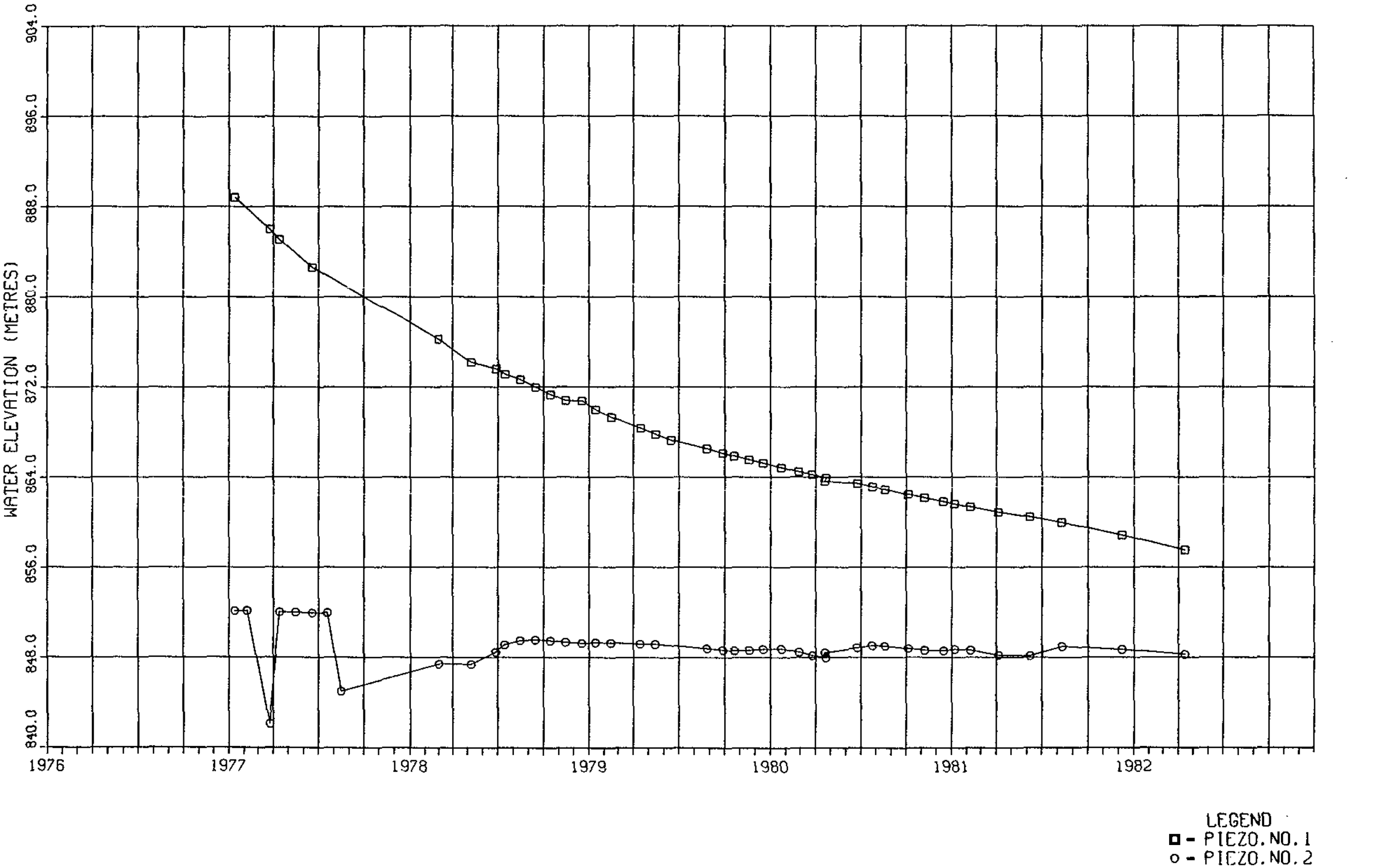
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-811





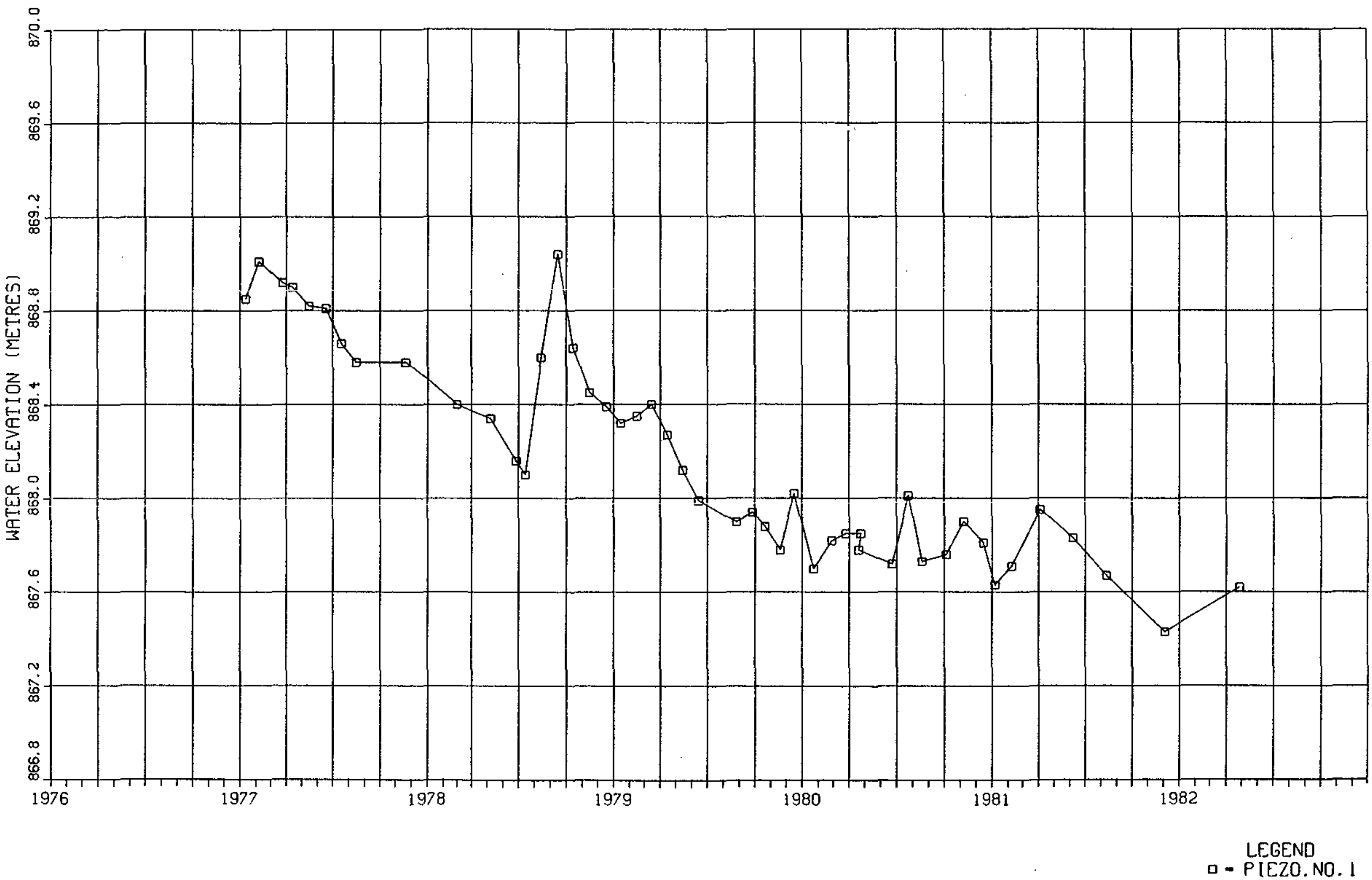
Golder Associates

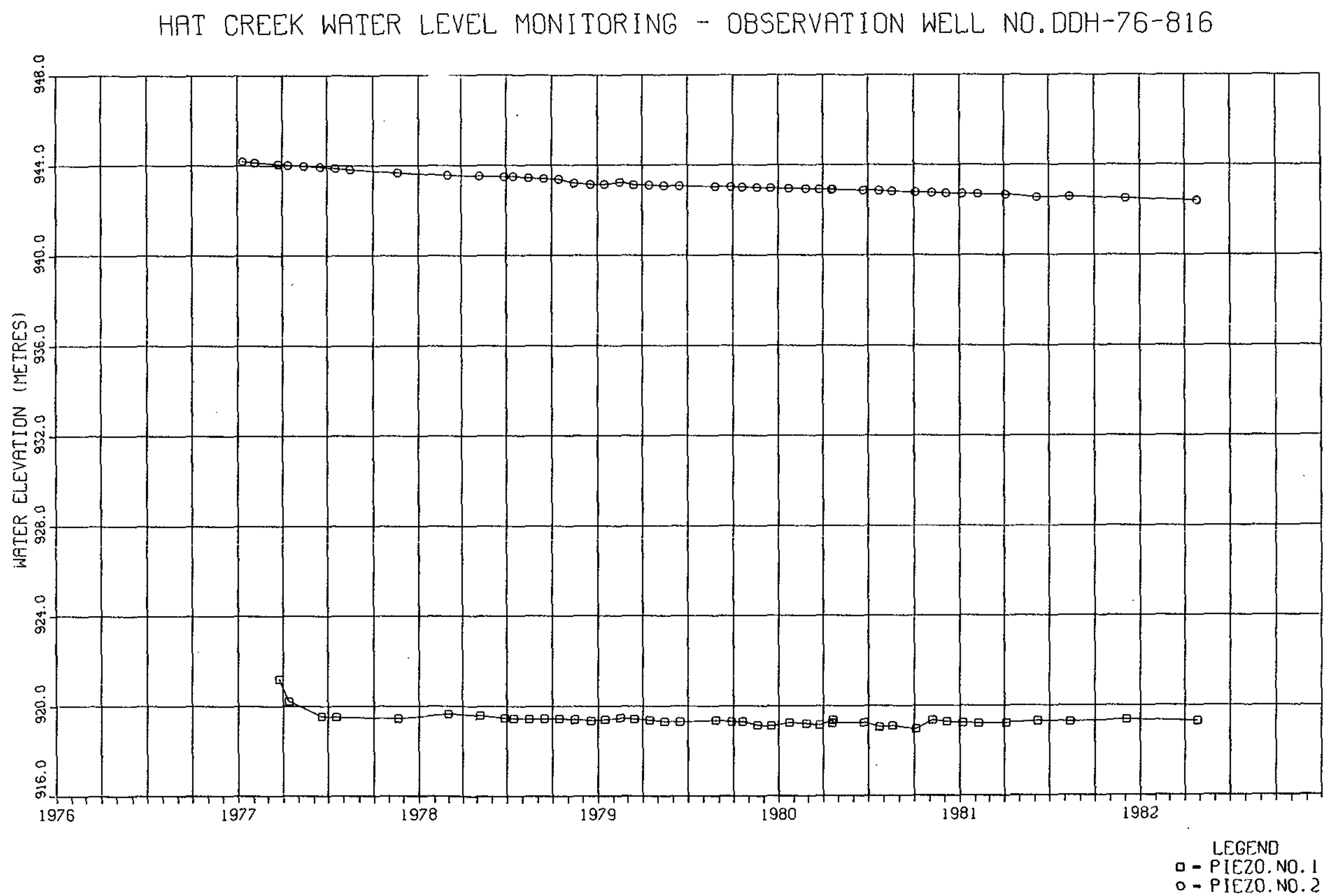
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-814

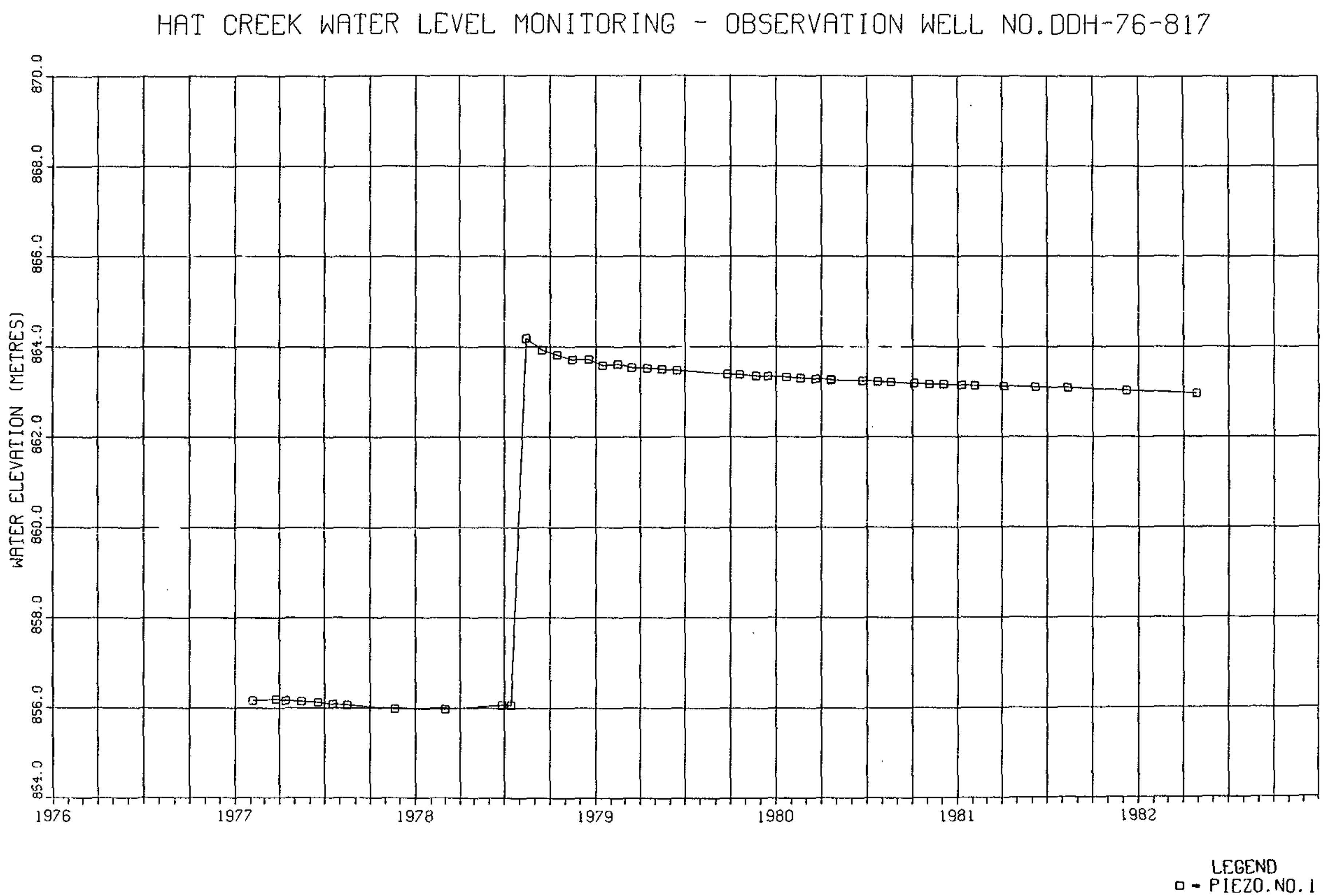


Golder Associates

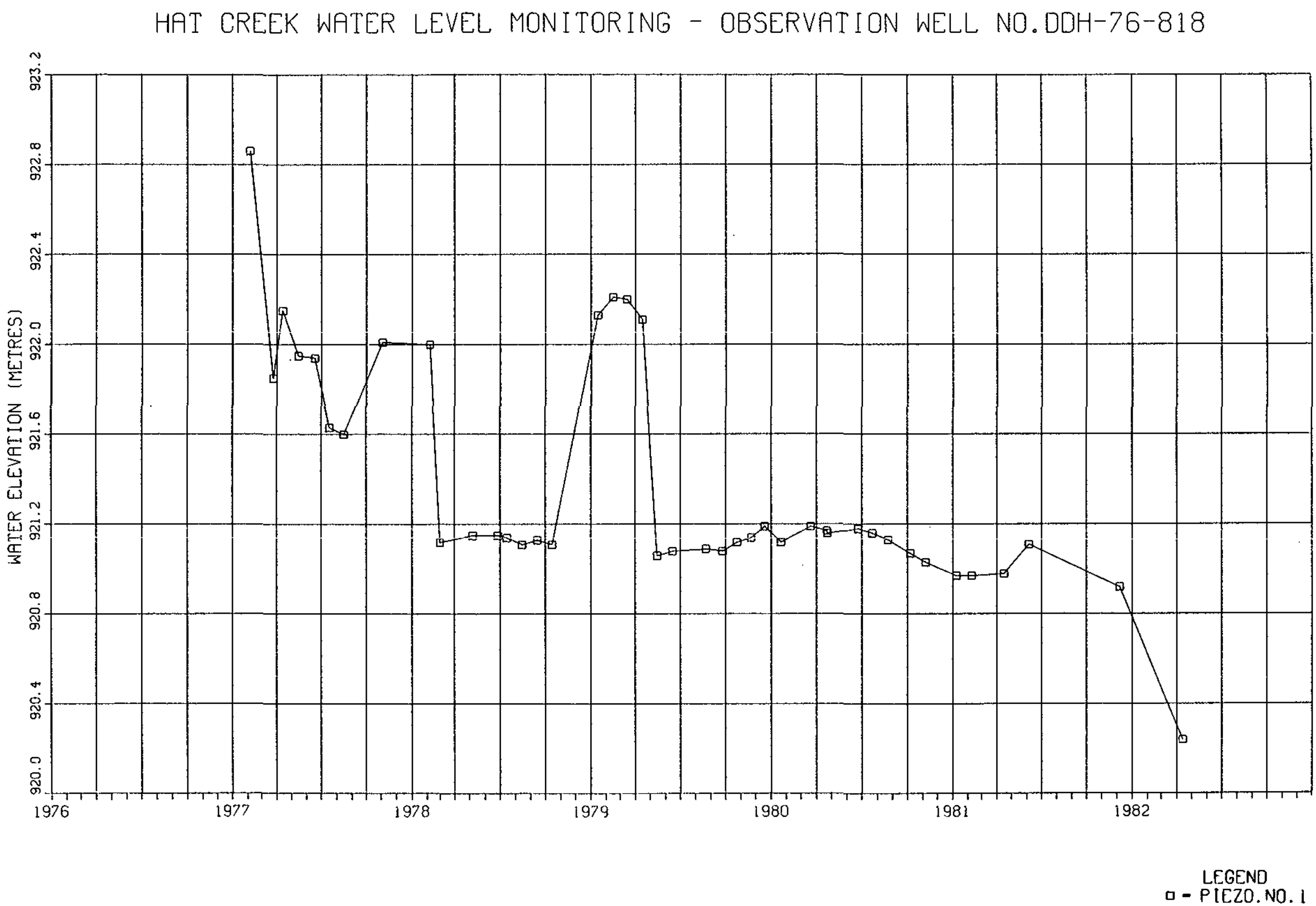
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-815




HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-816
WATER ELEVATION (METRES)
948.0
944.0
940.0
936.0
932.0
928.0
924.0
920.0
916.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1
○ - PIEZO.NO.2

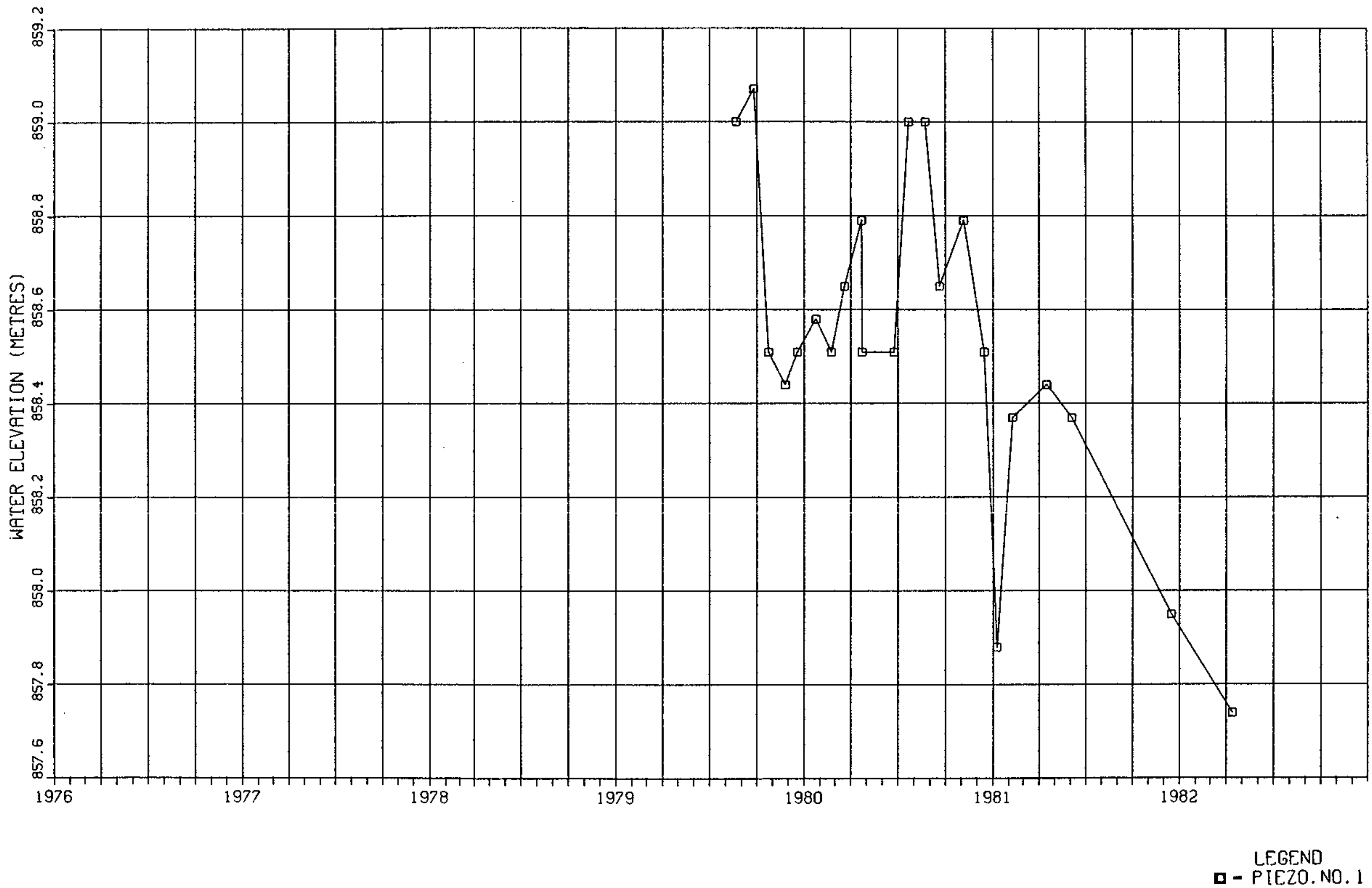


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-817
WATER ELEVATION (METRES)
870.0
868.0
866.0
864.0
862.0
860.0
858.0
856.0
854.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

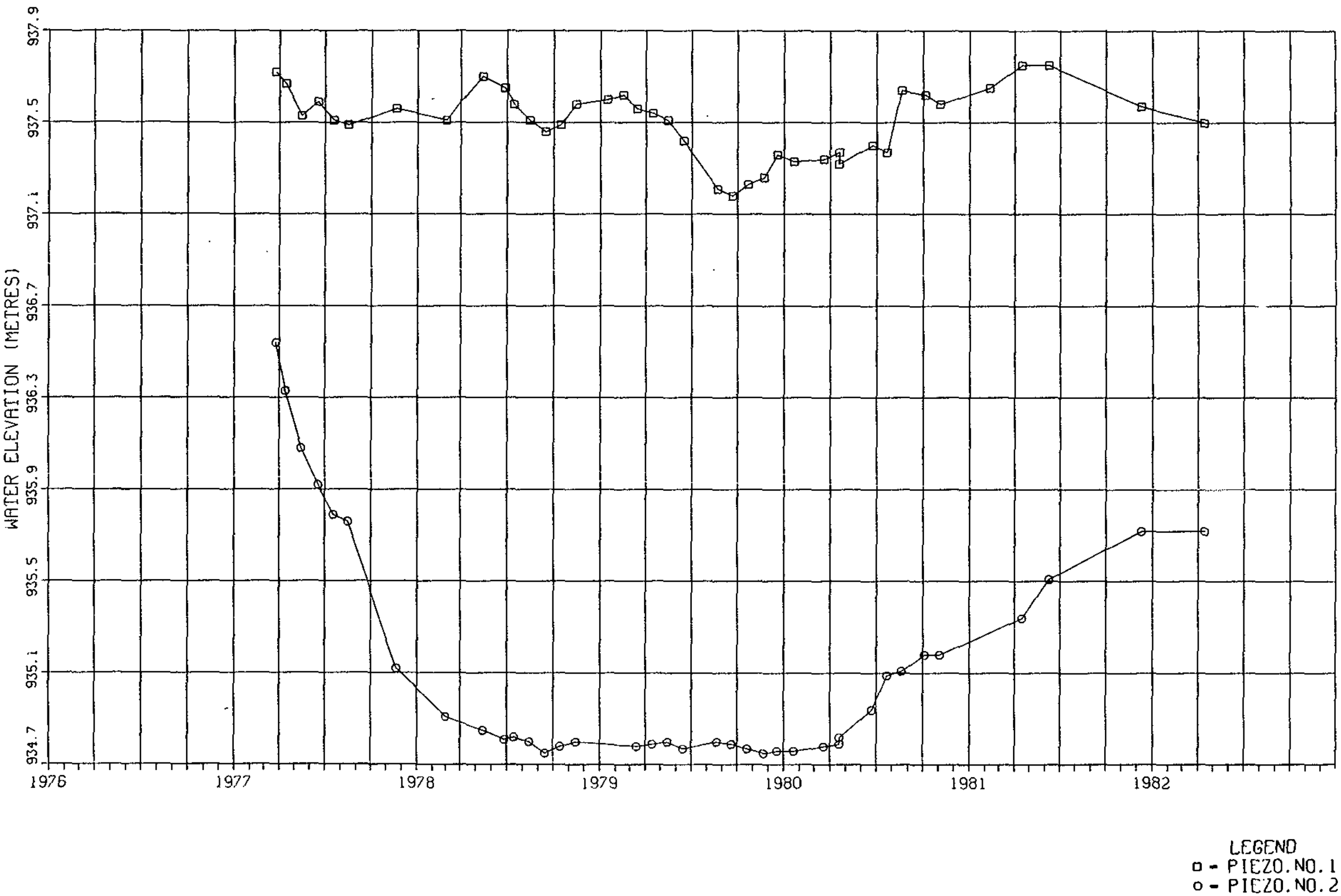


Golder Associates

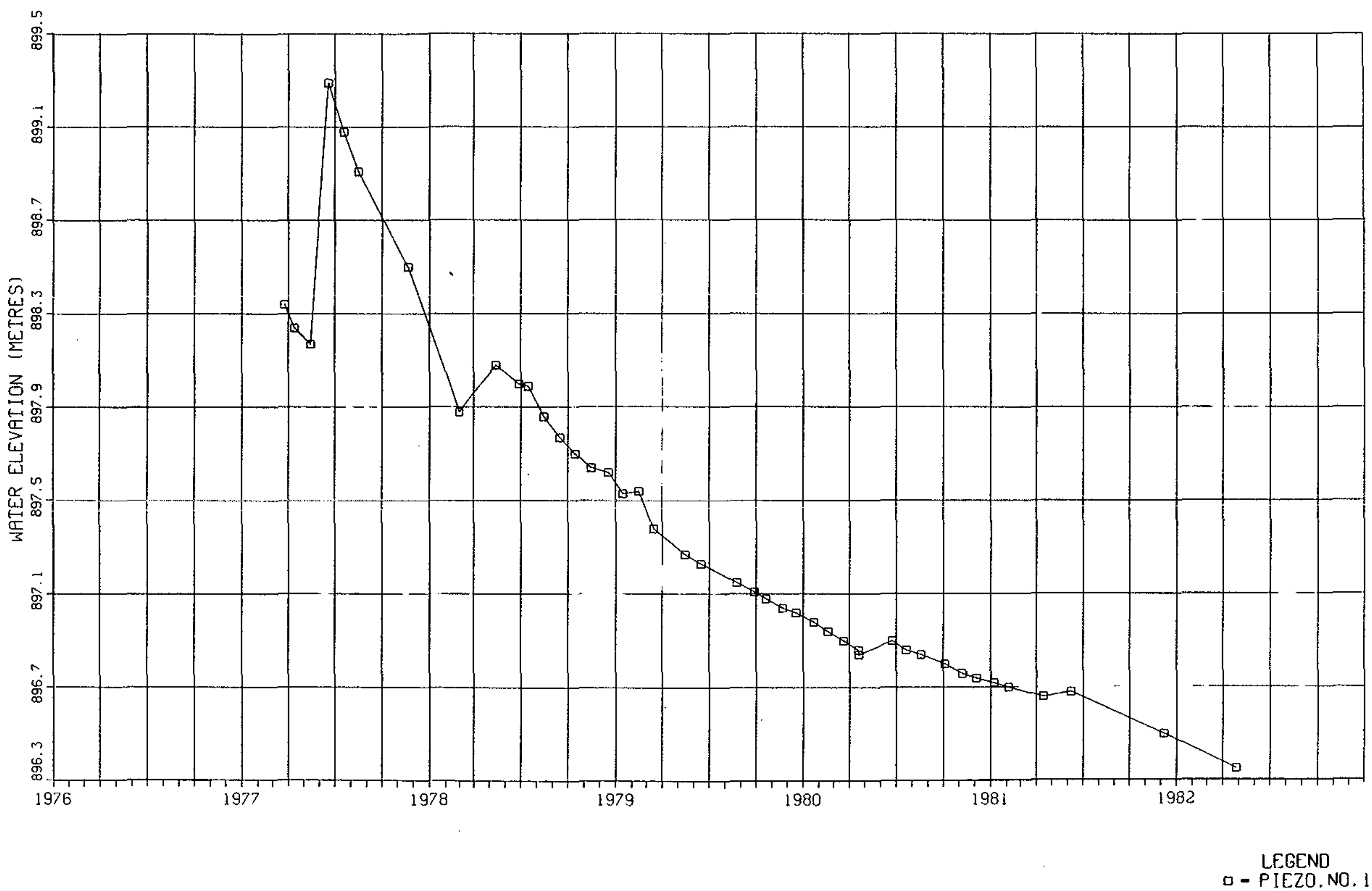
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-819



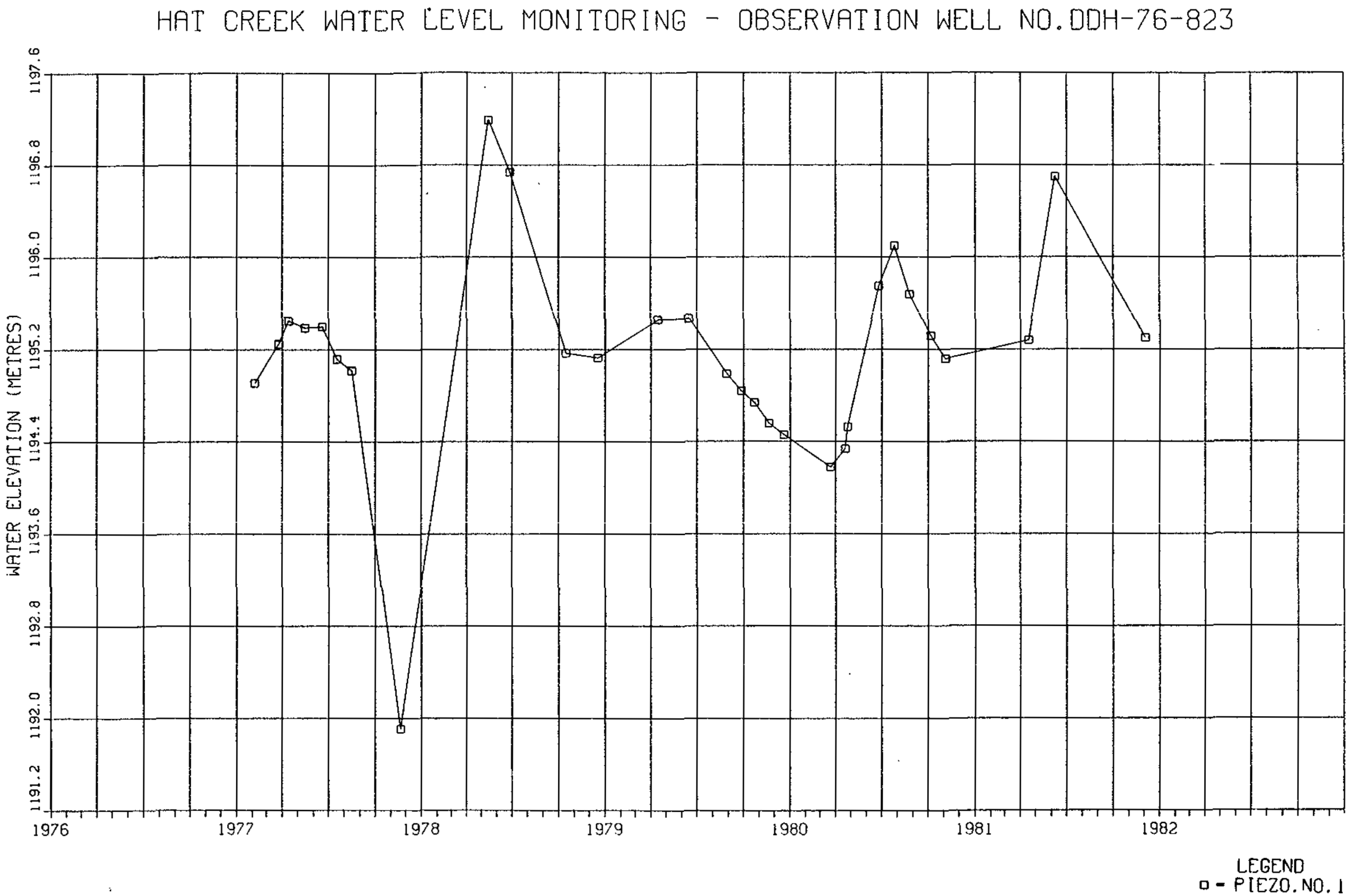
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-820



HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-821



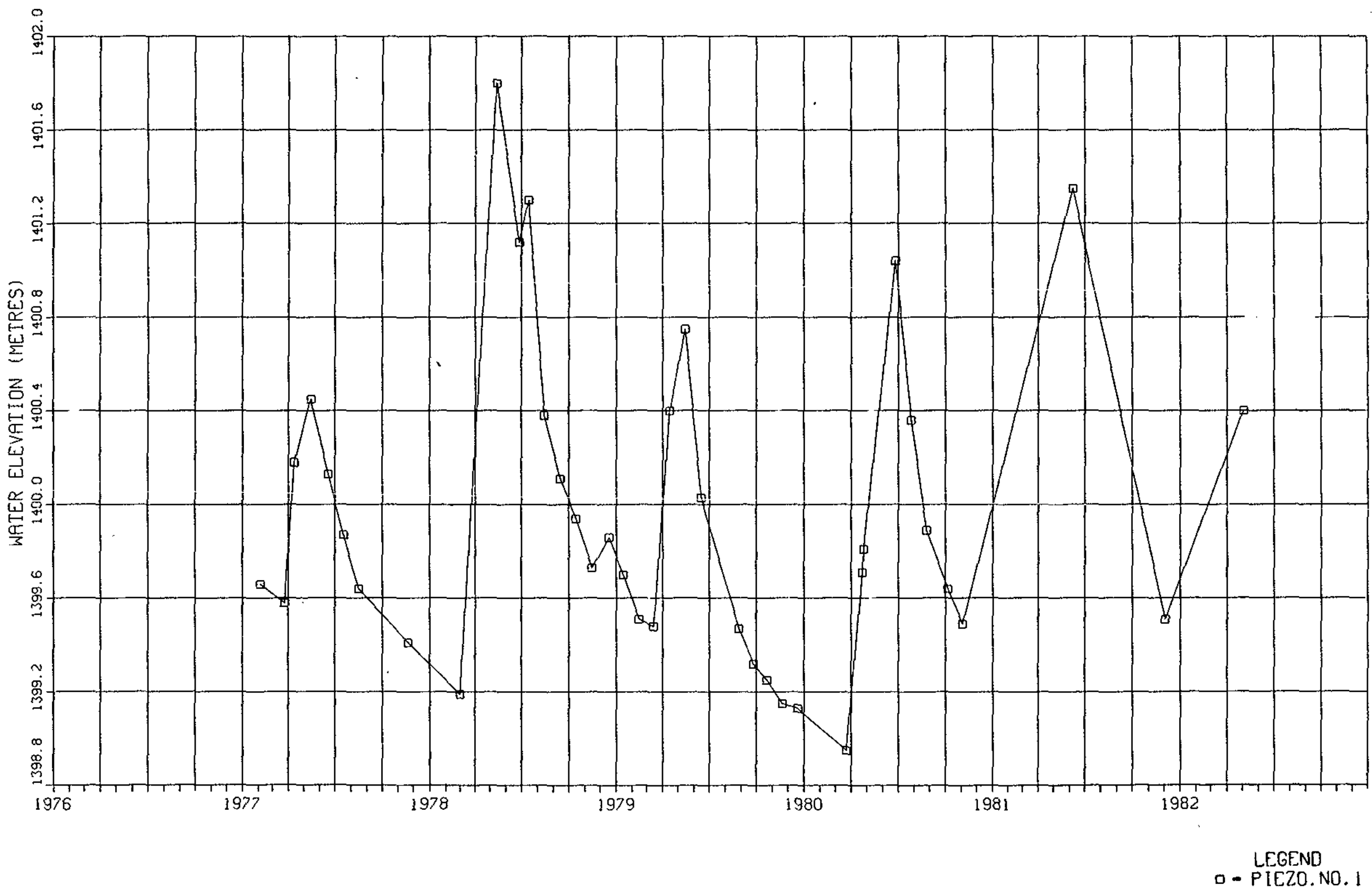
Golder Associates

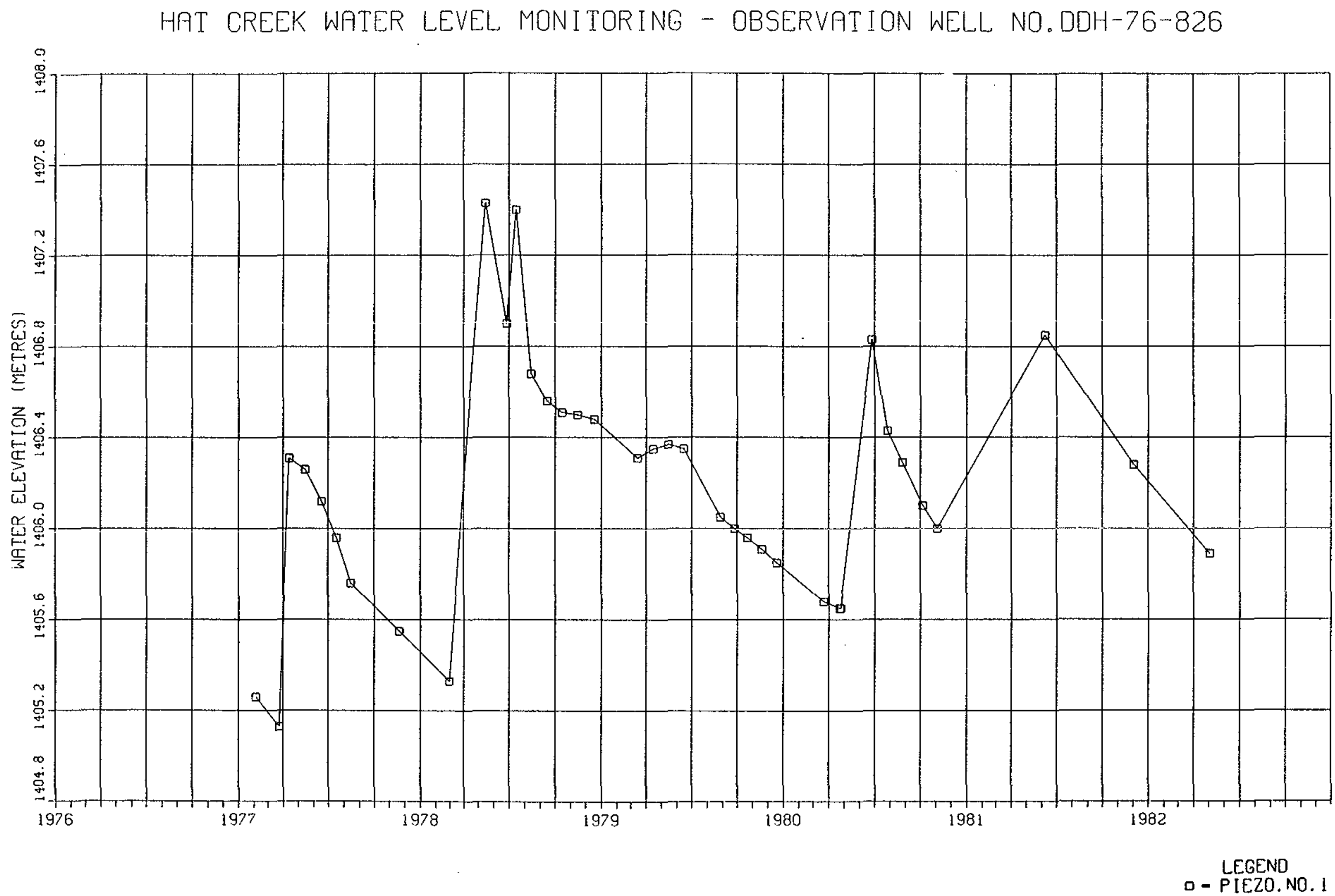


Golder Associates

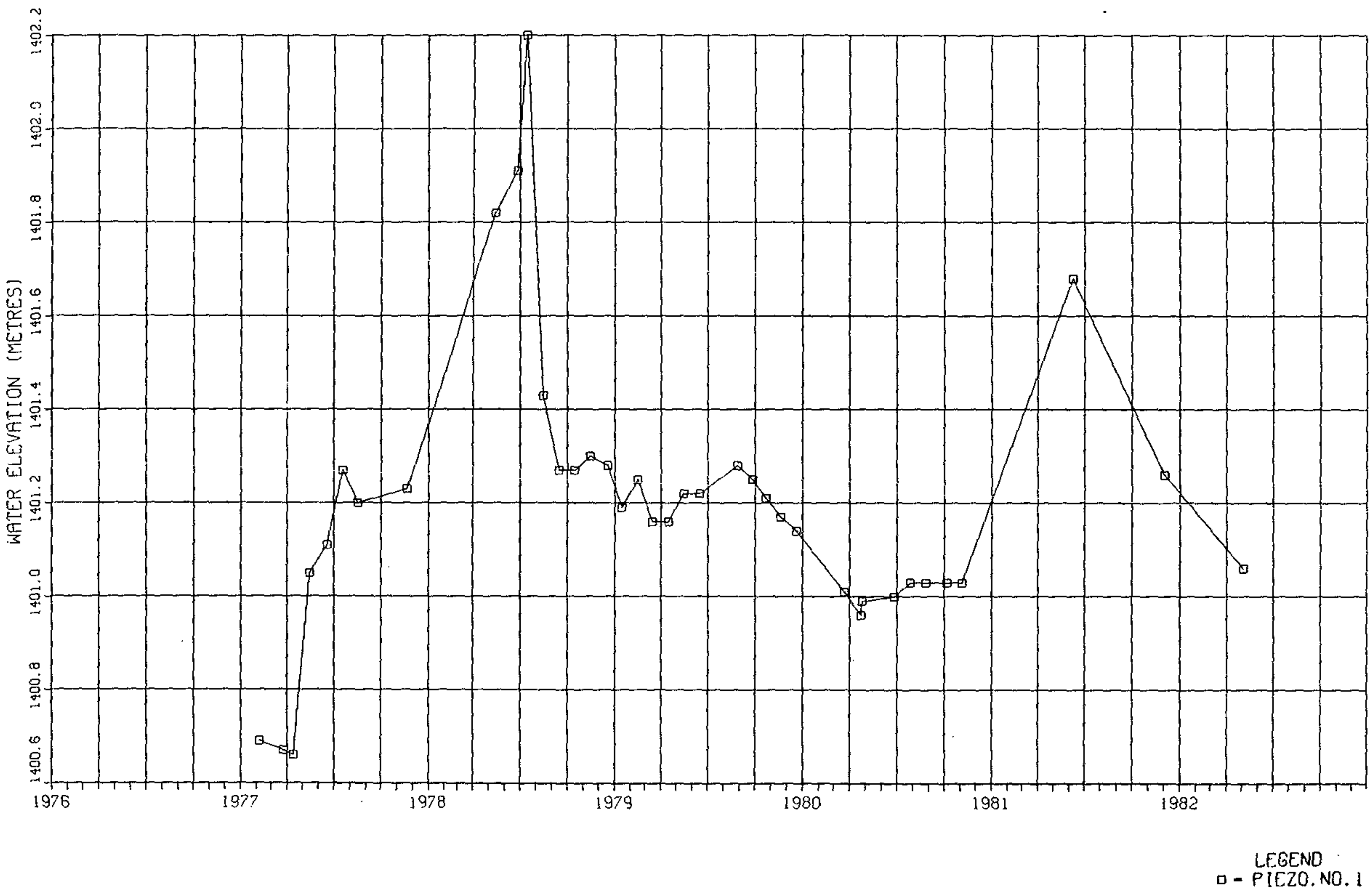

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-824
WATER ELEVATION (METRES)
1130.0
1129.6
1129.2
1128.8
1128.4
1128.0
1127.6
1127.2
1126.8
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

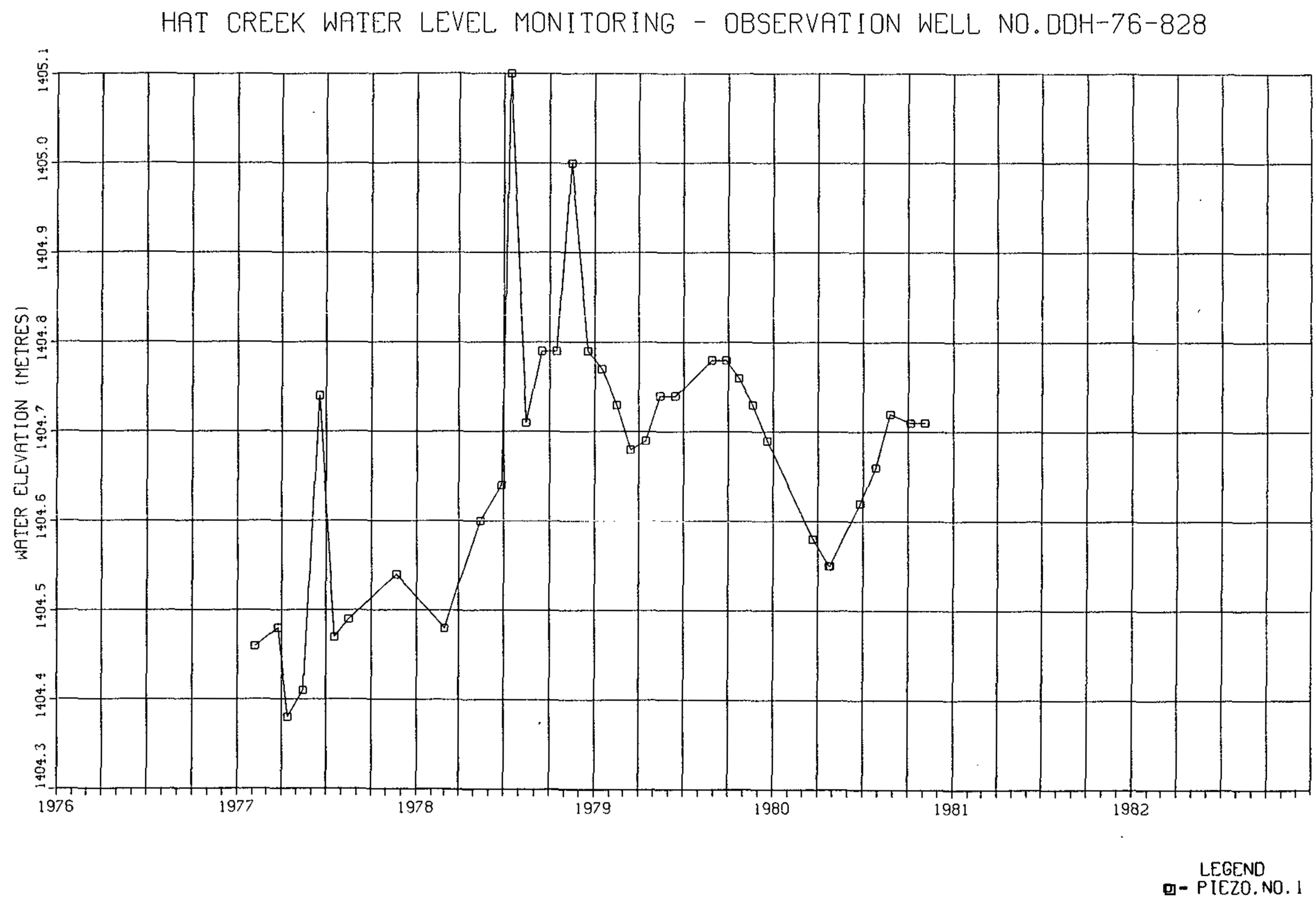
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-825

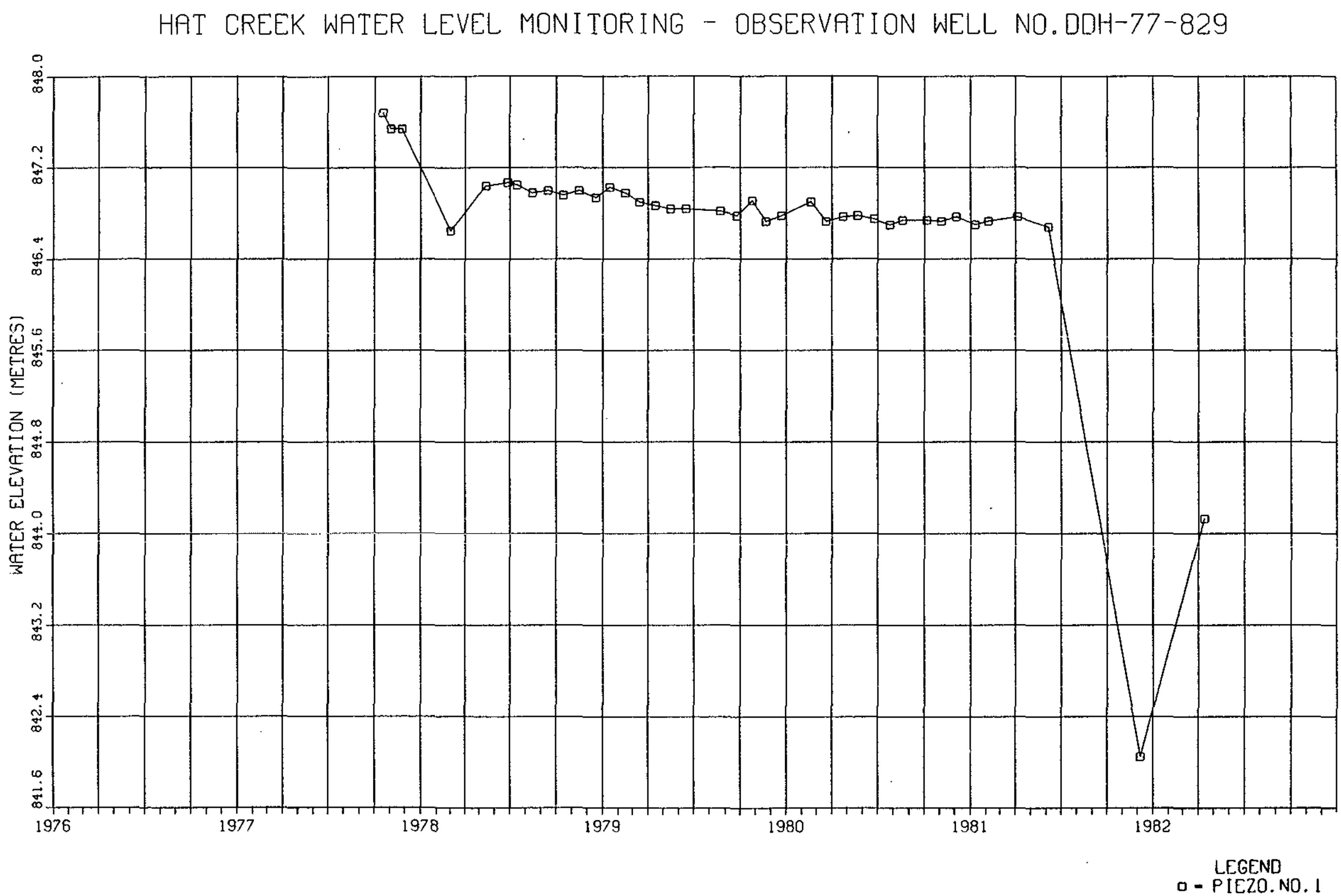



HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-826
WATER ELEVATION (METRES)
1408.0
1407.6
1407.2
1406.8
1406.4
1406.0
1405.6
1405.2
1404.8
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

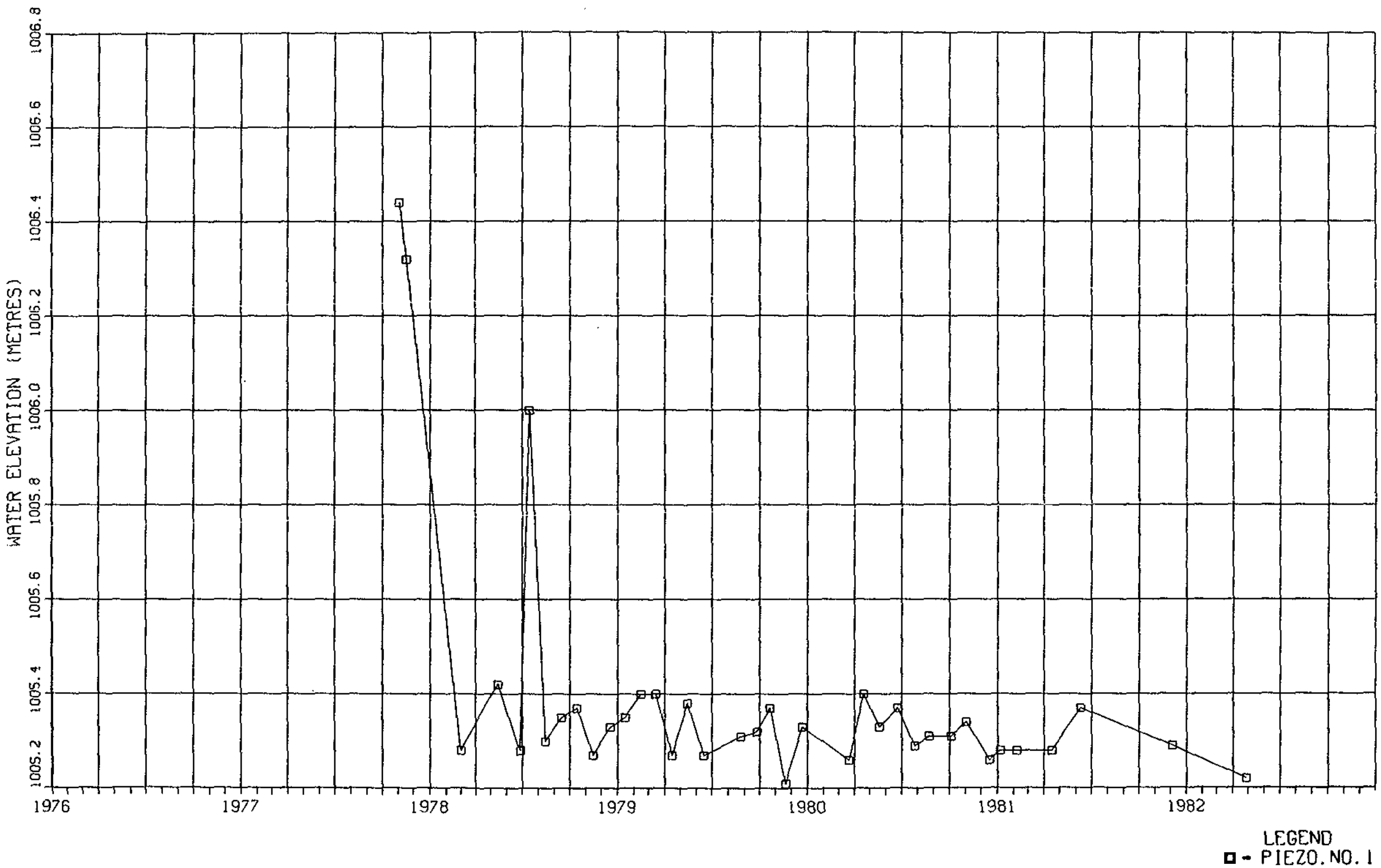
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-827



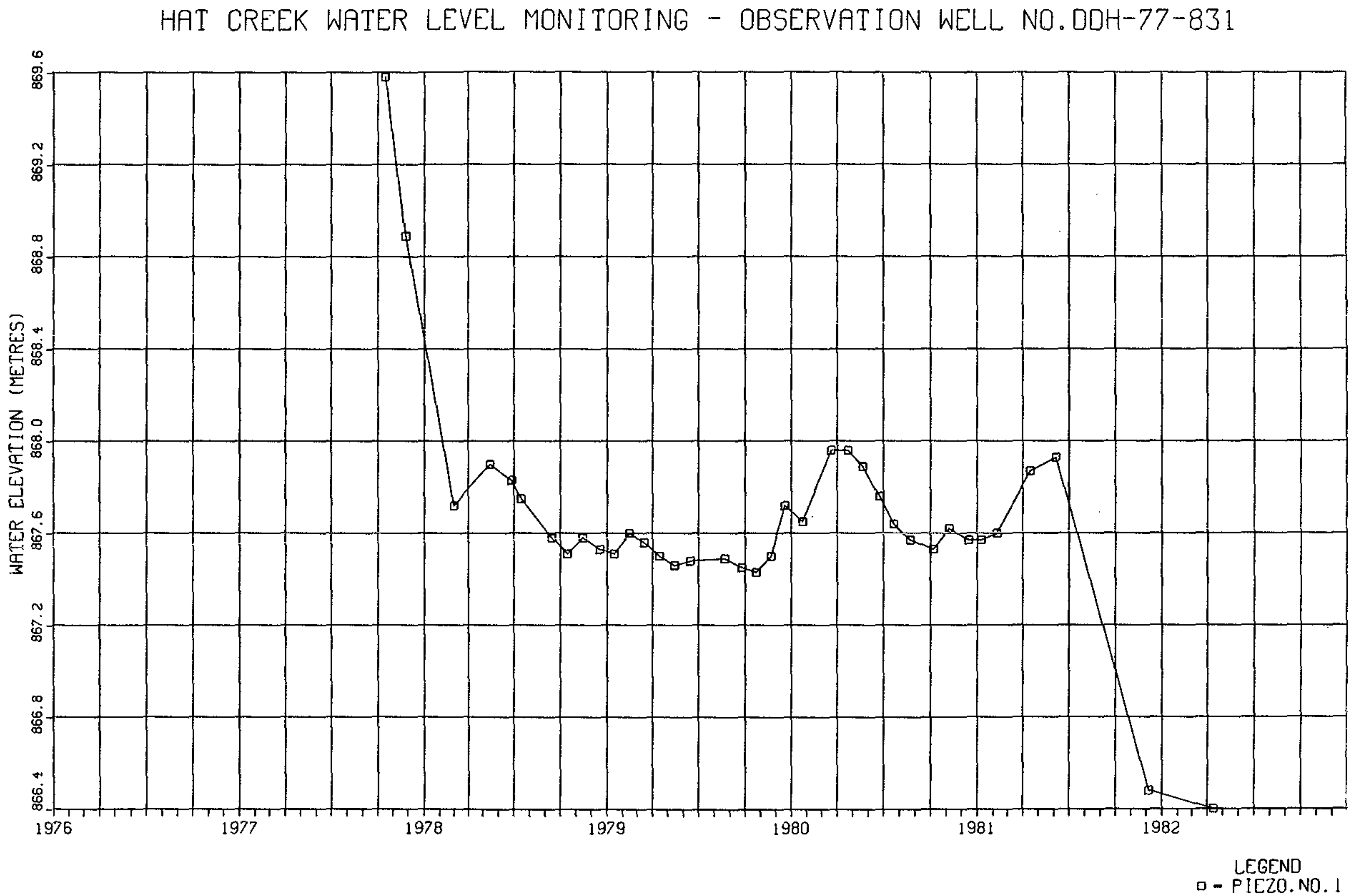

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-76-828
WATER ELEVATION (METRES)
1405.1
1405.0
1404.9
1404.8
1404.7
1404.6
1404.5
1404.4
1404.3
1976
1977
1978
1979
1980
1981
1982
LEGEND
PIEZO.NO.1

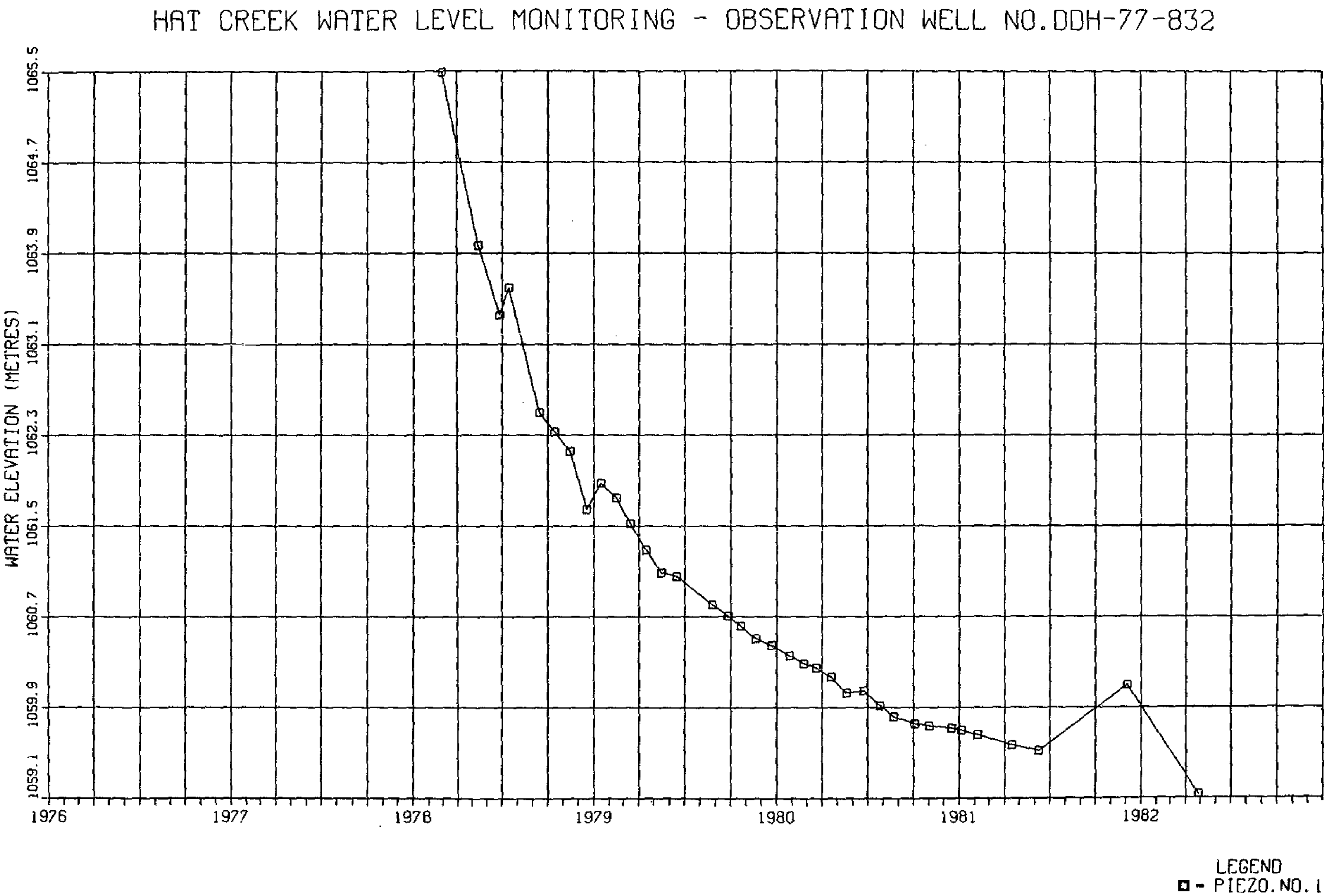

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-829
WATER ELEVATION (METRES)
848.0
847.2
846.4
845.6
844.8
844.0
843.2
842.4
841.6
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-830



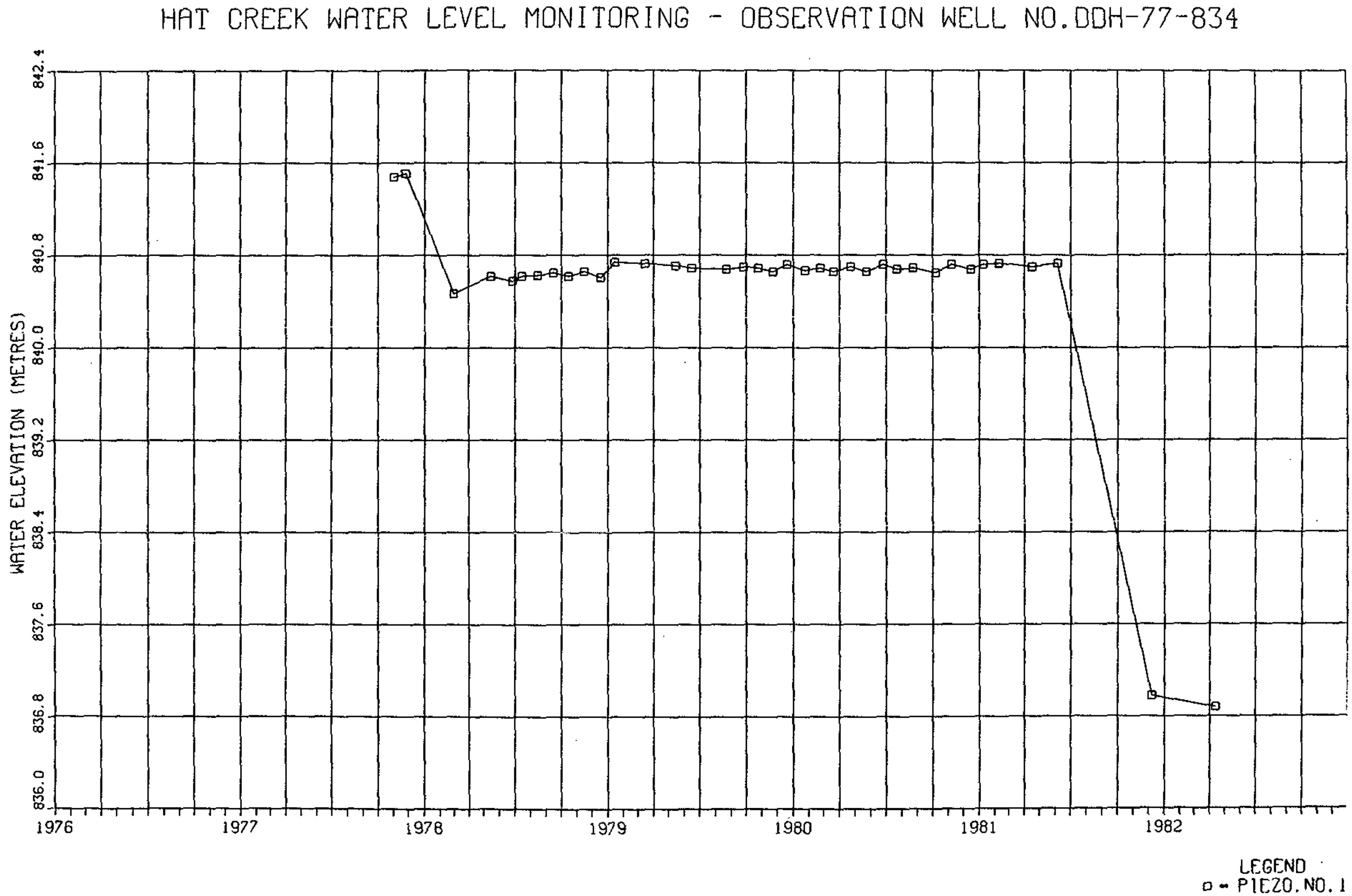
Golder Associates


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-831
WATER ELEVATION (METRES)
869.6
869.2
868.8
868.4
868.0
867.6
867.2
866.8
866.4
1976
1977
1978
1979
1980
1981
1982
LEGEND
▫ - PIEZO.NO.1

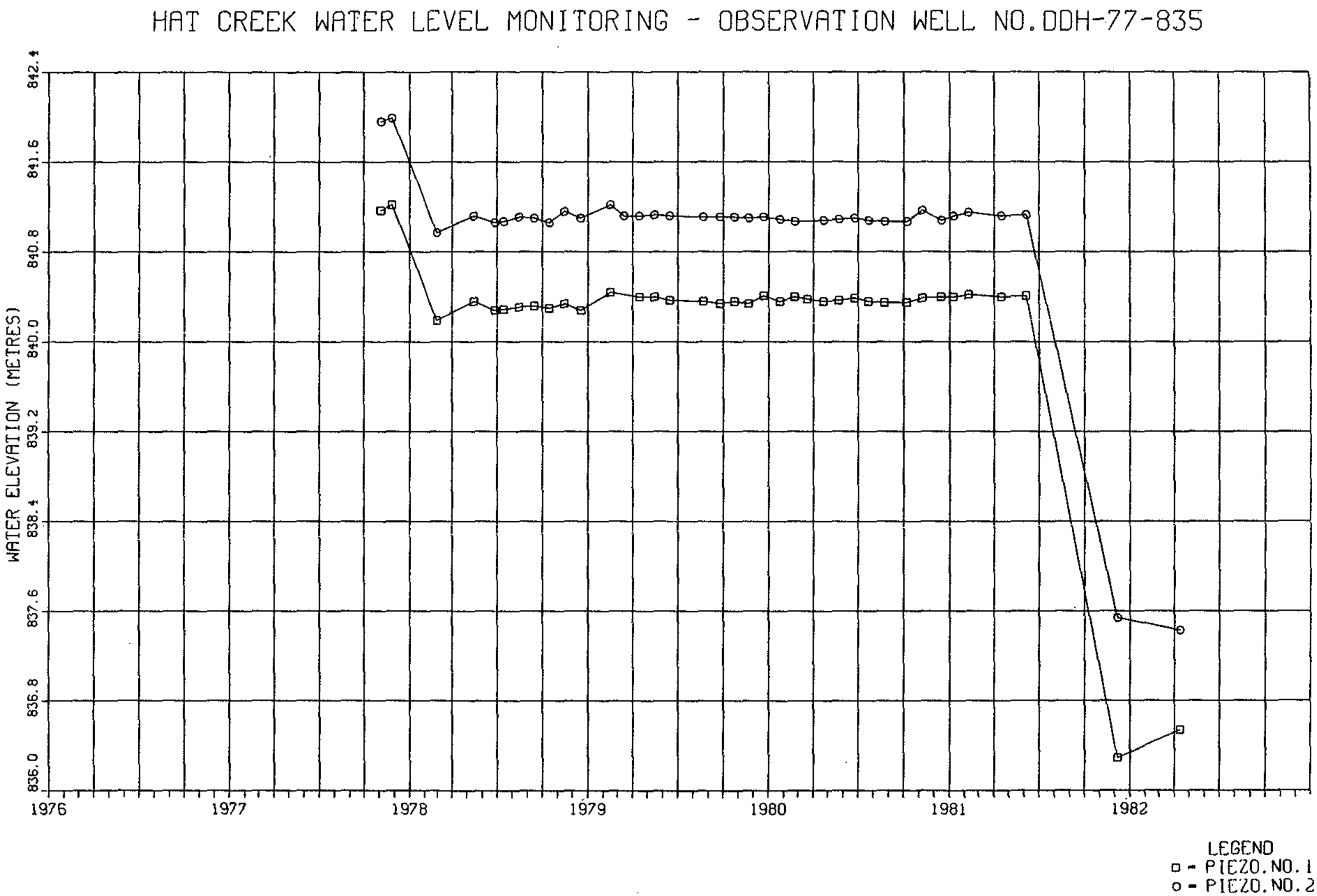


Golder Associates

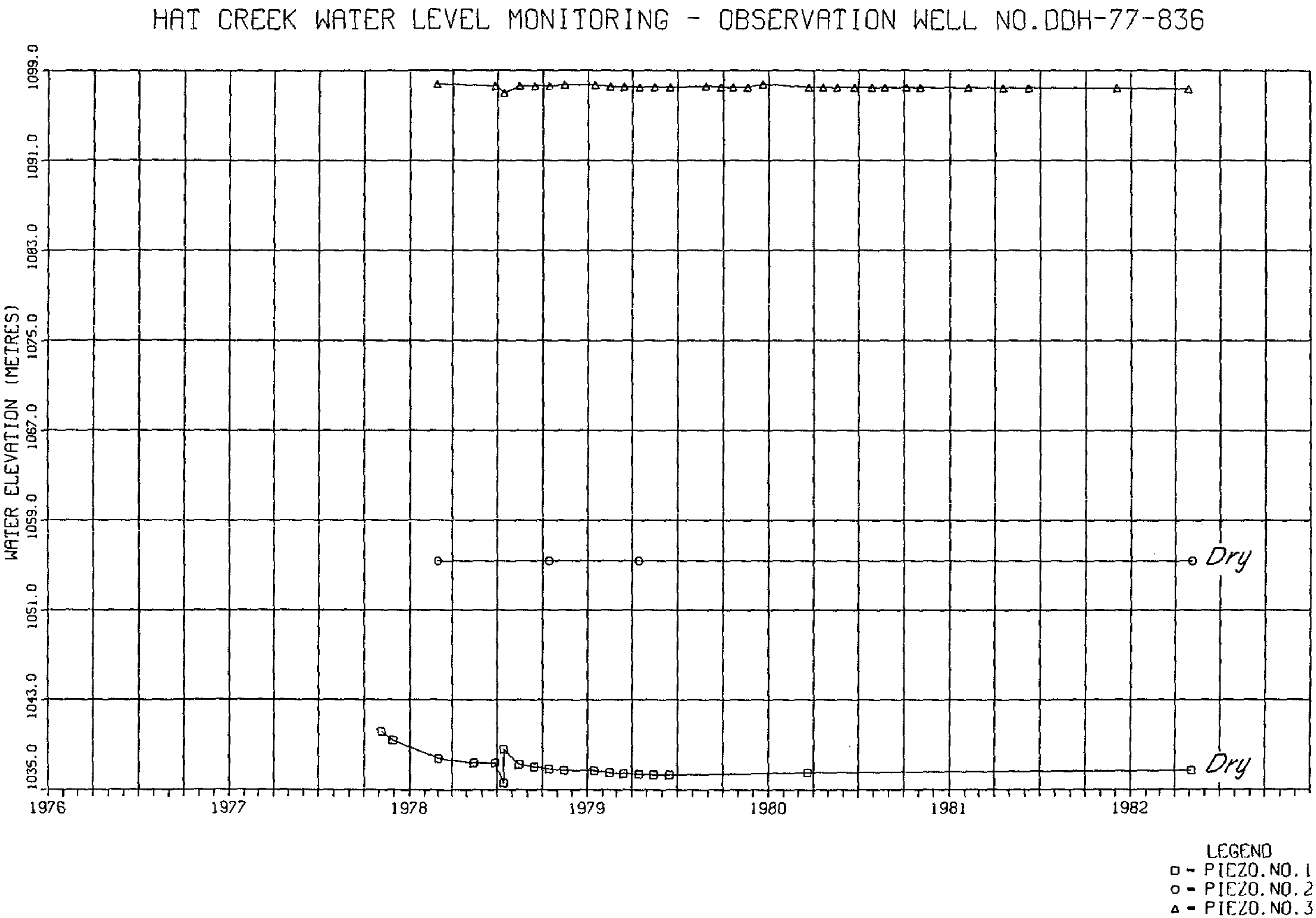
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-833



Golder Associates

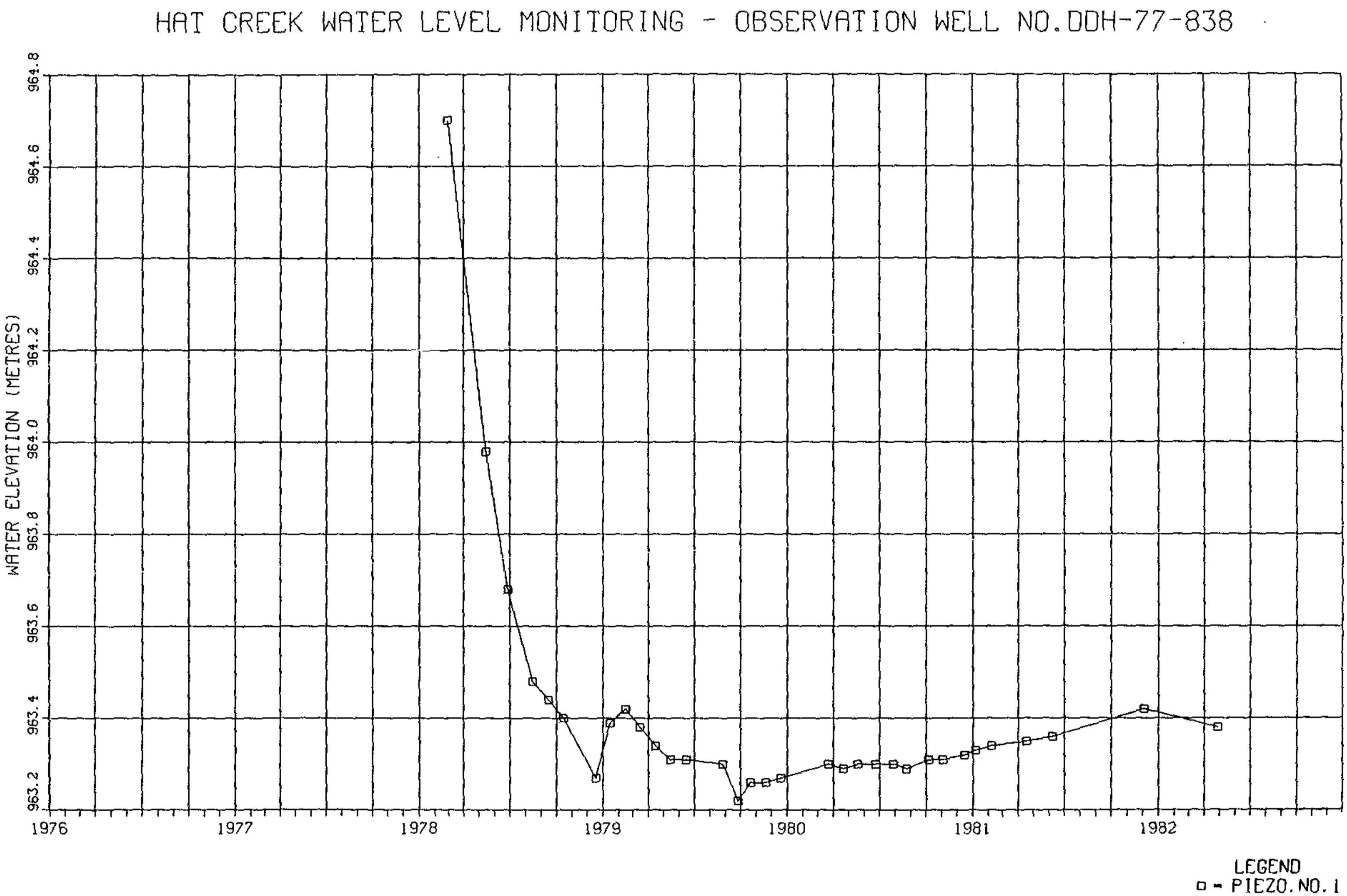


Golder Associates



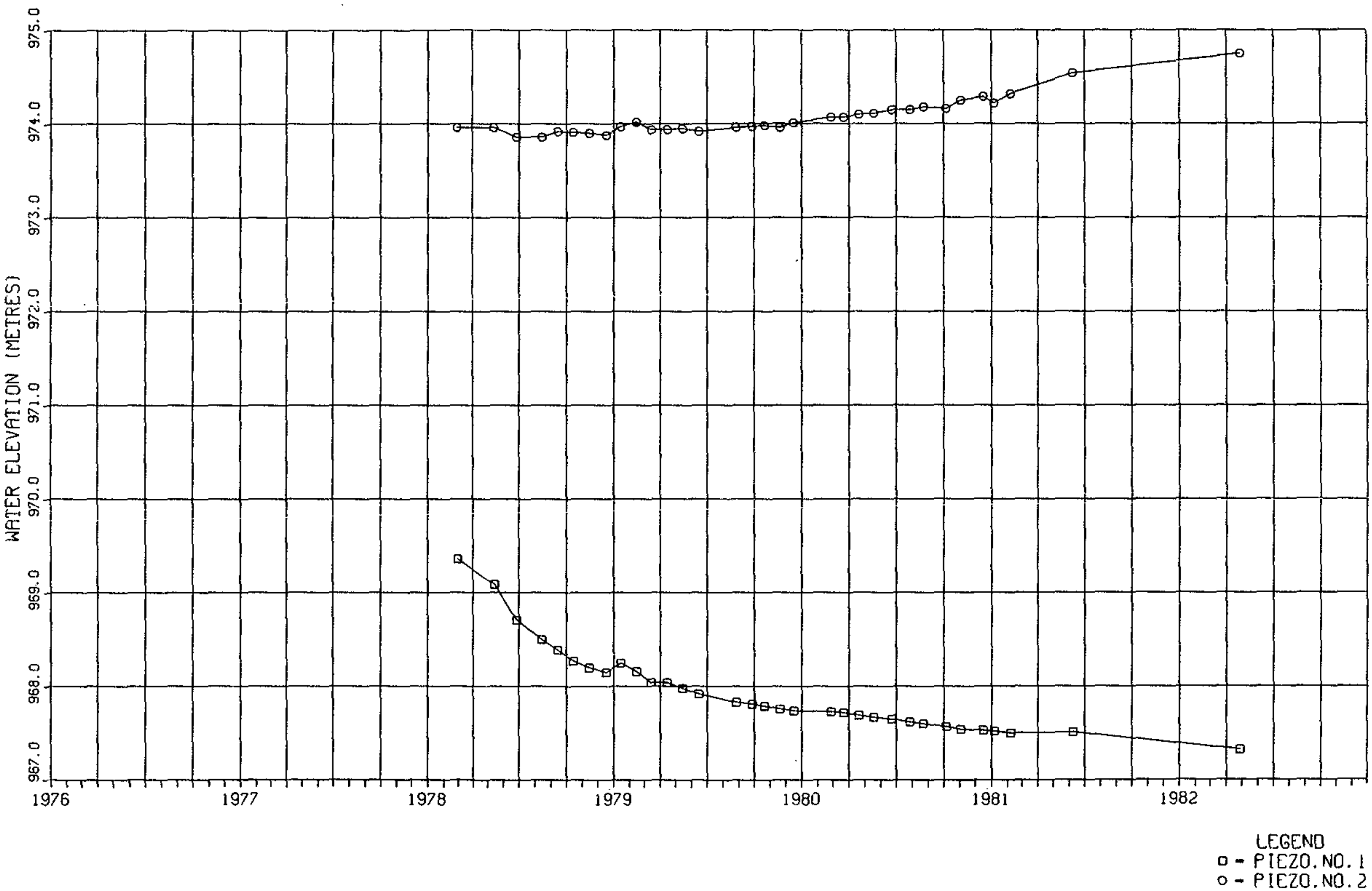
Golder Associates

Golder Associates

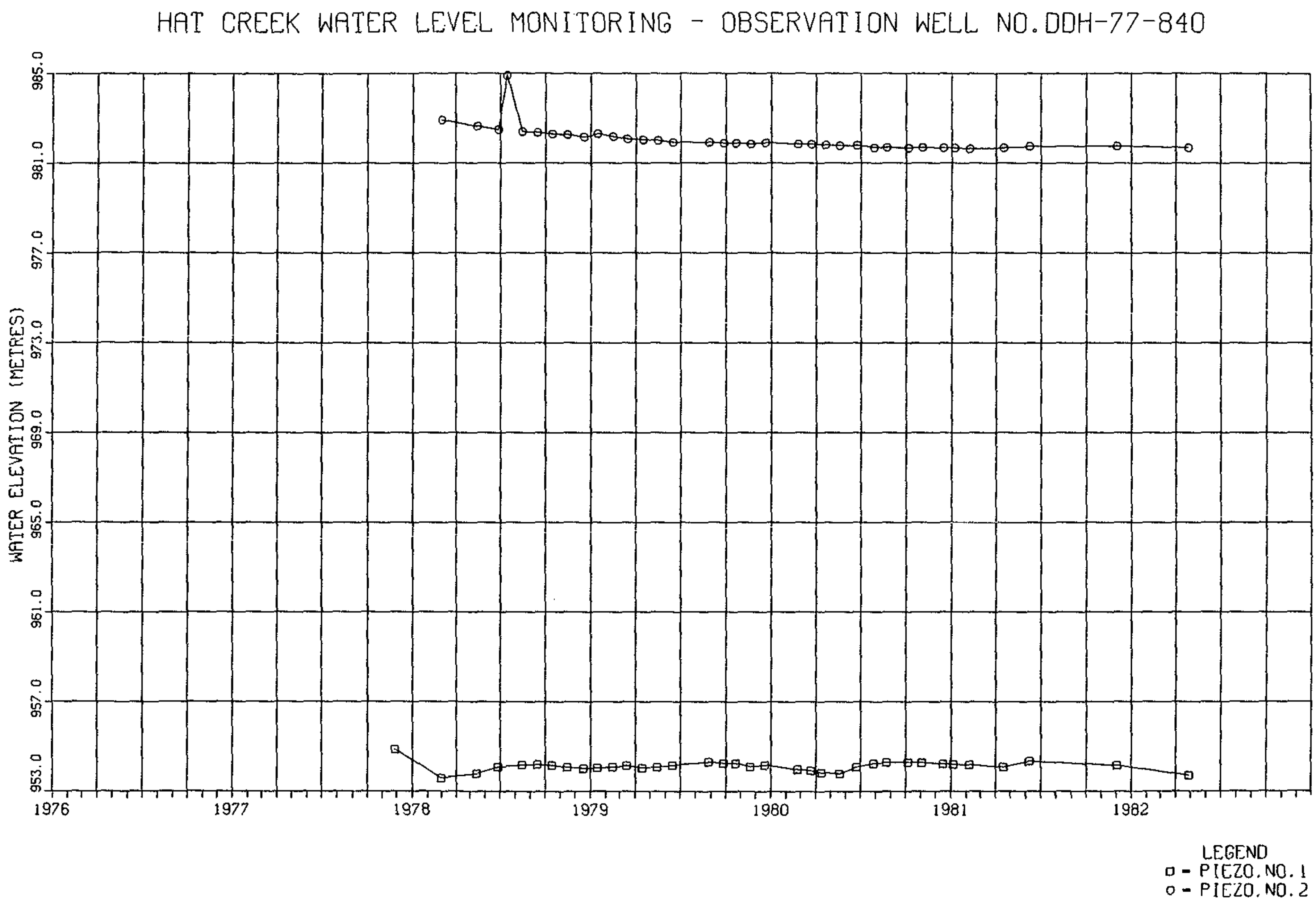


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-839

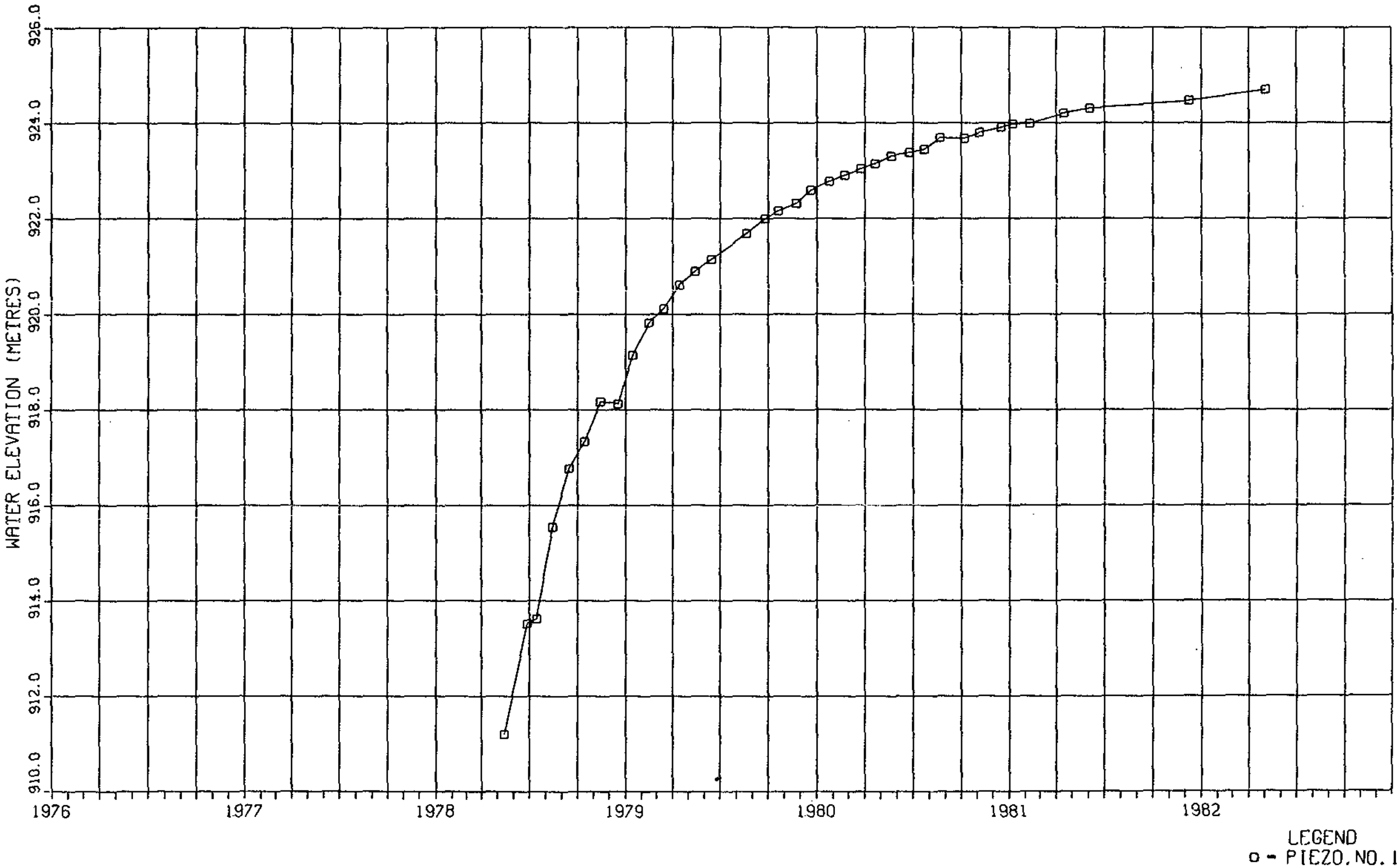


Golder Associates



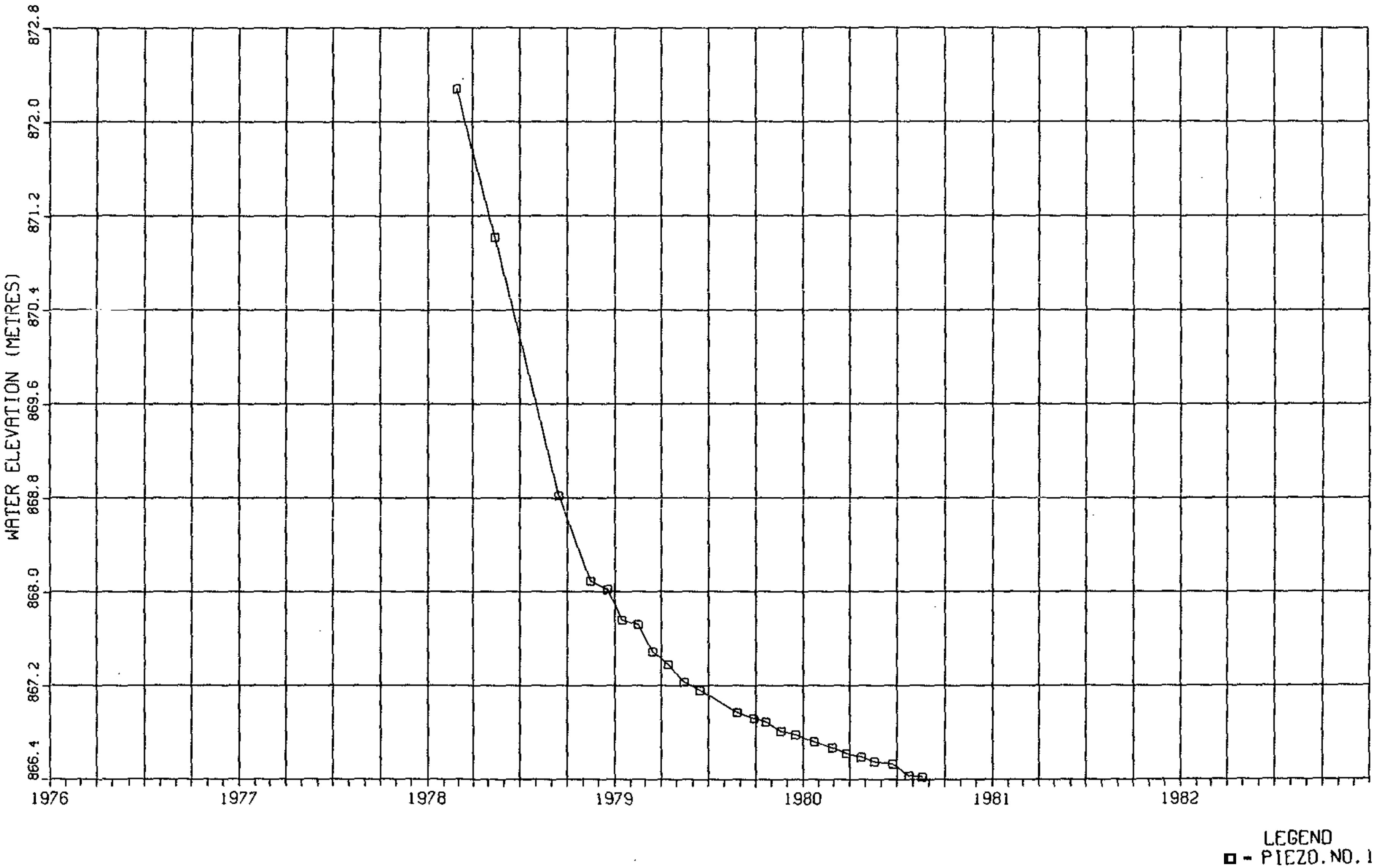
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-842



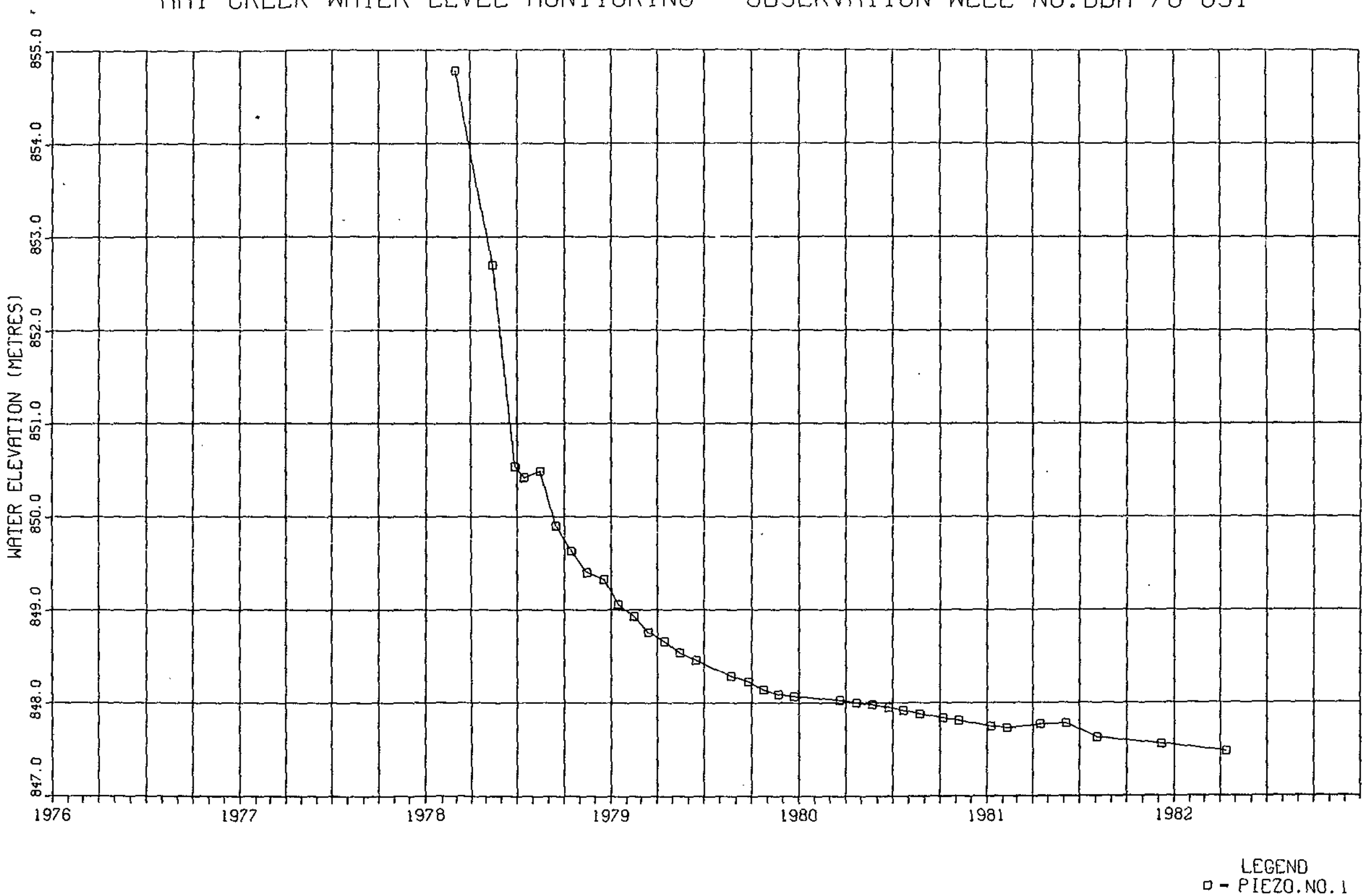
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-77-846



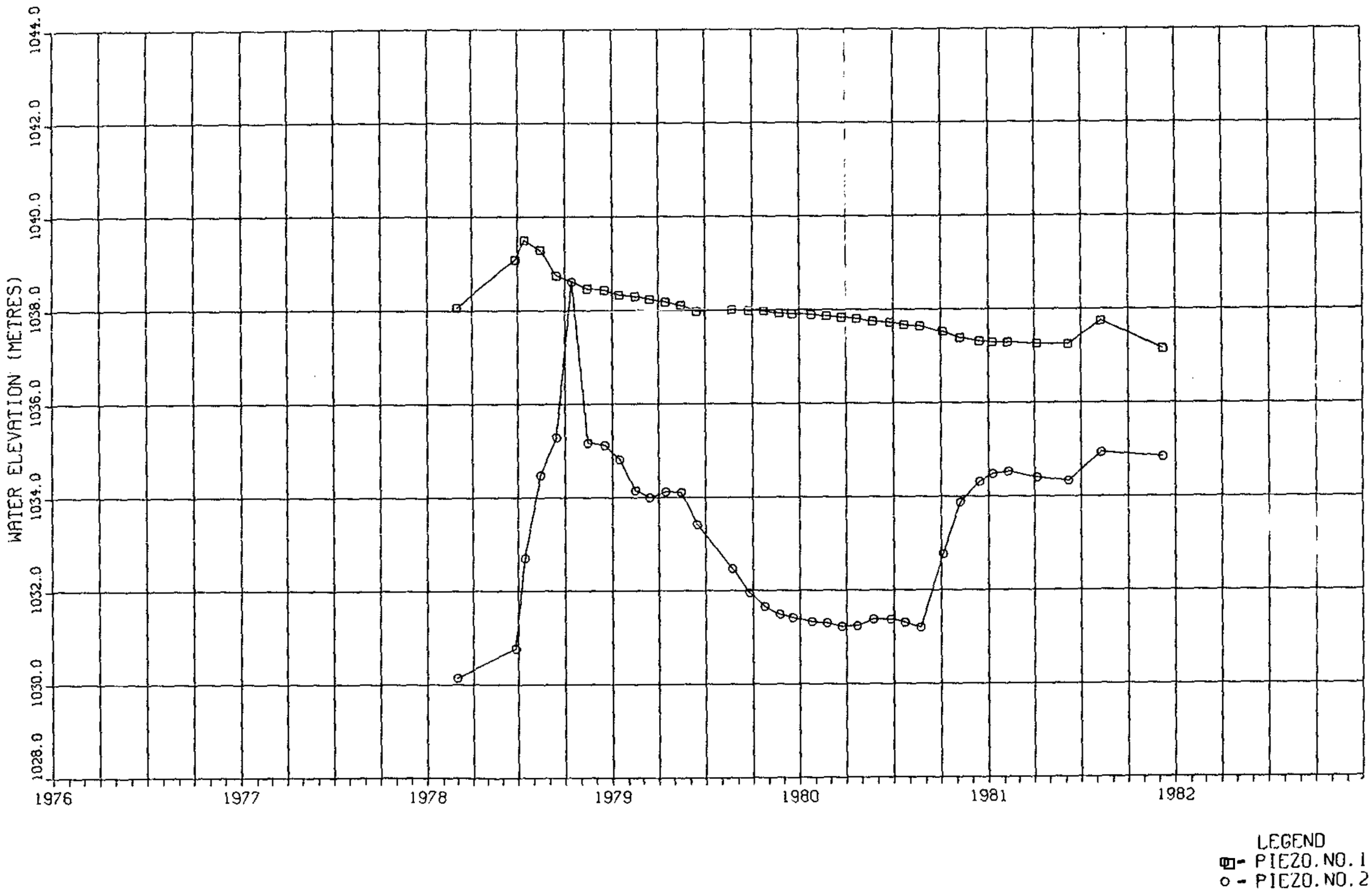
Golder Associates

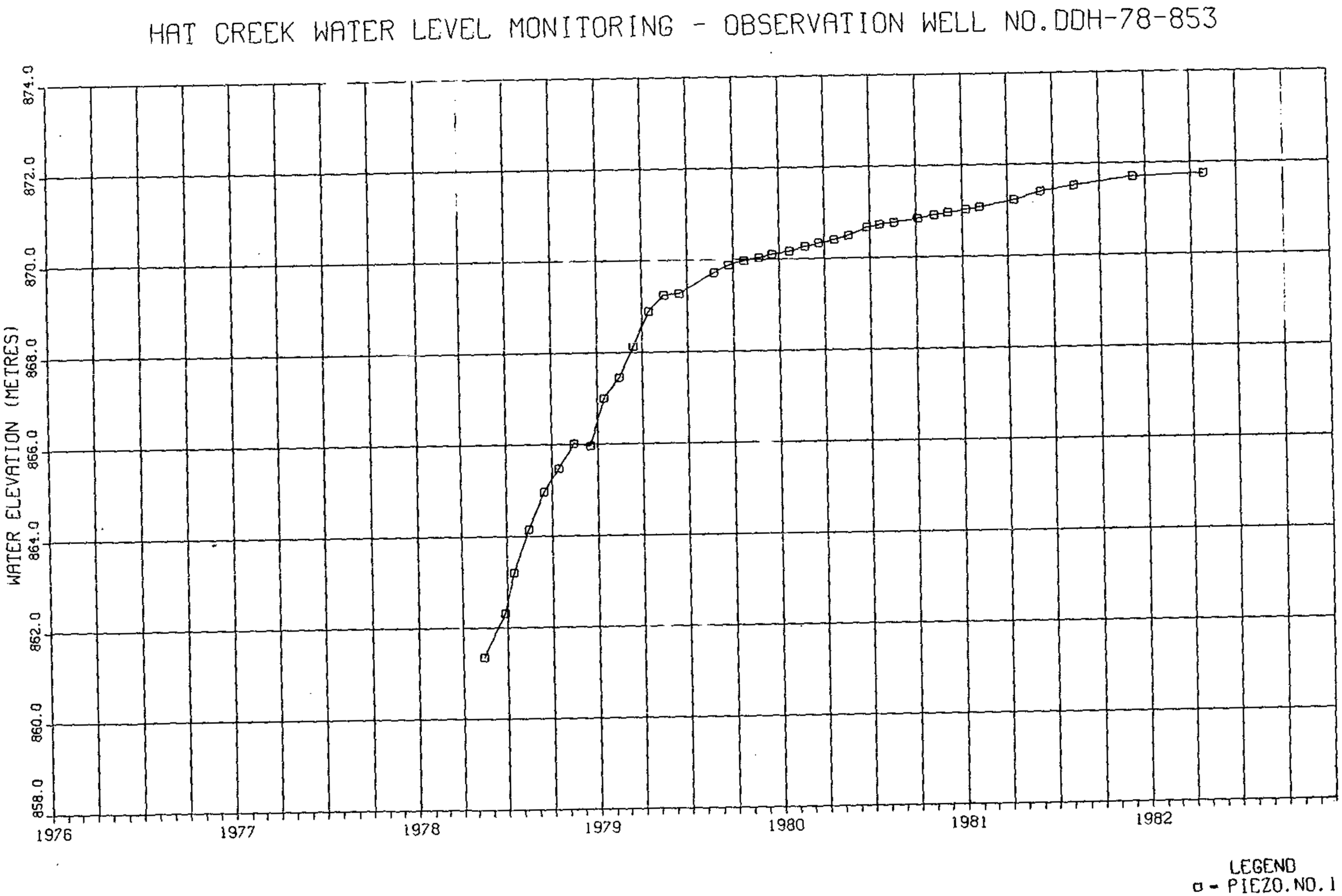
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-851



Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-852

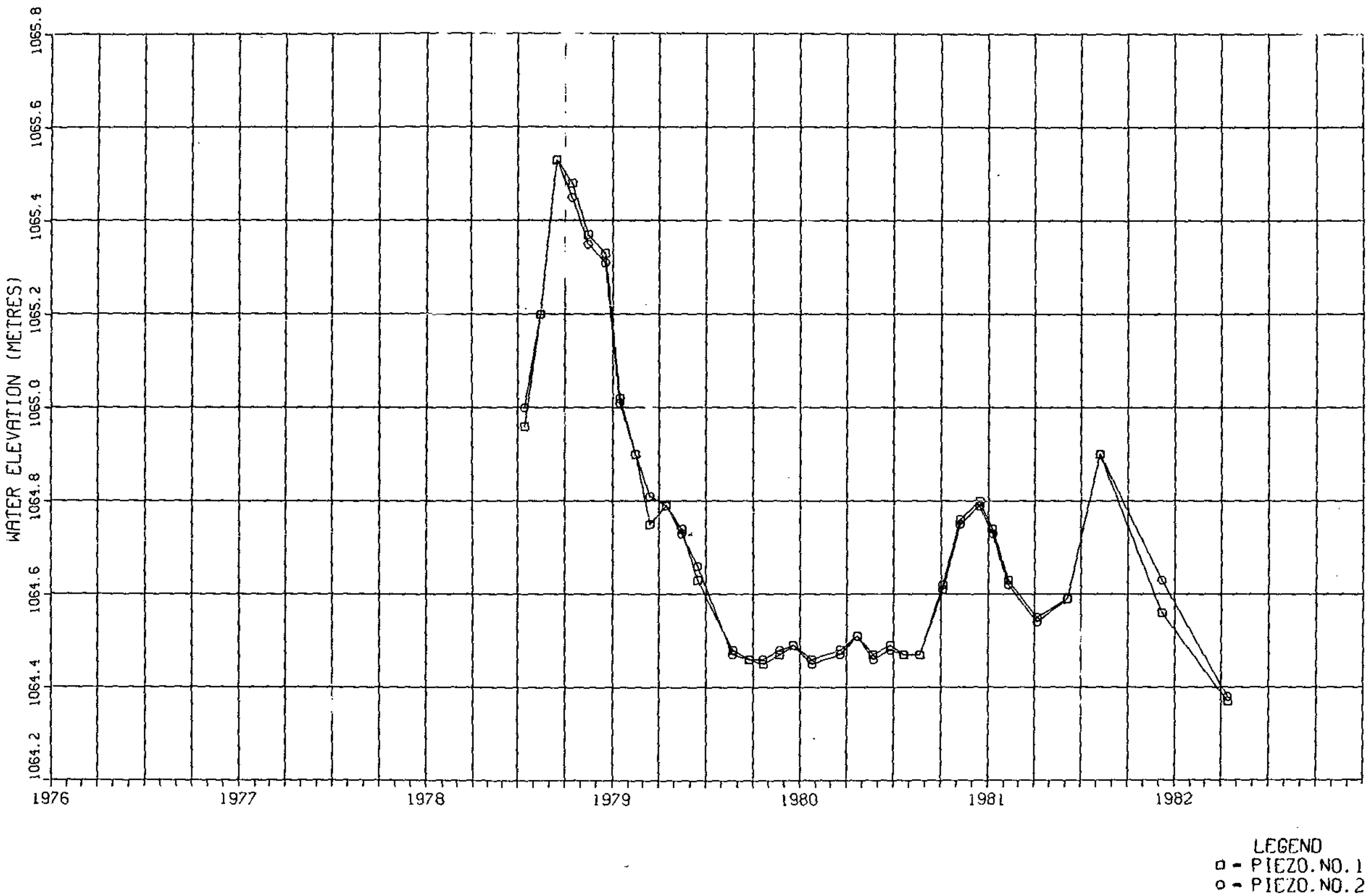




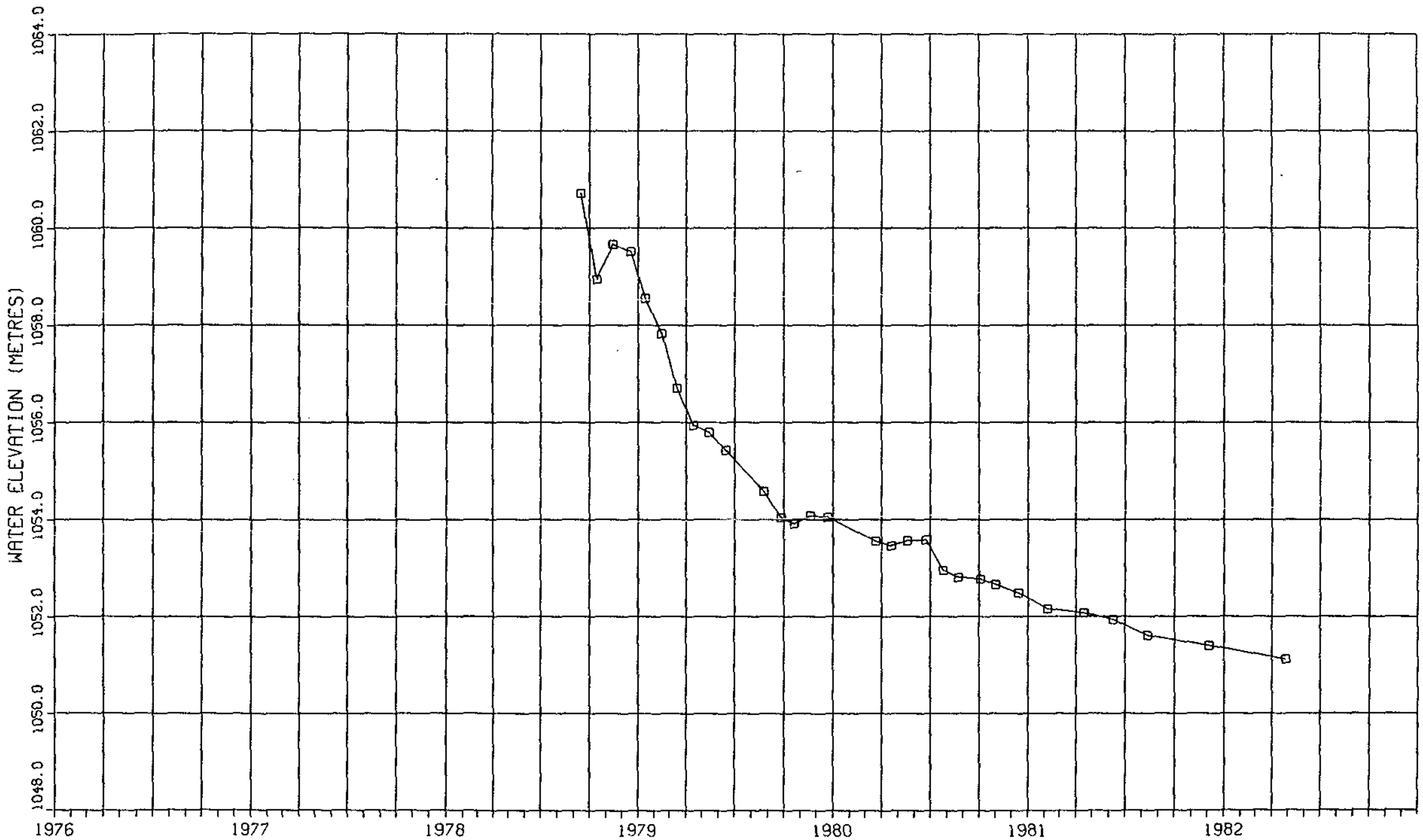
Golder Associates

Golder Associates

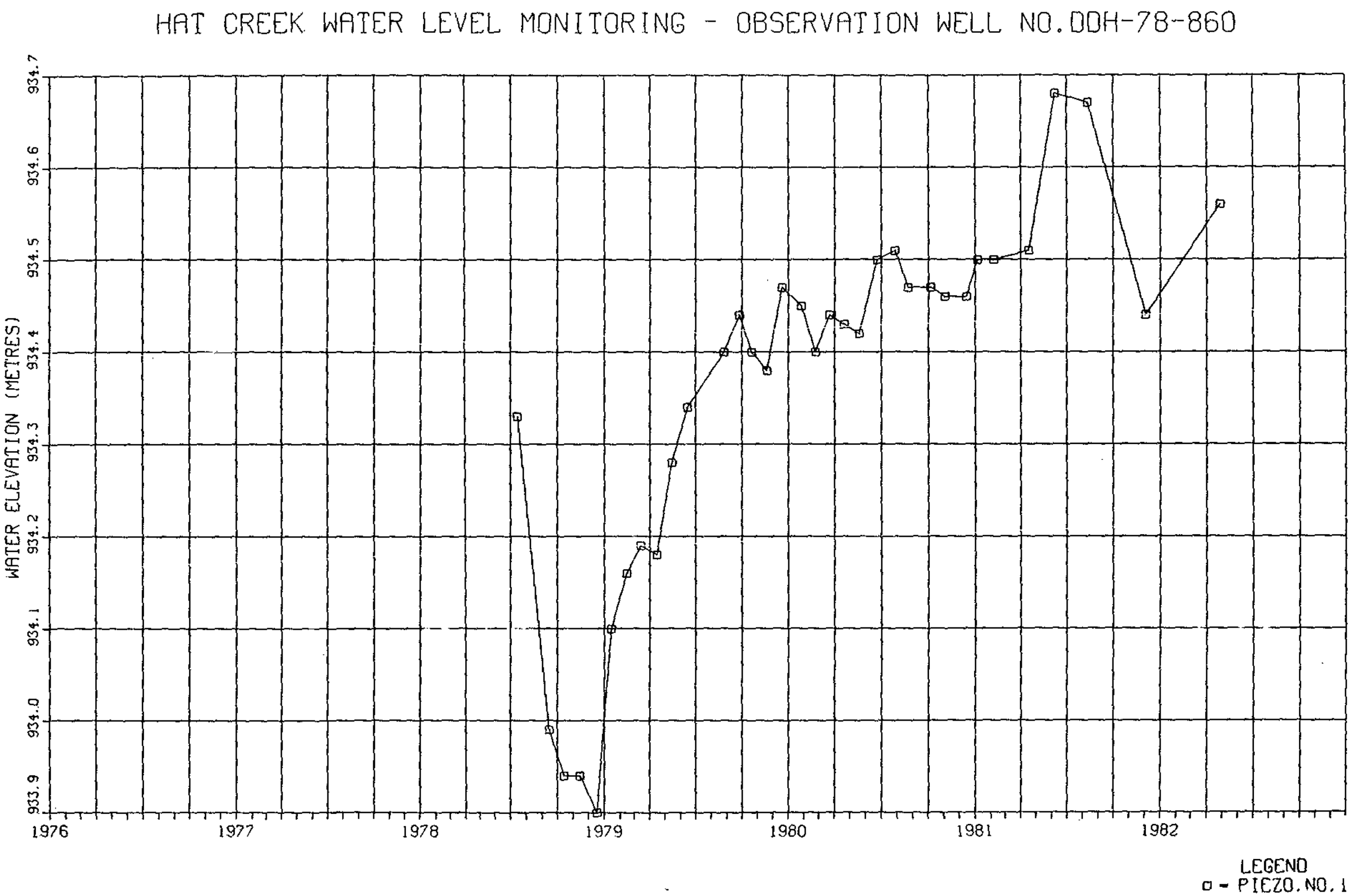
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-857

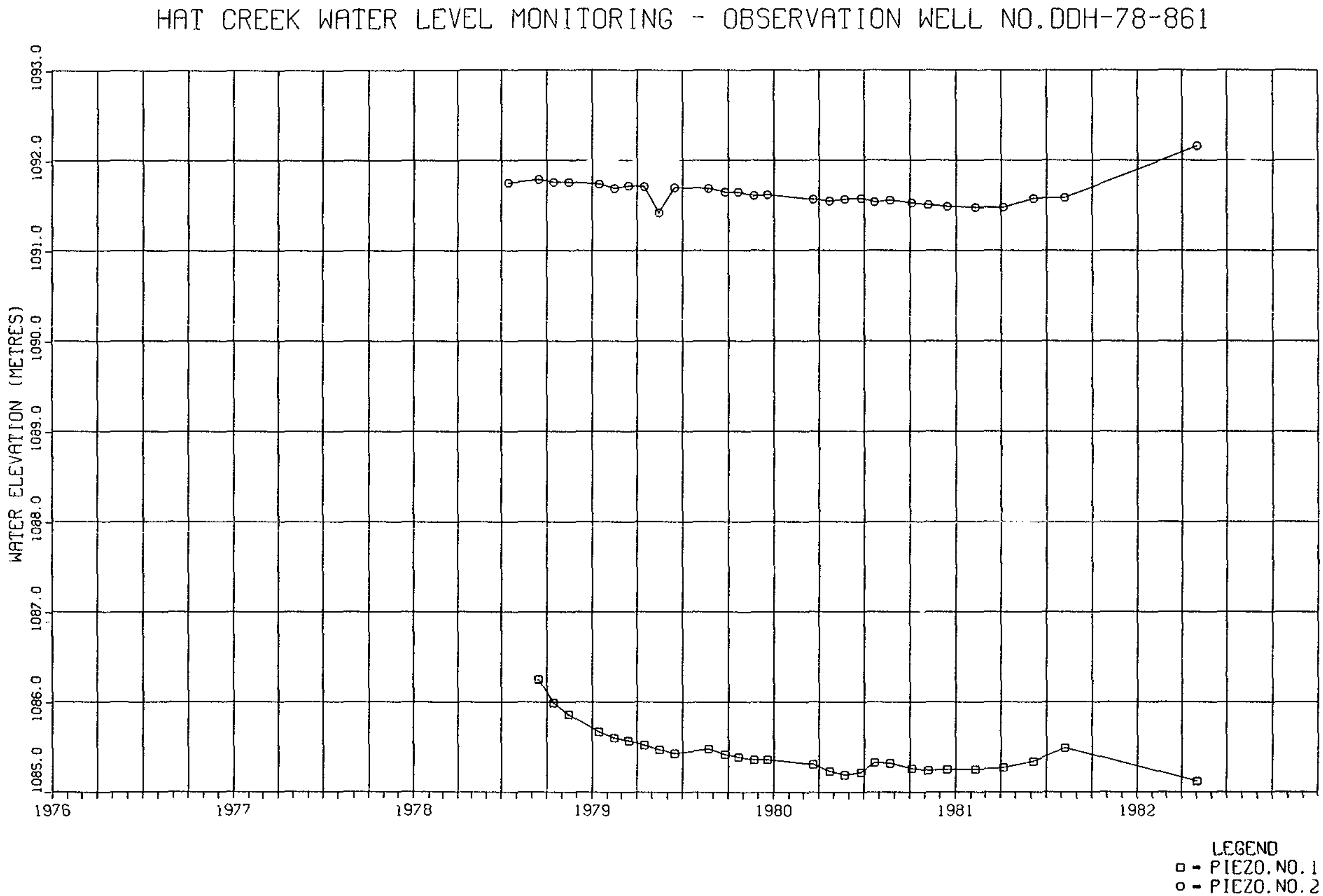


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-858



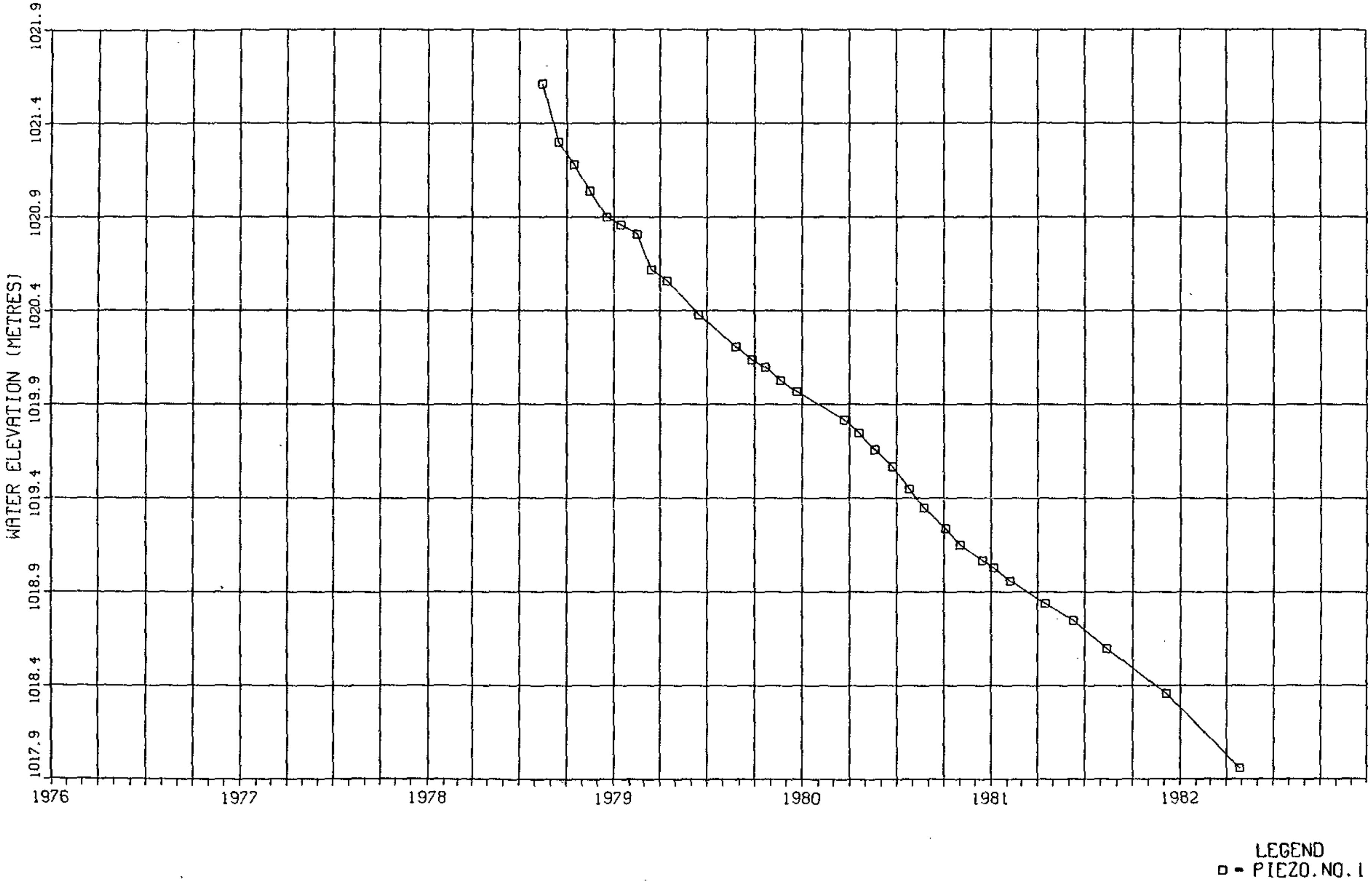
Golder Associates


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-860
WATER ELEVATION (METRES)
934.7
934.6
934.5
934.4
934.3
934.2
934.1
934.0
933.9
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

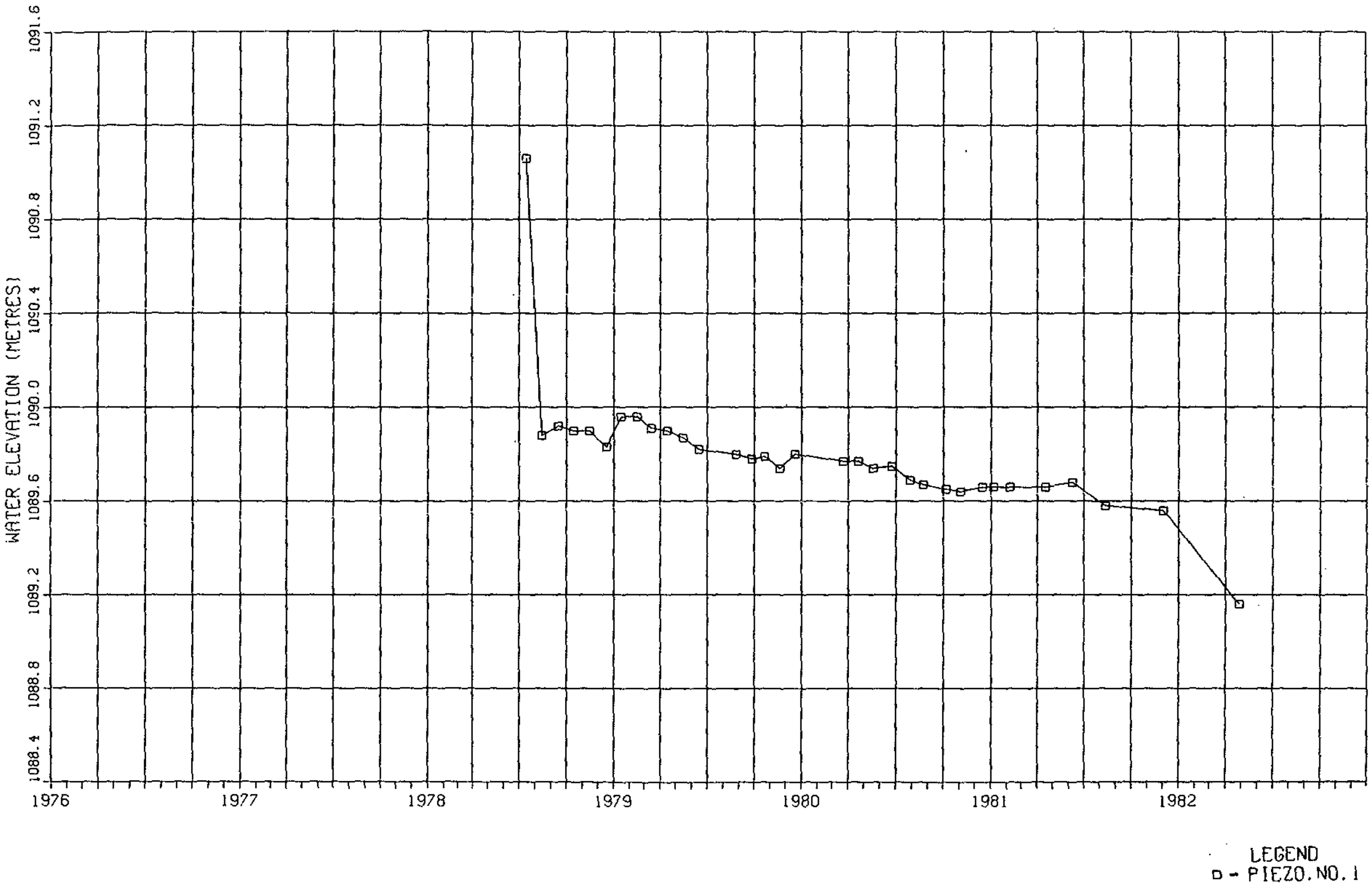


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-862

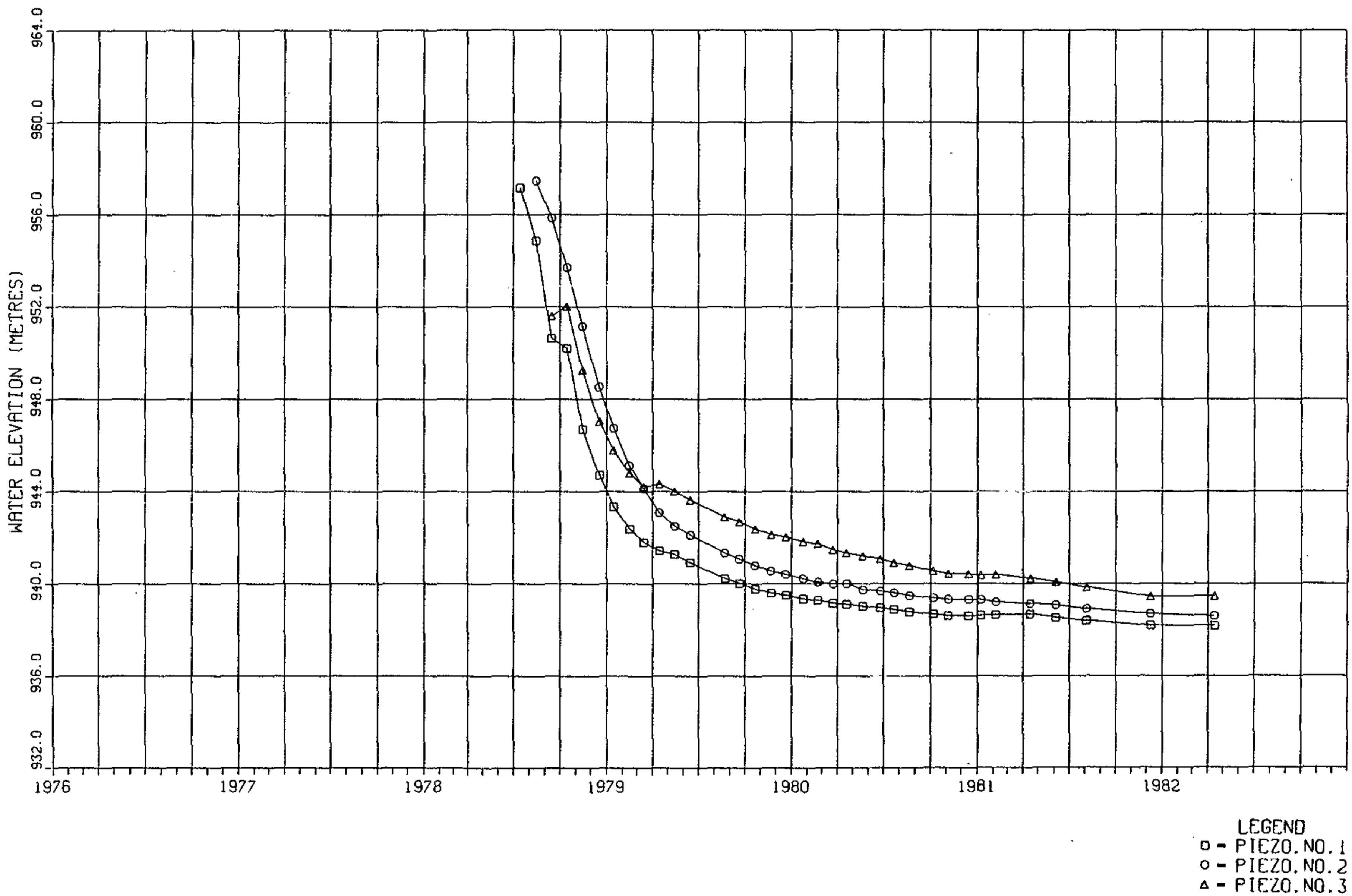


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-863

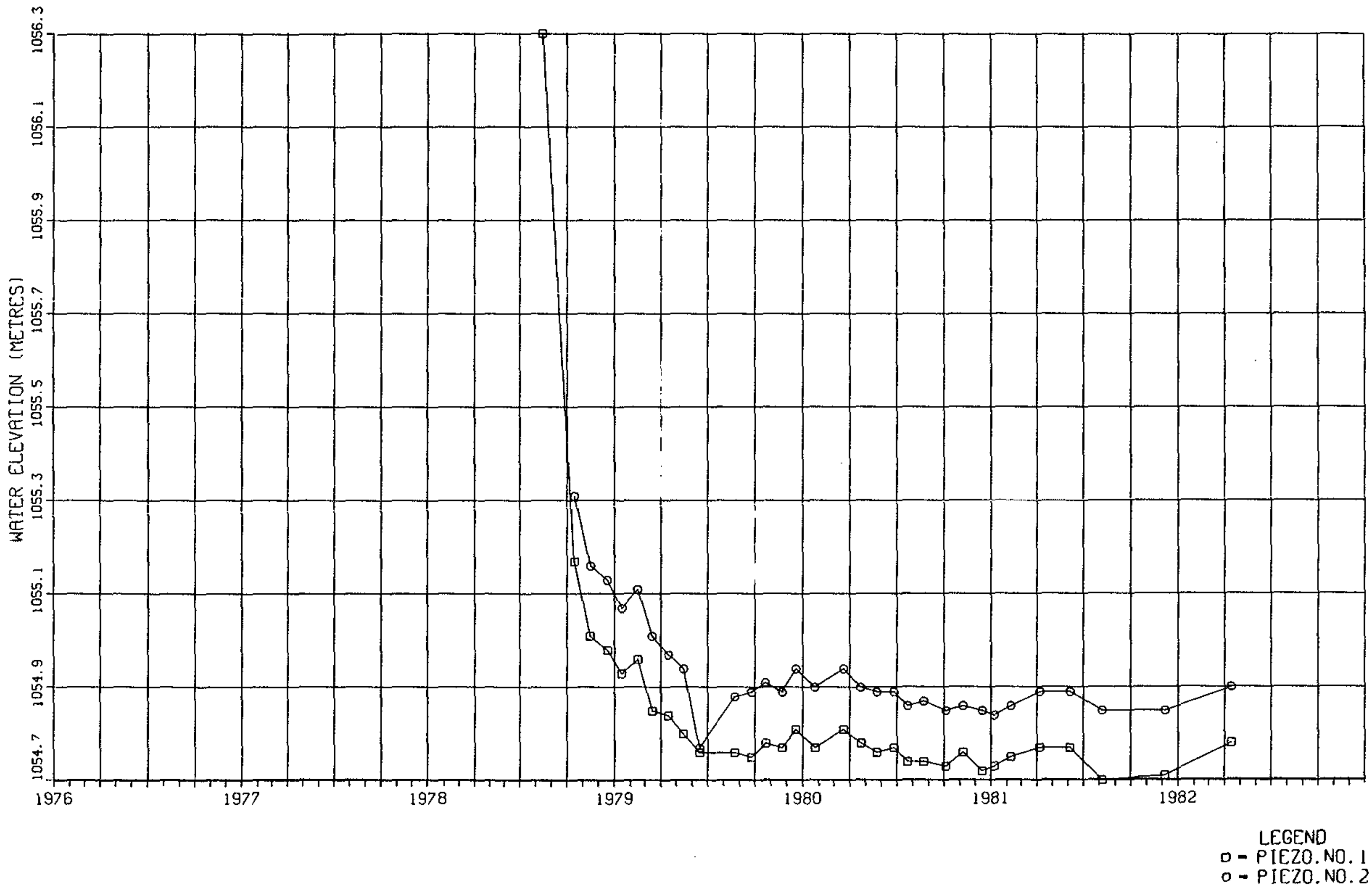


Golder Associates

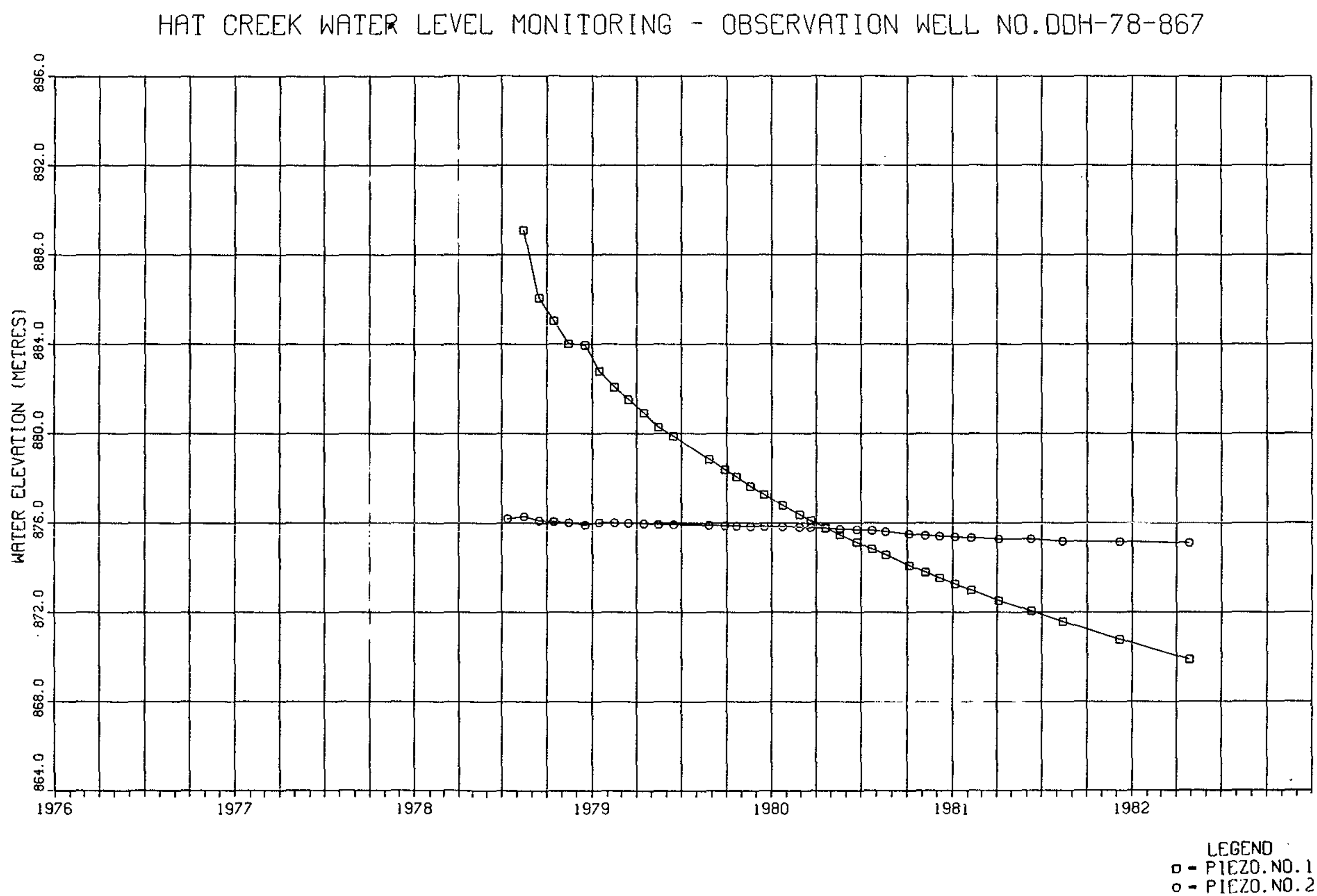
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-865



HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-866

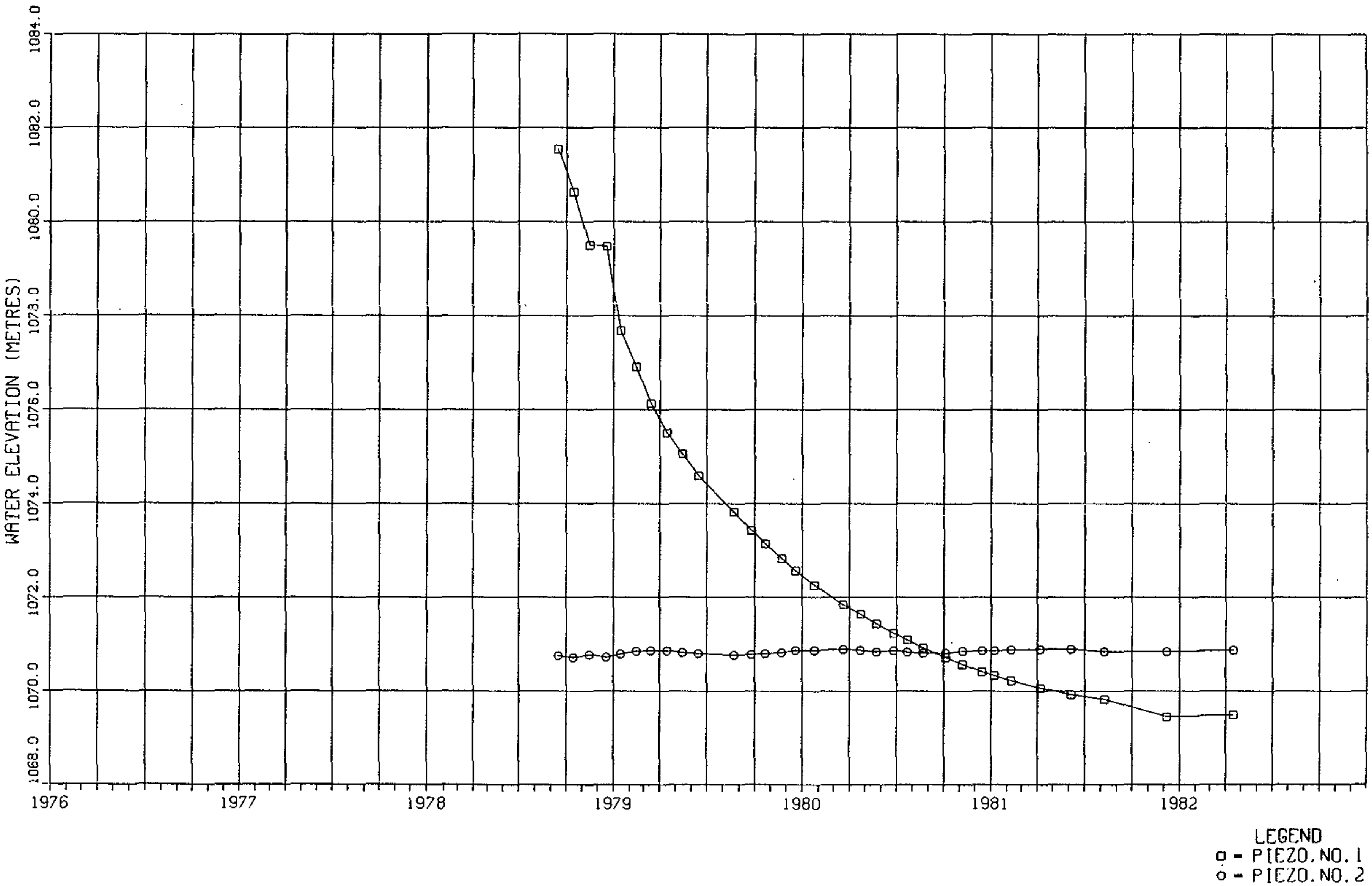


Golder Associates



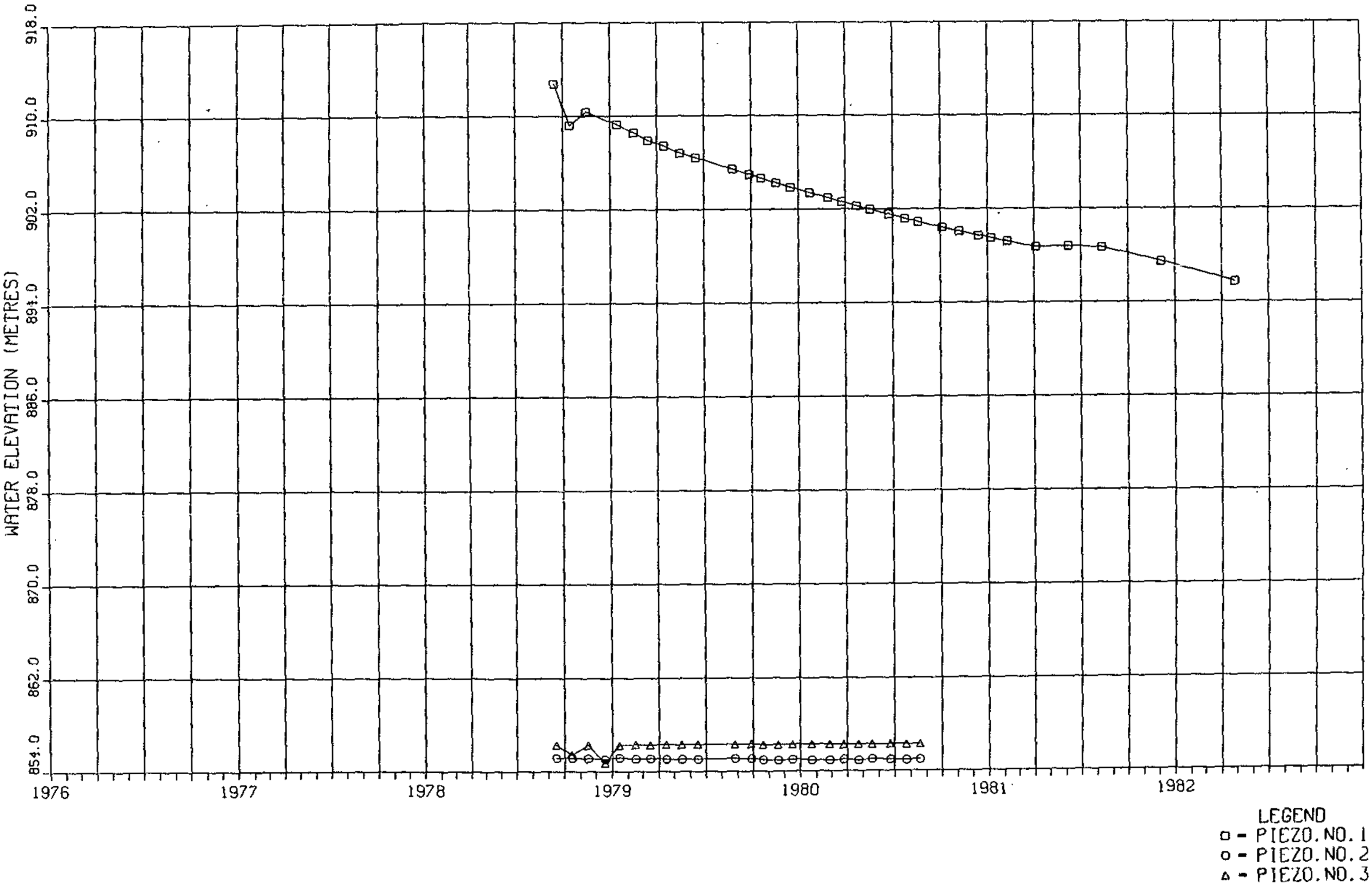
Golder Associates

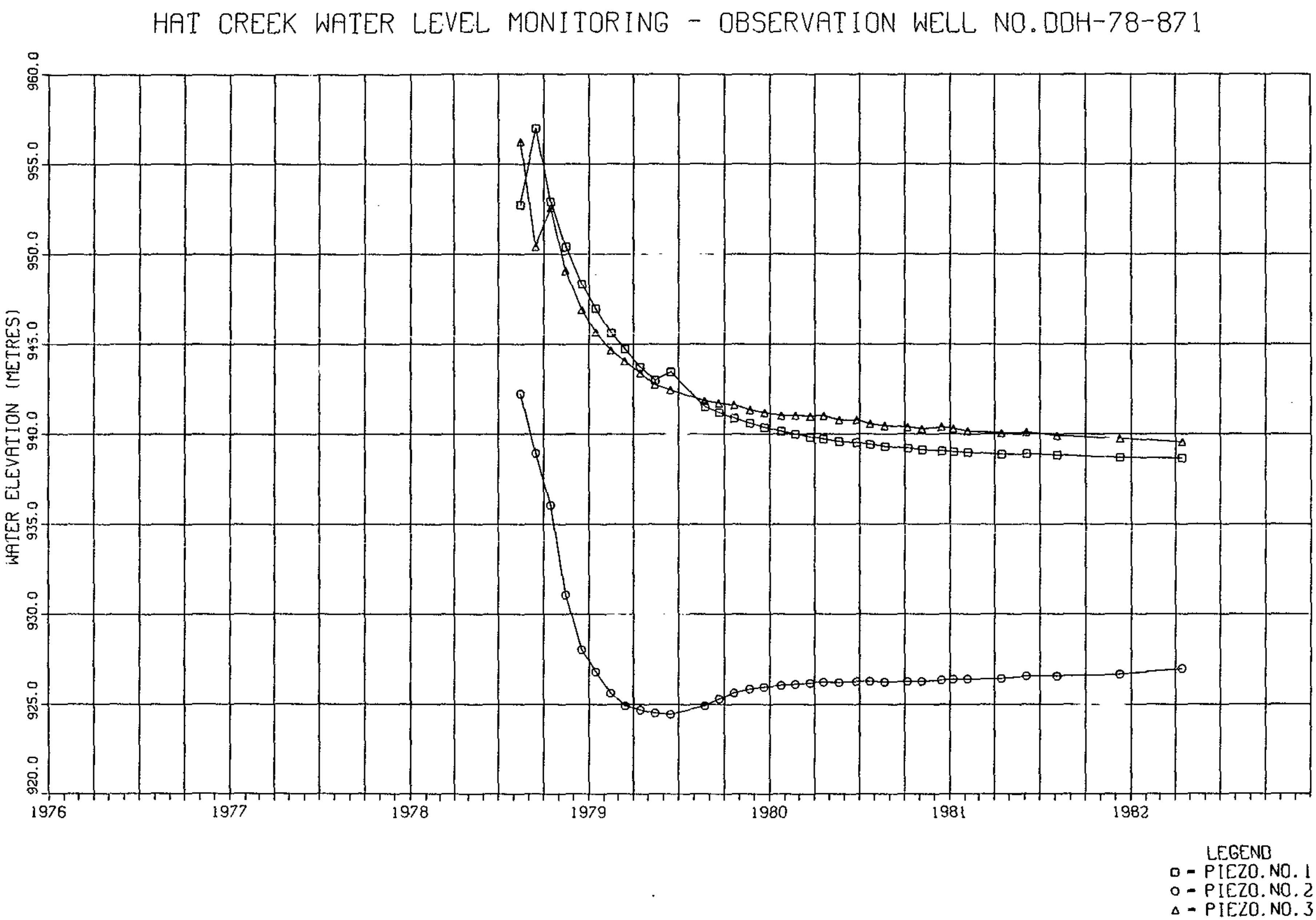
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-868



Golder Associates

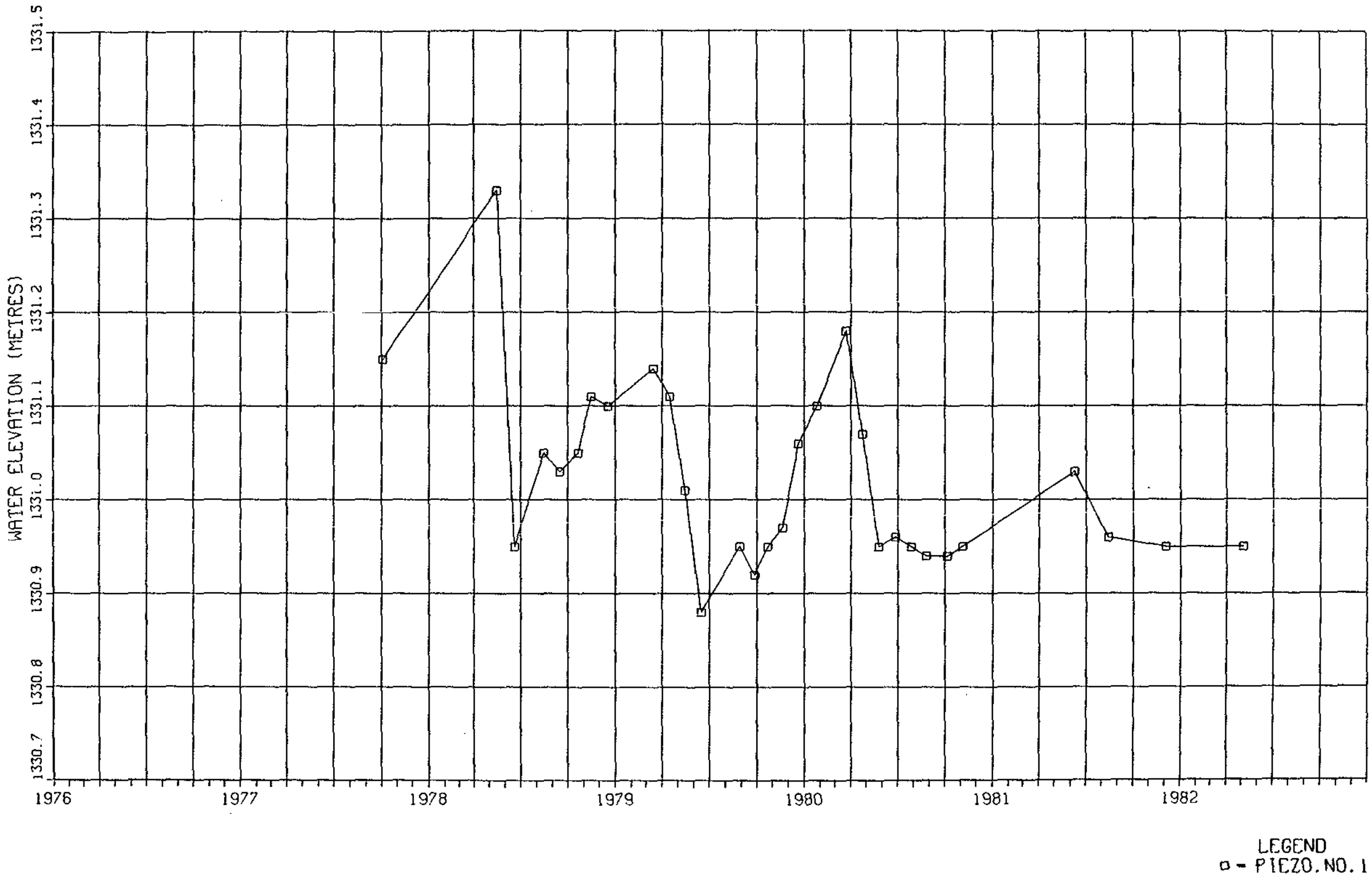
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.DDH-78-870

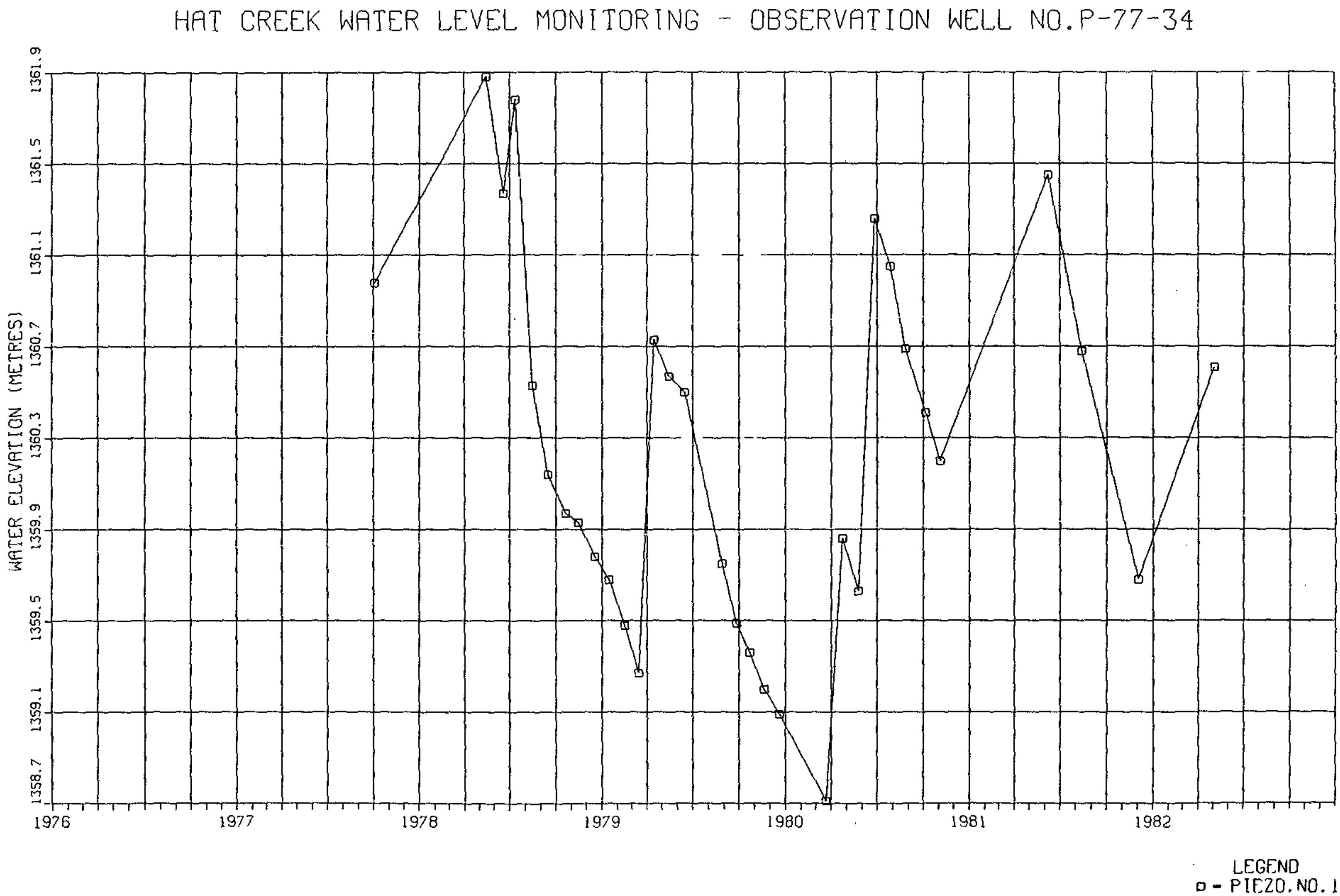




Golder Associates

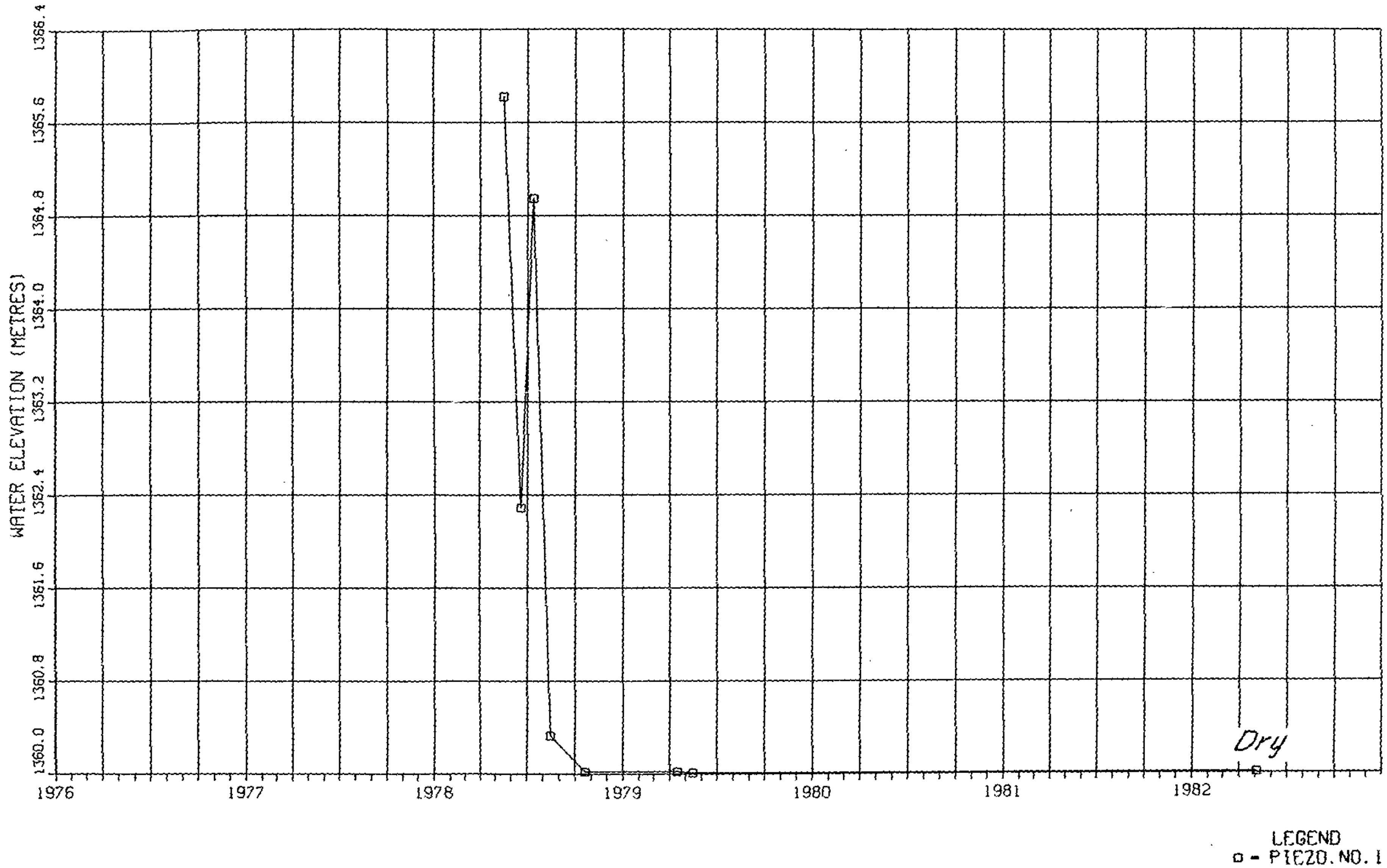
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-32





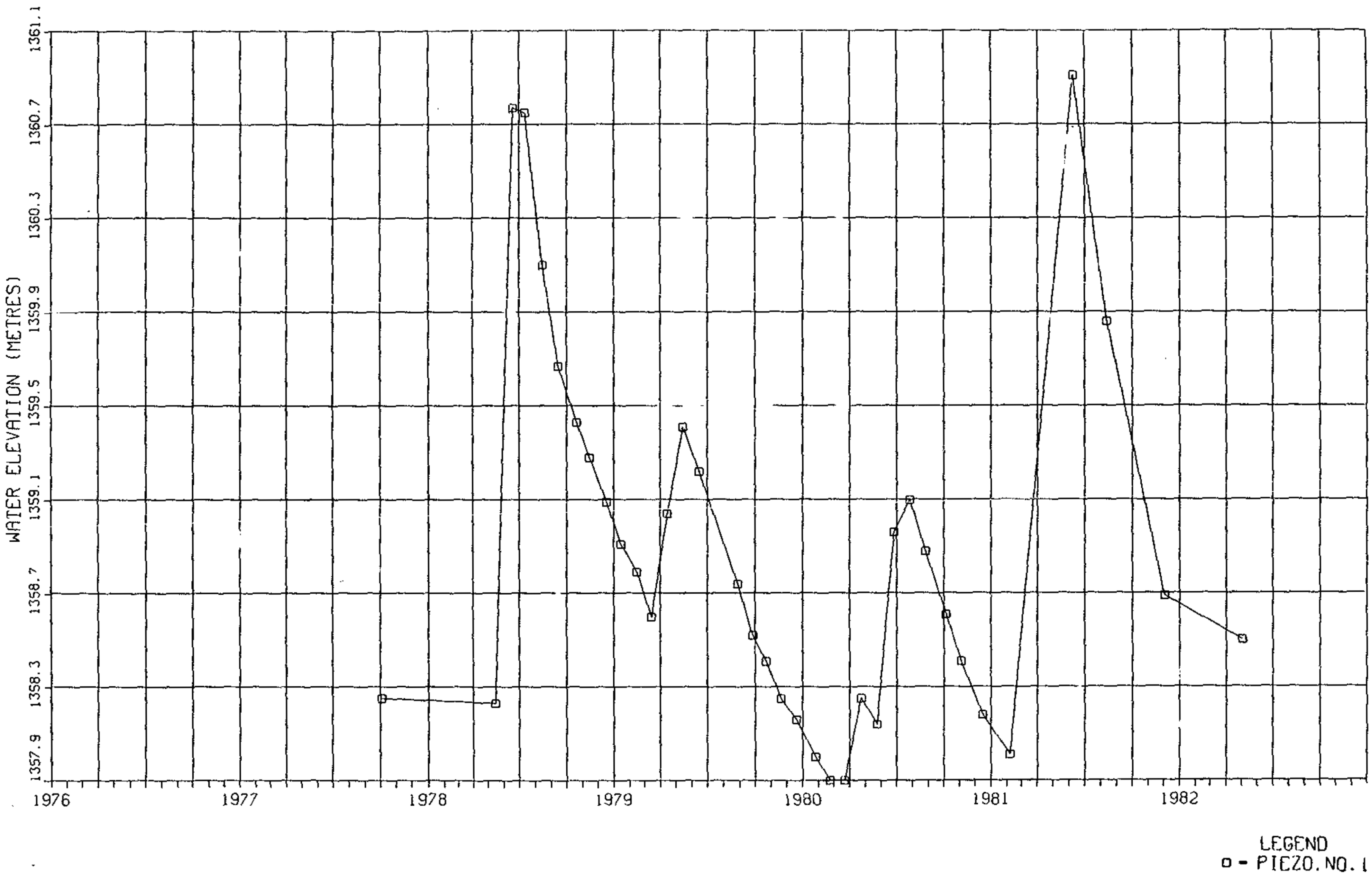
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-35

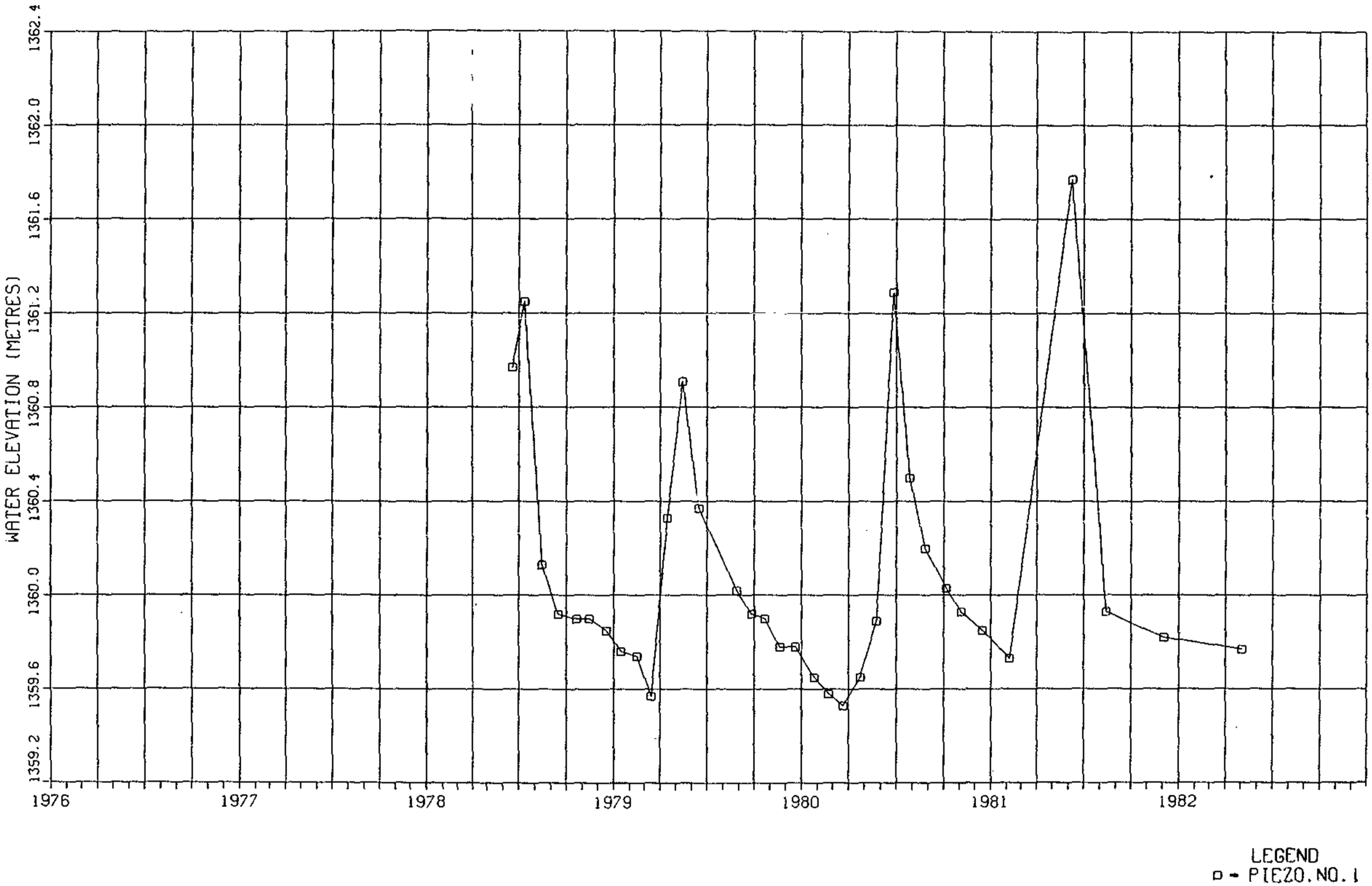


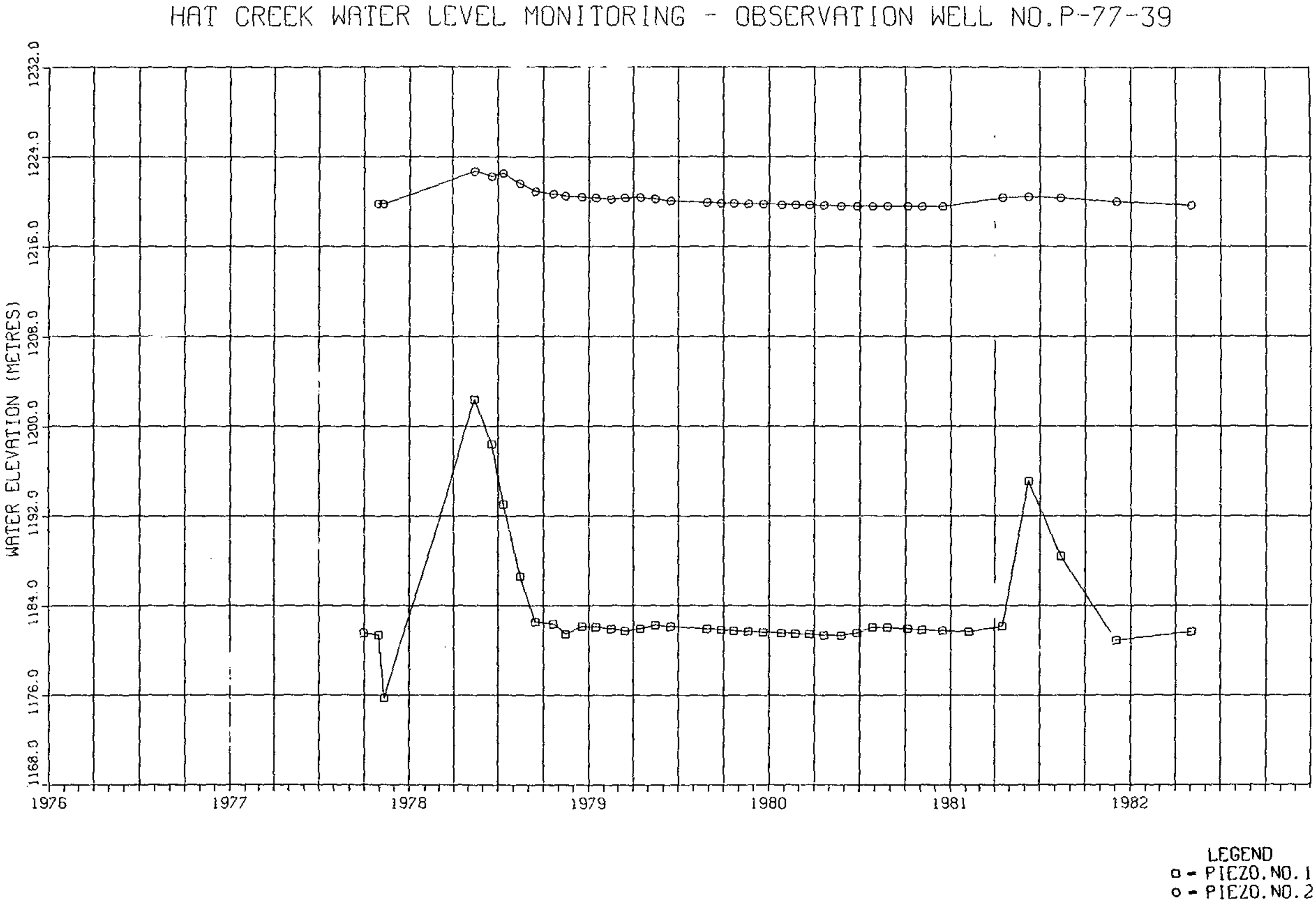
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-36

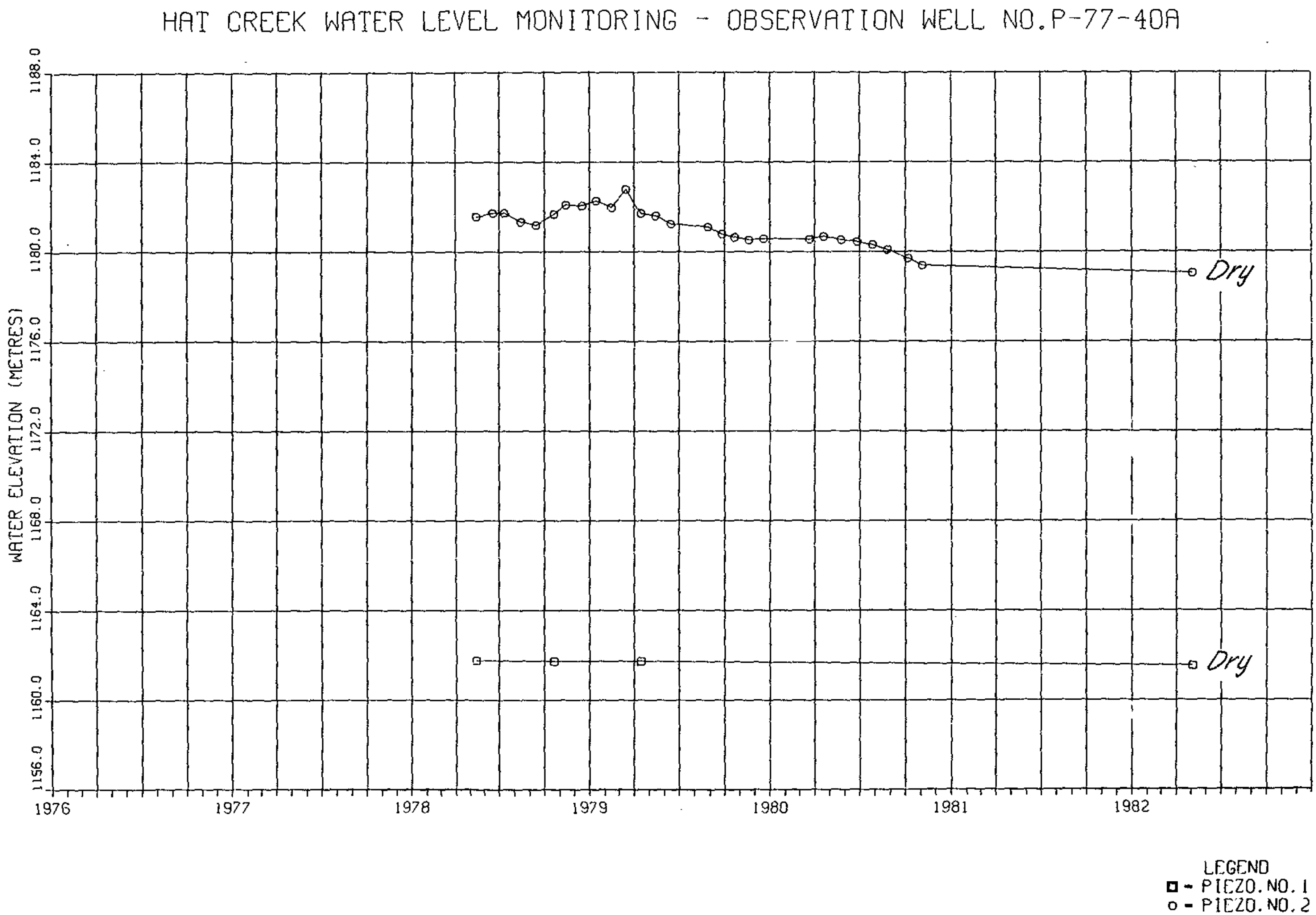


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-37



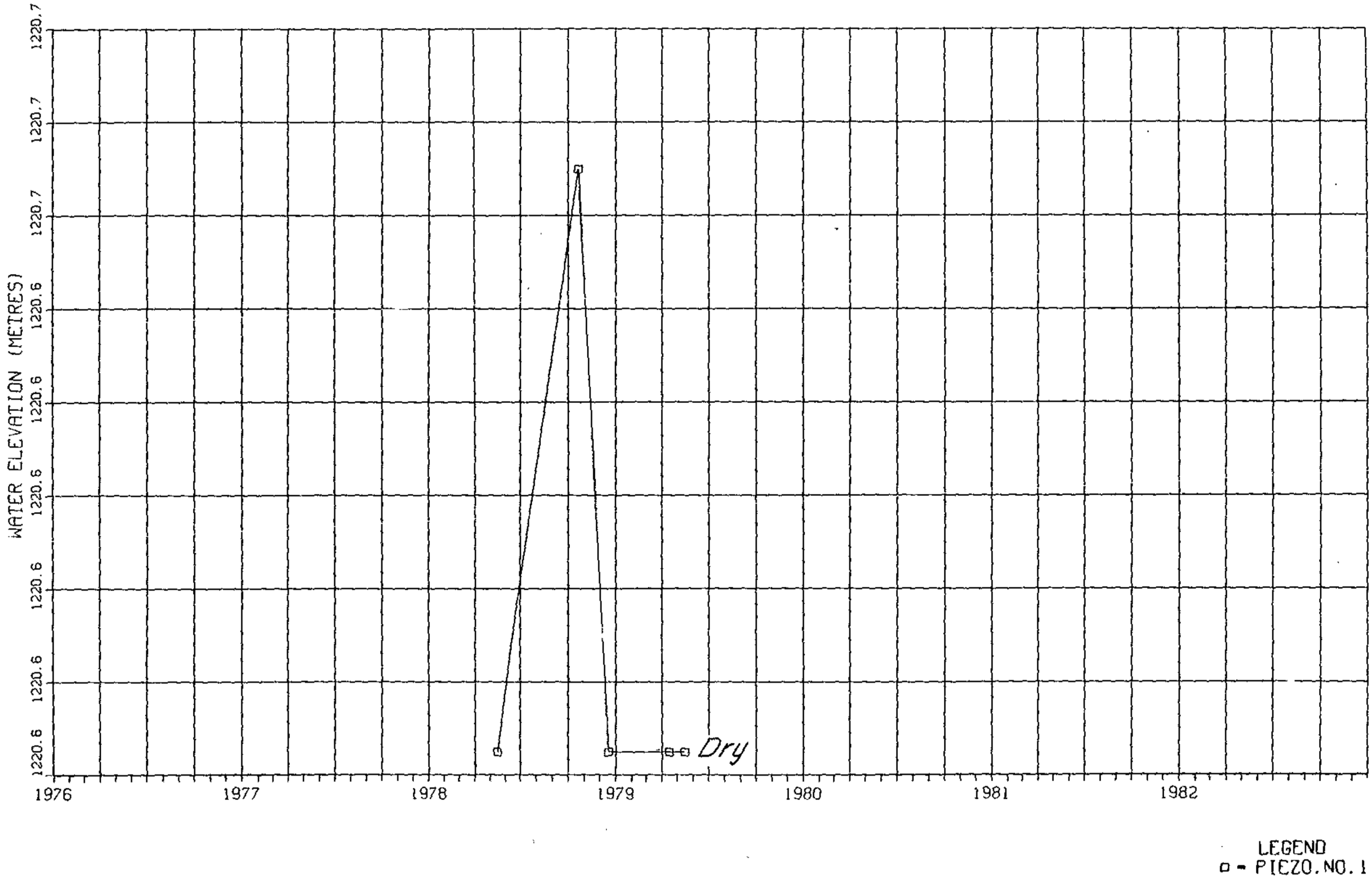


Golder Associates

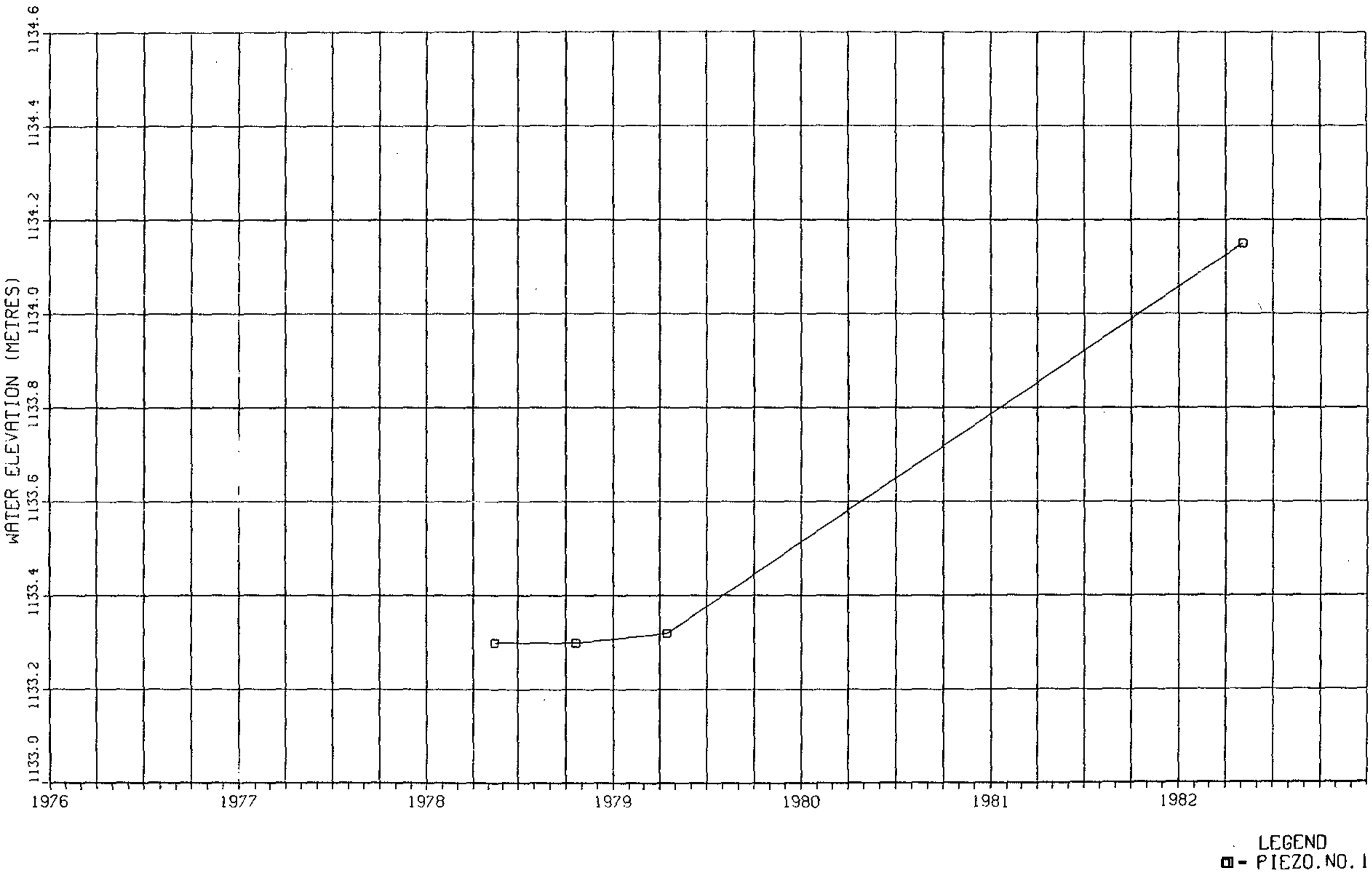


Golder Associates

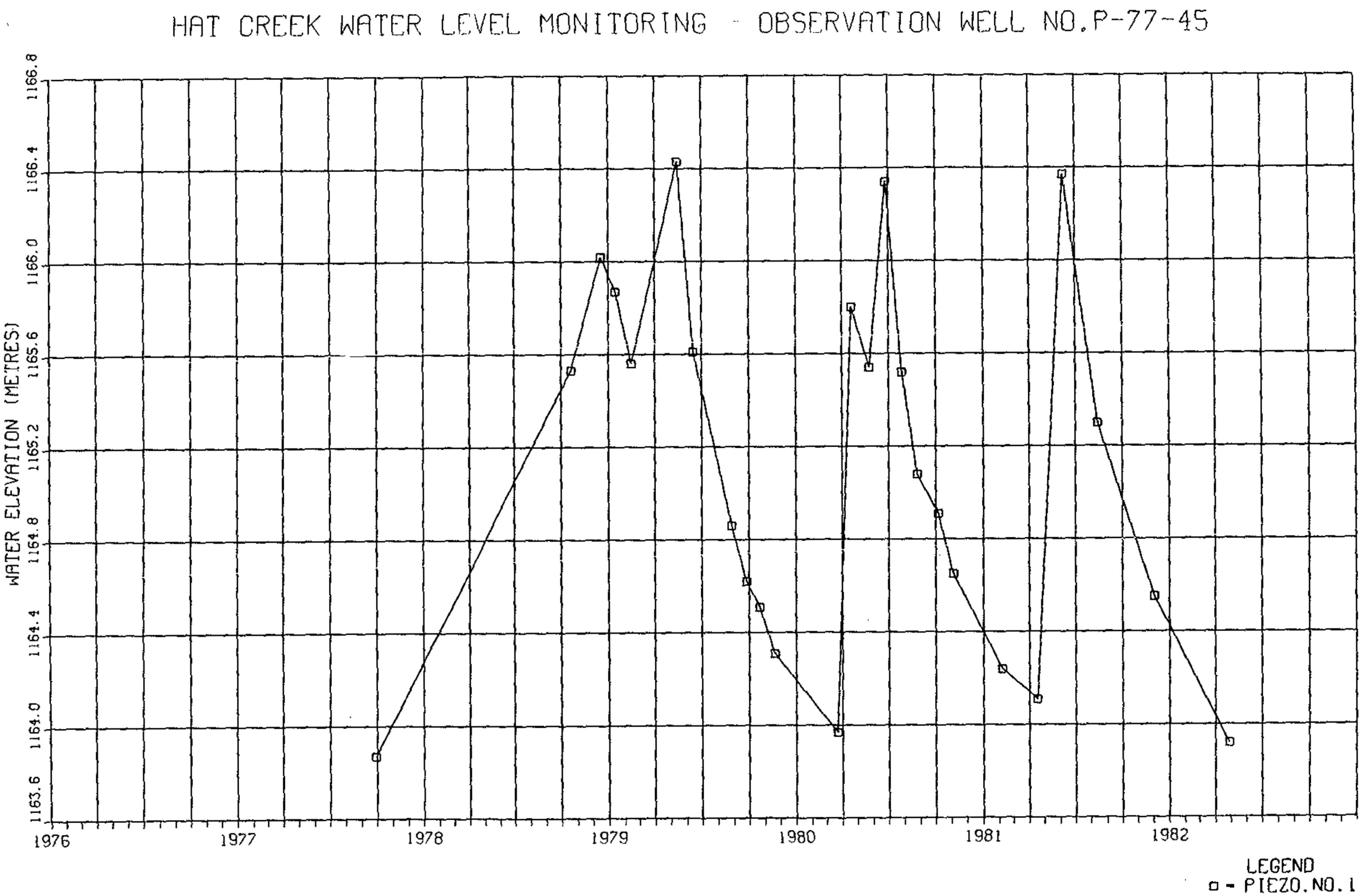
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-41



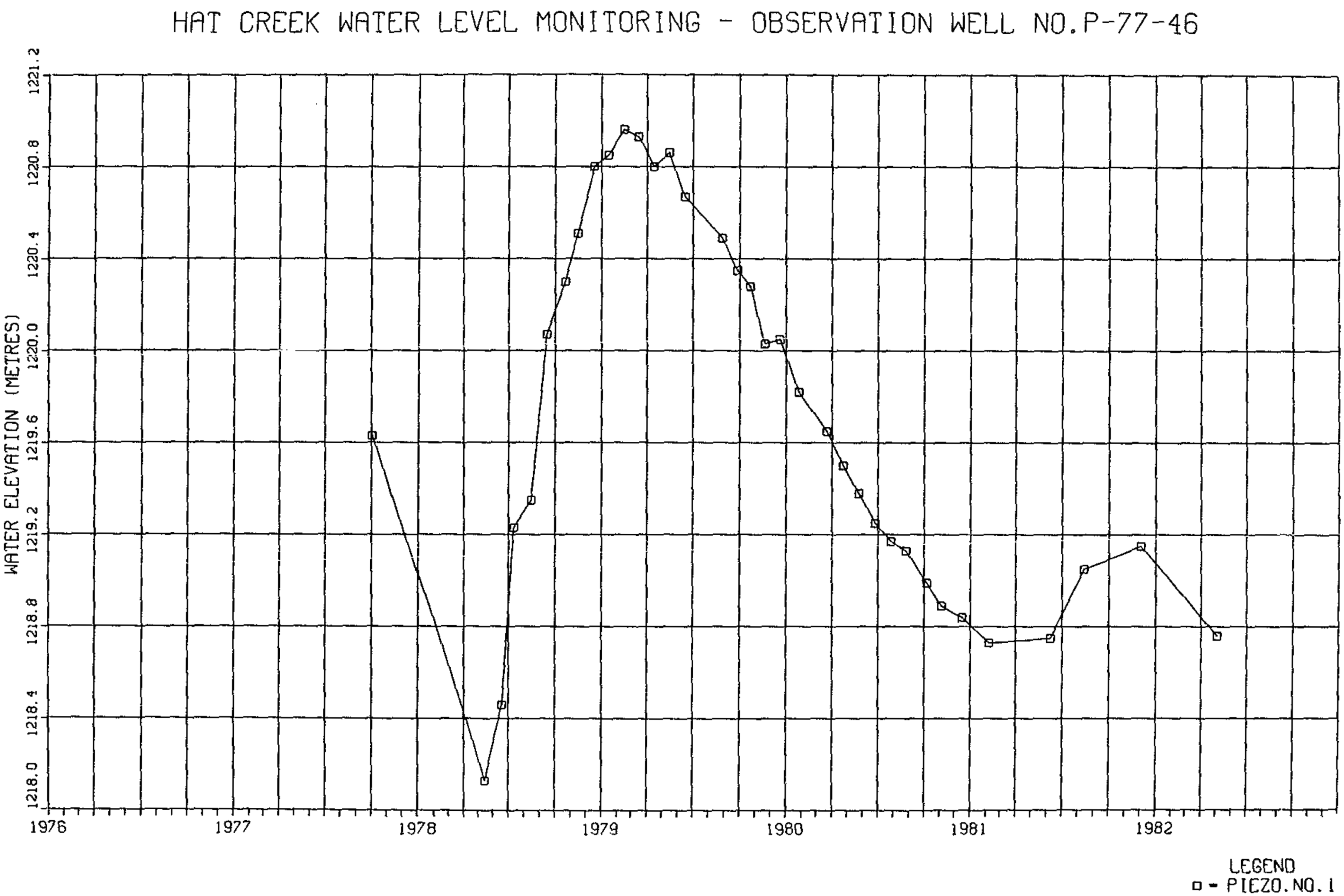
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-44



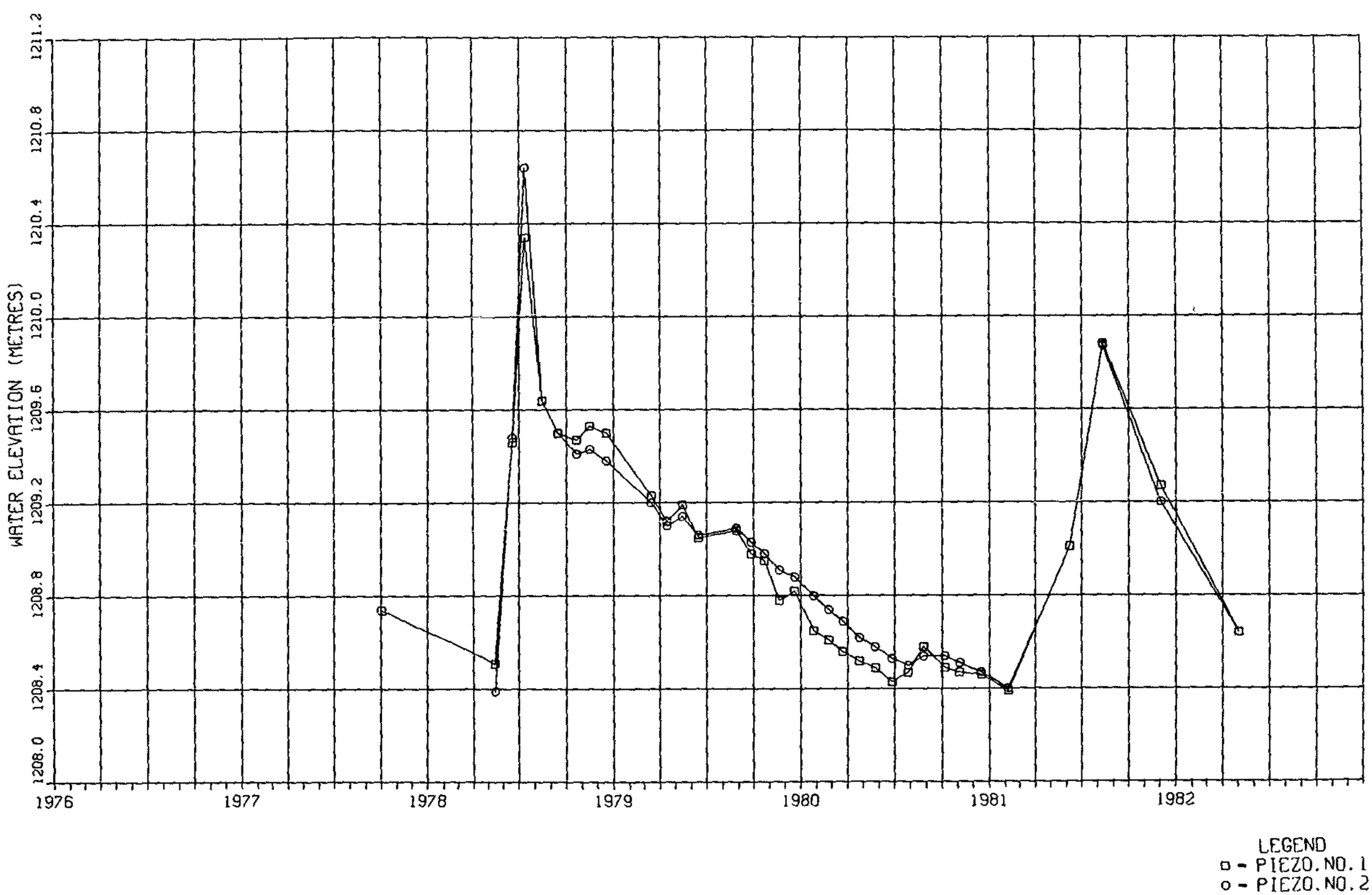
Golder Associates



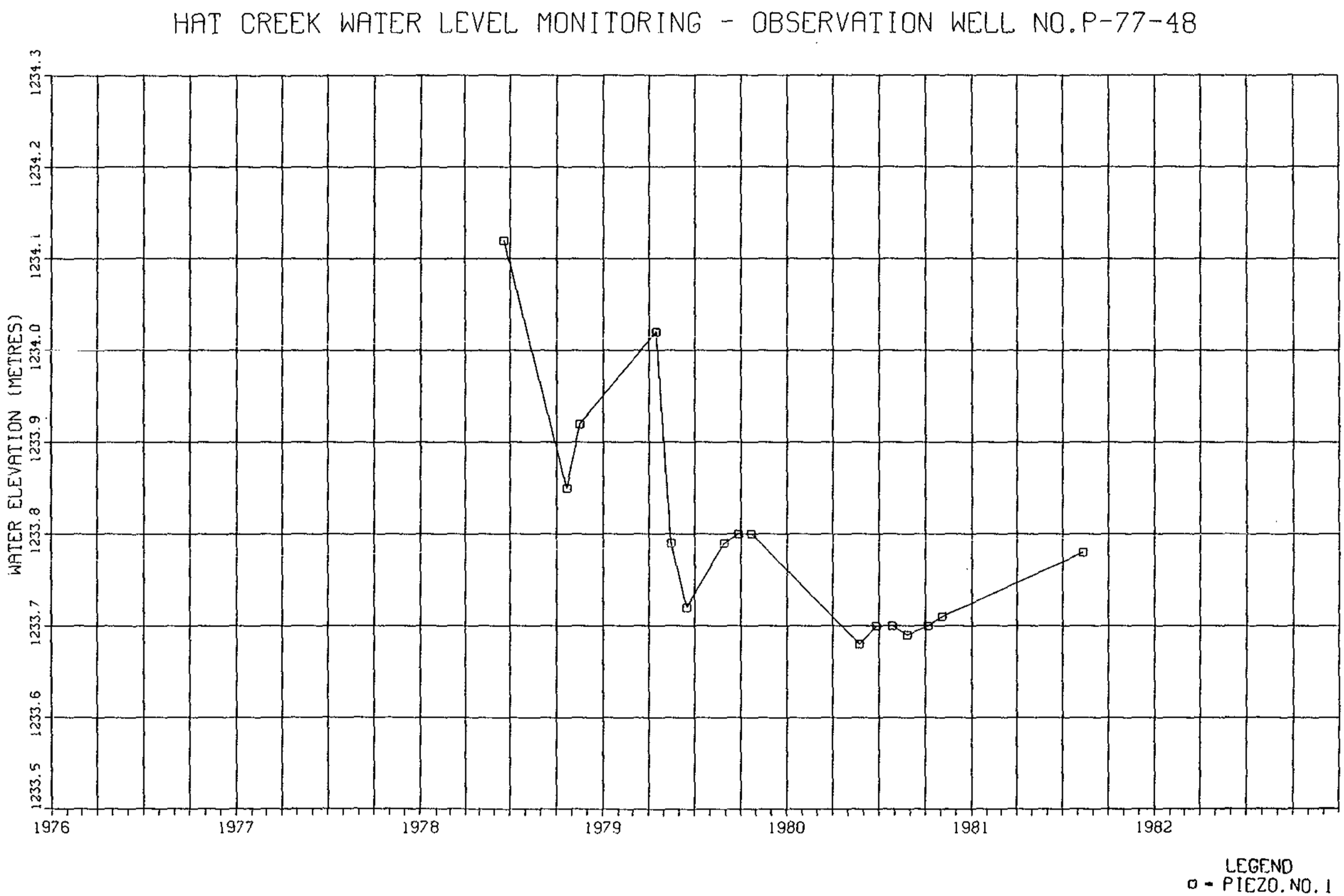
Golder Associates


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-46
WATER ELEVATION (METRES)
1221.2
1220.8
1220.4
1220.0
1219.6
1219.2
1218.8
1218.4
1218.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-47

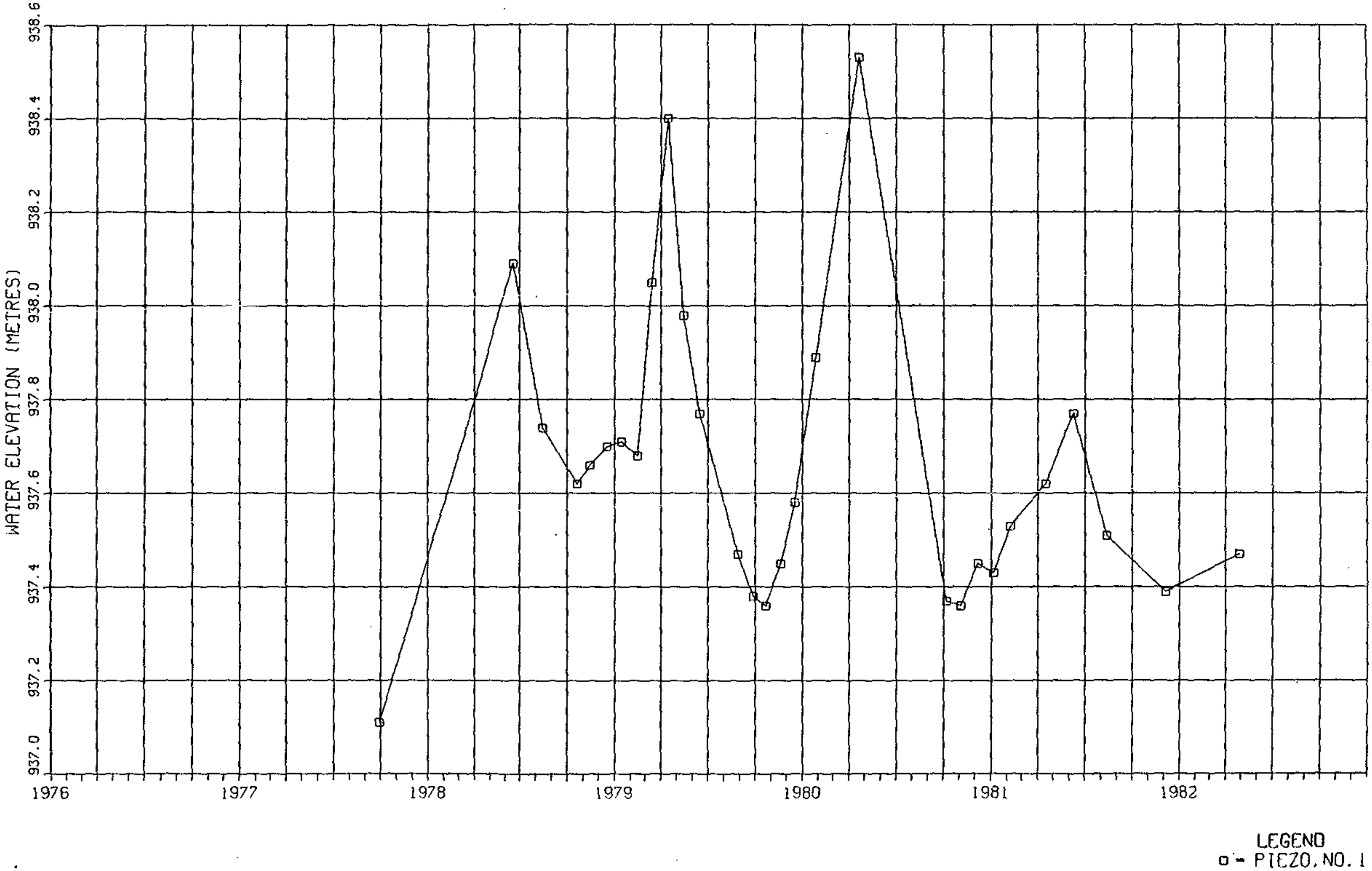


Golder Associates

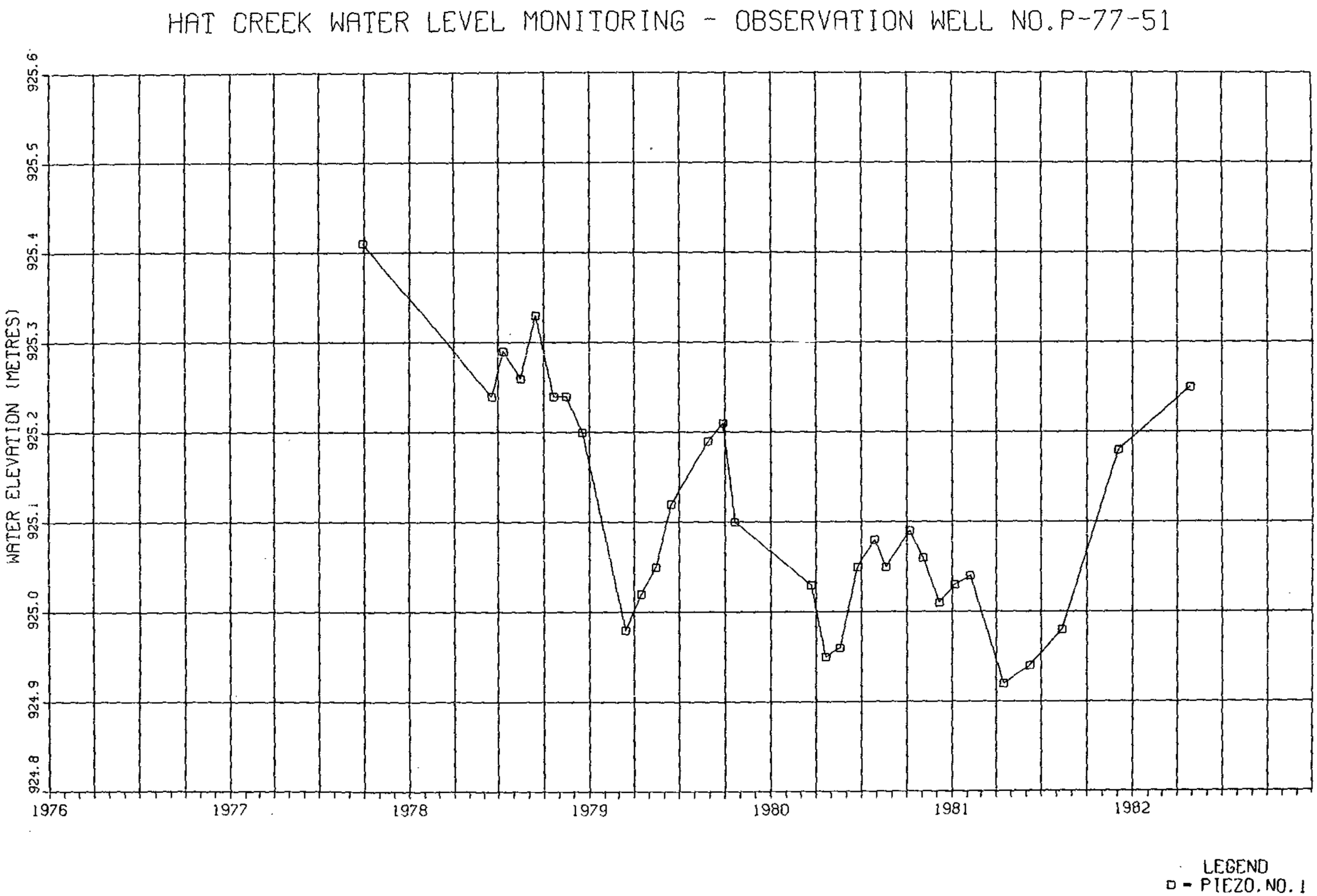


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-50

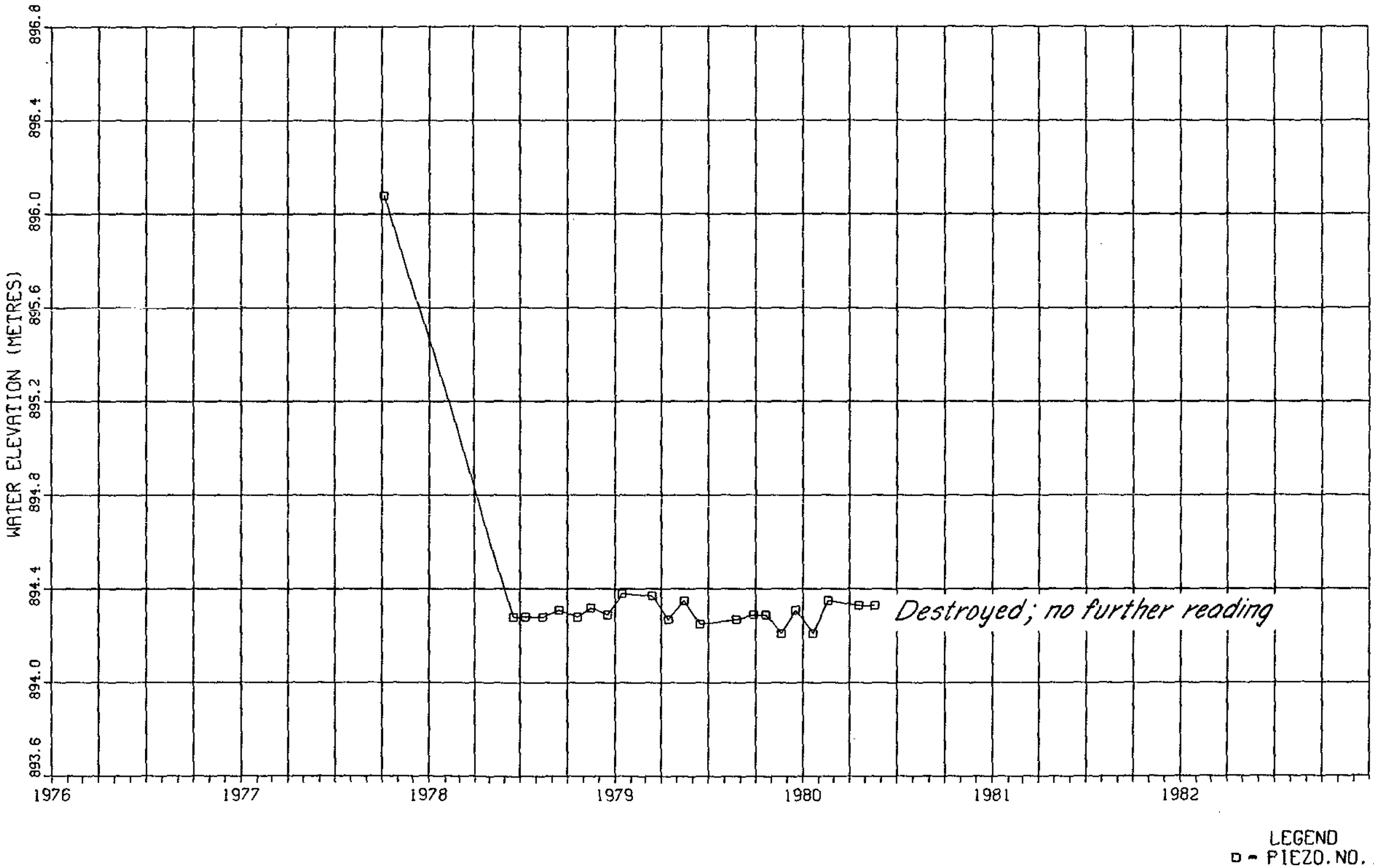


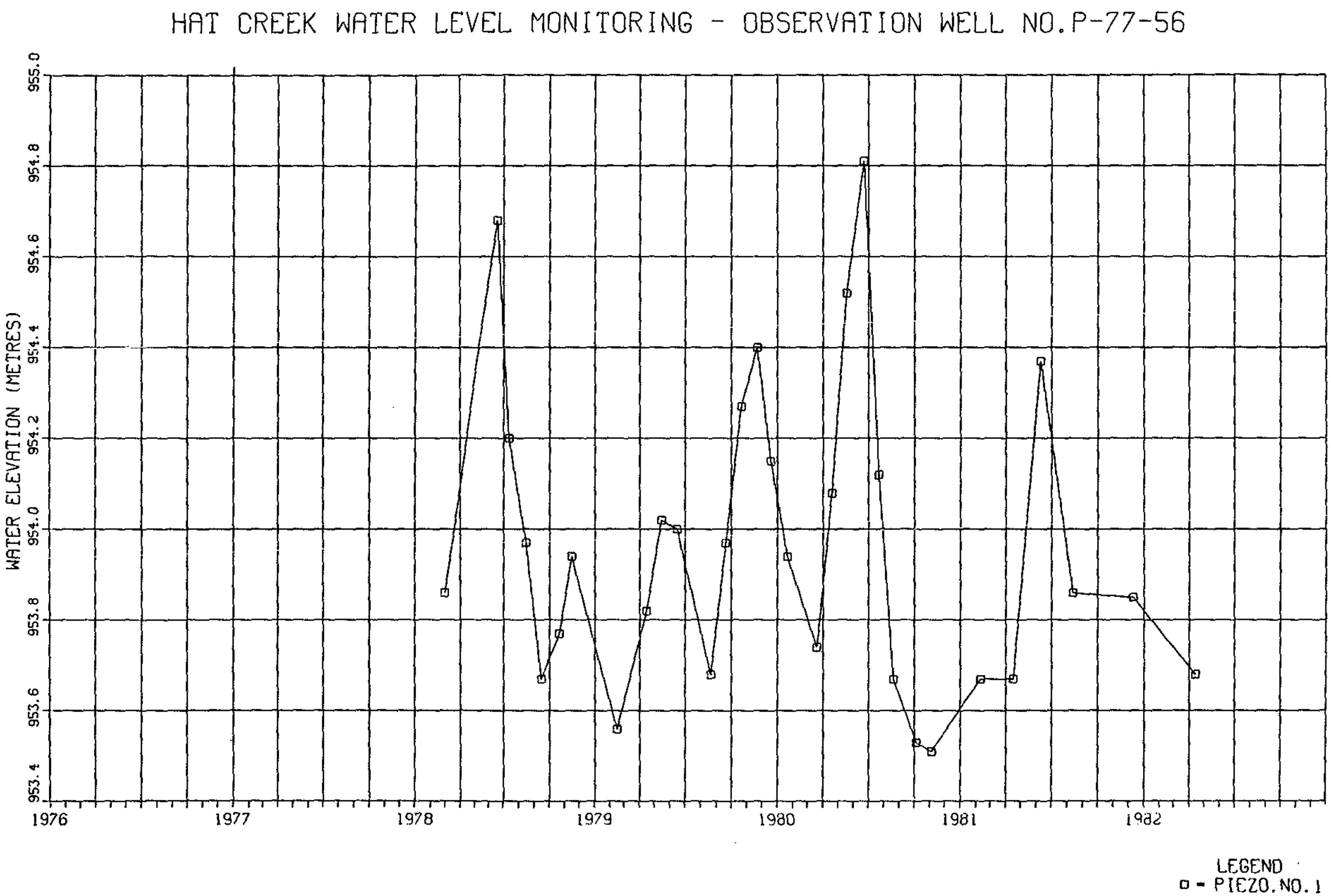
Golder Associates



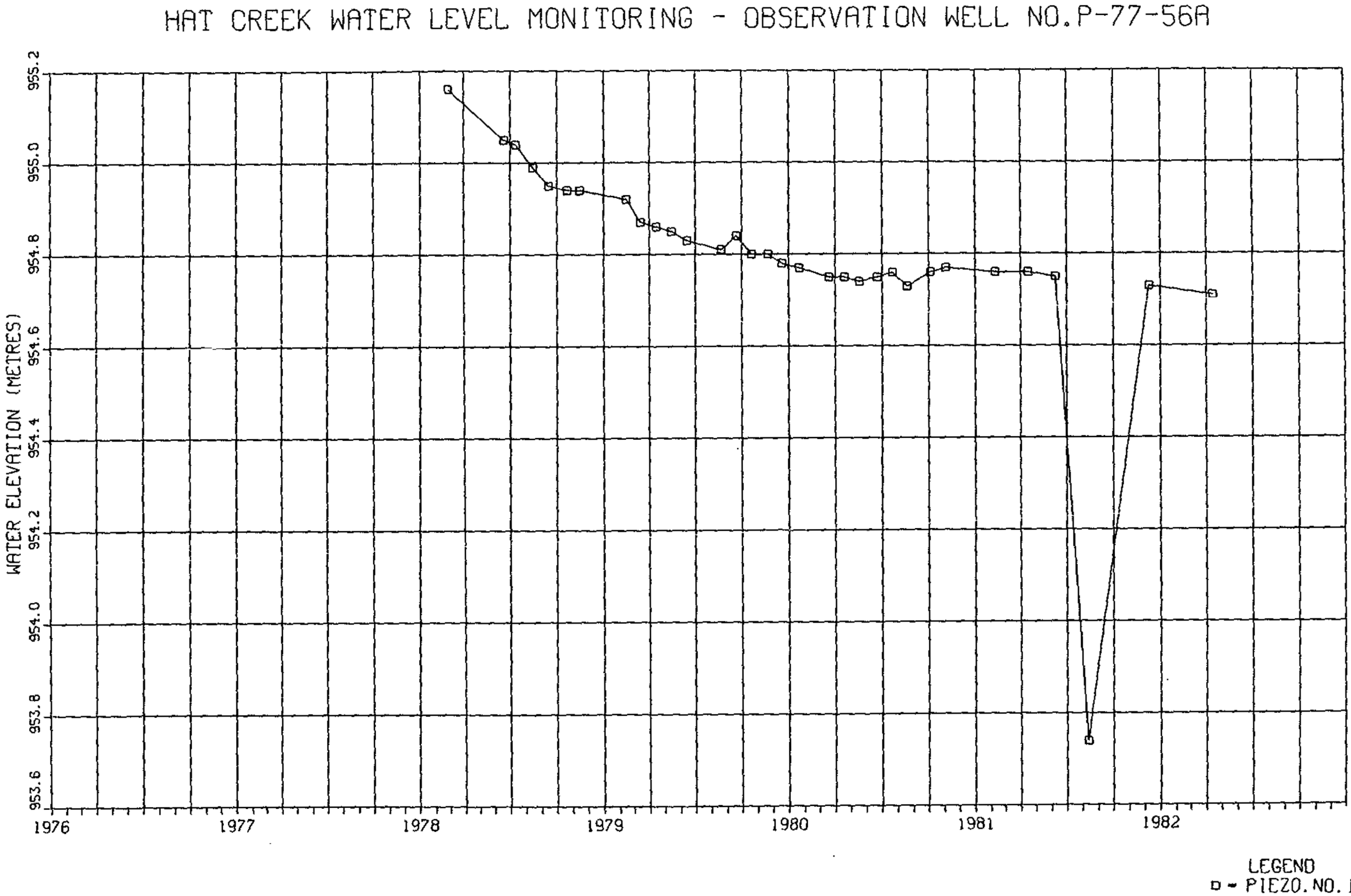
Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-52

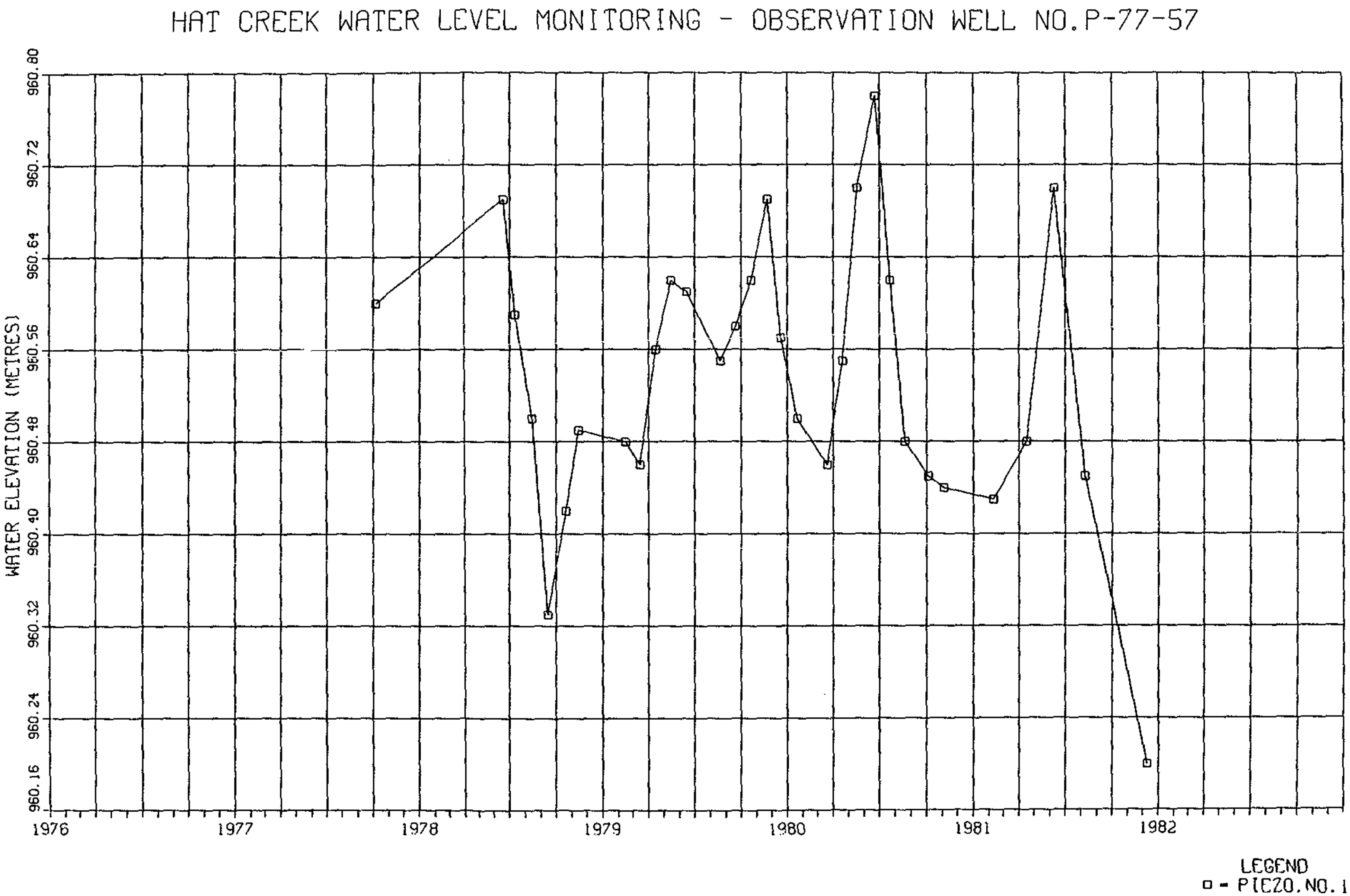


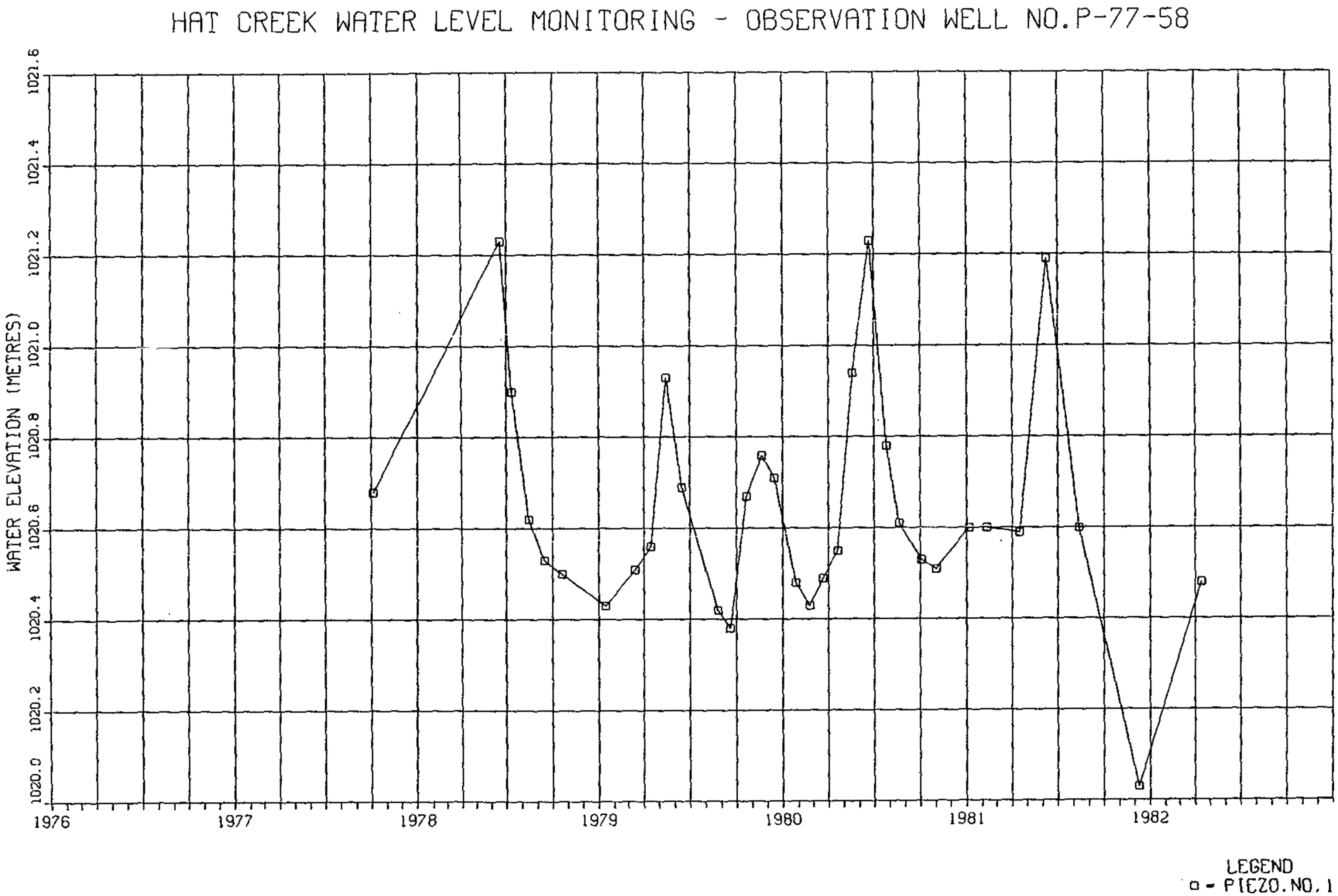


Golder Associates



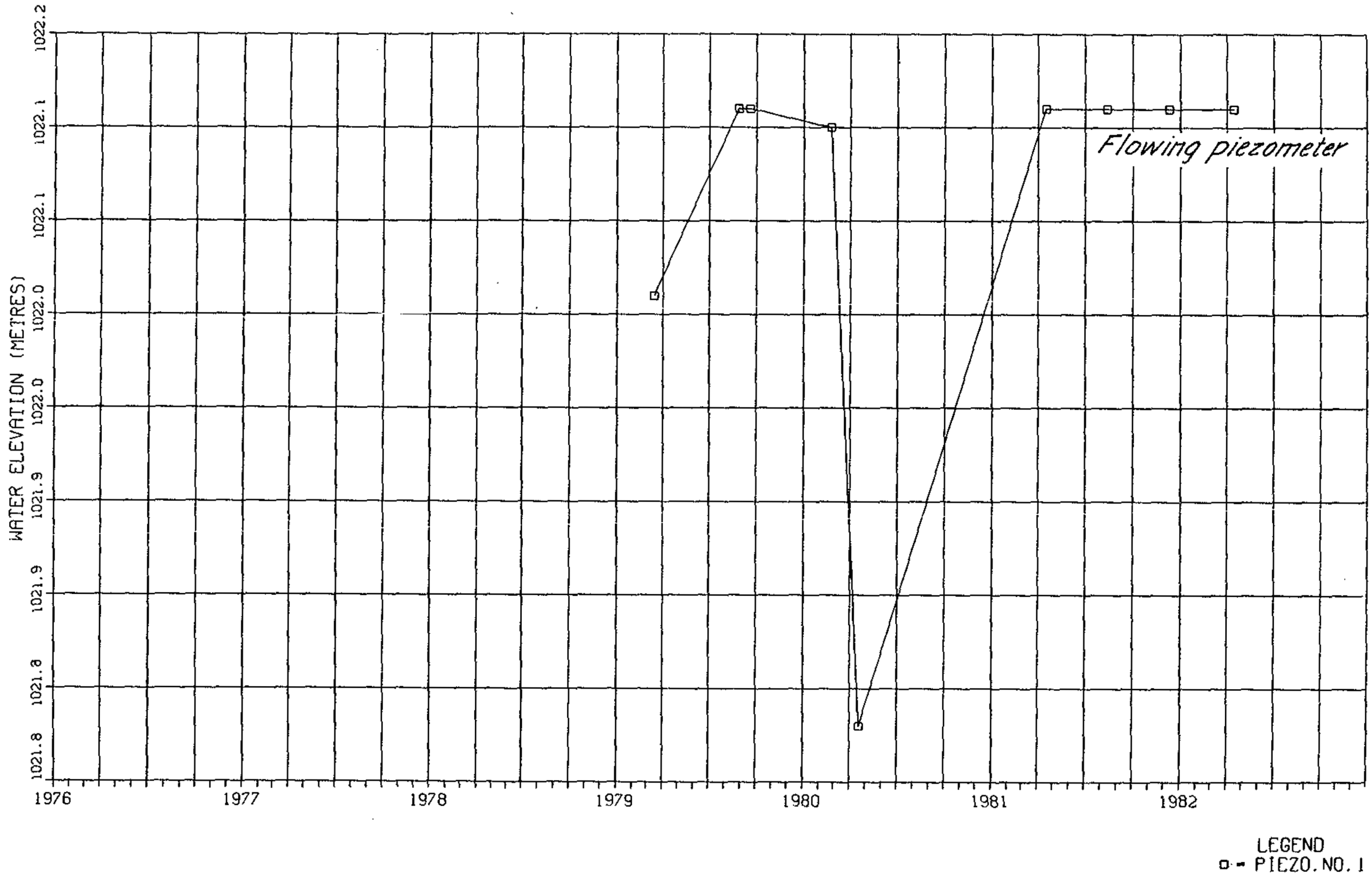
Golder Associates


HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-57
WATER ELEVATION (METRES)
960.80
960.72
960.64
960.56
960.48
960.40
960.32
960.24
960.16
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

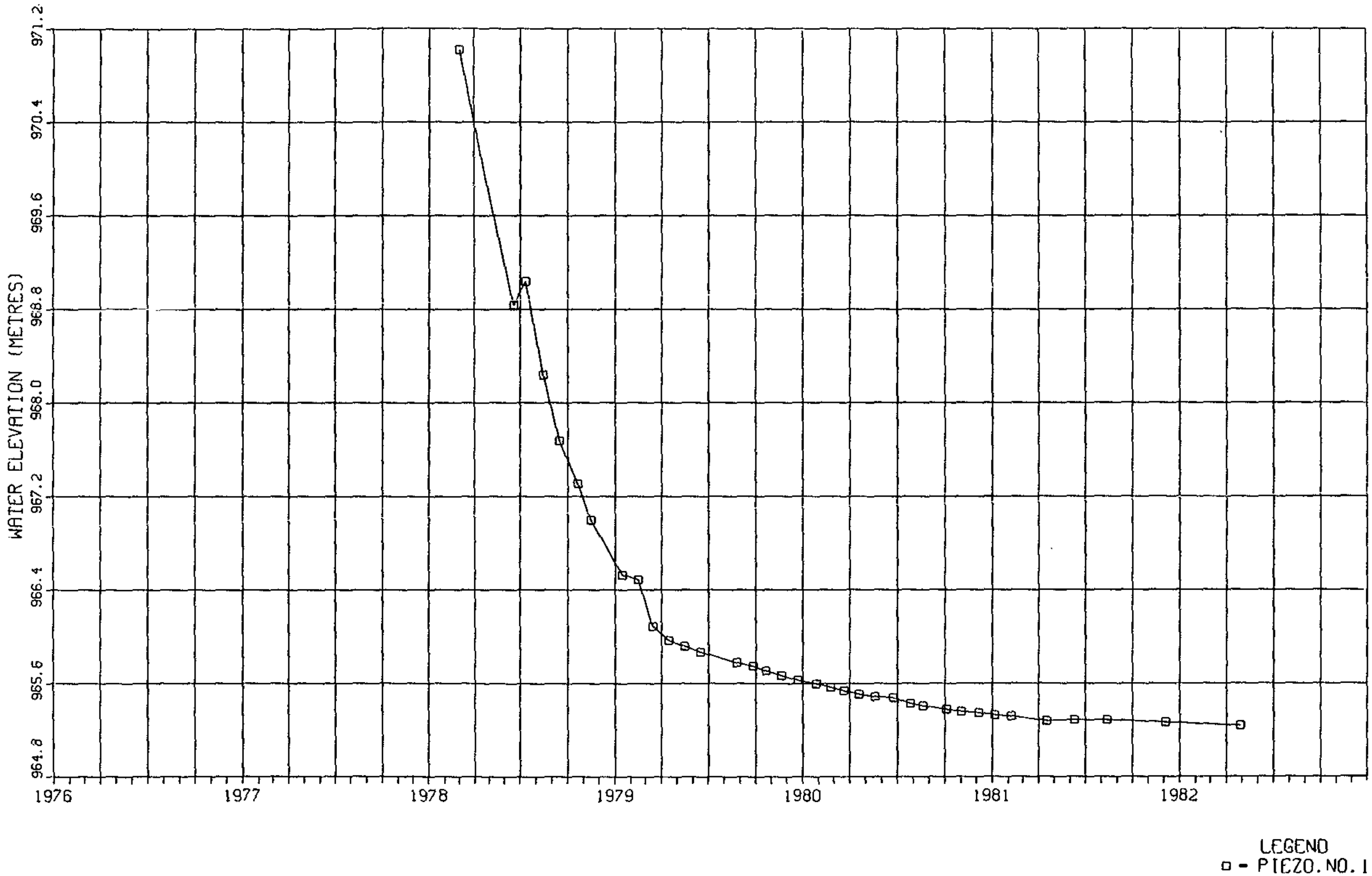


Golder Associates

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-59

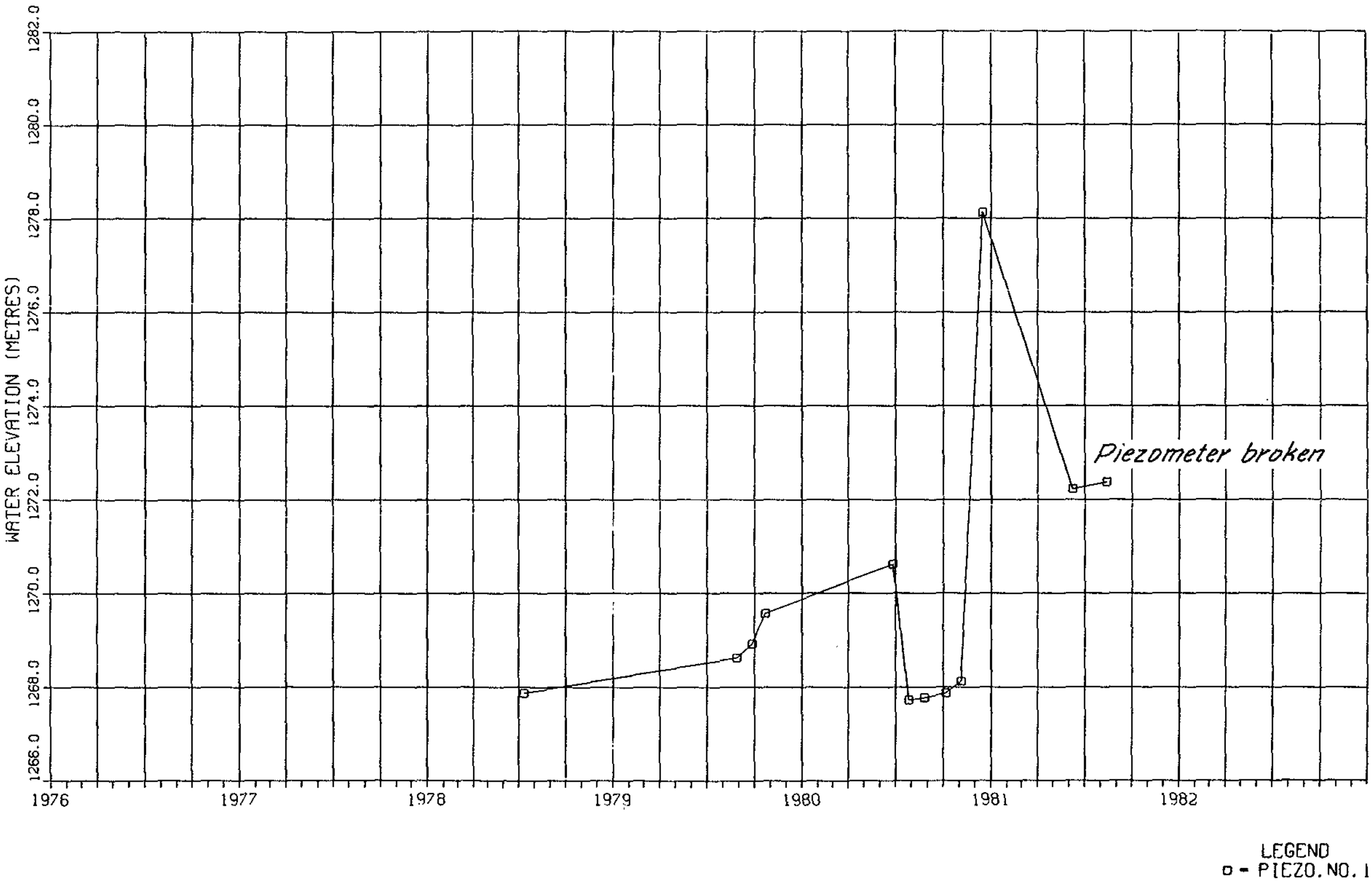


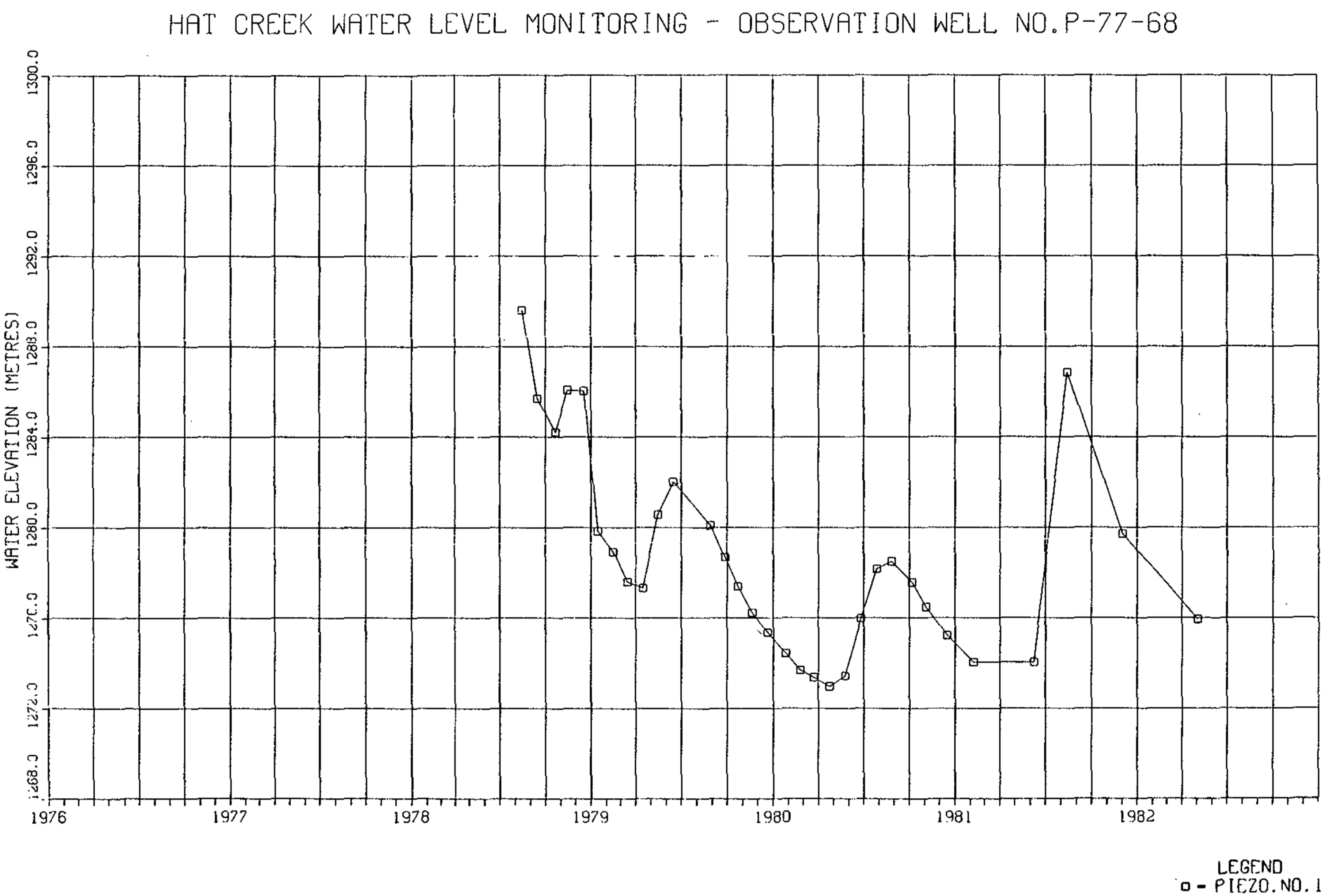
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-61



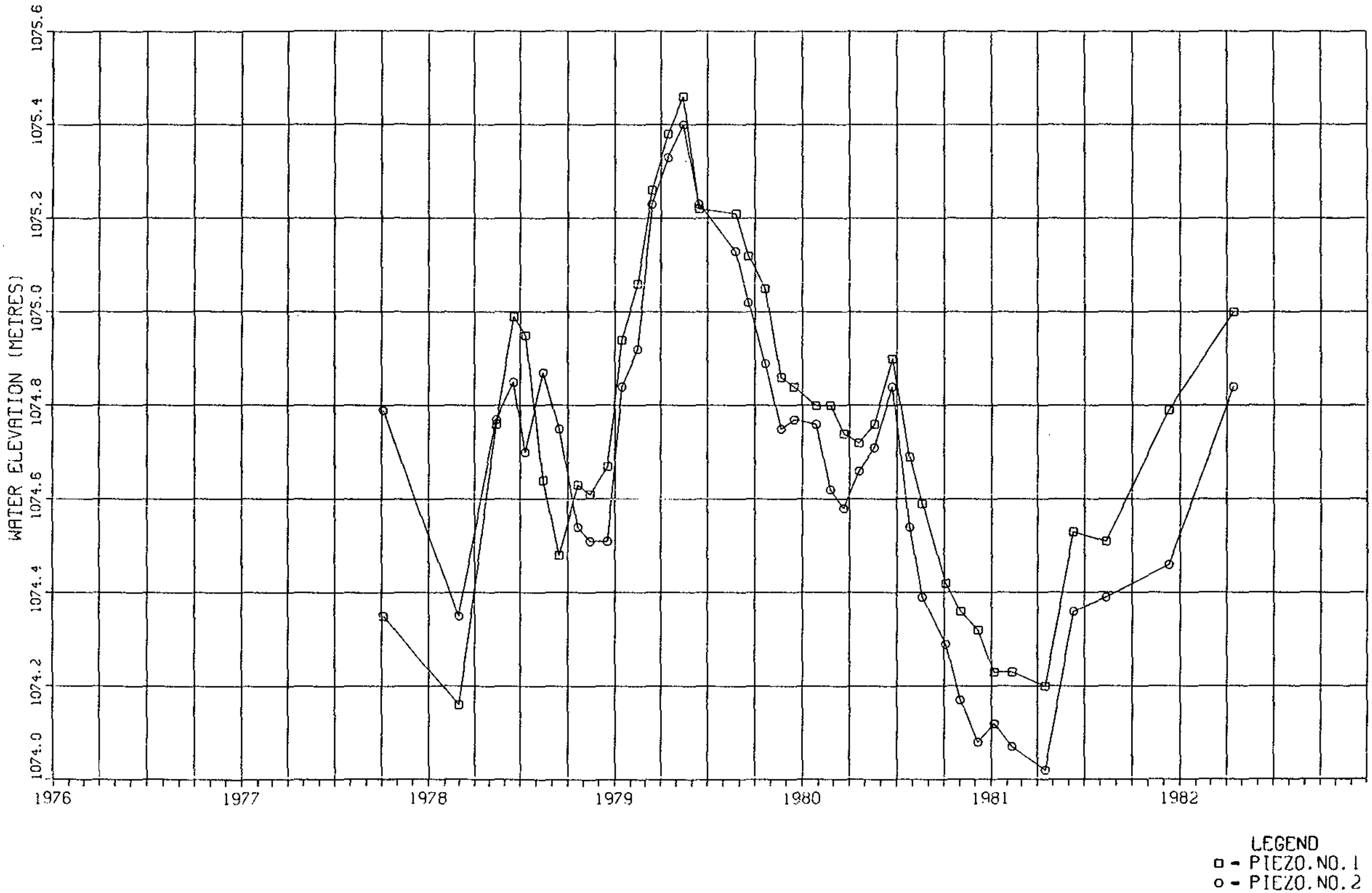
Golder Associates

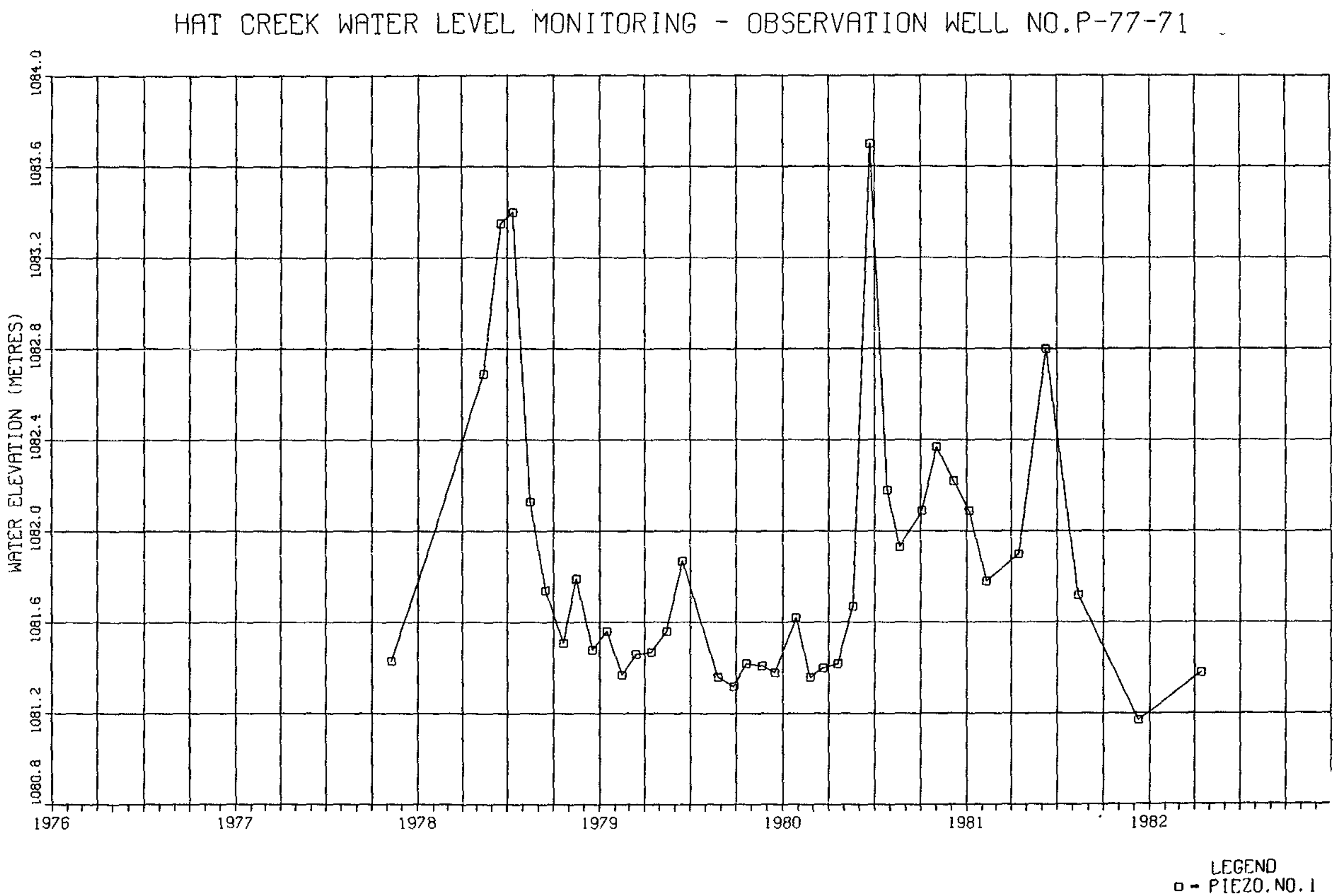
HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-67




HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-68
WATER ELEVATION (METRES)
1300.0
1296.0
1292.0
1288.0
1284.0
1280.0
1276.0
1272.0
1268.0
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-70




HAT CREEK WATER LEVEL MONITORING - OBSERVATION WELL NO.P-77-71
WATER ELEVATION (METRES)
1084.0
1083.6
1083.2
1082.8
1082.4
1082.0
1081.6
1081.2
1080.8
1976
1977
1978
1979
1980
1981
1982
LEGEND
□ - PIEZO.NO.1

APPENDIX E

HYDROGEOLOGIC LOGS

HYDROGEOLOGICAL LOGS

1.0 INTRODUCTION

The following hydrogeological logs summarize information on all boreholes where subsurface hydrogeological data has been obtained.

In order to show all data in a compact log, it was necessary to use a number of abbreviations and a symbolic notation. The following notes explain these abbreviations. The note numbers refer to the numbers shown in parenthesis at the head of each column in the logs.

2.0 REFERENCE ELEVATION

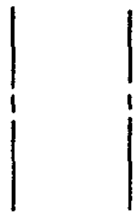
All depth measurements are given in metres relative to surveyed ground level.

(1) Lithologic Terminology Used in Logs

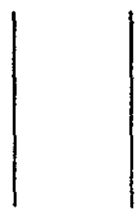
Lithology of boreholes has been determined from hydrogeologists' field descriptions and interpretation of geophysical logs (where applicable).

3. Completed Construction

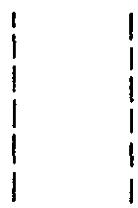
a) Hole



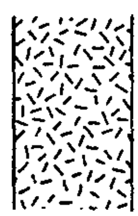
drilled hole casing removed



drilled hole casing left in place



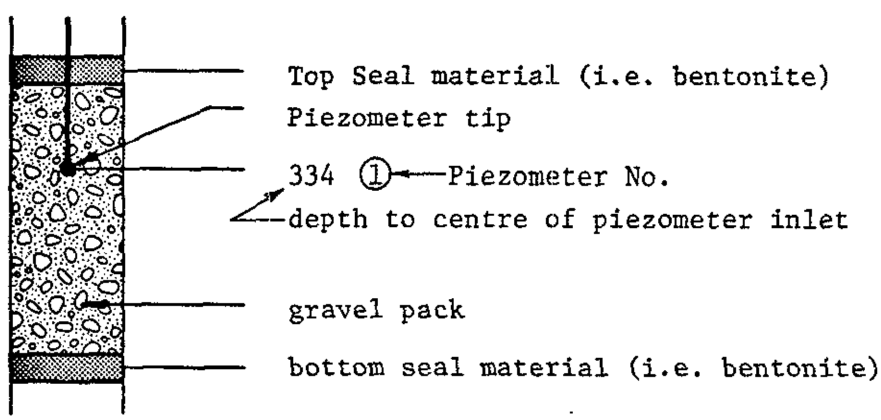
drilled open hole



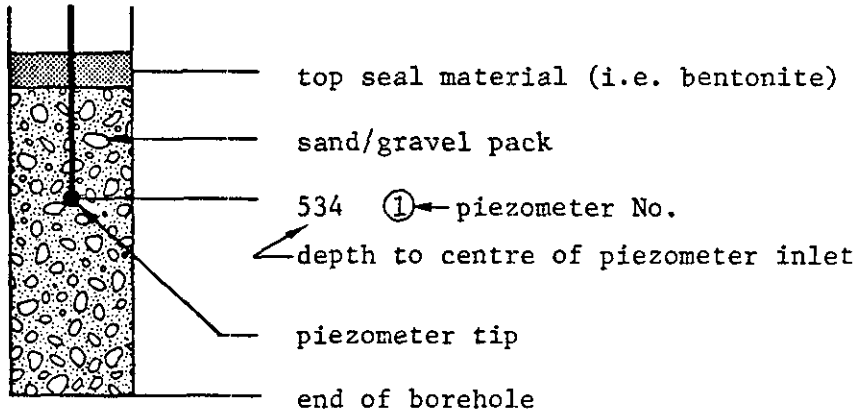
drilled hole known to have caved or squeezed.

b) Piezometer

Standard Double Seal Piezometer Arrangement

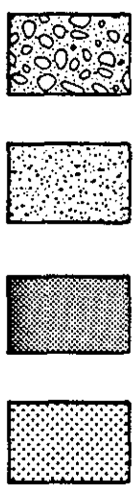


Standard Top Seal Piezometer Arrangement



Type of Piezometer tip: - perforated 25 mm Ø PVC pipe approx. 1.2 m long), wrapped with permeable fabric.

c) Types of Backfill



Gravel

Sand

Bentonite

Cement Grout

HYDROGEOLOGIC LOG

DRILLHOLE No. RH-82-102
Sheet 1 of 3

Job No.

Project: HAT CREEK COAL DEVELOPMENT - GROUNDWATER EXPLORATION

Type of drilling: AIR ROTARY

Rig: BUCYRUS ERIE 24R

Drilling fluid: AIR/WATER/FOAM

Coordinates: E 5993991
N 5626229.0

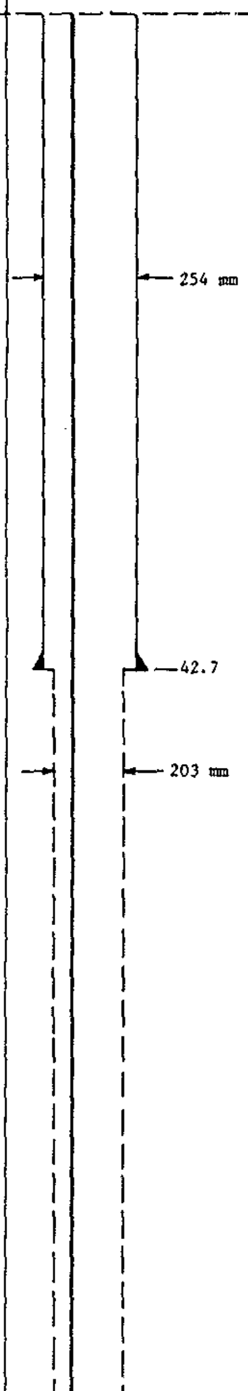
Angle from horizontal: VERTICAL

Bearing: ------ ° Azimuth

Reference elevation: 96517 m

Elevation type: surveyed [X] altimeter [ ] from map [ ]

Purpose of hole: HYDROGEOLOGY

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling: (2) Depth (m)	During Drilling: (2)(4) Water Level (m)	During Drilling: (5) Water Flow (l/s)	During Drilling: (6) Other	After Drilling: (2)(7) Water Level (m)	After Drilling: Permeability (8): (2) Depth (m)	After Drilling: Permeability (8): Method	After Drilling: Permeability (8): Value (m/s)	Comments
f-m Angular GRAVEL with some silty SAND occasional boulders (depth scale 0–90 m: 10, 20, 30, 40, 50, 60, 70, 80, 90)										

Contractor: DRILLWELL	Logged by: DJM	✱ NOTE: Bracketed numbers refer to notes preceding the logs.
Date started: 12/06/82	Checked by:	
Date finished: 20/06/82	Date:	**Golder Associates** — Scale: 1:500

HYDROGEOLOGIC LOG

DRILLHOLE No. RH-82-102
Sheet 2 of 3

Project: HAT CREEK COAL DEVELOPMENT - GROUNDWATER EXPLORATION

Type of drilling: AIR/ROTARY
Rig: BUCYRUS ERIE 24R
Drilling fluid: AIR/WATER/FOAM

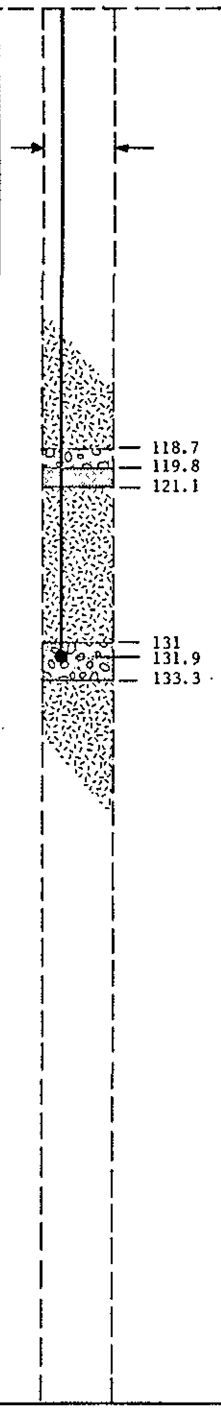
Coordinates: E 599399.1
N 5626229.0
Angle from horizontal: VERTICAL
Bearing: ------ °Azimuth

Reference elevation: 965.7
Elevation type: surveyed [X] altimeter [ ] from map [ ]
Purpose of hole: HYDROGEOLOGY

Job No.

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling: (2) Depth (m)	During Drilling: (2)(4) Water Level (m)	During Drilling: (5) Water Flow (l/s)	During Drilling: (6) Other	After Drilling: (2)(7) Water Level (m)	After Drilling: Permeability (8): (2) Depth (m)	After Drilling: Permeability (8): Method	After Drilling: Permeability (8): Value (m/s)	Comments
90 – 120.1: f.-m angular GRAVEL with some silty SAND and occasional boulder										
120.1 – 123.1: silty SAND & GRAVEL with some clay (TILL)										
123.1 – 137.8: silty coarse, angular SAND and fine GRAVEL with some coal fragments										
137.8 – 168.9: silty CLAY with some f. sand and occasional SILT layers										
168.9 – 175.0: SILT & f. SAND										
175.0 – 180: silty, f-c SAND and GRAVEL with trace of CLAY										

Depth scale: 90, 100, 110, 120, 130, 140, 150, 160, 170, 180



Contractor: DRILLWELL
Date started: 12/06/82
Date finished: 20/06/82
Logged by: DJM
Checked by:
Date:

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

Golder Associates

Scale: 1:500

HYDROGEOLOGIC LOG

DRILLHOLE No. RH-82-102
Sheet 3 of 3

Job No.

Project: HAT CREEK COAL DEVELOPMENT - GROUNDWATER EXPLORATION

Type of drilling: AIR/ROTARY

Rig: BUCYRUS ERIE 24R

Drilling fluid: AIR/WATER/FOAM

Coordinates: E 599399.1
N 5626229.0

Angle from horizontal: VERTICAL

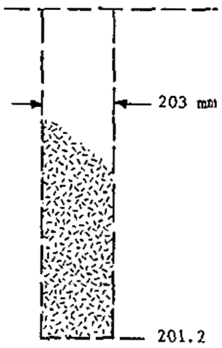
Bearing: ------ °Azimuth

Reference elevation: 965.7

Elevation type: surveyed [X] altimeter [ ] from map [ ]

Purpose of hole: HYDROGEOLOGY

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling: (2) Depth (m)	During Drilling: (2)(4) Water Level (m)	During Drilling: (5) Water Flow (l/s)	During Drilling: (6) Other	After Drilling: (2)(7) Water Level (m)	After Drilling: Permeability (8): (2) Depth (m)	After Drilling: Permeability (8): Method	After Drilling: Permeability (8): Value (m/s)	Comments
180										
Silty, dense f.-c SAND and f. GRAVEL with trace of clay										
190 — 190.8										
Green, soft clayey SILTSTONE										
201.2										
END OF BOREHOLE										



Contractor: DRILLWELL
Date started: 12/06/82
Date finished: 20/06/82

Logged by: DJM
Checked by:
Date:

* NOTE: Bracketed numbers refer to notes preceding the logs

Golder Associates

Scale: 1:500 Metric

HYDROGEOLOGIC LOG

DRILLHOLE No. RH-82-103
Sheet 1 of 3

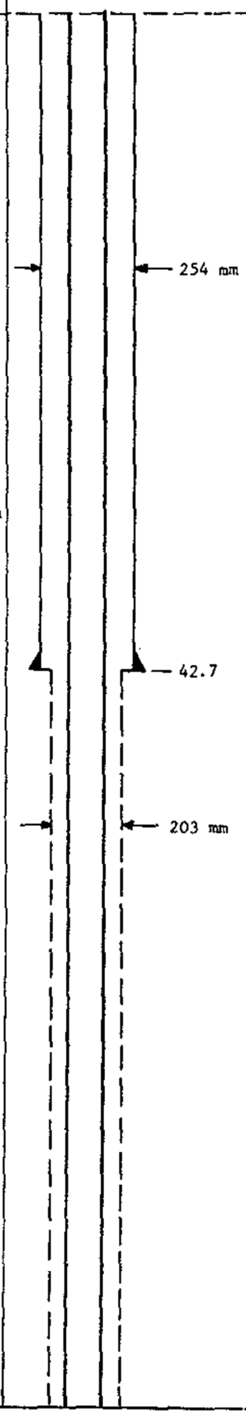
Job No.

Project: HAT CREEK COAL DEVELOPMENT - GROUNDWATER

Type of drilling: AIR/ROTARY
Rig: BUCYRUS ERIE 24R
Drilling fluid: AIR/WATER/FOAM

Coordinates: E 599056.4
N 5626420.7
Angle from horizontal: VERTICAL
Bearing: ------ °Azimuth

Reference elevation: 948.6 m
Elevation type: surveyed [X] altimeter [ ] from map [ ]
Purpose of hole: HYDROGEOLOGY

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling (2) Depth (m)	During Drilling (2)(4) Water Level (m)	During Drilling (5) Water Flow (l/s)	During Drilling (6) Other	After Drilling (2)(7) Water Level (m)	After Drilling Permeability (8) (2) Depth (m)	After Drilling Permeability (8) Method	After Drilling Permeability (8) Value (m/s)	Comments
Brown, subround to angular, f-m GRAVEL with some silty SAND and occasional thin clayey layers between 76 m and 102.4 m										

Depth scale (m): 10, 20, 30, 40, 50, 60, 70, 80, 90

Contractor: DRILLWELL
Date started: 20/06/82
Date finished: 29/06/82
Logged by: DJM
Checked by:
Date:

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

Golder Associates

Scale 1:500 Metric

HYDROGEOLOGIC LOG

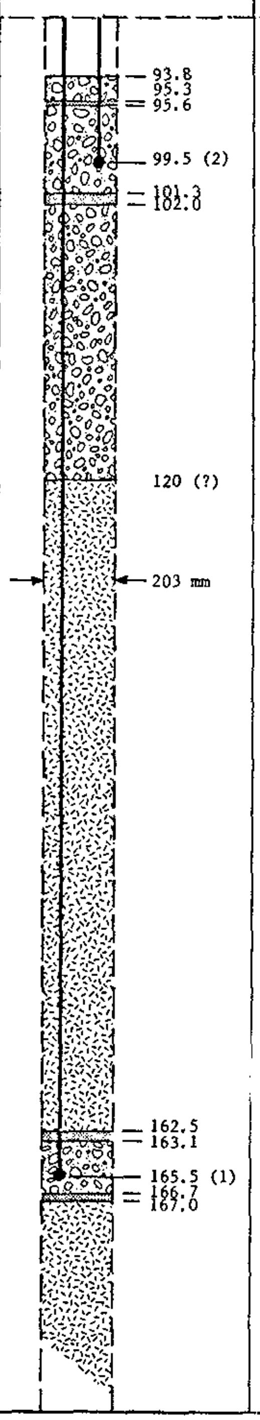
DRILLHOLE No. RH-82-103
Sheet 2 of 3

Job No.

Project: HAT CREEK COAL DEVELOPMENT – GROUNDWATER EXPLORATION
Type of drilling: AIR/ROTARY
Rig: BUCYRUS ERIE 24R
Drilling fluid: AIR/WATER
Coordinates: E 599056.4
N 5626420.7
Angle from horizontal: VERTICAL
Bearing: ------ ° Azimuth
Reference elevation: 948.6
Elevation type: surveyed [X] altimeter [ ] from map [ ]
Purpose of hole: HYDROGEOLOGY

(1) (2) * Lithology	(2) (3) Completed Construction	During Drilling: (2) Depth (m)	During Drilling: (2)(4) Water Level (m)	During Drilling: (5) Water Flow (l/s)	During Drilling: (6) Other	After Drilling: (2)(7) Water Level (m)	After Drilling: Permeability (8): (2) Depth (m)	After Drilling: Permeability (8): Method	After Drilling: Permeability (8): Value (m/s)	Comments
90 – 102.4: f.-m GRAVEL with some silty SAND										
102.4 – 108.2: Grey CLAY and fine SAND										
108.2 – 132.0: Interbedded clayey SAND and GRAVEL, clayey fine SAND and CLAY										
132.0 – 138.4: Silty fine SAND with c. SAND and f. GRAVEL										
138.4 – 155.4: Silty CLAY										
155.4 – 170.7: Silty f-c SAND and f GRAVEL						112.65				
170.7 – 175.6: Clayey, silty SAND and GRAVEL (TILL)										
175.6 – 180: Green to brown clayey SILTSTONE										

Depth scale: 90, 100, 110, 120, 130, 140, 150, 160, 170, 180



Contractor: DRILLWELL
Date started: 20/06/82
Date finished: 25/06/82
Logged by: DJM
Checked by:
Date:

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

Golder Associates

Scale: 1:500 Metric

HYDROGEOLOGIC LOG

DRILLHOLE No. RH-82-103
Sheet 3 of 3

Job No.

Project: HAT CREEK COAL DEVELOPMENT - GROUNDWATER EXPLORATION

Type of drilling: AIR/ROTARY
Rig: BUCYRUS ERIE 24A
Drilling fluid: AIR/WATER/FOAM

Coordinates: E 599056.4
N 5626420.7
Angle from horizontal: VERTICAL
Bearing: ------ °Azimuth

Reference elevation: 948.6
Elevation type: surveyed [X] altimeter [ ] from map [ ]
Purpose of hole: HYDROGEOLOGY

(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 (m/s)	Comments
180										
Green to brown clayey SILTSTONE	203 mm									
189.0	189.0									
190 END OF BOREHOLE										

Contractor: DRILLWELL
Date started: 20/06/82
Date finished: 25/06/82

Logged by: DJM
Checked by:
Date:

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

Golder Associates

Scale: 1:500 Metric

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Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
B.C. HYDRO
ON THE
HAT CREEK PROJECT
DIVERSION STUDY
FINAL REPORT

VOLUME 1 - MAIN TEXT

CL 7445
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December, 1982

822-1523B

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SELECTION AND EVALUATION

SUMMARY AND CONCLUSIONS

B.C. Hydro has proposed a coal-fired thermal power plant and associated open pit mine in the Upper Hat Creek Valley. Alternative power plant capacities of 800 MW and 2240 MW have been considered. Since the open pit would be located in the valley bottom astride the present Hat Creek channel, it would be necessary to divert Hat Creek and its tributary Finney Creek around the open pit. Previous studies by Monenco Consultants Pacific and the Hydroelectric Design Division of B.C. Hydro examined possible diversion alternatives and recommended a canal as the most economical arrangement.

A canal around the open pit mine could pose certain problems; local instability of the pit slopes could result in failure of the canal, or increased pore water pressures in the pit slopes, caused by leakage from the canal, could lead to instability. The consequences of canal leakage or failure could be serious. As a result of misgivings about the canal diversion, the present study was authorized to re-examine the alternatives with particular emphasis on a tunnel arrangement, since it also offered possibilities for some pit slope drainage. The use of polyethylene pipes was also investigated. Polyethylene pipe has only recently become available in diameters large enough for this use. During the course of the study, the pipeline arrangement became increasingly attractive because of its lower cost and its simplicity, and therefore it was investigated in some detail.

After considering the canal, the tunnel and the pipeline alternatives, the pipeline arrangement was selected as the recommended diversion method for both the 800 MW and 2240 MW Schemes. Hat Creek could be diverted by a small diversion dam located a short distance upstream of the open pit. A series of single, large diameter fibreglass reinforced pipe would convey the water to the pit rim, where twin, large diameter polyethylene pressure pipes (Sclairpipe or Driscopipe) would convey the water around the east side of the pit on the excavated slopes.

Initially, the pipe would follow the present Hat Creek channel. As the open pit expanded, the polyethylene pipes would be moved, one at a time, onto pit benches. From three to eight moves would be required depending on the project size. Polyethylene pressure pipe is extremely tough and lightweight, and so it can be moved easily. Beyond the pit, a section of fibreglass reinforced pipe would carry the water past the leachate lagoons, thereafter discharging into the original Hat Creek channel.

The principal advantages of the pipeline over the previously recommended canal arrangement, or the tunnel, would be reliability against seepage or failure, and lower cost. Since there would be two pipes around the critical open pit region, simplicity and flexibility would also be provided.

The overall length of the various diversion layouts considered ranged between about 4,000 m for the simplest pipeline arrangement for the 800 MW Scheme to about 9,000 m for the tunnel arrangement for the 2240 MW Scheme.

The initial capital cost of the recommended pipeline diversion would be approximately $16 million (1982 dollars) for the 800 MW Scheme and $19 million for the 2240 MW Scheme. This compares to $26 million for a canal arrangement and $50 million for a tunnel arrangement. Simpler and more economical pipeline arrangements are possible and these were considered. Recommended arrangements for both schemes and abandonment are described.

We thank you for the opportunity of carrying out these studies. We have pleasure in submitting this final report.

Yours very truly,

GOLDER ASSOCIATES

G. E. Rawlings

G.E. Rawlings, P. Eng.

GER/bjh
822-1523B

1.0 INTRODUCTION

1.1 Background

B.C. Hydro has proposed a coal-fired thermal power plant and associated open pit mine in the Upper Hat Creek Valley between Cache Creek and Lillooet, B.C. The valley lies approximately mid-way between the Fraser River valley to the west and the Thompson River valley to the east and runs roughly south to north. The elevation of the valley bottom ranges between approximately 800 and 1000 m. The Upper Hat Creek valley is broad, flat and it is presently used chiefly for agriculture.

The existence of coal deposits in the region has been known for over a century and numerous attempts have been made to exploit the resource. The first power plant feasibility and preliminary environmental impact reports for a coal-fired thermal plant were carried out in 1975. Since then, several conceptual design studies have been completed. Initially, the power plant capacity was to be 2240 MW with 240 MW of this required for the project operation, leaving a net output of 2000 MW. However, in the spring of 1982, B.C. Hydro also decided to investigate a scaled down version with a power plant capacity of 800 MW. This limited scale project would be less complex, less costly, and have a smaller open pit mine.

The open pit for the proposed project would be located at the northern end of the Upper Hat Creek valley in the No. 1 Coal Deposit and, depending on the scale of the project, it would be about 1.6 km to 2.2 km in diameter, (see Drawing 1). Both the 800 MW and the 2240 MW Schemes would have a design life of about 35 years. After this time, it might be decided to exploit the ultimate resource of this pit, or to exploit the No. 2 Coal Deposit a few kilometres south of the first pit upstream in Hat Creek.

Hat Creek itself is a small, meandering stream most of the time, but it is subject to occasional high flows during the freshet season: the peak recorded 24-hour flow was 14.6 m^3/s in June 1964. Since the proposed open pit mine would be located astride Hat Creek, diversion of the creek around the open pit would be required.

Previous studies recommended a canal as the preferred diversion arrangement. However, because of concern about the safety of a canal, Golder Associates were commissioned to carry out the present study to re-examine the diversion alternatives. Sigma Engineering Ltd. provided the hydrological and hydraulic engineering input to the work.

1.2 Terms of Reference

The objective of this study was to re-examine alternative arrangements for diverting Hat Creek around the open pit. Specifically, the study was intended to:

- review the use of a canal for the diversion of Hat Creek as recommended in previous studies;
- investigate the geotechnical aspects, hydraulics and benefits of improved pit slope stability in using a tunnel for the diversion of Hat Creek;
- investigate other possible diversion arrangements for Hat Creek;
- establish the technical and economic feasibility of each of the above diversion alternatives;
- recommend the most suitable diversion arrangement.

Initially, the study was confined only to the 2240 MW Scheme, but later it was extended to include the 800 MW Scheme.

1.3 Acknowledgements

This study was undertaken in close cooperation with the BCH Mining Department. The kind assistance of Dr. G. Lange, Mr. W. Fothergill and Mr. H. Kim is particularly noted.

1.4 Methodology

The purpose of the preliminary design study described in this report was to firm up the design of the Hat Creek Diversion arrangements to the point where the main design parameters could be chosen, their technical feasibility assured and a reliable cost estimate prepared. The approach to selecting the best diversion arrangement involved the identification of all possible alternatives and the elimination of the less promising as soon as that could be done with confidence. This approach is formally known as a "Branch and Bound" procedure. All available constraints are exploited to help narrow the range of alternatives. Branch and bound is a search technique which involves the following type of reasoning for every choice: if when trying to choose on grounds of cost between two alternatives which are equally satisfactory from a technical point of view, an over-estimate of the cost of Alternative 1 is less than an under-estimate of the cost of Alternative 2, then Alternative 2 can be safely eliminated. If, however, the over-estimated cost of Alternative 1 is greater than the under-estimated cost of Alterantive 2, neither can be eliminated at that point, both have to be retained until more accurate cost information allows bounds to be drawn which are sufficiently restrictive to distinguish the better alternative.

This is an efficient process in that engineering effort is progressively concentrated on the more promising alternatives. When it is combined with regular meetings of all those involved (including the client), it keeps everyone informed of the progress of the study and helps to ensure that nothing is overlooked.

Several different types of canal, pipeline, and tunnel arrangements were investigated, and the less attractive were eliminated until only a few alternatives were left for consideration. Since at this stage, the pipeline diversion arrangement compared very favourably with the other arrangements, it was selected for further detailed investigation and costing. Similar approaches were adopted for both the 800 MW and the 2240 MW Schemes.

A branch and bound search was also adopted for the assessment of the various tunnel alternatives. Details of this procedure are to be found in Appendix A.

1.5 Previous Studies

1.5.1 Monenco, Hat Creek Diversion Study, 1977

The first major study on the Hat Creek Diversion was a conceptual design study carried out by Monenco Consultants Pacific (1977). Monenco considered a wide range of alternatives for diversion around the proposed open pit, including those utilizing flow by gravity, by pumping and by upstream diversion of the watershed. Consideration was given to canals, tunnels, flumes, pipes and chutes. Also, a preliminary examination was made of utilization of Hat Creek water for the power plant water supply.

From these alternatives, Monenco selected those considered to be the most practical and economical, namely:

(1) A canal arrangement whereby the creek would be diverted upstream of the pit perimeter at sufficient elevation to flow by gravity in a canal along the east hillside of the valley, around the edge of the pit excavation and thence by pipe or chute running down into Hat Creek.

(2) A tunnel alternative which would use a lined tunnel conduit for the centre section of the canal close to the pit.

(3) A pumping arrangement which would provide regulation of the creek flow by upstream storage reservoirs, and then pump the water through a pipeline above the pit perimeter.

(4) A water supply arrangement which would pump water from Hat Creek for use at the power plant.

Of these schemes, Monenco concluded that the most economical arrangement was that of a gravity diversion in an earth-lined canal. The chief disadvantage of the canal arrangement as identified by Monenco was the need to replace a short central portion of the canal by tunnel after several years of pit operation because the expanding pit would encroach on the canal route. The water supply arrangement was identified as requiring additional work.

Monenco was restricted in its study of the tunnel alternative by the lack of geological information east of the pit at that stage.

1.5.2 HEDD, Diversion of Hat and Finney Creeks
Preliminary Design Report, 1978

In March 1978, the Hydroelectric Design Diversion (HEDD) of B.C. Hydro completed a preliminary design report on the Hat Creek diversion. They confined their work to the water supply arrangement and to the canal diversion arrangement.

HEDD presented some work on the water supply arrangement in their report, but it was ruled out on the basis of poor water quality and the high cost of remedial water treatment.

HEDD reviewed and refined the canal diversion arrangement in some detail along with a breakdown of cost estimates. Later in the present report, their recommended canal diversion arrangement is described but with some modifications (a pipe section between Ambusten Creek and Medicine Creek) as shown in Drawing 5.

Additional hydrological studies carried out by HEDD identified the design flow; these are reviewed in the present report in Section 2.

Some field investigation work was carried out in the form of drill holes and test pits for the canal and diversion dams and conduits. This information is reviewed in Section 3 of the present report.

1.5.3 Station Design Manual, 1980

In February 1980, HEDD released a "Station Design Manual" for the diversion of Hat and Finney Creeks. This report presents the preliminary design of the Hat Creek diversion, limiting itself to the canal arrangement only.

1.5.4 HGPD, Report on 1981 Site Investigation for Hat and Finney Creek Diversions and Access Road, 1982

Detailed site investigations for the proposed canal and impounding or diversion structures were carried out by the Hydroelectric Generation Projects Division (HGPD) of B.C. Hydro in the summer of 1981. The design of the canal was not revised although recommendations were given for design changes. The study is reviewed in Section 3 of the present report.

2.0 HYDROLOGY

2.1 General

The flow in Hat Creek can vary over a wide range. For most of the year, flows are quite low except during spring when snow is melting in the basin. Spring floods usually peak in May or June, and since these are the largest floods, they govern the diversion design capacity.

Stream flow records from three gauging stations are available for Hat Creek, although the period of record is rather short and intermittent. The station with the most useful records is Station 08LF061, located on Hat Creek immediately downstream of Medicine Creek, near the proposed point of diversion. This station has records dating from 1961 to 1977; hydrological information is presented in Drawing 2.

Hydrological analyses of flows in Hat Creek basin were carried out by Monenco Consultants Pacific (1977), by Beak Consultants (1977) and by HEDD (1978).

In the present study, a flow frequency analysis was made using a computer program developed at the University of British Columbia (Russell, 1982). This indicated results of the same order as the HEDD study and it was, therefore, decided to continue using the design flows derived in the comprehensive HEDD study.

The HEDD study used as criteria for the design of the diversion facilities, the 100-year return period flood as a normal operating condition and the 1000-year return period flood as an emergency condition.

Since failure of the diversion during time of flood could have very severe consequences for the operation of the project, a conservative approach to design seemed appropriate at the present stage. All the arrangements were therefore designed with sufficient capacity to pass the estimated 1000-year peak flood of 27 m^3/s. The design discharge capacities are summarized in Table 1.

At the final design stage, more hydrological data will be available, so that it would be possible to refine the estimates of flows and frequencies. At the same time, it would be desirable to carry out a risk/cost analysis, which would consider such factors as the consequences of flooding the pit to determine the most appropriate return period

TABLE 1

Summary of Adopted Design Discharge Capacities
(from HEDD, 1978, Table 5.3)

Facility	Design Discharge (m^3/s)	Recurrence Intervals	Flood Type	Drainage Area (km^2)
Emergency Spillways	79.0	PMF	PMF	350
Main Diversion				
Emergency Condition	27.0	1000	Snowmelt	350
Normal Condition	18.0	100	Snowmelt	350
Finney Creek Diversion	5.5	1000	Rainstorm	13

Notes:

(1) Capacities shown assume no upstream storage

(2) Probable Maximum Flood (PMF) is a maximized combination of snowmelt and rainstorms

(3) Spillway design discharge is based on PMF of 106 m^3/s less diversion capacity of 27 m3/s

to use for the design floods. However, for present purposes, the estimated 1000-year flood provides a reasonable and adequate basis for the comparison of alternative arrangements.

3.0 GEOTECHNICAL

3.1 Review of Previous Work

Prior to 1979, no geotechnical investigations had been carried out related to diversion schemes. However, some of the overburden drilling in the area of the east side of the valley in connection with the open pit was of use in considering the location of a diversion canal. This was the only data that was used directly by Monenco in preparing their report in 1976.

Subsequently in 1977, HEDD carried out overburden drilling for the canal scheme proposed by Monenco. This consisted of 3 holes at the headworks damsite, 2 holes at the pit rim damsite, 9 holes along the diversion canal route, 2 holes along the discharge conduit, and 2 and 3 holes at storage damsites No. 2 and 3, respectively: a total length of 472 m of drilling was performed. In addition, 16 test pits were excavated by backhoe at locations along the canal route at the damsites and in potential borrow areas. Laboratory testing was carried out on samples taken from the drillholes and test pits. This investigation concluded that, in general, the proposed canal diversion scheme could be founded on competent clayey, sandy gravels or clayey tills, but that in some areas of sand and gravel, an impervious lining would be needed in the canal. The report also concluded that cutoffs or blankets would be needed beneath both the headworks dam and the pit rim dam in order to control leakage in the pervious alluvial materials infilling Hat Creek valley bottom. Finally, it was concluded that adequate borrow materials for lining would be available for use in the canal. Details of the site investigation work are to be found in Section 4 of the HEDD Preliminary Design Report, 1978.

Further site investigations for the proposed canal diversion scheme were carried out in 1981 by HGPD. Details of the investigation program are reported in reference HGPD (1982). Rotary triconed and diamond drillholes were sunk in overburden and bedrock. These consisted of 5 holes at the headworks damsite, 4 holes at the pit rim damsite, 2 and 3 holes at the Ambusten Creek and Medicine Creek canal crossings, respectively, 5 holes along the diversion canal; a total length of 665 m of drilling was performed. Permeability testing was carried out in drillholes. In addition, 129 test pits were excavated by backhoe along the canal route and at the headworks and pit rim damsites. No further investigation of potential borrow areas was carried out. Laboratory testing of core samples from drillholes and disturbed samples from test pits was performed consisting of index tests, and shear, consolidation and swelling tests.

This study concluded that the entire length of the canal would likely require lining, whereas in the earlier 1978 study only the sections around the pit and the Ambusten and Medicine Creek crossings were thought to need lining. A potential slide area close to the canal just downstream from the headworks dam was identified and monitored with piezometers and a slope indicator installation. The report found that the section of the canal from Ambusten Creek to Medicine Creek "would be founded on a thin blanket of ablation and basal till overlying very soft, weak, highly bentonitic, undifferentiated volcaniclastics of rhyolitic composition". Flatter downhill side slopes were recommended for this section of the canal. However, the topography is steep and gullied, and in this report we recommend that this section of the canal be replaced by pipeline, see Section 4.3.2.

The report suggests that the downstream slopes of the embankment for the Ambusten and Medicine Creek crossings should be slightly flatter to accommodate possible weaknesses in the foundations. The embankment crossings are large and in order to ensure minimal settlement, it would be necessary to construct them of material well compacted in layers. Such embankments would be costly (see Table 7). Smaller embankments could result in substantial savings; such would be the case for pipeline crossings. In any event, pipeline crossings would be less sensitive to differential settlements between the embankment and the abutments than would be the case for a canal. Interfaces between engineered structures and natural ground can be a source of weakness leading to failure of canals, see Section 4.3.4. The report concludes that control of seepage through and under both the headworks dam and the Pit Rim Dam could be effected by using upstream blankets of till rather than slurry trench cutoffs to bedrock: a cutoff to bedrock had been proposed beneath the Pit Rim Dam in the HEDD 1978 report. The use of blankets seems reasonable, although we would recommend the installation of pumping wells in the alluvial materials at the toe of the Pit Rim Dam in order to intercept seepage towards the pit. These wells would also act as relief wells to control seepage pressures under the downstream toe of the dam.

The Headworks and Pit Rim Dams have not been re-analyzed for stability. For present feasibility purposes and, bearing in mind the current status of the canal arrangement, the general outline of the embankments reported in the HEDD 1978 report are acceptable.

3.2 Geology of the Diversion Scheme

The geological conditions along the route of the alternative diversion schemes have been adequately covered in previous reports (Golder Associates, 1977 and 1978; HGPD, 1982). The exception to this is the geology of the eastern escarpment which had not been systematically studied except close to the Medicine Creek Waste Embankment. Included in the present study, therefore, was a program of field investigation designed to obtain sufficient data for tunnel feasibility purposes.

Canal or pipeline routes would be predominantly on surficial deposits comprising colluvium, glacial till, outwash sands and gravels and slide debris. In places, they would also lie on Tertiary rocks. The engineered structures (Headworks Dam, Pit Rim Dam) would also lie on alluvial deposits; the geotechnical aspects of all these materials are covered in the earlier reports and in Sections 3.1, 4.3.4, and 4.5.5 of this report.

The eastern escarpment is geologically complex and required considerable geological interpretation to provide the necessary basis for the assessment of a tunnel alternative. Details of that work are covered in Appendix A (Volume 2).

3.3 Field Investigation

The early phases of the review of the diversion alternatives narrowed the potential tunnel alignments down to those passing through the eastern excarpment. For that reason, it was necessary to define the geology of that area and to interpret the facts acquired in terms of their geotechnical significance for tunnelling. The field work carried out is described briefly below, details are contained in Appendix A, Volume 2.

3.3.1 Geophysical Survey

Magnetic and resistivity surveys were carried out across the grain of the structure which runs lengthwise along the eastern escarpment. Excellent results were obtained from the magnetic survey but the resistivity method provided poor definition and was discontinued. Addendum 3 to Appendix A provides the results of the work. By means of this survey, it was possible to identify the geological structure and relate it to the stratigraphic sequence encountered in the drilling.

3.3.2 Geological Mapping

Although exposure is generally poor on the escarpment, there are significant outcrops which permit the stratigraphy and structure to be established. The work was carried out jointly by Golder Associates and Mr. H. Kim of BCH.

3.3.3 Diamond Drilling

Core drilling, mostly in angled holes, was carried out to obtain typical sections through the sequence at intervals along the escarpment. Recoveries were initially inhibited by the highly brecciated and altered nature of the andesite. In situ permeability testing was carried out and piezometers left in all holes.

3.3.4 Rock Testing

Routine tests were carried out on site for index purposes. These included moisture content, Atterberg Limits, uniaxial compression tests and point load strengths. Slake durability testing was carried out in Vancouver as well as petrographic description of representative rock types. The results of the testing may be found in Appendix A.

4.0 DIVERSION ALTERNATIVES

4.1 General

Three different alternative methods of creek diversion have been considered in this study, namely: canals, tunnels, and pipelines. They have been grouped together into various arrangements in order to provide the preferred engineering solution to the particular geotechnical, hydrological and mining constraints imposed.

The arrangements also differ for the scheme being considered (800 MW or 2240 MW) and for the period for which the arrangement would be operative (i.e. during the life of the pit or a long-term abandonment).

The main alternative arrangements would comprise:

(1) Canal in which the creek would be diverted sufficiently far upstream of the pit to permit gravity flow by canal around the eastern side and then by conduit back into Hat Creek. Some sections might be replaced by pipe (see Section 4.3).

(2) Canal/Tunnel/Pipe - a similar arrangement to the canal scheme except that the water would be conveyed past the pit in a tunnel.

(3) Pipeline Arrangements - various alternatives have been considered in which pipelines could replace both canal and tunnel for layouts both within and outside the pit.

These alternatives are considered for various layouts in subsequent sections.

4.2 Constraints

Constraints are imposed on the Hat Creek diversion alternatives by a number of factors including topography, geology and mine planning.

Diversion Dam and Intake

Any diversion alternative would require a dam and intake structure to divert the creek and create a siltation pond for the deposition of bedload. For cost purposes, it would be desirable to minimize the size of this structure. Moreover, to minimize seepage pressures in the pit slopes from such an impoundment, it should be kept at least several hundred metres upstream from the southern edge of the pit. For canal diversions, the structure must be above elevation 975 m for the canal to bypass the pit at an adequate distance from the pit perimeter.

For the tunnel alternatives, the most suitable elevation for the upstream tunnel portal would be about 970 m, although elevations as low as 960 m would be feasible. To convey water from Hat Creek at these higher elevations, it would be necessary to divert the creek some distance upstream. However, if the diversion dam were too far upstream, it could interfere with the mining of the No. 2 Deposit which might be mined some time during the life of the No. 1 Pit diversion. However, it is likely that alternative diversion arrangements would be made if part of the runoff from the Hat Creek catchment were intercepted by the No. 2 Deposit pit. Broadly, the No. 2 Deposit northerly limit has been treated as a constraint.

Pit Region

The mine plans developed by BCH have been treated as fixed and any diversion arrangement has been designed to accommodate them. A canal would have to lie between a safe distance from the edge of the planned pits (1980 design for the 2240 MW Scheme; 1982 design for the 800 MW Scheme), and the steep topography east of the pit. A pipeline would have to lie on the pit benches at the appropriate elevation, although for the final pipeline location, the pipe should be placed at constant grade to minimize sedimentation problems. A tunnel alternative would be constrained mainly by the geology, as discussed later.

Pit Exit Region

At the northern exit of the pit, there would be a congested area of haul roads and conveyor embankments which could be up to 30 m in height. A concrete pipe under these embankments would be vulnerable to settlement damage. Polyethylene pipe is not designed to withstand heavy superimposed loads. For the 800 MW Scheme, the congested area would be less than for the 2240 MW Scheme; however, in general it would be desirable for the diversion to bypass these areas.

Leachate and Sedimentation Lagoons

Downstream of the pit, there would be leachate and sedimentation lagoons, and Hat Creek would need to be diverted past these. Protection of the leachate lagoon against failure would be particularly important, and, for this reason, the diversion should be kept above and as far away as possible from this structure. This would apply especially to the pipeline arrangements for which the maximum height of the leachate lagoon would be an important consideration since sufficient head must be provided for the pipeline to carry the water past the lagoon.

Indian Reserve and Harry Creek

The location of the Indian Reserve places a constraint on the layout of the diversion at the downstream end. No encroachment on the reserve can be tolerated.

The elevation of the downstream portal of any tunnel section would likely be at approximately elevation 950 m beside the Harry Creek channel. Discharge into this channel would cause severe erosion and potential flooding in the Reserve below. Thus, the return of the diversion waters to Harry Creek must be between the sedimentation lagoon and the Indian Reserve boundary and would need to be by conduit.

4.3 Canal Scheme

4.3.1 General

The canal diversion arrangement as shown on Drawings 3 and 4 is similar to the diversion method recommended in the previous Monenco and HEDD studies, except that changes have been made as follows:

- the section of canal between Ambusten Creek and Medicine Creek has been replaced by a Fibreglass Reinforced Pipe (F.R.P.), since there was some doubt as to the practicability of a canal through this area of gullied steep side slopes;

- Fibreglass Reinforced Pipe has been substituted for Corrugated Steel Pipe (C.S.P.) for the discharge conduit in order to provide a more durable alternative.

Tables 2 and 3 summarize the canal arrangement parameters and hydraulic characteristics. The principal components of the diversion are given in Section 4.3.2.

4.3.2 Canal Diversion Arrangement

The main elements of a diversion accomplished predominantly by canal would be as follows:

(a) Headworks Dam and Intake Structure

Hat Creek would be diverted into the canal by an earthfill headworks dam upstream of the pit. The location of the dam remains unchanged from that shown in the previous HEDD (1978) study. It would be immediately downstream of Anderson Creek so that no minor diversion of that creek would be required. The water level behind the dam during normal operating conditions would be at about elevation 975 m. An emergency spillway would be provided for floods in excess of the 1000-year design discharge capacity of 27 m^3/s of the canal.

(b) Diversion Canal

The same canal is shown for both the 800 MW scheme and the 2240 MW scheme (see Figures 3 and 4, respectively). In both cases, the canal would be located at approximately elevation 975 m. This is the optimum elevation, sufficiently high above the pit boundary and still below the steeper parts of the cliffs east of the pit. For the larger 2240 MW pit, a portion of the canal would lie within the 35-year pit perimeter after several years of mine operation; it would have to be replaced by a conduit or a tunnel. For the smaller 800 MW pit, the canal might not

TABLE 2

Canal Diversion Parameters

	800 MW and 2240 MW Schemes
Intake and Diversion Dam	
Max. reservoir water level	976 m
Average dam height	15 m
Intake to Ambusten Creek	
Diversion method	Canal
Length	1500 m
Mean gradient	0.02%
Ambusten Creek to Medicine Creek	
Diversion method	2.7 m dia F.R.P.
Length	1700 m
Mean gradient	0.35%
Medicine Creek to Discharge Conduit	
Diversion method	Canal
Length	3175 m
Mean gradient	0.02%
Discharge Conduit	
Type	1.8 m dia F.R.P.
Length	2200 m
Mean gradient	6.8%
TOTAL LENGTH OF DIVERSION	8575 m

TABLE 3

Canal Geometrical and Hydraulic Characteristics
(Adapted from HEDD, 1978)

	800 MW and 2240 MW Schemes
Geometrical Characteristics	
Total length	4575 m
Depth	4.0 m
Invert width	1.2 m
Side slopes	2.5:1
Gross cross-sectional area	44.8 m^2
Gradient	0.02%
Hydraulic Characteristics	
Assumed friction factor	Manning n = 0.025
Flow Depth	
27 m^3/s (1000 year flood)	3.4 m
18 m^3/s (100 year flood)	2.9 m
Average Velocity	
27 m^3/s (1000 year flood)	0.82 m/s
18 m^3/s (100 year flood)	0.73 m/s

need to be replaced. In both schemes, the length of the section of canal to be replaced would depend on the location of the boundary of the pit as mining proceeded, which would depend in turn on pit slope stability considerations.

(c) F.R.P. Pipeline Sections and Creek Crossings

The hillside between Ambusten Creek and Medicine Creek is gullied and it slopes as steep as 30 per cent, giving rise to doubts about the practicability and the safety of a canal along this section. Construction of a 2.7 m diameter F.R.P. would be much simpler in this steeper area, since the overall width of a cut or an embankment is less for a pipe than for a canal. Moreover, since the radius of curvature for a pipe can be less than that for a canal, sharper bends could be made and the pipe could follow the original ground contours more closely.

A further advantage of using a pipeline for this section is that the creek crossings would be greatly simplified. Because the pipe grade does not need to be exactly horizontal, and the width of proposed foundation can be smaller than that for a canal, the embankments would be much reduced in size from those originally planned.

However, the use of a pipe does have some disadvantages. Firstly, large diameter pipe is expensive; secondly, the pipe, together with the canal-pipe transition structures, would suffer a head loss of about 6 m, considerably more than that of the canal, which would have a head loss of only 0.34 m over the same distance.

(d) Fibreglass Reinforced Pipe Discharge Conduit

The original concept of a corrugated steel discharge pipe has been replaced by 1.8 m diameter fibreglass reinforced pipe (F.R.P.) for greater durability. The routing would be around the eastern side of the coal blending area, rather than the west side as shown in the original HEDD (1978) report.

(e) Pit Rim Dam and Pumping

A pit rim dam and pumping system would be required to intercept runoff between the headworks and the pit rim; it would be necessary to pump the water up to the main diversion canal. The components would include an earthfill dam, emergency spillway, pumphouse and pipeline. The pit rim dam facilities and their locations would be very similar to those described in the HEDD Preliminary Design Report (1978).

(f) Minor Diversions

In the canal arrangement, a few minor diversions would be required to convey local inflows into the main diversion system. Two small diversions would be required where the diversion pipe crosses Ambusten Creek and Medicine Creek. Flow in these two small creeks would be intercepted by a small diversion structure, and would be channelled in a small diversion pipe discharging into the main diversion system.

The largest of the minor diversions would be a 2.7 km diversion of Finney Creek into the headworks reservoir. It is described in the HEDD Preliminary Design Report (1978).

4.3.3 Future Required Changes

On expansion of the pit with time, the pit boundary would infringe on the diversion canal route and part of the canal would have to be replaced with some other diversion method. The length of canal that would have to be replaced would depend on the final pit boundary, and would be significantly more for the larger 2240 MW pit than for the smaller 800 MW pit. For the larger 2240 MW pit, the canal would have to be replaced at an earlier stage in the life of the mine. The HEDD report of 1978 concluded that a realigned canal would be possible, but as discussed in Section 4.3.4 below, this is not thought to be practicable.

Two possible methods could be used to replace the canal in this area, either a tunnel or a pipeline. Of these two methods, the use of a tunnel would seem to be more logical. Some of the advantages of a tunnel arrangement would be its reliability and physical disassociation from the mining activities. The high cost, the main disadvantage of the tunnel, would be reduced, since tunnel construction need not begin for several years and these costs would be discounted to mine development costs at Year 1. Furthermore, by the time the tunnel would be required, the geology of the eastern pit area would be much better known, aiding tunnel design and construction considerably. The alternative arrangements incorporating a tunnel section are covered in Section 4.4.

The replacement of the central canal section with a pipeline would not offer significant advantages over using a pipeline as the primary diversion method from the beginning of mine development. The pipeline arrangements are discussed in Section 4.5.

4.3.4 Geotechnical Considerations

An important conclusion in the HEDD report of 1978 was that realignment of the canal onto the ultimate pit slopes, after some 12 years when the pit encroached on the canal, would be more economical than replacement by a tunnel or conduit. This presupposes that the ultimate pit slopes would be stable without creep movements. It is now felt that such a judgement cannot be made at this stage and, in fact, would not be possible until many years of mining experience had been gathered in this particular area. Therefore, it is concluded that replacement of the earlier canal by a realigned canal is not necessarily a practicable or economical solution. Our current knowledge indicates that it is unlikely to be a workable alternative and that the tunnel alternative would be needed.

It was also concluded that over the full length of the Hat Creek Diversion canal, seepage losses, if a plastic liner were incorporated, would be about 20 l/sec. In our opinion, much would depend on the care with which the plastic liner was installed and whether or not rupture would occur due to earth movements. Even so, the quantity of seepage is not of primary importance but rather it is the excess hydrostatic pressures that are set up in the pit walls by such seepage that are of concern. Therefore, the question of leakage in overall economic terms is linked to the question of its impact on pit slope stability. In the HEDD report, the following statement, Section 6.2, page 6-14, was made: "A canal lining combining both a plastic membrane and an impervious till lining is considered self-healing in terms of the movement anticipated in such areas." In our opinion, this statement is open to serious objection, when no estimate of the "movement anticipated" has yet been made.

Experience in the performance of recent "well engineered" canals would lend towards a cautious judgement regarding the likelihood of canal failures. Catastrophic collapses have taken place in three large new canal constructions in recent years. These are as follows (see references for details):

- Elbe Seiten Canal, West Germany, 1976 (NCE, 1976, and Hager, 1977)
- Nurenberg Canal, West Germany, 1979 (NCE, 1979)
- Ruahihi Canal, New Zealand, 1981 (NCE, 1981, and NCE, 1982).

In addition, a plastic liner failure caused a rupture in a reservoir at Kircheuim in Germany in 1977: the estimated damage cost $10 million.

The purpose in citing these examples is to point out that they occurred as a result of the oversight of apparently small design details, the results of which were minor seepages eventually leading to failures. Examples of canals in the French Alps at Lyonne and at Gap that have been destroyed by landslide movements are given in Gignoux and Barbier (1955).

Close to the area of the pit developments, major landslides up to 50 M^3 occurred towards the end of the 19th century near Ashcroft, B.C., in the glacial outwash gravel, sand and silt deposits on the Thompson River. These slides were caused by irrigation of the bench lands. The actual quantities of water involved are not known, although irrigation was thought to have been carried out by flooding open ditches The slides resulted in severe damage to the Canadian Pacific Railroad tracks and the railway eventually obtained an injunction to prevent the farmers from irrigating the land. A total of eight major landslides occurred.

The slides have been well documented by Stanton (1897 and 1904) and summarized by Skermer (1982). The soils are similar to the glacial outwash deposits that appear on the east side of the Hat Creek valley. The Ashcroft slides appear to be layered with slickensided bentonitic clay. At the time of writing this report (September 1982), CP rail was closed down for three to four days because of reactivation of one of these old slides in the glacial deposits on the left bank of the Thompson River. The movements were attributed to toe erosion by the river and irrigation by farmers of the benchlands above. The slide, over 300 m in length, was observed over a two-day period to be moving at an average rate of 30 mm per hour, after which it failed suddenly.

Although it was agreed that the primary cause of the earlier slides was the application of water to the land by means of irrigation, discussion ranged around the presence of clay beds at the bedrock contact. Some people believed that the set of sliding was in such a clay seam. Stanton, in his careful examination of the Great North Slide, concluded that no such clay seam exists, although boulder clay (till) underlies the silt and overlies the black shale bedrock. Stanton, therefore, concluded that the slide failed by softening of the silt as a result of increase in water content. Skermer, however, examined the slide debris at the site of the most northerly of the slides and found that, in fact, the silt was layered with very thin seams of clay. This clay is slickensided, indicating that sliding has taken place on these clay layers. The clay appears to be bentonitic in origin. Bentonite is the extremely weak clay mineral that is the set of the major landslides and soil creeps that are seen in the Hat Creek Valley above the coal deposits. In retrospect, it is not unreasonable that this type of clay mineral should be found redeposited as thin seams within the glacial lake sequences of clay layered silts found downstream in the Thompson River. Similar deposits of the bentonitic clay materials are found elsewhere upstream in the Bonaparte drainage basin and these, too, could have been washed into the glacial lake that occupied the Thompson River valley at the close of the glacial periods.

Similarly, irrigation of glacial outwash benchland on the right banks of the Thompson River just south of Spences Bridge caused a disastrous landslide in 1905. The slide swept rapidly across the river and dammed it for four to five hours. Ten people were killed (see Drysdale, 1913).

In our opinion, there is a serious risk involved in diverting Hat Creek in a canal around the perimeter of the pit where men and equipment are working below.

In the early years of mining, the weak clay rocks would not be exposed, and the pit slopes would be in layered glacial outwash materials consisting of silts, sands, and gravels. If leakage out of the canal did occur, it is quite possible that failure in the sands and gravels could take place as a result of erosion by piping of fine soils along preferred layers within those deposits or by sliding on thin clay seams as seems to be the case at Ashcroft. Piping is a common and well documented mode of failure of earth dams on layered granular soils. Furthermore, as mining progresses deeper into the pit, stress relief and creep of the underlying claystones might cause shear movements along the canal which could aggravate such leakages, and lead to canal rupture.

In summary, therefore, the geotechnical arguments against the canal are twofold. Firstly, leakage out of the canal could impair the stability of the pit slopes. Secondly, the reverse could happen, insofar as slope instability, unrelated to canal leakage, would lead to canal rupture. Although slope instability and canal failure are interrelated, in practice, cause and effect are likely to be inseparable issues. The pragmatic solution is to adopt another means for diversion of Hat Creek. Such an alternative means should be either leakproof, or alternatively, far enough removed from the pit slopes that leakage could not possibly impair stability. Alternatively, any means of diversion on, or close to, the pit slopes should be capable of accepting, without damage, slow creep movements in the foundation soils in the order of a few metres or more.

4.4 Canal/Tunnel/Pipe Alternative

4.4.1 General

Although the earlier Monenco and HEDD studies indicated that the canal close to the pit would probably have to be relocated into tunnel at a certain time as the pit encroached on the canal alignment, no detailed studies were carried out on that aspect. The current study gave detailed consideration to all arrangements which could involve a tunnel as a variation to either the canal or pipeline layouts for both the 800 MW and 2240 MW Schemes during the operational phases of the pit and for long-term abandonment. A tunnel layout has not been treated as a completely separate alternative but merely as a variation on the canal or pipeline arrangements described in Section 4.3.

A brief account is given in this section of the studies carried out to select a tunnel layout and for the choice of optimum routes within that layout. The preferred arrangement is described in detail. Appendix A (Volume 2) gives a complete account of the tunnel studies.

4.4.2 Layouts Considered

Three main tunnel layouts were considered for detailed study (see Drawing 5).

(A) A pressure tunnel running under the eastern side of the pit and driven sufficiently deeply below the pit to avoid interaction with slopes of a pit excavated to recover the total coal recovery. The tunnel would encounter surficial deposits and the Medicine Creek Formation.

(B) A free-flow tunnel driven along an alignment between the east margin of the pit and the eastern escarpment through weak volcaniclastics and surficial deposits.

(C) A free-flow tunnel at a higher level than layout B which would be driven largely through the volcaniclastic rocks of the eastern escarpment. Alternative routes within this overall layout could be chosen to avoid, or take advantage of, particular rock sequences.

It was also hoped at the outset of the study that a suitable tunnel could be selected which, in addition to providing the requirements of a safe and economic diversion, would also help to drain the eastern pit slopes by intercepting seepage from the escarpment.

Layouts A and B proved to have two main difficulties in common: they both would intercept considerable lengths of surficial deposits, probably under high heads of ground water (definitely in the case of A, possibly in the case of B); both tunnels would be driven partly through claystones and siltstones of a bentonitic composition with inherent problems of squeezing and slaking. An appraisal of the current tunnelling methods capable of dealing with high-head water inflows in granular surficial sediments (freezing, grouting and dewatering) showed that it would be impractical to attempt to drive a tunnel with such major constraints over the lengths and the depths being contemplated at Hat Creek. For that reason, these layouts were not considered further.

Layout C offered a choice of routes through the eastern escarpment with upstream portals either in the Medicine Creek or Hat Creek Valleys and downstream portals close to Harry Creek to the west of the escarpment. The initial evaluation of the layouts showed that Layout C was feasible and merited a detailed study. For that reason, a program of investigation was set up to assess the tunnelling problems which might be encountered, to select the appropriate method of tunnel excavation, to establish the parameters on which a tunnel design and costing could be based, and hence recommend the optimum route.

4.4.3 Tunnel Routes

The geology of the eastern embankment area was poorly known at the start of the study; an investigation was planned to obtain further data to enable a tunnel feasibility assessment to be made. This comprised geological mapping, geophysical survey, diamond drilling, field and laboratory testing. The details of the methods used and the results obtained are contained in Volume 2, Appendix A.

Once the geology had been accurately defined, the area was zoned into geotechnical units with distinct properties. These were designated G to G5 and they were ascribed "tunnelling quality indexes" which enabled them to be considered in relation to tunnelling methods (see Section 4.4.4).

Tunnel routes were then selected on the basis of topography and geologic reasons. Four routes were identified (T1 to T3A) and the proportions of the various geotechnical units were assessed. The routes were as follows:

Tunnel 1 would be driven primarily in surficial deposits but also for some distance through the Upper Volcaniclastics, they would be dry or under a modest head only. The route would be at the western edge of the escarpment. The upstream portal would be in Hat Creek.

Tunnel 2 would be driven through surficials and the stronger but brecciated rocks of the sequence, the altered andesites. The upstream portal would be in Hat Creek.

Tunnel 3 would be driven through the Lower Volcaniclastics at the eastern side of the escarpment but with an upstream portal in Medicine Creek.

Tunnel 3A would be driven through the Upper Volcaniclastics but from an upstream portal in the Hat Creek Valley. It would also be necessary to traverse the surficials and altered andesites.

Studies were undertaken on these four routes to chose the appropriate excavation method which would satisfy the requirements of all the geotechnical zones through which the tunnel would be driven. Cost estimates were then produced for all four alternatives in order to select the optimum tunnel route for inclusion in the main diversion studies for the canal/tunnel/pipeline comparisons (see Appendix A).

4.4.4 Tunnelling Methods

Comparisons have been made of the state-of-the-art methods of tunnel excavation and support relating to the various geotechnical units. Advice was provided by our two tunnelling consultants, Mr. A.A. Mathews in respect of "hard ground" tunnelling, and Dr. Z. Eisenstein in respect of "soft ground" tunnelling. The methods considered included the following:

- Hand excavation for surficial deposits where there is not a water problem;
- Shield-excavator for surficials or weak rocks where support is needed at the face but the ground can be excavated by a cutter or backhoe-type boom;
- Drill-and-blast for rock where it is uneconomic to invest in a machine;
- Part-face tunnel boring machine (road-headers) for suitable ground where a mechanized approach can be used but which requires flexibility;
- Full-face tunnel boring machine where uniform rock conditions exist and the capital cost can be justified.

Although particular excavation techniques are preferable for specific geotechnical units, it is generally impracticable to change tunnelling methods in any one alignment. In consequence, the method applicable to the dominant geotechnical unit is likely to be that for the complete tunnel. Thus, the following methods have been recommended for the four routes:

- T1 - Shield-excavator in surficials and rock; local drill and blasting;
- T2 - Hand excavation in surficials, drill-and-blast in rock;
- T3 - Shield-excavator in surficials and Lower Volcaniclastics; some local drill and blasting;
- T3A - Shield-excavator in surficials and Lower Volcaniclastics; drill and blast in altered andesites.

4.4.5 Tunnel Support

It has been assumed that all tunnel alternatives would require temporary support and final lining for hydraulic reasons; in some cases these might both serve the same purpose.

Techniques available include:

- Concrete segmental lining placed as an integral part of the excavation cycle;
- Rock bolting;
- Shotcrete with or without mesh;
- Cast-in-place concrete lining, locally reinforced;
- Steel sets.

After selection of the excavation method, it was possible to choose the appropriate tunnel support/lining methods. The conclusions were as follows:

T1 - concrete segmental lining

T2 - cast-in-place concrete lining in surficials and locally in rock, rock bolting, shotcrete and mesh in rock;

T3 - concrete segmental lining in surficials and lower Volcaniclastics, local rock bolting and shotcrete;

T3A - concrete segmental lining in surficials and lower Volcaniclastics; drill-and-blast with shotcrete locally.

All routes might require the use of steel sets over short sections.

4.4.6 Tunnel Design

From the evaluation of geotechnical behaviour during tunnelling, it has been concluded that either a circular or horseshoe-shaped tunnel would be admissable, since for long-term stability a concrete or shotcrete lining would be required and external ground pressures are not excessive.

A diameter of approximately 2.4 m (8 ft.) is required for hydraulic reasons and this is at the lower limit of the efficient use of men and machines in a heading. It is anticipated, therefore, that the tunnel would be driven at 3.0 m diameter.

Excavation would likely proceed from two headings although, if a machine were to be utilized, the larger part of the work would be carried out from one end only.

Tunnel portals have not been given detailed consideration in this feasibility study although they would need to in later phases.

4.4.7 Selection of Preferred Route

Comparison of the various alternative routes for a tunnel diversion have resulted in the recommendation of Route T3A. The details of the comparison are given in Appendix A.

The selection of the most appropriate tunnel route considers both the inherent tunnel characteristics and the tunnel as an integral part of the diversion scheme. For the purposes of route comparisons, differences in hydraulic operating efficiency and maintenance costs during the life of the structure are considered to be minor.

Factors considered in the selection of the preferred route include cost, remoteness from the pit, geological conditions and implied uncertainties and construction preferences. Since costs for Route T2 are within the range of T1 and T3, the first choice is primarily based on geological and construction conditions. Normally, the use of a machine for tunnel excavation, as opposed to conventional mining, contains a greater uncertainty on the construction outcome because of the inflexibility of machine operation. In this case, however, the excavation of Route T2 by drill-and-blast should consider the real possibility of serious problems arising from the combination of extensive lengths of tunnel of low RQD and adverse water conditions, complicated by the presence of a sub-parallel fault. Such ground water conditions are unlikely to present major problems in the more competent rocks traversed by T3 and T3A and, furthermore, the uniformity of the G4 geotechnical unit makes machine excavation reasonably reliable.

Thus, tunnel excavations by machine for Routes T1, T3 or T3A is preferred to conventional driving of Route T2, on the basis of certainty of construction outcome. An added benefit of this choice is the ability to utilize a precast concrete lining for both construction and operational functions. This lining method is most suited to a free-flow or low pressure tunnel for the present geological conditions.

The comparison of Routes T3 and T3A considers the saving in cost of tunnel ($2 million) for the shorter route, relative to the greater costs of pipeline/canal and earthworks structures at the Medicine Creek crossing. It is estimated that the extra costs for pipeline/canal/earthworks associated with the shorter tunnel route are less than $2 million, especially if the favoured alternative of the pipeline, instead of canal, is considered. Major aspects in this comparison are the uncertainty related to the slide zone identified on Route T3 and the variable geology within the block faulted zones in that area of tunnelling. Since no other major factors influence the comparison of T3 and T3A routes, Route T3 is eliminated from further consideration.

The greater cost of Route T3A compared to that for Route T1 is essentially a consequence of its greater length; as noted earlier, unit costs are very similar. This cost difference can be directly compared to two major differences between the two routes:

(1) proximity to the ultimate pit rim;
(2) differences in construction problems as a consequence of having to cross the altered andesite unit twice.

Route T1 is located approximately 400 m distant from the pit rim. The adequacy of this separation must consider the potential for seepage from the tunnel modifying the ground water conditions around the pit, the possibility of deep-seated pit slope failures affecting the tunnel, and the uncertainty regarding the ultimate position of the pit rim. This last factor is influenced by the life of the scheme, the discovery of new deposits, and modified pit slope angles. It should be noted that unless special precautions and lining construction practices are adopted, some seepage through the precast lining into the rock is to be expected.

The geological investigations indicate that the contacts between the andesite and the Upper and Lower Volcaniclastics that would be crossed by Route T3A are not expected to present significant tunnelling problems. An allowance for the different tunnelling conditions in the andesite unit has been made in the construction and cost estimate. Thus, on the basis of the above discussion of the various factors affecting the choice of the tunnel route, it is recommended that Route T3A be adopted as the preferred alternative.

4.4.8 Canal/Tunnel/Pipe Arrangements

In addition to the tunnel route (described above), the principal components of the tunnel diversion arrangement are given below. Drawings 6 and 7 illustrate this arrangement for the 800 MW and 2240 MW Schemes, respectively. The arrangement would be the same for both schemes. Table 4 summarizes the diversion tunnel parameters.

(1) Headworks Dam and Intake

The design and location of the headworks dam for the tunnel arrangement would be identical to that of the canal arrangement described earlier.

(2) Headworks Dam to Tunnel Portal

The diversion of Hat Creek from the headworks dam at elevation 975 m to the tunnel portal at 962 m follows essentially along

TABLE 4

Tunnel Diversion Parameters

	800 MW and 2240 MW Schemes
Intake and diversion dam	
Max. reservoir water level	976 m
Average dam height	15 m
Intake to Ambusten Creek	
Diversion method	Canal
Length	1,500 m
Mean gradient	0.02%
Ambusten Creek to tunnel portal	
Diversion method	2.4 m dia F.R.P.
Length	2,230 m
Mean gradient	0.63%
Tunnel Section	
Type	Concrete Segmental Lining
Diameter	2.4 m (probably 3.0 m driven)
Length	3,395 m
Mean gradient	0.59%
Discharge conduit (west)	
Type	F.R.P. or C.P.P.
Diameter	1.8 m dia
Length	1,950 m
Mean gradient	6.4%
TOTAL LENGTH OF DIVERSION	9,075 m

the same route as for the initial portion of the canal arrangement. This is shown on Drawings 6 and 7, and described in Section 4.3.2.

The use of large diameter pipes to the tunnel portal was also considered. However, the limited allowable head loss would require the use of oversize and, hence, expensive pipes.

(3) Discharge Conduit

From the tunnel exit portal close to Harry Creek at approximate elevation 942 m, the water would be returned to the original Hat Creek channel in a conduit, similar to that for the canal arrangement. Two pipeline routes were considered and are shown on Drawings 6 and 7.

The west discharge conduit route, which is preferred, would travel along the west side of Harry Creek skirting around the coal blending area and then discharging to Hat Creek, just upstream of the Indian Reserve boundary. This route is relatively short, but the pipe would have to be kept away from the coal stockpiles, since it would be costly to design the pipe for high overburden pressures and repair of the pipe would be difficult. Fibreglass Reinforced Pipe (F.R.P.) or Corrugated Polyethylene Pipe (C.P.P.) would be used. An alternative discharge conduit route wast of Harry Creek was also considered, but this would be 360 m longer (18 per cent).

(4) Pit Rim Dam and Pumphouse

A pit rim dam and pumphouse would be required immediately upstream of the pit to intercept seepage and local inflows. A pumphouse and a pipeline would be provided to pump the water up to the main diversion.

(5) Minor Diversions

The same minor diversions as for the canal arrangement would be required for the tunnel scheme. These were discussed in Section 4.3.2.

4.5 Pipelines

4.5.1 General

Pipeline arrangements would use a combination of two or three polyethylene pressure pipes laid on pit benches to divert Hat Creek water past the pit. Initially the pipe would be laid alongside the present Hat Creek channel and, as the pit developed, the pipes would be moved further out. It should not be necessary to have more than a limited number of pipe moves during the life of the pit. They would be moved one at a time during periods of low flows when one pipe would be empty.

The pipeline arrangement is simple and economical and Hat Creek would remain in the valley, where in a sense it "belongs". Different pipeline arrangements are shown in Drawings 8 to 12.

4.5.2 Pipe Materials

Several different pipe materials were investigated for their suitability for use in the Hat Creek Diversion. In recent years, new types of pipe materials have become available. The pipes considered were large diameter polyethylene pressure pipe, fibreglass reinforced pipe and corrugated polyethylene pipe. More conventional materials such as corrugated steel pipe and concrete pipe were also considered. Table 5 summarizes the main advantages and the disadvantages of these different materials.

TABLE 5

Comparison of Pipe Materials

Polyethylene Pressure Pipe

Advantages:	Very tough, durable, lightweight and easily movable, can be free standing above ground installation, leakproof, continuous joints (butt fusion).
Disadvantages:	High cost, maximum diameter is 1.5 m, joints difficult to test.

Fibreglass Reinforced Plastic Pipe

Advantages:	Durable, leakproof, available in larger diameters up to 6 m, double O-ring joints can be pressure tested, easy to repair, lightweight.
Disadvantages:	High cost, not easily movable since pipe is not as tough.

Corrugated Polyethylene Pipe

Advantages:	Tough, durable, flexible, lightweight.
Disadvantages:	High cost, limited experience and availability, low pressure range.

Corrugated Steel Pipe

Advantages:	Low cost, lightweight, wide experience.
Disadvantages:	Leaks, not as durable, not easily movable.

Concrete Pipes

Advantages:	High pressure capability, high external loads, wide experience.
Disadvantages:	High cost, rigid, heavyweight, not movable.

Note: Toughness refers to high impact resistance.

Polyethylene Pressure Pipe

Polyethylene pressure pipe is manufactured from a high molecular weight, high density resin, and is available in Canada from Dupont as Sclairpipe or from Phillips Petroleum as Driscopipe.

Polyethylene pressure pipe is an extremely tough (high impact resistance), durable, flexible and light-weight material. It absorbs impact loads over a wide temperature range allowing simple moving and handling, installation and above ground placement. The pipe is available in inside diameters of up to 1.5 m (59 inches) and is designed for internal pressures of up to 0.31 MPa (45 psi). The nominal diameter of the pipe is 1.6 m (63 inches OD). Short lengths of pipe can be joined by butt fusion into a number of 100 to 200 m continuous lengths, connected by bolted, flanged couplings.

In general, it is a reliable material and it is widely used in the mining industry for tailings pipe lines. Its chief disadvantages are the high cost and the lack of sizes above 1.5 m diameter. Also important is the problem that there is no simple method to test the butt fusion joints, although when properly carried out they are as strong as the pipe itself.

For the Hat Creek diversion, polyethylene pressure pipe has the advantage that it could be installed on the ground surface without burial and it could be readily moved as necessary by unbolting the flanged connections, and hauling the pipe with a 'dozer to its new location.

Fibreglass Reinforced Pipe

Fibreglass Reinforced Pipe (F.R.P.) (also known as Fibreglass Reinforced Plastic Pipe) consists of a plastic pipe around which is wound a tape made of continous glass fibre strands impregnated with resin. The pipe is smooth on the inside and the maximum pressure capacity of the pipe is determined by the thickness of the pipe wall, and can be specified over a wide range.

The advantage of F.R.P. is that it is available in larger diameters than polyethylene pressure pipe and, therefore, friction losses can be minimized. However, it is not as robust as polyethylene pipe, and it would either have to be supported on the sides by earth beams, or buried in a shallow trench. It is not, therefore, as easily movable. F.R.P. would be most suitable for conveying Hat Creek water along the approaches to the pit, and downstream of the pit where the pipe would not have to be moved during pit expansion. Large diameter F.R.P. has been used frequently for penstocks in hydro-electric projects, for example, 450 m of 2.7 m diameter F.R.P. was recently supplied to Ontario Hydro.

Corrugated Polyethylene Pipe

The third type of pipe considered was a new product called Spirolite made by Gulf Plastics Division in Georgia, USA. It is a corrugated pipe, smooth on the inside with reinforcing corrugations on the outside. Its advantages are that it is available in larger sizes than polyethylene pressure pipes and it is tougher then F.R.P. It is also somewhat more flexible. As for F.R.P., it is intended to be buried. The main disadvantage is that it is only able to withstand limited internal pressures. Only limited experience with the material is available to date and the pipe is not yet available from a Canadian manufacturer.

Other Pipe Materials

Materials also considered but rejected were corrugated steel pipe (CSP), precast concrete sewer pipe and prestressed embedded concrete cylinder pipe. Much experience is available with the use of these materials. Corrugated steel pipe is attractive because of its low costs, but it is not as durable as the other available products. More importantly, it is not leakproof and it could, therefore, contribute to pit slope instability through slow, undetected water leakage. For the tunnel alternative, CSP pipe could be used for the east discharge conduit since it would be located far from the pit and leakage would not impair pit slope stability.

Precast concrete and prestressed concrete pipes were considered but were rejected because of their high cost, very heavy weight and because they would be difficult to move about.

Recommended Pipe Material

Two types of pipe material are recommended for the Hat Creek Diversion. Within or around the pit, polyethylene pressure pipes would be used. No other pipe currently available would be suitable because the pipe in this section must be robust, leakproof and lightweight enough to permit easy pipe moves.

On the approaches to the pit and downstream of the pit, fibreglass reinforced pipe would be used. It was selected over polyethylene pressure pipe bcause F.R.P. is available in larger diameters so that a single pipe could used; it was preferred to corrugated polyethylene pipe because of F.R.P.'s pressure capabilities.

With the possibility of newer pipe materials in the future, the selected pipe materials should be reinvestigated for the final design.

4.5.3 Pipe Diversion Layouts

A number of pipeline diversion layouts were considered ranging from a simple low-level diversion scheme to more complex higher-level diversion schemes. Generally, the higher the pipeline is in elevation and the further the diversion from the valley bottom, the more complex and more expensive it becomes. Four of the pipeline diversion layouts considered are discussed below.

Low Level

The low-level pipeline diversion layout would be the simplest and the most economical method of diverting Hat Creek. It has been limited to the 800 MW Scheme. An overview of the low-level diversion arrangement is shown in Drawing No. 8. Hat Creek would be diverted by a low dyke just upstream from the pit rim into twin polyethylene pressure pipes. Initially, the pipe would be laid along the valley bottom, but as the pit excavation proceeded, it would be necessary to relocate the pipelines on the pit benches. Immediately past the downstream pit rim and the conveyor embankments, the water would be discharged into the original Hat Creek channel. A leachate lagoon by-pass conduit would be provided, located on the eastern side of the valley and slightly below the normal leachate lagoon level. During extreme floods, Hat Creek water would be required to pond upstream of the leachate lagoon to provide sufficient head to convey it through the by-pass conduit. Below the leachate lagoon, the water would be returned directly into the original Hat Creek channel. This channel might be relocated slightly to avoid the sedimentation ponds.

The advantages of the low-level diversion method would be its low cost and its simplicity. The principal disadvantage would be that the leachate lagoon by-pass conduit would lie below the normal leachate lagoon level. Since failure of the leachate lagoon would have very serious consequences, it would be desirable to have the by-pass conduit at a higher elevation than the maximum lagoon level and to one side. Another disadvantage would be that the section of diversion between the conveyor embankments and the leachate lagoon would be open channel and therefore subject to possible contamination from mine activities.

Mid Level

The mid-level pipeline diversion layout would be a compromise between the low-level route and a much higher diversion route. The mid-level diversion arrangement is shown for the 800 MW and the 2240 MW projects on Drawings No. 9 and 10, respectively. The two main disadvantages of the low-level layout discussed above would be eliminated. A summary of the pipeline diversion parameters for the mid-level layout are shown on Table 6 for both the 800 MW and 2240 MW Schemes.

Hat Creek would be directed into a single large diameter fibreglass reinforced pipe (F.R.P.) by a diversion dam which for the 800 MW Scheme would be located a short distance upstream of the pit rim boundary. Since additional head would be required for the mid-level route in comparison with the low-level route, the dam would be higher. For the 2240 MW project, the dam location would be moved upstream to Medicine Creek to gain the required elevation. The fibreglass reinforced pipe would carry Hat Creek from the dam to the pit rim. Twin polyethylene pressure pipes would carry the flow around the pit. As before, the polyethylene pipes would be laid on the pit benches, and they would be moved as the pit excavation proceeded. From the conveyor embankments, the flow would be carried in a single, large diameter fibreglass reinforced pipe to a point beyond the leachate lagoon. Beside the leachate lagoon the pipe would be located above the maximum lagoon level and separated from the lagoon by a roadway. This location would facilitate the repair or maintenance of either the lagoon or the pipe without each affecting the other.

The mid-level route would offer the advantage of simplicity; moreover, the problem associated with proximity of the diversion to the leachate lagoon, would be satisfactorily treated.

High Level (20 m^3/s)

The high-level diversion layout is shown on Drawing 11, illustrating the many different combinations of pipes, routings, and design flows that would be possible.

TABLE 6

Pipeline Diversion Parameters
(Mid-Level Layout)

	800 MW Scheme (m)	2240 MW Scheme (m)
Intake and diversion dam		
Maximum reservoir water level	898	923
Average dam height	10	15
Intake to pit rim		
Pipe type	F.R.P.	F.R.P.
Pipe diameter	2.4	2.4
Length	420	1080
Within Pit		
Pipe type	Twin PPP	Twin PPP
Pipe diameter	1.5	1.5
Initial pipe length	2 x 1470	2 x 2050
Final pipe length	2 x 1700	2 x 2840
Approx. number of pipe moves	3 to 4	Approx. 8
Embankment section		
Number of embankments	1	2
Pipe type	Twin PPP	Twin PPP
Pipe diameter	1.5	1.5
Length	2 x 500	2 x 730
Embankment to past leachate lagoon		
Pipe type	F.R.P.	F.R.P.
Pipe diameter	2.4	2.4
Length	1155	635
Discharge conduit after leachate lagoon		
Pipe type	F.R.P.	F.R.P.
Pipe diameter	2.1	2.1
Length	135	135
Open Channel		
Approximate length	600	600
Total length of diversion		
Initial length	4280	5230
Final length	4510	6020

Note: F.R.P. is Fibreglass Reinforced Pipe
PPP is Polyethylene Pressure Pipe

A dam would be provided upstream of the pit near Medicine Creek to raise the water in Hat Creek to a level that would allow it to flow in a canal or pipeline along the side slopes of the first bench above the valley floor. In the pit, the flow would be accommodated in twin polyethylene pipes laid on benches. The pipes would either slope down to and along the valley bottom as described for the mid-level scheme, or they could be extended to discharge into Hat Creek further downstream, thus avoiding the leachate lagoons.

With the 800 MW mine development plan, it might not be necessary to move the pipe during the entire life of the mine. The bench on which the pipes would be located is scheduled for construction in year 3, but excavation of this bench initially would involve only a relatively minor change in mine plan. It might be possible to leave the creek in the valley bottom for the first one or two years of operation, either in the original location or carried in twin polyethylene pipes as in the other pipeline arrangements.

With the arrangement as illustrated, the capacity would be about 20 m^3/s which approximately represents the 180-year flood. This seems a reasonable level of safety by normal standards, but if the scheme were adopted, it would be necessary to carry out much more thorough risk-cost analyses to find the optimum design.

For the larger 2240 MW pit, the polyethylene pipes within the pit would have to be moved several times as the pit expanded and hence much of the attractiveness of the high-level route is lost. Furthermore, the longer length of pipe required to convey the flow around the larger pit would use up additional head. This means that the diversion downstream of the pit would need to be located close to the valley bottom and, therefore, closer to the leachate lagoons. Since the high-level route for the 2240 MW project would offer no advantages over the mid-level route described earlier, the high-level route for the 2240 MW project was not considered further.

Triple Pipe

The triple pipe diversion layout is shown on Drawing No. 12. It would use three polyethylene pressure pipes around the pit rather than two. Three pipes would offer considerably more capacity and less head loss than two pipes and, therefore, the scheme would allow the pipes to exit from the pit at a higher elevation. This would allow much more flexibility in locating the exit from the pipe and thus routing around the congested conveyor embankment region would be simplified. The increased pressures available would also permit extra flexibility in pipe routing near the leachate lagoon. However, the cost of an additional pipe would be substantial and since with careful mine planning a twin pipe system would be sufficient, the three pipe system was not studied further.

4.5.4 Diversion Dams

For the mid-level pipeline layouts, a diversion dam would be located a few hundred metres upstream of the pit. Most of the time, the dam would not be required to impound water. The height of the dam was chosen to provide sufficient head to ensure full flow in the diversion pipe for the design flow. The locations for the 800 MW and 2240 MW Schemes and typical cross-sections are shown on Drawing 13. A better location for the diversion dams for the 2240 MW Scheme would appear to be about 200 m upstream of the location proposed by HEDD (1978) for the pit rim dam. This is just downstream of the confluence of Medicine Creek and Hat Creek. This location would appear to have advantages over one further downstream namely better abutments for the dam and lower dam height. This would result in a reduced dam volume, for the same dam crest elevation, the level of the valley bottom being about 5 m higher at this upstream location. The crest length of the dam would be about 50 m less compared to that at the downstream location. Furthermore, on the left abutment at the downstream pit rim dam location, a recent soil slide has occurred. Drillholes have been sunk at the proposed pit rim dam location and for final design further drillholes would need to be put down at the upstream location shown here. However, assuming similar foundation conditions, the dam cross-section shown on Drawing 13 has been designed. A glacial till core inclined slightly upstream is proposed together with sand and gravel shells. The core width has been designed to the standard criterion that at any elevation it should not be less than the height of embankment above that elevation. For a dam designed to the crest elevation of 925 m at the upstream location of the 2240 MW Scheme, the volume of fill would be about 25,000 m^3.

By suitable selection from the copious borrow materials available, it would be possible to eliminate the need for filters which are difficult to construct in small dams. Consequently, steep downstream slopes would be possible using a rockfill downstream toe and pumping from the proposed under-seepage control pressure relief wells. These wells would be spaced at approximately 15 to 20 m intervals and located at the downstream toe of the dam. Approximately 12 such wells would be needed; their depth would vary between 10 to 30 m. If they were taken down to the claystone, they would minimize subsurface flows from the Hat Creek Valley into the open pit. The minimum diameter of the wells would be 0.15 m and they should be surrounded by a gravel filter pack of 0.15 m minimum thickness. Water pumped from the wells could be directed into the diversion pipe downstream of the valve. Details for the final designs of the diversion dam can be found in USBR (1973).

The intake pipe, which is shown on Drawing 13, would consist of a concrete pressure pipe located beneath the dam in line with the existing Hat Creek channel upstream.

A reinforced concrete intake structure would be located at the upstream toe of the dam. A small settling basin would be required upstream of the intake to settle out bed load in the creek. The trash rack in front of the intake would be of sufficient area to minimize velocities and hydraulic losses through it. The trash rack surface would be sloping to permit floating debris to collect at the water surface. An access road leading down to the intake would be required for trash rack cleaning. The intake itself would have tapered wing walls and a well rounded entrance to keep the entrance losses as low as possible. A reinforced concrete transition structure would convey the flow from the rectangular intake to the circular concrete pressure pipe under the dam.

The end of the concrete pressure pipe downstream of the dam would be flanged for attachment to the F.R.P. Stop logs and a butterfly valve would be provided to allow for short-term inspection and repair of the F.R.P.

Under operating conditions with the water level at the top of the pipe at the intake, the capacity would be about 12 m^3/s. Thus, in general, compared to the maximum flood on record in Hat Creek of 14.6 m^3/s, it can be seen that the diversion dam would rarely pond water. If the dam were to impound water to the design flood elevations shown on Drawing 13, the diversion would carry the 1000-year design flood of 27 m^3/s. During a 35-year mine life, the probability of this occurring would be 3-1/2 per cent.

Flows in excess of the design flood would be discharged over an emergency spillway. A spillway constructed of placed rockfill should be adequate, and a suitable location would be around the centre of the dam in line with the existing Hat Creek channel on the downstream side. The spillway would rarely, if ever, be used, and more elaborate designs using reinforced concrete, reinforced rockfill or gabions do not seem warranted. However, an alternative, less expensive method of constructing a lined spillway channel would be to use shotcrete and a more detailed study should be made on the design slope. Basically, this would consist of a shotcrete channel lined with conventional concrete side walls running down the surface of the dam. The toe of the apron would be constructed of conventional concrete and would be of sufficient depth to prevent undermining of the apron. The use of shotcrete simplifies placement and its use would be economical if a concrete plant were located nearby.

As an alternative design to a zoned earthfill dam, consideration might be given to an overflow-throughflow rockfill diversion dam employing an impermeable membrane on the upstream face made of clay, bitumen mastic or concrete. Depending on the availability of rockfill, such a dam might be simpler and less expensive than a compacted zoned embankment. In that case, the spillway could be eliminated, since the rockfill embankment acts as such. Details can be found in Stephenson (1979).

5.0 COSTING OF DIVERSION ARRANGEMENTS

5.1 Basis For Costing

An experienced independent estimator, Bellevue Consultants Inc., was retained to provide detailed up-to-date cost estimates for the canal and tunnel diversion arrangements; those two reports have been submitted separately to BCH. The pipeline costs were estimated by Sigma Engineering Ltd. All cost estimates are in 1982 dollars and are compatible in terms of labour rates, profit, overhead and contingency markups.

The earthwork costs provided by Bellevue Consultants and price quotations from pipe manufacturers were used as the basis for unit costs for different elements of the pipeline diversion arrangement produced by Sigma.

5.2 Cost of Diversion

The capital costs of the different diversion arrangements are given in Tables 7 to 10 and are summarized below:

Canal arrangement	800 MW and 2240 MW	$ 26 Million
Tunnel arrangement	2240 MW	$ 48 Million
Pipeline arrangement (mid-level layout)	800 MW	$ 16 Million
Pipeline arrangement (mid-level layout)	2240 MW	$ 19 Million

Allowances have been made for engineering (15 per cent), contingencies (20 per cent) and corporate overhead (5 per cent). No allowances were made for inflation, operation, pipe moves or future canal relocation. Costs for the pipeline layout include both initial and final capital costs when the total length of pipe has been installed.

These costs have been taken into account in the selection of the recommended diversion arrangement discussed in Section 6 of this report.

6.0 SELECTION OF RECOMMENDED DIVERSION ARRANGEMENT

6.1 General

The selection of the recommended diversion arrangement was made by comparison of the alternatives on the basis of cost, potential problems, constructional and operational aspects. The three diversion arrangements are compared in tabular form in Table 11.

6.2 Capital Cost

The cost of the tunnel arrangement ($48 million) is almost twice the cost of the canal arrangement ($26 million) and three times the cost of the pipeline arrangement ($16 to $19 million). The high cost of tunnel is a major adverse factor in its consideration as a preferred diversion alternative.

TABLE 7

Canal Diversion Costs for 800 MW or 2240 MW Schemes
(All Costs at 1982 Price Levels)

	$ Thousands
Headworks Dam	
Dam	1,230
Spilllway	540
Diversion Canal/Pipe	
Intake	190
Canals	3,440
Pipe	3,500
Creek Crossings	1,600
Pipe - Canal Transition Structures	60
Diversion Conduit	
Intake	330
Pipe	3,340
Outlet Works	140
Pit Rim Dam	
Dam	1,980
Spillway	490
Pumphouse and Pipeline	270
Finney Creek Diversion	
Headworks Structure	90
Canal	750
Outlet Structure	180
SUBTOTAL	18,130
Engineering, Contingencies and Overhead (totalling 45 per cent)*	8,158
TOTAL	$ 26,288
TOTAL INITIAL COST	$ 26 Million

* This is made up of Engineering 15 per cent; Contingencies 20 per cent, and Corporate Overhead 5 per cent, all compounded, as in the original design cost estimates by HEDD (1978).

TABLE 8

Canal/Tunnel/Pipe Diversion Costs for 2240 MW Schemes
(All Costs at 1982 Price Levels)

	$ Thousands
Headworks Dam	
Dam	1,230
Spilllway	540
Diversion Canal/Pipe	
Intake	190
Canal	1,100
Pipe	3,500
Creek Crossings	1,600
Canal - Pipe Transition Structures	30
Discharge Conduit	
Pipe	2,230
Outlet Works	140
Pit Rim Dam	
Dam	1,980
Spillway	490
Pumphouse and Pipeline	270
Finney Creek Diversion	
Headworks Structure	90
Canal	750
Outlet Structure	180
SUBTOTAL	14,320
Tunnel (T3A Alternative)	19,080
SUBTOTAL	33,400
Engineering, Contingencies and Overhead (totalling 45 per cent)*	15,030
TOTAL	$48,430
TOTAL CAPITAL COST	$ 48 Million

* This is made up of Engineering 15 per cent; Contingencies 20 per cent; and Corporate Overhead 5 per cent, all compounded, as in the original design cost estimates by HEDD (1978).

TABLE 9

Pipeline Diversion Costs for 800 MW Scheme
Mid-Level Layout
(All Costs At 1982 Price Levels)

	$ Thousands Final Location	Initial Location
Intake		
Embankment	$ 780	$ 780
Intake Structure	140	140
Spillway - Emergency	70	70
Pipeline		
Fibreglass Reinforced Pipe	820	820
Polyethylene Pressure Pipe	3,200	2,800
Pit Rim - Embankment Region	2,800	2,800
Leachate Lagoon Bypass	2,700	2,700
Open Channel	410	410
SUBTOTAL	$ 10,920	$ 10,520
Engineering, contingencies and overhead (totalling 45 per cent)*	4,914	4,734
TOTAL	$ 15,834	$ 15,254
TOTAL CAPITAL COSTS	$ 16 million	$ 15 million

* This is made up of Engineering 15 per cent; Contingencies 20 per cent; and Corporate Overhead 5 per cent, all compounded as in the original design cost estimates by HEDD (1978).

TABLE 10

Pipeline Diversion Costs for 2240 MW Scheme
Mid-Level Layout
(All Costs At 1982 Price Levels)

	$ Thousands Final Location	Initial Location
Intake		
Embankment	$ 1,200	$ 1,200
Intake Structure	170	170
Spillway - Emergency	90	90
Pipeline		
Fibreglass Reinforced Pipe	1,900	1,900
Polyethylene Pressure Pipe	5,400	3,900
Embankment Region	2,340	2,340
Leachate Lagoon Bypass	1,800	1,800
Open Channel	410	410
SUBTOTAL	$ 13,310	$ 11,810
Engineering, contingencies and overhead (totalling 45 per cent)*	5,990	5,314
TOTAL	$ 19,300	$ 17,124
TOTAL CAPITAL COSTS	$ 19 million	$ 17 million

* This is made up of Engineering 15 per cent; Contingencies 20 per cent; and Corporate Overhead 5 per cent, all compounded, as in the original design cost estimates by HEDD (1978).

TABLE 11

Comparison of Diversion Arrangements

Capital Cost	Canal	Canal/Tunnel/Pipe	Pipeline
800 MW	$26 million	$48 million	$16 million
2240 MW	$26 million	$48 million	$19 million
<u>Potential Problems</u>			
Sudden failure	Canal breach possible	Canal section - possible Tunnel section	Butt fusion joint failure, physical damage
Slow leakage	Probable	Canal upstream of tunnel	None
Pit slope stability	Decreased	Possibly increased	No effect
Ease of repair	May be hard to locate leaks, but repair is simple. Repair of canal breach difficult.	Tunnel repair difficult except at low flows	Easy to locate and repair leaks
<u>System Components</u>			
Dams	Headworks and pit rim dams	Headworks and pit rim dams	Diversion dam
Pumphouse	Required at pit rim	Required at pit rim	None
Creek crossings	Ambusten and Medicine Creeks	Ambusten and Medicine Creeks	None
Minor diversions	Finney, Ambusten, Medicine Creeks	Finney, Ambusten, Medicine Creeks	2240 MW Scheme only: short Finney Creek diversion
Interference with mine	Coal blending areas	Coal blending areas	Open pit mine, pit exit region, leachate lagoon
Ease of construction	Simple	Tunnelling more complex than alternatives	Simple
<u>Operational Aspects</u>			
Inspections	Required	Required	Required
Maintenance frequency	High	Moderate	Moderate
Adaptability to changes in mine plans	Not readily adaptable, rigid	No conflicts with fore-seeable mine plans	Flexible
Interference with mine operation	Moderate to low	Minimal	Moderate

6.3 Potential Problems

6.3.1 Pit Slope Failures

The reliability of the diversion system is of primary concern since the consequences of failure could be very serious, resulting in shutdown of the mine and powerplant and danger to men and equipment working in the pit below.

The main geotechnical issue is the influence on pit slope stability as the result of the implementation of any of the various diversion alternatives. The insidious nature of leakage from a canal around the pit on pit slope stability is a concern. Plastic liners are not leakproof, since in construction practice it is virtually impossible to seal the joints. Futhermore, leakage tends to be concentrated at specific locations and this may be particularly harmful in terms of inducing slope instability.

The first real evidence of instability would, in fact, be slope movement. For this reason, a canal around the pit is not recommended. A canal failure upstream would likely be less damaging, since although it would discharge into the pit, the sudden flow of water would not necessarily in itself induce pit slope instability.

Similarly, although failure of a pipeline would cause a flood into the pit, it would not, in general, lead to pit slope instability unless the flow continued for a long-term saturating the ground.

The difference from a geotechnical viewpoint between canal and pipeline failures is that the pipe does not leak slowly, although it could perhaps burst and that would be readily detectable. It would not, however, cause long-term buildup of pore water pressures in the slope materials, which is the factor leading to slope instability. Experience with pipes used for tailings disposal shows that pipes can fail when they are empty, if joints pull apart because of temperature contractions. However, water flowing in the pipes moderates the temperature variations, and contractive failure is less likely. In practice, contractive failures are guarded against by snaking the pipe to increase the effective length.

6.3.2 Canal

The canal could fail by sudden breach of the embankment, or by slow leakage. Embankment breach, which would result in flooding of the pit, could be caused by high flows overtopping and eroding the embankment or by piping through an embankment damaged by ice, animals or vegetation. For most monthly Hat Creek flows, the pit perimeter drainage collection system could be sized to handle an emergency spill from a breach. Emergency repair of a breached canal embankment should be possible during operation since adequate earthmoving equipment would be available at the mine site. Canal reliability against sudden failure could be significantly increased by a regular inspection and maintenance program.

The most serious disadvantage of the canal method of diversion relates to pit slope stability (see Section 6.3.1 above). It is the effect of slow leakage from a canal around the pit slope that is the major cause of concern. Sudden failure of the canal could be seen and dealt with, and might only result in a concentrated flow of water towards the pit. Undetected leakage might go on for years and might result in uncontrollable pit slope failure. Only with the most extensive monitoring equipment could canal leakage be detected.

The repair of slow leakage, although generally not an emergency, would be difficult and would present two problems. Firstly, the section of canal under repair would have to be drained of water. During low flow months, this could be done using the mine drainage system or a small temporary by-pass pipeline and a pump. The more difficult problem would be locating the leak. A considerable length of canal might have to be reconstructed in order to be sure that the leak had been repaired.

6.3.3 Pipeline

In terms of maintenance, the advantage of the pipeline arrangement would be that by using two pipes, one pipe could be shut down for repairs while the other pipe continued to divert the flow. The mine drainage system in the region of the pipelines could also be sized to handle, on an emergency basis, average Hat Creek flows.

For the pipeline arrangement, it is possible for either a pipe joint or the pipe itself to fail. Experience with polyethylene pipe has shown that failure occurs most commonly at a butt fusion joint. Usually, such failures have been traced to poor jointing procedures in cold weather conditions. Unfortunately, there is presently no reliable method for testing butt fusion joints after they are made, but reliability can be increased by ensuring good joining procedures.

Failure of the pipe itself rarely occurs, and when it does it is usually due to accidental mechanical damage, for example, by being driven over by construction equipment. Such physical damage could be minimized by warning signs, and protective ditches, berms and earth covers over the pipe. Should the pipeline be located over a coal seam, or rock surface, sufficient bedding material would be provided to cushion and insulate the pipe from a possible coal fire.

The fire hazard was considered, and apart from coal seams possibly being exposed in the northern corner of the pit at about elevation 850 m, the pipeline would not lie on coal benches. Furthermore, experience suggests that spontaneous combustion does not take place in in situ coal, i.e., on exposed coal seams, but rather in stockpiles of excavated coal (Dr. B. Dutt, personal communication). The fire hazard would not, therefore, appear to be a serious risk.

6.3.4 Tunnel

Although the tunnel would be the most reliable diversion arrangement, it could also be subject to possible failure. Experience shows that most difficulties occur at or near tunnel portals. Careful consideration would need to be given to designs. Upstream of the tunnel, on the steeper side slopes, the canal and the pipe sections could be subject to failure.

Repair of a tunnel problem would be exceedingly difficult, and it would be unlikely that the mine drainage system would be able to handle the Hat Creek flows for the length of time needed for repairs to be completed.

6.4 System Components

Another major factor in the choice of a diversion arrangement is the siting of the system components in relation to the topography and the planned mine structures. In most cases, simple arrangements result in greater reliability since there are fewer components to give problems.

Diversion dams would be relatively expensive structures and their size should be minimized. Ponding of water immediately above the pit rim would be undesirable because of the effect on pit slope stability.

The canal arrangements and the tunnel alternative would require both a headworks dam and a pit rim dam. Both dams would pond substantial amounts of water behind them. The pit rim dam would also require a pumphouse and pipe line to pump water up to the main diversion.

The pipeline arrangement would require only one diversion dam. It would pond water only in flood, since water would simply flow into the pipe intake structure at heads up to the top of the pipe for flow up to about 12 m^3/s. This is seen to be a significant advantage.

The canal and tunnel arrangements would require that the diversion cross Ambusten Creek and Medicine Creek. These creek crossings would be major embankments. No creek crossings would be required for the pipeline arrangement.

The canal and tunnel arrangements would require several minor diversions. The largest would be the diversion of Finney Creek around to the headworks dam. Additionally, small diversions would be required at Ambusten Creek and Medicine Creek. For the pipeline arrangement in the 2240 MW Scheme, a short diversion for Finney Creek would be required. For the 800 MW Scheme, no minor diversions at all would be required when using a pipeline.

Construction of the canal would be quite simple being primarily an earth moving operation, although in the steeper regions it would require larger quantities of embankment fill. Pipeline construction would be straightforward and would consist primarily of some trenching and the assembly of pipe sections. The tunnel would be the most complex arrangement to construct (see Appendix A.)

6.5 Operational Aspects

Ease of operation is also of importance in the selection of a diversion arrangement. Maintenance, interference of the diversion with mine operation and adaptability to future changes to the mine plan have been considered.

In terms of inspection, monitoring, and maintenance, the canal arrangement would be the most demanding. Since failure of the canal might be by slow leakage and detected only by slope movement, a network of seepage and earth movement monitoring instruments would be required. The diversion dam and the canal embankment would be regularly inspected.

The pipeline arrangement would also require a regular inspection program. However, apart from flushing out sediment deposits in pipe depressions, little regular maintenance would be required.

The maintenance requirements for the tunnel alternative would be minimal.

From a mining point of view, it would be desirable for the diversion system to have minimum interference with the operation of the mine. For the canal arrangement, interference would be minimal for the first few years of mine operation. But as the pit expanded, especially for the 2240 MW Scheme, the pit slopes would infringe upon the canal route. At this stage, either a tunnel or a pipeline would be required to bypass the pit section.

The pipeline alternative would have perhaps the greatest effect on mining operations, since it would be located within the pit. At intervals, as the pit expanded, the pipe would have to be relocated. However, for the 800 MW pit, only three pipe moves would be required, and in Year 10 of mine operations, the pipe would be in its final location. For the 2240 MW pit, additional pipe moves would be required.

Of the different diversion arrangements considered, the tunnel alternative would have the least interference with mine operations since it would be physically removed from the mine area.

As mining proceeded, the pit plan would undoubtedly evolve. The outline would change, the bench elevations would be different, the life of the mine would probably become longer and Pit No. 2 might be brought into operation.

Of the three diversion arrangements considered, the canal arrangement would be the least adaptable to future changes in the mine plan. Its location would be fixed and canal re-routing would require reconstruction.

The pipeline arrangement would have the greatest flexibility in accommodating mine plan changes because the pipe could be easily moved to any location when required. Fixed structures such as the diversion intake might have to be rebuilt if additional head were required.

The tunnel, although not movable, would not conflict with changes in the mine plan because it would be located outside the open pit, but a handsome price is paid for this advantage.

6.6 Recommended Diversion Arrangement

The relative merits of the various diversion arrangements are shown on Table 12.

After comparison of the different diversions on the basis of cost, potential problems, system components, and operational aspects, the mid-level pipeline arrangement is recommended as the optimum choice for both the 800 MW and 2240 MW Schemes during operation. The recommendation for the long-term (abandonment) solution for the 800 MW Scheme is also a pipeline, but for the 2240 MW Scheme the choice is more complicated and dependent on the planning options exercised during operation. This is treated further in Section 7.3.

The pipeline arrangement was chosen primarily because of its low cost but also because it would have the greatest flexibility and, being leakproof, would have no effect on pit slope stability. The pipeline would follow closely the original grade of Hat Creek, meaning that, for most of its length, it would be in the valley bottom. Pipes are manufactured to close standards and repairs would be easy. For this reason,

TABLE 12

Conclusions of Diversion Alternatives Study

	Canal	Canal/Tunnel/Pipe	Low-Level Pipe	Mid-Level Pipe	High-Level Pipe	Triple Pipe
800 MW During Operation	S	S	US	(R)	S	S
800 MW Abandonment	US	S	US	US	(R)	S
2240 MW During Operation	US	S	US	(R)	S	S
2240 MW Abandonment	US	(R)	US	US	US	S

(R) - Recommended

S - Suitable technically, possibly economically

US - Unsuitable, either economically or technically leakage problems, compared to those of the canal, would be virtually eliminated. Elimination of the rim dam, with the inherent problems of seepage and pump maintenance, adds to the reasons for selecting the pipeline arrangements over the canal.

7.0 PREFERRED ARRANGEMENT: PIPELINE DETAILS

7.1 Design, Construction, and Operation

The layouts of the recommended mid-level pipeline arrangements have been outlined in Section 4, and the parameters are summarized in Table 6. The routes are shown as Drawings 9 and 10. The principal components would be:

(1) Low diversion dam and intake structure
(2) F.R.P. from intake to upper pit rim
(3) Twin polyethylene pressure pipes within the pit
(4) Twin polyethylene pipes in the pit exit region
(5) F.R.P. alongside the leachate lagoon
(6) F.R.P. discharge conduit
(7) Open channel
(8) Minor diversions

Details of the diversion dam and intake are described in Section 4.5

From the intake structure to the pit rim, a 2.4 m diameter F.R.P. would be used. This would be either an above ground or a shallow buried installation. A typical above ground installation is shown on Drawing 14. F.R.P. was chosen for this section because its large diameter reduces head loss and because it has the ability to withstand internal pressures if the intake were surcharged during extreme floods.

The pipe would be supplied in lengths and connected by bell and spigot joints, see details shown on Drawing 14. The joints can be pressure tested with the pipe length. This section of F.R.P. pipe would remain in place for the duration of the diversion.

Within the open pit mine, twin, 1.5 m diameter polyethlyene pressure pipes would be used. Initially, the pipe would be laid alongside the present Hat Creek channel, and as the pit excavation proceeded it would be necessary to relocate the pipelines on the pit benches. For the 800 MW project, about three or four pipe moves would be required over the life of the mine, and the pipe would be in its final location after year 10. The larger 2240 MW pit would require a few additional pipe moves.

The polyethylene pipes could be laid directly onto the pit benches, although depending on the bench materials, a small amount of surface preparation might be required. Some lateral support in the form of earth berms or steel support anchors placed at 40 m intervals would also be required to contain movements caused by temperature contraction or expansion, see Drawing 14. The pipe should be protected from external damage by either ditches, earth berms or protective earth fill. Additional study would be required to examine the best method for dealing with any icing problems.

A single pipe would have a capacity of about 14 m^3/s, which is approximately the size of the largest flood peak on record. Together, the two pipes would have sufficient capacity to discharge the 1000-year flood. During much of the year, flows would be quite low (less than 3 m^3/s), so that it should be relatively simple to move the pipes one at a time when necessary. With polyethylene pipe, the joints would be butt-fused into longer lengths which would be flanged and bolted together. During pipe moves, the flanges would be unbolted, and since the pipe is very tough and flexible, it would be simply pulled or rolled (on a specially prepared bed or on skids) to the new location. Butterfly control

control valves at the transition between the fibreglass pipe and polyethylene pipe section would be used to shut off the pipe being moved. Details are shown on Drawing 14.

The conveyor and haul road embankments are immediately downstream of the open pit. Since no pipe relocation is foreseen in this area, a single large diameter F.R.P. pipe could be used. However, twin, 1.5 m diameter polyethylene pressure pipes were selected because of their increased flexibility and safety. Beneath the embankments, the polyethylene pipes would be placed inside corrugated steel pipe culverts designed to withstand the high external loads.

From the conveyor embankment and alongside the leachate lagoon, Hat Creek would be diverted through a single 2.4 m diameter F.R.P. In sections, the pipe would be laid on an embankment to permit a constant downward grade to minimize sedimentation problems. Alongside the leachate lagoon, the pipe would be located on a bench with pipe invert at all times above the maximum leachate level. This would facilitate inspection, maintenance and repair of either the lagoon or the pipe without disrupting the operation of one or the other. Locating the pipe above and further away from the leachate lagoon would not be possible without significantly increasing cost.

From the leachate lagoon, down to approximately the 830 m elevation, a 2.1 m diameter F.R.P. pipe would be used as a discharge conduit, emptying into an open channel. An energy dissipation structure would be required at the lower end of the pipe, see Drawing 14.

A short section of open channel would pass Hat Creek around the sedimentation lagoons. Since this area is relatively flat, the channel would not be much more than a re-alignment of the present Hat Creek channel.

One of the advantages of the mid-level pipeline arrangement is that the diversion dam would be close to the pit. As a result, no other minor diversions would be required for the 800 MW scheme, and only a short section of diversion for lower Finney Creek would be needed for the 2240 MW scheme. All other creeks would flow directly into the drainage area above the diversion dam.

The location of the pipe for the different stages of pit development for the 800 MW project are shown on Drawings 15 and 16. The pipe would be in its final location by year 10 of its operation.

7.2 Repair of Damaged Polyethylene Pipe

If part of the polyethylene needed repair, the water flow in that pipe would be diverted into the second polyethylene pipe. The mine drainage system could be used to handle the flows from the diversion on an emergency basis for most months of the year when the flows are not too high. Two types of repair are available: mechanical repair and fusion repair.

The mechanical method would likely be the initial method of repair since it is fast and immediately available. It consists of a full wrap around type repair clamp, with an integral gasket. The minimum clamp length should be at least twice the nominal pipe diameter. The clamp is tightened around the pipe evenly and securely. For above ground installations, this method is suitable for non-pressurized services. The high pull out forces in a pressure line would require a buried line with a compacted backfill. This should pose no difficulty, since the pipeline would only be pressurized appreciably during design flood flows and most of the time it would flow only partially full.

If the pipe were buried under a protective fill, the fill could be removed for a distance near the damaged section. The damaged section could be cut out, a replacement section prepared, and a butt fusion machine used to install the replacement section. However, if the pipe were to be buried and the protective fill could not be removed for a sufficient distance, it would be necessary to use a flanged spool piece. The damaged section would be cut out, and the butt fusion machine would be lowered into the trench to install a flange assembly on each pipe end. A flanged spool piece would be made from a spare pipe to fit precisely the resulting gap and the flanged connections bolted up.

7.3 Long-Term Considerations and Abandonment

If at the end of a 35 to 41-year life of the mine, it was decided not to exploit the total coal resource, some form of permanent abandonment of the pit would be necessary. A diversion arrangement would be required to ensure a continued supply of potable water to the Indian Reserve downstream unaffected by stability or environmental problems in the pit itself. Ideally, a maintenance-free system should be left.

For the 800 MW pit, as planned, at a mine life of 35 years, it is tentatively proposed that a high level pipeline could be located on the eastern gravel benches sufficiently far removed from the rim of the pit that the pipe would be unaffected by any continuing pit slope instability. A higher elevation dam, approximately at the junction of Hat Creek and Anderson Creek, would need to be built to supply sufficient head. The route would be similar to that followed by the canal arrangement shown on Drawing 4. The present capital cost of this abandonment arrangement might be about $32 million at 1982 price levels. However, the viability of this arrangement depends on the stability of the pit slopes and the extent to which the rim is likely to migrate eastwards with time. It should be possible to assess this prior to the abandonment of the 800 MW Pit.

The above arrangement would not be possible for the 2240 MW Scheme at the end of a 41-year mine life because of the larger pit size. For this scheme, a possible method of abandonment might be continued stabilization of the pit slope and maintenance of the pipeline. A more likely and permanent solution would be to divert the water using the tunnel alternative, in conjunction with a pipe and canal upstream, the headworks dam and a pipeline downstream. Such an arrangement might cost about $44 million at 1982 price levels, assuming that the pit rim dam and pumping facilities would not be needed.

It is more likely that mining would continue after the the 35 to 41-year period, and that the total resource of the No. 1 Pit would be extracted. Moreover, if the No. 2 Pit were also exploited, the No. 1 Pit could then be backfilled with waste material from that pit. In that case, the abandonment might be much simpler, since the pipe could be relocated on the ultimate pit slopes while mining continued. After backfilling the pit, consideration could be given to returning the creek to the centre of the valley in either a pipeline or a new channel constructed on the waste backfill. With this arrangement, the expensive tunnel alternative would not be required for either the 800 MW or 2240 MW schemes. The final total resource pit geometry would probably be the same for both schemes, and the only difference would be in the rate of mining and, therefore, the life of the mine. This would only affect the diversion arrangement to the extent that for the smaller scheme, i.e. longer mine life, more maintenance might have to be provided to secure the pipes in the pit against progressive slope instability. These alternatives are still conjectural and entirely dependent on the long-term plans for both the No. 1 and No. 2 Coal Deposit.

8.0 FUTURE INVESTIGATIONS

8.1 General

This study has been based on meterological data, mine plans and other information provided by B.C. Hydro. The information obtained and the level of detail presented in this report is considered adequate for a feasibility study. However, further work would be required for final design.

8.2 Hydrology

The design flows for Hat Creek have been based on a regional flood frequency study, since only a few years of Hat Creek flow data were available. At the final design stage, the design flows should be re-examined incorporating data available to that date. Meteorological and flow measurements, especially at Station 08LF061, near Upper Hat Creek, should be maintained.

8.3 Risk/Cost Analysis of Flooding Pit

In conjunction with a further hydrological study at the final design stage, a risk/cost analysis, investigating the consequence of flooding the pit, should be undertaken to determine the optimum design flood. In the present study, design floods have been based on an emergency capacity of 27 m^3/s corresponding to the 1000-year flood.

However, few diversions are designed for floods as rare as the 1000-year event, and the possibility of lower diversion capacity and accepting some risk of flooding the pit should be investigated. The water spilled into the pit would be contaminated and it would need to be treated and eventually disposed of on site in order to honour the current policy of zero discharge of pollutants from the project. A risk/cost analysis would investigate the probability of Hat Creek discharges, diversion capacity, treatment capacity, and the consequences of flooding the pit and the associated costs.

8.4 Pipe Materials

At the final design stage, new pipe materials would likely have become available, and the pipe materials should be reviewed. Particularly desirable would be an increase of the 1.5 m diameter maximum size for the polyethylene pressure pipe. The substitution of corrugated polyethylene pipe for fibreglass reinforced pipe should also be re-investigated.

8.5 Tunnel Investigation

Any further consideration of the tunnel as a means of diversion could be put off until the later stages of operation when abandonment procedures have been selected.

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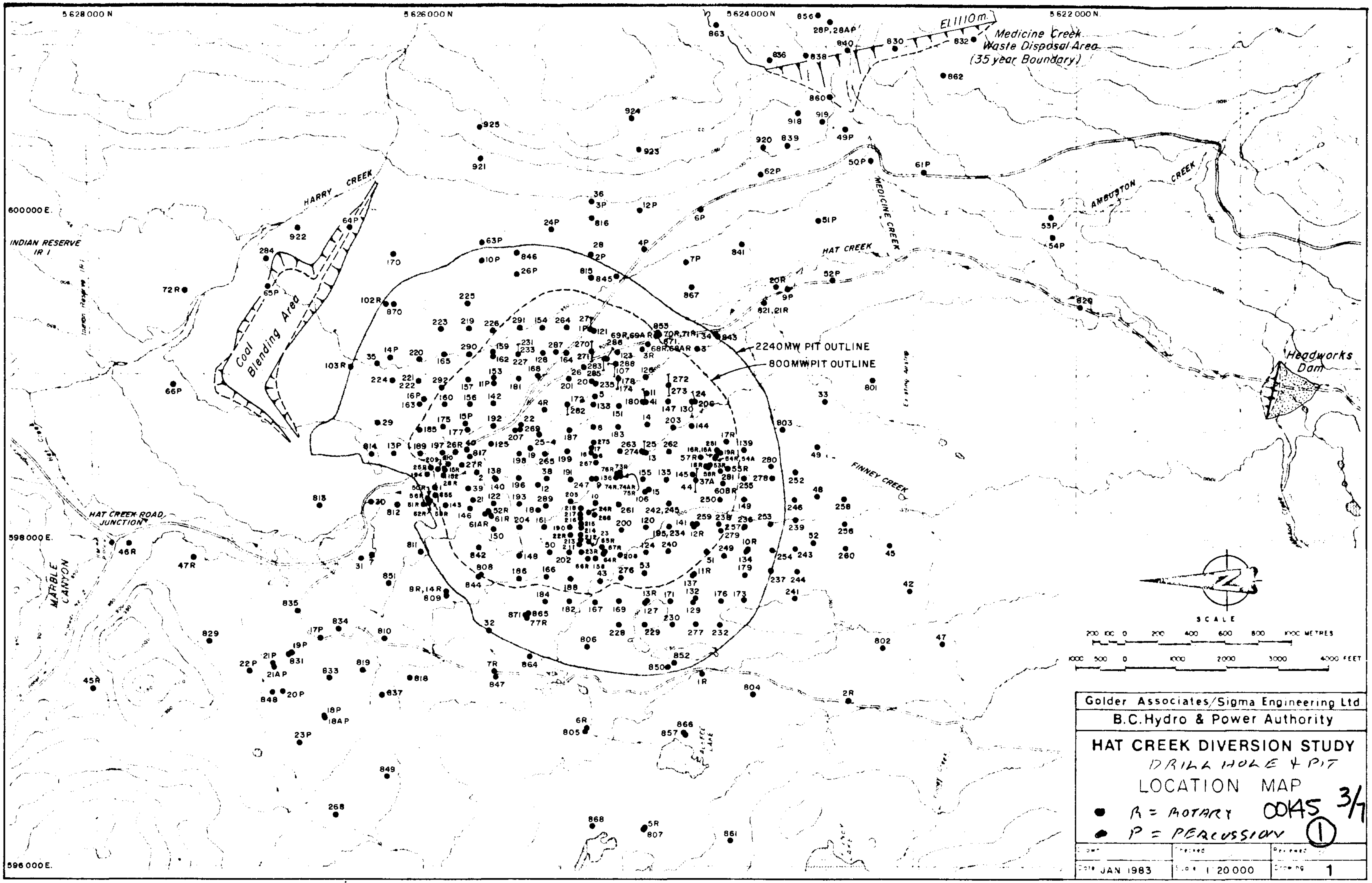
Medicine Creek Waste Disposal Area (35 year Boundary)
El.1110m.
Coal Blending Area
2240MW PIT OUTLINE
800MW PIT OUTLINE
HARRY CREEK
MEDICINE CREEK
HAT CREEK
AMBUSTON CREEK
FINNEY CREEK
Headworks Dam
INDIAN RESERVE IR 1
HAT CREEK ROAD JUNCTION
MARBLE CANYON
5 628 000 N
5 626 000 N
5 624 000 N
5 622 000 N
600000 E.
598000 E.
596000 E.
SCALE
Golder Associates/Sigma Engineering Ltd
B.C.Hydro & Power Authority
HAT CREEK DIVERSION STUDY
DRILL HOLE & PIT
LOCATION MAP
R = ROTARY
P = PERCUSSION
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1
JAN 1983
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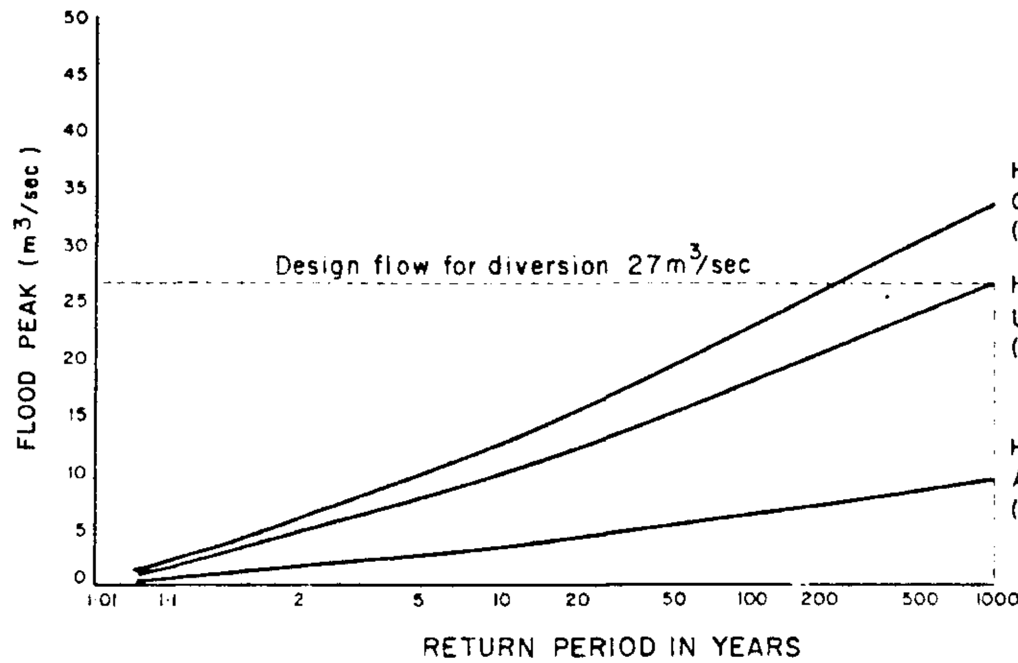
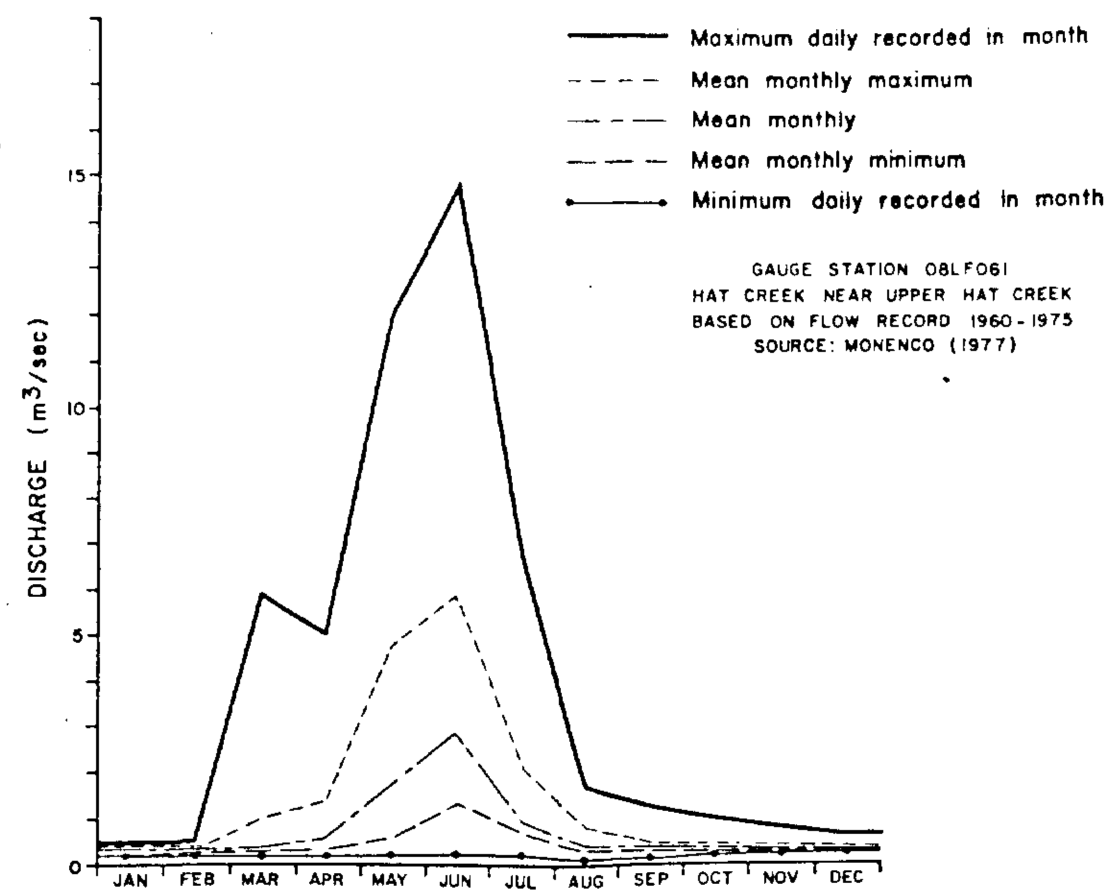
FLOOD FREQUENCY CURVES DERIVED FROM REGIONAL DATA



MONTHLY FLOW IN HAT CREEK JUST ABOVE THE MINE SITE



Source : B.C. Hydro H.E.D.D. Report 913, 1978

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HAT CREEK DIVERSION STUDY

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HYDROLOGY INFORMATION

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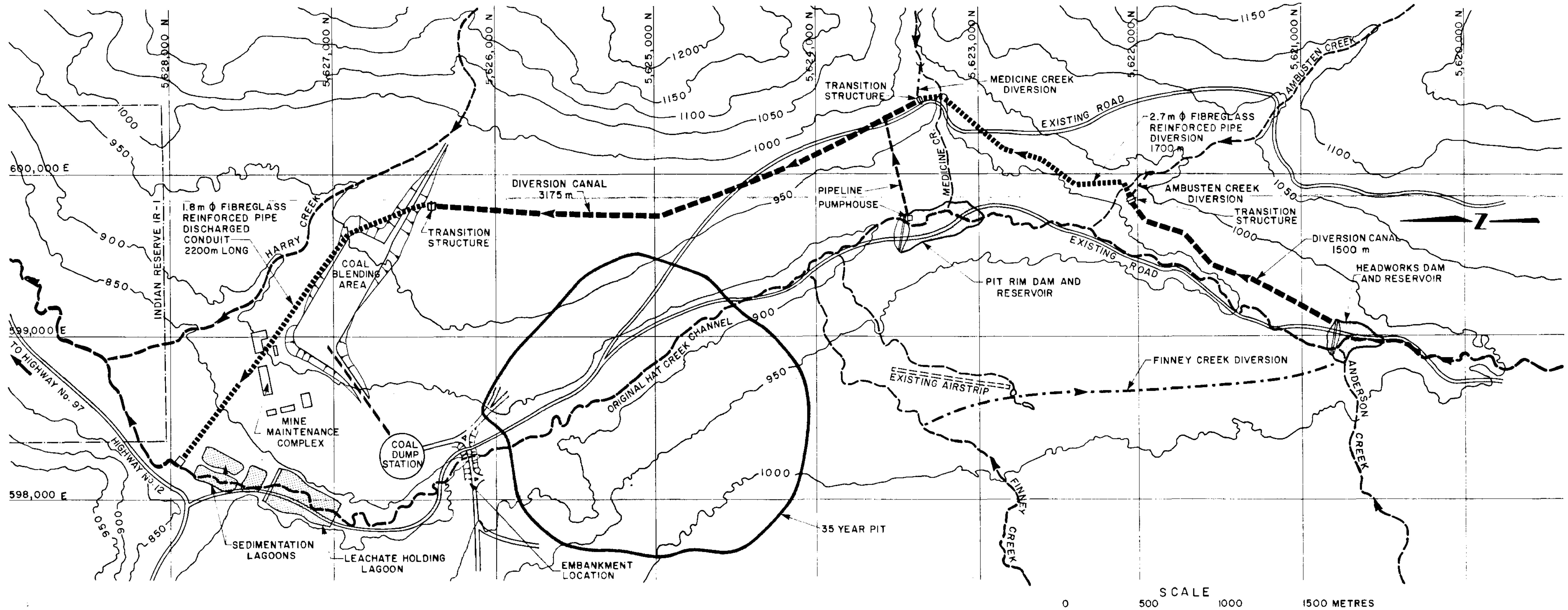
Date SEP 1982	Scale N.T.S.	Drawing 2



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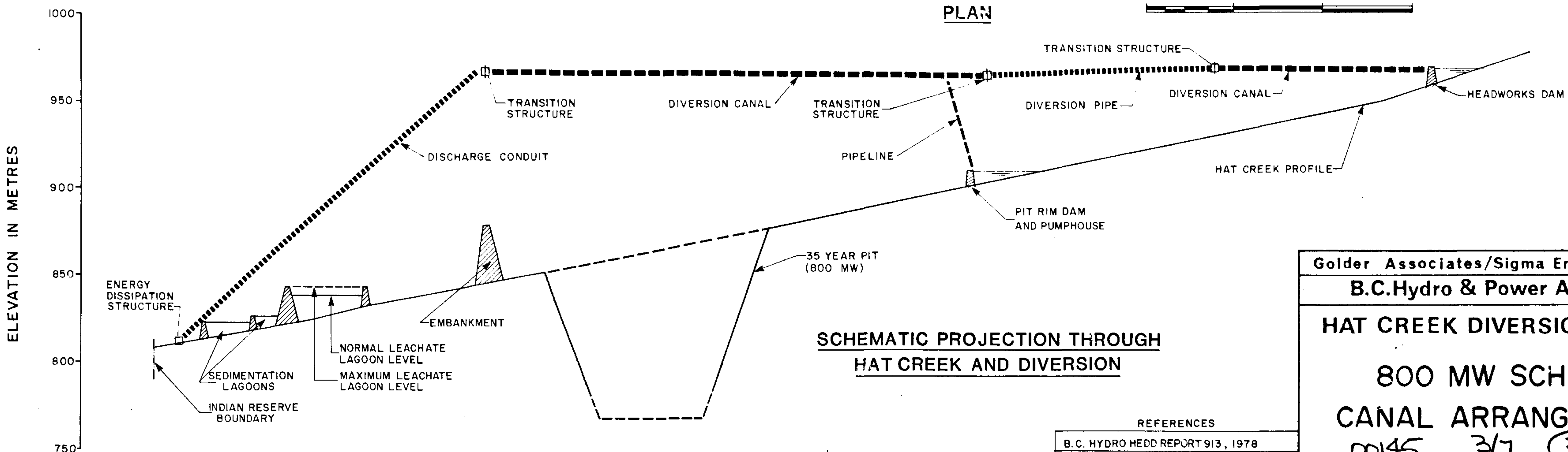

SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

REFERENCES
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B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

800 MW SCHEME
CANAL ARRANGEMENT

00145 3/7 3

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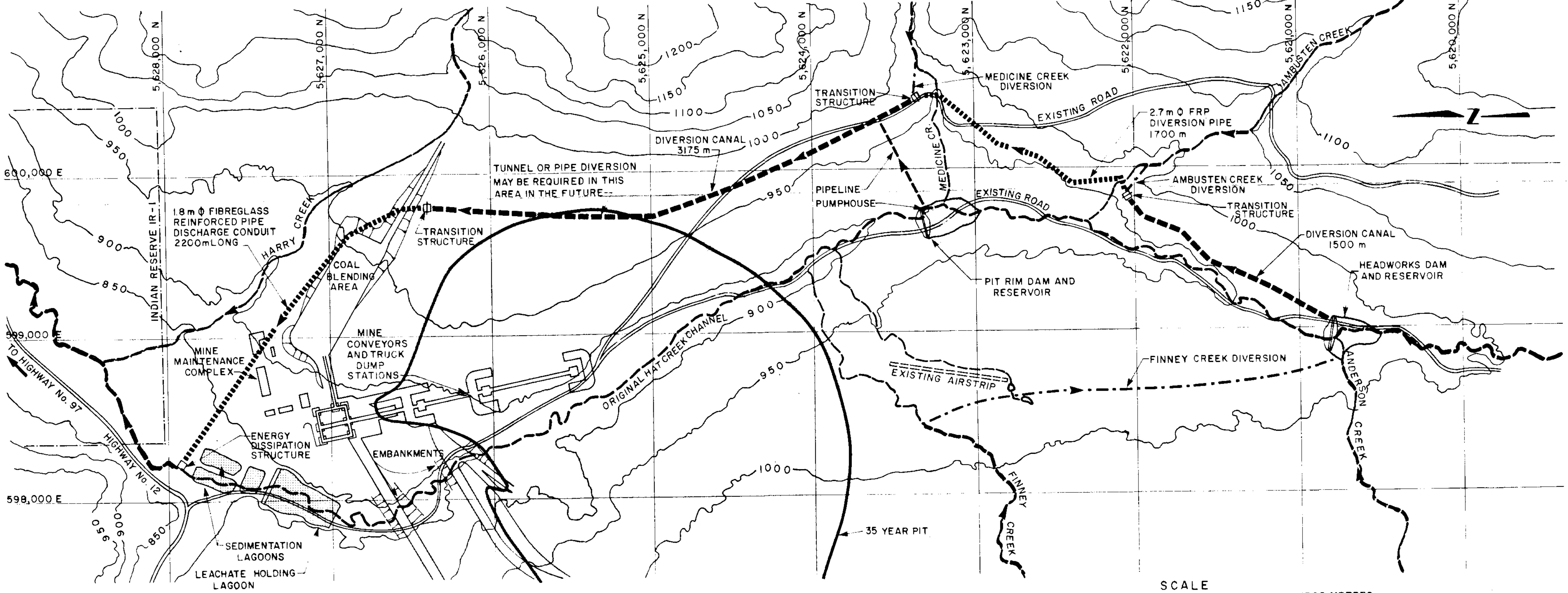
Date Sept. 1982	Scale As shown	Drawing 3



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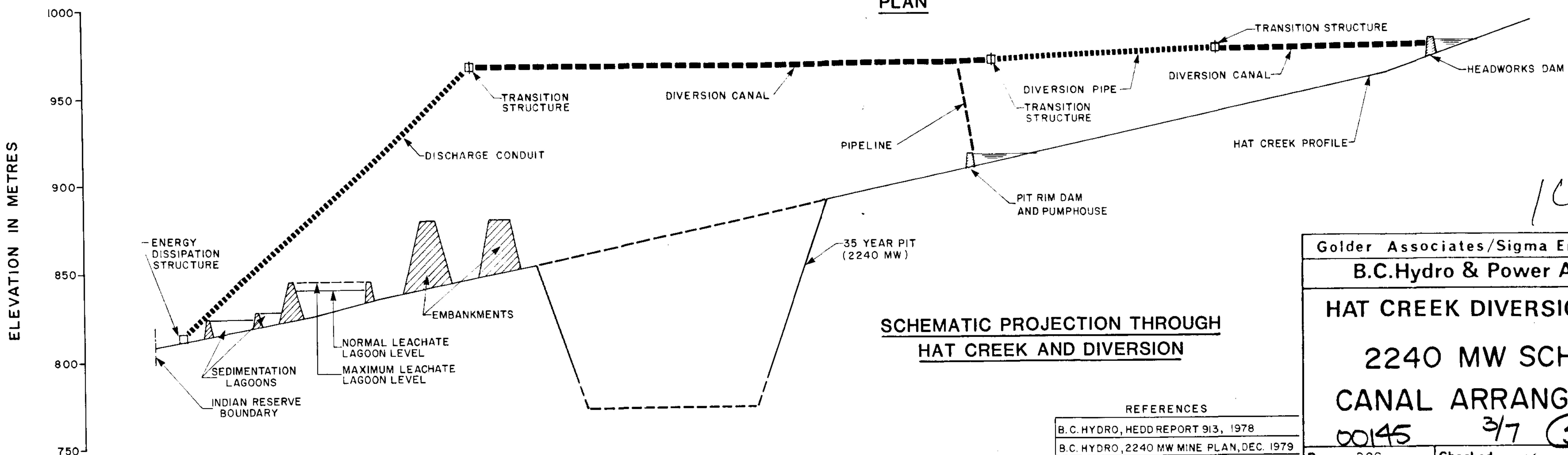

SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

145

REFERENCES
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HAT CREEK DIVERSION STUDY

2240 MW SCHEME CANAL ARRANGEMENT

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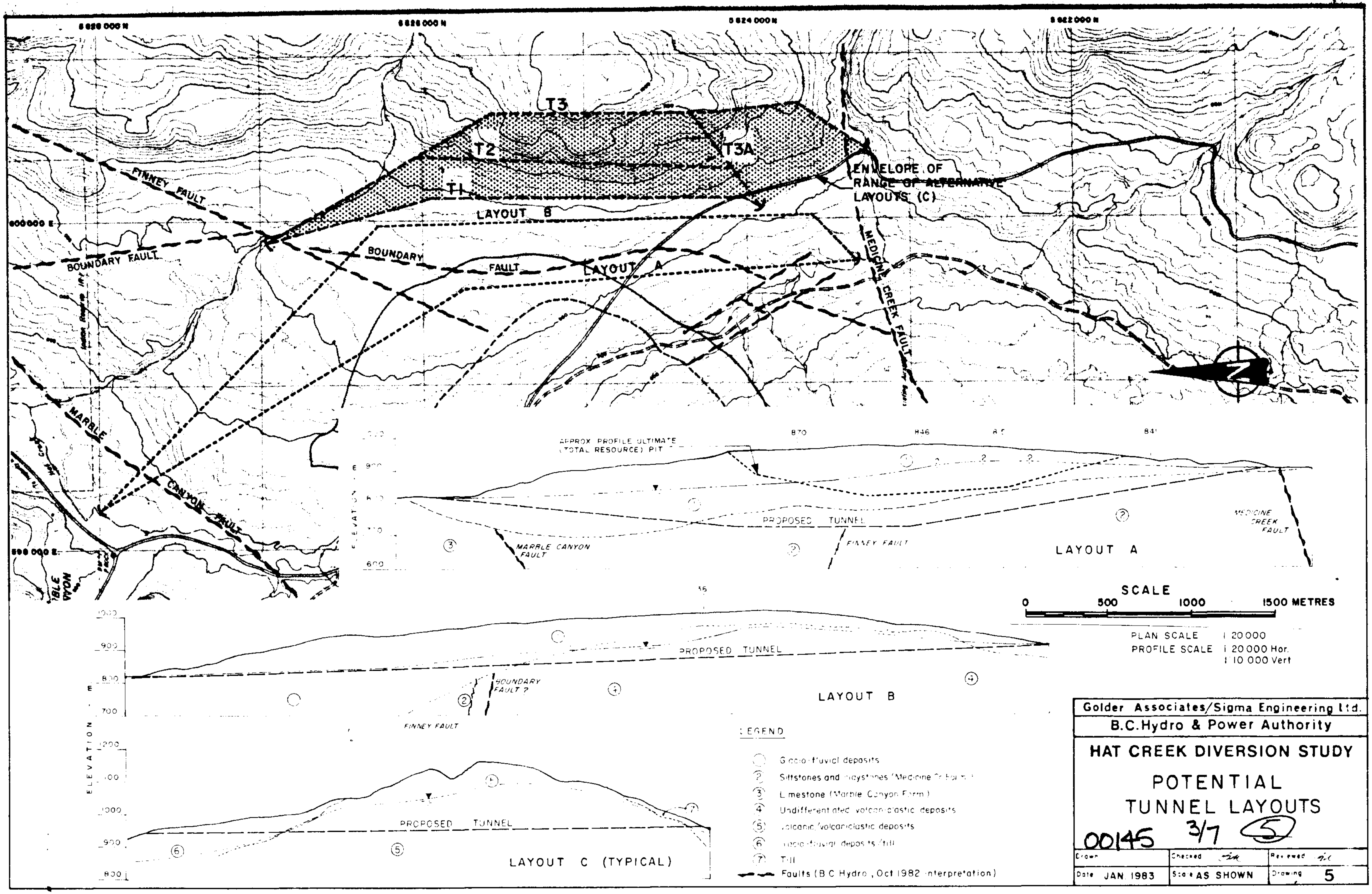
Date Sept. 1982	Scale As shown	Drawing 4



5 628 000 N
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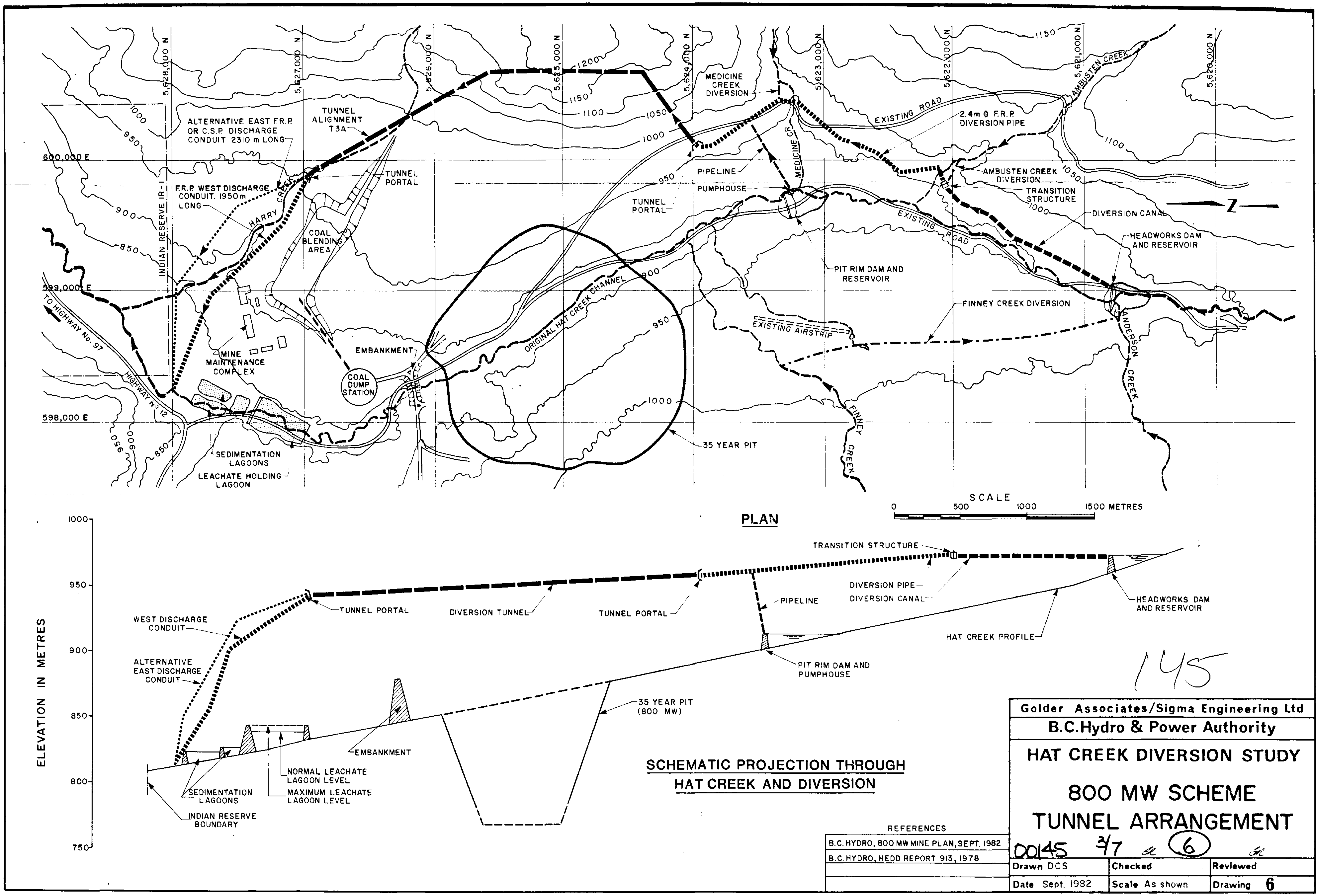
5 624 000 N
5 622 000 N
600 000 E
598 000 E
T3
T2
T3A
T1
ENVELOPE OF RANGE OF ALTERNATIVE LAYOUTS (C)
LAYOUT B
FINNEY FAULT
BOUNDARY FAULT
BOUNDARY
FAULT
LAYOUT A
MEDICINE CREEK FAULT
MARBLE CANYON FAULT
APPROX. PROFILE ULTIMATE (TOTAL RESOURCE) PIT
PROPOSED TUNNEL
MARBLE CANYON FAULT
FINNEY FAULT
MEDICINE CREEK FAULT
LAYOUT A
SCALE
0 500 1000 1500 METRES
PLAN SCALE 1:20 000
PROFILE SCALE 1:20 000 Hor. 1:10 000 Vert
PROPOSED TUNNEL
BOUNDARY FAULT ?
FINNEY FAULT
LAYOUT B
ELEVATION
PROPOSED TUNNEL
LAYOUT C (TYPICAL)
LEGEND
Glacio-fluvial deposits
Siltstones and claystones (Medicine Cr. Form.)
Limestone (Marble Canyon Form.)
Undifferentiated volcaniclastic deposits
Volcanic/volcaniclastic deposits
Glacio-fluvial deposits/till
Till
Faults (B.C. Hydro, Oct. 1982 interpretation)
Golder Associates/Sigma Engineering Ltd.
B.C. Hydro & Power Authority
HAT CREEK DIVERSION STUDY
POTENTIAL TUNNEL LAYOUTS
00145 3/7 5
Date JAN. 1983
Scale AS SHOWN
Drawing 5



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HAT CREEK DIVERSION STUDY

800 MW SCHEME
TUNNEL ARRANGEMENT

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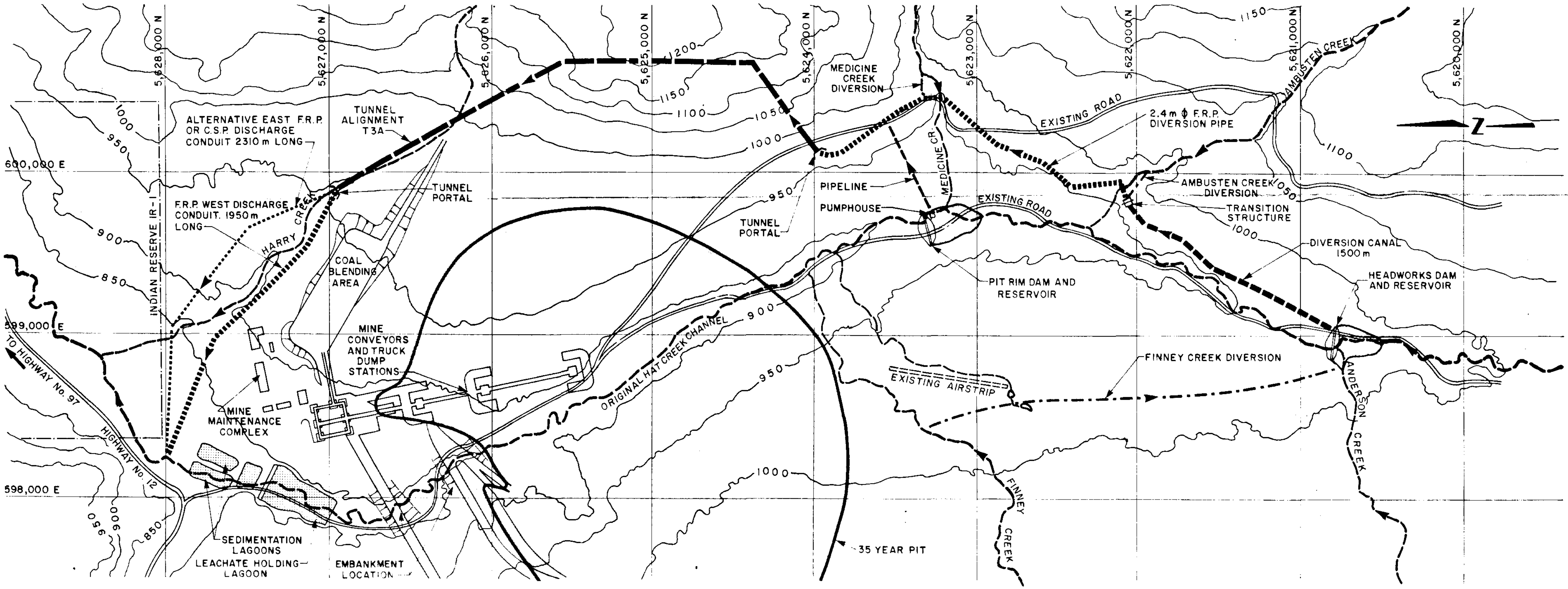
Drawn DCS	Checked	Reviewed
Date Sept. 1982	Scale As shown	Drawing 6



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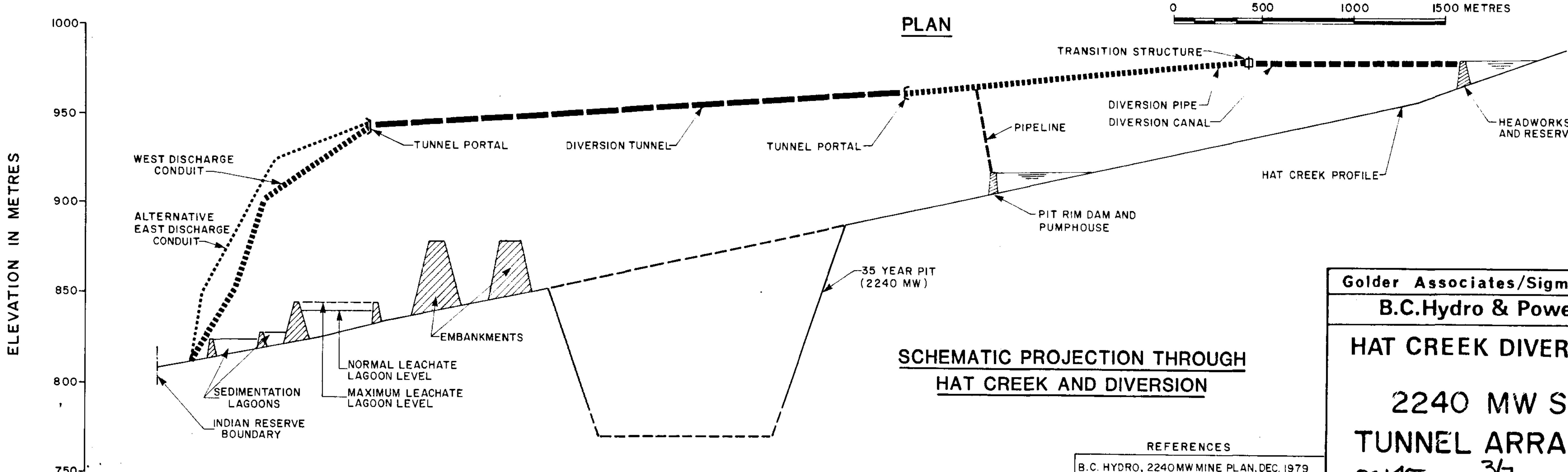

SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

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HAT CREEK DIVERSION STUDY

2240 MW SCHEME
TUNNEL ARRANGEMENT

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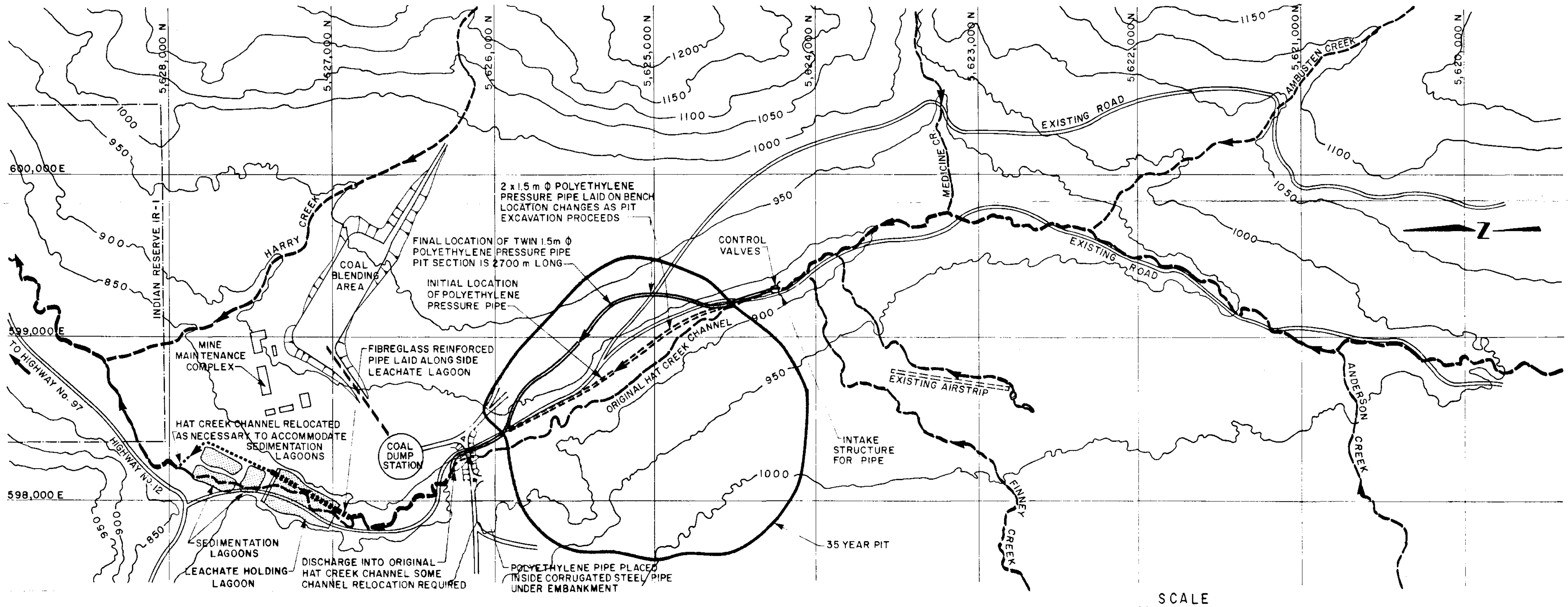
Drawn DCS	Checked	Reviewed
Date Sept. 1982	Scale As shown	Drawing 7



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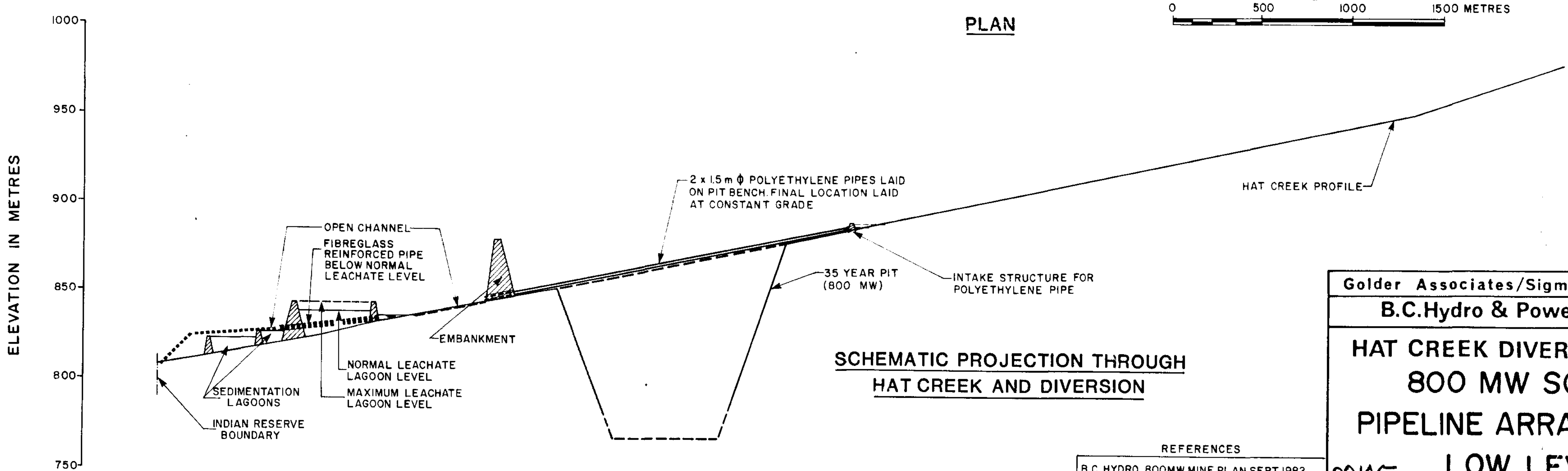

SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

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HAT CREEK DIVERSION STUDY
800 MW SCHEME
PIPELINE ARRANGEMENT
LOW LEVEL 3/7 8

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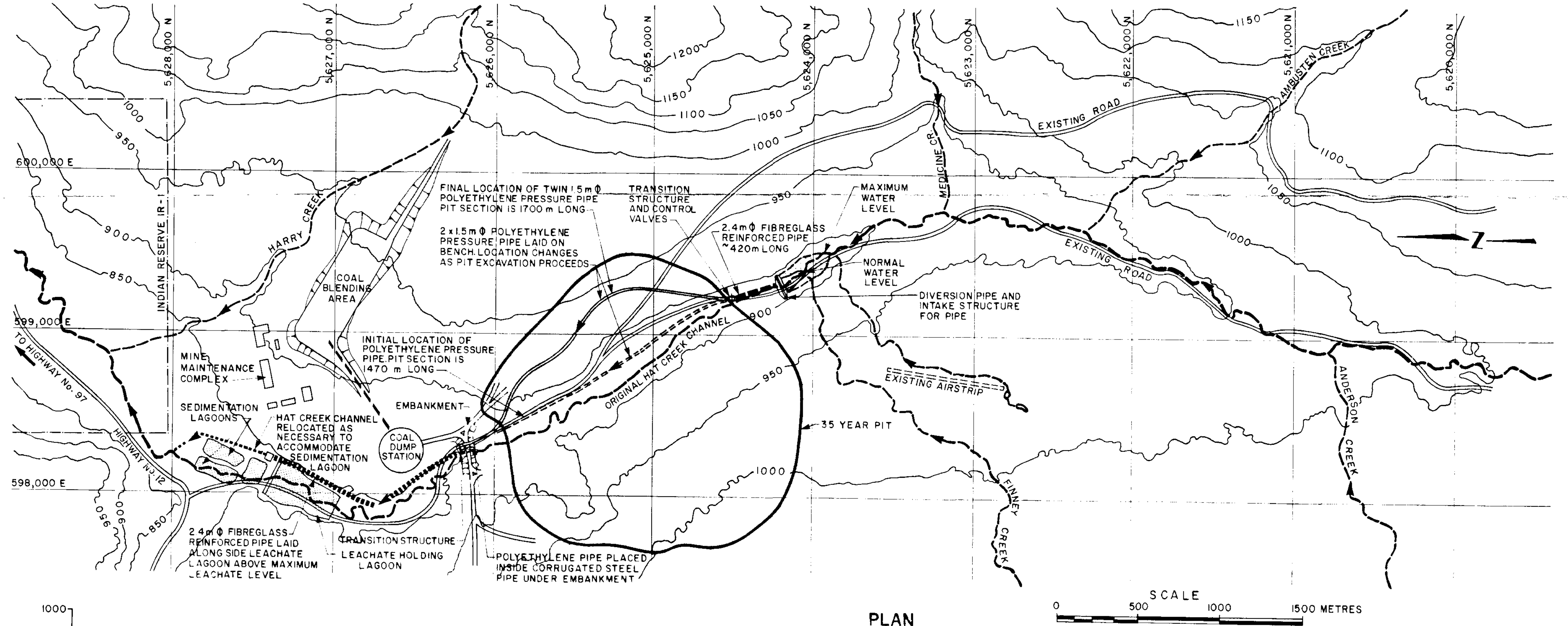
Drawn DCS	Checked	Reviewed
Date Sept. 1982	Scale As shown	Drawing 8



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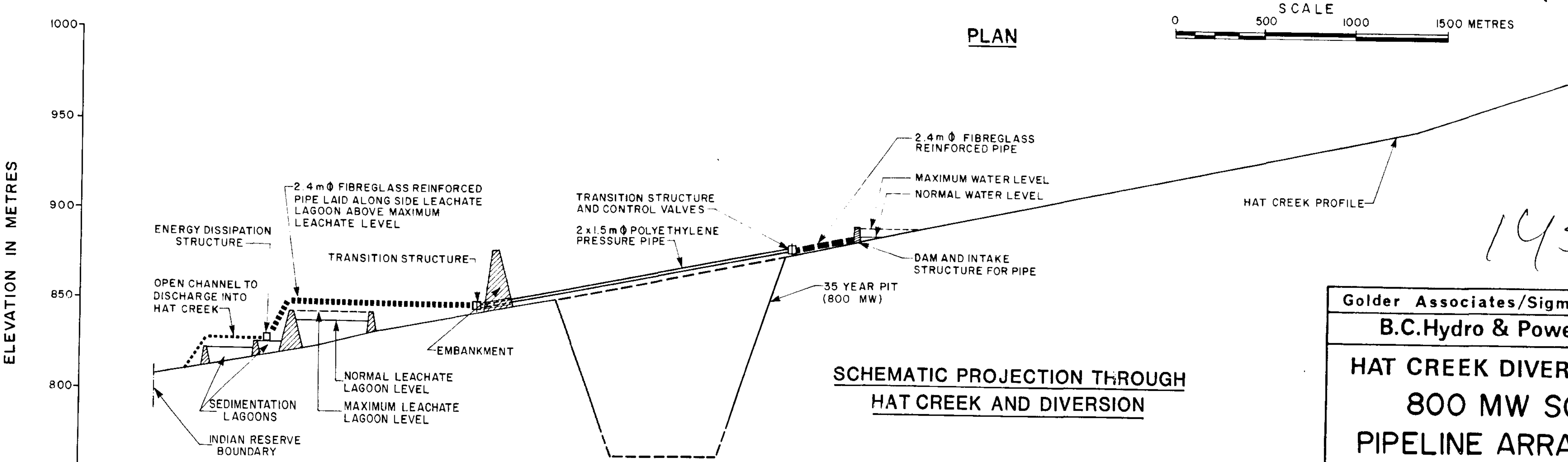

SCHEMATIC PROJECTION THROUGH
HAT CREEK AND DIVERSION

145

REFERENCES
B.C. HYDRO 800 MW MINE PLAN, SEPT 1982

Golder Associates/Sigma Engineering Ltd

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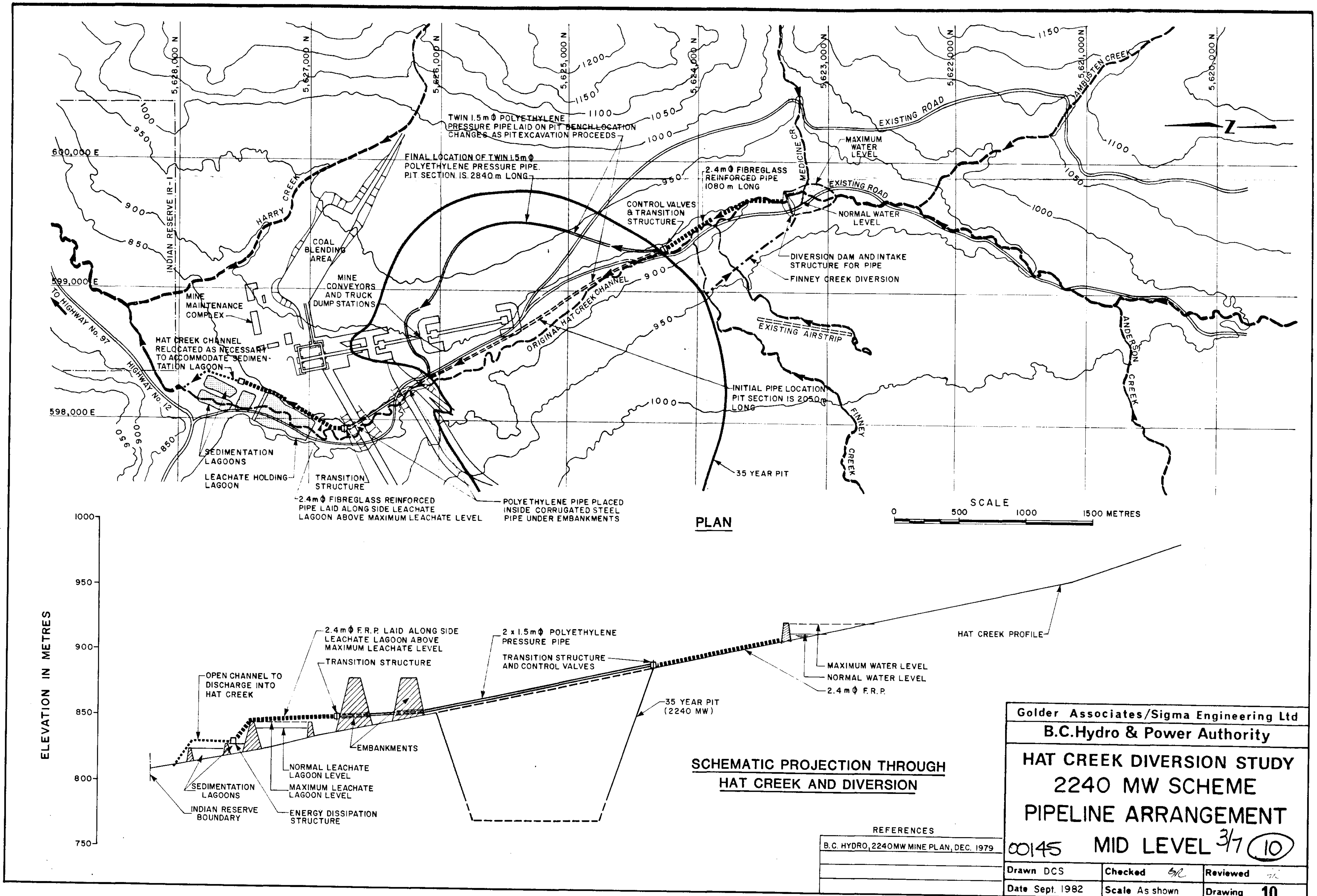
HAT CREEK DIVERSION STUDY
800 MW SCHEME
PIPELINE ARRANGEMENT

00145 MID LEVEL 3/7 (9)

Drawn DCS	Checked	Reviewed
Date Sept 1982	Scale As shown	Drawing 9



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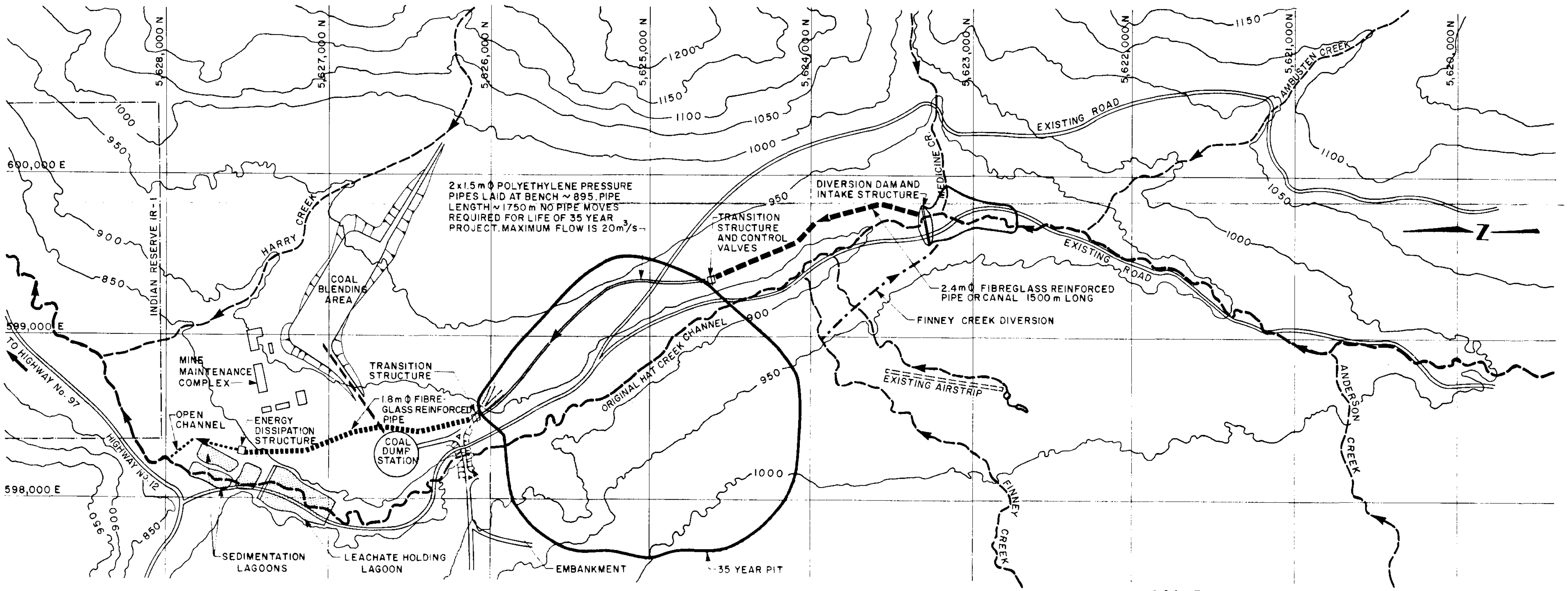
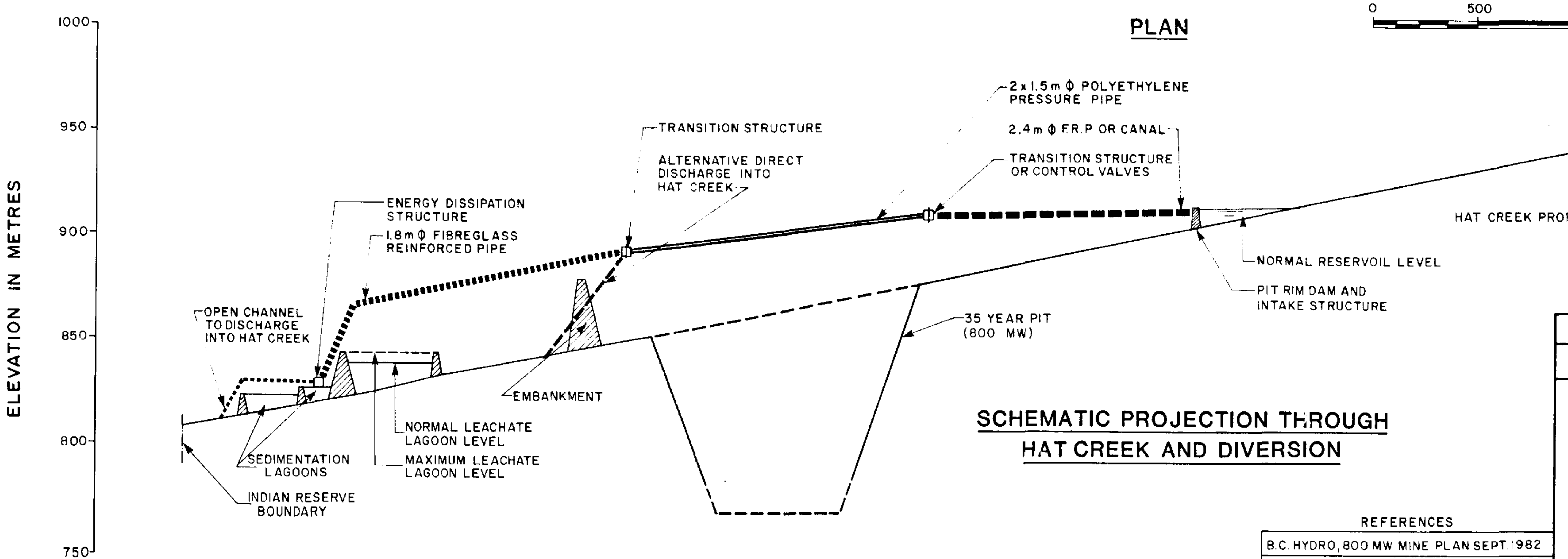
5,625,000 N
5,624,000 N
5,623,000 N
5,622,000 N
5,621,000 N
5,620,000 N
600,000 E
599,000 E
598,000 E
1200
1150
1100
1050
1000
950
900
850
INDIAN RESERVE IR-1
HARRY CREEK
TWIN 1.5 m Φ POLYETHYLENE PRESSURE PIPE LAID ON PIT BENCH-LOCATION CHANGES AS PIT EXCAVATION PROCEEDS
FINAL LOCATION OF TWIN 1.5 m Φ POLYETHYLENE PRESSURE PIPE. PIT SECTION IS 2840 m LONG
EXISTING ROAD
BAMBUSTEN CREEK
MAXIMUM WATER LEVEL
MEDICINE CR.
2.4 m Φ FIBREGLASS REINFORCED PIPE 1080 m LONG
CONTROL VALVES & TRANSITION STRUCTURE
NORMAL WATER LEVEL
DIVERSION DAM AND INTAKE STRUCTURE FOR PIPE
FINNEY CREEK DIVERSION
COAL BLENDING AREA
MINE CONVEYORS AND TRUCK DUMP STATIONS
ORIGINAL HAT CREEK CHANNEL
EXISTING AIRSTRIP
ANDERSON CREEK
MINE MAINTENANCE COMPLEX
TO HIGHWAY No. 97
HIGHWAY No. 12
HAT CREEK CHANNEL RELOCATED AS NECESSARY TO ACCOMMODATE SEDIMENTATION LAGOON
INITIAL PIPE LOCATION PIT SECTION IS 2050 m LONG
FINNEY CREEK
SEDIMENTATION LAGOONS
LEACHATE HOLDING LAGOON
TRANSITION STRUCTURE
35 YEAR PIT
2.4 m Φ FIBREGLASS REINFORCED PIPE LAID ALONG SIDE LEACHATE LAGOON ABOVE MAXIMUM LEACHATE LEVEL
POLYETHYLENE PIPE PLACED INSIDE CORRUGATED STEEL PIPE UNDER EMBANKMENTS
PLAN
SCALE
0
500
1000
1500 METRES
Z
ELEVATION IN METRES
1000
950
900
850
800
750
2.4 m Φ F.R.P. LAID ALONG SIDE LEACHATE LAGOON ABOVE MAXIMUM LEACHATE LEVEL
TRANSITION STRUCTURE
2 x 1.5 m Φ POLYETHYLENE PRESSURE PIPE
TRANSITION STRUCTURE AND CONTROL VALVES
HAT CREEK PROFILE
MAXIMUM WATER LEVEL
NORMAL WATER LEVEL
2.4 m Φ F.R.P.
OPEN CHANNEL TO DISCHARGE INTO HAT CREEK
35 YEAR PIT (2240 MW)
EMBANKMENTS
NORMAL LEACHATE LAGOON LEVEL
MAXIMUM LEACHATE LAGOON LEVEL
SEDIMENTATION LAGOONS
INDIAN RESERVE BOUNDARY
ENERGY DISSIPATION STRUCTURE
SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION
REFERENCES
B.C. HYDRO, 2240 MW MINE PLAN, DEC. 1979
Golder Associates/Sigma Engineering Ltd
B.C.Hydro & Power Authority
HAT CREEK DIVERSION STUDY
2240 MW SCHEME
PIPELINE ARRANGEMENT
00145
MID LEVEL 3/7
10
Drawn DCS
Checked
Reviewed
Date Sept. 1982
Scale As shown
Drawing 10



PLAN



SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

REFERENCES

B.C. HYDRO, 800 MW MINE PLAN SEPT. 1982

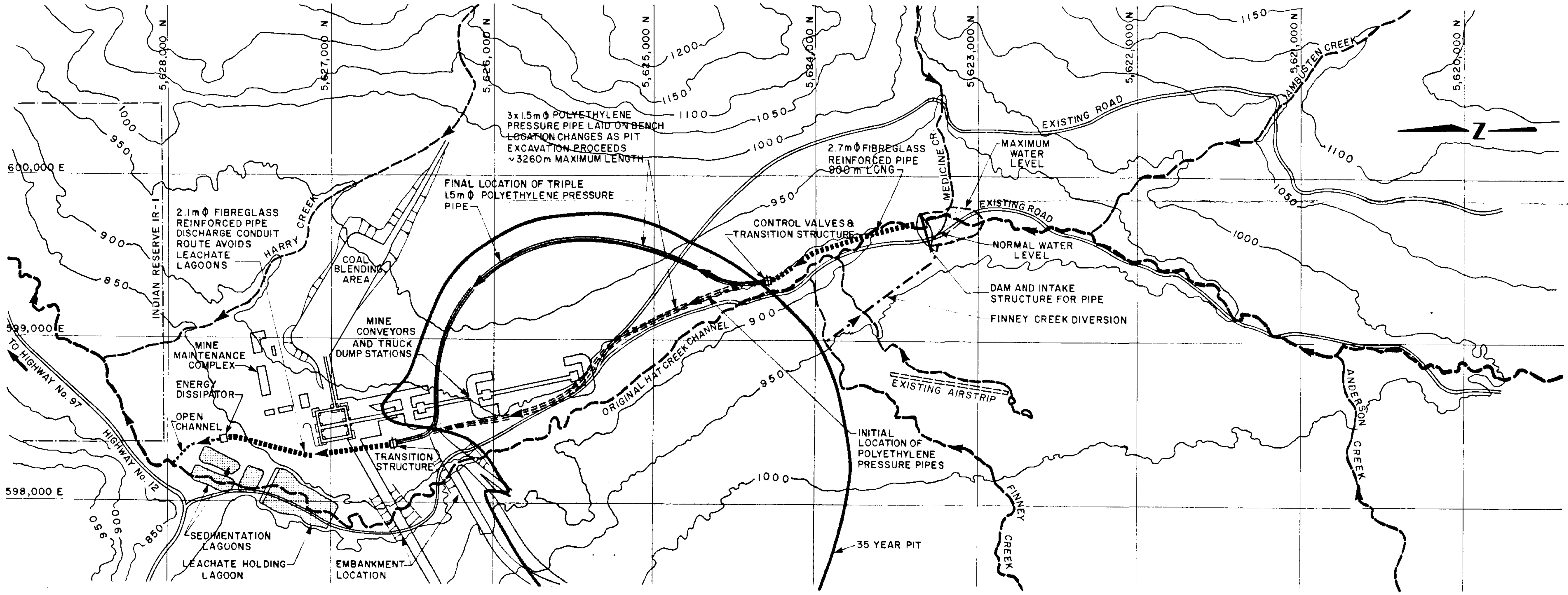
00145 3/7 (11)

Golder Associates/Sigma Engineering Ltd

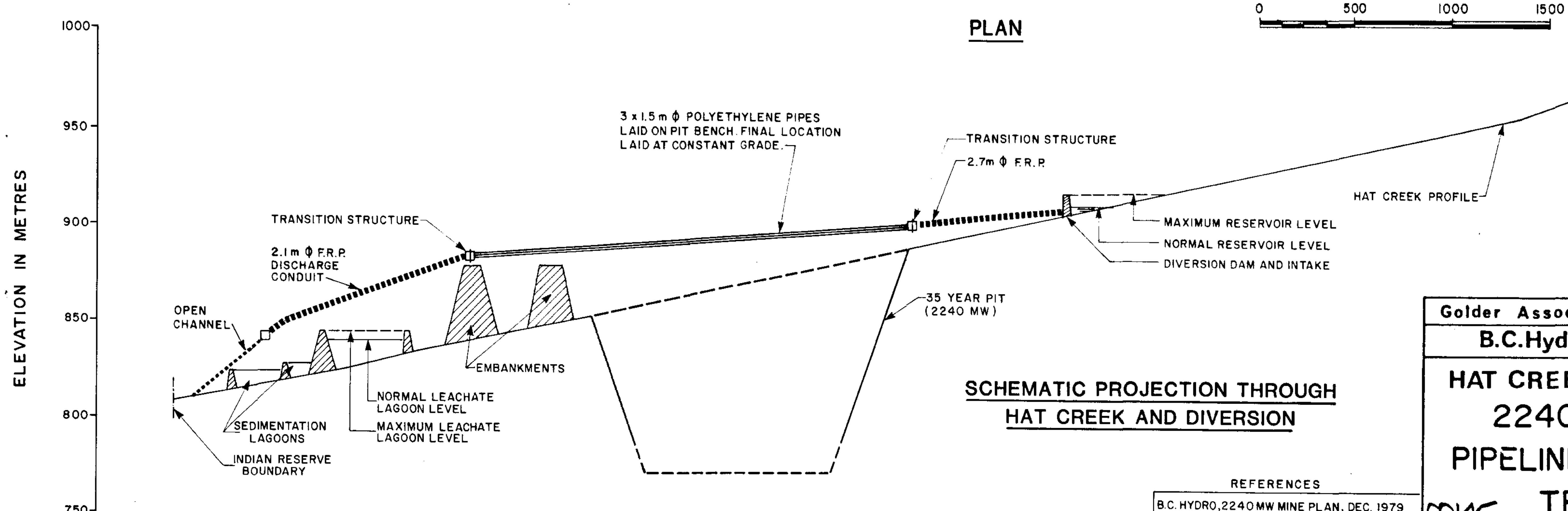
B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY
800 MW SCHEME
PIPELINE ARRANGEMENT
HIGH LEVEL (20 m³/s)

Drawn DCS	Checked	Reviewed
Date Sept. 1982	Scale As shown	Drawing 11



PLAN



SCHEMATIC PROJECTION THROUGH HAT CREEK AND DIVERSION

REFERENCES

B.C. HYDRO, 2240 MW MINE PLAN, DEC. 1979

Golder Associates/Sigma Engineering Ltd

B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

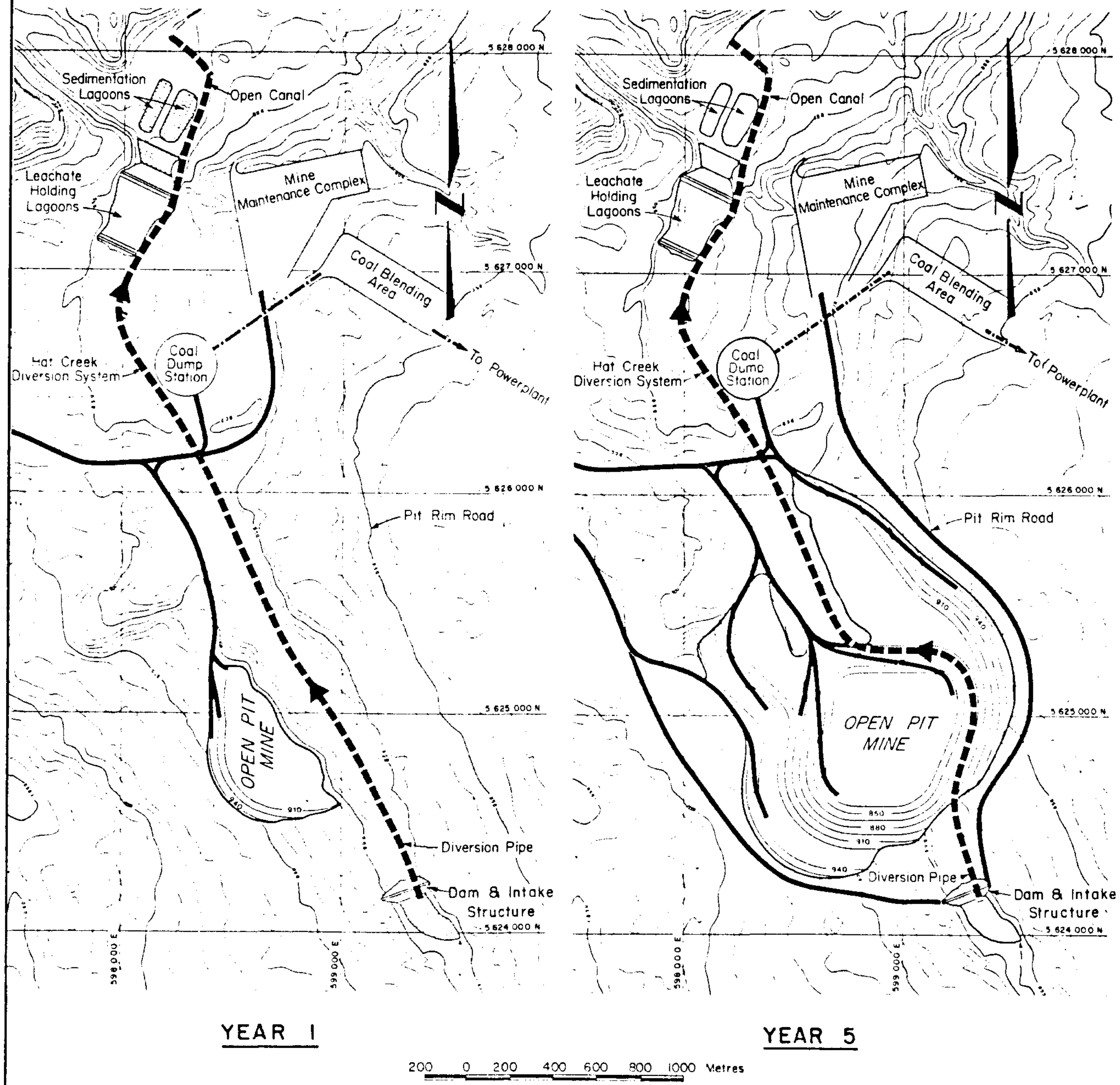
2240 MW SCHEME

PIPELINE ARRANGEMENT

TRIPLE PIPE 3/7 (12)

00145

Drawn DCS	Checked	Reviewed
Date Sept. 1982	Scale As shown	Drawing 12



Source: B.C. Hydro, 800MW Mine Plan, September 1982

Golder Associates/Sigma Engineering Ltd.

B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

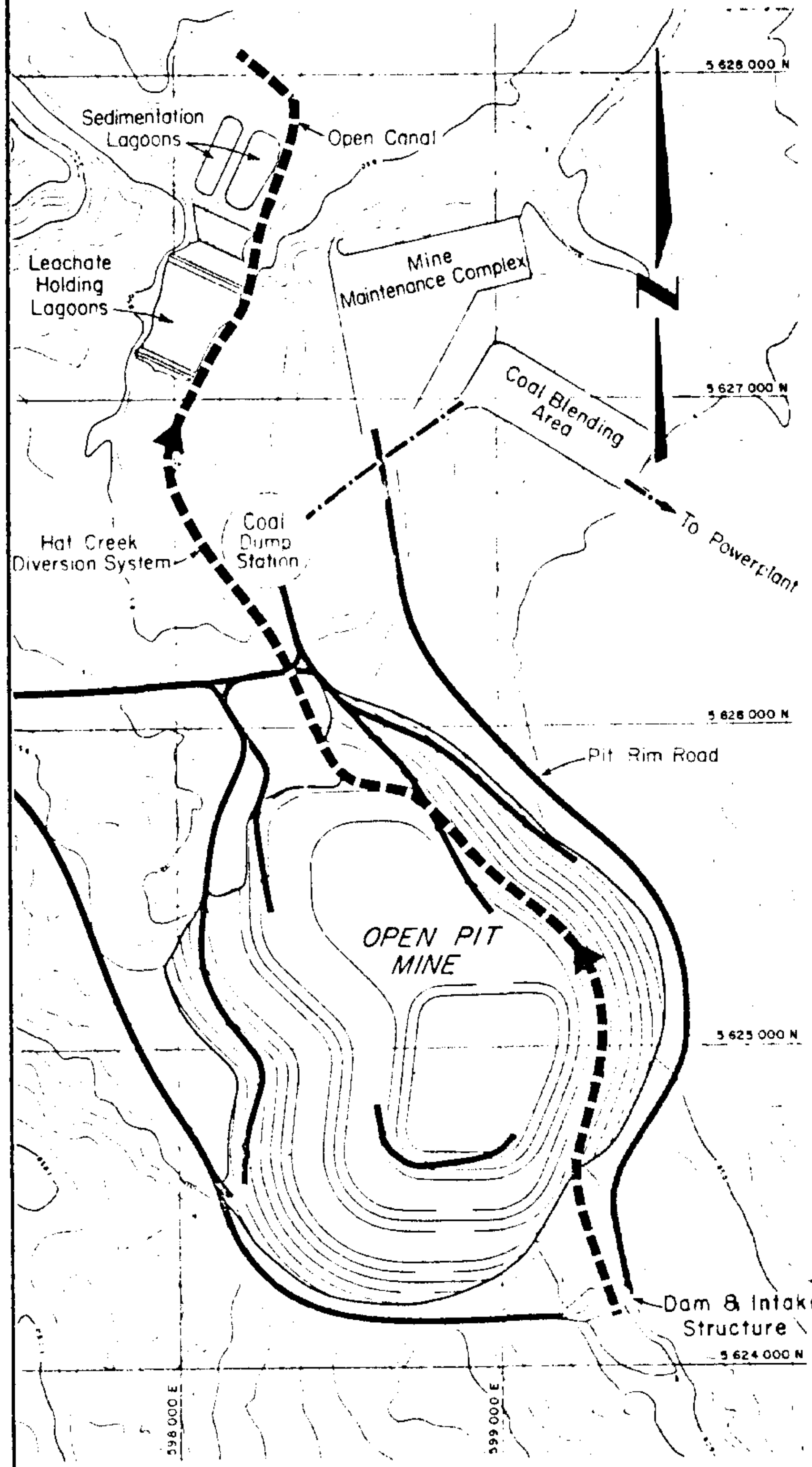
00145 800 MW SCHEME 3/7 13

PIPELINE ARRANGEMENT

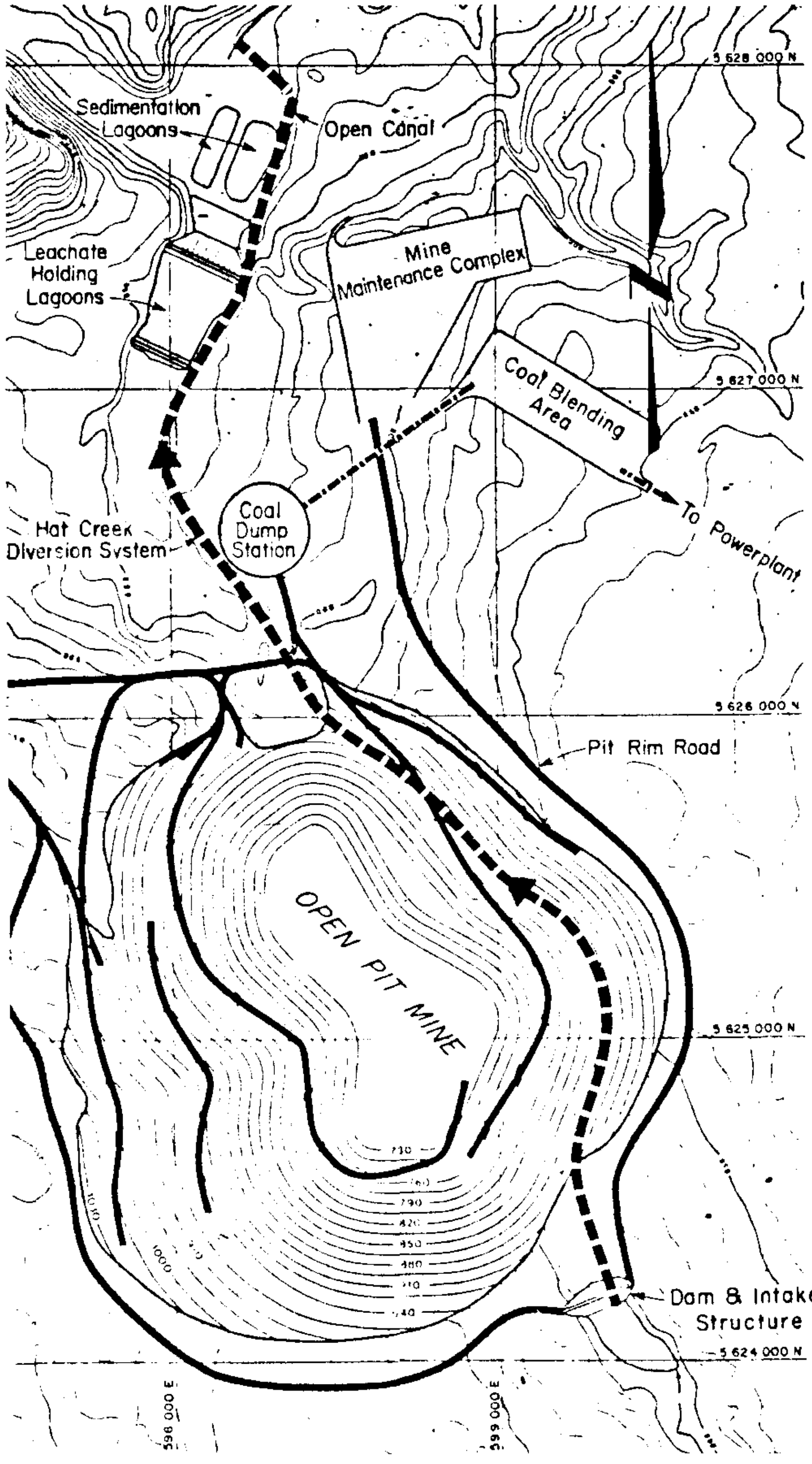
PIT DEVELOPMENT YR. 1 & YR. 5

Drawn J. NG	Checked	Reviewed
Date SEP 1982	Scale AS SHOWN	Drawing 15

5246



YEAR 10

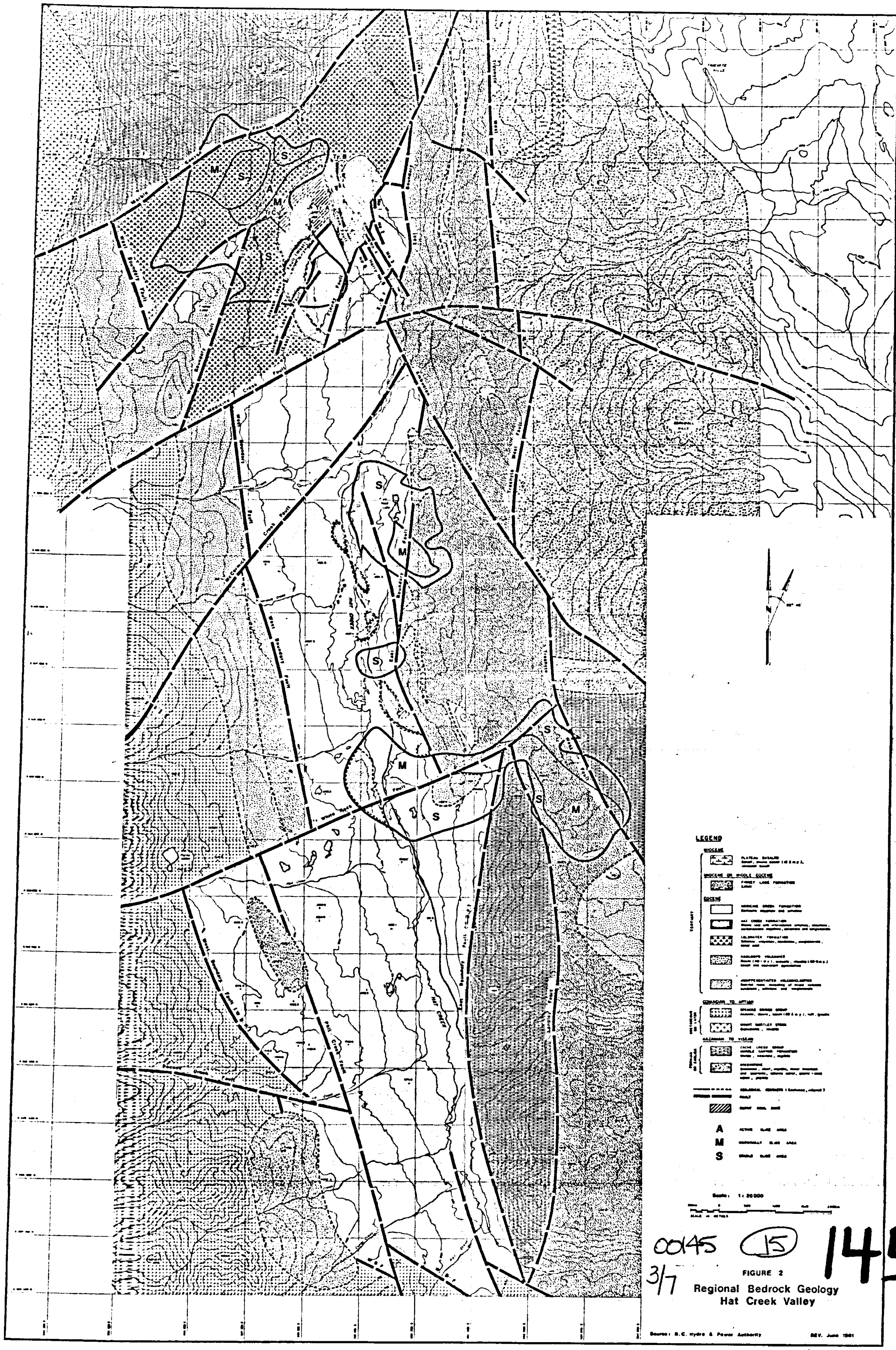


YEAR 35

200 0 200 400 600 800 1000 Metres

Source: B.C. Hydro, 800 MW Mine Plan, September 1982

Golder Associates/Sigma Engineering Ltd.		
B.C.Hydro & Power Authority		
HAT CREEK DIVERSION STUDY 00145 800 MW SCHEME 3/7 14 PIPELINE ARRANGEMENT		
PIT DEVELOPMENT YR. 10 & YR. 35		
Drawn J. NG	Checked	Reviewed
Date SEP 1982	Scale AS SHOWN	Drawing 16

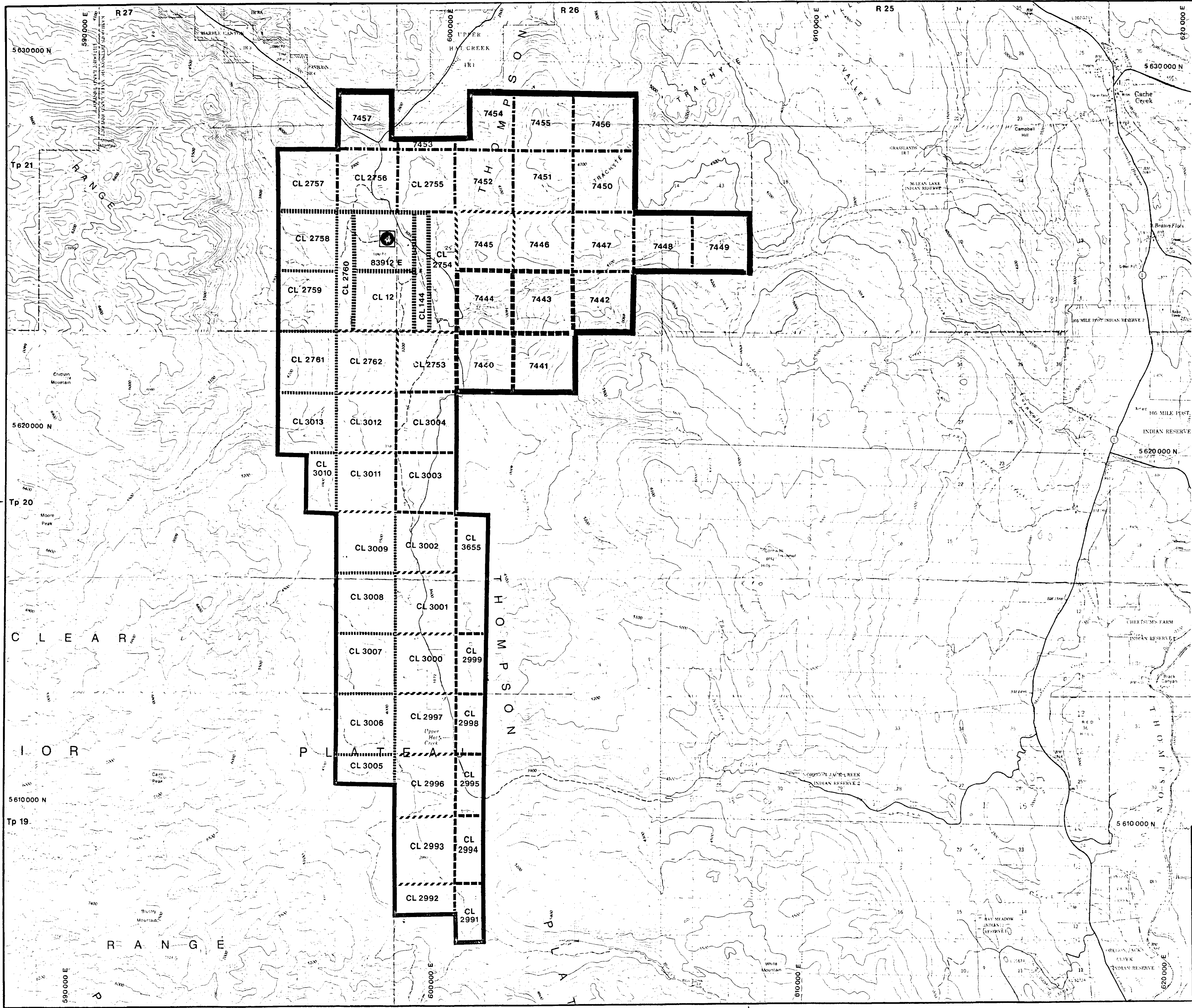


00145 15
3/7
145

FIGURE 2
Regional Bedrock Geology
Hat Creek Valley

Source: B.C. Hydro & Power Authority
REV. June 1981

HC - HAT CREEK 82(I)A



T.N.
N.G.
1°21'
22° 11'
Grid north
Magnetic north

CROWN GRANT

GROUP 1

GROUP 2

GROUP 3

GROUP 4

From National Topographic System Maps : 92-I/11, 92-I/12, 92-I/13, 92-I/14

1000 0 1000 2000 3000 4000 m
SCALE IN METRE : 1 : 50000

B.C. HYDRO
HAT CREEK PROJECT – MINING DEPARTMENT

00145
3/7
16

COAL LICENCES
UPPER HAT CREEK VALLEY

145

HC- HAT CREEK 80(1)A

DATE March 1983	FIGURE 3	R
DWN	DWG No.	

REPORT No.

D

D75-1001

CONFIDENTIAL

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
B.C. HYDRO
ON THE
HAT CREEK PROJECT
DIVERSION STUDY

FINAL REPORT
VOLUME 2, APPENDIX A

C.L. 7445

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Vancouver, British Columbia

5 of 7

December, 1982 822-1523

APPENDIX A

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1. Drillhole Logs
2. Petrographic Reports
3. Magnetic and Resistivity Surveys
4. Classification Parameters Used in NGI Tunnelling Quality Index (Q)

SUMMARY AND CONCLUSIONS

This Appendix gives an account of the work that was carried out during 1982 on the tunnel aspects of potential creek diversions at Hat Creek.

Previous work for the 2240 MW Pit showed that it would be necessary to move the proposed canal diversion into a tunnel approximately in Year 12, when the pit had encroached eastward to such an extent that the safety of the canal was jeopardized. No detailed investigation was carried out to assess the viability or cost of a tunnel diversion at that stage.

The 1982 studies, which examined the various methods of diversion for both the 2240 MW and the 800 MW Pits, considered not only diversion layouts in which the tunnel would one component of a canal/tunnel/pipeline arrangement, but other layouts also where diversion would be solely by tunnel. The possibility of using the tunnel schemes to enhance drainage of the pit slopes was also considerd.

Three main tunnel layouts were examined:

- a pressure tunnel located under the pit through weak rocks and granular deposits (Layout A)
- a tunnel located between the pit and the eastern escarpment, also through weak rocks and surficials (Layout B)
- a tunnel through the eastern escarpment predominantly through stronger rocks (Layout C).

Evaluation of these three alternatives showed that because of the depth of surficials, the potential pressures and inflows of ground water during excavation and limitations on methods of tunnelling through both surficials and weak claystones, Layouts A and B should not be considered further. Layout C merited detailed investigation; the summer field work program was oriented towards locating and proving tunnel alignments through the eastern escarpment.

Existing data was assessed and geological mapping and a geophysical survey were carried out to determine the geological framework of the eastern escarpment. On this basis, a program of investigation was carried out including diamond core drilling and field and laboratory testing to enable the definition of geotechnical units, and hence the selection of alternative tunnel routes through the escarpment. At the same time, current methods of tunnelling were studied to decide which would be the most appropriate methods of excavation for the types of ground anticipated.

Five geotechnical units (G1 to G5) were established, and four possible tunnel alignments (T1 to T3A) were selected for detailed evaluation. Methods of excavation for the four alignments and potential problems were considered. Requirements for tunnel support and lining were compared, as well as schedules and costs for different excavation techniques.

Cost estimates for the four routes are:

	$ M
T1	16.25
T2	18.07
T3	20.93
T3A	19.08

However, factors other than cost must be taken into consideration in the selection of a preferred alternative, namely: geological conditions and implied uncertainties, construction preferences (ease, simplicity, etc.) and remoteness from the pit (safety under abandonment plans).

The conclusions of the study were that the tunnel alignment T3A is the preferred route for inclusion in the overall diversion studies (see Main Text). This route is shown on Figure 12. It would involve tunnelling predominantly in the weak claystones/siltstones and sandstones for most of the length, with stretches towards the north and south ends in highly brecciated or jointed, altered andesite. Both portals would be developed in glacial deposits, and a significant length of the tunnel at the downstream end would be in dry granular surficials. Excavation would be by an excavator shield (see Figure 11 and Table 4), with a precast segmental lining. No major construction problems are foreseen, but either further investigation would be necessary, or a suitable contract should be formulated to provide for risk and responsibility at a level compatible with the degree of geological uncertainty.

HAT CREEK
TUNNEL ROUTE SELECTION AND EVALUATION

1.0 INTRODUCTION

Studies by Monenco Consultants Pacific (1977) of the various alternative arrangements for diverting Hat Creek and Finney Creek around the proposed open pit during operation of the Hat Creek Mine considered the need for a tunnel as part of the scheme. Such a tunnel appeared likely to be routed through the escarpment east of the pit. It could be driven either prior to excavation of the pit, or at a later date when the pit had expanded to a point at which the stability of a canal located between the advancing pit edge and the escarpment could be endangered. The scheme recommended by Monenco consisted of a canal diversion around the rim of the east side of the pit up until Year 26 (later amended to Year 12 by HEDD), when a tunnel would be driven to provide a permanent diversion for the creeks.

The objective of the present study by Golder Associates (in association with Sigma Engineering Ltd.) has been to reconsider the various methods of diverting the creeks around the pit in light of revised pit plans and new geotechnical data. The extent to which a deep level tunnel could achieve drainage of the east pit slopes, and hence improve slope stability, was given particular emphasis.

The main text of this Diversion Study Report (see Volume 1) covers the hydrological aspects of the study, the alternative overall layouts examined and the design and costing of the recommended arrangements for each of the two mine schemes (800 MW and 2240 MW), both for the long term (on abandonment) and during pit operations.

This Appendix covers all the considerations necessary for detailed appraisal and selection of the tunnel section of the diversion routes which could potentially form part of the recommended arrangement. A suggested alignment for the preferred arrangement for long term abandonment of the 2240 MW Scheme is presented.

2.0 DIVERSION BY TUNNEL

2.1 Requirements

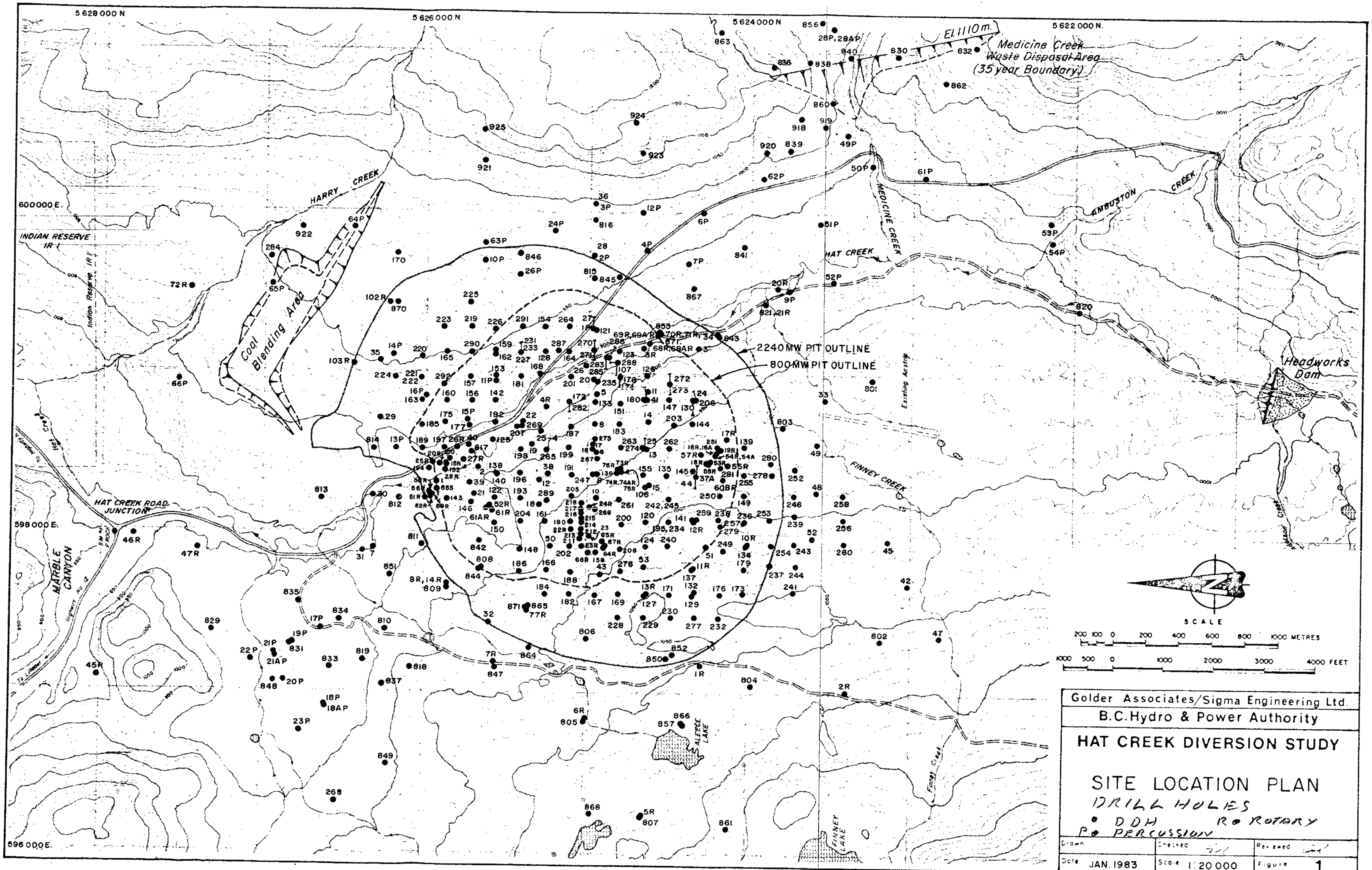
Excavation of the open pit for exploitation of the Hat Creek No. 1 Coal Deposit requires diversion of both Hat Creek and Finney Creek prior to and during the mining operation. The topography of the Hat Creek valley in the vicinity of the pit determines that the shortest and most economic routes lie to the east of the pit (see Figure 1). The east slopes of the valley are formed by a series of wide flat terraces, developed on glacio-fluvial granular sediments. A steep escarpment bounds these terraces to the east and represents the outcrop of a series of volcaniclastic and flow rocks. The ultimate limits of the pit slopes after the long term degradation of the walls of the final 35-year pit are likely to be close to the toe of the escarpment.

Diversion of the Hat Creek flows around the pit by a tunnel may be favoured for the following reasons:

- o because a tunnel provides a more economic alternative to canal or pipeline
- o because a tunnel provides a safer means of diverting the creek flows
- o because the integrity of the canal or pipeline cannot be assured after cessation of mining.

The tunnel is required to pass a 1000-year flood flow of 27 m^3/sec. It may be designed as a free-flow or pressure tunnel. The need to provide a tunnel lining depends on the hydraulic requirements, or whether or not tunnel support has had to be installed and on whether leakage from the tunnel to the surrounding formations is possible.

Tunnel alignments are dependent on the topography defining the portal locations, the pit configuration and the geology. The routes are also constrained by the Indian Reservation downstream of the pit and the need to return the water to the creek approximately at the Hat Creek Road Junction (G.R. 5628000N, 5982250E).



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B.C. Hydro & Power Authority

HAT CREEK DIVERSION STUDY

SITE LOCATION PLAN

DRILL HOLES

• DDH R• ROTARY

P• PERCUSSION

Drawn	Checked	Reviewed
Date JAN. 1983	Scale 1:20 000	Figure 1

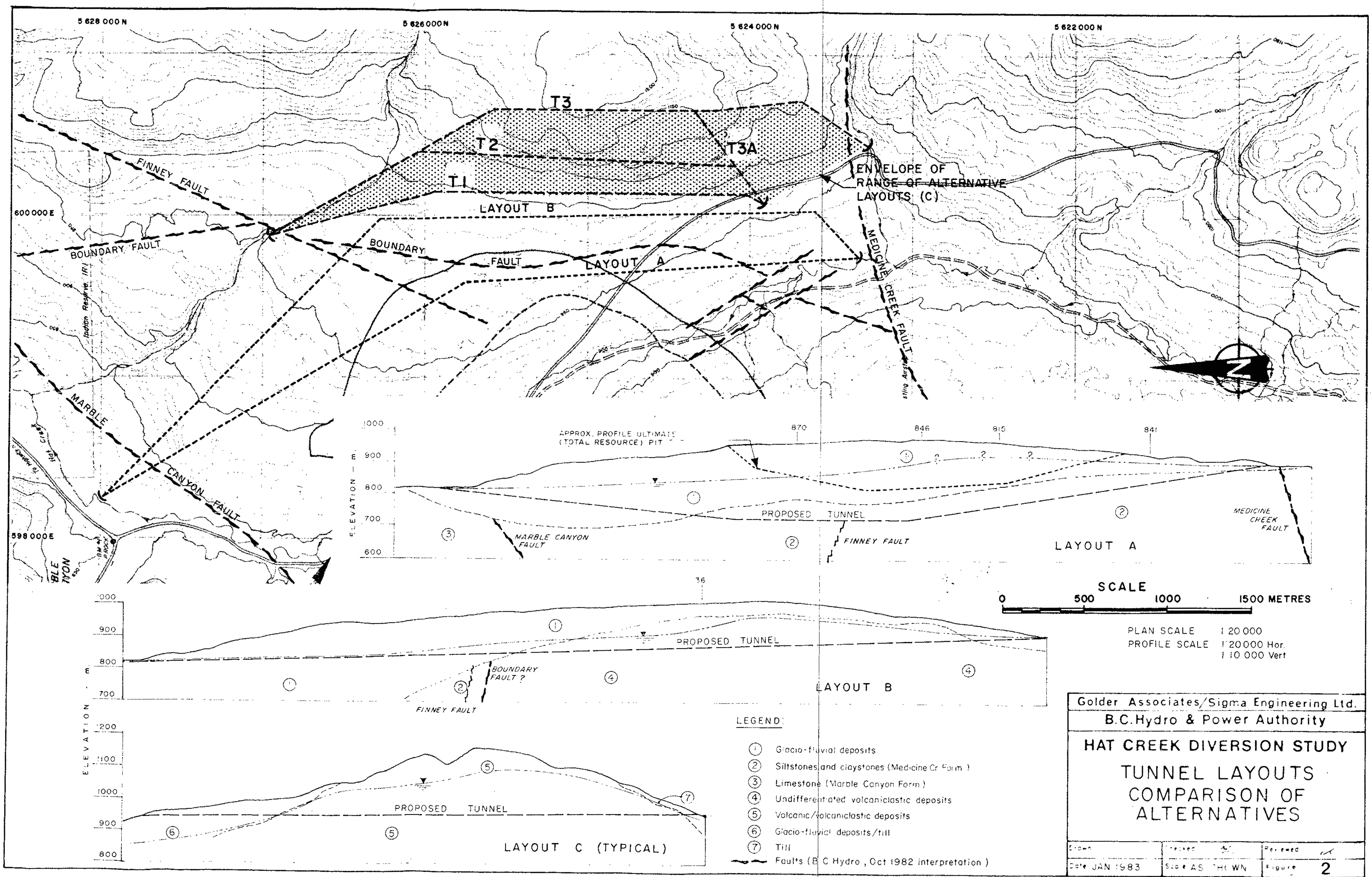
2.2 Alternative Tunnel Arrangements

As part of the considerations for various canal and pipeline diversion arrangements described in the Main Text, three alternative tunnel layouts were proposed for initial review. These have been designated Alternative Layouts A, B, and C, and are shown in Figure 2.

Alternative Layout A is a deep pressure tunnel aligned through the eastern half of the pit. Various arrangements of vertical/inclined shafts and tunnel could be incorporated into this arrangement. The upstream portal would lie at approximate elevation 920 m, just downstream from the confluence of Medicine and Hat Creeks. The downstream portal would lie at approximate elevation 810 m at the Hat Creek Junction. The tunnel would have to be sited at an elevation sufficiently low to avoid danger not only from the existing 35-year pit, but also from any future exploitation of the total coal resource.

Alternative Layout B would be driven from the same upstream portal, but would be aligned to the northeast of the pit rim through volcaniclastic deposits. The route would need to be judiciously selected to avoid the hard rocks of the escarpment on the east, and to lie below the thick glacio-fluvial deposits. The outlet portal would also lie at elevation 810 m at the Hat Creek Road Junction. This alternative would be a free-flow tunnel and would be driven at a uniform grade over most of its length.

Unlike Alternative Layouts A and B, Alternative C is not solely a tunnel diversion scheme, but really one element in a canal/tunnel and/or pipeline scheme which would comprise a diversion at a high level round the pit, minimizing the tunnel length and making use of the glacio-fluvial terraces as much as possible for siting surface structures.


5 628 000 N
5 626 000 N
5 624 000 N
5 622 000 N
600 000 E
598 000 E
T3
T2
T1
T3A
ENVELOPE OF
RANGE OF ALTERNATIVE
LAYOUTS (C)
LAYOUT B
LAYOUT A
FINNEY FAULT
BOUNDARY FAULT
BOUNDARY
FAULT
MEDICINE CREEK FAULT
MARBLE CANYON FAULT
APPROX. PROFILE ULTIMATE
(TOTAL RESOURCE) PIT
870
846
815
841
PROPOSED TUNNEL
MARBLE CANYON
FAULT
FINNEY FAULT
MEDICINE
CREEK
FAULT
LAYOUT A
ELEVATION — m
36
BOUNDARY
FAULT ?
FINNEY FAULT
LAYOUT B
LAYOUT C (TYPICAL)
SCALE
0 500 1000 1500 METRES
PLAN SCALE 1:20 000
PROFILE SCALE 1:20 000 Hor.
1:10 000 Vert
LEGEND
1 Glacio-fluvial deposits
2 Siltstones and claystones (Medicine Cr. Form.)
3 Limestone (Marble Canyon Form.)
4 Undifferentiated volcaniclastic deposits
5 Volcanic/volcaniclastic deposits
6 Glacio-fluvial deposits/till
7 Till
Faults (B.C. Hydro, Oct 1982 interpretation)
Golder Associates/Sigma Engineering Ltd.
B.C. Hydro & Power Authority
HAT CREEK DIVERSION STUDY
TUNNEL LAYOUTS
COMPARISON OF
ALTERNATIVES
Date JAN 1983
Scale AS SHOWN
Figure 2

For this reason, the diversion structure on Hat Creek would have to be some considerable way upstream to permit gravity flow without the need for pumping. Where this route lies close to the pit, a tunnel section through the escarpment would be necessary. Various options for portals and alignments exist within the escarpment and have been considered. Figure 2 shows an envelope which bounds all the alternative tunnel routes considered.

For consideration of the various design and construction aspects of potential tunnels, this Appendix has been set up in the following way:

<u>Section 3</u> - describes the geology of the east side of the pit area and escarpment.

<u>Section 4</u> - discusses construction methods appropriate to the geology for various alternative layouts.

<u>Section 5</u> - evaluates Alternative Layouts A, B and C (described above) and discusses the logic of selecting Alternative Layout C for detailed study.

<u>Sections 6 through 9</u> - deal with the detailed investigation of the alternative tunnel routes through the escarpment within Layout C, the anticipated conditions, comparative costs, and the recommended route for inclusion in the overall scheme discussed in the main text of the report.

3.0 GEOLOGY OF EAST SIDE OF PIT

3.1 General

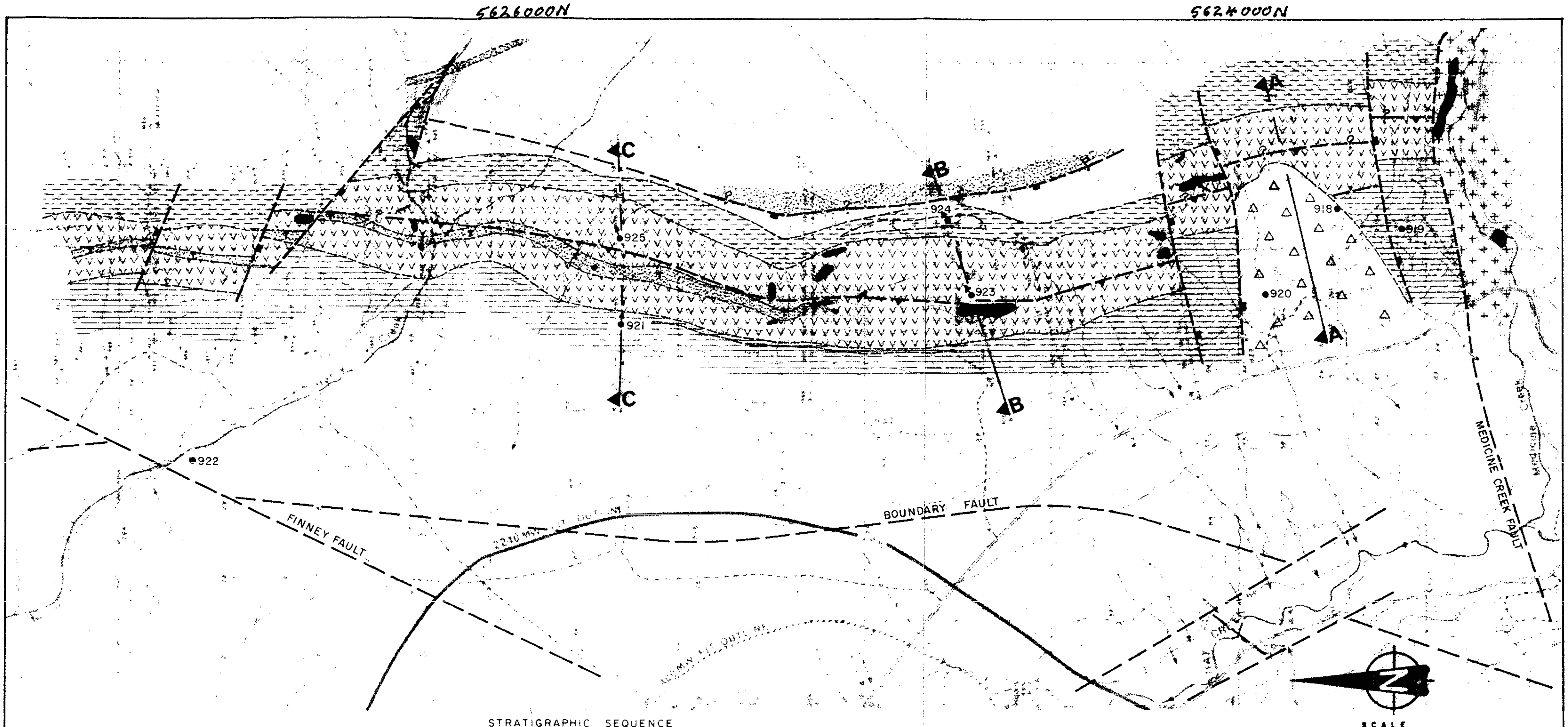
A comprehensive discussion of the regional geology covering the east side of the pit may be found in Golder Associates' reports, dated March, 1977 and December, 1978. No detailed attempt has been made to re-examine the regional geology during this study. The current investigation was focussed on the geology within the eastern escarpment of the Hat Creek Valley along the corridor containing the proposed diversion tunnel routes between Medicine and Harry Creeks.

In addition to the re-interpretation of existing data carried out for this investigation, new data on the eastern escarpment was obtained from detailed geological mapping, core drilling, magnetic and resistivity profiling across the eastern escarpment, petrographic analysis of core samples and rock testing.

As a result of the drilling and geophysical works, a good stratigraphic and structural framework has been established along the proposed diversion tunnel alignments through the eastern escarpment. This has permitted the definition of geotechnical units for tunnelling purposes.

3.2 Stratigraphy of East Side of Pit

The stratigraphy within the eastern escarpment in the area of the proposed diverson tunnel alignments is shown in Figures 3 and 4. Stratigraphic units have been subdivided into the sequence (in order of increasing age) shown on Table I. Figure 5 shows the comparative stratigraphic sequences encountered in the various holes drilled within the eastern escarpment. The following sections describe the stratigraphic units in detail.



LEGEND

Geological boundary (approximate)

Geological boundary (conjectural)

Fault (definite)

Fault (assumed)

Fault (conjectural)

Normal displacement showing downthrown side

Reverse - teeth on upper plate

Geological section (See Figure 4).

925 Drillhole - 1981/82 tunnel diversion program, showing orientation of hole.

NOTE: Surficial, colluvial and glacial deposits are not represented

STRATIGRAPHIC SEQUENCE

Glacio-fluvial sediments (shown on sections only)

Slide debris

Upper volcaniclastics

Altered Andesites

Lower volcaniclastics

Sandstone, siltstone, claystone

Outcrop

Tuffaceous conglomerate interbed

Interbedded tuffaceous sediments, basalt or lithic breccia

SCALE

100 50 0 100 200 300 400 500 METRES

500 250 0 500 1000 1500 2000 FEET

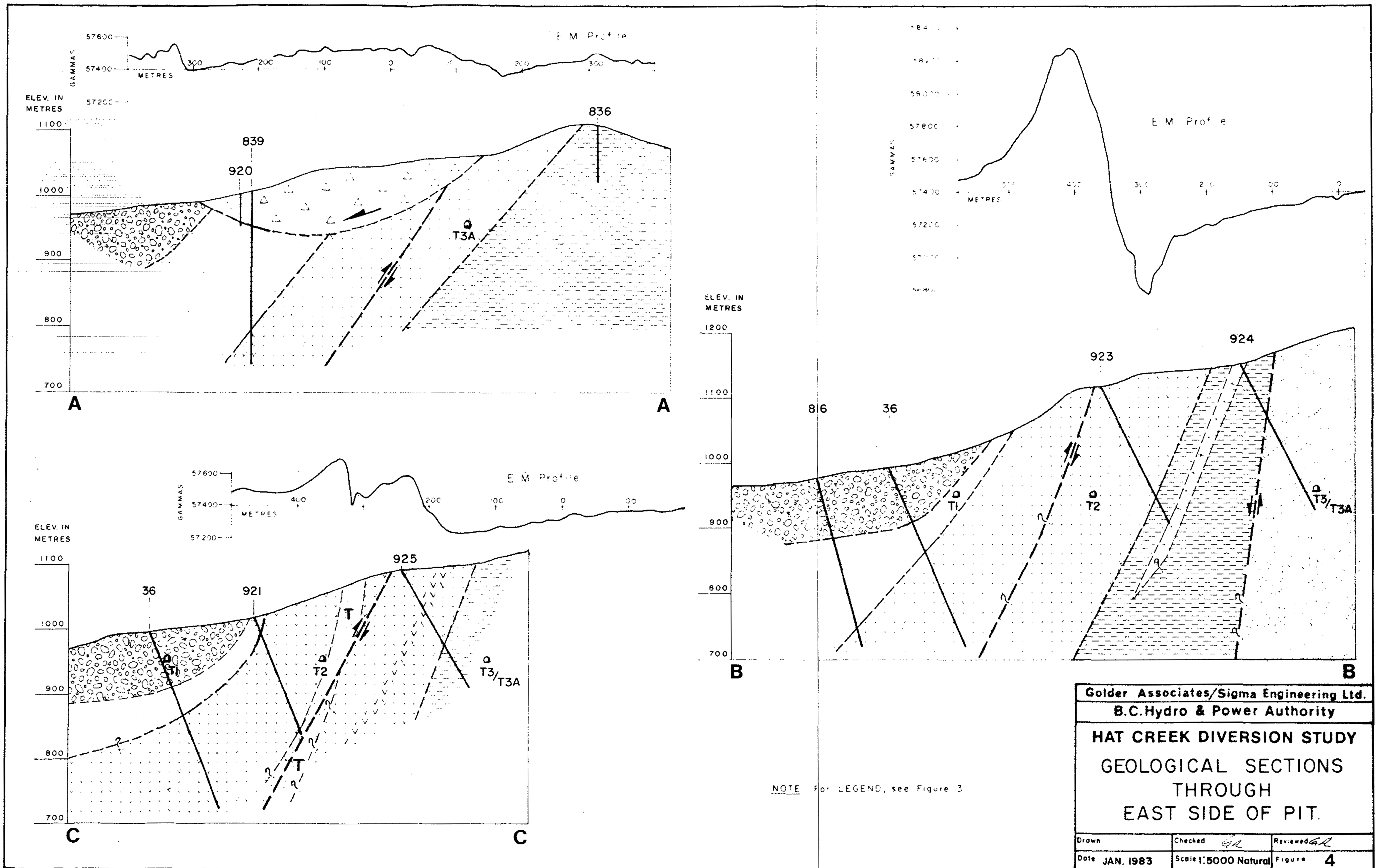
Golder Associates/Sigma Engineering Ltd.

B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

GEOLOGY
EAST SIDE OF PIT

Drawn	Checked	Reviewed
Date JAN 1983	Scale 1 10 000	Figure 3



Golder Associates/Sigma Engineering Ltd.
B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

GEOLOGICAL SECTIONS THROUGH EAST SIDE OF PIT.

Drawn	Checked GR	Reviewed GR
Date JAN. 1983	Scale 1:5000 Natural	Figure 4

TABLE 1

STRATIGRAPHIC SEQUENCE IN EASTERN ESCARPMENT

Quarternary	Recent	- Colluvium, alluvium, slide debris.
	Pleistocene	- Glacial sediments, till.
Tertiary	Kamloops Group	- Upper volcaniclastics - sandy siltstone, clayey siltstone, tuffaceous sandstone, conglomerates, and minor andesites.
		- Altered andesites - breccias, agglomerates, and occasional conglomerates.
		- Lower volcaniclastics - siltstone, claystone, tuffaceous sandstone and siltstone, conglomerate, lithic breccia, and minor carbonaceous claystone/siltstone.
Permian	Cache Creek Group	- Non-tuffaceous sandstone, siltstone, and claystone.
		- Marble Canyon Limestone - marble, limestone, argillite.
		- Greenstone, chert, argillite with limestone and quartzite.

3.2.1 Distribution of Deposits

The various tunnel layouts have all been selected to pass through rocks east of the Finney Fault, which represents the easterly limit of the No. 1 Coal Deposit (Golder Associates, 1978). Uniform claystones and siltstones of the Medicine Creek Formation are present between the Finney Fault and the volcanic/volcaniclastic rocks of the escarpment. Tunnels of Alternative Layout A would lie within this rock sequence.

The volcanic/volcaniclastic sequence forming the escarpment strikes nearly north-south, parallel to the ridge and abuts against older rocks forming the higher ground to the east on which the power plant is sited. Tunnels of Alternative Layouts B and C would lie within this sequence.

The glacio-fluvial deposits infill a channel cut into the Medicine Creek Formation rocks. This channel deepens and widens to the north where it lies over the Marble Canyon Limestone of Permian age (see Figure 6).

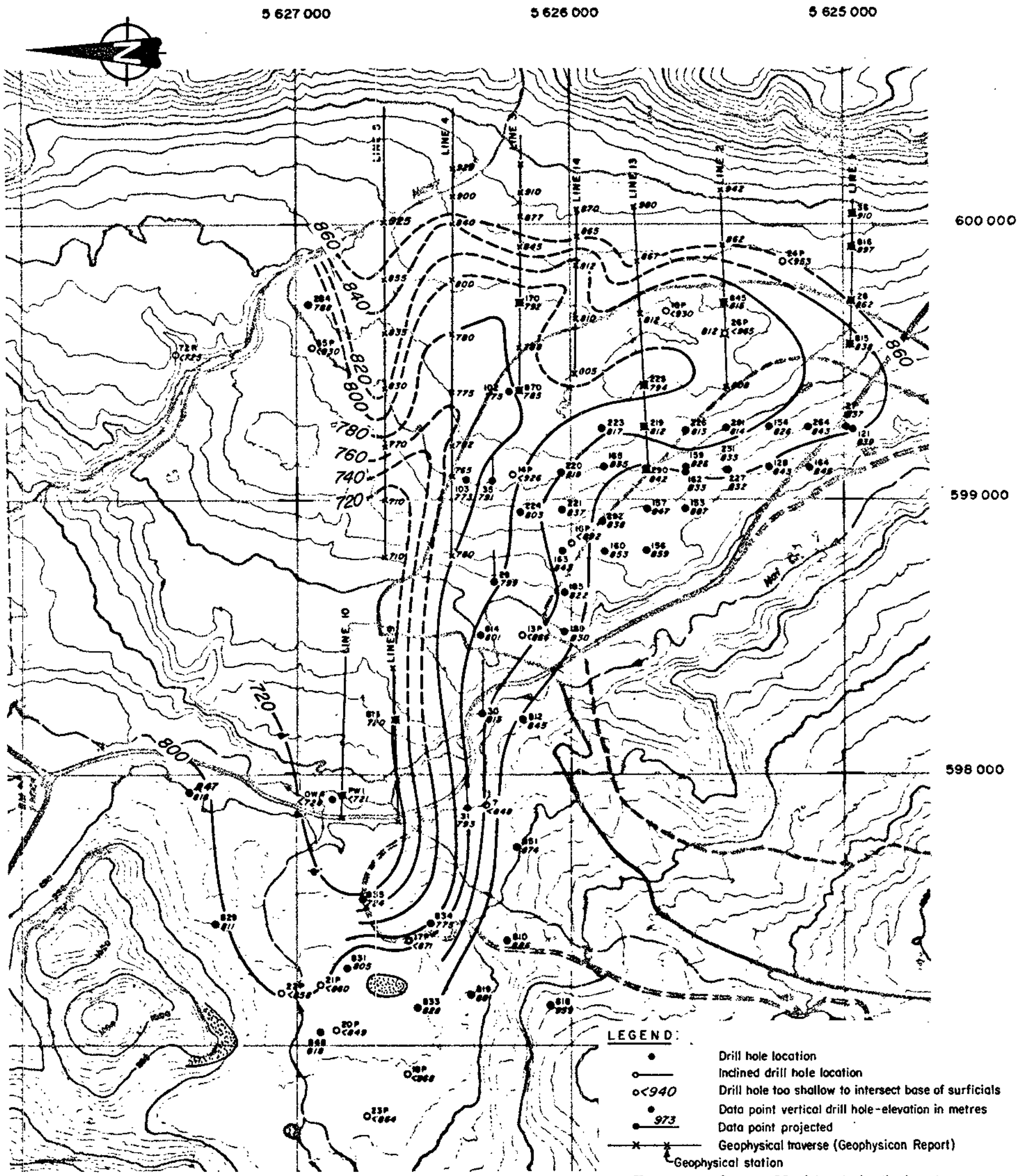
The rocks west of the escarpment have generally been well investigated in previous years. The escarpment rocks are generally poorly known, except at the south end on the axis of the Medicine Creek waste embankment. The current investigation was therefore concentrated on obtaining data for those tunnel layouts passing through the rocks forming the escarpment. The geology of this area, as deduced from the 1982 investigations, is shown on Figures 3 and 4.

3.2.2 Surficial Deposits

These deposits are composed almost entirely of glacial till and glacio-fluvial outwash sediments in the vicinity of the diversion tunnel alignments.

CONTOURS ON THE BASE OF THE SURFICIALS - NORTHERN PIT RIM.

Figure 6



LEGEND:

- Drill hole location
- Inclined drill hole location
- o<940 Drill hole too shallow to intersect base of surficials
- 973 Data point vertical drill hole - elevation in metres
- Data point projected
- Geophysical traverse (Geophysicon Report)
- Geophysical station
- Contours - 20m. interval - elevation in metres
- Contour inferred from geophysics.
- Outcrop boundary

NOTE: Geophysical contours based on 110% of inferred depth, to compensate for presence of basal fill.

PROJECT NO. 822-1524B DRAWN G.A. REVIEWED DATE Nov. '82

Golder Associates

SCALE 1:20 000

(a) Slide Debris

Slide debris is confined to the area located approximately 500 m north of Medicine Creek (Figure 2). A 31.7 m thick section of slide debris was encountered in hole DDH81-920 which was drilled in 1981 for the south portal investigation. Drillhole data, geophysical interpretation, and geomorphological expression were used to define the limits of the slide debris which typically consists of a multilithological mixture of boulders (primarily volcanic), cobbles, and gravels with a clay matrix.

(b) Glacio-fluvial Sediments

These deposits infill a trough eroded into the Tertiary rocks located between the faulted eastern margin of the coal and the escarpment to the east. Contours on the base of that trough are shown in Figure 6. The sediments are typically composed of moderately dense to dense gravel and cobbles in a silt, clay, and sand matrix, and local beds of medium to coarse sand. Occasional large boulders are also encountered.

(c) Glacial Till

One metre of glacial till was encountered near the bottom of DDH82-922, at a depth of 23.9 m, in the area of the northern portal adjacent to Harry Creek. The till is typically overconsolidated and composed of gravel and cobbles in a silty sand matrix. Other layers of till are inferred to exist in this portal area.

3.2.3 Upper Volcaniclastics

This unit is moderately well defined and apparently consistent along the base of the eastern escarpment (Figure 3). The upper volcaniclastics were penetrated by Drillholes DDH82-921, DDHs81-918 and 920, DDH76-816, DDH78-839, and DDH75-36 (logs of holes drilled during this investigation are attached as Addendum 1). Drillholes 918 through 920, and 839, are located in the southern (upstream) portal area, resulting in greater subsurface stratigraphic control here than in the northern portal area.

The upper volcaniclastics generally consist of a sequence of sandy siltstone, clayey siltstone/siltstone, sandstone/tuffaceous sandstone, conglomerate/tuffaceous conglomerate, andesite and/or basalt flows and volcanic breccias. The stratigraphic thickness of the upper volcaniclastics is not well established, but based on Drillhole DDH76-816, which penetrated the base of the unit, these rocks are at least in the order of 140 m (460 ft).

The top of the unit is not visible. It is possible that the upper volcaniclastics represent the lateral stratigraphic equivalent of the Medicine Creek Formation, although the possibility also exists that a fault separates this unit from the claystones proved further west.

3.2.4 Andesite

The andesite comprises the majority of the rocks of the eastern escarpment and has resisted erosional processes to form a prominent topographic feature. These rocks do occur in small isolated outcrops, but they are normally mantled by a thin veneer of colluvium and/or glaciofluvial deposits. The colluvium may form appreciable thicknesses at the base of the escarpment. These rocks are characteristically dark to medium grey and aphanitic; the unit is typically highly fractured and locally brecciated. Results of petrographic thin section analyses indicate the andesite has undergone varying levels of hydrothermal alteration, varying in intensity of alteration from slight to high. The more highly altered rocks may contain as much as 80 to 85 per cent alteration products; these products were difficult to identify definitively under the microscope, but appear to be smectite clays. The pervasiveness of alteration ranges from local occurrences along fractures within fairly intact rock, to being ubiquitous in crystal-lithic tuff-breccias where intense alteration has occurred in both the matrix and lithic fragments. The more highly altered rocks are usually very friable and relatively lower in strength (see Section 6.2.4).

Drillholes DDHs78-839, 81-918, 82-921, 923 and 925, penetrated the andesite. Based on several lines of evidence (discussed in Section 3.3), this suite of rocks appears to be repeated by a high angle reverse fault (Figure 4).

Stratigraphic thickness is in excess of 100 m. However, the fault controlled thickness of the sequence ranges between 230 and 260 m.

3.2.5 Lower Volcaniclastics

The lower volcaniclastics are only moderately well defined, and occur in limited outcrop in the area just north of Medicine Creek, and in an outcrop adjacent to Harry Creek (Figure 3). This lithologic unit is characterized by an interbedded sequence of siltstone, claystone, minor beds of conglomerate, tuffaceous sandstones and siltstones and lithic breccias. At a depth of 90 m, in DDH82-924, a sequence of primarily tuffaceous claystone/siltstone/sandstone is in fault contact with a sequence of non-tuffaceous, finely bedded sandstone, siltstone and claystone. Many of the finely bedded sandstone beds exhibit cross-bedding, soft sediment deformation and well preserved faulted bedding offsets.

These rocks are thought to represent the Cache Creek group of Permian age and, for engineering purposes, have been grouped with the tuffaceous rocks on the west side of the fault.

The lower volcaniclastic sequence was penetrated by drillholes DDH82-923, 924 and 925, DDH78-836 and 838. The stratigraphic thickness of the lower volcaniclastics is uncertain, as none of the drillholes penetrated the entire sequence of rocks, but 250+ m have been proven.

3.3 Structure of the Eastern Escarpment

It is not the intent of this report to present a complete discussion of the structural geology of the Hat Creek Basin. For this, reference should be made to previous Golder Associates Reports (1977, 1978), the Hat Creek Project Mining Report (1979), the Hat Creek Project Coal Liquefaction Report (1980) and to Kim (1978).

The eastern escarpment of the Hat Creek Basin, through which the proposed diversion tunnel would pass, is characterized by a sequence of steeply dipping (average 75° to 80° to the west) volcanic and volcaniclastic rocks described in Section 3.2. These rocks have been disturbed by at least two fault sets. One fault set trends nearly north-south, to slightly east of north; the trend of the second fault set varies between N50°W to N80°E. The dips of these faults are not well known, as the density of drillholes and drillhole depths preclude detailed analysis. The presence of these faults is substantiated by drillhole data, outcrops, and surface and down-hole geophysical data. The north-south trending fault set is apparently older, as it appears to have been offset in several locations by the cross faults.

The north-south set is represented predominantly by a major fault parallel to the ridge which repeats the andesite sequence (see Figure 4).

It is located almost entirely within the altered andesite and extends to the north from Medicine Creek to just north of Harry Creek. Correlation of down-hole neutron logs, and repetition of three alteration zones within the andesite sequence, indicate that the fault has repeated the sequence. The locations of the drillholes preclude a dip any shallower than 56° to the west or 24° to the east. The most probable dip direction is to the west, based on bedding dips and the assumption that the faulting would most likely occur along planes of greatest weakness, i.e. subparallel to bedding planes. This fault has been truncated at its extreme ends; just north of Harry Creek, the fault has been cut off by a cross cutting normal fault. In the area just north of Medicine Creek, it has been offset in an east stepping en echelon pattern by three cross-cutting south-side-down, normal faults. A major east-west fault truncates the north-south fault at Medicine Creek; this fault juxtaposes the west dipping sequence of the upper and lower volcaniclastics and altered andesite against the south dipping sequence of basalt and volcaniclastic rocks.

3.4 Hydrogeology

The tunnel routes investigated pass through two hydrogeological units; the surficial materials and the bedrock volcanics/volcaniclastics, and claystones and siltstones. Both units exhibit variations within hydrogeological parameters, with the surficial materials generally constituting the major water bearing units.

The eastern escarpment is primarily located in a zone of ground water recharge with decreasing hydraulic heads with depth. The distribution of piezometric levels within the bedrock is probably controlled by the hydraulic conductivity of the rock and the structural geological conditions present. It is likely that the permeability of the ground through which the tunnel will pass is highly anisotropic, but no quantitative data is available to substantiate this. Both horizontal and vertical ground water flow components may be in operation. Recharge to the bedrock units is via infiltration of precipitation to the saturated zone and from downslope seepage through bedrock to the east. The general direction of ground water flow in the saturated materials is downward and toward the ground water discharge areas in Hat Creek. In the area of the northern portals of Layouts A and B, ground water flow has a vertically upward component with hydraulic heads increasing with depth, characterizing a ground water discharge area.

4.0 ALTERNATIVE METHODS OF TUNNELLING

The most suitable method of excavating a tunnel depends on the geological conditions, the size, shape, and length of the tunnel and, to a lesser extent, the construction time constraints and end-use requirements. This section of the report:

- o briefly summarizes tunnel geometry considerations in the selection of the tunnelling method;
- o reviews, in terms of the general geological conditions anticipated, the various excavation methods available;
- o evaluates for the particular conditions at Hat Creek the design constraints on the selection of tunnelling techniques.

The inter-relationship of various technical factors affecting the choice of the excavation method is given in Figure 7.

Section 5 evaluates in greater detail the various methods of tunnelling for Alternative Layouts A, B, and C described in Section 2.0, and discusses which of these layouts merits detailed further investigation. Suitable excavation methods for the various routes within the preferred alternative layout, based on the interpretation of the geological data for each tunnel route, are described in Section 6.0.

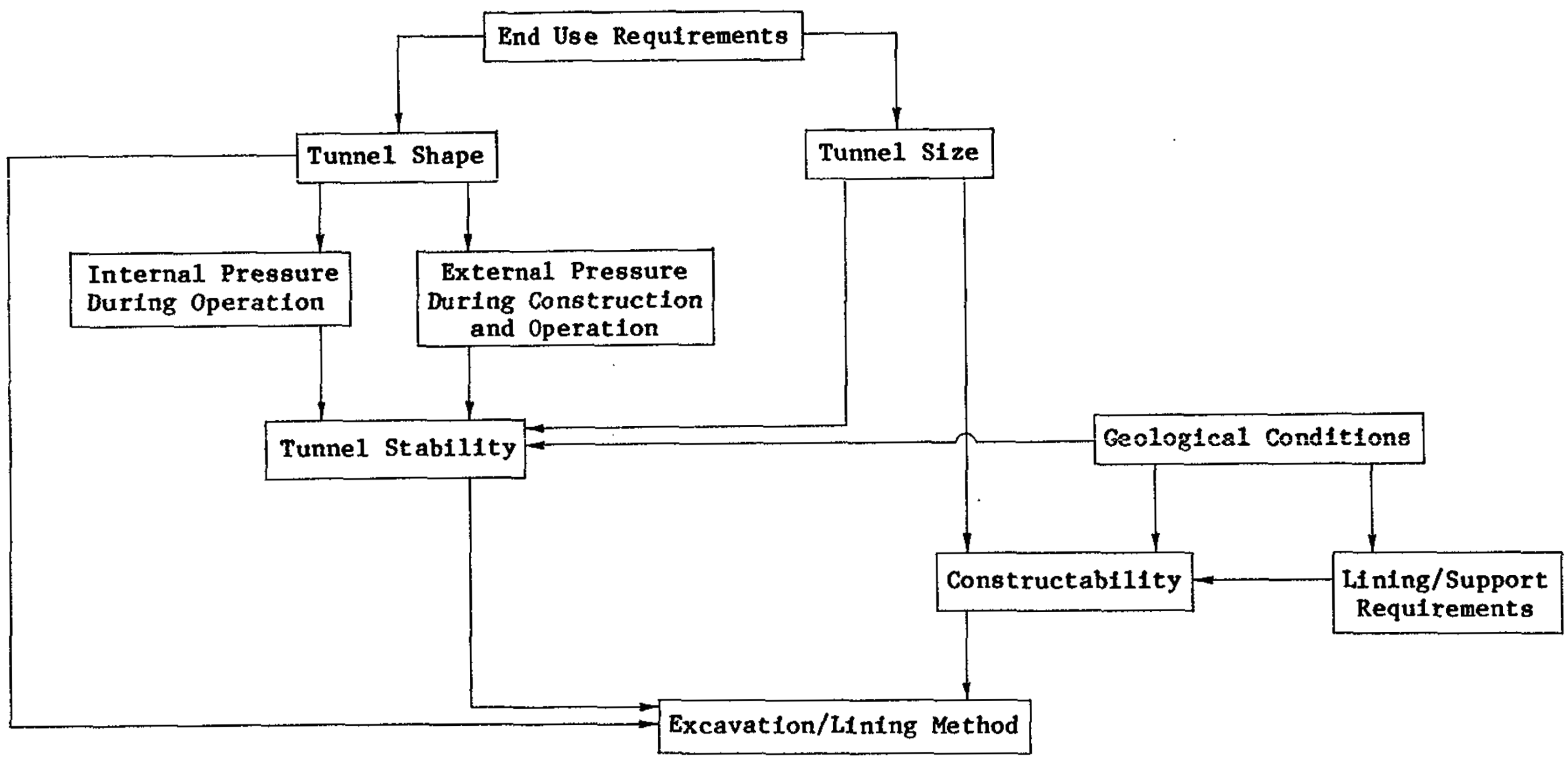
4.1 Tunnel Geometry

The minimum average tunnel size that can be constructed using modern equipment is controlled by the requirements for mucking, loading and drilling vehicles. Technical limitations exist in the use of mechanized boring machines larger than about 9.2 m (30 ft) diameter.

For the diversion tunnel, a diameter of about 2.4 m (8 ft) is required for hydraulic reasons. This is at the lower limit for the

FIGURE 7

TECHNICAL FACTORS AFFECTING CHOICE OF EXCAVATION METHOD



efficient use of men and equipment in the congested heading area. For this reason, both conventional and machine-driven tunnels are rarely less than 3.0 m (10 ft) in effective diameter. This factor is the basis for the choice of tunnel size for the diversion alternatives under consideration.

The shape of the tunnel is determined in the first instance by end-use requirements (hydraulic or vehicular, for example). Stability considerations during both construction and operation of the tunnel may also strongly influence the shape of the tunnel. If these requirements are not critical, the tunnel shape selected may be a simple consequence of the method of excavation. A circular tunnel is ideal for resisting both external and internal pressures, and is also compatible with the construction requirements of many tunnelling machines. The flat floor of a D or horseshoe-shaped tunnel is a distinct construction advantage if a conventional excavation method is used. If a flat invert is used for the final tunnel shape, concrete volumes and hydraulic efficiency are not adversely affected.

In competent soil or rock, where structural linings are not required to ensure stability, tunnel shape is not a significant geotechnical design consideration.

4.2 Construction Methods

4.2.1 Soft Ground

The relatively low strength and the high deformability of tunnels in soft ground strongly influence the method of tunnelling. Soft ground tunnels can be excavated either by hand or machine, with or without the protection of a shield. Unless the soil is continuous and cohesive, some form of immediate support to the roof, and even the face, may be necessary. In many instances, a pre-constructed lining is erected under the protection of a shield.

Hand Mining utilizes manual labour and pneumatic equipment. Excavation proceeds continuously, with the mucking cars bringing in the support (temporary lining and lagging). Concreting is often performed immediately following excavation but out of cycle, so as not to interfere with the advance. Excavation rates are slow - in the order of 3.0 m/day (10 ft/day). Hand mining is most suitable for small tunnels, where grade and direction change rapidly, or for short sections where high initial capital equipment costs make mechanized methods prohibitive.

Mechanized Excavation in soft ground can be carried out using a rotating wheel cutter mole, a rotating drum (annulus) cutter, or a part-face excavator-digger. The part-face machine uses a rotating or swivelling bucket arm on an extension boom. The mole or drum frequently uses a simultaneously placed precast lining for thrust reaction in the weak ground. Only circular shapes can be excavated, albeit with high advance rates [approximately 25 m/day (80 ft/day)] and little ground disturbance. The excavator-digger is more versatile in terms of the range of ground conditions and cross-sectional shapes that can be handled. Advance rates are higher than for hand-mining, but are usually less (in the order of 50 per cent) than those achieved with full-face machines under their respective preferred geological conditions.

4.2.2 Rock

Excavation of tunnels in rock are carried out by conventional drill-and-blast, part-face (roadheader) type machines, or boring machines.

The Drill, Blast, and Mucking procedure is applicable to the full range of tunnel size, shape, and geological conditions, although unit excavation and support costs may be extremely high in certain types of ground. The technique has the important advantage over machine excavation in its adaptability to variations in geology. Construction outcomes are thus more predictable, especially if topographical conditions limit the level of pre-construction geological investigation. The development of modern support systems (bolts and shotcrete), and the increasing use of mechanization, has greatly enhanced the cost and construction time aspects of conventional tunnelling.

Roadheader Machine-Driven Tunnels have resulted in very competitive tunnelling performances being achieved with the recently developed roadheader technology. These machines attempt to exploit the advantages of mechanized excavation, while retaining the benefits of minimal equipment cost and flexibility in coping with variable and unforeseen ground and support requirements. Advance rates are limited by the rate at which energy can be dissipated at the boom type cutter head. Excavation is limited to rock of low or moderate strength up to 138 MPa (20,000 psi).

Full-Face Boring Machines are now able to cope with all uniform ground conditions; roller bit technology has advanced to the stage where no technical limitations exist as to the type of rock that can be excavated. However, very hard or abrasive rock presents serious constraints on economic viability as dictated by advance rates and roller costs. Average advance rates are in the order of 150 m/week, (500 ft/week), or 2 to 3 times the rates achieved by conventional excavation. The advantages of the boring machine are: elimination of blast vibrations, rock disturbance and overbreak; reduction in concrete and muck handling quantities; and high rates of advance. The main disadvantages are: the limitation to a circular shape; the minimum radius of curvature and maximum tunnel grades that can be excavated; the high initial cost; and the rock quality required. The use of a tunnel boring machine is not recommended in rocks with an RQD less than about 50 per cent. Boring machines are custom designed for the strength and hardness of the rock expected. Therefore, if rock conditions differ from those anticipated or are variable over the length of the tunnel, operations are likely to be difficult or inefficient.

4.3 Tunnelling Method Selection Criteria

The interpretation of the above factors for the diversion tunnel under study results in the following considerations and constraints on the choice of the excavation method:

(a) The length of the proposed tunnel alternatives is adequate to justify the high initial capital cost of a full-face boring machine or roadheader. The construction of a new machine with features especially suited to the expected ground conditions is expected to be cost-effective, if the choice were made to use mechanized excavation.

(b) In terms of end-use requirements, tunnel size and shape do not present any constraints on the method of excavation. A circular tunnel shape would be acceptable if either a full-face or part-face machine were used. Tunnel sizes of the order of 3 m (10 ft) diameter can be readily excavated with all excavation techniques.

(c) The choice of excavation method is dictated almost exclusively by the geological conditions anticipated. The aspects of significance in this regard are:

- Variability of geology along the proposed route. This will affect the type and length of excavation in each rock type;

- The uncertainty of the geological predictions. Part-face machines or conventional tunnelling is favoured where ground conditions are not readily predicted;

- Stability of the ground against erosion. If a concrete lining is required for hydraulic efficiency or to prevent erosion, a tunnelling method which efficiently integrates temporary and permanent support may be more cost effective;

- Stability of the tunnel against ground and ground water pressures. Although overburden pressures are relatively low, sections of weak ground may need specialized excavation and support techniques, or may require the use of circular sections to control squeezing/swelling ground.

An evaluation of constructability and optimum methods of excavation is given in Section 7.0, following a review of geological conditions and the selection of a suitable overall tunnel layout (Alternatives A, B or C) in Section 5.0.

5.0 EVALUATION OF ALTERNATIVE LAYOUTS

5.1 Layout A

The location of the possible tunnel alignment for Layout A is shown on Figure 2, and a geological sketch along the tunnel profile is shown on the same figure.

Layout A would encounter bentonitic claystones/siltstones of the Medicine Creek Formation for the upstream two-thirds of the tunnel length. The downstream third would be driven through glacio-fluvial sands and gravels, or possibly glacio-lacustrine silts. It is estimated that up to 120 m head of ground water could be operative on the tunnel during construction if it were excavated prior to the development of the pit itself. Although an average hydraulic conductivity of 1×10^{-6} m/sec in the surficial soils is not high, piping failures at the face would be likely, as well as zones of high inflow where more open gravels occur.

It is anticipated that some serious problems could result due to softening, squeezing and swelling of the weak rocks which contain expansive clay minerals. Difficulties could be encountered in advancing a tunnel boring machine under these conditions and such conditions could be exacerbated by the potentially high ground water pressures.

The depth of the pit would require that a tunnel along the alignment indicated would need to be driven at approximately elevation 725 m to avoid interaction with the pit excavation. This would necessitate a shaft or a long section of inclined tunnel, either of which would likely present problems in the materials described above.

However, the primary problem of this tunnel layout would be that of ground water control in the surficial deposits. This problem has been reviewed in detail by our tunnelling consultants, Mr. A.A. Mathews and Dr. Z. Eisenstein.

The potential methods of water control considered have included grouting, freezing, compressed air, and dewatering. Grouting from the surface or ahead of the face is considered to be either impractical or ineffective. The advance rate would be very slow, tunnelling costs would be high, and there would be a serious risk of having to close down the tunnel as the result of excessive inflows. Freezing was considered to be beyond the available technology, and uneconomical for the tunnel lengths contemplated. The use of compressed air for water pressures up to 60 m head and beyond is not considered viable; such highly specialized tunnelling operations are also expensive.

Although costs would be high, some form of dewatering is considered to be the only viable method of constructing the tunnel within the surficial materials below the water table. Even with dewatering, stringent design and construction standards would have to be met. Firstly, the construction of the tunnel would require some form of full-face closed shield machine, with the attendant lack of flexibility in dealing with other rock sequences that might be encountered. Secondly, unless the lining were completely impervious (anticipated to be both difficult and expensive in these materials), considerable water inflow would occur in the long term, substantially modifying the operational function of the diversion tunnel.

In consequence of this qualitative but careful appraisal of tunnelling conditions, Alternative Layout A was discarded from further consideration.

5.2 Layout B

Alternative Layout B is also shown on Figure 2. A tunnel on this alignment would skirt the pit excavation, but would be driven half in the upper volcaniclastic sequence and half in surficial deposits. High ground water heads could also be locally operative on this tunnel excavation near the rock/surficial contact, although generally the heads in the surficials would likely be low at the downstream end. It has been estimated that the average inflows in rock, assuming a medium hydraulic conductivity of 5 x 10^{-6} m/sec, could be 7 x 10^{-4} m^3/sec/m of tunnel.

Similar tunnelling problems, but not as severe, are anticipated for Layout B as for Layout A. Ground water control, even for the more modest heads of this layout, would be at the limit of, or beyond, state-of-the-art methods in current tunnelling practice. For this reason, Layout B was also discarded.

5.3 Layout C

Layout C, as shown on Figure 2, is an envelope of the various routes that could be selected which pass through the stronger rocks of the east escarpment. Various alignments and different portals can be developed for a number of tunnel routes which could be driven through this ground. When Layout(s) C are compared with those of A and B, it is immediately apparent that there are several major advantages: more resistant, and hence stronger rocks, form the core of the escarpment; geological reconnaissance indicates that these rocks are continuous over the proposed routes; surficial deposits would be encountered only at higher elevations compared to the other layouts, and operative ground water heads would likely be much lower; ground water levels and permeabilities would likely be much lower in the rock sequences of the escarpments than in the troublesome surficial deposits of the other layouts; a range of different rock sequences appear to exist, permitting the optimization of tunnel routes.

This route would be largely within volcanic/volcaniclastic sequences. Ground water inflows would depend on the hydraulic conductivity of the rock and the prevailing ground water heads outside the tunnel. Based on an average hydraulic conductivity of 5×10^{-6} m/sec for these rocks, ground water inflows are estimated as 1×10^{-3} m^3/sec/m length of tunnel. However, higher inflows are likely to occur where the tunnel breaks through into a more fractured or faulted zone, but these higher flows will probably be short lived. Ground water inflows are not anticipated from the surficial materials, although some perched ground water may be encountered, producing temporary inflows.

The tunnelling difficulties anticipated in Layout C are amenable to current technology and no insurmountable problems are expected. For this reason, the initial screening of the three main tunnel alternative layouts for the proposed diversion concluded that further detailed investigation should be carried out only on Layout C. In consequence, a program of investigation was carried out in Summer, 1982, which centred on the escarpment east of the planned pit excavation. Section 6.0 describes the work carried out, the results obtained, and the choice of routes for detailed evaluation.

6.0 INVESTIGATIONS FOR PREFERRED ROUTES

6.1 Methods

The field investigations were concentrated along the east escarpment (Alternative Layout C) to obtain data for selecting and evaluating optimum tunnel routes.

The diversion tunnel investigation was divided into two phases. The first phase was the field program, consisting of field reconnaissance, diamond drilling, permeability testing, geophysical mapping, and on-site laboratory testing of core samples. The second phase involved the analyses of both previously and newly acquired data to develop a geological and geotechnical framework within which to evaluate the various diversion tunnel alignments. Geotechnical units were established as the result of this work (see Section 6.3).

Geological mapping and the geophysical surveys were carried out during May, 1982. Some further limited geophysical traverses were done during June. Drilling commenced at the beginning of June, 1982, and was completed by mid-July, 1982.

6.2 Results of Field Work

6.2.1 Field Mapping

Limited additional field mapping was undertaken for this investigation. Reliance was placed upon mapping accomplished in previous years by Golder Associates and B.C. Hydro. Occasional field visits to key outcrops were undertaken to further enhance the understanding of the stratigraphy and structure of the eastern escarpment.

The available data from rock exposures is very limited as there is less than 5 per cent total outcrop exposed on the eastern escarpment. However, once the drilling results had been analyzed and the data calculated, it was found necessary to remap selected outcrops in relation to the subsurface data.

6.2.2 Diamond Core Drilling

The 1982 drilling contract was carried out by Coates Enterprises Ltd. A total of 917 m of HQ triple tube diamond core drilling was completed in a total of 5 drillholes. All but one hole (DDH82-922) were angle holes. The locations of these holes, as well as of all holes drilled to date at Hat Creek, are shown on Figure 1. In addition to obtaining core samples from the drilling, through-the-bit packer tests, directional and geophysical surveys were conducted during the drilling operations as well. The downhole geophysical surveys were performed by Roke Oil Enterprises Ltd. of Calgary, Alberta, and included neutron, density, and gamma-ray logs. These downhole geophysical logs were most useful in the correlation of the andesite sequence and support the hypothesis of fault displaced repetition of the sequence (Section 3.3).

The drillholes were oriented so as to intersect the stratigraphic units as close to perpendicular as practicable; allowance had to be made for the broken nature of the ground in selecting the angle for drilling. The core obtained was logged immediately after each drill run and while still in the split inner tube. It has been our experience in previous drilling programs at Hat Creek that the drill core soon deteriorates on exposure, thus leading to possible erroneous lithologic descriptions if not described as soon as possible. Geological logs of all holes drilled during the 1982 program are contained in Addendum 1.

6.2.3 Geophysical Mapping

Magnetic and electrical methods of geophysical mapping were carried out along survey lines on the eastern escarpment to assist in the geological mapping of the potential corridor. It was noted early in the field program that the magnetic technique was more successful than the electrical resistivity technique, and that the volcanic sequences had a significantly stronger magnetic response than the sedimentary sequences. Hence, magnetic mapping predominated. The magnetic and resistivity profiles and a magnetic feature plan are presented in Addendum 3.

6.2.4 Materials Testing

The majority of the material evaluation was conducted by Golder Associates at the site laboratory, with somewhat limited testing carried out in Vancouver. On-site laboratory testing consisted of uniaxial compression testing, point load testing, Atterberg limits, and moisture contents. Slake durability tests were carried out in Vancouver. All testing was conducted according to ASTM standards and recommended procedures. The results of the laboratory testing are included in the drilling logs (Addendum 1), and are summarized for each geotechnical zone in Table 2.

The results of the point load testing were found to be extremely variable and erratic. This is attributed to the fact that the specimens frequently contained significant structure and inhomogeneities which influenced the failure. No sensible correlation of results with uniaxial compressive strength could be established and point load testing was terminated after completion of drillhole DDH82-921.

Uniaxial compressive strength testing was undertaken for purposes of material classification for the geotechnical units G3 (the altered andesite) and G4 (the lower volcaniclastic), for which core was available. For both units, the strengths were found to be highly variable and ranged between wide extremes. The reason for this is the pervasive nature of bedding, jointing and fracturing on the scale of the specimens. Thus, in many cases, where failure of the specimen occurred along such discontinuities, low strengths representative of rock mass behaviour were obtained. In the minority of cases, failure of intact rock material was evident and much higher strengths were recorded.

For Geotechnical Unit G3, unconfined compressive strengths for the laboratory specimens ranged from near zero to about 80 MPa, with an average of 23 MPa. The corresponding range and mean values for the rock mass were zero to 35 MPa, and an average of 8 MPa.

Unit G4 was found to be noticeably weaker, with rock mass strength in the range of zero to 11 MPa, and a mean value of about 2 MPa. The low values are attributed to the existence of alteration and weathering.

Because of the dominant and extensive influence of structure on the strength and behaviour of rock materials in the present instance, the stability of the tunnel and the mode of failure is essentially related to the rock mass strength. Thus, this strength parameter, rather than intact strength, has been compared to the magnitude of regional stresses to determine the potential for stress related squeezing problems during excavation.

6.2.5 Petrographic Analysis

The results of the petrographic analysis of selected samples proved most helpful. All volcanic specimens submitted for analysis proved to be andesitic in composition, and most had undergone some degree of hydrothermal alteration. What had been termed rhyodacite, dacite, and rhyolite in previous investigations along the eastern escarpment, are termed andesite or andesite breccias for this report. Even dark grey vesicular rocks were found to be andesitic in chemistry. Alteration products are typically amorphous smectite clays and opaline minerals.

The petrographic reports are included as Addendum 2.

6.2.6 Slake Durability Tests

The slake durability test is an index test to evaluate the weathering resistance of shales, claystones, siltstones, and other clay bearing rocks. The test procedure and its development are described in Franklin and Chandra (1971). Slake durability tests were conducted on eleven samples; five from geotechnical unit G3, and six from G4. The test measures the percentage breakdown of the material after rotating the samples under saturated condition for a specified duration, and then subsequently drying them.

TABLE 2

SUMMARY OF ROCK TESTING RESULTS

GEOTECHNICAL UNIT	UNIAXIAL STRENGTH		MOISTURE CONTENT (%)	ATTERBERG LIMITS		SLAKE DURABILITY ID (%)
	INTACT	ROCK MASS		LL	PL	
G2 Upper Rock	5.6 - 13.7 Average 7.4*		15.5 - 37.0 Average 27.0	36 - 137 Average 78	16 - 80 Average 42.7	
G3 and	0.4 - >78.1 Average 23.0	0.0 - 35.1 Average 8.5	11.3 - 23.2 Average 16.7			17.5 - 94.4
G4 Lower Rock	0.4 - >70.9 Average 15.5**	0.0 - 11.1 Average 2.0	7.8 - 39.5 Average 17.9	9.3 - 84.7 Average 50.9	20.6 - 40.3 Average 31.1	0.0 - 93.4

* Source of data from Drillhole 816

** This value includes a small number of unusually strong lithic breccias. Deletion of these values yields an average intact uniaxial strength of 5.4 MPa.

The andesites of G3 generally exhibited a high slake durability, ranging between 74.6 and 94, with an average value of 73.9 per cent (second-cycle slaking). A value of 17.5 per cent was recorded for an andesite breccia, while another andesite breccia resulted in a value of 94.4 per cent. The susceptibility of the breccias to slaking is largely dependent on the composition of the matrix materials, i.e. whether they are chloritic, bentonitic, or lithic. Breccias with a high bentonite content in the matrix will slake more readily than those with a lithic matrix.

The slake durability of geotechnical unit G4 is somewhat lower than for G3. Values ranged from 0 to 93.4 per cent, with an average of 61.6 per cent. Of the samples tested, claystone, siltstone, and silty sandstone yielded fairly high values (in the 60 to 90 per cent range), while the lower values were associated with coarse sandstones and tuffaceous sandstones (0 and 50 per cent, respectively). It is apparent from the limited sampling that the finer grained silty sandstone and siltstones/claystones are more durable than the coarse grained sandstones.

6.2.7 Hydraulic Conductivity Testing

Hydraulic conductivity testing was carried out in drillholes DDH82-923 and 924 as drilling proceeded. The method of testing employed a double packer system to test the length of open hole below the drill rods. Where a zone was tested, the rods were pulled back and the double packer inserted into the drill rods and lowered to the desired depth. The packers acted to seal the borehole and prevent the water from escaping through the drill rods. Following the installation of the packer system, the water level was allowed to stabilize, and then a falling head permeability test run. The test involved pouring a slug of water down the rods and monitoring the decay of excess head. The tests were analyzed using the Hvorslev (1951) method. Table 3 presents details of the testing.

Analysis of the data indicates the hydraulic conductivity of the volcanic/volcaniclastic materials to be variable, ranging between 4 x 10^{-4} m/sec and 3 x 10^{-8} m/sec. This variation in hydraulic conductivity values is probably a reflection of the degree of fracturing and interconnection of such fractures within the rock mass. During the drilling of drillhole DDH82-923, circulation of drilling fluid was lost at a depth of approximately 236.5 m. (The water level in the open hole following loss of fluid was noted at a depth of approximately 46 m, elevation 1064 m.) The material encountered at a depth of 236 m was identified as a very soft, highly weathered clay, and may have been associated with the fracturing of the lithic breccia.

The hydraulic conductivity of the siltstone and claystone tested from the Cache Creek Group was between 2.7 x 10^{-7} and 3.7 x 10^{-8} m/sec. Further hydrogeological work carried out this year (1982 Geotechnical Update) has identified the hydraulic conductivity of similar sediments, for the rock in the Medicine Creek Formation, to range between 1.0 x 10^{-6} to 8.8 x 10^{-13} m/sec, with a median value of 4 x 10^{-11} m/sec.

Hydraulic conductivity of the surficial materials in the area of the northern pit rim is calculated to range between 1 x 10^{-6} to 6 x 10^{-9} m/sec, based on previous investigations (1977/78 Geotechnical Report) and the 1982 Geotechnical Update. The variable hydraulic conductivity of this material reflects the varying proportions of clay, silt, sand and gravel found in these stratified deposits.

TABLE 3

SUMMARY OF PACKER PERMEABILITY TESTING

HOLE NUMBER	INTERVAL TESTED (m)	BASIC TIME LAG (sec)	HYDRAULIC CONDUCTIVITY (m/sec)	LITHOLOGY
DDH82-923	47.2 - 49.7	35.5	4.3×10^{-4}	Andesite
	90.3 - 93.3	180	5.8×10^{-6}	Andesite
	129.5 - 132.5	7,500	1.4×10^{-7}	Andesite
	166.1 - 167.6	375	4.6×10^{-6}	Andesite, Breccia
	185.5 - 189.5	30,000	3.5×10^{-8}	Tuffaceous Sandstone
	244.2 - 250.0	1,300	4.8×10^{-7}	Lithic Breccia
DDH82-924	138.7 - 141.1	4,500	2.7×10^{-7}	Claystone/ Siltstone
	207.6 - 212.4	19,800	3.7×10^{-8}	Claystone/ Siltstone

6.3 Geotechnical Evaluation

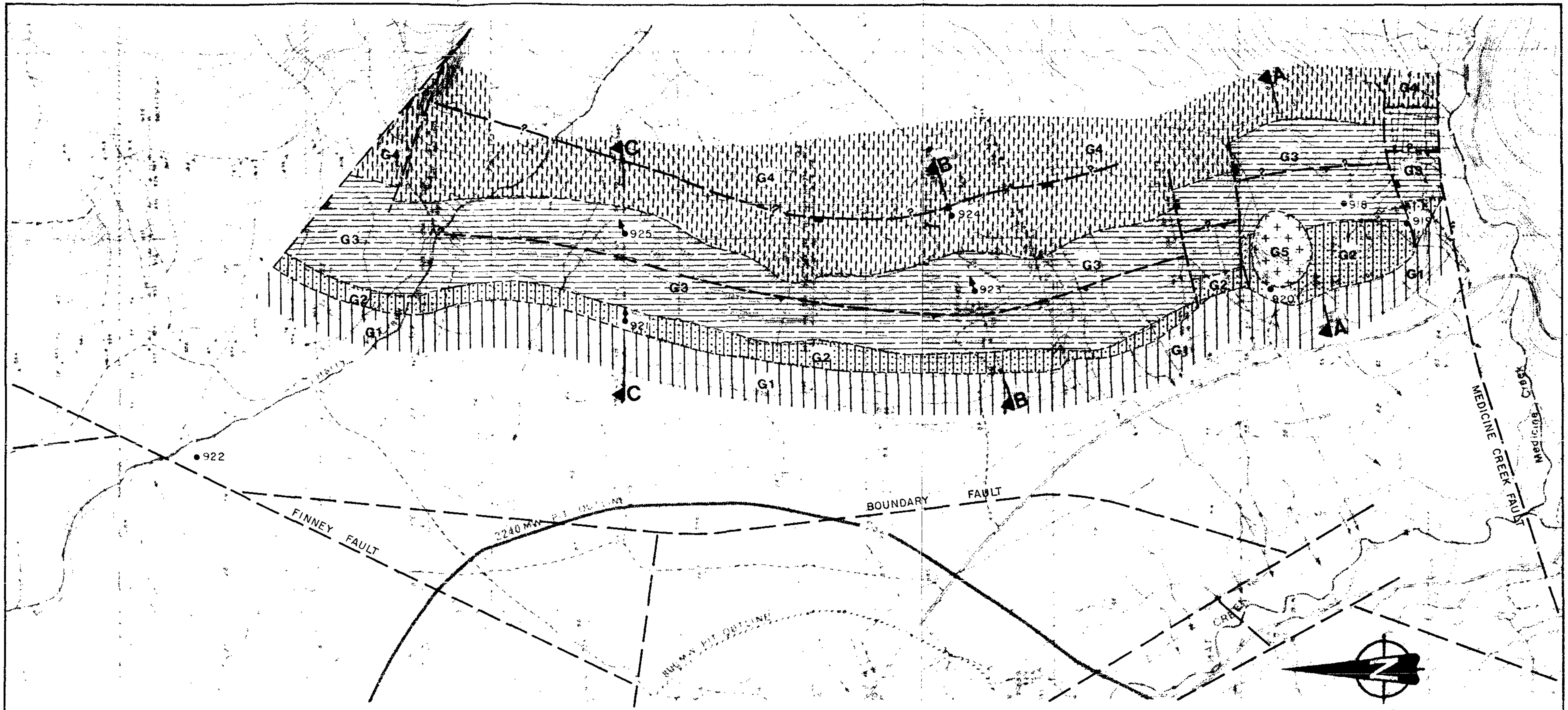
6.3.1 Methods of Evaluation

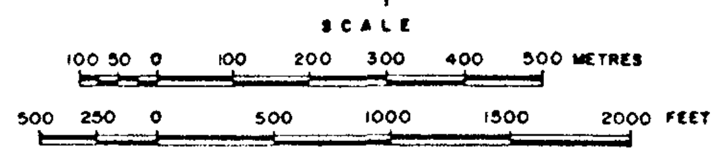
The general methodology adopted in the evaluation of the potential tunnelling conditions through the east escarpment at Hat Creek consisted of first establishing the geological units which could be represented by a relatively uniform set of geotechnical properties, and which were also distinctly different from the properties of other units. Conditions within several groups of rock types could then be represented by a single set of parameters, and hence defined as Geotechnical Units.

Five separate units have thus been chosen for characterization in terms of tunnelling conditions:

- G1 - fluvioglacial deposits (surficials)
- G2 - upper volcaniclastic beds (forming the west side of the escarpment)
- G3 - altered andesite rocks
- G4 - lower volcaniclastic beds (present to the east of the andesite rocks)
- G5 - slide debris.

A map of the distribution of the geotechnical zones was drawn up for the approximate tunnel invert elevation (955 m O.D.) and is shown as Figure 8. On the basis of this, and within the area of Layout C, a set of alternative tunnel routes was selected for detailed evaluation. The location of the routes selected for evaluation is shown on Figure 9. The lengths and sequences of each of the geotechnical units which are intersected by the respective tunnel routes were determined. In this way, the estimated ground conditions and stability behaviour could be assessed and a construction method proposed. This method takes into account the relative lengths of tunnel in each geotechnical unit, the sequence of excavations, and the potential for significant variations from the anticipated ground conditions.





LEGEND

- – – – – Geotechnical unit boundary
- –?– –?– Geotechnical unit boundary (conjectural)
- ——— Fault (definite)
- — — — Fault (assumed)
- –?–?– Fault (conjectural)
- Normal displacement showing downthrown side.
- Reverse - teeth on upper plate
- ▲——▲ Geological section (See Figure 4).
- 925 Drillhole - 1981/82 tunnel diversion program, showing orientation of hole.

GEOTECHNICAL UNITS

G1	Glacio-fluvial deposits
G2	Upper volcaniclastic beds
G3	Altered andesite
G4	Lower volcaniclastic beds and Cache Creek Formation
G5	Landslide debris

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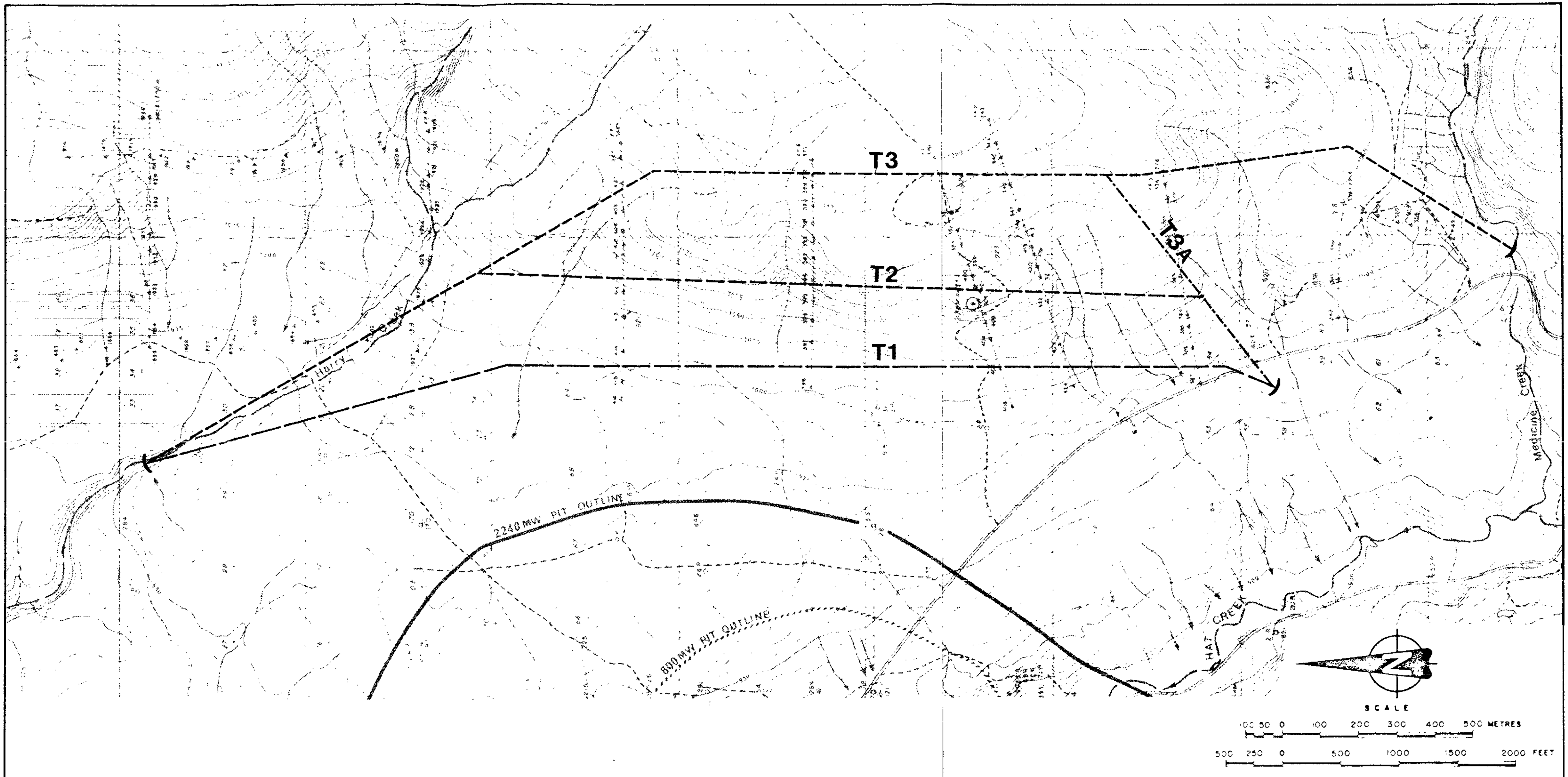
B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

DISTRIBUTION OF GEOTECHNICAL UNITS AT TUNNEL INVERT ELEVATION (955m. O.D.)

Drawn	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure 8



Golder Associates/Sigma Engineering Ltd.

B.C. Hydro & Power Authority

HAT CREEK DIVERSION STUDY

LOCATION OF TUNNEL ROUTES INVESTIGATED

Drawn	Checked	Reviewed
Date JAN. 1983	Scale 1:10 000	Figure 9

The anticipated ground conditions and support requirements form the basis for the determination of construction methods described in Section 7.0. Estimates of advance rates and tunnelling costs, presented in Section 8.0, are deduced from a careful synthesis of the construction procedure.

The evaluation of stability conditions and support requirements for Units G2, G3, and G4 has been based on the use of the NGI empirical system for estimating tunnel support requirements (Hoek and Bray, 1980). This evaluation is not applicable to the surficial deposits of unit G1. This system is based on a compilation of actual performances from many case histories of tunnel construction and addresses both structurally and stress controlled types of failures. The system determines a stability/support index by estimating the values of three quotients:

1) Rock Quality Designation/Number of Joint Sets (RQD/Jn) - an indicator of the effective joint block size;

2) Joint Roughness/Joint Alteration (Jr/Ja) - an indicator of the frictional strength of the discontinuities, and hence the rock mass;

3) Joint Water/Stress Factor (Jw/SRF) - a measure of the active destabilizing stresses.

The tunnelling quality index, Q, is then calculated from the equation:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

The value of Q is then entered in an empirically derived chart to determine the level and type of support required, depending on the size of the tunnel and its application. Calculations for the derivation of Q for each of the other geotechnical units are presented in subsequent sections. A classification of the parameters used in the calculation of Q is included in Addendum 4.

6.3.2 Geotechnical Unit G1

This unit consists of widespread glacial deposits of till and outwash typically composed of sand, gravel, and some boulders in a silty clayey matrix; horizons of sand or silt are likely to be present too. The unit is weak, friable and cohesive, with an unconfined strength in the order of 0.5 MPa. It is generally overconsolidated and dry, but exhibits some potential for swelling and strength reduction on exposure to water. The water table is likely to be below the proposed tunnel elevation for the routes investigated.

Although overburden depths vary from zero to about 50 m along the route, temporary support in the form of steel sets would be required immediately behind the face because of the low strength. Protection of the unit against hydraulic erosion during operation of the tunnel would also be necessary using either a cast-in-place or precast concrete lining and contact grouting. It might be advantageous to integrate temporary and permanent support if this is compatible with the excavation method. If conventional excavation by hand mining were adopted, steel sets would be required for support. The design of the permanent lining would need to consider nominal external swelling pressures.

6.3.3 Geotechnical Unit G2

This unit is a sequence of weathered, massive-thinly bedded weak sandstones, siltstones, and conglomerates. The unconfined compressive strength is in the order of 2 to 5 MPa. There is evidence of the presence of expansive clay, possibly resulting from alteration, which have low strengths and potentially high swelling pressures on exposure to water. The unit is sampled in 155 m of drill core from three locations, diamond drillholes DDH74-36, DDH77-839, and DDHS82-921.

The tunnelling quality index is calculated as follows:

Parameter	Description	Value
RQD	Variable, averaging 70	70
Jn	One main and one random joint set	3
Jr	Discontinuous joints with some slickensiding	2
Ja	Some silty, clayey joint alterations	3
Jw	Very low permeability with about 40 m potential head	0.66
SRF	Non-existent to mild squeezing potential	2.5
	Q(G2) = 4.1 (fair quality)	

An unsupported tunnel driven through geotechnical unit G2 could be considered as marginally stable in the short term; however, shotcrete and occasional spot bolting would be required for temporary support. Instability would likely be local and in the form of squeezing-type excessive deformations, especially where ground water is encountered. An additional shotcrete lining thickness, and a concrete paved invert, would be required to protect against hydraulic erosion.

6.3.4 Geotechnical Unit G3

This unit consists of slightly to completely altered andesite, intensely fractured and locally brecciated, highly variable in quality possibly with some 25 per cent altered to expansive clays. Fractures and joints are commonly chlorite or opal coated. Unconfined compressive strengths are in the order of 8 MPa for the rock mass. Stability and tunnelling conditions are very much dependent upon the intensity and quality of joints and fractures. This unit has been extensively sampled from outcrops from 810 m of drillholes from 7 locations.

The tunnelling quality index is calculated as follows:

Parameter	Description	Value
RQD	Predominantly 0 - 20 range with some more competent bands	10
Jn	Four joint sets	12
Jr	Joints are smooth and planar	1.0
Ja	Only minor joint alteration	1.0
Jw	Minimal pressure head and low permeability	0.66
SRF	Mild squeezing expected	5
	Q(G3) = 0.03 (extremely poor)	

Extensive support would thus be required for a tunnel driven through geotechnical unit G3 because of the low strength of the heavily fractured rock mass, compared to the overburden pressures. Support in the form of 7 to 15 cm of shotcrete on the roof and sides would be required, with limited sections requiring steel sets. Routine excavation using an excavator shield is not considered possible, or at best marginal.

6.3.5 Geotechnical Unit G4

This unit is defined as the lower volcaniclastic series. It is composed of weakly to moderately strong, medium-thick bedded, slightly weathered tuffaceous sandstones, siltstones, and claystones. Occasional zones of bentonite with moderately altered lithic breccias are also present. The unit also includes non-tuffaceous sandstone, siltstone, and claystone supposedly of the Cache Creek Group, which were encountered east of the fault intersected in DDH82-924. Unconfined strength is in the order of 2 MPa. The presence of clay minerals in the breccia results in low strengths and a susceptibility to swelling. The unit is sampled from a total of 420 m of core from drillholes DDH78-836 and 838, DDH82-923 and 924.

The tunnelling quality index is calculated as follows:

Parameter	Description	Value
RQD	Ranges from 45 to 100, averaging 70	70
Jn	Three regular plus one random joint set	6
Jr	Smooth undulating joints	2
Ja	Some joints slightly altered, chloritized	2
Jw	Potentially 100 to 130 m ground water head, with very low permeabilities	0.66
SRF	Overburden stress equal to or greater than mass strength, some squeezing probably	2.5
	Q(G4) = 1.2 (poor)	

Tunnelling conditions in geotechnical unit G4 are expected to be better than for Units G1 and G3, but not as good as for the G2 unit. Nominal temporary support in the form of 2 to 5 cm of shotcrete would be required. Steel sets might be required in very short sections of difficult ground. The marginal stability, low material strength, and susceptibility to moisture would require that some form of hydraulic surface (concrete lining) be adopted.

6.3.6 Geotechnical Unit G5

Toward the south (upstream) end of the east escarpment, a rock slide is present which, it is considered, developed in post-glacial times. It affects rocks of geotechnical units G2 and G3. Drillhole DDH81-920 intersected the edge of the slide, but its greatest depth is not known; assumptions have been made in the preparation of Section AA on Figure 4. The slide debris is highly variable and composed of local rock types set in a clayey or sandy matrix. Because of its importance to tunnelling, this zone has been considered as a separate geotechnical unit.

6.4 Selection of Tunnel Routes

6.4.1 Factors Influencing Route Selection

The complexity of geological conditions within the escarpment on the east side of the pit (as discussed in Sections 3.0 and 6.0), and their controlling influence on the method and cost of construction, have necessitated the selection and comparison of a number of alternative tunnel routes. This section considers the interpreted geological/geotechnical conditions, together with project-related constraints on the positioning of the tunnel section of the diversion and discusses the selection of the routes which were identified for the subsequent detailed economic comparisons.

The main factors impacting the detailed selection of the route are:

- o geological and hydrogeological conditions
- o constructional aspects (refer to Section 4.0)
- o portal siting
- o hydraulic requirements
- o potential instability resulting from long term degradation of pit slopes.

Geological conditions profoundly affect the choice of routes. Of primary significance is the general stratigraphy and structure of the area, and the scope for aligning any route within one geological unit or a favourable combination of units. The geotechnical properties of each unit for tunnelling purposes and the expected behaviour of each unit of material on exposure, determine if the rock or soil is a viable host medium for the tunnel. Finally, the extent and reliability of the geological data must be considered; some routes could easily be written off on the basis of inadequate or unrepresentative data.

Certain routes may traverse a number of geotechnical units of widely different properties, with consequent limitations on the economic application of tunnelling techniques. In some cases, longer routes which simplify the mix of geological conditions may be preferable if the tunnelling technique can be standardized. The choice of excavation method is primarily a function of the geology.

Portal construction is a major consideration of most tunnels. In the present case, because of the flat slopes at both the upstream and downstream ends of the tunnel and the thickness of overburden present, particular emphasis needs to be given to these areas. The location of the portals is thus a major constraint on the selection of the tunnel routes.

Certain latitude is available in the vertical grade of the tunnel as controlled by flow requirements. The anticipated slope of the Hat Creek tunnel (1 to 2 per cent) is well within the acceptable range from the point of view of constructability. Minor adjustments in grade can also be tolerated to permit suitable portal locations to be selected.

The final slopes of both the 800 MW and 2240 MW pits will degrade on abandonment of the mine and become flatter with time. In this way, the margin of the pit will recede toward the east escarpment. At the present time, the extent to which this will occur is uncertain, but it must be treated as a constraint to the selection of tunnel routes. Any tunnel driven through geotechnical units G1 and G2 close to the pit perimeter could potentially be at risk. Tunnels driven through units G3 and G4 would be protected from pit encroachment.

The four routes selected for study are shown in Figure 9. The reasoning for the selection of these particular routes is as follows.

6.4.2 Route T1

This route is designed to be as short as possible, with a minimum number of alignment changes, and also to remain entirely in the overburden or the upper volcaniclastic sequence (Unit G2). It represents the route nearest the east rim of the pit. An effort has been made to minimize the length of tunnel in the surficials (Unit G1) because of the suspected inferior excavation conditions within that unit. The intake portal lies below the west face of the escarpment within the Hat Creek valley; the "dog-leg" has been introduced to ease the portal entry.

6.4.3 Route T2

This route is designed to maximize the proportion of tunnel in the altered andesite (Unit G3). This objective is compromised by the need to orient the intake section of the tunnel to ensure adequate cover, and to avoid the faulted and slide areas. At the outlet end of the tunnel, an alignment south of Harry Creek would avoid the faulted area detected further north by the geophysical survey. The andesite unit is considered to be sufficiently large in areal extent to permit a reasonably direct alignment without the potential problem of extensive mixed-face tunnelling conditions.

6.4.4 Route T3

Because of the general similarity in properties between the upper and lower volcaniclastic series (Units G2 and G4, respectively), excavation and support methods similar to those for Route T1 are envisaged. The optimum portal lies in the Medicine Creek Valley, and the initial section of the tunnel is aligned to the northeast to avoid the slide zone. The route attempts to locate the greatest proportion of its length in the lower volcaniclastics. Except for the Harry Creek section, the tunnel is about 950 m distant from the 35-year pit rim location. About 20 per cent of the tunnel is located in the altered andesite, and four basic changes in the excavation method would be required along the tunnel length.

6.4.5 Route T3A

This route is essentially a variation of Route T3, whereby the location of the intake portal in the Hat Creek Valley permits the faulted area and slide zone at the southern part of the escarpment to be avoided. A slight advantage is also obtained in that the total length of tunnel in Unit G3, required to be excavated through the shield, is less.

The subdivisions of each route into lengths of tunnel in each geotechnical unit are shown in Figure 10. The lengths of tunnel in surficial deposits are estimated from bedrock surface contour maps. Because of the gradually sloping topography and the depth of overburden in the portal areas, an allowance of 100 m has been made in the total tunnel length for portal construction in each of the variants.

PROJECT NO. 822-1523 B DRAWN G.A. REVIEWED GM DATE Sept. '82

ROUTE T1		
GEOTECHNICAL UNIT	LENGTH (metre)	EXCAVATION METHOD
G1	60	Outlet portal
	100	Drill and blast with steel sets (C3)
	1620	Excavator – shield (C1)
G2	50	Drill and blast with shotcrete (C5)
	700	Excavator shield (C1)
G1	450	Excavator shield (C1)
	50	Drill and blast with steel sets (C3)
	40	Inlet portal
Total length 3070 m.		

ROUTE T2		
GEOTECHNICAL UNIT	LENGTH (metre)	EXCAVATION METHOD
G1	60	Outlet portal
	50	Drill and blast with steel sets (C2)
	680	Hand mining with steel sets (C6)
G2	140	Drill and blast with shotcrete (C4)
G3	1910	Drill and blast with shotcrete (C4)
	100	Drill and blast with steel sets (C2)
G1	270	Hand mine (C6)
	40	Inlet portal
Total length 3250 m.		

ROUTE T3		
GEOTECHNICAL UNIT	LENGTH (metre)	EXCAVATION METHOD
G1	60	Outlet portal
	50	Drill and blast with steel sets (C3)
	680	Excavator shield (C1)
G2	140	Excavator shield (C1)
G3	150	Drill and blast with shotcrete (C5)
	300	Excavator shield (C1)
G4	100	Drill and blast with shotcrete (C5)
	50	Drill and blast with steel sets (C3)
	1570	Excavator shield (C1)
G3	55	Drill and blast with shotcrete (C5)
	400	Excavator shield (C1)
G1	290	Excavator shield (C1)
	50	Drill and blast with steel sets (C3)
	40	Inlet portal
Total length 3935 m.		

ROUTE T3A		
GEOTECHNICAL UNIT	LENGTH (metre)	EXCAVATION METHOD
G1	60	Outlet portal
	50	Drill and blast with steel sets (C3)
	680	Excavator shield (C1)
G2	140	Excavator shield (C1)
G3	150	Drill and blast with shotcrete (C5)
	300	Excavator shield (C1)
	100	Drill and blast with shotcrete (C5)
	50	Drill and blast with steel sets (C3)
G4	1400	Excavator shield (C1)
G3	200	Excavator shield (C1)
G2	60	Excavator shield (C1)
G1	215	Excavator shield (C1)
	50	Drill and blast with steel sets (C3)
	40	Inlet portal
Total length 3495 m.		

- Designations C1 to C6 refer to recommended excavation methods shown on Table 4.

Golder Associates

ALTERNATIVE ROUTES GEOTECHNICAL SUBDIVISION

Figure 10

7.0 TUNNEL EXCAVATION AND SUPPORT METHODS

7.1 Potential Methods of Excavation In Geotechnical Units

The various methods of tunnel excavation have been covered generally in Section 4.0. The criteria for selecting the excavation method for the range of conditions anticipated at Hat Creek are covered in Section 4.3. It was concluded in that analysis that the appropriate excavation method would be dictated almost entirely by the geological conditions.

From the evaluation of geotechnical behaviour anticipated during tunnelling, it is also concluded that either a circular or horseshoe-shaped tunnel would be admissable since, for long term stability, a concrete or shotcrete lining would be required, and external ground pressures are not excessive. Therefore, for each of the tunnel routes identified (T1, T2, T3 and T3A), consideration must be given to the geology as reflected by the geotechnical units to permit the optimum method of tunnelling to be selected. The following paragraphs examine two aspects: firstly, the possible methods which could be utilized in excavating the geotechnical units described in Sections 6.3.2 to 6.3.6; and, secondly, the distribution of those units along the tunnel routes in order to select an appropriate excavation method. These are summarized on Table 4.

7.1.1 Geotechnical Unit G1

The glacio-fluvial deposits and till of geotechnical unit G1 could be excavated by hand, or by part-face or full-face soft ground shield tunnelling machine such as the excavator with shield protection shown in Figure 11. A concrete lining (segmental in the case of the machine, cast-in-place for hand mining) would be required immediately behind the face for support. With an excavator-shield type of machine, the precast lining could be installed under the protection of the shield tail and this lining could serve as both temporary and permanent support. The lining would also be used as the thrust reaction for the machine advance. The use of a machine could be justified only for those tunnel routes where the same machine could be utilized for the majority of the excavation of the tunnel. Some local drilling and blasting might be required at the portals and where large boulders are encountered which could necessitate mucking out through the shield where that is used.

7.1.2 Geotechnical Unit G2

The siltstones, sandstones and conglomerates of geotechnical unit G2 (upper volcaniclastics) could be excavated by a variety of means: an excavator shield, drill-and-blast, roadheader, or full-face boring machine. The selection of method would be dependent on the length of that unit which would be intersected because, generally, only short sections of G2 would be encountered over a complete tunnel length. Thus, unit G2 would be excavated by the dominant method selected for any particular tunnel route. It could be supported by segmental lining, by shotcrete, or by steel sets.

7.1.3 Geotechnical Unit G3

Altered and brecciated andesites comprise the bulk of geotechnical unit G3. The tunnelling quality index Q gives an 'extremely poor' rating, largely reflecting the number of joint sets and very low RQD. In addition, a north-south trending fault affects this unit, repeating the andesitic sequence; it is likely that this would provide difficult tunnelling conditions, exacerbated by the fact that it is aligned parallel to the tunnel. Full support would generally be required for a tunnel driven through this sequence. This unit is considered to be unsuitable for a full-face tunnel boring machine because of the broken nature of the ground and the high degree of variability.

Drill-and-blast techniques would be appropriate for unit G3. However, for tunnels intersecting only limited lengths of this unit and if a machine were selected for other reasons, the machine could be used for construction expediency for this unit also.

The most appropriate lining would be shotcrete, with or without bolts, but a cast-in-place lining or segmental lining might also be used where such an approach provided economies within the context of the overall tunnelling technique selected. This unit might require some localized drill-and-blast excavation through the shield face if that technique were selected for overall use.

7.1.4 Geotechnical Unit G4

Unit G4 comprises tuffaceous sedimentary rocks of the lower volcaniclastic unit and non-tuffaceous sedimentary rocks supposedly of the Cache Creek Group. This unit is considered to have superior tunnelling properties to the other units except G2. It appears to be reasonably uniform in the strike direction (data from the geophysical survey, Addendum 3), but is weak; it could be excavated by any form of machine or by drill-and-blast. However, because it is incompletely known at the present time, it is recommended that the method that is selected should have a high degree of flexibility. This would preclude a full-face boring machine, and favour a shield-excavator utilizing segmental linings. Some drill-and-blast work through the shield face might be required, and shotcrete or cast-in-place lining used over those localized sections.

7.1.5 Geotechnical Unit G5

This unit, comprising slide debris, has been avoided in all the tunnel routes selected, and consequently does not need to be considered in selecting a tunnel excavation method.

7.2 Selection of Excavation Method for Tunnel Routes

As will be seen from Section 7.1, a range of options of excavation methods exist for most geotechnical units, although there are some major preferences. All tunnel routes would intersect several geotechnical units (see Figure 10); Routes T3 and T3A intersect all units; Route T2 does not intersect Unit G4 and Route T1 does not intersect Units G3 or G4. Clearly the choice of basic excavation method should be appropriate to the geotechnical unit which dominates a specific tunnel route. In addition, the choice of a tunnelling machine method requires assurance that it could be utilized over a sufficient length of tunnel to justify the relatively high capital cost of the machine.

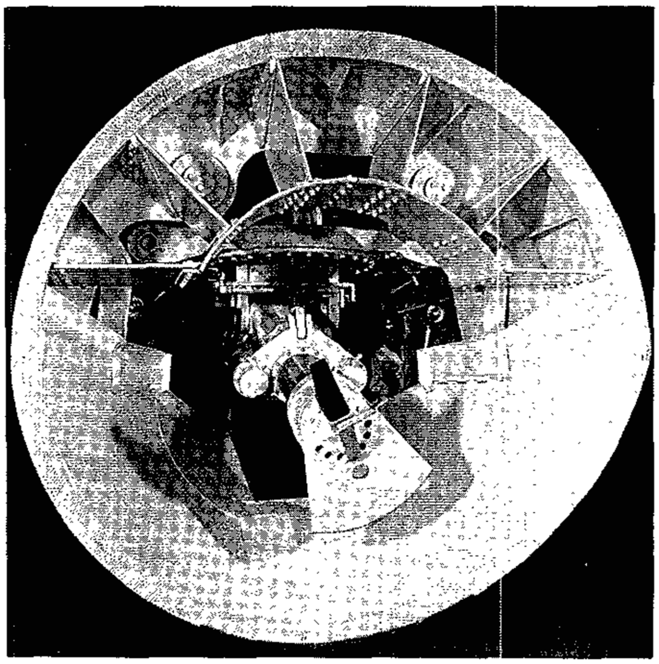
For Route T1, the greatest length would be in Unit G1 and an excavator shield is recommended. It could also cope with Unit G2. Some local drill-and-blasting would also be needed.

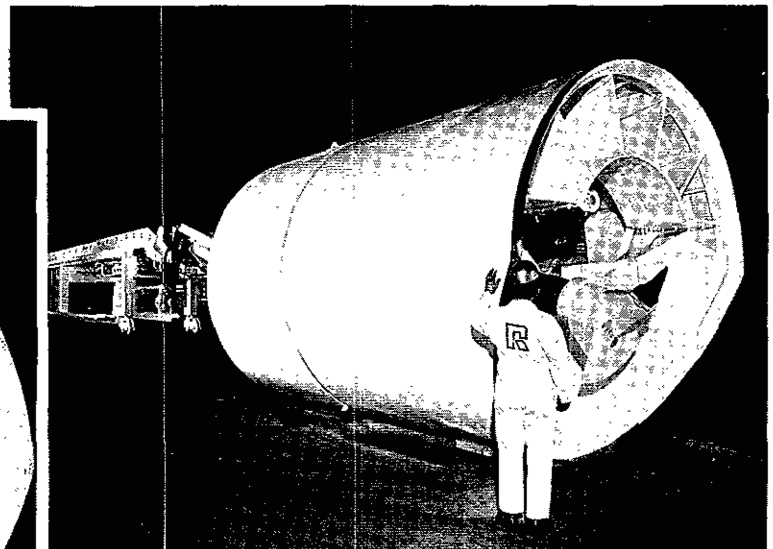
For Route T2, the greatest length would be in Unit G3 and drill-and-blast with a shotcrete lining is recommended, except through Unit G1 where hand mining would be needed. Some hand mining and cast-in-place lining would also be necessary in association with the drilled and blasted length.

For Route T3, the greatest length would be in Unit G4 and an excavator shield is recommended for the complete length, which includes Units G1, G2 and G3. An allowance has been made for drill-and-blast through the shield where necessitated by difficult ground.

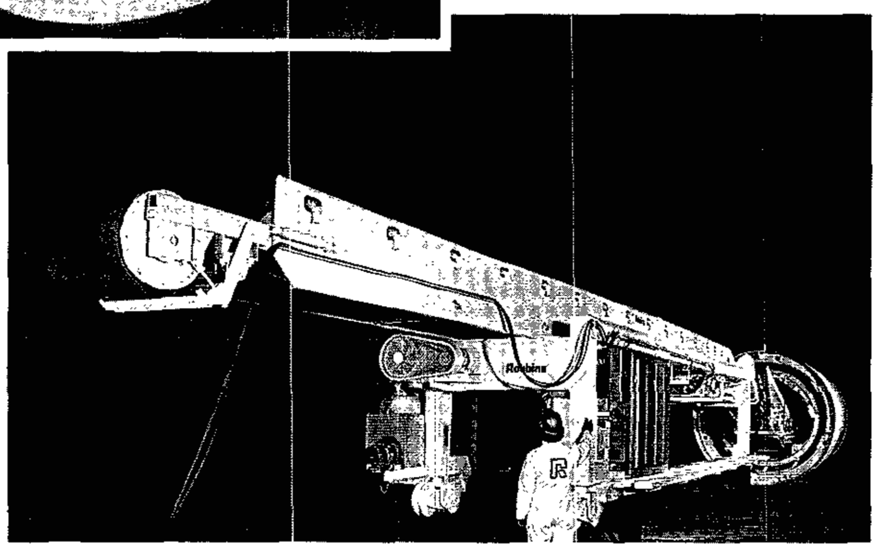
For Route T3A, the greatest length would also be in Unit G4 and the same methods as for Route T3 are appropriate.

model 127S-164 Robbins Tunnel Machine





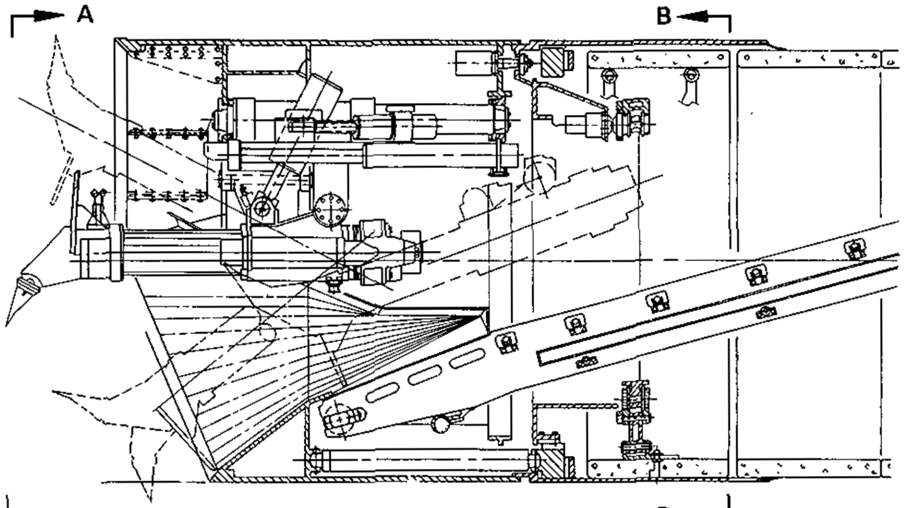
EXCAVATOR SHIELD
DIAMETER 12 ft 1 in. (3,67 m)

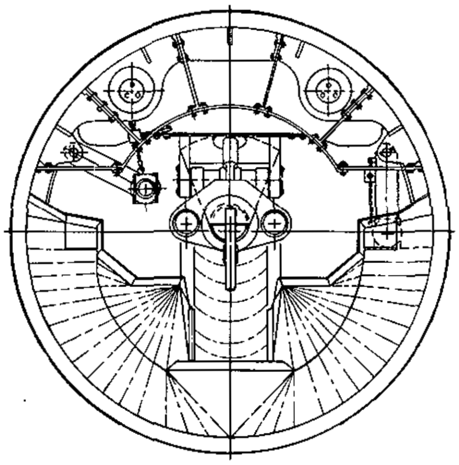


Project Information:

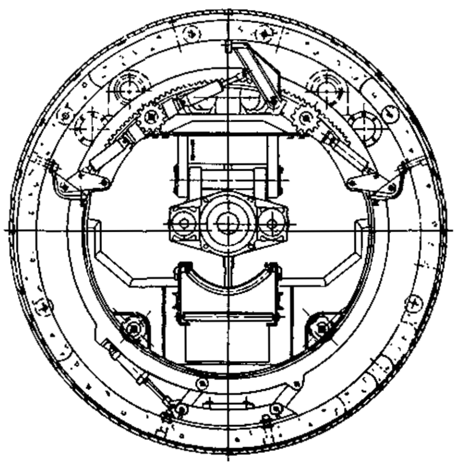
LOCATION	Bern Sewer Tunnel / Bern, Switzerland
MATERIAL	Gravel with Sand & Clay
SUPPORT	Precast Concrete Segments
TUNNEL LENGTH	5,080 ft (1550 m)

The Robbins Company Box C8207, 7615 S. 212th St. Kent, WA 98031, USA Phone (206) 872-0500 Cable ROBBORE KENW Telex 32-8711





SECTION A-A



SECTION B-B

Specifications:

TYPE	Excavator Shield
DIAMETER	12 ft 1 in. (3,67 m)
HORSEPOWER	450
THRUST	3,420,000 lbs (1.500.000 kilos)
WEIGHT	60 tons (55 metric tons)
CUTTERS	Ripper/Scraper

Features:

1. Heavy structural shield with thick cutting edge for high strength.
2. Tail shield is hydraulically articulated for improved steering.
3. Roof-mounted boom-type hydraulic excavator with powerful ripper excavates full face and beyond shield cutting edge.
4. Joy stick excavator controls and quadrant control of thrust jacks simplify excavating and steering operations.
5. Fast retract on thrust system speeds lining erection.
6. Breasting hood in upper half of shield and a deep muck apron in bottom half provide face control.
7. Heavy thrust ring equalizes load on tunnel lining and protects thrust cylinders.
8. A thrust cylinder skewing device compensates for shield roll.
9. Large belt conveyor handles high volume of muck without overloading.
10. Retractable machine conveyor for cleaning.
11. Single pick-up rotary segment erector for placing heavy precast concrete segments in the tail shield.

Table 4 summarizes the respective lengths over which the above excavation methods would be utilized for each tunnel route. Figure 10 shows this information in the form of summary tunnel route logs.

In addition to the non-routine methods which might be necessary (drilling and blasting through the shield, for example), all estimates in subsequent sections have allowed for the possible encounter of faulted, sheared or weak (squeezing) ground and local high water inflows. Greatly reduced advance rates apply to all these special operations.

7.3 Tunnel Portals

Tunnel portals have been investigated to a limited extent. In 1981, the upstream portal area of Route T3 was investigated by drilling. In the 1982 investigation, the upstream portal area of T1/T2/T3A and the downstream portal area of all the tunnels were investigated by geophysics (Geo-Physi-Con, 1982). No detailed work has been carried out on tunnel portal design for this report.

For the purposes of route selection and costing, it has been assumed that all portals would require specific measures and careful initial tunnelling; some would require cut-and-cover sections, others might require slope stabilization works. The portal costs have been treated in a non-specific way as lump-sum items.

TABLE 4

EXCAVATION METHODS

DESIGNATION	CONSTRUCTION METHOD	GEOLOGICAL UNITS APPLICABLE	ROUTE T1	ROUTE T2	ROUTE T3	ROUTE T3A
C1	Excavator shield using pre-cast segmental lining.	G1, G2, G3, G4	2770	–	3380	2995
C2	Drill-and-blast with steel sets and cast-in-place concrete lining.	G1, G3	–	150	–	–
C3	Drill-and-blast with steel sets and cast-in-place concrete lining. Mucking through the shield.	G1, G4	150	–	150	150
C4	Drill-and-blast with shotcrete, bolts and paved invert.	G2, G3	–	2050	–	–
C5	Drill-and-blast with shotcrete, bolts and paved invert. Mucking through the shield.	G2, G3, G4	50	–	305	250
C6	Hand mining with steel sets and cast-in-place concrete lining.	G1	–	950	–	–
TOTAL (Excluding Portals)			2970	3150	3835	3395

8.0 CONSTRUCTION ESTIMATES AND COSTS

8.1 Construction Schedules

The schedules for the excavation and support of the tunnels have been prepared on the basis of the following assumptions:

- The tunnels either lie above the water table or have minimal inflow problems.

- In the soft ground sections, it is assumed that the face can be controlled without compressed air.

- For those sections excavated by shield excavator, only one heading is adopted.

- Those sections advanced manually, or by drill-and-blast, are excavated from either one or two headings depending on access. For example, a short section interrupting the excavator would be advanced on one heading only.

- For drill-and-blast sections excavated ahead of and through the shield, greatly reduced advance rates are estimated and applied, i.e. in Routes T1, T3, and T3A. These reductions have been necessary because of the obstructions caused by the inability to back-off the machine.

- A 24 inch rail system with California switch and diesel locomotives are adopted for heading excavation and muck handling.

- In the softer shield excavated ground, a 3.2 m (10.6 ft) circular tunnel, with grouted precast 150 mm (6 inch) thick segmental lining, is adopted.

- o For Route T2 excavated by drill-and-blast, a shotcrete lining with a paved invert has been assumed. For those sections requiring steel sets, and in the hand mined section, a 3.4 m (11 ft) horseshoe section with a 300 mm (12 inch) thick circular concrete lining has been adopted.

- o A convenient muck dumping site close to the portals is available.

The advance rates and synthesis of construction procedures for each method of excavation have been calculated by Bellevue Consultants Ltd. (1982), in a detailed construction estimate based on a careful study of the excavation cycle, mucking requirements, and sequence of support and lining. These advance rates, shown in Table 5, are considered to be average advance rates for extended periods of tunnelling. The advance rate for the shield has a major impact on the total driving time, and is based on the use of a ring beam erector to lift segments, rotate these into position and then expand and lock into place after the shield is cleared.

Tunnel construction is carried out on three shifts per day, of duration of 8, 7-1/2 and 7 hours each, and six days per week; day 7 is allocated for maintenance. Equipment and personnel efficiency factors are based on year-round operation from a camp at the construction site, and at elevation 915 m.

In the compilation of the schedule, it is assumed that the contact grouting of the precast lining is performed concurrently with excavation, while cast-in-place concrete lining is poured and grouted after completion of the excavation. The schedules for the four alternative routes thus deduced are given in Table 6.

TABLE 5

AVERAGE TUNNEL DRIVING RATES

CONSTRUCTION METHOD		RATE (m/day)
C1	Excavator Shield	25.6
C2	Drill-and-Blast (steel sets)	10.4
C3	Drill-and-Blast (steel sets) Through Shield	4.6
C4	Drill-and-Blast (shotcrete)	12.0
C5	Drill-and-Blast (shotcrete) Through Shield	4.6
C6	Hand Mining (steel sets)	8.2

Note: These are for advance of the tunnel heading only, and do not include concreting and invert paving where required.

TABLE 6

CONSTRUCTION SCHEDULES

ACTIVITY	DURATION (weeks)			
	T1	T2	T3	T3A
Mobilization, Move-in Site Grading, Access	2	2	2	2
Portal Preparations	2-1/2	3	2-1/2	2-1/2
Shield Excavation, Precast Lining and Grouting	18	-	22	19-1/2
Drill-and-Blast Excavation (steel sets)	5-1/2	1-1/2*	5-1/2	5-1/2
Drill-and-Blast Excavation (shotcrete)	2	14*	11	9
Hand Excavation (steel sets)	-	9-1/2*	-	-
Concrete Lining	4	15**	6	4
Invert Paving	-	6	-	-
TOTAL TIME	34	51	48	42-1/2
TOTAL LENGTH (m)	2970	3150	3835	3395

* Driving from both ends has been assumed.
** A much faster concreting rate has been assumed than is possible with the short sections in T1, T3 and T3A.

In addition to the above estimates of construction times, two additional schedule items should be noted in the overview of alternative routes:

- o For Routes T1, T3, and T3A where the shield excavator is used, 6 to 7 months delivery should be allowed for the construction of this specially designed piece of equipment. No other long lead time items are required.

- o For all alternatives, the total time from the "decision to construct" to the completion of the tunnel for water conveying is expected to be approximately two years.

Total construction times for Routes T1, T2, T3, and T3A are estimated to be 34, 51, 48, and 42-1/2 weeks, respectively. The relatively long duration for Route T2 results from the need to concrete some 1500 m of tunnel (primarily in the hand mined section) after excavation is complete. Depending on conditions encountered, it may be possible to reduce this time by utilizing a heavy shotcrete lining (placed concurrently with excavation) and a paved invert. However, current indications of geological conditions indicate that the proposal for such a lining method is presently not prudent.

8.2 Cost Estimating Assumptions

The following conditions and assumptions apply to the estimate of tunnel construction costs:

- o All necessary facilities (fuel, electricity, water) are assumed to be provided at site at a reasonable cost.

- o Costs of delivery for plant, equipment and materials to the site are included in the estimates at the following rates:

Concrete, delivered to tunnel portal	\$ 60/cu.yd
Reinforcing steel (ready for installation)	\$ 0.5/lb
Grout (shotcrete)	\$ 100/cu.yd
Welded wire fabric	\$ 0.5/lb
Steel sets (6WF25)	\$ 1.0/lb
Timber (for lagging)	\$ 0.4/fbm
Power	\$ 0.06/kwhr
Camp Costs	\$ 50/man-day

- All construction items are purchased and then salvaged.

- All costs are expressed in 1982 Canadian dollars.

- Labour, including fringes and benefits, is estimated at \$25/hr man for a composite crew. Ten per cent is added for overtime premiums (3 shifts/day), and 50 per cent is added for Saturday work (6-day week). Payroll insurance and taxes are estimated at 30 per cent for underground work, and 20 per cent for outside work.

- Costs include a contingency of 5 per cent and a profit of 10 per cent of total labour and equipment costs or 40 per cent of total labour costs, whichever is highest.

8.3 Cost Estimating Procedure

A simplified cost estimating procedure has been established, based on the detailed preliminary construction estimates originally prepared by Bellevue Consultants Inc. (1982). This procedure was prepared to enable alternative routes and modified ground conditions to be readily compared without the need for a complete revision of the basic estimate.

The procedure categorizes all cost items into three types:

- o Lump sum costs which are *independent* of tunnel length (i.e. mobilization, portals, etc);
- o Direct unit excavation and support costs which can be accurately expressed as \$/m length of tunnel;
- o Escalation type costs.

Unit costs are considered to be essentially valid, for changes to the excavated length of tunnel in question, up to 30 per cent. Escalation type costs are those which are basically proportional to a direct cost item, such as labour, man-hours, or equipment. Such costs include overhead, profit, etc.

Based on the assumptions given earlier, unit costs have been calculated for the various excavation methods likely to be used in the tunnels. These are shown in Table 7. These costs reflect the different methods of excavating similar ground with and without the obstruction of the excavator shield. The costs are direct costs only for excavation, temporary support, final support, lining, and (where applied) invert paving and grouting. They do not include costs for mobilization, portal construction, camp and subsistence, overheads, profit, and contingencies.

Details of the lump sum costs and escalation costs synthesized from the preliminary construction estimates are given in Table 8. The accuracy of these cost indices has been checked by recalculation of costs for the original conditions of the preliminary construction estimate.

Using the cost indices given in Tables 7 and 8 for the four alternative tunnel routes, estimates for the contractors' bid price (including profit, contingency, and all contractors' overhead) have been calculated. These are given in Table 9.

TABLE 7

SUMMARY UNIT COSTS

FOR TUNNEL EXCAVATION AND SUPPORT

CONSTRUCTION METHOD		COST ($/m)
C1	Excavator Shield, including precast lining and grouting	1,672
C2	Drill-and-Blast, steel sets, concrete lining and grouting	4,348
C3	Same as C2, except through shield	9,080
C4	Drill-and-Blast, shotcrete, bolts, paved invert	1,731
C5	Same as C4, except through shield	5,951
C6	Hand Mining, steel sets, concrete lining and grouting	4,500

TABLE 8

SUMMARY COST DETAILS
LUMP SUMS AND ESCALATION ITEMS

ITEM	COST (x $1,000)
Lump Sum Costs	
1) Mobilization and Move-in	800
2) Furnish Plant and Equipment	
- either shield	2,000
plus extras	1,370
- or drill-and-blast	2,787
3) Site Grading and Access	94
4) Portal Excavation and Support	79
5) Portal Structures	126
6) Demobiliation and Salvage (see Note 3)	175 - 40% of Item 2
Unit Costs (per metre of tunnel)	
7) Excavation, Support and Lining	Use Table 7
Escalation Costs	
8) Camp and Subsistence	See Note 1
9) Contractors Overhead and General Expenses	25% of Items 1 thru 7
10) Contractors Profit	
- Greater of either	10% of Items 1 thru 9
or	See Note 2
11) Contingency	5% of Items 1 thru 10

Note 1 - $50/man-day is equivalent to:
For T1 type tunnel - 84 + 0.46/m
For T2 type tunnel - 88 + 0.45/m
For T3 type tunnel - 74 + 0.50/m

Note 2 - For T1 type tunnel - 119 + 0.66/m
For T2 type tunnel - 112 + 0.69/m
For T3 type tunnel - 104 + 0.71/m

Note 3 - Percentage may be varied, depending on length of tunnel.

TABLE 9

ESTIMATED TUNNEL COSTS

Route	Cdn $M
T1	16.25
T2	18.07
T3	20.93
T3A	19.08

A typical breakdown of the costs for Routes T2 and T3, representative of drill-and-blast and shield excavation respectively, are given in Table 10.

8.4 Route Comparisons and Discussions

The estimates of construction schedules and costs afford a good comparison of the relative technical and non-technical factors relevant to each of the four alternative tunnel routes. Based on the project constraints and the current interpretation of geological conditions, the selected routes are considered to represent the broad range of most reasonable route options.

Construction costs are most heavily dependent upon the geological conditions encountered, and thus the excavation techniques proposed. Provided that the excavation methods and advance rates adopted are validated, the estimates can be regarded as reasonably accurate. An accuracy limit of $\pm$10 per cent is considered appropriate.

The considerable differences in construction duration between the four routes arise firstly from the differences in respective tunnel lengths, but also from the effect of the construction method. For Route T2, the considerable length of hand mined section required to be concrete lined after completion of excavation, presents an additional burden on construction time.

TABLE 10

TYPICAL COST BREAKDOWN

ITEM	DRILL-AND-BLAST (i.e. T2)	SHIELD EXCAVATION (i.e. T3)
Labour (including taxes, fringes, and overtime)	6.5	8.4
Supplies	3.2	3.9
Equipment Operating Costs	0.9	1.2
Equipment Rental	1.6	2.2
Permanent Materials	2.7	1.4
Subcontracts	0.2	0.3
TOTAL	15.1	17.2
Contingency (5 per cent)	0.9	1.0
Profit (approximately 10 per cent of total)	2.1	2.7
ESTIMATED BID PRICE	18.1	20.9

NOTE: Costs Are in Millions of Dollars

Cost estimates also follow predictable trends with respect to tunnel lengths, despite the diversity of tunnelling methods. The total cost per metre of tunnel for each of the four routes lies in the narrow range of $5,300 to $5,600/m. It should be noted that while this represents the average expected cost for each tunnel route, the probability of variation from this cost (i.e. the uncertainty in actual cost outcome) is greater for the shield excavated tunnels along Routes T1, T3, and T3A than for the hand excavated/drill-and-blast tunnel Route T2. Because of this, it is common practice to undertake more detailed site investigations when mechanized excavation methods are contemplated.

The advance rate assumed for the excavator shield of 25.6 m/ day, has a major influence on the estimated construction cost. Faster rates have been achieved with this type of machine in similar ground, although slower rates have been more common in the past. The above rate has been adopted bearing in mind the historical progress in tunnelling technology and the fact that the machine would be constructed with special features matched to the anticipated ground conditions.

The long term performance of the segmental lining in erodable ground, particularly in the surficial deposits, was carefully studied. The current lining installation and sealing technology, and the lack of high or fluctuating internal pressures, permit the adequate long term control of leakage, erosion, and stability using precast segmental concrete linings. Consequently, the efficiency of combining temporary and permanent support and the integration of the lining and excavation process, is particularly advantageous under the present geological conditions and design constraints.

In general, the selection of construction equipment, design of excavation support systems and prediction of performance during construction and costing, appear satisfactory and more than adequate for the current objectives of obtaining a refined preliminary cost estimate for several potential tunnel routes. The estimate is considered to be slightly conservative. This is intentional, since at this stage in the conceptual design of the project, some geological uncertainties remain.

9.0 RECOMMENDED ROUTE

The selection of the most appropriate tunnel route considers both the inherent tunnel characteristics and the tunnel as an integral part of the diversion scheme. For the purposes of route comparison, differences in hydraulic operating efficiency and maintenance costs during the life of the structure are considered to be minor.

Factors considered in the selection of the preferred route include cost, remoteness from the pit, geological conditions and implied uncertainties and construction preferences. Since costs for Route T2 lie within the range of those of T1 and T3, the first choice is primarily based on geological and construction conditions. Normally, the use of a machine for tunnel excavation, as opposed to conventional mining, contains a greater uncertainty on the construction outcome because of the inflexibility of machine operation. In this case however, the excavation of Route T2 by drill-and-blast should consider the real possibiltiy of serious problems arising from the combination of extensive lengths of tunnel in ground of low RQD and adverse ground water conditions, complicated by the presence of a subparallel fault. Such ground water conditions are unlikely to present major problems in the more competent rocks crossed by T3 and T3A and, furthermore, the uniformity of the G4 geotechnical unit makes machine excavation reasonably reliable.

Thus, tunnel excavations by machine for Routes T1, T3 or T3A is preferred to conventional driving of Route T2, on the basis of certainty of construction outcome. An added benefit of this choice is the ability to utilize a precast concrete lining for both construction and operational functions. This lining method is most suited to a free-flow or low pressure tunnel for the present geological conditions.

The comparison of Routes T3 and T3A considers the saving in cost of tunnel ($2 M) for the shorter route, relative to the greater costs of pipeline/canal and earthworks structures at the Medicine Creek crossing. It is estimated that the extra costs for pipeline/canal/earthworks associated with the shorter tunnel route are much less than $2 M, especially if the favoured alternative of the pipeline, instead of the canal, is considered. Since no other major factors influence the comparison of T3 and T3 A routes, Route T3 is eliminated from further consideration.

The greater cost of Route T3A compared to that for Route T1 is essentially a consequence of its greater length; as noted earlier, unit costs are very similar. This cost difference can be directly compared to two major differences between the two routes:

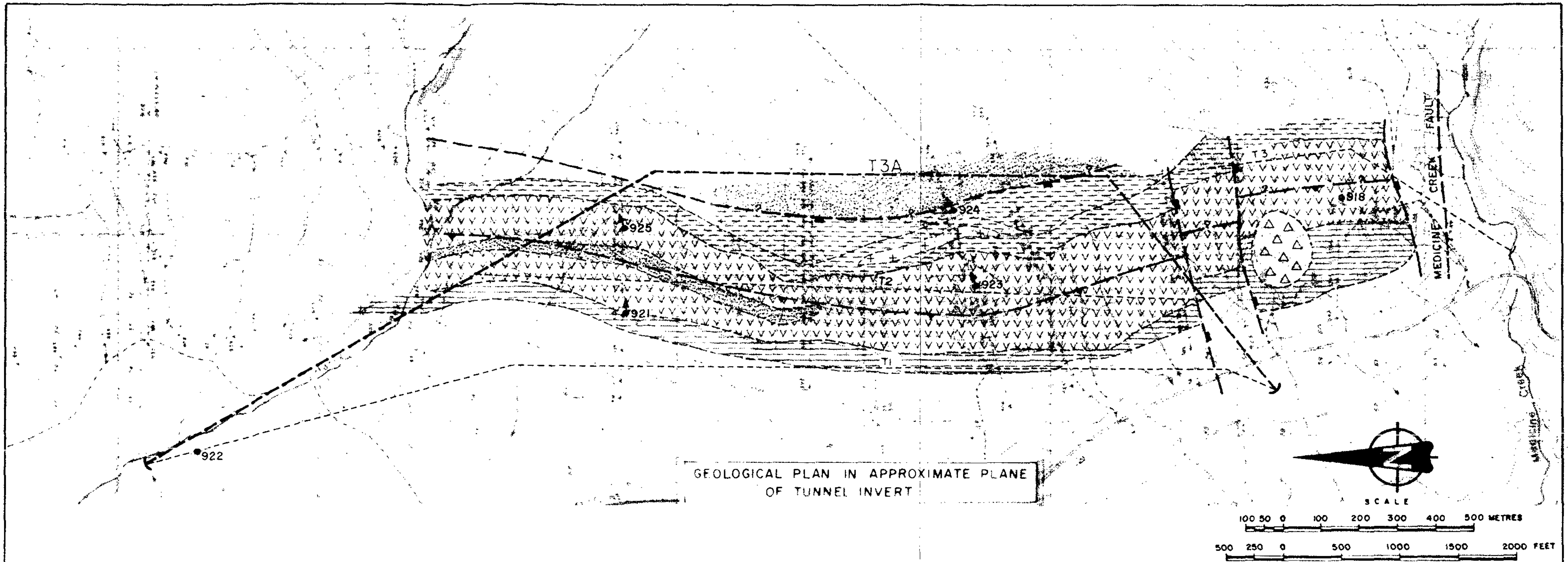
(1) proximity to the ultimate pit rim
(2) differences in construction problems as a consequence of having to cross the andesite unit twice.

Route T1 is located approximately 400 m distant from the pit rim. The adequacy of this separation must consider the potential for seepage from the tunnel modifying the ground water conditions around the pit, the possibility of deep seated pit slope failures affecting the tunnel, and the uncertainty regarding the ultimate position of the pit rim. This last factor is influenced by the life of the scheme, the discovery of new deposits and modified pit slope angles. It should be noted that unless special precautions and lining construction practices are adopted, some seepage through the precast lining into the rock is to be expected. To prevent such seepage two options area available, both cheaper than the cost difference between T1 and T3A, but the lack of construction experience prevents an absolute guarantee of results:

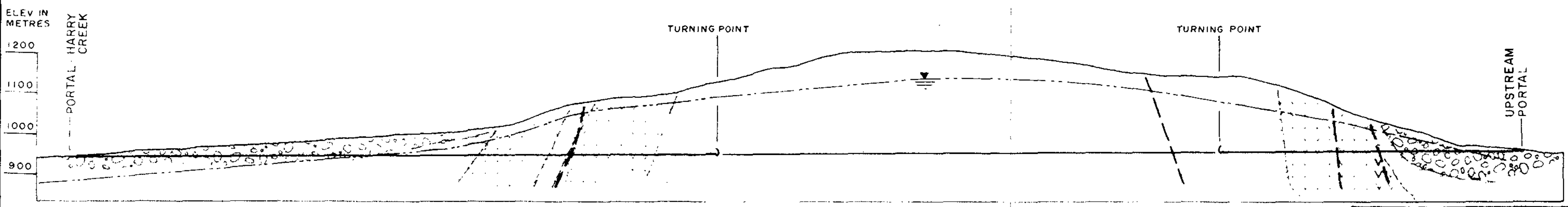
(1) Special design and construction of the precast segments to improve the water tightness.

(2) Installation of an additional coating inside the precast lining to act as an impermeable membrane.

The geological investigations indicate that the contacts between the andesite and the upper and lower volcaniclastics that would be crossed by Route T3A are not expected to present significant tunnelling problems. An allowance for the different tunnelling conditions in the andesite unit has been made in the construction and cost estimates. Thus, on the basis of the above discussion of the various factors affecting the choice of the tunnel route, it is recommended that Route T3A be adopted as the preferred alternative. Figure 12 shows the alignment and profile along the recommended Route T3A. It is emphasized that further investigation would be desirable at a later date to verify conditions assumed for this route, and to establish machine design data.



GEOLOGICAL PLAN IN APPROXIMATE PLANE OF TUNNEL INVERT



GEOLOGICAL SECTION THROUGH RECOMMENDED ROUTE T3A

NOTE For LEGEND, see Fig 3

Golder Associates/Sigma Engineering Ltd.	
B.C.Hydro & Power Authority	
HAT CREEK DIVERSION STUDY	
TUNNEL ROUTE T3A ALIGNMENT AND PROFILE	
Drown	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure 12

10.0 REFERENCES

(1) BELLEVUE CONSULTANTS, 1982. "Hat Creek Diversion Tunnels Preliminary Construction Estimate for Golder Associates."

(2) BRITISH COLUMBIA HYDRO AND POWER AUTHORITY, December 1979. "Hat Creek Project Mining Report."

(3) BRITISH COLUMBIA HYDRO AND POWER AUTHORITY, March 1981. "Hat Creek Project Coal Liquefaction Report."

(4) FRANKLIN, J.A. and CHANDRA, R., 1971. "The Slake Durability Test." International Journal of Rock Mechanics and Mining Sciences, Vol. 9, pp. 325-341.

(5) GEO-PHYSI-CON CO. LTD., June 1982. "Geophysical Survey Hat Creek Coal Project."

(6) GOLDER ASSOCIATES, December 1978. "Hat Creek Project Preliminary Engineering Work - Geotechnical Study 1977-78."

(7) GOLDER ASSOCIATES, March 1977. "Hat Creek Geotechnical Study."

(8) KIM, H., October 1979. "Depositional Environment and Stratigraphic Subdivision - Hat Creek #1 Deposit", CIM Bulletin.

(9) MONENCO CONSULTANTS PACIFIC LTD., January 1977. "Hat Creek Diversion Study."

ADDENDUM 1

Drillhole Logs

EXPLANATION OF SOME TERMS USED ON CORED BOREHOLE LOGS

The logging procedure involved identification of rock type, engineering descriptions of the rock materials and rock masses, per cent recovery, and rates of advance. On the logs, under the heading "Drilling Progress", details are given of timing, bit type, bit diameter in millimetres, and casing depths (m) and diameters (mm). Rate of advance is given in minutes per metre for each core run.

Rock quality designation (RQD) is an empirical measure defined as the percentage ratio of the total length of rock core fragments more than 100 mm long to the total drilled length of the rock (see Figure A1). Rock quality can then be described according to the table below, though it should be noted that RQD can be effected by drilling equipment and technique. RQD is normally assessed for each core run in rock.

ROCK QUALITY DESIGNATION

RQD (Percentage)	Description of Rock Quality
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Core recovery is a percentage of the total length of core recovered to the total drilled length, usually assessed for each core run.

In the bedding/foliation column, the angle between the bedding or foliation relative to the core axis is noted. When not noted, no bedding or foliation was visible.

The fracture index is the ratio of the total number of natural fractures to the total drilled length per core run and is given in units of number per metre. This value usually considers purely natural fractures in rock core. However, fractures are frequently induced by drilling, and a visual assessment of the relative numbers of natural and induced fractures has been made. Hence, on this logging program, RQD has been calculated on the total number of fractures, and fracture index on the natural fractures. A visual representation of core recovery and fracture indices is presented on Figure A1.

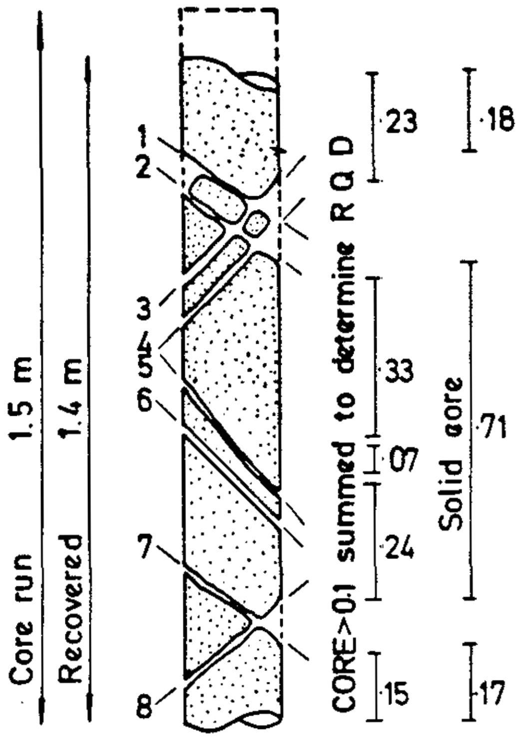
Under the "description" column, details of the colour, grain size, estimated strength, and other natural and/or structural qualities relative to engineering are given, together with a soil or rock name typed in upper case. A brief explanation of some of the descriptive terms used is given on Figure A2. Where joints are present, the descriptive terms shown on Figure A3 have been used, together with details of infilling material, if present.

ABBREVIATIONS USED ON DRILLHOLE LOGS, SAMPLE AND TEST DATA

Symbol	Description	Unit
ATT	Atterberg Limits	
W%	Water Content	%
LL	Liquid Limit	%
PL	Plastic Limit	%
PI	Plasticity Index	%
UCS	Uniaxial Compressive Strength	MPa
Pt. Load Is 50	Point Load Test	MPa
P.A.	Petrographic Analysis	
L.C.	Lost Core	
SD	Slake Durability Test	%
FHT	Falling Head Test	
k	Hydraulic Conductivity	m/sec

CORE RECOVERY AND FRACTURE INDICES

Figure A1



Core recovery $= \frac{1.40}{1.50} = 93\%$

Solid core recovery $= \frac{1.06}{1.50} = 71\%$

RQD $= \frac{0.95}{1.50} = 63\%$

Fracture index $= \frac{8}{1.50} = 5.3$ /m run

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ROCK DESCRIPTION

Figure A 2

The tables given below and overleaf are taken from the format proposed for mining and civil engineering purposes by the Geological Society of London Engineering Group in their Quarterly Journal of Engineering Geology Vol. 10 no. 4 1977 to which reference should be made for the full details of the recommended method.

Descriptive indices for rock material are: **Colour, grain size, weathering and alteration, rock type and strength.**

Rock colour

1	2	3
Light	Pinkish	Pink
Dark	Reddish	Red
	Yellowish	Yellow
	Brownish	Brown
	Olive	Olive
	Greenish	Green
	Bluish	Blue
		White
	Greyish	Grey
		Black

Grain size

Term	Particle size	Retained on BS Sieve No. (approx)	Equivalent Soil Grade
Very coarse-grained	>60 mm	2 in	Boulders + Cobbles
Coarse-grained	2–60 mm	8	Gravel
Medium-grained	60 microns-2 mm	200	Sand
Fine-grained	2–60 microns		Silt
Very fine-grained	<2 microns		Clay

Note: grains >60 microns diameter are visible to the naked eye

Weathering/alteration grades

Term	Description	Grade
Fresh	No visible sign of rock material weathering.	IA
Faintly weathered	Discoloration on major discontinuity surfaces.	IB
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the material may be discoloured by weathering and may be somewhat weaker than in its fresh condition.	II
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or corestones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or corestones	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Rock material strength

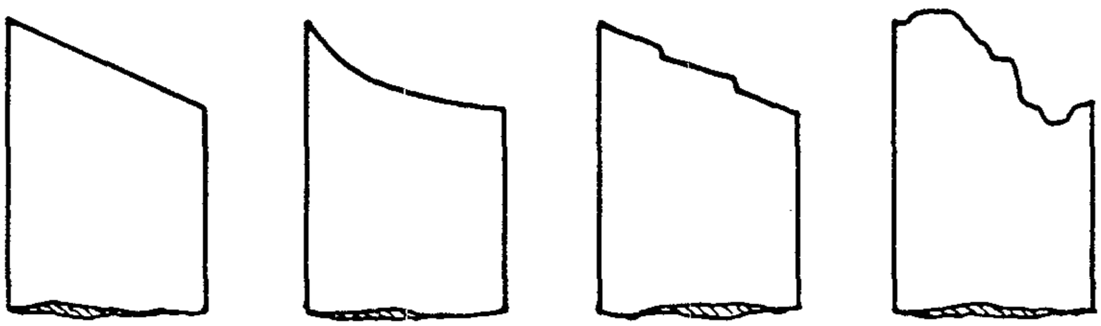
Term	Unconfined Compressive Strength MN/m^2 (MPa)	Field estimation of hardness
Very strong	>100	Very hard rock–more than one blow of geological hammer required to break specimen
Strong	50–100	Hard rock–hand held specimen can be broken with a single blow of geological hammer.
Moderately strong	12·5–50	Soft rock–5 mm indentations with sharp end of pick.
Moderately weak	5·0–12·5	Too hard to cut by hand into a triaxial specimen.
Weak	1·25–5·0	Very soft rock– material crumbles under firm blows with the sharp end of a geological pick.
Very weak rock or hard soil	0·60–1·25	Brittle or tough, may be broken in the hand with difficulty.
Very stiff	0·30–0·60*	Soil can be indented by the fingernail.
Stiff	0·15–0·30	Soil cannot be moulded in fingers
Firm	0·08–0·15	Soil can be moulded only by strong pressure of fingers.
Soft	0·04–0·08	Soil easily moulded with fingers
Very soft	0·04	Soil exudes between fingers when squeezed in hand.

* The compressive strengths for soils given above are double the unconfined shear strengths.

PROJECT NO. 822-1523 DRAWN - REVIEWED DATE Dec. '82

PLANARITY AND ROUGHNESS OF JOINTS IN DRILL CORE

Figure A 3



Planar (P) | Curved (C) | Stepped (S) | Irregular (I)

PLANARITY

Polished (P) Slickensided (K) | Smooth (S) | Rough (R) | V. Rough (V)

ROUGHNESS

PROJECT NO. 822-1523 DRAWN - REVIEWED DATE Dec. '82

Type of drilling: ROTARY CORE, MUD FLUSH

Coordinates: 5625648.9 N / 600278.6 E

DRILLHOLE No. 82-921

Sheet 1 of 10

Rig: LONGYEAR 44

Dip: 65°

Location: HAT CREEK EASTERN ESCARPMENT

Bit: TRICONE/HQ OVERSIZE

Azimuth: N90°E

Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10, 20)	R. Q. D. (0, 50)	Core Recovery % (0, 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation (1, 2)	Legend	Description
												Tricone HW casing to 21.3 m.
					2							
					4							
				6								6.4 Wash sample #1.
				8								
				10								
				12								
				14								
					1005.4							15.2 - 23.5 Light to medium brown; stiff; SANDY SILT with some fine-medium gravel, some sand layers; TILL.
				16								16.5 Wash sample #2.
												17.7 Wash sample #3.
				18								
												19.2 Wash sample #4.
				20								20.2 Wash sample #5.

Contractor: COATES

Date started: 3-6-82

Date finished: 10-6-82

Logged by: D.P.F./W.D.C.

Checked by: D.F.

Date: 18-8-82

Remarks: HW casing to 21.6 m.

Golder Associates

Scale: 1 : 50 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE, MUD FLUSH
Coordinates: 5625648.9 N, 600278.6 E
Rig: LONGYEAR 44
Dip: 65^0
Bit: TRICONE/HQ OVERSIZE
Azimuth: $N90^0E$

DRILLHOLE No. 82-921

Sheet 2 of 10

Location: HAT CREEK EASTERN ESCARPMENT

Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Depth	Reduced Level	Water Level	Test Results	Description
								TILL Cont'd.
4/6 AM								
	5	95	95	22			W% = 15.5 Non-plastic	21.9 - 22.0 MC and ATT.
	11	100	100	24	996.4			24.2 - 41.8 Moderately to heavily weathered; fractured; medium-dark grey; moderately strong; ANDESITE; locally heavily altered and very friable.
	9	70	70					
	13	0	35	26				
	7	0	70					27.4 - 32.9 Zone of intense alternation, very friable and weak.
		20	80	28			UCS = 23.47 MPa	28.2 - 28.5 Uniaxial.
	2	85	100	30				
	5	6	10	32				
		0	25					
	22	0	10	34				
	20	0	10	36				
AM	30	0						
PM	30	0	0	38				
	20	0	0	40		PZ. #2 40 m 8/24/82		

Contractor: COATES
Date started: 3-6-82
Date finished: 10-6-82
Logged by: D.P.F./W.D.C.
Checked by: DF
Date: 18-8-82
Remarks:

Golder Associates

Scale: 1 : 50 metric

PROJECT NO. ...... DRAWN ...... REVIEWED ...... DATE ......

Type of drilling: ROTARY CORE, MUD FLUSH
Coordinates: 5625648.9 N, 600278.6 E

DRILLHOLE No. 82-921
Sheet 3 of 10

Rig: LONGYEAR 44
Dip: 65°
Location: HAT CREEK EASTERN ESCARPMENT

Bit: TRICONE/HQ OVERSIZE
Azimuth: N90°E
Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Description
	20		0							ANDESITE Cont'd. 40.2 - 41.8 Fresh; dark grey, coarse gravel size fragments of obsidian returned.
	16		20		978.8					
	13	18	40	42						41.8 - 47.5 Moderate-heavily weathered; dark grey-reddish brown; moderately strong; ANDESITE BRECCIA; locally contains angular to subangular coarse gravel size obsidian fragments in reddish brown matrix; very strong.
	15	20	60	44			Pt.load Is 50 = 1.6 MPa			43.7 - 43.8 Pt. load
	13		43							
	19	0	44	46						
	18	0	63		973.1					47.5 - 70.4 Moderate-heavily weathered; reddish brown-medium grey; moderately strong-weak; ANDESITE locally highly altered and bentonitic.
	7	0	58	48						47.5 - 48.6 Highly weathered; angular andesite clasts in clay/silt matrix; very weak; FAULT GOUGE.
	25		60	50						
	22	0	46							51.0 - 53.6 Numerous fractures filled with green clay.
25	16	15	100	52			Pt.load Is 50 = 6.5 MPa			52.8 - 53.0 Pt. load.
15	10		87	54						54.0 - 70.4 Zone of intense alteration; highly bentonitic; locally brecciated; weak.
30 6/5 AM	20		20	56						
20		60	100						2.5	57.5 - 58.0 Breccia zone.
23	15	60	82	58				60° 50°	1.3	
30	19	60	75	60				45°	2.7	

Contractor: COATES
Date started: 3-6-82
Date finished: 10-6-82
Logged by: D.P.F./W.D.C.
Checked by: DF
Date: 19-8-82
Remarks:

Golder Associates

Scale: 1 : 50 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling	ROTARY CORE, MUD FLUSH	Coordinates	5625648.9 N / 600278.6 E	DRILLHOLE No. 82-921 Sheet 4 of 10
Rig	LONGYEAR 44	Dip	65°	Location HAT CREEK EASTERN ESCARPMENT
Bit	TRICONE/HQ OVERSIZE	Azimuth	N90°E	Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend
	13	0	45								
	9	95	100	62			W%=23.2 LL=36.8 PL=28.6 PI= 8.2		20		
	11	23	45				Pt.load Is 50 = 0		5		
	18	0	35	64				20°			
	30	60	75	66			P.A.	25°	3.3		
	30	30	100				Pt.load Is 50 = 0.5 MPa	35°	4		
	24	30	75	68			UCS = 4.01 MPa	30°	3.5		
	15	60	65				Pt.load Is 50 = 0 DIA. 0.41 MPa Axial.		2.7		
	36	0	100	70	950.2			60°	10		
	8	0	65						10		
	26	0	50	72							
5/5	21	0	25								
AM	30	0	100								
5/5 PM	41	0	80	74							
	24	0	83								
	23	0	62	76							
	18	0	63								
	25	0	22	78							
				80							

Description

ANDESITE Cont'd.

60.5 - 68.3 Highly weathered; flow banded andesite; very friable; weak; highly bentonitic; locally brecciated.

61.6 - 61.8 MC, ATT.

62.2-62.5 Pt. load.

66.5 P.A.: Altered flow banded andesite.

67.4 - 67.6 Pt. load.

68.3 - 70.4 Zone of intense alteration; highly bentonitic; locally brecciated.

68.9 - 69.1 Uniaxial.

69.9 - 70.1 Pt. load.

70.4 - 187.7 Slightly weathered; dark grey; aphanitic; strong; ANDESITE; highly fractured; occasional clay infillings along fractures; locally brecciated; local zones of moderate-heavy alteration.

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor COATES	Logged by D.P.F./W.D.C.	Remarks:
Date started 3-6-82	Checked by DF	Golder Associates
Date finished 10-6-82	Date 19-8-82	Scale: 1 : 50 metric

Type of drilling: ROTARY CORE, MUD FLUSH	Coordinates: 5625648.9 N / 600278.6 E		DRILLHOLE No. 82-921 Sheet 5 of 10
Rig: LONGYEAR 44	Dip: 65^{0}		Location: HAT CREEK EASTERN ESCARPMENT
Bit: TRICONE/HQ OVERSIZE	Azimuth: N90^{0}E		Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	21	0	89									
	12	0	47									81.6 - 81.8 Breccia zone.
	26	20	58	82								82.7 - 82.8 Breccia zone.
	21	48	81	84								83.8 - 83.9 Breccia zone.
	18	54	96				UCS = 2.97 MPa					84.4 - 88.1 Alteration zone; moderately strong; fractured; locally exhibits flow banding. 85.0 - 85.2 Uniaxial.
6/6 AM	28	66	100	86			UCS = 7.22 MPa					86.3 - 86.6 Uniaxial.
	23	0	55				P.A.		16			86.6 P.A.: Andesite breccia.
	29	0	0	88								
	29	0	0									
	6	0	20	90			PL & LL Non-plastic					90.2 - 90.3 ATT.
	23	0	10									
	27	0	0				P.A.					91.4 P.A.: Trachytic andesite.
	16	0	0	92								
	30	0	100									93.4 - 96.3 Breccia zone.
	14	45	95	94					7.3			
	22	25	65						9			
	22	0	100	96			Pt.load Is 50 = 0.5 MPa					96.3 Pt. load.
	23	35	85					50^{0}				97.1 - 98.1 Breccia zone.
	8	0	90	98			Pt.load Is 50 = 0					97.8 - 97.9 Pt. load.
6/6 PM	20	66	100	100								

Contractor: COATES	Logged by P.P./W.D.C.	Remarks:	
Date started: 3-6-82	Checked by: DF	Golder Associates	Scale: 1 : 50 metric
Date finished: 10=-6-82	Date: 19-8-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE, MUD FLUSH	Coordinates: 5625648.9 N, 600278.6 E	DRILLHOLE No. 82-921	Sheet 6 of 10
Rig: LONGYEAR 44	Dip: 65°	Location: HAT CREEK ESCARPMENT	
Bit: TRICONE/HQ OVERSIZE	Azimuth: N90°E	Reference elevation: DRILL FLOOR	

Drilling Progress	Rate of Advance min./m	R.Q.D.	Core Recovery %
	22	0	76
	20	53	100
	26	13	100
	23	53	79
	17	0	100
	35	0	88
	27	77	95
	26	20	100
	33	60	100
	23	0	72
	21	0	0
7/6 AM	16	15	100
	7	20	100
	25	0	90
	38	0	0
	10	0	100
	20	30	100
	20	0	80
	20	40	85
	10	0	80
	18	45	100

Depth	Test Results	Bed./Fol.	Fracture Index
approx. 101.5	UCS = 7.87 MPa	45°	
approx. 102		32°	
approx. 114		65°	13.3
approx. 114.2	UCS = 1.95 MPa		
approx. 117.4	Pt.load Is 50 = 1.3 DIA 51.9 MPA AXIAL.		12
approx. 119.3	UCS = 5.17 MPa	54°	

Description

ANDESITE Cont'd.

100.8 - 103.4 Zone of well developed flow banding.

101.4 - 101.6 Uniaxial.

103.4 - 105.2 Highly altered breccia zone.

106.0 - 107.7 Highly weathered zone, heavily altered; moderately weak; soft; typically brecciated with chloritic matrix; Brecciated zones highly bentonitic.

107.7-109.0 Breccia zone with chloritic matrix; friable.

110.0 - 113.6 Zone of intense fracturing.

114.7 - 116.4 Breccia zone, locally chloritized.

114.1 - 114.3 Uniaxial.

117.4 Pt. load.

119.2 - 119.3 Uniaxial.

Depth scale: 102, 104, 106, 108, 110, 112, 114, 116, 118, 120

Contractor: COATES	Logged by: P.P./W.D.C.	Remarks:	
Date started: 3-6-82	Checked by: D.F.	Golder Associates	Scale: 1 : 50 metric
Date finished: 10-6-82	Date: 19-8-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE, MUD FLUSH	Coordinates: 5625648.9 N, 600278.6 E	DRILLHOLE No. 82-921	Sheet 7 of 10
Rig: LONGYEAR 44	Dip: 65°	Location: HAT CREEK EASTERN ESCARPMENT	
Bit: TRICONE/HQ OVERSIZE	Azimuth: N90°E	Reference elevation: DRILL FLOOR	

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	22	20	100				UCS = 1.97 MPa	40°	8			120.8 - 121.0 Breccia zone, chloritic matrix. 121.9 - 122.1 Breccia zone. 120.8 - 121.0 Uniaxial.
	28	0	55	122				53°				
	26	0	10									
	24	0	65	124								
	16	0	20									
7/7 PM	25	0	20	126								126.2 - 127.2 Lost core.
	30	0	0								L.C.	
8/7 AM	10	0	100									
	33	0	0	128								
	14	0	65					50°				
	15	0	20	130								130.1 - 135.5 Zone of intense fracturing; fractures closely spaced (< 1 cm); fractured locally are filled with chlorite.
	13	0	35	132								
	16	40	65									
		70	100									
	10	0	30	134			Pt. load Is 50 = 0 DIA. 0.28 MPa AXIAL.					134 Pt. load.
	5	0	100									
	18	0	45					58°	20			
	12	30	80	136								136 - 140 Lost core.
	10	0	35								L.C.	
8/6 PM	20	0	66	138								
	23	0	3	140								

Contractor: COATES	Logged by: D.P.F./W.D.C.	Remarks:	
Date started: 3-6-82	Checked by: D.F.	Golder Associates	Scale: 1 : 50 metric
Date finished: 10-6-82	Date: 19-8-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling	ROTARY CORE, MUD FLUSH	Coordinates	5625648.9 N 600278.6 E	DRILLHOLE No. 82-921 Sheet 8 of 10
Rig	LONGYEAR 44	Dip	65°	Location HAT CREEK EASTERN ESCARPMENT
Bit	TRICONE/HQ OVERSIZE	Azimuth	N90°E	Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	23	0	3									140.5 - 141.7 Lost core.
	33	0	20								L.C.	
	40	0	100	142								
	35	0	51									
		0	51									
	33	0	50									
		0	60	144								144.5 - 147.8 Highly fractured and chloritic.
	33	0	100									
	50	0	82									
	32	0	100	146								146.3 - 146.6 Breccia zone with chloritic clay matrix.
	29	0	39									
	30	0	33	148								
	17	0	20									
9/6 AM	17	0	60	150								
	17	0	10									151.8 - 152.0 Shear zone.
	28	0	50	152								
	30	0	90									
		0	90									
	8	0	35									
	14	0	5	154								
	12	0	0	156								
	28		85					40°				
	22	40	90	158				45°	9			158.2 - 158.3 Shear zone.
	15	0	65					40°				159.7 - 159.8 Shear zone, clay gouge. 159.7 - 159.8 MC & ATT.
				160								

Contractor COATES	Logged by P.P./W.D.C.	Remarks:	
Date started 3-6-82	Checked by D.F.	**Golder Associates**	Scale: 1 : 50 metric
Date finished 10-6-82	Date 19-8-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

DRILLHOLE No. 82-921

Sheet 9 of 10

Type of drilling	ROTARY CORE, MUD FLUSH
Coordinates	5625648.9 N / 600278.6 E
Rig	LONGYEAR 44
Dip	65^{o}
Location	HAT CREEK ESCARPMENT
Bit	TRICONE/HQ OVERSIZE
Azimuth	$N90^{o}E$
Reference elevation	DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	26	65	100									
	20	25	100									
	13	0	60									
	17	0	45	162								
	13	40	85									
	16	0	30	164								
	11	50	70	166				35^{o}	7			
	11	20	45					45^{o}	5			
AM	11	40	80	168				40^{o}	2.9			168.3 - 187.7 Zones of intense fracturing and breccia; slightly chloritic; breccia matrix; occasionally bentonitic.
PM	16	65	100	170				38^{o}				
	13	82	92					30^{o}				
	16	61	100	172				32^{o}	4.4			
	20	71	100					30^{o} 50^{o}	1.1			
	24	71	100	174					3.3			
	15	85	100	176			P.A.	50^{o}	2.6			176.4 P.A.: Trachytic andesite.
	13	100	100					50^{o}	1.3			
	15	100	100	178								
				180								

Contractor	COATES	Logged by	P.P./W.D.C.
Date started	3-6-82	Checked by	D.F.
Date finished	10-6-82	Date	19-8-82

Remarks:

Golder Associates

Scale: 1 : 50 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE, MUD FLUSH

Coordinates 5625648.9 N 600278.6 E

DRILLHOLE No. 82-921

Sheet 10 of 10

Rig LONGYEAR 44

Dip 65^{0}

Location HAT CREEK EASTERN ESCARPMENT

Bit TRICONE/HQ OVERSIZE

Azimuth $N90^{0}E$

Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	11	98	100					65^{0}	1.3			
	11	95	100	182				60^{0}	2			
	32	0	50									
	23	98	98	184				35^{0} 58^{0}				
	20	100	100	186			UCS = 3.74 MPa					185.4 - 185.6 Uniaxial.
		0	26									
		0	50									
10/6	15	75	100									
AM	40		100									
	8	100	100	188	832		UCS = 9.41 MPa		5.8			188.7 - 188.9 Uniaxial.
				190								188.6 END OF HOLE

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor COATES

Date started 3-6-82

Date finished 10-6-82

Logged by P.E.P.

Checked by D.F.

Date 19-8-82

Remarks: Piezometer #1 plugged at 36.3m.

Golder Associates

Scale: 1 : 50 metric

Type of drilling: ROTARY CORING	Coordinates: 5626797.3 N / 599915.4 E	**DRILLHOLE No. 82-922**	Sheet 1 of 2
Rig: LONGYEAR 44	Dip: -90°	Location: HAT CREEK	
Bit: TRICONE/HUDDY	Azimuth: Ø	Reference elevation: 960.3 m	

Drilling Progress	Rate of Advance min./m (10, 20)	R. Q. D. (0, 50)	Core Recovery % (0, 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												0 - 3.5 Tricone brown silty/clayey coarse SAND/fine GRAVEL (quartz, rock fragments)
				2								
					956.9							3.35 - 23.9 Coring gravel and cobbles in brown silty/clay/sand matrix; locally contains thin beds of sand; GLACIOFLUVIAL DEPOSITS.
	10		20	4								
	10		30	6								
	25		35									
	17		30	8								
	19		40									
	12		48	10								
	12		30									
	15		45	12								11.9 - 12.2 Uniaxial
	16		75									
	21		57									12.3 - 12.4 Pt. load
				14							L.C.	
	12		31									
	40		100									
	13		11	16								
	9		20									
	17		17	18								
	15		17									
	11		0	20								Coarse grey angular sand in cuttings, no water pressure.

Contractor: COATES	Logged by: N.M	Remarks:	
Date started: 15 - 6 - 82	Checked by: D.F.	**Golder Associates**	Scale: 1 : 50 metric
Date finished: 15 - 6 - 82	Date: 17 - 8 - 82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORING	Coordinates 5626797.3 N 599915.4 E		**DRILLHOLE No.** 82-922 Sheet 2 of 2
Rig LONGYEAR 44	Dip 90°		Location HAT CREEK
Bit HUDDY FACE INJECT	Azimuth Ø		Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10, 20)	R. Q. D. (0, 50)	Core Recovery % (0, 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												COARSE SAND cont'd
	7.6		22									Coarse to fine, subrounded gravel and coarse sand in wash.
	11		13	22								
	12		10									22.2 Small amount of light brown silt matrix recovered.
	13		15	24	936.4							23.9 - 25.0 Dark brown; well rounded gravel and cobbles in a silty sand matrix; TILL.
			20	25								
												935.3 END OF BOREHOLE

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor COATES	Logged by P.P.	Remarks:	
Date started 15 - 6 - 82	Checked by D.F.	**Golder Associates**	Scale: 1 : 50 metric
Date finished 15 - 6 - 82	Date 17 - 8 - 82		

Type of drilling: ROTARY CORE/POLYMER FLUSH	Coordinates: 5624713.3 N / 600362.3 E	DRILLHOLE No. 82-923 — Sheet 1 of 13
Rig: LONGYEAR 44	Dip: 60^{0}	Location: HAT CREEK EASTERN ESCARPMENT
Bit: HUDDY, "BLACK" DIAMOND IMPREGNATED	Azimuth: $N73^{0}E$	Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Fracture Index
16/6 PM	12	61	94	7f
	13	85	100	9f
	7	9	46	L.C. / 3f / 2f / L.C.
	11	21	69	9f
	15	0	65	5f
	22	59	100	8f
	13	34	72	11f
	15	71	100	9f
	12	98	100	5f
	13	14	80	5f
	14	69	69	3f
	11	72	100	7

Depth	Reduced Level	Test Results	Bed./Fol.	Description
0 – 2.13				Triconed to 2.13 m.
2.13	1112.3	UCS = 29.5 MPa	12^{0}	2.13 - 6.96 Slightly weathered; finely bedded; light brownish grey; moderately strong to strong; ANDESITIC TUFF, flow banding, fractures occasionally rehealed. 2.0 - 2.3 Uniaxial. 4.8 - 5.7 Lost core. 5.7 - 6.0 Heavily weathered. 6.4 - 6.9 Lost core.
6.96	1107.4	UCS = 1.56 MPa	65^{0}	6.96 - 12.3 Slightly weathered; massive; moderately strong, ANDESITIC AGGLOMERATE, occasional pyroclasts, flow banded or flattened lapilli. 7.9 - 9.1 Clayey gravel. 7.6 - 7.8 Uniaxial. 9.1 - 14.3 Massive andesite.
12.3	1102.1	UCS = 7.15 MPa; UCS = 5.56 MPa		12.3 - 24.4 Slightly weathered; massive; vari-coloured; moderately strong to strong; ANDESITE BRECCIA. 13.4 - 13.6 Breccia; coarse gravel size clasts in chlorite matrix. 15.3 - 16 Intensely fractured zone, slicken-sided rock fragments. 12.4 - 12.6 Uniaxial. 14.5 - 14.6 Uniaxial.

Contractor: COATES
Date started: 16-6-82
Date finished: 24-6-82
Logged by: P.P./N.G.M.
Checked by: D.F.
Date: 27-7-82
Remarks: HW Casing to 2.13 m

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH
Coordinates: 5624713.3 N, 600362.3 E

DRILLHOLE No. 82-923

Sheet 2 of 13

Rig: LONGYEAR 44
Dip: 60°
Location: HAT CREEK EASTERN ESCARPMENT

Bit: HUDDY, "BLACK" DIAMOND IMPREGNATED
Azimuth: N73°E
Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
										I		ANDESITE BRECCIA Cont'd.
	15	60	100				UCS = 13.6 MPa		7f			20.8 - 20.9 Uniaxial.
	13	91	100	22					4f			
	13	32	78	24	1090.0				6f			
	13	57	100				UCS = 2.99 MPa		9f			24.4 - 84.9 Slightly weathered; massive; fine grained; green-grey; moderately strong; ANDESITE; locally brecciated, highly fractured and jointed.
	15	77	100	26					3f			25.0-25.4 Fractured and weathered zone. 25.8 - 26.1 Uniaxial.
	15	100	100	28					2f			
	11	70	98	30			UCS = 7.30 MPa		5f			29.3 - 29.5 Uniaxial.
	10	0	52						10f			
	20	0	100	32			Pt. load Is 50=4.7 MPa (DIA.)		10f			32.7 - 32.8 Pt. load.
	15	10	95						8f			33.4 - 33.6 Intensely fractured zone.
17/6 AM	20	0	63	34					3f			
	12	22	80						6f			35.7 - 35.9 Fractured zone.
	13	0	85	36					4f			36.1 - 36.3 Fractured zone.
	12	0	75						1f			37.3 - 38.7 Shear zone.
	14	0	78	38			UCS = 11.60 MPa					38.8 - 39.0 Uniaxial
	13	11	75	40					5f			39.6 - 39.9 Fractured zone.

Contractor: COATES
Date started: 16-6-82
Date finished: 24-6-82
Logged by: P.P./N.G.M.
Checked by: D.F.
Date: 27-7-82
Remarks:

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH	Coordinates: 5624713.3 N, 600362.3 E	DRILLHOLE No. 82-923	Sheet 3 of 13
Rig: LONGYEAR 44	Dip: 60°	Location: HAT CREEK EASTERN ESCARPMENT	
Bit: HUDDY, "BLACK" DIAMOND IMPREGNATED	Azimuth: N73°E	Reference elevation: DRILL FLOOR	

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
	10 20	0 50	0 50									ANDESITE Cont'd.
	15	0	100						6f			40.1 - 40.3 Fractured zone.
	13	0	13	42								
	13	0	50				S.D.= 74.6%					42.4 - 42.6 Slake-durability test.
	11	0	20									
	17	10	60	44					3f			
	8	0	17									
	9	0	78	46			Pt.load Is 50 = 4.75 MPa		3f			46.6 - 46.8 Pt. load.
	11	0	94						5f			47.2 - 49.7 Packer test - FHT.
	8	0	3	48			Packer FHT K = 4.33 x 10^{-4} m/s					
	13	0	85									
17/6 PM	17	0	34	50								
	17	0	73						5f			
	14	0	95	52		52.6 8/24/82	UCS = 10.86 MPa		16f			52.1 Becoming chloritic. 52.1 - 52.3 Uniaxial.
	14	13	91	54			UCS = 22.7 MPa		12			54.1 - 54.2 Uniaxial.
	10	0	33	56								
	14	0	3						L.C.		L.C.	56.8 - 58.2 Lost core.
	13	0	100	58								
	39	0	60									
	14	0	50	60								

Contractor: COATES	Logged by: P.P./N.G.M.	Remarks: 42.1 m : Bit changed.	
Date started: 16-6-82	Checked by: D.F.	Golder Associates	Scale: 1 : 100 metric
Date finished: 24-6-82	Date: 27-7-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH
Coordinates: 5624713.3 N, 600362.3 E

DRILLHOLE No. 82-923

Rig: LONGYEAR 44
Dip: 60^{0}
Location: HAT CREEK EASTERN ESCARPMENT

Bit: HUDDY,"BLACK" DIAMOND IMPREGNATED
Azimuth: $N73^{0}E$
Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	17	0	50						2, 2			
	15	0	30	62			Pt.load Is 50 = 0 DIA. .23 MPa AXIAL.					63.4 - 63.5 Pt. load.
	20	0	40	64								
	19	0	73									
	19	0	30	66								
	22	0	33									
	22	0	1	68					L.C.		L.C.	68.0 - 70.2 Lost core.
	13	0	0	70					L.C.			70.4 - 71.4 Highly fractured.
18/6 AM	11	8	53						6f			
	12	6	69	72					3f			
	15	100	100				UCS = 1.21 MPa		1f			73.5 - 73.8 Uniaxial.
	13	0	50	74					2f			
	18	0	65									
	20	0	83						3f			
	13	0	50	76					1f			76.8-79.0 Brecciated Andesite tuff.
	13	33	86				S.D. = 17.5%		4f			76.9 - 77.0 Slake-durability test.
	11	81	91	78			UCS = 3.92 MPa		2f			78.8-79.0 Uniaxial.
	11	7	60	80					1f			

Contractor: COATES
Logged by: P.P./N.G.M.
Remarks:

Date started: 16-6-82
Checked by: D.F.

Date finished: 24-6-82
Date: 27-7-82

Golder Associates

Scale 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

DRILLHOLE No. 82-923

Sheet 5 of 13

Type of drilling	ROTARY CORE/POLYMER FLUSH
Coordinates	5624713.3 N / 600362.3 E
Rig	LONGYEAR 44
Dip	60^0
Bit	HUDDY,"BLACK" DIAMOND IMPREGNATED
Azimuth	$N73^0E$
Location	HAT CREEK EASTERN ESCARPMENT
Reference elevation	DRILL FLOOR

Drilling Progress	Rate of Advance min./m	R. Q. D.	Core Recovery %	Reduced Level	Test Results	Fracture Index
	11	7	60			1f
	9	57	100		UCS = 16.94 MPa	6f
	10	23	77			5f
	14	40	100			
	13	50	100	1029.0	UCS = 4.33 MPa	6f
	12	60	100			11f
	11	50	100	1025.7	UCS = 4.81 MPa	5f
	13	57	100			6f
	9	47	94		Packer FHT	4f
	12	10	93		K = 5.79×10^{-6} m/s	5f
18/6 PM	11	70	100		UCS = 11.0 MPa	11f
	11	56	100		UCS = 18.04 MPa	13f
	6	36	100			15f
	7	39	100			13f

Depth scale: 82, 84, 86, 88, 90, 92, 94, 96, 98, 100

Description

ANDESITE Cont'd.

- 80.1 - 83.6 Highly brecciated tuff.
- 81.0 - 81.2 Uniaxial.
- 83.7 - 84.8 Grey; moderately strong; fine grained, tuff.
- 84.9 - 85.0 Uniaxial.
- 85.4 - 88.7 Weathered, massive; dark grey to black; fine grained ANDESITE FLOW; typically vesicular or amygdaloidal; locally contains pyrite.
- 87.2 - 87.4 Uniaxial.
- 88.7-110.0 Weathered, dark grey to black, fine grained; ANDESITE; locally heavily fractured and brecciated.
- 90.3 - 93.3 Packer test - FHT.
- 91.6 - 91.8 Fractured zone.
- 93.1 - 93.3 Uniaxial.
- 95.4 - 95.6 Uniaxial.

Contractor	COATES	Logged by	P.P./N.G.M.
Date started	16-6-82	Checked by	D.F.
Date finished	24-6-82	Date	27-7-82

Remarks:

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE/POLYMER FLUSH

Coordinates 5624713.3 N
600362.3 E

DRILLHOLE No. 82-923

Sheet 6 of 13

Rig LONGYEAR 44

Dip 60°

Location HAT CREEK EASTERN ESCARPMENT

Bit HUDDY,"BLACK" DIAMOND IMPREGNATED

Azimuth N73°E

Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												ANDESITE Cont'd.
	8	60	100				UCS = 7.81 MPa		13f			100.2 - 100.4 Uniaxial.
	11	22	100						23f			
	9	95	100	102					6f			
	15	32	91	104			UCS = 8.35 MPa		23f			103.8 - 104.0 Uniaxial.
	21	33	92						17f			105.7 - 105.8 Breccia zone, chlorite matrix.
	16	0	87	106					13f			
	12	49	100	108					15f			
	13	72	95	110	1004.4		UCS = 19.39 MPa		10f			109.1 - 109.3 Uniaxial.
	13	82	100				UCS = 12.3 MPa		5f			110 - 120 Moderately weathered; massive; locally vesicular; black/rusty brown ANDESITE flow; locally chloritized and/or oxidized iron; occasional breccia zones.
	14	93	100	112					8f			111.2 - 111.4 Uniaxial.
	15	78	100	114					13f			
	15	92	100	116			UCS = 9.41 MPa		8f			115.5 - 115.6 Uniaxial.
	34	82	100				UCS = 19.32 MPa		9f			117.7 - 117.9 Uniaxial.
	22	82	100	118					5f			
	14	52	100	120					11			

Contractor COATES

Date started 16-6-82

Date finished 24-6-82

Logged by P.P./N.G.M.

Checked by D.F.

Date 27-7-82

Remarks:

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling	ROTARY CORE/POLYMER FLUSH	Coordinates	5624713.3 N 600362.3 E	DRILLHOLE No. 82-923 Sheet 7 of 13
Rig	LONGYEAR 44	Dip	60°	Location HAT CREEK EASTERN ESCARPMENT
Bit	HUDDY,"BLACK" DIAMOND IMPREGNATED	Azimuth	N73°E	Reference elevation DRILL FLOOR

Rate of Advance min./m	R. Q. D.	Core Recovery %	Depth	Reduced Level	Test Results	Fracture Index	Description
				994.4			ANDESITE Cont'd.
							120.0-142.3 Weathered;massive; medium to dark grey; moderately strong; ANDESITE; locally brecciated and altered.
21	100	100			UCS = 778.07 MPa	0	121.4 - 121.6 Uniaxial.
16	100	100	122			3f	
16	93	100			UCS = 2.97 MPa	3f	123.8 - 124.0 Uniaxial.
16	50	100	124			7f	
21	59	100	126		UCS = 6.3 MPa	9f	126.4 - 126.7 Uniaxial.
	50	100	128				128.5 1 cm thick chlorite seam.
	62	95	130		Packer FHT		129.5 - 132.5 Packer test - FHT.
	42	100			K = 1.39 x 10^{-7} m/s		
17	58	100					131.9 P.A.: Andesite breccia.
			132		P.A.		132.0-132.5 Breccia zone.
7	13	79			UCS = 10.24 MPa	11f	132.1 - 132.3 Uniaxial.
25	0	76	134				
16	35	88				15	135.0-135.6 Breccia zone recemented with chlorite.
21	0	49	136				
27	53	86	138			3	138.2 - 138.9 Gravel to cobble size agglomerate.
33	29	82			UCS = 7.01 MPa	5	139.0-139.2 Uniaxial.
			140				

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor COATES	Logged by P.P./N.G.M.	Remarks:
Date started 16-6-82	Checked by D.F.	**Golder Associates**
Date finished 24-6-82	Date 27-7-82	Scale: 1 : 100 metric

Type of drilling ROTARY CORE/POLYMER FLUSH
Coordinates 5624713.3 N
600362.3 E
Rig LONGYEAR 44
Dip 60^{0}
Bit HUDDY,"BLACK" DIAMOND IMPREGNATED
Azimuth N73^{0}E

DRILLHOLE No. 82-923

Sheet 8 of 13

Location HAT CREEK EASTERN ESCARPMENT

Reference elevation DRILL FLOOR

Rate of Advance min./m	R.Q.D.	Core Recovery %	Depth	Reduced Level	Test Results	Fracture Index	Description
							ANDESITE Cont'd.
22	0	100				5	
20	7	92	142	972.1		14	
14	50	100				4	142.3 - 169.1 Coarse sand to cobble size andesite clasts in a chloritic matrix; ANDESITE BRECCIA.
19	72	100	144		UCS = 20.39 MPa	4	144.8 - 145.1 Uniaxial.
17	99	100	146			3	
24	95	100	148		UCS = 23.04 MPa	5	147.1 - 147.2 Uniaxial.
26	59	90			UCS = 11.81 MPa	8	149.1 - 149.3 Uniaxial.
19	82	100	150		UCS = 11.87 MPa	5	150.9 - 151.1 Uniaxial.
33	100	79				3f	
46	0	50	152		S.D. = 94.4%		152.0 - 152.1 Slake-durability test.
11	25	50				1f	
34	90	100				2f	
15	73	100	154		UCS = 13.86 MPa	2f	154.2 - 154.5 Uniaxial. 154.8 - 155.1 Highly fractured.
16	72	100	156			4f	
17	77	100	158			5f	158.0-158.1 Chloritic seam and alteration zone.
17	83	100			UCS = 14.97 MPa	6f	158.2 - 158.4 Uniaxial.
			160				

Contractor COATES
Date started 16-6-82
Date finished 24-6-82
Logged by P.P./N.G.M.
Checked by D.F.
Date 27-7-82
Remarks:

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE/POLYMER FLUSH	Coordinates 5624713.3 N 600362.3 E	DRILLHOLE No. 82-923	Sheet 9 of 13
Rig LONGYEAR 44	Dip 60^0	Location HAT CREEK EASTERN ESCARPMENT	
Bit HUDDY,"BLACK" DIAMOND IMPREGNATED	Azimuth N73^{0}E	Reference elevation DRILL FLOOR	

Rate of Advance min./m	R.Q.D.	Core Recovery %	Depth	Reduced Level	Test Results	Bed./Fol.	Fracture Index	Description
								ANDESITE BRECCIA Cont'd.
14	53	97			UCS = 13.39 MPa		6f	161.3 - 161.5 Uniaxial.
11	61	92	162				7f	162.7 - 162.8 Intensely fractured zone; recemented and altered.
18	67	100	164				5f	
16	83	100			UCS = 40.57 MPa		4f	165.9 - 166.1 Uniaxial.
10	70	100	166		Packer FHT P.A. K=4.63 x 10^{-6} m/s		7f	166.1 - 167.6 Packer test - FHT. 166.7 - 166.8 P.A.: Vesicular andesite.
10	100	100	168	945.3			3f	
17	95	100	170	944.4	UCS = 66.17 MPa		4f	169.1-170.0 Slightly weathered, massive; dark green/grey; very fine grained; vesicular ANDESITE; locally aphanitic; chloritic. 169.8 - 170.0 Uniaxial.
	100	100	172		S.D. = 89.1%		3f	170 - 178 Slightly weathered, medium to fine grained; ANDESITIC TUFF; locally exhibits graded beds. 171.7 - 171.9 Slake-durability test.
16	95	100	174		UCS = 15.75 MPa		3f	173.2 - 173.5 Uniaxial.
18	100	100			UCS = 7.43 MPa	46° 50°	2f	174.8 - 175.0 Uniaxial.
16	100	100	176				2f	
18	50	100	178	936.4			9	178.0-219.7 Slightly weathered, massive to finely bedded; light grey to green; medium-fine grained TUFF/TUFFACEOUS SANDSTONE.
18	77	100	180		UCS = 3.18 MPa		5	179.4 - 179.6 Uniaxial.

Contractor COATES	Logged by N.G.M.	Remarks:	
Date started 16-6-82	Checked by D.F.	**Golder Associates**	Scale: 1 : 100 metric
Date finished 24-6-82	Date 27-7-82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH	Coordinates: 5624713.3 N, 600362.3 E	DRILLHOLE No. 82-923	Sheet 10 of 13
Rig: LONGYEAR 44	Dip: 60°	Location: HAT CREEK EASTERN ESCARPMENT	
Bit: HUDDY,"BLACK" DIAMOND IMPREGNATED	Azimuth: N73°E	Reference elevation: DRILL FLOOR	

Rate of Advance min./m	R.Q.D.	Core Recovery %	Depth	Test Results	Bed./Fol.	Fracture Index	Description
							TUFFACEOUS SANDSTONE Cont'd.
10	100	100			49°	4f	
16	79	100	182	UCS = 13.94 MPa		4f	182.5 - 182.7 Uniaxial.
14	88	94	184			2f	
13	65	65				1f	
14	40	100	186	UCS = 1.59 MPa Packer FHT K = 3.48 x 10^{-8} m/s		7f	186.2 - 187.8 Highly weathered zone. 186.2 - 186.4 Uniaxial. 186.5 - 189.5 Packer test - FHT.
9	93	100	188			1f	
8	100	100	190			0f	189.3 - 190.8 Stratification, cross bedding, graded bedding.
7	95	100		UCS = 1.59 MPa		2f	190.8 - 191.1 Uniaxial.
6	100	100	192		58°	0f	
8	97	97	194			0f	
7	100	100	196	P.A. UCS = 0.78 MPa		0f	195.4 - 195.6 P.A.: Feldspathic Wacke (sandstone). 196.7 - 196.9 Uniaxial.
6	100	100	198			0f	
6	81	94	200	UCS = 2.93 MPa		2f	199.3 - 199.5 Uniaxial.

Contractor: COATES	Logged by P.P./N.G.M.	Remarks:
Date started: 16-6-82	Checked by D.F.	Golder Associates
Date finished: 24-6-82	Date 27-7-82	Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH	Coordinates: 5624713.3 N / 600362.3 E	DRILLHOLE No. 82-923	Sheet 11 of 13
Rig: LONGYEAR 44	Dip: 60^{0}	Location: HAT CREEK EASTERN ESCARPMENT	
Bit: HUDDY, "BLACK" DIAMOND IMPREGNATED	Azimuth: N 73^{0}E	Reference elevation: DRILL FLOOR	

Drilling Progress	Rate of Advance min./m (10, 20)	R. Q. D. (0, 50)	Core Recovery % (0, 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												TUFFACEOUS SANDSTONE Cont'd.
	7	100	100						0f			
	10	83	83	202					1f			
	11	83	100	204			UCS = 2.38 MPa		5f			203.4 - 203.6 Uniaxial.
	16	50	100				S.D. = 49.9% W% = 19.4		7f			204.9 - 205.1 Slake-durability test. 205.5 MC
	22	83	100	206					1f			
	12	73	100	208			UCS = 1.91 MPa		3f			207.0 - 207.3 Uniaxial.
	13	90	100									
	11	100	100	210			W% = 24.2 UCS = 1.18 MPa		2f			210.0 MC 210.6 - 210.7 Uniaxial.
	15	100	100	212			W% = 20.6 UCS = 4.73 MPa		4f			212.0 MC 212.3 - 212.4 Uniaxial.
	15	100	100	214					3f			
	12	100	100				W% = 20.6 UCS = 3.15 MPa		1			214.4 MC 215.8 - 216.0 Uniaxial.
	20	100	100	216				49^{0}	2			217.4 - 217.8 Soft Breccia zone. 218.0 MC 218.0 - 218.1 ATT
	16	98	99	218			W% = 32.5 LL=91.3 PL=33.8 PI=57.5	72^{0}	5			218.1 - 218.3 Uniaxial. 219.7 - 230.7 Slightly weathered, massive; brown and light grey; moderately weak; SILTY SANDSTONE; locally, medium-coarse sandstone.
				220	894.7		UCS = 1.66 MPa					

Contractor: COATES	Logged by: N.G.M.	Remarks:
Date started: 16-6-82	Checked by: D.F.	Golder Associates
Date finished: 24-6-82	Date: 27-7-82	Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH
Coordinates: 5624713.3 N, 600362.3 E
Rig: LONGYEAR 44
Dip: 60°
Bit: HUDDY, "BLACK" DIAMOND IMPREGNATED
Azimuth: N73°E

DRILLHOLE No. 82-923

Sheet 12 of 13

Location: HAT CREEK EASTERN ESCARPMENT
Reference elevation: DRILL FLOOR

Rate of Advance min./m	R.Q.D.	Core Recovery %	Depth	Reduced Level	Test Results	Bed./Fol.	Fracture Index	Description
								SILTY SANDSTONE Cont'd.
14					S.D. = 58.2%; UCS = 2.99 MPa		3f	220.4-220.6 Slake durability. 221.3 - 221.6 Uniaxial and ATT.
	100	100	222		LL 84.7; PL 40.0; PI 44.7	80°	3f	222 - 222.2 Medium grained, Andesitic tuff.
18	30	52				64°		
	85	85	224		W% = 33.3; LL 88.2; PL 40.3; PI 47.9		0f	224.4 - 224.5 ATT and MC.
19	67	83	226				2f	
20	83	100			UCS = 1.18 MPa		6f	227 - 227.2 Uniaxial.
22	100	100	228		W% = 27.4		4f	228.6 MC
17	83	83	230	883.7	UCS = 4.10 MPa; LL 52.1; PL 30.7; PI 21.4		3f	230.4 - 230.6 ATT and Uniaxial. 230.6 - 230.7 Slake-durability test.
11	59	100			S.D. = 93.4%; W% = 32.2		6f	230.7 - 250.3 Slightly weathered, massive and jointed/light to dark grey, LITHIC BRECCIA; coarse sand to cobble sized, subrounded to angular clasts in a fine grained matrix; occasional fine sandstone/siltstone horizons.
17	60	100	232				6f	231.0 MC
12	67	100	234				5f	
13	71	100	236		P.A.		6f	236.1 - 236.2 P.A.: Altered vesicular andesite breccia. 236.5 - 236.6 Very soft; highly weathered; clay.
16	99	100			UCS = 53.17 MPa		1f	237.0 - 237.2 Uniaxial.
13	100	100	238				2f	
			240					

Contractor: COATES
Date started: 16-6-82
Date finished: 24-6-82
Logged by: P.P./N.G.M.
Checked by: D.F.
Date: 27-7-82
Remarks:

Golder Associates

Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE/POLYMER FLUSH | Coordinates: 5624713.3 N, 600362.3 E | **DRILLHOLE No. 82-923**

Sheet 13 of 13

Rig: LONGYEAR 44 | Dip: 60° | Location: HAT CREEK EASTERN ESCARPMENT

Bit: HUDDY,"BLACK" DIAMOND IMPREGNATED | Azimuth: N73°E | Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
												LITHIC BRECCIA Cont'd.
	15	100	100				UCS = 32.30 MPa		3f			240.1 - 240.3 Uniaxial.
	19	100	100	242					1f			
	15	84	100				UCS = >70.89 MPa Pt.load		6f			243.3 - 243.5 Uniaxial and pt. load.
	18	96	100	244			Is 50 = 6.4 MPa Packer FHT		7f			244.2 - 250.0 Packer test - FHT.
	11	97	100	246			K = 4.8 x 10^{-7} m/s		5f			247.0 - 247.2 Uniaxial.
	12	99	100	248			UCS = 66.17 MPa		2f			
	9	95	95	250			UCS = 55.4 MPa		0			250.0 - 250.2 Uniaxial.
												END OF HOLE.

Contractor: COATES | Logged by: P.P./N.G.M. | Remarks:

Date started: 16-6-82 | Checked by: D.F.

Date finished: 24-6-82 | Date: 27-7-82

Golder Associates | Scale: 1 : 100 metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

DRILLHOLE No. 82-924

Sheet 1 of 13

Type of drilling: ROTARY CORE POLYMER FLUSH

Coordinates: 5624768.5 N / 600544.9 E

Rig: LONGYEAR 44

Dip: 60°

Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT (75 SERIES) FACE DISCHARGE

Azimuth: N 75° E

Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10 20)	R. Q. D. % (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instrumentation (1 2)	Legend	Description
				0								Tricone to 12.19.
				2								
				4								
				6								
				8								
				10								
				12	1133.0				1			12.2-59.2: Highly weathered; light grey; weak; BRECCIA; 0.5-1.0 cm clasts in a coarse sand to clay matrix; elongate clasts exhibit a preferred orientation; locally contains coarse grain sandstone.
	15	93	93						0			
		93	93	14			W% = 19.4 UCS = 0 MPa		0			13.4-13.7: M.C. and Uniaxial.
									0			
	11	100	100	16				56°	1			
	9	100	100					65°	2			
				18					0			
	8	100	100	20				55°	2			

Contractor: D.W. COATES

Logged by: PP/NGM

Remarks:

Date started: 27 JUNE 82

Checked by: DF

Date finished: 8 JULY 82

Date: 27 JULY 82

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH
Coordinates: 5624768.5 N / 600544.9 E
DRILLHOLE No. 82-924
Sheet 2 of 13

Rig: LONGYEAR 44
Dip: 60°
Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT (75 SERIES) FACE DISCHARGE
Azimuth: N 75° E
Reference elevation: DRILL FLOOR

Rate of Advance Min./m	R.Q.D. %	Core Recovery %	Depth m	Water Level	Test Results	Description
						BRECCIA cont'd
7	80	82	20–22		UCS = 0.40 MPa	20.1-20.4: Uniaxial.
9	88	100	22			
8	90	90	24		W% = 16.7	23.2: M.C.
9	100	100	26			26.0-26.2: Uniaxial.
6	93	93		PZ #2 26.8 m 8/24/82; PZ #1 27.2 m 8/24/82	UCS = 0 MPa	
5	100	100	28			
5	88	92	30			
5	82	99	32		W% = 13.1	31.2: M.C.
5	100	100				
5	100	100	34		UCS = 2.74 MPa	34.2-34.4: Uniaxial.
4	87	87	36			
5	100	100	38			
6	81	81	40			38.8-39.1: Claystone bed; gradational lower contact, sharp upper contact.

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor: D.W. COATES
Date started: 27 JUNE 82
Date finished: 8 JULY 82
Logged by: PP/NGM
Checked by: DF
Date: 27 JULY 82
Remarks:
Golder Associates
Scale: metric

Type of drilling: ROTARY CORE POLYMER FLUSH

Coordinates: 5624768.5 N, 600544.9 E

DRILLHOLE No. 82-924

Sheet 3 of 13

Rig: LONGYEAR 44

Dip: 60°

Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT (75 SERIES) FACE DISCHARGE

Azimuth: N 75° E

Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10, 20)	R. Q. D. % (0, 50)	Core Recovery % (0, 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instrumentation (1, 2)	Legend	Description
												BRECCIA cont'd
	5	100	100						1f 0.7			
	5	60	60	42					L.C. 3f 3.0			43.0: M.C.
	5	79	100	44			W% = 20.2 UCS = 1.43 MPa		6f 3.9			43.7-43.9: *Uniaxial.*
	6	79	79						L.C. 0f			
	8	100	100	46					2f 1.4			
	7	72	72	48					0 L.C.			49.9: M.C.
	7	100	100	50			W% = 20.2		1f .07			
	7	89	100						8f 5.3			
	5	97	100	52				50° 30°	3f 2.0			
	7	59	75	54				40° 30°	L.C. 4f 3.6			
	13	63	87	56				42°	7f 4.6			56.7-57.0: Uniaxial.
	8	53	100				UCS = 7.95 MPa W% = 16.7	27° 33° 27°	7f			57.0: M.C. 57.4-57.5: Dark green *tuff bed.*
	8	37	67	58	1086.0			33° 27°	5f			59.2-59.7: Slightly weathered; jointed; gray-green; fine grained; mod. strong; TUFFACEOUS FINE SANDSTONE/SILTSTONE; *contains* local coarse gravel beds.
				60								

Contractor: D.W. COATES

Date started: 27 JUNE 82

Date finished: 8 JULY 82

Logged by: PP/NGM

Checked by: DF

Date: 27 JULY 82

Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH
Coordinates: 5624768.5 N, 600544.9 E
DRILLHOLE No. 82-924
Sheet 4 of 13

Rig: LONGYEAR 44
Dip: 60°
Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT (75 SERIES) FACE DISCHARGE
Azimuth: N 75° E
Reference elevation: DRILL FLOOR

Rate of Advance Min./ (10–20)	R.Q.D. %	Core Recovery %	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Description
				1085.5					
8	56	72						3f	59.7-67.4: Moderately weathered; massive; jointed; green; fine-coarse gravel size clasts; moderately strong; LITHIC BRECCIA.
9	80	100	62					3f	62.8-63.1: Uniaxial.
9	73	100				UCS = 1.19 MPa		1f	63.2-63.3: Highly fractured.
9	73	100	64					4f	
8	60	94	66	1077.8		UCS = 0.40 MPa		2f	66.2-66.3: Clay gouge zone. 67.0-67.2: Uniaxial. 67.3: M.C.
12	73	100	68			W% = 11.3		5f	67.4-78.0: Moderately weathered; massive; jointed; light gray; fine grained; moderately weak; SANDY CLAYSTONE/CLAYEY SANDSTONE.
9	77	100	70				56°, 22°	2f	
9	67	100					8°, 12°	6f	
8	88	91	72			UCS = 1.18 MPa	8°	2f	72.2-72.4: Uniaxial.
9	53	100	74			W% = 15.7	12°, 6°	8f	74.1: M.C.
9	60	100	76					10f	75.9-77.3: Lithic breccia.
9	47	100		1067.2			14°	5f	
10	69	88	78			UCS = 0		3f	78.0-86.3: Moderately weathered; massive and jointed; green-gray; gravel-cobble sized clasts; moderately weak; LITHIC BRECCIA; fine grained tuffaceous matrix; bentonitic.
			80						

Contractor: D.W. COATES
Date started: 27 JUNE 82
Date finished: 8 JULY 82
Logged by: PP/NGM
Checked by: DF
Date: 27 JULY 82
Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5624768.5 N / 600544.9 E		**DRILLHOLE No. 82-924** Sheet 5 of 13
Rig: LONGYEAR 44	Dip: 60°		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT (75 SERIES) FACE DISCHARGE	Azimuth: N 75° E		Reference elevation: DRILL FLOOR

LITHIC BRECCIA cont'd

Rate of Advance Min./m	R.Q.D. %	Core Recovery %	Fracture Index
11	68	100	1
7	63	100	1, 3
9	33	90	4
11	50	72	3, 1
17	43	93	5
32	79	100	7, 2
	60	60	2
			L.C.
10	82	82	0, 3
15	52	100	6
	98	100	2
10	100	100	2, 1
9	64	100	4
15	100	100	4
13	86	100	3
6	63	80	5

Depth m	Reduced Level	Test Results	Bed./Fol.
~84		W% = 23.6	
~85		UCS = 1.79 MPa P.A.	
86.3	1058.9		
~87		W% = 14.9	
~88.3			80°
~90		UCS = 6.43 MPa	
~93.5		W% = 9.3	
94.0	1051.2		68°
~95			89°
~96		UCS = 7.72 MPa	
~97			84°
~98.2			78°
~99.7		W% = 13.0	

Depth scale: 82, 84, 86, 88, 90, 92, 94, 96, 98, 100

Description

83.4: M.C.

84.9-85.1: P.A.: Altered crystal-lithic tuff-breccia.

85.1-85.3: Uniaxial.

86.3-94.0: Slightly weathered; fissured; dark brown-gray; very fine grain, moderately strong; CLAYSTONE/SILTSTONE; locally grades to fine sandstone.
86.8: M.C.
86.9-87.1: Shear/gouge zone; slickensided clay surfaces.
88.2-89.0: Very fine sand and silt laminae (average 1 mm thick).
89.0-89.4: Occasional clasts; numerous shears; cross bedded.
90.0-90.2: Uniaxial.
92.9: M.C.

94.0-130.7: Slightly weathered; finely bedded; light-dark gray; fine grained; moderately weak; SANDSTONE; numerous thin silt and clay beds; occasional cross beds, beds locally sheared and offset.
95.4-95.6: Uniaxial.
99.4: M.C.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:
Date started: 27 JUNE 82	Checked by: DF	**Golder Associates**
Date finished: 8 JULY 82	Date: 27 JULY 82	Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH

Coordinates: 5624768.5 N / 600544.9 E

DRILLHOLE No. 82-924

Sheet 6 of 13

Rig: LONGYEAR 44

Dip: 60°

Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT (75 SERIES) FACE DISCHARGE

Azimuth: N 75° E

Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10 20)	R.Q.D. % (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation (1 2)	Legend	Description
												SANDSTONE cont'd
	13	43	80	102				85°	8, 8			
	10	100	100				UCS = 5.47 MPa	84°	3, 2			103.2-103.4: Uniaxial.
	9	93	100	104					2			
		86	86	106				72°	2, 3			107.2: M.C.
	7	89	100	108			W% = 13.0		5, 1			
	6	90	90						1			
	9	59	100	110				73°	6, 3			110.8-111.1: Claystone bed.
	10	80	80	112			UCS = 8.48 MPa, LL 42.9, PL 26.3, PI 16.6		2			111.7-114.3: Brownish gray; moderately strong; claystone bed. 112.5-112.8: ATT. and Uniaxial.
	27	27	100				W% = 10.5		22			113.1: M.C.
	8	82	82	114				70°	0, 1			
	7	100	100	116					5			
	7	100	100	118				80°	3, 2			
	11	43	93	120				65° TO 75°	6			120.9-121.0: Uniaxial.

Contractor: D.W. COATES

Date started: 27 JUNE 82

Date finished: 8 JULY 82

Logged by: PP/NGM

Checked by: DF

Date: 27 JULY 82

Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE POLYMER FLUSH	Coordinates 5624768.5 N / 600544.9 E	DRILLHOLE No. 82-924	Sheet 7 of 13
Rig LONGYEAR 44	Dip 60°	Location HAT CREEK EASTERN ESCARPMENT	
Bit PILOT (75 SERIES) FACE DISCHARGE	Azimuth N 75° E	Reference elevation DRILL FLOOR	

Drilling Progress	Rate of Advance Min./m	R. Q. D. %	Core Recovery %	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
	10 20	0 50	0 50							1 2		SANDSTONE cont'd
	10	11	100				UCS = 6.26 MPa W% = 10.5	70°	6			121.2-121.5: Claystone/ siltstone bed. 121.4: M.C.
	11	33	100	122				72° 68°	9 7			122.0-124.7: Laminated gray-green fine sandstone and dark gray-brown siltstone (beds generally 0.5-1.0 cm thick); laminae offset by small faults; stringers of carbonaceous material.
	11	59	100	124				75°	5 3			
	9	63	100	126				68°	5			
	9	47	91					68° 72°	4 5			
	13	70	100	128			UCS = 0 MPa W% = 8.7	68°	5 1			129.5-129.7: M.C. and Uniaxial.
	11	53	81	130	1014.5			70°	7			130.7-250.2: Slightly weathered; jointed; dark gray; very fine grained; moderately strong; CLAYSTONE/SILTSTONE; locally contains thin interbeds of fine sandstone.
	10	67	100	132				75° 65°	5 2			
	15	87	90					80°	1			
	18	43	100	134			W% = 9.5		4 4			134.7-135.0: M.C. and Uniaxial. 135.9-136.1: ATT.
	17	72	100	136			LL 44.5 PL 26.3 PI 18.2 UCS = 11.13 MPa		4			
	21	33	100	138					5 5			138.0-138.7: Core moderately fractured.
	13	57	57	140			Packer FHT		2			138.7-141.1: Packer test - FHT.

Contractor D.W.COATES	Logged by PP/NGM	Remarks:	
Date started 27 JUNE 82	Checked by DF	**Golder Associates**	Scale: metric
Date finished 8 JULY 82	Date 27 JULY 82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5624768.5 N / 600544.9 E	DRILLHOLE No. 82-924	Sheet 8 of 13
Rig: LONGYEAR 44	Dip: 60°	Location: HAT CREEK EASTERN ESCARPMENT	
Bit: PILOT (75 SERIES) FACE DISCHARGE	Azimuth: N 75° E	Reference elevation: DRILL FLOOR	

Rate of Advance Min./m	R. Q. D. %	Core Recovery %
15	20	50
19	72	100
15	92	94
13	80	100
12	79	100
12	100	100
12	100	100
12	87	100
13	100	100
7	100	100
19	100	100
11	72	100
13	95	100
11	47	100

Depth m: 142, 144, 146, 148, 150, 152, 154, 156, 158, 160

Test Results:
K = 2.74 x 10^{-7} m/s
W% = 8.5, UCS = 7.41 MPa
W% = 8.3, UCS = 3.89 MPa, S.D. = 88.7%

Bed./Fol.: 84°, 80°, 82°, 80°, 82°, 80°, 73°, 60°

Fracture Index: 6, 7, 7, 3, 6, 4, 1, 3, 2, 3, 5, 3, 0, 4, 3, 5, 6, 2, 5, 7

Instrumentation: 1 2

Description:

CLAYSTONE/SILTSTONE cont'd

144.1-144.3: M.C. and Uniaxial.

155.9-156.1: M.C. and Uniaxial.

157.0: Slake durability.

158.8-160.3: Fossil bivalves.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:
Date started: 27 JUNE 82	Checked by: DF	
Date finished: 8 JULY 82	Date: 27 JULY 82	Golder Associates — Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE POLYMER FLUSH	Coordinates 5624768.5 N / 600544.9 E	DRILLHOLE No. 82-924 Sheet 9 of 13
Rig LONGYEAR 44	Dip 60°	Location HAT CREEK EASTERN ESCARPMENT
Bit PILOT (75 SERIES) FACE DISCHARGE	Azimuth N 75° E	Reference elevation DRILL FLOOR

Rate of Advance Min./m	R.Q.D. %	Core Recovery %	Depth m	Test Results	Bed./Fol.	Fracture Index	Description
							CLAYSTONE/SILTSTONE cont'd
11	94	100			60°, 57°	5	
					54°	3	
11	67	100	162		47°, 50°	4	
					40°	1	
11	73	100	164		52°	5	
12	40	100	166	W% = 8.0 UCS = 0.79 MPa	32°, 48°	7	165.8-166.0: M.C. and Uniaxial.
					40°	8	
16	64	80			50°	3	
11	26	100	168		45°, 57°	3	168.0-168.3: Clay gouge zone and highly fractured core.
						8	169.7-169.9: Highly fractured, fractures parallel-subparallel to bedding.
13	73	100	170		43°	5	
13	60	100	172		45°	4	
						3	
13	70	70			40°	1	
12	42	100	174		42°, 48°	8	
15	90	100	176	W% = 7.8 UCS = 2.38 MPa	45°	3	175.5-175.8: M.C. and Uniaxial.
						4	
	70	100	178		46°	6	
						7	
19	56	100	180		45°	10	

Contractor D.W. COATES
Date started 27 JUNE 82
Date finished 8 JULY 82
Logged by PP/NGM
Checked by DF
Date 27 JULY 82
Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling	ROTARY CORE POLYMER FLUSH	Coordinates	5624768.5 N / 600544.9 E	DRILLHOLE No. 82-924
Rig	LONGYEAR 44	Dip	60°	Sheet 10 of 13
Bit	PILOT (75 SERIES) FACE DISCHARGE	Azimuth	N 75° E	Location: HAT CREEK EASTERN ESCARPMENT
				Reference elevation: DRILL FLOOR

Rate of Advance Min./m	R. Q. D. %	Core Recovery %	Depth m	Test Results	Bed./Fol.	Fracture Index	Description
							CLAYSTONE/SILTSTONE cont'd
13	58	100				6	
						18	181.0-181.6: Zone of intense shearing.
19	72	73	182	W% = 8.8 UCS = 2.86 MPa	53°	5	182.6-182.7: Breccia zone, recemented. 182.7-183.2: Lost core. 182.6-182.7: M.C. and Uniaxial.
18	50	100	184			5	
						11	
18	46	100				7	
			186			4	
15	86	100				8	
19	53	100	188			6	
17	40	93			42°	7	
			190	W% = 10.0 UCS = 2.70 MPa	40°	4	190.4-190.7: M.C. and Uniaxial.
15	43	100			42°	7	
			192			4	192.1-192.2: Fractured zone.
21	50	100			41°	5	193.6-193.9: Fractured zone.
19	93	100	194		37°	2	
					40°	3	
16	87	100	196		40°	2	
15	73	73			45°	0	
			198	W% = 10.5 UCS = 3.8 MPa		1	198.1-198.3: M.C. and Uniaxial.
14	13	100	200			2	199.4-200.0: Shear zone.

Contractor	D.W. COATES	Logged by	PP/NGM
Date started	27 JUNE 82	Checked by	DF
Date finished	8 JULY 82	Date	27 JULY 82

Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5624768.5 N, 600544.9 E		DRILLHOLE No. 82-924
			Sheet 11 of 13
Rig: LONGYEAR 44	Dip: 60^{0}		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT (75 SERIES) FACE DISCHARGE	Azimuth: N 75^{0} E		Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10 20)	R. Q. D. % (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend
	18	23	87					40^{0}	7, 2		
	15	80	100	202				40^{0}	3		
	28	96	100	204			W% = 11.0 UCS = 2.98 MPa		1, 3		
		76	100						7		
	25	70	93	206				45^{0}	2, 9		
	20	27	100	208				47^{0}, 35^{0}	7		
	15	93	100	210			Packer FHT K = 3.67×10^{-8} m/s	60^{0}	2, 1		
	11	50	100						6		
	10	75	100	212			W% = 9.5 UCS = 0 MPa	45^{0}, 40^{0}	2, 3		
	13	87	100	214				55^{0}, 50^{0}	2		
	11	93	100	216				50^{0}	3, 4		
	21	50	100					45^{0}	3		
		50	100	218			ATT. LL 34.8 PL 20.6 PI 14.2		3		
		100	100	220			P.A.	52^{0}			

Description

CLAYSTONE/SILTSTONE cont'd

201.2-201.5: Shear zone; clay matrix.

202.2-202.3: Gray-green; fine-medium grain; sandstone bed.

203.4-203.6: M.C. and Uniaxial.

207.6-212.4: Packer test - FHT.

210.4-210.5: Gray-green; fine-medium grain; sandstone; cleaves readily along bedding planes.
210.5-210.8: Very hard, dark gray; silty clay gouge.
210.5-212.9: Extensively slickensided.

212.7-212.9: M.C. and Uniaxial.

213.7-215.2: Interbedded claystone and gray-green; fine-medium grain sandstone with carbonaceous partings.

217.6-218.2: Clay gouge and breccia; soft clay with angular fragments of claystone.
218.0: ATT.

219.7-219.8: P.A.: Carbon bearing calcareous wacke.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:	
Date started: 27 JUNE 82	Checked by: DF	**Golder Associates**	Scale: metric
Date finished: 8 JULY 82	Date: 27 JULY 82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5624768.5 N, 600544.9 E		DRILLHOLE No. 82-924 Sheet 12 of 13
Rig: LONGYEAR 44	Dip: 60°		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT (75 SERIES) FACE DISCHARGE	Azimuth: N 75° E		Reference elevation: DRILL FLOOR

Columns: Drilling Progress | Rate of Advance Min./m (10, 20) | R. Q. D. % (0, 50) | Core Recovery % (0, 50) | Depth m | Reduced Level | Water Level | Test Results | Bed./Fol. | Fracture Index | Instrumentation | Legend | Description

CLAYSTONE/SILTSTONE cont'd

Rate of Advance Min./m	R. Q. D. %	Core Recovery %
20	75	100
20	60	63
18	89	100
17	56	100
20	40	100
15	70	93
17	50	100
18	53	69
14	33	87
17	80	100
33	16	67
11	75	100
14	47	100
14	37	100

Depth m: 222, 224, 226, 228, 230, 232, 234, 236, 238, 240

Fracture Index (top to bottom): 7, 3, L.C., 1, 5, 5, 2, 8, 5, 3, 9, 4, 3, 9, 3, 6, 4, 4, 3, 5, 8

Bed./Fol. (top to bottom): 49°, 42°, 40°, 42°, 43°, 48°, 55°, 50°, 50°

Instrumentation: 1

Legend: L.C.

Test Results:

W% = 9.3
UCS = 0 MPa

W% = 9.4
UCS = 4.33 MPa

Description:

222.1-222.3: M.C. and Uniaxial.
222.3-222.8: Lost Core.

228.0: Fossil bivalves.

230.9-231.0: Breccia/shear zone.

231.5-231.6: Breccia/shear zone.

233.3-233.5: M.C. and Uniaxial.

235.1-235.3: Highly fractured zone.

236.8-237.1: Highly fractured zone.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:
Date started: 27 JUNE 82	Checked by: DF	Golder Associates
Date finished: 8 JULY 82	Date: 27 JULY 82	Scale: metric

PROJECT NO. ...... DRAWN ...... REVIEWED ...... DATE ......

Type of drilling ROTARY CORE POLYMER FLUSH

Coordinates 5624768.5 N / 600544.9 E

DRILLHOLE No. 82-924

Rig LONGYEAR 44

Dip 60°

Location HAT CREEK EASTERN ESCARPMENT

Bit PILOT (75 SERIES) FACE DISCHARGE

Azimuth N 75° E

Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance Min./m	R.Q.D. %	Core Recovery %	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instrumentation	Legend	Description
												CLAYSTONE/SILTSTONE cont'd
	14	70	70					58°	2			240.7: Calcite veining.
	15	33	80	242				58°	3			241.4-241.8: Highly fractured and sheared zone.
	18	93	93	244			W% = 9.3 UCS = 11.13 MPa	52°	0			243.0-243.2: M.C. and Uniaxial.
	28	93	100					53°	0			
	30	98	100	246				54°	2			
									2			
	24	100	100	248				55°	3			
	26	50	66	250	894.9		W% = 9.1 UCS = 8.84 MPa					249.4-249.6: M.C. and Uniaxial.
												250.3 END OF HOLE

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor D.W. COATES

Date started 27 JUNE 82

Date finished 8 JULY 82

Logged by PP/NGM

Checked by DF

Date 27 JULY 82

Remarks:

Golder Associates

Scale: metric

DRILLHOLE No. 82-925

Sheet 1 of 11

Type of drilling: ROTARY CORE POLYMER FLUSH
Coordinates: 5625656.8 N, 600512.4 E
Rig: LONGYEAR 44
Dip: 60°
Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT(75 SERIES)/HUDDY "BLACK"
Azimuth: N 75° E
Reference elevation: DRILL FLOOR

Rate of Advance Min./m	R. Q. D.	Core Recovery %
11	53	67
14	84	90
22	55	89
20	0	50
18	0	80
22	0	39
21	0	20
33	50	63
28	60	77
19	21	50
21	0	75
20	19	85
20	27	83
15	28	92
32	30	100
30	60	100

Depth (m): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Reduced Level: 1093.0; 1090.2

Test Results: W% = 14.6, UCS = 0 MPa; UCS = 0.39 MPa

Bed./Fol.: 22°; 21°; 24°; 23°

Fracture Index: 0, 2, 1, 2, 1, 6, 7, 0, 0, 0, 0, 0, 1, 5, 6, 5, 3, 9, 6

Instrumentation: 1 2

Description

Tricone to 1.9 m: Sand, gravel, cobbles in silty clay matrix.

1.9-4.8: Weathered; very stiff to hard; brown; coarse SAND/SANDSTONE with silt/clay matrix; locally contains gravel and cobbles.

4.0-4.2: M.C. and Uniaxial.

4.8-90.4: Moderately weathered; highly fractured; gray; very fine grained; moderately strong; ANDESITE; locally highly altered and brecciated; most fractures and joints are iron oxide stained and often clay filled.
- 4.8-5.2: Highly fractured zone.
- 7.1-7.2: Breccia zone; clay matrix.
- 7.9-8.8: Highly fractured zone.

10.3-13.6: Highly altered and brecciated zone; angular andesite fragments in soft to firm yellow gray clay matrix.

15.6-15-8: Uniaxial.

17.2-17.5: Highly fractured zone; clay infillings.

19.3: Fault breccia.

Contractor: D.W. COATES
Date started: 10 JULY 82
Date finished: 21 JULY 82
Logged by: PP/NGM
Checked by: DF
Date: 20 AUG 82
Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5625656.8 N / 600512.4 E	DRILLHOLE No. 82-925 Sheet 2 of 11
Rig: LONGYEAR 44	Dip: 60°	Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT (75 SERIES)/HUDDY "BLACK"	Azimuth: N 75° E	Reference elevation: DRILL FLOOR

Rate of Advance Min./m	R. Q. D.	Core Recovery %	Depth m	Water Level	Test Results	Bed./Fol.	Fracture Index
18	6	89					8
10	16	60	22				6
19	43	85	24		UCS = 10.59 MPa	28°	8
17	16	54		PZ. #1			
22	11	66	26	25.3 m 8/24/82			
16	0	75					
20	0	52	28				6
19	0	100	30				10
							10
18	20	76	32				10
13	40	100			UCS = 63.59 MPa	15°	7
							9
14	22	95	34			19°	5
							9
19	23	100	36			15°	6
17	13	73			UCS = 21.86 MPa	16°	2
16	56	100	38				7
21	75	100	40				6

Description

ANDESITE cont'd

21.9-22.7: Fault breccia.

23.1-23.2: Uniaxial.

27.3-29.4: Highly fractured, brecciated, and altered zone; abundant clay in fractures.

32.5-32.7: Uniaxial.

34.3-34.4: Breccia zone.

36.9-37.1: Uniaxial.

37.5-40.2: Breccia zone and clay gouge; locally highly altered; some clasts exhibit well developed alteration rims.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:
Date started: 10 JULY 82	Checked by: DF	
Date finished: 21 JULY 82	Date: 20 AUG 82	Scale: metric

Golder Associates

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE POLYMER FLUSH	Coordinates 5625656.8 N / 600512.4 E	DRILLHOLE No. 82-925	Sheet 3 of 11
Rig LONGYEAR 44	Dip 60°	Location HAT CREEK EASTERN ESCARPMENT	
Bit PILOT(75 SERIES)/HUDDY "BLACK"	Azimuth N 75° E	Reference elevation DRILL FLOOR	

Drilling Progress	Rate of Advance Min./m (10 20)	R.Q.D. (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation (1 2)	Legend	Description
												ANDESITE cont'd
	19	46	100					18°	9			41.5-41.6: Breccia and clay gouge zone.
	13	50	100	42					4			42.0-44.5: Breccia zone; alteration rims.
	12	31	100			PZ. #2			6			
	12	25	88	44		43.3 m 8/24/82			8			
	17	33	100						6			
	18	44	94	46					6			
	20	23	100	48					9			
									7			
	15	9	94	50					7			
									2			49.5-50.6: Breccia zone with iron oxide stained clay infillings and matrix.
	16	77	100						3			
	11	26	100	52			UCS = 28.10 MPa		7			51.8-51.9: Uniaxial.
							S.D. 94.0%		5			53.0-53.2: Slake durability.
	14	20	90	54					8			
								10°	8			
	10	40	100	56			UCS = 35.13 MPa	15°	8			55.4-55.5: Breccia zone with clay gouge.
												55.9-56.0: Uniaxial.
	11	74	100						4			56.9-57.0: Breccia zone and clay gouge.
				58					2			57.6-58.0: Breccia zone and clay gouge.
	12	11	82						7			58.7-59.2: Breccia zone and clay gouge.
	13	61	94	60					6			

Contractor D.W. COATES	Logged by PP/NGM	Remarks:	
Date started 10 JULY 82	Checked by DF	**Golder Associates**	Scale: metric
Date finished 21 JULY 82	Date 20 AUG 82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5625656.8 N / 600512.4 E		DRILLHOLE No. 82-925 Sheet 4 of 11
Rig: LONGYEAR 44	Dip: 60°		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT(75 SERIES)/HUDDY "BLACK"	Azimuth: N 75° E		Reference elevation: DRILL FLOOR

Columns: Drilling Progress | Rate of Advance Min./m (10, 20) | R. Q. D. (0, 50) | Core Recovery % (0, 50) | Depth m | Reduced Level | Water Level | Test Results | Bed./Fol. | Fracture Index | Instru-mentation (1 2) | Legend | Description

ANDESITE cont'd

Rate of Advance Min./m	R. Q. D.	Core Recovery %
15	63	100
26	16	60
20	35	100
17	50	100
32	0	78
32	0	75
13	40	100
13	22	98
12	24	58
32	0	97
30	30	76
28	36	98
27	20	100
19	59	86
19	47	100
24	17	80
17	47	94
23	0	22

Depth (m): 62, 64, 66, 68, 70, 72, 74, 76, 78, 80

Fracture Index: 5, 8, 8, 8, 10, 5, 10, 7, 10, 5, 6, 9, 9, 5, 4, 3, 3, 7, 5

Bed./Fol.: 36° (approx. 61.6 m); 38° (approx. 67.3 m)

Test Results	Description
UCS = 62.45 MPa	62.9-63.1: Uniaxial.
UCS = 5.59 MPa	68.9-69.1: Uniaxial.
	70.8-71.3: Breccia zone with clay matrix.
	72.4-72.9: Breccia zone with clay matrix.
	75.3-75.9: Breccia zone with chlorite matrix.
UCS = 1.57 MPa	76.4-76.6: Uniaxial.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks: Hole caving; cemented interval 62.5 m - 80.8 m.
Date started: 10 JULY 82	Checked by: DF	
Date finished: 21 JULY 82	Date: 20 AUG 82	Golder Associates — Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE POLYMER FLUSH	Coordinates 5625656.8 N 600512.4 E	DRILLHOLE No. 82-925	Sheet 5 of 11
Rig LONGYEAR 44	Dip 60°	Location HAT CREEK EASTERN ESCARPMENT	
Bit PILOT(75 SERIES)/HUDDY "BLACK"	Azimuth N 75° E	Reference elevation DRILL FLOOR	

Drilling Progress	Rate of Advance Min./m	R. Q. D.	Core Recovery %	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instrumentation	Legend	Description
	10 20	0 50	0 50							1 2		ANDESITE cont'd
	23	0	22									
	5	0	20									
	30	0	30	82								
	20	0	42									
	15	13	60						1			
									4			
	16	0	89	84					7			
	27	0	100						7			85.6-85.8: Breccia zone with chlorite matrix; recemented.
	30	0	82	86								
	47	0	93									87.4-87.6: Uniaxial and P.A., P.A.: Pyroxene andesite.
	27	31	92				P.A. UCS = 2.76 MPa		7			87.8-88.0: Chloritic gouge zone.
	18	65	88	88					6			
									4			89.6-89.8: Recemented chloritic breccia.
	14	68	100	90	1004.6				7			90.4-115.5: Recemented chloritic ANDESITE BRECCIA.
									5			
	15	53	100	92					8			
	15	43	100						6			
	18	0	26	94								
	29	0	50	96								
	20	0	50									
	20	0	20	98								
	7	0	1	100								

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor D.W. COATES	Logged by PP/NGM	Remarks:
Date started 10 JULY 82	Checked by DF	Golder Associates
Date finished 21 JULY 82	Date 20 AUG 82	Scale: metric

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5625656.8 N, 600512.4 E		DRILLHOLE No. 82-925, Sheet 6 of 11
Rig: LONGYEAR 44	Dip: 60°		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT(75 SERIES)/HUDDY "BLACK"	Azimuth: N 75° E		Reference elevation: DRILL FLOOR

Rate of Advance Min./m (10, 20)	R.Q.D. (0, 50)	Core Recovery % (0, 50)
7	0	1
15	13	50
11	40	79
13	63	93
8	23	60
11	74	92
11	28	59
11	54	87
12	0	30
16	0	50
17	69	100
13	22	100
16	25	74
14	13	86
20	38	100
15	63	92

Columns: Drilling Progress | Rate of Advance Min./m | R. Q. D. | Core Recovery % | Depth m | Reduced Level | Water Level | Test Results | Bed./Fol. | Fracture Index | Instru-mentation (1 2) | Legend | Description

Depth m: 102, 104, 106, 108, 110, 112, 114, 116, 118, 120

Reduced Level: 979.5 (at 115.5 m)

Test Results: UCS = 0 MPa (101.1–101.3); UCS = 1.18 MPa (114.4–114.6)

Fracture Index: L.C., 5, 7, L.C., 4, 3, 1, L.C., 3, 3, 4, 4, 10, 5, 8

Description

ANDESITE BRECCIA cont'd

100.0-100.8: Lost Core

101.1-101.3: Uniaxial.

105.5-106.3: Lost core.

108.5-109.2: Lost core.

114.4-114.6: Uniaxial.

115.5-137.5: Moderately weathered; highly fractured; gray; very fine grained; moderately strong; ANDESITE; locally highly brecciated.

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:	
Date started: 10 JULY 82	Checked by: DF	Golder Associates	Scale: metric
Date finished: 21 JULY 82	Date: 20 AUG 82		

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH
Coordinates: 5625656.8 N, 600512.4 E
Rig: LONGYEAR 44
Dip: 60°
Bit: PILOT(75 SERIES)/HUDDY "BLACK"
Azimuth: N 75° E

DRILLHOLE No. 82-925

Sheet 7 of 11

Location: HAT CREEK EASTERN ESCARPMENT
Reference elevation: DRILL FLOOR

Rate of Advance Min./	R.Q.D.	Core Recovery %	Depth m	Reduced Level	Test Results	Bed./Fol.	Fracture Index	Description
							3	ANDESITE BRECCIA cont'd
14	66	86	122				2	
17	0	50						
14	0	37	124				L.C.	122.8-123.8: Lost core.
							4	
							4	
18	0	0					L.C.	124.4-125.3: Lost core.
16	0	33	126					
16	47	100	128		P.A. UCS = 3.35 MPa		7	127.6-127.9: Uniaxial. 127.6-129.8: P.A.: Altered andesite breccia.
13	43	100					1	129.0-129.3: Highly fractured zone.
							5	
21	27	90	130				8	
29	47	100	132				7	
							9	
21	23	100	134				10	
26	40	100			UCS = 13.73 MPa	30°	3	
							7	
21	25	88	136	957.5			6	
12	70	100	138			38°	4	137.5-197.4: Slightly weathered; interbedded; gray-dark gray; very fine to fine grained; moderately strong; laminated SILTSTONE/CLAYSTONE and fine SANDSTONE; locally conglomeritic.
							6	
8	100	100	140				3	138.1-138.6: Slumped bedding and soft sediment deformation; displaced bedding.

Contractor: D.W. COATES
Date started: 10 JULY 82
Date finished: 21 JULY 82
Logged by: PP/NGM
Checked by: DF
Date: 20 AUG 82
Remarks:

Golder Associates

Scale: metric

PROJECT NO. ...... DRAWN ...... REVIEWED ...... DATE ......

Type of drilling ROTARY CORE POLYMER FLUSH | Coordinates 5625656.8 N 600512.4 E | **DRILLHOLE No. 82-925** Sheet 8 of 11

Rig LONGYEAR 44 | Dip 60° | Location HAT CREEK EASTERN ESCARPMENT

Bit PILOT(75 SERIES)/HUDDY "BLACK" | Azimuth N 75° E | Reference elevation DRILL FLOOR

Drilling Progress	Rate of Advance Min./ (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instrumentation (1 2)	Legend	Description
									2			CLAYSTONE/SILTSTONE/SANDSTONE cont'd 140.5-140.8: M.C. and Uniaxial.
	9	40	100				W% = 25.4 UCS = 1.99 MPa	40°	10			
	6	69	91	142				56°	5			
									5			
	8	40	100	144				47°	3			145.0-145.2: Conglomerate bed.
	7	83	100						2			
				146			W% = 22.1 UCS = 8.74 MPa		2			146.3-146.6: M.C. and Uniaxial.
	14	79	100						4			
	14	79	100	148					4			
							W% = 31.6 UCS = 8.70 MPa S.D. 79.4%		4			148.8-150.9: Chloritic claystone. 149.2-149.5: M.C. and Uniaxial,slake durability. 150.1-150.4: Zone of intense fracturing.
	14	0	80	150					19			
	13	26	100	152					5			
									2			
	13	100	100	154					2			
	13	91	91						2			
									4			
	13	97	100	156					3			
	13	88	100	158					5			
									4			158.5-163.9: Medium grained; green-gray; sandstone.
	9	100	100	160			W% = 23.7 UCS = 0.59 MPa		0			159.7-159.9: M.C. and Uniaxial.

Contractor D.W. COATES | Logged by PP/NGM | Remarks:

Date started 10 JULY 82 | Checked by DF

Date finished 21 JULY 82 | Date 20 AUG 82

Golder Associates | Scale: metric

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling ROTARY CORE POLYMER FLUSH
Coordinates 5625656.8 N
600512.4 E
Rig LONGYEAR 44
Dip 60°
Azimuth N 75° E
Bit PILOT(75 SERIES)/HUDDY "BLACK"

DRILLHOLE No. 82-925

Sheet 9 of 11

Location HAT CREEK EASTERN ESCARPMENT

Reference elevation DRILL FLOOR

CLAYSTONE/SILTSTONE/SANDSTONE cont'd

Rate of Advance Min./m	R.Q.D.	Core Recovery %	Depth m	Test Results	Bed./Fol.	Fracture Index	Description
6	92	92				1, 0	
7	100	100	162			2	
14	100	100	164			1, 2	
	100	100	165			3	
18	100	100			36°	0, 2	167.7-169.4: Slightly weathered; massive; jointed; pale-dark green; very fine grained to gravel size clasts; moderately strong; tuffaceous breccia.
12	50	97	168	P.A.	38°	4	168.0: P.A.: Crystal lithic, altered tuff-breccia. 168.2-168.4: Highly fractured.
35	50	83	170	W% = 39.5 UCS = 0 MPa		1	169.8-170.1: M.C. and Uniaxial.
37	44	100			36°	5	
37	17	88	172			9	
29	70	100				4, 3	172.2-174.4: Slightly weathered; massive; jointed; green; fine to coarse grained; moderately strong sandstone/conglomerate.
43	50	80	174				
41	27	100		S.D. 0%		6, 9	175.0: Slake durability.
40	33	80	176			7	
18	23	100	178	W% = 34.8 UCS = 2.47 MPa		8, 4	177.6-177.8: M.C. and Uniaxial.
24	79	92	180				

Contractor D.W. COATES
Date started 10 JULY 82
Date finished 21 JULY 82
Logged by PP/NGM
Checked by DF
Date 20 AUG 82

Remarks:

Golder Associates

Scale: metric

PROJECT NO. .... DRAWN ...... REVIEWED ...... DATE ....

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Type of drilling: ROTARY CORE POLYMER FLUSH
Coordinates: 5625656.8 N, 600512.4 E
DRILLHOLE No. 82-925
Sheet 10 of 11

Rig: LONGYEAR 44
Dip: 60°
Location: HAT CREEK EASTERN ESCARPMENT

Bit: PILOT(75 SERIES)/HUDDY "BLACK"
Azimuth: N 75° E
Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10, 20)	R. Q. D. (0, 50)	Core Recovery % (0, 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Instru-mentation	Legend	Description
											CLAYSTONE/SILTSTONE/SANDSTONE cont'd
	11	100	100								181.0-183.0: Occasional angular, gravel size clasts.
	14	98	98	182							
	6	100	100	184							
	11	100	100	186							
	15	98	100								186.9-187.9: Claystone dike like feature; very sharp but irregular contacts.
	11	86	86	188							188.0-189.7: Breccia; gravel size clasts in coarse sand matrix.
	16	86	100	190							189.7-193.0: Grayish brown claystone with slickensided and polished shear surfaces. 190.6-190.8: M.C. and Uniaxial.
	15	70	70	192			W% = 22.4 UCS = 0.40 MPa				
	13	46	66	194							
	11	88	100					47°			
	11	100	100	196							
	15	100	100	198	897.6						197.4-200.0: Layered, dark brown; soft; carbonaceous; CLAYEY COAL/CARBONACEOUS CLAYSTONE.
	18	34	100	200			W% = 24.6 UCS = 0 MPa				199.0-199.3: M.C. and Uniaxial.

Fracture Index (top to bottom, 181–200 m):

2	1	0	0	0	0	3	1	0	4	3	3	3	2	5	1	2	3	6	5

Contractor: D.W. COATES
Logged by: PP/NGM
Remarks:
Date started: 10 JULY 82
Checked by: DF
Date finished: 21 JULY 82
Date: 20 AUG 82

Golder Associates

Scale: metric

Type of drilling: ROTARY CORE POLYMER FLUSH	Coordinates: 5625656.8 N 600512.4 E		DRILLHOLE No. 82-925 Sheet 11 of 11
Rig: LONGYEAR 44	Dip: 60^{o}		Location: HAT CREEK EASTERN ESCARPMENT
Bit: PILOT(75 SERIES)/HUDDY "BLACK"	Azimuth: N 75^{o} E		Reference elevation: DRILL FLOOR

Drilling Progress	Rate of Advance Min./m (10 20)	R. Q. D. (0 50)	Core Recovery % (0 50)	Depth m	Reduced Level	Water Level	Test Results	Bed./Fol.	Fracture Index	Instru-mentation	Legend	Description
					895							CARBONACEOUS CLAYSTONE cont'd
	15	100	100					60^{o} 50^{o}	0			200.0-203.0: Slightly weathered; massive; sheared; gray to dark gray; very fine grained; moderately strong to moderately weak; CLAYSTONE with carbonaceous/coaly bands, fine sandstone/siltstone bands.
	16	100	100	202 203			W% = 33.7 UCS = 0.32 MPa	65^{o} 60^{o}	0 1			202.3-202.5: M.C. and Uniaxial.
												203.0 END OF HOLE

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

Contractor: D.W. COATES	Logged by: PP/NGM	Remarks:	
Date started: 10 JULY 82	Checked by: DF	Golder Associates	Scale: metric
Date finished: 21 JULY 82	Date: 20 AUG 82		

ADDENDUM 2

Petrographic Reports

TIERRA CONSULTING
470 WEST 20TH AVE.
VANCOUVER, B.C.
V5Y-2C8
TEL.: 876-5778

Date 20-07-'82

Mr. G. Rawlings
Golder Associates
224 West 8th ave.
Vancouver, B.C. V5Y 1N5

re. Project 8221524 A

Dear Sir:

Enclosed please find petrographic descriptions, thin sections and remaining sample material for 5 core specimens submitted to me, via Coots Petrographic Services, for petrographic analysis.

All five samples were derived from andesitic volcanic flows, and contain well developped fluidal textures defined by subparallel plagioclase microlites. Spec. #3 66.5 m is a flowbanded andesite with flattened, zeolite filled vesicles. Spec. #4a 86.6 m and spec. 82-923 131.9 m are andesite breccias, in which the lithic fragments have been cemented together by pale green, amorphous and aphanitic clay minerals(?); these may be hydrothermal smectites. Small amounts of opal, chalcedony, and in the case of spec. 82-923 131.9 m small tridymite crystals, occur as interfragmental fillings as well. Specimens #9 176.4 m and #5 91.4 m are trachytic andesites, characterized by microcrystalline aggregates of subparallel plagioclase and by nondescript, intersertal material, which may in part be altered glass. These two samples contain only small amounts of hydrothermal clay minerals(?).

Hopefully these descriptions are satisfactory and of some help to you in your investigations. If you have any further questions regarding these samples, please do not hesitate to contact me at 876-5778.

Sincerely,

Peter van der Heyden
M.Sc. Geologist

Specimen # : 82-921 #3 66.5 m

Classification : Altered, flowbanded andesite

Mode :

Plagioclase	45-50%
Clayminerals	40-45%
Zeolites	5%
Indeterminate minerals and alteration products	5%
Quartz	<0.5%
Opaques	<1%

Handspecimen : White (altered, clayminerals?), aphanitic, flowbanded volcanic rock. Flowbanding is defined by flattened vesicles, which are partly filled with very fine grained, pink coloured, sferulitic zeolite(?) aggregates. These are commonly aligned in gently undulating trails. Irregularly shaped, aphanitic and somewhat glassy, grey-green planar, lenticular and elongate bodies, ranging up to several cm across, are probably unaltered domains; similar material occurs as thin rims around vesicles.

Thin section : This specimen is composed predominantly of very fine grained, subparallel plagioclase microlites (max. length 0.16 mm), which give the specimen a characteristic trachytic texture, and is further composed of minor amounts of cryptocrystalline interstitial material. No phenocrysts were observed. The feldspars are severely altered (white colour in handspecimen: clayminerals?). Alteration minerals could not be identified on account of their murky, clouded appearance under the microscope. Also, the definate composition of the prismatic microlites which form the bulk of this specimen, could not be ascertained microscopically. The crystals commonly have twins resembling Carlsbad types, and birefringence and optical orientation are compatible with plagioclase. However, small amounts of feldspathoids such as nepheline may be present. Depending on wether this rock is composed mainly of albite or more calcic plagioclase, it should be classified as a soda trachyte or as a andesite; the latter classification is here tentatively applied.

Flattened vesicles are partly filled with faintly radiating zeolite aggregates. Many vesicles have thin (0.1 - 0.4 mm), relatively dark, cloudy rims, which appear to be more altered than volcanic material further removed.

A small, lithic inclusion (0.65 mm across), composed of foliated(?) quartz and white mica, is present near the top of the section. Also, a small lenticular area, 2.5 mm long, composed of microcrystalline, granular polygonal quartz grains, is present along one side of the section.

Fine grained opaques are thinly scatterred throughout the volcanic component of this specimen. Opaques also occur in close association with zeolites in the vesicles.

Specimen # : 82-921 #4a 86.6 m

Classification : Andesite breccia

Mode :

Plagioclase	20-30%
Clay minerals(?)	20%
Nondescript alteration products	50-60%
Opal	2%
Chalcedony	tr
Opaques	<1%

Handspecimen : Severely altered and brecciated rock; cm scale, subangular, grey and black, aphanitic fragments, set in a very fine grained, red coloured, matrix. The matrix itself appears to be cut by fractures, which have been healed by aphanitic, cracked, pale green material of indeterminate composition. This material also occurs in irregular patches throughout the rock. It is quite soft, and resembles porcelain. It is quite possibly a hydrothermal clay mineral. (Note: in spec. 82-923 131.9 m the same mineral was tentatively identified as serpentine.)

Thin section : The specimen is a breccia, derived from a extrusive volcanic rock (flow), similar to specimen 82-921 #3 66.5 m. The severely altered lithic fragments show a relict trachytic texture, defined by subparallel plagioclase microlites. The intersertal matrix is composed of murky, brownish, hematitic (red colour) clayminerals(?). These may have been derived from intergranular glass; some fragments have characteristic perlitic cracks. The alteration texture is very patchy: cloudy and murky patches of altered palgioclase alternate with irregular hematitic patches and small pools of brownish, cracked, aphanitic clay minerals(?).

The breccia fragments have been cemented by the brown, aphanitic material (clay minerals?). Remaining cavities, as well as cross-cutting fractures, have been filled, commonly only partly, with micro-colloform opal. One small cavity near the bottom of the section is filled with chalcedony. Fine grained opaques (? may be very fine grained, high relief crystals) are locally present in the cores of small cavities and along the center of some fractures.

Note : a small fragment along the left side of the section has a well developped, relict flowbanding, defined by thin, dark red streaks. The majority of the fragments have a more even, trachytic or pilotaxitic texture, and the igneous component of this sample is therefor more like spec. # 82-921 #9 176.4 m than spec. 82-921 #3 66.5 m. However, both of these are andesitic volcanic rocks.

Specimen # : 82-921 #5 91.4 m

Classification : Trachytic andesite

Mode :	Plagioclase	85-90%
	Indeterminate material	10%
	Zeolites	<1%
	Clay minerals(?)	2-3%
	Zircon and apatite	tr
	Opaques	1%

Handspecimen : Grey, aphanitic flow rock, with thin, faintly defined reddish bands which probably represent flow layering. The specimen is somewhat glassy and has macroscopically visible perlitic cracks. Thin, discontinuous, reddish streaks (same material as in red stratified bands?) are oriented at approx. 45° to the stratification.

Thin section : The sample is composed mainly of microcrystalline plagioclase microlites whiach are oriented in typical fluidal fashion (trachytic). Intergranular spaces are filled with indetrminate intersertal material, which is commonly quite murky, and minor amounts of micro-granular opaques. Some of the intersertal material may be altered glass. Small prisms (up to 0.15 mm long) of altered (opaques) pyroxene or amphibole, some of which form micro-phenocrysts, are scattered throughout the specimen.

Oriented at approximately 45° to the fluidal (trachytic) texture are thin, discontinuous streaks or seams lined with zeolites(?) and filled with olive to brown coloured, amorphous clay minerals(?). The amorphous material is locally cracked. Zeolites are commonly sferulitic.

The seams are surrounded by narrow alteration rims, similar to the flattened vesicles in spec. # 82-921 #3 66.5 m. Their orientation with respect to the fluidal texture suggests that the seams are healed tension fractures.

Very rare, rounded inclusions of zircon (0.10 mm) and apatite (0.22 mm) are present in amongst the plagioclase microlites.

Specimen # : 82-921 #9 176.4 m

Classification : Trachytic andesite

Mode :	
Plagioclase	85-90%
Lamprobolite (basaltic hornblende)	5%
Nondescript intergranular material	5%
Glass (palagonite) & clay minerals	1-3%
Zeolites	tr
Quartz	<1%
Opaques	1%

Handspecimen : Grey, aphanitic, locally somewhat glassy looking rock; the glassy appearance on some fractured surfaces may be due to thin coatings of clay minerals. Thin red and white streaks define stratification planes, along which the core specimen tends to break. The white bands, which are not represented in thin section, contain minute, disseminated, red coloured crystals of indeterminate composition.

Thin section : Insofar as igneous character is concerned, this specimen is very much like the three samples described previously; it appears to be least altered and fractured, and only contains minimal amounts of secondary, deuteric minerals. Hence this specimen may be taken to be representative of the andesitic nature of this suite.

The sample consists typically of very fine grained, subparallel plagioclase microlites (up to 0.25 mm long), which give this specimen its characteristic trachytic or pilotaxitic texture under the microscope. The plagioclase may be albite, judging by low relief and small extinction angles of small Carlsbad twins.

Intergranular, brown lamprobolite, which commonly forms microphenocrysts up to 0.65 mm long, is locally more or less altered to fine, granular opaques; together with nondescript, partly vitric intergranular material, it forms the remainder of this specimen.

A few thin seams, parallel to the fluidal texture defined by plagioclase microlites, are filled with brownish palagonite or clay minerals, and minor zeolites. Other seams and small, irregular patches up to 0.5 mm across, contain clear, fine grained quartz.

Very fine grained opaque material, partly derived from basaltic hornblende, occurs disseminated in intergranular spaces throughout the sample.

Specimen # : 82-923 131.9 m

Classification : Andesite breccia

Mode :

Plagioclase	30%
Serpentine(?) or clay minerals(?)	30%
Nondescript material (incl. glass(?))	20%
Pyroxene	<10%
Opal, chalcedony, tridymite & quartz	5-10%
Zeolites(?)	?
Opaques	<1%

Handspecimen : Volcanic breccia with densely packed, grey and black, aphanitic to glassy, subrounded to angular fragments ranging from sub-mm scale to several cm across. The fragments, particularly the dark coloured ones, commonly have light coloured alteration rims. The interfragmental spaces are filled with aphanitic and very fine grained mineral aggregates: green (serpentine?), white (zeolites?), and lesser amounts of light blue (opal?) material.

Thin section : The fragments in this specimen are composed of very fine grained andesite with characteristic trachytic or pilotaxitic texture. Abundant subparallel plagioclase microlites (exact composition indeterminate due to fine grain size; aver. size 0.06 mm, microphenocrysts up to 0.15 mm) and lesser amounts of colourless pyroxene (hypersthene as well as pigeonite??), are set in a nondescript, brownish intersertal matrix. The matrix may be partially composed of devitrified glass (palagonite) and deuteric serpentine(?). A few small, granular opaques, up to 0.06 mm across, are widely scattered throughout the fragments; they are probably magnetite. The darker borders of the fragments appear to be due to higher concentrations of dark, murky alteration products (note that these borders are light coloured in handspecimen).

The interfragmental spaces are lined with pale brown (green in handspecimen!), radial serpentine(?) aggregates, and filled with colloform, bluish opal, tabular tridymite crystals, radial chalcedony, and granular quartz. Possibly minor amounts of radiating zeolites are present as well, but these were not definately distinguished from other radiating deuteric minerals. The material here tentatively identified as serpentine is commonly cracked, is quite soft (scratched by needle in handspecimen), and has negative relief wrt. Canada-balsam. Its macroscopic appearance is very similar to serpentine, but only XRD analysis would give conclusive information in this regard. Note that identical material in the three previously described specimens was tentatively identified as clay minerals of hydrothermal origin. If this material is indeed composed of clay minerals, the most likely variety would be one of the smectites, such as montmorillonite, saponite or nontronite. All of these are common hydrothermal alteration products of volcanic rocks, and can occur in hot spring environments. In terms of relief, montmorillonite or saponite are the most likely candidates.

VP

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Golder Associates
Attn. Mr. D. Finley
224 West 8th Ave.
Vancouver, B.C.
V5Y 1N5

Sept. 7, 1982
Inv. # 3460

Dear Sir:

Enclosed please find petrographic descriptions for 8 specimens from the Hat Creek area, as well as thin sections and remaining sample material.

The specimens have been classified as follows :

Spec. # 82-925	127.6-129.85 m	:	Altered andesite breccia
Spec. # 82-925	87.37-87.59 m	:	Pyroxene andesite
Spec. # 82-925	168 m	:	Crystal-lithic, clay rich (altered) tuff-breccia
Spec. # 82-923	236.1-236.2 m	:	Clay rich (altered) vesicular andesite breccia
Spec. # 82-923	195.4-195.55 m	:	Feldspathic wacke (sandstone)
Spec. # 82-923	166.7-166.8 m	:	Vesicular andesite
Spec. # 82-924	219.7-219.76 m	:	Carbon bearing calcareous wacke (sandstone)
Spec. # 82-924	84.9-85.1 m	:	Altered crystal-lithic tuff-breccia

As for the specimens from the same area which I have described on a previous occasion, I favor a hydrothermal type alteration for the development of the green, amorphous clay minerals in the volcanic specimens.

I have adjusted the format of my description somewhat for the sake of expedience, but you will find all the major features of these rocks are well covered. It must be realized that exact identification of the very fine grained, murky materials in the thin sections, is not possible under the microscope. Analyses of clay minerals would only be possible by XRD and/or DTA.

If you have any further questions regarding these specimens, please contact me at 876-5778.

Sincerely,

Peter van der Heyden

Specimen # : DDH 82-925 127.6-129.85 m

Classification : Andesite breccia (altered)

Mode :	
Plagioclase	40-45%
Pyroxene	5%
Unidentified material	10-15%
Matrix material	40%

Hand specimen : Severely altered breccia composed of bluish-grey, fine grained volcanic fragments upto several cm in size, set in a violet-blue and waxy, green matrix. The fracture pattern suggests cataclastic deformation of a primary cohesive volcanic rock.

Thin section : This specimen is a brecciated version of specimen # 82-925 87.37-87.59 m. The texture of the volcanic fragments is typically trachytic, as defined by abundant, subparallel plagioclase microlites and less common microphenocrysts. The specimen was clearly a extrusive volcanic rock (flow) prior to brecciation. The groundmass of the volcanic component is rather nondescript (cloudy, murky and fine grained), and may have been glassy prior to alteration. The specimen is classified as a andesite on account of its plagioclase content. Volcanic fragments are set in a brownish (green in hand specimen), very fine grained matrix which could not be identified under the microscope, but which in all likelihood contains mainly clay minerals.

Plagioclase : Microlites averaging 0.1-0.15 mm in length, microphenocrysts up to 0.45 mm. Both carlsbad and albite twinning is present.

Pyroxene : Rare, slender hypersthene(??) microphenocrysts up to 1.8 mm long. more commonly as small, stubby laths interstitial between plagioclase microlites.

Groundmass of volcanic component : Murky, unidentified material, possibly devitrified glass.

Matrix material : Brownish material, probably mainly composed of clay minerlas. The bluish material seen in hand specimen was not identified in thin section.

Specimen # : DDH 82-925 87.37-87.59 m

Classification : Pyroxene andesite

Mode :

Plagioclase	90-95%
K-feldspar	trace
Pyroxene	2-3%
Ilmenite(?)	trace
Nondescript material	5%
Clay minerals(?)	3%

Hand specimen : Grey, waxy, aphanitic to fine grained volcanic rock. The waxy, greenish material (smectite??) occurs along distinct perlitic cracks. This rock was evidently quite glassy at one time.

Thin section : The texture of this specimen is micro-porphyritic, with a local fluidal (trachytic) texture in a dominantly felted, microcrystalline groundmass which is composed mainly of plagioclase. Conspicuous perlitic cracks cut across the specimen. Late, secondary fractures partly follow the perlitic cracks, and have been healed by amorphous, colloform clay minerals(?) and locally contain small amounts of relatively coarse grained K-feldspar. On account of its plagioclase content this specimen is classified as a andesite.

Plagioclase : Dominantly as microlites in the groundmass; some microphenocrysts with carlsbad and albite twinning are also present. Maximum length is about 0.15 mm. Very small amounts of K-feldspar occur in late fractures.

Pyroxene : Conspicuous microphenocrysts up to 0.7 mm long, commonly forming small clusters up to 1.5 mm in size. Both ortho- (straight extinction) and clinopyroxene (inclined extinction) may be present.

Ilmenite(?) :Opaque prisms up to 0.8 mm long, scattered throughout the specimen.

Matrix : Nondescript, cryptocrystalline, intersertal material, possibly devitrified glass.

Clay minerals(?): Amorphous, brownish (green in hand specimen) material along fractures.

Specimen # : DDH 82-925 168 m (2 thin sections)

Classification : Crystal-lithic, clay rich (altered) tuff-breccia

Mode :	
Plagioclase	30-35%
Clay minerals	60%
Biotite + unidentified material	5-10%

Hand specimen : Very friable, greenish grey clastic rock composed mainly of amorphous, aphanitic clay minerals. Lithic fragments are angular to subangular, and range up to 2 cm in size.

Thin section : The microscopic character of this rock is that of a crystal-lithic tuff-breccia, which was deposited in a clay rich environment. However, the clay matrix of this specimen may very well be due to pervasive alteration of a very fine grained primary tuffaceous matrix. The subrounded form of some crystal fragments suggests some amount of sedimentary transport prior to final deposition. Most of the larger fragments are fine grained to aphanitic, altered, trachytic textured volcanics of andesitic composition. Crystal fragments are predominantly composed of plagioclase; a few small flakes of biotite were observed as well. A large volcanic fragment in one of the thin sections has typical perlitic cracks, indicative of it's glassy nature.

Plagioclase : Microlites in volcanic clasts, crystal fragments up to 1 mm in size. The latter commonly exhibit oscillatory zoning. They probably represent broken phenocrysts.

Clay minerals : Amorphous, brownish (grey-green in hand specimen) material in grounmass, probably mainly smectites.

Biotite and unidentified material : Thinly scattered throughout the specimen. Many fine grained fragments could not be identified due to small size and severe alteration.

Note : Fractures in thin section were aquired during sample preparation.

Specimen # : DDH 82-923 236.1-236.2 m

Classification : Clay rich (altered) vesicular andesite breccia

Mode :	Plagioclase	20%
	Pyroxene	10%
	Clay minerals	65%
	Opal	1%
	Nondescript material	3-5%

Hand specimen : This specimen is a breccia composed of very fine grained to aphanitic, grey, vesicular and amygdaloidal volcanic flow fragments, set in a matrix rich in greenish clay minerals. Vesicles are lined with blue-voilet, colloform material and locally filled with white zeolites.

Thin section : Volcanic fragments in this specimen are porphyritic andesites, consisting of plagioclase and pyroxene phenocrysts set in a altered, very fine grained matrix with plagioclase microlites and intersertal, rather nondescript murky material, which may be devitrified glass. The boundaries between fragments and matrix material are not very well defined on account of pervasive alteration of both. The clay rich matrix contains abundant lithic and crystal clasts. One of the lithic clasts appears to be a crystal tuff. Vesicular cavities in the volcanic clasts are lined with micro-colloform clay minerals(?) and thin coatings of opal. Locally very minor calcite is present as well. Zeolites were not observed in thin section, but they are present in hand specimen.

Plagioclase : Phenocrysts and crystal fragments up to 1.2 mm long, with carlsbad, albite and rare pericline twinning. These commonly exhibit oscillatory zoning. Also forms microlites in matrix of fragments.

Pyroxene : Conspicuous phenocrysts and crystal fragments up to 0.7 mm across. Locally formong small aggregate clots. Both clino- and orthopyroxene may be present.

Clay minerals : Mainly nondescript, brownish, very fine grained or amorphous material in the graoundmass. Also as micro-colloform coatings on some vesicle walls.

Opal : Thin, isotropic coatings of some vesicles. Blue-violet colour in hand specimen.

Nondescript material : Intersertal in volcanic fragments, possibly mainly devitrified glass.

Specimen # : DDH 82-923 195.4-195.55 m

Classification : Feldspathic wacke (sandstone)

Mode :

Plagioclase	70-75%
Quartz	10%
Biotite	2-3%
Muscovite	trace
Pyroxene(?) and hornblende(?)	1%
Matrix material (clay minerals?)	10-15%

Hand specimen : Extremely friable, light greenish-grey, medium grained, biotite bearing clastic (sandstone). A faint layering is oriented at about 30° to the length of the core.

Thin section : This specimen is a immature (both in a mechanical and in a chemical sense), moderately to poorly sorted, medium grained sanstone, composed predominantly of plagioclase grains set in a nondescript, very fine grained, somewhat brownish matrix, which is probably largely composed of clay minerals. Maximum grainsize is 2.7 mm. Other materials present in the clastic portion of this rock are quartz, feldspar, biotite, muscovite, pyroxene(?) and hornblende(?).

Plagioclase : Predominantly subangular to subrounded single crystal grains up to 1.2 mm across. Most grains exhibit twinning (carlsbad, albite and pericline) and many have oscillatory zoning. Many grains are fractured. A few lithic grains (andesite) are composed predominantly of very fine grained, trachytic and felted plagioclase microlites.

Quartz : The largest, subrounded grains in this specimen are composed of fine grained, deformed lithic quartz. Smaller grains with very fine grained, cherty texture and single grains of undeformed quartz are present as well.

Biotite and muscovite : Deformed flakes up to 1.2 mm in size, bent between feldspar grains. Muscovite is relatively rare.

Pyroxene(?) and hornblende(?) : Minor, small fragments with high relief, high birefringence and, in the case of hornblende(?), green pleochroism.

Specimen # : DDH 82-923 166.7-166.8 m

Classification : Vesicular andesite

Mode :	Plagioclase	70-75%
	Pyroxene	2-3%
	Alteration products	25-30%

Hand specimen : Severely altered, friable, grey vesicular volcanic rock. Some vesicles are lined with fine grained greenish material (crystalline). Others are filled with clay minerals.

Thin section : This specimen is a extrusive andesitic volcanic. It's texture is intersertal: a felted mass of plagioclase microlites encloses interstitial pyroxene and murky, brownish alteration products, probably mainly clay minerals. Very small amounts of zeolite and very fine grained green material (malachite??) are locally visible along vesicle walls.

Plagioclase : Abundant microlites forming a felted mass; maximum crystal length is about 0.45 mm.

Pyroxene : Small, anhedral, colourless grains up to 0.3 mm across, interstitial between plagioclase microlites.

Alteration products : Mainly brownish, amorphous, intersertal clay minerals, irregularly distributed throughout the specimen and as coatings on vesicle surfaces. Zeolites occur as very thin, colloform coatings as well as subhedral crystals on some vesicle walls and along irregular veinlets.

Specimen # : DDH 82-924 219.7-219.76 m

Classification : Carbon bearing calcareous wacke (sandstone)

Mode :

Quartz	40%
Plagioclase	2-3%
Muscovite	2%
Calcite	40-45%
Carbon	7-10%
Clay minerals(?)	2-3%

Hand specimen : Grey, fine grained, laminated, micaceous and calcareous clastic rock. The dark, thin laminae are rich in carbonized organic material (sticks, leaf fragments).

Thin section : This specimen is a fine grained, loosely packed, gritty wacke (sandstone), composed of angular quartz, feldspar and chert grains, and minor amounts of clastic muscovite, set in a rather murky matrix composed predominantly of calcite and lesser amounts of brownish clay minerals(?). The opaque carbon laminae, which define the macroscopic stratification of the sample, contain fragments with celular textures, indicative of their organic origin.

Quartz : Angular and subangular grains up to 0.2 mm across. Commonly undulose. Several grains are lithic, microcrystalline chert (about 5% of the clastic component).

Plagioclase : Angular and subangular grains up to 0.2 mm in size. Relatively minor component.

Muscovite : Small, subhedral flakes up to 0.22 mm across, commonly bent.

Calcite : Very fine grained, anhedral grains and amorphous material in matrix. It is not evident if the calcite is a primary component of this rock, or wether it is due to secondary replacement.

Carbon : Opaque laminae and rare celular fragments.

Clay minerals(?) : Murky, brownish, nondescript material distributed throughout the matrix.

Specimen # : DDH 82-924 84.9-85.1 m

Classification : Altered crystal-lithic tuff-breccia

Mode : Plagioclase 15%
Biotite 1-2%
Nondescript alteration products 80-85%

Hand specimen : Severely altered, greenish grey tuff-breccia. Abundant angular volcanic fragments, ranging up to several cm in size, are set in a green, clay rich matrix. The fragments themselves are commonly altered to white, fine grained material. Some of the dark fragments have a distinct glassy appearance. A few fragments are surrounded by limonitic alteration halos.

Thin section : Microscopic textures indicate that this specimen is a crystal-lithic tuff-breccia, with angular crystal fragments (mainly plagioclase) and angular to subrounded lithic fragments (andesite) set in a very fine grained, murky matrix, which may consist mainly of clay minerals. The severe alteration and fine grained nature of both matrix and many lithic fragments precludes definate identification of component minerals. Fluidal (trachytic and flow banded) and felted textures of most lithic fragments suggest derivation from extrusive andesitic volcanics.

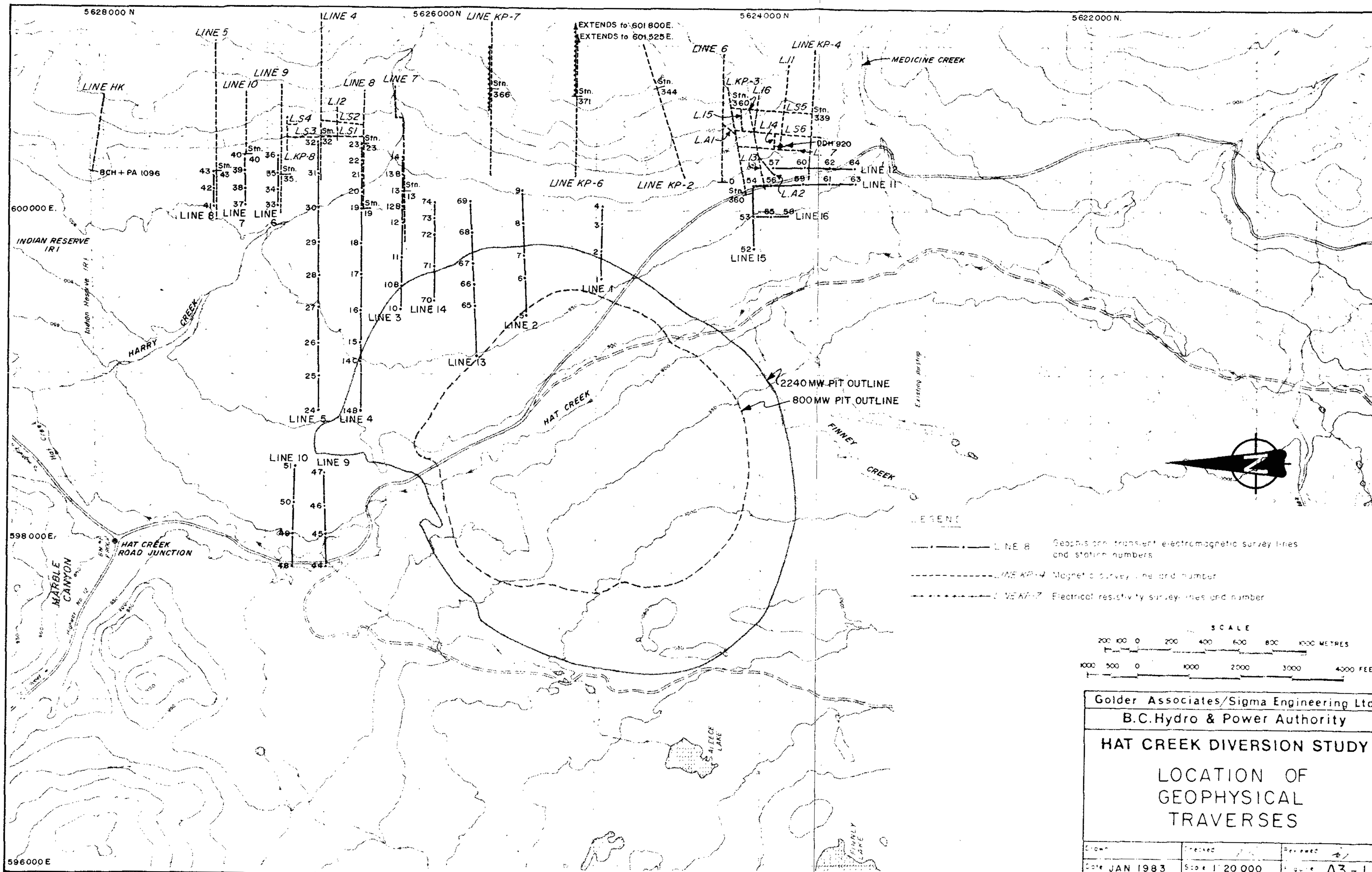
Plagioclase : Angular crystal fragments up to 1 mm in size, locally with carlsbad & albite twinning and oscillatory zoning. Microlitic plagioclase occurs in trachytic and felted lithic fragments.

Biotite : Small, subhedral flakes up to 0.3 mm across; these are the only recognizable crystal fragments apart from plagioclase.

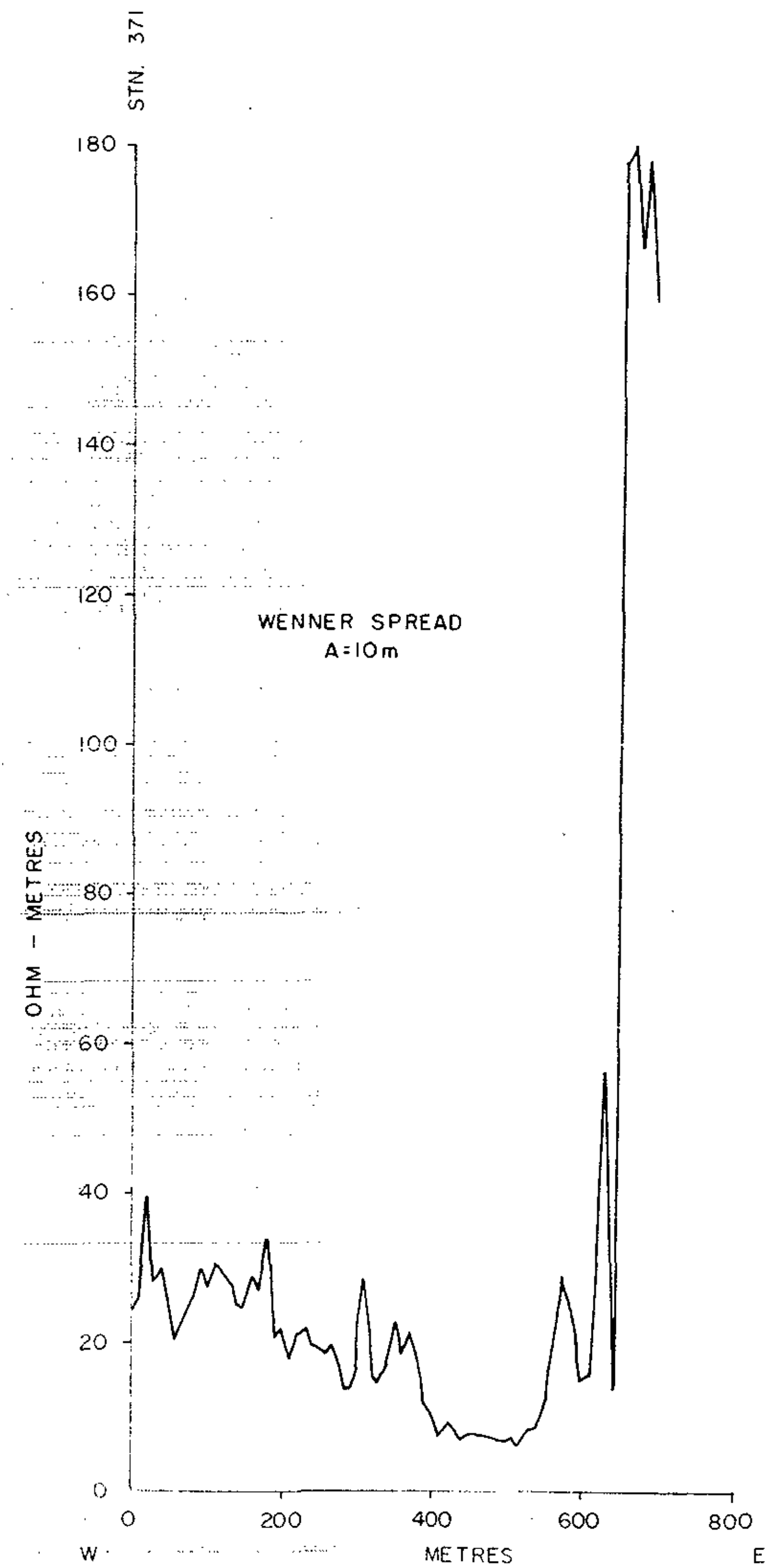
Nondescript material in matrix, alteration products : Very fine grained, murky material, which could not be identified under the microscope. Occurs both in lithic fragments and in the tuffaceous matrix. Probably predominantly composed of clay minerals, with localized limonitic patches.

ADDENDUM 3

Magnetic and Resistivity Surveys

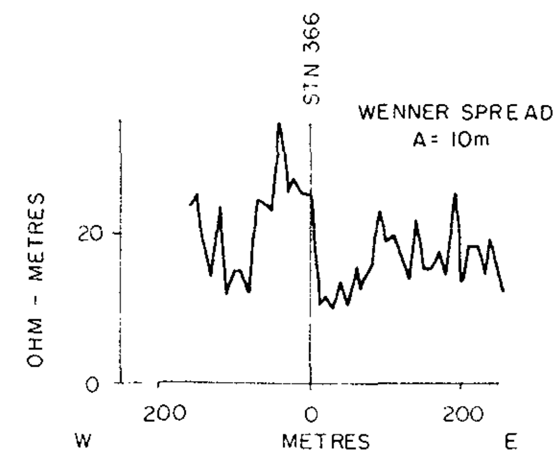


LINE 5
LINE 4
LINE KP-7
EXTENDS to 601 800E.
EXTENDS to 601 525E.
LINE 6
LINE KP-4
MEDICINE CREEK
LINE HK
LINE 10
LINE 9
LINE 8
LINE 7
LINE KP-6
LINE KP-2
LINE 12
LINE 11
LINE 16
LINE 15
LINE 1
LINE 2
LINE 3
LINE 14
LINE 13
INDIAN RESERVE IR1
HARRY CREEK
HAT CREEK
FINNEY CREEK
2240 MW PIT OUTLINE
800 MW PIT OUTLINE
HAT CREEK ROAD JUNCTION
MARBLE CANYON
DDH 920
8CH + PA 1096
5628000 N
5626000N
5624000 N
5622000 N.
600000 E.
598000 E
596000 E
LEGEND
LINE 8 Geophysical transient electromagnetic survey lines and station numbers
LINE KP-4 Magnetic survey line and number
LINE KP-7 Electrical resistivity survey lines and number
SCALE
200 100 0 200 400 600 800 1000 METRES
1000 500 0 1000 2000 3000 4000 FEET
Golder Associates/Sigma Engineering Ltd.
B.C.Hydro & Power Authority
HAT CREEK DIVERSION STUDY
LOCATION OF GEOPHYSICAL TRAVERSES
Date JAN 1983
Scale 1:20 000
Figure A3-1



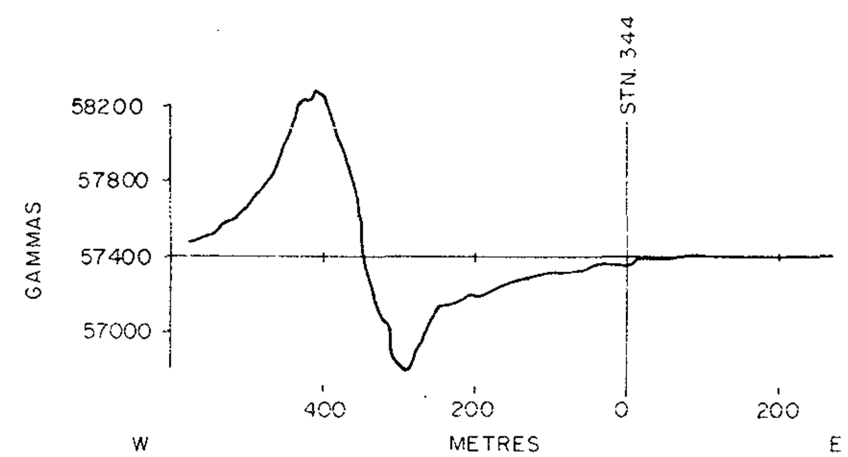
RESISTIVITY PROFILE

KP-6



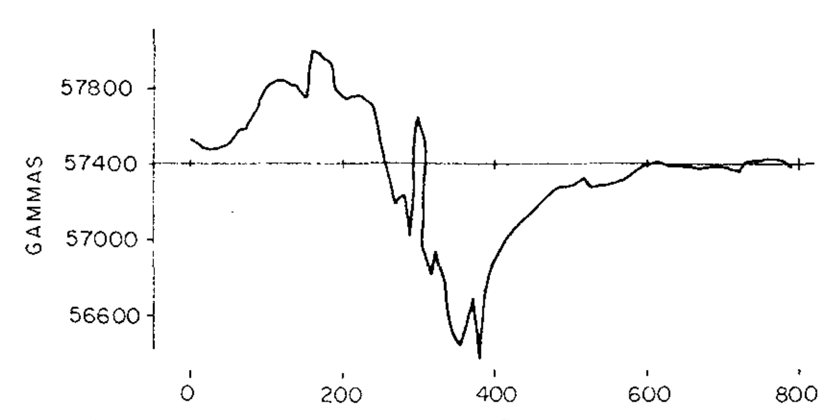
RESISTIVITY PROFILE

KP-7



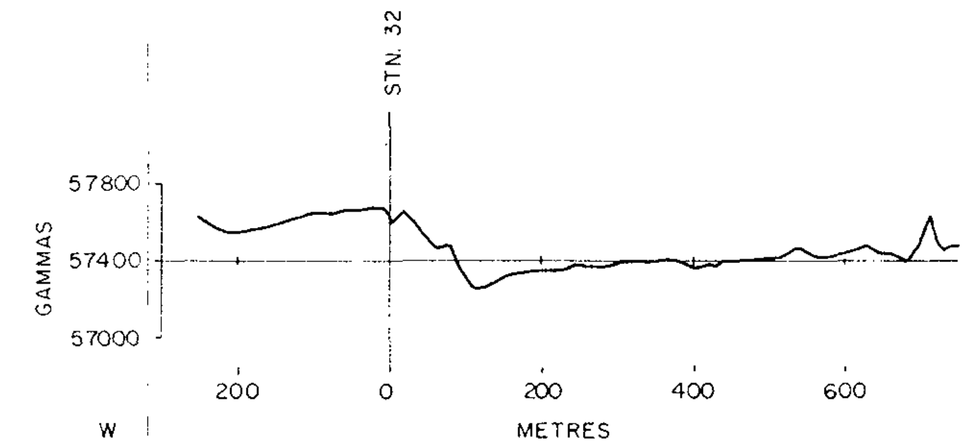
MAGNETIC GRADIENT PROFILE

KP-2



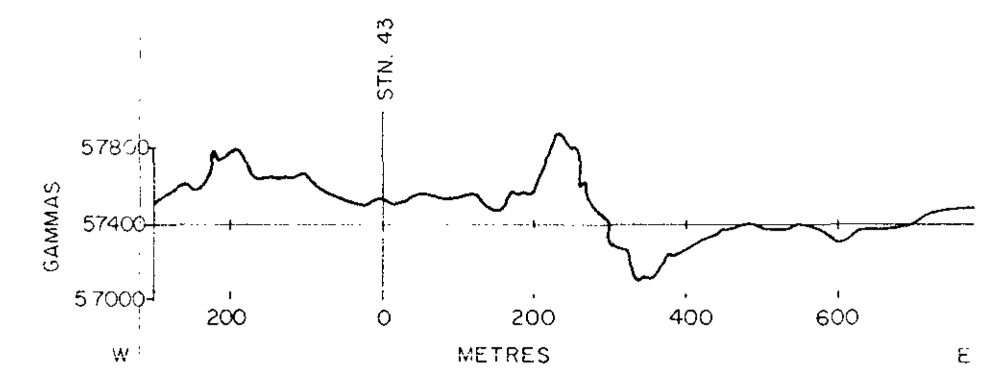
MAGNETIC GRADIENT PROFILE

LINE 6



MAGNETIC GRADIENT PROFILE

LINE 4



MAGNETIC GRADIENT PROFILE

LINE 5

For locations of traverses, see Fig. A3-1.

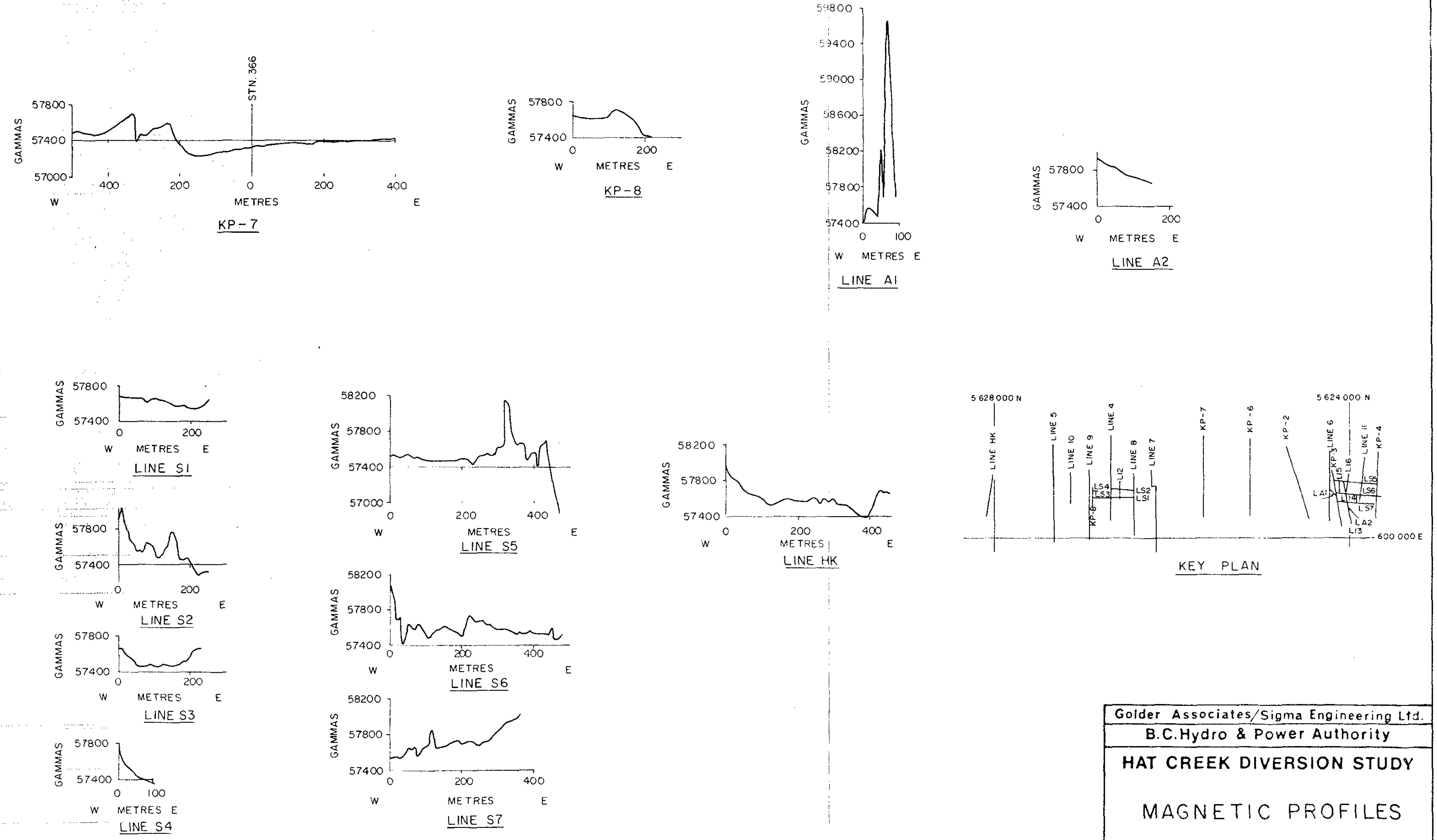
Golder Associates/Sigma Engineering Ltd.

B.C.Hydro & Power Authority

HAT CREEK DIVERSION STUDY

RESISTIVITY AND MAGNETIC PROFILES

Drawn	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure A3-2



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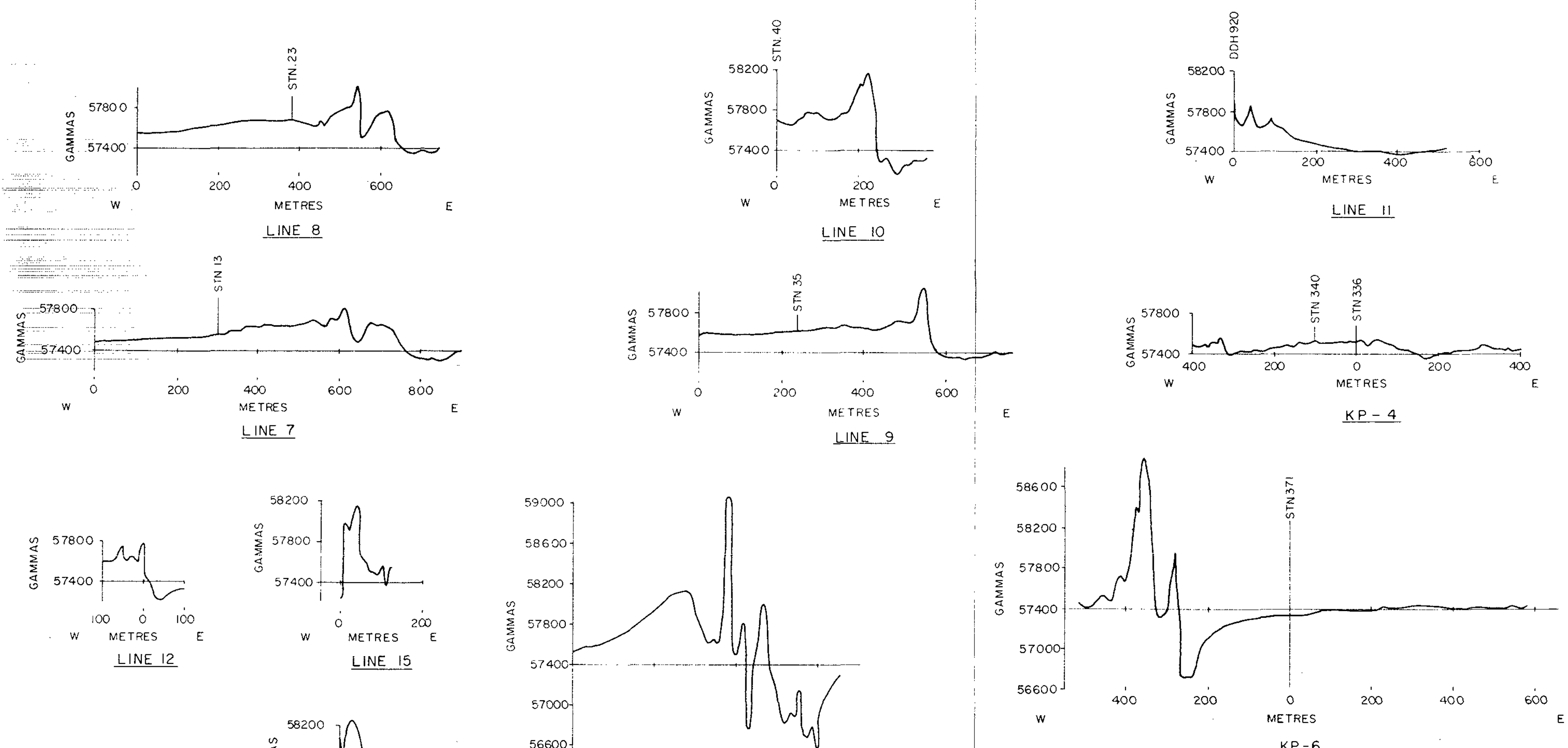
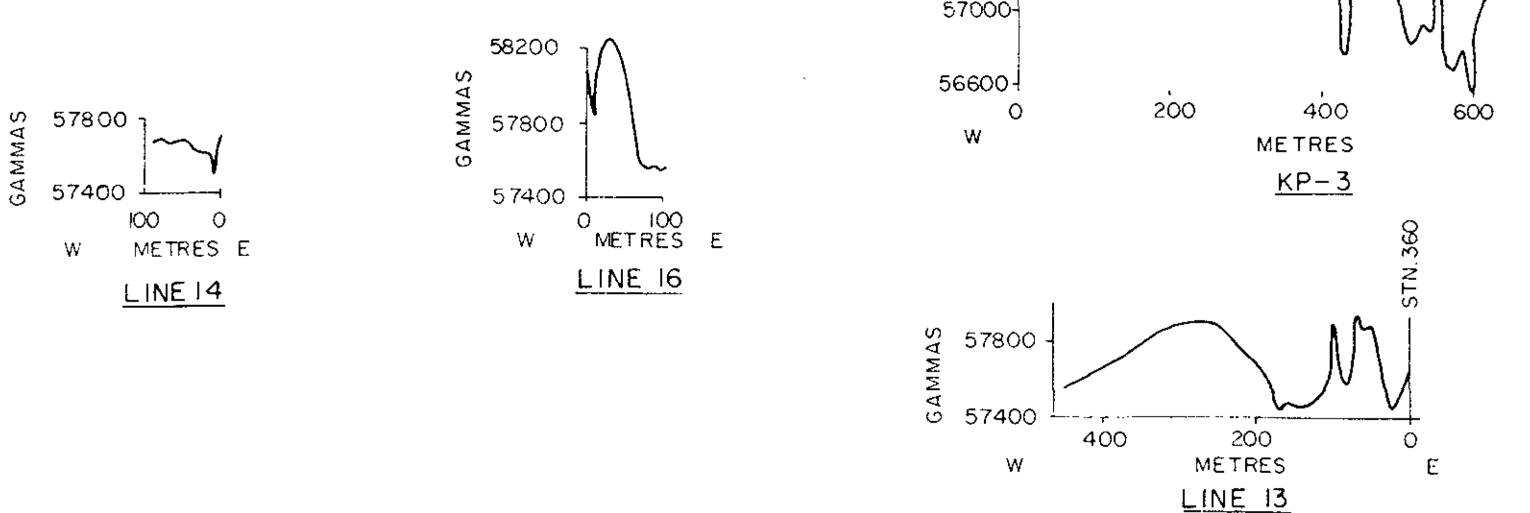
B.C.Hydro & Power Authority		
HAT CREEK DIVERSION STUDY		
MAGNETIC PROFILES		
Drawn	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure A3-3





For location of traverses, see Key Plan on Fig. A3-3.

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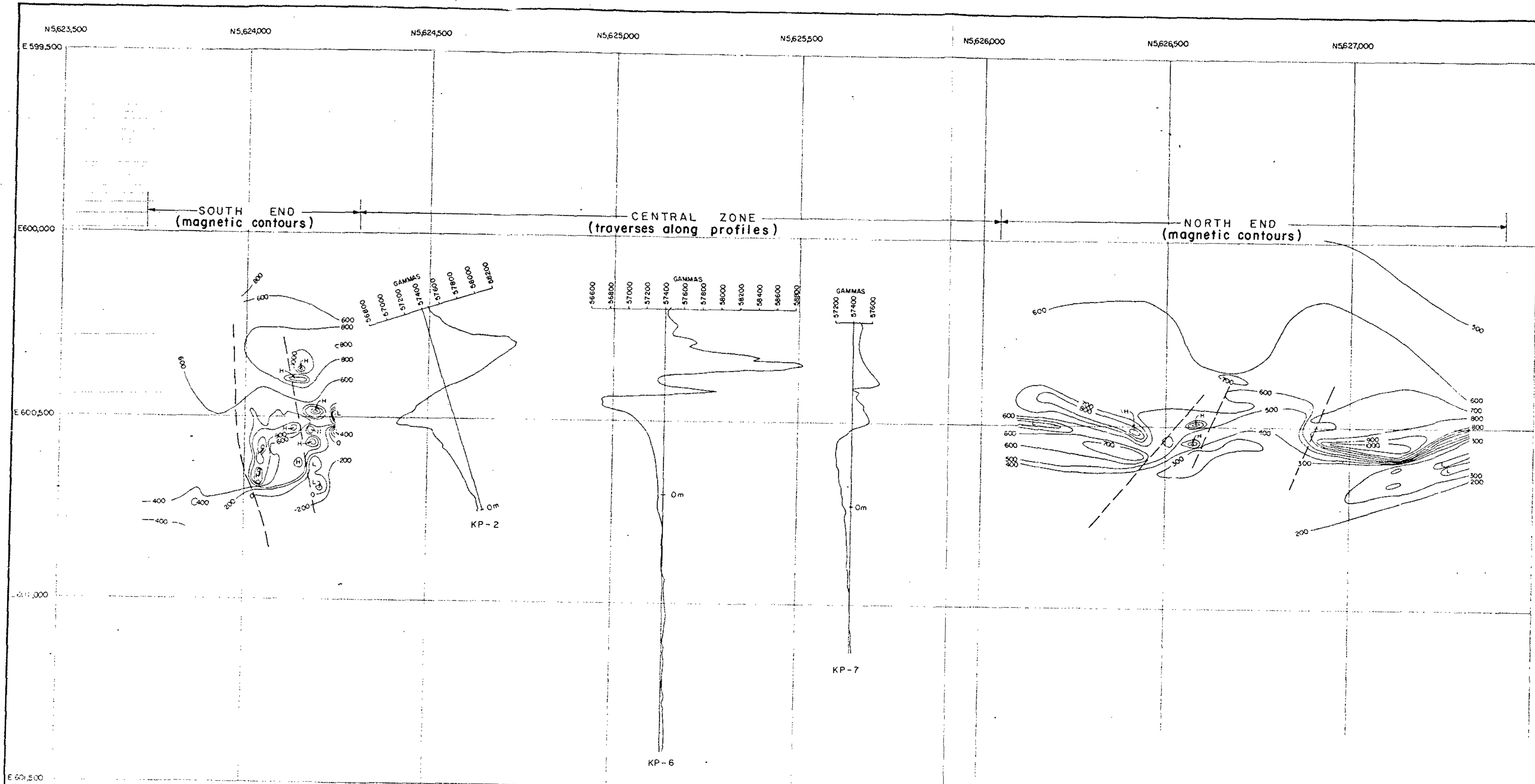
Drawn	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure A3-4



LEGEND

SOUTH END

—600— Magnetic Contour (gammas relative to base reference). Contour interval = 200 gammas

H Area of relatively high magnetic contrast.

L Area of relatively low magnetic contrast

Base reference = 57000 gammas

— — — Magnetically inferred fault

CENTRAL ZONE

Offset profiles shown for lines KP-2, KP-6 & KP-7 with line location as abscissae

NORTH END

—600— Magnetic contour (gammas relative to base reference). Contour interval = 100 gammas

H Area of relatively high magnetic contrast.

L Area of relatively low magnetic contrast.

Base reference = 57000 gammas.

— — — Magnetically inferred fault

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HAT CREEK DIVERSION STUDY

MAGNETIC FEATURE PLAN

Drawn	Checked	Reviewed
Date JAN. 1983	Scale AS SHOWN	Figure A3-5

ADDENDUM 4

Classification of Parameters Used in NGI Tunnelling Quality Index (Q)

TABLE 7 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

Description	*Value*	*Notes*
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as $\leq$ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$
G. Three joint sets plus random	12	2. For portals use $(2.0 \times J_n)$
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
G. Slickensided, planar	0.5	
c. No rock wall contact when sheared.		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
a. Rock wall contact.		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	—

	J_a	ϕ_r (approx.)
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)
b. Rock wall contact before 10 cms shear.		
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)
c. No rock wall contact when sheared.		
K. L. M. Zones or bands of disintegrated or crushed rock and clay (see G,H and J for clay conditions)	6.0 8.0 8.0 - 12.0	(6° - 24°)
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0	
Q. P. R. Thick, continuous zones or bands of clay (see G, H and J for clay conditions)	10.0 - 13.0 13.0 - 20.0	(6° - 24°)

1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm^2)
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0
D. Large inflow or high pressure , considerable outwash of fillings	0.33	2.5 - 10.0
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10

1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
2. Special problems caused by ice formation are not considered.

6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0

b. Competent rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

	SRF
P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.
2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :
   $RQD = 115 - 3.3J_v$ (approx.) where J_v = total number of joints per m^3
   ($RQD = 100$ for $J_v < 4.5$)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q . *The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may

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Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
B.C. HYDRO AND
POWER AUTHORITY
ON
HAT CREEK CONSTRUCTION
WATER SUPPLY

BRITISH COLUMBIA

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SUMMARY AND CONCLUSIONS

1.0 PURPOSE OF PROGRAM

The ground water exploration carried out by Golder Associates at the Hat Creek Project during 1981 was designed to provide a supply of water for the concrete batching plant and potable water for the camp requirements up to a maximum of 1700 m^3/d (19.7 l/s, 311 U.S. gpm). The program involved the following aspects, namely:

(i) selection of preferred areas, followed by the drilling completion, development and testing of test/observation wells to determine aquifer potential;

(ii) drilling of larger diameter production wells together with completion, development, and pump-testing of these wells;

(iii) specification of permanent pumps for the individual wells based on the required pumping rates.

2.0 WELL DRILLING AND PUMP TESTING

The Hat Creek and Marble Canyon valleys were identified as potential areas for ground water development. Test and production wells drilled in Marble Canyon near the junction with Hat Creek identified three aquifers within the alluvial sands and gravels. Pump testing of the screened aquifers indicated that only the shallow aquifer had potential for the required water supply. A production well was installed in this aquifer to provide an estimated 150 U.S. gpm (9.5 l/s). A test well provides a temporary back-up supply of 96 U.S. gpm (6.1 l/s) within the same aquifer.

One test well and one production well were drilled in the Hat Creek Valley between the north rim of the proposed pit and the junction in the Marble Canyon. An artesian sand and gravel aquifer was proved at depth. During the pump testing of these wells stabilisation of the water levels did not occur. It is concluded that although the aquifer does not have abundant recharge, there is potential for supply because of the large available drawdown. A production well was installed in this aquifer to provide 312 U.S. gpm (19.7 l/s) as a permanent supply. A test well provides a temporary back up supply of 100 U.S. gpm (6.3 l/s).

The pump tests show that the results of the various analytical methods are in good agreement; a median hydraulic conductivity of 5×10^{-5} m/s for the shallow sandy gravel aquifer in Marble Canyon has been calculated. The value of storage calculated for this aquifer is in the order of 1×10^{-4}. The report presents pump specifications and performance curves.

3.0 WATER QUALITY

Water samples collected by BCH during the program were analyzed by Eco-Tech Laboratories of Kamloops. For the parameters tested, the samples met the limits set by "Guidelines for Canadian Drinking Water, 1978." The waters are essentially alkaline, moderately hard and low in total dissolved solids.

The water is considered suitable for use in concrete mixing as specified in "Design and Control of Concrete Mixture," Canadian Portland Cement Association.

4.0 CONCLUSIONS AND RECOMMENDATIONS

The aquifers identified and tested north of the proposed Hat Creek open pit can supply the projected requirements of the concrete batching plant and needs of the construction camps. However, because the well installed into the deep aquifer at the junction of Hat Creek/ Houth Creek did not stabilize, monitoring will be required during performance. It is recommended that the following monitoring and sampling program be undertaken by BCH.

(1) Water levels and/or natural artesian overflow discharge rates should be recorded once a week in all wells.

(2) Sampling of drinking water for bacteriological parameters should be undertaken on a weekly basis.

(3) Sampling of drinking water for chemical parameters should be carried out twice a year.

(4) Sampling of water for the concrete batching plant should be extended to include silica and potassium. This analysis to be carried out annually.

1.0 INTRODUCTION

This report describes the work carried out by Golder Associates at Hat Creek, British Columbia, towards the ground water exploration for potential aquifers and the design, construction and testing of water supply wells for construction purposes. The terms of reference for undertaking this work are presented in our letter E/80/1737 to B.C. Hydro and Power Athority (BCH), dated November 27th, 1980. A supply of water for the concrete batching plant and potable water for camp requirements up to a maximum of 1700 m^3/d (19.7 l/s, 311 U.S. gpm) was specified. Three target areas considered to have ground water potential were selected for investigation; these are identified in Figure 1 of the terms of reference.

The contractual part of the program was carried out during June and July, 1981, by A and H Construction Ltd. of Abbotsford, British Columbia. Golder Associates hydrogeological staff maintained full time supervision throughout the entire field operation, analyzed the results and made recommendations for the siting and design of the production wells and the specifications for the pumps.

The approach taken in this ground water program was a combination of exploratory and testing procedures. The following steps were undertaken:

- selection of preferred areas for drilling;
- drilling of production-type observation wells to identify potential aquifers;
- air-lift test pumping during drilling to obtain an initial estimate of water availability;
- placement of permanent screened installations to permit the aquifers to be test-pumped by temporary submersible pump and such that those installations could be left as production and/or observation wells;
- test pumping of most likely aquifers;

- drilling of permanent production wells, installation of screens and development;
- test pumping of production wells;
- specification of permanent pumps.

2.0 TEST WELL PROGRAM

2.1 Drilling

An initial exploratory program was planned which would identify potential aquifers in the Hat Creek and Marble Canyon Valleys beyond the northern limit of the mine site. For this purpose, five observation holes (OW1 - OW5), each 6 inch (152 mm) in diameter, were drilled using an air-rotary truck-mounted drillrig. The locations of these wells are shown on Figure 1.

During drilling, air-lifted soil samples or rock cuttings were collected every 0.6 m for description. Grain size analyses were performed on selected samples in water bearing horizons. Air-lifted flows were recorded and water samples were taken by BCH for chemical analyses. In addition, temperature, pH and specific conductance of ground water were measured in the field. All records of the drilling are provided in the Borehole Logs in Appendix A, and grain size curves are given in Appendix B.

2.2 Aquifer Description

Observation wells drilled at the eastern limit of Marble Canyon (OW1, OW2 and OW3) encountered three potential aquifers.

A shallow sand and gravel zone was encountered between 20 and 26 m with recorded air-lifted flows of up to 4.3 l/sec (68 U.S. gpm); all three test holes intersected this aquifer. A deeper, gravelly, coarse sand zone was detected in OW2 at a depth of 30 to 32.6 m with an air-lifted flow of 1.0 l/sec (15 U.S. gpm); this potential aquifer was not encountered in either OW1 or OW3. The third potential aquifer was intersected by OW1 at a greater depth of 50 to 55 m. Air-lifted flows could not be recorded from this silty fine sand unit due to difficult drilling conditions (heaving), however, observations indicated this unit to have some potential.

To the east of Hat Creek, OW5 identified a shallow sandy gravel aquifer extending from 7.6 to 16.6 m below ground, with recorded air-lifted flows of 0.1 to 3.9 l/sec (1.5 - 61 U.S. gpm). This was considered to be an extension of the shallow sand and gravel aquifer which was encountered by OW1, OW2 and OW3. All observation wells in this area were drilled 3 m into limestone bedrock; water flows from the limestone were less than 0.1 l/sec (1.5 U.S. gpm). Figure 2 provides a schematic interpretation of the Marble Canyon aquifer system at the location investigated.

Observation well 4, drilled in the main Hat Creek Valley close to the junction of Houth Creek and Hat Creek, penetrated a thick sequence of silty clays overlying an artesian sand and gravel aquifer between 67.4 and 110.4 m below ground level. Air-lifted flows of up to 15 l/sec (237 U.S. gpm) were recorded. This well was not continued to bedrock because of sand which blocked the casing annulus, making the retrieval of samples difficult.

Grain size analyses were performed on selected samples from the water bearing horizons. These are included as Appendix B. The grain size distributions were subsequently used in well screen selection (see Section 2.3).

It was, therefore, concluded from the initial exploratory program that there were aquifers in the two separate areas which were capable of being exploited.

2.3 Completion

In the Marble Canyon areas, OW1 was used to screen and test the deepest aquifer. The screen in this hole was ultimately abandoned following testing, and a 19 mm diameter standpipe piezometer was later installed in the shallow aquifer encountered in that hole. OW2 was used to screen and test the middle aquifer and OW3 was used to screen and test the shallow aquifer.

OW5 was considered unsuitable for well screen installation; instead a 19 mm diameter standpipe piezometer wasinstalled to monitor water levels in the shallow aquifer in the Hat Creek Valley; OW4 was used to screen and test the deep aquifer in the Hat Creek Valley. Table 1 summarizes test well completion details.

The information obtained from the drilling was used to identify the water-bearing horizons and select the appropriate installations. Grain size distributions from these depths were used to select appropriate well screens. In all cases a well screen slot size that would retain 50 per cent of the formation was selected. The well screen slot sizes chosen for the observation wells are shown on the hydrogeological logs (Appendix A) and in Table 1.

TABLE 1

SUMMARY OF TEST WELL COMPLETION DETAILS

Well No.	Depth of Screened Interval Below Ground (m)	Slot Size (thousandths of an inch)	Comments
1	49.08 - 55.89	5	Screen ultimately replaced by sealed piezometer at 24.9 m.
2	30.28 - 32.92	100	
3	23.24 - 25.91	150	
4	104.11 - 106.75	80	
5	-	-	Open piezometer installed at 15.69 m.

Well screens were installed using the pull-back method. All holes were backfilled with pea gravel and sealed with bentonite below the screen, except OW4 where flowing artesian conditions were encountered. 152 mm Johnson telescopic stainless steel well screens with 127 mm riser pipe and K-packer were pushed down inside the casing with the drill rods. The casing was then pulled back to expose the screen. Well completion details are presented on the hydrogeological logs in Appendix A.

Following installation, the wells were developed initially by bailing and then alternately by surging and bailing in order to develop a natural filter around the screen. Observation well 4 was developed with air prior to bailing, the effect of this development being to increase the natural overflow from 2.4 l/sec (38 U.S. gpm) to 4.3 l/sec (68 U.S. gpm).

2.4 Pump Testing

Preliminary pump testing in the observation wells was carried out at the end of June; the information was subsequently used to identify aquifer potential and locate production well sites. A 4-inch submersible pump, installed above the screened section, was used to pump test each well. The pumped water was discharged via a trench into Hat Creek. Permission to discharge ground water into Hat Creek was given in a letter from the Waste Management Branch of the Ministry of the Environment (see Appendix D).

Prior to, during and after testing, water levels were monitored in the wells. Table 2 summarizes the basic data relating to the testing. Pump test readings recorded during the testing and analysis of the data are included in Appendix C-1.

TABLE 2

PUMPING TEST DATA FROM TEST WELL PROGRAM

Observation Well Number	Aquifer Tested	Pump Depth (m)	Pumping Rate 1/s	U.S.gpm	Elapsed (min)	Drawdown (m)
OW1	49.09-55.89 (Deep)	48.0	1.28	20.4	300	33.44
OW2	30.28-32.92 (Middle)	29.0	2.25	35.7	200	11.50
OW3	23.24-25.91	22.5	4.0	63.6	90	3.97
	(Shallow)	22.5	5.31	84.0	110	5.77
		22.5	6.51	103.2	550	7.21
OW4	104.11-106.75	30.18	7.17	113.4	1500	4.97

During the pumping of observation well OW1, the water remained cloudy with fine silt in suspension throughout the test. The water from this well was considered unsuitable for supply. Observation well OW2 was pumped at a rate of 2.25 1/sec (35.7 U.S. gpm) for 200 min; drawdown stabilized after 30 min. Observation well OW3 was pumped at varying rates, with drawdown in all cases stabilizing after less than 100 min. Observation well OW4 was pumped at 7.1 1/sec (113.4 U.S. gpm), but the drawdown did not stabilize with time. On cessation of pumping, natural overflow returned within one minute.

During the pump testing of observation wells OW3 and OW4, water samples were collected and sent for analysis to Eco-Tech of Kamloops. The results are included in Appendix D.

2.5 Discussion of Results

Analysis of pump test data was carried out using Theis and Jacob methods of analysis. These analyses are included in Appendix C-2 and C-3 respectively. Table 3 summarizes the aquifer parameters obtained from the analyses.

TABLE 3

Summary of Test Well Pumping Test Results

Well No.	Transmissivity m^2/s Theis	Transmissivity m^2/s Jacob	Hydraulic Conductivity m/s Theis	Hydraulic Conductivity m/s Jacob	Estimated Aquifer Thickness (m)	Comments
OW1	1.67×10^{-5}	1.97×10^{-5}	3.6×10^{-6}	3.8×10^{-6}	5.2	No calculation of storage possible
OW2	5.77×10^{-5}	5.71×10^{-5}	2.4×10^{-5}	2.4×10^{-5}	2.4	No calculation of storage possible
OW3	3.66×10^{-4}	5.81×10^{-4}	4.5×10^{-5}	5.7×10^{-5}	6.4	No calculation of storage possible
OW4 Leg 1	---	8.74 x 10-3	---	---	Unknown	No fit possible
Leg 2	---	1.38 x 10-3	---	---	Unknown	to Theis curve

The deep aquifer in Marble Canyon (tested by OW1) was not considered suitable for water supply purposes due to the presence of fine silt and very limited production capability. The middle aquifer (tested by OW2) was considered to have insufficient production capability for the purpose of water supply to the proposed construction camps and batching plant. However, it was considered feasible for this well and aquifer to provide a limited supply (1.9 - 2.5 l/s, 30 to 40 U.S. gpm) to the B.C. Hydro office complex at the junction of Highway 12 and Hat Creek Road.

The results of testing OW3 indicated that the shallow aquifer had potential for at least a portion of the required supply, and this site was chosen for a production well location.

Test results from OW4 were somewhat disappointing due to the fact that stabilization did not occur during pumping. However, potential for supply, due mainly to the large available drawdown, was evident and this site was also chosen for a production well location.

3.0 PRODUCTION WELL PROGRAM

3.1 Drilling

The sites for the production wells were selected following the preliminary pump test program, the conclusions of which are given in Section 2.5. Two 8 inch (203 mm) diameter production wells (PW1 and PW2) were drilled between July 1st and 10th. Figure 1 shows the location of these wells.

The production well in the Hat Creek Valley did not intersect bedrock and was terminated at 116.8 m due to sand blocking the annulus between the casing and rods. Production well 2 (PW2) was drilled into a depth of 33.5 m. The well was drilled 3 m into limestone bedrock.

3.2 Aquifer Description

Production Well PW1 penetrated similar sediments to those encountered in the drilling of OW4. A sandy gravel aquifer was identified between 71.64 and 116.46 m, underlain by silty sand to 116.7 m. During drilling, air-lifted water flows of 10 - 15 l/s (158 - 317 U.S. gpm) were recorded between 100 and 106 m.

The production well in Marble Canyon (PW2) penetrated a coarse sandy gravel aquifer between 19.5 and 29.6 m yielding water flows up to 11 l/s (74 U.S. gpm). Little water was obtained from the limestone bedrock.

The hydrogeological logs for both wells are included in Appendix A. Representative grain size distributions of the aquifer materials in both wells are provided in Appendix B.

3.3 Completion

The selection of the well screens for the production wells was based on the same criteria as for the test wells. A well screen slot size that would retain 50 per cent of the formation was selected. The well screen slot sizes chosen for the production wells are shown on the hydrogeological logs (Appendix A) and in Table 4.

PW1 was completed with 9.66 m of 120 slot 203 mm telescopic stainless steel wire-wound well screen between 110.35 and 100.69 m. Completion of PW2 was with 3.25 m of 200 slot 203 mm telescopic stainless steel well screen between 25.93 and 29.18 m. In both wells, a sediment trap was incorporated beneath the well screen while a riser pump and K-packer were welded above the screen.

Only PW2 was backfilled with gravel and sealed with bentonite below the screen. In PW1, the silty sand formation was allowed to cave into the hole beneath the tail-pipe. As in the completion of the test wells, the pull-back method was used to install the well screens. Both wells were developed by surging with air until the flushed water was sand free. Finally, the tail-pipes were bailed to remove any sediment trapped. Completion details are summarized in Table 4.

TABLE 4

SUMMARY OF PRODUCTION WELL COMPLETION DATA

Well No.	Screened Interval	Slot Size (thousandth inch)	Screen Type
PW1	110.35-100.69	120	Stainless Steel Wire-Wound
PW2	25.93-29.18	200	Stainless Steel Wire-Wound

3.4 Testing

Pump testing of production well PW1 commenced on July 20th, 1981 and continued through to July 28th, 1981. The well was pumped at a constant rate of 26.5 l/sec (420.7 U.S. gpm) for 11,890 min, with the pumped water discharged into Hat Creek. Water samples were taken at the start of and during the test. These were sent to Eco-Tech Laboratories of Kamloops for chemical analyses. The results are indicated in Appendix D and are discussed in Section 4.0.

Production well 2 was pump tested between July 24th and 28th, 1981. A pump rate of 6.3 l/sec (100 U.S. gpm) was intially selected; this was subsequently increased to 8.5 l/sec (135 U.S. gpm) to obtain maximum information from the test. A pumping rate of 9.46 l/s (150 U.S. gpm) was attempted to further increase drawdown in the well. This resulted in an electrical breakdown and hence the pump was throttled back to 8.5 l/s (135 U.S. gpm). The pumped water was discharged into Hat Creek.

During the pump testing of both production wells, drawdown was monitored in the pumped wells and nearby test wells. This data is contained in Appendix C-1.

Pump test hydrographs for both tests are included as Figures 3 and 4.

3.5 Discussion of Results

Data from the production well pump tests were again analyzed by Theis and Jacob methods. Analyses are included in Appendix C-2 and C-3 and the results are summarized in Table 5.

The results of testing PW1 showed that at a rate of 26.5 l/sec (420.7 U.S. gpm) stabilization was not approached. This aquifer does not appear to have abundant recharge and use for supply purposes will be limited by this fact. The choice of pumping rate for the operation of the well PW1 has been based on the pumping test record but as it could not be demonstrated that sufficient recharge was occurring to the aquifer, the optimum substainable pumping rate cannot be selected with total confidence. For this reason, the recommended pumping rate is lower than the test pumping rate and monitoring of well performance will be necessary during operation.

The geometry of the aquifer pumped by PW1 near the rim of the proposed pit is currently unknown. It is considered advisable that further investigation be carried out on the ground water regime in this area to ascertain whether there could be any adverse ground water impact on the proposed mine.

Results of testing PW2 showed that stabilization occurred at a pumping rate of 8.5 l/sec (135 U.S. gpm). Boundary effects were minimal since observation data indicate the cone of drawdown to be steep with a radius of influence not approaching the bedrock boundaries for this particular pumping rate.

After consideration of location, uses, aquifer parameters, available drawdown and possible interference effects, recommendations on pumping rates and uses for the production and observation wells were determined. Table 6 summarizes these recommendations.

TABLE 5

Summary of Production Well Pumping Test Results

Well No.	Transmissivity m^2/s Theis	Transmissivity m^2/s Jacob	Hydraulic Conductivity m/s Theis	Hydraulic Conductivity m/s Jacob	Storativity Theis	Storativity Jacob	Estimated Aquifer Thickness (m)	Comments
PW1 Leg 1	---	1.07×10^{-2}	---	---	---	---	Unknown	No fit possible
Leg 2	---	2.69×10^{-4}	---	---	---	---	Unknown	to Theis curve
OW4 Leg 1	---	9.7×10^{-3}	---	---	---	---	Unknown	No fit possible
Leg 2	---	2.75×10^{-4}	---	---	---	7.71×10^{-2}	Unknown	to Theis curve
PW2	2.27×10^{-4}	2.05×10^{-4}	2.27×10^{-5}	2.05×10^{-5}	---	---	10.0	
OW3	1.35×10^{-3}	2.17×10^{-3}	2.1×10^{-4}	3.39×10^{-4}	1.98×10^{-4}	1.52×10^{-4}	6.4	

The proposed methods and rates of pumping in the various wells are based on facts acquired during the study and on some assumptions. When production pumping begins, it will be necessary to monitor well performance in the initial days of operation and to 'fine-tune' the recommendations. It is proposed that an allowance should be made for the services of an hydrogeologist at that stage to make these observations and adjustments.

TABLE 6

WATER SUPPLY RECOMMENDATIONS

Well No.	(U.S.gpm)	(1/sec)	Recommended Use
PW1	312	19.7	Permanent supply for construction camp and batching plant.
PW2	150	9.5	Permanent supply for construction camp and batching plant.
OW4	100	6.3	Temporary supply for construction camp and batching plant. To be used for peak demand periods and/or emergency purposes.
OW3	96	6.1	Temporary supply for construction camp and batching plant. To be used for peak demand periods and/or emergency purposes.
OW2	36	2.3	Permanent supply for office complex.

4.0 WATER QUALITY

Water samples were collected from all wells either during drilling or during pump testing. The samples were kept chilled until delivered to Eco-Tech in Kamloops, where they were submitted for routine chemical analysis. The results are included as Appendix D.

Results for waters from the sand and gravel aquifers of Marble Canyon indicate an alkaline, moderately hard water, low in total dissolved solids, whereas waters from the Hat Creek aquifer are slightly softer with a lower dissolved solids. An apparent anomaly is seen in the high arsenic levels recorded in the water from OW4. Water analysed from PW1, pumping from the same aquifer, shows negligible arsenic and it is considered that contamination of the sample is responsible for the anomaly. For the parameters tested, the samples meet the limits set by "Guidelines for Canadian Drinking Water, 1978", published by authority of Health and Welfare, Canada. We recmmmend that BCH sample drinking water on a weekly basis for bacteriological parameters. Full chemical analyses of the drinking water should be undertaken once every six months.

Silica and potassium were not determined, but for the parameters analysed, it is considered that the water is suitable for concrete mixing as specified in 'Design and Control of Concrete Mixtures' published by the Canadian Portland Cement Association. We recommend that BCH extend the analyses currently carried out by including silica and potassium.

5.0 PERMANENT PUMP INSTALLATION

Using recommended maximum pumping rates from Table 6 and estimated total dynamic head requirements, appropriate submersible pumps and fittings were obtained and installed.

Figures 5 to 9 provide details of installations. Installation in PW2 was carried out on October 5th, 1981, just prior to commencement of a long term environmental pump test in this well (the subject of a separate report). Installation in OW2 occurred one day later on October 6th, 1981. Installation in OW4 and OW3 was carried out on November 5th and 6th, 1981, respectively.

At the time of writing, only the drop pipe had been obtained for PW1. Recommendations for pump type to be obtained have been made in Figure 8. The pump for this well will require 3-phase power which is not yet available on site. No long term pumping test on this production well is considered to be necessary for environmental purposes because the depth of the aquifer below creek level and the nature of the sediments overlying that aquifer, preclude the possibility of any influence by the production pumping on creek flows.

The detailed manufacturers' specifications for the recommended pumps are included as Appendix D.

We thank you for the opportunity of carrying out this interesting work.

Yours very truly,

GOLDER ASSOCIATES

G.E. Rawlings

G.E. Rawlings, P. Eng.

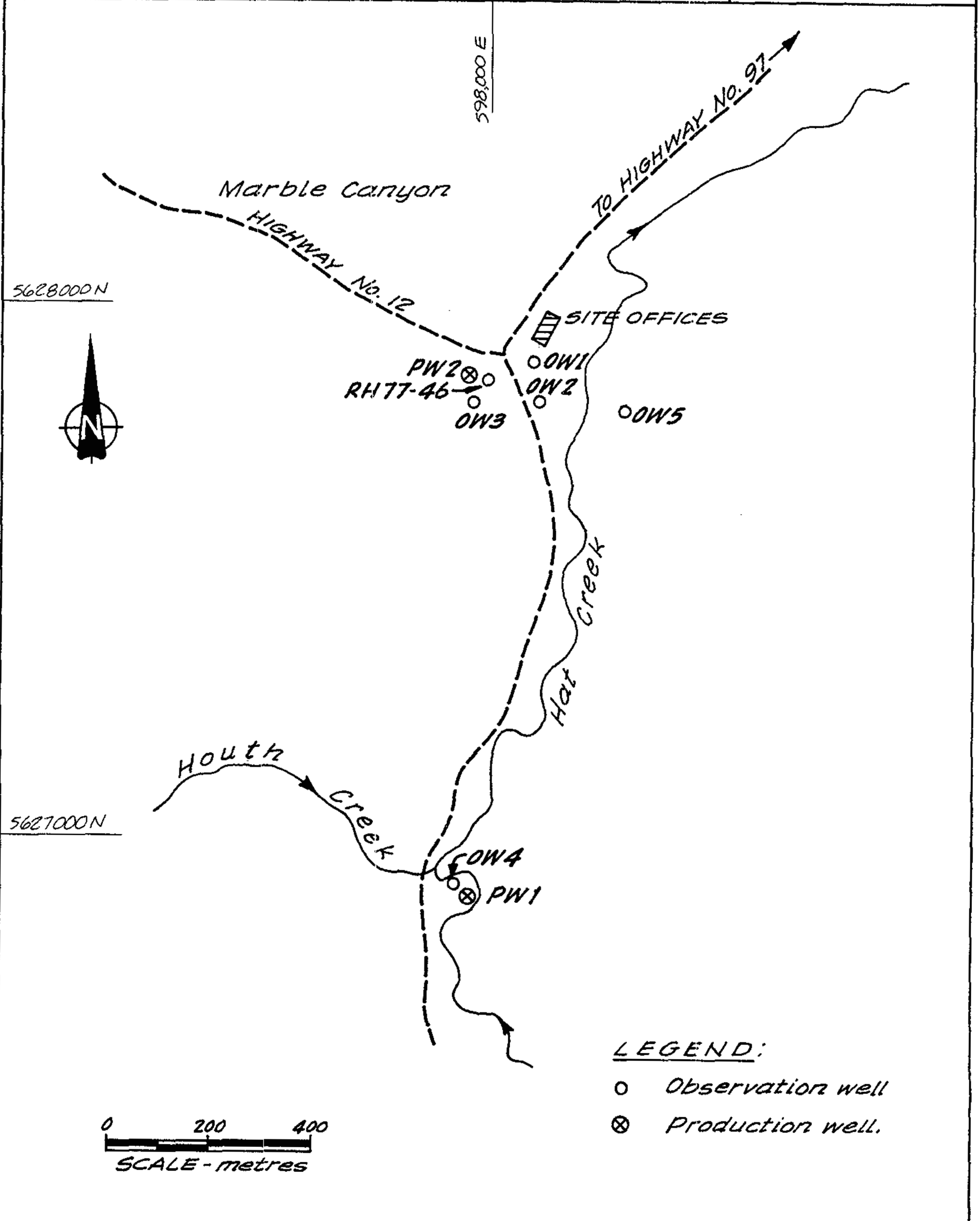
Rick Guiton

R. Guiton

GER/RG/bjh
812-1507

WELL LOCATION PLAN-
HAT CREEK CONSTRUCTION CAMP
WATER SUPPLY.

Figure 1



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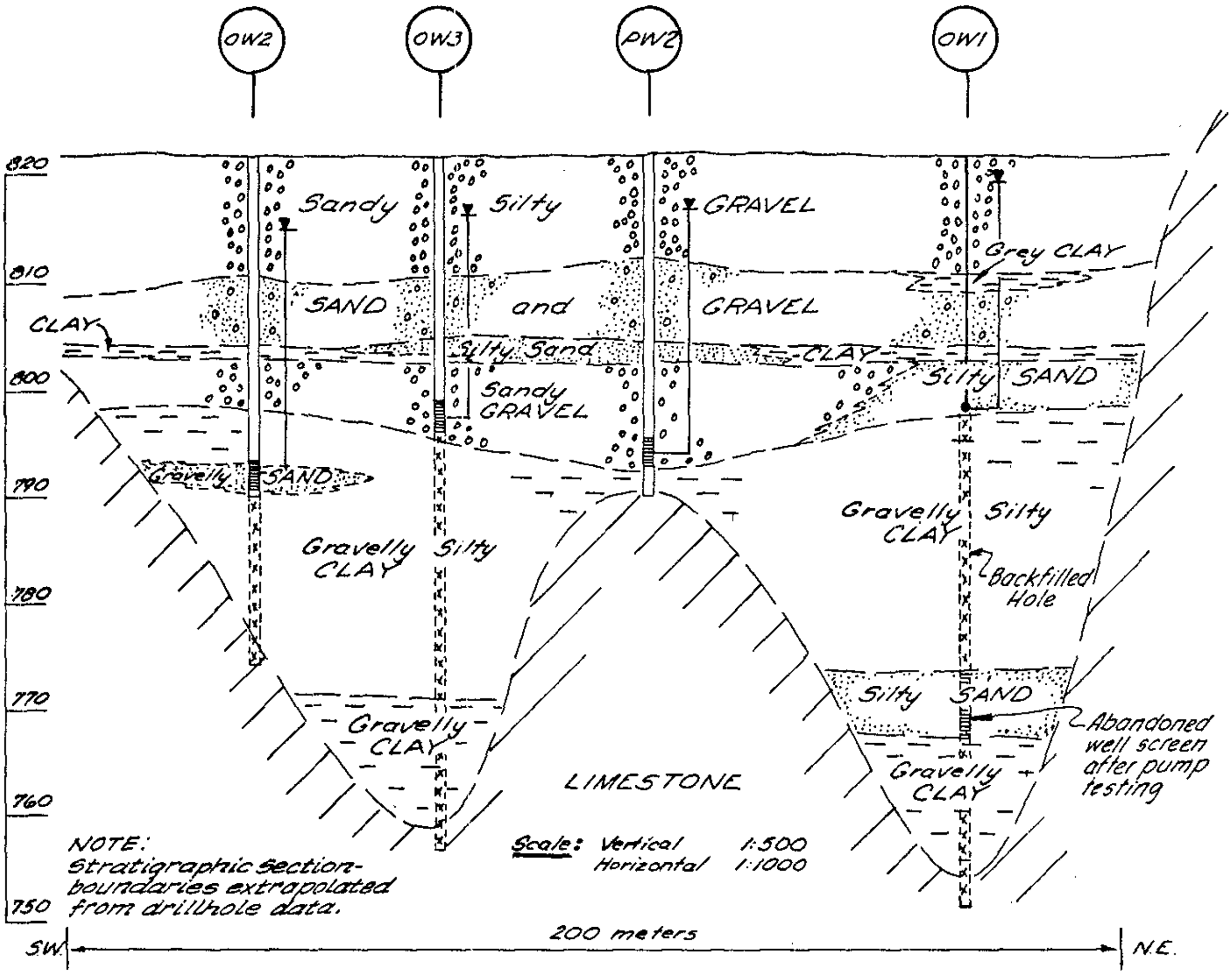
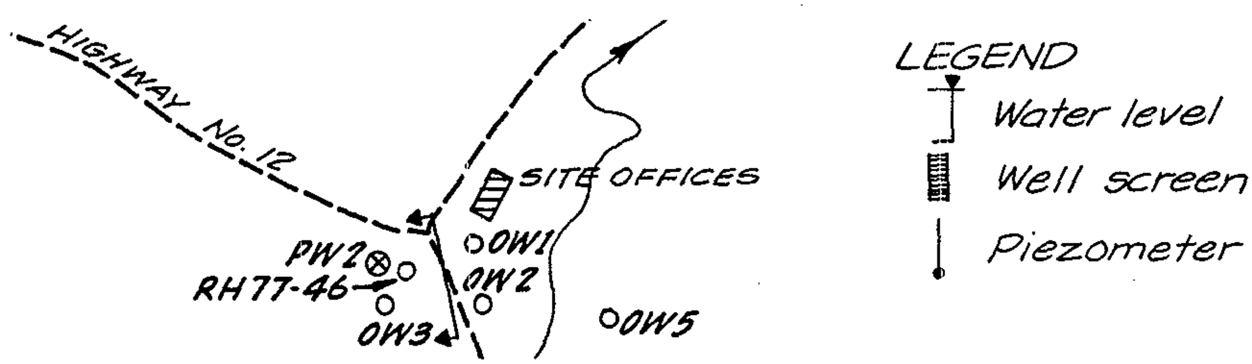
SCHEMATIC SECTION - MARBLE CANYON AQUIFER SYSTEM

Figure 2



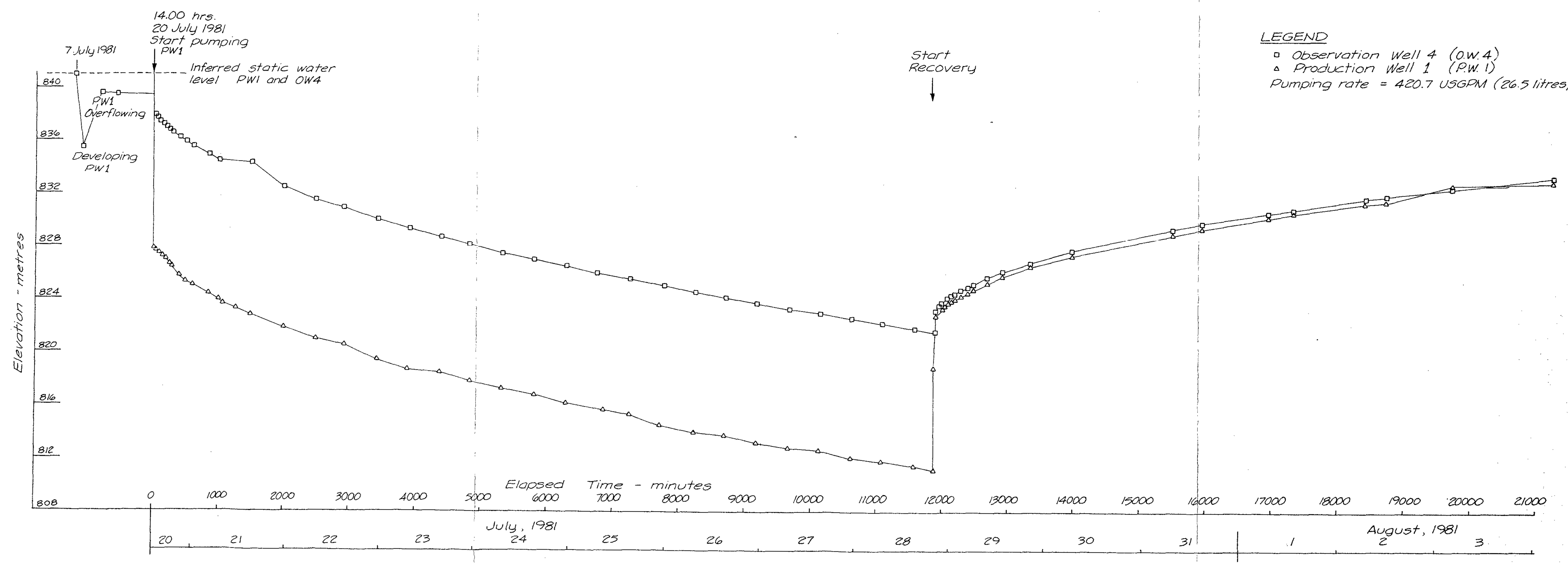


Location of Section

PROJECT NO. 812-1507 DRAWN R.O. REVIEWED DATE July '81

PUMP TEST HYDROGRAPHS
HAT CREEK CONSTRUCTION CAMP
WATER SUPPLY

Figure 3



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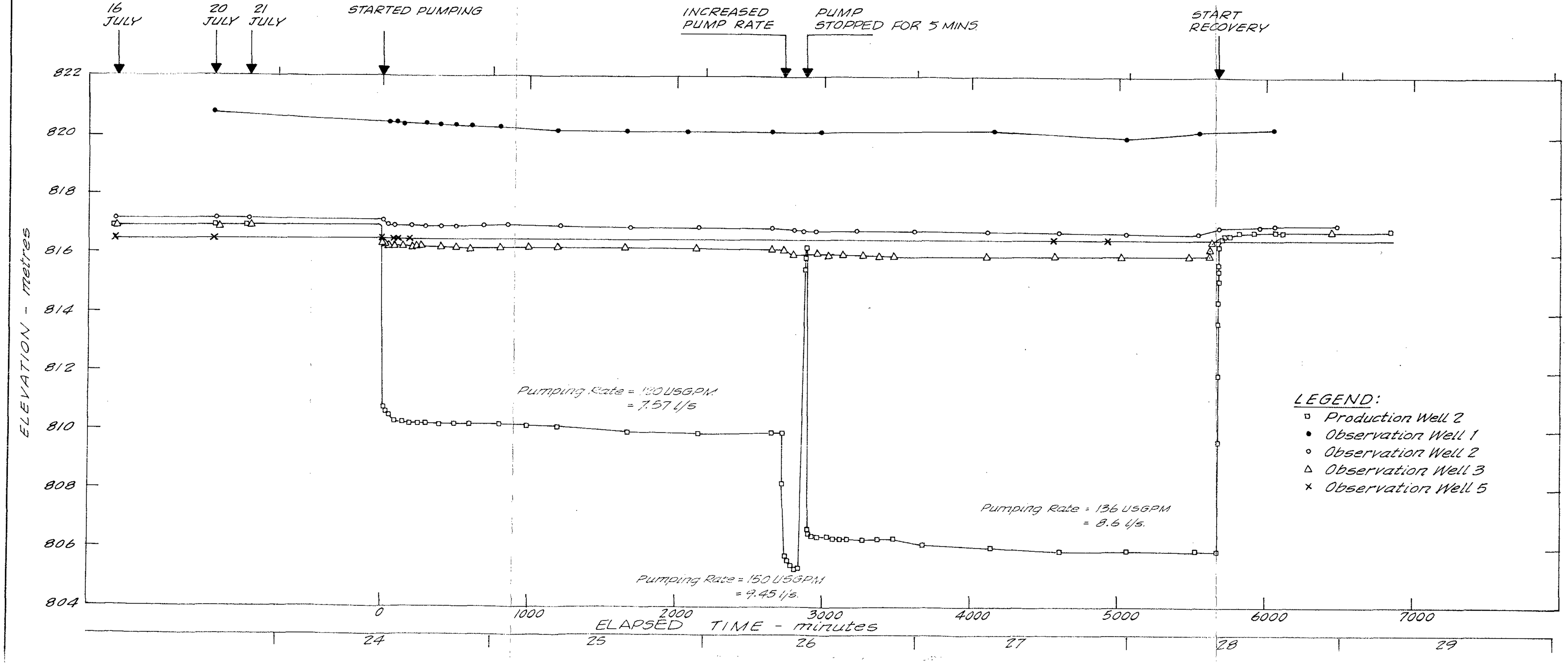
Drawn KD
Reviewed
Date Oct '81

8121507

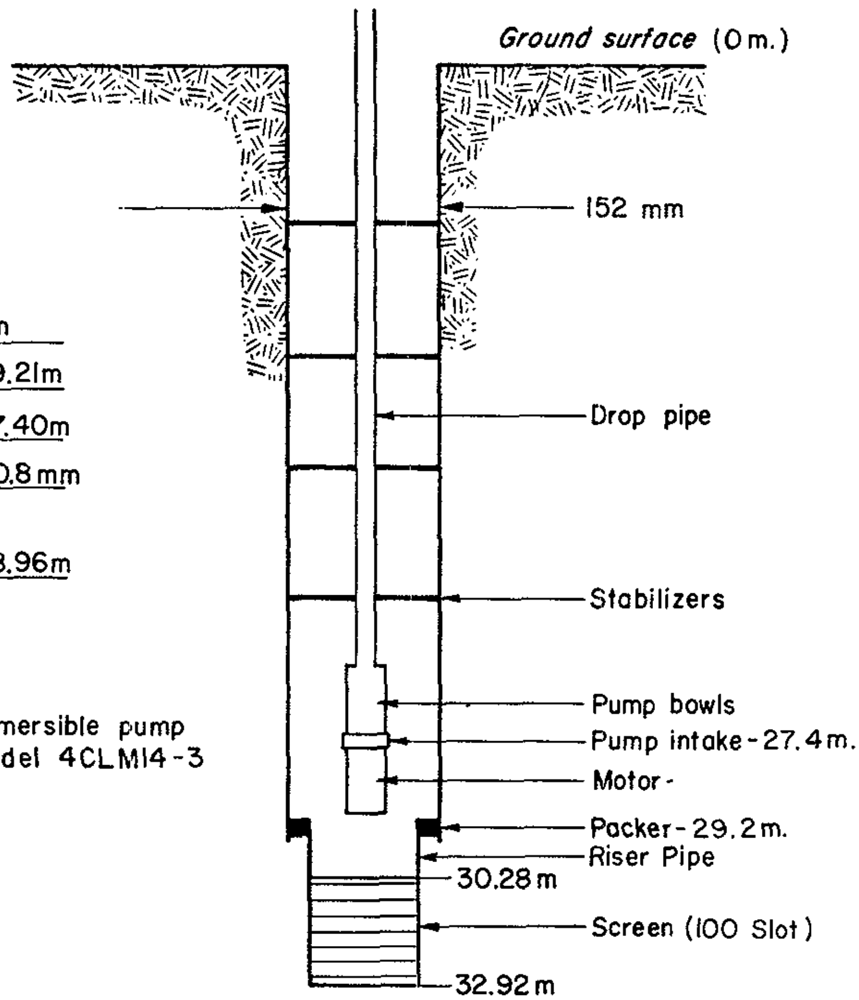
PUMP TEST HYDROGRAPHS
HAT CREEK CONSTRUCTION CAMP
WATER SUPPLY

Figure 4



PUMP INSTALLATION DATA - WELL No. OW2

Figure **5**



DETAILS:

Casing diameter: 152 mm

Depth to top of packer: 29.21m

Depth to pump intake: 27.40m

Diameter of drop pipe: 50.8 mm

Estimated total dynamic head to surface: 28.96m

Berkeley submersible pump
Model 4CLM14-3

NOTE: All depths measured from ground surface.

NOT TO SCALE

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PUMP INSTALLATION DATA - WELL No. OW3

Figure **6**

DETAILS:

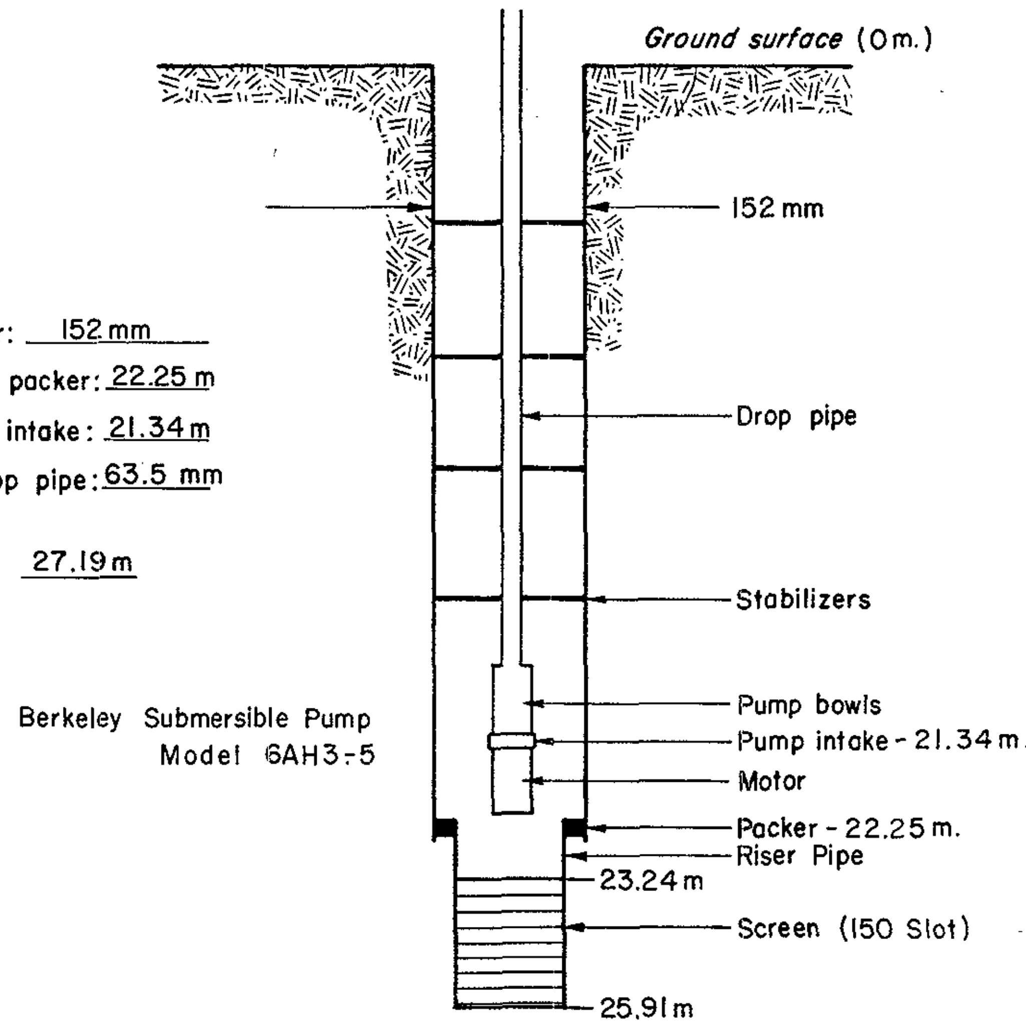
Casing diameter: 152 mm

Depth to top of packer: 22.25 m

Depth to pump intake: 21.34 m

Diameter of drop pipe: 63.5 mm

Estimated total dynamic head: 27.19 m



NOTE: All depths measured from ground surface.

NOT TO SCALE

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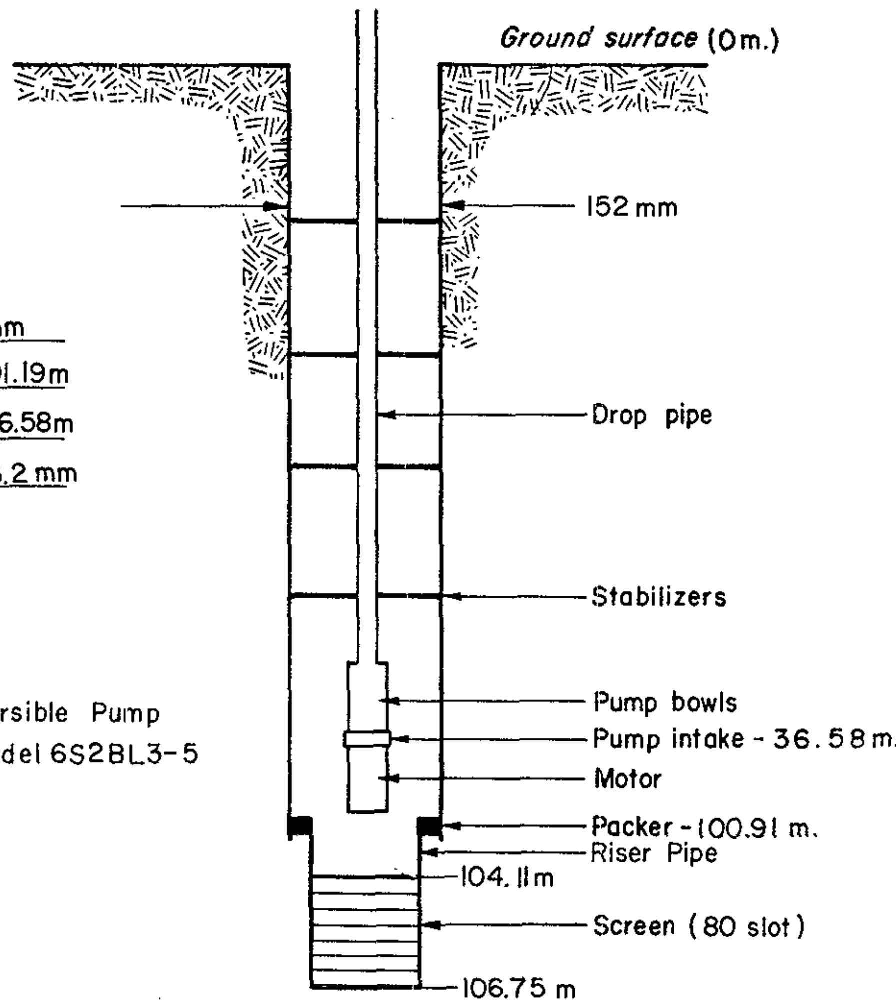
PUMP INSTALLATION DATA - WELL No. OW4

Figure **7**



DETAILS:

Casing diameter: 152mm

Depth to top of packer: 101.19m

Depth to pump intake: 36.58m

Diameter of drop pipe: 76.2 mm

Estimated total dynamic head : 42.12m

Berkeley Submersible Pump Model 6S2BL3-5

NOTE: All depths measured from ground surface.

NOT TO SCALE

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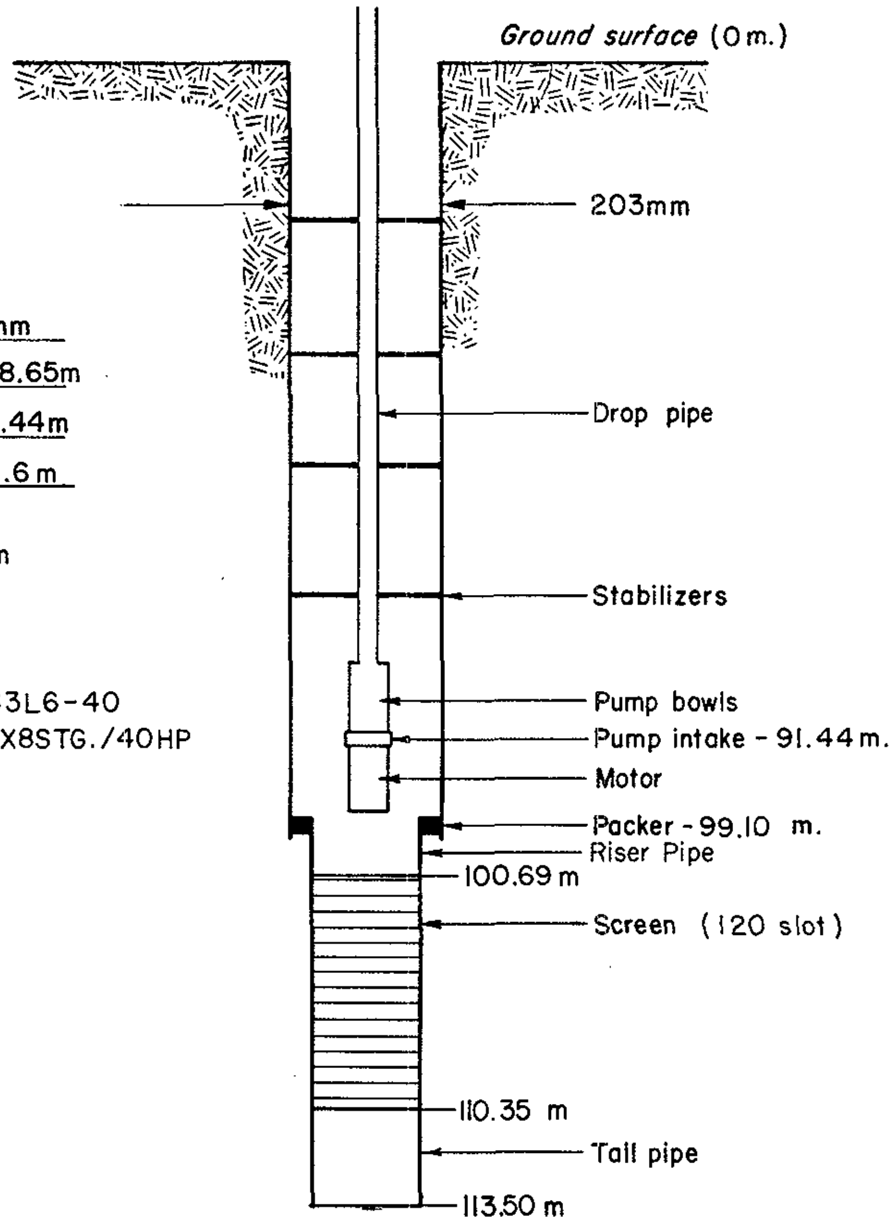
PUMP INSTALLATION DATA - WELL No. PW1

Figure **8**



DETAILS:

Casing diameter: 203mm

Depth to top of packer: 98.65m

Depth to pump intake: 91.44m

Diameter of drop pipe: 101.6 m

Estimated total dynamic head: 104.06 m

Recommended pumps:

1. Berkeley submersible 7S3L6-40
2. Goulds submersible UHX8STG./40HP

NOTE: All depths measured from ground surface.

NOT TO SCALE

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PUMP INSTALLATION DATA - WELL No. PW2	Figure 9

DETAILS:

Casing diameter: 203 mm

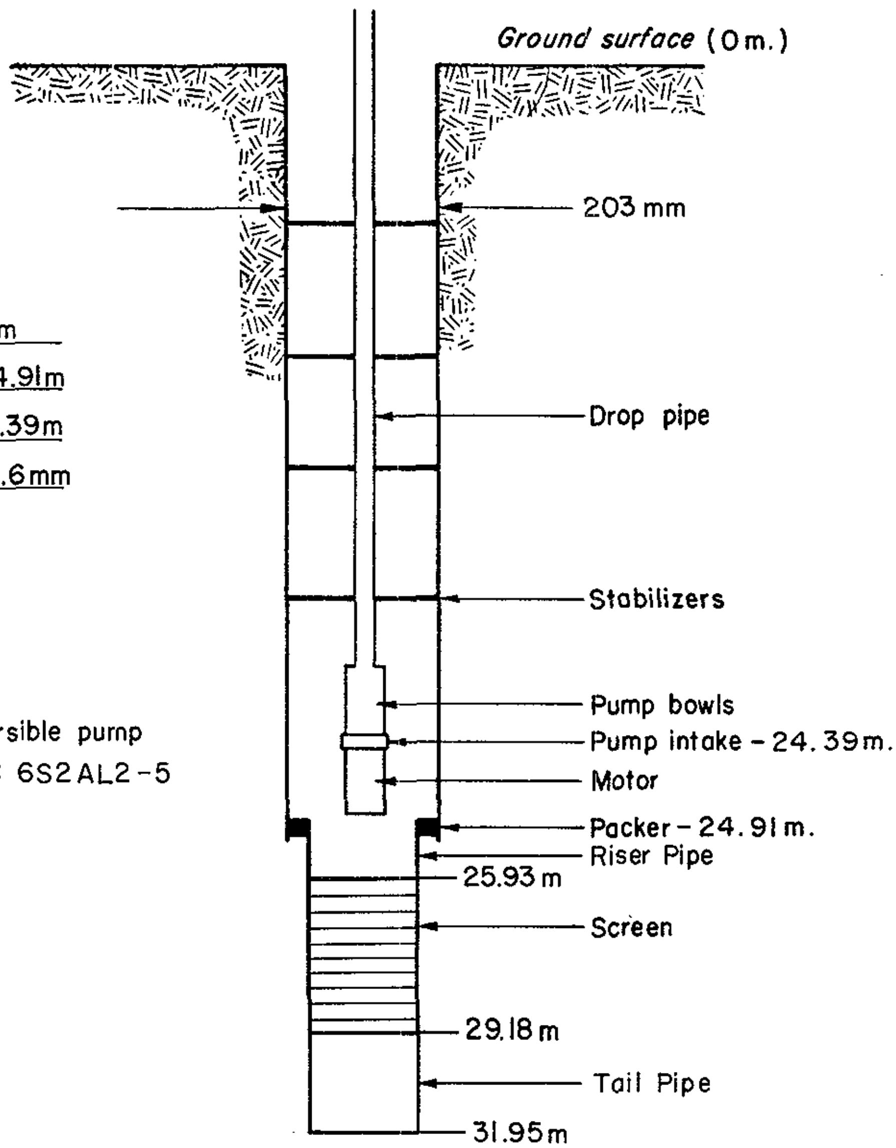
Depth to top of packer: 24.91 m

Depth to pump intake: 24.39 m

Diameter of drop pipe: 101.6 mm

Estimated total dynamic head: 28.35 m

Berkeley submersible pump
Model : 6S2AL2-5



NOTE: All depths measured from ground surface.

NOT TO SCALE

PROJECT NO. 812/507 DRAWN .... REVIEWED .... DATE ....

APPENDIX A

HYDROGEOLOGICAL LOGS

APPENDIX A

1.0 HYDROGEOLOGICAL LOGS: 1980 CONSTRUCTION CAMP WATER SUPPLY

The following hydrogeological logs summarize information on all boreholes where subsurface hydrological data has been obtained.

In order to show all data in a compact log, it was necessary to use a number of abbreviations and a symbolic notation. The following notes explain these abbreviations. The note numbers refer to the numbers shown in parenthesis at the head of each column in the logs.

2.0 REFERENCE ELEVATION

All depth measurements are given in meters relative to surveyed ground level.

(1) Lithologic Terminology Used in Logs

Lithology of boreholes used in 1980 has been determined from hydrogeologists' field descriptions of rotary cuttings.

(2) Completed Construction

a) Hole

drilled hole casing removed

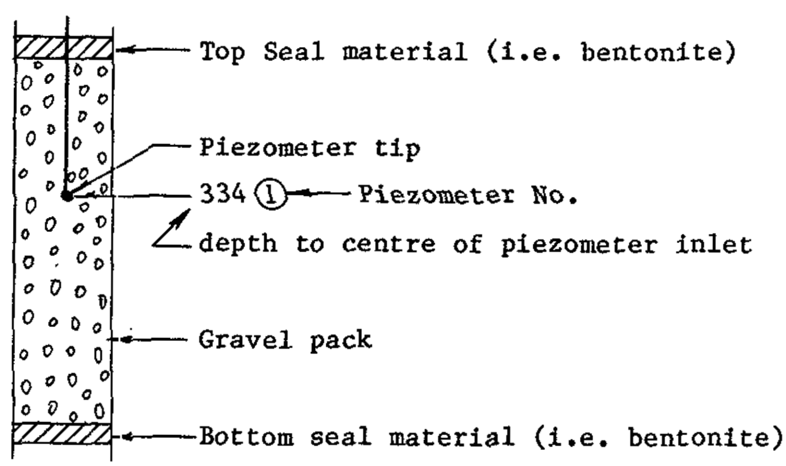
drilled hole casing left in place

drilled open hole

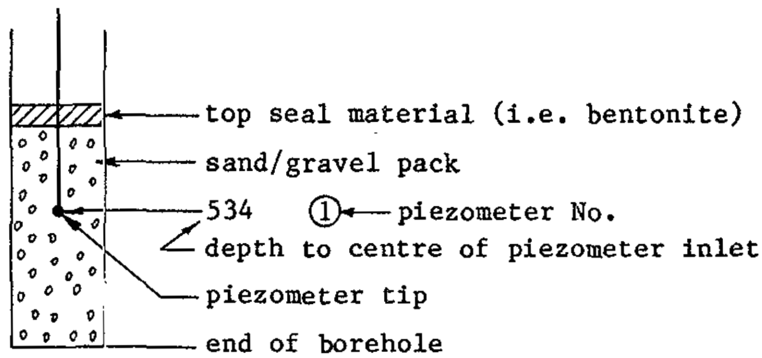
drilled hole known to have caved or squeezed

b) Piezometer

Standard Double Seal Piezometer Arrangement

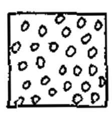


Standard Top Seal Piezometer Arrangement

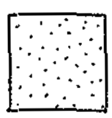


Type of Piezometer Tip: - perforated 25 mm Ø PVC pipe approx. 1.2 m long), wrapped with permeable fabric.

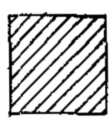
c) Types of Backfill



Gravel

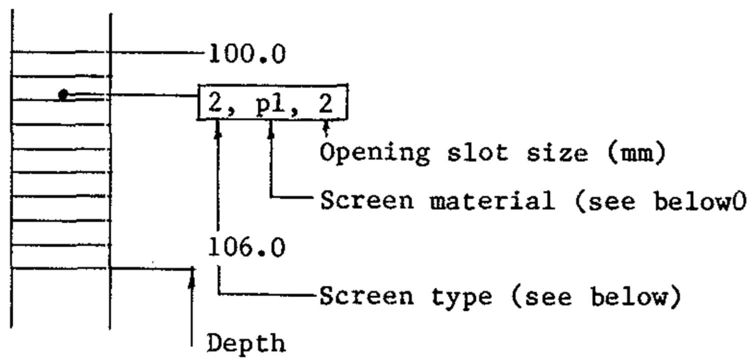


Sand



Bentonite

d) Well Screen



Screen type:	1)	Continuous wire wound
	2)	Slotted horizontally
	3)	Slotted vertically
	4)	Louvered openings

Screen Material	pl = plastic
	st = steel
	ss == stainless steel

(4) Water Level:

Water level measured when drilling had reached indicated depth (metres).

(5) Water Flow:

Water flows recorded while drilling was in progress (litre per second).

(6) Other:

fl = water flowing over top of casing

lw = losing water

Mw = making water

EC = electrical conductivity of water sample in micro mhos/cm

pH = field pH measurement of water sample

(7) Water Level:

Water level measured in piezometer on the date indicated in the comments column. The top number indicates water level with respect to reference elevation (positive values indicate artesian heads). The bottom number is the elevation of th total piezometric head.

(8) Permeability:

Depth = Depth range for permeable test (metres)

Method = Method used to determine hydraulic conductivity

Value = hyraulic conductivity determined metres per second (m/s)

fh = Falling head test in piezometer

rh = Rising head test in piezometer

pt = Pumping test

APPENDIX B

GRAIN SIZE DISTRIBUTION

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. OW1

Sample No.			
Notation	×	o	
Depth (M)	58.32	59.45	
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	0.14	0.14	
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

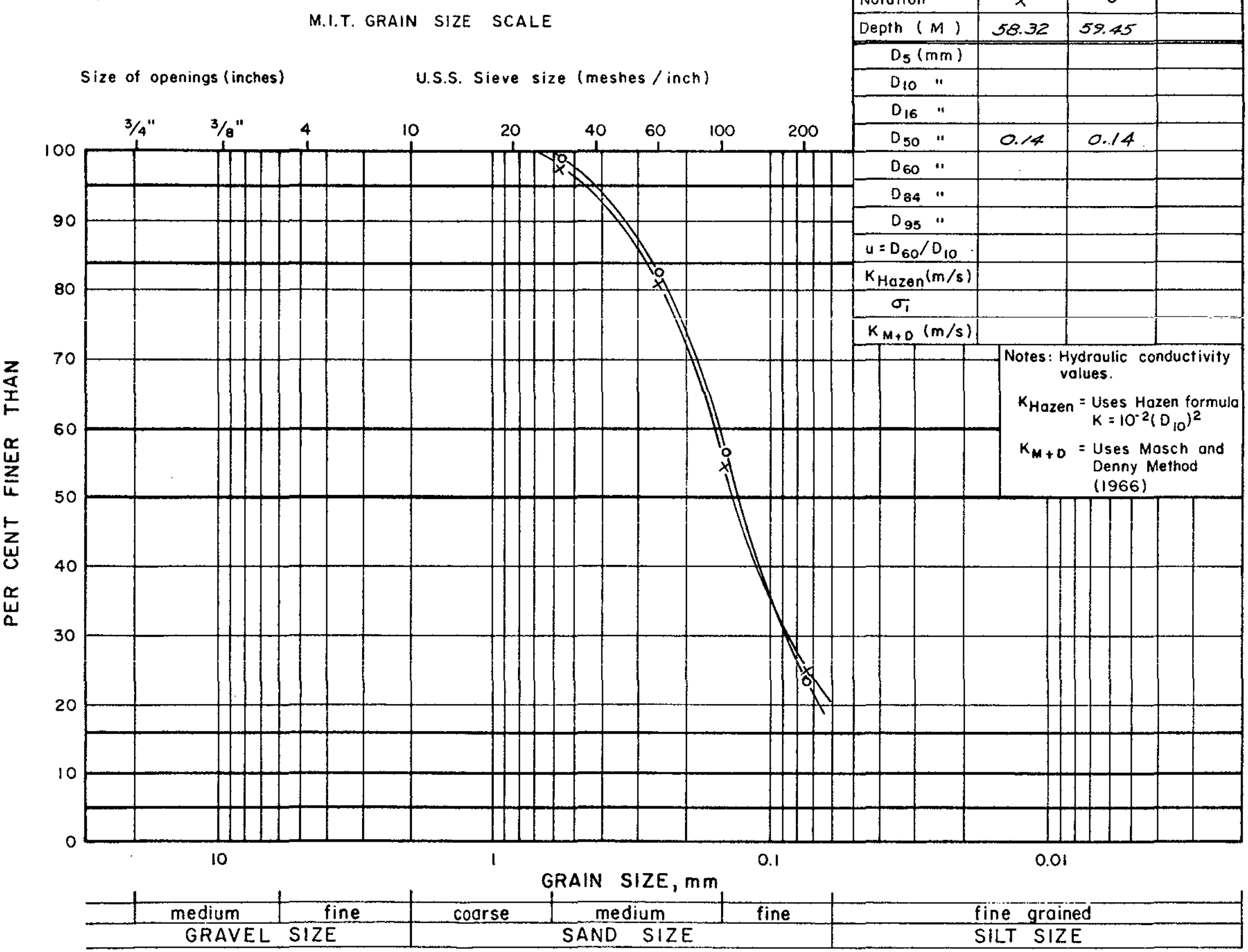


Figure B-1

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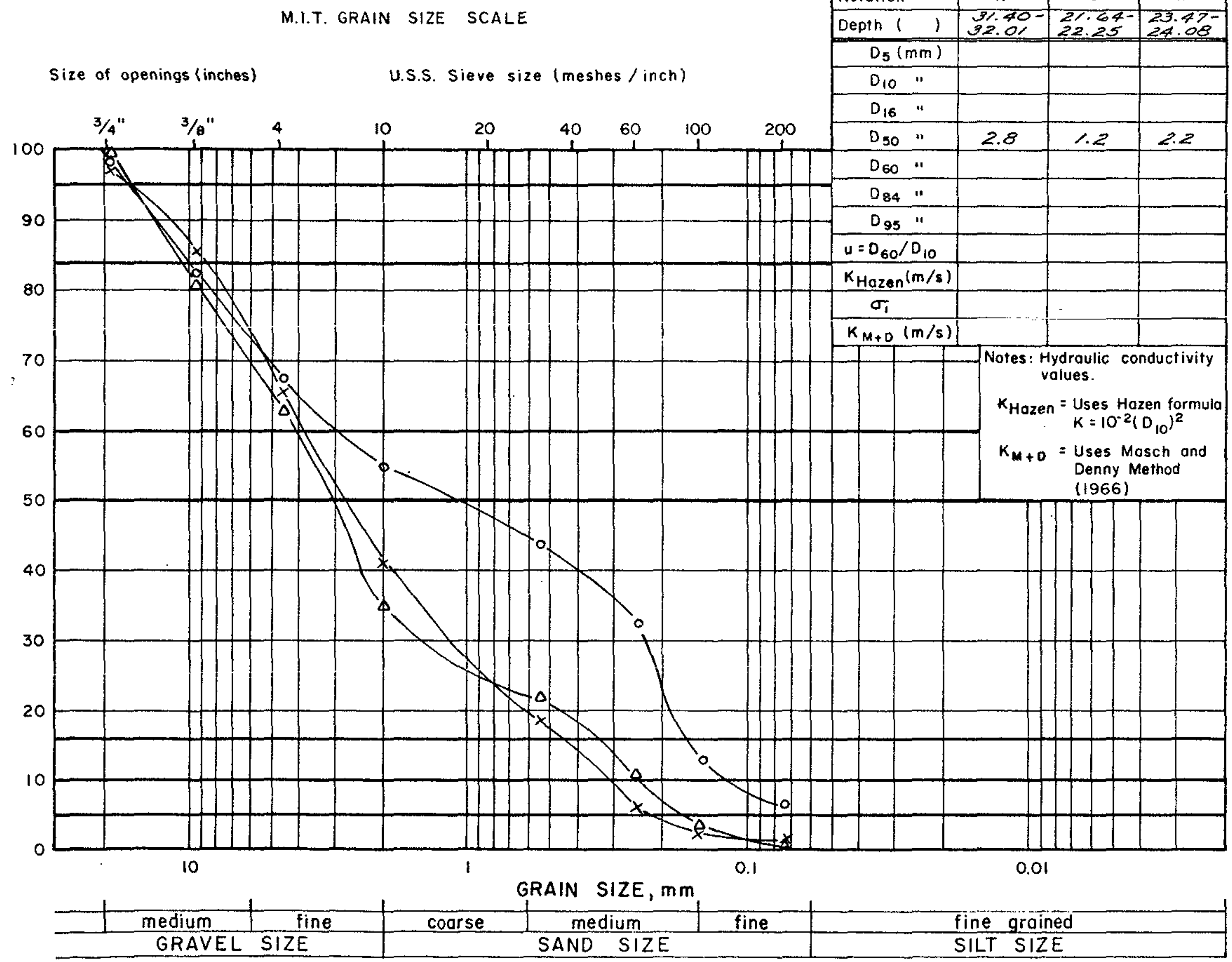
Project No. 812-1507 Drawn MB Reviewed ____ Date July, '81



Sample No.			
Notation	x	o	Δ
Depth ( )	31.40-32.01	21.64-22.25	23.47-24.08
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	2.8	1.2	2.2
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_i			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. OW2

Figure B-2

Golder Associates

Project No. 812-1507   Drawn M.B.   Reviewed ....   Date July, '81

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. OW2

Sample No.			
Notation	x	o	
Depth (M)	18.59 - 19.20	17.37 - 17.98	
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	3.8	3.8	
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

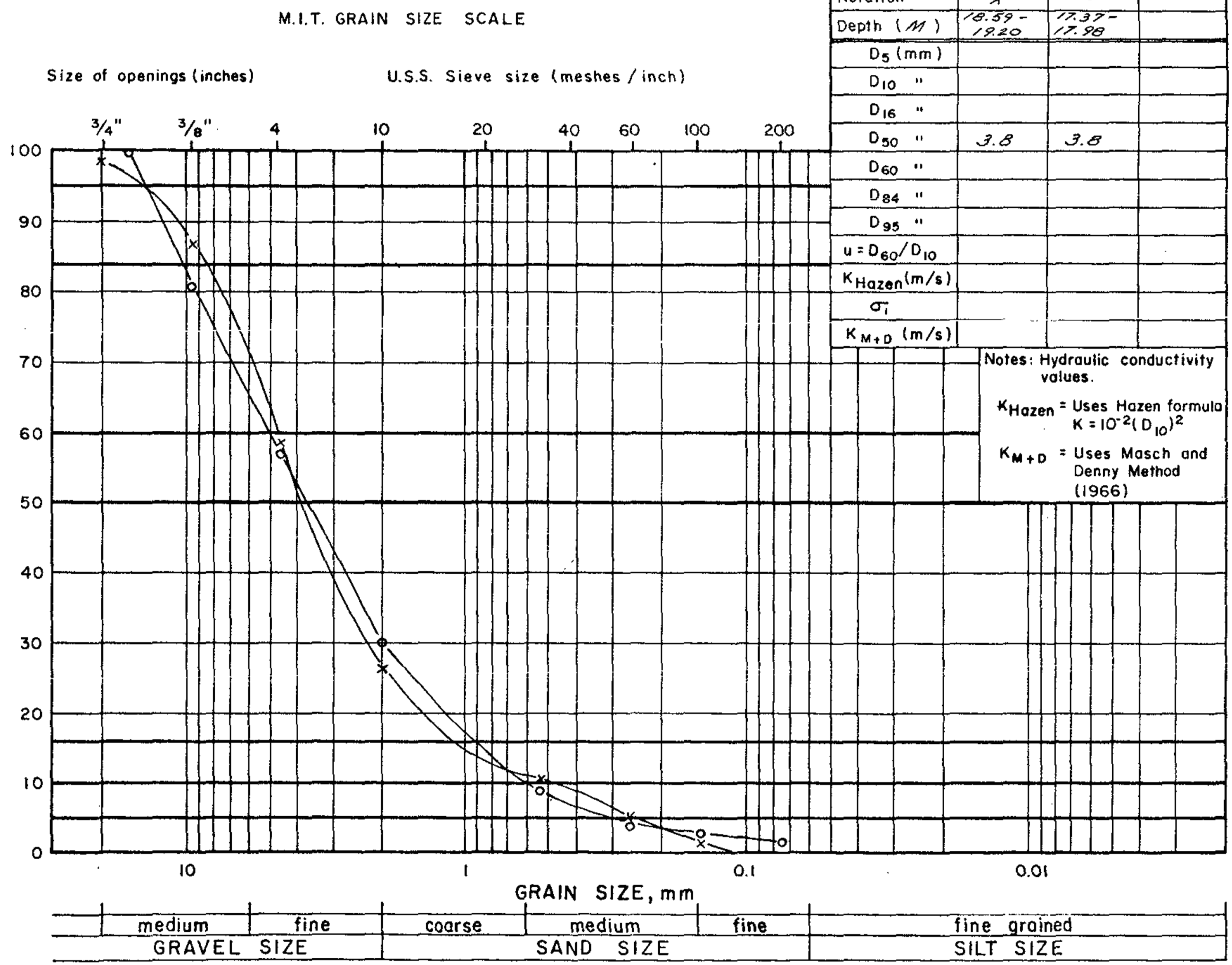


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Figure B - 3

812 1507 Drawn MB Date July, '81

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. OW 3

Sample No.			
Notation	x	o	
Depth (M)	21.34 - 21.95	24.29 - 25.0	
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	3.8	7.5	
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_i			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

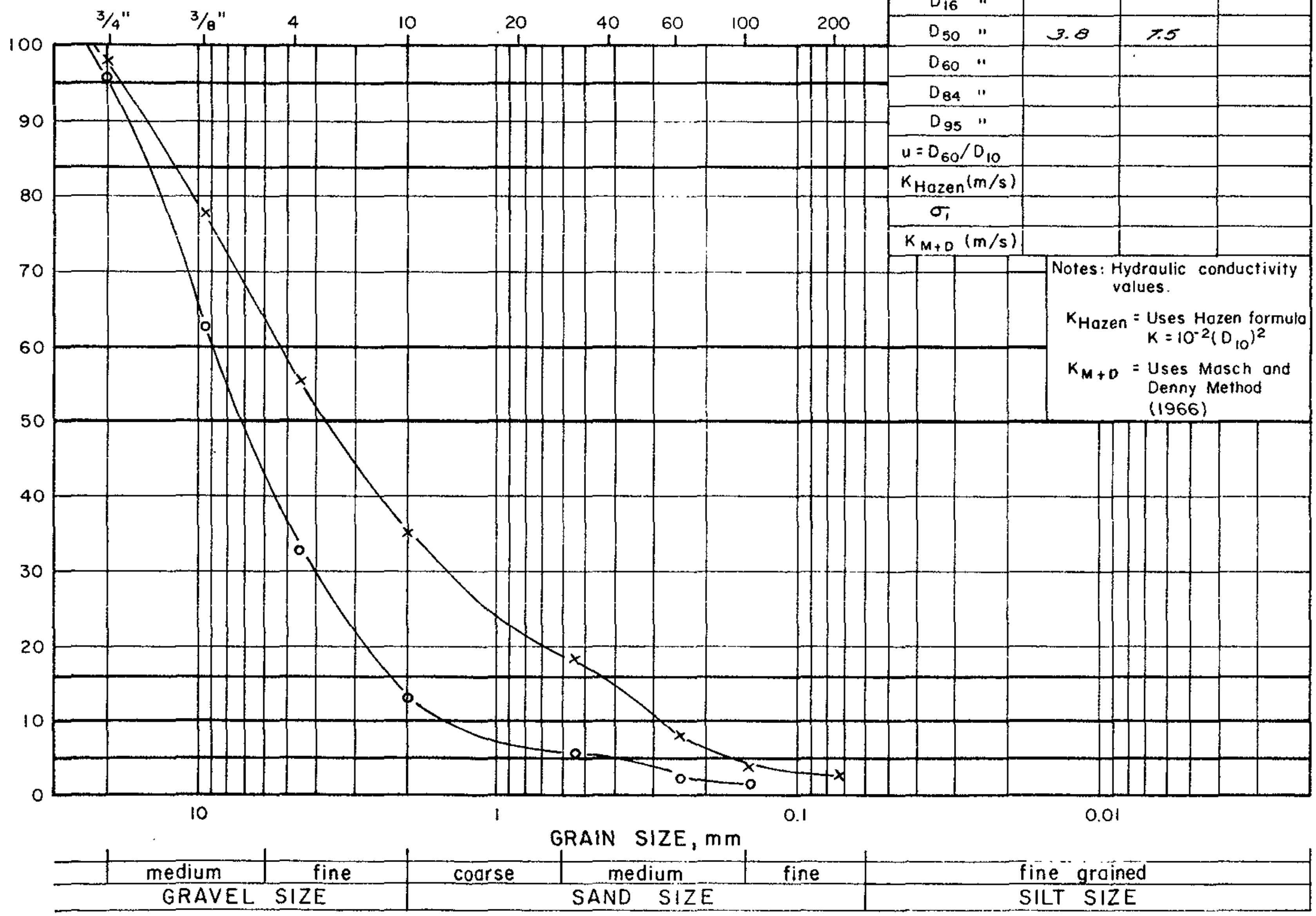

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Figure B-4

Project No. 812-1507 Drawn MB Reviewed .... Date July, '81

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. OW4

Sample No.			
Notation			
Depth (M)	109.75-110.36	104.87-105.48	101.21-101.82
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	2.0	4.0	1.6
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

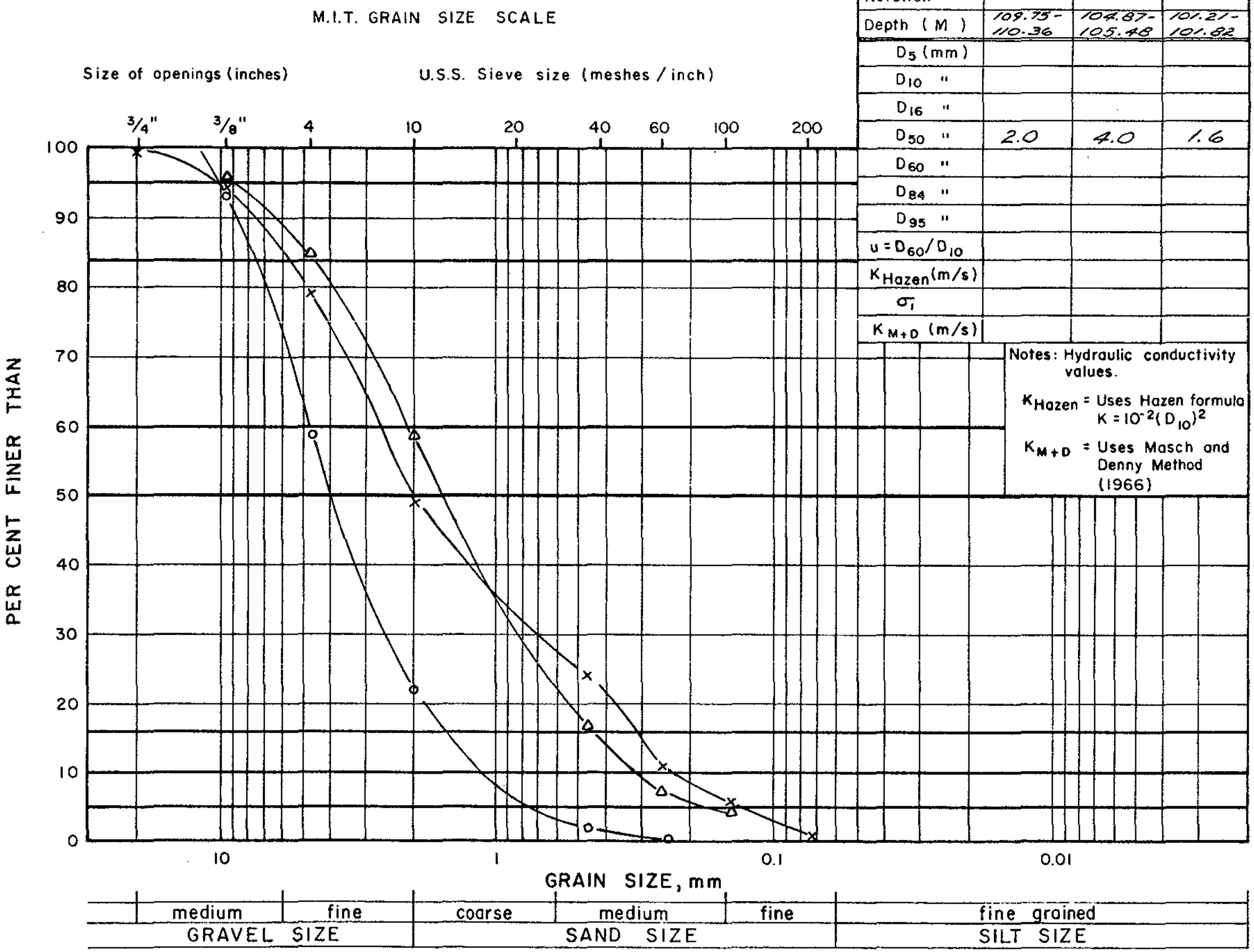

Figure B-5

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Project No. 812 1507 Drawn MB Reviewed ...... Date July, '81

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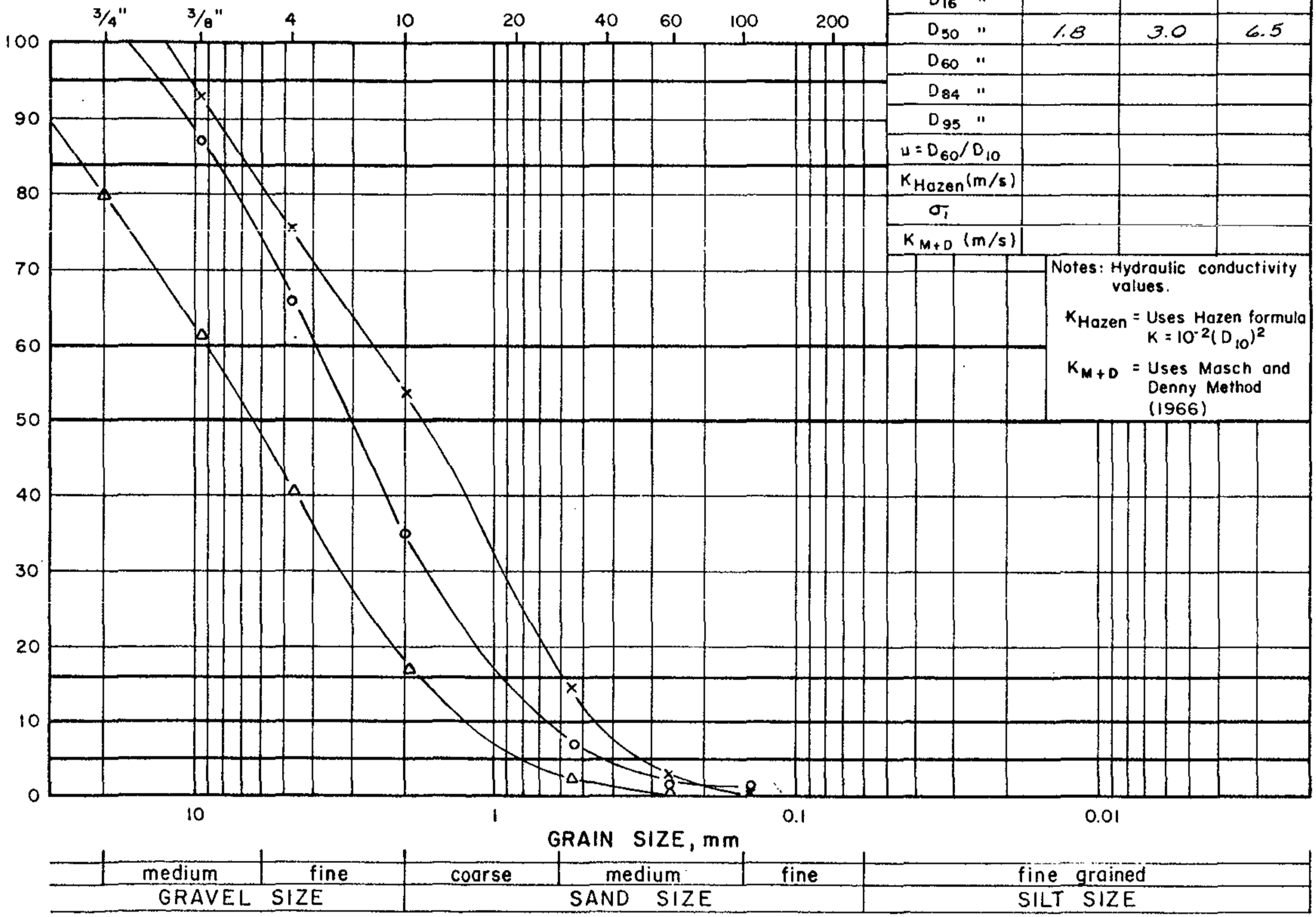
M.I.T. GRAIN SIZE SCALE

Sample No.			
Notation	x	o	Δ
Depth (M)	104.87-105.48	106.70-107.31	100.61-101.22
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	1.8	3.0	6.5
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)



Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. PW1

Figure B-6

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. PW1

Project No. 812-1507 Drawn MB Reviewed .... Date July, '81

Sample No.			
Notation	x		
Depth (M)	106.70 - 107.31		
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	8.0		
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

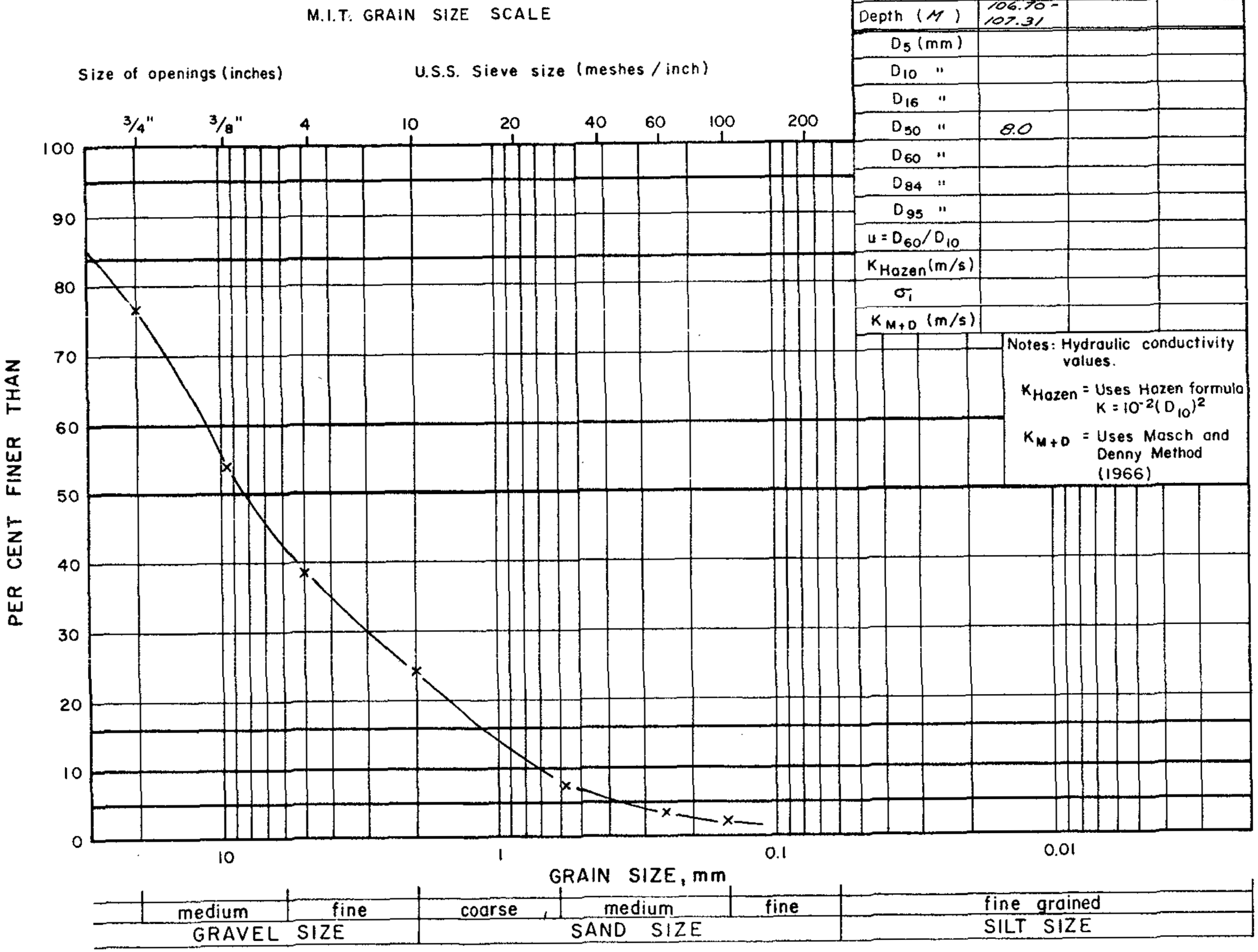


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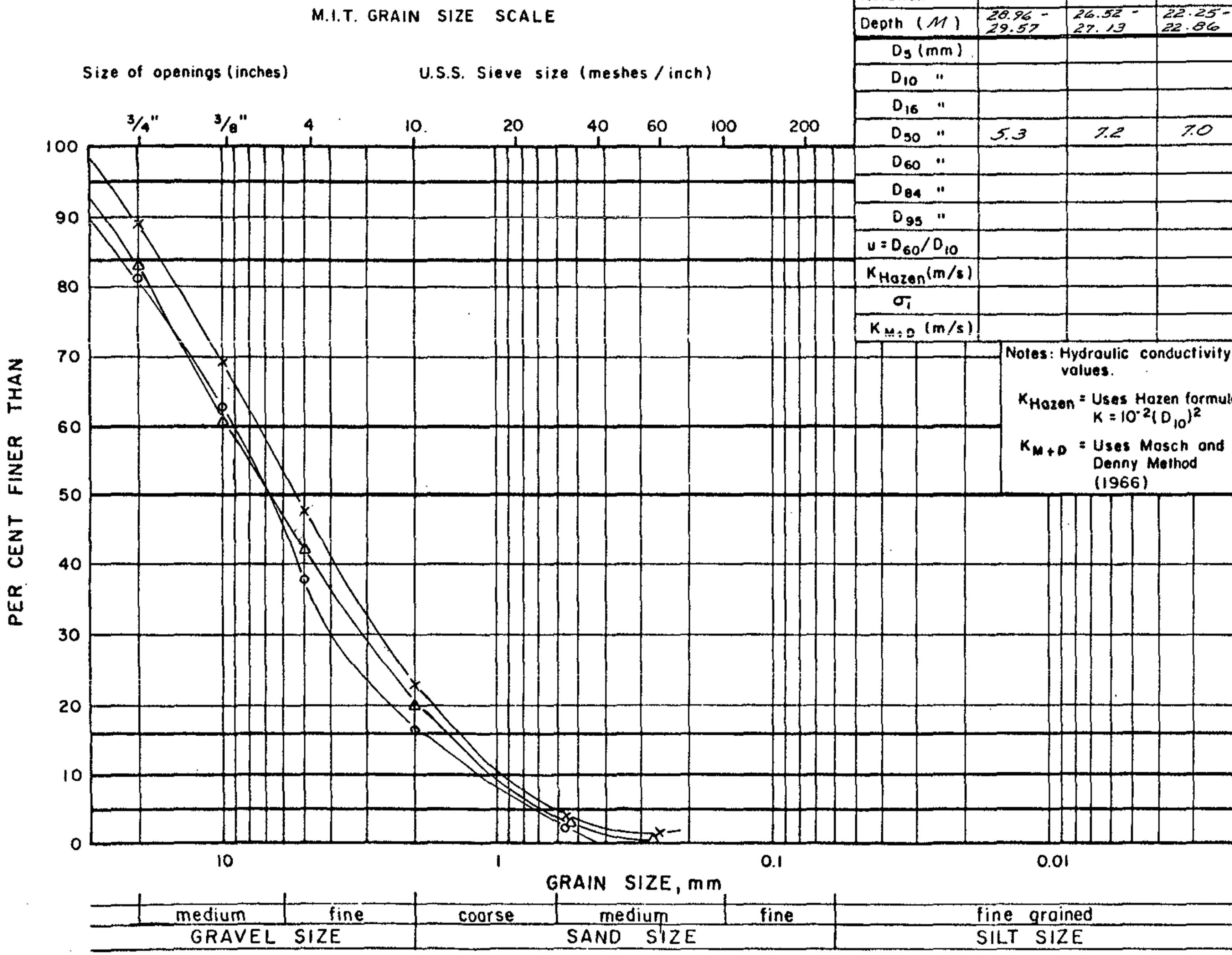
Figure B-7

Sample No.			
Notation	×	o	Δ
Depth (M)	28.96 - 29.57	26.52 - 27.13	22.25 - 22.86
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	5.3	7.2	7.0
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)



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Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. PW2

Figure B-8

Grain Size Distribution and Calculation of Hydraulic Conductivity Values

WELL No. PW2

M.I.T. GRAIN SIZE SCALE

Sample No.			
Notation	x		
Depth (M)	24.67-25.30		
D_5 (mm)			
D_{10} "			
D_{16} "			
D_{50} "	7.4		
D_{60} "			
D_{84} "			
D_{95} "			
$u = D_{60}/D_{10}$			
K_{Hazen} (m/s)			
σ_I			
K_{M+D} (m/s)			

Notes: Hydraulic conductivity values.

K_{Hazen} = Uses Hazen formula $K = 10^{-2}(D_{10})^2$

K_{M+D} = Uses Masch and Denny Method (1966)

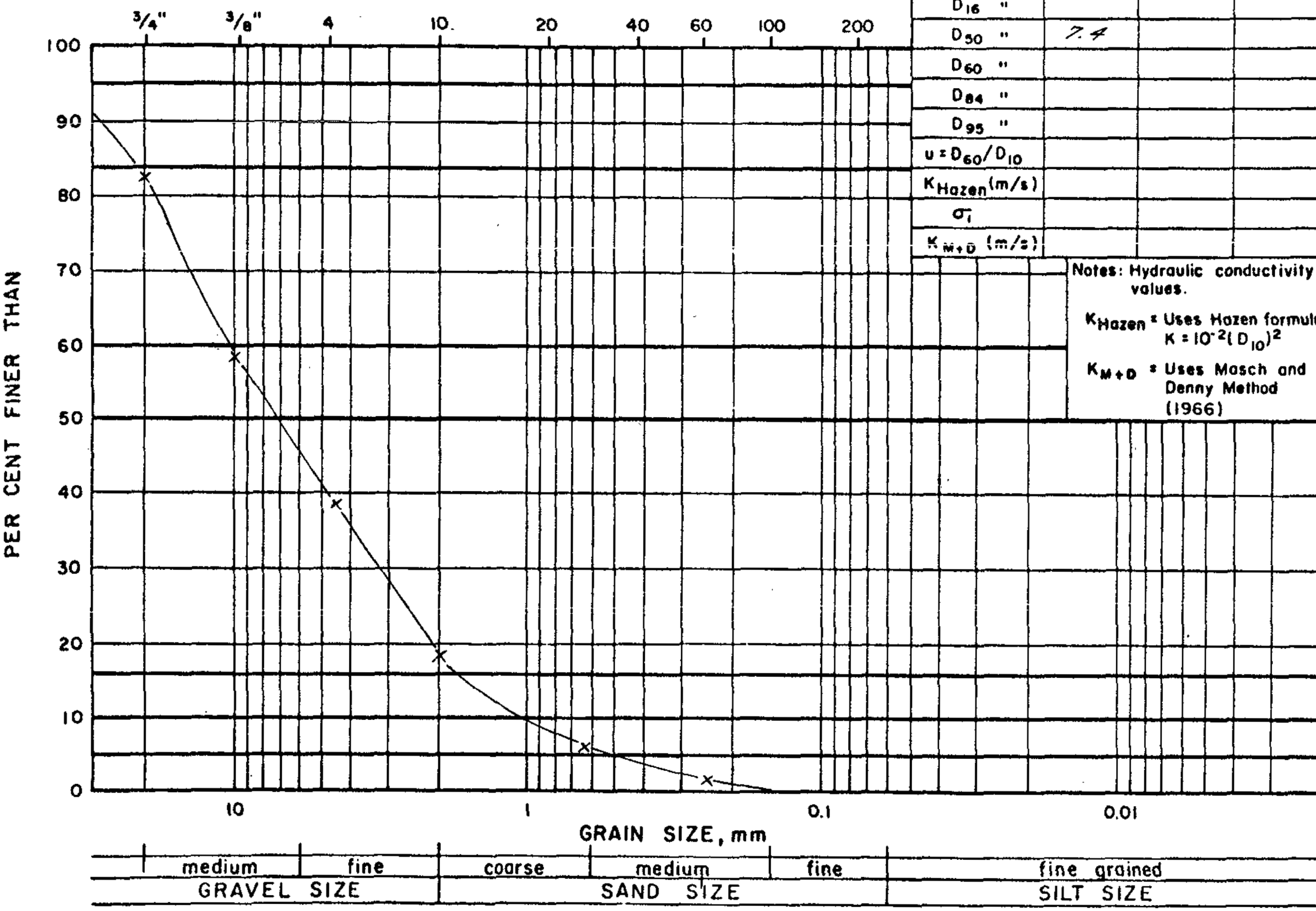

Figure B-9

Golder Associates

APPENDIX C

PUMP TEST RESULTS

C-1 Data

C-2 Theis Analysis

C-3 Jacob Analysis

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* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*                              GOLDER ASSOCIATES                               *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*        PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER -     OW1,              *
*                                                                              *
*                              12/11/81-11.29.29                               *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *

    PUMPED WELL NUMBER - OW1,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 30/ 6/81- 5.0/18
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  -  1/ 7/81- 5.0/ 0

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                          822.40 METRES
    DATUM POINT -                                      TOP OF 152MM CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -           .61 METRES
    DEPTH TO STATIC WATER LEVEL -                 4.55 METRES
    ELEVATION OF STATIC WATER LEVEL -           818.46 METRES
    TYPE OF OBSERVATION WELL -                         SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                 49.70 TO      55.80 METRES
    DISTANCE FROM PUMPING WELL -                  0.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                             152.00   MM
    PUMP TYPE -                                        SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                  TRIDENT DIGITAL,
    PUMPING RATE -                           1.288E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                               CONFINED
    AQUIFER DESCRIPTION -                              SILTY SAND,
    AQUIFER THICKNESS -                             0.  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - SCREEN ABANDONED AFTER PUMPING - PIEZOMETER INSTAL
```

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW1, ** 12/11/81-11.29.29 ** PAGE 2

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		823.01		
0	0	0	0	0.0			0.00		823.01		
81	6	26	13	0.0			3.89		819.12		BEFORE DEVELOPING
81	6	27	8	35.0			5.12		817.89		
81	6	27	13	20.0			9.80		813.21		AFTER BAILING NOT FULLY DEVELOPED
81	6	27	13	45.0			5.78		817.23		
81	6	27	14	5.0			5.48		817.53		
81	6	27	16	0.0			5.21		817.80		
81	6	28	15	5.0			4.55		818.46		
81	6	29	7	53.0			4.51		818.50		
81	6	29	8	12.0			4.52		818.49		
81	6	29	8	22.0			4.52		818.49		
81	6	29	8	32.0			4.52		818.49		
81	6	29	8	42.0			4.52		818.49		
81	6	29	9	8.0			4.53		818.48		
81	6	29	9	47.0			4.54		818.47		
81	6	29	10	7.0			4.54		818.47		
81	6	29	10	22.0			4.54		818.47		
81	6	29	11	17.0			4.54		818.47		
81	6	29	11	37.0			4.54		818.47		
81	6	29	12	27.0			4.55		818.46		
81	6	29	13	10.0			4.55		818.46		
81	6	29	14	10.0			4.55		818.46		
81	6	29	14	55.0			4.55		818.46		
81	6	29	16	37.0			4.54		818.47		
81	6	29	18	50.0			4.56		818.45		
81	6	29	19	12.0			4.56		818.45		
81	6	29	19	52.0			4.55		818.46		
81	6	29	20	21.0			4.54		818.47		
81	6	30	8	0.0			4.53		818.48		INSTALLED PUMP TO 47.4M BELOW GROUND
81	6	30	18	3.0			4.57		818.44		INSTALLED OPEN END PIEZOMETER
81	6	30	18	5.0			4.57	0.02	818.44		FLOW METER READING 6683111 START PUMP
81	6	30	18	6.0	1.0		9.45	4.90	813.56		WATER CLOUDY
81	6	30	18	9.0	4.0		14.25	9.70	808.76		
81	6	30	18	11.0	6.0		16.44	11.89	806.57		
81	6	30	18	15.0	10.0		18.79	14.24	804.22		
81	6	30	18	20.0	15.0		21.47	16.92	801.54		PUMPED WELL STORAGE
81	6	30	18	26.0	21.0		23.76	19.21	799.25		
81	6	30	18	31.0	26.0		25.04	20.49	797.97		
81	6	30	18	35.0	30.0		25.76	21.21	797.25		
81	6	30	18	40.0	35.0		26.49	21.94	796.52		
81	6	30	18	48.0	43.0		27.48	22.93	795.53		
81	6	30	19	18.0	73.0		29.39	24.84	793.62		
81	6	30	19	28.0	83.0		30.18	25.63	792.83		
81	6	30	19	38.0	93.0		31.23	26.68	791.78		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	30	20	12.0	127.0		32.59	28.04	790.42		WATER REMAINED CLOUDY
81	6	30	21	12.0	187.0		34.48	29.93	788.53		AV. PUMP RATE 17 IGPM
81	7	1	0	5.0	360.0		38.01	33.46	785.00		PUMP STOPPED, METER 6689283
81	7	1	7	30.0	805.0		6.74	2.19	816.27		OPEN PIEZOMETER REMOVED
81	7	2	9	0.0	2335.0		5.58	1.03	817.43		
81	7	3	8	0.0	3715.0		5.58	1.03	817.43		
81	7	6	9	35.0	8130.0		5.60	1.05	817.41		
81	7	7	7	50.0	9465.0		5.61	1.06	817.40		
81	7	8	9	35.0	11010.0		5.62	1.07	817.39		
81	7	9	13	0.0	12655.0		5.63	1.08	817.38		
81	7	10	8	0.0	13795.0		5.63	1.08	817.38		WELL SCREEN ABANDONED

RESIDUAL DRAWDOWN

OBSERVATION WELL - OW1,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (T1)	RATIO (T/T1)	DRAWDOWN (S)
805.0	445.0	1.81	2.19
2335.0	1975.0	1.18	1.03
3715.0	3355.0	1.11	1.03
8130.0	7770.0	1.05	1.05
9465.0	9105.0	1.04	1.06
11010.0	10650.0	1.03	1.07
12655.0	12295.0	1.03	1.08
13795.0	13435.0	1.03	1.08

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*                                                                              *
*                                                                              *
*                           GOLDER ASSOCIATES                                  *
*                                                                              *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*                                                                              *
*        PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER -     OW1,              *
*                                                                              *
*                                                                              *
*                           12/11/81-11.29.30                                  *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
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    PUMPED WELL NUMBER - OW1,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 30/ 6/81- 5.0/18
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  -  1/ 7/81- 5.0/ 0

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                         822.40 METRES
    DATUM POINT -                                     TOP OF 19MM PUC PIPE,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -          .61 METRES
    DEPTH TO STATIC WATER LEVEL -                2.49 METRES
    ELEVATION OF STATIC WATER LEVEL -          820.52 METRES
    TYPE OF OBSERVATION WELL -                        STANDPIPE PIEZOMETER
    DEPTH OF GRAVEL PACK INTERVAL -             23.06 TO     26.41 METRES
    DISTANCE FROM PUMPING WELL -                90.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                            152.00   MM.
    PUMP TYPE -                                       SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                 TRIDENT DIGITAL,
    PUMPING RATE -                         1.288E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                              CONFINED
    AQUIFER DESCRIPTION -                             SILTY SAND,
    AQUIFER THICKNESS -                           0.  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - STANDPIPE PIEZOMETER INSTALLED AFTER PUMPTESTING,
                       - OW1,
```

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW1, ** 12/11/81-11.29.30 ** PAGE 2

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		823.01		
0	0	0	0	0.0			0.00		823.01		
81	7	15	10	30.0	21145.0		3.70	1.21	819.31		SEALED PIEZOMETER INSTALLED
81	7	16	11	15.0	22630.0		3.45	0.96	819.56		
81	7	20	8	15.0	28210.0		2.78	0.29	820.23		
81	7	24	8	20.0	33975.0		2.65	0.16	820.36		
81	7	24	10	36.0	34111.0		2.49	0.	820.52		
81	7	24	10	50.0	34125.0		2.49	0.	820.52		PUMPING PW 2
81	7	24	10	50.5	34125.5		2.49	0.	820.52		
81	7	24	10	51.0	34126.0		2.50	0.01	820.51		
81	7	24	10	51.5	34126.5		2.50	0.01	820.51		
81	7	24	10	52.0	34127.0		2.51	0.02	820.50		
81	7	24	10	52.5	34127.5		2.51	0.02	820.50		
81	7	24	10	53.0	34128.0		2.51	0.02	820.50		
81	7	24	10	54.0	34129.0		2.50	0.01	820.51		
81	7	24	10	55.0	34130.0		2.52	0.03	820.49		
81	7	24	10	56.0	34131.0		2.51	0.02	820.50		
81	7	24	10	58.0	34133.0		2.51	0.02	820.50		
81	7	24	11	0.0	34135.0		2.51	0.02	820.50		
81	7	24	11	5.0	34140.0		2.53	0.04	820.48		
81	7	24	11	10.0	34145.0		2.56	0.07	820.45		
81	7	24	11	15.0	34150.0		2.55	0.06	820.46		
81	7	24	11	20.0	34155.0		2.56	0.07	820.45		
81	7	24	11	30.0	34165.0		2.58	0.09	820.43		
81	7	24	11	40.0	34175.0		2.58	0.09	820.43		
81	7	24	11	50.0	34185.0		2.59	0.10	820.42		
81	7	24	12	10.0	34205.0		2.54	0.05	820.47		
81	7	24	12	30.0	34225.0		2.54	0.05	820.47		
81	7	24	13	25.0	34280.0		2.59	0.10	820.42		
81	7	24	14	10.0	34325.0		2.54	0.05	820.47		
81	7	24	15	5.0	34380.0		2.55	0.06	820.46		
81	7	24	15	55.0	34430.0		2.58	0.09	820.43		
81	7	24	17	35.0	34530.0		2.61	0.12	820.40		
81	7	24	19	20.0	34635.0		2.63	0.14	820.38		
81	7	24	20	56.0	34731.0		2.64	0.15	820.37		
81	7	25	0	5.0	34920.0		2.70	0.21	820.31		
81	7	25	6	42.0	35317.0		2.76	0.27	820.25		
81	7	25	14	42.0	35797.0		2.72	0.23	820.29		
81	7	25	22	42.0	36277.0		2.75	0.26	820.26		
81	7	26	6	42.0	36757.0		2.72	0.23	820.29		
81	7	26	9	6.0	36901.0		2.70	0.21	820.31		
81	7	26	10	20.0	36975.0		2.70	0.21	820.31		
81	7	26	11	40.0	37055.0		2.71	0.22	820.30		
81	7	26	12	50.0	37125.0		2.71	0.22	820.30		
81	7	26	14	45.0	37240.0		2.71	0.22	820.30		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER - OW1, ** 12/11/81-11.29.30 ** PAGE 3

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	26	16	10.0	37325.0		2.72	0.23	820.29		
81	7	26	22	40.0	37715.0		2.76	0.27	820.25		
81	7	27	7	10.0	38225.0		2.79	0.30	820.22		
81	7	27	11	30.0	38485.0		2.83	0.34	820.18		
81	7	27	15	10.0	38705.0		2.91	0.42	820.10		
81	7	27	22	40.0	39155.0		3.06	0.57	819.95		
81	7	28	6	40.0	39635.0		2.82	0.33	820.19		
81	7	28	8	48.0	39763.0		2.79	0.30	820.22		
81	7	28	9	0.0	39775.0		2.80	0.31	820.21		STOPPED PUMPING PW 2
81	7	28	9	1.0	39776.0		2.80	0.31	820.21		
81	7	28	9	1.5	39776.5		2.81	0.32	820.20		
81	7	28	9	2.0	39777.0		2.81	0.32	820.20		
81	7	28	9	2.5	39777.5		2.82	0.33	820.19		
81	7	28	9	3.0	39778.0		2.81	0.32	820.20		
81	7	28	9	4.0	39779.0		2.81	0.32	820.20		
81	7	28	9	5.0	39780.0		2.81	0.32	820.20		
81	7	28	9	6.0	39781.0		2.79	0.30	820.22		
81	7	28	9	8.0	39783.0		2.78	0.29	820.23		
81	7	28	9	10.0	39785.0		2.75	0.26	820.26		
81	7	28	9	15.0	39790.0		2.73	0.24	820.28		
81	7	28	9	20.0	39795.0		2.77	0.28	820.24		
81	7	28	9	25.0	39800.0		2.75	0.26	820.26		
81	7	28	9	30.0	39805.0		2.76	0.27	820.25		
81	7	28	9	40.0	39815.0		2.74	0.25	820.27		
81	7	28	9	50.0	39825.0		2.74	0.25	820.27		
81	7	28	10	0.0	39835.0		2.72	0.23	820.29		
81	7	28	14	0.0	40075.0		2.71	0.22	820.30		
81	7	28	15	45.0	40180.0		2.70	0.21	820.31		
81	7	29	10	45.0	41320.0		2.70	0.21	820.31		
81	7	29	14	30.0	41545.0		2.63	0.14	820.38		
81	7	29	22	0.0	41995.0		2.65	0.16	820.36		
81	7	30	9	0.0	42655.0		2.69	0.20	820.32		
81	7	31	8	50.0	44085.0		2.73	0.24	820.28		
81	8	1	9	45.0	45580.0		2.76	0.27	820.25		
81	8	2	11	35.0	47130.0		2.74	0.25	820.27		
81	8	3	9	40.0	48455.0		2.75	0.26	820.26		
81	8	5	9	25.0	51320.0		2.79	0.30	820.22		
81	8	7	9	40.0	54215.0		2.79	0.30	820.22		
81	8	8	9	10.0	55625.0		2.83	0.34	820.18		
81	8	9	9	0.0	57055.0		2.82	0.33	820.19		
81	8	10	9	0.0	58495.0		2.77	0.28	820.24		

RESIDUAL DRAWDOWN

OBSERVATION WELL = OW1,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (T1)	RATIO (T/T1)	DRAWDOWN (S)
21145.0	20785.0	1.02	1.21
22630.0	22270.0	1.02	.96
28210.0	27850.0	1.01	.29
33975.0	33615.0	1.01	.16
34111.0	33751.0	1.01	0.00
34125.0	33765.0	1.01	0.00
34125.5	33765.5	1.01	0.00
34126.0	33766.0	1.01	.01
34126.5	33766.5	1.01	.01
34127.0	33767.0	1.01	.02
34127.5	33767.5	1.01	.02
34128.0	33768.0	1.01	.02
34129.0	33769.0	1.01	.01
34130.0	33770.0	1.01	.03
34131.0	33771.0	1.01	.02
34133.0	33773.0	1.01	.02
34135.0	33775.0	1.01	.02
34140.0	33780.0	1.01	.04
34145.0	33785.0	1.01	.07
34150.0	33790.0	1.01	.06
34155.0	33795.0	1.01	.07
34165.0	33805.0	1.01	.09
34175.0	33815.0	1.01	.09
34185.0	33825.0	1.01	.10
34205.0	33845.0	1.01	.05
34225.0	33865.0	1.01	.05
34280.0	33920.0	1.01	.10
34325.0	33965.0	1.01	.05
34380.0	34020.0	1.01	.06
34430.0	34070.0	1.01	.09
34530.0	34170.0	1.01	.12
34635.0	34275.0	1.01	.14
34731.0	34371.0	1.01	.15
34920.0	34560.0	1.01	.21
35317.0	34957.0	1.01	.27
35797.0	35437.0	1.01	.23
36277.0	35917.0	1.01	.26
36757.0	36397.0	1.01	.23
36901.0	36541.0	1.01	.21
36975.0	36615.0	1.01	.21
37055.0	36695.0	1.01	.22
37125.0	36765.0	1.01	.22
37240.0	36880.0	1.01	.22
37325.0	36965.0	1.01	.23
37715.0	37355.0	1.01	.27
38225.0	37865.0	1.01	.30
38485.0	38125.0	1.01	.34
38705.0	38345.0	1.01	.42
39155.0	38795.0	1.01	.57
39776.0	39416.0	1.01	.31
39776.5	39416.5	1.01	.32
39777.0	39417.0	1.01	.32
39777.5	39417.5	1.01	.33
39778.0	39418.0	1.01	.32
39779.0	39419.0	1.01	.32
39780.0	39420.0	1.01	.32
39781.0	39421.0	1.01	.30
39783.0	39423.0	1.01	.29
39785.0	39425.0	1.01	.26
39790.0	39430.0	1.01	.24
39795.0	39435.0	1.01	.28
39800.0	39440.0	1.01	.26
39805.0	39445.0	1.01	.27
39815.0	39455.0	1.01	.25
39825.0	39465.0	1.01	.25
39835.0	39475.0	1.01	.23
40075.0	39715.0	1.01	.22
40180.0	39820.0	1.01	.21
41320.0	40960.0	1.01	.21
41545.0	41185.0	1.01	.14
41995.0	41635.0	1.01	.16
42655.0	42295.0	1.01	.20
44085.0	43725.0	1.01	.24
45580.0	45220.0	1.01	.27
47130.0	46770.0	1.01	.25
48455.0	48095.0	1.01	.26
51320.0	50960.0	1.01	.30
54215.0	53855.0	1.01	.30
55625.0	55265.0	1.01	.34
57055.0	56695.0	1.01	.33
58495.0	58135.0	1.01	.28

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*                                                                              *
*                                                                              *
*                              GOLDER ASSOCIATES                               *
*                                                                              *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*                                                                              *
*         PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER -     OW2,             *
*                                                                              *
*                                                                              *
*                            12/11/81-11.27.44                                 *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *

    PUMPED WELL NUMBER - OW2,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 30/ 6/81-10.0/11
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 30/ 6/81-30.0/14

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                               823.60 METRES
    DATUM POINT -                                           TOP OF 152MM CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -                .61 METRES
    DEPTH TO STATIC WATER LEVEL -                      7.37 METRES
    ELEVATION OF STATIC WATER LEVEL -                816.84 METRES
    TYPE OF OBSERVATION WELL -                              SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                      30.50 TO     32.90 METRES
    DISTANCE FROM PUMPING WELL -                     122.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                                  152.00   MM
    PUMP TYPE -                                             SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                       TRIDENT DIGITAL,
    PUMPING RATE -                               2.258E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                                    CONFINED
    AQUIFER DESCRIPTION -                                   GRAVELLY COARSE SAND,
    AQUIFER THICKNESS -                                 0.  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - NONE,
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DATE			TIME		ELAPSED TIME	PRESSURE READING	DEPTH TO WATER	DRAWDOWN	WATER ELEVATION	DISCHARGE RATE	COMMENTS
YR	MON	DAY	HR	MIN	MINUTES	PSI	METRES	METRES	METRES	LITRES/S	
0	0	0	0	0.0			0.00		824.21		
0	0	0	0	0.0			0.00		824.21		
81	6	15	9	0.0			6.37		817.84		OPEN HOLE
81	6	25	10	0.0			6.70		817.51		SCREENED-UNDEVELOPED
81	6	26	15	45.0			7.22		816.99		AFTER DEVELOPING
81	6	27	8	40.0			7.16		817.05		
81	6	27	16	10.0			7.16		817.05		
81	6	28	15	7.0			7.16		817.05		
81	6	28	20	32.0			7.17		817.04		
81	6	28	21	30.0			7.41		816.80		STARTED PUMPING OW3 @ 20:48
81	6	28	21	27.3			7.45		816.76		
81	6	28	21	31.5			7.47		816.74		
81	6	28	21	35.0			7.48		816.73		
81	6	28	21	41.0			7.50		816.71		
81	6	28	21	49.0			7.51		816.70		
81	6	28	22	2.0			7.51		816.70		
81	6	28	22	8.0			7.51		816.70		
81	6	28	22	12.0			7.51		816.70		
81	6	28	22	17.0			7.51		816.70		
81	6	28	22	19.5			7.51		816.70		STOPPED PUMPING OW3
81	6	28	22	20.0			7.51		816.70		
81	6	28	22	20.5			7.51		816.70		
81	6	28	22	21.0			7.50		816.71		
81	6	28	22	21.5			7.49		816.72		
81	6	28	22	22.0			7.48		816.73		
81	6	28	22	22.5			7.47		816.74		
81	6	28	22	23.5			7.46		816.75		
81	6	28	22	24.5			7.43		816.78		
81	6	28	22	25.5			7.41		816.80		
81	6	28	22	27.5			7.37		816.84		
81	6	28	22	29.5			7.34		816.87		
81	6	28	22	34.5			7.28		816.93		
81	6	28	22	39.5			7.25		816.96		
81	6	28	22	44.5			7.23		816.98		
81	6	28	22	58.0			7.20		817.01		
81	6	29	7	55.0			7.18		817.03		
81	6	29	8	0.0			7.18		817.03		PUMPING STARTED OW3
81	6	29	8	.5			7.18		817.03		
81	6	29	8	1.0			7.18		817.03		
81	6	29	8	1.5			7.18		817.03		
81	6	29	8	2.0			7.19		817.02		
81	6	29	8	2.5			7.20		817.01		
81	6	29	8	3.0			7.21		817.00		
81	6	29	8	4.0			7.25		816.96		
81	6	29	8	5.0			7.28		816.93		

DATE YR	DATE MON	DATE DAY	TIME HR	TIME MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	29	8	6.0			7.32		816.89		
81	6	29	8	8.0			7.38		816.83		
81	6	29	8	10.0			7.43		816.78		
81	6	29	8	15.0			7.52		816.69		
81	6	29	8	20.0			7.57		816.64		
81	6	29	8	25.0			7.60		816.61		
81	6	29	8	30.0			7.62		816.59		
81	6	29	8	40.0			7.64		816.57		
81	6	29	8	50.0			7.65		816.56		
81	6	29	9	0.0			7.66		816.55		
81	6	29	9	20.0			7.66		816.55		
81	6	29	9	46.0			9.67		814.54		
81	6	29	9	50.0			9.67		814.54		INCREASED PUMP RATE
81	6	29	9	50.5			9.67		814.54		
81	6	29	9	51.0			9.68		814.53		
81	6	29	9	51.5			9.68		814.53		
81	6	29	9	52.0			9.68		814.53		
81	6	29	9	52.5			9.68		814.53		
81	6	29	9	53.0			9.68		814.53		
81	6	29	9	54.0			9.69		814.52		
81	6	29	9	55.0			9.70		814.51		
81	6	29	9	56.0			9.71		814.50		
81	6	29	9	58.0			9.72		814.49		
81	6	29	10	0.0			9.73		814.48		
81	6	29	10	5.0			7.75		816.46		
81	6	29	10	10.0			7.76		816.45		
81	6	29	10	15.0			7.77		816.44		
81	6	29	10	20.0			7.77		816.44		
81	6	29	10	30.0			7.78		816.43		
81	6	29	10	40.0			7.78		816.43		
81	6	29	10	50.0			7.79		816.42		
81	6	29	11	14.0			7.79		816.42		
81	6	29	11	34.0			7.79		816.42		
81	6	29	12	20.0			7.80		816.41		
81	6	29	13	10.0			7.81		816.41		
81	6	29	14	0.0			7.81		816.40		
81	6	29	14	50.0			7.82		816.40		
81	6	29	16	35.0			7.83		816.38		
81	6	29	19	0.0			7.84		816.37		STOPPED PUMPING OW3
81	6	29	19	1.0			7.84		816.37		
81	6	29	19	2.0			7.83		816.38		
81	6	29	19	3.0			7.80		816.41		
81	6	29	19	4.0			7.76		816.45		
81	6	29	19	5.0			7.71		816.50		
81	6	29	19	6.0			7.67		816.54		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER - OW2, ** 12/11/81-11.27.44 **

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	29	19	8.0			7.59		816.62		
81	6	29	19	10.0			7.53		816.68		
81	6	29	19	15.0			7.43		816.78		
81	6	29	19	20.0			7.37		816.84		
81	6	29	19	25.0			7.34		816.87		
81	6	29	19	30.0			7.31		816.90		
81	6	29	19	40.0			7.29		816.92		
81	6	29	10	50.0			7.27		816.94		
81	6	29	20	0.0			7.26		816.95		
81	6	29	20	20.0			7.25		816.96		
81	6	29	20	40.0			7.24		816.97		INSTALLED PUMP TO 28.4M BELOW GROUND
81	6	30	9	47.0			7.21		817.00		INSTALLED OPEN END PIEZOMETER
81	6	30	10	0.0			7.22		816.99		START PUMP
81	6	30	10	.3			13.02		811.19		
81	6	30	10	.5			17.38		806.83		
81	6	30	10	.8			20.55		803.66		
81	6	30	10	1.0			22.61		801.60		STOPPED PUMP
81	6	30	10	30.0			7.25		816.96		START PUMP
81	6	30	10	30.5			17.02		807.19		
81	6	30	10	30.8			20.70		803.51		STOPPED PUMP
81	6	30	11	10.0			7.27	-0.10	816.94		START PUMP
81	6	30	11	10.3	0.3		9.82	2.45	814.39		
81	6	30	11	10.5	0.5		10.97	3.60	813.24		
81	6	30	11	10.8	0.8		11.98	4.61	812.23		
81	6	30	11	11.0	1.0		13.11	5.74	811.10		
81	6	30	11	11.5	1.5		14.38	7.01	809.83		
81	6	30	11	12.0	2.0		15.32	7.95	808.89		
81	6	30	11	12.5	2.5		15.99	8.62	808.22		
81	6	30	11	13.0	3.0		16.51	9.14	807.70		
81	6	30	11	14.0	4.0		17.20	9.83	807.01		
81	6	30	11	15.0	5.0		17.82	10.45	806.39		
81	6	30	11	16.0	6.0		17.90	10.53	806.31		
81	6	30	11	18.0	8.0		18.22	10.85	805.99		
81	6	30	11	20.0	10.0		18.39	11.02	805.82		
81	6	30	11	25.0	15.0		18.59	11.22	805.62		
81	6	30	11	30.0	20.0		18.68	11.31	805.53		
81	6	30	11	35.0	25.0		18.73	11.36	805.48		
81	6	30	11	40.0	30.0		18.70	11.33	805.51		
81	6	30	11	50.0	40.0		18.65	11.28	805.56		
81	6	30	12	0.0	50.0		18.73	11.36	805.48		
81	6	30	12	10.0	60.0		18.76	11.39	805.45		
81	6	30	12	30.0	80.0		18.70	11.33	805.51		
81	6	30	12	50.0	100.0		18.71	11.34	805.50		
81	6	30	13	40.0	150.0		18.68	11.31	805.53		
81	6	30	14	30.0	200.0		18.72	11.35	805.49	2.26	STOPPED PUMP

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER - OW2, ** 12/11/81-11.27.44 ** PAGE 5

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	30	14	30.3	200.3		17.24	9.87	806.97		
81	6	30	14	30.5	200.5		15.73	8.36	808.48		
81	6	30	14	30.8	200.8		14.38	7.01	809.83		
81	6	30	14	31.0	201.0		13.52	6.15	810.69		
81	6	30	14	31.5	201.5		12.82	5.45	811.39		
81	6	30	14	32.0	202.0		10.98	3.61	813.23		
81	6	30	14	32.5	202.5		10.22	2.85	813.99		
81	6	30	14	33.0	203.0		9.66	2.29	814.55		
81	6	30	14	34.0	204.0		8.96	1.59	815.25		
81	6	30	14	35.0	205.0		8.52	1.15	815.69		
81	6	30	14	36.0	206.0		8.25	0.88	815.96		
81	6	30	14	38.0	208.0		7.94	0.57	816.27		
81	6	30	14	40.0	210.0		7.79	0.42	816.42		
81	6	30	14	45.0	215.0		7.56	0.19	816.65		
81	6	30	14	50.0	220.0		7.47	0.10	816.74		
81	6	30	14	55.0	225.0		7.39	0.02	816.82		
81	6	30	15	0.0	230.0		7.35	-0.02	816.86		
81	6	30	15	20.0	250.0		7.30	-0.07	816.91		
81	6	30	15	30.0	260.0		7.27	-0.10	816.94		
81	6	30	15	50.0	280.0		7.25	-0.12	816.96		
81	7	1	10	45.0	1415.0		7.22	-0.15	816.99		REMOVED OPEN END PIEZOMETER
81	7	2	9	5.0	2755.0		7.23	-0.14	816.98		
81	7	3	8	0.0	4130.0		7.24	-0.13	816.97		
81	7	6	9	38.0	8548.0		7.26	-0.11	816.95		
81	7	7	7	50.0	9880.0		7.27	-0.10	816.94		
81	7	8	9	30.0	11420.0		7.28	-0.09	816.93		
81	7	9	13	0.0	13070.0		7.29	-0.08	816.92		
81	7	10	8	0.0	14210.0		7.30	-0.07	816.91		
81	7	13	9	0.0	18590.0		7.32	-0.05	816.89		
81	7	15	11	30.0	21620.0		7.33	-0.04	816.88		
81	7	16	11	15.0	23045.0		7.34	-0.03	816.87		
81	7	20	8	15.0	28625.0		7.36	-0.01	816.85		
81	7	21	8	20.0	30070.0		7.35	-0.02	816.86		
81	7	24	10	39.0	34529.0		7.37	0.	816.84		
81	7	24	10	50.0	34540.0		7.37	0.	816.84		PUMPING STARTED PW2
81	7	24	10	52.5	34542.5		7.37	0.	816.84		
81	7	24	10	53.0	34543.0		7.37	0.	816.84		
81	7	24	10	53.5	34543.5		7.38	0.01	816.83		
81	7	24	10	54.0	34544.0		7.38	0.01	816.83		
81	7	24	10	54.5	34544.5		7.38	0.01	816.83		
81	7	24	10	55.0	34545.0		7.39	0.02	816.82		
81	7	24	10	56.5	34546.5		7.40	0.03	816.81		
81	7	24	10	57.5	34547.5		7.41	0.04	816.80		
81	7	24	10	58.5	34548.5		7.42	0.05	816.79		
81	7	24	11	.5	34550.5		7.43	0.06	816.78		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW2, ** 12/11/81-11.27.44 ** PAGE 6

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	24	11	2.5	34552.5		7.44	0.07	816.77		
81	7	24	11	7.0	34557.0		7.46	0.09	816.75		
81	7	24	11	12.0	34562.0		7.48	0.11	816.73		
81	7	24	11	17.0	34567.0		7.49	0.12	816.72		
81	7	24	11	22.0	34572.0		7.50	0.13	816.71		
81	7	24	11	32.0	34582.0		7.52	0.15	816.69		
81	7	24	11	42.0	34592.0		7.52	0.15	816.69		
81	7	24	11	52.0	34602.0		7.53	0.16	816.68		
81	7	24	12	10.0	34620.0		7.53	0.16	816.68		
81	7	24	12	30.0	34640.0		7.53	0.16	816.68		
81	7	24	13	30.0	34700.0		7.54	0.17	816.67		
81	7	24	14	20.0	34750.0		7.55	0.18	816.66		
81	7	24	15	8.0	34798.0		7.55	0.18	816.66		
81	7	24	15	58.0	34848.0		7.55	0.18	816.66		
81	7	24	17	37.0	34947.0		7.56	0.19	816.65		
81	7	24	19	22.0	35052.0		7.57	0.20	816.64		
81	7	24	21	0.0	35150.0		7.57	0.20	816.64		
81	7	25	0	1.0	35331.0		7.58	0.21	816.63		
81	7	25	6	40.0	35730.0		7.60	0.23	816.61		
81	7	25	14	40.0	36210.0		7.62	0.25	816.59		
81	7	25	22	40.0	36690.0		7.63	0.26	816.58		
81	7	26	6	40.0	37170.0		7.65	0.28	816.56		
81	7	26	9	5.0	37315.0		7.72	0.35	816.49		
81	7	26	10	25.0	37395.0		7.73	0.36	816.48		
81	7	26	11	40.0	37470.0		7.72	0.35	816.49		
81	7	26	16	10.0	37740.0		7.74	0.37	816.47		
81	7	26	22	40.0	38130.0		7.75	0.38	816.46		
81	7	27	7	12.0	38642.0		7.77	0.40	816.44		
81	7	27	15	8.0	39118.0		7.78	0.41	816.43		
81	7	27	22	45.0	39575.0		7.81	0.44	816.40		
81	7	28	6	45.0	40055.0		7.82	0.45	816.39		
81	7	28	8	57.0	40187.0		7.82	0.45	816.39		
81	7	28	9	0.0	40190.0		7.82	0.45	816.39		STOP PUMPING PW2
81	7	28	9	.5	40190.5		7.82	0.45	816.39		
81	7	28	9	1.0	40191.0		7.82	0.45	816.39		
81	7	28	9	1.5	40191.5		7.82	0.45	816.39		
81	7	28	9	2.0	40192.0		7.82	0.45	816.39		
81	7	28	9	2.5	40192.5		7.81	0.44	816.40		
81	7	28	9	3.0	40193.0		7.81	0.44	816.40		
81	7	28	9	4.0	40194.0		7.80	0.43	816.41		
81	7	28	9	5.0	40195.0		7.79	0.42	816.42		
81	7	28	9	6.0	40196.0		7.79	0.42	816.42		
81	7	28	9	8.0	40198.0		7.77	0.40	816.44		
81	7	28	9	10.0	40200.0		7.75	0.38	816.46		
81	7	28	9	15.0	40205.0		7.71	0.34	816.50		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW2, ** 12/11/81-11.27.44 ** PAGE 7

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	28	9	20.0	40210.0		7.69	0.32	816.52		
81	7	28	9	25.0	40215.0		7.67	0.30	816.54		
81	7	28	9	30.0	40220.0		7.66	0.29	816.55		
81	7	28	9	40.0	40230.0		7.64	0.27	816.57		
81	7	28	9	50.0	40240.0		7.63	0.26	816.58		
81	7	28	10	0.0	40250.0		7.62	0.25	816.59		
81	7	28	14	0.0	40490.0		7.59	0.22	816.62		
81	7	28	15	45.0	40595.0		7.56	0.19	816.65		
81	7	28	22	48.0	41018.0		7.55	0.18	816.66		
81	7	29	8	20.0	41590.0		7.54	0.17	816.67		
81	7	29	10	48.0	41738.0		7.55	0.18	816.66		
81	7	29	14	30.0	41960.0		7.53	0.16	816.68		
81	7	29	22	0.0	42410.0		7.53	0.16	816.68		
81	7	30	9	0.0	43070.0		7.53	0.16	816.68		
81	7	31	8	45.0	44495.0		7.52	0.15	816.69		
81	8	1	9	45.0	45995.0		7.51	0.14	816.70		
81	8	2	11	35.0	47545.0		7.51	0.14	816.70		
81	8	3	9	50.0	48880.0		7.51	0.14	816.70		
81	8	4	11	10.0	50400.0		7.51	0.14	816.70		
81	8	5	9	25.0	51735.0		7.51	0.14	816.70		
81	8	7	9	40.0	54630.0		7.52	0.15	816.69		
81	8	8	9	0.0	56030.0		7.53	0.16	816.68		
81	8	9	9	0.0	57470.0		7.53	0.16	816.68		
81	8	10	9	0.0	58910.0		7.53	0.16	816.68		

RESIDUAL DRAWDOWN

OBSERVATION WELL - OW2,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (T1)	RATIO (T/T1)	DRAWDOWN (S)
200.3	.3	801.00	9.87
200.5	.5	401.00	8.36
200.8	.8	267.67	7.01
201.0	1.0	201.00	6.15
201.5	1.5	134.33	5.45
202.0	2.0	101.00	3.61
202.5	2.5	81.00	2.85
203.0	3.0	67.67	2.29
204.0	4.0	51.00	1.59
205.0	5.0	41.00	1.15
206.0	6.0	34.33	.88
208.0	8.0	26.00	.57
210.0	10.0	21.00	.42
215.0	15.0	14.33	.19
220.0	20.0	11.00	.10
225.0	25.0	9.00	.02
230.0	30.0	7.67	-.02
250.0	50.0	5.00	-.07
260.0	60.0	4.33	-.10
280.0	80.0	3.50	-.12
1415.0	1215.0	1.16	-.15
2755.0	2555.0	1.08	-.14
4130.0	3930.0	1.05	-.13
8548.0	8348.0	1.02	-.11
9880.0	9680.0	1.02	-.10
11420.0	11220.0	1.02	-.09
13070.0	12870.0	1.02	-.08
14210.0	14010.0	1.01	-.07
18590.0	18390.0	1.01	-.05
21620.0	21420.0	1.01	-.04
23045.0	22845.0	1.01	-.03
28625.0	28425.0	1.01	-.01
30070.0	29870.0	1.01	-.02
34529.0	34329.0	1.01	0.00
34540.0	34340.0	1.01	0.00
34542.5	34342.5	1.01	0.00
34543.0	34343.0	1.01	0.00
34543.5	34343.5	1.01	.01
34544.0	34344.0	1.01	.01
34544.5	34344.5	1.01	.01
34545.0	34345.0	1.01	.02
34546.5	34346.5	1.01	.03
34547.5	34347.5	1.01	.04
34548.5	34348.5	1.01	.05
34550.5	34350.5	1.01	.06
34552.5	34352.5	1.01	.07
34557.0	34357.0	1.01	.09
34562.0	34362.0	1.01	.11
34567.0	34367.0	1.01	.12
34572.0	34372.0	1.01	.13
34582.0	34382.0	1.01	.15
34602.0	34402.0	1.01	.16
34620.0	34420.0	1.01	.16
34640.0	34440.0	1.01	.16
34700.0	34500.0	1.01	.17
34750.0	34550.0	1.01	.18
34798.0	34598.0	1.01	.18
34848.0	34648.0	1.01	.18
34947.0	34747.0	1.01	.19
35052.0	34852.0	1.01	.20
35150.0	34950.0	1.01	.20
35331.0	35131.0	1.01	.21
35730.0	35530.0	1.01	.23
36210.0	36010.0	1.01	.25
36690.0	36490.0	1.01	.26
37170.0	36970.0	1.01	.28
37315.0	37115.0	1.01	.35
37395.0	37195.0	1.01	.36
37470.0	37270.0	1.01	.35
37740.0	37540.0	1.01	.37
38130.0	37930.0	1.01	.38
38642.0	38442.0	1.01	.40
39118.0	38918.0	1.01	.41
39575.0	39375.0	1.01	.44
40055.0	39855.0	1.01	.45
40187.0	39987.0	1.01	.45
40190.0	39990.0	1.01	.45
40190.5	39990.5	1.01	.45
40191.0	39991.0	1.01	.45
40191.5	39991.5	1.01	.45
40192.0	39992.0	1.01	.45
40192.5	39992.5	1.01	.44
40193.0	39993.0	1.01	.44
40194.0	39994.0	1.01	.43
40195.0	39995.0	1.01	.42
40196.0	39996.0	1.01	.42
40198.0	39998.0	1.01	.40
40200.0	40000.0	1.01	.38
40205.0	40005.0	1.00	.34
40210.0	40010.0	1.00	.32
40215.0	40015.0	1.00	.30
40220.0	40020.0	1.00	.29
40230.0	40030.0	1.00	.27
40240.0	40040.0	1.00	.26
40250.0	40050.0	1.00	.25
40490.0	40290.0	1.00	.22
40595.0	40395.0	1.00	.19
41018.0	40818.0	1.00	.18
41590.0	41390.0	1.00	.17
41738.0	41538.0	1.00	.18
41960.0	41760.0	1.00	.16
42410.0	42210.0	1.00	.16
43070.0	42870.0	1.00	.16
44495.0	44295.0	1.00	.15
45995.0	45795.0	1.00	.14
47545.0	47345.0	1.00	.14
48880.0	48680.0	1.00	.14
50400.0	50200.0	1.00	.14
51735.0	51535.0	1.00	.14
54630.0	54430.0	1.00	.15
57470.0	57270.0	1.00	.16
58910.0	58710.0	1.00	.16

GOLDER ASSOCIATES

PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER - OW3,

25/08/81-13.36.44

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    PUMPED WELL NUMBER - OW3,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 29/ 6/81- 0.0/ 8
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 29/ 6/81- 0.0/19

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                          822.20 METRES
    DATUM POINT -                                      TOP OF 152MM CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -           .61 METRES
    DEPTH TO STATIC WATER LEVEL -                 5.83 METRES
    ELEVATION OF STATIC WATER LEVEL -           816.98 METRES
    TYPE OF OBSERVATION WELL -                         SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                 23.80 TO      26.20 METRES
    DISTANCE FROM PUMPING WELL -                 47.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                             152.00   MM
    PUMP TYPE -                                        SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                  TRIDENT DIGITAL,
    PUMPING RATE -                           6.516E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                               CONFINED
    AQUIFER DESCRIPTION -                              SANDY COARSE GRAVEL,
    AQUIFER THICKNESS -                             0.  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - NONE,
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DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		822.81		
0	0	0	0	0.0			0.00		822.81		
81	6	12	9	0.0			5.10		817.71		OPEN HOLE
81	6	15	9	0.0			5.10		817.71		OPEN HOLE
81	6	25	9	0.0			5.16		817.65		SCREENED-UNDEVELOPED
81	6	26	20	0.0			5.22		817.59		AFTER DEVELOPING
81	6	27	10	35.0			5.23		817.58		
81	6	27	16	0.0			5.24		817.57		
81	6	28	15	11.0			5.25		817.56		
81	6	28	20	37.0			5.25		817.56		INSTALLED PUMP TO 21.9M BELOW GROUND
81	6	28	20	47.0			5.62		817.19		INSTALLED OPEN END PIEZOMETER
81	6	28	20	48.0			5.62		817.19		START PUMP
81	6	28	20	49.0			7.79		815.02		
81	6	28	20	49.5			8.03		814.78		
81	6	28	20	50.0			8.16		814.65		
81	6	28	20	50.5			8.29		814.52		
81	6	28	20	51.0			8.48		814.33		
81	6	28	20	52.0			8.52		814.29		
81	6	28	20	53.0			8.60		814.21		
81	6	28	20	54.0			8.67		814.14		
81	6	28	20	56.0			8.74		814.07		
81	6	28	20	58.0			8.80		814.01		
81	6	28	21	3.0			8.83		813.98		
81	6	28	21	8.0			8.85		813.96		
81	6	28	21	13.0			8.84		813.97		
81	6	28	21	18.0			8.83		813.98		
81	6	28	21	28.0			9.74		813.07		INCREASED PUMP RATE
81	6	28	21	38.0			9.78		813.03		
81	6	28	21	48.0			9.59		813.22		DECREASED PUMP RATE
81	6	28	21	58.0			9.59		813.22		
81	6	28	22	8.0			9.59		813.22		
81	6	28	22	18.0			9.59		813.22		STOPPED PUMP
81	6	28	22	18.1			8.91		813.90		
81	6	28	22	18.2			8.29		814.52		
81	6	28	22	18.4			7.89		814.92		
81	6	28	22	18.5			7.63		815.18		
81	6	28	22	18.7			7.42		815.39		
81	6	28	22	18.8			7.21		815.60		
81	6	28	22	18.9			7.11		815.70		
81	6	28	22	19.0			7.01		815.80		
81	6	28	22	19.2			6.91		815.90		
81	6	28	22	19.3			6.81		816.00		
81	6	28	22	19.6			6.71		816.10		
81	6	28	22	19.9			6.61		816.20		
81	6	28	22	20.2			6.51		816.30		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW3, ** 12/11/81-11.19.40 **

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	28	22	20.5			6.41		816.40		
81	6	28	22	20.9			6.31		816.50		
81	6	28	22	21.5			6.21		816.60		
81	6	28	22	22.1			6.11		816.70		
81	6	28	22	23.1			6.01		816.80		
81	6	28	22	24.6			5.91		816.90		
81	6	28	22	27.6			5.81		817.00		
81	6	28	22	33.0			5.73		817.08		
81	6	29	7	55.0			5.63		817.18		
81	6	29	8	0.0			5.63	-0.20	817.18		START PUMP
81	6	29	8	.3	0.3		7.47	1.64	815.34		
81	6	29	8	.5	0.5		8.68	2.85	814.13		
81	6	29	8	1.0	1.0		9.17	3.34	813.64		
81	6	29	8	1.5	1.5		9.58	3.75	813.23		
81	6	29	8	2.0	2.0		9.89	4.06	812.92		
81	6	29	8	2.5	2.5		10.11	4.28	812.70		
81	6	29	8	3.0	3.0		11.08	5.25	811.73		INCREASED PUMP RATE
81	6	29	8	4.0	4.0		10.75	4.92	812.06		
81	6	29	8	5.0	5.0		10.96	5.13	811.85		
81	6	29	8	6.0	6.0		11.11	5.28	811.70		
81	6	29	8	8.0	8.0		11.21	5.38	811.60		
81	6	29	8	10.0	10.0		11.34	5.51	811.47		
81	6	29	8	15.0	15.0		11.44	5.61	811.37		
81	6	29	8	20.0	20.0		11.41	5.58	811.40		
81	6	29	8	25.0	25.0		11.40	5.57	811.41		
81	6	29	8	30.0	30.0		11.40	5.57	811.41	5.33	
81	6	29	8	40.0	40.0		11.40	5.57	811.41	5.35	
81	6	29	8	50.0	50.0		11.39	5.56	811.42	5.55	
81	6	29	9	0.0	60.0		11.39	5.56	811.42	5.30	
81	6	29	9	20.0	80.0		11.40	5.57	811.41	5.32	
81	6	29	9	50.0	110.0		11.38	5.55	811.43	5.29	
81	6	29	9	50.3	110.3		11.96	6.13	810.85		INCREASED PUMP RATE
81	6	29	9	50.5	110.5		12.17	6.34	810.64		
81	6	29	9	50.8	110.8		12.21	6.38	810.60		
81	6	29	9	51.0	111.0		12.34	6.51	810.47	7.12	
81	6	29	9	51.5	111.5		12.45	6.62	810.36		
81	6	29	9	52.0	112.0		12.50	6.67	810.31	6.44	
81	6	29	9	52.5	112.5		12.55	6.72	810.26		
81	6	29	9	53.0	113.0		12.60	6.77	810.21	6.44	
81	6	29	9	54.0	114.0		12.67	6.84	810.14	6.82	
81	6	29	9	55.0	115.0		12.71	6.88	810.10	6.44	
81	6	29	9	56.0	116.0		12.75	6.92	810.06	6.44	
81	6	29	9	58.0	118.0		12.77	6.94	810.04	6.63	
81	6	29	10	0.0	120.0		12.77	6.94	810.04	6.63	
81	6	29	10	5.0	125.0		12.79	6.96	810.02	6.55	

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	29	10	10.0	130.0		12.80	6.97	810.01	6.64	
81	6	29	10	15.0	135.0		12.82	6.99	809.99		
81	6	29	10	20.0	140.0		12.80	6.97	810.01	6.53	
81	6	29	10	30.0	150.0		12.80	6.97	810.01	6.58	
81	6	29	10	40.0	160.0		12.80	6.97	810.01	6.55	
81	6	29	10	50.0	170.0		12.79	6.96	810.02	6.55	
81	6	29	11	10.0	190.0		12.79	6.96	810.02	6.54	
81	6	29	11	30.0	210.0		12.78	6.95	810.03	6.52	
81	6	29	12	20.0	260.0		12.78	6.95	810.03	6.55	
81	6	29	13	10.0	310.0		12.75	6.92	810.06	6.55	
81	6	29	14	0.0	360.0		12.78	6.95	810.03	6.53	
81	6	29	14	50.0	410.0		12.76	6.93	810.05	6.56	
81	6	29	16	30.0	510.0		12.80	6.97	810.01	6.56	
81	6	29	19	0.0	660.0		12.84	7.01	809.97		STOPPED PUMP
81	6	29	19	.3	660.3		9.45	3.62	813.36		
81	6	29	19	.5	660.5		8.95	3.12	813.86		
81	6	29	19	.8	660.8		8.41	2.58	814.40		
81	6	29	19	1.0	661.0		8.07	2.24	814.74		
81	6	29	19	1.5	661.5		7.57	1.74	815.24		
81	6	29	19	2.0	662.0		7.23	1.40	815.58		
81	6	29	19	2.5	662.5		6.98	1.15	815.83		
81	6	29	19	3.0	663.0		6.80	0.97	816.01		
81	6	29	19	4.0	664.0		6.53	0.70	816.28		
81	6	29	19	5.0	665.0		6.35	0.52	816.46		
81	6	29	19	6.0	666.0		6.21	0.38	816.60		
81	6	29	19	8.0	668.0		6.08	0.25	816.73		
81	6	29	19	10.0	670.0		5.98	0.15	816.83		
81	6	29	19	15.0	675.0		5.86	0.03	816.95		
81	6	29	19	20.0	680.0		5.81	-0.02	817.00		
81	6	29	19	25.0	685.0		5.78	-0.05	817.03		
81	6	29	19	30.0	690.0		5.76	-0.07	817.05		
81	6	29	19	40.0	700.0		5.74	-0.09	817.07		
81	6	29	19	50.0	710.0		5.73	-0.10	817.08		
81	6	29	20	0.0	720.0		5.71	-0.12	817.10		REMOVED PUMP AND OPEN END PIEZOMETER
81	6	29	20	15.0	735.0		5.74	-0.09	817.07		
81	6	29	20	41.0	761.0		5.72	-0.11	817.09		
81	6	30	7	55.0	1435.0		5.72	-0.11	817.09		
81	6	30	9	55.0	1555.0		5.69	-0.14	817.12		
81	6	30	10	0.0	1560.0		5.69	-0.14	817.12		START PUMP IN OW2
81	6	30	10	.5	1560.5		5.69	-0.14	817.12		
81	6	30	10	1.0	1561.0		5.69	-0.14	817.12		
81	6	30	10	1.5	1561.5		5.70	-0.13	817.11		
81	6	30	10	2.0	1562.0		5.70	-0.13	817.11		
81	6	30	10	2.5	1562.5		5.71	-0.12	817.10		
81	6	30	10	3.0	1563.0		5.72	-0.11	817.09		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	30	10	4.0	1564.0		5.73	-0.10	817.08		
81	6	30	10	5.0	1565.0		5.74	-0.09	817.07		
81	6	30	10	6.0	1566.0		5.74	-0.09	817.07		
81	6	30	10	8.0	1568.0		5.73	-0.10	817.08		STOPPED PUMP IN OW2
81	6	30	10	25.0	1585.0		5.69	-0.14	817.12		
81	6	30	10	30.0	1590.0		5.69	-0.14	817.12		RESTART PUMP IN OW2
81	6	30	10	30.5	1590.5		5.69	-0.14	817.12		
81	6	30	10	31.0	1591.0		5.70	-0.14	817.12		
81	6	30	10	31.5	1591.5		5.70	-0.13	817.11		
81	6	30	10	32.0	1592.0		5.71	-0.12	817.10		
81	6	30	10	32.5	1592.5		5.72	-0.11	817.09		
81	6	30	10	33.0	1593.0		5.74	-0.09	817.07		
81	6	30	10	34.0	1594.0		5.76	-0.07	817.05		
81	6	30	10	35.0	1595.0		5.77	-0.06	817.04		
81	6	30	10	36.0	1596.0		5.78	-0.05	817.03		
81	6	30	10	38.0	1598.0		5.80	-0.03	817.01		STOPPED PUMP IN OW2
81	6	30	10	40.0	1600.0		5.80	-0.03	817.01		
81	6	30	10	42.0	1602.0		5.80	-0.03	817.01		
81	6	30	10	44.0	1604.0		5.78	-0.05	817.03		
81	6	30	10	46.0	1606.0		5.76	-0.07	817.05		
81	6	30	10	48.0	1608.0		5.75	-0.08	817.06		
81	6	30	10	50.0	1610.0		5.74	-0.09	817.07		
81	6	30	10	52.0	1612.0		5.73	-0.10	817.08		
81	6	30	10	54.0	1614.0		5.72	-0.11	817.09		
81	6	30	11	5.0	1625.0		5.70	-0.13	817.11		
81	6	30	11	10.0	1630.0		5.70	-0.13	817.11		RESTART PUMP IN OW2
81	6	30	11	10.5	1630.5		5.70	-0.13	817.11		
81	6	30	11	11.0	1631.0		5.70	-0.13	817.11		
81	6	30	11	11.5	1631.5		5.70	-0.13	817.11		
81	6	30	11	12.0	1632.0		5.71	-0.13	817.11		
81	6	30	11	12.5	1632.5		5.71	-0.12	817.10		
81	6	30	11	13.0	1633.0		5.72	-0.11	817.09		
81	6	30	11	14.0	1634.0		5.73	-0.10	817.08		
81	6	30	11	15.0	1635.0		5.74	-0.09	817.07		
81	6	30	11	16.0	1636.0		5.76	-0.07	817.05		
81	6	30	11	18.0	1638.0		5.78	-0.05	817.03		
81	6	30	11	20.0	1640.0		5.80	-0.03	817.01		
81	6	30	11	25.0	1645.0		5.84	0.01	816.97		
81	6	30	11	30.0	1650.0		5.85	0.02	816.96		
81	6	30	11	35.0	1655.0		5.86	0.03	816.95		
81	6	30	11	40.0	1660.0		5.87	0.04	816.94		
81	6	30	11	50.0	1670.0		5.88	0.05	816.93		
81	6	30	12	0.0	1680.0		5.89	0.06	816.92		
81	6	30	12	10.0	1690.0		5.89	0.06	816.92		
81	6	30	12	30.0	1710.0		5.90	0.07	816.91		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER - OW3, ** 12/11/81-11.19.40 ** PAGE 6

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	30	12	56.0	1736.0		5.90	0.07	816.91		
81	6	30	13	45.0	1785.0		5.90	0.07	816.91		
81	6	30	14	30.0	1830.0		5.90	0.07	816.91		STOPPED PUMP IN OW2
81	6	30	14	30.5	1830.5		5.90	0.07	816.91		
81	6	30	14	31.0	1831.0		5.90	0.07	816.91		
81	6	30	14	31.5	1831.5		5.90	0.07	816.91		
81	6	30	14	32.0	1832.0		5.90	0.07	816.91		
81	6	30	14	32.5	1832.5		5.89	0.06	816.92		
81	6	30	14	33.0	1833.0		5.88	0.05	816.93		
81	6	30	14	34.0	1834.0		5.87	0.04	816.94		
81	6	30	14	35.0	1835.0		5.86	0.03	816.95		
81	6	30	14	36.0	1836.0		5.85	0.02	816.96		
81	6	30	14	38.0	1838.0		5.82	-0.01	816.99		
81	6	30	14	40.0	1840.0		5.80	-0.03	817.01		
81	6	30	14	45.0	1845.0		5.77	-0.06	817.04		
81	6	30	14	50.0	1850.0		5.74	-0.09	817.07		
81	6	30	14	55.0	1855.0		5.73	-0.10	817.08		
81	6	30	15	0.0	1860.0		5.73	-0.10	817.08		
81	6	30	15	10.0	1870.0		5.71	-0.12	817.10		
81	6	30	15	22.0	1882.0		5.70	-0.13	817.11		
81	6	30	16	30.0	1950.0		5.70	-0.13	817.11		
81	7	2	9	10.0	4390.0		5.70	-0.13	817.11		
81	7	3	8	0.0	5760.0		5.70	-0.13	817.11		
81	7	6	9	40.0	10180.0		5.72	-0.11	817.09		
81	7	7	8	0.0	11520.0		5.73	-0.10	817.08		
81	7	8	9	30.0	13050.0		5.74	-0.09	817.07		
81	7	9	14	0.0	14760.0		5.74	-0.09	817.07		
81	7	10	8	0.0	15840.0		5.76	-0.07	817.05		
81	7	13	9	0.0	20220.0		5.78	-0.05	817.03		
81	7	15	11	30.0	23250.0		5.80	-0.03	817.01		
81	7.	16	11	15.0	24675.0		5.81	-0.02	817.00		
81	7	20	8	15.0	30255.0		5.83	0.	816.98		
81	7	21	8	20.0	31700.0		5.81	-0.02	817.00		
81	7	24	10	45.0	36165.0		5.83	0.	816.98		
81	7	24	10	50.0	36170.0		5.83	0.	816.98		START PUMPING PW2
81	7	24	10	51.5	36171.5		5.84	0.01	816.97		
81	7	24	10	51.0	36171.0		5.87	0.04	816.94		
81	7	24	10	51.5	36171.5		5.92	0.09	816.89		
81	7	24	10	52.0	36172.0		5.97	0.14	816.84		
81	7	24	10	52.5	36172.5		6.01	0.18	816.80		
81	7	24	10	53.0	36173.0		6.05	0.22	816.76		
81	7	24	10	54.0	36174.0		6.12	0.29	816.69		
81	7	24	10	55.0	36175.0		6.18	0.35	816.63		
81	7	24	10	56.0	36176.0		6.22	0.39	816.59		
81	7	24	10	58.0	36178.0		6.29	0.46	816.52		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	24	11	0.0	36180.0		6.34	0.51	816.47		
81	7	24	11	5.0	36185.0		6.42	0.59	816.39		
81	7	24	11	10.0	36190.0		6.46	0.63	816.35		
81	7	24	11	15.0	36195.0		6.48	0.65	816.33		
81	7	24	11	20.0	36200.0		6.50	0.67	816.31		
81	7	24	11	30.0	36210.0		6.52	0.69	816.29		
81	7	24	11	40.0	36220.0		6.53	0.70	816.28		
81	7	24	11	50.0	36230.0		6.53	0.70	816.28		
81	7	24	12	10.0	36250.0		6.56	0.73	816.25		
81	7	24	12	30.0	36270.0		6.56	0.73	816.25		
81	7	24	13	20.0	36320.0		6.57	0.74	816.24		
81	7	24	14	10.0	36370.0		6.58	0.75	816.23		
81	7	24	15	0.0	36420.0		6.58	0.75	816.23		
81	7	24	15	50.0	36470.0		6.59	0.75	816.23		
81	7	24	17	35.0	36575.0		6.60	0.77	816.21		
81	7	24	19	15.0	36675.0		6.60	0.77	816.21		
81	7	24	20	50.0	36770.0		6.61	0.78	816.20		
81	7	25	0	10.0	36970.0		6.62	0.79	816.19		
81	7	25	3	0.0	37140.0		6.64	0.81	816.17		
81	7	25	6	45.0	37365.0		6.65	0.82	816.16		
81	7	25	14	45.0	37845.0		6.68	0.85	816.13		
81	7	25	22	45.0	38325.0		6.71	0.88	816.10		
81	7	26	6	45.0	38805.0		6.72	0.89	816.09		
81	7	26	8	5.0	38885.0		6.72	0.89	816.09		
81	7	26	8	55.0	38935.0		6.73	0.90	816.08		
81	7	26	8	6.0	38886.0		6.73	0.90	816.08		
81	7	26	8	6.5	38886.5		6.74	0.91	816.07		
81	7	26	8	7.0	38887.0		6.75	0.92	816.06		
81	7	26	8	7.5	38887.5		6.77	0.94	816.04		
81	7	26	8	8.0	38888.0		6.78	0.94	816.04		
81	7	26	8	9.0	38889.0		6.79	0.96	816.02		
81	7	26	8	10.0	38890.0		6.81	0.98	816.00		
81	7	26	8	11.0	38891.0		6.82	0.99	815.99		
81	7	26	8	13.0	38893.0		6.83	1.00	815.98		
81	7	26	8	15.0	38895.0		6.85	1.02	815.96		
81	7	26	8	20.0	38900.0		6.86	1.03	815.95		
81	7	26	8	25.0	38905.0		6.87	1.04	815.94		
81	7	26	8	25.5	38905.5		6.88	1.04	815.94		
81	7	26	8	26.0	38906.0		6.88	1.05	815.93		
81	7	26	8	26.5	38906.5		6.90	1.07	815.92		
81	7	26	8	27.0	38907.0		6.91	1.08	815.90		
81	7	26	8	27.5	38907.5		6.92	1.09	815.89		
81	7	26	8	28.0	38908.0		6.93	1.10	815.88		
81	7	26	8	29.0	38909.0		6.96	1.13	815.86		
81	7	26	8	30.0	38910.0		6.97	1.14	815.84		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	26	8	31.0	38911.0		6.99	1.16	815.82		
81	7	26	8	33.0	38913.0		7.01	1.18	815.80		
81	7	26	8	35.0	38915.0		7.02	1.19	815.79		
81	7	26	8	40.0	38920.0		7.05	1.22	815.76		
81	7	26	8	45.0	38925.0		7.06	1.23	815.75		
81	7	26	8	50.0	38930.0		7.07	1.24	815.74		
81	7	26	8	55.0	38935.0		7.08	1.25	815.73		
81	7	26	9	5.0	38945.0		7.10	1.27	815.71		
81	7	26	9	15.0	38955.0		7.10	1.27	815.71		
81	7	26	9	25.0	38965.0		7.10	1.27	815.71		
81	7	26	9	45.0	38985.0		7.10	1.27	815.71		
81	7	26	10	5.0	39005.0		7.11	1.28	815.70		PUMP OFF 5 MINS AT 10.47
81	7	26	11	10.0	39070.0		6.98	1.15	815.83		
81	7	26	11	15.0	39075.0		7.00	1.17	815.81		
81	7	26	11	30.0	39090.0		7.01	1.18	815.80		
81	7	26	11	40.0	39100.0		7.01	1.18	815.80		
81	7	26	11	50.0	39110.0		7.02	1.19	815.79		
81	7	26	12	10.0	39130.0		7.02	1.19	815.79		
81	7	26	12	40.0	39160.0		7.03	1.20	815.78		
81	7	26	13	30.0	39210.0		7.03	1.20	815.78		
81	7	26	14	20.0	39260.0		7.03	1.20	815.78		
81	7	26	15	10.0	39310.0		7.03	1.20	815.78		
81	7	26	16	0.0	39360.0		7.04	1.21	815.77		
81	7	26	17	40.0	39460.0		7.05	1.22	815.76		
81	7	26	19	20.0	39560.0		7.05	1.22	815.76		
81	7	26	21	0.0	39660.0		7.06	1.23	815.75		
81	7	27	7	25.0	40285.0		7.10	1.27	815.71		
81	7	27	15	10.0	40750.0		7.13	1.30	815.68		
81	7	27	22	45.0	41205.0		7.13	1.30	815.68		
81	7	28	6	50.0	41690.0		7.14	1.31	815.67		
81	7	28	9	0.0	41820.0		7.15	1.32	815.66		
81	7	28	9	.5	41820.5		7.14	1.31	815.67		
81	7	28	9	1.0	41821.0		7.10	1.27	815.71		
81	7	28	9	1.5	41821.5		7.04	1.21	815.77		
81	7	28	9	2.0	41822.0		6.97	1.14	815.84		
81	7	28	9	2.5	41822.5		6.89	1.06	815.92		
81	7	28	9	3.0	41823.0		6.84	1.01	815.97		
81	7	28	9	4.0	41824.0		6.73	0.90	816.08		
81	7	28	9	5.0	41825.0		6.65	0.82	816.16		
81	7	28	9	6.0	41826.0		6.58	0.75	816.23		
81	7	28	9	8.0	41828.0		6.50	0.67	816.31		
81	7	28	9	10.0	41830.0		6.43	0.60	816.38		
81	7	28	9	15.0	41835.0		6.33	0.50	816.48		
81	7	28	9	20.0	41840.0		6.27	0.44	816.54		
81	7	28	9	25.0	41845.0		6.24	0.41	816.57		

DATE YR	DATE MON	DATE DAY	TIME HR	TIME MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	28	9	30.0	41850.0		6.22	0.39	816.59		
81	7	28	9	40.0	41860.0		6.19	0.36	816.62		
81	7	28	9	50.0	41870.0		6.17	0.34	816.64		
81	7	28	10	0.0	41880.0		6.16	0.33	816.65		
81	7	28	10	20.0	41900.0		6.14	0.31	816.67		
81	7	28	10	40.0	41920.0		6.13	0.30	816.68		
81	7	28	11	30.0	41970.0		6.12	0.29	816.69		
81	7	28	12	20.0	42020.0		6.10	0.27	816.71		
81	7	28	13	10.0	42070.0		6.10	0.27	816.71		
81	7	28	14	0.0	42120.0		6.10	0.27	816.71		
81	7	28	22	52.0	42652.0		6.06	0.23	816.75		
81	7	29	6	15.0	43095.0		6.04	0.21	816.77		
81	7	29	10	52.0	43372.0		6.06	0.23	816.75		
81	7	29	14	45.0	43605.0		6.04	0.21	816.77		
81	7	29	22	0.0	44040.0		6.03	0.20	816.78		
81	7	30	9	0.0	44700.0		6.03	0.20	816.78		
81	7	31	8	50.0	46130.0		6.01	0.18	816.80		
81	8	1	9	40.0	47620.0		6.00	0.17	816.81		
81	8	2	11	30.0	49170.0		6.00	0.17	816.81		
81	8	3	9	35.0	50495.0		5.99	0.16	816.82		
81	8	4	11	5.0	52025.0		5.99	0.16	816.82		
81	8	5	9	30.0	53370.0		5.99	0.16	816.82		
81	8	7	9	45.0	56265.0		6.00	0.17	816.81		
81	8	8	9	5.0	57665.0		6.00	0.17	816.81		
81	8	9	9	15.0	59115.0		6.00	0.17	816.81		
81	8	10	9	5.0	60545.0		6.00	0.17	816.81		

RESIDUAL DRAWDOWN

OBSERVATION WELL = OW5,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (T1)	RATIO (T/T1)	DRAWDOWN (S)
660.3	.3	2641.00	3.62
660.5	.5	1321.00	3.12
660.8	.8	881.00	2.58
661.0	1.0	661.00	2.24
661.5	1.5	441.00	1.74
662.0	2.0	331.00	1.40
662.5	2.5	265.00	1.15
663.0	3.0	221.00	.97
664.0	4.0	166.00	.70
665.0	5.0	133.00	.52
666.0	6.0	111.00	.38
668.0	8.0	83.50	.25
670.0	10.0	67.00	.15
675.0	15.0	45.00	.03
680.0	20.0	34.00	-.02
685.0	25.0	27.40	-.05
690.0	30.0	23.00	-.07
700.0	40.0	17.50	-.09
710.0	50.0	14.20	-.10
720.0	60.0	12.00	-.12
735.0	75.0	9.80	-.09
761.0	101.0	7.53	-.11
1435.0	775.0	1.85	-.11
1555.0	895.0	1.74	-.14
1560.0	900.0	1.73	-.14
1560.5	900.5	1.73	-.14
1561.0	901.0	1.73	-.14
1561.5	901.5	1.73	-.13
1562.0	902.0	1.73	-.13
1562.5	902.5	1.73	-.12
1563.0	903.0	1.73	-.11
1564.0	904.0	1.73	-.10
1565.0	905.0	1.73	-.09
1566.0	906.0	1.73	-.09
1568.0	908.0	1.73	-.10
1585.0	925.0	1.71	-.14
1590.0	930.0	1.71	-.14
1590.5	930.5	1.71	-.14
1591.0	931.0	1.71	-.14
1591.5	931.5	1.71	-.13
1592.0	932.0	1.71	-.12
1592.5	932.5	1.71	-.11
1593.0	933.0	1.71	-.09
1594.0	934.0	1.71	-.07
1595.0	935.0	1.71	-.06
1596.0	936.0	1.71	-.05
1598.0	938.0	1.70	-.03
1600.0	940.0	1.70	-.03
1602.0	942.0	1.70	-.03
1604.0	944.0	1.70	-.05
1606.0	946.0	1.70	-.07
1608.0	948.0	1.70	-.08
1610.0	950.0	1.69	-.09
1612.0	952.0	1.69	-.10
1614.0	954.0	1.69	-.11
1625.0	965.0	1.68	-.13
1630.0	970.0	1.68	-.13
1630.5	970.5	1.68	-.13
1631.0	971.0	1.68	-.13
1631.5	971.5	1.68	-.13
1632.0	972.0	1.68	-.13
1632.5	972.5	1.68	-.12
1633.0	973.0	1.68	-.11
1634.0	974.0	1.68	-.10
1635.0	975.0	1.68	-.09
1636.0	976.0	1.68	-.07
1638.0	978.0	1.67	-.05
1640.0	980.0	1.67	-.03
1645.0	985.0	1.67	.01
1650.0	990.0	1.67	.02
1655.0	995.0	1.66	.03
1660.0	1000.0	1.66	.04
1670.0	1010.0	1.65	.05
1680.0	1020.0	1.65	.06
1690.0	1030.0	1.64	.06
1710.0	1050.0	1.63	.07
1736.0	1076.0	1.61	.07
1785.0	1125.0	1.59	.07
1830.0	1170.0	1.56	.07
1830.5	1170.5	1.56	.07
1831.0	1171.0	1.56	.07
1831.5	1171.5	1.56	.07
1832.0	1172.0	1.56	.07
1832.5	1172.5	1.56	.06
1833.0	1173.0	1.56	.05
1834.0	1174.0	1.56	.04
1835.0	1175.0	1.56	.03
1836.0	1176.0	1.56	.02
1838.0	1178.0	1.56	-.01
1840.0	1180.0	1.56	-.03
1845.0	1185.0	1.56	-.06
1850.0	1190.0	1.55	-.09
1855.0	1195.0	1.55	-.10
1860.0	1200.0	1.55	-.10
1870.0	1210.0	1.55	-.12
1882.0	1222.0	1.54	-.13
1950.0	1290.0	1.51	-.13
4390.0	3730.0	1.18	-.13
5760.0	5100.0	1.13	-.13
10180.0	9520.0	1.07	-.11
11520.0	10860.0	1.06	-.10
13050.0	12390.0	1.05	-.09
14760.0	14100.0	1.05	-.09
15840.0	15180.0	1.04	-.07
20220.0	19560.0	1.03	-.05
23250.0	22590.0	1.03	-.03
24675.0	24015.0	1.03	-.02
30255.0	29595.0	1.02	0.00
31700.0	31040.0	1.02	-.02
36165.0	35505.0	1.02	0.00
36170.0	35510.0	1.02	0.00
36171.5	35511.5	1.02	.01

36171.0	35511.0	1.02	.04
36171.5	35511.5	1.02	.09
36172.0	35512.0	1.02	.14
36172.5	35512.5	1.02	.18
36173.0	35513.0	1.02	.22
36174.0	35514.0	1.02	.29
36175.0	35515.0	1.02	.35
36176.0	35516.0	1.02	.39
36178.0	35518.0	1.02	.46
36180.0	35520.0	1.02	.51
36185.0	35525.0	1.02	.59
36190.0	35530.0	1.02	.63
36195.0	35535.0	1.02	.65
36200.0	35540.0	1.02	.67
36210.0	35550.0	1.02	.69
36220.0	35560.0	1.02	.70
36230.0	35570.0	1.02	.70
36250.0	35590.0	1.02	.73
36270.0	35610.0	1.02	.73
36320.0	35660.0	1.02	.74
36370.0	35710.0	1.02	.75
36420.0	35760.0	1.02	.75
36470.0	35810.0	1.02	.76
36575.0	35915.0	1.02	.77
36675.0	36015.0	1.02	.77
36770.0	36110.0	1.02	.78
36970.0	36310.0	1.02	.79
37140.0	36480.0	1.02	.81
37365.0	36705.0	1.02	.82
37845.0	37185.0	1.02	.85
38325.0	37665.0	1.02	.88
38805.0	38145.0	1.02	.89
38885.0	38225.0	1.02	.89
38935.0	38275.0	1.02	.90
38886.0	38226.0	1.02	.90
38886.5	38226.5	1.02	.91
38887.0	38227.0	1.02	.92
38887.5	38227.5	1.02	.94
38888.0	38228.0	1.02	.95
38889.0	38229.0	1.02	.96
38890.0	38230.0	1.02	.98
38891.0	38231.0	1.02	.99
38893.0	38233.0	1.02	1.00
38895.0	38235.0	1.02	1.02
38900.0	38240.0	1.02	1.03
38905.0	38245.0	1.02	1.04
38905.5	38245.5	1.02	1.05
38906.0	38246.0	1.02	1.05
38906.5	38246.5	1.02	1.07
38907.0	38247.0	1.02	1.08
38907.5	38247.5	1.02	1.09
38908.0	38248.0	1.02	1.10
38909.0	38249.0	1.02	1.13
38910.0	38250.0	1.02	1.14
38911.0	38251.0	1.02	1.16
38913.0	38253.0	1.02	1.18
38915.0	38255.0	1.02	1.19
38920.0	38260.0	1.02	1.22
38925.0	38265.0	1.02	1.23
38930.0	38270.0	1.02	1.24

38935.0	38275.0	1.02	1.25
38945.0	38285.0	1.02	1.27
38955.0	38295.0	1.02	1.27
38965.0	38305.0	1.02	1.27
38985.0	38325.0	1.02	1.27
39005.0	38345.0	1.02	1.28
39070.0	38410.0	1.02	1.15
39075.0	38415.0	1.02	1.17
39090.0	38430.0	1.02	1.18
39100.0	38440.0	1.02	1.18
39110.0	38450.0	1.02	1.19
39130.0	38470.0	1.02	1.19
39160.0	38500.0	1.02	1.20
39210.0	38550.0	1.02	1.20
39260.0	38600.0	1.02	1.20
39310.0	38650.0	1.02	1.20
39360.0	38700.0	1.02	1.21
39460.0	38800.0	1.02	1.22
39560.0	38900.0	1.02	1.22
39660.0	39000.0	1.02	1.23
40285.0	39625.0	1.02	1.27
40750.0	40090.0	1.02	1.30
41205.0	40545.0	1.02	1.30
41690.0	41030.0	1.02	1.31
41820.0	41160.0	1.02	1.32
41820.5	41160.5	1.02	1.31
41821.0	41161.0	1.02	1.27
41821.5	41161.5	1.02	1.21
41822.0	41162.0	1.02	1.14
41822.5	41162.5	1.02	1.06
41823.0	41163.0	1.02	1.01
41824.0	41164.0	1.02	.90
41825.0	41165.0	1.02	.82
41826.0	41166.0	1.02	.75
41828.0	41168.0	1.02	.67
41830.0	41170.0	1.02	.60
41835.0	41175.0	1.02	.50
41840.0	41180.0	1.02	.44
41845.0	41185.0	1.02	.41
41850.0	41190.0	1.02	.39
41860.0	41200.0	1.02	.36
41870.0	41210.0	1.02	.34
41880.0	41220.0	1.02	.33
41900.0	41240.0	1.02	.31
41920.0	41260.0	1.02	.30
41970.0	41310.0	1.02	.29
42020.0	41360.0	1.02	.27
42070.0	41410.0	1.02	.27
42120.0	41460.0	1.02	.27
42652.0	41992.0	1.02	.23
43095.0	42435.0	1.02	.21
43372.0	42712.0	1.02	.23
43605.0	42945.0	1.02	.21
44040.0	43380.0	1.02	.20
44700.0	44040.0	1.01	.20
46130.0	45470.0	1.01	.18
47620.0	46960.0	1.01	.17
49170.0	48510.0	1.01	.17
50495.0	49835.0	1.01	.16
52025.0	51365.0	1.01	.16
53370.0	52710.0	1.01	.16
56265.0	55605.0	1.01	.17
57665.0	57005.0	1.01	.17
59115.0	58455.0	1.01	.17
60545.0	59885.0	1.01	.17

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*                                                                              *
*                                                                              *
*                             GOLDER ASSOCIATES                                *
*                                                                              *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*                                                                              *
*        PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER -     OW4,              *
*                                                                              *
*                                                                              *
*                            12/11/81-11,11,44                                 *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *

    PUMPED WELL NUMBER - OW4,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 27/ 6/81-30,0/16
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 28/ 6/81-30,0/17

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                          838,06 METRES
    DATUM POINT -                                      TOP OF 152MM PVC CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -          3,07 METRES
    DEPTH TO STATIC WATER LEVEL -                  ,24 METRES
    ELEVATION OF STATIC WATER LEVEL -           840,89 METRES
    TYPE OF OBSERVATION WELL -                         SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                104,10 TO    106,70 METRES
    DISTANCE FROM PUMPING WELL -                 21,50 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                             152,00   MM
    PUMP TYPE -                                        SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                  TRIDENT DIGITAL,
    PUMPING RATE -                          7,167E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                               CONFINED
    AQUIFER DESCRIPTION -                              SANDY GRAVEL,
    AQUIFER THICKNESS -                    UNDEFINED  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - WELL FLOWING AT 5,2 L/SEC AFTER BAILING,
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* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW4, ** 12/11/81-11.11.44 ** PAGE 2

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		841.13		
0	0	0	0	0.0			0.00		841.13		
81	6	24	14	0.0			0.00		841.13	4.26	AIR SURGED SCREENED-FLOWING ARTESIAN
81	6	25	8	0.0			0.00		841.13	3.90	
81	6	26	14	35.0			0.00		841.13	3.03	
81	6	27	8	15.0			0.00		841.13	2.84	
81	6	27	14	15.0			0.00		841.13	2.67	
81	6	27	15	3.0			0.00		841.13	5.05	
81	6	27	16	30.0			0.00	-0.24	841.13		START PUMPING AT 16.30
81	6	27	16	30.5	0.5		4.00	3.76	837.13		INITIAL METER READING 6476340
81	6	27	16	31.0	1.0		4.05	3.81	837.08	7.58	
81	6	27	16	31.5	1.5		4.12	3.88	837.01		
81	6	27	16	32.0	2.0		4.11	3.87	837.02	7.20	
81	6	27	16	32.5	2.5		4.22	3.98	836.91		
81	6	27	16	33.0	3.0		4.17	3.93	836.96	7.05	
81	6	27	16	34.0	4.0		4.19	3.95	836.94	7.35	
81	6	27	16	35.0	5.0		4.19	3.95	836.94	7.35	
81	6	27	16	36.0	6.0		4.19	3.95	836.94	7.42	
81	6	27	16	38.0	8.0		4.20	3.96	836.93	7.35	
81	6	27	16	40.0	10.0		4.21	3.97	836.92	7.50	
81	6	27	16	45.0	15.0		4.22	3.98	836.91	7.42	
81	6	27	16	50.0	20.0		4.23	3.99	836.90	7.58	
81	6	27	16	55.0	25.0		4.24	4.00	836.89	7.42	
81	6	27	17	0.0	30.0		4.23	3.99	836.90	7.27	
81	6	27	17	10.0	40.0		4.21	3.97	836.92	7.27	
81	6	27	17	20.0	50.0		4.20	3.96	836.93	7.35	
81	6	27	17	30.0	60.0		4.21	3.97	836.92	6.97	
81	6	27	17	50.0	80.0		4.22	3.98	836.91	7.20	
81	6	27	18	10.0	100.0		4.25	4.01	836.88	7.31	
81	6	27	19	0.0	150.0		4.31	4.07	836.82	7.23	
81	6	27	19	50.0	200.0		4.36	4.12	836.77	7.15	
81	6	27	20	40.0	250.0		4.41	4.17	836.72	7.18	
81	6	27	21	30.0	300.0		4.47	4.23	836.66	7.20	
81	6	27	23	10.0	400.0		4.57	4.33	836.56	7.21	
81	6	28	0	50.0	500.0		4.65	4.41	836.48	7.21	
81	6	28	2	30.0	600.0		4.73	4.49	836.40	7.20	
81	6	28	6	53.0	863.0		4.92	4.68	836.21	7.19	
81	6	28	9	10.0	1000.0		4.98	4.74	836.15	7.14	
81	6	28	11	0.0	1110.0		5.03	4.79	836.10		WATER SAMPLED PH 7.1 TX12C COND. 380
81	6	28	13	50.0	1280.0		5.14	4.90	835.99	7.11	
81	6	28	17	30.0	1500.0		5.21	4.97	835.92		FLOW METER READING 6618220
81	6	28	17	30.2	1500.2		3.07	2.83	838.06		
81	6	28	17	30.4	1500.4		2.81	2.57	838.32		
81	6	28	17	30.5	1500.5		2.67	2.43	838.46		
81	6	28	17	30.5	1500.5		2.57	2.33	838.56		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	6	28	17	30.6	1500.6		2.47	2.23	838.66		
81	6	28	17	55.0	1525.0		1.41	1.17	839.72		
81	6	28	17	57.0	1527.0		1.37	1.13	839.76		
81	6	28	18	0.0	1530.0		1.35	1.11	839.78		
81	6	28	18	10.0	1540.0		1.31	1.07	839.82		
81	6	28	18	20.0	1550.0		1.29	1.05	839.84		
81	6	28	18	33.0	1563.0		1.21	0.97	839.92		
81	6	29	7	20.0	2330.0		.88	0.64	840.25		
81	6	29	21	20.0	3170.0		.67	0.43	840.46		
81	6	30	7	25.0	3775.0		.58	0.34	840.55		
81	7	1	8	0.0	5250.0		.48	0.24	840.65		
81	7	2	8	0.0	6690.0		.38	0.14	840.75		
81	7	3	8	0.0	8130.0		.36	0.12	840.77		
81	7	6	14	30.0	12840.0		.25	0.01	840.88		
81	7	7	9	30.0	13980.0		.24	0.	840.89		
81	7	8	7	30.0	15300.0		.71	0.47	840.42		DEVELOPING PW1
81	7	8	18	30.0	15960.0		5.91	5.67	835.22		DEVELOPING PW1
81	7	9	8	0.0	16770.0		2.11	1.87	839.02		PW1 RECOVERING AFTER DEVELOPING
81	7	13	9	0.0	22590.0		1.64	1.40	839.49		
81	7	15	10	30.0	25560.0		1.70	1.46	839.43		
81	7	20	8	15.0	32625.0		1.64	1.40	839.49		
81	7	20	14	0.0	32970.0		1.74	1.50	839.39		
81	7	20	14	.5	32970.5		2.37	2.13	838.76		
81	7	20	14	1.0	32971.0		2.69	2.45	838.44		
81	7	20	14	1.3	32971.3		2.76	2.52	838.37		
81	7	20	14	1.5	32971.5		2.81	2.57	838.32		
81	7	20	14	1.8	32971.8		2.87	2.63	838.26		
81	7	20	14	2.0	32972.0		2.90	2.66	838.23		
81	7	20	14	2.3	32972.3		2.95	2.71	838.18		
81	7	20	14	2.5	32972.5		2.97	2.73	838.16		
81	7	20	14	2.8	32972.8		3.00	2.76	838.13		
81	7	20	14	3.0	32973.0		3.01	2.77	838.12		
81	7	20	14	3.3	32973.3		3.03	2.79	838.10		
81	7	20	14	3.5	32973.5		3.05	2.81	838.08		
81	7	20	14	3.8	32973.8		3.06	2.82	838.07		
81	7	20	14	4.0	32974.0		3.08	2.84	838.05		
81	7	20	14	4.3	32974.3		3.09	2.85	838.04		
81	7	20	14	4.5	32974.5		3.10	2.86	838.03		
81	7	20	14	4.8	32974.8		3.11	2.87	838.02		
81	7	20	14	5.0	32975.0		3.12	2.88	838.01		
81	7	20	14	5.5	32975.5		3.14	2.90	837.99		
81	7	20	14	6.0	32976.0		3.16	2.92	837.97		
81	7	20	14	7.0	32977.0		3.19	2.95	837.94		
81	7	20	14	8.0	32978.0		3.22	2.98	837.91		
81	7	20	14	10.0	32980.0		3.26	3.02	837.87		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW4, ** 12/11/81-11.11.44 ** PAGE 4

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	20	14	15.0	32985.0		3.33	3.09	837.80		
81	7	20	14	20.0	32990.0		3.37	3.13	837.76		
81	7	20	14	25.0	32995.0		3.42	3.18	837.71		
81	7	20	14	30.0	33000.0		3.45	3.21	837.68		
81	7	20	14	40.0	33010.0		3.51	3.27	837.62		
81	7	20	14	50.0	33020.0		3.57	3.33	837.56		
81	7	20	15	0.0	33030.0		3.63	3.39	837.50		
81	7	20	15	20.0	33050.0		3.71	3.47	837.42		
81	7	20	15	40.0	33070.0		3.83	3.59	837.30		
81	7	20	16	30.0	33120.0		4.03	3.79	837.10		
81	7	20	17	20.0	33170.0		4.22	3.98	836.91		
81	7	20	18	10.0	33220.0		4.44	4.20	836.69		
81	7	20	19	0.0	33270.0		4.66	4.42	836.47		
81	7	20	20	40.0	33370.0		5.00	4.76	836.13		
81	7	20	22	20.0	33470.0		5.35	5.11	835.78		
81	7	21	0	10.0	33580.0		5.66	5.42	835.47		
81	7	21	4	10.0	33820.0		6.32	6.08	834.81		
81	7	21	6	40.0	33970.0		6.70	6.46	834.43		
81	7	21	15	5.0	34475.0		6.85	6.61	834.28		
81	7	21	23	25.0	34975.0		8.81	8.57	832.32		
81	7	22	7	45.0	35475.0		9.75	9.51	831.38		
81	7	22	15	5.0	35915.0		10.43	10.19	830.70		
81	7	22	23	5.0	36395.0		11.24	11.00	829.89		
81	7	23	7	5.0	36875.0		11.97	11.73	829.16		
81	7	23	15	5.0	37355.0		12.54	12.30	828.59		
81	7	23	23	5.0	37835.0		13.13	12.89	828.00		
81	7	24	7	5.0	38315.0		13.73	13.49	827.40		
81	7	24	15	5.0	38795.0		14.24	14.00	826.89		
81	7	24	23	5.0	39275.0		14.77	14.53	826.36		
81	7	25	7	5.0	39755.0		15.29	15.05	825.84		
81	7	25	15	5.0	40235.0		15.76	15.52	825.37		
81	7	25	23	5.0	40715.0		16.29	16.05	824.84		
81	7	26	7	5.0	41195.0		16.77	16.53	824.36		
81	7	26	15	5.0	41675.0		17.14	16.90	823.99		
81	7	26	23	5.0	42155.0		17.60	17.36	823.53		
81	7	27	7	5.0	42635.0		18.03	17.79	823.10		
81	7	27	15	0.0	43110.0		18.38	18.14	822.75		
81	7	27	23	0.0	43590.0		18.78	18.54	822.35		
81	7	28	7	0.0	44070.0		19.15	18.91	821.98		
81	7	28	15	0.0	44550.0		19.50	19.26	821.63		
81	7	28	19	50.0	44840.0		19.71	19.47	821.42		
81	7	28	20	10.0	44860.0		19.71	19.47	821.42		STOPPED PUMPING PW1
81	7	28	20	10.5	44860.5		19.14	18.90	821.99		
81	7	28	20	11.0	44861.0		18.74	18.50	822.39		
81	7	28	20	11.5	44861.5		18.59	18.35	822.54		

DATE YR	DATE MON	DATE DAY	TIME HR	TIME MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	28	20	12.0	44862.0		18.50	18.26	822.63		
81	7	28	20	12.5	44862.5		18.44	18.20	822.69		
81	7	28	20	13.0	44863.0		18.39	18.15	822.74		
81	7	28	20	14.0	44864.0		18.32	18.08	822.81		
81	7	28	20	15.0	44865.0		18.27	18.03	822.86		
81	7	28	20	16.0	44866.0		18.23	17.99	822.90		
81	7	28	20	18.0	44868.0		18.18	17.94	822.95		
81	7	28	20	20.0	44870.0		18.13	17.89	823.00		
81	7	28	20	25.0	44875.0		18.06	17.82	823.07		
81	7	28	20	30.0	44880.0		18.01	17.77	823.12		
81	7	28	20	35.0	44885.0		17.97	17.73	823.16		
81	7	28	20	40.0	44890.0		17.93	17.69	823.20		
81	7	28	20	50.0	44900.0		17.86	17.62	823.27		
81	7	28	21	0.0	44910.0		17.81	17.57	823.32		
81	7	28	21	10.0	44920.0		17.75	17.51	823.38		
81	7	28	21	30.0	44940.0		17.67	17.43	823.46		
81	7	28	21	50.0	44960.0		17.57	17.33	823.56		
81	7	28	22	40.0	45010.0		17.39	17.15	823.74		
81	7	28	23	30.0	45060.0		17.19	16.95	823.94		
81	7	29	0	20.0	45110.0		17.03	16.79	824.10		
81	7	29	7	10.0	45520.0		16.87	16.63	824.26		
81	7	29	2	50.0	45260.0		16.59	16.35	824.54		
81	7	29	4	30.0	45360.0		16.35	16.11	824.78		
81	7	29	6	10.0	45460.0		16.08	15.84	825.05		
81	7	29	9	30.0	45660.0		15.66	15.42	825.47		
81	7	29	14	30.0	45960.0		15.10	14.86	826.03		
81	7	29	22	10.0	46420.0		14.37	14.13	826.76		
81	7	30	8	20.0	47030.0		13.54	13.30	827.59		
81	7	31	9	0.0	48510.0		11.92	11.68	829.21		
81	7	31	16	50.0	48980.0		11.48	11.24	829.65		
81	8	1	9	30.0	49980.0		10.67	10.43	830.46		
81	8	1	16	0.0	50370.0		10.37	10.13	830.76		
81	8	2	11	20.0	51530.0		9.57	9.33	831.56		
81	8	2	16	0.0	51810.0		9.38	9.14	831.75		
81	8	3	9	25.0	52855.0		8.78	8.54	832.35		
81	8	4	11	0.0	54390.0		7.99	7.75	833.14		
81	8	7	10	5.0	58655.0		6.29	6.05	834.84		
81	8	8	9	15.0	60045.0		5.85	5.61	835.28		
81	8	9	9	40.0	61510.0		5.40	5.16	835.73		
81	8	10	9	45.0	62955.0		5.01	4.77	836.12		

RESIDUAL DRAWDOWN

OBSERVATION WELL - OW4,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (TI)	RATIO (T/TI)	DRAWDOWN (S)
1500.2	.2	9376.00	2.83
1500.4	.4	3948.37	2.57
1500.5	.5	3001.00	2.43
1500.5	.5	2831.19	2.33
1500.6	.6	2460.02	2.23
1525.0	25.0	61.00	1.17
1527.0	27.0	56.56	1.13
1530.0	30.0	51.00	1.11
1540.0	40.0	38.50	1.07
1550.0	50.0	31.00	1.05
1563.0	63.0	24.81	.97
2330.0	830.0	2.81	.64
3170.0	1670.0	1.90	.43
3775.0	2275.0	1.66	.34
5250.0	3750.0	1.40	.24
6690.0	5190.0	1.29	.14
8130.0	6630.0	1.23	.12
12840.0	11340.0	1.13	.01
13980.0	12480.0	1.12	0.00
15300.0	13800.0	1.11	.47
15960.0	14460.0	1.10	5.67
16770.0	15270.0	1.10	1.87
22590.0	21090.0	1.07	1.40
25560.0	24060.0	1.06	1.46
32625.0	31125.0	1.05	1.40
32970.0	31470.0	1.05	1.50
32970.5	31470.5	1.05	2.13
32971.0	31471.0	1.05	2.45
32971.3	31471.3	1.05	2.52
32971.5	31471.5	1.05	2.57
32971.8	31471.8	1.05	2.63
32972.0	31472.0	1.05	2.66
32972.3	31472.3	1.05	2.71
32972.5	31472.5	1.05	2.73
32972.8	31472.8	1.05	2.76
32973.0	31473.0	1.05	2.77
32973.3	31473.3	1.05	2.79
32973.5	31473.5	1.05	2.81
32973.8	31473.8	1.05	2.82
32974.0	31474.0	1.05	2.84
32974.3	31474.3	1.05	2.85
32974.5	31474.5	1.05	2.86
32974.8	31474.8	1.05	2.87
32975.0	31475.0	1.05	2.88
32975.5	31475.5	1.05	2.90
32976.0	31476.0	1.05	2.92
32977.0	31477.0	1.05	2.95
32978.0	31478.0	1.05	2.98
32980.0	31480.0	1.05	3.02
32985.0	31485.0	1.05	3.09
32990.0	31490.0	1.05	3.13
32995.0	31495.0	1.05	3.18
33000.0	31500.0	1.05	3.21
33010.0	31510.0	1.05	3.27
33020.0	31520.0	1.05	3.33
33030.0	31530.0	1.05	3.39
33050.0	31550.0	1.05	3.47
33070.0	31570.0	1.05	3.59
33120.0	31620.0	1.05	3.79
33170.0	31670.0	1.05	3.98
33220.0	31720.0	1.05	4.20
33270.0	31770.0	1.05	4.42
33370.0	31870.0	1.05	4.76
33470.0	31970.0	1.05	5.11
33580.0	32080.0	1.05	5.42
33820.0	32320.0	1.05	6.08
33970.0	32470.0	1.05	6.46
34475.0	32975.0	1.05	6.61
34975.0	33475.0	1.04	8.57
35475.0	33975.0	1.04	9.51
35915.0	34415.0	1.04	10.19
36395.0	34895.0	1.04	11.00
36875.0	35375.0	1.04	11.73
37355.0	35855.0	1.04	12.30
37835.0	36335.0	1.04	12.89
38315.0	36815.0	1.04	13.49
38795.0	37295.0	1.04	14.00
39275.0	37775.0	1.04	14.53
39755.0	38255.0	1.04	15.05
40235.0	38735.0	1.04	15.52
40715.0	39215.0	1.04	16.05
41195.0	39695.0	1.04	16.53
41675.0	40175.0	1.04	16.90
42155.0	40655.0	1.04	17.36
42635.0	41135.0	1.04	17.79
43110.0	41610.0	1.04	18.14
43590.0	42090.0	1.04	18.54
44070.0	42570.0	1.04	18.91
44550.0	43050.0	1.03	19.26
44840.0	43340.0	1.03	19.47
44860.0	43360.0	1.03	19.47
44860.5	43360.5	1.03	18.90
44861.0	43361.0	1.03	18.50
44861.5	43361.5	1.03	18.35
44862.0	43362.0	1.03	18.26
44862.5	43362.5	1.03	18.20
44863.0	43363.0	1.03	18.15
44864.0	43364.0	1.03	18.08
44865.0	43365.0	1.03	18.03
44866.0	43366.0	1.03	17.99
44868.0	43368.0	1.03	17.94
44870.0	43370.0	1.03	17.89
44875.0	43375.0	1.03	17.82
44880.0	43380.0	1.03	17.77
44885.0	43385.0	1.03	17.73
44890.0	43390.0	1.03	17.69
44900.0	43400.0	1.03	17.62
44910.0	43410.0	1.03	17.57
44920.0	43420.0	1.03	17.51
44940.0	43440.0	1.03	17.43
44960.0	43460.0	1.03	17.33
45010.0	43510.0	1.03	17.15

45060.0	43560.0	1.03	16.95
45110.0	43610.0	1.03	16.79
45520.0	44020.0	1.03	16.63
45260.0	43760.0	1.03	16.35
45360.0	43860.0	1.03	16.11
45460.0	43960.0	1.03	15.84
45660.0	44160.0	1.03	15.42
45960.0	44460.0	1.03	14.86
46420.0	44920.0	1.03	14.13
47030.0	45530.0	1.03	13.30
48510.0	47010.0	1.03	11.68
48980.0	47480.0	1.03	11.24
49980.0	48480.0	1.03	10.43
50370.0	48870.0	1.03	10.13
51530.0	50030.0	1.03	9.33
51810.0	50310.0	1.03	9.14
52855.0	51355.0	1.03	8.54
54390.0	52890.0	1.03	7.75
58655.0	57155.0	1.03	6.05
60045.0	58545.0	1.03	5.61
61510.0	60010.0	1.02	5.16
62955.0	61455.0	1.02	4.77

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GOLDER ASSOCIATES

PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER =     OW5,

12/11/81-11.30.54
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    PUMPED WELL NUMBER - OW5,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK BC,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 27/ 6/81-30.0/16
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 28/ 6/81-30.0/17

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                          820.94 METRES
    DATUM POINT -                                      TOP OF 19MM PVC PIPE,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -           .61 METRES
    DEPTH TO STATIC WATER LEVEL -                 5.04 METRES
    ELEVATION OF STATIC WATER LEVEL -           816.51 METRES
    TYPE OF OBSERVATION WELL -                         STANDPIPE PIEZOMETER
    DEPTH OF GRAVEL PACK INTERVAL -              13.94 TO     19.22 METRES
    DISTANCE FROM PUMPING WELL -                326.40 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                             152.00   MM
    PUMP TYPE -                                        SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                  TRIDENT DIGITAL,
    PUMPING RATE -                          7.167E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                               CONFINED
    AQUIFER DESCRIPTION -                              SANDY GRAVEL,
    AQUIFER THICKNESS -                    UNDEFINED  METRES

TEST DETAILS
    WEATHER CONDITIONS - VARIABLE,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - NONE,
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* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = OW5, ** 12/11/81-11.30.54 **

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		821.55		
0	0	0	0	0.0			0.00		821.55		
81	7	16	11	15.0	27045.0		5.20	0.16	816.35		
81	7	20	8	15.0	32625.0		5.04	0.	816.51		
81	7	24	8	40.0	38410.0		5.04	0.	816.51		
81	7	24	10	50.0	38540.0		5.04	0.	816.51		START PUMP IN PW2
81	7	24	11	14.0	38564.0		5.04	0.	816.51		
81	7	24	11	30.0	38580.0		5.05	0.01	816.50		
81	7	24	12	40.0	38650.0		5.05	0.01	816.50		
81	7	24	13	30.0	38700.0		5.05	0.01	816.50		
81	7	24	19	45.0	39075.0		5.05	0.01	816.50		
81	7	25	9	30.0	39900.0		5.06	0.02	816.49		
81	7	27	11	30.0	42900.0		5.09	0.05	816.46		
81	7	28	8	50.0	44180.0		5.10	0.06	816.45		
81	7	28	9	50.0	44240.0		5.11	0.07	816.44		
81	7	29	14	20.0	45950.0		5.12	0.08	816.43		
81	7	30	9	0.0	47070.0		5.12	0.08	816.43		
81	7	31	8	30.0	48480.0		5.13	0.09	816.42		
81	8	1	9	50.0	50000.0		5.12	0.08	816.43		
81	8	2	11	45.0	51555.0		5.12	0.08	816.43		
81	8	3	9	50.0	52880.0		5.12	0.08	816.43		
81	8	5	9	20.0	55730.0		5.13	0.09	816.42		
81	8	7	9	30.0	58620.0		5.13	0.09	816.42		
81	8	8	8	50.0	60020.0		5.13	0.09	816.42		
81	8	9	9	0.0	61470.0		5.13	0.09	816.42		
81	8	10	8	55.0	62905.0		5.14	0.10	816.41		

RESIDUAL DRAWDOWN

OBSERVATION WELL - OW5,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (TI)	RATIO (T/TI)	DRAWDOWN (S)
27045.0	25545.0	1.06	.16
32625.0	31125.0	1.05	0.00
38410.0	36910.0	1.04	0.00
38540.0	37040.0	1.04	0.00
38564.0	37064.0	1.04	0.00
38580.0	37080.0	1.04	.01
38650.0	37150.0	1.04	.01
38700.0	37200.0	1.04	.01
39075.0	37575.0	1.04	.01
39900.0	38400.0	1.04	.02
42900.0	41400.0	1.04	.05
44180.0	42680.0	1.04	.06
44240.0	42740.0	1.04	.07
45950.0	44450.0	1.03	.08
47070.0	45570.0	1.03	.08
48480.0	46980.0	1.03	.09
50000.0	48500.0	1.03	.08
51555.0	50055.0	1.03	.08
52880.0	51380.0	1.03	.08
55730.0	54230.0	1.03	.09
58620.0	57120.0	1.03	.09
60020.0	58520.0	1.03	.09
61470.0	59970.0	1.03	.09
62905.0	61405.0	1.02	.10

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*                               GOLDER ASSOCIATES                               *
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*                                                                               *
*                                                                               *
*         PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER =      PW1,             *
*                                                                               *
*                                                                               *
*                              12/11/81-11.00.23                                *
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    PUMPED WELL NUMBER - PW1,
    CLIENT             - B.C.HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HATCREEK,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 20/ 7/81- 0.0/14
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 28/ 7/81-10.0/20

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                           838.34 METRES
    DATUM POINT -                                       TOP OF 200MM CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -            .45 METRES
    DEPTH TO STATIC WATER LEVEL -                 -2.10 METRES
    ELEVATION OF STATIC WATER LEVEL -            840.89 METRES
    TYPE OF OBSERVATION WELL -                          SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                 100.28 TO    109.91 METRES
    DISTANCE FROM PUMPING WELL -                   0.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                              203.00   MM
    PUMP TYPE -                                         SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                   TRIDENT DIGITAL,
    PUMPING RATE -                           2.656E+01 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                                CONFINED
    AQUIFER DESCRIPTION -                               VARIABLE SANDY GRAVEL,
    AQUIFER THICKNESS -                     UNDEFINED  METRES

TEST DETAILS
    WEATHER CONDITIONS -
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - WELL FLOWING BEFORE PUMPING,
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DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		838.79		
0	0	0	0	0.0			0.00		838.79		
81	7	20	14	0.0			-.10	2.00	838.89		FLOWING OVER CASING
81	7	20	14	.5	0.5		10.04	12.14	828.75		INITIAL METER READING 6847045
81	7	20	14	1.0	1.0		10.60	12.70	828.19		PUMP SET AT 75.14M BELOW GROUND
81	7	20	14	1.5	1.5		10.77	12.87	828.02		
81	7	20	14	2.0	2.0		10.78	12.88	828.01		
81	7	20	14	2.5	2.5		10.81	12.91	827.98		
81	7	20	14	3.0	3.0		10.82	12.92	827.97		
81	7	20	14	4.0	4.0		10.87	12.97	827.92		
81	7	20	14	5.0	5.0		10.93	13.03	827.86		
81	7	20	14	6.0	6.0		10.95	13.05	827.84		
81	7	20	14	8.0	8.0		10.98	13.08	827.81		
81	7	20	14	10.0	10.0		11.01	13.11	827.78		
81	7	20	14	15.0	15.0		11.07	13.17	827.72		
81	7	20	14	20.0	20.0		11.09	13.19	827.70		
81	7	20	14	25.0	25.0		11.11	13.21	827.68		
81	7	20	14	30.0	30.0		11.15	13.25	827.64		
81	7	20	14	40.0	40.0		11.20	13.30	827.59		
81	7	20	14	50.0	50.0		11.24	13.34	827.55		
81	7	20	15	0.0	60.0		11.28	13.38	827.51		
81	7	20	15	20.0	80.0		11.39	13.49	827.40		
81	7	20	15	40.0	100.0		11.57	13.67	827.22		
81	7	20	16	30.0	150.0		11.68	13.78	827.11		
81	7	20	17	20.0	200.0		11.84	13.94	826.95		
81	7	20	18	10.0	250.0		12.27	14.37	826.52		
81	7	20	19	0.0	300.0		12.34	14.44	826.45		INCREASED PUMP RATE
81	7	20	20	40.0	400.0		13.13	15.23	825.66		
81	7	20	22	20.0	500.0		13.53	15.63	825.26		
81	7	21	0	7.0	607.0		13.82	15.92	824.97		
81	7	21	4	10.0	850.0		14.49	16.59	824.30		
81	7	21	6	40.0	1000.0		14.91	17.01	823.88		
81	7	21	8	0.0	1080.0		15.26	17.36	823.53		
81	7	21	11	15.0	1275.0		15.60	17.70	823.19		
81	7	21	15	0.0	1500.0		16.09	18.19	822.70	26.57	
81	7	21	23	20.0	2000.0		17.07	19.17	821.72	26.60	
81	7	22	7	40.0	2500.0		17.93	20.03	820.86	26.67	
81	7	22	15	0.0	2940.0		18.36	20.46	820.43	26.90	
81	7	22	23	0.0	3420.0		19.54	21.64	819.25	26.53	
81	7	23	7	0.0	3900.0		20.24	22.34	818.55	27.31	
81	7	23	15	0.0	4380.0		20.51	22.61	818.28	26.64	
81	7	23	23	0.0	4860.0		21.10	23.20	817.69	26.46	
81	7	24	7	0.0	5340.0		21.74	23.84	817.05	26.66	
81	7	24	15	0.0	5820.0		22.19	24.29	816.60	26.49	
81	7	24	23	0.0	6300.0		22.78	24.88	816.01	26.41	

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	25	7	0.0	6780.0		23.27	25.37	815.52	26.46	
81	7	25	15	0.0	7260.0		23.64	25.74	815.15	26.30	
81	7	25	23	0.0	7740.0		24.56	26.66	814.23	26.68	
81	7	26	7	0.0	8220.0		25.00	27.10	813.79	26.81	
81	7	26	15	0.0	8700.0		25.99	28.09	812.80	26.43	
81	7	26	23	0.0	9180.0		25.82	27.92	812.97	26.44	
81	7	27	7	0.0	9660.0		26.24	28.34	812.55	26.64	
81	7	27	15	0.0	10140.0		26.47	28.57	812.32	26.42	
81	7	27	23	0.0	10620.0		26.93	29.03	811.86	26.42	
81	7	28	7	0.0	11100.0		27.28	29.38	811.51	26.37	
81	7	28	15	0.0	11580.0		27.63	29.73	811.16	26.34	
81	7	28	20	10.0	11890.0		27.92	30.02	810.87		METER READING 11015800
81	7	28	20	10.3	11890.3		20.15	22.25	818.64		WATER SAMPLED T%12C PH%77 EC%360
81	7	28	20	10.5	11890.5		17.32	19.42	821.47		
81	7	28	20	10.8	11890.8		16.70	18.80	822.09		
81	7	28	20	11.0	11891.0		16.51	18.61	822.28		
81	7	28	20	11.5	11891.5		16.45	18.55	822.34		
81	7	28	20	12.0	11892.0		16.39	18.49	822.40		
81	7	28	20	12.5	11892.5		16.35	18.45	822.44		
81	7	28	20	13.0	11893.0		16.32	18.42	822.47		
81	7	28	20	14.0	11894.0		16.27	18.37	822.52		
81	7	28	20	15.0	11895.0		16.23	18.33	822.56		
81	7	28	20	16.0	11896.0		16.20	18.30	822.59		
81	7	28	20	18.0	11898.0		16.16	18.26	822.63		
81	7	28	20	20.0	11900.0		16.12	18.22	822.67		
81	7	28	20	25.0	11905.0		16.06	18.16	822.73		
81	7	28	20	30.0	11910.0		16.02	18.12	822.77		
81	7	28	20	35.0	11915.0		15.98	18.08	822.81		
81	7	28	20	40.0	11920.0		15.94	18.04	822.85		
81	7	28	20	50.0	11930.0		15.88	17.98	822.91		
81	7	28	21	0.0	11940.0		15.83	17.93	822.96		
81	7	28	21	10.0	11950.0		15.77	17.87	823.02		
81	7	28	21	30.0	11970.0		15.67	17.77	823.12		
81	7	28	21	50.0	11990.0		15.57	17.67	823.22		
81	7	28	22	40.0	12040.0		15.40	17.50	823.39		
81	7	28	23	30.0	12090.0		15.22	17.32	823.57		
81	7	29	0	20.0	12140.0		15.05	17.15	823.74		
81	7	29	1	10.0	12190.0		14.90	17.00	823.89		
81	7	29	2	50.0	12290.0		14.61	16.71	824.18		
81	7	29	4	30.0	12390.0		14.38	16.48	824.41		
81	7	29	6	10.0	12490.0		14.12	16.22	824.67		
81	7	29	9	30.0	12690.0		13.70	15.80	825.09		
81	7	29	14	30.0	12990.0		13.67	15.77	825.12		
81	7	29	22	10.0	13450.0		12.96	15.06	825.83		
81	7	30	8	20.0	14060.0		12.12	14.22	826.67		

PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = PW1, ** 12/11/81-11.00.23 **

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	31	9	0.0	15540.0		9.95	12.05	828.84		
81	7	31	16	50.0	16010.0		9.51	11.61	829.28		
81	8	1	9	30.0	17010.0		8.69	10.79	830.10		
81	8	1	16	0.0	17400.0		8.29	10.39	830.50		
81	8	2	11	20.0	18560.0		7.60	9.70	831.19		
81	8	2	16	0.0	18840.0		7.40	9.50	831.39		
81	8	3	9	25.0	19885.0		6.72	8.82	832.07		
81	8	4	10	55.0	21415.0		6.02	8.12	832.77		
81	8	7	10	0.0	25680.0		4.88	6.98	833.91		
81	8	8	9	15.0	27075.0		4.42	6.52	834.37		
81	8	9	9	40.0	28540.0		3.98	6.08	834.81		
81	8	10	9	40.0	29980.0		3.60	5.70	835.19		

RESIDUAL DRAWDOWN

OBSERVATION WELL - PW1,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (T1)	RATIO (T/T1)	DRAWDOWN (S)
11890.3	.3	47561.00	22.25
11890.5	.5	23781.00	19.42
11890.8	.8	15854.33	18.80
11891.0	1.0	11891.00	18.61
11891.5	1.5	7927.67	18.55
11892.0	2.0	5946.00	18.49
11892.5	2.5	4757.00	18.45
11893.0	3.0	3964.33	18.42
11894.0	4.0	2973.50	18.37
11895.0	5.0	2379.00	18.33
11896.0	6.0	1982.67	18.30
11898.0	8.0	1487.25	18.26
11900.0	10.0	1190.00	18.22
11905.0	15.0	793.67	18.16
11910.0	20.0	595.50	18.12
11915.0	25.0	476.60	18.08
11920.0	30.0	397.33	18.04
11930.0	40.0	298.25	17.98
11940.0	50.0	238.80	17.93
11950.0	60.0	199.17	17.87
11970.0	80.0	149.63	17.77
11990.0	100.0	119.90	17.67
12040.0	150.0	80.27	17.50
12090.0	200.0	60.45	17.32
12140.0	250.0	48.56	17.15
12190.0	300.0	40.63	17.00
12290.0	400.0	30.73	16.71
12390.0	500.0	24.78	16.48
12490.0	600.0	20.82	16.22
12690.0	800.0	15.86	15.80
12990.0	1100.0	11.81	15.77
13450.0	1560.0	8.62	15.06
14060.0	2170.0	6.48	14.22
15540.0	3650.0	4.26	12.05
16010.0	4120.0	3.89	11.61
17010.0	5120.0	3.32	10.79
17400.0	5510.0	3.16	10.39
18560.0	6670.0	2.78	9.70
18840.0	6950.0	2.71	9.50
19885.0	7995.0	2.49	8.82
21415.0	9525.0	2.25	8.12
25680.0	13790.0	1.86	6.98
27075.0	15185.0	1.78	6.52
28540.0	16650.0	1.71	6.08
29980.0	18090.0	1.66	5.70

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*                                                                              *
*                                                                              *
*                              GOLDER ASSOCIATES                               *
*                                                                              *
*                                                                              *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                              *
*                                                                              *
*       PUMP TEST SUMMARY FOR WELL/PIEZOMETER  NUMBER -     PW2,               *
*                                                                              *
*                                                                              *
*                              12/11/81-11.09.12                               *
*                                                                              *
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    PUMPED WELL NUMBER - PW2,
    CLIENT             - B.C. HYDRO,
    PROJECT NAME       - HAT CREEK CONSTRUCTION CAMP WATER SUPPLY,
    PROJECT NUMBER     - 8121507,
    LOCATION OF TEST   - HAT CREEK B.C.,
    TYPE OF TEST       - CONSTANT RATE
    DATE PUMP STARTED  - 24/ 7/81-50.0/10
    (DAY/MO/YR-MIN/HRS)
    DATE PUMP STOPPED  - 28/ 7/81- 0.0/ 9

DATA ON OBSERVATION WELL
    GROUND ELEVATION -                            822.26 METRES
    DATUM POINT -                                        TOP OF 203MM CASING,
    HEIGHT OF DATUM ABOVE GROUND LEVEL -             .61 METRES
    DEPTH TO STATIC WATER LEVEL -                   5.91 METRES
    ELEVATION OF STATIC WATER LEVEL -             816.96 METRES
    TYPE OF OBSERVATION WELL -                           SCREENED WELL
    DEPTH OF SCREENED INTERVAL -                   25.93 TO     29.18 METRES
    DISTANCE FROM PUMPING WELL -                    0.00 METRES

DATA ON PUMPED WELL
    WELL DIAMETER -                               203.00  MM
    PUMP TYPE -                                          SUBMERSIBLE

FLOW MEASUREMENT
    FLOWMETER, TYPE -                                    DIGITAL,
    PUMPING RATE -                             6.308E+00 LITRES/S

AQUIFER DATA
    AQUIFER CONDITIONS -                                 CONFINED
    AQUIFER DESCRIPTION -                                SANDY COARSE GRAVEL,
    AQUIFER THICKNESS -                               0. METRES

TEST DETAILS
    WEATHER CONDITIONS - SUNNY, HOT, 30C,
    TESTED BY          - GOLDER ASSOCIATES,
    COMMENTS           - NONE,
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DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
0	0	0	0	0.0			0.00		822.87		
0	0	0	0	0.0			0.00		822.87		
81	7	16	11	15.0			5.89		816.98		
81	7	20	8	15.0			5.90		816.97		
81	7	21	8	20.0			5.90		816.97		
81	7	24	10	15.0			5.91		816.96		
81	7	24	10	50.0			5.91	0.	816.96		START PUMP
81	7	24	10	50.3	0.3		7.08	1.17	815.79	6.31	METER READING 4561200
81	7	24	10	50.5	0.5		8.95	3.04	813.92		
81	7	24	10	50.8	0.8		9.79	3.88	813.08		
81	7	24	10	51.0	1.0		10.22	4.31	812.65	6.31	
81	7	24	10	51.5	1.5		10.84	4.93	812.03		
81	7	24	10	52.0	2.0		11.07	5.16	811.80	6.31	
81	7	24	10	52.5	2.5		11.27	5.36	811.60		
81	7	24	10	53.0	3.0		11.41	5.50	811.46	6.31	
81	7	24	10	54.0	4.0		11.58	5.67	811.29		
81	7	24	10	55.0	5.0		11.71	5.80	811.16		
81	7	24	10	56.0	6.0		11.79	5.88	811.08	6.31	METER READING 4561800
81	7	24	10	58.0	8.0		11.93	6.02	810.94		
81	7	24	11	0.0	10.0		12.01	6.10	810.86		
81	7	24	11	5.0	15.0		12.15	6.24	810.72		
81	7	24	11	10.0	20.0		12.21	6.30	810.66		
81	7	24	11	15.0	25.0		12.26	6.35	810.61		
81	7	24	11	20.0	30.0		12.29	6.38	810.58		
81	7	24	11	30.0	40.0		12.34	6.43	810.53	6.31	METER READING 4564000
81	7	24	11	40.0	50.0		12.36	6.45	810.51		
81	7	24	11	50.0	60.0		12.39	6.48	810.48	6.31	METER READING 4566900
81	7	24	12	10.0	80.0		12.57	6.66	810.30		
81	7	24	12	30.0	100.0		12.59	6.68	810.28		
81	7	24	13	20.0	150.0		12.63	6.72	810.24		METER READING 4575900
81	7	24	14	10.0	200.0		12.63	6.72	810.24	6.18	METER READING 4580800
81	7	24	15	0.0	250.0		12.65	6.74	810.22	6.37	METER READING 4585850
81	7	24	15	50.0	300.0		12.65	6.74	810.22		METER READING 4590850
81	7	24	17	30.0	400.0		12.67	6.76	810.20	6.21	METER READING 4600700
81	7	24	19	10.0	500.0		12.68	6.77	810.19	6.26	METER READING 4610620
81	7	24	20	50.0	600.0		12.69	6.78	810.18	6.26	METER READING 4620540
81	7	25	0	10.0	800.0		12.71	6.80	810.16	6.25	METER READING 4640340
81	7	25	3	4.0	974.0		12.74	6.83	810.13		METER READING 4657500
81	7	25	6	50.0	1200.0		12.75	6.84	810.12	6.25	METER READING 4679950
81	7	25	14	50.0	1680.0		12.95	7.04	809.92	6.38	METER READING 4728500
81	7	25	22	50.0	2160.0		12.97	7.06	809.90	6.38	METER READING 4777100
81	7	26	6	50.0	2640.0		12.96	7.05	809.91	6.36	METER READING 4825500
81	7	26	8	5.0	2715.0		12.96	7.05	809.91		METER READING 4833550
81	7	26	8	5.3	2715.3		13.59	7.68	809.28		INCREASE PUMP RATE
81	7	26	8	5.5	2715.5		13.94	8.03	808.93		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	26	8	5.8	2715.8		14.20	8.29	808.67		
81	7	26	8	6.0	2716.0		14.24	8.33	808.63	7.57	
81	7	26	8	6.5	2716.5		14.44	8.53	808.43		
81	7	26	8	7.0	2717.0		14.55	8.64	808.32	7.57	
81	7	26	8	7.5	2717.5		14.60	8.69	808.27		
81	7	26	8	8.0	2718.0		14.60	8.69	808.27	7.57	
81	7	26	8	9.0	2719.0		14.62	8.71	808.25		
81	7	26	8	10.0	2720.0		14.66	8.75	808.21		METER READING 4833780
81	7	26	8	11.0	2721.0		14.68	8.77	808.19		
81	7	26	8	13.0	2723.0		14.71	8.80	808.16		
81	7	26	8	15.0	2725.0		14.67	8.76	808.20		METER READING 4834300
81	7	26	8	20.0	2730.0		14.72	8.81	808.15	7.57	METER READING 4834900
81	7	26	8	25.0	2735.0		14.72	8.81	808.15		METER READING 4835520
81	7	26	8	25.3	2735.3		15.40	9.49	807.47		INCREASE PUMP RATE
81	7	26	8	25.5	2735.5		15.90	9.99	806.97		
81	7	26	8	25.8	2735.8		16.12	10.21	806.75		
81	7	26	8	26.0	2736.0		16.42	10.51	806.45	9.46	
81	7	26	8	26.5	2736.5		16.66	10.75	806.21		
81	7	26	8	27.0	2737.0		16.82	10.91	806.05	9.46	
81	7	26	8	27.5	2737.5		16.83	10.92	806.04		
81	7	26	8	28.0	2738.0		16.90	10.99	805.97		
81	7	26	8	29.0	2739.0		16.97	11.06	805.90		
81	7	26	8	30.0	2740.0		17.01	11.10	805.86	9.84	METER READING 4836300
81	7	26	8	31.0	2741.0		17.06	11.15	805.81		
81	7	26	8	33.0	2743.0		17.11	11.20	805.76		
81	7	26	8	35.0	2745.0		17.10	11.19	805.77	8.83	METER READING 4837000
81	7	26	8	40.0	2750.0		17.24	11.33	805.63		
81	7	26	8	45.0	2755.0		17.24	11.33	805.63	9.59	
81	7	26	8	50.0	2760.0		17.30	11.39	805.57		
81	7	26	8	55.0	2765.0		17.36	11.45	805.51	9.46	
81	7	26	9	5.0	2775.0		17.40	11.49	805.47		
81	7	26	9	15.0	2785.0		17.45	11.54	805.42	9.43	
81	7	26	9	25.0	2795.0		17.52	11.61	805.35		
81	7	26	9	45.0	2815.0		17.53	11.62	805.34	9.44	
81	7	26	10	5.0	2835.0		17.58	11.67	805.29	9.46	
81	7	26	10	49.0	2879.0		7.38	1.47	815.49		PUMP STOPPED AT 10.47
81	7	26	10	49.5	2879.5		7.04	1.13	815.83		ELECTRICAL FAILURE
81	7	26	10	50.0	2880.0		6.95	1.04	815.92		
81	7	26	10	51.0	2881.0		6.83	0.92	816.04		
81	7	26	10	52.0	2882.0		6.73	0.82	816.14		
81	7	26	10	53.0	2883.0		6.63	0.72	816.24		PUMP RESTARTED AT 10:53
81	7	26	10	54.0	2884.0		14.30	8.39	808.57		
81	7	26	10	54.5	2884.5		14.96	9.05	807.91		
81	7	26	10	55.0	2885.0		15.44	9.53	807.43	8.52	
81	7	26	10	55.5	2885.5		15.62	9.71	807.25		

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	26	10	56.5	2886.5		15.86	9.95	807.01		
81	7	26	10	57.0	2887.0		15.94	10.03	806.93		
81	7	26	10	58.0	2888.0		16.03	10.12	806.84		
81	7	26	10	59.0	2889.0		16.09	10.18	806.78		METER READING 4857600
81	7	26	11	1.0	2891.0		16.16	10.25	806.71		
81	7	26	11	3.0	2893.0		16.28	10.37	806.59	8.83	
81	7	26	11	8.0	2898.0		16.30	10.39	806.57		
81	7	26	11	13.0	2903.0		16.33	10.42	806.54		
81	7	26	11	18.0	2908.0		16.35	10.44	806.52		
81	7	26	11	23.0	2913.0		16.36	10.45	806.51		
81	7	26	11	30.0	2920.0		16.37	10.46	806.50	8.54	METER READING 4861800
81	7	26	11	40.0	2930.0		16.41	10.50	806.46		
81	7	26	11	50.0	2940.0		16.42	10.51	806.45		
81	7	26	12	10.0	2960.0		16.43	10.52	806.44	8.58	METER READING 4867250
81	7	26	12	40.0	2990.0		16.44	10.53	806.43		METER READING 4871350
81	7	26	13	30.0	3040.0		16.49	10.58	806.38	8.66	
81	7	26	14	20.0	3090.0		16.54	10.63	806.33	8.64	
81	7	26	15	10.0	3140.0		16.55	10.64	806.32	8.63	
81	7	26	16	0.0	3190.0		16.59	10.68	806.28	8.64	
81	7	26	17	40.0	3290.0		16.61	10.70	806.26	8.62	
81	7	26	19	20.0	3390.0		16.62	10.71	806.25	8.64	
81	7	26	21	0.0	3490.0		16.63	10.72	806.24	8.61	
81	7	26	24	20.0	3690.0		16.76	10.85	806.11	8.69	
81	7	27	7	20.0	4110.0		16.90	10.99	805.97	8.61	METER READING 5024380
81	7	27	15	10.0	4580.0		16.96	11.05	805.91	8.64	
81	7	27	22	50.0	5040.0		17.02	11.11	805.85	8.66	
81	7	28	6	50.0	5520.0		16.98	11.07	805.89	8.61	
81	7	28	9	0.0	5650.0		16.98	11.07	805.89		STOPPED PUMP METER READING 523520
81	7	28	9	.3	5650.3		13.25	7.34	809.62		
81	7	28	9	.5	5650.5		10.98	5.07	811.89		
81	7	28	9	.8	5650.8		9.20	3.29	813.67		
81	7	28	9	1.0	5651.0		8.45	2.54	814.42		
81	7	28	9	1.5	5651.5		7.75	1.84	815.12		
81	7	28	9	2.0	5652.0		7.45	1.54	815.42		
81	7	28	9	2.5	5652.5		7.26	1.35	815.61		
81	7	28	9	3.0	5653.0		7.13	1.22	815.74		
81	7	28	9	4.0	5654.0		6.98	1.07	815.89		
81	7	28	9	5.0	5655.0		6.87	0.96	816.00		
81	7	28	9	6.0	5656.0		6.78	0.87	816.09		
81	7	28	9	8.0	5658.0		6.66	0.75	816.21		
81	7	28	9	10.0	5660.0		6.58	0.67	816.29		
81	7	28	9	15.0	5665.0		6.46	0.55	816.41		
81	7	28	9	20.0	5670.0		6.39	0.48	816.48		
81	7	28	9	25.0	5675.0		6.35	0.44	816.52		
81	7	28	9	30.0	5680.0		6.32	0.41	816.55		

* PUMP TEST SUMMARY FOR WELL/PIEZOMETER NUMBER = PW2, ** 12/11/81-11.09.12 **

DATE YR	MON	DAY	TIME HR	MIN	ELAPSED TIME MINUTES	PRESSURE READING PSI	DEPTH TO WATER METRES	DRAWDOWN METRES	WATER ELEVATION METRES	DISCHARGE RATE LITRES/S	COMMENTS
81	7	28	9	40.0	5690.0		6.28	0.37	816.59		
81	7	28	9	50.0	5700.0		6.26	0.35	816.61		
81	7	28	10	0.0	5710.0		6.25	0.34	816.62		
81	7	28	10	20.0	5730.0		6.23	0.32	816.64		
81	7	28	10	40.0	5750.0		6.22	0.31	816.65		
81	7	28	11	30.0	5800.0		6.20	0.29	816.67		
81	7	28	12	20.0	5850.0		6.19	0.28	816.68		
81	7	28	13	10.0	5900.0		6.17	0.26	816.70		
81	7	28	14	0.0	5950.0		6.16	0.25	816.71		
81	7	28	15	40.0	6050.0		6.16	0.25	816.71		
81	7	28	17	15.0	6145.0		6.15	0.24	816.72		
81	7	29	6	15.0	6925.0		6.12	0.21	816.75		
81	7	29	14	45.0	7435.0		6.11	0.20	816.76		
81	7	29	22	0.0	7870.0		6.11	0.20	816.76		
81	7	30	9	0.0	8530.0		6.10	0.19	816.77		
81	7	31	8	50.0	9960.0		6.09	0.18	816.78		
81	8	1	9	40.0	11450.0		6.08	0.17	816.79		
81	8	2	11	30.0	13000.0		6.07	0.16	816.80		
81	8	3	9	35.0	14325.0		6.07	0.16	816.80		
81	8	4	11	5.0	15855.0		6.07	0.16	816.80		
81	8	5	9	35.0	17205.0		6.09	0.18	816.78		
81	8	7	9	0.0	20050.0		6.09	0.18	816.78		
81	8	8	9	5.0	21495.0		6.08	0.17	816.79		
81	8	9	9	15.0	22945.0		6.13	0.22	816.74		
81	8	10	9	10.0	24380.0		6.08	0.17	816.79		

RESIDUAL DRAWDOWN

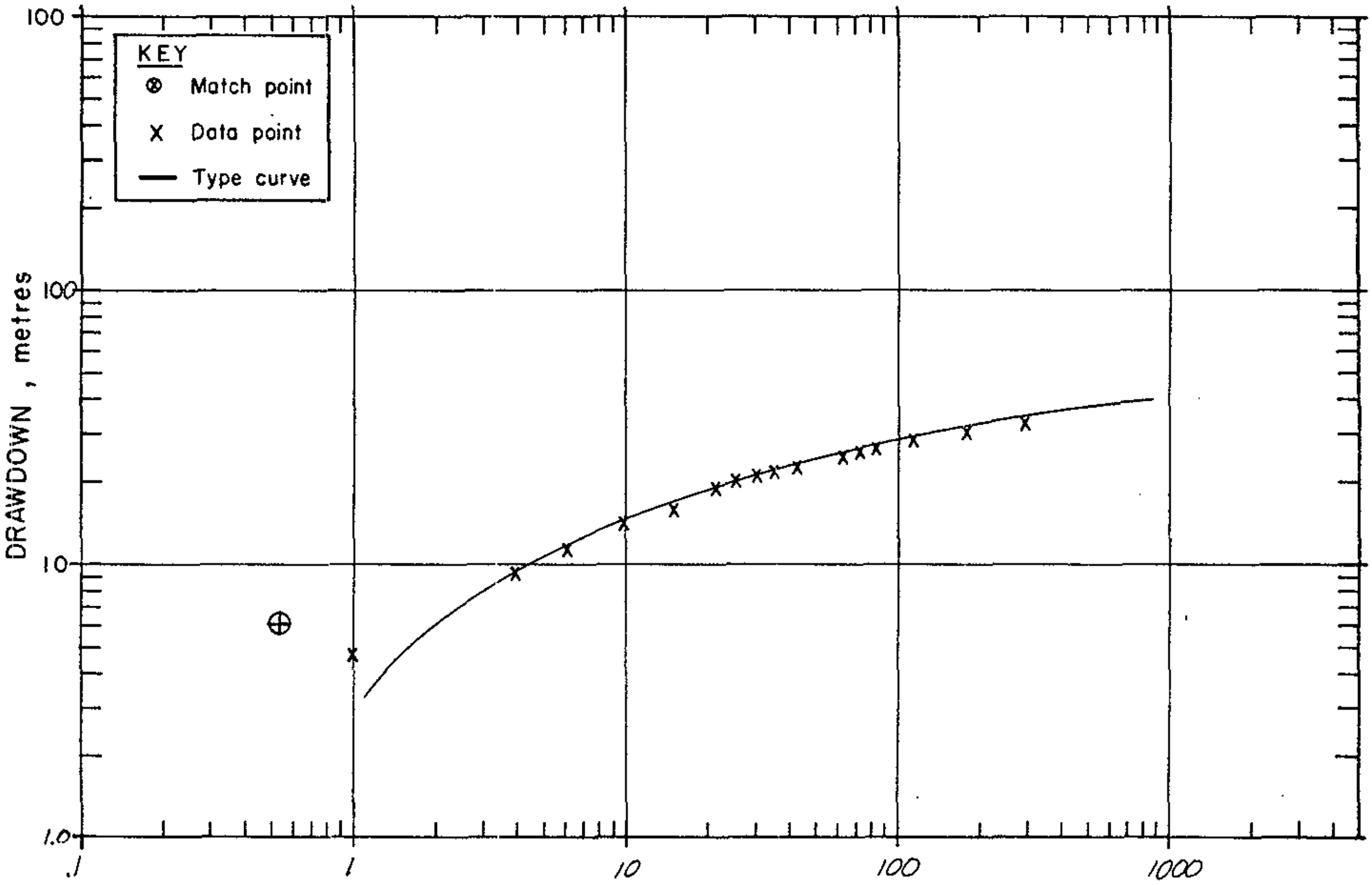
BSERVATION WELL - PW2,

ELAPSED TIME (T)	TIME SINCE PUMP STOPPED (TI)	RATIO (T/TI)	DRAWDOWN (S)
5650.3	.3	22601.00	7.34
5650.5	.5	11301.00	5.07
5650.8	.8	7534.33	3.29
5651.0	1.0	5651.00	2.54
5651.5	1.5	3767.67	1.84
5652.0	2.0	2826.00	1.54
5652.5	2.5	2261.00	1.35
5653.0	3.0	1884.33	1.22
5654.0	4.0	1413.50	1.07
5655.0	5.0	1131.00	.96
5656.0	6.0	942.67	.87
5658.0	8.0	707.25	.75
5660.0	10.0	566.00	.67
5665.0	15.0	377.67	.55
5670.0	20.0	283.50	.48
5675.0	25.0	227.00	.44
5680.0	30.0	189.33	.41
5690.0	40.0	142.25	.37
5700.0	50.0	114.00	.35
5710.0	60.0	95.17	.34
5730.0	80.0	71.63	.32
5750.0	100.0	57.50	.31
5800.0	150.0	38.67	.29
5850.0	200.0	29.25	.28
5900.0	250.0	23.60	.26
5950.0	300.0	19.83	.25
6050.0	400.0	15.13	.25
6145.0	495.0	12.41	.24
6925.0	1275.0	5.43	.21
7435.0	1785.0	4.17	.20
7870.0	2220.0	3.55	.20
8530.0	2880.0	2.96	.19
9960.0	4310.0	2.31	.18
11450.0	5800.0	1.97	.17
13000.0	7350.0	1.77	.16
14325.0	8675.0	1.65	.16
15855.0	10205.0	1.55	.16
17205.0	11555.0	1.49	.18
20050.0	14400.0	1.39	.18
21495.0	15845.0	1.36	.17
22945.0	17295.0	1.33	.22
24380.0	18730.0	1.30	.17

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 1 WELL No. OW1 **Figure** C.2.1

HY - 4

THEIS CURVE ANALYSIS



CALCULATIONS:

$$T = \frac{Q\ 10^{-3}\ W(u)}{4\pi s} = \frac{(1.28)\ 10^{-3}\ (1)}{12.57\ (6.1)} = 1.67 \times 10^{-5} \text{ metres}^2/\text{sec.}$$

$$S = \frac{240\ Ttu}{r^2} = \frac{240\ (\quad)(\quad)(\quad)}{(\quad)^2} = -$$

WHERE:

r = Radius from pumped well ..-.. (metres)

Q = Pumping rate ..1.28.. (litres/sec.)

T = Transmissivity (metres²/sec.)

S = Storage coefficient (fraction)

s = Drawdown ..6.1.. (metres)

t = Time since pumping started ..-.. (minutes)

W(u) ..1.. } Match point parameters from

u ..1.. } standard Theis type curve.

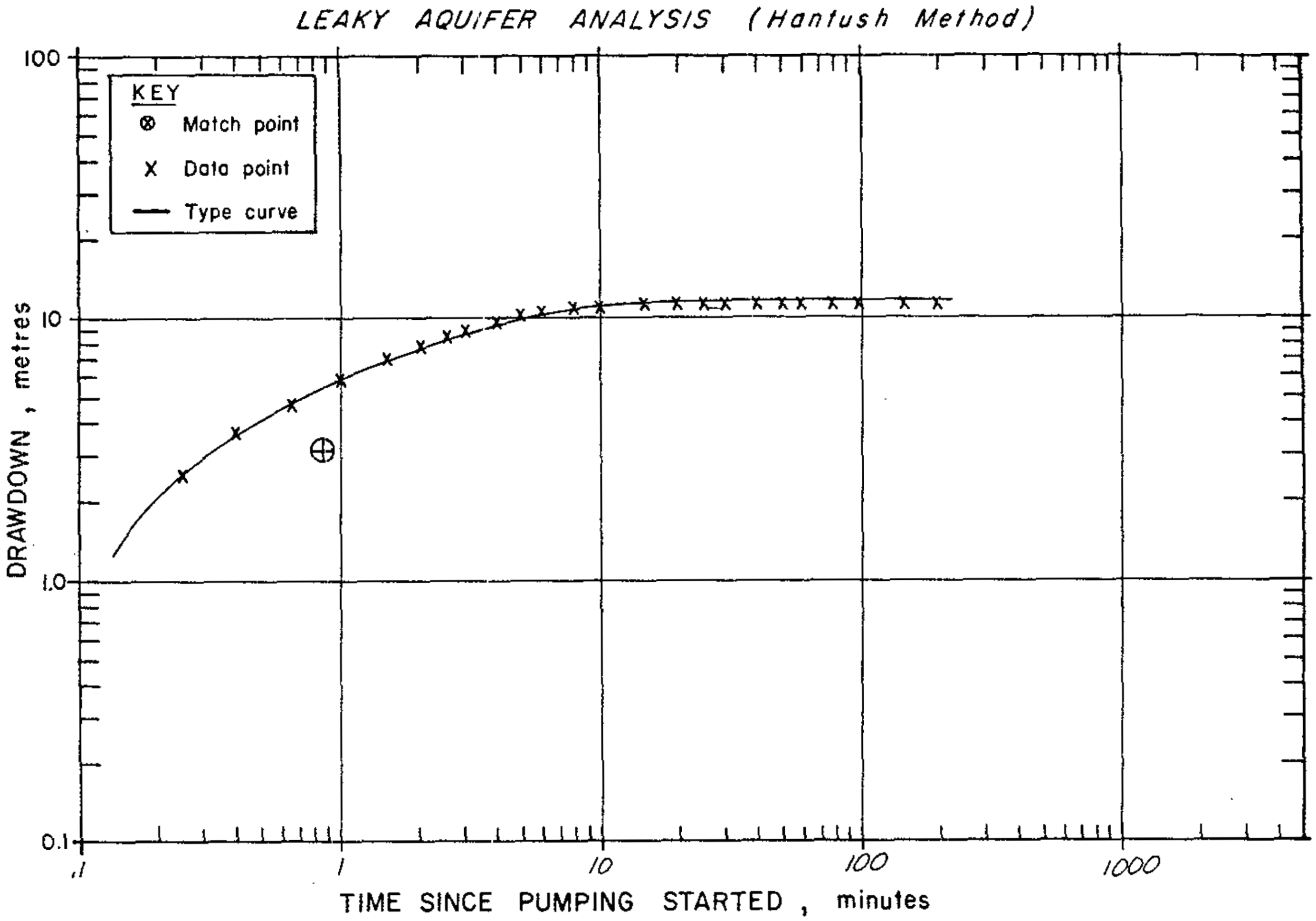
Date Jan. '82

812-1507

Golder Associates

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 2
Well No. OW2 Data observed in OW2

Figure C.2.2



CALCULATIONS:

$$T = \frac{Q\,10^{-3}\,W(u,r/B)}{4\pi s} = \frac{(2.25)\,10^{-3}\,(\;1\;)}{12.57\;(\;3.1\;)} = 5.77\times10^{-5}\ \text{metres}^2/\text{sec.}$$

$$S = \frac{240\,Ttu}{r^2} = \frac{240\,(\quad)(\quad)(\quad)}{(\quad)^2} = -$$

$$P = \frac{Tm'\,(r/B)^2}{r^2} = \frac{(\quad)(\quad)(\quad)^2}{(\quad)^2} = \quad - \quad \text{metres/sec.}$$

WHERE:

r = Radius from pumped well ... - ... (metres)
Q = Pumping rate ... 2.25 ... (litres/sec.)
m' = Average thickness of aquitard - (metres)
T = Transmissivity (metres²/sec.)
S = Storage coefficient (fraction)
P = Hydraulic conductivity of aquitard (metres/sec.)

s = Drawdown ... 3.1 ... (metres)
t = Time since pumping started ... - ... (minutes)

W(u, r/B) ... 1 ...
u ... .1 ...
r/B ... .15 ...
Match point parameters from Hantush leaky aquifer type curve

HY-7
812-1507 Drawn R.D. Date Jan. '81

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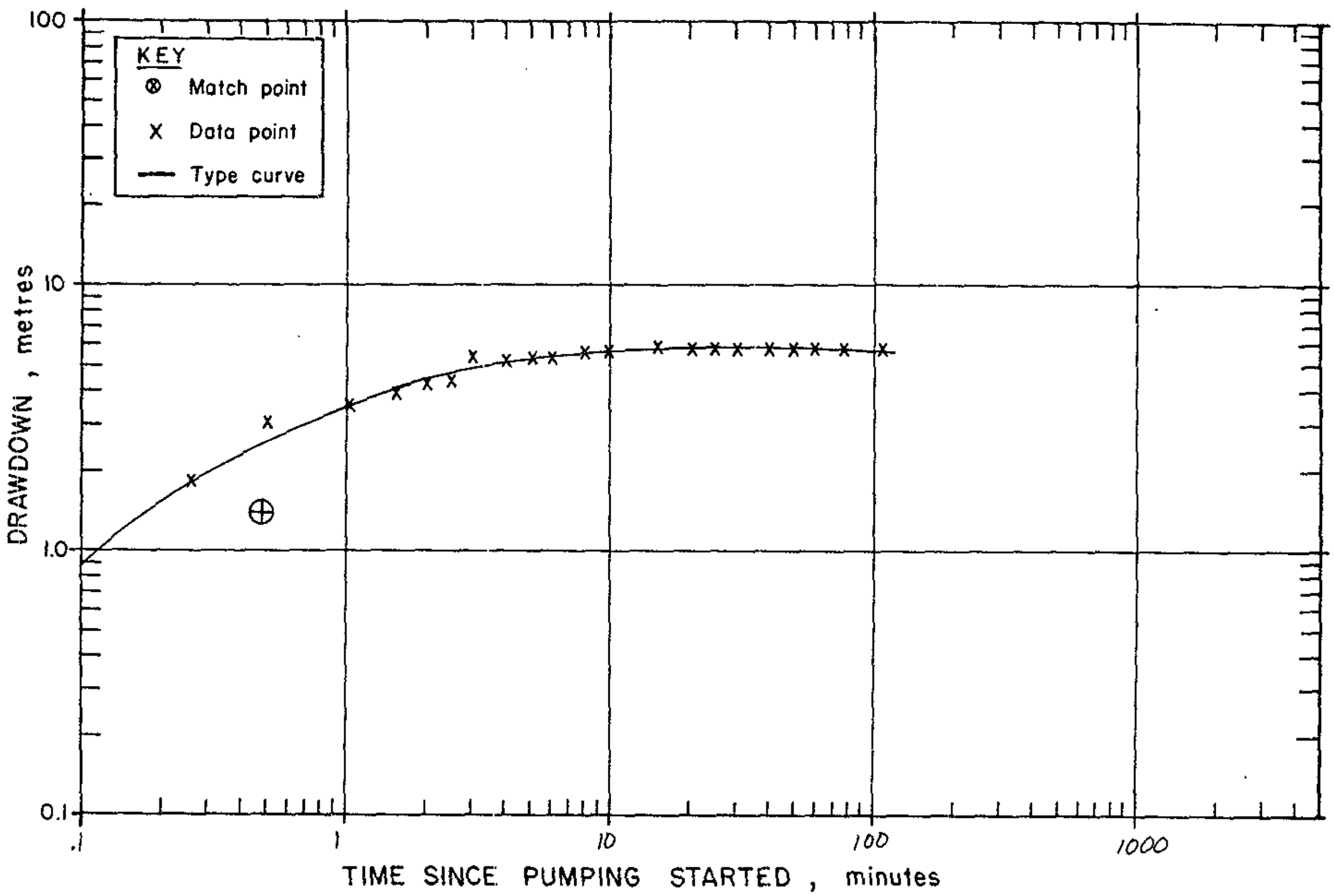
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 3
Well No. OW3 Data observed in OW3

Figure C.2.3

LEAKY AQUIFER ANALYSIS (Hantush Method)



CALCULATIONS:

$$T = \frac{Q\,10^{-3}\,W(u,r/B)}{4\pi s} = \frac{(3.31)\,10^{-3}\,(\,1\,)}{12.57\,(\,1.45\,)} = 2.91\times10^{-4}\ \text{metres}^2/\text{sec.}$$

$$S = \frac{240\,Ttu}{r^2} = \frac{240\,(\quad)(\quad)(\quad)}{(\quad)^2} = -$$

$$P = \frac{Tm'\,(r/B)^2}{r^2} = \frac{(\quad)(\quad)(\quad)^2}{(\quad)^2} = -\ \text{metres/sec.}$$

WHERE:

r = Radius from pumped well - (metres)
Q = Pumping rate 5.31 (litres/sec.)
m' = Average thickness of aquitard - (metres)
T = Transmissivity (metres²/sec.)
S = Storage coefficient (fraction)
P = Hydraulic conductivity of aquitard (metres/sec.)

s = Drawdown 1.45 (metres)
t = Time since pumping started - (minutes)

W(u, r/B) 1
u .1
r/B .15
Match point parameters from Hantush leaky aquifer type curve

HY-7 812-1507 RD Jan. 82

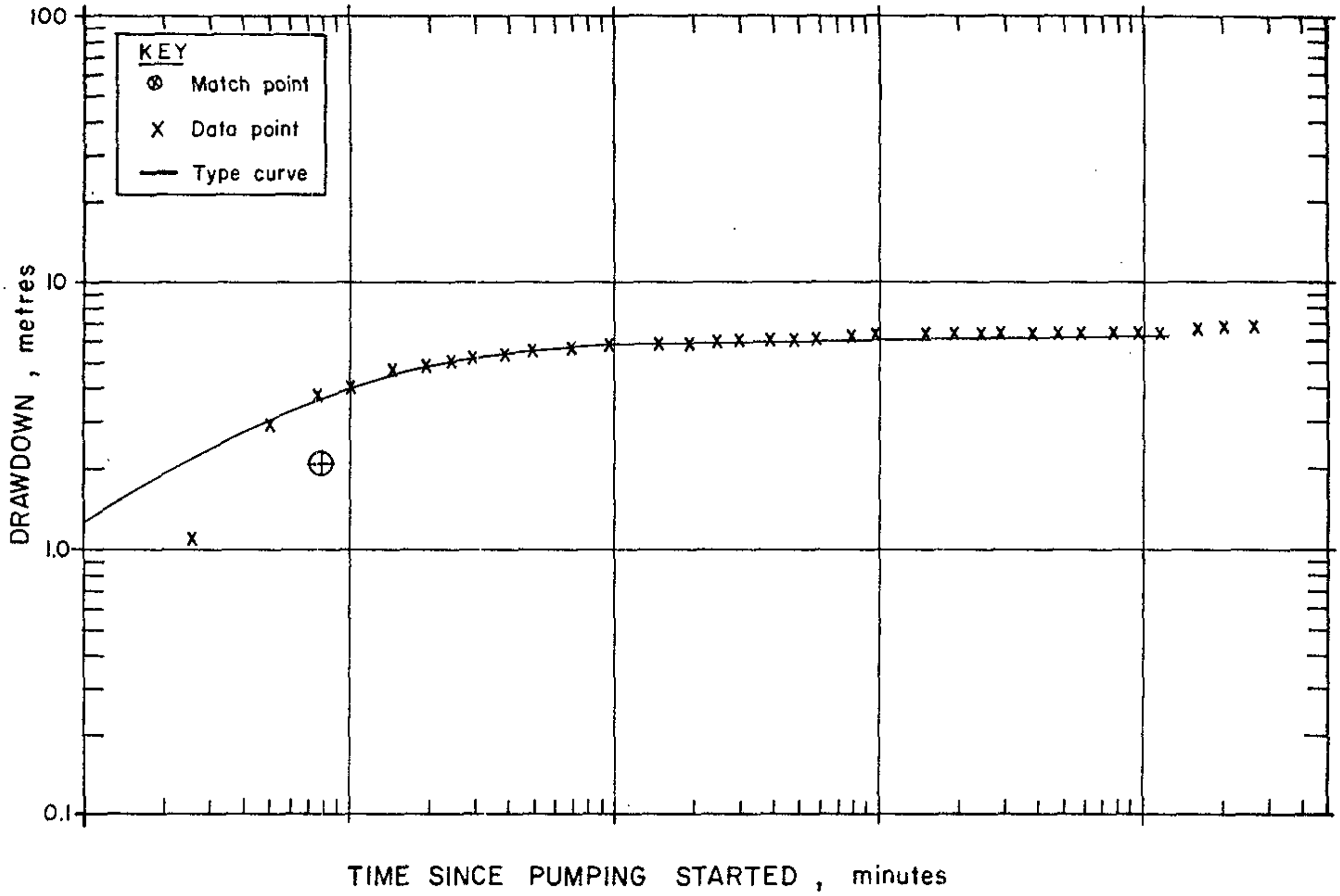
Golder Associates

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 6

Well No. PW2 Data observed in PW2

Figure C.2.4

LEAKY AQUIFER ANALYSIS (Hantush Method)



CALCULATIONS:

$$T = \frac{Q\,10^{-3}\,W(u,r/B)}{4\pi s} = \frac{(6.3)\,10^{-3}\,(1)}{12.57\,(2.2)} = 2.27\times10^{-4} \text{ metres}^2\text{/sec.}$$

$$S = \frac{240\,Ttu}{r^2} = \frac{240\,(\quad)(\quad)(\quad)}{(\quad)^2} = -$$

$$P = \frac{Tm'\,(r/B)^2}{r^2} = \frac{(\quad)(\quad)(\quad)^2}{(\quad)^2} = - \quad \text{metres/sec.}$$

WHERE:

r = Radius from pumped well - (metres)

Q = Pumping rate 6.3 (litres/sec.)

m' = Average thickness of aquitard .... (metres)

T = Transmissivity (metres^2/sec.)

S = Storage coefficient (fraction)

P = Hydraulic conductivity of aquitard (metres/sec.)

s = Drawdown 2.2 (metres)

t = Time since pumping started - (minutes)

W(u, r/B) 1
u .1
r/B .15
Match point parameters from Hantush leaky aquifer type curve

HY-7

812-1507 Drawn R.D. Revised Date Jan. '82

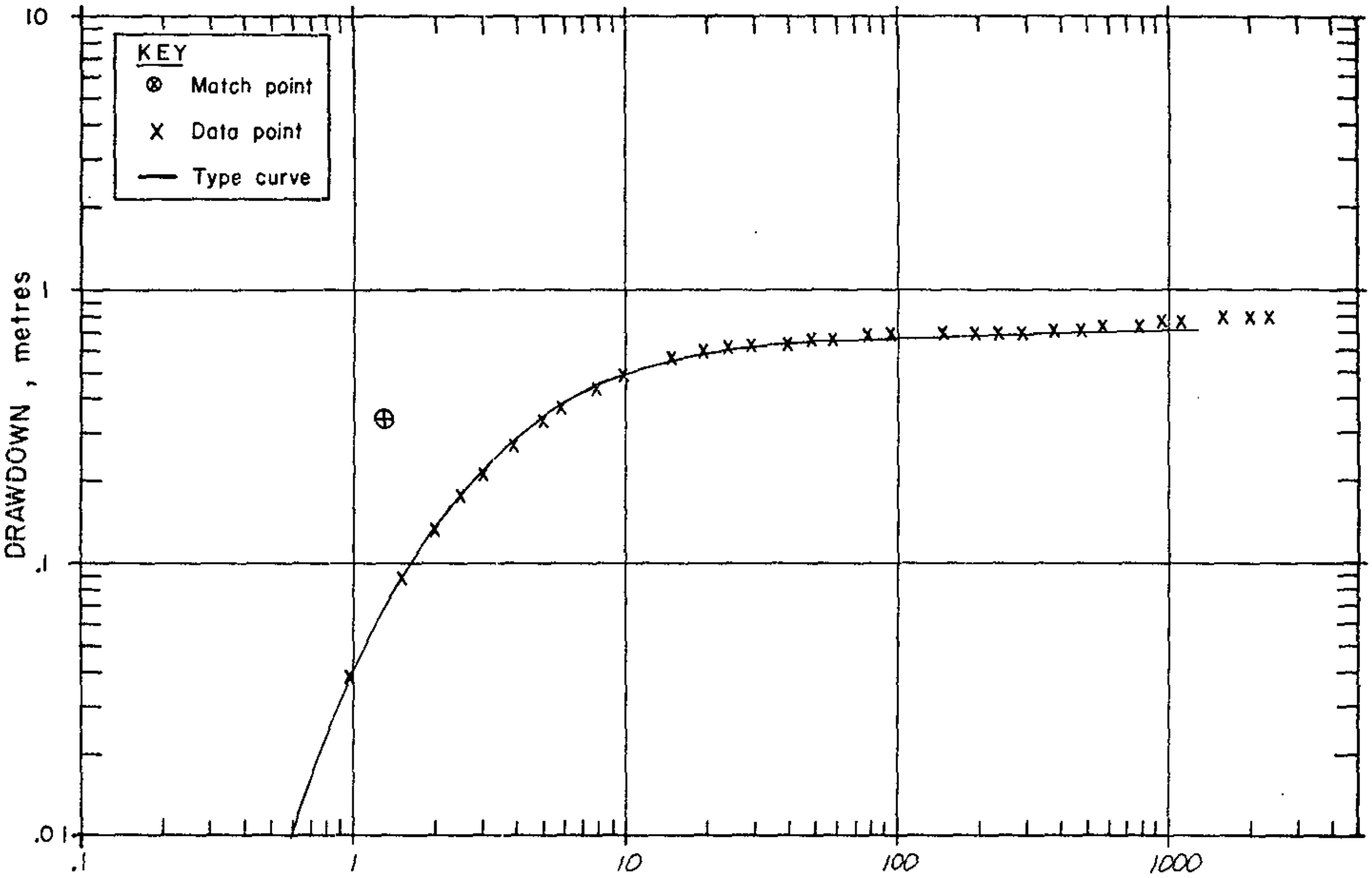
Golder Associates

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 6 **Figure** C. 2.5

Well No. PW2 Data observed in OW3

HY-7

LEAKY AQUIFER ANALYSIS (Hantush Method)



CALCULATIONS:

$$T = \frac{Q\,10^{-3}\,W(u,r/B)}{4\pi s} = \frac{(6.3)\,10^{-3}\,(1)}{12.57\,(.37)} = 1.35\times10^{-3}\ \text{metres}^2/\text{sec.}$$

$$S = \frac{240\,Ttu}{r^2} = \frac{240\,(1.35\times10^{-3})(1.35)(1)}{(47)^2} = 1.98\times10^{-4}$$

$$P = \frac{Tm'\,(r/B)^2}{r^2} = \frac{(\quad)(\quad)(\quad)^2}{(\quad)^2} = -\ \text{metres/sec.}$$

WHERE:

r = Radius from pumped well 47 (metres)

Q = Pumping rate 6.3 (litres/sec.)

m' = Average thickness of aquitard .... (metres)

T = Transmissivity (metres²/sec.)

S = Storage coefficient (fraction)

P = Hydraulic conductivity of aquitard (metres/sec.)

s = Drawdown .37 (metres)

t = Time since pumping started 1.35 (minutes)

W(u, r/B) 1
u 1
r/B .6
Match point parameters from Hantush leaky aquifer type curve

812-1507 R.O. Jan. '82

— **Golder Associates** —

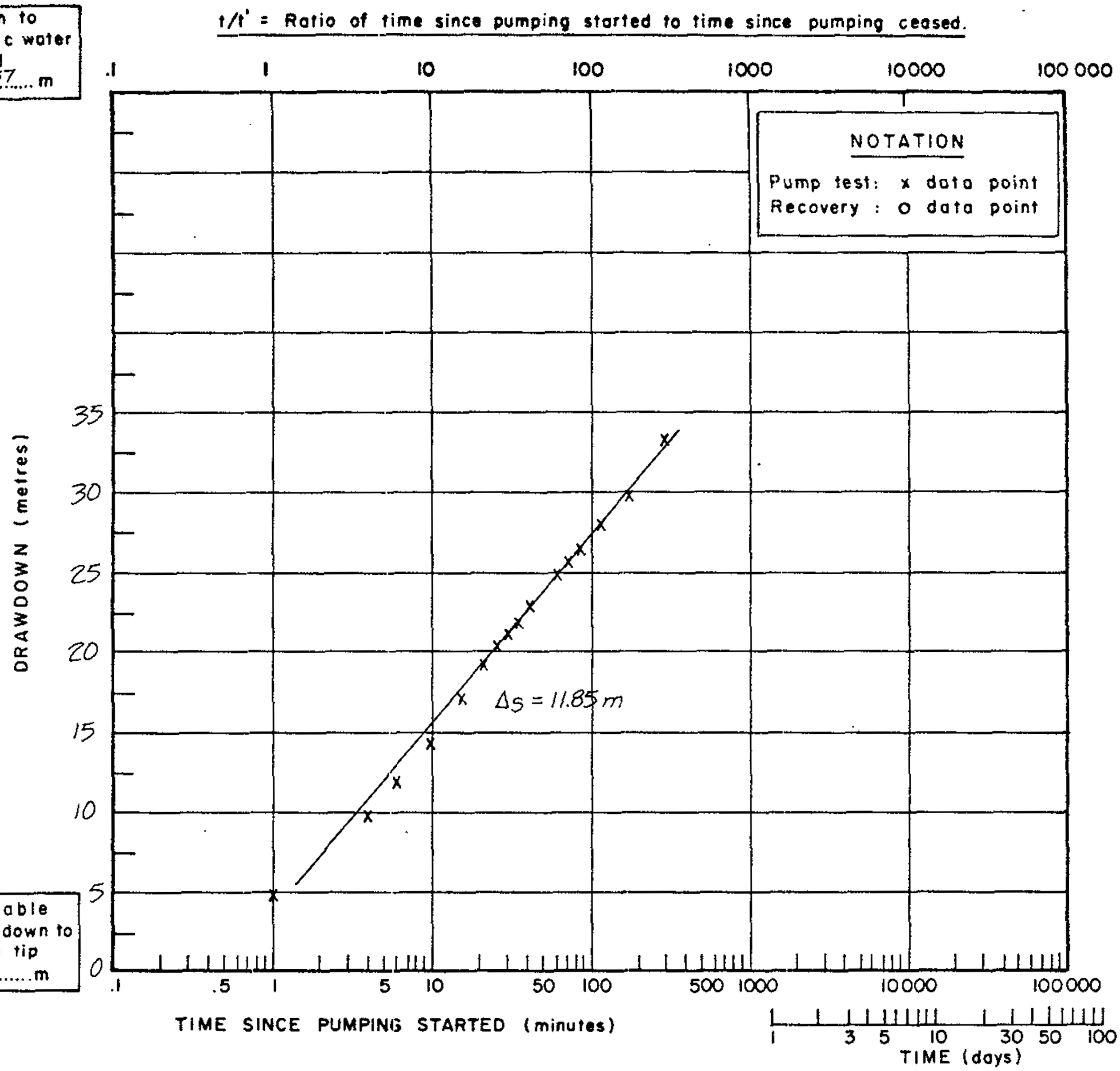
HY-1

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 1

Well No. OW1 Data observed in OW1

Figure C.3.1

Depth to static water level 4.57 m



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
$\frac{1.83 \times 1.28}{10^{+4} \times 11.85} = 1.97 \times 10^{-5}\ m^2/s$	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –
$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –

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

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

WHERE r = Radius from pumped well ...–... (metres)

Q = Pumping rate ...1.28... (litres/sec.)

t_0 = Time intercept for zero drawdown ...–... (min.)

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

Δs = Drawdown (metres per log cycle)

T = Transmissivity (metres²/sec.)

S = Storage coefficient (fraction)

Project No. 812-1507 Drawn R.O. Reviewed ...... Date Jan '82

Golder Associates

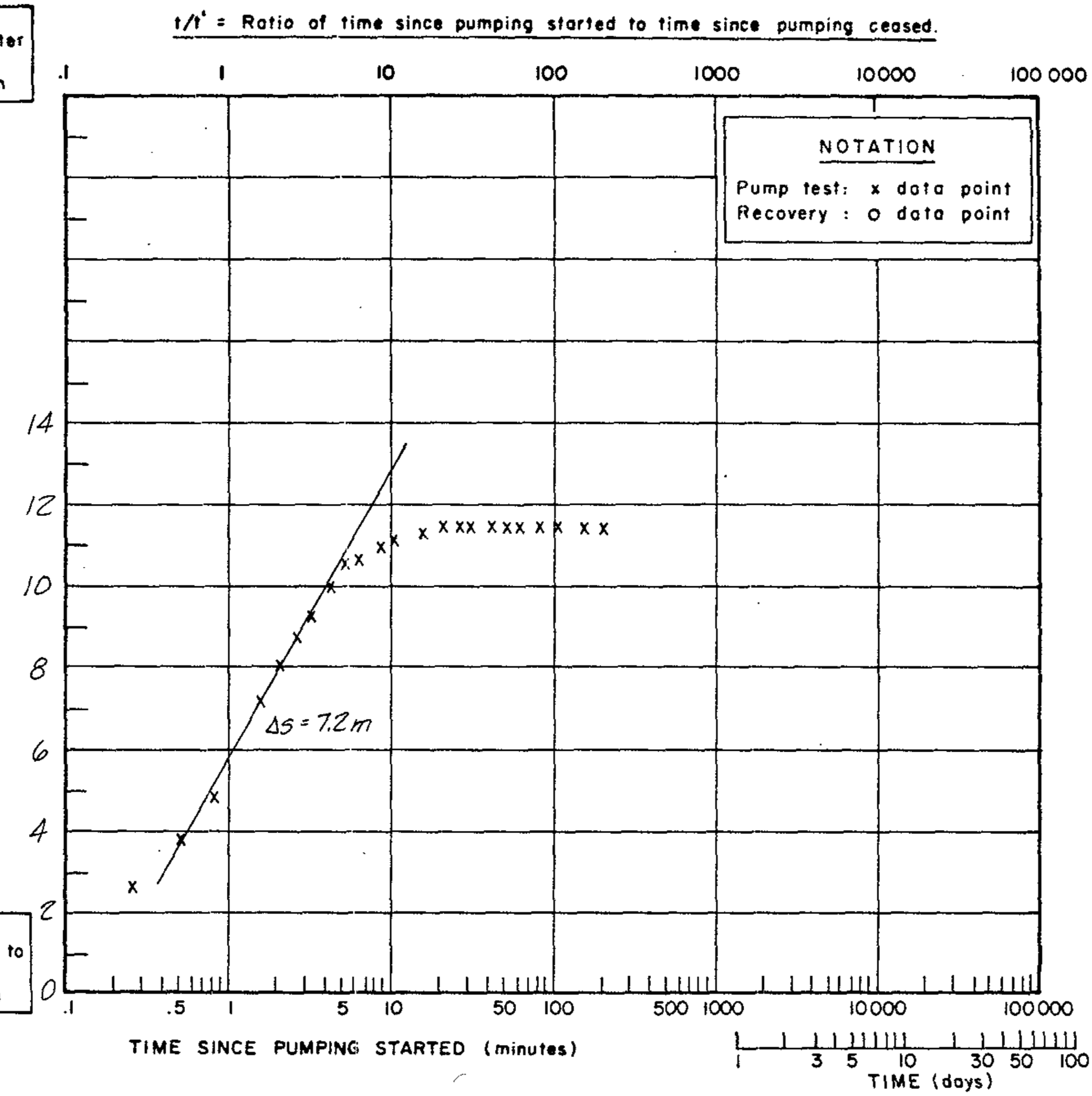
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 2

Well No. OW2 Data observed in OW2

Figure C.3.2

HY-I

Depth to static water level 7.37 m



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
$\frac{1.83 \times 2.25}{10^{+4} \times 7.2} = 5.71 \times 10^{-5}\,m^2/s$	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –
$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ –

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

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

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

Q = Pumping rate ...2.25... (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

Golder Associates

Project No. 812-1507 Drawn R.D. Reviewed .... Date Jan. '82

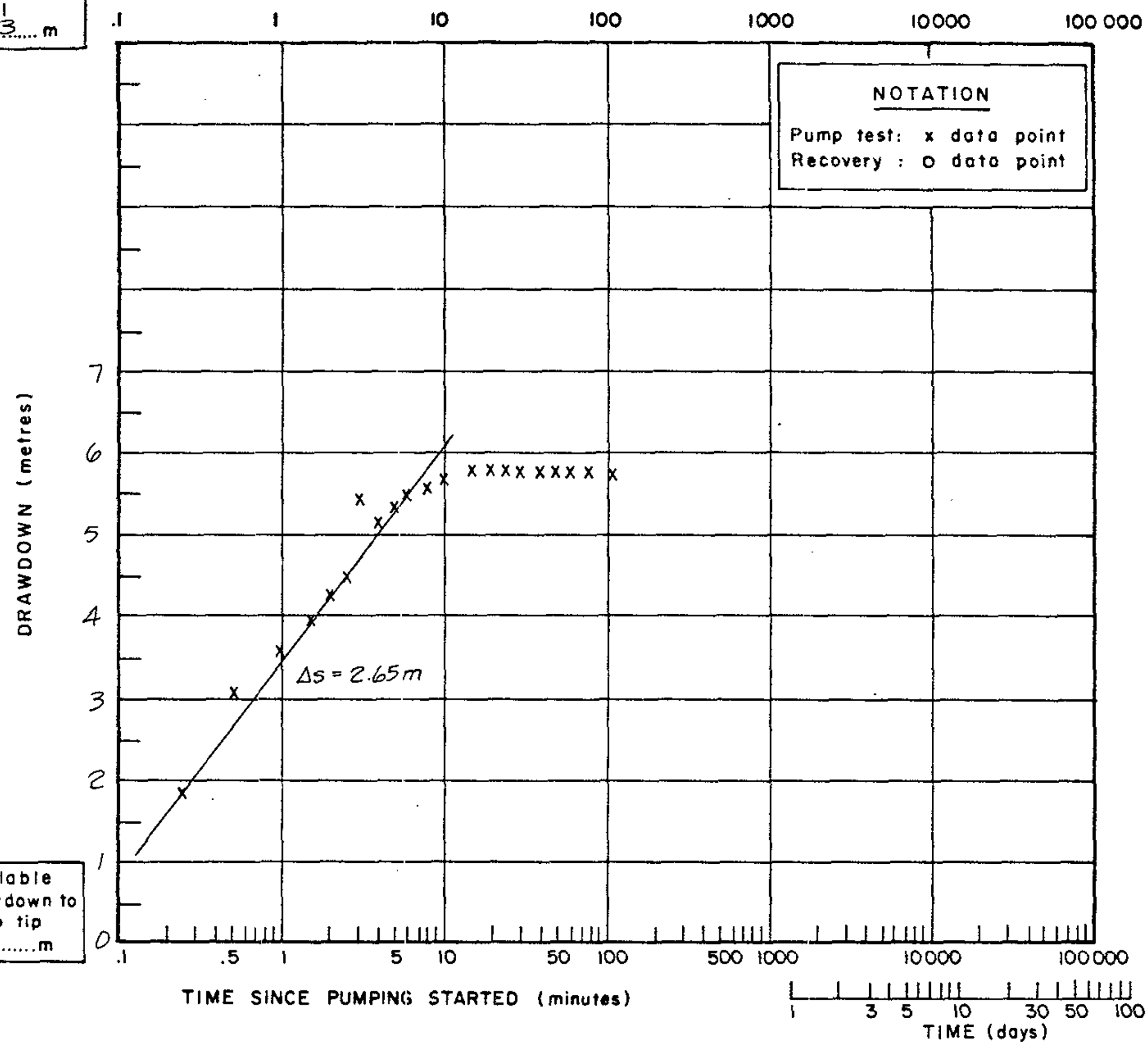
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 3

Well No. OW3 **Data observed in** OW3

Figure C. 3.3

Depth to static water level 5.83 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
$\frac{1.83 \times 5.31}{10^{+4} \times 2.65} = 3.66 \times 10^{-4}\ m^2/s$	$\frac{1.83 \times \text{.........}}{10^{+4} \times \text{.........}} =$ —
$\frac{1.83 \times \text{.........}}{10^{+4} \times \text{.........}} =$ —	$\frac{1.83 \times \text{.........}}{10^{+4} \times \text{.........}} =$ —

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

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

<u>WHERE</u> r = Radius from pumped well — (metres)
Q = Pumping rate 5.31 (litres/sec.)
t_0 = Time intercept for zero drawdown — (min.)
t_{min} = Approx. minimum value for which u < 0.01

Δs = Drawdown (metres per log cycle)
T = Transmissivity (metres^2/sec.)
S = Storage coefficient (fraction)

Golder Associates

Project No. 812-1507 Drawn R.Q. Reviewed .... Date Jan. '82

HY-1

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 4

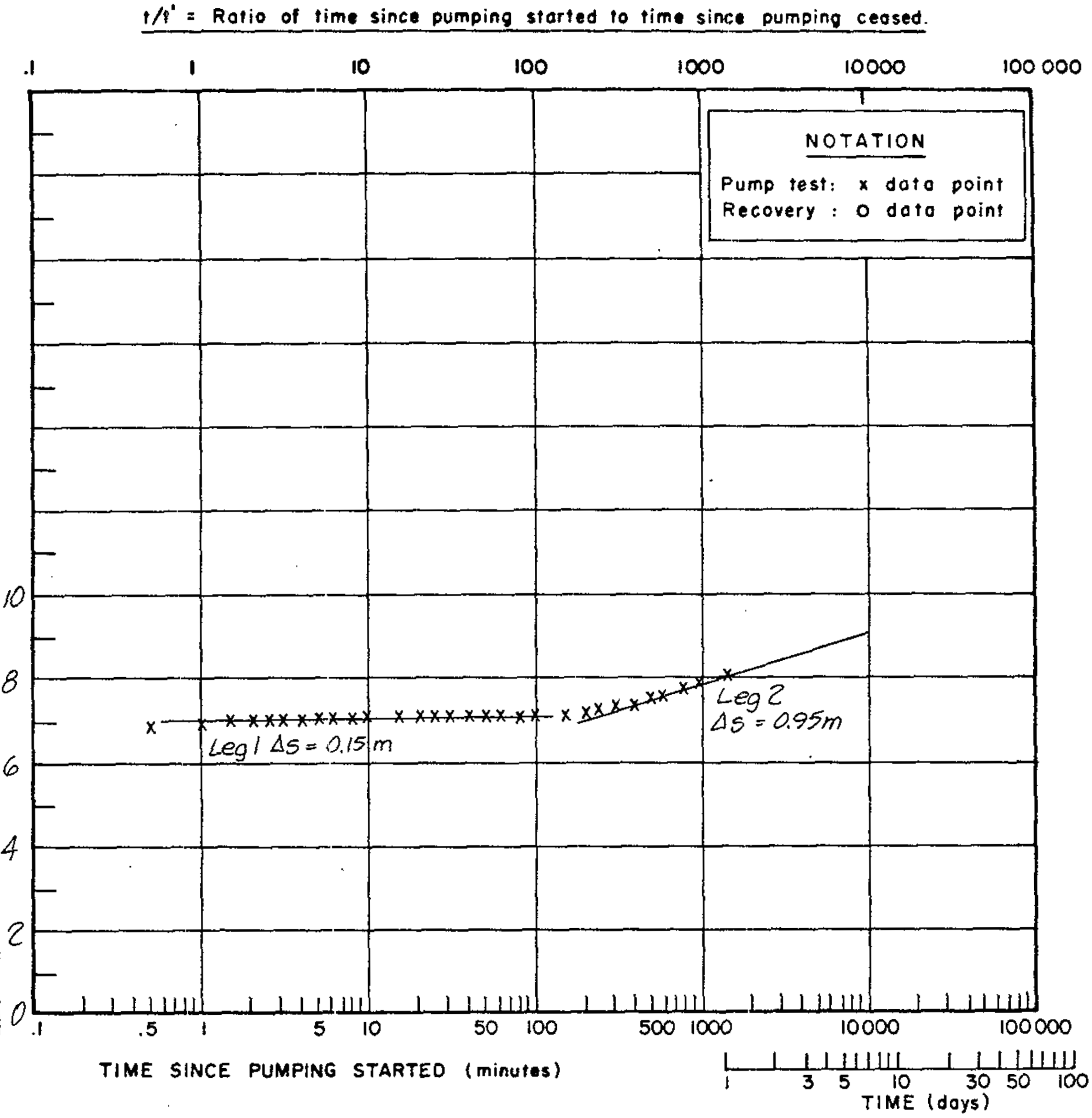
Well No. OW4 Data observed in OW4

Figure C.3.4

HY-1

Depth to static water level +2.83 m

Available drawdown to piezo tip .... m



CALCULATIONS

		Pumping	Recovery
Leg no. 1	$T = \frac{1.83\,Q}{\Delta s \times 10^4}$	$\frac{1.83 \times 7.17}{10^{+4} \times .15} = 8.74 \times 10^{-3}\,m^2/s$	$\frac{1.83 \times ....}{10^{+4} \times ....} =$
Leg no. 2	$T = \frac{1.83\,Q}{\Delta s \times 10^4}$	$\frac{1.83 \times 7.17}{10^{+4} \times .95} = 1.38 \times 10^{-3}\,m^2/s$	$\frac{1.83 \times ....}{10^{+4} \times ....} =$

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

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

WHERE r = Radius from pumped well ....–.... (metres)
Q = Pumping rate ....7.17.... (litres/sec.)
t_0 = Time intercept for zero drawdown ..–.. (min.)
t_{min} = Approx. minimum value for which u < 0.01

Δs = Drawdown (metres per log cycle)
T = Transmissivity (metres²/sec.)
S = Storage coefficient (fraction)

Project No. 812-1507 Drawn R.D. Reviewed .... Date Jan. '82

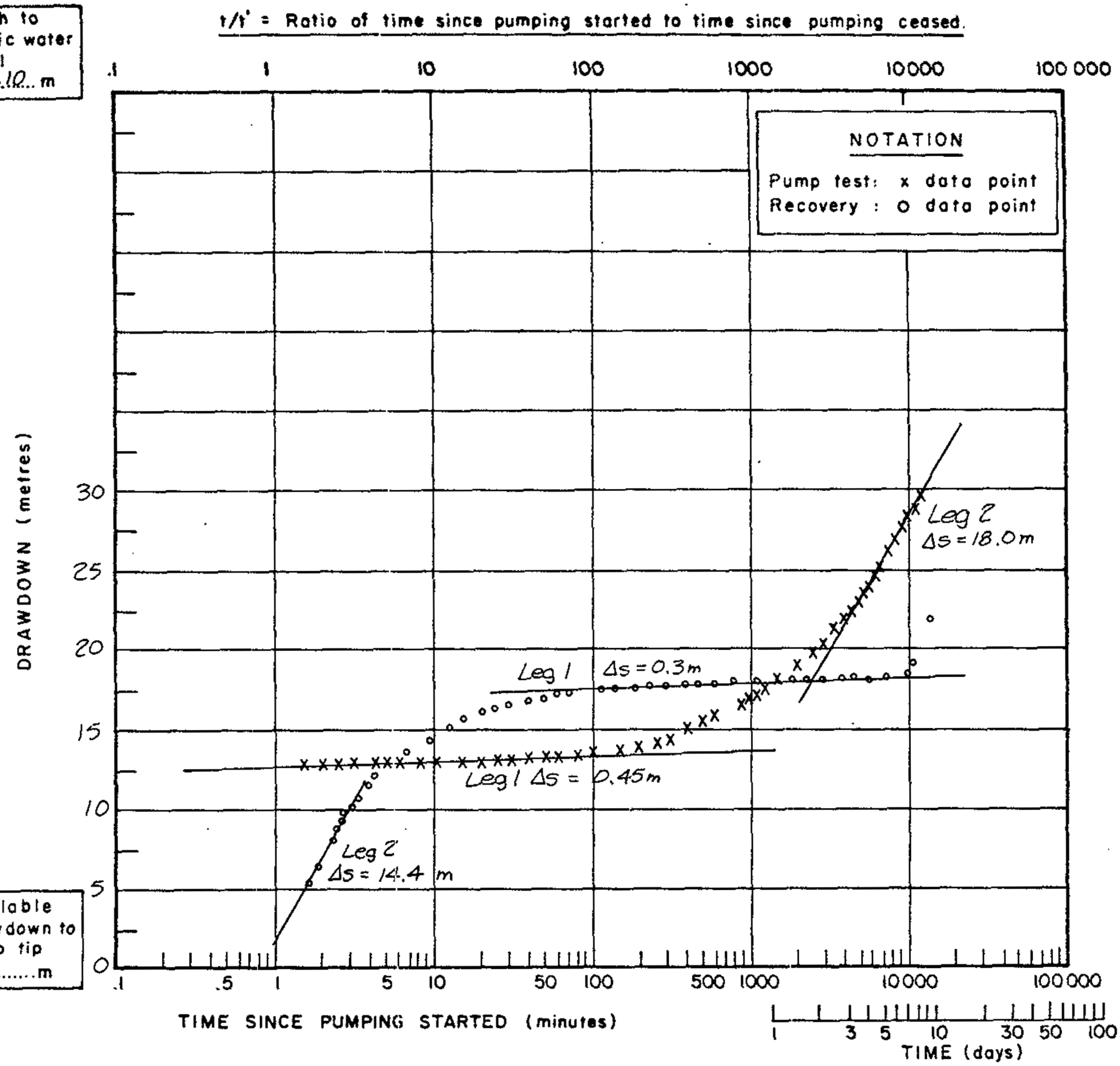
Golder Associates

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 5
Well No. PW1 **Data observed in** PW1

Figure C.3.5

HY-1

Depth to static water level +2.10 m



Available drawdown to piezo tip .... m

CALCULATIONS

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

Leg no. 2 $T = \frac{1.83\,Q}{\Delta s \times 10^4}$

Pumping	Recovery
$\frac{1.83 \times 26.5}{10^{+4} \times .45} = 1.07 \times 10^{-2}\ m^2/s$	$\frac{1.83 \times 26.5}{10^{+4} \times .3} = 1.61 \times 10^{-2}\ m^2/s$
$\frac{1.83 \times 26.5}{10^{+4} \times 18} = 2.69 \times 10^{-4}\ m^2/s$	$\frac{1.83 \times 26.5}{10^{+4} \times 14.4} = 3.36 \times 10^{-4}\ m^2/s$

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

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

<u>WHERE</u> r = Radius from pumped well – (metres)

Q = Pumping rate 26.5 (litres/sec.)

t_0 = Time intercept for zero drawdown – (min.)

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

Δs = Drawdown (metres per log cycle)

T = Transmissivity (metres²/sec.)

S = Storage coefficient (fraction)

Project No. 812-1507 Drawn R.O. Reviewed .... Date Jan. '82

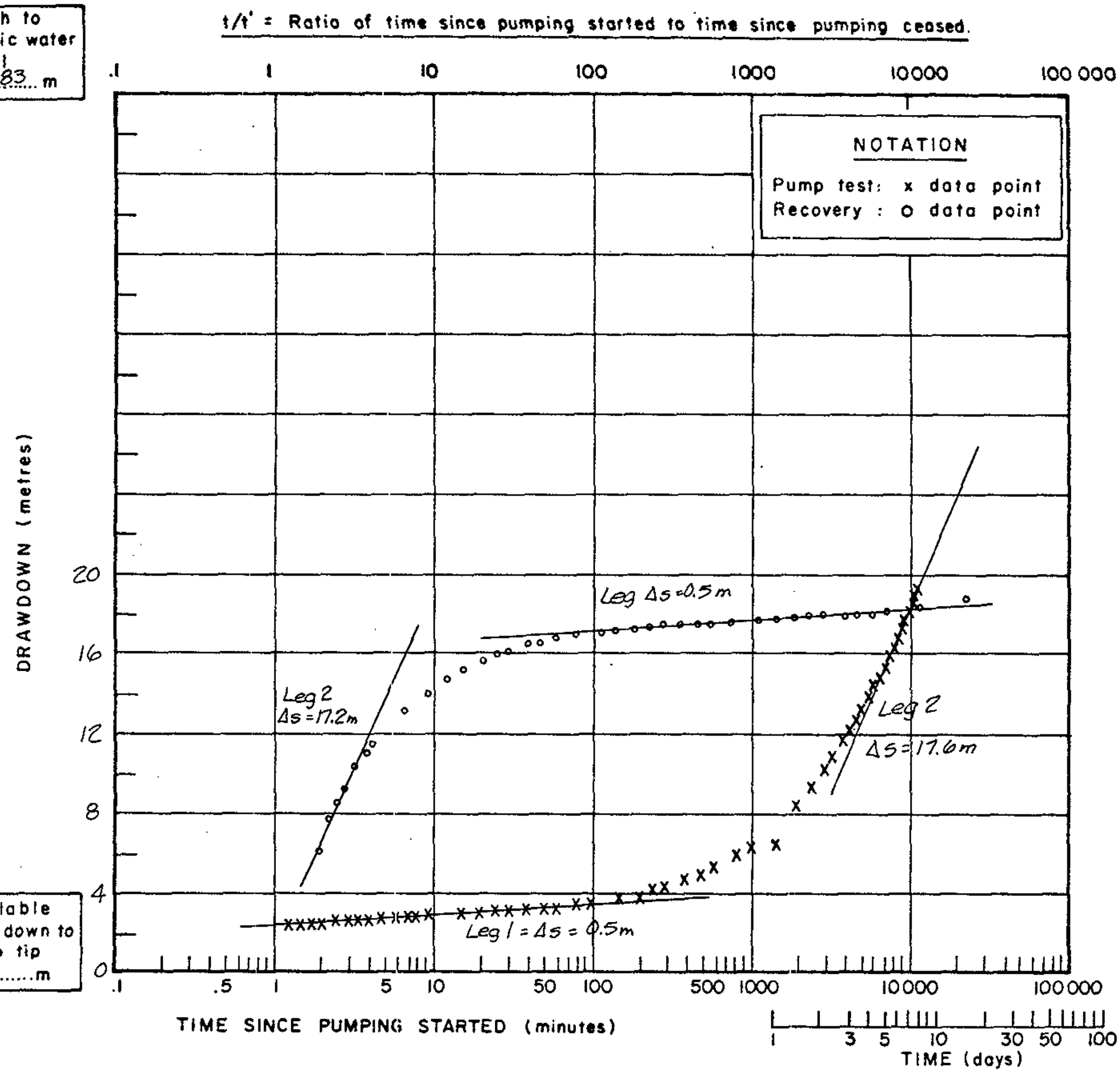
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 5

Well No. PW1 Data observed in OW4

Figure C.3.6

HY-1

Depth to static water level +2.83 m



Available drawdown to piezo tip ....m

CALCULATIONS

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

Leg no. 2 $T = \frac{1.83\ Q}{\Delta s \times 10^4}$

Pumping	Recovery
$\frac{1.83 \times 26.5}{10^{+4} \times 5} = 9.7 \times 10^{-3}\ m^2/s$	$\frac{1.83 \times 26.5}{10^{+4} \times 5} = 9.7 \times 10^{-3}\ m^2/s$
$\frac{1.83 \times 26.5}{10^{+4} \times 17.6} = 2.75 \times 10^{-4}\ m^2/s$	$\frac{1.83 \times 26.5}{10^{+4} \times 17.2} = 2.81 \times 10^{-4}\ m^2/s$

$$S = \frac{135\ T.\ t_0}{r^2} = \frac{135\ (2.75 \times 10^{-4})(960)}{(21.5)^2} = 7.71 \times 10^{-2}$$

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

<u>WHERE</u> r = Radius from pumped well 21.5 (metres)

Q = Pumping rate 26.5 (litres / sec.)

t_0 = Time intercept for zero drawdown 960 (min.)

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

Δs = Drawdown (metres per log cycle)

T = Transmissivity (metres²/sec.)

S = Storage coefficient (fraction)

Project No. 812-1507 Drawn R.O. Reviewed .... Date Jan. '82

Golder Associates

HY-I

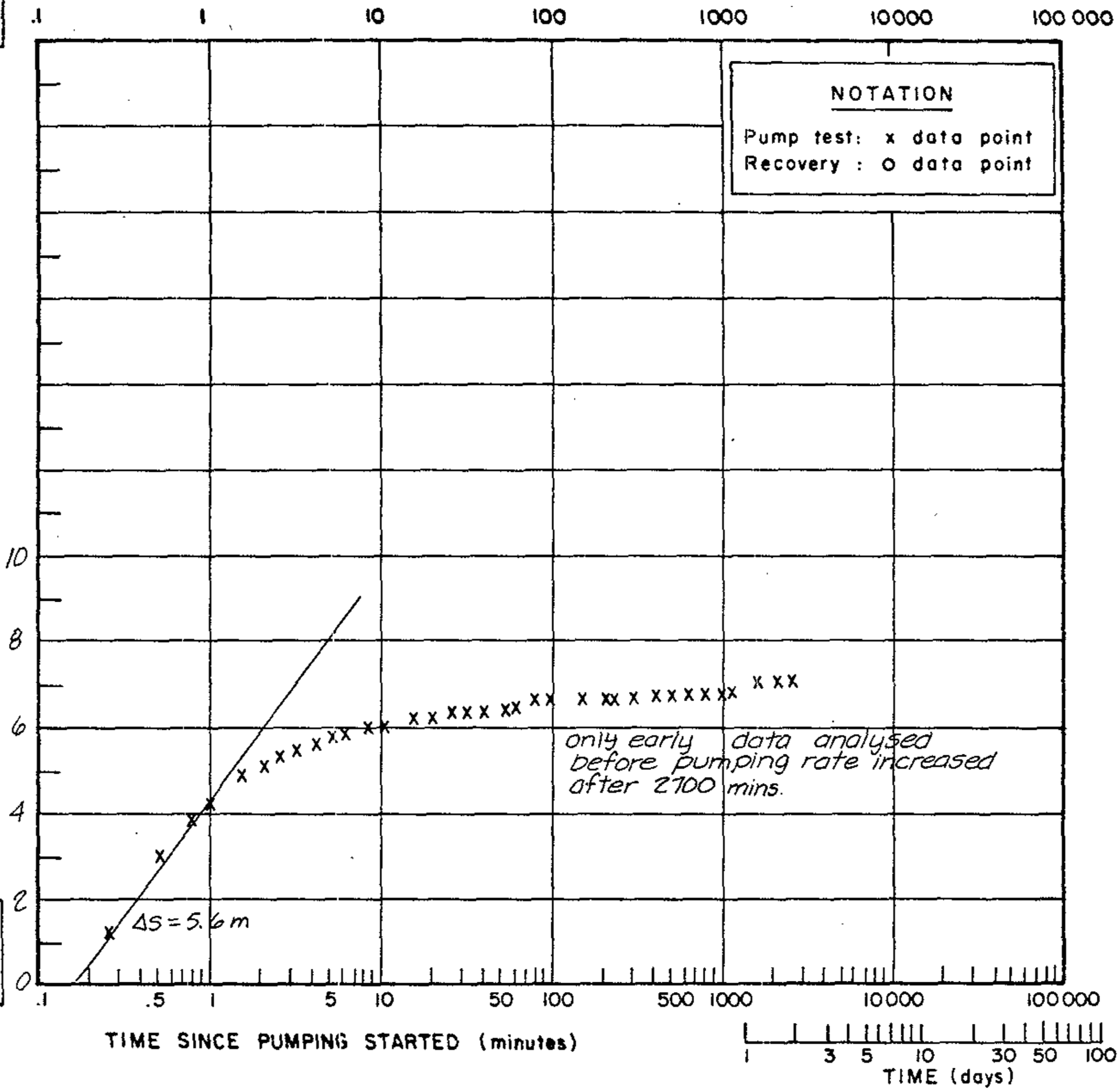
TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 6

Well No. PW2 Data observed in PW2

Figure C. 3.7

Depth to static water level 5.91 m

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



Available drawdown to piezo tip ....m

CALCULATIONS

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

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

Pumping	Recovery
$\dfrac{1.83 \times 6.3}{10^{+4} \times 5.6} = 2.05 \times 10^{-4}\,m^2/s$	$\dfrac{1.83 \times \ldots}{10^{+4} \times \ldots} =$
$\dfrac{1.83 \times \ldots}{10^{+4} \times \ldots} =$	$\dfrac{1.83 \times \ldots}{10^{+4} \times \ldots} =$

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

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

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

Q = Pumping rate 6.3 (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

Project No. 812-1507 Drawn R.D. Reviewed .... Date Jan. '82

Golder Associates

TIME - DRAWDOWN GRAPH FOR PUMP TEST No. 6

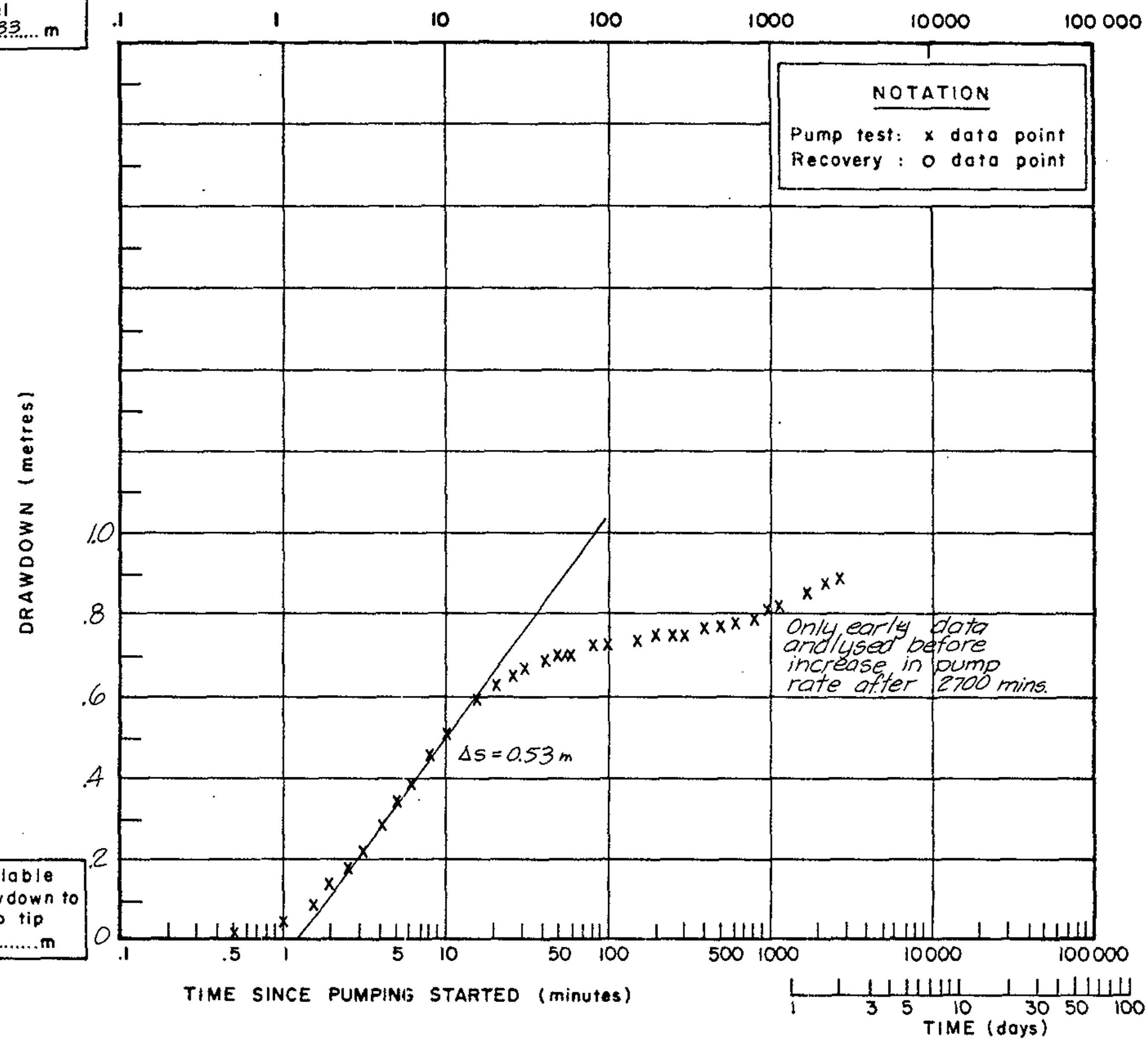
Well No. PW2   Data observed in OW3

Figure C.3.8

HY-I

Depth to static water level 5.83 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
$\frac{1.83 \times 6.3}{10^{+4} \times .53} = 2.17 \times 10^{-3}\ m^2/s$	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$
$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$ —	$\frac{1.83 \times \ldots}{10^{+4} \times \ldots} =$

$$S = \frac{135\,T.t_0}{r^2} = \frac{135\,(2.17\times10^{-3})(1.15)}{(47)^2} = 1.52 \times 10^{-}$$

$$t_{min} = \frac{.42\,r^2 S}{T} = \frac{.42\,(47)^2(1.52\times10^{-4})}{(2.17\times10^{-3})} = 64.9 \text{ minutes}$$

WHERE r = Radius from pumped well 47 (metres)
Q = Pumping rate 6.3 (litres/sec.)
t_0 = Time intercept for zero drawdown ...... (min.)
t_{min} = Approx. minimum value for which u < 0.01

Δs = Drawdown (metres per log cycle)
T = Transmissivity (metres²/sec.)
S = Storage coefficient (fraction)

Project No. 812-1507 Drawn R.D. Reviewed .... Date Jan. 82

Golder Associates

APPENDIX D

HYDROCHEMISTRY

Province of
British Columbia

Ministry of
Environment

Waste Management Branch
Thompson Nicola-Cariboo
Region
106, 1050 West Columbia
Kamloops, B.C.
V2C 1L4
Phone: 374-5981

YOUR FILE E/81/904

OUR FILE Area TNRD - Cache Cree

RECVD. AT GA VANCOUVER
JUN 11 1981
FILE No.

June 4, 1981

Golder Geotechnical Consultants Ltd.
224 West 8th Avenue
Vancouver, British Columbia
V5Y 1N5

Attention: D.E. Kneale

Dear Sirs:

Groundwater discharge to Hat Creek

In response to your letter of May 21, 1981, please be advised that the discharge of potable water from groundwater wells to surface waters does not require a Pollution Control Permit.

Yours very truly,

Kenneth A. Evans, P. Eng.
Head, Air-Industrial Section

KAE/du

June 16th, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Authority
Box 12121 - 555 West Hastings Street
Vancouver, B.C.

ATTENTION: Mr. F.G. Hathorn

SAMPLE IDENTIFICATION: Aquifer water samples collected June 9th, 1981, and received labelled as follows:

AQUIFER 1 - 40 ft. depth, 10°C, med. to coarse sand with some fine gravel & trace silt.

AQUIFER 2 - 70 ft. depth, 11°C, med. to coarse gravel with some sand.

PARAMETER	AQUIFER 1	AQUIFER 2
pH (units)	8.01	7.96
Conductivity (uhmos/cm^2)	318.	298.
Alkalinity - Total ($CaCO_3$)	234.	205.
Hardness ($CaCO_3$)	243.	227.
Calcium (Ca)	64.	60.4
Magnesium (Mg)	19.9	18.2
Sodium (Ma)	17.3	11.8
Sulfate	50.2	47.6
Chloride	1.2	1.4
Dissolved Solids	318.	292.
Carbonate alkalinity (as $CaCo_3$)	< 0.5	< 0.5
Bicarbonate alkalinity (as $CaCo_3$)	234.	205.

NOTE: All results in mg/l unless otherwise noted.

ECO-TECH LABORATORIES LTD.
Sandra H. Taylor
Chief Chemist

SHT/te

OW4

June 29, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Authority

ATTENTION: Mr. E.G. Hathorn

SAMPLE IDENTIFICATION: Water sample collected June 17, 1981 and received labelled as follows:
Ground Water #1, RH 81-87 @ 262'

PARAMETER	
pH (units)	7.92
Dissolved Solids	235.
Conductivity (umhos/cm^2)	286.
Alkalinity - Total ($CaCO_3$)	184.
Alkalinity - Carbonate ($CaCO_3$)	∠0.5
Calcium (Ca)	40.4
Chloride (Cl)	∠0.5
Hardness ($CaCO_3$)	171.
Magnesium (Mg)	16.8
Sodium (Na)	16.8
Sulfate (SO_4)	21.4
Alkalinity - Bicarbonate ($CaCO_3$)	184.

NOTE: All results in mg/l unless otherwise noted.
∠ = Less Than

ECO-TECH LABORATORIES LTD.
Sandra M. Taylor
Chief Chemist

ST/te

During Pumping

July 6, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Authority

ATTENTION: Mr. F.G. Hathorn

SAMPLE IDENTIFICATION: 2 Water Samples received June 30, 1981
Labelled: "Observation Wells #3 & #4 -
Construction Water Supply
Hat Creek"

PARAMETER	O.W. #3	O.W. #4
COLLECTED	2:00P.M. 06/29	11:00A.M. 06/28
pH (units)	7.44	8.00
Dissolved Solids - Total	336.	255.
Conductivity (umhos/cm^2)	278.	237.
Alkalinity - Phen ($CaCO_3$)	<0.5	<0.5
Alkalinity - Total ($CaCO_3$)	207.	176.
Alkalinity - Bicarbonate ($CaCO_3$)	207.	176.
Alkalinity - Carbonate ($CaCO_3$)	<0.5	<0.5
Hardness ($CaCO_3$)	229.	120.
Calcium (Ca)	58.3	19.1
Magnesium (Mg)	19.9	17.3
Chloride (Cl)	<0.5	<0.5
Fluoride (F)	0.10	0.60
Sulfate (SO_4)	58.0	52.0
Phosphorus - Total (P)	0.026	0.052
Nitrite (N)	<0.005	<0.005
Nitrate (N)	<0.005	0.038
Sodium (Na)	11.8	51.1

...../2

July 6, 1981

PARAMETER	O.W.#3	O.W.#4
Iron (Fe)	$\angle$0.05	$\angle$0.05
Arsenic (As) (ug/l)	$\angle$2.0	12.0
Barium (Ba)	$\angle$1.0	$\angle$1.0
Boron (B)	0.12	$\angle$0.05

NOTE: All results in mg/l unless otherwise noted.
$\angle$ = Less Than

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Sandra M. Taylor
Chief Chemist

ST/te

OW4

Eco-Tech
LABORATORIES LTD.

ENVIRONMENTAL TESTING
GEOCHEMISTRY
ANALYTICAL CHEMISTRY

783 Notre Dame Drive, Kamloops, B.C. V2C 5N8 — Telephone (604) 372-9700

July 29, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Authority

ATTENTION: Mr. F. G. Hathorn

SAMPLE IDENTIFICATION: Water sample received June 23, 1981
Collected June 19, 1981
Labelled: "R H 81-87 # 2"

PARAMETER	
pH (units)	7.93
Conductivity (uhos/cm)	252.
Hardness ($CaCO_3$)	134.
Alkalinity - Total ($CaCO_3$)	168.
Alkalinity - Carbonate ($CaCO_3$)	/0.5
Alkalinity - Bicarbonate ($CaCO_3$)	168.
Calcium (Ca)	23.9
Magnesium (Mg)	17.8
Sodium (Na)	28.5
Chloride (Cl)	/0.5
Sulfate (SO_4)	55.2
Total Dissolved Solids	276.

NOTE: All results in mg/l unless otherwise noted.
/ = Less Than.

ECO-TECH LABORATORIES LTD.
Sandra M. Taylor
Chief Chemist

ST/te

OW4

ECO-TECH LABORATORIES LTD.

ENVIRONMENTAL TESTING
GEOCHEMISTRY
ANALYTICAL CHEMISTRY

783 Notre Dame Drive, Kamloops, B.C. V2C 5N8 — Telephone (604) 372-9700

July 29, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Authority

ATTENTION: Mr. F. G. Hathorn

SAMPLE IDENTIFICATION: Water sample received June 23, 1981
Collected June 19, 1981
Labelled: "R H 81-87 # 2"

PARAMETER	
pH (units)	7.93
Conductivity (uhos/cm)	252.
Hardness ($CaCO_3$)	134.
Alkalinity - Total ($CaCO_3$)	168.
Alkalinity - Carbonate ($CaCO_3$)	$\angle$0.5
Alkalinity - Bicarbonate ($CaCO_3$)	168.
Calcium (Ca)	23.9
Magnesium (Mg)	17.8
Sodium (Na)	28.5
Chloride (Cl)	$\angle$0.5
Sulfate (SO_4)	55.2
Total Dissolved Solids	276.

NOTE: All results in mg/l unless otherwise noted.
$\angle$ = Less Than.

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Chief Chemist

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GEOCHEMISTRY
ANALYTICAL CHEMISTRY

783 Notre Dame Drive, Kamloops, B.C. V2C 5N8 — Telephone (604) 372-9700

July 29, 1981

ANALYTICAL RESULTS

CLIENT: B.C. Hydro & Power Auhtority

ATTENTION: Mr. F. G. Hathorn

SAMPLE IDENTIFICATION: 2 Water Samples received labelled:
"Raw Sample P.W. #1" and "Soda Springs"

PARAMETER	RAW SAMPLE P.W. #1	SODA SPRINGS
DATE	20/7/81; 15:30	22/7/81
pH (units)	7.98	6.50
Total Dissolved Solids	228.	1850.
Conductivity (uhos/cm)	316.	2190.
Alkalinity - Total ($CaCO_3$)	177.	1840.
Alkalinity - Carbonate ($CaCO_3$)	/0.5	/0.5
Alkalinity - Bicarbonate ($CaCO_3$)	177.	1840.
Hardness ($CaCO_3$)	165.	1820.
Calcium (Ca)	36.6	396.
Magnesium (Mg)	17.5	200.
Sodium (Na)	23.3	83.0
Chloride (Cl)	/0.5	1.3
Sulfate (SO_4)	28.6	75.0
Iron (Fe)	/0.01	0.14
Nitrite (N)	/.005	/0.005
Nitrate (N)	0.006	1.56
Fluoride (F)	0.37	1.25
Phosphorus (Total)(P)	0.011	0.036
Zinc (Zn)	/0.02	0.03
Arsenic (As)	/0.002	/0.002
Copper (Cu)	/0.01	0.02
Lead (Pb)	/0.05	0.05
Managanese (Mn)	0.04	0.17

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ENVIRONMENTAL TESTING
GEOCHEMISTRY
ANALYTICAL CHEMISTRY

PARAMETER	RAW SAMPLE P.W. #1	SODA SPRINGS
Barium (Ba)	∠0.5	∠0.5
Boron (B)	∠0.05	0.15

NOTE: All results in mg/l unless otherwise noted.
∠ = Less Than.

ECO-TECH LABORATORIES LTD.
Sandra M. Taylor
Chief Chemist

ST/te

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783 Notre Dame Drive, Kamloops, B.C. V2C 5N8 — Telephone (604) 372-9700

ANALYTICAL RESULTS   August 6, 1981

CLIENT: B.C. Hydro & Power Authority

ATTENTION: Mr. F.G. Hathorn

SAMPLE IDENTIFICATION: Water sample received July 30, 1981
Labelled "PW1 Elapsed time 11890 mins., Raw unfiltered sample"
Collected July 28, 1981

PARAMETER	
pH (units)	8.00
Dissolved Solids (Total)	224.
Conductivity (umhos/cm)	294.
Alkalinity - Total ($CaCO_3$)	185.
Alkalinity - Carbonate ($CaCO_3$)	<0.5
Alkalinity - Bicarbonate ($CaCO_3$)	185.
Hardness ($CaCO_3$)	157.
Calcium (Ca)	31.2
Magnesium (Mg)	19.0
Sodium (Na)	20.7
Chloride (Cl)	<0.5
Sulfate (SO_4)	27.6
Iron (Fe)	0.05
Nitrite (N)	<0.003
Nitrate (N)	<0.003
Fluoride (F)	0.42
Phosphorus (Total) (P)	0.043
Zinc (Zn)	<0.02

...../2

August 6, 1981

Arsenic (As)	/.002
Copper (Cu)	/0.01
Lead (Pb)	/0.05
Manganese (Mn)	0.04
Barium (Ba)	/0.5
Boron (B)	/0.05

NOTE: All results in mg/l unless otherwise noted.
/ = Less Than.

Sandra M. Taylor

ECO-TECH LABORATORIES LTD.
Sandra M. Taylor
Chief Chemist

ST/te
CC to Golder Associates - Dave Banton
CC to B.C.H. Cache Creek - Paul Imada

APPENDIX E

PUMP SPECIFICATIONS

PUMP SPECIFICATIONS FOR BERKELEY MODEL NO. 4CLM14 - 3 (O.W. 2)

Figure E-1

BERKELEY

BERKELEY PUMP COMPANY

4 CLM SUBMERSIBLES

FOR 4" I.D. WELLS

PERFORMANCE	2100
DATE	12-1-77
PAGE	8
SUPERSEDES	
Performance 2100 Page 8	
Dated 3-1-72	

45 GPM SERIES

Setting in Feet	4CLM8-1½				4CLM10-2				4CLM14-3				4CLM23-5			
	DISCHARGE PRESSURE IN POUNDS PER SQUARE INCH															
	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60
	GALLONS PER MINUTE															
0		63	49	31		65	55	43			61	54				62
20		57	42	19		61	50	37		64	58	50			64	60
40	64	51	34			56	45	30		62	54	46			62	58
60	59	44	22		62	52	39	20	65	59	51	42		64	60	56
80	53	36			58	47	32		62	56	47	37		63	58	54
100	46	26			53	41	24		60	52	43	32		61	56	52
120	39				48	34			56	49	39	26		59	54	50
140	30				42	27			53	45	34	19	63	57	52	49
160	18				36				50	40	28		60	56	50	45
180					29				46	35	21		58	53	48	42
200					20				41	30			56	51	46	39
220									37	23			54	49	43	37
240									32				52	46	40	34
260									25				49	44	37	31
280									18				47	41	34	27
300													45	38	31	23
320													42	35	28	18
340													39	32	24	
360													37	29	19	
380													33	25		
400													31	21		
420													26			
440													22			

RISER PIPE 2"

NOTE: Best performance will obtained when operating within the heavy lines. For other conditions check 4CL and 4CM.

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

PUMP SPECIFICATIONS FOR BERKELEY MODEL NO. 6AH3-5 (O.W.3)

Figure E-2

BERKELEY ®

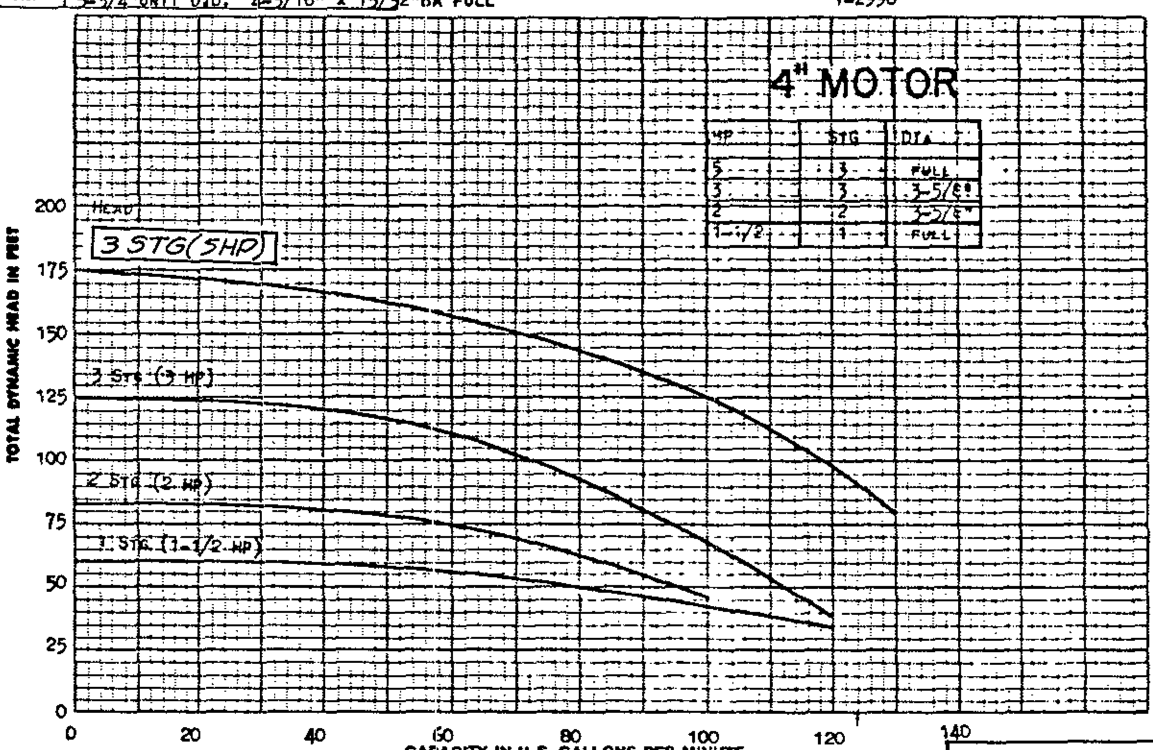
BERKELEY PUMP COMPANY

SUBMERSIBLE TURBINE PUMPS
6" AND 7" BOWLS
PERFORMANCE CURVES

CURVE	2500
DATE	12-1-77
PAGE	1.51
SUPERSEDES	
Curve 2500 Page 1.51 Dated 4-16-73	

	Bowl	Impeller
Material	PLASTIC	PLASTIC
Patt. No.	H-2135	M-4876
Mach. No.	H-2135	M-4876
Diameter	5-3/4" UNIT O.D. 4-3/16" x 15/32" BA FULL	

Based on T- *
*T-2990 Performance for MULTI Stage Pump.
T-2995
T-2996
T-2998



C-6628 Issue Date 7-5-77 (Supersedes C-6628 Dated 1-18-73) 3600 NOMINAL R.P.M. MODEL 6AH

Depth To Water Level In Feet	6AH1-1½				6AH2-2				6AH3-3				6AH3-5				6AH5-7½			
	DISCHARGE PRESSURE IN POUNDS PER SQUARE INCH																			
	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60
	CAPACITY IN GALLONS PER MINUTE AT DISCHARGE PRESSURE SHOWN																			
0		92				100				115	81				124	80				122
20						75				101	60				110	58				115
40	108								120	86				126	94				125	107
60	40				84				106	66				114	68				118	99
80					46				90				130	100	20			126	110	90
100									73				118	78				119	102	78
120									44				105	42			128	111	94	60
140													85				121	102	83	
160													56				114	94	66	
180																	106	84		
200																	98	68		
220																	88			
240																	76			
260																	56			
280																				

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

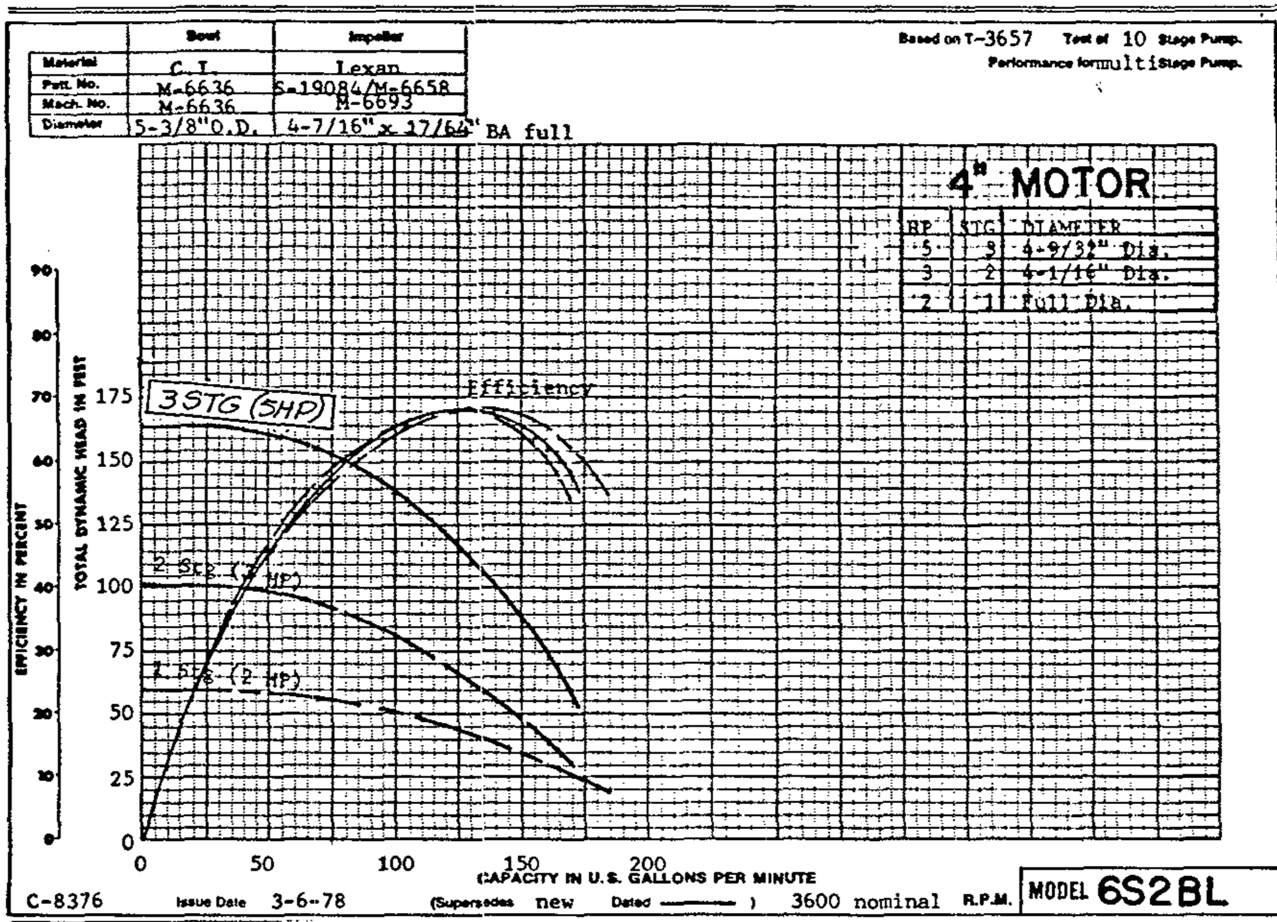
PUMP SPECIFICATIONS FOR BERKELEY MODEL NO. 6S2BL3-5 (O.W.4)

Figure E-3

BERKELEY PUMP COMPANY

SUBMERSIBLE TURBINE PUMPS
6" AND 7" BOWLS
PERFORMANCE CURVES

CURVE	2500
DATE	4-14-78
PAGE	2.01
SUPERSEDES	
Curve 2500	Page 2.01
Dated 12-1-77	



C-8376 Issue Date 3-6-78 (Supersedes new Dated ———) 3600 nominal R.P.M. MODEL 6S2BL

Depth To Water Level In Feet	6S2BL1-2				6S2BL2-3				6S2BL3-5				6S2BL4-7½				6S2BL6-10			
	DISCHARGE PRESSURE IN POUNDS PER SQUARE INCH																			
	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60
	CAPACITY IN GALLONS PER MINUTE AGAINST DISCHARGE PRESSURE SHOWN																			
0		120				152	75				147	100			175	150				163
20	182					125				165	130	65			165	138			170	156
40	137				159	89				151	108			177	154	123			165	149
60	25				134				168	135	79			167	142	104		171	159	141
80					101				155	114			180	158	128	84		167	152	133
100					35				140	99			171	147	111	145		161	144	123
120									121	15			162	133	90		173	154	135	113
140									99				149	117	60		162	146	126	101
160									62				137	97			156	138	115	85
180													122	73			148	129	105	63
200													102				140	119	91	
220													81				132	109	72	
240													40				122	97	15	
260																	112	78		
280																	100	47		
300																	83			
320																	60			

PROJECT NO. 812-1507 DRAWN .... REVIEWED .... DATE Mar. '82

PUMP SPECIFICATIONS FOR BERKELEY MODEL NO. 7S3L-40 (P.W. I)

Figure E-4

BERKELEY PUMP COMPANY

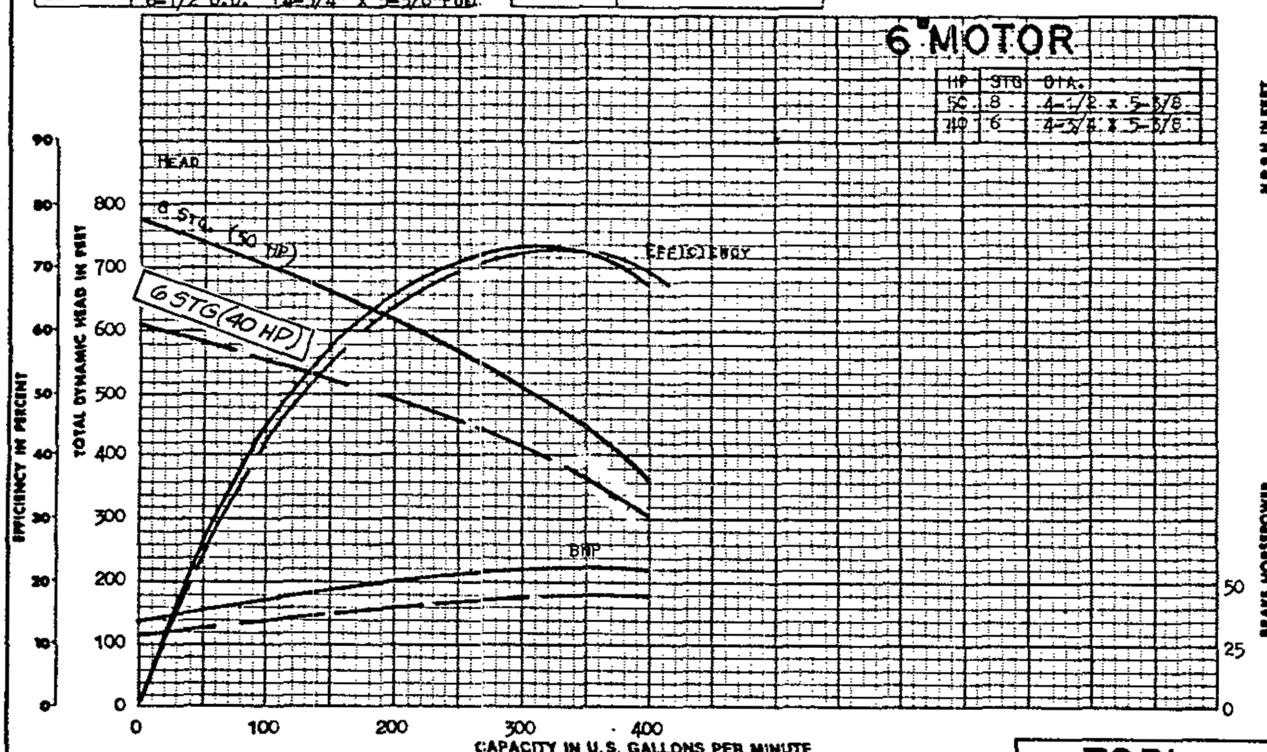
SUBMERSIBLE TURBINE PUMPS
6" AND 7" BOWLS
PERFORMANCE CURVES

CURVE	2500
DATE	7-1-76
PAGE	5.01
SUPERSEDES	
Curve 2500	Page 5.01
Dated 6-1-76	

	Bowl	Impeller
Material	C.I.	Brz.
Patt. No.	M-1975	M-2103
Mach. No.	M-1975	M-2103
Diameter	6-1/2"O.D.	4-3/4" x 5-3/8"Full

Letter	Impeller Diameter

Based on T- 3228 Test of 3 Stage Pump.
Performance for 6 & 8 Stage Pump.

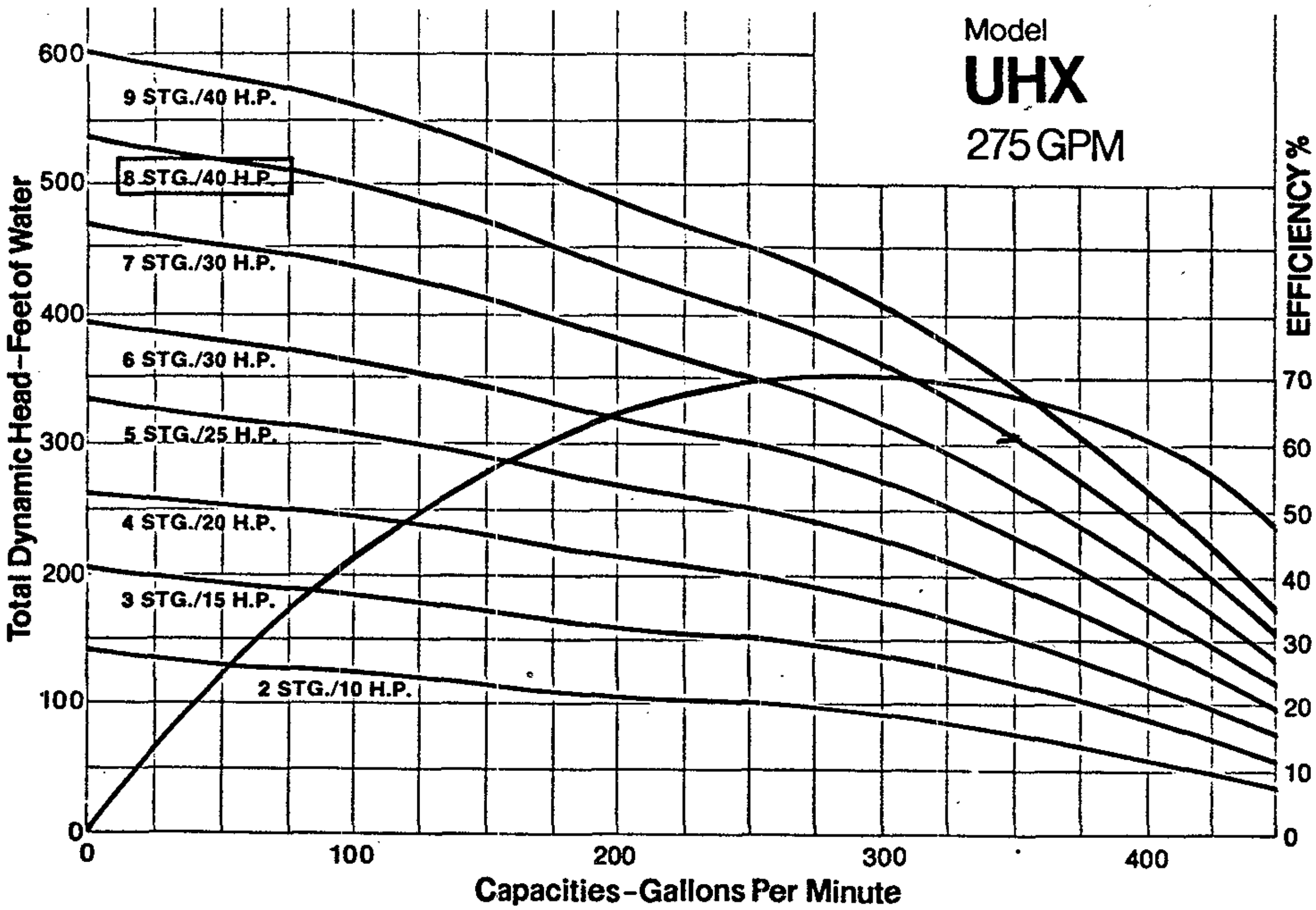


C-8142 Issue Date 4-2-76 (Supersedes NEW Dated -) 3600 Nom. R.P.M. MODEL 7S3L

PROJECT NO. 812-1507 DRAWN .... REVIEWED .... DATE Mar. '82

PUMP SPECIFICATIONS FOR GOULD'S MODEL
NO. UHX 8STG /40 HP (P.W. I)

Figure E-5



Horsepower		30						30						40					
Stages		6 STAGE						7 STAGE						8 STAGE					
Tank Pressures		0	20	30	40	50	60	0	20	30	40	50	60	0	20	30	40	50	60
Depth to Water	25 ft.				448	430	408					445	430						443
	50 ft.			447	428	407	385				444	428	406					442	428
	75 ft.		446	427	406	383	365			443	427	407	392				441	427	413
	100 ft.		425	405	382	363	335		442	426	406	391	370			440	426	412	396
	125 ft.	441	403	380	360	334	305		425	405	390	369	350		440	425	410	395	378
	150 ft.	421	378	358	332	304	273	439	405	389	369	348	325		425	409	394	377	361
	175 ft.	400	356	331	302	270	211	420	388	368	347	324	300	436	408	393	376	360	341
	200 ft.	375	330	300	268	210		403	368	345	323	300	265	421	392	375	359	341	321
	250 ft.	325	257	198	160			363	320	293	265	223	183	387	356	338	318	300	268
	300 ft.	250	145					315	258	215	180	143	90	353	346	290	262	225	190
	350 ft.	135						250	183	132	90			311	258	220	188	158	118
	400 ft.							165	78					250	183	150	113		
	450 ft.													175	105				
	500 ft.													95					
	600 ft.																		

PROJECT NO. 812-1502 DRAWN ...... REVIEWED ...... DATE Mar. '82

PUMP SPECIFICATIONS FOR BERKELEY MODEL NO. 6S2AL2-5 (P.W.2)

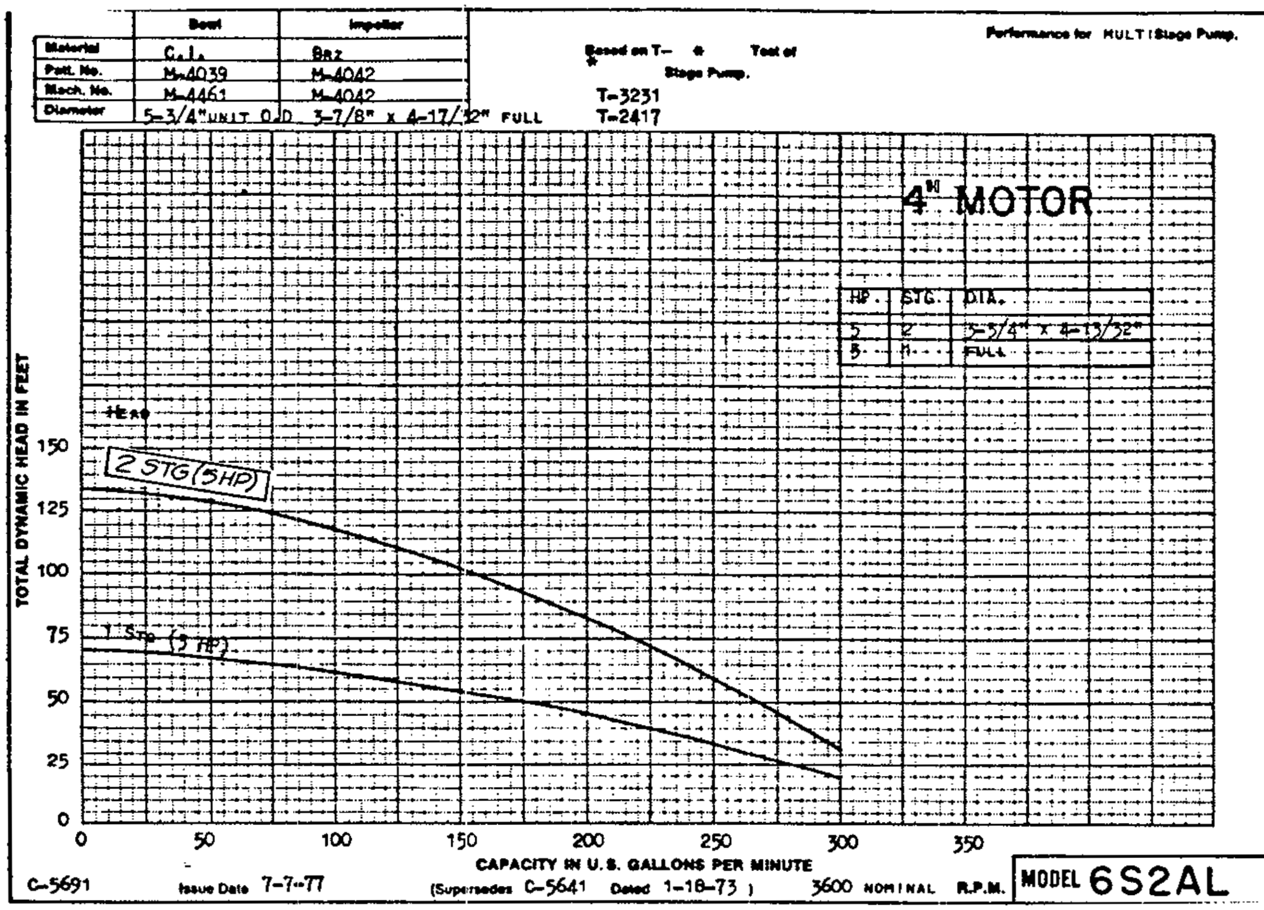
Figure E-6

BERKELEY®

BERKELEY PUMP COMPANY

6S2AL SUBMERSIBLE PERFORMANCE

PERFORMANCE	2500
DATE	6-1-78
PAGE	8
SUPERSEDES	
NEW	



Depth To Water Level In Feet	6S2AL1-3				6S2AL2-5				6S2AL3-7				6S2AL4-10			
	DISCHARGE PRESSURE IN POUNDS PER SQUARE INCH															
	0	20	40	60	0	20	40	60	0	20	40	60	0	20	40	60
	CAPACITY IN GALLONS PER MINUTE AT DISCHARGE PRESSURE SHOWN															
0		195				275	180				248	180			285	260
20	300	54				250	125			286	220	135			265	237
40	225				285	195	25			259	199	95		290	243	212
60	115				250	140			293	230	142			270	217	187
80					207	40			265	200	105		293	250	194	156
100					155				237	157			275	225	167	126
120					95				209	115			256	200	128	40
140									170				231	173	100	
160									126				205	140	60	
180													180	109		
200													150	65		
220													115			
240													80			
260																

PROJECT NO. 812-1507 DRAWN .... REVIEWED .... DATE Mar. 82