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FOREWORD

This report presents the results of work carried out to evaluate the feasibility of using a system of horizontal drains to lower groundwater levels in a hillside in the Mid-levels area of Hong Kong in order to improve the hillside stability. It should be extremely useful to all those concerned with the stabilisation of slopes in Hong Kong and elsewhere. Particularly noteworthy is the review of international practice with regard to horizontal drain installations, which is thought to be the most comprehensive and up-to-date review of its kind. In addition, details are provided of the field trials undertaken to evaluate the problems involved in drilling and installing long horizontal drains into the colluvium and weathered rocks encountered in Hong Kong.

A large number of individuals and organisations provided information for the preparation of this report. The assistance of the following is gratefully acknowledged :

Dr A.J. da Costa Nunes (Tecnosolo, Brazil); Prof. J.N. Hutchinson (Imperial College of Science and Technology, England); Prof. T.C. Kenney (University of Toronto, Canada); Dr G. Pilot (Laboratoire Central des Ponts et Chaussées, France); Dr D.L. Royster (Tennessee Department of Transportation, USA); Mr D.D. Smith (California Department of Transportation, USA); the late Prof. Ch. Veder (Graz Technical University, Austria); Canterbury City Council, England; the Snowy Mountains Hydro-Electric Authority, Australia; Binnie & Partners (Hong Kong); Brickell, Moss & Partners; Enpack (HK) Ltd; Fugro (Hong Kong) Ltd; Gammon (Hong Kong) Ltd; Harris & Sutherland (Far East); Intrusion-Prepakt (Far East) Ltd; Maunsell Geotechnical Services; Mott, Hay & Anderson Far East; P & T Civil Engineers Ltd; Scott Wilson Kirkpatrick & Partners; Ove Arup & Partners Hong Kong Ltd.

The report was prepared in the Geotechnical Control Office under the general direction of Mr. J.C. Shelton, Chief Geotechnical Engineer/Island West. In addition to the authors, many other individual staff members contributed in some way towards the work described.

E.W. Brand

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Principal Government Geotechnical Engineer

CONTENTS

1. INTRODUCTION

The Territory of Hong Kong is situated at the mouth of the Pearl River on the south eastern coast of China. It has a land area of only 1 050 sq km and a population of nearly six million. It consists of Hong Kong Island, the Kowloon peninsula and the 'New Territories' (Figure 1). The New Territories comprise a small piece of the Chinese mainland north of Kowloon and more than 200 small islands, the largest of which is Lantau. The population is concentrated in a number of distinct geographical locations dictated largely by the terrain. High concentrations of building development and population exist along the north side of Hong Kong Island, over the entire Kowloon peninsula, and in the new towns of Shatin, Tsuen Wan, Tuen Mun, Taipo and Fanling in the New Territories. The other islands, including Lantau, are relatively undeveloped and sparsely populated.

The natural terrain of the majority of the Territory is very hilly, with natural slopes commonly exceeding 30°. The average annual rainfall is 2 225 mm, nearly 80% of which falls between May and September. Rainfall intensities can be high, with a 24-hour rainfall of 200 mm and a one-hour rainfall in excess of 50 mm occurring fairly frequently.

The geology of Hong Kong, which is summarised in Figure 1, has been described by Ruxton (1960) and Allen & Stephens (1971). The main rock types are granite and acid volcanic rocks, which together cover the major portion of the Territory, and are by far the most important from an engineering point of view.

The rocks of Hong Kong are deeply weathered, with depths of decomposition up to 60 m in the granitic rocks and up to 20 m in the volcanic rocks. The completely decomposed granitic rock invariably contains large corestones. Colluvium, which is the debris from mass movement, carpets the lower slopes of most of the hillsides to a thickness of up to 30 m. It is sometimes in a loose state, with a high permeability, and frequently gives rise to perched water-table conditions. It is also prone to the formation of 'pipes' or 'tunnels' as a result of internal erosion, and these features can be of major significance to the hydrogeology of an area (Nash & Dale, 1984).

Hong Kong experiences severe slope stability problems, and large numbers of failures are not uncommon during intense rainstorms. This situation has been described by Lumb (1975, 1979) and, more recently, by Brand (1984, 1985). The densely populated Mid-levels area of Hong Kong Island has historically suffered a number of significant slope failures, one of the most tragic of which was the landslide at Po Shan (Vail, 1984).

Over a two-year period, a major geotechnical study of the Mid-levels area was carried out to assess the need for slope stabilisation measures (Geotechnical Control Office, 1982). One result of this study indicated that deep subsurface drainage measures would bring about a significant improvement to the stability of the natural hillside above Po Shan Road. This hillside rises at an angle of approximately 30° and consists of a surface layer of colluvium up to 12 m thick overlying a layer of completely decomposed volcanic rock up to 17 m thick which, in turn, is underlain by slightly decomposed volcanic rock (Figure 2). The hillside is heavily vegetated and many boulders can be seen lying on the ground surface.

Initial design calculations indicated that horizontal drains were likely to be a satisfactory means of lowering the groundwater levels, and that these would be by far the most economical means available. The use of these, however, would entail the installation of horizontal drains through very difficult ground conditions of far greater length than had previously been used in Hong Kong, or indeed in most countries of the world. For this reason, it was necessary for field trials to be carried out to assess the difficulties that would be encountered in the drilling and installation of these drains up to lengths of 70 m in colluvium and weathered rocks. In addition, a review was made of the experience with horizontal drains in several countries of the world by contacting various individuals and organisations who are well-known for their expertise in this field.

An account is given in this report of the trial installation of horizontal drains at Po Shan and of their short-term performance. Details are also given of the relevant information collected internationally about installation problems with and long-term performance of this type of drain. This document should prove useful to anyone involved in the design and installation of horizontal drains for the stabilisation of slopes anywhere in the world.

2. REVIEW OF AVAILABLE INFORMATION ON HORIZONTAL DRAINS

2.1 SOURCES OF INFORMATION

For this review, the following sources of information were used :

(a) published literature,

(b) information provided on request by individuals and organisations worldwide, and

(c) information provided on request by individuals, consulting engineers and contractors in Hong Kong about local horizontal drain installations.

The following individuals and organisations overseas provided information:

Dr. A.J. da Costa Nunes (Tecnosolo, Brazil); Prof. J.N. Hutchinson (Imperial College of Science and Technology, England); Prof. T.C. Kenney (University of Toronto, Canada); Dr. G. Pilot (Laboratoire Central des Ponts et Chaussées, France); Dr. D.L. Royster (Tennessee Department of Transportation, USA); Mr. D.D. Smith (California Department of Transportation, USA); Prof. Ch. Veder (Graz Technical University, Austria); Canterbury City Council, England; the Snowy Mountains Hydro-Electric Authority, Australia.

The following companies in Hong Kong provided information :

Binnie & Partners (Hong Kong); Brickell, Moss & Partners; Enpack (HK) Ltd; Fugro (Hong Kong) Ltd; Gammon (Hong Kong) Ltd; Harris & Sutherland (Far East); Intrusion-Prepakt (Far East) Ltd; Maunsell Geotechnical Services; Mott, Hay & Anderson Far East; P & T Civil Engineers Ltd; Scott Wilson Kirkpatrick & Partners; Ove Arup & Partners Hong Kong Ltd.

2.2 INTERNATIONAL EXPERIENCE WITH HORIZONTAL DRAINS

In this review, reference will be made to both published and unpublished information, and the sources of this will be cited in each case by the name of the 'author' or organisation followed by the date. Unpublished sources are included in the list of references (Section 6) along with published sources.

2.2.1 Use of Horizontal Drains in Australia

The only information available at present on the Australian use of horizontal drains relates to the construction of the Murray 2 power station in southeastern Australia, on the borders of Victoria and New South Wales. The following information on the scheme was kindly supplied to the Geotechnical Control Office (GCO) by the Snowy Mountains Hydro-Electric Authority (1983).

The power station, built for the Snowy Mountains Hydro-Electric Authority in the mid-1960's, is located in an excavation in decomposed granite up to 60 m in depth with an overall slope angle of 26.5°. The initial groundwater level was fairly near the ground surface, and stability analyses indicated that it was necessary to lower the groundwater level to ensure stability. A series of horizontal drains was designed to perform this operation.

The drains, which were installed by means of a special auger machine designed for horizontal drilling, were generally 30 m in length with every fourth drain being 60 m long. The holes were nominally 100 mm in diameter and inclined at 5° above the horizontal. No casing was used during drilling, but a perforated PVC liner was installed on completion of each hole. No filters were used to prevent clogging of the perforated PVC pipe. Some of the augered holes collapsed before the PVC liner could be installed, and in these instances a new hole was drilled nearby. No precautions were taken to ensure that the holes remained straight, and it has since been determined that many deviated drastically from their intended direction.

The final 3 m of the drain liner was left unperforated, and an attempt was made to seal the annulus between the pipe and the hole, in order to direct flow into the pipe, by using a flange/collar arrangement on this section of pipe. This arrangement was unsatisfactory, and eventually the annulus was sealed around the drain outlet by packing it with rags soaked in cement paste.

A basic pattern of drains was fixed at the design stage on approximately 15 m c/c horizontally and 12 m c/c vertically. Where strong seepages were noted during construction, extra drains were installed.

It was found during construction that the groundwater was located in confined cells, each separated by some vertical geological confining barrier. When drainage was provided to a particular cell, the cell quickly emptied. The groundwater level was monitored by a series of piezometers around the site. From the observed water levels, the drains appear to have had a significant effect in lowering the groundwater level.

It was stated that many of the drains, which have now been installed for approximately seventeen years, have ceased to flow. These have been flushed out, but with no effect, and so it is assumed that the drains have not malfunctioned but that the water table has been drawn down to below the level of the drains. The plot of groundwater levels on the sections supplied by the Snowy Mountains Hydro-Electric Authority seems to confirm this.

The only maintenance undertaken on the installation is the removal of algae and weeds from around the outlets and the repacking of the annulus seal.

2.2.2 Use of Horizontal Drains in Austria

Veder & Lackner (1984) reported details of the remedial works to slope failures following the completion of highway cuttings for the 'Circumvention Grimmenstein' section of the 'Sudautobahn' freeway A2 in Austria. The original ground, which sloped at 18° and showed evidence of past failures, consisted of mica slate and highly decomposed gneiss. Two or three months after the cuttings were completed, during a period of heavy rain, failures took place. A detailed investigation was carried out which highlighted ground conditions where complicated geology gave rise to complex water tables within the slope. By a careful study of the water tables, it was possible to install horizontal drains, which consisted of 120 mm fabric wrapped PVC tubes, into areas where water pressures were high to successfully stabilise the slips. This case study is important, as it demonstrates that even slopes with complex groundwater regimes can be stabilised by the use of horizontal drains, provided that a careful investigation is carried out to identify areas of main water flow. The drains are then installed in these areas. The difference in cost between providing toe support with associated slope trimming and providing horizontal drains was of the order of 8:1 (Veder & Lackner, 1984).

A second case study presented by Veder & Lackner (1985) concerned the stabilisation of a bridge abutment. The abutment was constructed on a valley side which had a 20° inclination. Soon after construction, the abutment began to move at the rate of about 10 mm per month.

The methods considered for stabilisation of the abutment were anchoring, the construction of a retaining wall downhill of the abutment, construction of a new abutment with deeper foundations, and the relieving of groundwater pressure by horizontal borings. The last method of stabilisation was chosen, as this was far less expensive than any of the alternatives considered. Two horizontal drains up to 70 m in length were drilled in the slope to pass beneath the abutment and intersect the zone between decomposed mica slate and the underlying sound mica. These drains were successful in relieving high water pressures which existed in this zone. Immediately after installation of the horizontal drains, movements of the bridge abutment ceased. Flows from the drains reduced with decrease in water pressure in the slope. This does not indicate that the drains were losing efficiency, merely that a reduction in the head causing flow occurred.

2.2.3 Use of Horizontal Drains in Brazil

Extensive use is made of horizontal drains for the lowering and control of groundwater levels in Brazil. However, the Authors are not aware of any papers, in English, which detail construction and performance of any of these drains.

Da Costa Nunes (1985) stated that sub-horizontal drains, installed in soil slopes which have continued to move after drain installation, are destroyed by the continual slope movement. This is due to shearing of the drains. Where complete stabilisation is achieved this problem does not occur.

He also reports that drain clogging has been found to be a problem where drains have been installed in soils with a high content of iron-salts. These drains were found to clog quickly due to the formation of iron-hydroxide gels in the drain liner perforations. He cited two cases where this problem had been experienced in the vertical and horizontal drainage systems of dams, these being the Refinaria Gabriel Pussos dam in Betim, Minas Gerais, and the Joâo Penido dam in Juiz de Fora, Minas Gerais.

Because of the occurrence of the above problems, da Costa Nunes commented that demands for maintenance of horizontal drains have been found to be very extensive.

2.2.4 Use of Horizontal Drains in France

Pilot (1983) confirmed that horizontal drains were frequently installed by the Laboratoire des Ponts et Chaussées, piezometers being installed to monitor drain performance. He confirmed that problems of drain clogging have been encountered due to siltation, root penetration through perforations and deposition of calcium.

It is reported by Pilot & Schluck (1969) that a drainage system based on horizontal drains, which was installed to stabilise a section of the Nancy-Metz motorway, was still performing effectively after 15 years,although a careful maintenance programme was required to ensure drain efficiency.

A technical note produced by Cartier & Virollet (1980) gives guidance for the monitoring and maintenance of sub-horizontal drains. It is recommended that pore pressures, drain flows and rainfall are monitored weekly through the first year to check the drain response and identify anomalies. Recommended maintenance to be carried out on drains which appear to be losing efficiency consists of hard brushing and flushing until clear water begins to flow from the drain. The addition of certain chemicals to the flushing water is also found to be beneficial. The technical note also recommends that, where clogged drains cannot be cleared, these must be replaced, and that a continued programme of drain monitoring should be undertaken to confirm drain performance.

Amar et al (1973) described a French case study for the stabilisation of an embankment which included the provision of a drainage tunnel with associated horizontal drains to remove water from the natural hillside. Although interesting, the paper does not unfortunately provide much detailed information on the performance of the drainage works.

2.2.5 Use of Horizontal Drains in Great Britain

Hutchinson (1983) stated that information on the measured performance, reliability and maintenance of horizontal drains was sadly lacking, this being confirmed by Kenney (1983). Hutchinson reported that he had embarked on a review exercise in 1976-77 when preparing his general report on slide stabilisation for the IAEG Prague Symposium (Hutchinson, 1977) and at that time could find very little information. Concerning the life of horizontal drains, he believes that unfiltered drains may fail rather rapidly, in say two to four years, whereas properly filtered drains should work satisfactorily for much longer periods. Hutchinson had no strong feelings on the need for a grouted impermeable invert to the drain, but thought that it should be an advantage as long as the water had 'a tendency to escape into the subadjacent soil'.

An early use of horizontal drains in Great Britain was at Cod Beck, Northumberland in 1955 for the stabilisation of a river bank. These drains ceased to function after six years, but no details of the site or why the drains failed are reported (Robinson, 1967).

Robinson (1967) gave details of a scheme in England where horizontal drains were used to stabilise an old landslip on the A660 near Otley in Wharfedale in 1964. Initially, two experimental drains were installed, one 33 m long and one 58 m long, each consisting of a 38 mm diameter galvanized iron pipe slotted all round and surrounded by a 114 mm diameter porous concrete filter. These drains were drilled with little difficulty through head deposits into shale using a Boyles BBS10 rig, water flush and conventional drill bits. The drains yielded 6.8 cu m/day initially, which increased to 9.1 cu m/day in wet periods. Because of the success of the two experimental drains, a more comprehensive scheme consisting of ten drains was designed. The 38 mm galvanized iron pipe was replaced by 32 mm PVC tube for the new drains. In fact, only eight drains were installed, as these gave a discharge of 45.5 cu m/day.

Robinson (1967) stated that the resistance to rotation of the 150 mm diameter casing in the horizontal boreholes was very great, and this caused mechanical trouble. In order to allow the full length of 60 m to be attained, it was necessary to use 100 mm diameter casing which necessitated discarding the filter around the pipe. The drain pipe was inserted through the drill rods, and by using an annular bit, the rods could be withdrawn over the drain, which was left in place. The omission of the filter surround allowed a smaller hole to be drilled, and this proved to be a solution to the problem of drilling at this site.

The drains at this site have shown a heavy encrustation of iron hydroxide around their outlets, which has precipitated from the iron-rich groundwater. If such a precipitate were to occur in the filter, it could cause a serious loss of efficiency with time, which could only be remedied by replacing the whole system. Robinson reported that, because trouble had been experienced in the USA with grass root growth in the pipe near the outfall, the last 6 m of pipe was left unperforated.

Details of a scheme to stabilise a large landslip in London clay at Beacon Hill were provided by Canterbury City Council (1985). The saucer-shaped slip, which has a diameter of 120 m and a maximum depth of 17 m, dates back to pre-1872, and continual movement occurred until the carrying out of remedial works in 1976. These included the sinking of four 4 m diameter caissons with a total of ninety horizontal drains radiating from them. The horizontal drains, which consisted of 100 mm diameter porous concrete pipes placed in pre-drilled holes, were up to 52 m in length and were designed to intersect the slip plane and pass 6 m beyond it.

Shortly after completion of the works, an average of 315 litres of water per week was being removed by the horizontal drains. In the winter following installation, the movement of the slip was 6 mm compared with 13 mm in the previous winter, despite double the amount of rainfall. In the winter of 1977/78, no movement of the slip was recorded. Because of the low permeability of the clay, a considerable time period was required for the works to lower overall pore water pressures and become fully effective, and this is reflected by the gradual slowing down of movement and the failure of piezometers to register reductions in pore water pressures in the months immediately following completion of the works.

2.2.6 Use of Horizontal Drains in the United States

It appears from the literature that more use is made of horizontal drains for the control and prevention of landslides in the USA than anywhere else in the world. Horizontal drains were first used there for the stabilisation of landslides in about 1939 (Smith & Stafford, 1957), although the method did not gain widespread acceptance until the early 1970's (Royster, 1977, 1980).

An early method of drain construction consisted of drilling a 50 mm diameter hole which was subsequently reamed out to 150 mm and cased with a 100 mm perforated metal pipe (Smith & Stafford, 1957). It is reported by Royster (1977, 1980) that, as drilling bit development proceeded, a 100 mm modified fishtail bit was used to do the drilling in one operation, and a 50 mm black pipe casing, perforated with three rows of 10 mm holes at 75 mm centres at the quarter points, was used instead of the 100 mm perforated metal pipe. He reported that over the years improvements in drilling machines, materials and procedures have made the use of such drains more attractive, the most significant changes being the use of PVC pipe as a liner for the drilled hole, heavy walled flush-coupled drill rods, expandable drag and roller bits and drilling machines capable of developing extremely high thrust and torque.

Royster (1977, 1980) outlined a frequently used installation procedure which involves the use of an expendable roller or drag bit which is attached to the drill stem with a slotted adapter. Drill stem is added in 3 m sections as the hole is advanced. Water pumped at the rate of about 2 litres/second is used to cool the bit and flush the cuttings from the hole. Once the required depth is reached, the bit is 'knocked off' by reversing the direction of rotation of the drill stem. Slotted PVC pipe is then inserted through the drill stem as it is extracted from the hole. Horizontal drains more than 300 m long have been drilled using this technique and equipment, and drains around 100 m in length are commonly drilled and cased during an eight hour shift (Smith, 1980).

Royster (1984) reported that the Tennessee Department of Transportation had installed several thousand feet of horizontal drains in the past ten to twelve years. The maximum length of these drains was 183 m, these being successfully installed in an embankment composed of highly weathered shale with some sand and silt material (Royster, 1980). The Tennessee Department of Transportation have found horizontal drains to be absolutely reliable and beneficial in most cases, provided that they are tailored to suit the site conditions. They have not developed a clean-out device or procedures for maintaining their installations, and they rarely undertake any monitoring apart from visually observing stability, as they do not have the time or the personnel to do this work.

Royster (1977, 1980) reported that the drilling machines used have been specially developed for the installation of horizontal drains, the machines used by the Tennessee Department of Transportation being capable of producing 3 000 Nm (2 200 lb ft) of torque at 150 rpm. The drill carriage, which has a 3.35 m stroke, has the capacity to apply up to 41 000 N (9 200 lb) of thrust to the drill bit. The thrust as well as the torque is reduced somewhat with depth due to side friction, and this may become especially important where there is considerable drift or hole deflection.

Royster (1977, 1980) described the PVC liner used by the Tennessee Department of Transportation, which has 38 mm inside diameter and has a two slot circumferential configuration with a 120° c/c separation. The number of slots per row ranges from 22 per 305 mm using 1.27 mm slots, to 42 per 305 mm for the 0.25 mm slot size. The outer 1.5 to 3.0 m of the liner is made solid, and the annular space between the liner and the wall of the hole is packed with bentonite or some other impervious material to a depth of 0.6 to 0.9 m to direct flow into the liner and to prevent erosion around it. The PVC pipe must be held in place as the drill string is withdrawn because friction and binding between the drill string and PVC pipe tend to pull out the PVC pipe.

Regarding the flushing medium, Royster (1977, 1980) reported that, although water has been the principal flushing and cooling agent used in horizontal drilling, air is being used more and more, either by itself or with water. Air flushing has been used successfully on some projects where the action of the return drill water caused the softer more friable materials to erode severely. This erosion produced deep channels beneath the drill stem, and as pressure was applied to the drill bit, the drill string would flex into the eroded channel and break. By changing to air as a flushing medium, this problem was overcome. Royster noted that it is important to ensure that cuttings are properly cleared and are not allowed to get trapped between the drill stem and the wall of the hole, otherwise severe torque problems may result. He stated that 'down-the-hole' percussive drilling has proved successful up to 91 m in rock formations that have relatively consistent hardness and are not badly fractured or jointed.

The Mississippi Highway Department's experience of using horizontal drains demonstrates that they can be successful, but they caution that there are areas in which the technique needs to be improved, particularly in horizontal and vertical control (Ruff, 1980). In one of their projects near Redwood, the Department installed 68 drains of average length 94.5 m, with a maximum length of 128 m, in a saturated stratum of medium-dense sand containing gravel and clay seams (Ruff, 1980).

Royster (1980) emphasized that the success of a drilling project depends a great deal on the skill, technique and creativity of the driller, who must be able to decide on which flushing medium to use, the type of bit required and when to adjust the spindle speed and thrust. He also highlighted the importance of making specifications for the installation of horizontal drains rigid enough to ensure that the project objectives are met and yet not be so rigid that they severely penalise the contractor for not completing all holes. The many variables in the subsurface make horizontal drilling extremely speculative and very risky,and there will be times when holes simply cannot be completed to the specified or desired depths.

Smith (1984) reported that the California Department of Transportation had installed horizontal drains which had functioned satisfactorily for well over 40 years. He recommended a high pressure water system for cleaning the drains, the water pressure being between 11 000 kPa and 21 000 kPa with a minimum flow of 2.3 litres/second. He confirmed that there is a problem of water flow in the annulus surrounding the drain liner and suggested that, when this occurs, the annulus can be grouted by inserting an inflatable rubber packer to the desired depth and then grouting with a tremie tube. Solid PVC or steel casing should be used for the grouted length, so that grout does not enter the casing.

Smith (1984) provided useful details of 'drop off bits' which are used when drilling in unstable ground. These bits are simple adaptions of standard drilling items. Firstly, a standard drill bit adaptor is modified by machining a 'J'-shaped slot in the end. A tricone roller bit, which has a sleeve and pin welded over the threaded portion of the bit, is then inserted into the adaptor and locked into place by rotating the bit counter clockwise so that the pin engages into the tail portion of the 'J' slot (Figure 3). The boring is then made using this device, and the bit is 'dropped off' at the end of the hole by simply rotating the drill rod a quarter turn counter-clockwise and applying a low water pressure.

Smith (1984) also described techniques for overcoming problems caused by sand flowing into the borehole when new lengths of drill rod are connected, and also during subsequent drain liner installation. One technique used by the California Department of Transportation is to weld a 25 mm threaded pipe nipple to the thread end of the rock roller bit and then attach a one-way valve (Figure 4). This eliminates a rush of water and sand back through the bit and into the drill rod when adding new lengths of rod. This nipple and valve arrangement can be attached to a conventional bit as well as a 'drop off' bit.

To avoid jamming of the drain liner in the drill rods, a special 'nut' section with an interior gland and rubber 'O' ring is inserted between the first section of drill rod and the bit adaptor. As the drain liner is pushed through the snug fitting rubber ring, it acts as a wiper and prevents most of the sand and water from entering the drill rod. The drain liner must have an end cap and be inserted through the 'O' ring 'nut' section before the 'drop off' bit with the one-way valve is released.

Details of several case studies are reported by Smith (1980). In 1941, at Cloverdale near San Francisco, 97 horizontal drains were installed as part of the corrective measures to a large landslide. These drains, which were in loose broken shale, consisted of 50 mm perforated steel casings placed in 100 mm diameter predrilled holes of average length 16.7 m.

Fifteen years after installation, appreciable quantities of water appeared in various places along the toe of the cut slope. Upon investigation, it was found that the drains had ceased to function properly because of heavy deposits of rust, gypsum and root growth. A drain-cleaning operation restored the function of some of the drains, and more drains were installed over a period of years. In 1974, 147 drains were examined, and 40% had flows. Many of the original drains, which were now 33 years old, were still functioning. However, most of the steel casings were severely rusted, and root growth had clogged many of the drains.

At Nojoqui Grade in California, an embankment failure occurred in 1940. As part of the remedial works, 42 horizontal drains were installed below the reconstructed embankment toe in the saturated foundation area, which consisted of soft poorly bedded claystone and siltstone. These drains, which ranged in length from 22 to 58 m, consisted of 50 mm perforated steel casings inside 100 mm diameter predrilled holes. The first record of any drain cleaning was in December 1962, about 21 years after installation. Of the original 42 drains, only 27 could be found, and most of these had heavy accumulations of roots, rust and silt. No appreciable increase in flow occurred after cleaning, but this was done at the end of a hot dry summer. In late 1974, a road-widening scheme led to the destruction of most of the original drains. Some of those which were uncovered during the course of the work were still functioning, although the casings were mostly rusted through. Those drains that had exposed outlets had large accumulations of rust and algae on the ground below the casing. Thirty-two new drains, which ranged in length from 46 to 137 m, were installed during the road widening in 1975. An average of 84 m of drain was drilled and cased in an eight hour shift. On further inspection in 1978, the remaining original drains, which were now 38 years old, were so badly rusted that any disturbance by cleaning would have totally destroyed them.

The first installation in California to utilise both PVC and steel casing was in November 1969, at Pacific House, in the Sierra Nevada Mountains. Ten horizontal drains, ranging in length from 47.5 to 64.9 m, were installed in saturated clayey soil and decomposed coarse grained granitic debris. Four of the drains consisted of 38 mm PVC casing, while the remainder consisted of conventional 50 mm perforated steel casing.

In November 1978, the site was inspected, and wet spots were found around the drains. Two of the drains showed more water coming from around the casing than through it. Heavy root growth from willows had plugged most of the drains. The 10 year old PVC casing was in excellent condition but the steel casing had started to rust, although it would probably have lasted a further 20 to 30 years.

Smith (1980) found that dense growth of water-seeking plants around the drain outlets tends not only to conceal the drains but also to curtail their performance by extensive root growth within the first 3 to 6 m of the drain opening. He therefore suggested that solid pipe should be used for the outer 6 m of each drain to discourage roots from entering the drains.

Smith (1980) considered that the long-term performance of horizontal drains is also a function of the pH and mineral content of the groundwater. High acidity (low pH), or the presence of corrosive elements commonly found in fault zones or highly mineralised areas, may significantly shorten the life of drains, particularly when steel casing has been used. Groundwater highly charged with calcium or iron may also reduce the performance of the drain by plugging the slots or perforations. He suggests that the lithologic characteristics of the formations in which drains are placed are a definite factor in their long-term performance. An installation located in moderately hard broken rock will usually have a long life because of a minimal amount of silt and clay size particles that can gradually build up around the outside of the drain and block the passage of groundwater. Conversely, drains placed in fine sands or silts may have a shorter life because of an abundance of fines. These may require more frequent cleaning or replacement.

Smith (1980) considered it important that detailed drain cleaning and maintenance records are kept for each drain, but perhaps the single most important factor in the long-term performance of horizontal drains is a well developed and well executed programme of inspection, repair and cleaning.

Smith (1980) concluded that a 40-year life span is about the maximum that can be expected for drains that have steel casings, and perhaps 30 years is a more practical limit. By means of an effective maintenance and cleaning programme, this 40-year life may be extended if PVC pipes are used.

2.3 HONG KONG EXPERIENCE WITH HORIZONTAL DRAINS

Horizontal drains have been used to lower water tables in man-made slopes and behind retaining walls in Hong Kong since about 1973. The drains usually installed are very short compared with those used in the USA and seldom exceed 20 m, although drains 50 m long have been installed at Ngau Chi Wan (PWD Contract 411/75), and 33 m long drains have been installed in the cut slopes at Clearwater Bay Road (PWD Contract 535/74). These are the longest known installations in Hong Kong.

It has been recommended by Tong & Maher (1975) that a grouted impermeable invert is provided to a drain in order to prevent the flow of water between the drain liner and the hole. They argued that, if such flow occurs, it is liable to infiltrate back into the ground nearer the slope surface, which would have a detrimental effect on slope stability.

The written replies received from companies in Hong Kong in response to GCO enquiries indicate that differences of opinion exist on the suitability of horizontal drains for use as a long-term slope stabilisation method, and on the need to provide a grouted impermeable invert to a horizontal drain. Most companies had made little use of long horizontal drains (over 25 m), experience being limited to drains of 20 m or less. Whereas some firms considered that a grouted invert or annulus to the drain was necessary to prevent water from re-entering the ground nearer the slope face, others thought a solid invert to the drain liner was sufficient for this purpose. Some considered that a grouted invert was desirable but questioned the practicality of forming this under site conditions.

Although horizontal drain schemes have been installed to lower or prevent rises in groundwater tables in slopes, very little monitoring has been undertaken to determine their effectiveness. Where limited monitoring has been undertaken, the results are often inconclusive because of lack of data either before or after installation.

Sufficient information is available for some installations to indicate that they are successfully controlling water level rises (Clearwater Bay Road; the remedial works to the 1972 Po Shan failure; slope behind 14-16 Po Shan Road; Grenville House, May Road), although it is not known to what extent they have reduced the water levels. However, other installations appear to be ineffective, with evidence of seepages from the slopes, walls or weepholes but no flow occurring from the drains themselves (Pearl Gardens, Conduit Road; wall behind Po Shan Mansions; 4 -

6 Po Shan Road; Evelyn Court, Cloudview Road).

Two rows of horizontal drains up to 10 m in length and spaced at approximately 5 m centres were installed in a large cut slope on Tsing Yi Island, as seepages were observed after construction of the slope and piezometers indicated a high ground water level. This measure was intended to lower and control the groundwater level, thus increasing the factor of safety. As the slope subsequently failed, it could be concluded that the drains did not adequately control water levels.

As stated earlier, the longest drains known to have been installed in Hong Kong were the 50 m drains at Ngau Chi Wan. These drains, which are provided with impermeable inverts, were installed in 1978. Although some of these drains are producing water, and the flow increases due to rainfall, limited piezometric monitoring before and after installation indicates that no drawdown of the water table due to the installation of the drains has occurred.

Cases of actual drain installations were cited in some of the written replies, for instance horizontal drains installed between 1976 and 1978 on the Tuen Mun Road at locations of observed seepage, and on the Tsing Yi SW projects around 1979. No monitoring of these installations was undertaken apart from a visual check on flows. As significant flows occurred for some time, the drains were considered successful. A site inspection by GCO in December 1983 revealed that most of the drains on the Tuen Mun Road, which appear to be in rock rather than soil, are still flowing. Much vegetation was growing around the drain positions but this had not caused any blockage to the drains themselves. Large differences in flow between individual drains were noted, some discharging considerable quantities of water, whilst others were producing a mere dribble (Plates 1 & 2).

Another example cited in the correspondence was that of drains installed in a large cut slope below No. 1 May Road. It was stated that these were still flowing nine years after installation, although no comparison has been made of the quantity of flow now compared with that produced shortly after installation. A site inspection was carried out in December 1983, when it was noted that these drains were provided with an impermeable invert. Of the three rows of drains installed, flow was evident from the bottom row only. Of the 71 drains installed in the bottom row, 55 had no flow, four were wet and twelve had significant flows.

It is interesting to note that, in a recent contract for the installation of approximately 2000 m of horizontal drain for the Hong Kong Housing Authority, grouted impermeable inverts were specified. These drains were up to 18 m in length. Under the contract, the successful contractor was required to carry out a full-scale trial installation of the grouted drain, which was subsequently exhumed for inspection. The trial drain, which was 10 m in length, was installed near the top of an existing platform. Unfortunately, because of its location, no groundwater was encountered during the installation. The invert of the drain was grouted in three stages. Upon exhumation, it was found that a good impermeable invert had in fact been formed by the procedure which was developed by the contractor (Plates 3 to 10). A serious shortcoming of this trial was the fact that no groundwater was encountered. Strong groundwater flow may seriously affect the grouting procedure, washing the grout from the drain. In conversation, the contractor stated that he would be confident of this procedure for drains up to 25 m in length.

Binnie & Partners (Hong Kong) (1983a) reported that a series of 10 m long horizontal drains installed at Ko Chiu Road in N.E. Kowloon are flowing but have not produced any marked reduction in water level. In contrast, they reported that temporary drains installed for the Housing Department at Tin Wan in Aberdeen had a dramatic effect on limiting the rise in groundwater level, reducing the rise from 2 m/hour to 0.2 m/hour (Binnie & Partners (Hong Kong), 1983b). Despite this, uncertainties over the long term performance and maintenance obligations resulted in Binnie & Partners (Hong Kong) recommending a permanent drainage scheme which did not rely on horizontal drains.

Ove Arup & Partners Hong Kong Ltd (1983) advised that a drainage tunnel option at Hong Kong Gardens was chosen over a horizontal drain scheme as it was thought that this was far more likely to succeed in reducing the water table to the required level than a horizontal drain scheme. Maintenance problems were also likely to be encountered on a horizontal drain scheme, and it was felt that it would be difficult to ensure that preventive maintenance was in fact carried out on a privately owned lot.

2.4 NOTES ON SOME AVAILABLE DESIGN METHODS

Very little of the literature examined gives any reliable basis for the determination of drain spacing and drain row separation, existing design methods suffering from certain limitations. For example, the two-dimensional approach of the kind used by Choi (1974) treats the drain as a continuous slot, whereas three-dimensional analysis considers the drain as a line sink (Choi, 1977, 1983; Nonveiller, 1981). Theoretically, a drain modelled as a line sink cannot flow because it has an infinitesimally small periphery. However, in each case, element or grid size has caused the models to behave as a drain with a finite radius, and hence results have been obtained.

Choi (1977) compared his finite element model with a sand model and found that the two were in agreement. Work by the Authors suggests that Choi's effective drain radius was very large, i.e. 10 to 20% of the upstream wetted thickness above bedrock. This is consistent with the comparatively large drains installed in the model, i.e. 10 mm radius drains for a 150 mm upstream head.

Prellwitz (1978) presented a technique for estimating the maximum and minimum limits for drain spacing based on the theory for steady vertical infiltration into subsoil drains. However, it is rather unwieldly to use.

Kenney et al (1977) carried out model tests to determine the effects of drain length and spacing on groundwater pressures. These results were used in conjunction with stability calculations to produce design charts for a range of drain lengths and spacings to give approximately the desired improvement in slope stability. Their models were based on an 18° slope which was formed using glass beads. No attempts were made to evaluate the effects of varying drain radius. Because of the geometry employed, these models were not applicable to Hong Kong conditions.

It must be concluded that further work is required to develop theoretical techniques, and possibly produce design charts, for the design of horizontal drainage systems.

3. TRIAL DRAIN INSTALLATION

3.1 PROGRAMME

A site investigation was planned to obtain detailed information on ground conditions and hydrogeology for the design of a drainage scheme. The investigation confirmed colluvium deposits overlying completely decomposed volcanic rock which was underlain by slightly weathered volcanic rock. It became apparent from a costing exercise that a drainage scheme comprising horizontal drains would probably be the most economic solution. It was known, however, that it would be difficult to carry out horizontal drilling and drain installation in these ground conditions, and a variation to the contract was therefore incorporated to allow for the installation of a series of long horizontal drains as a trial. This work examined :

(a) the practical difficulties of drilling very long (over 70 m) holes using standard site investigation drilling rigs,

(b) the type of drain detail suitable for long horizontal drains,

(c) the practicalities of providing a grouted invert or a grouted annulus to the horizontal drains, and

(d) the short-term performance of such drains in dewatering the hillside at Po Shan.

Two sites were chosen as suitable for the installation of the trial drains, one adjacent to the Po Shan slip remedial works (Vail & Attewill, 1976) and one behind the 750 ft service reservoir (Figure 5). These sites were chosen because of the relative ease of access and because detailed piezometric data were available for each of them. It was originally envisaged that four drains should be installed at each site. Grouted impermeable inverts to the drains were to be provided at one location and not the other in order to establish the difference in performance of the two different types of drain.

After the completion of this work by Enpack (HK) Ltd, another contractor, Gammon (Hong Kong) Ltd., offered to drill an additional hole using the 'ODEX' system to evaluate the effectiveness of this method for drilling long sub-horizontal holes. This work was undertaken at their own expense.

3.2 DRILLING TRIALS

3.2.1 Drilling Trial by Enpack (HK) Ltd

All the holes were drilled using rotary coring rigs. With the exception of one hole, which was drilled with a Boyles BBS 15 machine, Craelius D900 drill rigs were used. The specifications of the drilling rigs are given in Table 1, and details of the casing and core barrels used are given in Table 2. Techniques of wash boring through soil combined with rotary coring through boulders and rock were used to advance the holes.

Drilling was largely carried out at low rotational speeds of approximately 60 rpm with carbide casing shoes, and up to approximately 200 rpm with impregnated diamond core bits. The diamond bits, which are expensive, were particularly prone to damage in the gravelly intercorestone material. The contractor therefore preferred to use the cheaper carbide casing shoes despite their short life of 5 m in grades III & IV rock and only 2 m in grade II rock.

Because of difficulties in drilling borehole H3, which resulted in the breaking of the casing, modified equipment was subsequently used. A casing shoe with every second carbide tooth offset outwards by about 2 mm was used to give extra clearance and hence cut down friction on the casing. In addition, it was found necessary to use a TNW surface-set diamond bit attached to BW casing to drill from 95 to 105 m in grades III to V rock in borehole H2. Both modified pieces of equipment were successful.

The drill rigs were supported on either wooden or steel pipe scaffolding. The thrust was transferred to the ground by grouting in 2 m and 4 m anchor pins in rock and soil respectively, the anchor holes being drilled by the machine using a TNW barrel at 45° to the vertical. By drilling the anchor holes on the same plane as the drain hole, transverse moments which tended to twist the machine out of alignment were avoided.

The contractor subsequently chose to tie the drill frame to the anchor pin directly, using a cable and tensioner. The use of a cable tie is not regarded as totally satisfactory, as it is too elastic, and much thrust ends up being absorbed by the drilling platform. A similar anchorage should have been drilled behind the rig so that pull-out forces could have been transferred more directly into the ground. Pullout was found to be the critical loading mechanism because of the jamming caused by gravel particles. Plate 11 illustrates a drilling rig in position, and Figure 6 shows how the various thrusts should be absorbed.

Wash boring was used to advance the holes. In this technique, flushing water passes down the centre of the casing, which is being continually rotated and surged, and returns up the outside of the casing, bringing the soil particles with it. When boulders or rock are encountered, the material is penetrated by drilling with a core barrel down the centre of the casing. The casing is then reamed into the hole formed by the core barrel.

The main problem in using this type of drilling occurred because of the variable nature of the ground being penetrated. The most difficult aspect was the gravelly and cohesionless nature of much of the material between boulders. This frequently caused hole collapse on withdrawal of casing and jamming of the core barrel in the casing, or casing within casing, or either core barrel or casing in boulders. This problem was accentuated after the water table was penetrated, as material tended to be washed into the hole because no hydrostatic head existed in the hole to support its walls. The narrow clearance between the casing and boulders would not allow these gravelly fragments to be flushed out without further grinding between the casing and boulder. Because of these problems,drilling was slow.

In the case of borehole H3, the HX casing broke at 31 m depth after having been drilled to 62 m. This failure was probably due to excessive friction between the casing and the ground, and possibly represents the limit for a single drive of HX casing in this material. After this failure, the contractor adopted the practice of telescoping casing to reduce friction. Typical casing depths were : PX - 10 m, HX - 45 m, NX - 80 m.

Because of the short life of the casing shoes, frequent withdrawal of the casing was required. As previously stated, collapse of the hole occurred when the casing was withdrawn to change the casing shoe, particularly when the hole was beneath the water table. The continual loss of material must have enlarged the hole and weakened the surrounding ground, as at times the holes were streaming continuously with sand-sized and gravel-sized particles for up to two hours after the casing was reamed back to the end of the hole.

The speed of drilling varied greatly because of the variety of materials being drilled. Grade V soils were most easily penetrated, and rates of up to 25 m per day were achieved. Boulders interspersed with soil and gravel gave the most difficult drilling conditions, reducing penetration to 1 to 2 m per day. The rates achieved are summarized in Figures 7 and 8, which show drilling progress in terms of operating days for the Po Shan side and Reservoir side drains respectively. The average rate in bouldery material was 3.5 m per day, as shown by the material drilled for holes H6, H7 and H8 on the Po Shan side and all the drilling in colluvium on the Reservoir side. Where wash boring could be used in less bouldery material, rates increased to an average of about 7.5 m per day.

The drilling accuracy is affected by two basic factors: correct initial rig alignment and drill string deviation. In the case of the holes drilled in the trial, initial alignment of the drill rigs was carefully carried out using a precision inclinometer to set the vertical angle and survey marks for horizontal alignment. A borehole survey was carried out using an Eastman Whipstock type RMSS single shot photographic survey tool. This uses a camera to photograph a magnetic compass and dipmeter while the tool is at rest in the hole.

Tables 3 and 4 show borehole deviations from their designed trajectory at 70 m depth for horizontal and vertical deviations respectively. For horizontal deviations, the shallowest survey of the borehole is assumed to indicate drill alignment for boreholes H2, H3A, H4 and H5, whilst for boreholes H6, H7 and H8, the initial alignment was obtained by surveying drill rods placed in the hole.

All initial inclinations are assumed to be accurate. As can be seen from Table 3, initial misalignments were as important as actual borehole deviation in causing horizontal inaccuracy. If the error in drill rig alignment and that due to rod deviation had been cumulative, a lateral deviation of more than 11 m could have occurred in borehole H3A. No consistent horizontal drift direction occurred, and an examination of all survey records suggests that the main deviation occurred in the first 15 m of drilling. Vertical deviations similarly showed no pattern, the maximum being 4.0 m for borehole H3A at 70 m depth.

3.2.2 Drilling Trial by Gammon (Hong Kong) Ltd Using 'ODEX'

In December 1983, Gammon (Hong Kong) Ltd. undertook a drilling trial using the Atlas Copco 'ODEX' drilling system. In this method, the casing is advanced using a down-the-hole hammer with an eccentric lobe under-reamer, which cuts a hole slightly larger than the casing. The lobe is designed to expand on drill string rotation and retract on reversal of direction of rotation for removal up the inside of the casing. The drilling rig used in the trial was an Acker MP MKIV, this being a large track-mounted rig producing 6 800 Nm torque and 7.2 tonne thrust.

The hole was drilled at a nominal 17° upslope angle from the sitting out area in front of the Po Shan slip remedial works (Figure 5). The drilling was carried out to 35 m using the ODEX 165 in conjunction with 7 inch (178 mm) casing and a COP 62 hammer. At this depth, rock was encountered. The hole was advanced beyond rock into soil without the use of casing for support, and subsequently the HW drill rod snapped at 37.7m. Following unsuccessful attempts to retrieve the hammer, ODEX 115 was used to advance the hole past this obstruction. Shortly after drilling commenced, rock was again encountered, and the contractor decided to continue drilling without the use of casing. The hole was advanced through rock and soil to a depth of 70 m with a 115 mm bit on the COP 42 hammer. The hole was subsequently surveyed, and a $1\frac{1}{2}$ inch (38 mm) galvanized pipe drain liner installed.

The decision to change from ODEX 115 to an open hole was taken because of excessive wear on the reamer lobe and slow drilling progress. Rates of penetration of 30 m per day for soil and 10 m per day for rock and soil were achieved using this technique. Hole deflection using ODEX was minimal, but the hole climbed 0.8 degrees in 15 m of open-hole drilling. From the collar to 52 m depth, a change in lateral direction of 2.0 degrees occurred.

3.2.3 Experience Gained from Drilling Trials

While the conventional rotary drilling rigs presently used in Hong Kong are capable of drilling horizontally to 70 or 80 m, this must be regarded as the limit of their range in mixed boulder and soil material, because of slow drilling and difficulty in casing withdrawal. The problem of hole collapse on repeated casing withdrawal is potentially serious. Because of this problem, it is considered that an alternative drilling method should be adopted; for example, by use of an expanding drill bit which can be removed through the casing for replacement. To obtain a sufficiently high penetration rate in hard rock, a down-the-hole hammer would probably be required. The Gammon drilling trial indicated that excessive wear of the under-reamer could be anticipated when using ODEX for horizontal drilling. This problem can probably be overcome by using stabilizers on the drill rod to support it within the casing.

Because of the difficult nature of the ground, any drilling system is likely to encounter problems which can only be overcome by modification of equipment and techniques during drilling. The drilling rigs suitable for drilling long horizontal drains require high thrust and pullout capacities. The work of Enpack indicates that a pullout greater than six tonnes is required to overcome friction on casing withdrawal. In addition, the torque should be greater than 5 500 Nm. A twin-chuck system which enables the bit to be advanced independently of the casing is desirable. This facility would allow the casing to be rotated and would facilitate the use of expanding bits. Such a system enables a maximum number of drilling options. Directional control is not likely to be successfully or repeatedly achieved in mixed boulders and soils. Some means of surveying the hole is essential to identify borehole deviation away from the water table which would explain why some holes remain dry while others flow continually.

3.3 INSTALLATION OF DRAIN LINERS

3.3.1 Summary

During the installation of the trial horizontal drains, problems arose which were not anticipated, and some predicted problems did not occur. At the beginning of the trial, the main concern was the provision of a horizontal drain which would not clog and would prevent the loss of fines through its openings. This did not prove to be a problem in the short term, but PVC drain liners were found to be of inadequate strength. In order to install 70 m drains under the conditions prevailing at the site, it was found necessary to use drain liners made of a stronger material.

The original requirement of providing a drain with openings only along the top lost priority, as screens were eventually placed totally below the water table, the major part of the drain liner being unperforated. An adequate drain detail was finally achieved after much experimentation.

Originally, the type of drain considered as ideal had a perforated upper portion and an impervious invert, which ensured that all draining water flowed in the drain and could not infiltrate into the soil closer to the slope surface. As a horizontal drain liner is necessarily smaller than the hole in which it is placed, an annular void will exist between the external wall of the drain liner and the formed hole. If the lower half of this annulus is left ungrouted and the soil surrounding the hole does not collapse onto the drain liner, water intercepted by the drainage hole will be conducted along this void regardless of whether or not the invert of the drain liner is solid and will infiltrate into the slope nearer the slope surface. However, the American literature, which is the most comprehensive on horizontal drain installations, does not mention the use of grouted impermeable drain inverts. One can only assume that these have never been used or considered in the USA, although perforations are confined to the tops of the pipes. In fact, the only papers demonstrating the need for, and detailing the construction of, such an invert were found in Hong Kong publications (Choi, 1974; Tong & Maher, 1975). Herbert (1982) of the Institute of Geological Sciences, England, confirmed that the efficiency of horizontal drains in dewatering that part of the slope below the drain is dependant on making the drain invert impermeable. However, he questioned the practicality of doing this and suggested that it would be easier to make the drain liner unperforated above the water table and then fully grout this portion.

The attraction of the patented 'Enpack drain' is that its installation includes a grouted invert. Because of the limited experience with this type of drain, trials were carried out at Enpack's depot to demonstrate the effectiveness of the grouting technique. The technique was considered complicated but met with limited success.

It was decided to install an Enpack drain at one location. During the installation in borehole H5, the procedures required to form the impermeable invert were demonstrated to be complicated, and doubt was expressed over the likely success of forming such an invert under site conditions. The difficulties were further aggravated by the fact that water was flowing from the hole fairly rapidly during installation. This particular installation is discussed in more detail later. Because of the installation difficulties experienced with the 'Enpack drain' and the limited benefit of such a drain in intercepting infiltrating groundwater, this design was abandoned.

An alternative type of drain was used which took the form of a screen installed below the water table at the end of an unperforated section of pipe. Because of significant water flow around the borehole/drain annulus, attempts were made to grout the annulus up to the screen. Such a drain can only lower the water table into which the screen is placed but, if successfully grouted, does prevent the water from seeping back into the slope nearer the slope face (Figure 9).

It was originally envisaged that the choice of screen type would be a critical factor in drain design if a loss of fines through the drain openings was to be avoided. The grading curves obtained for samples taken from vertical boreholes at Po Shan showed a wide even soil grading for all completely decomposed volcanic rock, including inter-corestone material. The fine material led to the adoption of a composite filter of sand and Terram 1000 fabric for the pumping well used for a pumping test carried out at Po Shan. Because the placement of such a filter in a horizontal drain is extremely difficult, it was decided to install drains with a filter formed of Terram 1000 alone. This was fixed to a 50 mm diameter PVC pipe, which was perforated with five rows of 7.5 mm diameter holes, using a contact adhesive (Plate 12). This type of drain liner was used for horizontal drains H3A, H4 and H5. No loss of fines through these holes was noted.

Because of concern over the long-term performance of the filter fabric with respect to clogging, alternative designs of screen were used in the other holes. The first of these was made from 50 mm nominal diameter (ND) PVC pipe with a wall thickness of 4.5 mm. This was slotted with 1 mm wide slots at 12 mm centres extending to the diameter (Plate 13). The slotting reduced the rigidity of this pipe to such an extent that these drain liners broke during installation, and none could be successfully installed. A close copy of this type of screen was manufactured from 40 mm ND galvanized iron pipe. The slots in this were formed by sawing with a hack-saw, and were approximately 1.5 mm wide (Plate 14). Drain liners of this type were used in drainage holes H8 and H6, the latter installation being re-galvanized electrolytically after slotting. These screens showed clear water flow within six hours of installation, indicating that an adequate filter pack had formed over the slots.

In the final drainage hole, H7, a commercially available stainless steel wedge wire well screen of 0.5 mm slot width was installed. This screen was of 50 mm outside diameter, and was used so that a corrosion resistant screen could be evaluated over a long period, and the supposed benefits of a wedge wire configuration in resisting clogging could be assessed. During installation, this screen was found to be significantly less rigid than the slotted galvanized iron pipe.

3.3.2 Details of Individual Drain Installations

This section is included because the problems encountered and lessons learnt may be of value to others attempting to install horizontal drains. The drain installation sequence adjacent to the Po Shan slide remedial works is shown in Figure 10. Figure 11 shows the drain positions in relation to existing piezometers, and Figure 12 presents a section showing horizontal drain positions.

(1) Horizontal Drain H3A. This borehole behaved as expected during drilling but was not typical. The hole was drilled quickly, and fell 4° from its original inclination of 10° upwards. Because of this, it penetrated the main water table without extended drilling, the hole being completed at 70 m in grade II rock.

The drain liner installed used 24 m of Terram covered screen of the type shown in Plate 12. This was placed easily within the NX casing and the casing withdrawn. A mercury tilt switch was used to orientate the drain liner so that the solid invert was placed downwards. The switch was fitted into a PVC coupler, and this was placed at mid-screen length. The switch assembly, which can be seen in Figure 13, works by the make/break action of a mercury droplet moving within a glass capsule and bridging the gap between two contacts. The device was orientated on the screen so that the on/off action occurred with the drain in the correct orientation.

Following installation, the drain showed an estimated leakage from around the drain annulus of 20 to 30% of the total flow, which reduced in two days to a minor seepage. Approximately one month after installation, the annular seepage increased to half of the total flow and then abruptly stopped. Annular flow increased again after tropical storm Joe (11 to 14 October 1983) to 20% of total flow. The leakage from around the drain appeared to have been controlled by collapse and subsequent washing away of the fallen material. This flow increased with rainfall.

(2) Horizontal Drain H4. This drain was drilled to 77 m and completed in corestone/soil material. The borehole rose from an initial inclination of 10° to 12.5° at 41 m before falling back to 11.3° at the borehole end. This resulted in the drain being 2.16 m above the projected borehole position, but this was higher than that anticipated based on the falling trajectory of borehole H3A. The result was that the borehole did not penetrate the water table and therefore did not produce any water even after casing withdrawal.

The borehole was screened in a similar manner to H3A, but the annulus between the unperforated pipe and borehole was grouted with 0.25 cu m of grout, an amount which was insufficient to grout the theoretical volume of the annulus. Because the borehole was inclined upwards, grouting was designed to take place from the drill rig end of the borehole, up the hole towards a return pipe. The annulus between the hole and the inserted drain liner was hand-packed to a depth of 4 m from the drain outlet using fist-size balls of bentonite/cement mixture wrapped in plastic flyscreen mesh. The outer 0.5 m was sealed with mortar. This is shown in Figure 14. In addition to the provision of a mercury tilt switch, the PVC pipes were marked on assembly to show correct alignment. It was found that the manual markings were sufficient to obtain correct orientation provided that the drain was not binding in the hole.

(3) Horizontal Drain H5. This drain hole was drilled to a depth of 80.3 m at a 10° inclination. It is estimated that it intercepted the perched and main water tables at about 40 m and 71 m respectively.

The drain installation was of the patented Enpack type mentioned earlier. This consists of a PVC pipe containing perforations in the upper half which are covered with filter fabric. The pipe also contains slots along the invert and on either side along the diameter. After placing in the hole, the lower half of the annulus between the drain liner and the hole is grouted. This is achieved by pushing a twin hose grout pipe up the drain liner and pumping in a two-part reactive grout mix. This is intended to flow through the slots in the drain invert and fill the lower half of the annulus between the drain liner and the hole. After the annulus is filled to the mid-height of the drain liner, excess grout flows into the drain liner through the slots along its diameter. On removal of the grouting rods, the rods and grout mixing head are intended to clean excess grout from the invert of the PVC drain liner.

A number of problems were encountered during drain installation. Firstly, the drain was fitted with small pairs of 'wheels', formed of small bearings, which were intended to make the drain liner self-orientating in the borehole. These 'wheels' added to the diameter of the drain liner and caused it to be a moderately tight fit in the NX casing. It is uncertain whether this was the cause, but it was very difficult to hold the drain liner in the borehole while winching out the casing. The drain liner had to be repeatedly pushed back into the borehole as the casing was withdrawn.

When subsequently the borehole was surveyed by passing the survey tool up the drain liner, the drain was found to be blocked at a depth of 42 m. It was concluded that the drain liner had broken and allowed soil to fall into it. A compression failure of the liner may have occurred due to the large force required to hold it in position and force it up the borehole during casing withdrawal. Alternatively, a tensile break may have occurred during casing withdrawal. Soil may have collapsed onto the drain liner as the casing was withdrawn, holding it in place. Some of the soil may have been washed into the casing by the flowing water and jammed the drain liner inside the casing. On continued casing withdrawal, the drain would be stretched and may have broken.

Despite the failure of the drain liner, grouting of the drain invert was carried out on the first 42 m of the drain in order to ascertain its likely success under site conditions. Whilst it was not proven whether the drain invert was successfully grouted or not, the complicated procedure made it a highly suspect operation under the difficult site conditions. Also, it is quite possible that a large percentage of the grout will be washed out of any drain which has a large water flow during the grouting operation. For these reasons, this type of drain was abandoned in favour of an alternative design.

The borehole, which had been producing large quantities of water through and around the casing, only produced a trickle after grouting. This reduction in flow may have been the result of the collapse, or the grouting technique may have grouted the complete annulus between the drain liner and the hole, or a combination of the two.

(4) Horizontal Drain H2. Borehole H2 was drilled in stages. Because of the failure of the borehole to produce water at 82 m depth, a borehole survey was carried out. This indicated that the borehole had risen from an initial vertical inclination of 6° to 10°. At this inclination, it was anticipated that the hole would reach the water table at 92 m. Because the drill rig had reached the practical limit for NX casing, it was decided to try and drill further, forming an uncased hole by the use of 30 m of BX casing fitted with a TNW surface-set diamond bit to act as a single-tube core barrel. This technique was used to drill to 105 m. Some water began to flow from the hole after it reached a depth of 95 m.

Several attempts were then made to install a drain liner. The drain used was complex. It had initially 16 m of slotted PVC screen, followed by a packer made from a motorcycle inner tube which was inflated through a 6.3 mm OD nylon tube. The screen had fitted to its outside 4.8 mm OD nylon tubes to act as piezometers and designed to monitor screen loss changes. The entire unit was designed to be grouted up to directly behind the packer where excess grout would pass into the main drain. On initial installation, the drain was placed to 92 m depth and could be pushed no deeper. However, on casing withdrawal, sand and gravel particles jammed the drain liner in the casing, so that it was pulled out with the casing. Following this, the hole was re-reamed to 83 m with NX casing and the borehole probed to 95 m depth with 13 mm diameter GI pipe. The drain liner could not now be pushed up the casing because of a stream of soil particles which were being washed down the casing. To overcome this problem, the end cap was cut off the drain liner, and it was pushed up the hole while flushing soil particles down the drain with water supplied from a GI pipe pushed up the drain centre. This had to be repeated, as once again the drain liner was pulled out on casing withdrawal. Eventually, the drain liner was successfully installed to a depth of 82 m.

The annulus was grouted until grout returned into the drain, but no attempt was made to use the packer, as it had received such abuse that it was considered unlikely to be intact. Also, the use of a packer which requires a low inflation pressure is difficult if water rather than air is used for inflation, as it is necessary to compensate for elevation changes between the packer and the drain outlet when determining inflation pressures. Lastly, because of borehole collapse and undoubtedly enlarged areas, the idea of using a packer at all was abandoned.

Because of the difficulty in intersecting the shallow dipping water table at a reasonable depth, further drilling on the Reservoir side was abandoned.

The installation of this drain emphasized several problems, namely :

(a) the lack of directional control,

(b) the need for regular borehole survey during drilling,

(c) the extreme difficulty of installing a drain liner inside casing whilst soil particles are being washed down it, and

(d) from (c), the need to socket the end of the casing into rock to prevent soil inflow.

(5) Horizontal Drain H8. Borehole H8 was drilled on the Po Shan side at 6° inclination to a depth of 70.2 m, the hole being cased to a depth of 67.8 m. The drain liner first installed comprised 12 m of slotted 50 mm ND PVC screen, the remainder being unperforated pipe. On first installation, the drain liner became jammed by soil particles at only 16 m penetration and could not be pushed further up the hole. The pipe had to be broken out with the casing and subsequently recoupled. The casing was redrilled and the drain reinstalled after soil particles had stopped flowing down the casing. On withdrawal of the casing, the drain liner again jammed in the casing, and attempts to hold it in place led to drain failure in compression. Following this, the casing was removed with the drain liner inside and the casing reamed back to 70 m depth.

Because of the repeated problems of drain liner jamming and breakage, it was decided to increase the clearance between the drain and casing and to strengthen the drain liner by using galvanized iron pipe of 40 mm ND, which was slotted as previously described. A galvanized iron drain liner, comprising 12 m of screen and the remainder unperforated pipe, was installed without problems. The drain was grouted in place with 0.42 cu m of cement grout with a w/c ratio of 0.55. The theoretical grout volume to the grout return was 0.40 cu m, but no return of grout occurred. During grouting, a jetting head on the end of a 13 mm diameter GI pipe was placed inside the drain so that, if any grout overflow onto the screen occurred, it could be flushed out by jetting water through this pipe. It was in fact not required in this case, as the grout did not reach the screen.

(6) Horizontal Drain H6. This borehole was drilled to 70 m at a 6° inclination. Because the small Boyles BBS15 drill rig used had extreme difficulty drilling this length, it was decided not to continue drilling into fresh rock.

A drain liner comprising 15 m of slotted galvanized screen, which had been regalvanized after slotting, and 55 m of unperforated GI pipe, was installed without difficulty. Despite pumping 0.54 cu m of cement grout into a void of theoretical volume 0.33 cu m to the end of the grout return pipe, no grout reached the return pipe, and blowing up the grout return showed no resistance.

(7) Horizontal Drain H7. Borehore H7 was similar to boreholes H6 and H8. Instead of a galvanized slotted iron screen, a wedge wire stainless steel screen of 12 m length was installed. Because of the problems of grout failing to reach grout returns at 40 to 45 m depth on previous holes, three grout returns at 15, 30 and 45 m were placed in the hole. After pumping in 0.43 cu m of grout, the 15 m grout return showed some return flow and then became blocked. The theoretical hole volume at this depth was only 0.13 cu m. No further grout returns were noted, despite the fact that a total of 1.18 cu m of grout was pumped into the hole. However, as a fissure or geological pipe in the ground 2.7 m above

borehole H6 began to exude grout, grouting was suspended. The cement grout used had a w/c ratio of 0.5 and had retarder added.

It is interesting to note that, on withdrawal of the casing from borehole H7 during drain installation, the flow from the adjacent horizontal drain, H6, ceased but began to flow again later. This may have indicated a connection between the boreholes and an increase in screen loss on borehole H7, which led to the flow building up again in H6.

(8) Drain Installation in Gammon Trial Borehole. A 40 mm diameter galvanized iron drain liner consisting of 12 m of slotted screen, the rest being unperforated pipe, was installed to a depth of 55 m in the 70 m long hole. The drain liner could not be advanced beyond this point because of hole collapse. The annulus between the drain liner and the hole was sealed at the slope surface with hand-rammed bentonite/cement balls to a depth of about 4 m.

Following heavy rains, it was found that approximately half of the total flow from the drain was occurring as leakage around the outside of the drain liner. This suggests that the soil surrounding the drain liner had not collapsed onto the drain liner. This is hardly suprising considering that most of the drilling beneath the water table was in rock.

3.3.3 Conclusions and Recommendations on Drain Liner Installations

The installation of the drain liners during this trial led to the following conclusions :

(a) The drain liners need to be robust to withstand the forces exerted during casing withdrawal. Galvanized iron or stainless steel pipe of 40 mm diameter is of satisfactory strength.

(b) It is extremely difficult to install a drain liner whilst any sand or gravel particles are being washed down the casing.

(c) Because of (b), it is desirable to socket the casing into bedrock so that no particles can flow into the casing.

(d) A clearance of 25 mm between the drain liner connectors and the casing is adequate to prevent the drain liner from jamming in the casing.

(e) Correct drain liner orientation can be achieved by careful assembly and marking.

(f) A slot width of 1.5 mm is adequate to prevent loss of fines in CDV material.

(g) The grouting of the pipe section up to the screen can be difficult for the following reasons :

(1) The Borehole may collapse and block the grout flow.

(2) Large voids may be present, both natural and as a result of drilling.

(3) If grout is pumped up the hole to a position beneath the water table, it will be washed either through or into the screen by groundwater flow, unless the drain is filled with water and sealed to prevent inflow. If the drain is sealed, problems can be expected with groundwater damaging the initial borehole collar seal.

(h) Failure to grout the annulus may lead to substantial water pressure build up behind the collar seal.

As a result of the above conclusions, the following recommendations can be made :

(a) For long drains in completely decomposed volcanic rock (CDV), galvanized iron, stainless steel or material of similar strength should be used.

(b) To prevent particle inflow into the casing being caused by groundwater flow during drain liner installation, the casing should be socketed into rock.

(c) A clearance of 25 mm between the casing and drain liner connectors should be maintained to prevent jamming of the drain liner within the casing.

(d) Slots of width 1 to 1.5 mm should be provided in the screen; these can adequately filter the CDV at Po Shan.

(e) The annulus between the drain liner and the hole should be grouted to prevent water flow down the annulus and hence recharge into the ground below the drain. A grout pipe perforated along its entire length and extending to the limit of the grouted zone should be provided to enable grouting to be carried out beyond collapsed zones. A screen jetting pipe should be placed in the drain prior to grouting, to enable the screen to be flushed if grout reaches the screen.

(f) If a decision is taken not to grout the annulus, then a short length of screen should be provided immediately behind the collar seal to prevent a build up of water pressure. The arrangement is shown in Figure 15.

4. PERFORMANCE OF TRIAL HORIZONTAL DRAINS

The performance of a horizontal drain installation is governed not only by the length and spacing of the drains but also by the hydrogeology of the slope into which the drains are placed. A good knowledge of the hydrogeology of the slope is therefore fundamental to the understanding of the performance of any horizontal drain scheme.

4.1 HYDROGEOLOGY OF THE HILLSIDE

4.1.1 General

The study of the hydrogeology of the hillside at Po Shan was directed specifically towards the construction of drainage measures to lower the main and any significant perched water tables. For the purpose of drainage design, the infiltration from the surface and bedrock may be combined as the inflow into the main aquifer. A simplified model of this is shown in Figure 16. The infiltration from surface and bedrock is particularly important, as it provides for all recharge into the system, and without this the system would drain and eventually dry up. This point is important in understanding the operation of a drainage system. Emphasis was therefore placed on determining the inflow into the system. To do this, a reliable estimate of the mass ground permeability was required. A pumping test was undertaken as part of the site investigation to obtain this information, a summary of the results being at Appendix B.

For an examination of the hydrogeology of the hillside reference should be made to Figures 17, 18, 19 and 20. These show respectively on 14 June 1983, contours of piezometric level, perched water tables, bedrock topography and wetted thickness above bedrock of the main water table. This date was chosen because it represents typical wet season conditions but is not immediately after a major storm, which would give rise to a high transient water level.

As can be seen from Figure 17, the water surface has a somewhat irregular downslope gradient and does not follow surface contours precisely. This indicates varying transmissivity and infiltration over the slope. Figure 18 shows that perched water tables exist at two locations on the slope. The perched water table adjacent to the Po Shan slip remedial works appears to be connected to the main water table. This is discussed in more detail later. Figure 19 indicates the irregularity in bedrock levels, and this leads to a very complex pattern of wetted thickness, as shown in Figure 20. A feature worthy of note is the thinning of the wetted thickness around boreholes PD20 and PD26. Figure 21 shows the flow per unit width in the main water table above bedrock on 14 June 1983. This is derived on the basis of the permeability value calculated from the pumping test results and data from piezometers and bedrock levels.

From an examination of the flow across sections X1 to X5 (Figures 18 and 21), a general increase in flow downslope between sections X1 and X4 can be observed. However, below section X4, the flow apparently decreases significantly on the basis of wetted thickness, if the permeability is considered to be unchanged. In fact, an increase in permeability or a return of flow to bedrock is the likely explanation. This phenomenon is indicated in Figure 22, from which the downslope flow at section X3 is approximately 110 cu m/day over a slope width of 161 m, i.e. 0.68 cu m/day per unit width.

The majority of piezometers installed in the Po Shan area have only been monitored since June 1983, and they were read only manually throughout the 1983 wet season. It appears from these readings, and those from a few piezometers installed during earlier studies, that storm responses are not as important as seasonal piezometric rises. Following a wet start to the year, the piezometric contours were high (Figures 17 and 18), and it is thought unlikely that water levels would rise beyond these by more than 1 m seasonally and 1.5 m due to storms of a 1 in 10 year return period.

4.1.2 Flow in the Main Water Table

From the measured piezometric levels within the main water table and the contoured bedrock profile, an analysis of flow direction and rate was carried out using the mean value of permeability derived from the pumping test. This was done by dividing the area into a series of square blocks on a 15 m grid spacing. The flow through the sides of each block was estimated using Darcy's law, corrections being made for slope geometry. These values were used to determine an average flow rate and direction through each block. A computer programme was written to carry out these calculations.

The accuracy of the flow estimates are thought to be low because of variations in permeability of the ground and flow through bedrock. It was not possible to take these factors into account in the analysis. The results are presented in Figure 21. This bears a close resemblance in form to Figure 20, which shows the wetted thickness above bedrock. Of particular significance is the variation of flow downslope between flow lines. It appears that flow through bedrock may account for this variation, which is particularly noticeable around boreholes PD20 and PD26 where the wetted thickness has been shown to decrease. However, the concept of flow through bedrock at this location is in conflict with the results from the pumping test, which indicate that the area in question behaves as an imperfect barrier. Some of this flow deficit in the area of PD20 and PD26 may be accounted for by flow into the perched water table which exists in this area. The major part, however, almost certainly results from variations in the ground permeability.

4.1.3 Perched Water Tables

The two areas indicated in Figure 18 had consistent perched water tables throughout the wet season. These areas are in the vicinity of boreholes PD33 and ML7 on the west side and boreholes PD14, P15, PD20 and ML6 on the eastern side. Perching apparently occurred at the base of the colluvium. In the case of the eastern side perched water table, it would seem that, when the main water table is high, the phreatic surface rises above the colluvium base, leading to a split in groundwater flow between the main and perched water tables. This additional flow in the perched water table helps, in part, to account for the deficit in the amount of water flowing in the main aquifer in this region, as shown in Figure 21.

The likelihood of such a connection between the perched and main water tables is reinforced by the fact that the main water table was lowered by horizontal drains H6 and H8 even though these only penetrated the perched water table. This effect was indicated by piezometers installed in boreholes PD15, PD16 and PD17.

4.1.4 Inflow into the Main Aquifer

Four methods were used to estimate the surface and bedrock infiltrations into the decomposed rock stratum. These were based on :

(a) examination of the net inflow or outflow from the grid elements of a mesh using measured water levels and bedrock contours (bedrock defined as grades II and III continuous rock),

(b) examination of the increase in groundwater flow downslope over the site,

(c) comparison of total groundwater flow to the area of infiltration upslope, and

(d) examination of the steady-state drawdown characteristics of the trial horizontal drains.

To enable an examination of variations in net inflow and outflow over the site to be carried out, a computer program was written. This program is based on a node centred grid. The net inflow per unit area into each grid element is equivalent to the flow term Q in the equation :

$$\frac{d}{dx}(Tx\frac{dh}{dx}) + \frac{d}{dy}(Ty\frac{dh}{dy}) = Sy\frac{dh}{dt} + Q \quad .....(1)$$

where T is the transmissivity, Sy is the Specific yield, and Q is the inflow per unit area

Provided that dh/dt is small, then Q is equal to the left-hand side terms in the equation.

On 14 June 1983, dh/dt was small, and therefore Sy dh/dt was also small in comparison with the left-hand side terms. However, no meaningful estimate of Q could be made because of the rapid variations in T on a 15 m grid base. This was due to the irregular bedrock surface combined with significant bedrock permeability, which meant that the Q values found represented local inflow/outflow from the bedrock.

To examine the increase in groundwater flow downslope over the site, reference is made to Figure 22. This represents the flow across sections X1 to X5 shown in Figure 21. Between sections X1 and X4, the trend is for an increase in downslope flow of 0.48 cu m/day per metre in plan, which is equivalent to an infiltration of 3×10^{-3} cu m/day per unit area.

However, the flow apparently decreases for section X5. Likely explanations for this apparent flow decrease are either an increase in permeability or a return of flow to bedrock.

For the comparison of total groundwater flow to the area of infiltration upslope, the flow quantity at section X3 has been estimated as 110 cu m/day (Figure 21). On the basis of a plan catchment area of 4.06×10^4 sq m this gives a value of infiltration of 2.71×10^{-3} cu m/day per unit area.

An examination of the steady-state drawdown characteristics of the trial horizontal drains was carried out by use of the theory of steady-state radial flow into a well for an area under constant infiltration. From this, it is possible to estimate the infiltration into the zone of influence surrounding a well group. In doing this, it is necessary to assume, and maintain, a small drawdown so that the principle of superposition may be used. This requirement arises because the hillslope aquifer is unconfined, and excessive drawdown will distort the main flow pattern, which is downslope flow, because the aquifer thickness will change.

Initially, a dry season case was examined. On 19 January 1984, the three discharging horizontal drains on the eastern side were closed and the head allowed to build up until 7 February 1984, after which the drains were opened. The head build up during this period showed a relatively minor rate of change towards the end of the period. The head build up caused by the drain closure is shown on Figure 23, along with an approximate 70 m radius zone of influence. With a permeability value from the pumping test of 0.172 m/day, an infiltration rate of 1.72×10^{-3} cu m/day per unit area perpendicular to the slope, or 1.9×10^{-3} cu m/day per unit area in plan, was calculated. The water balance value of 1.7×10^{-3} cu m/day corresponds to the flow before drain closure. A similar exercise was carried out using the drawdown occurring due to drain installation.

For comparison, the water levels on 28 June 1983 before drain installation and those on 29 September 1983 were chosen. These two periods followed storms which occurred on 17 June and 8 September respectively. Piezometric levels outside the zone of influence of the drains were approximately 0.2 m lower on 29 September than on 28 June, and so the difference in piezometric level within the zone of influence minus 0.2 m gives the actual drawdown due to the drains. The head reduction between these dates is presented in Figure 24. Allowing for the slight overall reduction in water level, the zone of influence was estimated as 82 m. This corresponds to an infiltration rate of 2.1×10^{-3} cu m/day per unit area over the period.

From the above work, an overall average infiltration rate into the main water table, during the 1983 wet season, of 2.4×10^{-3} cu m/day per unit area was obtained. A corresponding value for the dry season of 1.9×10^{-3} cu m/ day per unit area was calculated, both of these values being for projected plan area. These values are surprisingly close, indicating the importance of infiltration from bedrock. The mean precipitation over the period from 28 June 1983 to 29 September was 7.4×10^{-3} cu m/day per unit area, 32% of which infiltrated into the main water table.

4.1.5 Estimation of Aquifer Properties after Drain Installation

Apart from the pumping test carried out during the site investigation, tests were carried out on the trial horizontal drain installations to obtain values for permeability, storativity and effective drain diameter. A brief description of each test is as follows :

(a) After prolonged closure, the drain tap was opened to allow flow at a controlled rate for one hour, and the head decline within the drain was monitored together with adjacent piezometers. From these results, Jacob plots (Cooper & Jacob, 1946) were produced, from which values for permeability and storativity were obtained.

(b) The drain tap was closed, and the head build up in the drain was monitored. The results were analysed using Horner and American Petroleum Institute Methods (Dake, 1978) to determine permeability and effective drain diameter.

The equipment used in carrying out these tests is shown on Plate 15.

Because of the delayed yield behaviour of the aquifer, it was decided to restrict flow rates and the flow period to one hour in an attempt to prevent a change from confined to unconfined aquifer characteristics. This led to some problems in obtaining a good plot of increase in head with time after drain closure. However, good estimates of local transmissivity to the drains were obtained. A mean value of permeability of 0.78 m/day was calculated, which is 4.5 times that of the value obtained from the pumping test.

The poor plots of increase in head with time were used to obtain a value of effective drain diameter. The large value obtained for local permeability, together with an apparently large value for effective drain diameter, leads to the conclusion that significant loosening of the ground surrounding the drain was caused by drain installation.

4.2 EFFECTS OF TRIAL HORIZONTAL DRAINS ON GROUNDWATER REGIME

The performance of the horizontal drains is indicated reasonably well by Figure 24, which shows the reduction in water level between 28 June 1983 and 29 September 1983. As the difference in water level between these dates averaged 0.2 m outside the zone of influence of the drains, the apparent reduction in water level within the zone of influence should be reduced by 0.2 m to give a more accurate picture. As can be seen, the drawdown obtained is somewhat irregular.

The effect of the drains is also illustrated by Figure 23, which shows the increase in head after closure of the drains for a period of seventeen days. The head build up is much more even than the drawdown shown by Figure 24, as no rainfall occurred during this period. An elongation in the head build up perpendicular to the downslope direction can be seen. This is contrary to the drawdown pattern due to a single well or well group in a sloping aquifer predicted by Hantush (1963). He suggested that such a system should produce a drawdown pattern elongated in the downslope direction.

The effect on total drain flow of closing one drain was determined by closing the tap on the middle drain, H7, and measuring the total flow from the drain group. The head build up in drain H7 was also measured and found to increase 1.15 m above the screen height. An idealised head profile is presented in Figure 25. It is interesting to note that the total flow from the drains decreased only marginally from 19.86 cu m/day to 19.10 cu m/day after the closure of drain H7. The majority of flow from H7 was directed into drain H6. There is some evidence of hydraulic connection between horizontal drains H6 and H7, as witnessed during the installation of drain H7. When the casing was withdrawn from around drain H7, flow in drain H6 temporarily ceased.

Two other interesting facts emerged from the tests described in Section 4.1.5. Firstly, because a head build up in the drains due to closure of the taps was maintained without leakage from around the outside of the drain, the indications are that the adjacent soil had collapsed onto the drain liner. Also, because there was no increase in water level in the perched water table, it can be assumed that no transfer of water between aquifers occurred. One conclusion made is that, in this particular case, it was unnecessary to form a grouted impermeable invert or annulus because the surrounding soil had sealed the annulus by collapsing onto the drain liner. In this situation, the provision of a drain liner with an impermeable invert is all that is required.

The second point worthy of note concerns the filtering action of the screen. After the initial drain installation, the discharge cleared of fines very quickly, indicating that a natural filter had built up around the screen. The drains were subsequently closed for seventeen days, during which time a head build up of 5.5 m occurred above the screens. Upon reopening, a significant loss of fines occurred during the first 24 hours, during which time a new filter formed around the screen and the discharge subsequently cleared. Obviously a filter that had initially been satisfactory, failed under the large pressure differential that existed on drain reopening. It may be possible to use the technique of turning off the drains, allowing a head build up, and subsequently opening them, to flush drains that show a significant falling off in efficiency.

The flows from the drains are shown in Figures 26 and 27, together with associated rainfall. As may be expected, drain flow is influenced by rainfall. Flows from drains H6, H7 and H8 had reduced to approximately 14 cu m/day by October 20. Throughout the dry season, flows continued to decrease, and in February 1984, flows from drains H6, H7 and H8 were 3.5, 8.0 and 8.4 cu m/day respectively.

4.3 EFFECTS OF GROUNDWATER CHEMISTRY ON DRAIN PERFORMANCE

Groundwater samples were taken from the trial horizontal drains in January 1984 and sent to the Public Works Laboratories for analysis. The analysis report stated that the water samples contained very low concentrations of substances which may be precipitated from solution, but there was very little danger of the drainage slots being clogged provided that the water quality is maintained.

However, the water samples were slightly acidic, and the presence of dissolved carbon dioxide and chloride ions would be detrimental to zinc coatings on galvanized iron.

5. CONCLUSIONS AND RECOMMENDATIONS

The study has demonstrated that horizontal drains can be effective in reducing the water table at Po Shan. Drains H6, H7 and H8 lowered the water table by over 3.0 m at a distance of 10 m from the drain group. A scheme comprising several rows of drains can be expected to produce appreciably larger drawdowns than these three isolated drains.

The available methods for the design of horizontal drainage schemes contain certain simplifications which make their applicability suspect. Some design methods use two-dimensional analysis, which may be invalid unless the drain spacing is small. The three-dimensional design methods take no account of the effective hydraulic drain radius. In addition, any design method needs to take account of infiltration, as this will lead to a significant rise in groundwater levels downstream of any system of horizontal drains.

The main problems associated with any system of long horizontal drains will be those associated with the drilling of long enough holes and with the installation of the drain liners. The drilling methods adopted need to safeguard against hole collapse in order to avoid loosening the soil over large areas of the hillside. To ensure that the hole does not collapse, casing should not be withdrawn beneath the water table in soil until after the installation of the drain liner. To be able to comply with this requirement in holes greater than 70 m in length, 'down-the-hole' hammers and under-reaming bits are required. Any contractor undertaking such work must have a variety of such drilling options available, because difficulties are invariably encountered. Only by having flexibility to change between alternative drilling methods can these be overcome. The drilling machinery itself must be of high pullout capacity and torque, yet be light enough to be moved between holes easily. This can best be achieved by the use of feed beam drills with separate hydraulic power packs.

Drain liner installation poses problems below the water table. It is concluded from the trials that a substantial clearance between drain liner and casing must be maintained to prevent soil particles from jamming them together. Alternatively, a minimal clearance could perhaps be used in conjunction with some form of 'wiper', so that any gap is too small to allow soil particles to enter the annulus. The drain liner must be of sufficient strength to prevent breakage under the forces experienced during installation. Stainless steel is recommended as being the most suitable material, because it has high strength and is corrosion resistant.

There is little evidence to indicate that drain performance at Po Shan will deteriorate with time due to screen clogging. Chemical tests on samples of groundwater have been carried out, and these have not identified any compounds likely to clog the screen by precipitation. Also, the literature does not generally consider clogging to be a major problem, as long as the groundwater does not contain substances that will precipitate onto the screen. A regular programme of maintenance must be implemented to clear vegetation growth from around drain outlets to prevent this from reducing drain performance.

If siltation or fungal growth eventually leads to a reduction in drain performance due to partial clogging, then it may be possible to restore their original performance by flushing out the drains using the technique of closing the taps to build up a head and then opening them. It has been shown that, with screen apertures of 1.5 mm, the large head differential on opening the taps may be sufficient to destroy the original natural filter but, with head reduction, a new filter will form. Before this operation is carried out, it will be necessary to consider the effects of such a procedure on slope stability.

The long-term effectiveness of a drainage system can be verified only by monitoring the groundwater level reductions. This requires extensive piezometric monitoring, which is best achieved using automatic piezometer systems. The results of this early monitoring at Po Shan will verify the adequacy of the design or highlight areas where additional drains are required. Apart from piezometric monitoring, drain discharge will be monitored and relationships of flow versus piezometric head established. Once this has been done, monitoring can be reduced and occasional checks made to ensure that the flow versus head relationships have not deteriorated.

In summary, a drainage scheme comprising long horizontal drains is considered to be an economic and feasible solution for improving the stability of the natural hillside at Po Shan. Problems may arise during drain installation, but these can be minimized by careful selection of equipment and method of operation. Fewer problems will be encountered if the installation is carried out in the dry season when the groundwater levels are low.

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TABLES

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Table 1 - Basic Specification of Drill Rigs Used by Enpack (HK) Ltd

Drill Rig	Power Plant	Rated Capacity with HW Rods (Vertical)	Maximum Thrust (tonne)	Maximum Pullout (tonne)
Craelius D900	Diesel 29.5 kW	325 m	4.4	5.9
Boyles BBS15	Diesel 19.4 kW	183 m	N/A	N/A

Table 2 - Details of Casing and Core Barrels Used by Enpack (HK) Ltd

Casing Type	Casing OD (mm)	Core Barrel	Barrel OD (mm)	Depth Range (m)
PX	140	No core barrel		0-14
HX	114	T2-101	101	0-62
NX	89	TNW	73	39-95
BX	73	No core barrel		45-105

Table 3 - Borehole Deviations in Horizontal Plane at 70 m Depth

Borehole	Initial Angular Error	Deviation (m) at 70 m due to Initial Angular Error	Drill String Lateral Deflect-ion (m) at 70 m	Total Borehole Deviation (m) (+ ve Indicates RH Direction)
H2	0°28'	0.57	-0.44	0.13
H3A	5°28'	6.67	-4.57	2.10
H4	-1°02'	-1.26	0.62	0.64
H5	-2°30'	-3.05	3.67	0.64
H6	0°59'	1.19	Not measured	
H7	-0°45'	-0.93	Not measured	
H8	1°28'	1.80	Not measured	

Table 4 - Borehole Deivations in Vertical Plane at 70 m Depth

Borehole	Vertical Deviation (m) at 70 m Depth (+ ve = Upwards)
H2	3.54
H3A	-4.00
H4	2.00
H5	-0.36
H6	1.27
H7	-1.02

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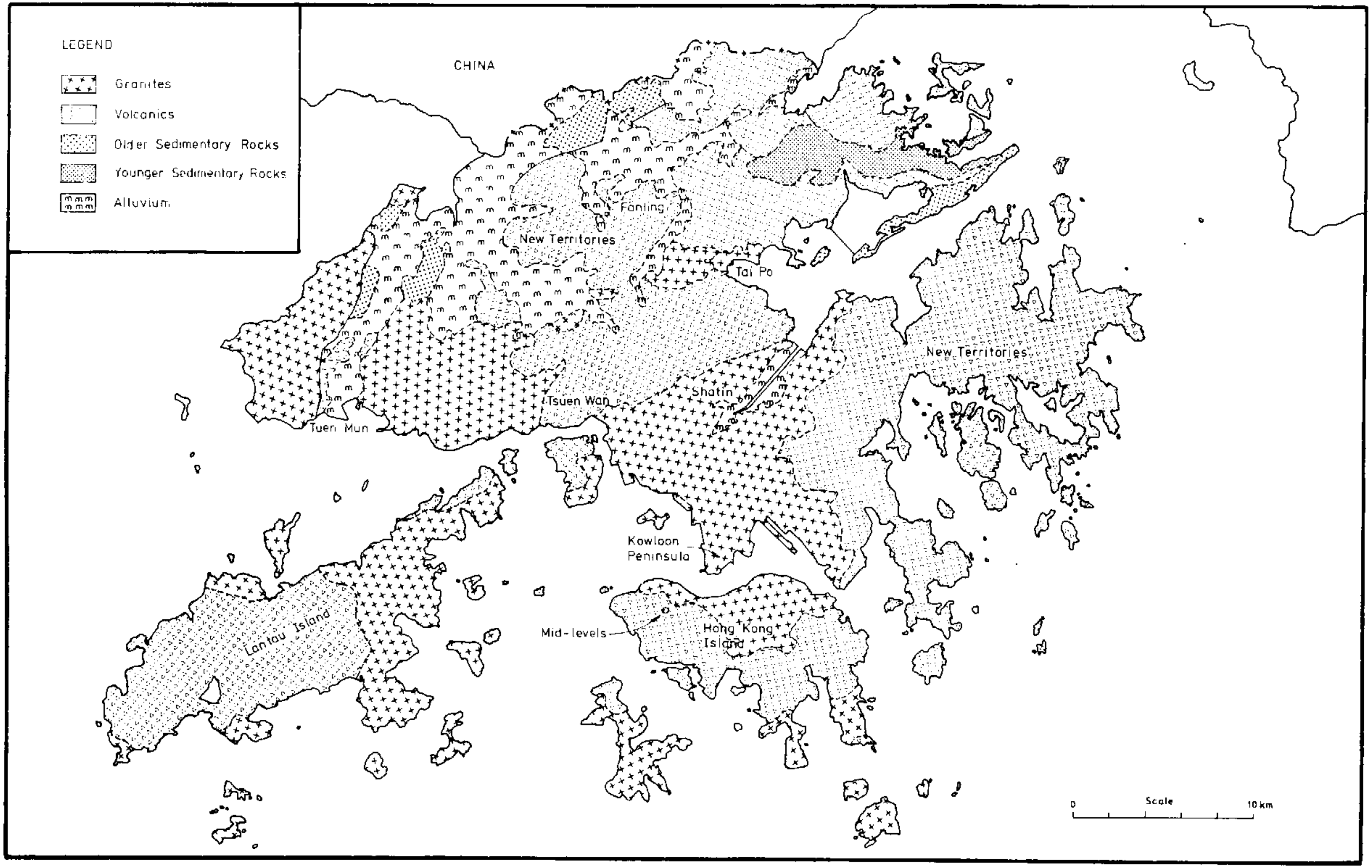
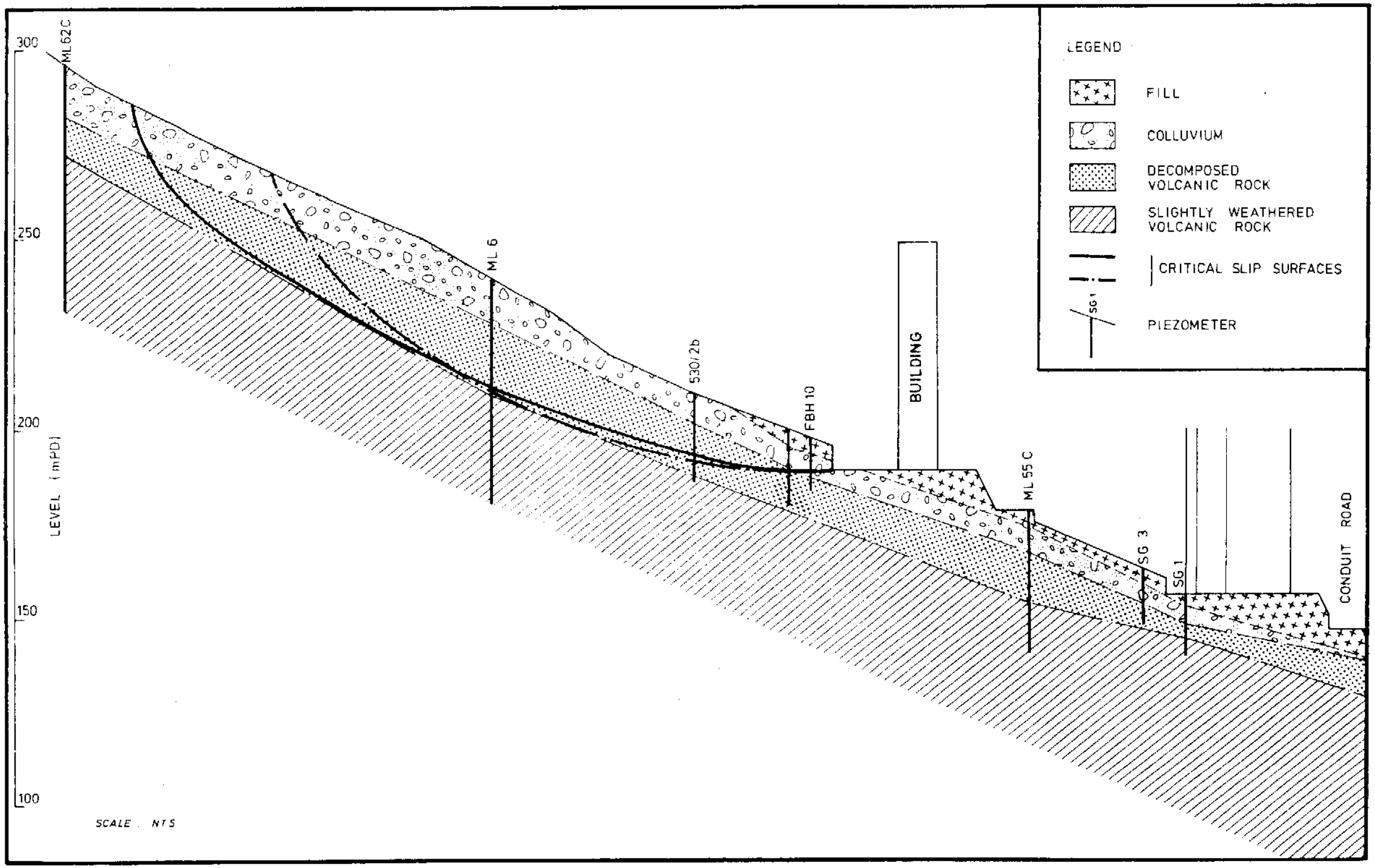


Figure 1 - Geology of Hong Kong



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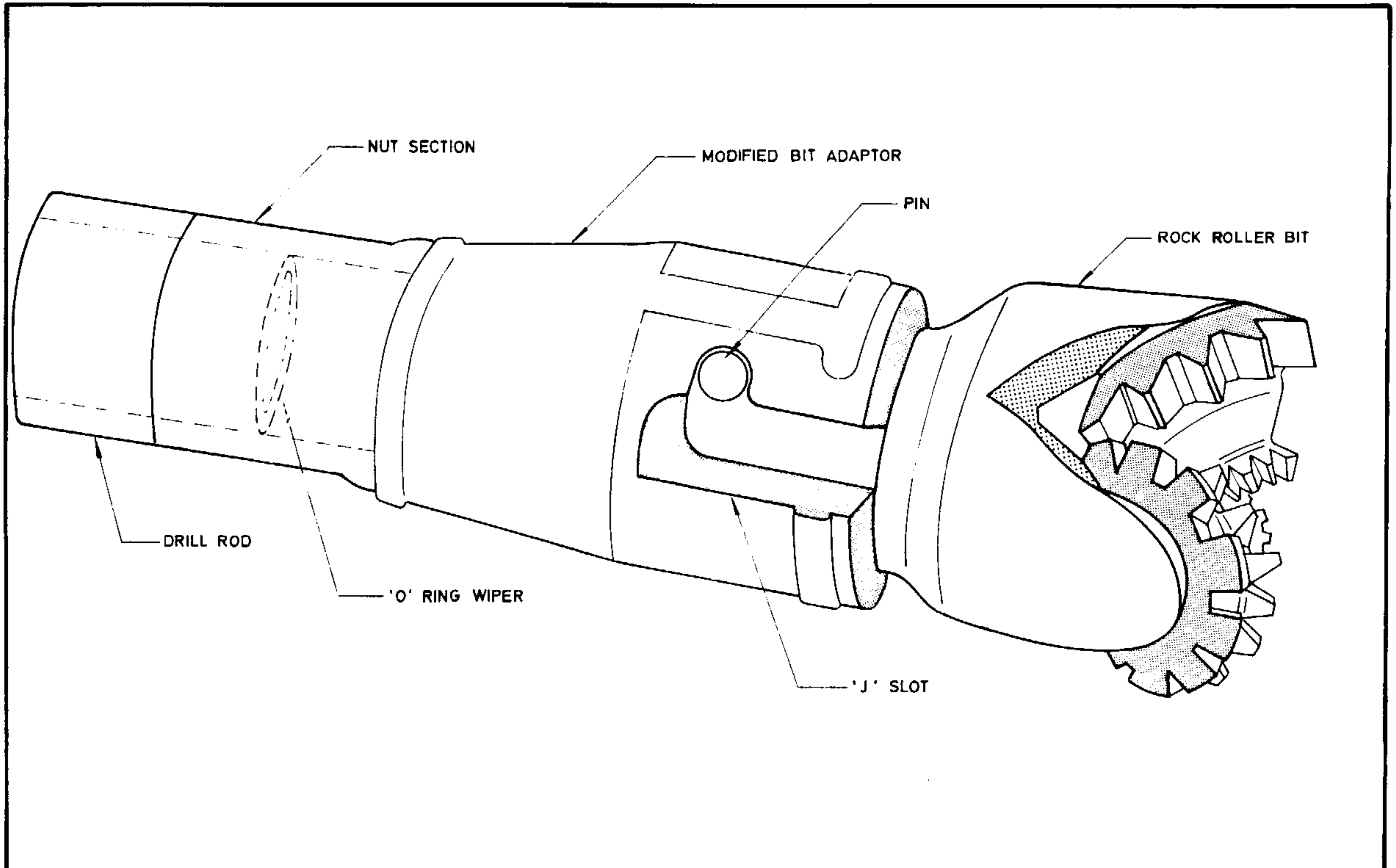

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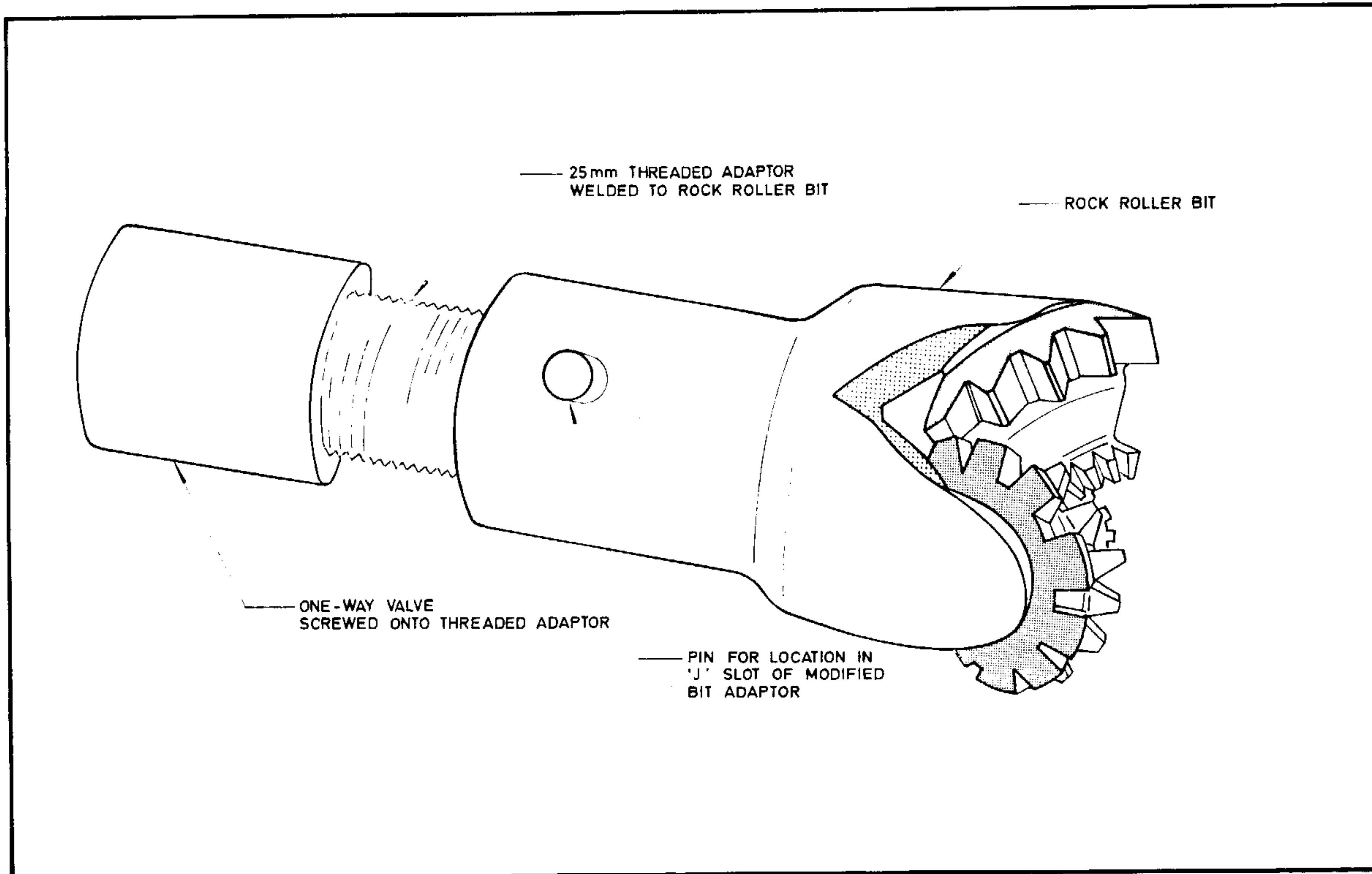

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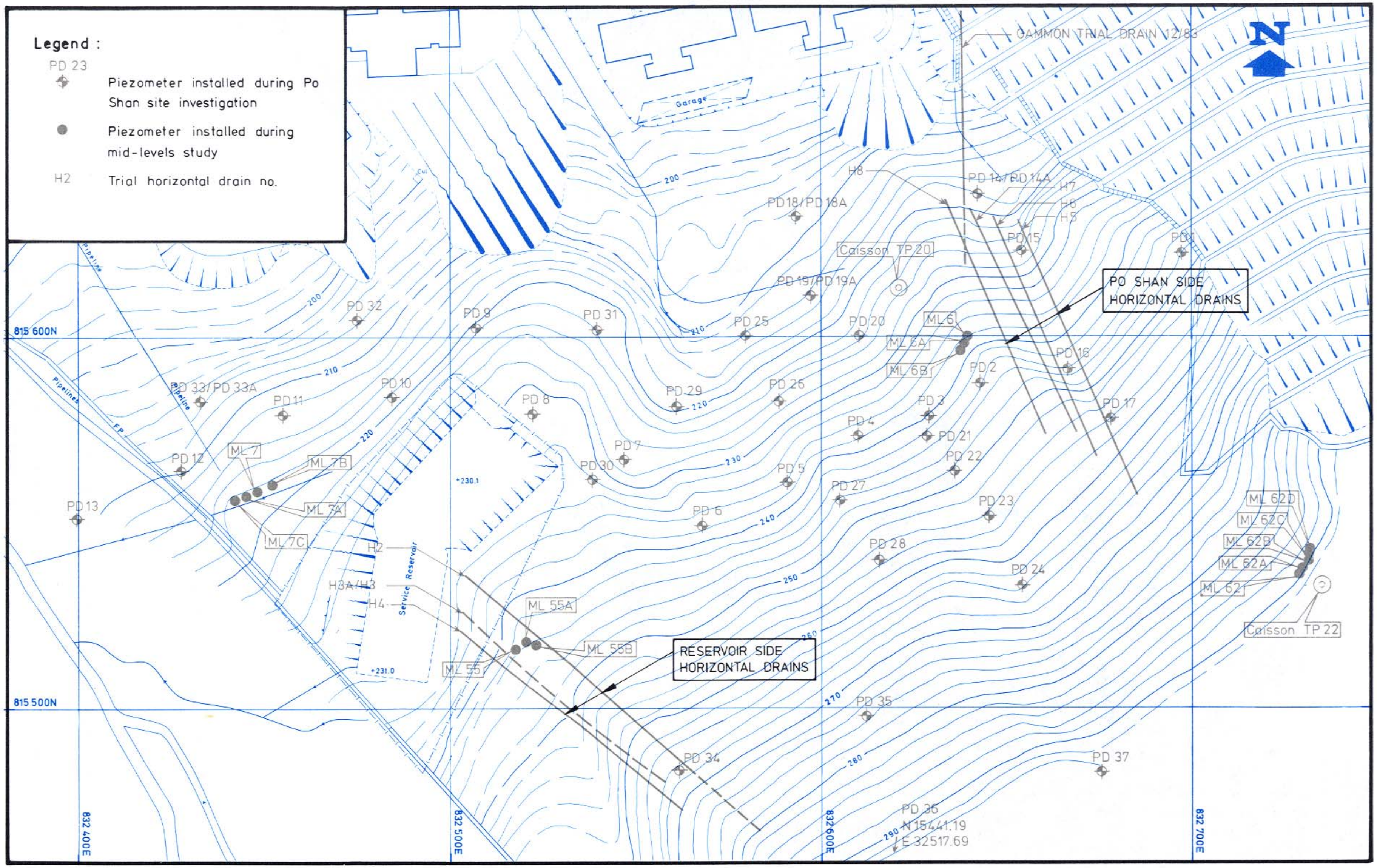

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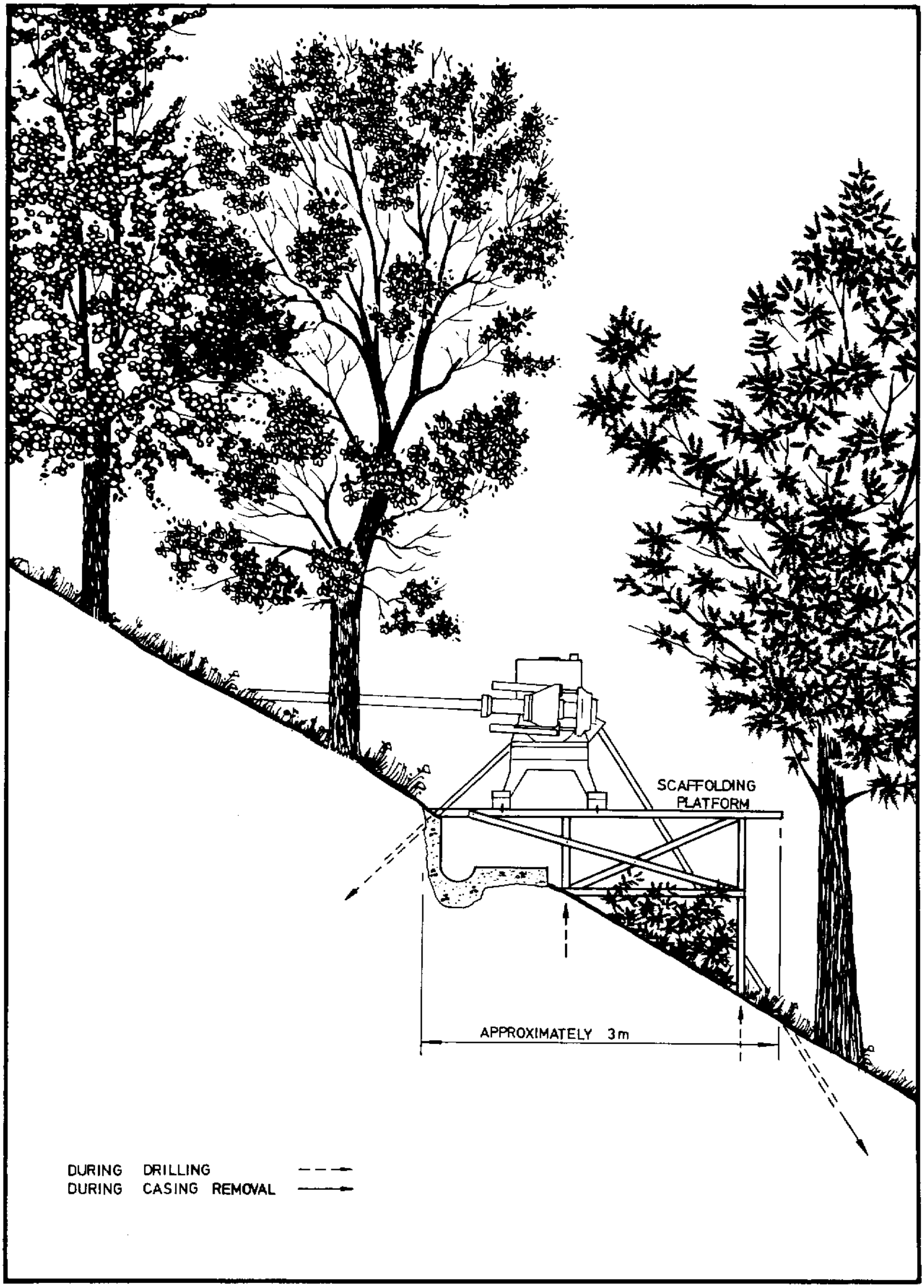

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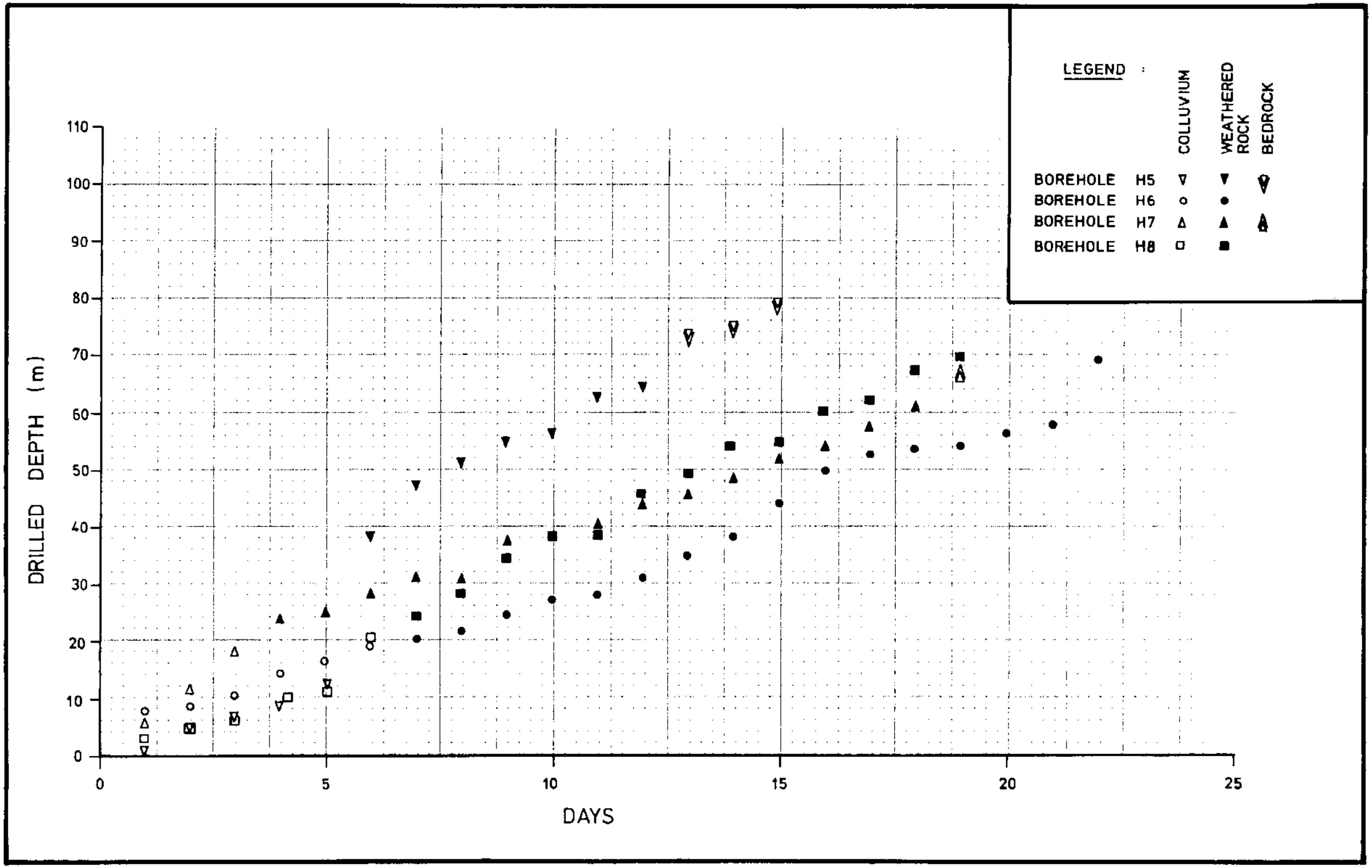


Figure 7 - Drilling Progress for Horizontal Drains - Po Shan Side

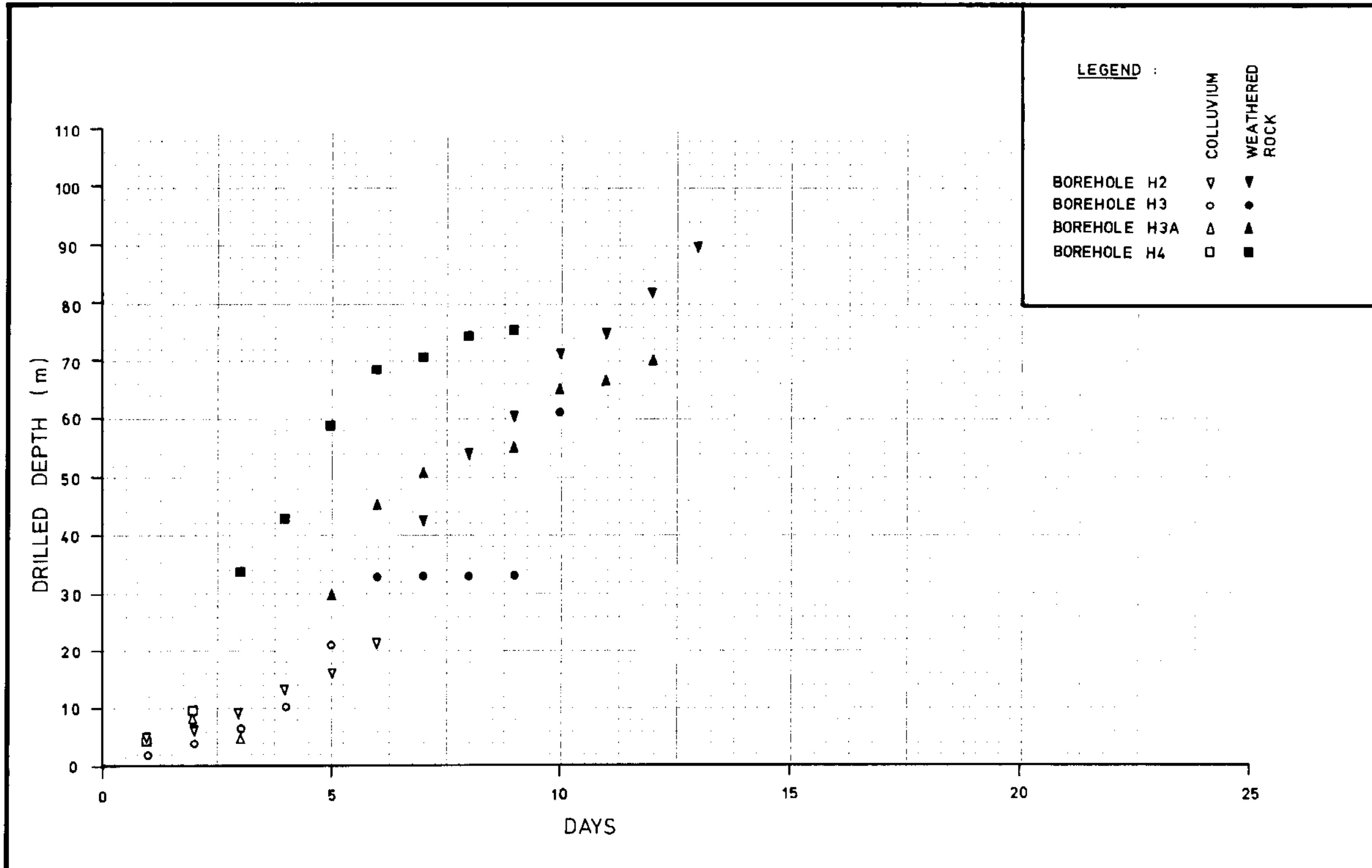
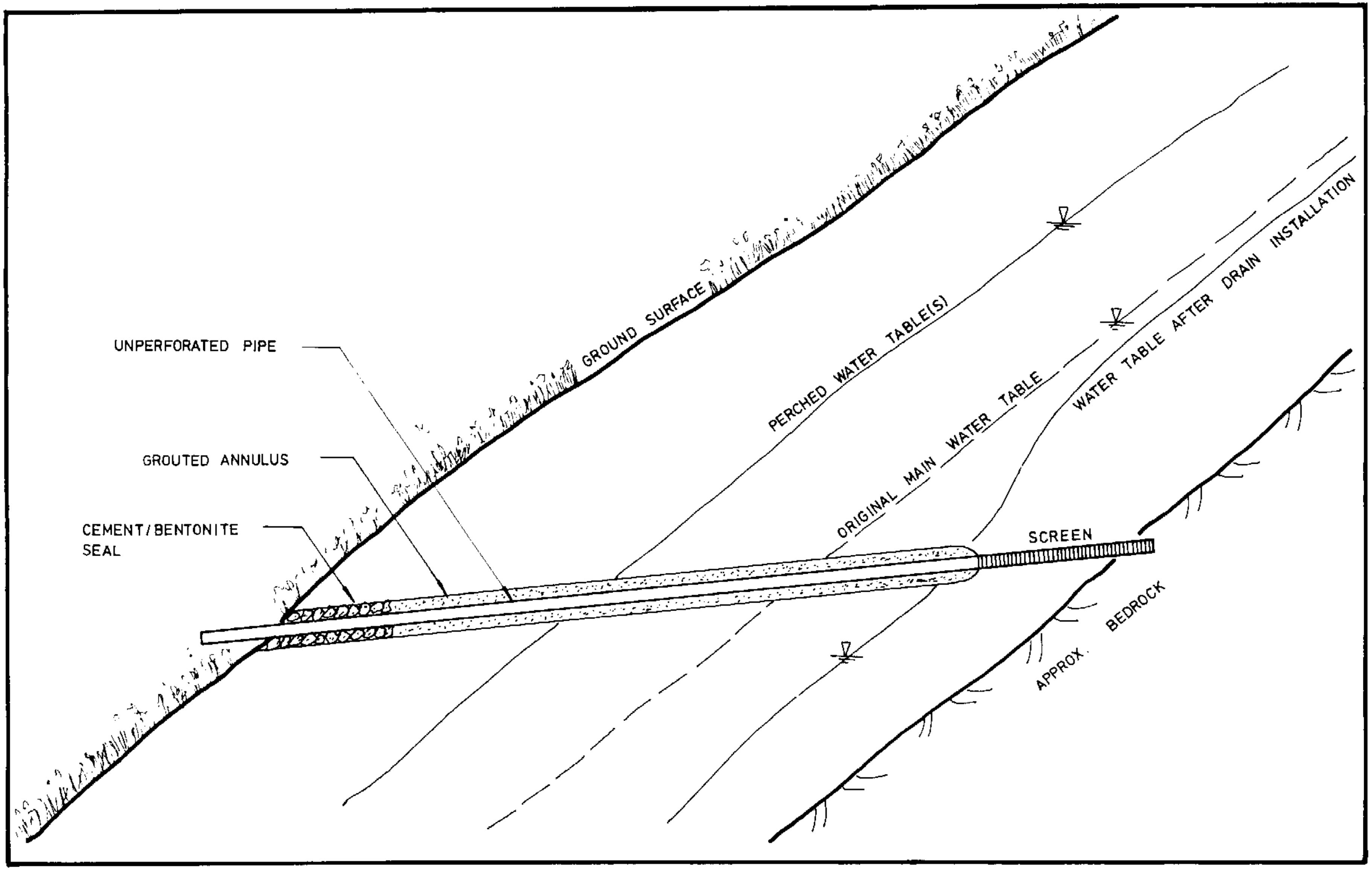


Figure 8 - Drilling Progress for Horizontal Drains - Reservoir Side



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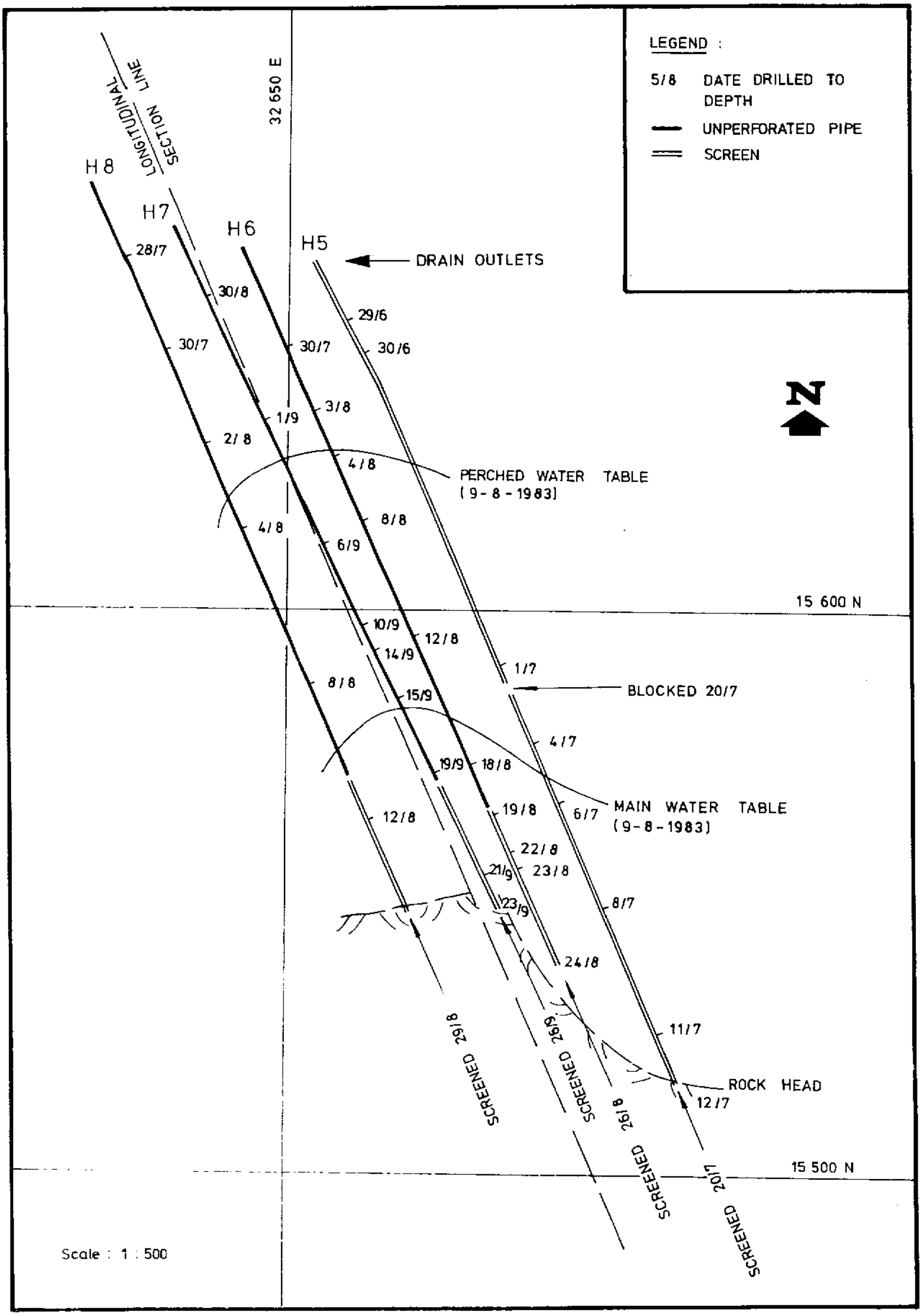

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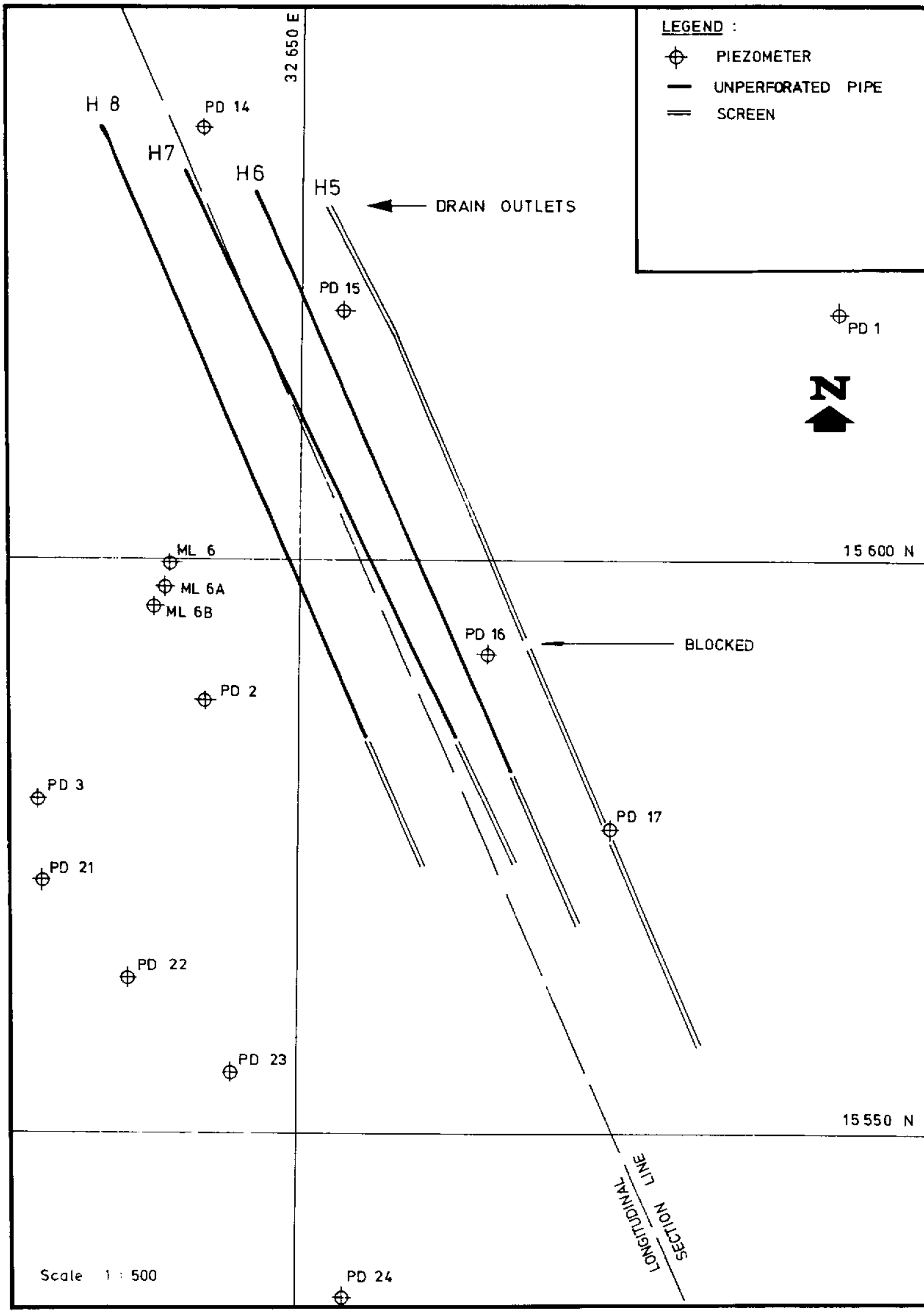

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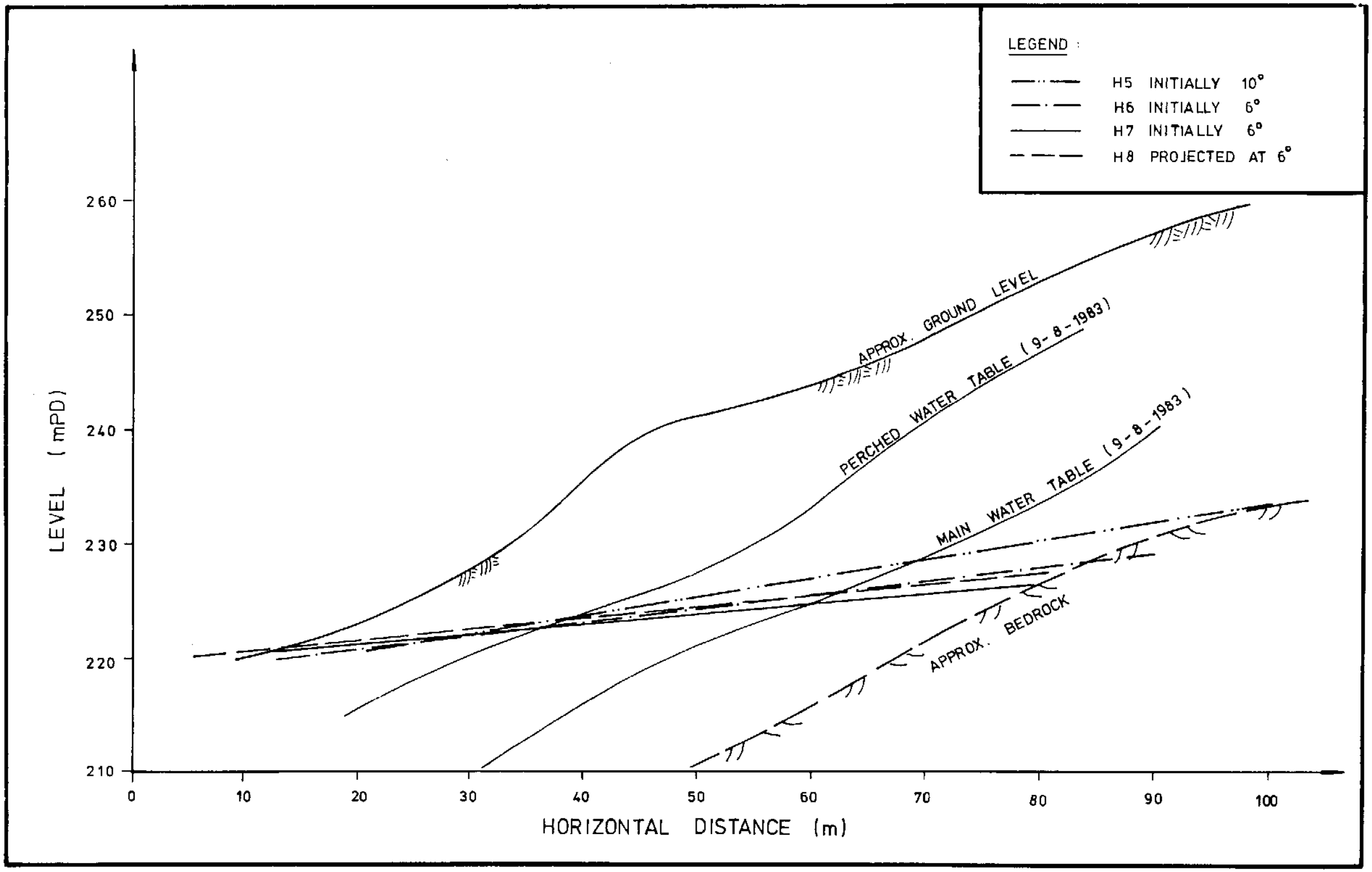

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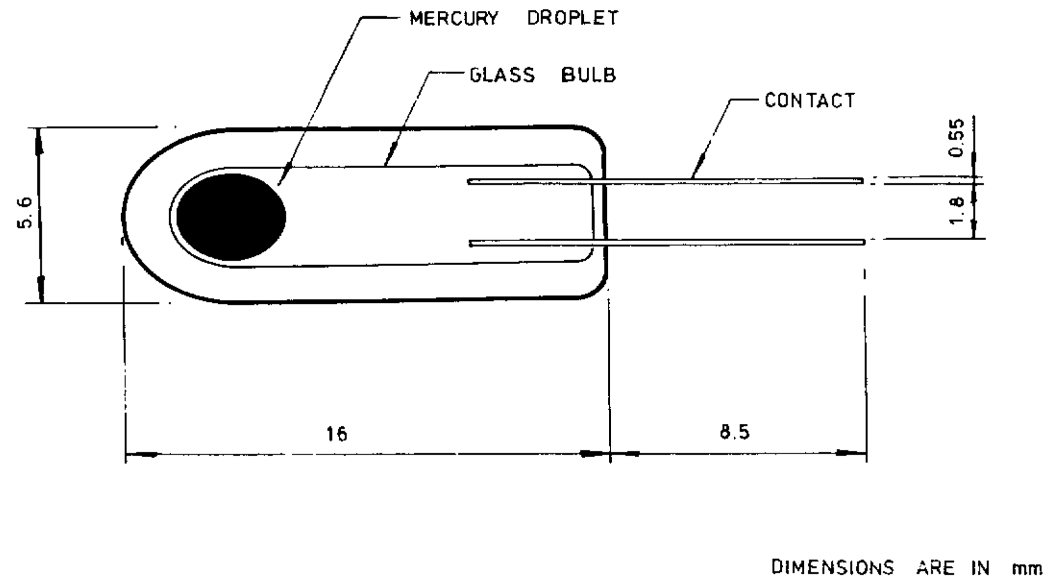
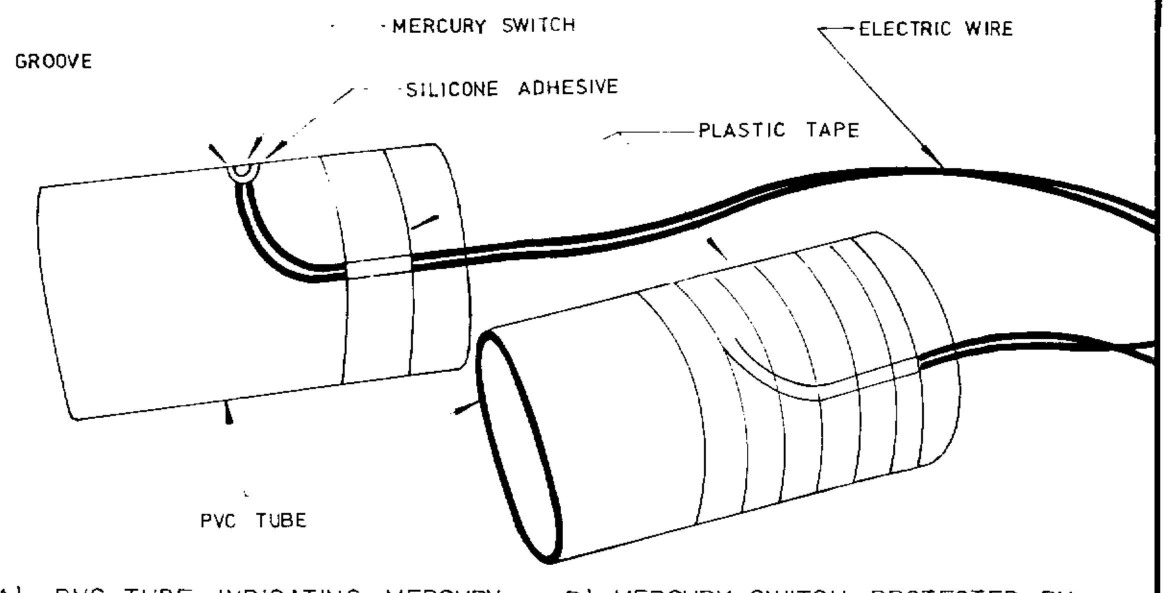


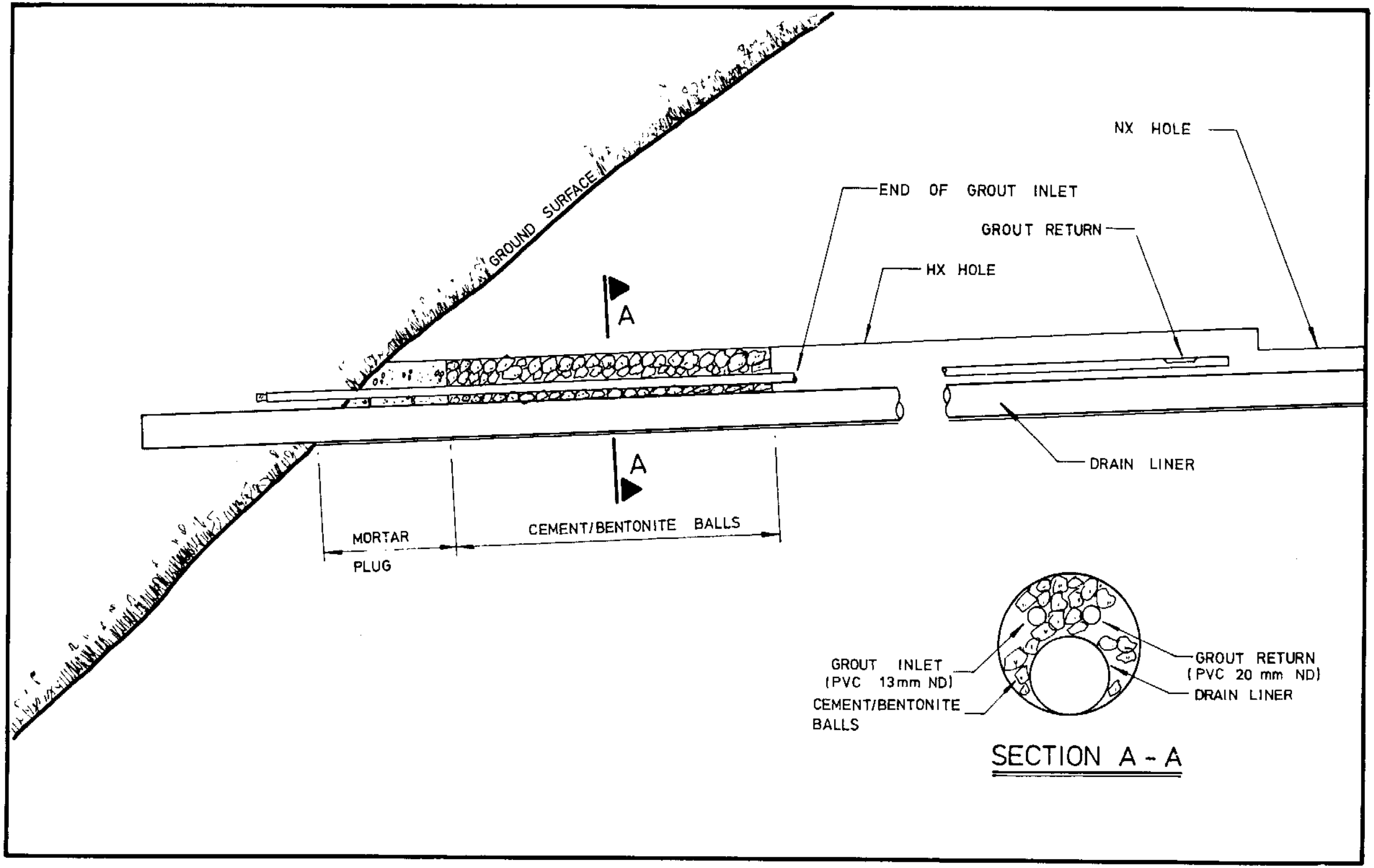
DETAILS OF MERCURY TILT SWITCH



A) PVC TUBE INDICATING MERCURY SWITCH LOCATION

B) MERCURY SWITCH PROTECTED BY PLASTIC TAPE BEFORE INSTALLATION

Figure 13 - Mercury Tilt Switch



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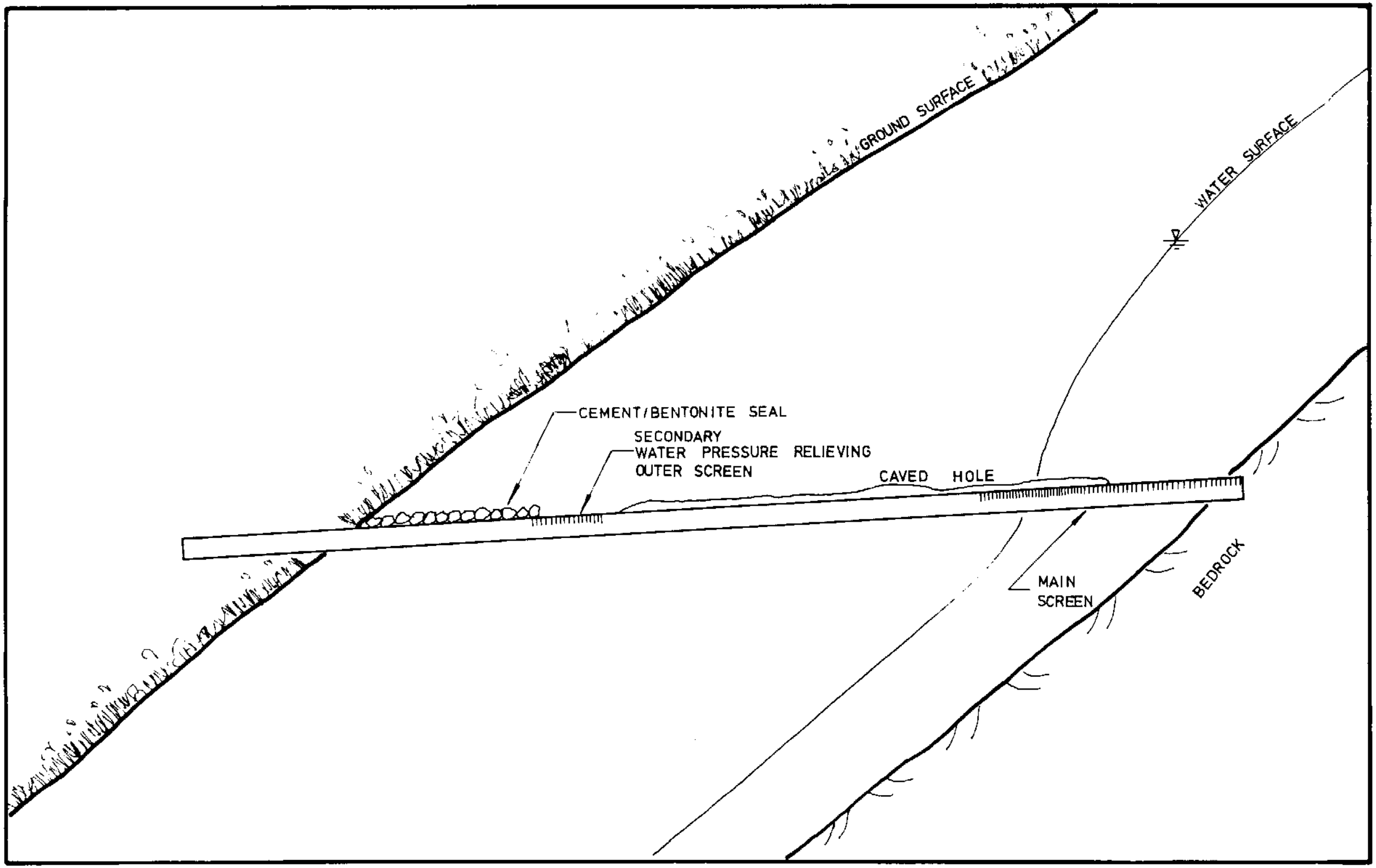

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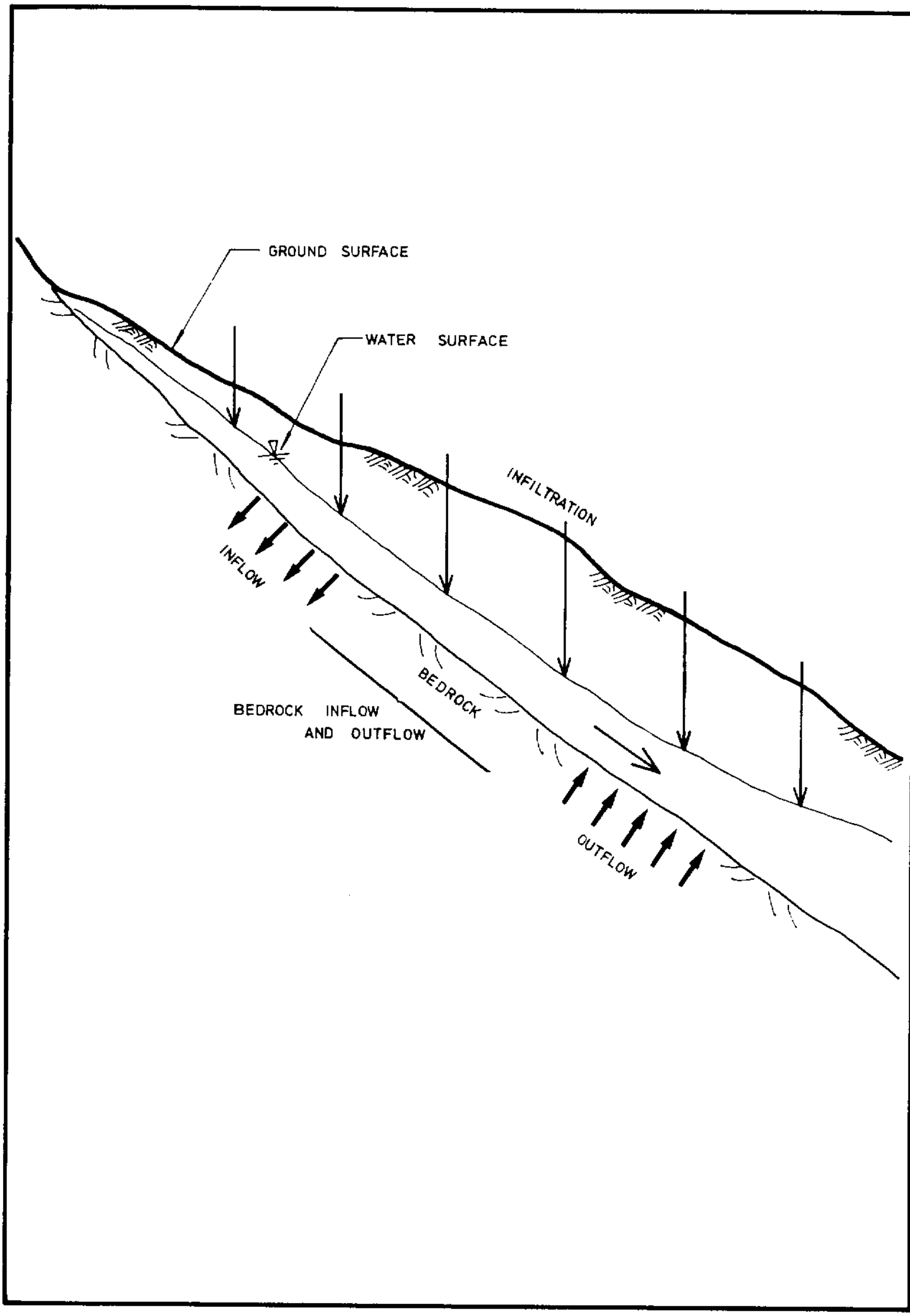

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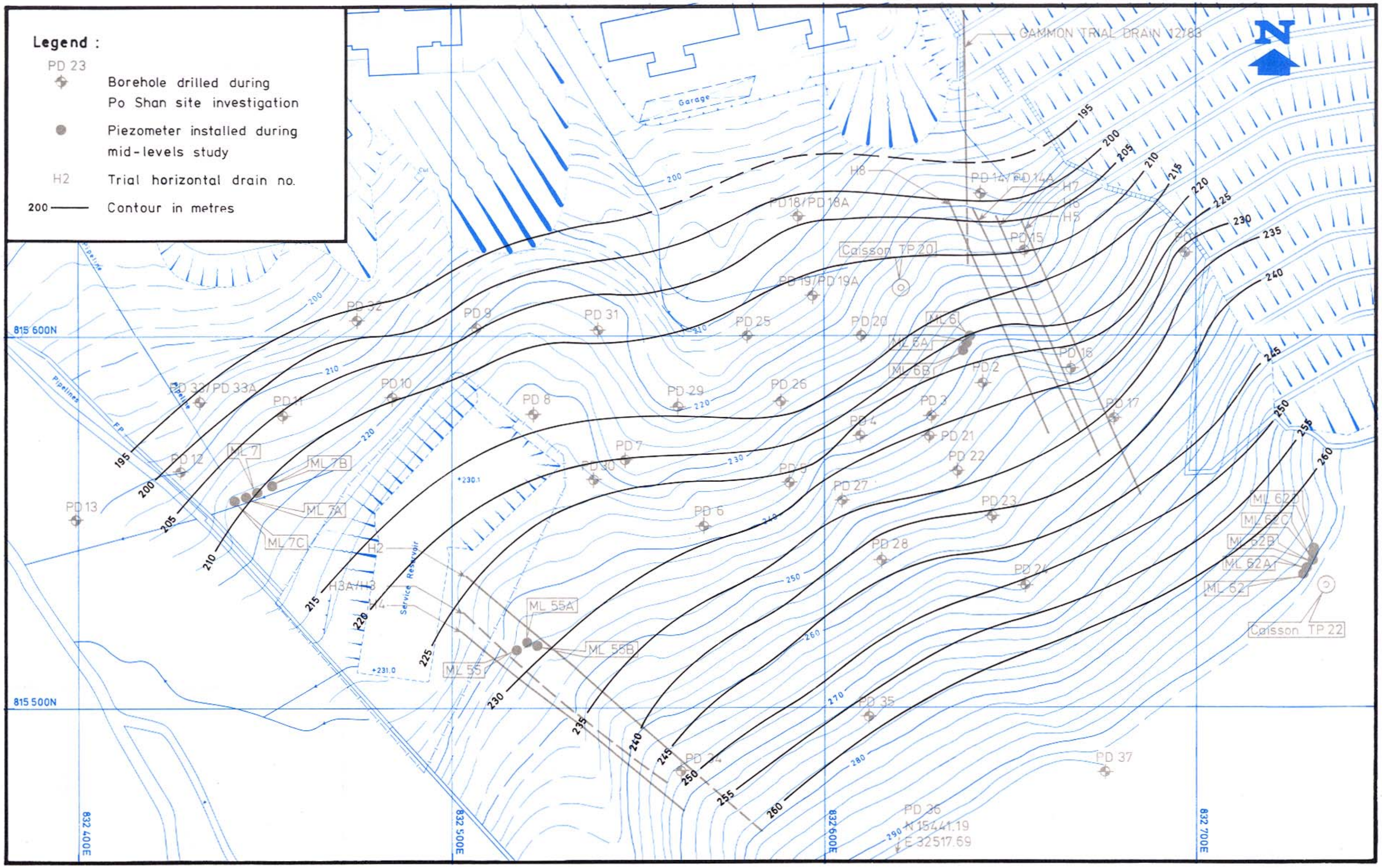

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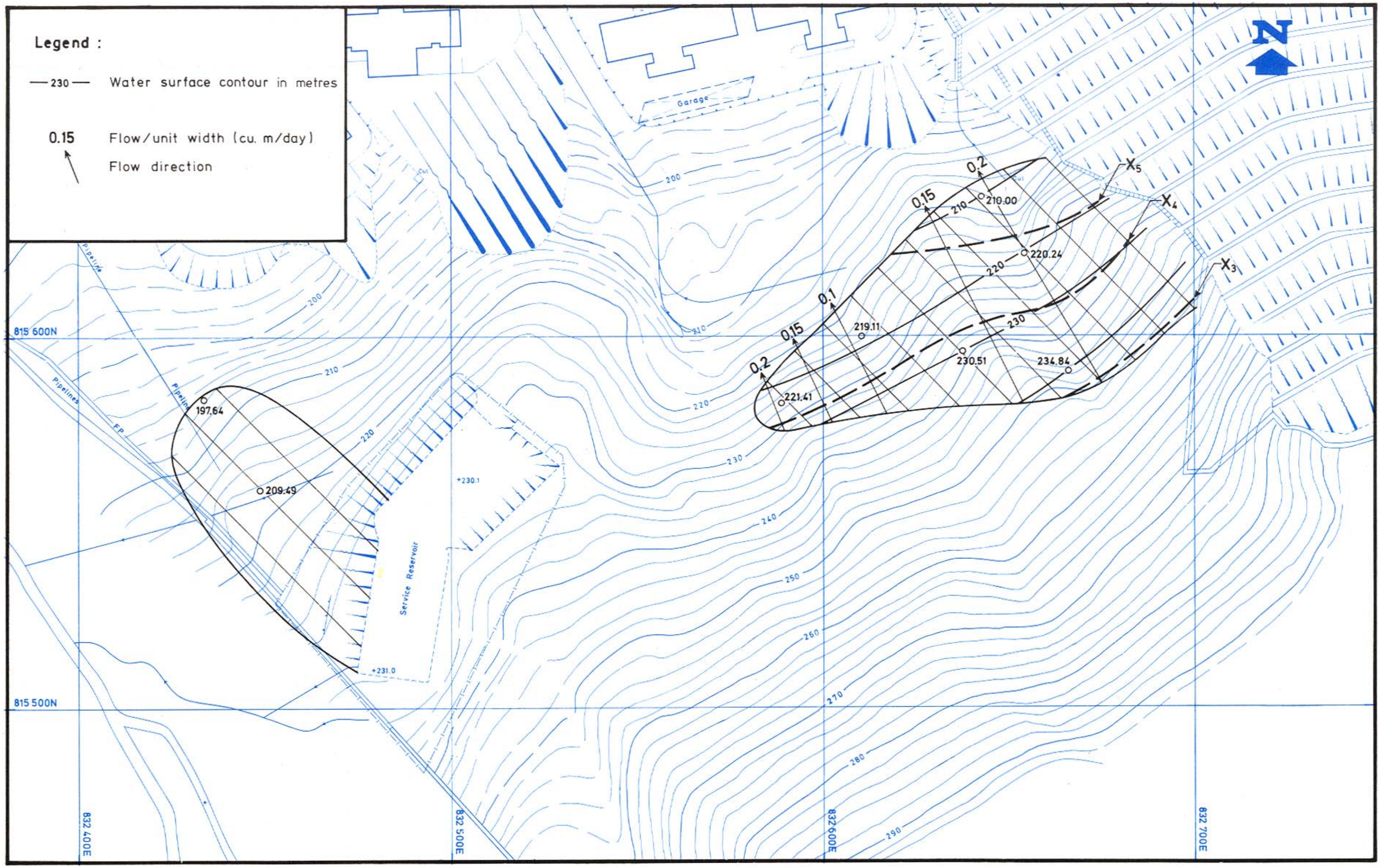

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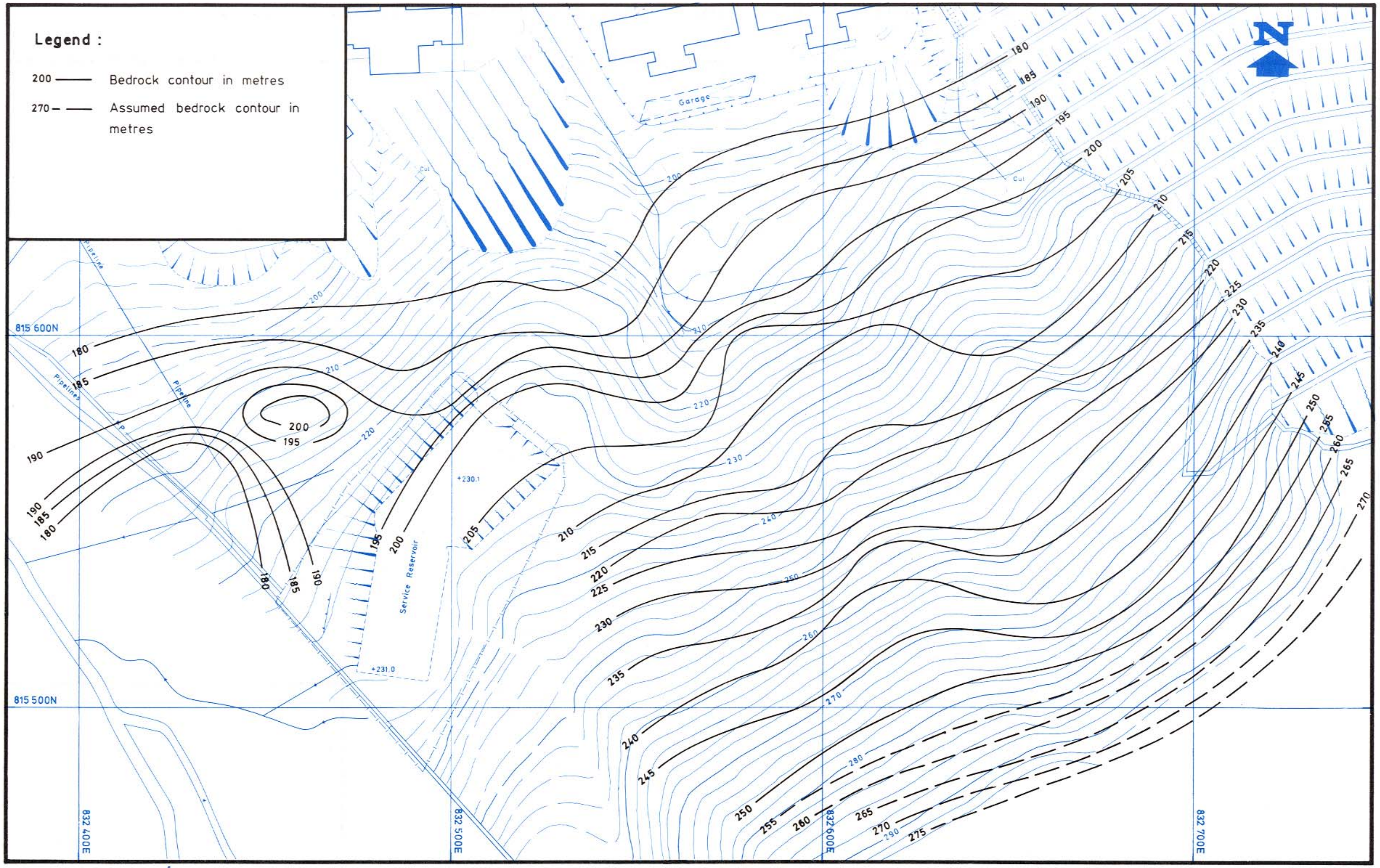

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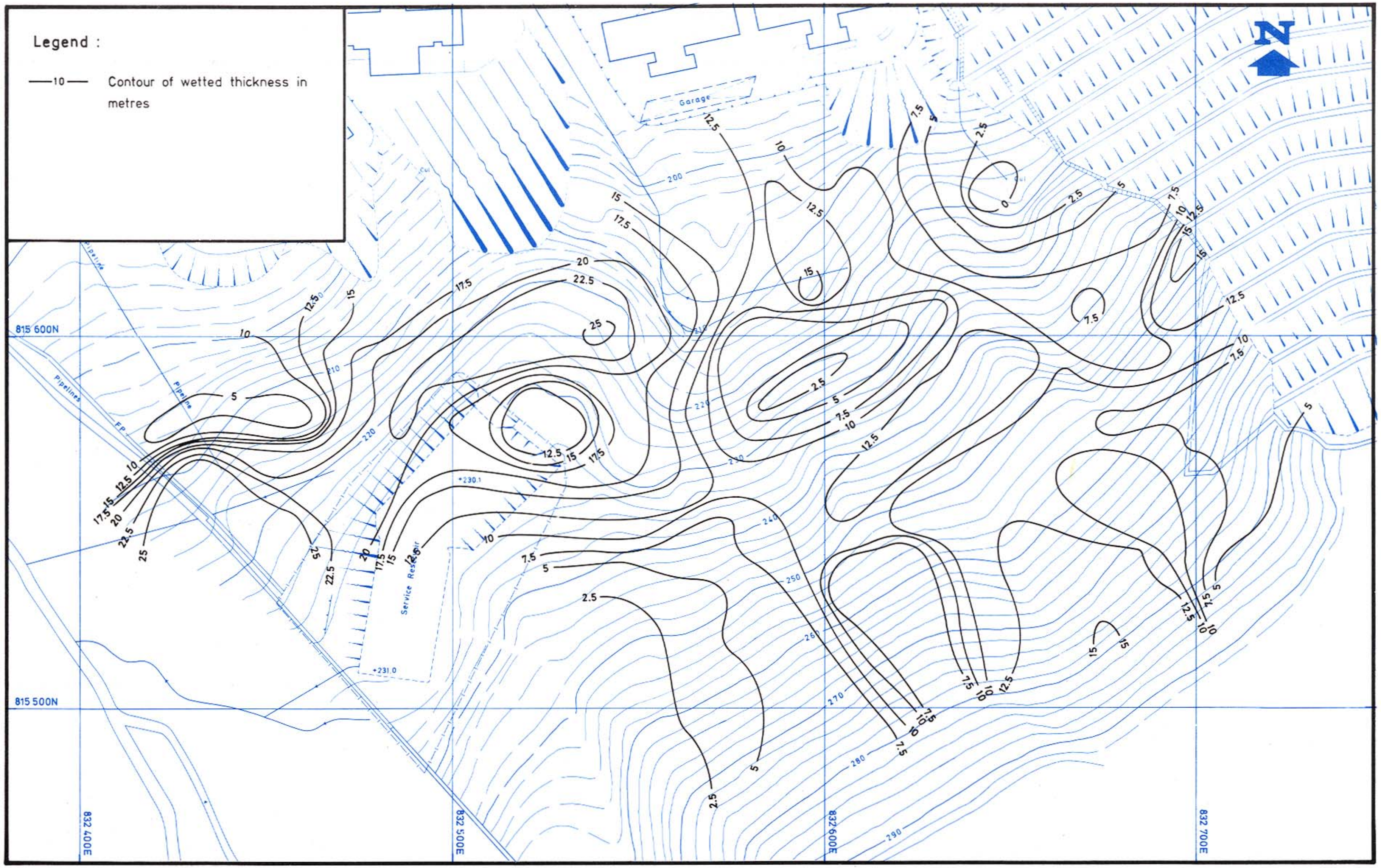

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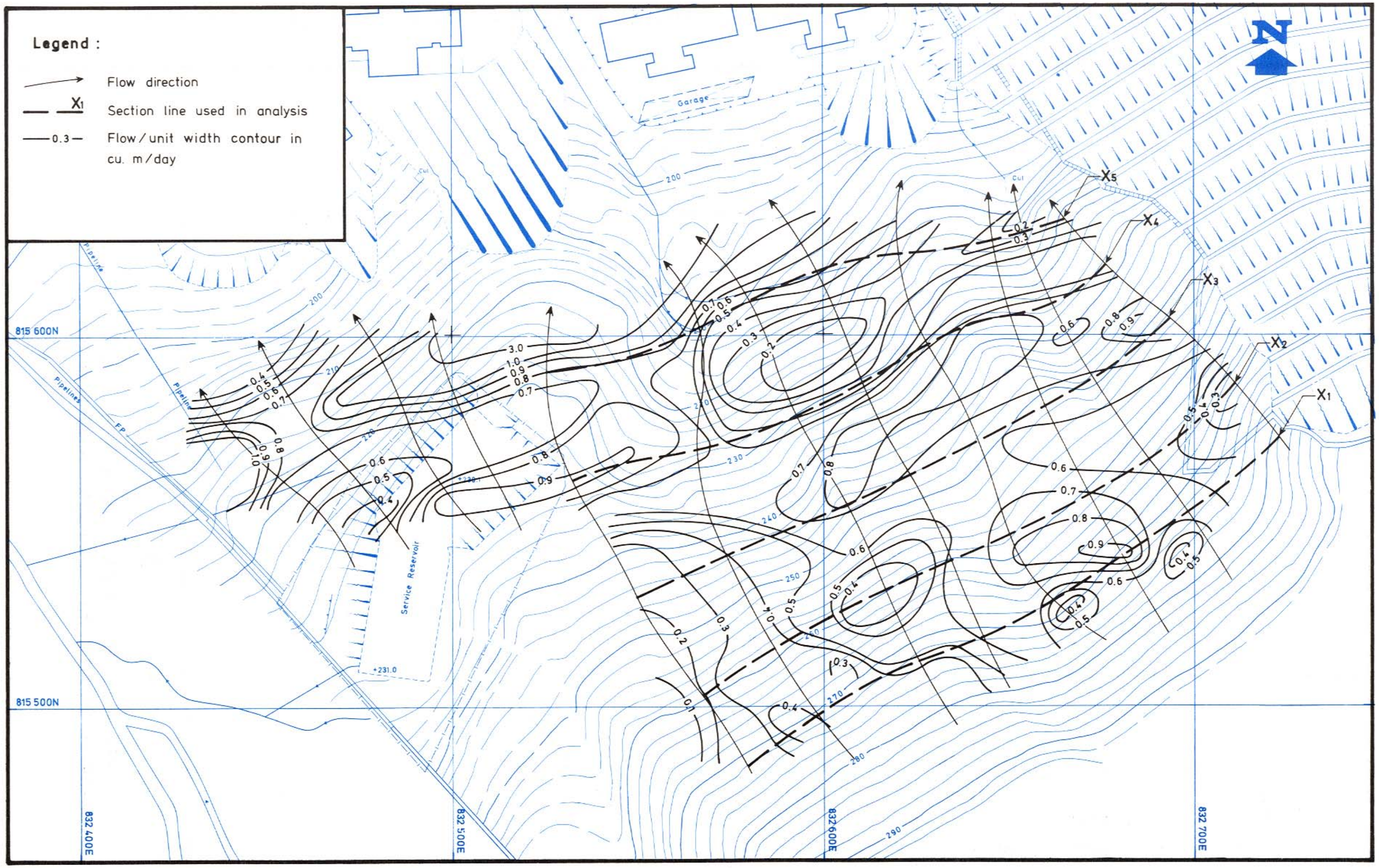

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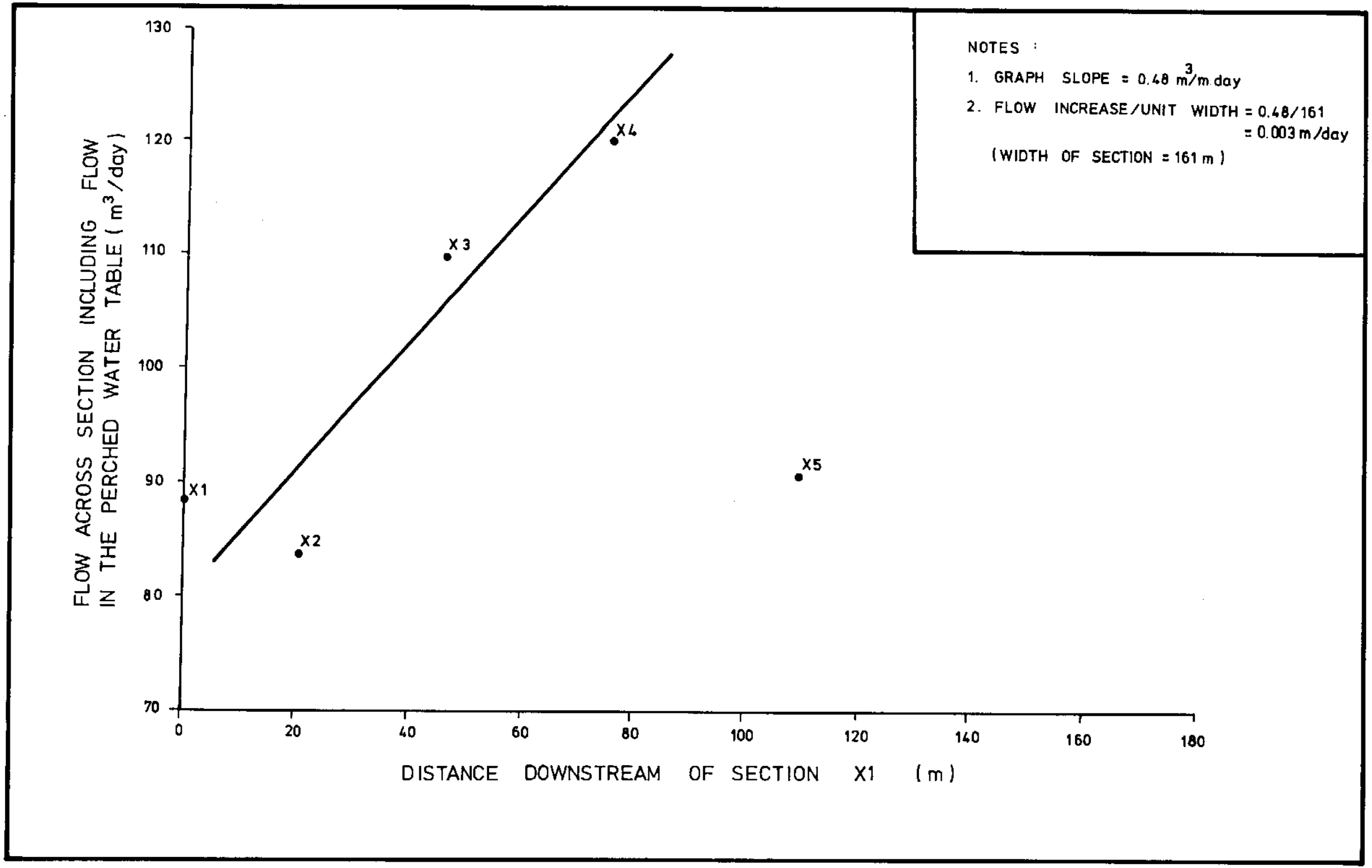

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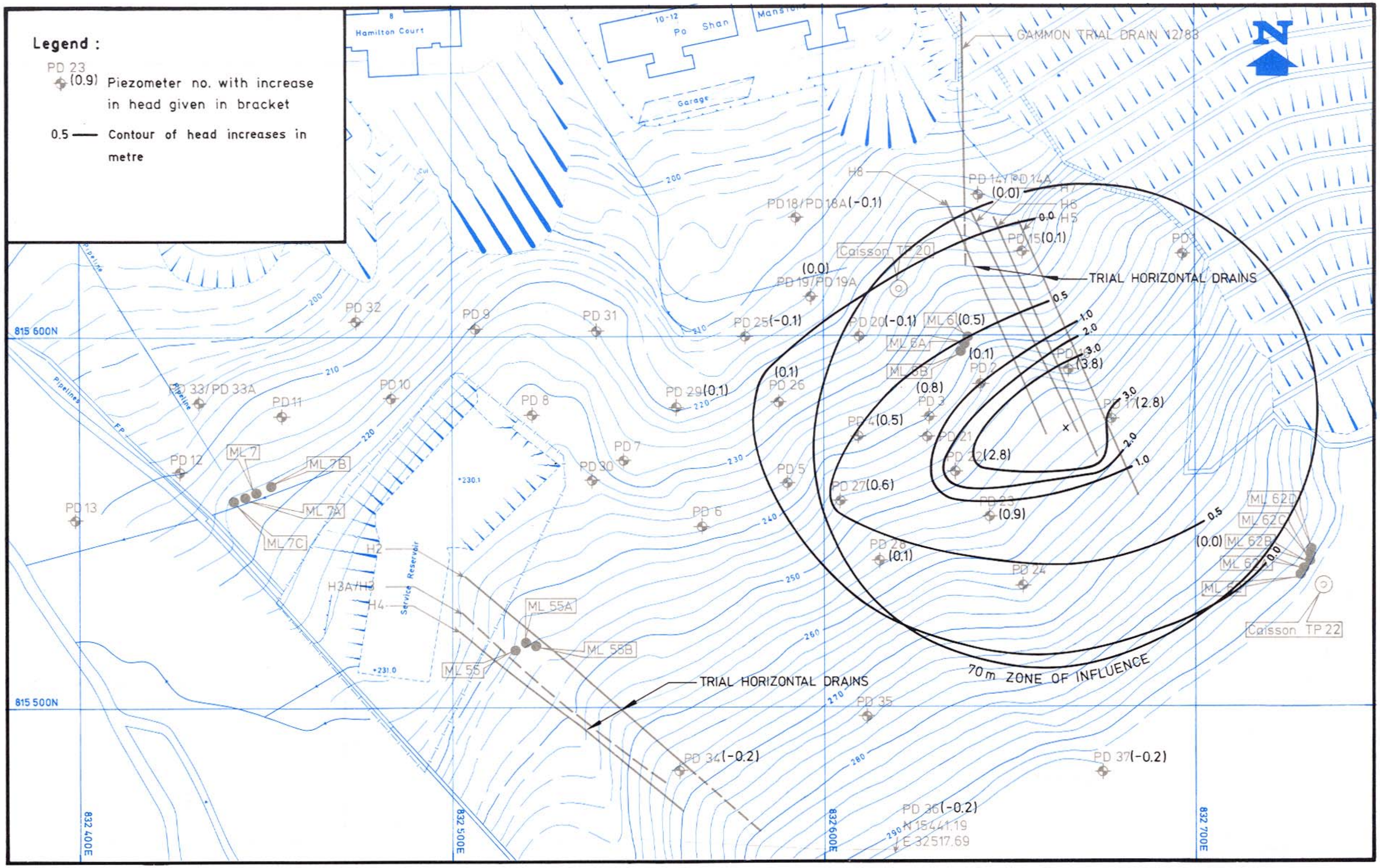

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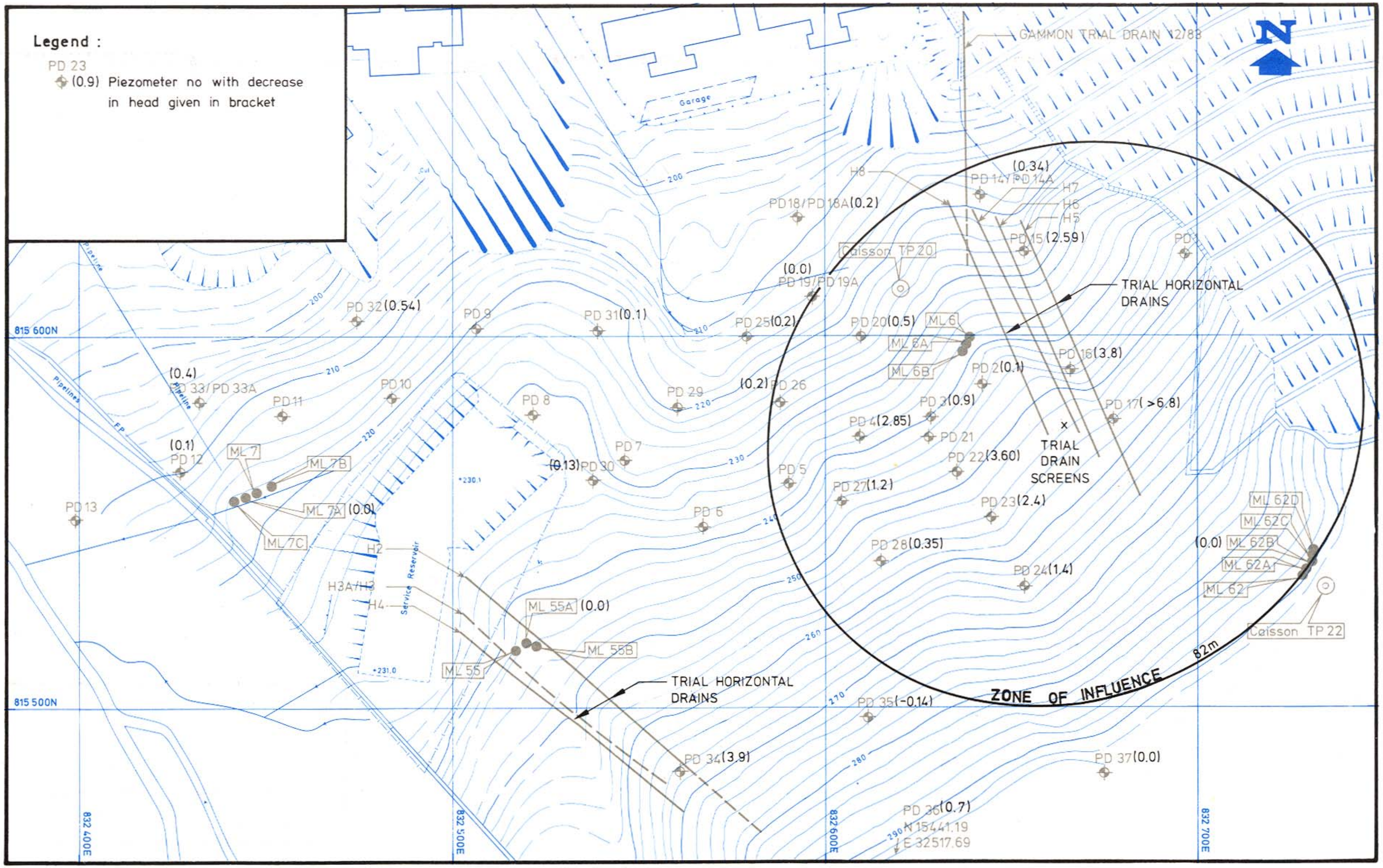

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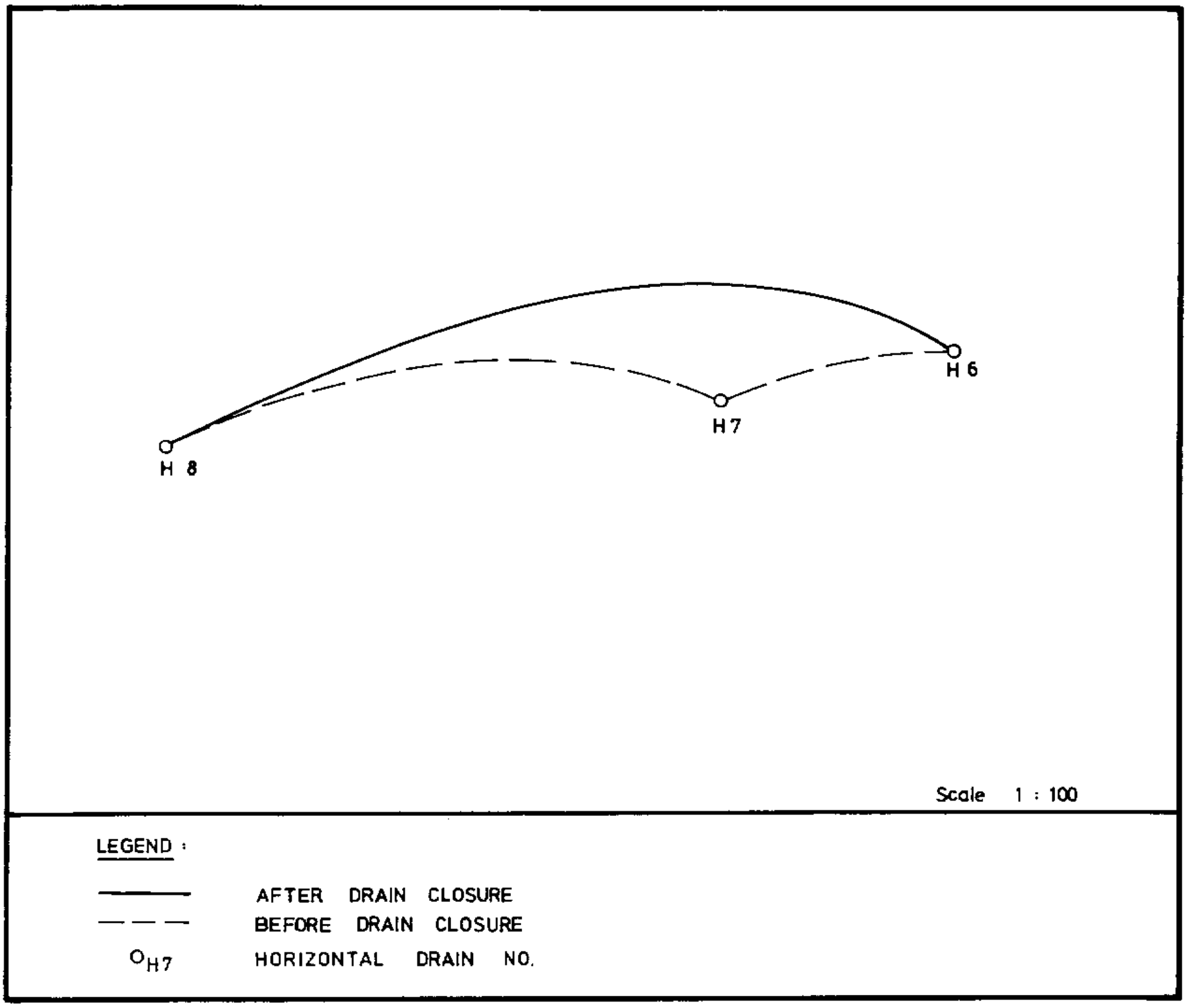

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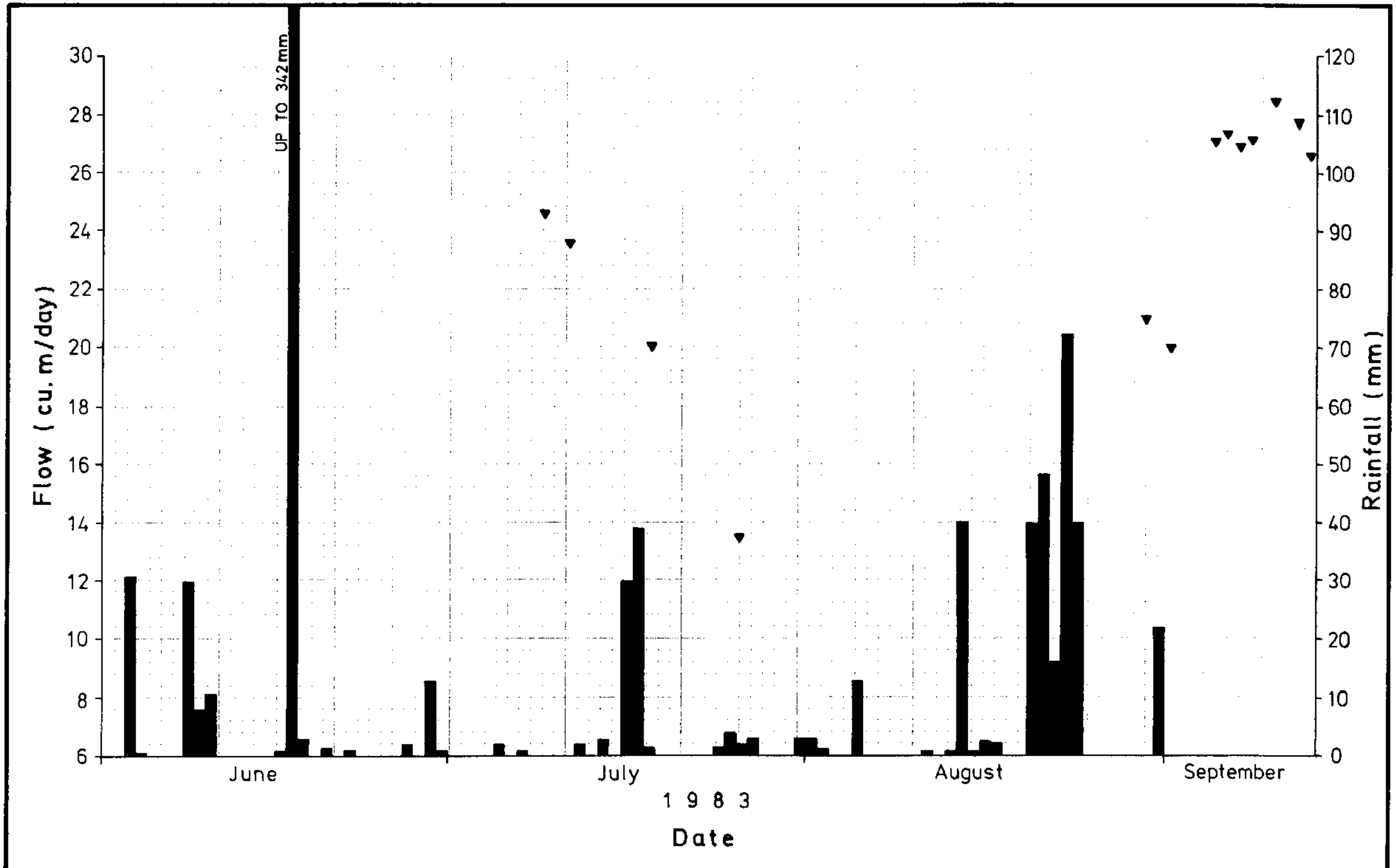

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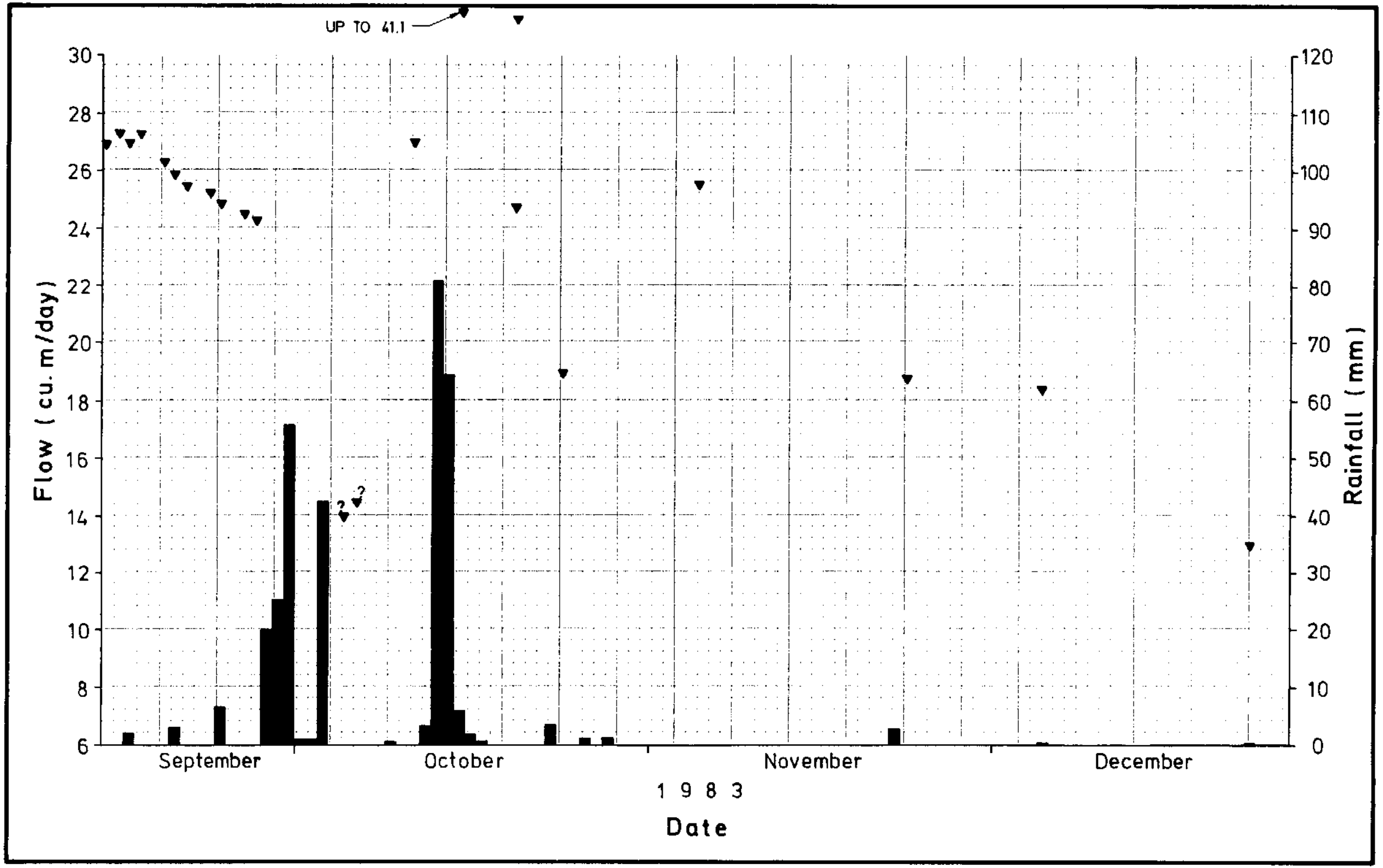

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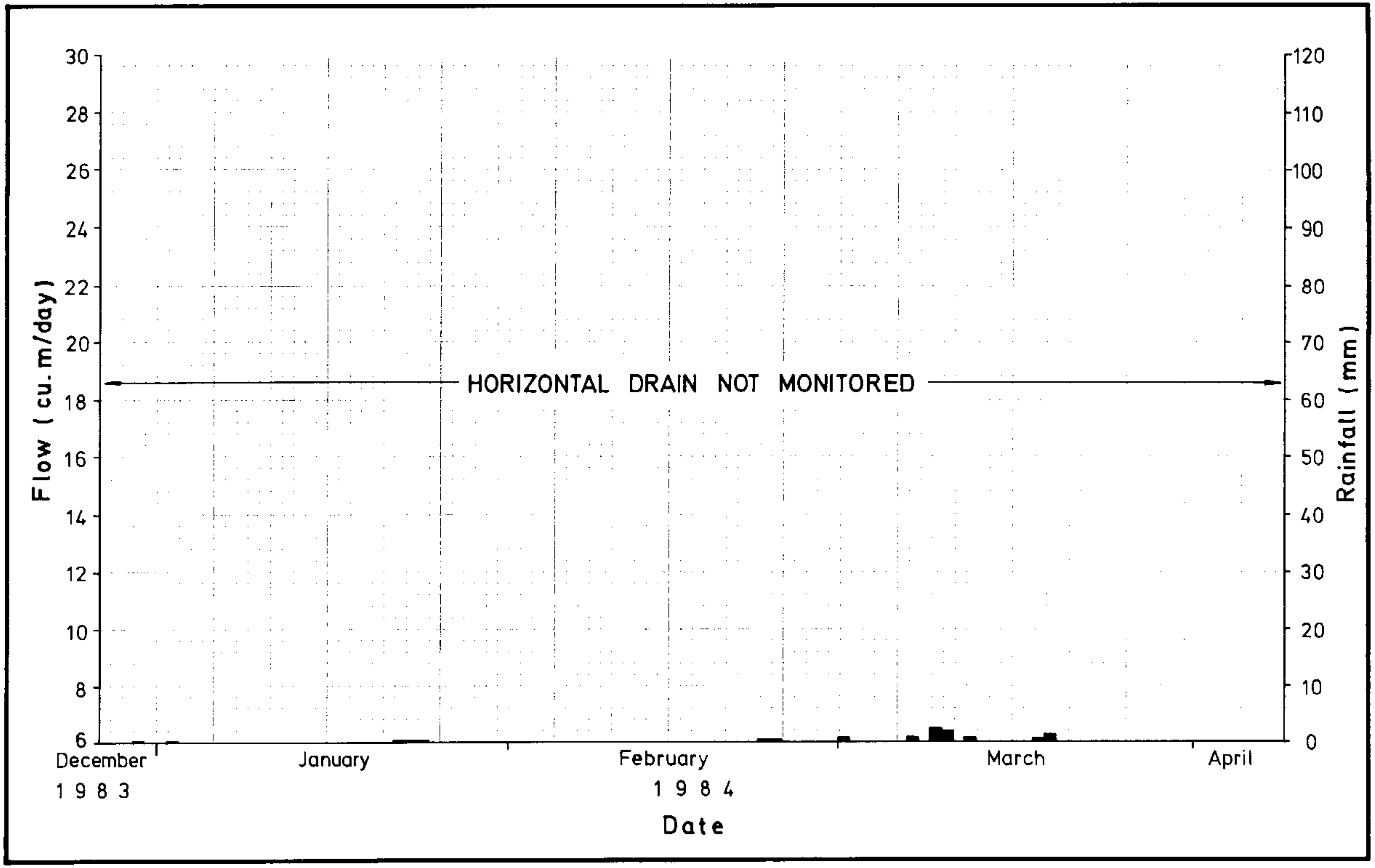

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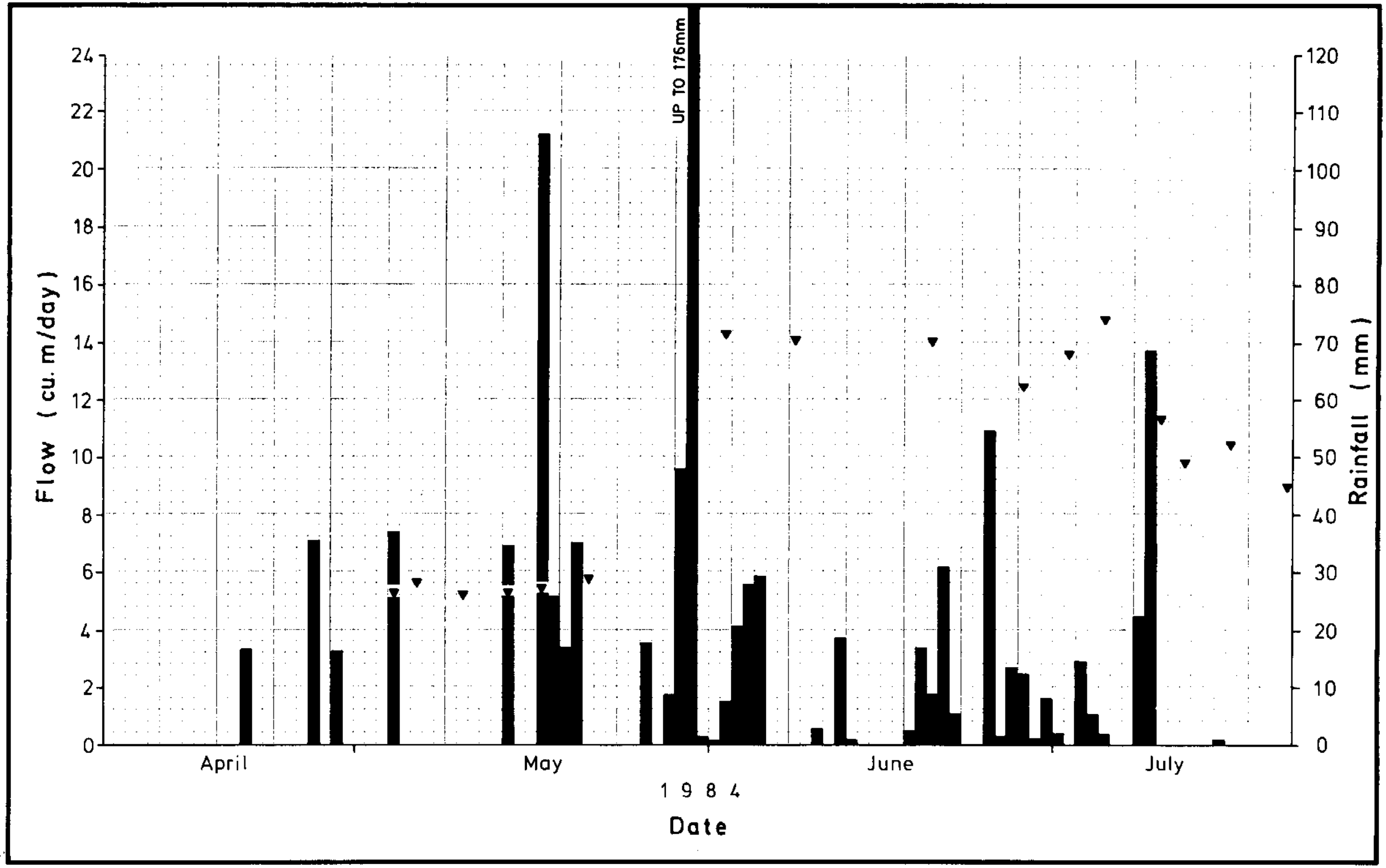

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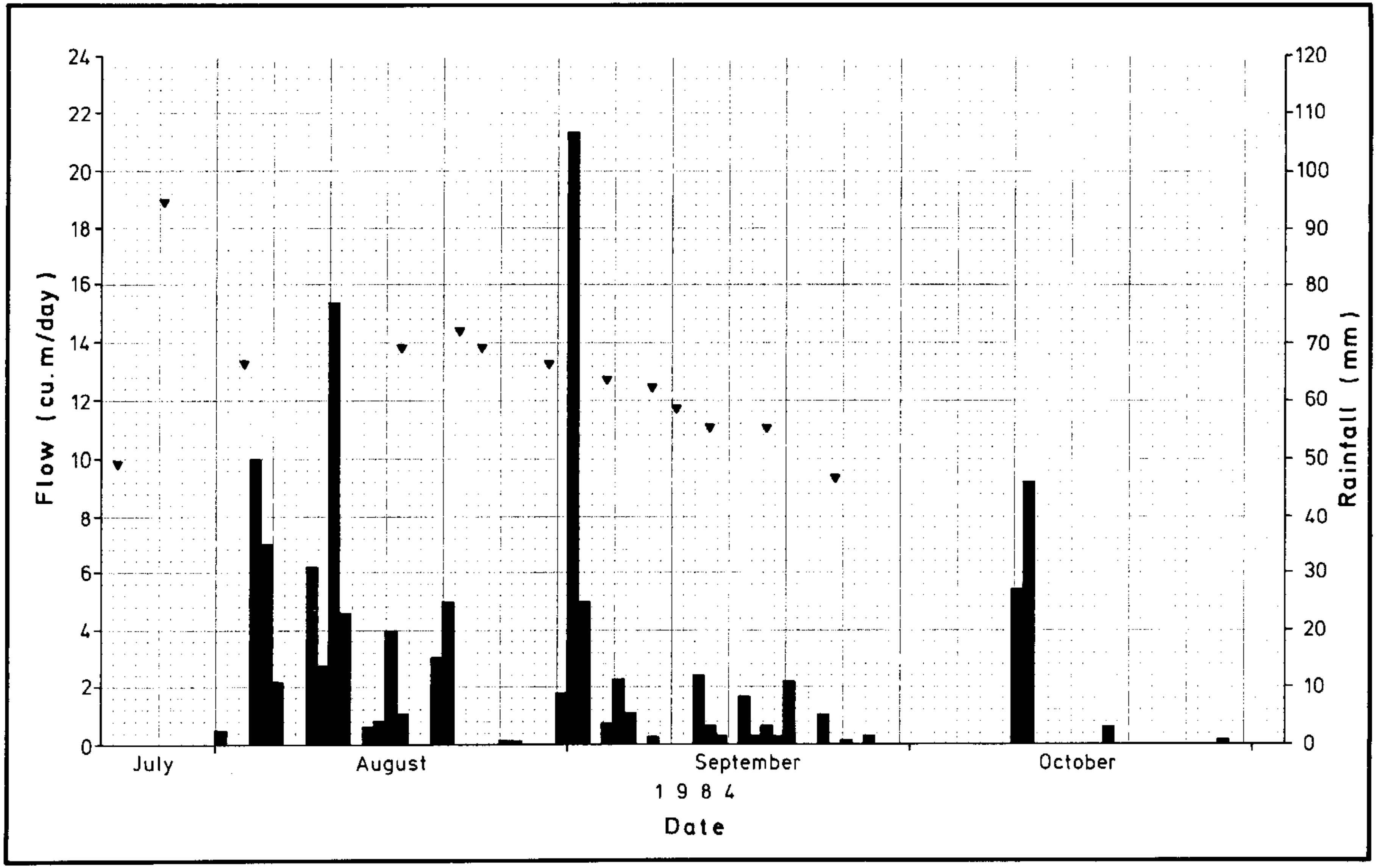

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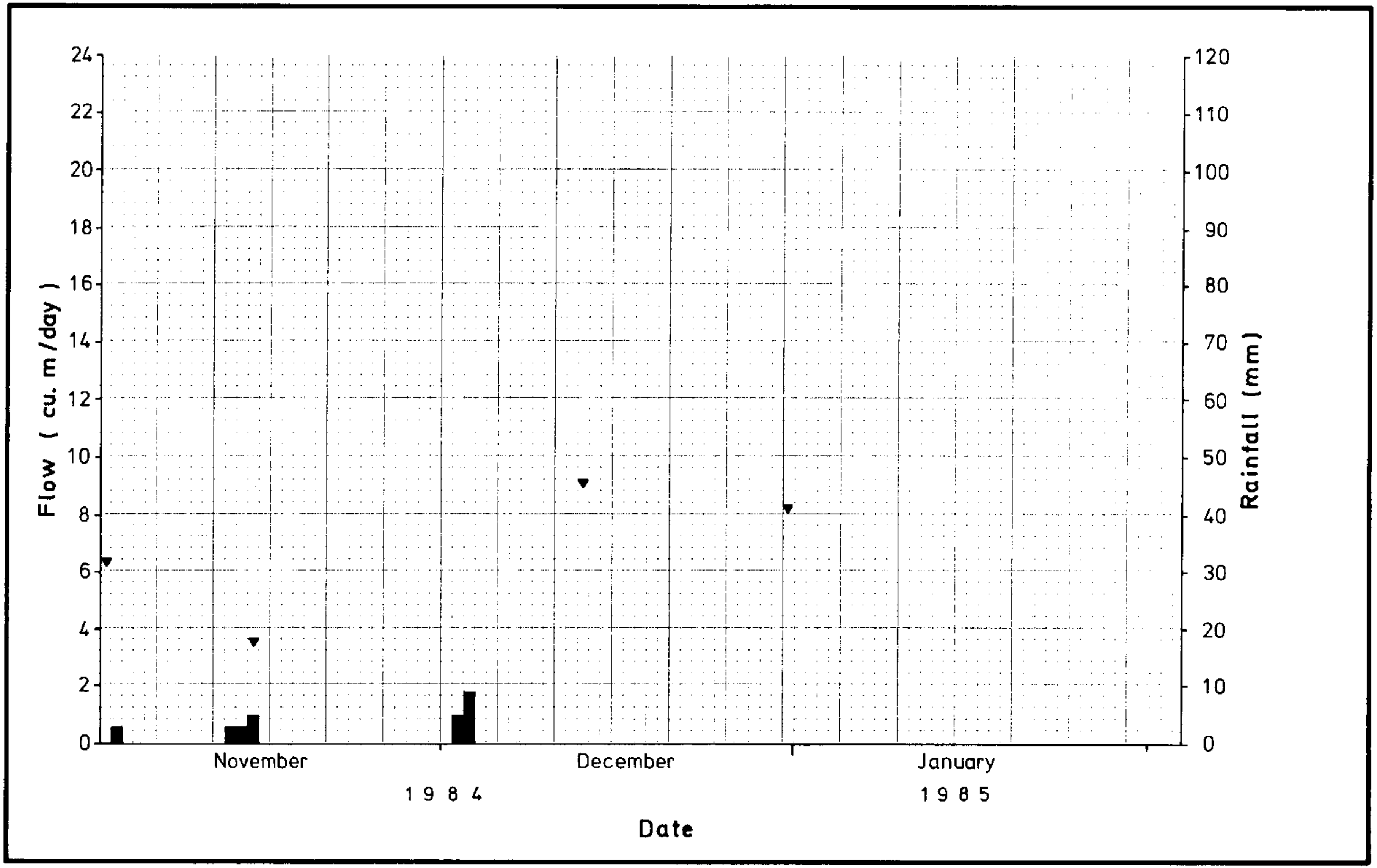

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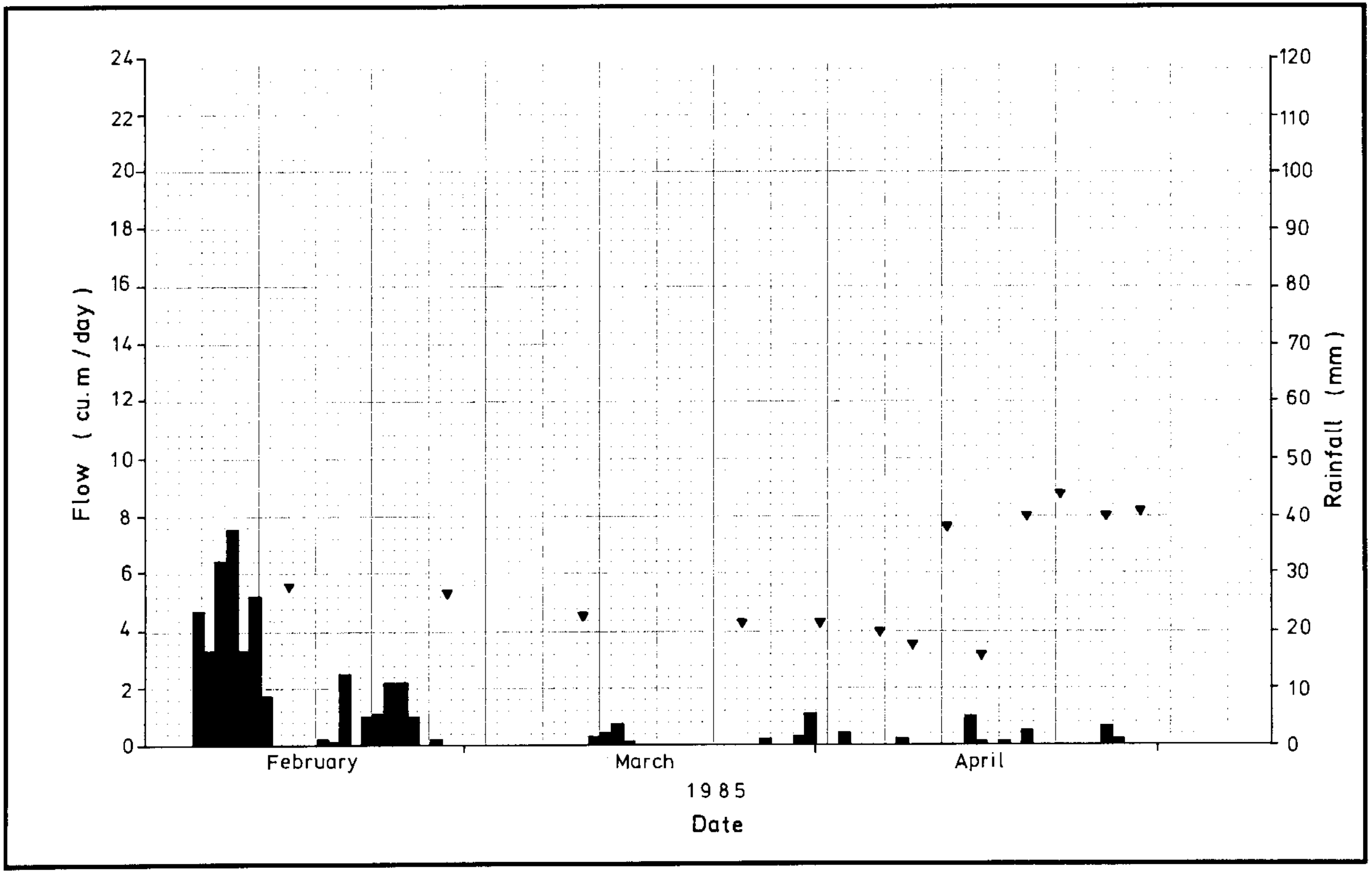

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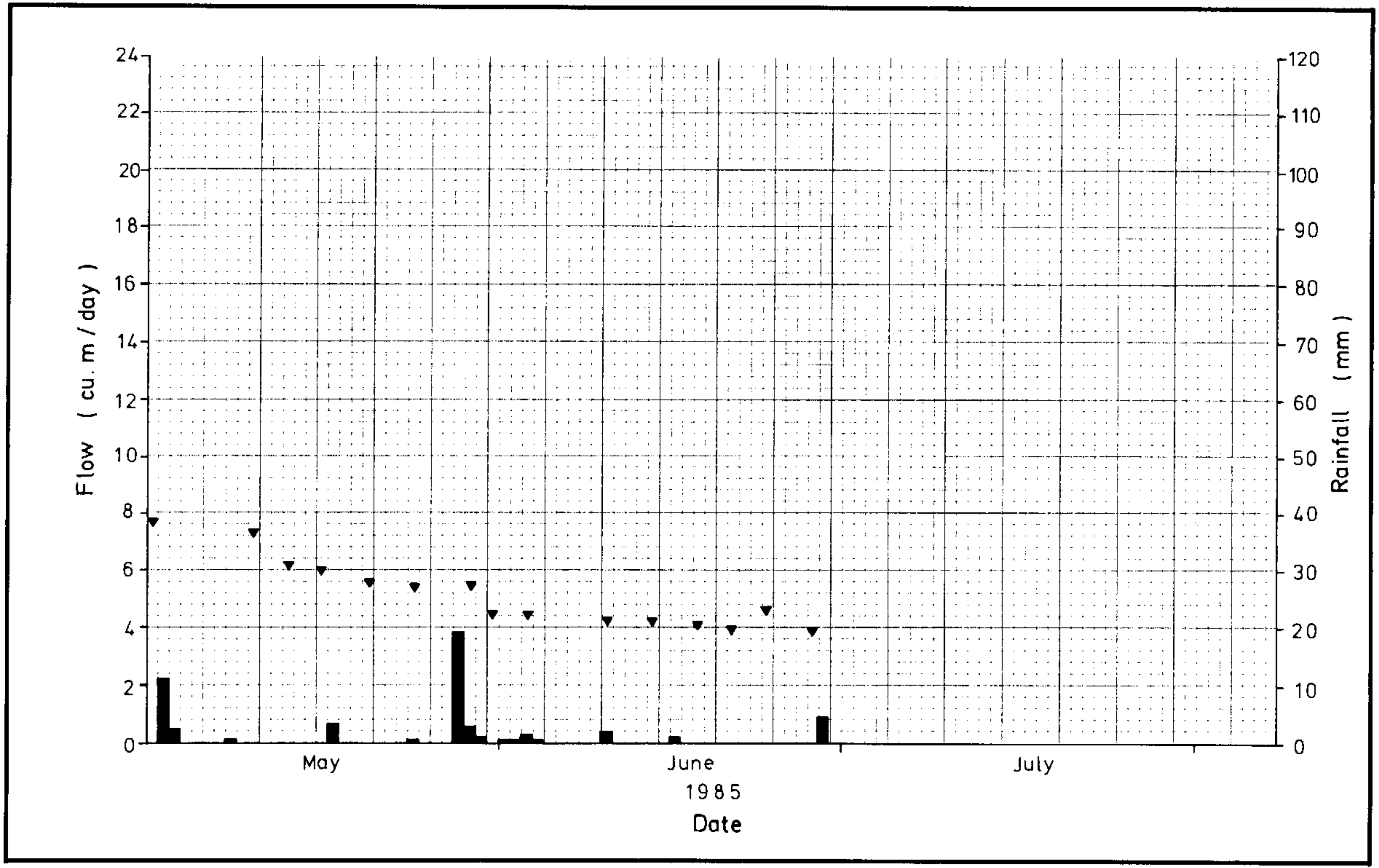

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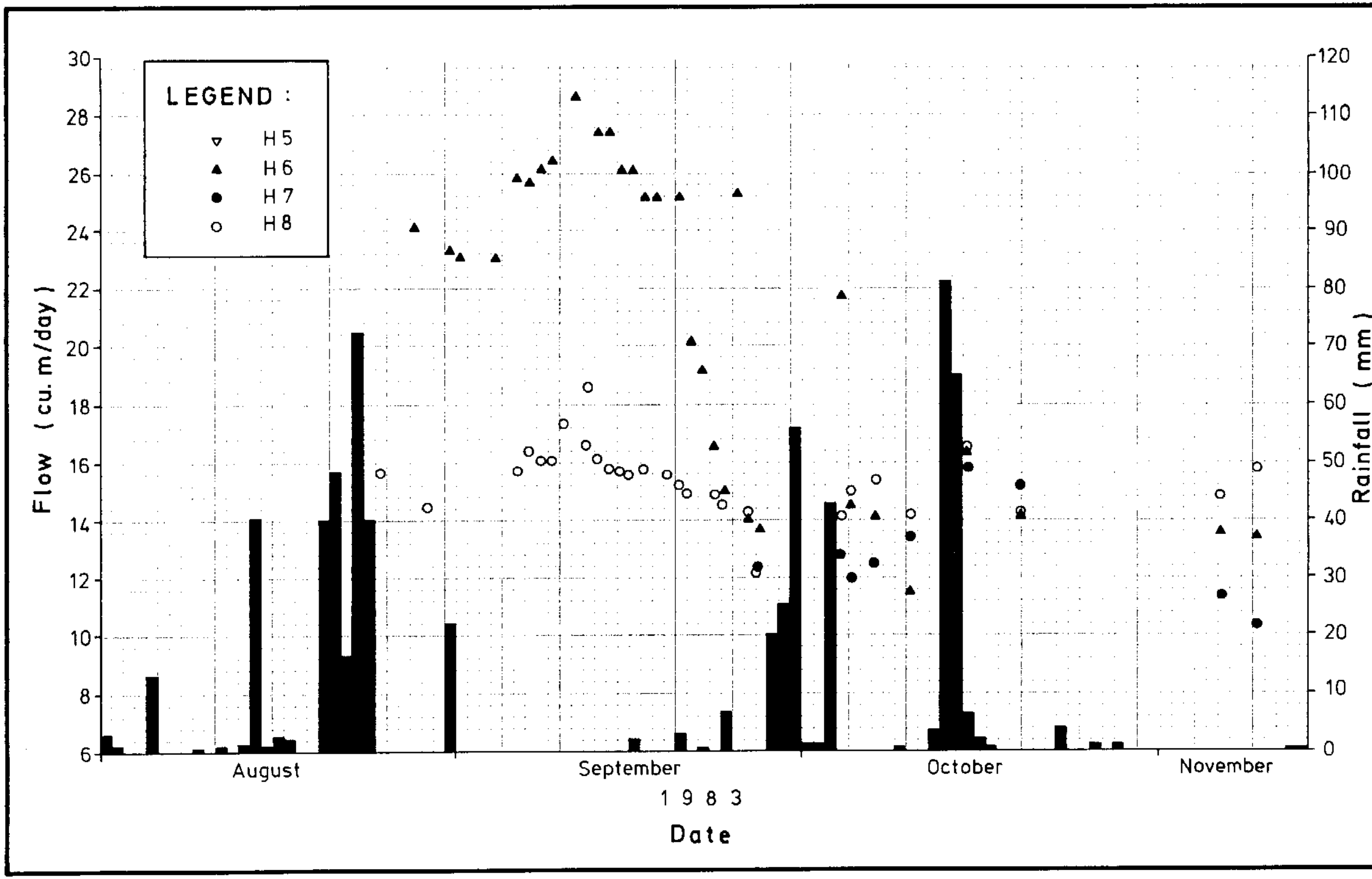

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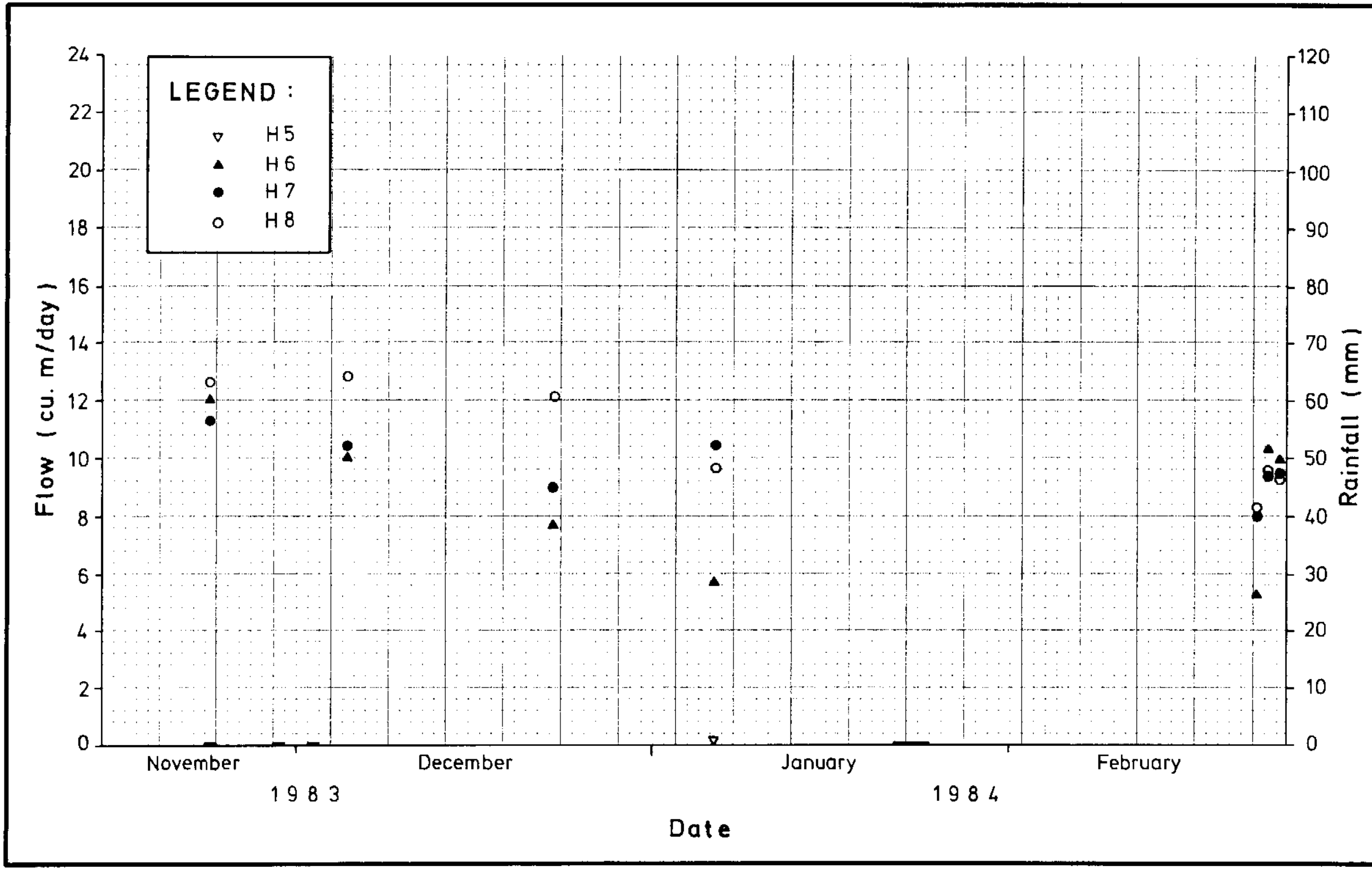

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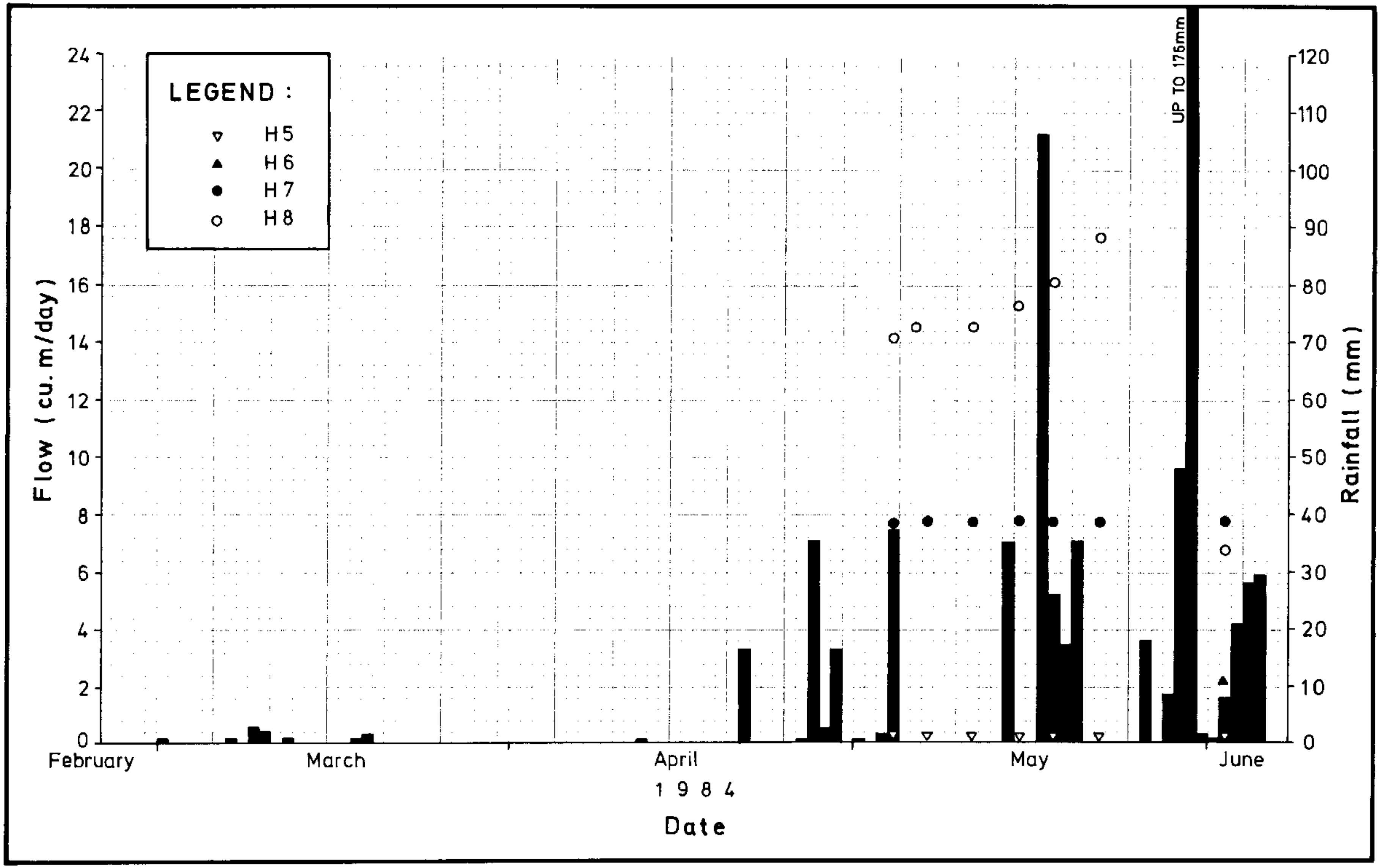

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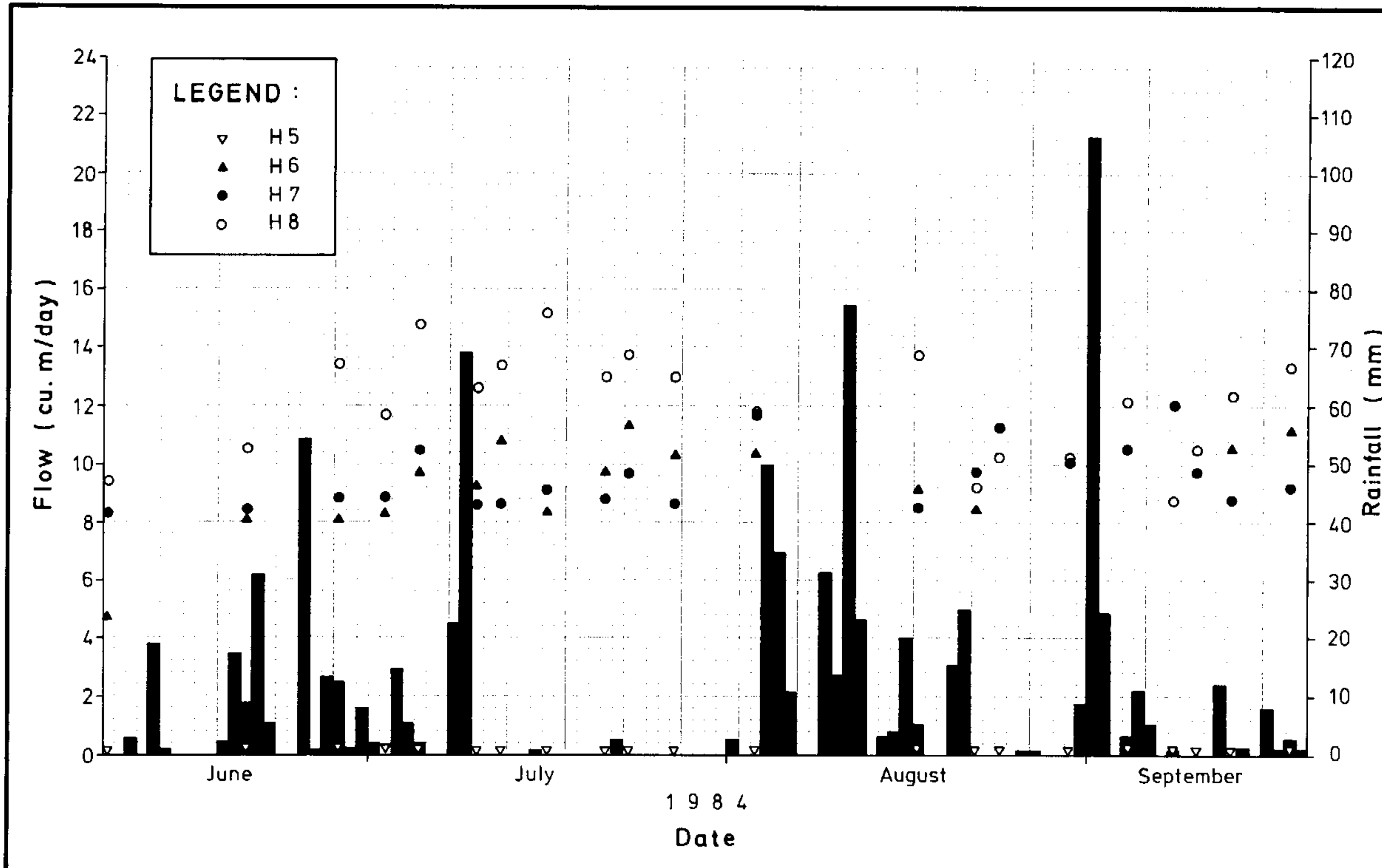

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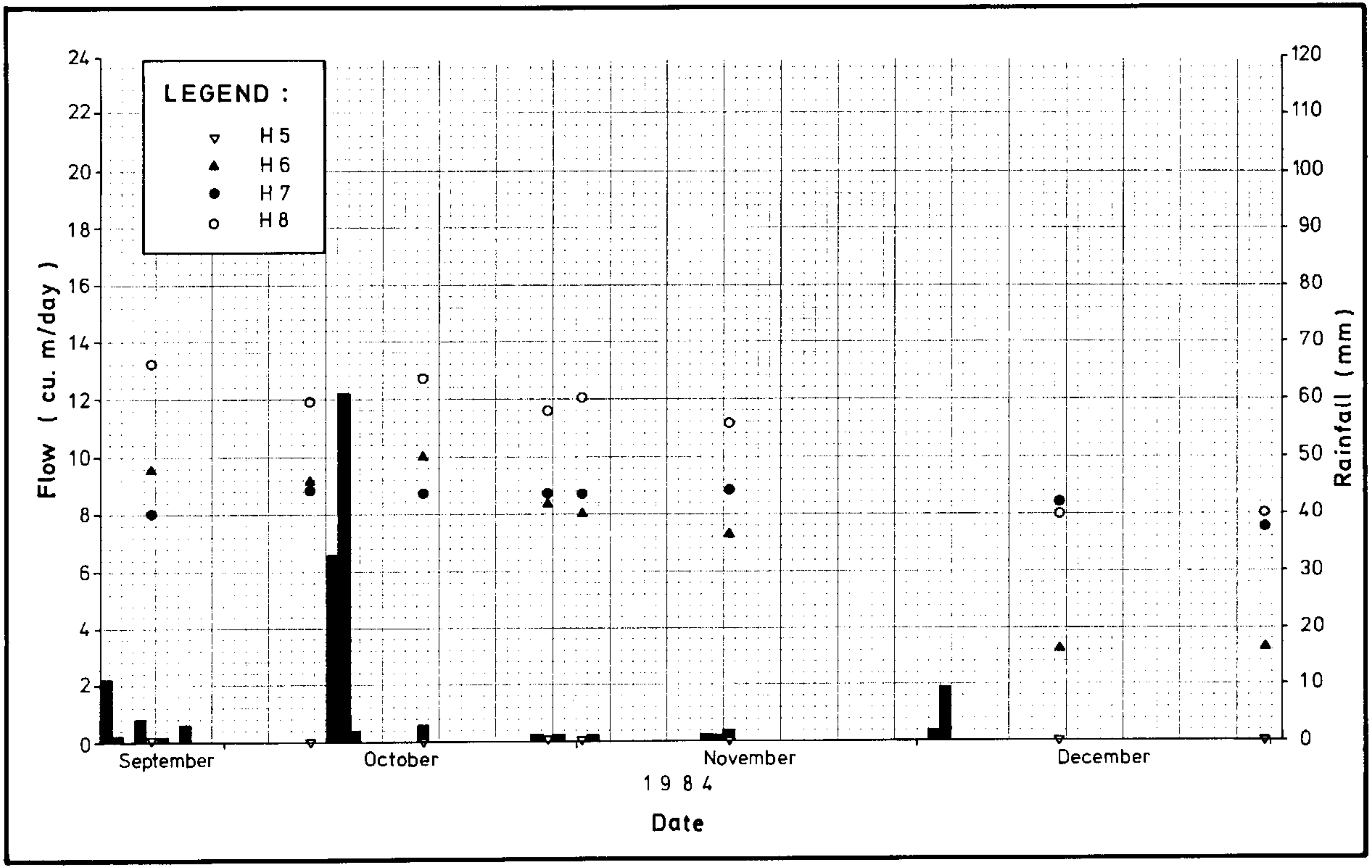

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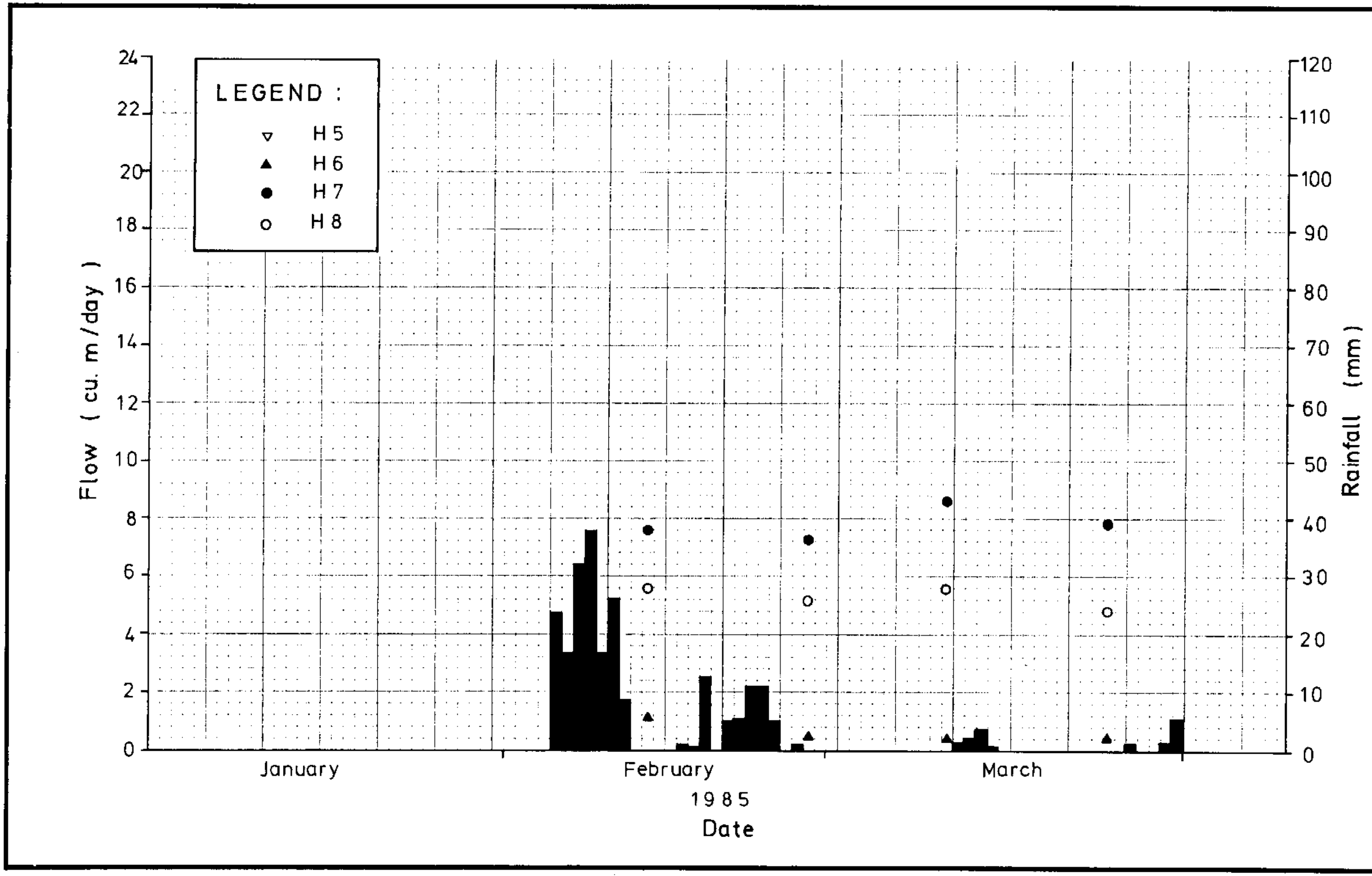

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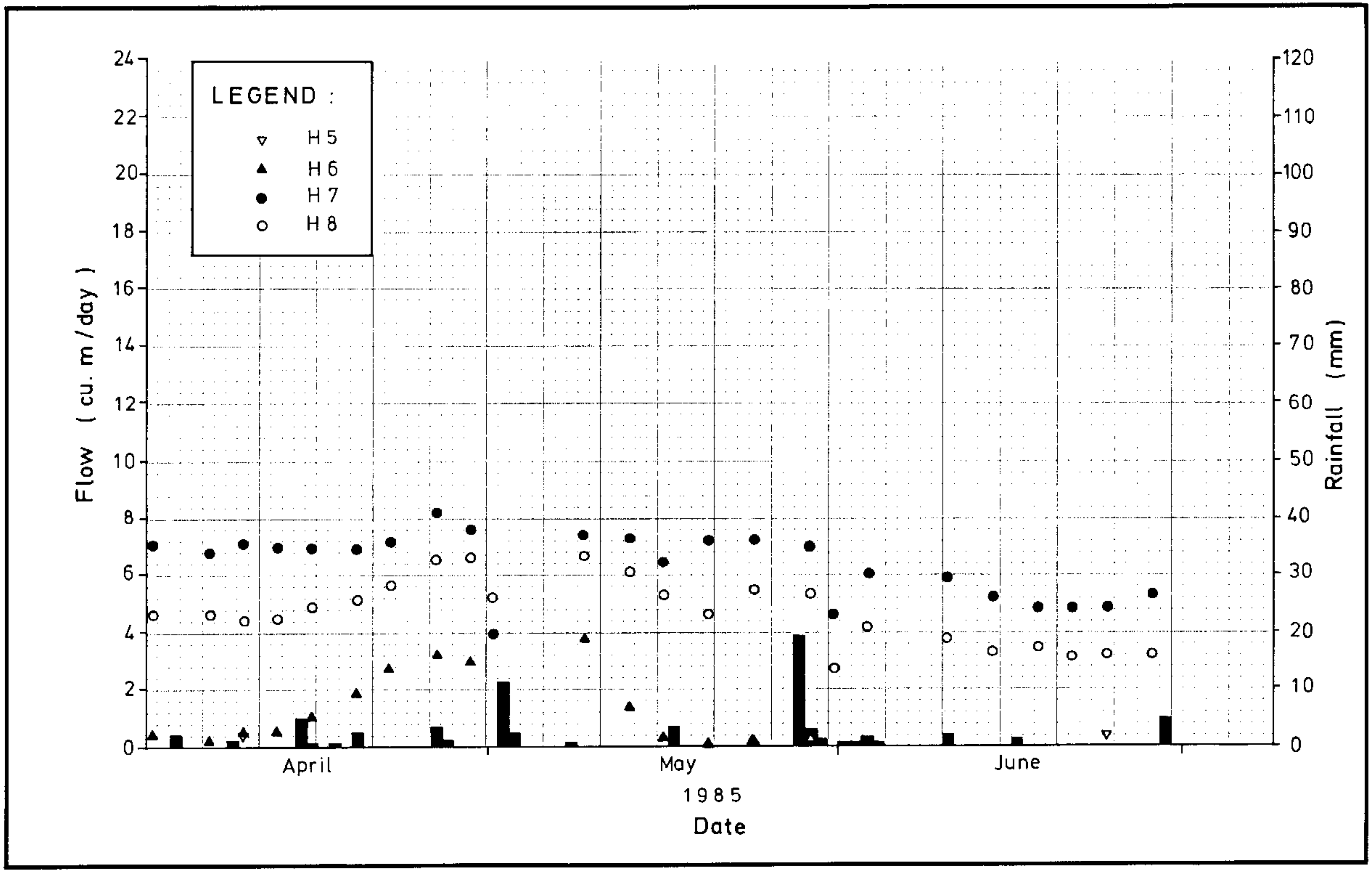

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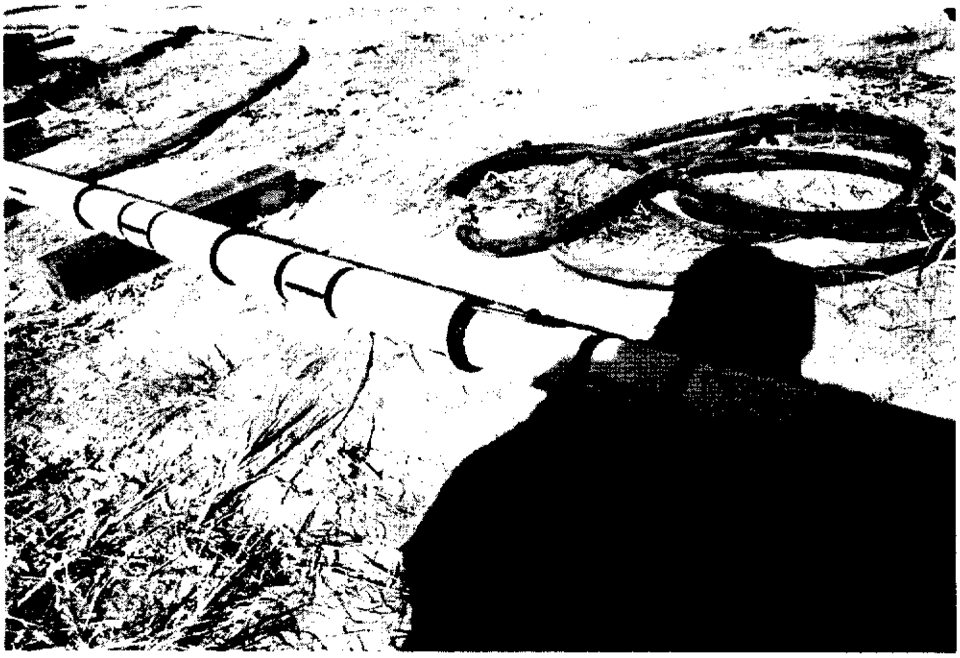

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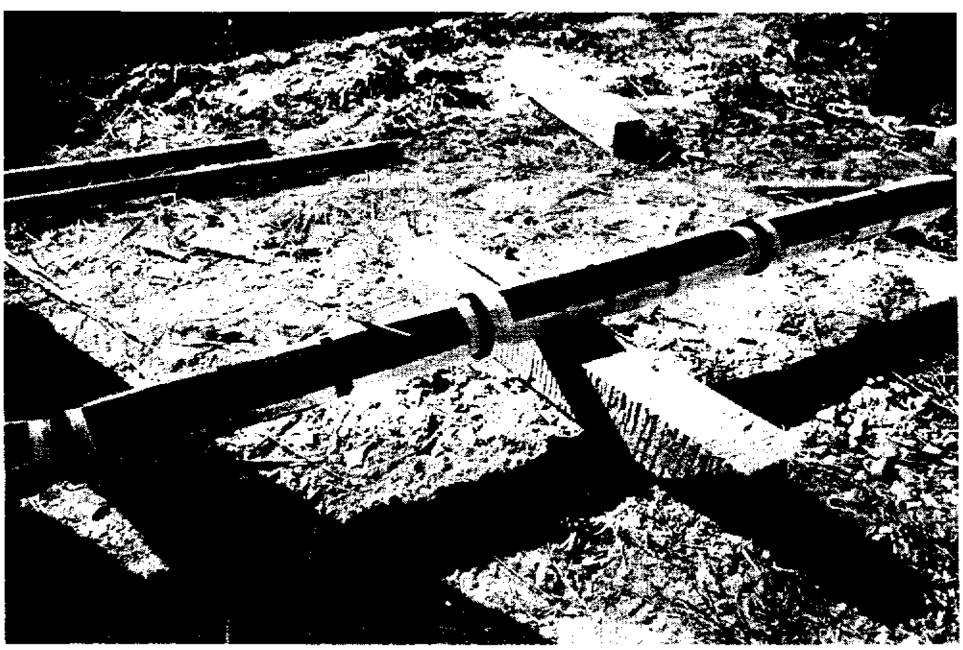

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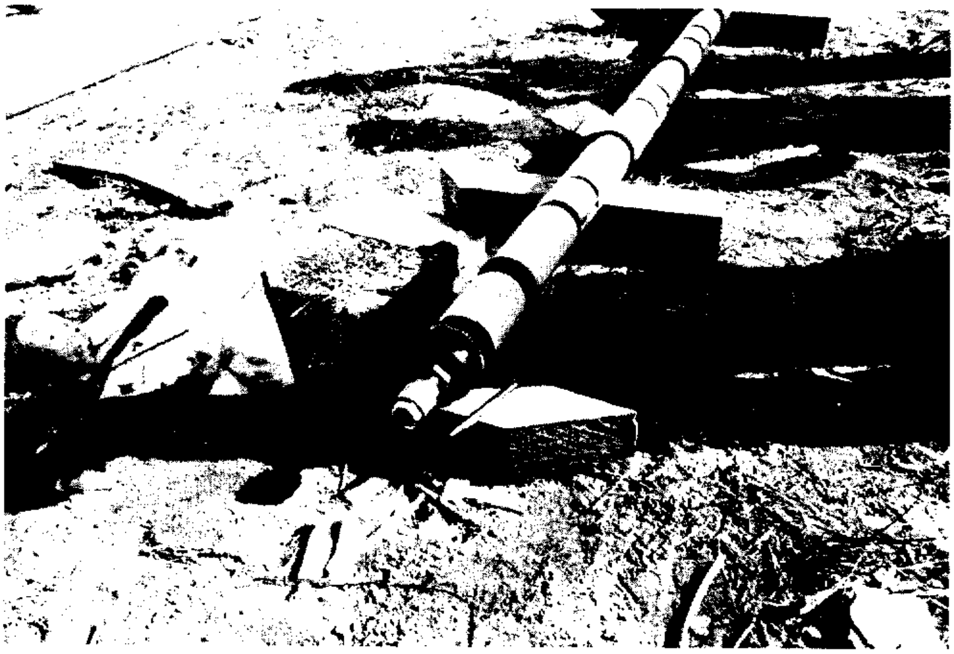

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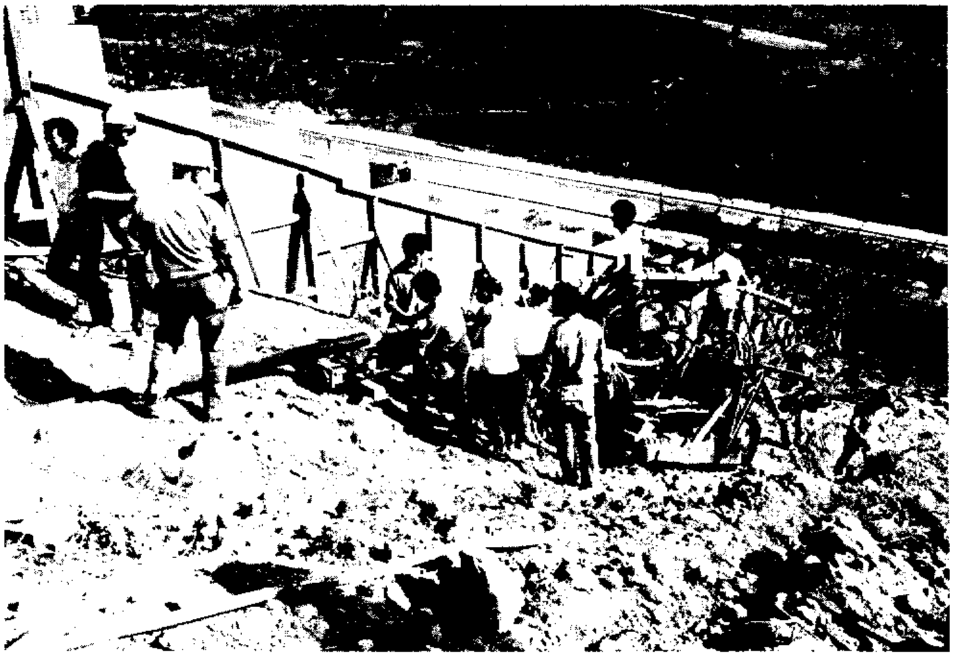

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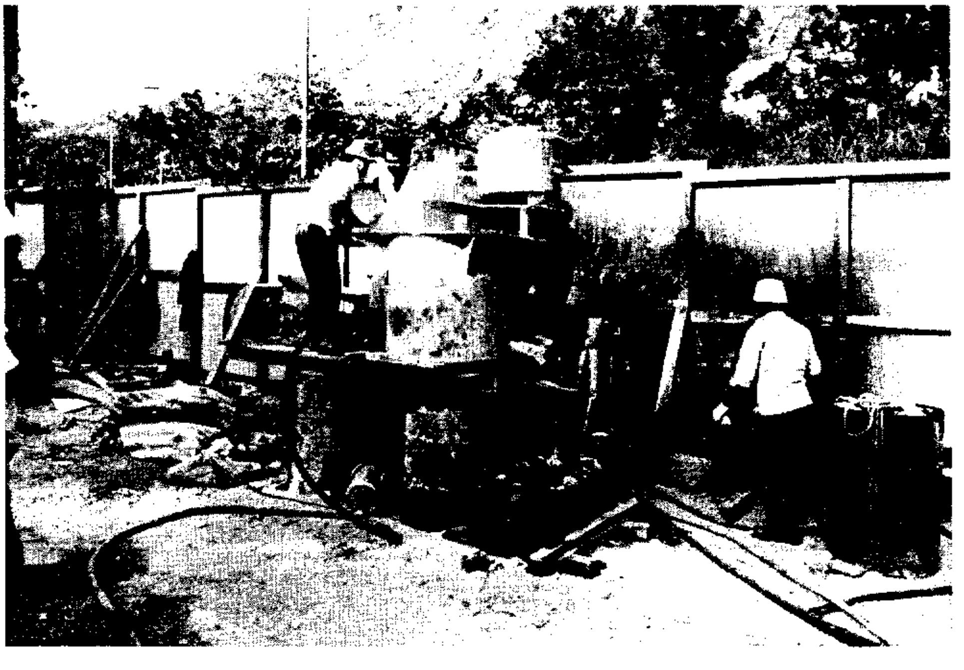

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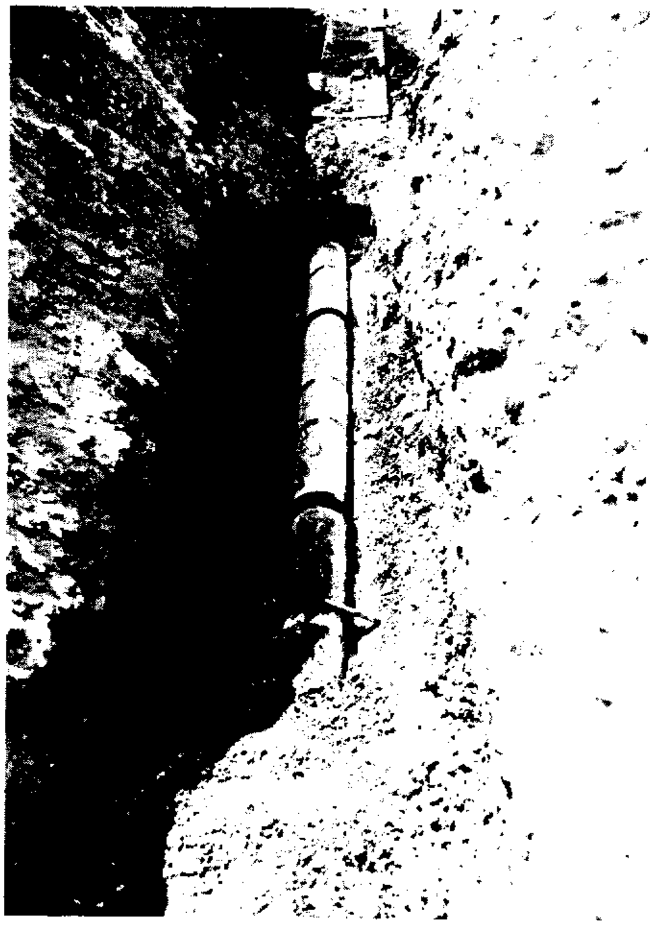

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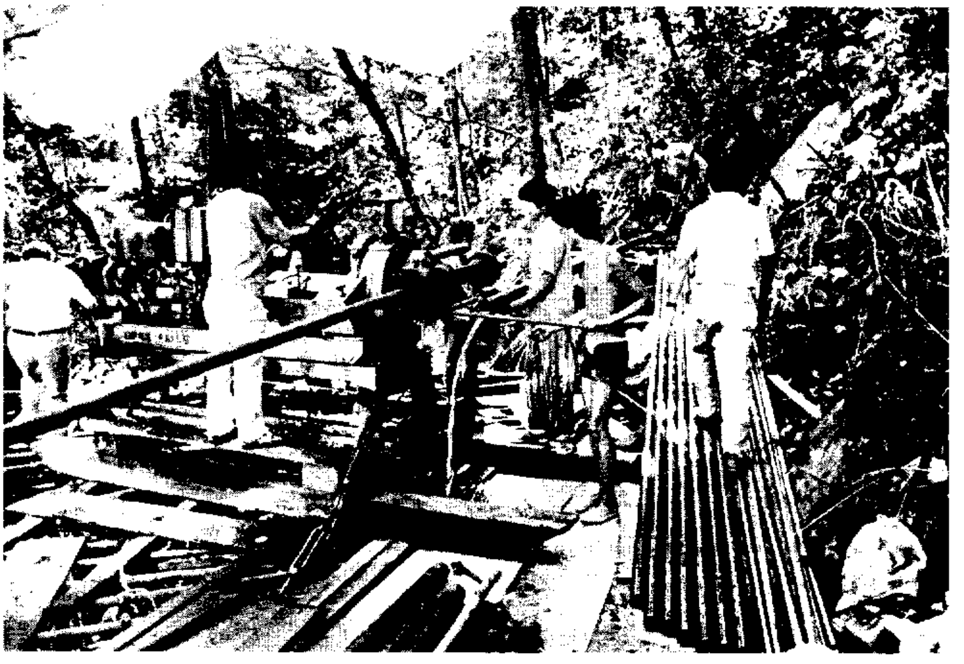

Plate 11 - Drilling Operation in Progress
(September, 1983)

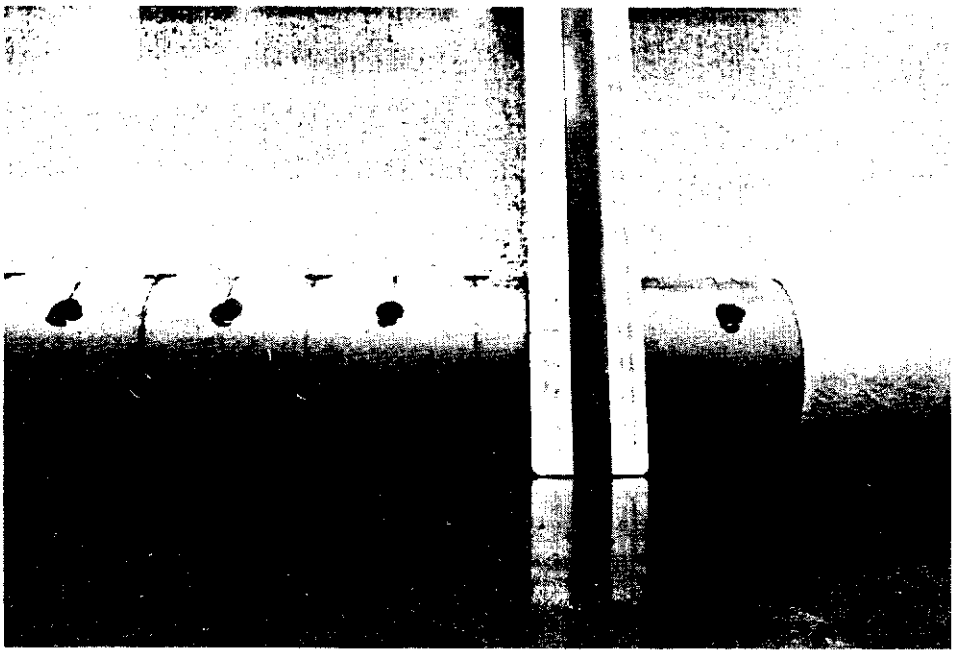

Plate 12 - Section of 50 mm Diameter PVC Drain Liner Showing 7.5 mm Diameter Drainage Holes at 50 mm c/c. Terram 1000 Filter Fabric Is Attached to the Pipe Using Contact Adhesive, as Shown at RHS of Photograph (October, 1983)

Plate 13 - 50 mm Diameter Slotted PVC Screen
(September, 1983)

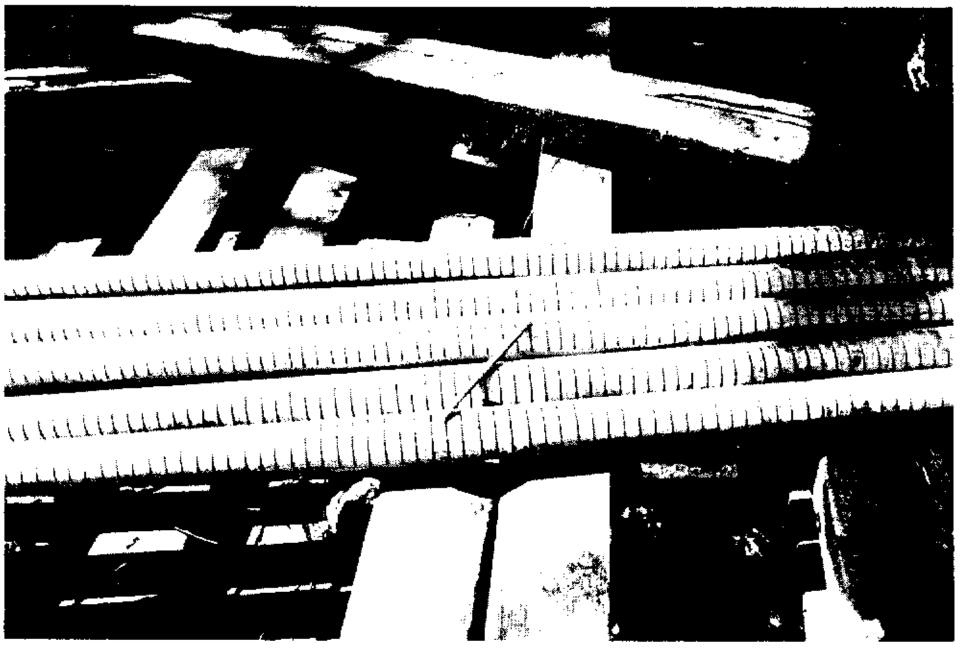

Plate 14 - 40 mm Diameter Slotted Galvanised
Iron Pipe Screen
(September, 1983)

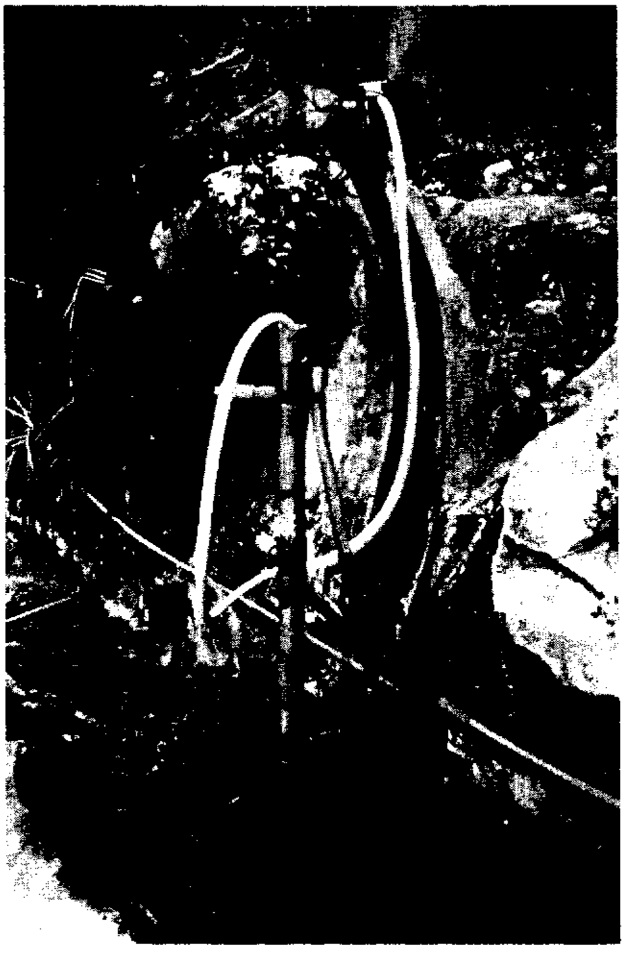

Plate 15 - Apparatus Used for Carrying Out Tests on Horizontal Drains. Note Tap on Drain, Pressure Gauge and Flowmeter (January, 1984)

APPENDICES

APPENDIX A

DEFINITIONS OF TERMS

DEFINITION OF TERMS

Aquifer

An aquifer is a permeable water bearing stratum.

Unconfined Aquifer

An unconfined aquifer is a permeable water bearing stratum only partly filled with water and overlying a relatively impervious layer. Its upper boundary is formed by a free water surface at atmospheric pressure.

Confined Aquifer

A confined aquifer is an aquifer with impervious upper and lower boundaries in which the piezometric head is coincident with or higher than the base of the upper confining boundary.

Transmissivity (T)

Transmissivity is the product of the average hydraulic conductivity (permeability) and the thickness of the aquifer.

Permeability (K)

Permeability is the rate of flow per unit area of a medium under unit hydraulic gradient.

Specific Yield (S_y)

Specific yield is the volume of water released or stored per unit surface area of the aquifer per unit change in the component of head normal to that surface. This term relates to unconfined aquifers only.

Storativity (S)

Storativity is the volume of water released or stored per unit surface area of the aquifer per unit change in head for a confined aquifer. This value depends on the elasticity of the aquifer material and the fluid.

Horner Plot

A Horner plot is a plot of

$$h \text{ versus } \log_{10} \frac{t + \Delta t}{\Delta t}$$

where h = head,
t = pumping time,
Δt = time after pumping has ceased

Jacob Plot

A Jacob plot is a plot of

$$h \text{ versus } \log_{10} t$$

where h = head,
t = time since the start of pumping

APPENDIX B

SUMMARY OF PUMPING TEST AT PO SHAN

Summary of Pumping Test

A pumping test was carried out on the west side of the Po Shan slip remedial works in borehole 21 (Figure B1). The site of the test was on a steep hillside of 26° average slope in completely decomposed volcanic rock.

The results showed a delayed yield behaviour and considerable inhomogeneity of the ground. Despite this, a reasonably reliable estimate of transmissivity, and hence permeability, was obtained. However, the value for specific yield is uncertain, as the test was affected by rainfall after approximately 10.5 days of pumping. The mean value of specific yield calculated is very low but, despite uncertainties, it is unlikely to be less than half the actual average value. This reflects the effect of boulders in reducing the pore space available for storage, and the high water retention of the well-graded decomposed volcanic rock.

Average values of transmissivity, permeability, storativity and specific yield are given in Table B1. Because multiple analysis techniques on a number of boreholes were used to determine these values, the standard deviation/mean value was calculated. These values, which are shown in Table B1, are probably realistic indications of the reliability of the parameters, the results with a low standard deviation/mean value ratio being more reliable than those with a high value. No correlation between boulder content above bedrock and permeability could be found, and no allowance was made for flow to or from bedrock.

The drilling of the trial horizontal drains in the vicinity of the pumping test is not expected to have had a significant influence on the results obtained.

From the analysis, an effective borehole diameter of 400 mm was calculated for a borehole of diameter 200 mm using oil well test procedures.

Table B1 - Average Hydraulic Parameters for Completely Decomposed Volcanic Rock at Po Shan

Parameter	Symbol	Average Value	$\frac{\text{Standard Deviation}}{\text{Mean Value}}$
Transmissivity	T	1.51 sq m/day	0.33
Permeability	K	0.172 m/day = 1.99×10^{-6} m/s	0.30
Storativity	S_A	2.06×10^{-3}	0.94
Specific Yield *	S_Y	3.01×10^{-2}	1.50

* Based on vertical drawdown for aquifer tilted at 26°

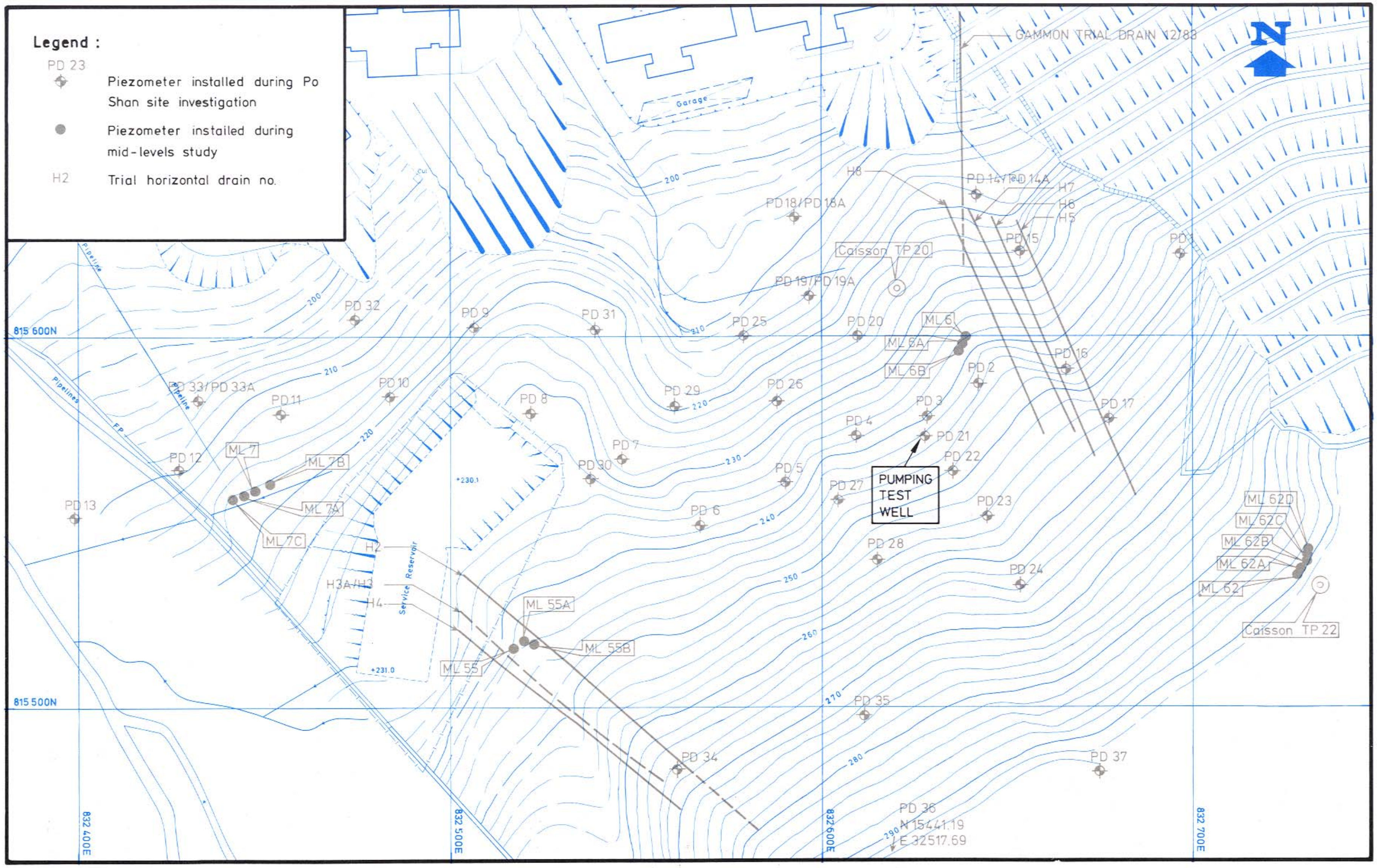

Figure B1 - Location of Pumping Test Well