MID-LEVELS STUDY

REPORT ON GEOLOGY, HYDROLOGY AND SOIL PROPERTIES

GEOTECHNICAL CONTROL OFFICE
PUBLIC WORKS DEPARTMENT HONG KONG

MID-LEVELS STUDY

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PUBLISHED JANUARY 1982

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FOREWORD

The Mid-levels area is a densely urbanised zone of Hong Kong where the scale of development over recent years has dramatically modified the natural slopes. On 29th May 1979, the Temporary Restriction on Building Development (Mid-levels) Ordinance was enacted which prohibited further building up to 31st December 1981, so that Government could carry out a study of the geotechnical factors that should govern future development in the area. The restriction applied to an extensive area from Hong Kong University to Glenealy Valley.

The technical direction of the study was a joint venture between the principal consultants and the Geotechnical Control Organisation. A Steering Committee co-ordinated and advised on the technical procedures and interpretation of the results of the various investigations.

This report was prepared as part of the study in order to explain the methods used and summarise the principal technical conclusions on geology, hydrology and soil properties.

The section on geology and the related drawings give a zonal interpretation. In many lots the ground conditions will be different from those established for the relevant zone. Sharp changes in sub-surface profiles were detected during the study, but these are not generally expected to have a significant effect on zonal behaviour. Data obtained by site investigations for future developments will be used to up-date the geological drawings in this report and revisions will be issued at intervals.

The hydrology section includes background information and specific illustrations of local behaviour. The study period included the latter part of the 1979 wet season and extended to the end of the 1981 wet season. No exceptional storms occurred during 1981 and the year was drier than average. The zonal behaviour was based on the 1979 and 1980 data.

The shear strength testing was carried out at the GCO Laboratories at North Point under the direction and control of the consulting engineers. Standard equipment was modified so that non-standard tests could be tried. The shear strength assessment was based solely on testing carried out on high quality soil samples obtained specifically for the study.

A considerable amount of basic factual data was collected and generated by the study team. Numerous reports were prepared during the study; and those that include basic factual data are included in references 1 to 39. Because this information could be of assistance to those working in the Mid-levels area the Geotechnical Control Office maintains a Data Bank of basic factual information from where basic data can be obtained. The sub-surface data for the study came from various sources:

a) Site investigations carried out specifically for the study under the technical supervision of the consulting engineers, who described the cores and prepared the logs.

b) Site investigations previously carried out by Government and private developers.

c) Excavations at current construction sites that were logged by the consulting engineers with the consent of the developer.

The data available from previous investigations were of variable quality and an assessment of reliability was made so that doubtful information was not allowed to have an unreasonable influence.

The piezometer records for the study are available through the GCO Data Bank as will be the records of selected piezometers which will continue to be read by the Geotechnical Control Organisation.

This study dealt with the general slopes and not with the local slopes at each individual lot. Further site investigation will be necessary especially to obtain information for foundation design and local slope stability.

S. Rodin
Geotechnical Adviser
Hong Kong Government

Members of the Steering Committee

Chairman :	Mr. S. Rodin	-	Geotechnical Adviser, Hong Kong Government
Members :	Dr. E.W. Brand	-	Principal Government Geotechnical Engineer, Hong Kong Government
	Dr. A.W. Malone	-	Government Geotechnical Engineer/ Buildings, Hong Kong Government
	Mr. D.J. Eastaff	-	Binnie & Partners (Hong Kong)(to December 1978)
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	Dr. D.J. Henkel	-	Ove Arup & Partners Hong Kong Ltd
	Prof. P. Lumb	-	University of Hong Kong

Consulting Engineers

Principal consultants:

Ove Arup & Partners Hong Kong Ltd

Binnie & Partners (Hong Kong)

The following consultants were appointed to advise on specific aspects of the study:

Halcrow International Partnership

Dr. B.P. Ruxton	-	Consultant geologist
Dr. R. Herbert	-	Hydrology Unit, Institute of Geological Sciences, Wallingford, UK
Dr. M.G. Anderson	-	Department of Geography, Bristol University, UK

Site Investigation Contractors

Gammon (Hong Kong) Ltd

Bachy Soletanche Group

GEOLOGY

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(bound in separate volume)

<u>LIST OF DRAWINGS</u>
(bound in separate volume)

1. INTRODUCTION

This study consisted of an examination of the slope stability of the Mid-levels residential area of Hong Kong Island. The study began in July 1979 following several reports on the stability of localised areas of Mid-levels.

This report gives an interpretation of the geology of the Mid-levels study area.

For the purpose of the study, Mid-levels was divided into five areas based on major drainage lines, bounded to the south by Lugard Road and two arbitrary lines to the north. These five areas are named from east to west, Glenealy, Seymour, Central, Po Shan and University and are shown in Figure 1.1. For ease of reference the topographic features within each part have been named and are shown in Figure 1.2.

The study began with a desk study reviewing all the available information in the form of news reports, photographs and maps. This was followed by a study of past failures and a preliminary site investigation which included field mapping of Victoria Peak. The latter area was included because colluvium is derived from it. A subsequent more detailed and extensive site investigation was carried out over the study area.

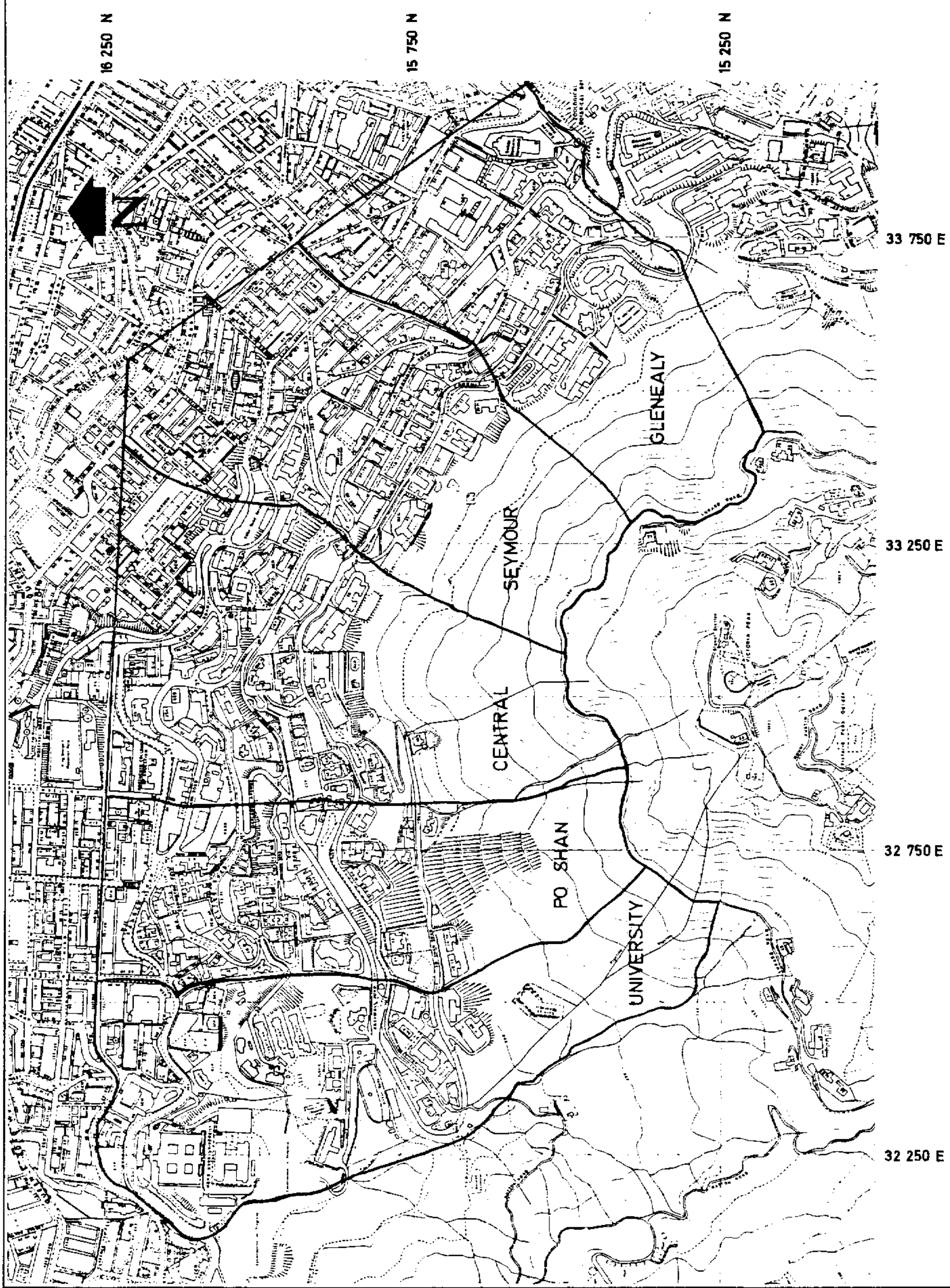
The seismicity of Mid-levels was not examined during this study.

This report on the geology of the Mid-levels area presents the results and conclusions of all the previous work. The text presented here should be read in conjunction with the drawing volume. It should be noted however, that the accuracy of the drawings showing subsurface geology is dependent upon the distance between relevant site investigation stations, see Drawing G19.

The system of soil and rock descriptions used throughout this study is explained in reference 24.

LOCATION OF THE STUDY AREAS

FIGURE 1.1



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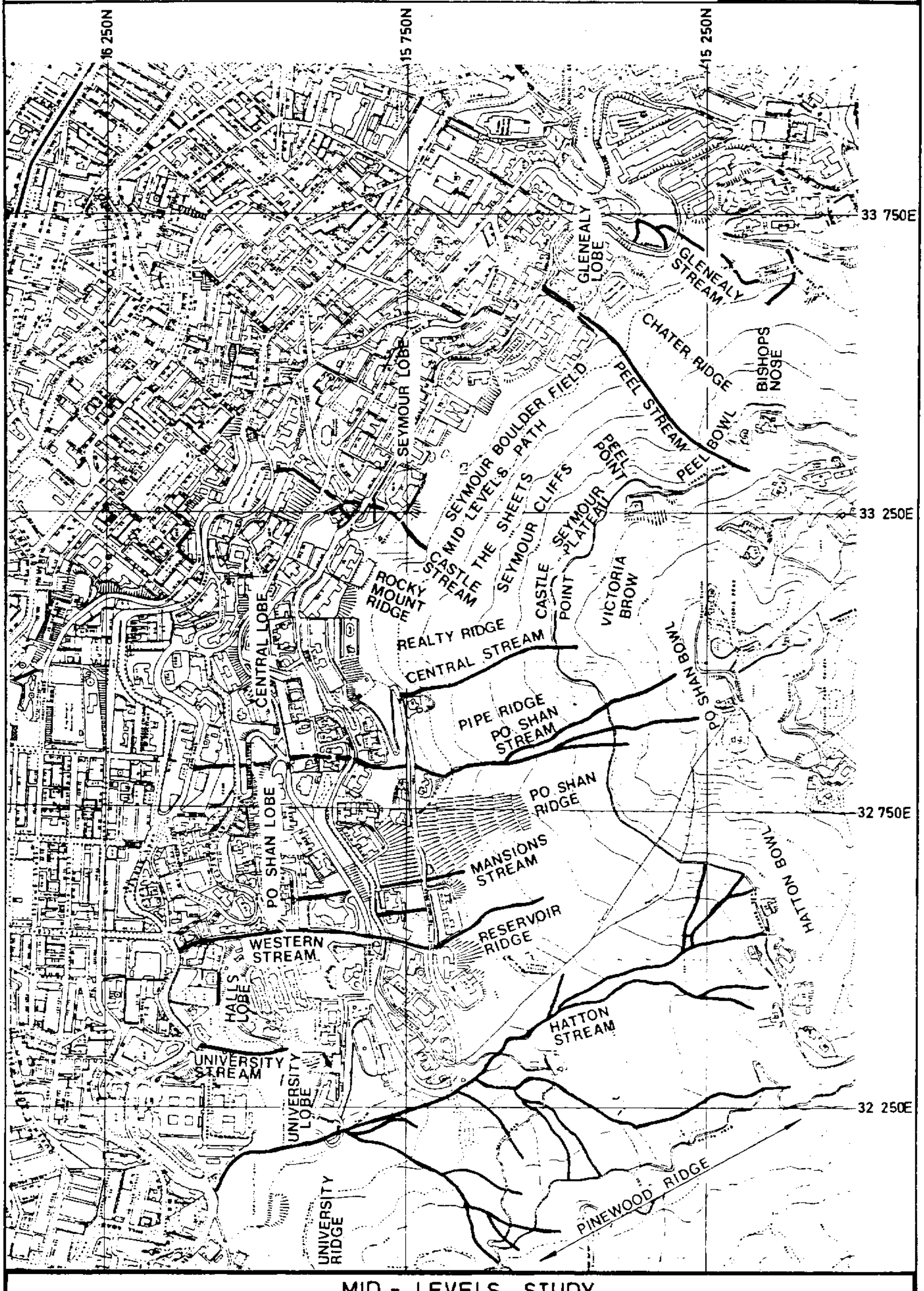
GEOGRAPHY

FIGURE 1 · 2



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2. TOPOGRAPHY

The Mid-levels is situated on the northern slopes of Hong Kong Island. The most prominent topographic feature is Victoria Peak which stands above the study area to an elevation of 550 mPD, see Figure 1.1. The slopes to the north of Victoria Peak fall rapidly away at about 35° and become almost vertical at the Seymour Cliffs which are approximately 100 m in height. Below these cliffs is a ramp slope which extends from about 150 mPD to flatter slopes of approximately 18°. The lower slopes extend down to the harbour edge in a broad concave profile and are characterised by low ridges dissected by streams radiating from the steeper slopes above. At the extreme west the topography changes into a large re-entrant, the Hatton Bowl, flanked by Pinewood Ridge which extends northwards again towards the harbour. The major topographic break to gentler lower slopes in the Hatton Bowl area is at 190 mPD.

The lower areas from 170 mPD down to the harbour edge have been completely urbanised since the beginning of the century, the only major addition being Po Shan Road in 1924 which is also the highest road at 190 mPD. The urbanisation has resulted in extensive modification of the original topography by cutting and filling to construct roads and buildings.

3. MAIN GEOLOGICAL FEATURES

This chapter describes the main geological features of Mid-levels, while subsequent chapters deal with particular aspects of the geological studies.

The rocks of Mid-levels consist of volcanics of the Repulse Bay Formation which have been intruded by the Hong Kong Granite. Dykes have subsequently intruded both rock types.

A summary of the geological history of Mid-levels is given in Table 3.1.

Alteration of the volcanics prior to the granite intrusion resulted from regional metamorphism. Contact metamorphism has produced a zone of slightly coarser grained volcanics while a chilled margin in the granites is slightly finer grained.

The volcanic rocks occur in the southern part of the area where rock outcrops are common. A layer of decomposed volcanic rock occurs in this area between outcrops. The decomposed rock gradually thickens to the north and reaches a maximum near the granite contact in the western part of the area.

Fresh granite outcrops are uncommon. The best exposure lies immediately below Seymour Cliffs. Decomposed granite thickens from this exposure in a generally northerly direction and is thickest near the northern boundary of the area.

The boundary between the granite and volcanic rocks crosses the study area along an irregular line. It can be seen in an outcrop immediately below Seymour Cliffs, see Drawing G20. Elsewhere it has been detected from subsurface information.

The contact has been faulted in several locations. Fault E striking in a NW direction offsets the contact to the north some 300 metres, see Drawing G20. Several other smaller faults have been identified which are parallel to the two main photolineament strikes. These are discussed in Chapter 6.

Colluvial deposits blanket most of the slopes. Three classes of colluvium produced at different times have been identified in the field.

TABLE 3.1 SUMMARY OF GEOLOGICAL HISTORY IN MID-LEVELS

Period	Age in million years	Events
QUATERNARY Recent Pleistocene	2	Deposition of colluvium and other superficial deposits
TERTIARY Palaeocene	65	Intrusion of dolerites
CRETACEOUS Upper		Earth Movements : Folding, faulting and widespread shearing
CRETACEOUS Lower	135	Completion of cooling of Hong Kong granite
JURASSIC Upper		Intrusion of Hong Kong granite
JURASSIC Middle and Lower		Earth Movements : Folding and regional metamorphism of Repulse Bay Formation
	180	Extrusion of Repulse Bay Formation lavas and tuffs

After Allen and Stephens (1971) - Reference 41

4. ROCK UNITS

4.1 Granite

The Hong Kong Granite which has been intruded into the volcanics underlies the majority of the developed area of Mid-levels and the lower undeveloped slopes in the Glenealy and Seymour Areas. The granite extends across the harbour to Kowloon.

The rock is generally coarse grained with approximately equal sized, subrounded grains of 2 mm to 4 mm diameter. Within about 0.5 m of the contact with the volcanics, the granite occasionally becomes a fine grained microgranite.

Microscope examination of thin sections of granite show that it is composed of orthoclase feldspar, plagioclase feldspar, quartz and biotite. The quartz forms 40% to 60% of the rock with about 5% biotite and the remainder feldspar. The orthoclase feldspar and plagioclase feldspar usually occur in the proportion 2:1. Occasionally the rock contains a trace of muscovite. The feldspars are generally twinned and show at least slight turbidity due to incipient alteration. Thin-sections show the crystals to be generally closely interlocked, and this, together with the coarse grain size indicates that the rock cooled slowly at depth.

The overall colour of the rock is grey or pinkish grey when fresh. The quartz is a uniform, transparent grey; the orthoclase is usually white from incipient alteration but may be pink or brownish pink; the plagioclase is usually white and the biotite is dark brownish green. Occasionally the rock becomes greenish grey in colour as encountered in drillhole ML100A between about 48 m and 52 m depth (reference 26), where hydrothermal alteration of the biotite has produced green chlorite which pseudomorphs the biotite or penetrates the entire rock as fine crystals. This altered zone contains closely jointed areas and veins of granite pegmatite, which may have been the pathways for the hydrothermal fluids. The joints are coated or cemented by chlorite or mixtures of feldspar and quartz.

Veins of fine-grained microgranite can be seen cutting the granite in exposures and very occasionally in drillcores. These veins are typically 200 mm to 300 mm in thickness and often are not planar for more than a few metres. Thinner veins of 10 mm to 20 mm thickness have been found and are most conspicuous in exposures of decomposed granite. They have not been found in sufficient quantity to determine whether they are more common near the volcanic contact. Veins of almost pure feldspar were encountered in drillholes ML8, ML112 and ML122, see reference 26.

Xenoliths of volcanic rocks are extremely rare. A few cobble or gravel-sized fragments were seen on the rock exposures above Conduit Road within about 10 m of the contact.

The jointing of the granite, its contact with the volcanic rocks and its weathering are discussed in Chapters 6 and 7.

4.2 Volcanics

The majority of the volcanic rocks in Mid-levels belong to the 'Pyroclastic Group' of the Repulse Bay Formation, the rest are 'Coarse Tuff' as defined in reference 41. The latter consists entirely of uniform coarse tuff, while the pyroclastics include fine tuff, coarse tuff, lapilli tuff and ignimbrites. The 'Coarse Tuffs' are only found along Pinewood Ridge, see Drawing G20.

The best exposures of the least decomposed volcanic rocks are in the Seymour Cliffs. Other smaller exposures are found in the vicinity of Lugard Road and above. Isolated exposures also occur at lower levels on the steep slopes, generally in stream beds.

Decomposed volcanic rock underlies virtually the entire upper slopes between hard rock exposures. Superficial deposits cover most of the decomposed volcanics except in the vicinity of the summit of Victoria Peak and along the crest of Pinewood Ridge.

Knobbly tuff is common in the eastern part of the area including the summit of Victoria Peak. Knobbly tuff describes those rocks which have been recrystallised by metamorphism and contain aggregations of quartz crystals. Boulders of this tuff are common in colluvium throughout the study area.

Lapilli tuff outcrops on the summit of Victoria Peak and can be identified by its distinctive lens-shaped lapilli.

The pyroclastic volcanic rocks can have a slightly porphyritic texture due to feldspar phenocrysts, while the 'coarse tuff' rocks are uniformly medium grained. Feldspar and quartz are the most common minerals in the volcanic rocks.

When fresh the rocks range between pale and dark grey in colour which on weathering becomes yellowish brown. The slightly porphyritic texture is still evident in all the samples of decomposed volcanics recovered. No grade VI decomposed volcanics were found.

The jointing of the volcanics, their contact with the granite and their weathering are discussed in Chapters 6 and 7.

4.3 Dykes

Dykes are most common in the volcanic rocks and have been found in several locations, mainly close to known faults or photolineaments. These have been identified as dolerite dykes.

Another dyke, probably of porphyritic granite, has been sampled in drillhole ML9A, University Area, see reference 26.

All the dykes are generally steeply dipping but their irregular contacts may dip as low as 30°. The dyke contacts are sharp.

The dykes are variable in thickness, commonly about 0.2 m across but a maximum of 1.5 m was recovered from drillhole ML50C, University Area, see reference 26.

Most dykes have very closely spaced fractures, however no slickensides were identified, possibly due to the generally high rate of decomposition of the rock.

The dolerite dykes all have a uniformly fine grained texture while the porphyritic granite has a fine groundmass with medium grained quartz phenocrysts.

Decomposed dolerite dykes have a distinctive dark yellowish brown colour and their silty feel makes identification easy. One moderately decomposed dolerite dyke, sampled in drillhole ML50 (reference 26), has a dark green colour and moderate strength from field estimates on drillcores. No fresher dolerite dykes were recovered.

The granitic porphyry recovered in drillhole ML59A was completely decomposed to a sandy silt with a white to pinkish yellow brown colour.

One aplite dyke was also recovered in the above drillhole. It is 0.3 m thick, white in colour and consists of completely decomposed slightly sandy silt.

5. METAMORPHISM

Thermal metamorphism of the volcanic rocks has been caused by the instrusion of the granite. On Victoria Peak and in the Mid-levels area, an attempt has been made to map zones of volcanic rocks with different grades of metamorphism based on a classification of the various mineral assemblages found in those rocks. Rock samples have been collected and petrological slides made for examination by microscope.

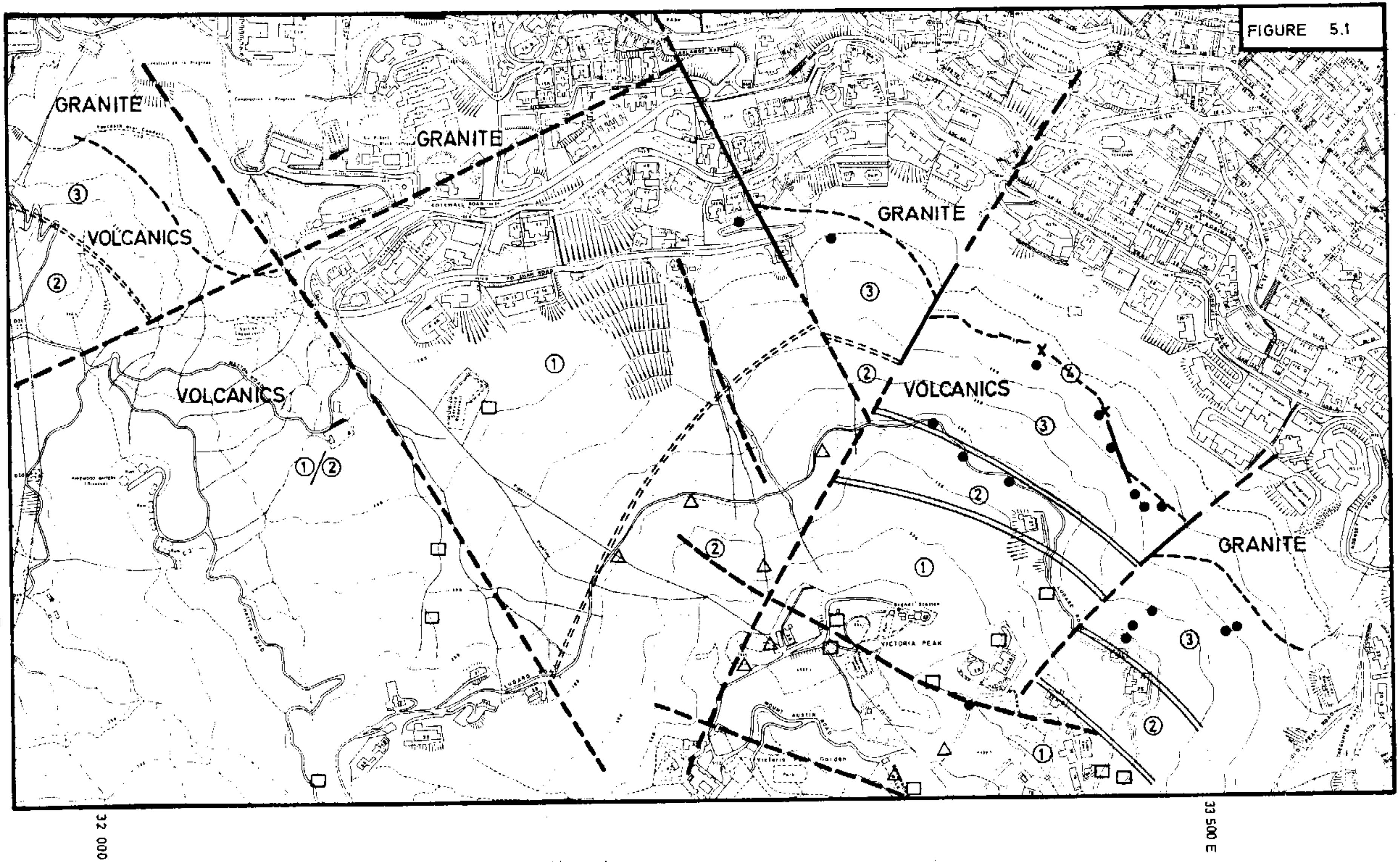
Figure 5.1 shows the approximate position of the boundaries (isograds) between the various metamorphic grades as determined from an examination of rock specimens and studies in the field.

The highest grades of metamorphism occur as would be expected close to the granite contact, with progressively lower grades further away; however there are exceptions to this pattern. Rocks subjected to medium grade metamorphism were found high on Victoria Peak and a considerable distance from the granite contact. Previous workers (reference 41) have explained this by postulating a rise in the granite contact beneath the Peak or a separate granite intrusion, or alternatively have suggested that the index metamorphic minerals have formed by metasomatism.

The variation in grade on the top of Victoria Peak could be explained by variation in the original rock mineralogy.

The variation in grade is also complicated by faults which may have displaced rock units since metamorphism, see Figure 5.1.

The general low grade of metamorphism within the area between Hatton Stream, Central Stream, Lugard Road and Kotewall Road may be explained by block-faulting of the entire area downwards, possibly hinging near Lugard Road. In this way low-grade metamorphic rock originally some distance from the granite is brought into juxtaposition with it.



LEGEND :-

Fault

Photolineament (possible fault)

Granite Contact

Isograd - approximate

Isograd - uncertain

Metamorphic grade:

① - zone, □ sample with thin section - Low grade A

② - zone, △ sample with thin section - Low grade B

③ - zone, ● sample with thin section - Medium grade

④ - zone, X sample with thin section - High grade

PROPOSED ZONES OF METAMORPHIC GRADES

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6. STRUCTURE

6.1 Joints

Joints are present in all the rocks of the Mid-levels area. Tectonic joints are predominant in the volcanic rocks. They also occur in the granite but in this rock type the dominant joints are sheeting joints, as exposed in Seymour Cliffs, which are formed by stress relief during erosion.

The spacing of joints, their continuity and infilling are significant for their effect on rock mass permeability and slope stability of soil or rock exposures. For example, permeability tests in drillholes in bedrock show granite to be less permeable than the volcanics. This is mainly due to the increased spacing of joints in the granite compared to the volcanics. In exposed rock scarps joints provide potential slip surfaces. Similarly the stability of decomposed rock profiles may be reduced by relict joints.

6.1.1 Joints in granite

Joints in the granite range from medium to very widely spaced and are widely spaced on average. For example, sheet joints exposed in Seymour Cliffs are spaced 1 m to 3 m apart and joints normal to them (cross-joints) from 1 m to 10 m apart. In weathered profiles the joint spacing, measured from drillholes, appears to decrease to a minimum of 0.2 m within about 1 m of rockhead but many of these fractures may have been formed by the weathering process. By comparison in drillholes ML51 and ML121, the granite is very widely jointed up to rockhead. Occasionally very closely jointed zones up to about 1 m thick were found in drillholes, for example ML100, see reference 26.

The orientation of granite joints measured from the rock exposures of Seymour Cliffs is summarised below.

Joint Set	Dip Direction	Dip	Surface Properties
Major (Sheet)	NE-NNE	30-40°	50m continuity, slightly curved
Minor 1	SW-SSW	60-80°	20m continuity
2	NW or SE	80-90°	20 m continuity, planar
(rare) 3	E or W	30-50°	40m continuity

The cross joints (sets 2 and 3) continue across the granite/volcanic contact. The sheeting joints terminate at the contact but joints of limited continuity of the same orientation are occasionally developed in the lower 20 m of the volcanics.

Some drillcores from the Glenealy and Seymour areas showed joints in granite rock inclined between 30° and 40° which may belong to the sheeting joint set. These joints appeared down to various depths below the surface of in situ material and are tabulated below. No relict joints of the same inclination were found in the completely decomposed granite sampled in these drillholes, presumably having been obscured by decomposition.

Drillhole	Depth (m)	Drillhole	Depth (m)
ML2	12.7	ML105	14.5
ML100A	30.8	ML106	32.3
ML101	17.4	ML107	23.4
ML102	14.4	ML110A/B	12.6
ML103	32.5	ML113	28.0

Lack of exposure prevented confirmation of these joint orientations elsewhere in the study area.

In fresh rock exposures joints are generally closed and only slightly ironstained.

Staining of joints is usually found within 10 m below rockhead but rarely below 20 m. The infilling of joints by washed-in soil occurs only within one or two metres of an exposed rock surface. Very occasionally drillholes have recovered cores with joints filled by up to 10 mm thickness of feldspar or feldspar with quartz that has presumably been deposited by hydrothermal action.

The thickness of decomposed rock extending into the rock fabric each side of joints generally decreases with depth below rockhead. Similarly joint openings are usually at a maximum adjacent to rockhead and decrease with depth.

With further decomposition of the rock mass, the joints remain as relict features with coatings of limonite or manganese dioxide. These relict joints in exposures and drillholes are usually less than 10 mm thick and may contain a thin layer of clay. Occasionally the relict coating is slickensided, as seen in chunam strip CS6, (reference 27).

An example of joints influencing rock slope stability is evident in Seymour Cliffs. Lying on sheet joints above Conduit Road are many blocks of granite which slid from the cliff face along locally decomposed joints. On the lower vegetated slopes a number of slips have occurred probably by the movement of colluvium or decomposed rock along a basal sheet joint which may be relict.

6.1.2 Joints in volcanics

Joints in the volcanic rocks range from closely to widely spaced and are closely spaced on average.

Joint orientations were measured from volcanic rock exposures in the Glenealy and Seymour Areas and from impression tests in ML153.

Joint Set		Dip Direction	Dip	Surface Properties
Major		NW or SE	80-90°	100m continuity, planar
Minor (rare)	1	SW-SSW	60-80°	20m continuity
	2	E or W	30-50°	40m continuity
	3	NE-NNE	30-40°	a few metres continuity, only near the granite-volcanic contact. This corresponds to sheet joints in granite.

The joints are generally closed at depths greater than about 10 m below rockhead. Peel Point is the exception where staining of joints and the infilling of open joints was observed in drillhole ML153 to depths of more than 45 m. Here the rock mass appears to have been loosened by erosion of rocks supporting either side of this prominent topographic feature.

Relict joints in decomposed volcanic rock profiles have thinner infillings, are spaced more regularly, have a lower continuity and undulate more than in granite. Good examples were found throughout the area in test pits and caissons, see references 27 and 28. In some pits or caissons relict joints were absent and are therefore presumably widely spaced, such as in caisson K2 Ring 2, or cannot be distinguished due to the great variation in colour and texture of the decomposed rock.

6.2 Faults

Faults in the study area have been identified from field exposures or information from drillholes, see Drawing G20. They have two main strikes, approximately NE/SW and NW/SE. This corresponds with the main photolineaments discussed in 6.2.1.

Dykes are commonly found near faults or photolineaments.

Each fault identified in the study area forms part of a photolineament, see Drawing G20 and Figure 6.1 respectively.

Faults A, B and D are not exposed but they offset the granite/volcanic contact and probably the metamorphic isograds also. Fault A is associated with pegmatites exposed along the Mid-levels Path, and Fault D corresponds with a zone of deep weathering in the granite. The linear trend of the faults suggests that they are inclined almost vertically.

Fault C is exposed in Seymour Cliffs and strikes 165° with a dip 60°W. It truncates the local doming of the granite/volcanic contact. A subsidiary fault shown in the 1:1000 geological plans (reference 25) is exposed in Seymour Cliffs. It is steeply inclined to the SE and strikes approximately 060°. This fault offsets the granite/volcanic contact but not Fault C, which therefore must have been formed later.

Fault E which offsets the granite/volcanic contact has been intersected by drillholes BP4A and BP8, see Drawing G34. Steeply dipping sheared decomposed granite was recovered in the former and dolerite dyke in the latter. The rocks either side of this fault suggest that movement was greater at the downslope end, that is analogous to a scissors movement. It is a normal fault, the west side moving down relative to the east. The location and orientation of adjacent photolineaments to the west suggest that block faulting down may have occurred, see Figure 6.1.

Faults have been identified in at least two cuttings excavated for building development in Mid-levels. A sheared zone about 0.5 m wide in decomposed volcanics occurs at the rear of 41C Conduit Road and aligns with Fault E. Similarly two faults in the cutting for the Engineering Building, Phase 1a Redevelopment at the Hong Kong University were found. One fault trends SE and is truncated by a second which forms part of a prominent photolineament along the Hatton Stream Valley, see Figure 6.1.

The permeability characteristics of faults in Mid-levels are difficult to determine because few drillholes have sampled them. In addition any permeability change due to the material in fault zones alone, as determined from piezometric observations, was in many cases masked by the change in permeabilities of the different decomposed rock types either side of the faults. The presence of a clay layer or shattered zone for example, will radically affect the permeability across and along them.

6.2.1 Photolineaments

A photolineation study has been made of the western end of Hong Kong Island including the Mid-levels area using high flight aerial photographs taken in 1945, 1956 and 1963. The photographs taken in 1945 and 1956 are particularly useful as the tree and shrub cover is sparse. A considerable amount of the finer ground detail is lost in photographs taken after 1960 owing to re-afforestation.

It has been assumed that any lineations found in the Mid-levels area could represent faults which have weathered differentially.

The majority of the photolineaments identified follow relatively straight valleys, see Figure 6.1. Their straightness suggests that the faults are almost vertical.

Two main strikes are evident, approximately NE/SW and NW/SE. These correspond with photolineaments elsewhere in the Colony.

Additional evidence for the conclusion that the lineations are associated with faults can be interpreted from the drillhole and piezometer data obtained in the Kotewall Road/Robinson Road Area (ML52, reference 26) where a lineation aligned approximately NE/SW is indicated in Drawing G20. The lineation follows the granite/volcanic contact and piezometer readings on either side of the lineation show a difference in levels. Several dykes were recovered in the drillcore. A nearby photolineament could also be associated with faulting.

6.3 Granite/Volcanic Contact

The granite/volcanic contact is complex, being irregular in shape and often sharp on a handspecimen scale, but on a macro scale may form a mixed zone tens of metres wide. There are a number of features such as dykes, pegmatites and inter-fingering of each rock type which have been found at the intruded contact. These types of feature which may not necessarily occur together are shown diagrammatically in Figure 6.2. The nature of the contact varies along its length. For the sake of clarity however, it is shown on plans and sections as a single line. The intruded contact between the granite and volcanics which has not been faulted appears within the study area only east of Central Stream. To the west the contact appears to be entirely faulted. The contact is also offset by faults to a small extent at Castle Stream in the centre of Seymour Cliffs, at Peel Stream and Glenealy Stream.

It is exposed at the base of Seymour Cliffs where it reaches a maximum elevation of about 290 m. In either direction it drops in elevation across the hill slopes and is abruptly moved downslope by Fault E, see drawing G20. The shape of the exposed granite/volcanic contact suggests that it may be locally domed in that area.

At least three drillhole groups ML5, ML31 and ML52 have sampled the granite/volcanic contact zone, see reference 26. These drillhole groups all recovered a cyclic sequence of decomposed granite and volcanics indicating interleaving of the two rock types. It is thought that granite stringers extend into the volcanics as irregular shaped bodies, forming a contact zone, although this is not seen at the exposed contact in the Seymour Cliffs. Faulting has probably displaced the contact at ML52 in at least one direction and possibly two, that is parallel to the north easterly trending contact, and north westerly trending fault. The contact between Hatton Stream and Oaklands Avenue corresponds to part of a distinct photolineament, see Figure 6.1. This lineament together with the distribution of zones of rocks subjected to different grades of metamorphism suggest that the contact may be faulted along this line.

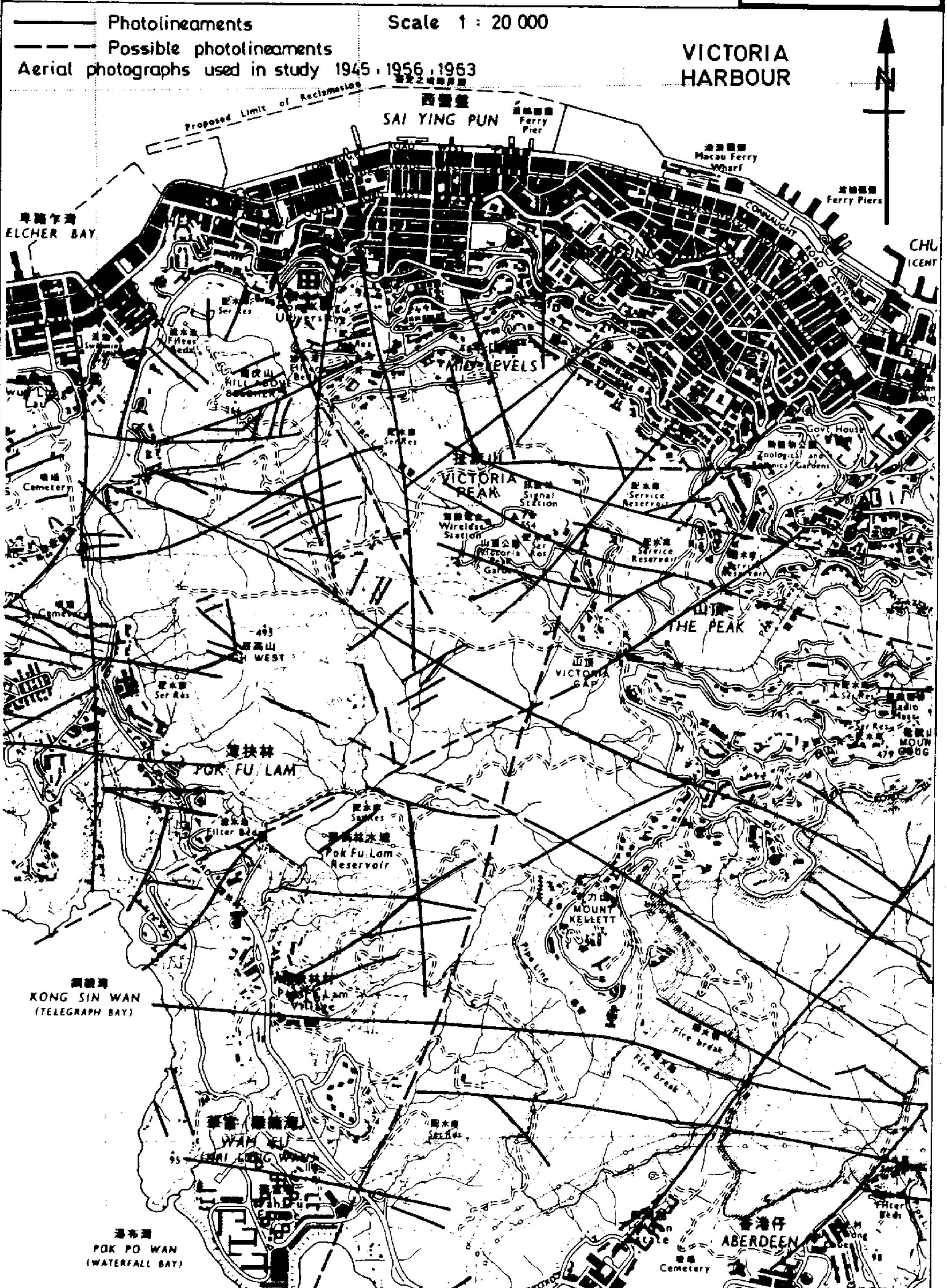
A gradational change in grain size occurs in both rock types as the intruded contact is approached. The generally coarse-grained granite reduces in grain size while the reverse occurs in the generally fine-grained volcanics. The effect is marked in the volcanics which have been recrystallised to a slightly coarser-grain size for hundreds of metres whereas the granite has a finer chilled margin which has a maximum of about one metre thickness. Such a gradational change can be seen where the intruded contact is exposed in Seymour Cliffs.

In general, any change of material properties across the contact zone particularly in the decomposed layer may be considered gradational due to its variable nature.

The granite/volcanic contact is thought to be inclined to the south, away from the centre of the granite body. In the area of Central Fan, combinations of drillholes suggest that the contact most likely dips southwards at a moderate to steep angle. Drillhole ML153 was located about 110 m southwest of the contact exposed in Seymour Cliffs. The drillhole did not penetrate the granite/volcanic boundary at its base level of 214 mPD which indicates that the contact is inclined at least 25° in a southerly direction. The exposure of the intruded contact in Seymour Cliffs indicates that it is irregular in shape and so extrapolation of its orientation is difficult. Faults trending NW/SE and NE/SW which offset the contact in plan are almost vertical as indicated by the straightness of their photolineaments. The only measureable faulted contact is exposed in Seymour Cliffs, that is Fault C which dips southwestwards at about 60°.

PHOTOLINEAMENTS

FIGURE 6.1



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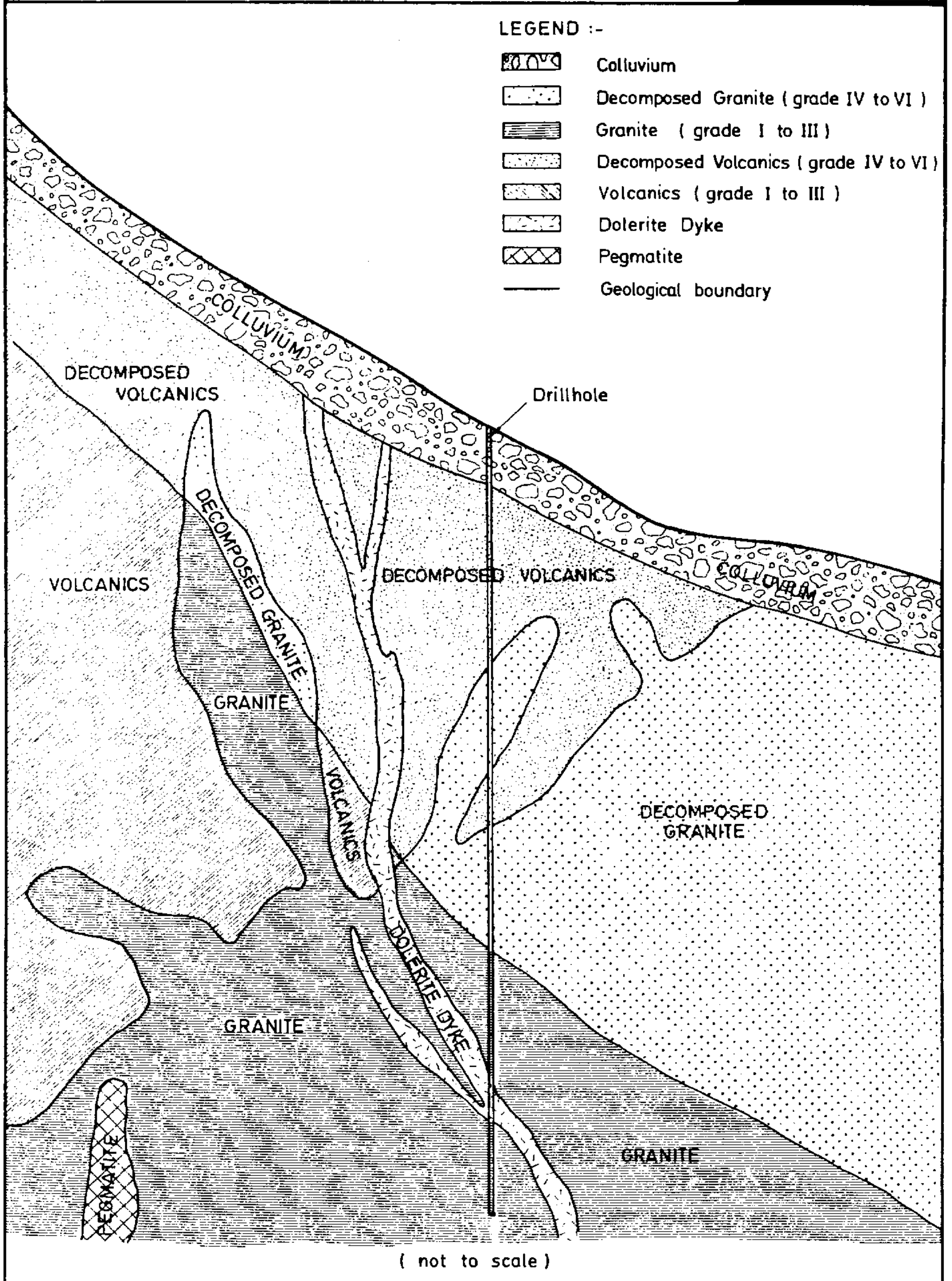
DIAGRAMMATIC REPRESENTATION OF THE GRANITE / VOLCANIC CONTACT

FIGURE 6.2



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7. WEATHERING

This chapter discusses the weathering of in situ rocks which includes both chemical (decomposition) and mechanical processes. The decomposition of colluvium is discussed in Chapter 8. An explanation of weathering processes of the rocks in Hong Kong and details of the mineralogy of the weathering products are given in references 42 and 43 respectively.

7.1 Weathering of Granite

Chemical decomposition is dominant, mechanical weathering being restricted to small outcrops.

The granite is more readily decomposed and eroded than the volcanics in Mid-levels.

Granite bedrock underlies the northern part of the study area, and apart from where it is exposed in the upper slopes is overlain by a layer of decomposed rock.

7.1.1 Depth of decomposition

In general the thickness of the decomposed layer increases northwards in Glenealy and Seymour Areas while in the remaining areas the thickness decreases northwards. The thickness of the decomposed layer is usually 20 m to 50 m but can be up to 90 m; this is shown in the geological sections G1 to G18. There is a gradational increase in the thickness of the decomposed layer across the granite/volcanic contact zone. The rate of thickening is related to the attitude of rockhead. In the University Area between drillhole ML50 and ML50C rockhead dips about 45° in a northerly direction. In the region of Realty Ridge and Rocky Mount Ridge deep weathering of granite adjacent to the volcanic contact may indicate local undermining of the volcanics or a rapid drop in bedrock topography at the contact, see Drawings G13 and G14.

Thicker zones of decomposed granite occur irregularly throughout Mid-levels, see reference 25. Two zones 90 m thick were found, one near the northern extremity of Seymour Area, the other near the centre of Po Shan Area. The former extends downhill from Castle Stream and may be due to enhanced decomposition around possible extensions of Faults D and F. The latter coincides with the intersection of Fault E and the granite/volcanic contact which may also be faulted, (Fault F). The location of these faults helps to explain this thick decomposed zone.

The presence of colluvium will encourage decomposition by protecting the decomposed layer from erosion and maintaining a moist environment. This can be seen at chunam strip CS2 (reference 27) where thick decomposed rock underlies colluvium adjacent to a rock outcrop.

7.1.2 Decomposed rock profile

The idealised decomposed rock profile described in reference

42 is very rarely developed in the study area. Such an idealised profile consists of a decreasing proportion of corestones upwards from rockhead. This corresponds to Zones A, B and C defined in reference 44. The development of a decomposed rock profile corresponds to the rate of decomposition of corestones which is influenced by many factors including fluctuations in the groundwater table, removal of surface material by erosion and climate. The size of corestones is initially determined by joint spacing. Larger corestones are likely to survive longer. In the early stages of development the decomposed rock profile is thin and corestones are common. With continued decomposition the decomposed layer thickens and the zone of corestones remains near rockhead. This process may continue until virtually all corestones have decomposed.

Observations in the study area showed that corestones were more common in the decomposed rocks on the upper slopes than on the lower slopes. This general variation in corestone frequency may be related to a less mature decomposed rock profile on the upper slopes compared to the lower slopes. The former could be derived from a relatively high groundwater table and increased erosion rates ensuring a thinner decomposed rock layer compared to the latter. Glenealy Valley is a good example of high erosion rates restricting the development of a more mature decomposed rock profile.

Granite corestones are generally uncommon but may exceed 8 m diameter. Local concentrations of corestones in decomposed granite were found in chunam strip CS11, drillholes ML107A, 110 and 113, and in recent foundation excavations in Hong Kong University between Knowles Building and University Drive. In the latter area corestones 2 m - 8 m across were found in the top 10 m of the decomposed rock sequence and in several cases at ground level. These unusually large corestones are probably due to very widely spaced joints. This effect may be due to decomposed material in the weathered granite profile having been washed away leaving a concentration of corestones at the surface.

The highest grade of decomposed rock (grade VI) was only found in granite in individual drillholes scattered over the whole area. The details are tabulated below including the thickness of material intermediate between grades V and VI.

Drillhole	Granite Thickness grade VI (m)	Granite Thickness grade V/VI (m)
Glenealy Area		
ML100A	0.4	-
Seymour Area		
ML19B	-	1.9+
ML112A	0.5	-
Central Area		
ML4	0.5	5.8
University Area		
ML59	2.0	6.2

In addition a thickness of approximately 1 m of grade VI was observed in recent construction excavations between Knowles Building and University Drive, Hong Kong University.

The material classified as completely decomposed granite (grade V) can vary in its material properties, see Soil Properties Report.

The least decomposed (grade II) granite is generally white or light yellowish brown in colour with occasionally patchy black staining by precipitated iron and manganese oxides. With increasing decomposition at shallower depths the feldspars become progressively stained more red, purple or brown. Throughout the study area the variation in colour and strength is complex and must be due to differences in original mineralogy, to the period of weathering, the frequency of jointing and proximity to drainage channels and areas of erosion.

7.2 Weathering of Volcanics

7.2.1 Mechanical weathering

Volcanic rocks are exposed in a slightly decomposed state as rock scarps throughout the upper slopes. The relatively undecomposed boulder screes below each scarp show that they are actively eroding. Close examination of the rock faces indicate that the growth of tree-roots in joints plays an important role in levering-off individual blocks. The profile of some scarps indicate that very large falls of rock blocks have occurred, possibly involving many hundreds of cubic metres. This scale of slip must have been caused by a combination of decomposition along joints, a rise in water pressure and possibly by earthquakes. The rock within 5 m of the back-scarps of larger slips shows extensive loosening by the opening of joints, as shown in exposure log EL3 OAP reference 27.

7.2.2 Chemical decomposition

Due to their finer grain size the decomposition of the volcanic rocks appears slower than the granite resulting in thinner decomposed rock profiles.

The joint spacing of the volcanics is closer than the granite and so corestones are rarely greater than 1 m across. Throughout the study area corestones in the volcanics are more common than in the granites, but usually appear only within 5 m to 7 m above rockhead. Upon further decomposition the corestones become highly decomposed and, in areas of knobbly tuff, are gravelly due to the resistance of the quartz aggregations.

When completely decomposed the relict shapes of the corestones in the volcanics are retained more commonly than in the granite, possibly due to the lower permeability of the volcanic soil and consequently the lower mobility of the soluble staining minerals.

The corestones are grey or greenish when fresh with brown patinas. Relict corestones are either brownish green, white or purple depending on the original rock type.

A decomposed rock profile containing corestones set in a soil matrix may be distinguished from colluvium of similar grading by its relict fabric. The corestones usually retain a regular arrangement and orientation, and may be separated by iron-stained relict joints. Also the fabric of the corestones can be traced into the matrix. By contrast, colluvium contains no continuous relict fabric and may contain rock fragments of various types. Within this study the distinction was difficult in the area of Realty Ridge where the in situ decomposed rock and the colluvium consist of a similar rock type. Relict fabric within the in situ material was rare and the careful study of samples and exposures was necessary.

8. SUPERFICIAL DEPOSITS

8.1 Colluvium

Colluvium is a mixture of soil and rock debris that has moved downslope under the force of gravity.

A mantle of colluvium covers the north facing slopes in the Mid-levels study area, see Drawing G20. Large colluvium 'lobes' cover the more gently sloping lower slopes while a relatively uniform blanket of colluvium is found on the upper slopes. The areas without any colluvial cover include the slope immediately below Seymour Cliffs, some valley floors on the lower slopes and between the harbour and the snout of colluvial lobes. These areas are defined by the zero colluvium thickness contour on Drawing G22.

Although some individual granite boulders have been found in colluvium in Mid-levels particularly over the eastern part, it can all be regarded as volcanic colluvium.

8.1.1 Colluvium class division

Despite the heterogeneous nature of the colluvium found in the study area three classes of colluvium have been identified from field exposures, investigation excavations and drill core samples, on the basis of different matrix density, colour and state of decomposition of cobbles and boulders. These are defined in Table 8.1. Colluvium found at site investigation stations has been classified and details are given in Appendix A.

As the three classes of colluvium could be identified with the naked eye no systematic microscope examination of colluvium thin sections was made.

It is believed that these classes of colluvium were produced during different geological time periods, as reflected by the increasing grade of decomposition of the cobbles and boulders, class 1 most and class 3 least. This proposal is supported by the colluvial sequence, class 1 overlain by class 2 which in turn is overlain by class 3. In the oldest, class 1, most cobbles and boulders are completely decomposed while those of classes 2 and 3 have patinas of decreasing thickness, see Table 8.1. Evidence from elsewhere in the Colony supports this proposed colluvium class division.

To date there is insufficient evidence to give each class an accurate age. The considerable climatic changes associated with the Pleistocene Era could explain the three cycles of colluvial deposition and erosion. This is discussed in detail in section 9.3.

Testing of colluvium in the laboratory has concentrated on class 2 rather than the other classes due to the latter's relative scarcity in the field. Therefore only engineering properties of class 2 colluvium are known with any degree of certainty, refer to the Soil Properties Report.

TABLE 8.1 DEFINITIONS OF COLLUVIUM CLASSES IN MID-LEVELS

CLASS 1 (oldest)

Matrix	- stiff to very stiff, mottled dark red and yellow brown slightly clayey sandy SILT, with some gravel in the east.
Cobbles and Boulders	- commonly comprising 75 to 100% but may be as low as 25%, subrounded minor angular, mainly highly to completely decomposed, some moderately decomposed, no patinas.
	(Type location, University Area, described in exposure log EL2B, reference 27)

CLASS 2 (intermediate age)

Matrix	- firm to stiff, mottled dark red and yellowish brown, clayey sandy SILT, with some gravel in the east.
Cobbles and Boulders	- commonly comprising 0 to 50% and up to 80%, subangular to subrounded, mainly moderately to highly decomposed, occasionally slightly decomposed. Patinas commonly 30 to 60 mm thick and may be up to 100 mm thick.
	(Type location, Central area, described in exposure log EL2H, reference 10)

CLASS 3 (youngest)

Matrix	- soft to firm, uniform pale brown to yellowish brown, slightly clayey sandy SILT, with some gravel in the east.
Cobbles and Boulders	- commonly comprising 25 to 50% in the west, 50 to 75% and up to 100% in the east, angular to subrounded, mainly slightly to moderately decomposed with patinas generally a few mms and up to 10 mm.
	(Type location, exposures in cuttings along Lugard Road)

Within any one colluvium class there is considerable variation from one deposit to another caused by several factors. For example, the local landform upslope, the type and state of decomposition of the parent material and the amount of erosion and decomposition that has occurred after initial deposition.

Weathering can lead to modification of the colluvium and lead to difficulties in assessing the class. For example, class 2 colluvium matrix normally has a dark red and yellow brown colour which is often mottled. However where it is not covered by class 3 colluvium, the matrix has a uniform pale yellow brown colour, see Drawing G21. This is probably due to leaching out of the dark reds and brown over a long period of decomposition. In addition the density of the matrix over the top metre or so is reduced by voids being produced from rotted tree roots, animal activity and the washing out of clay and silt. The uniform yellow brown colour and lower density similar to those of class 3 colluvium make classification difficult. The patina thickness on cobbles and boulders of class 2 colluvium is thicker than that of class 3 and this diagnostic feature must be used for positive distinction between colluvium classes in this state. All geological and geomorphological information was assessed before each colluvial deposit was classified.

8.1.2 Characteristics of the colluvium classes

a) Class 1 (oldest)

Class 1 colluvium has only been identified in 8 locations in Mid-levels and is the least common of the three colluvium classes, refer to Drawing G21. These deposits are generally only 1 m or 2 m thick, but a maximum thickness of 16.5 m has been found in drillhole H4 in the Central Area.

Class 1 colluvium has been observed to remain stable on steep slopes over considerable periods. For example, class 1 colluvium occurs at the base of an almost vertical cutting up to 3 m high behind the filter beds between University Drive and Kotewall Road, University Area. This cutting has been standing unsupported for at least 80 years. As a further example outside the study area, part of the almost vertical road cutting for Clearwater Bay Road near Choi Hung is about 5 m high and consists of class 1 colluvium. It has been standing unsupported for approximately 30 years.

A high percentage of cobbles and boulders was found in class 1 colluvium. Some are quite angular as in drillhole ML53B while the majority are subrounded as found at the filter beds exposure, see reference 26. The subrounded cobbles and boulders indicate erosion and reworking. Boulders in class 1 colluvium are generally less than 1 m across.

The characteristics of class 1 colluvium are summarised in Table 8.1. The matrix consistency is stiff to very stiff as determined by simple field tests. This may be due to consolidation by subsequent colluvium deposits, washing in of clay and silt from above, dessication or cementation.

This matrix consistency and high cobble and boulder content help explain the stability of class 1 colluvium forming steep cuttings.

The matrix of class 1 colluvium has a characteristic dark red and yellow brown colour caused by the cyclic leaching effects of a fluctuating groundwater table. This phenomenon is also seen in class 2 colluvium but not in class 3.

As the completely decomposed cobbles and boulders are intact most if not all of the decomposition of class 1 colluvium has occurred in place, that is after deposition.

b) Class 2

The most extensive and thickest colluvium deposits fall within class 2. Deposits of 10 m to 15 m thick are common and have been found up to 36 m thick in the Seymour Area, see Drawing G22. They extend from at least elevation 300 mPD in test pit TP36, Po Shan Area to the northern limit of colluvium, approximately elevation 15 mPD. The lower parts of the undeveloped slopes are covered by class 2 colluvium except the area without colluvium immediately below Seymour Cliffs, see Drawing G20. Seymour Cliffs were probably also once covered by colluvium as suggested by remnant patches of class 2 colluvium. For instance, drillhole ML110 recovered 22 m of class 2 colluvium some of which appeared to be in a loose state with several voids suggesting that it may have reslipped since original deposition.

The thick and extensive class 2 colluvium deposits suggest that they occurred as massive slides capable of transporting boulders up to 3 m across. Boulders of this size have been found in class 2 colluvium in many locations, for example, the private caissons at 23 Po Shan Road, Po Shan Area (reference 28) and an exposure in Hospital Road cutting, Central Area (reference 27).

Patinas 30 mm to 100 mm thick of moderately to completely decomposed rock occur on the majority of cobbles and boulders in class 2 colluvium. As with class 1 colluvium this suggests that decomposition occurred in place after colluvium deposition as the patinas would probably not remain intact with movement downslope.

The properties of class 2 colluvium are intermediate between those of classes 1 and 3 which are explained by its intermediate age, see Table 8.1.

As indicated earlier in this chapter the nature of colluvial deposits vary from one area to another depending on several factors including the parent material. This can be seen with class 2 colluvium on Realty Ridge and Chater Ridge compared with that seen on the slopes between Po Shan Road and Lugard Road, see Drawing G21. The colluvium in the former location, as determined by field observations has a high percentage of gravel in its matrix compared with that in the latter area. This is explained by the type of parent material found upslope. Outcrops of knobbly tuff are more prolific above Realty Ridge compared with the adjacent area to the west. The gravel is thought to be the knobbly fragments, possibly lapilli, which have a high quartz content enabling them to resist decomposition and survive movement.

The colluvium deposits which include class 2 on Pinewood Ridge, adjacent to University Area, are considered to be 'less mature', that is they have not developed to the same extent as deposits to the east. For example, the colluvium deposits are thinner, of the order of 1 m to 2 m along the Conduit Path at elevation about 145 mPD. The boulders in this colluvium are subangular suggesting a relatively short distance of travel from the point of origin. The matrix content is low however, probably due to some erosion and reworking removing the fines fraction. Towards the crest of Pinewood Ridge the colluvium has the appearance of a decomposed rock profile that has been subjected to soil creep only. This sequence of soil creep developing into colluvium can be expected in the formation of some colluvium deposits.

c) Class 3

Class 3 colluvium is defined in Table 8.1. This colluvium is found on the steep upper slopes, except the area adjacent to the western side of the Po Shan slide remedial works and immediately below Seymour Cliffs. This colluvium generally has a thickness of the order of 2 m to 3 m but reaches a maximum thickness of 12 m in drillhole ML7, University Area, see reference 19. Field observations indicate that the matrix of this colluvium has the lowest consistency of any found in Mid-levels, see Table 8.1.

The cobbles and boulders are generally slightly to moderately decomposed with patinas generally less than 10 mm. This relatively low state of boulder decomposition and matrix consolidation compared to that of classes 1 and 2 indicate that class 3 is the most recent.

The proportion of cobbles and boulders compared to matrix in class 3 colluvium is generally about 25% to 50% in the west and 50% to 75% in the east. However there are several matrix free boulder deposits (see Drawing G21) which consist of boulders generally 1 m or 2 m and up to 6 m across. The largest boulders are most common below the rock scarps of Seymour Cliffs. The boulder deposit rock types correspond with the rock exposures upslope. These boulders may be subrounded to subangular. Significant subrounded boulder deposits are found on both edges of Po Shan Area and are considered to be the result of reworking and erosion which has washed out the fines material and concentrated boulders in the valleys, see Drawing G21. All streams on the upper slopes flow through or over boulders lying along their valley floors. Those boulders found along Castle Stream and Peel

Stream are generally angular to subangular. This difference in boulder angularity to the Po Shan area may be due to a shorter transport from their point of origin or more recent erosion and deposition. The extensive boulder deposit below Seymour Cliffs contains angular boulders frequently 4 m to 6 m across forming an interlocking layer about 5 m thick. These boulders have probably fallen from the exposed rock cliffs above. The distance of travel of the boulders would have been greatly enhanced by the ricochet surface provided by the sheeting joints in the granite exposure below the cliff.

8.1.3 Genetic development of a colluvium deposit

The most detailed investigation of colluvium at any one site within Mid-levels was carried out near 23 Po Shan Road where some 19 caissons were monitored, 4 of which were logged in detail, see reference 28. All the colluvium logged falls within class 2 but considerable variation was found.

The different types of class 2 colluvium were described as 'colluvium matrix', 'boulder field' and 'bouldery colluvium'. These three types reflect the genetic development of the colluvium deposit and these processes could have occurred throughout the study area.

'Colluvium matrix' refers to colluvium primarily consisting of soil matrix with a low percentage of cobbles and boulders similar to the majority of class 2 colluvium in Mid-levels. 'Boulder field' and 'bouldery colluvium' both consist almost entirely of tightly packed boulders with soil matrix only filling the voids between boulders. These two types differ in the boulder shape and type of matrix infilling. In the 'boulder field' type the cobbles and boulders are angular and the matrix is made up of silt and clay. However the 'bouldery colluvium' has rounded cobbles with a sandy matrix.

The processes which the three types of colluvium have undergone are interpreted as follows. The 'colluvium matrix' is the original undisturbed colluvium deposit. No structures were found in this colluvium type which would be expected from 'flow effects' such as shearing, laminations or segregation of cobbles and boulders from matrix and therefore individual 'flows' could not be identified.

The 'boulder field' and 'bouldery matrix' types are considered as being reworked 'colluvium matrix' by a migrating stream generally following the present day Po Shan Stream. Refer to Figure 1.2. Erosion of the 'colluvium matrix' would result in washing out of the fines leaving the cobbles and boulders which would roll and creep towards the stream course. This process can be seen in stream beds in Mid-levels today. During later erosion and deposition the voids between cobbles and boulders were filled with fine material from slope wash, particularly when the stream had migrated away. The difference between the 'boulder field' and 'bouldery colluvium' is that the latter has been reworked more vigorously or over a longer time or that streams have virtually continuously flowed through it. The thin patinas on cobbles and boulders in both the 'boulder field' and 'bouldery colluvium' probably mainly formed after the infilling of matrix occurred in the former.

The mechanism of producing a 'bouldery colluvium' as described above may help to explain why some precolluvium valleys, such as that below Po Shan Stream, have remained buried with colluvium, while the majority have been exposed and coincide with the present day valleys. For example, it is possible that Po Shan Stream not only flows along the present day surface nullahs but also underground through 'bouldery colluvium' near the base of colluvium.

8.1.4 Colluvium structure

Any structure found within colluvium can help to provide an understanding of the mode of formation and the state of colluvium during deposition, for example, the nature of the colluvium lying in contact with in situ material.

Both diffuse and sharp contacts have been found throughout the area, see Drawing G21. A sharp contact is interpreted as reflecting colluvium deposition causing little or no disturbance of the underlying in situ decomposed rock. This could occur if the flow of colluvium is laminar. In contrast a diffuse contact may have been caused by the incorporation or mixing of the in situ material with colluvium. Diffuse contacts examined in this study are generally approximately 0.5 m thick and up to about 2 m thick in test pit TP74 on Realty Ridge, see reference 27. They may consist entirely of fine material or a mixture of cobbles or boulders within a matrix. A diffuse contact could have resulted from turbulent flow.

Sharp and diffuse contacts do not occur in any regular pattern and are therefore considered to be the function of specific colluvium flow events with local irregularities. The properties of a colluvial flow can change and both sharp and diffuse boundaries may result within a single flow. For example, on steep in situ material slopes the colluvium flow will accelerate, increasing pore pressures so that 'rafting' of boulders is possible, see exposure log EL3B, Po Shan Area (reference 27). On shallower slopes the colluvium flow will slow down and tend to dilate, lowering pore pressures.

The contact between colluvium and in situ material generally dips in a N to NE direction between 15° and 30°. However, locally the contact may dip up to 70° and the dip direction may vary considerably, see Drawing G21. This would suggest that the original in situ material surface was quite irregular in parts before being covered by colluvium.

The colluvium was found to be heterogeneous and no evidence of continuous or extensive layering was found. However two examples of very local layering were found at the contact with in situ material. In the private caisson CSA6 at 3-5 Kotewall Road, Central Area (reference 16), a zone 0.4 m thick of multicoloured finely layered silt or clayey silt with laminae less than 1 mm to about 5 mm thick was observed immediately above in situ material. No slickensides were found. Coloured layering 10 mm thick in silt was also found at 5.8 m in the drill core from ML53B, University Area (reference 26). Again no slickensides were found.

The layering and laminations are interpreted as occurring during laminar flow.

The heterogeneity of colluvium causes large variations in permeability, see Chapter 10.

No shearing was found in any of the exposures or samples of colluvium from the Mid-levels Area. It should be noted that a polished plane with slickensides was found in colluvium in the cutting for the Hok Tau Road, during its construction in 1965/66. In addition 'clayey slip bands' and 'sheared texture' in colluvium are described in logs of drillholes (R series) along Robinson Road, see reference 14. Verification of these descriptions was not possible as these samples could not be located.

Turbulence structures have been described in colluvium at only two locations in Mid-levels, that is, exposure log EL7H (reference 10) and at one location in the Po Shan footpath cutting.

8.2 Alluvium

Deposits of uniform silty sand were recorded in old drillholes near the north east corner of the study area, between a level of -5 m and +20 mPD. They lie directly on completely decomposed granite and may be reworked decomposed granite that has been washed downslope by Castle and Peel Streams. The deposits lie beyond the limit of colluvium except near Gough Street where they may be overlain by colluvium. This possible alluvium is recorded only in old drillholes and poor sampling may have led to misidentification of in situ decomposed granite as alluvium.

No alluvial or marine clays have been found either within or beneath the colluvium.

8.3 Fill

The fill material encountered in this investigation ranges from relatively uniform gravelly silty sand that is distinctly layered, such as that in test pits TP73 or TP97, to a loose deposit of bricks as in test pit TP86, see reference 27.

Fill was encountered at almost every site investigation station throughout the developed areas. The thickness of fill found during the study was usually less than 2 m. Drillhole ML60, recovered 9 m and a maximum of 12.8 m was recorded in drillhole IP8, see references 26 and 14 respectively.

Field observations and standard penetration test results indicate that the fill is generally loose. Rock fragments are commonly cobble size but boulders may be encountered, for example drillhole ML111, see reference 26. Fill is usually distinguished by its low density or presence of man-made materials such as brick or concrete. Where the latter is absent, identification may be difficult particularly where the deposit resembles colluvium.

The fill is usually derived from building foundation platforms cut into the hillside; for example, the fill in drillhole ML111 (reference 26) could have been derived from the cutting in colluvium immediately upslope.

9. GEOMORPHOLOGY

The landforms of the Mid-levels have been mapped at a scale of 1:5000 from aerial photographs and details are given in reference 40. This has since been further correlated with the geological data and some minor revisions have been made and included in Drawing G25. This drawing shows landforms classified by the processes that formed them into erosion and deposition zones. The erosion zones have been further subdivided into:

a) freefall zones: areas of rock outcrop usually at angles greater than 60° where movement of material downslope is by wedge or toppling failure.

b) basal erosion zones: areas characterised by rapid material removal by landsliding causing the retreat of slopes uphill into the landforms above.

c) transportation zones: funnel shaped areas through which debris passes from the basal erosion zones towards the stream channels.

d) stream channels: these may have considerable erosive power due to a coarse bedload derived from the colluvium.

The deposition zones are those areas where landforms have been created or modified by the deposition of colluvium.

Since large sections of the study area have been classed as erosion zones in Drawing G25, which includes a number of slip scarps, a more detailed study of these past failures has been undertaken, see Table 9.1. The terms used in the table are defined in Figure 9.1.

The subcolluvial topography, shown on Drawing G23, is discussed in sections 9.1 and 9.2, for the Glenealy, Seymour and Central areas, and the Po Shan and University areas respectively.

9.1 Glenealy, Seymour and Central Areas

9.1.1 Features of the subcolluvial topography

Comparison of the subcolluvial topography, shown in Drawing G23 with the predevelopment topography, shows that valleys in the surface of the in situ materials are sometimes partly or completely filled by colluvium. In the Glenealy, Seymour and Central Areas there are three valleys that are completely filled by colluvium.

One such valley diverges from the present Peel Stream at about the position of the Mid-levels Path, passes through the position of the reservoir and joins Glenealy Valley just south of Chater Hall. This valley was probably the original course for water collected in the large catchment of Peel Bowl. It was blocked by slip debris which now forms the colluvium on Chater Ridge, diverting the stream northwards. The original course of the stream explains certain features of Glenealy

TABLE 9.1 DETAILS OF PAST FAILURES

MID-LEVELS STUDY AREA - GLENEALY, SEYMOUR AND CENTRAL AREAS

Failure Number	Type	Date	Failure Scar Maximum Width(m)	Failure Scar Maximum Plan Length (m)	Failure Scar Depth	Debris Maximum Width(m)	Debris Maximum Plan Length(m)	Slip Material	Slope Angle	Comments
1	N	?	4	8	2	5	15?	DG	35°-40°	Debris seen in TP 5 OAP
2	N	?	4	3+	2	5	?	G+DG	45°	Debris removed by stream
3	N	?	3	8	2	4	?	Coll 3	45°	Debris removed by stream
4	N	pre '45	20+	?	7	?	?	v	40°-50°	Part of a large rock scarp
5	N	pre '45	8	40	5	10	?	v	40°-50°	
6	N	1966	15	40	5	15	200	DV+Coll	40°-50°	Debris flowed into stream course
7	N	?	10	35	4	?	?	Coll 3	40°	Debris removed by slip No. 8
8	N	1966	5	18	3	8	15	Coll 3	40°	
9	N	?	4	18	2.5	?	?	DV+Coll	40°-50°	Debris removed by stream
10	N	1966	4	8	2	15	70	DV+Coll	35°-40°	Depression of slip area in 1963
11	N	pre '24	9	8	2	8	?	DV+Coll	35°-40°	
12	N	pre '24	10	16	2-3	5	20	DV+Coll	35°-40°	Lower debris removed by stream
13	N	1979	4	5	2	8	25	G	35°-40°	Granite blocks sliding on sheeting joints

TABLE 9.1 (continued) DETAILS OF PAST FAILURES

MID-LEVELS STUDY AREA - GLENEALY, SEYMOUR AND CENTRAL AREAS (continued)

Failure Number	Type	Date	Failure Scar Maximum Width(m)	Failure Scar Maximum Plan Length(m)	Depth	Debris Maximum Width(m)	Debris Maximum Plan Length(m)	Slip Material	Slope Angle	Comments
Old Peak Road										
	C	1966	20	40	3	20	40	Coll	30°-35°	
Chater Hall Garden										
	C	1966	30	40	4	20	80	Coll+DG?	30°-40°	Debris flowed into stream course

MID-LEVELS STUDY AREA - PO SHAN & UNIVERSITY

1	N	1966	17	13	2	7	28	Coll 3	35°	
2	N	1966	7	6	2	4	6	Coll 3	30°-35°	
3	C	1966	15	22	2-5	?	?	Coll 3	-	Debris cleared from Piccadilly Mansions lot
4	N	1966	9	15	2	6	28	Coll 3	25°-30°	
5	N	1966	8	6	2	8	10	Coll 3	25°-30°	
6	N	1966	12	11	2	11	13	Coll 3	25°-30°	
7	N	1966	11	11	5	10	14	DV	25°-30°	
8	N	1966	10	11	2	6	22	DV	25°-30°	
9	N	1966	8	10	2	10	33	Coll 3	25°-30°	

TABLE 9.1 (continued) DETAILS OF PAST FAILURES

MID-LEVELS STUDY AREA - PO SHAN & UNIVERSITY (continued)

Failure Number	Type	Date	Failure Scar Maximum Width(m)	Failure Scar Maximum Plan Length(m)	Failure Scar Depth	Debris Maximum Width(m)	Debris Maximum Plan Length(m)	Slip Material	Slope Angle	Comments
10	N	1966	4	6	2	4	24	DV	25°-30°	
11A	N	1966	7	7	2)	11	110	DV	25°-30°	Debris in stream course
11B	N	1966	11	13	2)					
12	C	1966	9	12	5	9	28	DV	-	Joins 11A & B in stream
13	C	1966	7	6	2	7	4	DV	-	Debris stopped on path
14	C	1966	8	6	2	8	4	DG	-	Debris stopped on path
15	N	1966	14	11	5	?	?	DG	25°-30°	Debris obscure on photograph
16	N	1966	6	6	2	6	11	Coll 3?	25°-30°	Joins 11A & B in stream
17	C	1972	23	5	12	29	100	Coll 3 & DV	25°-30°	Hamilton Court (11/5/72) (16/6/72)
18	Nc	1972	50	50	20	70	210	Coll 3 & DV	35°	Po Shan Slide (18/6/72)
19	N	1966	25	60	5	27	60	Coll 2	35°)	Rear of Po Shan Mansions
20	N	1966	25	60	5	25	80	Coll 2	35°)	

NEW TERRITORIES (example of largest only)

Failure Number	Type	Date	Failure Scar			Debris		Slip Material	Slope Angle	Comments
			Maximum Width(m)	Maximum Plan Length(m)	Depth	Maximum Width(m)	Maximum Plan Length(m)			
Castle Peak (co-ordinates 13400E 29100N)										
	N	?	80	64	5	80	320	Coll 2?	30°	Cliff above slope topography
Kai Keung Leng (co-ordinates 27400E 26700N)										
	N	?	60	35	5	60	150	Coll ? & DV	35°	Ridge, ravine topography similar to Pinewood Ridge
Pak Tai To Yau (co-ordinates 31300E 36600N)										
	N	62/63	30	24	5	18	120	Coll 3?	35°	Slip occurs in 'bowl' topography

Note: Abbreviations used:

- Coll 3 - colluvium and the class number
- DV - decomposed granite
- N - natural slope
- C - cut slope

See Figure 9.1 for definition of terms

Valley. Glenealy is a deep, mature valley and its erosion required more water than would be available from its present, small catchment. Also its depth is much less uphill of the point of entry of the original stream. The infilled valley is clearly shown in the cutting behind the reservoir. Water flow within the colluvium at the base of the infilled valley may have been responsible for the slip in Chater Hall gardens in 1966.

A second infilled valley is exposed in the cutting behind Rocky Mount Garden and extends northwards at least as far as Robinson Road. Castle Stream may have originally flowed down this valley but was diverted eastwards to its present course when the valley was blocked with colluvium. Drillholes along the valley, namely ML120 and ML121 (reference 26) indicate no concentration of boulders in the bottom or any signs of eluviation of fines.

The third infilled valley extends from Breezy Terrace, across High Street to Hospital Road. The valley cuts into what is now a prominent ridge, and therefore is probably a very old erosion feature. This assumption is strengthened by the fact that most of the infilling colluvium may be the oldest, class 1, type.

Other valleys in the surface of in situ material are only partly filled. Presumably here, falls of colluvium have never been large enough to block the stream which continued to flow over, and possibly through, the colluvium.

The upper limit of development follows a distinct break of slope in the subcolluvial topography. This break occurs at about 150 mPD but locally varies between 120 and 160 mPD in the Glenealy, Seymour and Central Areas.

9.1.2 Thickness of colluvium

From the study of the colluvium thickness and of the subcolluvial topography the following features are noted:

a) The thickest deposits, between 15 m and 35 m thick, lie in buried depressions or valleys. Today these deposits remain upstanding to form small ridges or lobe-shapes.

   The thick colluvium at the Jewish Recreation Club, Conduit Road lies in a distinct depression with finer material at the base. The depression forms the head of a shallow sub-colluvial valley which extends north-eastwards towards Aberdeen Street.

b) Colluvium covers substantial ridges in the surface of in situ material. Considerable erosion of the adjacent areas must have occurred since deposition of the colluvium.

   The thickness of colluvium on ridges ranges from zero at the eastern end of Lyttelton Road to a maximum of 15 m at Cliffview Mansions on Conduit Road.

c) The thinnest deposits lie on the upper slopes, in Castle and Glenealy Streams and at the northern and north eastern limits of the area. In the first two locations this is due to continuing erosion.

The loss of colluvium cover on the lowest slopes is also thought to be due to erosion, possibly by a previous rise in sea-level. The alternative explanation that colluvium flows never reached the lower slopes is very unlikely. Boulders up to 5 m across have been found within some of the most northerly deposits suggesting that the energy of the colluvium flows was not at its limit at these locations.

9.2 University and Po Shan Areas

9.2.1 Features of the sub-colluvial topography

A study was made of the sub-colluvial topography (surface of in situ material) given in Drawing G23.

The method of study involved an analysis of the topography at two levels:

a) a clinographic study, that is a study of the average slope angle between successive contours, of the whole topography which would reveal any broad characteristics

b) a study of the stream profiles and slope profiles along the drainage divides which would reveal more localised characteristics.

The clinographic curves, stream and slope profiles were appraised statistically using 'best units analysis' (reference 45) which divides the profile into straight segments and curved elements that have a degree of internal uniformity. This method permits any number of profiles to be analysed with the same degree of objectivity thereby allowing direct comparison and is also a smoothing technique so that anomalies are discarded.

The major break in slope of the sub-colluvial topography is at 160 mPD below Po Shan Road and separates the higher steeper slopes from the lower gentler slopes. It does not appear to be related in any way to the granite/volcanic contact zone. Sharp breaks of slope are a common feature throughout Hong Kong as for example in the North Kowloon and Castle Peak areas. The 160 mPD break found in the University and Po Shan areas is most likely related to some former, higher sea level possibly of Pliocene or early Pleistocene age.

Although the granite/volcanic contact zone is strongly reflected in the depth of decomposed in situ material, it has no surface expression on the lower slopes of the sub-colluvial topography either in the streams or along the drainage divides. This may be due to the coarseness of the sub-colluvial contouring but the density of drilling in the contact zone has been high and therefore gross features should have been picked up. It therefore appears that the lower slopes of the sub-colluvial topography represent a mature landscape where different bedrock types are hardly reflected in slope profiles.

From the 'best units analysis' of the profiles along the drainage divides, there is persistent evidence for a step in the topography below the major break in slope; see Figure 9.2. This takes the average form of flatter slopes of 14° from 110 mPD to 150 mPD and steeper slopes averaging 20° from 75 mPD to 110 mPD. Below this steeper section is another flatter section at 11° with a break to 6° at 55 mPD. This sequence, which is also present in the stream profiles though less noticeably, probably reflects changes in the erosion cycle due to climatic and sea-level fluctuations during the Pleistocene and have since been largely eradicated in the streams.

9.2.2 Colluvium classes related to topography

A study was made of the spatial relationships of the 3 classes of colluvium both within the original topography and the sub-colluvial topography (Drawing G23).

The slope angles for the sub-colluvial topography on which the three classes of colluvium are resting, as calculated from 'best units analysis', are summarised in Table 9.2.

Within the sub-colluvial topography class 1 colluvium has been located on only a small range of slopes mostly at 12° on the tops and sides of ridges. On slopes of 15° or greater no class 1 colluvium has been found. Deposits are present at 140 mPD just below the main break in slope and overlie decomposed volcanics. At lower elevations of 60 mPD to 90 mPD Class 1 colluvium overlies decomposed granite, see Drawings G21 and G23. Class 2 colluvium is present over the whole area up to 300 mPD both above and below the major break in slope except in some of the stream courses in the lower area. On the higher slopes class 2 is overlain by class 3 which extends irregularly down to the major break in slope at 160 mPD. Class 3 colluvium is absent from the Po Shan Ridge and in the zone of basal erosion above 10-16 Po Shan Road, see Drawing G25.

The colluvium surface slope angles for classes 2 and 3 prior to construction in the area are summarised in Table 9.3.

The major break in slope for the pre-construction topography is at 180 mPD which is uphill from the break in slope of the sub-colluvial topography at 160 mPD. Therefore the major break in slope has migrated upslope with the deposition of class 2 colluvium but has been considerably smoothed where class 3 colluvium is present. On the higher slopes class 3 rests on slope averaging 4° steeper than those with class 2, see Table 9.3. On the lower slopes, the deposition of class 2 has had a smoothing effect on the topography which is evident from a smaller range in slope angles and an average slope angle of 2° less than that of the sub-colluvial topography, see Tables 9.2 and 9.3. This difference may however be caused by the coarseness of contouring in the sub-colluvial topography.

TABLE 9.2 SLOPE ANGLES ON WHICH COLLUVIUM CLASSES ARE STANDING - SUB COLLUVIAL SLOPE ANGLES

	Upper Slopes			Break of slope m.P.D.	Lower Slopes		
	max	average	min		max	average	min
CLASS 1	-	-	-	-	13°	12°	9°
CLASS 2	32°	26°	20°	160	34°	18°	6°
CLASS 3*	31°	24°	17°	-	-	-	-

* only up to 350 mPD

TABLE 9.3 PRE-CONSTRUCTION COLLUVIAL SLOPE ANGLES

	Upper Slopes			Break of slope m.P.D.	Lower Slopes		
	max	average	min		max	average	min
CLASS 2	40°	31°	26°	180	22°	16°	11°
CLASS 3	45°	35°	16°		-	-	-

The relative positioning of the colluvium classes with class 1 overlain by class 2 which is overlain by class 3 accords with their relative ages, see Chapter 8. There are no instances where the sequence is broken with class 3 in contact with class 1.

9.2.3 Thickness of colluvium

The colluvium thickness before development was studied in conjunction with the sub-colluvial topography, Drawing G23, and the following facts emerged:

a) The thickest deposits of colluvium (in excess of 20 m) are located in buried valley depressions or on valley sides and today remain upstanding to form ridges.

b) The general thickness of deposits on the sub-colluvial ridges is 10 m with a maximum of 15 m above the Old Halls.

c) The thinnest deposits (less than 5 m) are in the stream courses around the University. Other thin deposits are present on Po Shan Ridge where class 2 colluvium is resting on 32° slopes.

The above suggests that during the cycle of deposition relating to class 2 colluvium, all the sub-colluvial valleys were completely buried with thicknesses in excess of 20 m such that the ridges also became buried with at least 10 m of colluvium. During this period of deposition, the original streams migrated over the surface of the colluvium but still taking approximately the same course. During and after deposition, these streams eroded out the colluvium to form the present ridge and valley topography of the lower slopes. On the higher slopes, class 2 colluvium was easily removed where it was sitting on steep angled slopes.

The present day University Stream appears to be underfit, that is, the present day stream size does not appear to have the erosive capacity to have formed the valley in which it now flows. The present stream has a small catchment area and yet it has incised through the colluvium down to in situ material in its lower reaches, see Drawing G22, and has eroded out a considerable amount of colluvium in what appears to be a dendritic pattern around the Old Halls and Sir Robert Black College. These tributary streams are now dry. Furthermore, buried stream-bed deposits have been logged in two localities along University Drive. The drying up of the streams can be explained in two ways:

a) stream capture by either Hatton Stream or Po Shan Stream of the upper catchment above Po Shan Road

b) a wetter climate than at present during the erosion of class 2 which would mean that all today's streams in the area are underfit.

9.3 General Model of the Geomorphological Development of the Mid-levels Area

9.3.1 Introduction

The Hong Kong Granite which now forms the centre of the Harbour Basin was intruded into the series of volcanics which make up the Repulse Bay Formation, during the Upper Jurassic period. The development of the present topography probably began during the

Late-Tertiary when the granite cupola was uncovered. As the granite is less resistant to weathering than the volcanics, differential denudation has continued to the present forming a basin partially surrounded by volcanic capped hills.

The processes instrumental in the denudation of the Harbour Basin will not have been constant over the whole area or with time, the main determinants relating to vegetation, geology and climate thereby resulting in a variety of different landforms. In the Mid-levels area, geological differences within the volcanics have had a profound effect and have resulted in two contrasting landscapes:

a) resistant knobbly tuff at the volcanic/granite contact has resulted in the Seymour Cliff topography whereas

b) less resistant coarse and fine tuffs have resulted in the gentler re-entrant topography such as that of the Hatton Bowl.

Denudation has not been a simple process of weathering and direct removal by erosion alone but has included reworking and deposition locally. The growth and eventual modification of depositional landforms is generally cyclic and largely related to changes in climate and/or sea level.

9.3.2 Cyclical fluctuations during the Pleistocene

Pleistocene fluctuations in both climate and sea level may have been responsible for the deposition and partial removal of the three classes of colluvium found in the Mid-levels. The reasons for this will now be discussed.

A general world outline of the Pleistocene fluctuations in sea level has been established, references 46 and 47. During glaciations large amounts of water were locked up as ice resulting in a fall in sea level. During interglacials sea level rose again. However, during the same period there was a continuous enlargement of the ocean basins so that with each successive cycle the sea fell and rose to successively lower levels.

As regards climatic fluctuations there is no world model just as there is no general world climate today. The major influencing factors for Hong Kong would have been a general southern shift of the thermal equator due to the growth of the northern ice sheets. There was also a general cooling of the earths surface by 5°C in equatorial regions and associated lowering in sea surface temperatures leading to a reduction in rates of evaporation and less rainfall. Also with each successive fall in sea level Hong Kong's climate would experience increased seasonality as the sea receded and the area was cut off from the moderating effect of the warm ocean currents. All these factors led to a sparser vegetation cover.

9.3.3 Cyclical production of colluvium

The likely scenario for deposition and erosion of colluvium in Mid-levels is therefore as follows:

a) onset of a glaciation producing a fall in sea level, reduction in temperature and rainfall (which becomes more seasonal) leading to changes in vegetation;

b) the change to sparser vegetation cover increases erosion by raindrop impact eroding coarser material into stream channels;

c) during the glaciation, an increase in rainfall seasonality leads to local slipping of the unprotected soil downslope and is the beginning of colluvium production;

d) the supply of debris exceeds the capacity of streams and runoff to remove it, leading to a clogging and build up of loose debris particularly in higher areas with steeper slopes;

e) at the end of a glaciation when climate and sea level revert to their previous state, that is an increase in rainfall and rise in water table, there is mass movement of the debris down to the lower slopes;

f) during the following interglacial there is a period of erosion due to streams adjusting their profiles to the new sea level and renewed weathering of slopes not protected by colluvium in preparation for the beginning of the next cycle.

Earthquakes may have also contributed to the volume of colluvium deposited.

9.3.4 Deposition and erosion of the three classes of colluvium

Using the above model the depositional history of colluvium in the study area identified as classes 1, 2 and 3 can now be proposed.

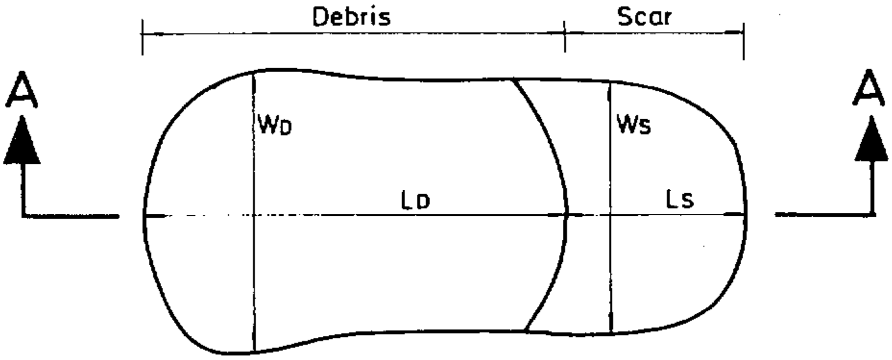
The earliest deposits of colluvium that have been located are termed class 1. There may have been earlier deposits resulting from previous cycles, but all evidence for them has been eroded away. This history therefore begins with the deposition of class 1 immediately following a glaciation. It is not known how much class 1 colluvium was produced but if later cycles are an indication then it could have blanketed most of the slopes. During the subsequent interglacial there was a long period of erosion in which most of class 1 was reworked and removed from all except the flatter slopes at around 9° to 13°. During this long interglacial period, those upper slopes which had been stripped of soil during colluvium production underwent extensive weathering prior to the next cycle, while the lower slopes, largely stripped of class 1 also matured and formed gullies.

With the onset of a new glaciation another cycle began and at the beginning of the next interglacial large volumes of class 2 were brought down to completely cover the lower slopes. Because class 2 is very common in the study area and because it exhibits a range of degree of decomposition, it may not be the product of only one glacial and interglacial cycle but several, with possibly only minor intervening erosion. Class 2 production may have been locally catastrophic as in the stripping of Seymour Cliffs. The interglacial separating class 2 production from class 3 production was shorter than the one separating class 1 and class 2 production resulting in extensive reworking and erosion of colluvium without its complete removal. Erosion was preferentially in today's colluvium free valleys and where class 2 was sitting on steep slopes such as Po Shan Ridge. However, some valleys may have contained streams which flowed through 'bouldery colluvium', resulting in a loss of erosive power, leaving the colluvium in place.

By the onset of the last glaciation, there had not been widespread weathering of the upper residual soil slopes prior to the final cycle, because these slopes were largely still protected by class 2 colluvium and resulted in smaller quantities of class 3 being produced. Since the last glaciation the rainfall may not have been sufficiently large or intense to cause class 3 to flow onto the lower slopes, but mainly reslipping into valleys where reworking occurs, largely as a result of basal erosion by streams.

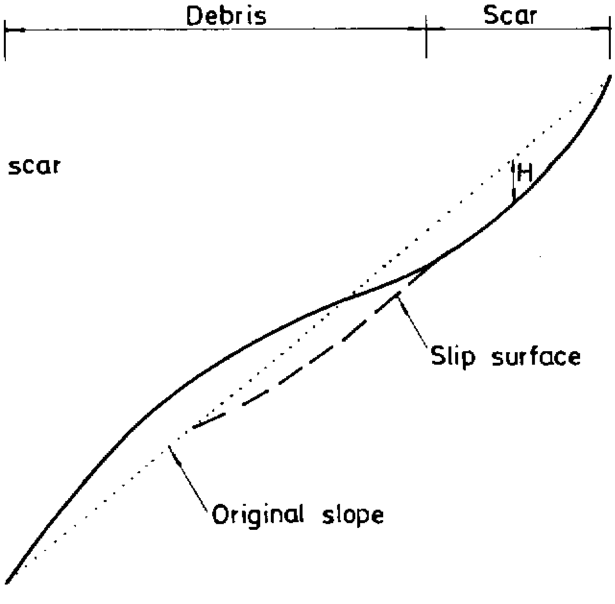
DEFINITION OF TERMS USED IN TABLE 9.1

FIGURE 9.1



LS = Length of scar
WS = Width of scar
LD = Length of debris
WD = Width of debris

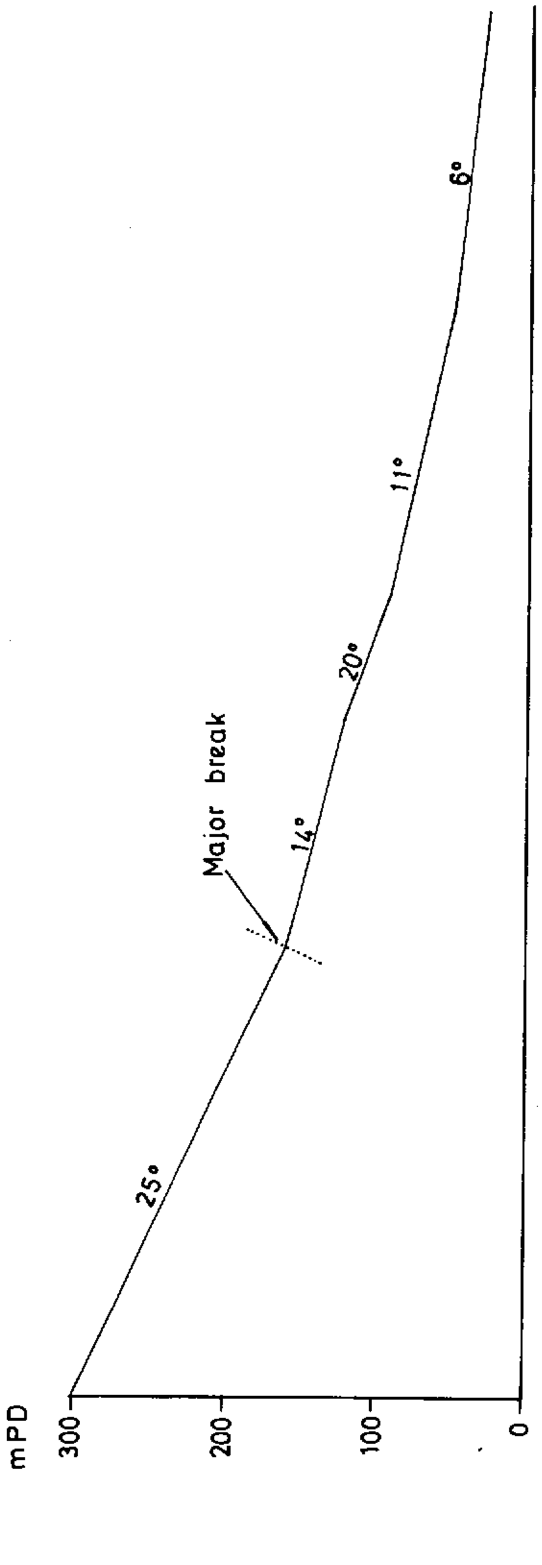
H = Depth of scar



SECTION A-A

MID-LEVELS STUDY

GENERALISED CROSS SECTION OF SUB-COLLUVIAL TOPOGRAPHY | FIGURE 9·2



MID-LEVELS STUDY

10. HYDROGEOLOGICAL FEATURES

10.1 Voids

Many voids have been observed in colluvium and detected in decomposed volcanic rock. Refer to Appendix B. Appendix B does not include the many observations of the highly voided surface colluvium deposits in the Glenealy, Central and Seymour areas. These deposits lie generally above Conduit Road and range from boulder scree to colluvium with a high proportion of boulders and voids.

Throughout the study area voids were observed on the upper slopes and occasionally along stream courses. On the steep upper slopes, some of these voids are undoubtedly due to piping. Pipes and voids have been seen elsewhere in the territory and are not regarded as unique phenomena in Mid-levels.

Colluvial deposits near the ground surface contain numerous voids varying from a few to hundreds of millimetres across. These are primarily due to tree roots, ant and animal activity, see exposure log EL5B reference 27.

The development of voids due to groundwater flow may be encouraged by the presence of cobbles and boulders, particularly if patinas exist. In several locations a gap of a millimetre of so between a cobble or boulder and its surrounding matrix has been observed in colluvium, see exposure log EL6B and chunam strip CS4 reference 27. Where this occurs below the groundwater table, water seeps from this space. This seepage increases where a completely decomposed patina surrounds the cobble or boulder.

Several of the observed voids are associated with boulders, more commonly occurring immediately below a boulder, see Appendix B. Such a configuration will preserve voids, as the boulder, provided it is wider, will protect the void from collapse. This phenomenon has been detected in colluvium, and in the core stone zone in decomposed volcanics immediately above bedrock.

The presence of voids or pipes will drastically affect the permeability locally of the material in which they occur. Large variations of permeability have been measured in colluvium some of which may be due to voids.

However, cobbles and boulders may also close voids. For example, as erosion of pipes enlarges them, any boulders that straddle them may eventally fall into the enlarged void blocking the pipe. This could cause significant changes in local groundwater conditions.

10.2 Perched Water Tables

Perched water tables have been observed in colluvium in Mid-levels.

Perching will occur wherever there is a source of water and a decrease in permeability with depth. General permeability values of colluvium and decomposed rock that have been assessed from field tests are given in Table 10.1. From this table it can be seen that there is a decrease of permeabilities from colluvium through decomposed rock to bedrock. This change in permeability could account for the perched water tables.

An alternative explanation for perching in colluvium would be a layer of particularly low permeability material such as clay at the base of colluvium or the top of decomposed rock. No such clay layer has been observed in the field or detected in particle size analyses. The particle size analyses have shown colluvium with clay contents exceeding 30% in nine locations but only three of these (TP4H, TP31 and TP82) can be shown to be close to the surface of in situ material. Similarly, perching might be produced by a clayey layer at the top of the in situ material. Particle size analyses show the clay content of decomposed volcanics exceeds 20% in only four locations, and only one example, TP4H, appears to be related to the surface of in situ material.

The clay content in colluvium is between 10% to 40% whereas that of the decomposed rocks is mainly less than 10%, refer to the Soil Properties Report. These results would suggest a reversal of the permeabilities that have been measured in colluvium and decomposed rock. It would appear that the interconnection of voids is greater in colluvium than in decomposed rock.

TABLE 10.1 ASSESSED AQUIFER PROPERTIES

Material	Permeability ($\times 10^{-6}$ m/s)
Boulder field colluvium	> 1000
Young colluvium (class 3)	50 - 350
Intermediate colluvium (class 2)	5 - 20
Old colluvium (class 1)	n.e.(1)
Decomposed granite (IV-VI)	7 - 10
Decomposed volcanics (IV-VI)	4
Granite rock (I-III)	0.2
Volcanic rock (I-III)	0.9

Notes : n.e. - no estimate available

1. Although no estimate is available, class 1 colluvium permeability is probably close to the low end of the class 2 permeability range.

11. CONCLUSIONS

The colluvial deposits in the study area have been derived almost completely from the volcanics rather than granite. Virtually all the colluvium is considered to have been deposited as a series of debris flows during deposition periods rather than as a single event or as a continuous process. One exception is the boulder field below Seymour Cliffs which probably initially formed as one event but has since been added to by smaller rock falls.

There have been at least three cycles of erosion followed by deposition in the study area probably related to climatic variations during the Pleistocene Era. These active periods may also have been associated with a time of more frequent or higher magnitude earthquakes. During each deposition period colluvium was produced, class 1 first and class 3 last. We are currently in an erosion period.

The three classes of colluvium can be identified in the field. Laboratory testing of colluvium has concentrated on class 2 colluvium, the most abundant, so it is difficult to identify the different colluvium classes from test results alone. The tests to date have not identified significant differences in the engineering properties of the matrix.

Colluvium classes 1 and 2 were initially deposited over most of the lower slopes. Most of class 1 was eroded prior to the deposition of class 2 and is only found in isolated areas. Class 3 colluvium has been deposited over most of the upper slopes.

Colluvium classes 1 and 2 were in a more fluid state during deposition than class 3 which is reflected by the relative distances each travelled before coming to rest.

The colluvium has been deposited upon a deeply eroded surface of in situ material.

The volcanic rocks have been divided into four zones which have been subjected to different grades of metamorphism. This is mainly due to contact metamorphism from the granite intrusion. Generally the grade of metamorphism decreases away from the granite contact.

The granite/volcanic contact is complex, varying in character along its length. It may be considered as a zone in the western part of the study area often with irregular granite stringers intruding the volcanics. This is not as common in the eastern part. The contact is faulted to the north some 300 m along a NW/SE trending fault near the junction of Po Shan and Central Areas. The contact between Hatton Stream and Oaklands Avenue may be faulted as it forms part of a photolineament and would explain the existence of low grade metamorphic rocks adjacent to the contact with granite. The two faults identified above may be part of a block faulting movement.

Photolineaments and faults trend in two main directions, NE/SW and NW/SE across the area. These are consistent with trends outside Mid-levels. Dykes are often found along fault lines or photolineaments.

The granite decomposes at a faster rate than the volcanics. At the granite/volcanic contact in the decomposed rock layer, there is a gradational increase in the thickness of decomposed rock going from the volcanics to the granite.

The decomposed rock layer generally thins towards the harbour in the western part of the area but thickens in the eastern part. There are two main localised thick zones of decomposed rock. These reach a thickness of 90 m, one at the western end of Robinson Road, the other extending from Peel Stream to the north east.

Numerous voids, some of which are due to piping, occur in colluvium and decomposed volcanics on the upper slopes. They are often adjacent to boulders which help prevent their collapse. These voids will have a significant local effect on the permeability of colluvium and decomposed volcanics.

GLOSSARY OF TERMS

Bedrock rock that is in situ and grade III or better.

Colluvium material transported by gravity. Scree and cliff debris are included in such deposits.

Decomposition the breaking down of rocks through chemical processes related to weathering.

Ignimbrite pyroclastic rock deposited at temperatures high enough to produce welding of volcanic fragments forming a flow banding fabric.

Isograd the boundary of a zone of rocks subjected to a particular metamorphic grade.

Knobbly tuff a tuff containing spherical or irregular aggregations of quartz which are more resistant to decomposition than the adjacent fine groundmass and give outcrops or boulders a nodular appearance.

Lapilli tuff a tuff that contains fragments of pyroclastic material 4 mm to 32 mm in diameter, often with a lens shape.

Metamorphic grade the extent of metamorphism measured by the amount or degree of difference between the original parent rock and the metamorphic rock. It indicates in a general way the pressure and temperature environment in which the metamorphism took place.

Metasomation the process by which a new mineral of partly or wholly different chemical composition may grow in the body of the old mineral.

Nodular tuff synonym for knobbly tuff.

Patina thin pale coloured outer layer produced by weathering.

Phenocryst the relatively large crystals which are found in a finer grained groundmass, constituting the texture termed porphyritic.

Polygonisation the inversion of a crystal to a network of interlocked equal-sized grains.

Porphyritic a textural term for those igneous rocks in which phenocrysts are set in a finer groundmass.

Pseudomorph a mineral whose outward crystal form is that of another mineral species which has been replaced by substitution or chemical alteration.

Pyroclastic rocks explosively ejected from a volcanic vent and containing fragments derived from the original material.

Sericitisation the metamorphism or decomposition of feldspar to a mass of fine mica crystals.

Tuff a rock formed of compacted volcanic ash.

Coarse Tuff - coarse grained tuff

Fine Tuff - fine grained tuff

Weathering the group of processes such as the chemical action of air and rain water and the mechanical action of changes in temperature, whereby rocks on exposure decay to soil.

Xenolith rock fragments that are foreign to the body of igneous rock in which they occur.

APPENDIX A

COLLUVIUM CLASSIFICATION

1) Rock type of boulders and cobbles

G	Granite
V	Volcanic

2) Weathering grade

Determined by state of cobbles and boulders

Grade	A	Rock fragments fresh
	B	Fragments have a decomposed rind or patina
	C	Fragments moderately decomposed on average
	D	All fragments highly to completely decomposed

Up to two symbols may be used to indicate variation.

3) Proportions

Defined as the proportion of cobbles and boulders

0	0%
1	1% - 25%
2	25% - 50%
3	50% - 75%
4	75% - 100%

4) Matrix

G	Gravel
S	Sand
T	Silt
C	Clay

Up to two symbols
Major constituent shown last
If matrix is absent no symbols are given. e.g. GVA4

EXAMPLES

1) Colluvium with 50% - 75% of volcanic cobbles and boulders, moderately decomposed or with thin patinas, in a sandy silt matrix
VBC 3 ST

2) Colluvium with 25% - 50% cobbles and boulders in a gravelly silt matrix, the rock fragments comprise highly to completely decomposed granite and moderately decomposed volcanics
GDVC 2 GT

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
DRILLHOLES					
ML1	22.6	-	7.6/VB3ST	15.0/VB4CT	Sharp contact with DG
ML1A	21.0+	-		?VGB23ST	Drillhole terminated in colluvium
ML2A	1.5	-		1.5/VAB3CT	Sharp contact with DG
ML3	4.5	-	2.5/VB1ST	2.0/VA3TC	Sharp contact with DG
ML3A	3.8	-		3.8/VB4GT	Sharp contact with DG
ML4	9.0	-	9.0/VB1T	-	Diffuse contact with DV
ML5	16.3	-	16.3/VC1GT	-	Sharp contact with DG
ML5D	6.6	-	6.6/ VBD3GT	-	Diffuse contact with DV
ML5E	7.3	-	4.0/VB23GT overlying 3.3/VC1GT	-	Diffuse contact with DV
ML10	10.0	-	10.0/VA2TS	-	Sharp contact with DV
ML11	4.3	-	4.3/GVB2ST	-	Sharp contact with DG
ML13	12.0	-	12.0/VC2T	-	
ML18	7.8	-	7.8 /VGB3 CT	-	Sharp contact with DG
ML18A	7.2+		7.2+/VGB2 GT		Drillhole terminated in colluvium
ML19	17.9		10.0/VB2ST overlying 7.9/VGC2ST		Sharp contact with DG
ML19A	11.3+		11.3+/VC3 ST		Drillhole terminated in colluvium

APPENDIX A

COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
ML19B	17.2	-	5.0/VB2ST overlying 9.0/VC2TS overlying 3.2/GDVCIT	-	Sharp contact with DG
ML19C	13.1	-	6.7/VC1ST overlying 6.4/OST	-	Some mixing with DG at base
ML20	20.2	-	3.4/1GC overlying 16.8/VBGD 1ST		
ML20A	8.4+		8.4+/VCB2 GT		Drillhole terminated in colluvium
ML30	11.3	-	2.3/VB4T overlying 9.0/OST	-	Diffuse contact with DG
ML31	6.6	-	5.3/VC2GT	1.3/VB1GT	Diffuse contact with DV
ML31A	6.7		6.2/VC2GT	0.5/VB0GT	Diffuse contact with DV
ML31B	5.4		5.4/VBC3GT		Diffuse contact with DV
ML31C	4.6		4.6/VC1GT		Diffuse contact with DV
ML31D	4.9		3.4/VC1GT overlying 1.5/OST		Lower fine colluvium may be DV
ML31E	6.0		6.0/VB3GT		Sharp contact with DV
ML31F	4.1		4.1/VC1GT		Diffuse zone at contact 0.5m thick
ML100	17.4	-	13.8/VC3S overlying 3.6/VC2SG	-	Sharp contact with DG

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
ML100A	15.2		15.2/VB3CT		Sharp contact with DG
ML101	1.5	-	1.5/VB3TS	-	Most colluvium removed by previous excavation
ML102A	21.4	-	14.4/VB3CT	7.0/VB3CT	Sharp contact with DG
ML102B	21.1		2.7+VB3CT		Only lower 2.7m of colluvium recovered
ML103	11.8		9.9/VBC4GS overlying 1.9/VBC1GT		Sharp contact with DG
Ml103A	12.0		9/VB34TS overlying 3.0/VB3GT		Thickness of fill uncertain Sharp contact with DG
ML104	13.9		8.5/VBC2SG overlying 5.4/VCD3GS		Very decomposed lower layer - approaches Class 1
ML104A	14.3		14.3/VB23 GS		Sharp contact with DG
ML107	5.0			5.0/VB3GS	Sharp contact with DG
ML107A	4.5			4.5/VB2ST	Zone of slight mixing at contact 0.2 to 0.4m thick
ML108	7.6			7.6/VB2GS	Sharp contact with DG
ML108A	7.0			7.0/VAB4 GT	Sharp contact with DG
ML110	21.8		21.8/VC2GT		Possibly re-worked. Sharp contact with granite

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
ML110A	20.9		20.9/VBC3 GT		Possibly re-worked. Sharp contact with granite
ML110B	17.2		13.2/VBC3 ST over-lying 4.0/VC1ST		Possibly re-worked diffuse contact with DG
ML111	11.6		3.4/VBC2GS overlying 8.2+/VCD23 TS		Very decomposed lower layer approaches Class 1
ML111A	33.0		16.0/VB2GT overlying 17.0/VB1ST		Diffuse contact with DG
ML112	21.6	4.9/ VD23ST	7.2/VBD12 GT overlying 9.5/VCD 23ST		Boundary with Class 1 uncertain
ML112A	0.5		0.5/VB1CT		Sharp contact with DG
ML112B	5.3		5.3/VB1ST		Possibly all mixed with DG
ML113	13.3		4.0/OTS	9.3VB3TS	Diffuse lower layer due to maxing with DG
ML120	9.0		5.0/VBC1CT overlying 4.0/VBCGD 1ST		Sharp contact with DG
ML121	19.0		10.8/ VBCGD23ST overlying 5.0/ GDVBC23TS overlying 3.2/VBC3TG		Sharp contact with DG

APPENDIX A

COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
ML121A	18.8		9.7/VBC1GT overlying 9.1/VC1ST		Sharp contact with DG
ML122	8.9		8.9/VCD23 ST		Diffuse contact with DG Decomposition of colluvium approaches Class 1
ML122A	10.2		10.2/VBC2 ST		Slightly more gravelly near base. Sharp contact with DG
ML153	3.6			3.6/VC1TS	Sharp contact with DV
H4	24.5	16.5/ VD12ST	8.0/VCD12 ST		Not an MLS borehole
BH15	16.0	11.0/ VD12ST	5.0/VCD12 TS		Not an MLS borehole

APPENDIX A

COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
TEST PITS					
TP2 OAP	2.0+			2.0+/VC4GT	
TP3 OAP	2.0			2.0/VB3GT	Sharp contact with DG
TP4 OAP	1.0+			1.0+/VB3GT	
TP5 OAP	0.8		0.8/GD3GT		
TP7 OAP	3.0		3.0/VBC12 GT		
TP74	2.3		2.3/VC2GT		
TP75	3.4+			3.4+/VBC 23GT	
TP76A	4.3+			2.0/VC2GT overlying 2.3+/VC3 GT	
TP77	1.4+			1.4+/VC23 GT	
TP77A	1.5+			1.5+/VC1GT	
TP78	2.5+			2.5+/VB2T	
TP78A	2.6		2.6/VC1GT		
TP80	1.4			1.4/VC1GT	
TP80A	2.7			2.7/VBC2GT	
TP80B	1.8			1.8/VCB2GT	
TP82	2.3			2.3/VC1ST	
TP85	0.5			0.5/VC1GT	
TP86	2.0+		2.0+/GDVC 1ST		

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
TP87	1.0+		1.0+/VBC2 ST		
TP88	1.4+		1.4+/VC1CT		
TP90	3.0			3.0/VBC23 GT	
TP96	3.0		3.0/VBC1GT		
TP98	0.3		0.3/VBC1GT		
TP98A	2.2+			2.2+/VBC 23GT	
TP99	0.4			0.4/VC1GT	
CHUNAM STRIPS					
CS1	3.0 length			VBC1GS	Inclined strip does not indicate true thickness of colluvium
CS2	23.0 length			12.0/VA4 overlying 11.0/VB1GT	Inclined strip does not indicate true thickness of colluvium
CS3	21.0 length		16.0.VB2CT	5.0/VB1GT	
CS5	5.0 length		5.0/VB1GT		Diffuse contact with DG
CS6	2.0 length		2.0/VAB1TS		Diffuse contact with DG
CS8	8.5+ length		5.0/VGB1ST overlying 3.5+/VGB 2ST		

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
GLENEALY, SEYMOUR AND CENTRAL AREAS

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
CS10	8.5+ length			8.5+/VB2GT	
CS11	14.2+ length		12.0/VB3TG overlying 2.2+/VBC12 GT		
EXPOSURES					
EL1 OAP	4.0+		4.0+/VBC4 SG		Sharp contact with DV
EL2 OAP	1.0+			1.0+/VB4GS	
CAISSONS					
K1	7.0+		5.0+/VBD4 GT	2.0/VB2ST	
K2	1.0			1.0/VC1ST	Sharp contact with DV
K3	6.0+			6.0+VBC3GT	
K4	7.0			7.0/VB23GT	Sharp contact with DG
PK1	1.5		1.5/VB3CT		Sharp contact with DG
PK2	4.0		2.5/VB4CT overlying 1.5/VC1CT		Diffuse contact with DG

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
PO SHAN AREA

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
DRILLHOLES					
ML6	10.2	-	10.2/VC1T	-	
ML9	21.4	-	11.6/VBC3T overlying 9.8/VC1ST	-	Sharp contact with DG
ML12	6.92	-	6.92/VBC2T		
ML23	16.15	-	16.15/VBC3ST		
ML23B	11.21	-	11.21		4.55 m of fill overlying colluvium
ML51	15.2	2.8/ GDVC3ST	12.4/VBC2ST	-	
ML51C	15.28	2.28/ GVCD4T	13.0/VCD3GT	-	
ML52	8.0	-	8.0/VCD3GT	-	
ML52A	9.2	-	9.2/VBC2T	-	
ML54	12.0	-	5.3/VBC1ST overlying 6.7/VCD1ST	-	
ML54C	12.28	-	12.28/VBD2T	-	
ML55A	5.6	-	2.2/VD4ST	3.4/VBC4ST	Sharp contact with DV
ML56	9.84	-	9.84/VC1T	-	Sharp contact with DV
ML56C	9.02	-	9.02/VC1ST	-	Sharp contact with DV
ML58	7.8	-	7.8/VB3CT	-	
ML58C	6.34	-	6.34/VBD1T	-	4.22 m of fill

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
PO SHAN AREA

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
ML62	5.93	-	5.93/VCD2T	-	
ML62C	7.68	-	3.27/VBC3T	4.41/VC2T	
ML63	9.3	-	9.3/VBC3GT	-	1.5 m of fill overlying colluvium
TEST PITS					
TP2H	1.5+	-	1.5+/VB4T	-	
TP3H	1.8+	-	1.8+/VC2T	-	
TP4H	1.0+	-	1.0+/VC1T	-	
TP5bH	0.7+	-	0.7+/VBC3T	-	
TP7H	1.8+	-	1.8+/VC3T	-	
TP23	1.5+	-	-	1.5+/VC2T	
TP23A	2.5+	-	-	2.5+/VC2T	
TP24	3.7+	-	-	3.7+/VCD3ST	
TP27	1.0+	-	-	1.0+/VC2T	
TP28	1.0+	-	-	1.0+/VC1T	
TP31	5.7	-	5.7/VD1T	-	Sharp contact with DV
TP36	5.6	-	5.6/VD1ST	-	Diffuse boundary with DV
CHUNAM STRIPS					
CS4	7.0+	-	7.0+/VC1ST	-	
CS7	5.0+	-	5.0+/VC1ST	-	
CS12	1.4	-	1.4/VC1ST	-	Sharp contact with DG

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
PO SHAN AREA

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
EXPOSURES					
EL2H	2.0+	-	-	2.0+/VB2ST	
EL6H	3.0+	3.0+/ GVD1ST	3.0+/VC3ST	-	Persistent seepage from class 1
CAISSONS					
TP20	2.8-6.5	-	2.8-6.5/VBC 2ST	-	Sharp contact with DV
TP22	7.0	-	4.0/VBC3ST	3.0/VBC2ST	Diffuse contact with DV
PC2	5.4+	-	3.8+/VBC4CT overlying 1.6/VC1ST	-	Sharp contact with DV
PC8	8.5+	-	8.5+/VBC4CT	-	Sharp contact of boulder field with DV
PC11	7.7+	-	4.8+/VBC2T overlying 2.9/VBC4CT	-	Sharp contact of boulder field with DV
PC17	4.7	-	2.5/VB3ST overlying 2.2/VC1ST	-	Sharp contact with DV
No. 7) 66 Bonham Road	0.8+	-	0.8+/VBC2T	-	Sharp contact with DV
No.10) 66 Bonham Road	0.8+	-	0.8+/VBC2ST	-	
No.15) 66 Bonham Road	0.6+	-	0.6+/VBC3ST	-	
No.16) 66 Bonham Road	3.6+	-	3.6+/VBC2ST	-	

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
UNIVERSITY AREA

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
DRILLHOLES					
OH1 *	8.0	-	3.3	2.7	
OH2 *	15.5	-	11.7	3.8	
OH3 *	19.5	8.5	11.0	-	Diffuse boundary with DG
OH4 *	17.2	-	17.2	-	
OH5 *	4.7	-	4.7	-	
HKUL1 *	11.2	-	11.2	-	
HKUL2 *	12.3	4.5	7.8	-	
HKUL3 *	12.3	-	12.3	-	
ML7	12.19	-	-	12.19/VB 2T	Sharp contact with DV
ML14	4.5	2	-	2.5	Class 1 - red and grey colour layers parallel to slope above contact
ML16	2.6	-	2.6	-	
ML24	13.0	-	13/VC1T	-	Sharp contact with DG
ML24A	11.9	-	11.9/VC1T	-	
ML25	4.3	-	4.3/VD1T	-	
ML26	13.1	-	13.1/VC1T	-	
ML27	11.5	-	11.5/VC1T	-	
ML50	6.2		6.2/VCD2T	-	Sharp contact with CDV, 1.8m fill overlying colluvium
ML50C	8.85	2.55/ VD3T	4.5/VBC2T		
ML53	Nil				Horizontal layering 0.5 m thick (slope wash?) between 2 colluvium types
ML53B	7.2	2.5/VGD 4T	4.7/VBC3T		
ML59	12.1	-	12.1/VCD1ST		Sharp contact with CDV
ML59A	13.07	-	13.07/VCD3 GT	-	
ML60	Nil	-			All fill
ML60C	Nil				All fill

APPENDIX A
COLLUVIUM CLASS AT EACH SITE INVESTIGATION STATION
UNIVERSITY AREA

INVESTIGATION	TOTAL COLLUVIUM THICKNESS (m)	COLLUVIUM CLASSES THICKNESS (m)/CLASSIFICATION			REMARKS
		Class 1	Class 2	Class 3	
TEST PITS					
TP1B	1.5+	-	-	1.5+/VC2T	
TP2B	2.0+	-	-	2.0+/VB4T	
TP3B	1.0+	-	-	1.0+/VC2T	
TP4B	4.5+	-	4.5+/VCD2ST	-	
TP25	2.8	-	-	2.8/VBC2T	
TP26	3.8	-	-	3.8/VBC2ST	
TP29	2.8	-	-	2.8/VBC2T	
TP30	1.9	-	-	1.9/VCD2T	
TP33	2.9+	-	-	2.9+/VBC2T	
TP35	3.4+	-	-	3.4+/VC2ST	
CHUNAM STRIPS					
CS15A	8.0+	-	-	8.0+/VBC1 ST	
EXPOSURES					
EL1B	1.0+	-	1.0+/VC2T	-	Sharp contact with DG
EL2B	2.0+	1.0+/ VCD4T	1.0+/VCB2 ST	-	Sharp and irregular contact between the two classes of colluvium
EL3B	2.0+	-	-	2.0+/VBC2T	
EL4B					Boulder field
EL5B	1.2+	-	-	1.2+/VBC2T	
EL6B	1.2+	-	-	1.2+/VBC1T	
EL7B	6.5	2.0/ VD2ST	4.5/VC2ST	-	
CAISSONS					
C1)) HKU	0.85+	0.85+/ VD2ST	-	-	
C2)Redev-	0.85+	-	0.85/VD2ST	-	
C14)elopment)	0.85+	0.85+/ VD3ST	-	-	
C16))	2.6	0.8/ VD4ST	1.8/VBC3ST	-	Sharp contact with DG
C34)	2.7	-	2.7/VBC3ST	-	Diffuse contact with DG
C40	1.5	-	1.0+/VBC2ST	-	Sharp contact with DG
C41	1.5	-	1.0+/VBC2ST	-	

Notes: * denotes drillholes formed prior to the Mid-levels Study

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
GLENEALY, SEYMOUR AND CENTRAL AREAS

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
Test pits				
TP85	33245 E 15463N	1.0 to 1.4 m	Cylindrical, 100 to 200 mm dia. more than 2 m long, in CDV. Probably an animal burrow. Now contains ant nests.	Ref. 27
Chunam strips				
CS2	33450 E 15620 N	0 to 5 m	Voids between boulders in scree deposit, up to 0.3 m across.	
Caissons				
PK1	33213 E 15873 N	1.6 to 2.1 m	Irregular voids, some lens - shaped, up to 0.1 m across, in colluvium matrix. Voids interconnect. Ant burrows.	
K4	33528 E 15380 N	4.0 to 6.6 m	Irregular shape up to 0.3 m across around or between boulders in colluvium. Formed by internal erosion from adjacent stream.	
Exposures				
Slip behind TP5 OAP	33615 E 15275 N	0.5 m	Approx. cylindrial; tapering into slope; 0.25 m dia. max; smooth sides with debris in base; in thin colluvium over DG.	Observation by PAR

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
GLENEALY, SEYMOUR AND CENTRAL AREAS

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
Hornsey Road	33690 E 15360 N	0 to 1.0 m	Lens-shaped voids upto 0.1 m across in fine colluvium matrix. Appearance as voids in PK1 - ant burrows.	Observation by PAR
EL3H	32907 E 15742 N	2 m	Rounded shape up to 0.2 m diameter between boulders in colluvium. Seen to issue water in storms.	Ref. 10

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
PO SHAN AREA

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
<u>Drillholes</u>				
2039/H3	32,745E 15,720N	Drillhole drilled at 10° above horizontal Ground level 195 mPD	Void about 0.3 m diameter at about 6.7 m below original ground level and about 3.7 m below base of Class 2? Colluvium. Void definitely in decomposed volcanics and associated with boulder below.	Drawing No. 0173/410 in "Report on Slope Stability of Crown Land above Po Shan Road" by Maunsells Drillhole drilled above Po Shan Road in December 1972 after Po Shan Slip, hence not in slip material
MPS101	32,644E 15,646N	10.21 - 11.13 m Ground level 213.36mPD	Void 0.92 m diameter? in completely decomposed volcanics with corestones adjacent.	Maunsells Report mentioned above Drillhole drilled in 1973 after Po Shan Slip
ML56	32,555E 15,714N	Depth of channels 9.84 - 10.41 m Ground level 176.74 mPD	In diffuse contact zone between Class 2 colluvium and completely decomposed volcanics. Channels 15 mm diameter, 0.2 m long on surface of core sample. Unlikely to be due to drilling flush water.	In A category core sample (photograph available)

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
PO SHAN AREA

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
ML62	32,728E 15,535N	10.78-12 m Ground level 289.48 mPD	Void in completely decomposed volcanics. Water issuing from ground about 20 m downs-slope as pumped into drillhole. Void able to carry 16 gpm but sides eroding. (Partly through 1966 slip debris?)	Witnessed by T.Hui/ B.W. Taylor/ N.R. Townsend during permeability test at 12 m. Drilling water lost at 7m and did not return Colluvium to about 7.7 m depth
ML62		16.9 - 11.97 m	Void in completely decomposed volcanics in core-stone zone. Void carries 16 gpm+ (possibly leaking seal for permea-bility test.	Permeability test at 16.97 - 17.97 m. Water rose to top of standpipe then dropped.
ML62C	32,730E 15,540.8N	10.6 - 12.3 m Ground level 289.35 MPD	High permeability $20,000\times10^{-6}$ m/s from constant head test in completely decomposed volcanics.	
ML62D	32,730E 15,542.0N	10.7 - 12 m Ground level 289.72 mPD	High permeability 30×10^{-6} m/s from constant head test in completely decomposed volcanics Four constant head and falling head tests between 12 and 22 m in slightly and moderately decomposed volcanic corestones in highly decomposed volcanic matrix gave permeability values $5-7\times10^{-6}$ m/s	

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
PO SHAN AREA

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
<u>Test pits</u>				
TP27	32,741E 15,385.6N	0.6 m Ground level 397.1 mPD	Void 100x40 mm and 50 mm deep Discontinuous, possibly hole from boulder. Void occurs in Class 3 colluvium.	Found by F.H. Li as test pit excavated.
TP28	32,725.5E 15,365.2N	1.6 m Ground level 393.19 mPD	Void 150x10 mm and 0.5 m+ deep. Irregular direction? Located beneath boulder in Class 3 Colluvium.	Found by F.H. Li as test pit excavation
<u>Chunam Strips</u>				
CS4	32,608E 15.625N	Depth from top of slope 0.5 - 3.5 m. Ground level bottom of slope 187.3mPD	Void diameter 10 mm+. Depth 0.5 - 3.5 m. Associated with boulders, often beside or below.	Found by H. Choy when logging chunam strip.
<u>Caisson</u>				
TP20	32,620E 15,612.7N	About 1m Ground level 223.04 mPD	Void about 0.2 m diameter, smooth sides, beneath a slightly decomposed volcanic boulder. Void extends about 1+ m, irregular direction and occurs in Class 2 colluvium.	Found by T. Hui/ N.R. Townsend as Caisson excavated.
Mansions Ridge Tensiometer Array (about 3m south of TP20)	32,620.2E 15,609N		Star drill holes #1 0.2 m void under boulder at 3.2 m #2 0.3 m holes under boulder at 2.4 m and 3.0 m. Possibly void at base of hole at 3.75 m (backfilling took more material than expected).	

APPENDIX B
PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
PO SHAN AREA

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
Exposures				
Area behind Po Shan Mansions (about elevation 220 mPD)		Exposed	Streams issue from ground - collected by surface channels.	

APPENDIX B

PIPES/VOIDS IN COLLUVIUM AND DECOMPOSED VOLCANICS
UNIVERSITY AREA

LOCATION	COORDINATES	DEPTH OF VOID FROM GROUND LEVEL	VOID AND MATERIAL DESCRIPTION	DATA SOURCE
Drillholes				
ML53B	32,314.6E 15,995N	6.2 m Ground level 85.73 mPD	Void wedge shape 80x40 mm in Class 1 colluvium.	In A category core. Photograph available.
M4 Tensiometer installation	32,510E 16.640N	Ground level 202.2 mPD	Void in completely decomposed volcanics about 4.5-5 m below base of Class 2 colluvium.	Noted by drillers on site. Permanent seepage measured downslope by Halcrow International Partnership.
Exposures				
South of Piccadilly Mansions	32,450E 15,590N	Ground level 208 mPD Exposed in colluvium	Void exposed in Class 3 colluvium about 0.2 m diameter with smooth sides and dips about 60° from horizontal into slope, extending 1 m+.	Found by B.W. Taylor in field.
Hong Kong University excavation for foot-bridge over nullah	32,335N 15,985N	1.5 m Ground level 84 mPD	2 voids about 100x50 mm and 200 mm deep immediately under boulders in Class 2 colluvium or	Logged by H. Choy and B.W. Taylor in exposure.

HYDROLOGY

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TABLES

FIGURES

DRAWINGS
(in separate volume)

1. INTRODUCTION

1.1 Scope of Report

This report presents and develops groundwater information that is relevant to the stability of the Mid-levels area. This is mainly piezometric levels for use in stability analyses, but also includes descriptions of the hydrology and hydrogeology of the study area and a review of the suction work. The effects of development on groundwater are also discussed.

1.2 The Study Area

1.2.1 Location

The Mid-levels area lies to the north of Victoria Peak between Lugard Road and Queen's Road. The extent of the present study area is between these bounds, Glenealy Valley to the east, and Pinewood/University Ridge to the west (Figures 1.1 and 1.2). For the purpose of the study, the area has been divided into five sub-areas. From east to west these are Glenealy, Seymour, Central, Po Shan and University, the boundaries of which are shown in Figure 1.1.

A distinct north-south division of the area occurs along the line of Conduit Road/Po Shan Road. To the north of this line the area is very intensively developed, greatly reducing the possibility for direct infiltration of rainfall into the ground. To the south and above the line, the slopes are undeveloped and in general are covered by trees and scrub vegetation.

1.2.2 Geology

The geology of the study area is fully described in the Geology Report and only the basic geology is outlined here.

The overall geology consists of volcanic ash deposits which were folded and later intruded into by a large body of granite which extended from Kowloon to Victoria Peak. Subsequent erosion of the granite has produced the harbour and the present day landscape.

The granite is relatively uniform in appearance throughout the area, while a wide variety of volcanic rock types have been found such as coarse, fine and lapilli tuff. The granite is more readily decomposed than the volcanics which results in an increase in the thickness of the decomposed layer from the volcanics in the south to the granite in the north. Within this overall pattern, there is a considerable variation in the thickness of the decomposed layer which appears to be mainly due to faults.

Colluvium, considered to be soil and rock debris which has moved downslope under the action of gravity, is generally absent from the upper slopes. In the mid-slopes the thickness varies considerably but is generally between 5 m and 20 m. In some areas old valleys and ridges have been completely obscured by colluvium. Elsewhere, streams have cut through the colluvium blanket. There is no colluvium on the lowest slopes.

The colluvium generally consists of cobbles and boulders set in a matrix of gravelly or sandy silt but, below cliffs, younger colluvium with numerous boulders and little or no matrix is found. In general, the older the colluvium the greater the degree of decomposition of the cobbles and boulders. This has implications on the mass permeability of the colluvium and is discussed in section 3.2.

1.2.3 Geomorphology

The topography of the Mid-levels area is the result of modification by erosion and deposition over millions of years since the rocks were first uplifted and exposed to the atmosphere. The geomorphology of the study area is described in the Geology Report. In very general terms, the landforming process involves the following events.

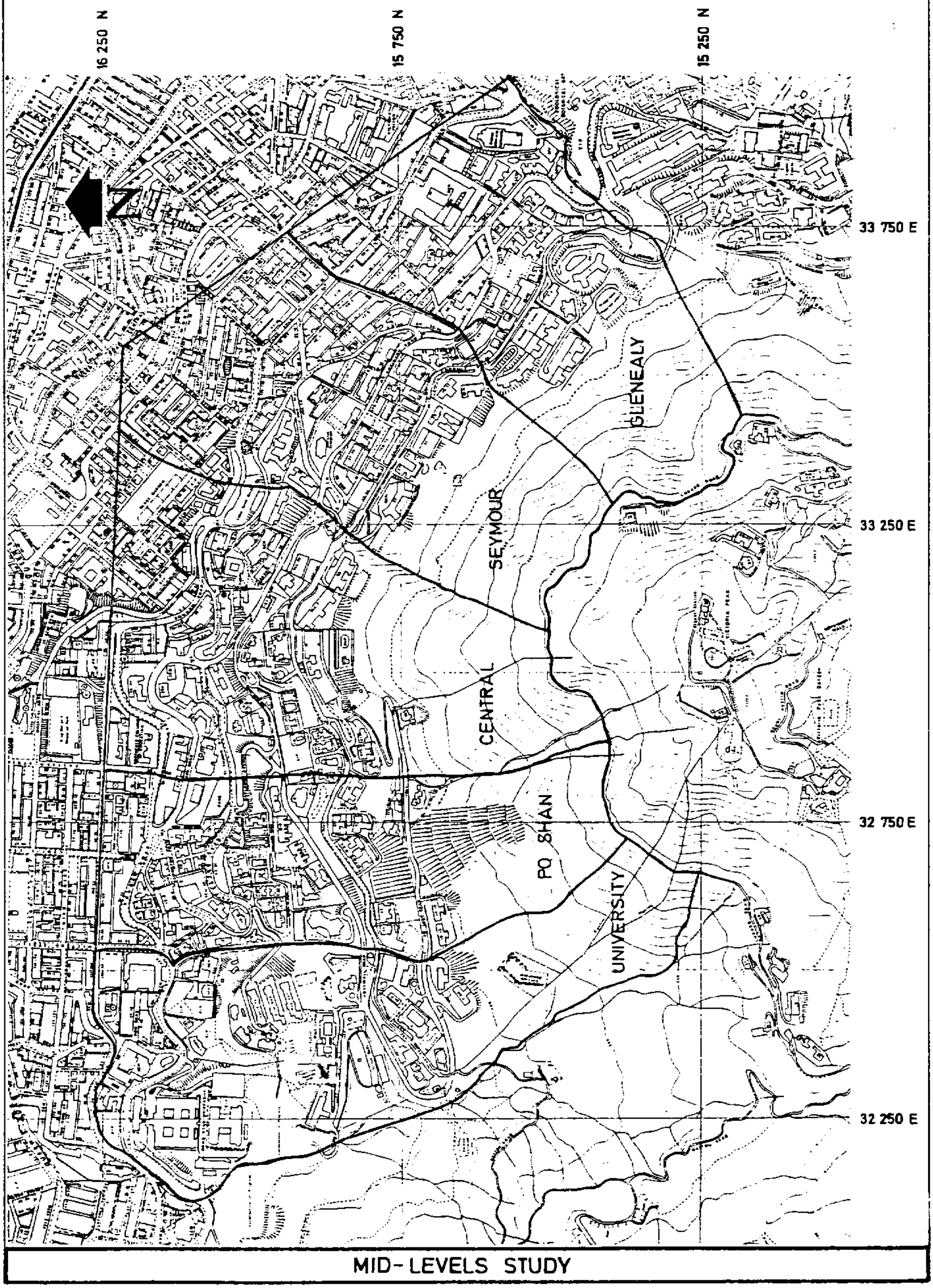
The rocks chemically decompose and weaken. This decomposition begins at the ground surface and gradually penetrates the rock mass. In many areas the decomposed surface layer slips from the upper slopes to form colluvium on the lower slopes. Erosion also occurs in the form of rock falls from exposed rock cliffs. Streams, forming along preferential drainage routes, gradually wash away the finest decomposed soils and transport them out to sea. Most of the present day erosion is concentrated in stream channels and on the upper, steeper slopes. On the lower slopes deposition occurs, resulting in an overall catenary profile.

The varied geology of Hong Kong results in different rates and amounts of erosion, which creates a very diverse landscape. The major controls on the rate of erosion are the presence of faults and the resistance of different rocks to weathering.

These controls have influenced the hydrology of the Mid-levels area. The major stream channels are straight and are aligned to the northeast or the northwest, that is, along the most common direction of faults throughout the Territory. Erosion has been concentrated along these lines of weakness. The overall form of Victoria Peak is a cone that has been incised and occasionally offset by faults. The form of the Peak is especially rugged and prominent in the region of Seymour Cliffs and this may, in part, be due to the greater resistance to erosion of the volcanic rocks near the granite contact.

LOCATION OF THE STUDY AREAS

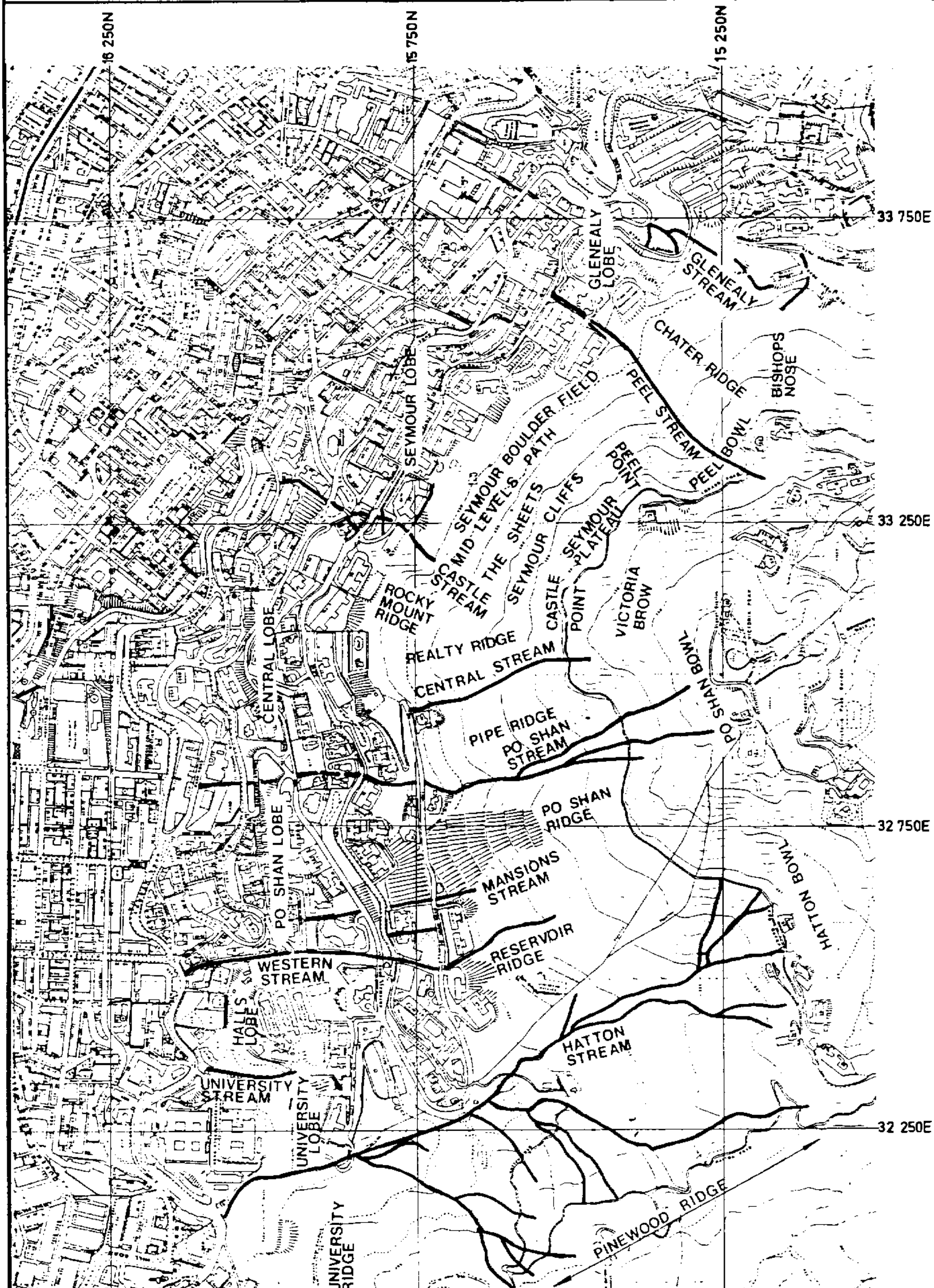
FIGURE 1.1



MID-LEVELS STUDY

GEOGRAPHY

FIGURE 1·2



MID - LEVELS STUDY

2. SURFACE WATER

2.1 Rainfall

2.1.1 Climate

Hong Kong has a variety of weather from season to season, with distinct wet and dry seasons. The winter monsoon, blowing from the north and northeast, brings dry weather. It usually begins in September and lasts until mid March and occasionally until May. Typically, only 15% of the year's rainfall falls in the months of October to March.

Summer is the wet season in Hong Kong, with the summer monsoon blowing from the south and southwest. It is not as persistent as the winter monsoon and consequently rainfall is very variable. Annual rainfall measured at the Royal Observatory (RO) has varied between 900 mm (in 1963) and 3100 mm (in 1973) and has averaged 2225 mm. Monthly mean and extreme rainfalls based on RO measurements from 1884 are given in Figure 2.1.

2.1.2 Rainfall in the Mid-levels

Twenty two tilting bucket recording raingauges have been installed in recent years by the GCO in connection with slope studies. These gauges are remotely monitored to provide 15 minute totals and the system is described in reference 48. Approximate locations of those in and around Mid-levels are shown in Figure 2.2. Apart from the gauge at the Peak Radio Station, all the gauges are sited on the roofs of buildings as these are the only sizeable unsheltered areas available.

The GCO gauges have not been installed for long enough to be of use in estimating long term annual average rainfall in the area. Dates of commissioning of the gauges are given in Figure 2.2. Isohyets of annual rainfall have been drawn by the RO (reference 49) based on rather fewer gauges, but with 25 years of data. These are shown in Figure 2.2. The data from the GCO gauges is not sufficient to justify any changes. Gauge RHM is most at variance with the isohyets and is suspected of over-reading.

However, average rainfall varies by less than 10% over the study area and such a difference is not significant given the degrees of uncertainty in other variables.

2.1.3 Storm rainfall

Data from the RO raingauge has already been analysed and the results were originally published in 1968 but have recently been updated (reference 50). Rainfall estimates are available for return periods up to 1000 years for durations from instantaneous to 31 days. There is no reason to dispute this work, and a selection of results is presented in Table 2.1 based on observations up to 1977. Long duration rainfalls are important for some groundwater responses, and we have made probability estimates for 3 and 6 month durations.

Gumbel distributions, as used in the RO analyses, were fitted by eye for 1884 - 1977. Estimates for return periods greater than 100 years have not been made due to poor fits.

TABLE 2.1 STATISTICAL ESTIMATES OF EXTREME RAINFALLS

Duration	Rainfalls for various return periods (1, 2)				
	2	5	10	100	1000
Hours: 1	68.2	88.5	102	144	185
2	94.3	128	150	219	286
4	119	169	203	307	409
8	146	213	257	395	530
24	214	309	371	567	759
Days: 2	253	366	440	675	904
4	311	441	527	796	1061
7	358	495	585	868	1148
15	483	642	747	1078	1402
31	668	847	965	1336	1700
Months: 3	1200	1480	1660	2240	-
6	1800	2250	2550	3480	-

Notes:

1. Rainfalls in mm, return periods in years.
2. Based on Royal Observatory records, 1884-1939 and 1947-77. Durations up to 31 days from reference 50; longer durations from probability analyses of data from references 49 and 52. The latter have not been extrapolated beyond 100 years in view of the possible unsuitability of the fitted probability distributions.

2.1.4 Rainfall in 1979 and 1980

1979 was a wetter year overall than average, with 2615 mm recorded at the RO. 1980 was very dry and the annual total was 1711 mm at the RO. Daily rainfalls are shown in Figure 2.3. Few major weather systems produced significant rainfall in Hong Kong in 1979-80. Major storms are summarised in Table 2.2. The most significant was the combination of Tropical Storm (TS) Gordon and Typhoon Hope at the start of August 1979. The rainfall in those seven days had a return period of about 7.5 years. Severe Tropical Storm (STS) Mac in late September 1979 was also important with a higher one day total. The storm of 30 August to 3 September 1980, although small, is of interest because it was isolated and has the most complete record of piezometer responses.

It is not possible to assign a return period to a whole year's weather. The annual maximum falls during 1979 and 1980 for various durations from 1 hour to 6 months are given in Table 2.3, together with an assessment of the return periods of these events. This is also shown graphically in Figure 2.4. All the 1980 events were minor, with return periods less than 2 years, i.e. less than the mean annual maxima. For 1979, short durations (up to 4 days) had return periods below 5 years and medium durations (4 to 12 days) were mostly about 7-8 year events. Durations from 12 to 26 days, dominated by TS Gordon and Typhoon Hope, had return periods of approximately 10 years.

Professor Lumb (reference 51) has put forward a useful visual guide to the severity of rainfall for slope stability. Figure 2.5 shows a plot of one day rainfall against the previous 15 days rainfall for various events. Historically, the points that plot furthest from the origin are associated with slope failures. 1979 and 1980 are shown on this plot, where it can be seen that only one event, in 1979, falls within Lumb's predictive zone for severe events - one day rainfall of 209 mm falling on 2.8.79.

Critical rainfall events are discussed further in Chapter 5.

2.1.5 Rainfall in 1966 and 1973

Use has been made of the rainfall records of 1966 and 1973, two notable wet years. These have been used in the groundwater modelling studies, and in assessing the statistical model relating rainfall and pore water pressure (see sections 5.2 and 5.3). The daily rainfall patterns for these years are shown in Figure 2.6 and observed maxima for various durations are included in Table 2.3 and plotted in Figure 2.4. It can be seen that 1966 had high return period rainfalls for all durations up to about 31 days, particularly in the 4 to 8 hour range. Short duration rainfalls in 1973 were not very significant but the seasonal durations were much rarer and the year had the largest rainfalls on record for 3, 6 and 12 months.

TABLE 2.2 MAJOR STORMS IN 1979-80

Dates	Name	Rainfall at RO(mm)	Maximum daily rainfall (mm)
29 July - 4 Aug 1979	TS(1)Gordon, Typhoon Hope	548	209 (2 Aug)
8-11 Aug 1979	-	189	71 (9 Aug)
23-26 Sept 1979	STS Mac	361	245 (23 Sept)
3-8 May 1980	(Thunderstorms)	223	86 (3 May)
11-14 July 1980	STS Ida(1,2)	185	74 (12 Jul)
26-28 July 1980	Typhoon Kim(2)	136	56 (28 Jul)
30 Aug - 3 Sept 1980	-	206	70 (1 Sept)

Notes: 1. (S)TS - (Severe) Tropical Storm

2. Ida and Kim did not pass very close to Hong Kong.

TABLE 2.3 OBSERVED MAXIMUM RAINFALLS

Duration		Observed maxima and estimated return periods (1, 2 & 3)							
		1966		1973		1979		1980	
		Rain	R.Period	Rain	R.Period	Rain	R.Period	Rain	R.Period
Hours:	1	108	14	76	2.8	82	3.5	44	2
	2	166	17	101	2.5	124	4.5	58	2
	4	273	47	194	8.5	148	3.3	65	2
	8	330	34	222	5.7	208	4.5	86	2
	24	401	14	257	2.9	245	2.6	102	2
Days:	2	460	12	262	2.1	253	2.0	145	2
	4	572	15	288	1.8	438	4.9	202	2
	7	680	21	354	2.0	548	7.5	223	2
	15	841	19	677	6.3	779	12	291	2
	31	1043	16	1012	13	914	7.4	507	2
Months:	3	1655	10	1999	38(4)	1553	6.7	916	2
	6	2248	5	2994	30(4)	2477	8.3	1465	2

Notes:

1. Rainfalls in mm, return periods in years
2. Observations at Royal Observatory
3. Probability analyses as Table 2.1
4. The relatively low return periods for long durations in 1973 results from the poor fit of the chosen probability distributions. In both cases, 1973 is the wettest year on record.

2.2 Evaporation and Evapotranspiration

Evaporation and evapotranspiration data are published by the RO for King's Park Meteorological Station, Kowloon (reference 49). Evaporation is measured using two U.S. Weather Bureau Class 'A' pans and potential evapotranspiration measured using three irrigated lysimeters. Full details are given in reference 53.

The mean annual evaporation at King's Park between 1958 and 1975 was 1700 mm. Evaporation is significant when considering water loss from reservoirs for example, but is not relevant in the present study as there are no free water surfaces.

The mean potential evapotranspiration at King's Park between 1952 and 1975 was 1377 mm. The mean monthly values and those for 1979 and 1980 are given in Table 2.4. In the absence of other data the values measured at King's Park have been used for the Mid-levels.

TABLE 2.4 POTENTIAL EVAPOTRANSPIRATION (mm)

Years	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1979	41	38	27	78	80	112	139	106	71	113	75	45	925
1980	49	56	22	57	90	150	176	165	129	140	96	94	1224
1952-75	83	76	89	102	132	133	153	148	136	131	106	88	1377

2.3 Runoff Measurements

As part of the Mid-levels Study, six nullahs and seven catchwaters have been monitored during 1980 using automatic flow recording equipment. The locations of the monitoring points are given on Drawing H1 and full details of the installations and the 1980 results are given in reference 36. Table 2.5 gives basic details of the installations with comments on the reliability of the data up to February 1981. All the stream gauges have concrete V-shaped weirs while the catchwaters have a mixture of sharp crested weirs and orifices. Pressure bulb recorders have been used at all but one site.

No installations were commissioned before June 1980 and therefore only limited results are available. In addition, 1980 had fewer storms than normal with no extreme events and this has restricted the rating of the gauges.

Not all flow entering the developed area from upslope has been measured as this was not the objective of the monitoring programme. The purpose and main use of the surface flow measurements has been in the estimation of infiltration.

TABLE 2.5 STREAM AND CATCHWATER GAUGING STATIONS

Weir no.	Location	Catchment area (m^2)	Date commissioned	Comments at February 1981
1	Hornsey Road	43,000	3.7.80	Good
2	7 Conduit Road	61,200	30.6.80	Poor rating
3	37 Conduit Road	53,000	18.6.80	Recorder problems. No reliable storm data
4	30 Po Shan Road	30,200	13.6.80	No significant flow recorded. Moderate rating
5	24 Po Shan Road	98,700	13.6.80	Questionable storm data
6(c)	Off Old Peak Road	29,300	1.8.80	Good storm data, see note 4
7(c)	Granite Sheets	28,900	7.8.80	No reliable storm data. 40° V-notch, see note 4
8(c)	Above 30 Po Shan Rd	4,800	7.7.80	Good storm data
9(c)	Between 8 & 10-12 Po Shan Road	36,200	8.7.80	Problems with leakage. Questionable storm data. 60° V-notch, see note 4
10	Hatton Road Nullah	341,800	7.6.80	Good
11(c)	Po Shan Slide	17,000	9.7.80	Unreliable before 6.8.80. 45° V-notch, see note 4
12(c)	37 Conduit Road (West)	7,900	7.7.80	Good. See note 4
13(c)	37 Conduit Road (East)	7,900	22.4.80	Manual only

Notes: 1. (c) Denotes gauging station on catchwater.

2. Locations and catchments are shown on Drawing H1.

3. Ratings, flow records and hydrographs in reference 36.

4. At the very low heads encountered sharp crested V-notch weirs are liable to blockage by debris such as leaves, sticks and paper. Those weirs with 60° or less notches are indicated.

2.4 Infiltration and Recharge

Part of the rain that falls on the ground surface runs off directly and part infiltrates into the ground; the latter part is further subdivided. A portion is absorbed in the unsaturated zone, meets soil moisture deficits and is subsequently lost to the atmosphere by evapotranspiration. The remainder recharges the saturated groundwater system. Some of the recharge subsequently emerges at the surface as springs and seepages; some may meet part of the evapotranspiration demand by capillary rise.

Relationships involving permeability and/or antecedent conditions are often used to estimate infiltration and recharge. Permeability models are not appropriate where conditions are very variable, such as in the upper slopes. Simple averages of permeability are not adequate as they conceal the importance of local, possibly small, high permeability zones and pipes. However, the infiltration tests carried out in the colluvium of the upper slopes indicate that it can accept rainfall intensities of more than 40 mm/hour.

Soil moisture deficits immediately prior to the most important storms of 1979 and 1980 were low, between 0-10 mm. This is in general agreement with the fact that cumulative monthly rainfall exceeds potential evapotranspiration for the months April to September (reference 53). Thus, antecedent conditions do not influence infiltration and recharge from major storms in the wet season sufficiently to have an important effect on piezometer responses. Even during dry periods the antecedent conditions are likely to have only a small effect on the infiltration and recharge from major storms.

No simple model was found for a relationship between infiltration during storms and various measures of the storms, such as duration, average intensity and total rainfall, using both 1980 data for Mid-levels and data from elsewhere in Hong Kong for other years.

2.4.1 Analysis of storms

PWD have analysed 43 storms for a variety of catchments in Hong Kong (reference 54). They were interested in storm runoff for flood estimation and so derived a ϕ index curve from the results. (ϕ index is the constant loss rate such that the volume of rainfall above the ϕ index equals the volume of direct runoff.) We have reanalysed the results for storm loss, i.e. the proportion of rainfall infiltrating into the ground. The results are very scattered with an average loss of 65% and a standard deviation of 18%.

Data on storm runoff for the Mid-levels is sparse as 1980 has been exceptionally dry and there were no quantitative stream measurements during 1979. Storm loss estimates have been made for the significant storms with available data from reliable stream gauges. Baseflow was separated in the same way as the PWD analyses discussed above. The results are given in Table 2.6 and are included in Figure 2.7. The majority of the loss estimates are high which is more likely to be due to the small storms being analysed and the generally dry antecedent conditions in 1980, rather than differences between catchments. They are not, however, at variance with the other data.

Storm loss has an inverse relationship with total rainfall (see Figure 2.7) and duration, but these are not well enough defined in the Mid-levels area to use. Nor was a useable relationship found between storm loss and average intensity, even when storms were grouped by duration. The overall average loss is 66% of total storm rainfall.

TABLE 2.6 STORM LOSSES

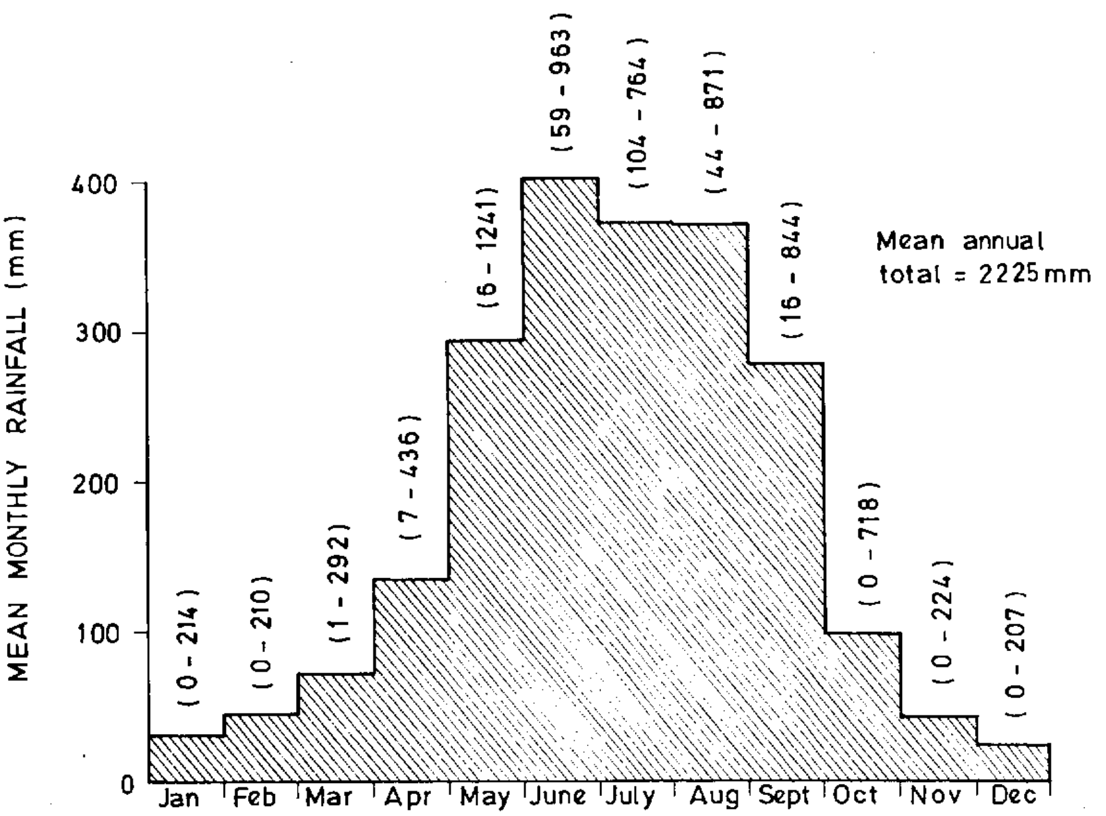
Weir no.	Date	Duration of storm runoff	Storm runoff (m^3)	Rainfall (mm)	Catchment rainfall (m^3)	Storm loss %	Comments
1	28.7.80	07.00-22.00	120	48	2065	94	
1	1.9.80	04.00-07.00	120	73	3140	96	
2	28.7.80	08.00-17.00	2250	48	2940	24	Possibly affected by basin repairs 26.7.80
2	1.9.80	04.00-21.00	2050	100	6120	66	Estimated after 16.45
5	1.9.80	04.00-24.00	1650	100	9870	83	
6	1.9.80	04.00-07.00	220	73	2140	90	
8	26.7.80	17.00-19.00	250	63	300	14	
8	28.7.80	07.00-11.00	120	40	190	27	
8	1.9.80	04.00-07.00	70	73	350	80	
10	26.7.80	17.00-23.00	4640	64	21900	79	
10	1.9.80	04.00-13.00	6490	76	26000	75)	Overlapping storm hydro-
10	1.9.80	10.30-12.30	336	9.5	3250	90)	graphs. Combined storm loss 73%
12	26.7.80	17.00-19.00	100	63	500	80	
12	1.9.80	04.00-07.00	130	73	575	77	

Notes: 1. Based on catchment areas in Table 2.5.

2. Plots of storm rainfall and runoff hydrographs are given in reference 36.

MEAN MONTHLY RAINFALL

FIGURE 2·1



Notes

1. Observed monthly maxima and minima given in parenthesis
2. Monthly maxima, minima and mean values based on rainfall recorded at the Royal Observatory from 1884 - 1977
3. Mean annual total based on rainfall recorded at the Royal Observatory during the 30-year period ending December 1980

PROJECT RAINGAUGES

FIGURE 2.2

LOCATION PLAN

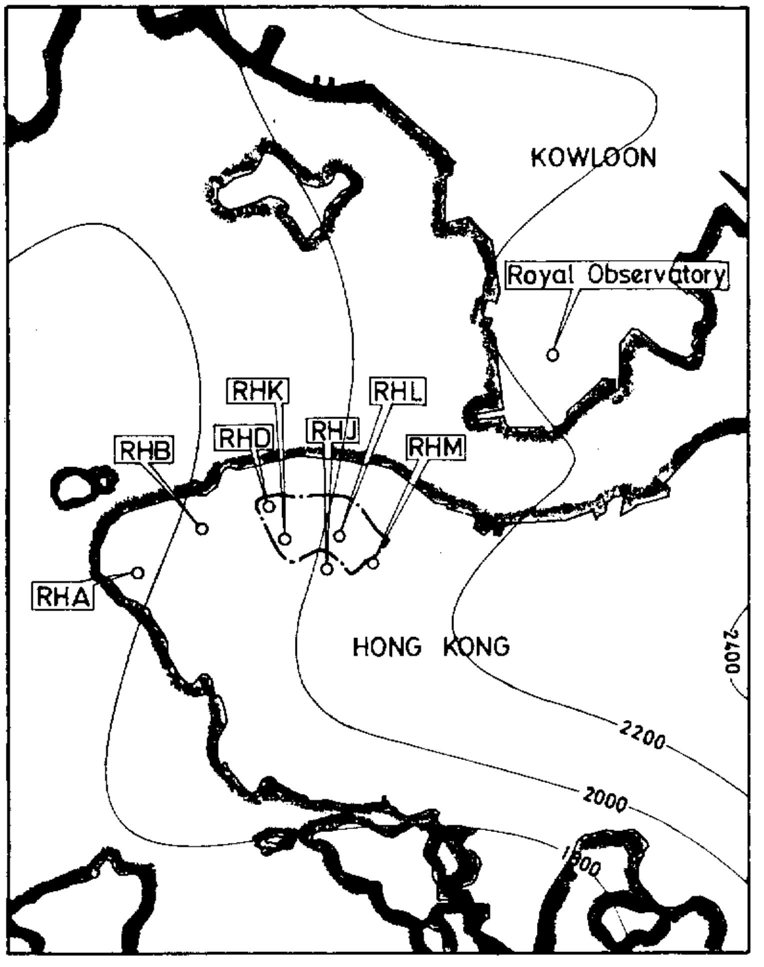
Scale 1 : 100,000

Rainguage with name or number

Mean annual rainfall 1952 - 1976 (mm)

Mid-levels study area

Based on :- 1 100,000 Topographic map, series HM 100 CL
- GCO drawing GCE 6A
- Reference 47



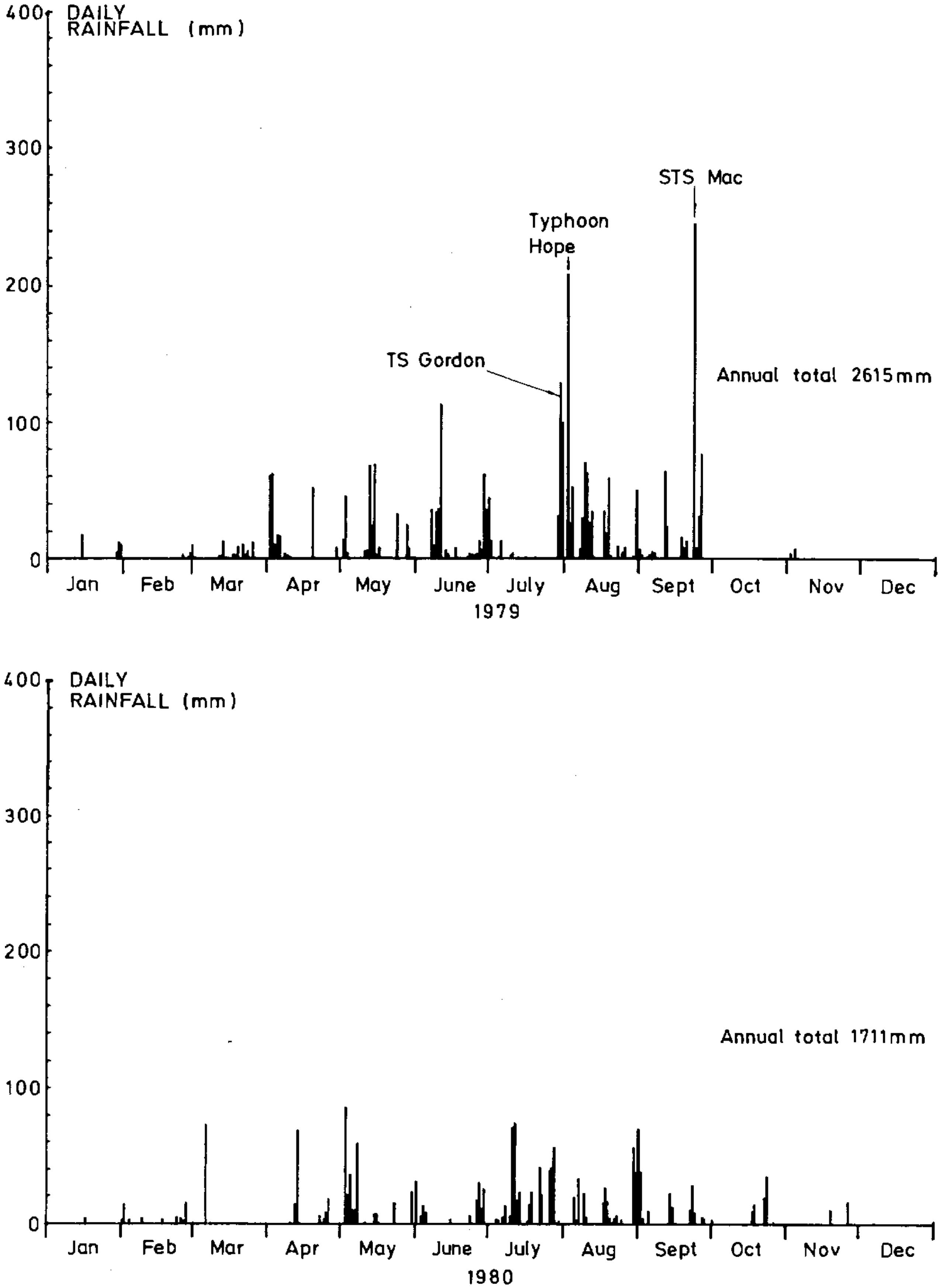
Details of gauges

Gauge number	Installation date	Location	Elevation of rim (m PD)	Rainfall 1980 (mm)	Remarks
Royal Obs.	1947	Kowloon		1711	
RHA	8.9.78	St. Clare's Girls School	107.7	896	On roof of building
RHB	8.9.78	Kwun Lung Lau Estate	95.9	1672	On roof of building
RHD	12.9.78	HK University Knowles Bldg.	122.2	1779	On roof of building
RHJ	13.9.79	Peak Radio Station	521.6	1908	Ground level
RHK	17.8.79	Hamilton Court, Po Shan Road	257.5	1602	On roof of building
RHL	23.9.79	Buxey Lodge, Conduit Road	189.1	1591	On roof of building
RHM	15.8.79	Queen's Gardens Old Peak Road	176.0	2064	On roof of building. Registers rain when dry elsewhere

MID-LEVELS STUDY

RAINFALL AT ROYAL OBSERVATORY, 1979-80

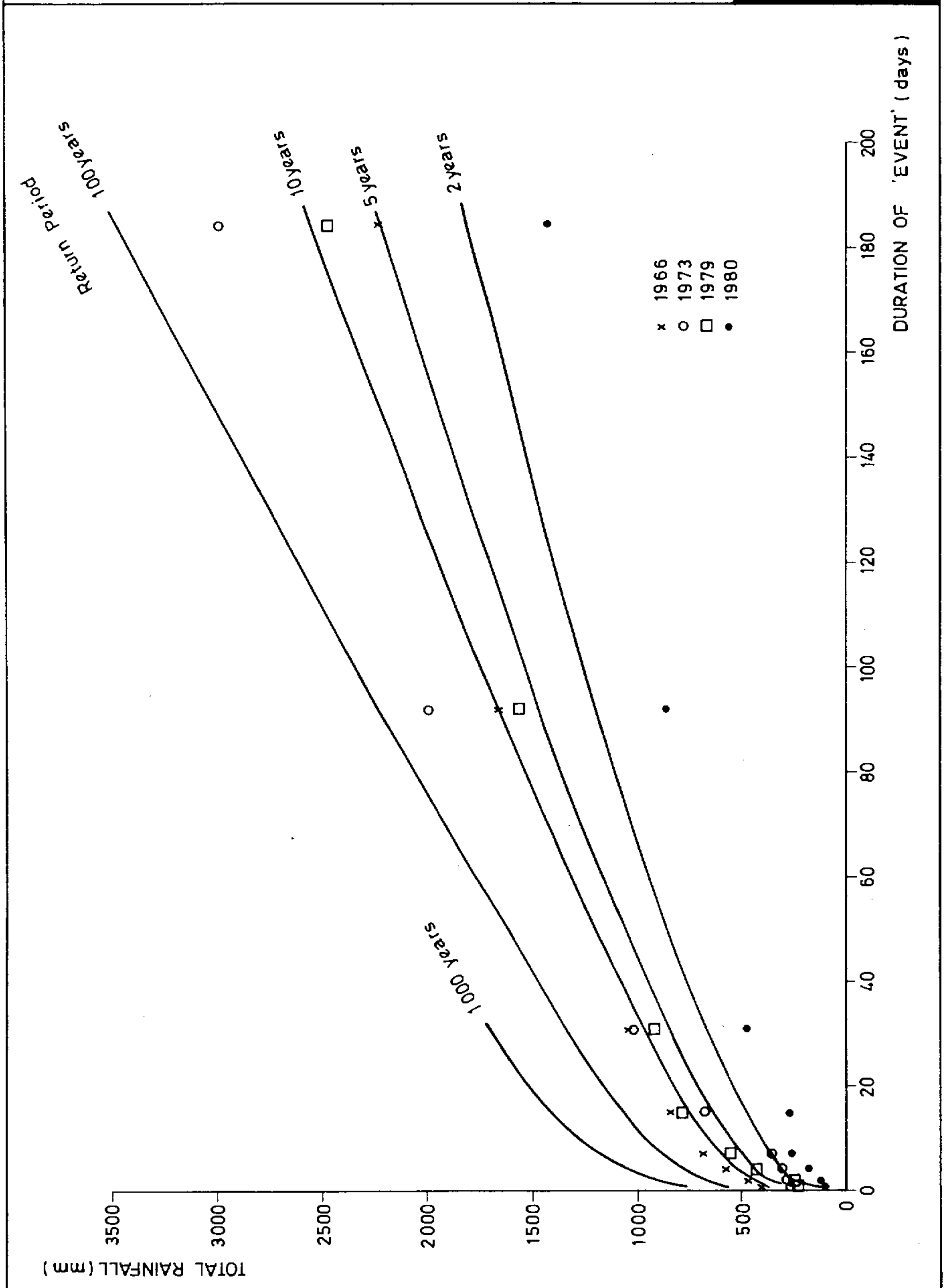
FIGURE 2.3



MID-LEVELS STUDY

EXTREME RAINFALLS

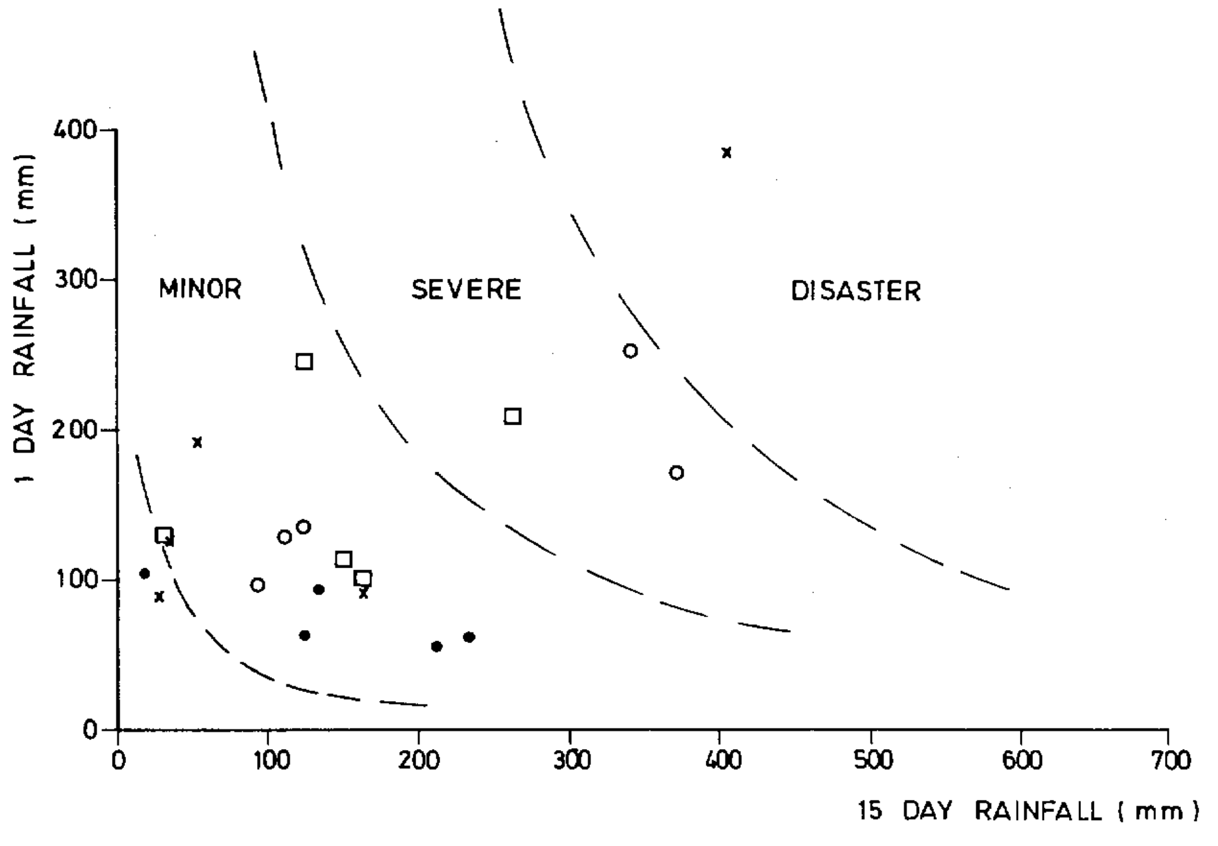
FIGURE 2·4



MID-LEVELS STUDY

1-DAY AND PREVIOUS 15-DAY RAINFALLS

FIGURE 2·5

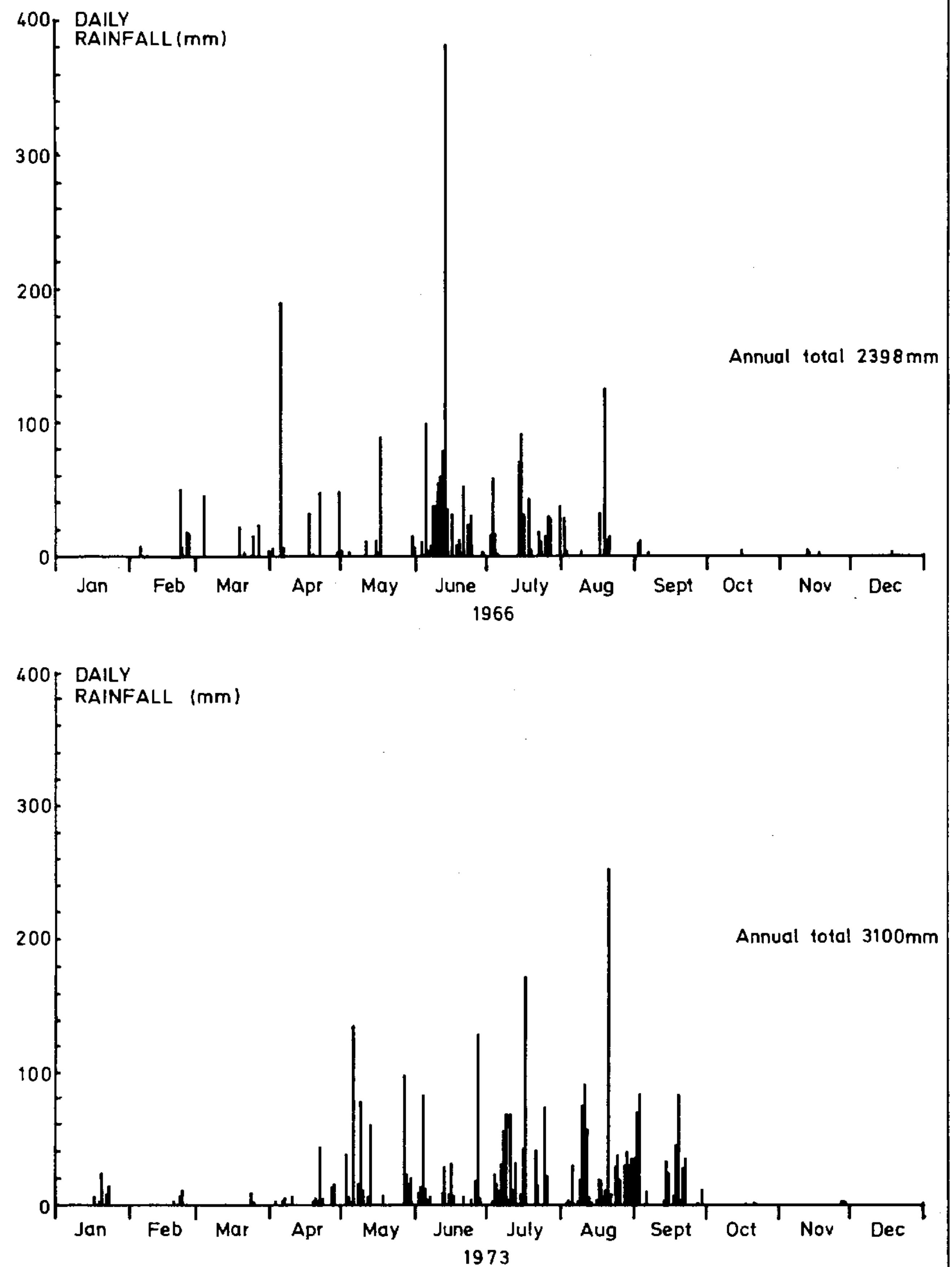


× 1966
o 1973
□ 1979
• 1980

Disaster Event:	Colony-wide damage: more than 50 slips in one day
Severe Event:	Wide spread damage: 10 to 50 slips in one day
Minor Event :	Local damage : less than 10 slips in one day

After Lumb (44) but with event boundaries modified

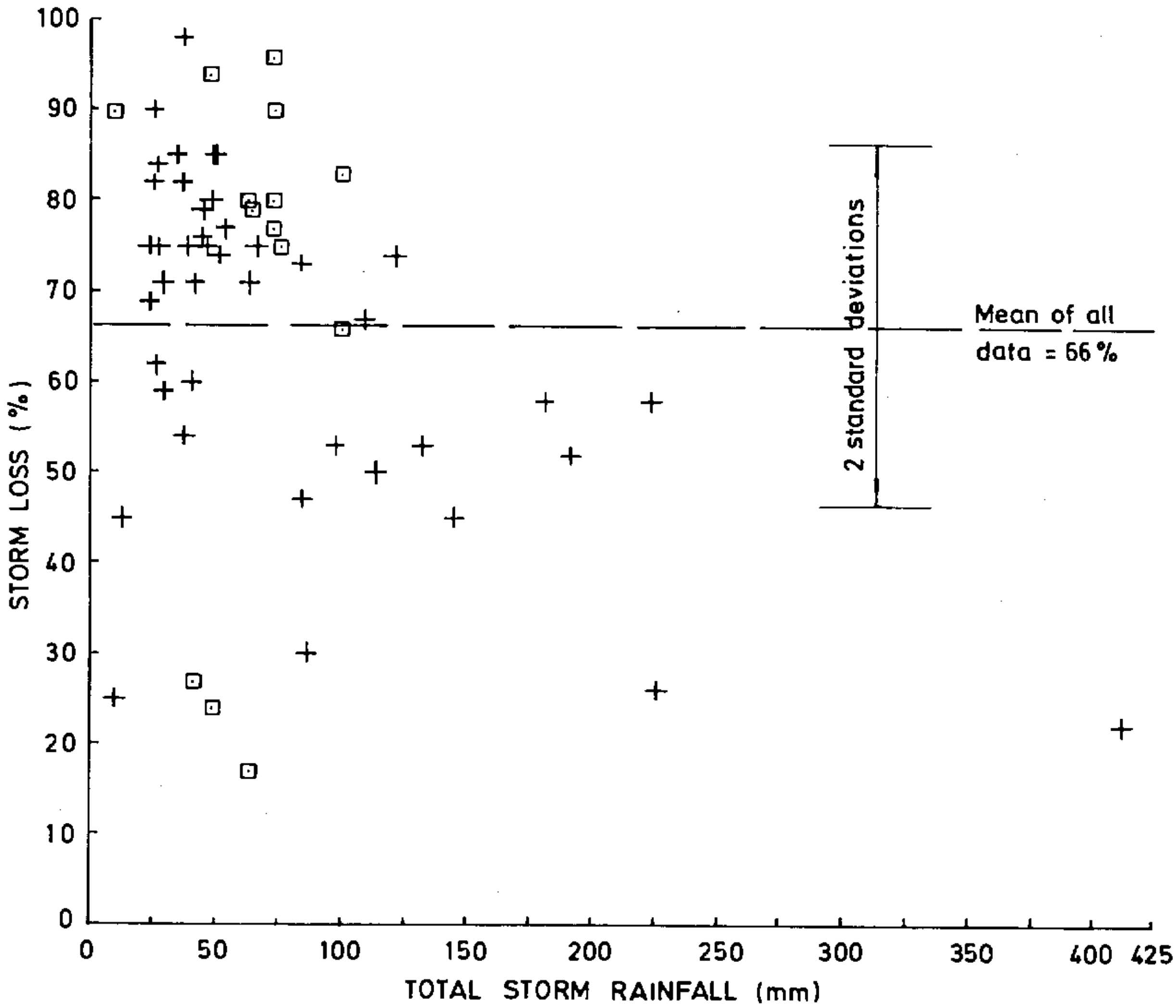
RAINFALL AT ROYAL OBSERVATORY, 1966 AND 1973 | FIGURE 2.6



MID-LEVELS STUDY

STORM LOSSES AND TOTAL STORM RAINFALL

FIGURE 2.7



□ Mid-levels data from Table 2.6

\+ Results from elsewhere in Hong Kong (47)

3. HYDROGEOLOGY

3.1 Introduction

This chapter describes the hydrogeology of the study area. Section 3.1 identifies the aquifer units and their main features, and the groundwater systems and their boundaries. Sections 3.2 and 3.3 review the available estimates of aquifer properties. The studies of soil suction are summarised in section 3.4. Section 3.5 describes the groundwater systems in detail, and discusses piezometric responses to rainfall.

3.1.1 Aquifers

The geology of the study area has been outlined in section 1.2 and is fully described in the Geology Report. The materials can be grouped into reasonably homogeneous aquifer units as follows:

a) volcanic bedrock (weathering grades I - III)

b) granite bedrock (I - III)

c) decomposed volcanics (IV - VI)

d) decomposed granite (IV - VI)

e) boulder field colluvium (class 3)

f) young colluvium (class 3)

g) intermediate and old colluvium (classes 2 and 1)

The bedrock aquifers are similar, are the least permeable and probably have the lowest storage. It is likely that they are not significant aquifers at depth as little water is found when tunnelling in Hong Kong. However they are important to this study, providing a reservoir of groundwater and some recharge to the higher aquifers with which they are in hydraulic continuity.

The decomposed rock aquifers are the major ones and, together with the bedrock aquifers, contain the main water table. There is little difference between the decomposed volcanic and granite aquifers except in permeability. The main water table is discontinuous across the contact (see section 3.5). This is probably not the case, however, across the fault. The northern (downslope) boundary of the main groundwater system is the sea of Victoria harbour, which is effectively a fixed head sink. Some groundwater is discharged as seepage.

The colluvium aquifers contain transient and permanent perched water tables which are of major importance for stability. These are connected to the main water table by both saturated and unsaturated flow. There is a major change in character of the colluvium aquifers between the upper and lower slopes. This is due to differences in the colluvium and to the effects of urbanisation.

The upper slopes are generally covered by young (class 3) colluvium which is more open, and hence more permeable, than the older colluvium (see section 3.2). There is little or no development in this area and most of the ground surface is still exposed. High rates of recharge can occur. The lower slopes are mainly covered by older colluvium. They are often paved and less natural recharge from the surface can occur. The majority of recharge to the main water table comes from the perched water tables. In places where the main water table rises above the surface of in situ material, water flows into the colluvium. The perched water tables have no fixed boundaries. In places they are effectively terminated by seepage faces in cuts or at the downslope boundary of the colluvium. In other areas perched water tables merge with the main water table.

3.1.2 Pipes

Pipes are continuous or partly continuous subsurface voids formed by internal erosion of naturally occurring voids in the soil matrix. They are normally only found in steep slopes where hydraulic gradients are sufficient to cause such erosion, but can be sizeable (200 mm diameter or more) and form partially continuous networks running downslope. They have been reported in many catchments and are recognised as a major component of the hydrological process where they occur as they can carry water rapidly downslope and possibly to depth (references 55 and 56).

The upper colluvium slopes are well suited to the formation of pipes. The steep slopes provide sufficient hydraulic gradient to initiate and develop pipes from naturally occurring voids and preferred flow paths. A number of voids, and some pipes, have been reported from test pits, drillholes and at the surface, particularly in the upper slopes; details are given in the Geology Report. No attempt was made to establish whether all the voids are pipes nor to carry out a systematic survey of the occurrence of pipes. The definite and possible pipes reported occur in colluvium (usually class 3) and occasionally in decomposed volcanics in the upper slopes.

The main effect of pipes is to greatly increase the overall permeability and infiltration rates of class 3 colluvium. This effect is discussed in section 3.2. They may have a greater effect on downslope than vertical permeability.

3.2 Permeability

To understand the flow of groundwater, particularly when using groundwater models of various types, a good knowledge of in situ permeabilities is required. The presence of an interface between permeable and less permeable underlying material is especially important because of the potential for perched water tables. The presence of a less permeable overlying material could lead to confined conditions with the potential for rapid rises in pore water pressures.

The following sections summarise the permeability measurements made, discuss the results obtained, and present assessed general permeabilities for the various soil and rock types.

3.2.1 Permeability measurements

During the Mid-levels Study, values of permeability for the various strata were determined using several methods. A summary of all measured and inferred permeabilities is given in Table 3.1.

Drillhole permeability tests were performed at most of the drilling locations. Generally constant head tests were carried out in colluvium and decomposed rock, although rising and falling head and USBR open end tests were occasionally used. Water absorption (packer) tests were performed in rock, usually of weathering grade III or fresher. Full details of the methods used and the test results have been presented in reference 29.

Infiltration tests (generally double ring) were performed in a number of trial pits and caissons, mainly in colluvium. Full details and the results of tests can be found in reference 32.

Permeability values have been calculated from flow measurements when carrying out the percolation stage prior to laboratory triaxial tests.

Estimates of permeability have been made from the measured velocity of a wetting band during a 1980 storm at two sites where automatically monitored vertical profiles of tensiometers have been installed. At the time of the estimates, suctions were close to zero and flows were controlled by gravity.

Data from large scale pumping tests elsewhere in Hong Kong in decomposed granite have been incorporated in Table 3.1.

During the course of the Mid-levels Study, falling head tests were carried out in the majority of the piezometers monitored. The object of these tests was to check the piezometer responses. They do not accurately measure material permeabilities as they may not saturate soil above the water table and there is a tendency to measure the permeability of the piezometer tip and filter in highly permeable soils. The derived permeabilities have therefore not been included.

3.2.2 Discussion of results

Permeability is the rate of flow of a fluid through a material and it may be measured directly by laboratory or field tests.

Flow through a soil or rock mass occurs only via pores, voids, pipes, or discontinuities, rather than through the matrix. Any permeability test should be as representative as possible. For this reason field tests are preferable to laboratory tests which can only test a small volume of material, generally only the matrix without any soil or rock mass features such as joints.

TABLE 3.1 SUMMARY OF MEASURED PERMEABILITY VALUES

Material	Method	Number of results	Mean permeability (x 10^{-6} m/s)
Colluvium (All types)	Drillhole test	47	7.4
	Infiltration test	34	48
	Laboratory test	20	0.3
	Wetting band	3	37
Colluvium (class 2)	Drillhole test	32	1.8
	Infiltration test	11	7.5
	Laboratory test	18	0.2
	Wetting band	1	20
Colluvium (class 3)	Drillhole test	15	86
	Infiltration test	23	75
	Laboratory test	2	15
	Wetting band	1	68
Decomposed granite	Drillhole test	71	1.0
	Infiltration test	1	6.1
	Laboratory test	23	6.3
	Pump test	6	10
Decomposed volcanics	Drillhole test	31	1.7
	Infiltration test	6	3.7
	Laboratory test	25	0.1
Granite rock	Drillhole test	20	0.2
Volcanic rock	Drillhole test	9	0.9

Permeability varies significantly due to many factors which are listed below:

a) Soil

   i) grain size; the distribution of the finest fractions is particularly important

   ii) arrangement of particles; sorting, stratification, orientation of grains

   iii) presence of highly permeable zones such as gravel or voids

   iv) density of soil

b) Soil and rock

   i) presence of discontinuities, their infilling, continuity, and interconnection.

Soil and rock masses are generally heterogeneous resulting in different permeabilities in the horizontal and vertical directions. Field permeability tests using boreholes primarily measure the horizontal permeability.

Apart from pumping tests, field permeability tests conducted in boreholes requiring a change in head of water are susceptible to many errors. For example:

a) leakage along the casing and around packers

b) clogging of pores by fines in the test water (this occurs with constant and falling head tests but can be avoided by using rising head tests)

c) air locking due to bubbles in the soil or test water, (this phenomenon is more common in the unsaturated zone above the groundwater table).

As permeability is subject to so many variables, and field permeability tests susceptible to numerous sources of error, it is not surprising that results vary considerably and that repeatability is often poor. Permeability test results are often quoted as a range to take into account these factors. This range is commonly one order of magnitude.

A greater emphasis has been put on the results of the larger scale permeability tests in assessing general values for the various soil and rock types.

A further consideration is that drillhole permeability tests give values tending to the horizontal permeability, whereas infiltration tests and wetting band estimates give values tending to the vertical permeability.

a) colluvium

The results of the borehole and infiltration tests are summarised and plotted against depth on Figure 3.1. The scatter is very large, but a possible gradual reduction in permeability with depth is suggested. Mean values from the other test methods are also shown.

Three classes of colluvium have been recognised (Geology Report), class 1 being the oldest and most decomposed, class 2 intermediate and class 3 being the youngest. In general, class 1 and 2 colluvium are found at lower elevations with class 3 mainly on the upper slopes. Very little colluvium has been recognised as class 1 and no permeability tests were carried out in this type of material. As might be expected, the younger colluvium has a more open structure than the old decomposed material and this is confirmed in the evaluation of mean permeabilities for the various colluvium classes (Table 3.1).

An important subdivision within class 3, the youngest colluvium, is the differentiation between material having substantial amounts of fine matrix and that comprising all, or almost all, boulders and cobbles with very little matrix. The latter type of colluvium is known as a boulder field, the most significant of which occurs above 7-37 Conduit Road (Seymour Boulder Field, Figure 1.2 and Geology Report). No permeability tests were carried out in this type of colluvium, but one borehole test close to the edge of the boulder field (ML113B at 4.22 m depth) gave a permeability of 120×10^{-6} m/s. Three infiltration tests, again close to the edge of the boulder field (caisson K4 on Chater Ridge, Figure 1.2) also suggested a high permeability. In all three tests, the measured permeability was affected by the presence of voids or pipes within the colluvium and the values measured were limited by the maximum rate of flow available from the test apparatus. This was equivalent to a permeability of about 350×10^{-6} m/s. It is therefore considered that the permeability of the boulder field colluvium is probably greater than 10^{-3} m/s.

General values of permeability for class 2 and 3 have been assessed on the basis of relevance of the test type. It is considered that the wetting band and infiltration test results are best estimates of in situ mass permeabilities for the colluvium. Assessed general values are given in Table 3.2.

b) decomposed rock (weathering grades IV to VI)

The results of the borehole tests and one infiltration test in decomposed granite are presented on Figure 3.2. The mean values from laboratory test, wetting band, and pump test results are also shown. There is no significant difference between the measured permeabilities of highly decomposed granite (IV) and completely decomposed granite (V and VI).

As zones of high permeability have most effect on ground water movement larger scale tests were given more weight and a range of 7-10 x 10^{-6} m/s was assessed for the decomposed granite.

The results of the permeability tests in decomposed volcanics are presented on Figure 3.3. Although a value for wetting band advance is shown, this value is based on a best estimate from tensiometers which are suspected of not fully equilibrating to changes in soil suction. It is possible, therefore, that the permeability thus estimated may be a lower bound. Confidence is therefore placed on the infiltration test results and in the absence of a good wetting band value, a general permeability of 4 x 10^{-6} m/s is suggested.

c) bedrock (weathering grades I to III)

The results of twenty borehole tests in granite are summarised in Figure 3.4. They have a mean value of 0.2 x 10^{-6} m/s.

The nine borehole test results in volcanic rock (Figure 3.5) have a mean value of 0.9 x 10^{-6} m/s. The relative permeabilities of granite to volcanic rock are consistent with the wider joint spacing in the granite.

On the basis that borehole permeability tests generally underestimate mass permeabilities, the above average values are considered to be lower bounds; the highest measured values may reflect bulk permeabilities.

3.3 Storage Coefficients

The storage coefficient of an aquifer measures the amount of water taken in (or released from) storage for a unit change in piezometric head. Thus the piezometric response of an aquifer is closely related to its storage coefficient.

Different values apply in unconfined and confined situations when the aquifer is fully saturated. In the former case, changes in water content are principally in the zone of movement of the water table, and the storage coefficient is independent of aquifer thickness. In a confined aquifer, storage is due to the compressibility of the whole depth of aquifer and storage coefficients depend on saturated thickness.

3.3.1 Available estimates

Estimates of storage can be obtained from pump tests and laboratory measurements. Pump test results are available only for the decomposed granite aquifer. Four laboratory tests were carried out on decomposed granite.

From mathematical modelling the following storage coefficients were found to give good correlation between observed and predicted piezometric response:

Material	Conditions	Storage coefficient	Comments
Colluvium	Unconfined	0.05	
Decomposed granite	Unconfined	0.03	Associated permeability 7×10^{-6} m/s
Decomposed granite	Unconfined	0.02	Associated permeability 5×10^{-6} m/s
Decomposed granite	Confined	0.001	Typical aquifer thickness of 20 m

Gravity drainage tests carried out on four decomposed granite samples gave storage coefficients between 0.02 and 0.04, averaging 0.03.

3.3.2 Assessed storage coefficients

Assessed storage coefficients are summarised in Table 3.2.

a) Colluvium. In the absence of any field estimates, a range of 0.05-0.1 is estimated for unconfined colluvium based on the modelling studies and experience elsewhere in the world. No distinction has been made between colluvium classes.

b) Decomposed granite (IV-VI). Pump tests and laboratory results both give an unconfined coefficient of 0.03. The confined specific storage coefficient estimates range between 1×10^{-4} and 5×10^{-5} (modelling).

c) Decomposed volcanics (IV-VI). No estimates are available.

d) Granite and volcanic bedrock (I-III). No estimates are available. Storage will be in the joint systems, and will depend on jointing intensity and continuity. It will also depend on joint thickness, whether they are formed by movement or the action of water, and on the type and amount of infilling.

TABLE 3.2 ASSESSED AQUIFER PROPERTIES

Material	Permeability ($\times 10^{-6}$ m/s)	Storage coefficient unconfined	confined
Old colluvium (Class 1)	n.e	0.05-0.10	n.e
Intermediate colluvium (class 2)	5-20	0.05-0.10	n.e.
Young colluvium (class 3)	50-350	0.05-0.10	n.e.
Boulder field colluvium	> 1000	0.05-0.10	n.e.
Decomposed granite (IV-VI)	7-10	0.03	$5\text{-}10 \times 10^{-5}$
Decomposed volcanics (IV-VI)	4	n.e.	n.e.
Granite rock (I-III)	0.2	n.e.	n.e.
Volcanic rock (I-III)	0.9	n.e.	n.e.

Notes: n.e. - no estimate available

(1) Although no estimate is available, class 1 colluvium permeability is probably close to the low end of the class 2 permeability range.

3.4 Suction

The measurement of soil suctions was considered an essential part of the study, firstly to ascertain whether residual suctions were maintained in the soil, and secondly to gain knowledge of water flow and pore water pressures above the main groundwater table.

Five sites within the Mid-levels Study area were instrumented with tensiometers to investigate suctions at locations with differing conditions of geology, topography, groundwater, and surface cover. The sites are Realty Ridge, Chater Ridge, Chater Hall, Merry Terrace and Mansions/Reservoir Ridges (Figure 3.6). The sites and instrumentation have been fully described in reference 32. Plots of soil suction with time have been presented for 1979 and 1980 (reference 32). This section of the report briefly discusses and summarises the results of the suction work.

3.4.1 Discussion of results

Each instrumented site has its own unique conditions which affect the local suction and groundwater systems. As a result a single overall summary for all sites is not appropriate. This discussion, therefore, is limited to the most important observations made, independent of the particular site. It is considered, however, that the points discussed are equally applicable to other parts of the study area.

a) range of measured suction in colluvium

All measured suctions in colluvium have fallen to zero or near zero values during the monitoring period. In some cases small positive pressures have developed. Positive pressures up to $5kN/m^2$ were developed in the colluvium at several locations at varying depths, including the in situ boundary, in response to the storm from 30.8.80 to 1.9.80. This storm had a return period of less than two years, the equivalent 1 in 10 year storm producing almost three times the rainfall of the 1980 storm. Significantly higher positive pressures may, therefore, be expected to occur within the colluvium in wetter years or in response to storms of longer return periods or different durations. The build up of small positive pressures within the vertical colluvium profile is compatible with the variation in permeability within the colluvium (reference 29).

The majority of tensiometers in colluvium are within 3.5 m of the ground surface, although four vertical profiles (Realty and Chater Ridge) extend to about 10 m below ground surface. Three of the profiles instrument the full depth of colluvium and cross the in situ boundary, whereas all the tensiometers in the K3 profile at Chater Ridge are installed in colluvium. Apart from two groups of tensiometers under chunam surfaces and one group under a concrete slab, all tensiometers are installed below an exposed or vegetated ground surface.

The evidence suggests that suctions fall to near zero values to depths of 10 m below ground in colluvium with an exposed surface. Figures 3.7 and 3.8 show the minimum suctions recorded in colluvium at Realty and Chater Ridge on 1.9.80 together with values recorded on 30.1.81, a day at the end of 2 months without rainfall. All suctions on 30.1.81 were greater than 10 kN/m^2 and at several locations the suctions were higher than the upper limit of measurement of the tensiometers (75-80 kN/m^2). Dry season suctions are very variable, and are controlled by the proximity of the ground surface (in exposed areas) rather than the height above the base of colluvium.

b) range of measured suctions in decomposed granite

Five tensiometers at Chater Ridge and fifteen at Chater Hall are installed in decomposed granite. The tensiometers at Chater Ridge (K4) have shown zero suctions and positive pressures but this is not unexpected as caisson K4 is adjacent to a topographic hollow which is a major drainage course (Peel Stream, Figure 1.2). Consequently the main water table is close to the lower tensiometers in this group. Small positive pressures were still being recorded by these tensiometers in January 1981.

Six of the tensiometers at Chater Hall (Group C) are between 0.5 m and 4.5 m from an exposed sloping face (vertical depths of 1.1 m and 6.8 m). All these tensiometers recorded zero or near zero suctions on 1.9.80 although the deepest three tensiometers had only been installed two weeks prior to that date. Figure 3.9 shows the range of suctions recorded at this location.

The Group B tensiometers at Chater Hall are between 0.5 m and 4.5 m beneath a chunam-covered decomposed granite cut slope (vertical depths of 1 m and 7.5 m). All six tensiometers have maintained a residual suction of about 20 kN/m^2 throughout the period of monitoring. The effect of the chunam surface protection is discussed separately below. Figure 3.10 shows the range of suctions recorded at this location.

The lower three tensiometers of Group D at Chater Hall are installed in decomposed granite which is overlain by a thin veneer of colluvium, about 1 m thick. These tensiometers were installed in August 1980. Tensiometers in the colluvium at this location have recorded zero suctions, whereas the limited evidence from the deeper tensiometers suggests that this may not be the case for suctions in the underlying, decomposed granite at shallow depths.

c) range of measured suctions in decomposed volcanics

Tensiometers installed in decomposed volcanics at Realty Ridge suggest that, at depth, residual suctions are maintained throughout wet periods. At caisson K2, where the decomposed volcanics are overlain by 1 m of colluvium, tensiometers from 2.5 m to 10 m depth maintained a small suction during and after the storm of 30.8.80 - 1.9.80. A minimum residual suction of about 10 kN/m^2 was maintained at a depth of 10 m below ground surface throughout the period of monitoring. Figure 3.11 shows the range of recorded suctions by the vertical profile of tensiometers in decomposed volcanics at caisson K2. Several tensiometers elsewhere on the site however, recorded zero or near zero suctions to a depth of 4 m in response to the 1980 storm.

d) flow of water above the main groundwater table

A plot of total water potential with elevation which shows the direction of water flow for vertical profiles of tensiometers is given for the deepest vertical profile in colluvium (K3, Chater Ridge) in Figure 3.12.

The concept of total water potential plotted against elevation potential is used to illustrate the direction of flow of water above the main groundwater table. The direction of flow between two points in space depends on the relative water pressures which consist of two parts:

i) their relative elevations,

ii) the relative water pressures or suctions acting at each point.

The sum is referred to as the total water potential.

In deriving the elevation/total water potential plot an arbitrary datum in elevation is selected, which for the present purpose has been taken to be just below the lowest tensiometer of the vertical profile considered. As the positions of the tensiometers are fixed, each tensiometer can be given an elevation potential, equal to the elevation of the tensiometer above the selected datum. For convenience, the elevation potential has been converted to units of pressure:

$$\text{elevation potential } (kN/m^2) = \left[\begin{array}{l}\text{elevation head}\\ \text{above datum}\end{array} (m)\right] \gamma\omega \ (kN/m^3)$$

Total water potential, plotted on the horizontal axis, is defined:

$$\text{total water pressure } (kN/m^2) = \text{elevation potential } (kN/m^2) + \text{pore water pressure } (kN/m^2)$$

A suction is considered as a negative pressure of the same magnitude.

Hence, if the sum of elevation potential and suction is the same for two adjacent points no flow will occur between the two points as the total water potentials are also equal. This situation would plot as a vertical line on an elevation/total water potential plot and would represent a hydraulic gradient of 0 (i.e. hydrostatic).

If, however, the suction magnitude increases with elevation at a rate greater than the elevation potential, flow would have an upward component as the total water potential decreases with increasing elevation. Vertically upward flow would be indicated by a gradient of -1 on the elevation/total water potential plot.

Conversely a line having a positive gradient on the same plot would indicate flow with a downward component, with a limit of +1 indicating vertically downward flow.

The plot in Figure 3.12 shows total water potentials before and after the storm of 30.8.80 - 1.9.80. The points plotted for 30.8.80 are immediately prior to rainfall and show downward flow to be occurring from a depth of about 3 m below ground surface with suctions being close to zero. Above 3 m the downward flow is less strong and in the upper 1.5 m hydrostatic conditions exist with suctions over 10 kN/m^2 close to the surface.

Minimum suctions were recorded on 1.9.80 with all suctions falling to zero and small positive pressures developed locally. In this near saturated condition (Soil Properties Report) downward flow is implied throughout the whole profile (hydraulic gradient = +1). By 10.9.80 the conditions had returned to those prior to the storm.

The dry season total water potentials again show a downward component of flow below about 3 m depth but strong upward flow above 3 m, with a hydraulic gradient of approximately -1.

e) effect of surface cover

Group A and B tensiometers at Chater Hall are installed beneath a chunam-covered cut slope. Figure 3.13 is a cross section through the slope showing the extent of chunam. Due to the existence of a pre-colluvial drainage channel, now infilled, part of the slope face is of colluvium whilst part is decomposed granite (reference 32). Group A tensiometers are installed in colluvium, Group B in decomposed granite. The chunam cover at both locations is similar and is in good condition, the drainage channels above the instrument groups have been checked for leakage and appear to be sound (reference 32).

The tensiometers in decomposed granite have recorded a residual suction of 20 kN/m^2 throughout the monitoring period, although they still respond rapidly to rainfall. The tensiometers in colluvium however, have both a very rapid response to rainfall and the suctions drop to zero or near zero values. This is due to infiltration into the colluvium above the chunammed surface and throughflow to the location of the tensiometers and probably also a result of limited direct infiltration through the chunam surface.

After a rainfall event the suctions appear initially to rise at rates similar to that for exposed slopes (Groups C and D), but the rate of rise is not maintained as suctions increase. This suggests that the chunam cover is reducing evapotranspiration compared to that occurring from the exposed slopes. The initial rise is due to gravity drainage of the soil.

Chunam cover can be effective in reducing infiltration into soil slopes but the cover must extend sufficiently far above the slope to be effective and it must be in good condition. The required extent is dependent on the permeability of the covered soil. For soil of higher permeability, a more extensive areal chunam cover is required.

Four tensiometers are installed to a depth of 3 m below the concrete slab of the car park of a residential block located in Central Mid-levels (reference 32). The upper tensiometer, at 1 m depth, is in fill, the tensiometer at 1.9 m depth is at the base of a thin layer of colluvium and the lower two are in decomposed granite. Since installation, the upper tensiometer has recorded a small suction of between 5 and 10 kN/m^2 but does show a damped response to rainfall. The lower three tensiometers have always recorded zero suction or positive pressures. A positive hydrostatic pressure from about 1.7 m below ground surface is implied. As the main groundwater in the general area is well below this, it is probable that a perched water table has been formed by water leaking from drains and/or services in the vicinity.

3.4.2 Conclusions

Soil suctions in colluvium of the upper slopes reduced to zero to depths of at least 10m below an exposed or vegetated ground surface in response to rainfall during the study period.

Positive pore water pressures of up to 5 kN/m^2 developed in the colluvium in response to minor storm events. Significantly higher positive pore pressures are likely to occur within the colluvium as a result of greater rainfall events than were monitored in 1980.

Soil suctions in decomposed granite reduced to zero at vertical depths of at least 3 m below an exposed ground surface in response to modest rainfall. For decomposed volcanics, the depth to which suctions dropped to zero in 1980 was 4 m although the in situ material was overlain by a thin cover of colluvium.

Where the main groundwater table is at depth, soil suctions in decomposed volcanics at depth were not significantly affected by rainfall during 1980. A residual suction of 10 kN/m^2 was maintained throughout the monitoring period at a depth of 10 m below an exposed ground surface.

Predominantly downward flow of water occurs in the colluvium of the upper slopes during wet periods. During dry periods the flow direction close to the ground surface is upward but below about 3 m depth the flow direction has a downward component.

Chunam surface cover, provided that it is in good condition, reduces infiltration into soil slopes. However, tensiometer behaviour suggests that wetting and evaporation does take place through such a surface. It is important that the chunam is of sufficient areal extent outside the slope for it to be effective. The greater the permeability of the soil, the greater the required extent of chunam.

3.5 Observed Groundwater Conditions

About four hundred piezometers have been monitored during the study. Some of these were installed for the study, others were already in existence. Their locations are shown on Drawing H2.

Hydraulic gradients in most of the study area are steep and therefore piezometric levels must be corrected for non-hydrostatic conditions. This has been done by assuming that flowlines are parallel to the ground surface. All levels have been corrected unless otherwise stated.

Where piezometric data are not available, high hydraulic gradients have a further implication when predicting pore pressure rises due to infiltration. For materials of high permeability such as class 3 colluvium, downslope flow must be considered. A simple estimate of pore pressure build up using a wetting band approach (reference 51), and neglecting throughflow, is not adequate in these situations. This is discussed further in section 5.4.3.

3.5.1 Main water table

The main water table is continuous between the decomposed rock and bedrock aquifers and has been observed in most piezometers installed in those materials.

Hydraulic gradients of the main water table are generally high and approximately the same as topographic gradients. This is greater than 30° in parts of the upper slopes reducing to 10° - 15° in the lower parts of the study area. The main water table is discontinuous across the volcanic/granite contact; in the University area levels in decomposed granite are about 10 m lower. This has been observed at several locations and is illustrated for three groups of piezometers in Figure 3.14. The discontinuity probably represents a seepage face between two materials of different permeabilities.

There are two major sources of recharge to the main water table:

a) Infiltration from rainfall. This recharge first passes through the colluvium aquifer, where present. Some recharge is direct from stream beds such as Hatton Stream, mainly in the upper and intermediate slopes.

b) Leakage from services. This is a major source of recharge in parts of the urban area is discussed in sections 3.5.2 and 4.2.

The main water table discharges to the sea, with some discharge to seepages. These are springs in a few cases, but are usually in man-made cuts. They are discussed separately in section 3.5.4. There are major drainage measures in a few locations. (section 4.4)

In some localised areas, particularly downslope, the decomposed rock aquifers may be overlain by colluvium which has a lower permeability than the in situ material. In these situations it is possible that confined, or partially confined, conditions exist in the main water table with the potential for high pore water pressures. No such areas, however, have been positively identified.

There are vertical flows within the main water table. In particular, strong upward hydraulic gradients from bedrock to the decomposed rock aquifer have been observed in some areas. An example for a series of piezometers in the granite aquifers, is shown on Figure 3.15.

3.5.2 Perched water tables

Perched water tables have been observed in many piezometers throughout the study area. Perched water tables are not always present over the whole study area and their characteristics vary from place to place. Some are transient and only exist during storms. Some are semi-permanent, and others have existed throughout the study period.

Perching will occur wherever there is a significant decrease in permeability with depth, provided that there is sufficient recharge from above. Permeability data (Table 3.2) indicates that such a change occurs between colluvium and both decomposed volcanics and granite.

None of the piezometer series have sufficiently closely spaced tips near the base of the colluvium to provide conclusive evidence that this is the major perching level. It can however be inferred from several series that include tips above and below the interface that show different levels (Figure 3.16). There are a few locations that indicate perched water tables in decomposed volcanics. Most of these are thought to indicate downward hydraulic gradients and flow, rather than true perching. Perching may occur at higher levels in the colluvium. Suction data (section 3.4) indicates that some positive pore pressures develop through the colluvium, but have not identified any major perching levels.

It was assumed for this study that perched water tables, when they occur, are based on the interface between colluvium and decomposed rock.

Three sources of recharge have been identified by considering the patterns of piezometer and tensiometer responses, groundwater levels on sections, the nature of seepages, and a chemical survey of seepages and of a limited number of piezometers. The chemical survey is summarised in section 4.1.2. These sources of recharge are infiltration from storms, leakage from services, and upward recharge from the main water table. They can be identified by the following characteristics:

a) Infiltration from storms. Marked piezometer and seepage response to rainfall. Drop in suctions and downward flow indicated by a vertical profile of tensiometers. There must be suitable areas for recharge to occur. Chemistry would indicate no contamination by sewers.

b) Leakage from services. Usually identified by chemistry or by the presence of perched water tables in areas without natural recharge. Leakage from storm sewers is suggested in completely paved downslope areas where piezometers show storm responses. Erratic fluctuations in piezometric levels suggest leaking fresh or flushing water mains.

c) Upward recharge from the main water table. Suggested by seasonal patterns in perched water tables and similar responses to rainfall in colluvium and the underlying layer. Requires the main water table to be above the colluvium base upslope.

The three recharge sources can, and do, occur in combination.

Figure 3.17, shows the response of piezometer group ML113 to recharge occurring into the colluvium initially by infiltration (rises in ML113B(U) and (L) on 12.7.80) and later by upward flow from the main water table (15.7.80 - 16.7.80).

3.5.3 Piezometric responses to rainfall

Two components, seasonal and storm, have been identified in the observed fluctuations in piezometric levels. The seasonal component reaches a low at the beginning of the wet season or later, and a high some time after the end of the wet season. The storm component rises and falls in response to each significant rainstorm. A more detailed classification has been proposed and is shown in Figure 3.18. In practice the divisions merge into each other.

The main water table shows both types of response. The seasonal rise in 1980 is shown in Drawing H3. 1980 was a relatively dry year, and recorded rises were not as great as those in 1979.

However, there are less data available for 1979. Larger rises were recorded in the upper slopes than the lower, reaching 15 m in the Seymour area. Rises were smaller in the Po Shan and University areas than elsewhere, and very small in the area of the Po Shan remedial works where sub-surface drainage has been installed.

Main water table response to the storm of 30.8.80 to 3.9.80 (206 mm at Royal Observatory) is shown on Drawing H4. The largest observed responses were in the region of Conduit Road at the foot of the Seymour Boulder Field with 6 m being recorded just above Conduit Road. The rises in this area cause a 'pressure wave' rise downslope through the developed area with an effect as low as Caine Road. The shape of the contours of rise in water table on Drawing H4 show this effect to be limited to the area known as the Seymour Lobe (Figure 1.2). Observed rises elsewhere in the study area were generally less than 2 m. Rises in the regions of the valleys within the study area are noticeably smaller than those on the ridges.

Perched water table response to the storm of 30.8.80 to 3.9.80 is shown on Drawing H5. Rises are largest in the undeveloped colluvium slopes and at the general break in slope along Conduit Road and Kotewall Road. The most significant rises are again in the region of Conduit Road at the foot of the Seymour Boulder Field where high colluvium permeability (section 3.2) is expected to result in significant and rapid throughflow. Rises in the urban area generally are small, except where influenced by services.

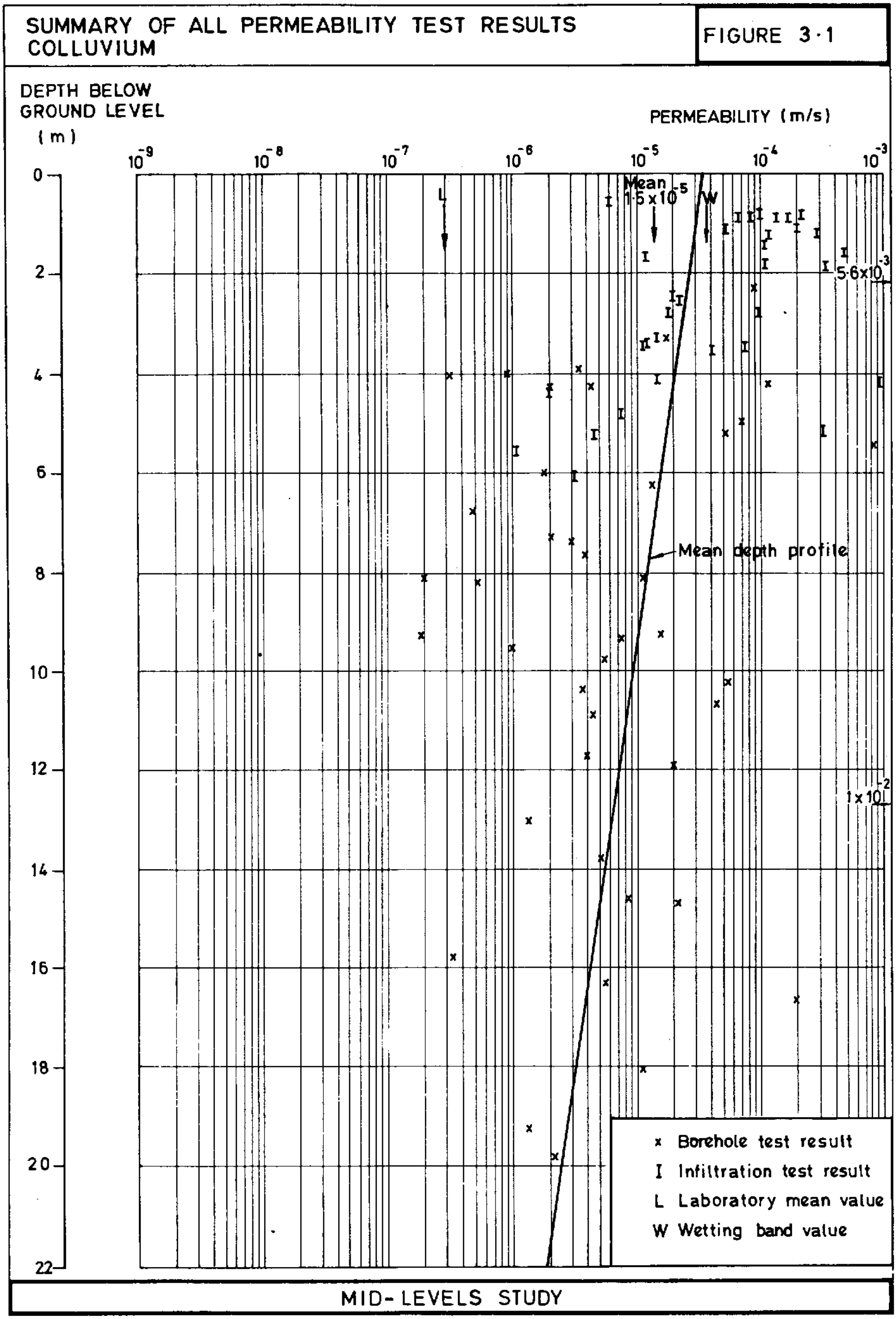
3.5.4 Seepages

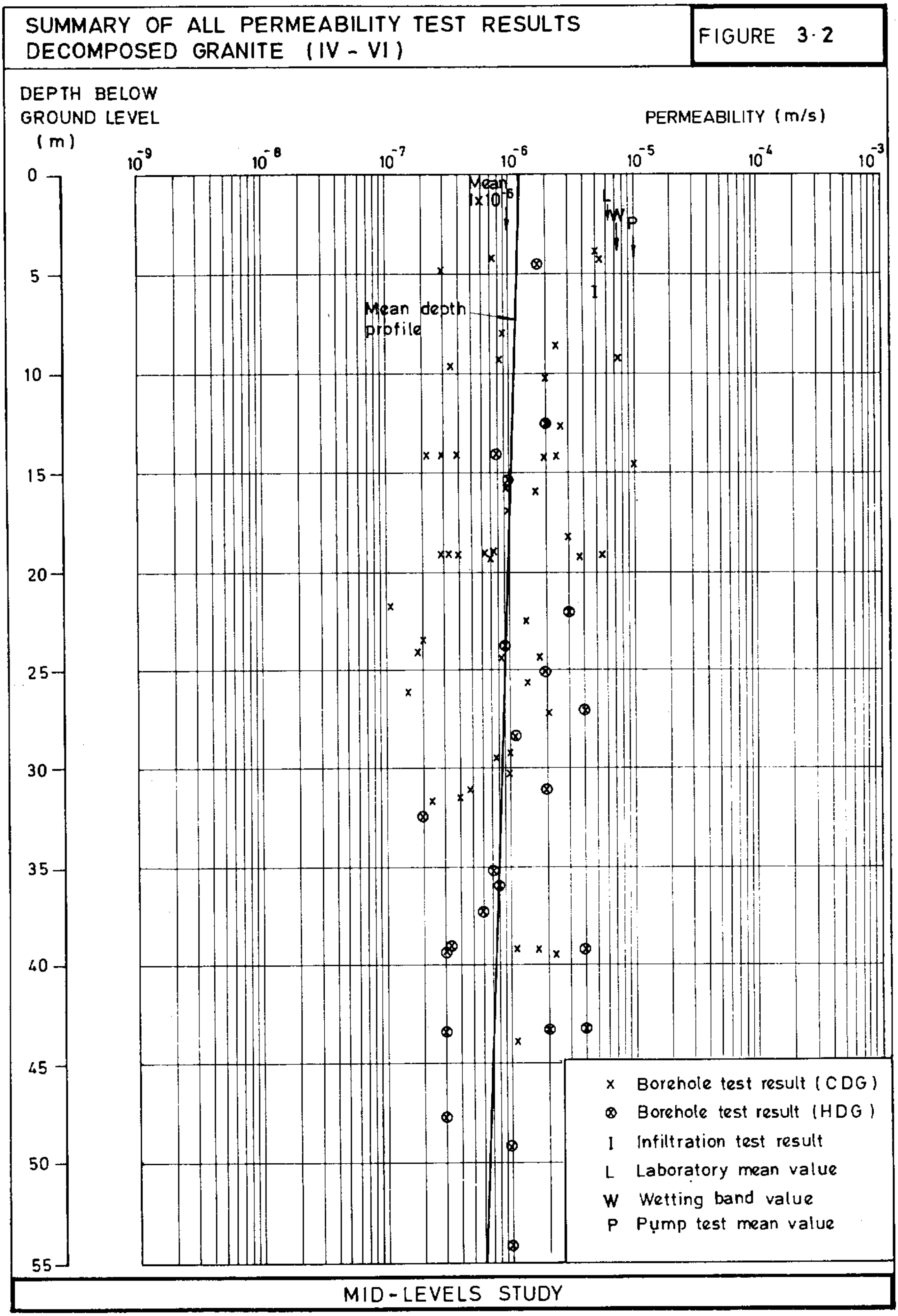
Groundwater discharges at many seepages throughout the area. These are natural springs in the upper slopes and from weepholes in retaining walls and chunam slopes in the developed area. Some originate from the main water table, and some from the perched. They have similar response characteristics to piezometers, with both persistent and storm types.

Flows from some seepages have been monitored as part of the surface water measurement programme. Full details are given in reference 36.

Samples of seepages have been analysed for chemical composition in order to determine the origins of groundwater. This survey is described in section 4.1.2.

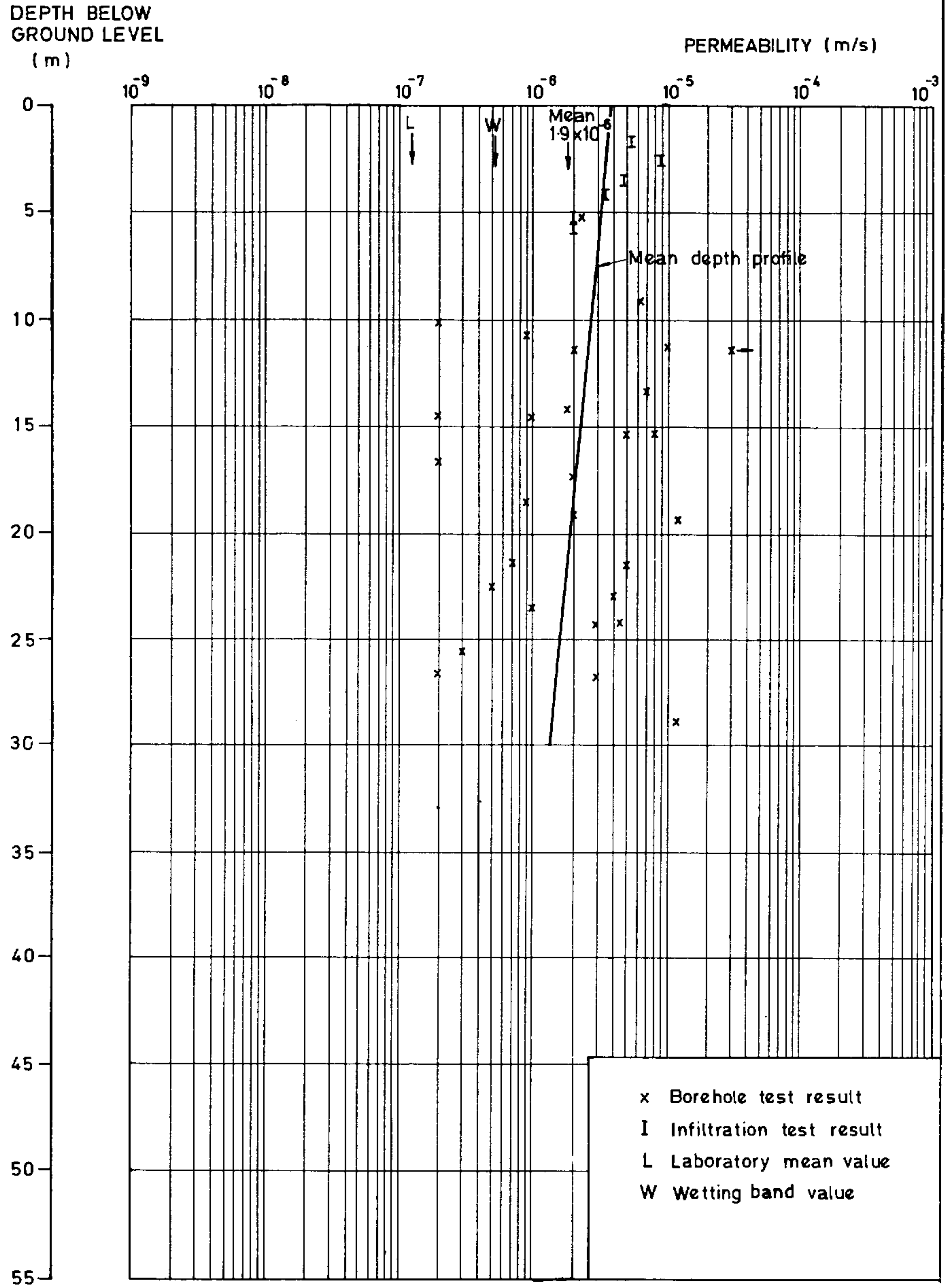
Drawing H6 summarises all of the available information about seepages; water table, source of water, type of flow and whether flows have been measured or chemical analyses done.


SUMMARY OF ALL PERMEABILITY TEST RESULTS
COLLUVIUM
FIGURE 3·1
DEPTH BELOW
GROUND LEVEL
(m)
PERMEABILITY (m/s)
10^{-9} 10^{-8} 10^{-7} 10^{-6} 10^{-5} 10^{-4} 10^{-3}
0 2 4 6 8 10 12 14 16 18 20 22
Mean $1{\cdot}5\times10^{-5}$
L
W
$5{\cdot}6\times10^{-3}$
1×10^{-2}
Mean depth profile
× Borehole test result
I Infiltration test result
L Laboratory mean value
W Wetting band value
MID-LEVELS STUDY


SUMMARY OF ALL PERMEABILITY TEST RESULTS
DECOMPOSED GRANITE (IV - VI)
FIGURE 3·2
DEPTH BELOW
GROUND LEVEL
(m)
PERMEABILITY (m/s)
Mean
1x10-6
Mean depth
profile
L
W
P
I
x Borehole test result (CDG)
⊗ Borehole test result (HDG)
I Infiltration test result
L Laboratory mean value
W Wetting band value
P Pump test mean value
MID-LEVELS STUDY

SUMMARY OF ALL PERMEABILITY TEST RESULTS DECOMPOSED VOLCANICS (IV - VI)

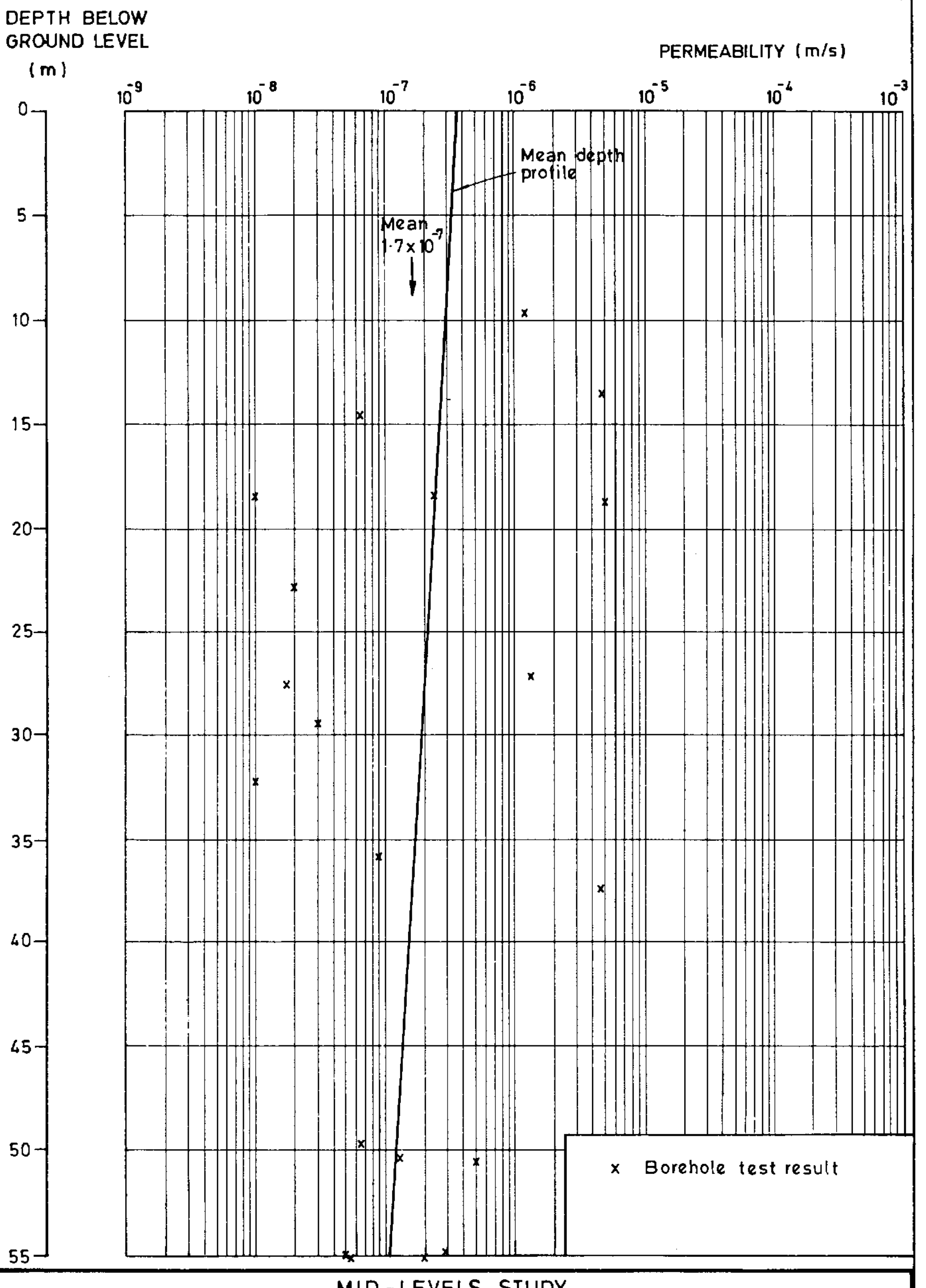
FIGURE 3·3



MID-LEVELS STUDY

SUMMARY OF ALL PERMEABILITY TEST RESULTS
GRANITE ROCK (I-III)

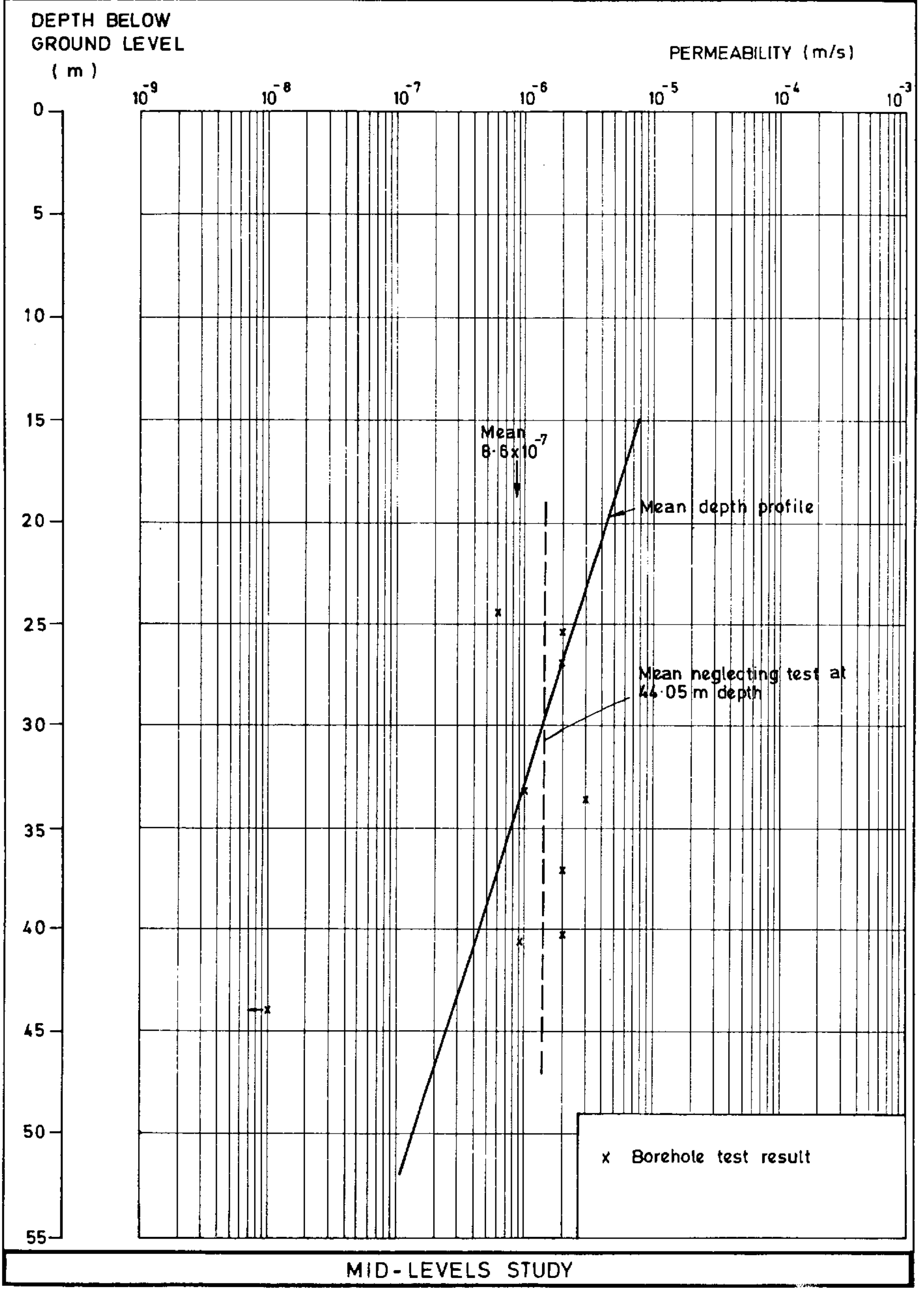
FIGURE 3·4



MID-LEVELS STUDY

SUMMARY OF ALL PERMEABILITY TEST RESULTS
VOLCANIC ROCK (I – III)

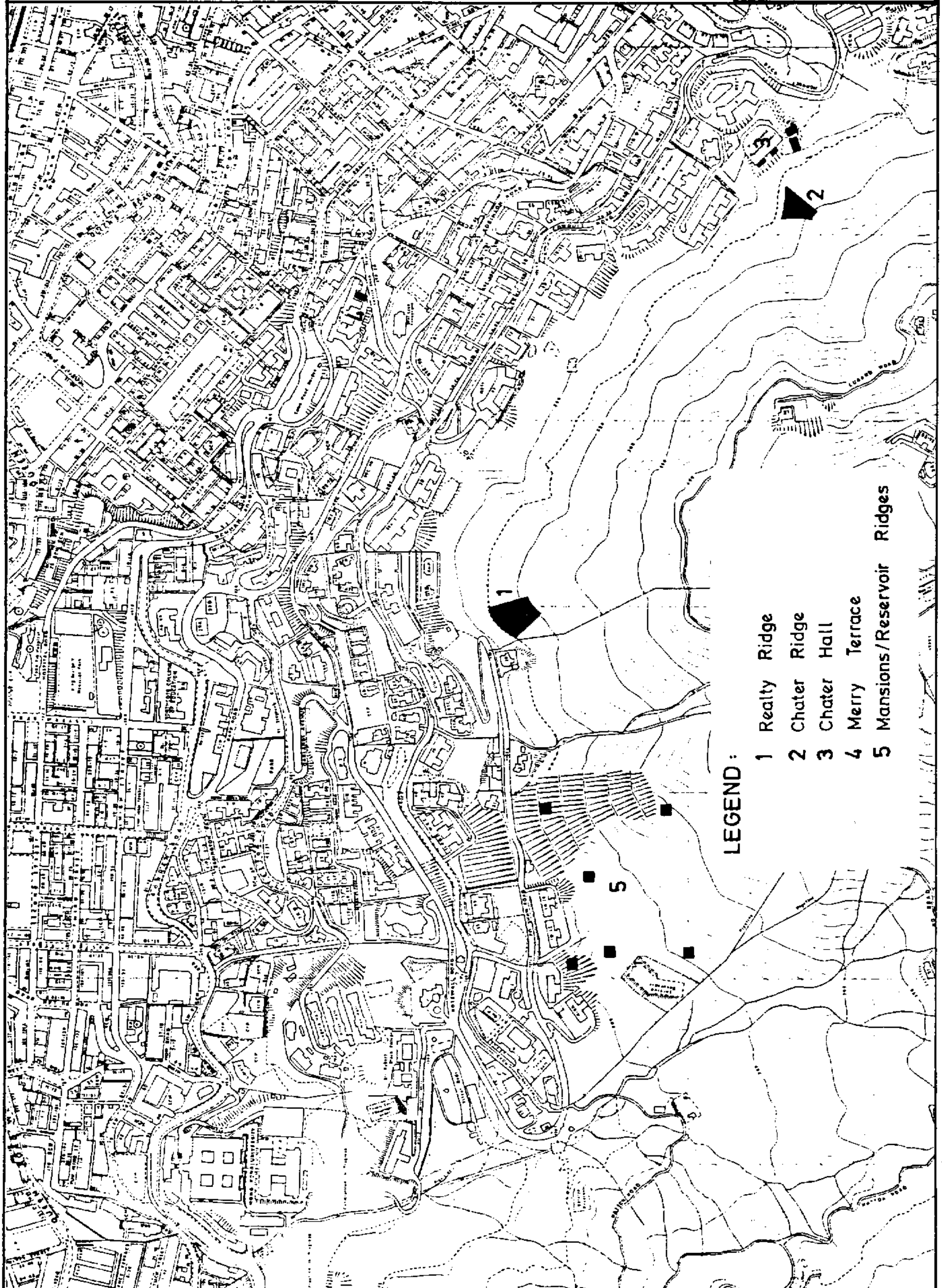
FIGURE 3·5



MID-LEVELS STUDY

SUCTION INSTRUMENTATION SITES. LOCATION PLAN

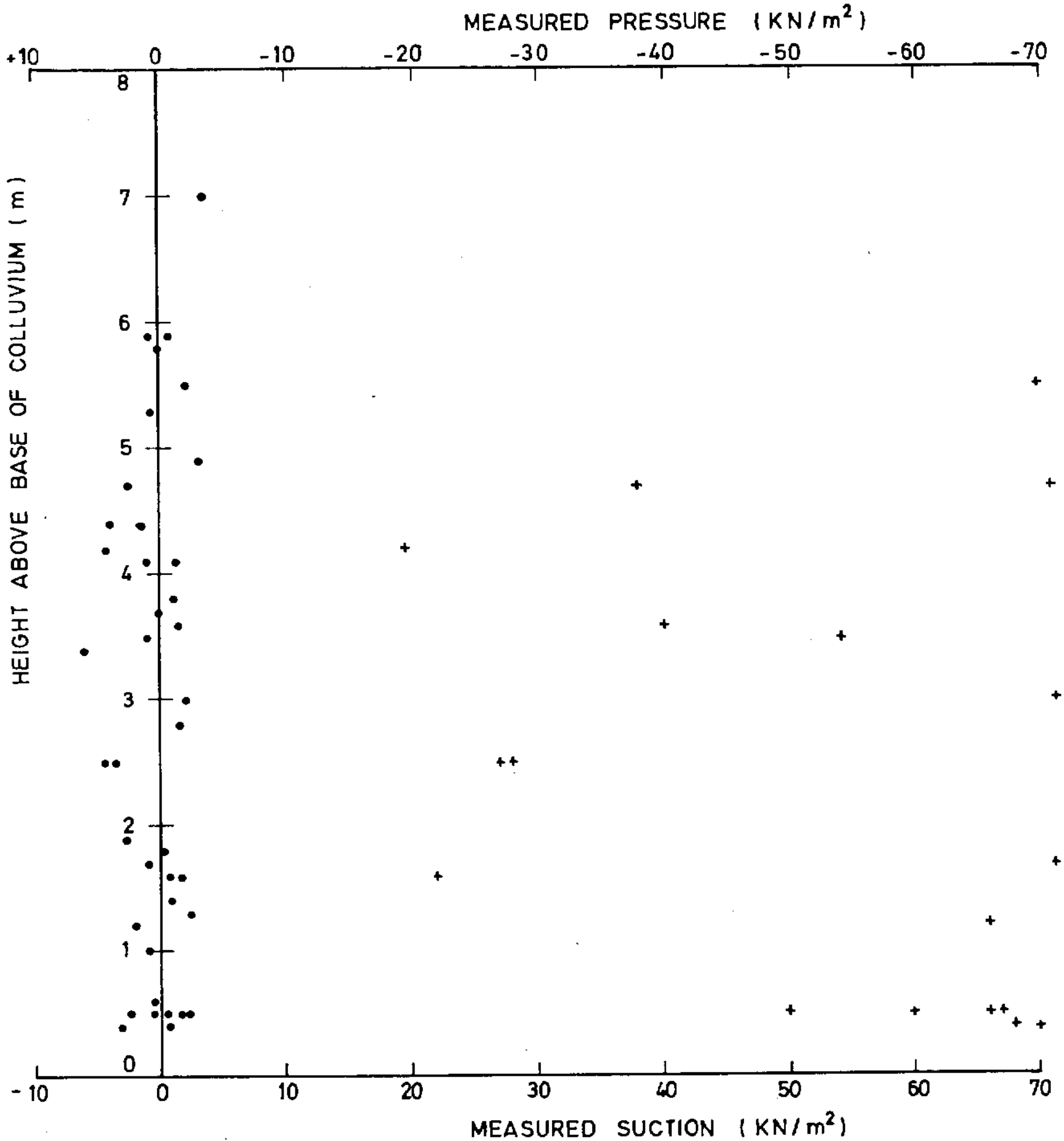
FIGURE 3.6



MID-LEVELS STUDY

MEASURED RANGE OF SUCTION IN COLLUVIUM AT REALTY RIDGE

FIGURE 3·7



• Minimum suction recorded on 1 · 9 · 80

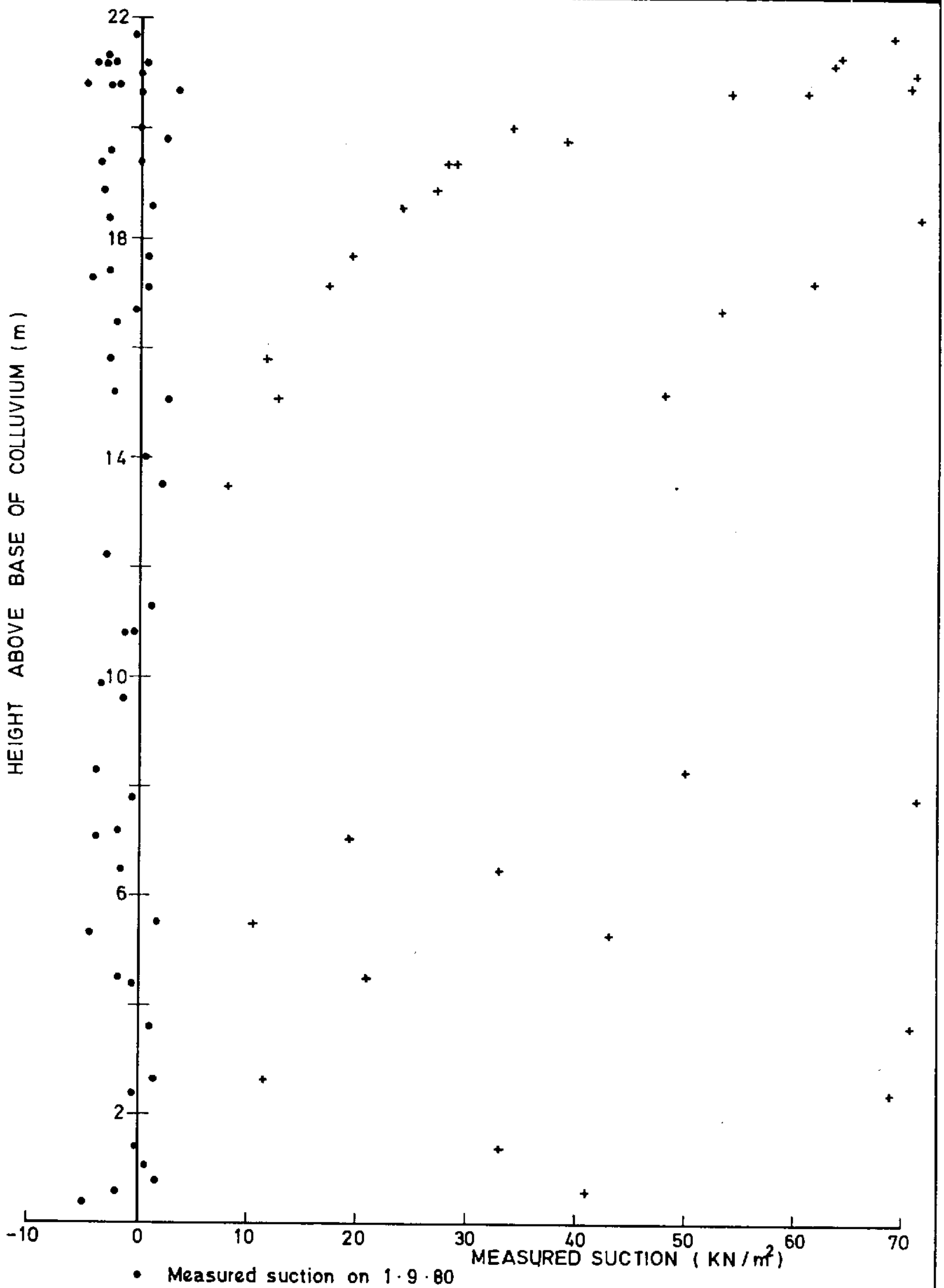
+ Maximum suction recorded on 30 · 1 · 81

19 tensiometers show cavitation on 30 · 1 · 81 (suction > 75KN/m²)

MID-LEVELS STUDY

MEASURED RANGE OF SUCTION IN COLLUVIUM AT CHATER RIDGE

FIGURE 3·8



• Measured suction on 1·9·80

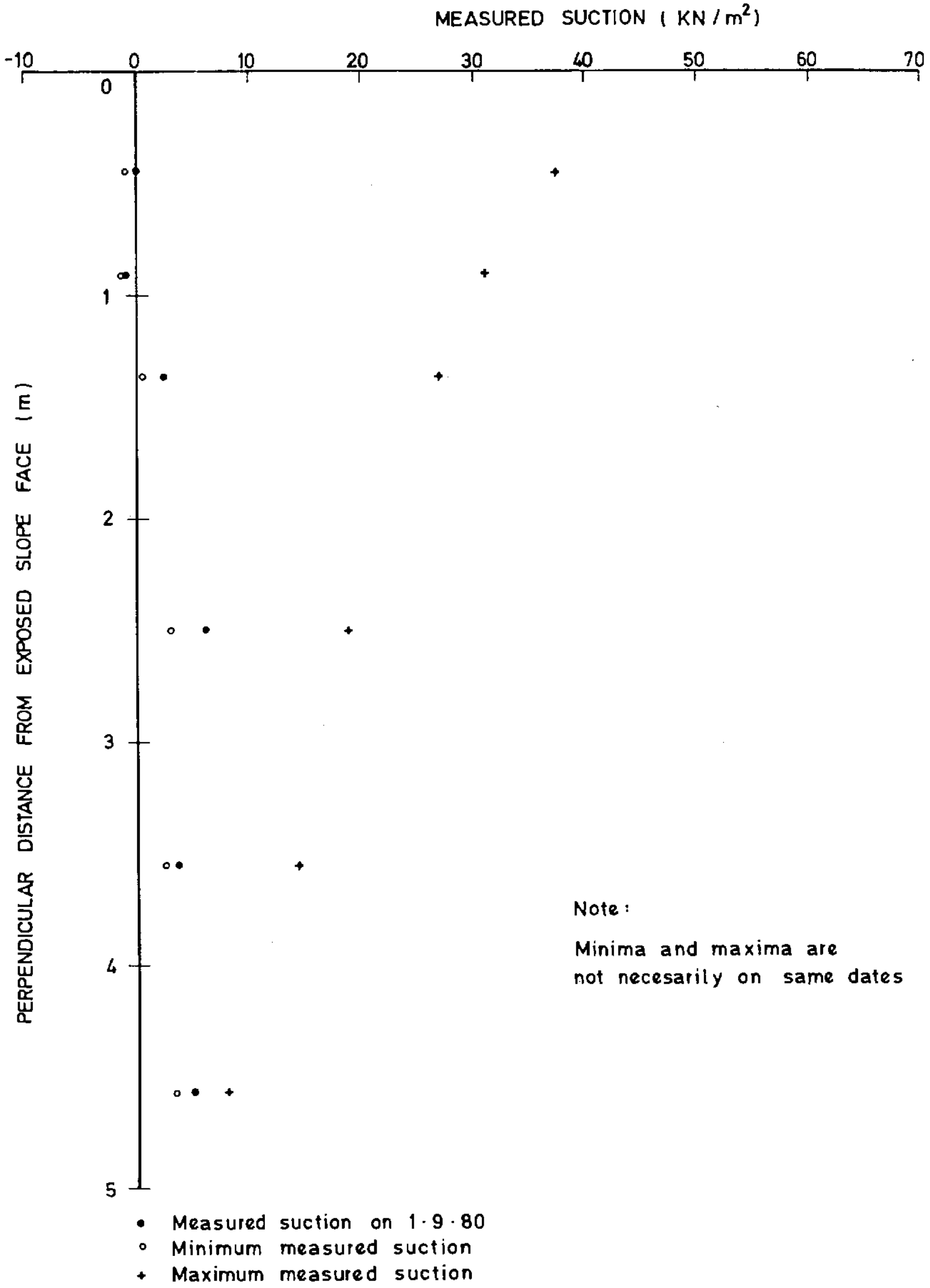
+ Measured suction on 30·1·81

11 Tensiometers show cavitation on 30·1·81 (suction > 75 KN/m²)

MID-LEVELS STUDY

MEASURED RANGE OF SUCTION IN EXPOSED DECOMPOSED GRANITE AT CHATER HALL

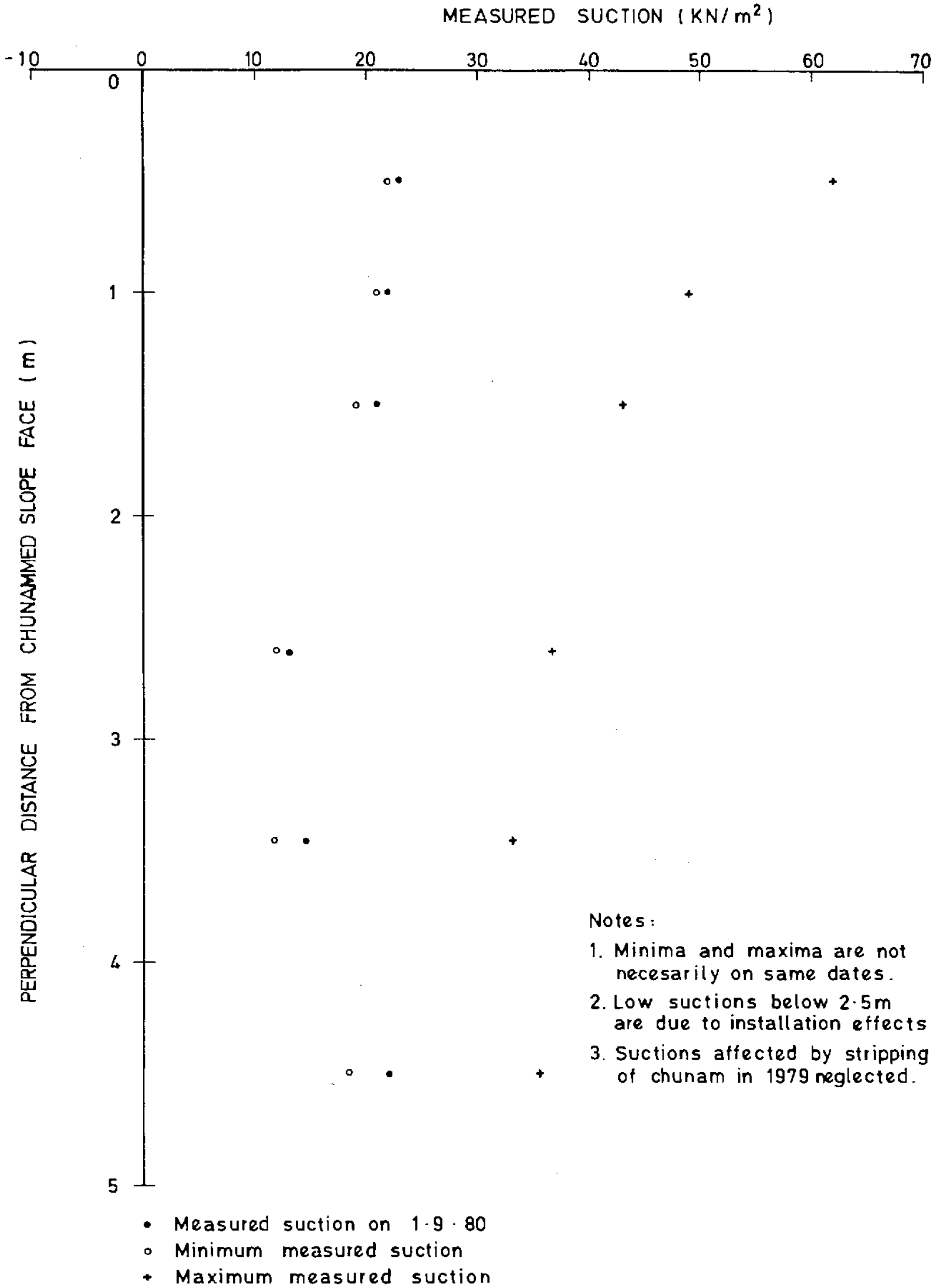
FIGURE 3·9



MID-LEVELS STUDY

MEASURED RANGE OF SUCTION IN CHUNAMMED DECOMPOSED GRANITE AT CHATER HALL

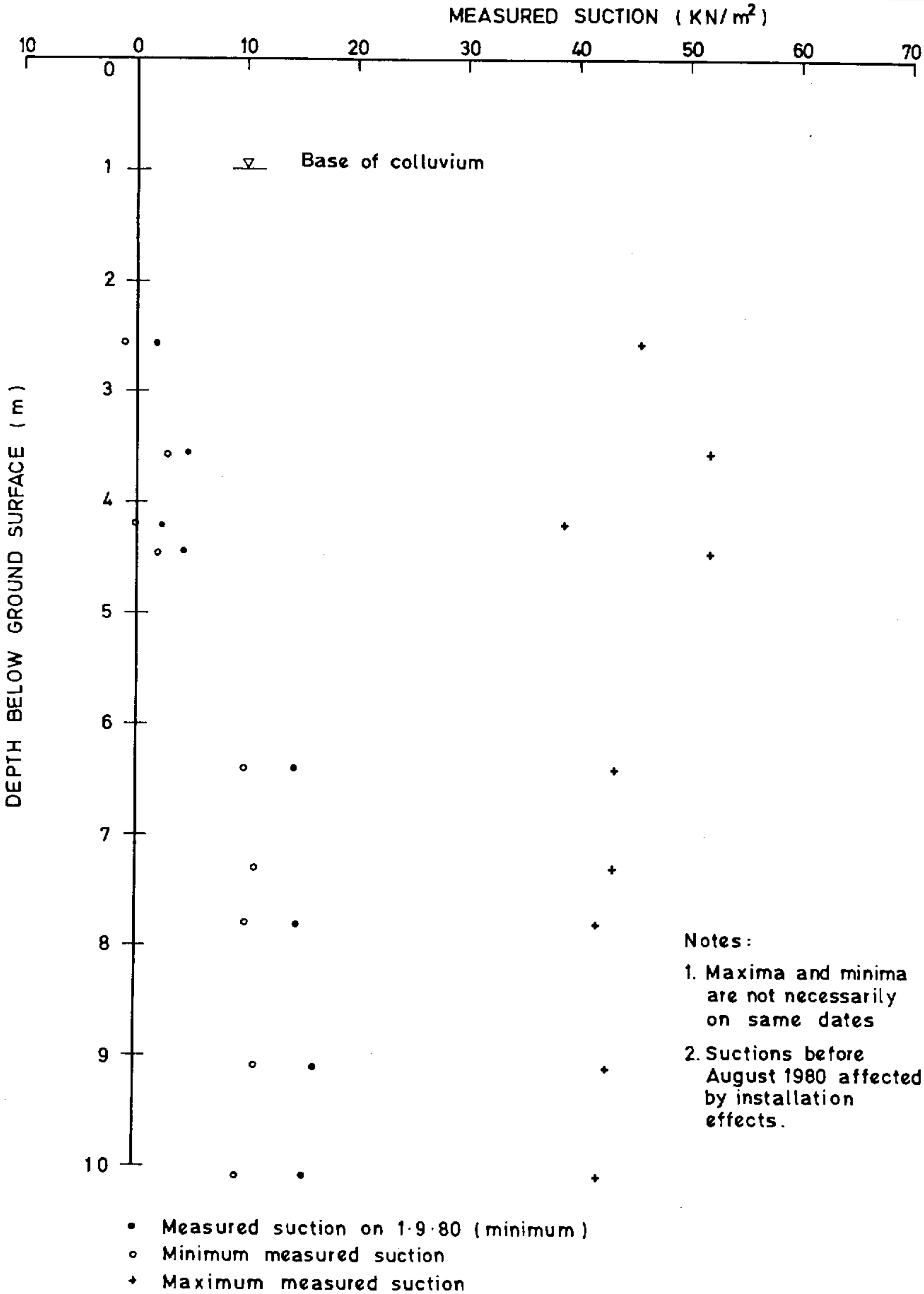
FIGURE 3·10



MID-LEVELS STUDY

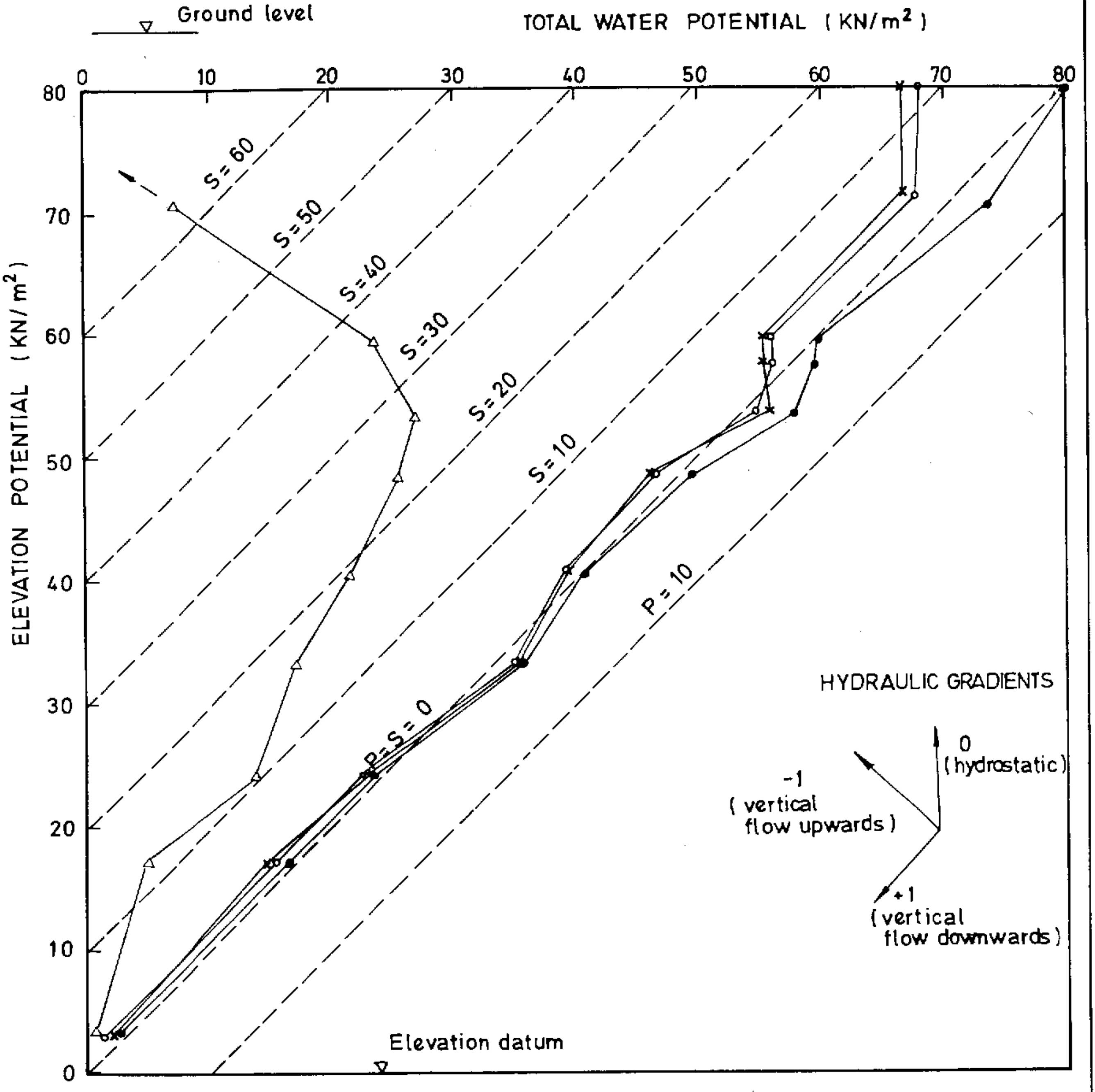
MEASURED RANGE OF SUCTION IN DECOMPOSED VOLCANICS AT REALTY RIDGE

FIGURE 3·11



MID-LEVELS STUDY

TOTAL WATER POTENTIALS IN COLLUVIUM AT CHATER RIDGE	FIGURE 3·12

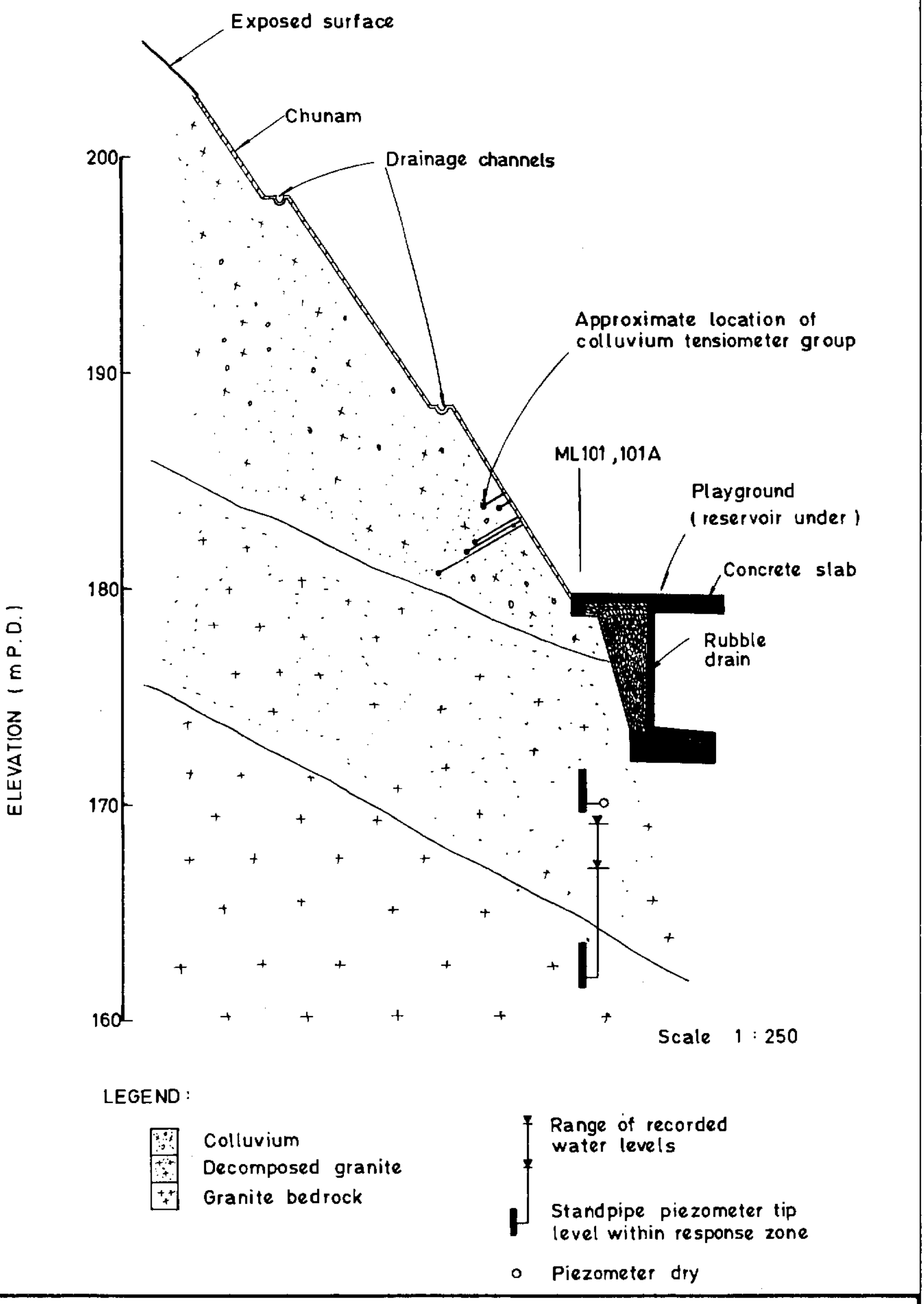


- ○ Total water potential on 30·8·80 (pre-storm)
- ● Total water potential on 1·9·80 (storm)
- x Total water potential on 10·9·80 (post-storm)
- △ Total water potential on 30·1·81 (dry season)
- S Suction (KN/m²)
- P Pressure (KN/m²)

MID-LEVELS STUDY

CROSS SECTION THROUGH CHUNAMMED SLOPE AT CHATER HALL

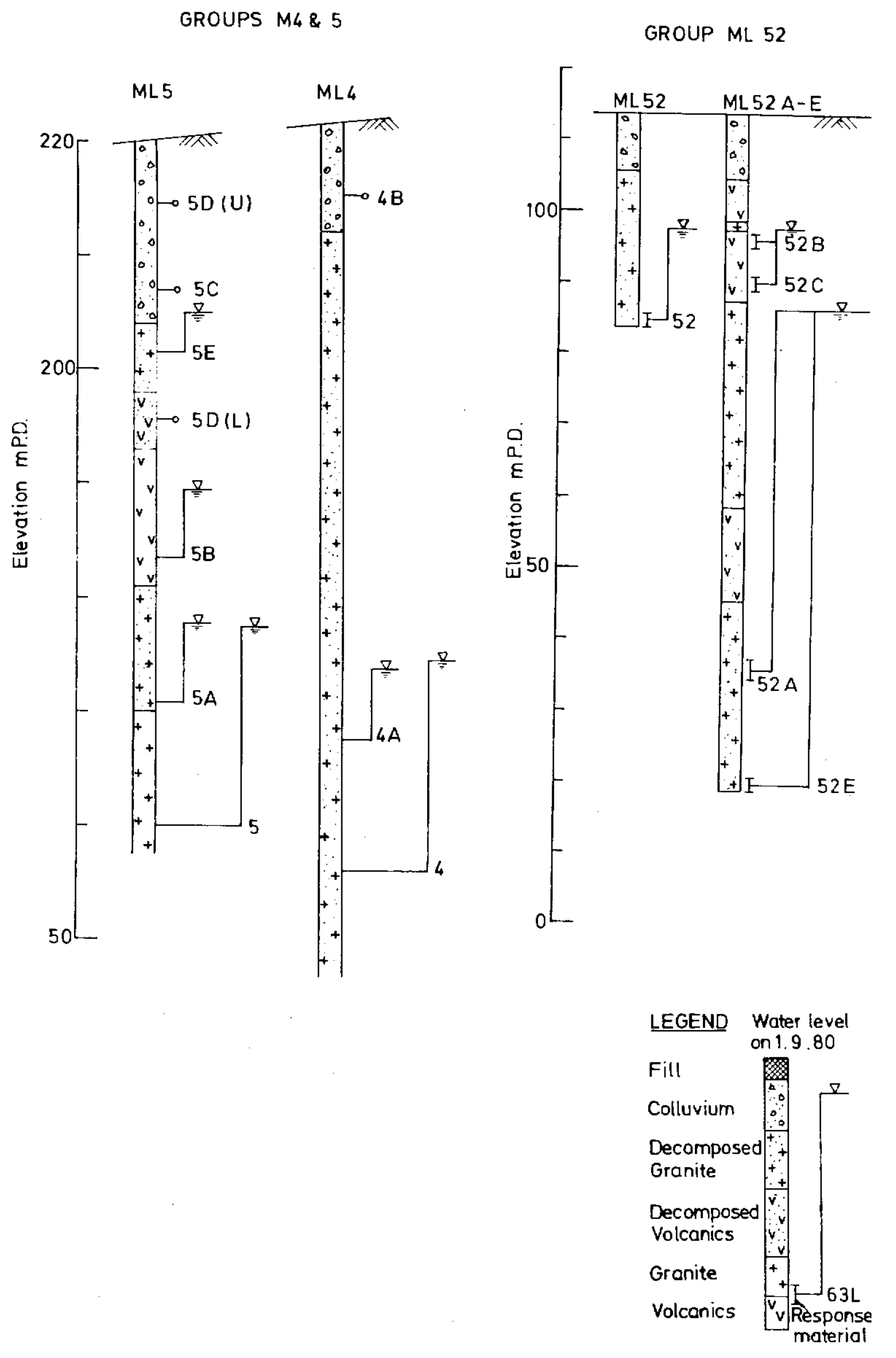
FIGURE 3·13



MID-LEVELS STUDY

MAIN WATER TABLE CONDITIONS AT GRANITE / VOLCANIC CONTACT

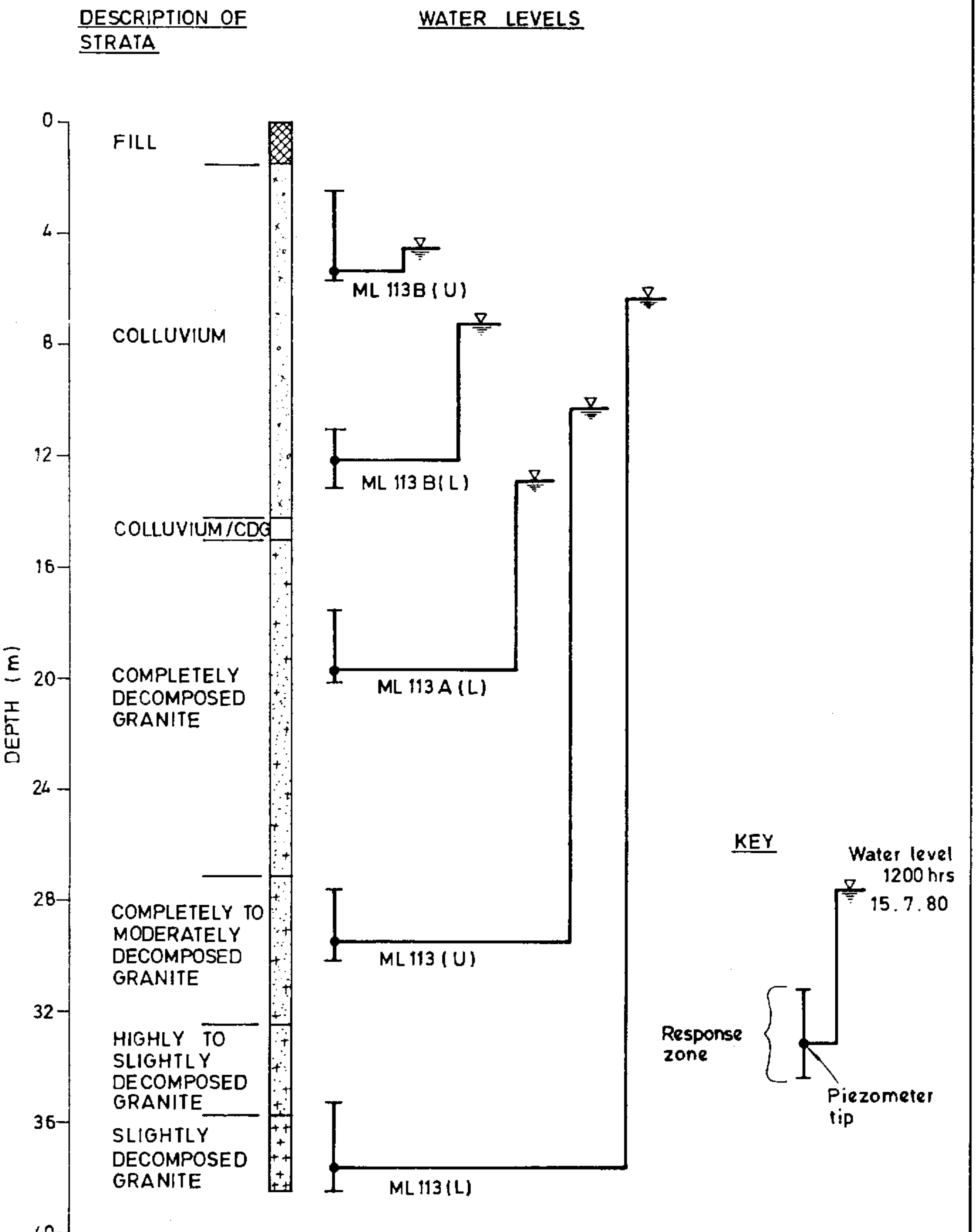
FIGURE 3.14



MID-LEVELS STUDY

PIEZOMETER GROUP ML 113 SHOWING UPWARD FLOW FROM BEDROCK

FIGURE 3·15

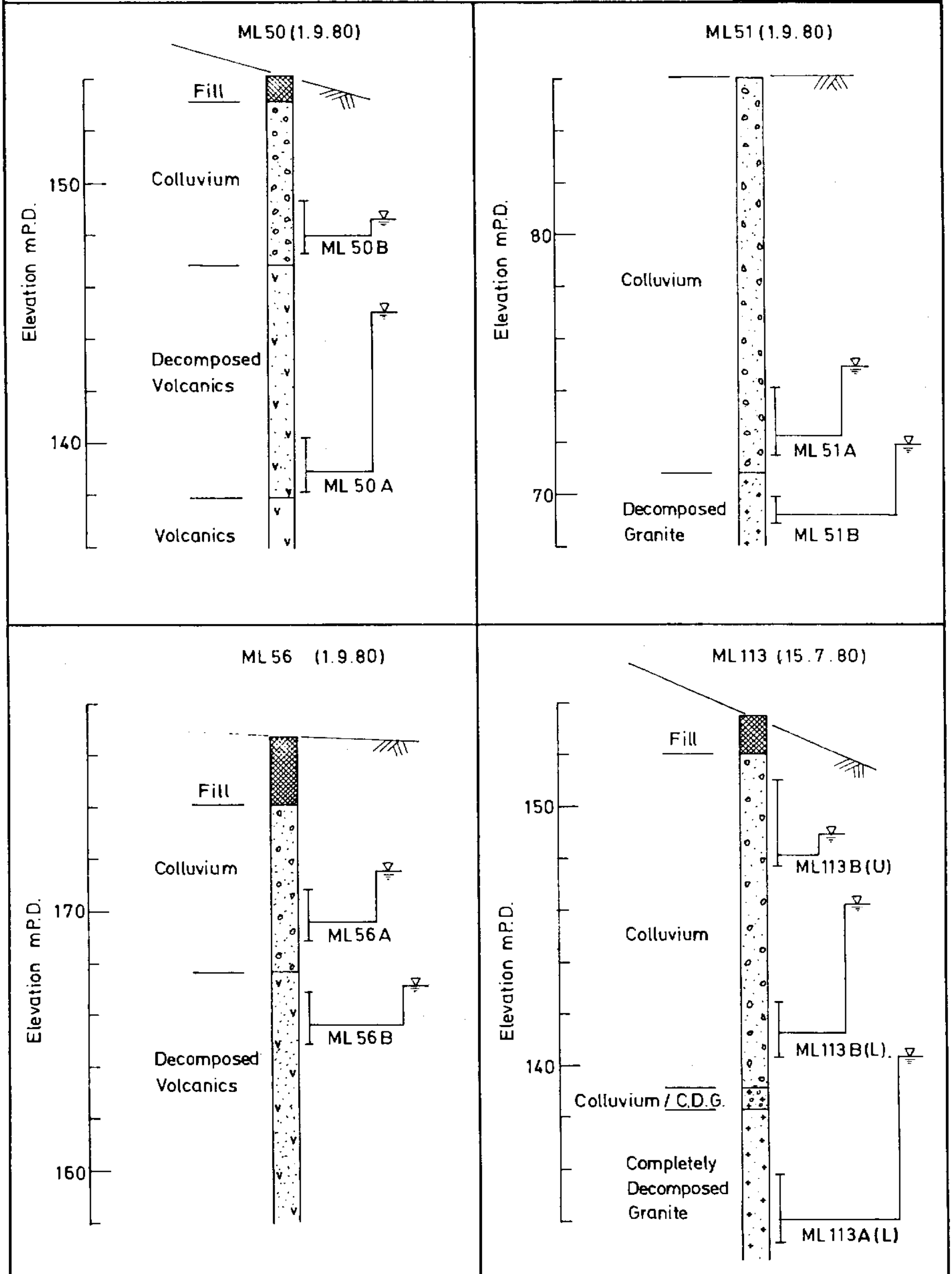


Note : Water levels not corrected for non-hydrostatic conditions

MID-LEVELS STUDY

PIEZOMETERS IMPLYING PERCHING AT BASE OF COLLUVIUM

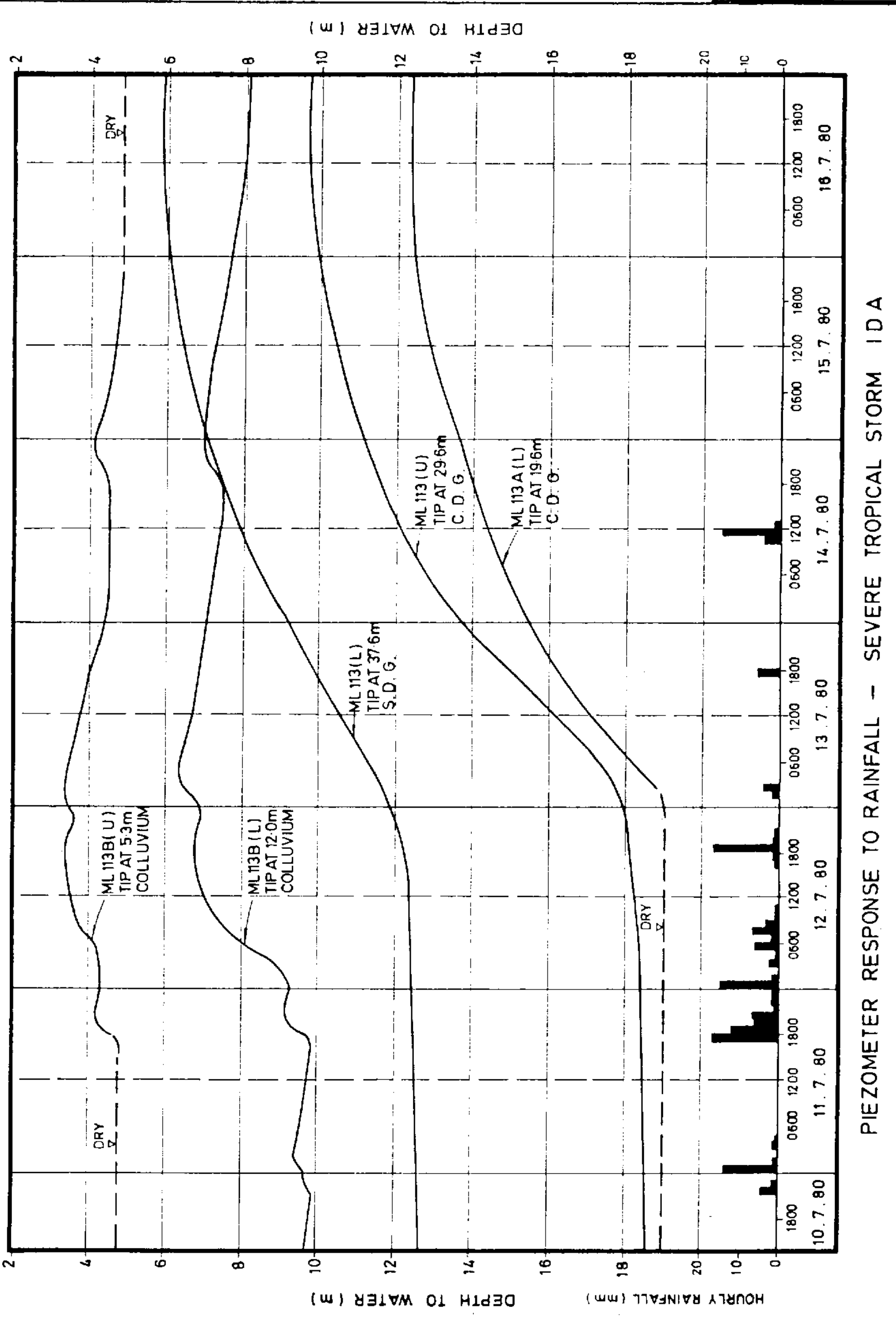
FIGURE 3.16



MID-LEVELS STUDY

PIEZOMETER GROUP ML 113 SHOWING STORM RESPONSE

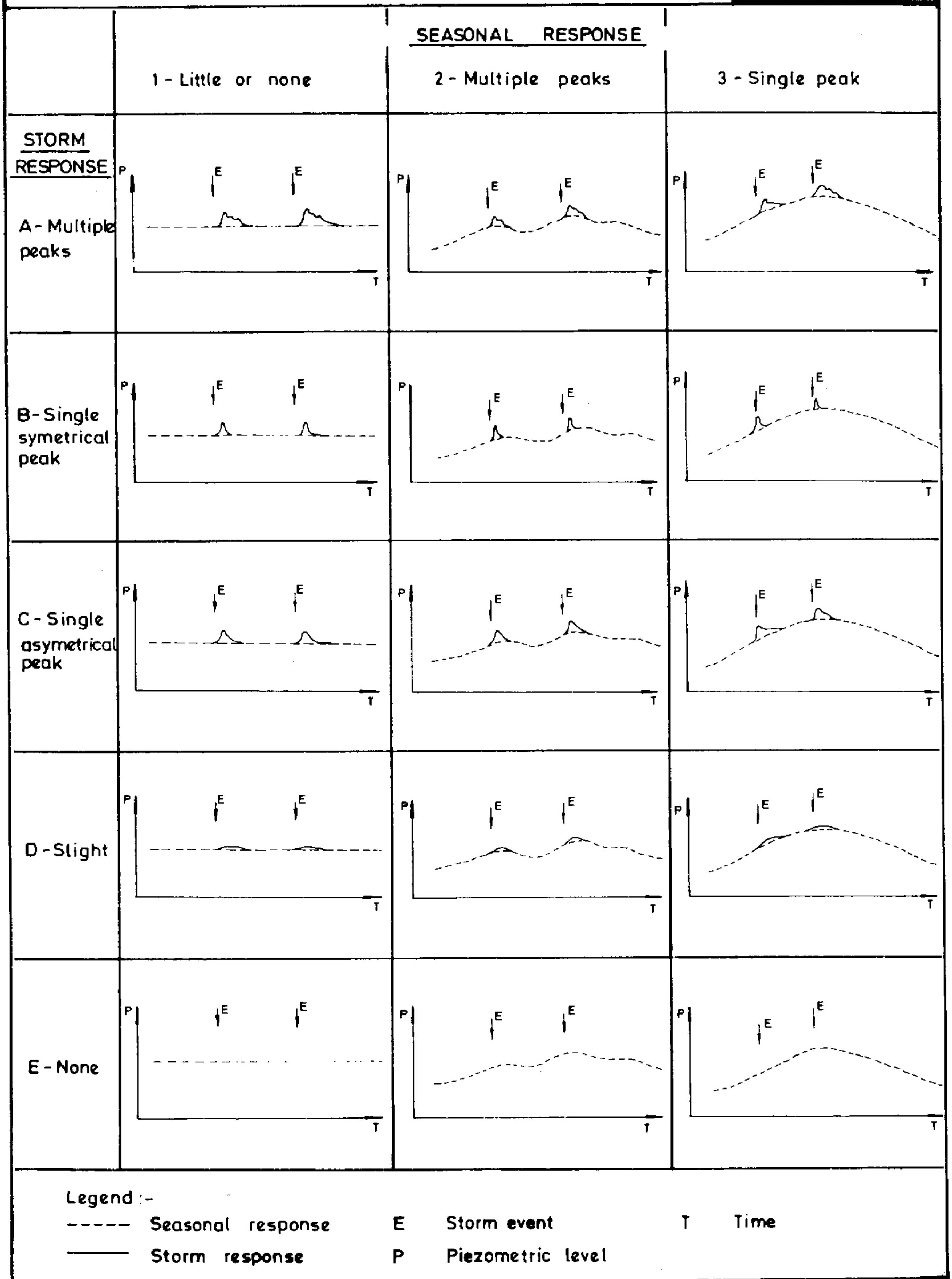
FIGURE 3·17



MID-LEVELS STUDY

PIEZOMETER RESPONSE CATEGORIES

FIGURE 3.18



MID - LEVELS STUDY

4. EFFECTS OF DEVELOPMENT

4.1 Leakage from Services

Some leakage from water bearing services is to be expected and has previously been recognised as a potential major cause of high groundwater levels in, for example, the Geotechnical Manual for Slopes (reference 44). It has been positively identified as such with three types of evidence; piezometry, including suction instrumentation, chemistry, and leakage tests. Its significance is twofold, causing perched water tables in areas that would not otherwise receive recharge, and raising main water table levels.

There are four types of water bearing service in the area:

a) Fresh water mains throughout the urban area.

b) Flushing water mains. These are almost as widespread as fresh water mains but are less dense in the upper and intermediate slopes. Water is pumped into this system from the harbour.

c) Sewers. Their extent and location are unknown, but probably corresponds closely with the fresh water system.

d) Storm drains, including streams and culverted streams. The extent of major drains is shown on Drawing H7.

4.1.1 Piezometric evidence

The piezometric evidence for leakage is presented in section 3, where the origins of perched water tables are discussed. Permanent perched water tables and rises in the main water table have been recorded.

4.1.2 Groundwater chemistry

A chemical survey was made of groundwater from seepages, piezometers and one caisson. It was intended to help decide the origins of groundwaters, particularly in perched water tables.

The survey provides clear evidence of substantial contributions to groundwater from leaking services, but firm conclusions on the proportions of groundwater, fresh water, flushing water and sewage cannot be reached from the analyses carried out. Analyses for certain trace elements would probably give more conclusive results. It was expected that the flushing water would be highly saline, but samples from three domestic premises were found to be only slightly brackish. (Only one, sample M, was fully analysed.)

Two summaries of the analyses are presented in Figures 4.1 and 4.2. The first figure classifies the sample locations by conductivity, which is an indicator of sewage and flushing water.

Conductivity increases downslope, indicating that sewage, and possibly saline flushing water, proportions increase.

The second figure groups the sampling locations according to fluoride concentration. Mains water is dosed with 0.6 mg/l fluorides while the standard fluoride level in sea water is 1.4 mg/l. The low fluoride samples are generally in the upper slopes, mostly above the mains network. High fluoride levels have been measured in samples taken from above Conduit Road in the vicinity of the Seymour Boulder Field. Immediately above this area granite sheeting joints are exposed close to the granite/volcanic contact. It is possible that the high fluoride concentrations may be derived from the granite.

The predominance of samples throughout the study area with fluoride concentrations close to the dosing level for mains water suggests that a significant amount of recharge is coming from services.

4.1.3 Leakage tests

There have been no direct measurements of leakages from the fresh or flushing mains, or from sewers, except for tests on a few service reservoirs. However the Water Supplies Department carries out regular night flow tests on the fresh water system and soundings and visual inspections of the flushing system.

Minimum night flow (MNF) tests are a way of detecting changes in leakage. Valving is adjusted so that all supplies to an area are through one point, where flow is measured. The minimum flow during the test period (usually 24:00 to 05:00 in Hong Kong) is measured. Any increases between tests implies an increase in leakage. However there are always legitimate nocturnal demands and some leakage takes place inside premises and does not recharge the aquifer. MNFs reflect, but do not measure, leakage to the aquifer.

About half of the MNF to large areas (100,000 people or more) which are well maintained could be leakage to the ground. No estimates of the proportion of MNF that becomes recharge are available for Hong Kong.

4.1.4 Leakage estimates from groundwater modelling studies

The inclusion of recharge, other than by direct infiltration from rainfall, was necessary in the groundwater models in order to reproduce observed behaviour during the dry season. The final calibration of the Glenealy/Seymour model required an average additional recharge of 2,300 mm/year to maintain observed groundwater levels. It is probable that less than 1,100 mm/year of this is attributable to recharge from bedrock, suggesting that the remainder may be attributable to leakage.

In the Po Shan/University model the additional recharge was made compatible with the possible recharge from rainfall to bedrock upslope, and was less than implied by aquifer properties and groundwater levels. This suggests that there may be additional recharge from leaking services.

4.1.5 Conclusions

Leakage from services does occur and this leads to increased ground water levels and reduced stability. In some areas the reduction in stability may be significant.

4.2 Cuts and Earth Retaining Structures

The study did not include investigation of the effects of individual cuts and earth retaining structures. Provided that effective drainage is incorporated, these features should lower groundwater levels locally. An example is given in Figure 4.3 where a single level basement may have increased groundwater levels. The piezometric responses could indicate that the basement is having a temporary damming effect causing rapid rises in pore water pressures.

4.3 Surface Protection

Chunam or sprayed concrete surface protections which are maintained in good condition will significantly reduce the amount of rainfall infiltrating directly into a slope. However, it has been shown that infiltration above a protected slope affects suctions under the protection. Thus chunam and other protections only reduce the size of suction changes after rainfall when they extend above the slope. Protective layers on the surface do not prevent recharge from leaking services and drains.

Chunam and sprayed concrete surface protections reduce the rate of evaporation from the slope surface and suctions increase slowly in dry spells. If water can infiltrate into a covered slope the suctions near the surface will generally be lower when compared with a similar slope without surface protection. This adverse effect of protection can also be seen when comparing the suction behaviour in a vegetated slope with a slope poorly covered by chunam which does not prevent indirect infiltration. After heavy rainfall early in the wet season the suction will be reduced in both slopes. Suction near the surface of the vegetated slope will generally increase quickly, due to evapotranspiration, before the start of the next rainfall event. However, in the chunam protected slope the suction may remain low. After the next rainfall event suctions may drop to lower levels in the protected slope due to the initial 'wetter' conditions of the soil. This effect would occur progressively during the wet season resulting in significantly lower residual suctions in the near surface part of the protected slope.

4.4 Drainage

4.4.1 Surface drainage

The effects of storm drains on groundwater level are discussed above (4.1). Locations of the major storm drains are shown on Drawing H7. In summary, they are a potentially major source of recharge during storms and have been identified as causing large piezometric responses within the study area. This problem is likely to be more severe in major storms when drains occasionally become blocked or surcharged.

4.4.2 Groundwater drainage

The major groundwater drainage works known in the study area are shown on Drawing H7. They consist of:

a) Po Shan remedial works; horizontal drainage wells into decomposed volcanics and bedrock and a drainage blanket under parts of the reformed slope.

b) Horizontal drainage wells into decomposed volcanics behind 10-16 Po Shan Road.

c) A tunnel in decomposed granite at Hong Kong Gardens.

d) Sai Ying Pun drainage tunnels in decomposed granite in the lower slopes.

Flow measurements have been made of both sets of tunnels and of some of the horizontal drains (reference 36).

Some preliminary conclusions can be drawn:

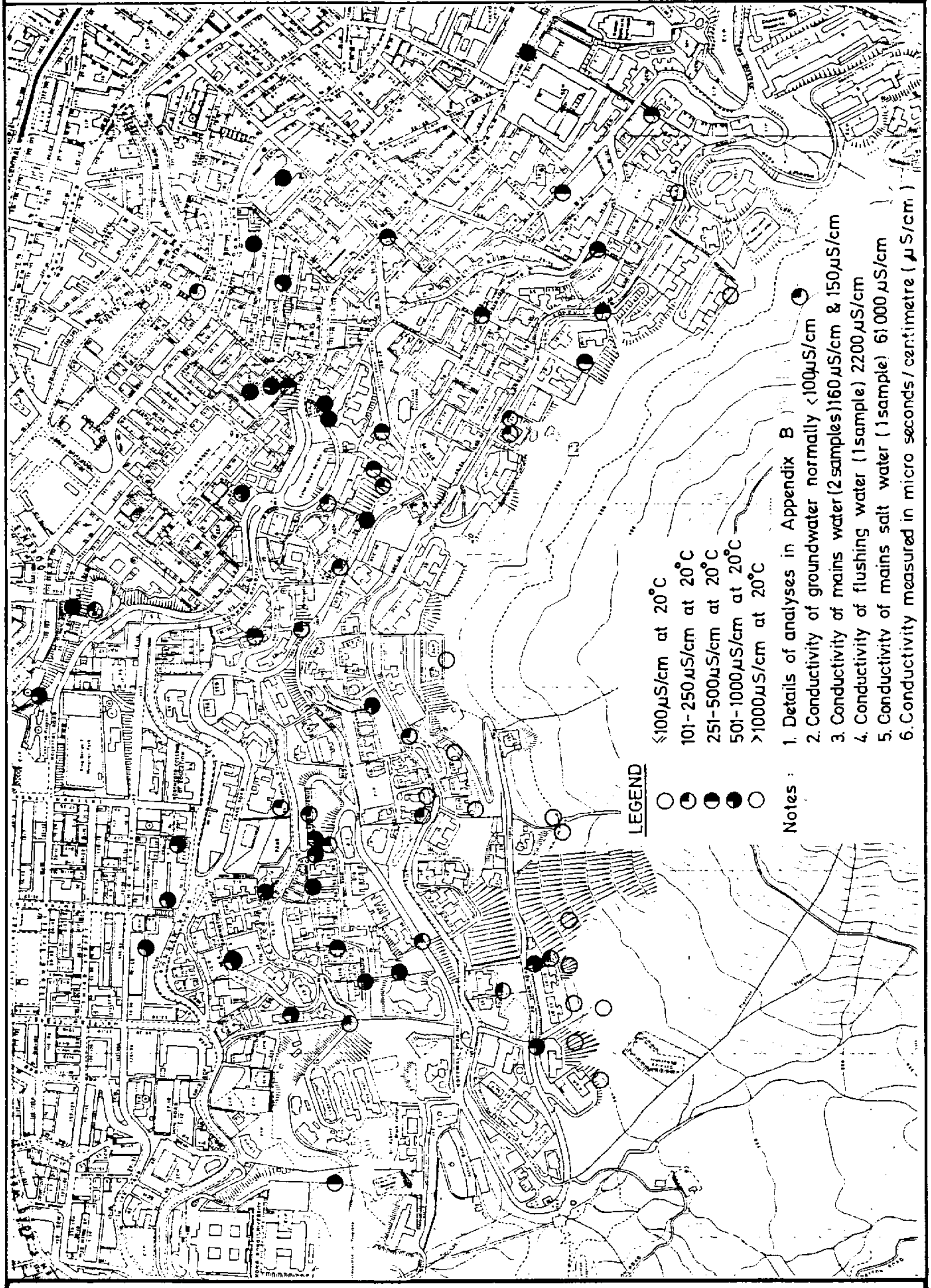
a) Seasonal and storm responses in the main water table are reduced by the horizontal drains in Po Shan remedial works and behind the buildings on Po Shan Road when compared with adjacent areas. The effect is greater for the longer drains in the remedial works which penetrate to bedrock.

b) There is insufficient evidence to draw any conclusions about the effects of horizontal drains on colluvium responses.

c) The effect of the drainage tunnels is to lower the main water table in the vicinity permanently.

GROUNDWATER CHEMISTRY : CONDUCTIVITY

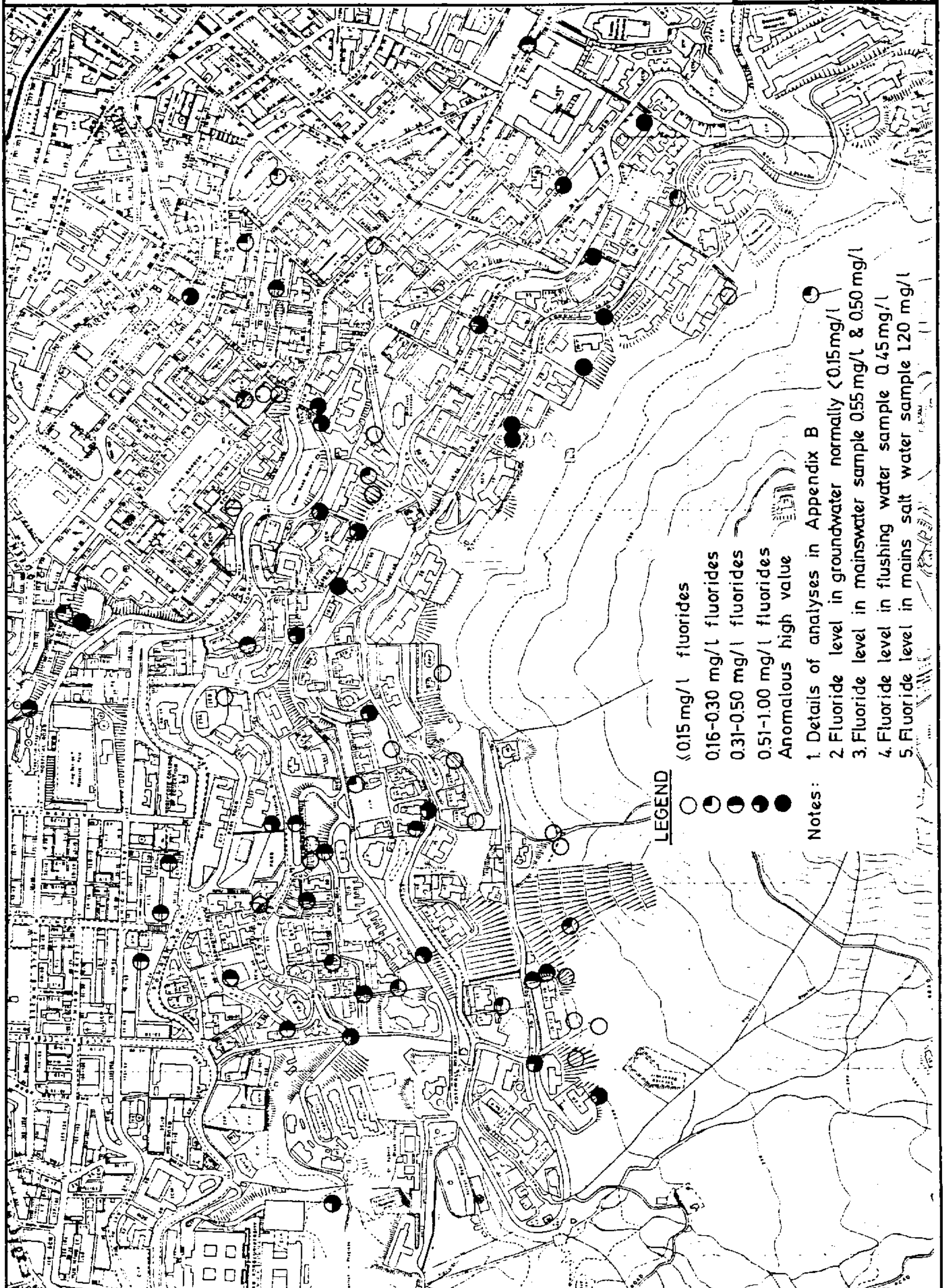
FIGURE 4.1



MID-LEVELS STUDY

GROUNDWATER CHEMISTRY : FLUORIDE CONCENTRATIONS

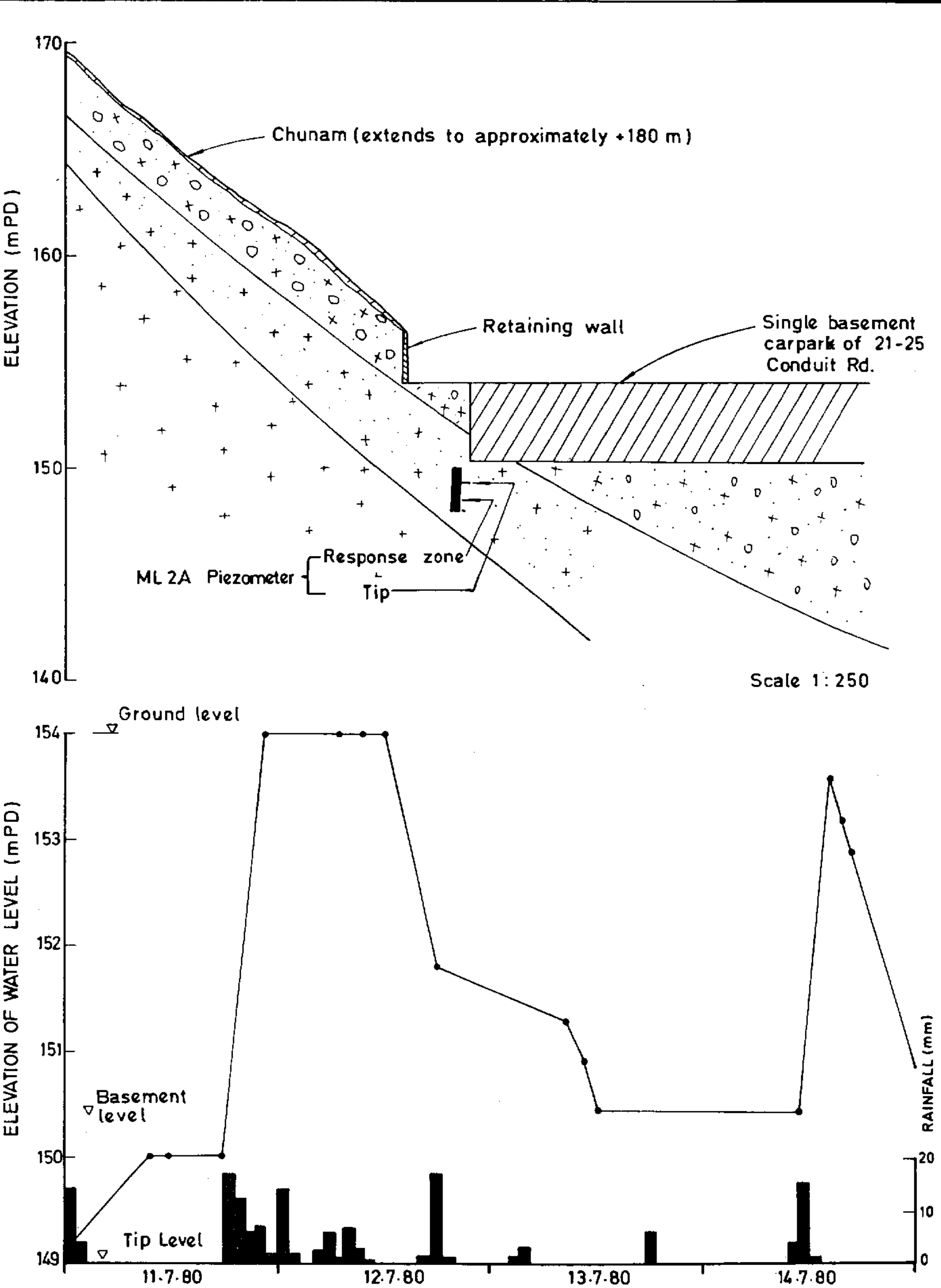
FIGURE 4.2



MID - LEVELS STUDY

EFFECT OF BASEMENT ON PORE WATER PRESSURES

FIGURE 4.3



MID-LEVELS STUDY

5. PREDICTION OF PIEZOMETRIC LEVELS

5.1 Introduction

5.1.1 Methods of analysis

Three approaches have been made to the analysis of piezometric levels:

a) a statistical correlation of groundwater response with rainfall

b) groundwater modelling of vertical two dimensional sections

c) extrapolation of observed piezometer responses

Each of these studies is reviewed below (sections 5.2 to 5.4).

5.1.2 Levels required for stability analysis

There are few detailed piezometer records in Mid-levels that start before July 1979, and many start in 1980. Extrapolation from one or two wet seasons' data to the 1 in 10 year events is possible, although it contains uncertainties, and 1 in 10 year values and levels that could result from leakage have been estimated.

Return periods cannot be sensibly evaluated for piezometric levels that are dominated by the effects of urbanisation, i.e. in areas affected by leaking services. Fluctuations caused by changes in the amount of leakage are often larger than the natural fluctuations. In these areas, two estimates were made of perched water table levels; 1 in 10 year levels ignoring the effects of services, and reasonably possible levels that could be caused by leaking services. Fluctuations in the main water table due to leaking services seem to be less important and less separable than those in the perched water tables and only 1 in 10 year levels were estimated.

Levels of a particular return period do not necessarily result from a rainstorm of the same return period. Many factors as well as total rainfall affect the piezometric level, including:

a) storm duration
b) temporal distribution of rainfall during the storm
c) antecedent ground conditions
d) additive effects of previous storms
e) timing of the storm relative to any seasonal fluctuations

The best way to estimate levels of a particular return period is by probability analysis of a long sequence of levels. These can be observed data or generated from a long rainfall record. The transformation of rainfall data to levels can be accomplished by statistical models, such as those discussed in section 5.2, or the groundwater models discussed in section 5.3.

Neither of the modelling approaches can be applied to the whole study area. Final 1 in 10 year levels were mainly based on extrapolations of observed piezometer responses and assume a correspondence between the return periods of rainstorms and piezometric levels.

5.2 Statistical Correlation of Groundwater Response

An attempt has been made to relate groundwater levels and rainfall through a statistical approach. A full description of the work can be found in reference 35.

The results for selected piezometers are good in view of the simple models used and the extremely short record available for calibrations. There were only two major storm events in 1979-80, both near the beginning of the period and these dominate the fitting process. The satisfactory models confirm that one and two variable equations can be used to model the observed piezometer responses. Other forms of model and alternative variables could be tested and would no doubt lead to reasonable results for other piezometers. However only 12 piezometers have adequate results at present. These alone cannot be used to develop estimates of piezometric levels throughout the area, but they provide a check on estimates by other means.

5.3 Groundwater Modelling

One section through the Glenealy/Seymour area and one through the Po Shan/University area have been modelled mathematically (reference 57). The locations of the sections are shown in Figure 5.1.

The model studies confirm the following:

a) groundwater response is greatest near the foot of the undeveloped slopes in the vicinity of Conduit Road and Robinson Road

b) groundwater response in the lower slopes is relatively small

c) there is considerable leakage from services within the developed area

d) the predicted 1 in 10 year levels are not substantially different from the highest water levels during recent extreme rainfall years.

5.4 Extrapolation of Observed Responses

The following sections discuss the extrapolation of data for individual piezometers to obtain estimates of 1 in 10 year levels. The results showed patterns that fitted our understanding of the hydrogeology, and thus permitted interpolation in areas without piezometers.

Piezometers showing a mainly storm response were treated differently from those with a mainly seasonal pattern. The methods have the same underlying principle: a 1 in 10 year rise is added to a typical level which occurs before the 1 in 10 year event, be it a season or a storm. In both cases, 1979 data have been used to derive the typical levels although it was a wetter than average year. The typical level is the less significant component so variations in it will have little effect on the final 1 in 10 year levels.

A distinction has also been made between the storm analyses for the main and perched water tables. The principle of adding a 1 in 10 year rise to a typical base level has been retained for perched water tables, but account is taken of their base on the surface of in situ material. Other methods have been necessary for areas of the perched water tables where there is insufficient piezometric data. These areas are the uppermost slopes and parts of the lower, developed slopes.

5.4.1 Seasonal responses

The 1 in 10 year levels of seasonally responding piezometers were estimated by adding a 1 in 10 year seasonal rise to a typical beginning of wet season level. The seasonal low in the first half of 1980 was taken for this typical level. It is due to the rainfall of 1979, an above average year. The 1980 wet season started late, allowing piezometer levels to decline further, and this level is probably representative of typical levels at the beginning of the wet season. Long (4 year) records are only available for five piezometers. Comparison of average seasonal lows and 1980 seasonal lows is inconclusive, but does not conflict with the approach adopted. The variability of seasonal highs is greater than that of lows for all of these piezometers, indicating that seasonal rises are more important than initial levels.

1 in 10 levels were estimated for all seasonally responding piezometers in the University and Po Shan areas and the results compared to observed maxima in 1979, where such data exist. 65% were within 1 m and 40% within 0.5 m. 1979 peaks were higher in about half the cases. This suggests 1979 peak values can be used as 1 in 10 levels for such piezometers, within the level of accuracy that can be achieved with the limited data available. 1979 peak seasonal levels were used in the Glenealy, Seymour and Central areas.

5.4.2 Storm responses in the main water table

Predictions of 1 in 10 year levels for piezometers with storm responses were made in a similar way; the response to a 1 in 10 year event was added to a typical 'pre-event' level. The average piezometric level during the 1979 wet season, after discounting responses to storms, was used. This is illustrated in Figure 5.5A. As with the seasonal responses, there is greater variability in peaks than lows. Using an initial level which is due to a wet year has only a slight effect on predicted 1 in 10 year levels.

In many cases 1979 data were not available to estimate the initial level and it was deduced from 1980 data. Initial levels in 1979 and 1980 were compared for 19 piezometers in the University and Po Shan areas. This is illustrated in Figure 5.2. When expressed as saturated depths of aquifer, the ratio of 1979 to 1980 varied between 0.98 and 1.45, averaging about 1.2. This is less than the ratio of rainfalls over the two wet seasons (1.7), and may suggest that flow does not take place equally throughout the depth of the aquifer. For the Glenealy, Seymour and Central areas seasonal responses for all piezometers with both 1979 and 1980 data were compared, Figures 5.3 and 5.4. An average seasonal response ratio of 1.9 and 1.5 were selected for decomposed rock and bedrock respectively.

It has been observed that different piezometers are sensitive to different lengths of storm event. This is a function of the delay before rises start, the rate of rise and the rate of decay. For example, piezometric levels in the undeveloped colluvium slopes reach a peak and start to decline within hours of heavy rainfall. A second rainfall here would have less effect than in the lower slopes where a peak is not reached until well after heavy rainfall.
A critical duration of storm has been estimated for each piezometer by inspecting the record of responses or, with short records, by comparison with nearby piezometers.

The method for a piezometer in decomposed volcanics is illustrated in Figure 5.5B where the responses for all storms are plotted against storm rainfall. The duration of each storm is marked and approximate relationships shown for durations of 1, 4 and 7 days. The mean response for each storm duration is extrapolated up to the 1 in 10 year rainfalls for that duration and that giving the largest rise is taken to represent the critical storm for that piezometer.

None of the piezometer records is long enough to investigate satisfactorily whether responses vary linearly with storm size. (The analysis shown in Figure 5.5B is for a piezometer with a particularly good record of storm data.) The scatter caused by variations in rainfall patterns between storms is enough to disguise any changes in response rate with increasing storm intensities. Non-linearities are not likely to be large within the limited range of extrapolation.

Critical durations in decomposed granite vary throughout the study area. In the University area they are almost all greater than 14 days, as would be expected from the generally seasonal responses. However, in the eastern study areas shorter storms are important and critical durations are normally less than 7 days. The piezometers in decomposed volcanics are mostly in the western areas; their critical durations are generally 2-4 days. The few faster responding piezometers do not have greater storm responses. Colluvium responses are generally greatest to storms of 2-4 days except in young (class 3) colluvium. There are few piezometers in this material, but they suggest that storms of one day or less are the most critical.

Storm responses are generally 2 m-3 m in decomposed granite, being larger higher in the slope and towards University Ridge and in the Conduit Road area. This reflects the greater recharge in these areas and the dissipation of storm responses as they flow through the aquifer.

A similar pattern is seen in decomposed volcanics with responses over 3 m in the higher slopes, and as low as 1 m near the contact. Lower rises adjacent to Po Shan slip and behind Po Shan Mansions seem to reflect the groundwater drainage measures in these areas.

5.4.3 Perched water tables

Perching is thought to occur at the interface between colluvium and in situ material, whether this is decomposed volcanics or granites. There is no evidence of higher general perching layers, although small positive pore water pressures have been recorded by suction instrumentation. There is some suggestion of perching layers in decomposed volcanics, but this is inconclusive. Thus, in the derivation of 1 in 10 year levels, the surface of in situ material has been taken as the base of all perched water tables.

Where transient perched water tables occur, the colluvium aquifer would normally be dry between storms and the surface of in situ material has been taken as the typical pre-event level. In permanent or semi-permanent perched water tables the average 1979 pre-storm level was used, as for the main water table analyses.

The derivation of 1 in 10 year rises is as discussed above for the main water table and is based on the extrapolation of the most sensitive observed responses at each location. With transient perched water tables, the observed responses are assumed to start from the surface of in situ material. In several cases this level is not known accurately, or only occasional perched levels have been observed, and analyses have not been possible.

There are some areas where no perching can occur. These are areas without colluvium or where there is effective natural or artificial drainage. Major man-made drainage measures (see Drawing H7) include a drainage blanket in the Po Shan remedial works which is expected to prevent any perching. Little or no perching is expected in the colluvium in the area of the Sir Ellis Kadoorie Primary School and King George V Memorial Park close to the northern boundary of the Central area. A network of drainage tunnels (Sai Ying Pun Tunnels) has been installed in this area (see Drawing H7) and they may be helping to reduce perching of water in the colluvium.

Several piezometers in colluvium along Conduit Road close to the foot of the Seymour Boulder Field (Figure 1.2) and in the upper slopes, show very large and rapid responses to storm events.

There are eight automatically monitored piezometers in the upper colluvium slopes of the Glenealy, Seymour and Central areas which are all installed as close as practically possible to the base of the colluvium. Because of the need to ensure that the response zone does not penetrate the in situ boundary and the need for the dip tube of the monitoring unit to be at least 0.5 m above the piezometer tip (reference 32), a perched head of at least 1.5 m above the colluvium base would be required before it was detected. Observations from the suction work suggest that heads of only 0.5 m occurred during 1980.

None of the eight piezometers recorded any perched heads during the 1980 wet season, but this could be because no major storms occurred during 1980. However, piezometer ML55B in the University area recorded substantial storm responses during 1980.

As a result of the paucity of data, a different approach was adopted for the upper slopes where extensive depths of colluvium occur. Two extreme situations were considered.

The first situation was to consider only vertical infiltration into colluvium without any outflow downslope or into the underlying in situ material. This approach models rainfall on horizontal ground. It is, however, unduly pessimistic in the long term on slopes as a result of neglecting water movement downslope. A runoff coefficient of 34% and an increase in storage of 5% (storage coefficient 0.05) to bring the colluvium to complete saturation were assumed. The resulting build up of perched head with no outflow of water was determined for 1 in 10 year storm events up to a duration of about 400 hours. This is shown as a plot of perched head versus event duration in Figure 5.6, as the vertical infiltration line (solid line).

The second situation considered was that of steady state flow downslope in the colluvium with no flow into the underlying in situ material. The steady state situation infers that the outflow at the downslope end of the catchment being considered equals the rate of recharge.

Darcy's Law was used to calculate the perched head necessary to maintain the steady state at the downslope ends of the catchments. Again a runoff coefficient of 34% together with a mean permeability of 1×10^{-4} m/s with slope angle of 30° were assumed.

The outflow per unit width was considered in terms of hydraulic gradient (slope angle), flow velocity (permeability) and depth of flow (depth of perched head). This can be expressed per unit width as:

$$\text{outflow} = kih \ \text{m}^2/\text{s}$$

$$\text{recharge} = \frac{L\,(1-r)\,R}{t} \ \text{m}^2/\text{s}$$

where L = catchment length (m)
R = depth of rainfall from 1 in 10 year storm event (m)
r = run-off coefficient
k = mean permeability (m/s)
i = mean hydraulic gradient
h = head at downslope end of catchment (m)
t = duration of 1 in 10 year storm event (s)

The resulting perched head (h) for 1 in 10 year storm events up to a duration of about 400 hours for various catchment lengths are shown in Figure 5.6 as steady state flow lines (dashed lines).

The intersection between the vertical infiltration line and the steady state flow line for each catchment length gives the maximum possible 1 in 10 year perched head at the downslope end of that catchment. This has been plotted in Figure 5.7 for catchment lengths up to 400 m. This method was used for the estimation of 1 in 10 year perched water levels in the upper colluvium slopes. It was found that the levels predicted by this method were in agreement with levels predicted from extrapolation of piezometric data in the middle slopes in the region of Conduit Road.

Estimates are also required of natural 1 in 10 year rises in the urban and partially urbanised areas. This colluvium is more uniform and less permeable. Recharge will be substantially lower because of the extent of paving. There are no piezometers that can be used to estimate rises in the University and Po Shan areas. The general absence of observed perched heads during 1979 and 1980 in the University area suggests that rises will be slight.

Most of the piezometers in colluvium in the Glenealy, Seymour and Central areas are situated in the middle and lower developed slopes. There is no evidence to suggest that, in general, the observed rises in these piezometers are not due to rises in the natural groundwater tables. These piezometers have, accordingly, been used in the predictions of the 1 in 10 year perched water levels.

5.5 Discussion

5.5.1 Review of analyses

The three methods of analysis described above have provided different types of result. The statistical correlation does not assume that the return period of rainfall events and piezometric levels correspond. The analysis is based on levels, rather than changes in level. Only 12 point estimates of 1 in 10 year levels have resulted from the analysis, but the method has been shown to be satisfactory and more study would lead to further estimates. The 12 values are insufficient to interpolate throughout the area and can only be used for comparison with the other methods.

The modelling studies have provided qualitative results. Much of the understanding of flows within the Mid-levels slopes resulted from attempting to model them. The models have helped in assessing aquifer properties and estimating recharge and they have assisted in understanding responses to different rainfalls. The main conclusion drawn is that groundwater responses do not vary in character as rainfall increases.

The method of extrapolating observed responses is the least sophisticated but has several advantages. It can be used for almost all piezometers. Any background knowledge of each piezometer can be incorporated. Comparing results areally and showing sensible patterns leads to greater confidence in the results. This regionalisation or pooling of results effectively makes more data available at each location.

5.5.2 Reliability of results

The reliability of the estimated 1 in 10 year levels is governed by the amount of data available; the methods of analysis are adequate for the amount of data and the limited extrapolation required. The data were limited in two ways:

a) in time, as almost all piezometer records were for 18 months or shorter

b) areally, with substantial areas without piezometric data

The short length of records is the less severe problem, although it is unfortunate that 1980 was such a dry year. The pooling of data by comparison of piezometers gives a good picture of the areal variation of responses. In addition, 1979 levels in seasonally responding piezometers were very similar to estimated 1 in 10 year levels, so that little or no extrapolation is required. The responses to several storms are contained in the records of piezometers with storm responses, giving more confidence in these extrapolations.

The areal lack of data is more severe. Hydraulic gradients are steep, but not uniform as there are many perturbations due to changes in aquifer geometry, topography and recharge, and possibly aquifer properties. Thus, although responses in an area without piezometers may be well defined by neighbouring piezometers, the absolute level at which these responses occur is not known. It has, of course, been inferred from sections and contoured, but only minor changes in contour positions will significantly alter estimated 1 in 10 year levels. This applies equally to the main and perched water tables. There are fewer piezometers in the perched water table, but the elevation component has been defined by the surface of in situ material. This is as accurately known as the main water table elevation.

5.5.3 Effects of leakage from services

Leaking services have two effects. They can cause a general permanent rise in piezometric levels over an area by steady leakage from mains. There may be occasional changes in level as leaks occur, enlarge or are repaired. Leaking storm drains will give storm responses in the urban area where they would not otherwise occur. The responses may change in time as leaks alter, are repaired, or blockages of pipes occur. Neither type of effect is predictable in time or areally.

The two effects have different consequences in the two water tables. The main water table does not show major storm responses due to leakage, although the perched water table does. Steady leakage can cause localised perched water tables, but is less noticeable in the main water table being only one of the three recharge components. It is also difficult to separate as its areal distribution is unknown.

The observed levels and responses of the main water table can be taken as reflecting the effects of average leakage. Major rises in level are unlikely, because the large increases in leakage necessary would probably be noticed, and because the effects seem to be dispersed.

Two situations can be considered for perched water tables: with and without leakage. With leakage, levels can reach the ground surface except for the thickest parts of the colluvium, and where local drainage reduces them.

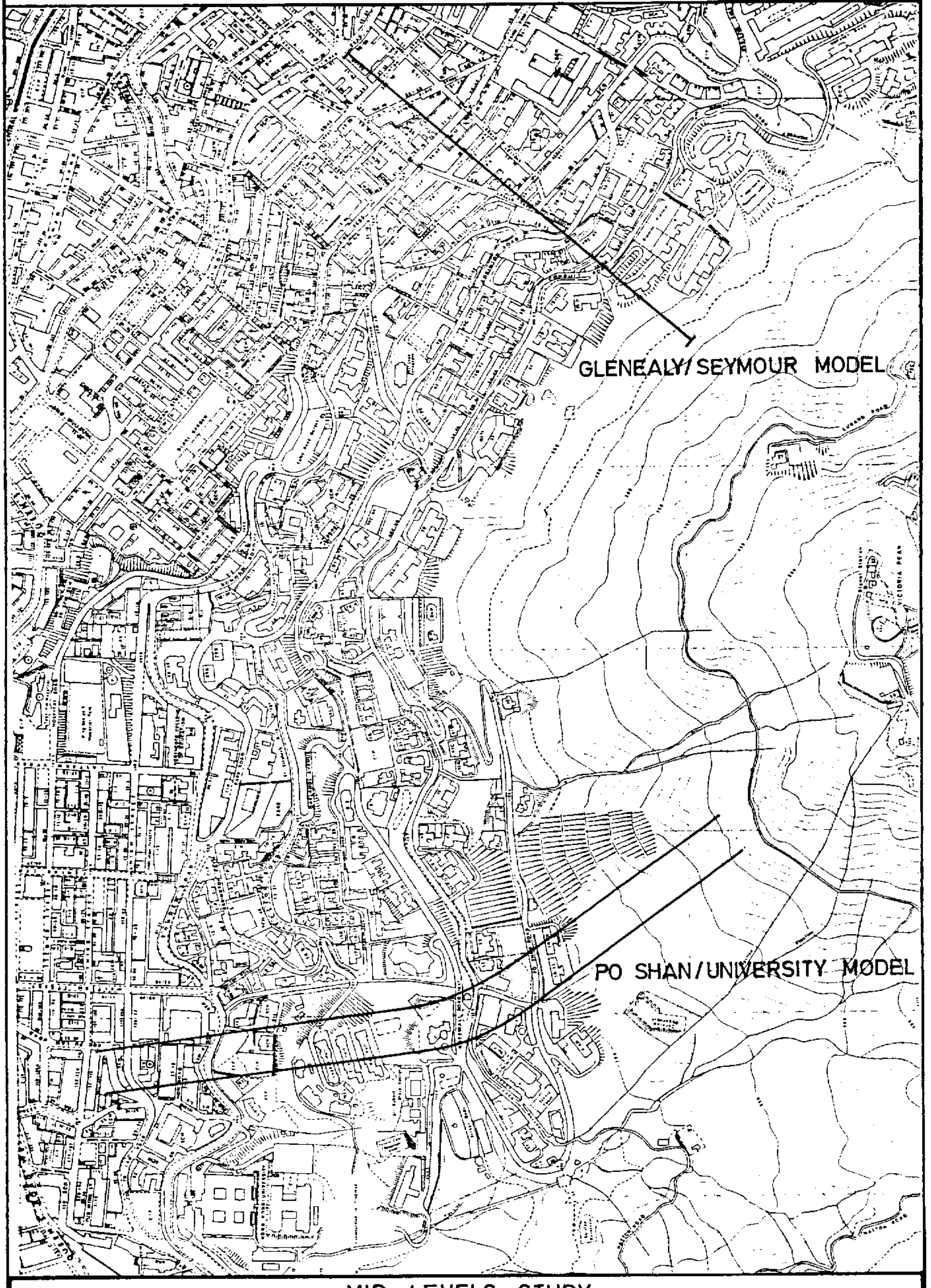
Without leakage, there would be no separate perched water table under parts of the urban area, as there is no significant recharge from rainfall in these areas. In other urban areas, the perched water tables are maintained by downslope flow.

5.6 Conclusions

Piezometric levels were estimated throughout the study area. In the main water table they are 1 in 10 year levels, estimated by extrapolated observed responses. For perched water tables they are 1 in 10 year levels estimated by extrapolating observed responses where such data is available, and by considering recharge and throughflow in other areas. No additional allowances have been made for the effects of leakage, except that the effect of present leakage is included in recent observations.

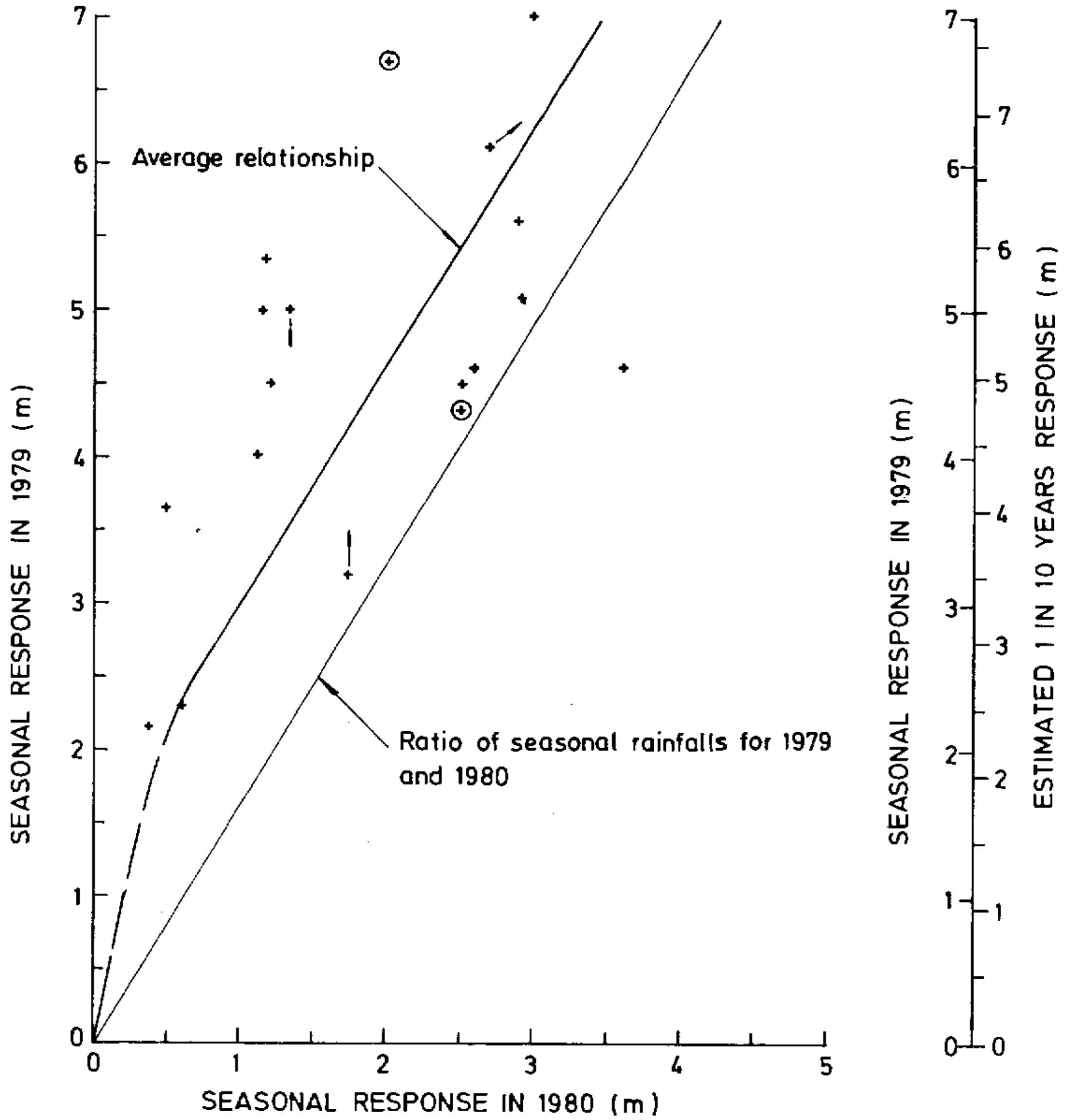
LOCATION OF GROUNDWATER MODELS

FIGURE 5.1



MID - LEVELS STUDY

EXTRAPOLATION OF SEASONAL RESPONSE : UNIVERSITY AND PO SHAN AREAS	FIGURE 5.2



+ Piezometers in University and Po Shan areas showing Type 2 or 3 seasonal response

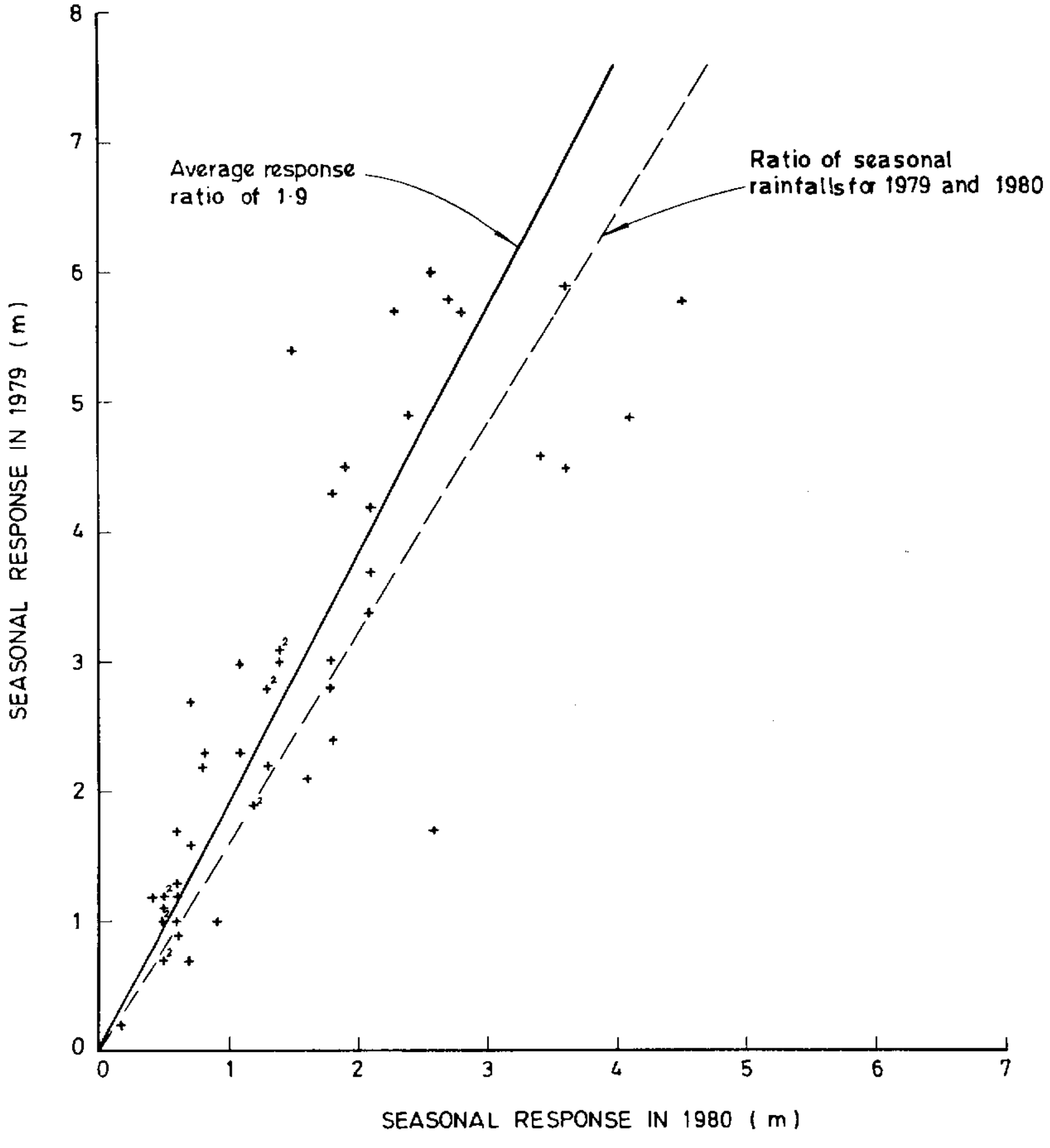
⊕ Piezometers with data for 1977 and 1978, which had similar seasonal rainfalls to 1980 and 1979 respectively

—- Indicates an estimate which may have to be altered in direction shown

Scaling from 1979 to 1 in 10 years is linear and by a factor of 1.1

EXTRAPOLATION OF SEASONAL PIEZOMETER RESPONSE DECOMPOSED ROCK : GLENEALY, SEYMOUR & CENTRAL AREAS

FIGURE 5·3



+² Indicates two data points

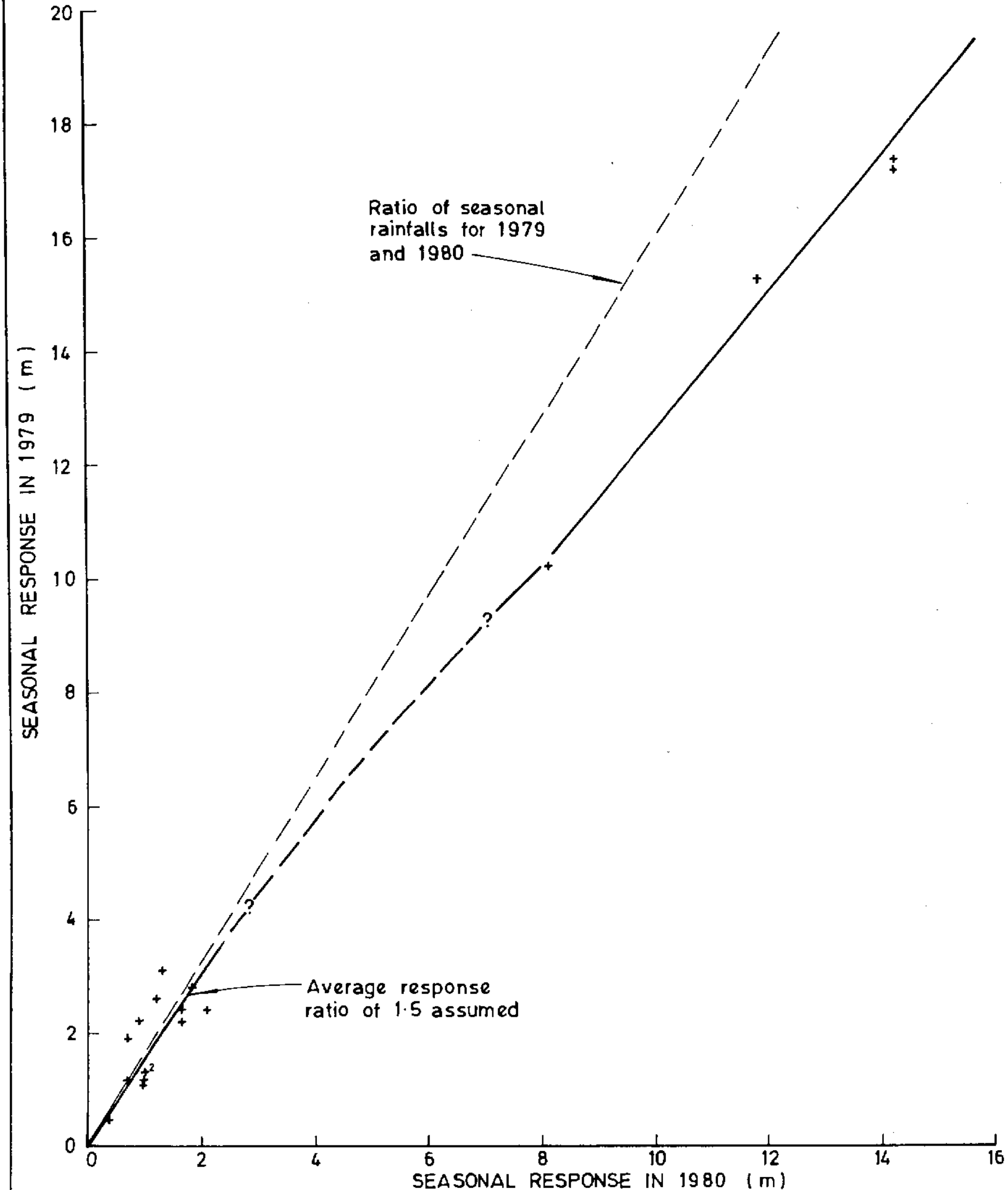
Data from piezometers in Glenealy, Seymour and Central areas.

Seasonal response in 1979 is considered as equivalent to a 1 in 10 year seasonal response.

MID-LEVELS STUDY

EXTRAPOLATION OF SEASONAL PIEZOMETER RESPONSE BEDROCK : GLENEALY, SEYMOUR AND CENTRAL AREAS

FIGURE 5·4



$+^2$ Indicates two data points

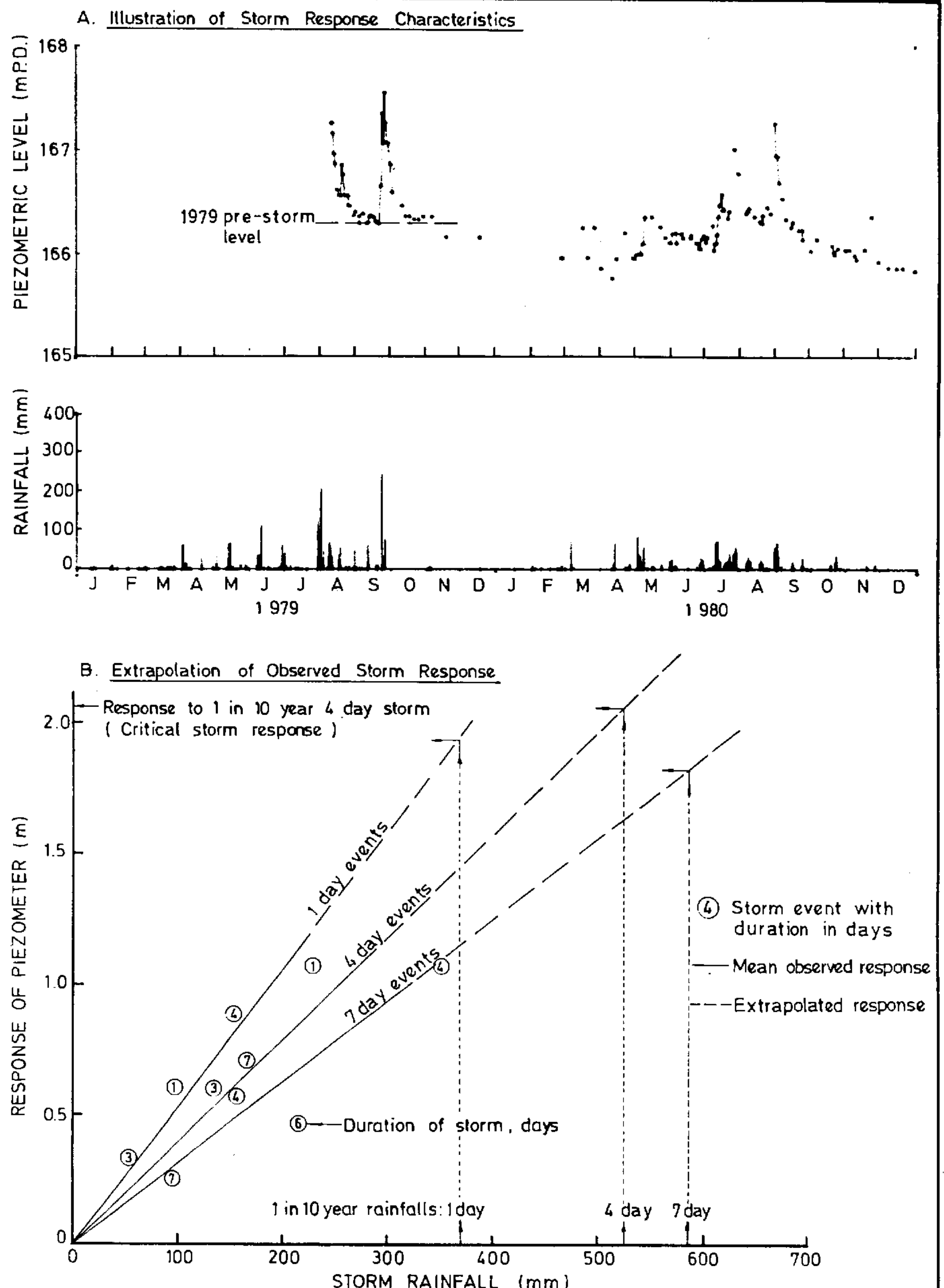
Data from piezometers in Glenealy, Seymour and Central areas

Seasonal response in 1979 is considered as equivalent to a 1 in 10 year seasonal response.

MID-LEVELS STUDY

EXTRAPOLATION OF STORM RESPONSE

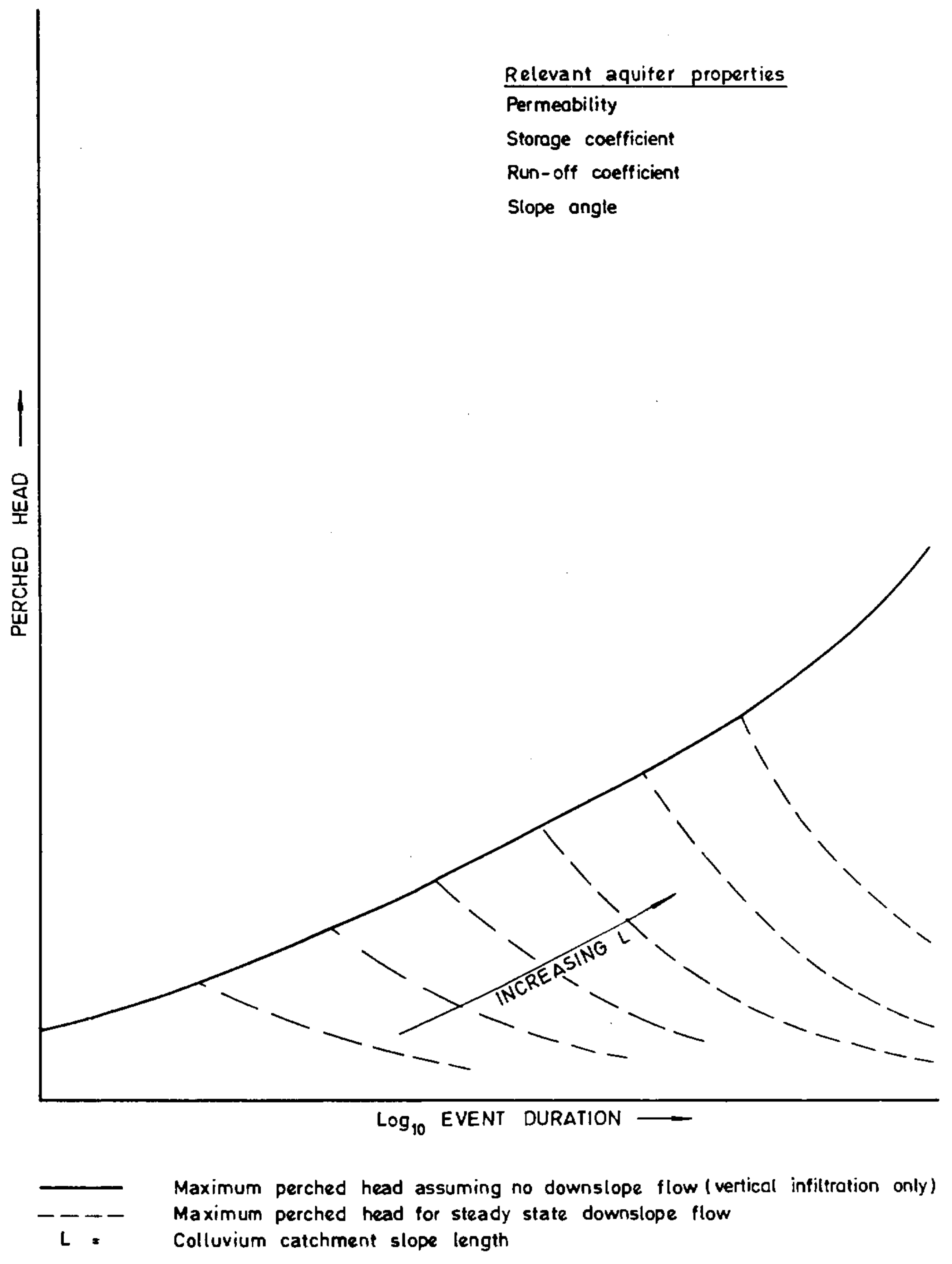
FIGURE 5.5



MID LEVELS STUDY

PERCHED HEAD

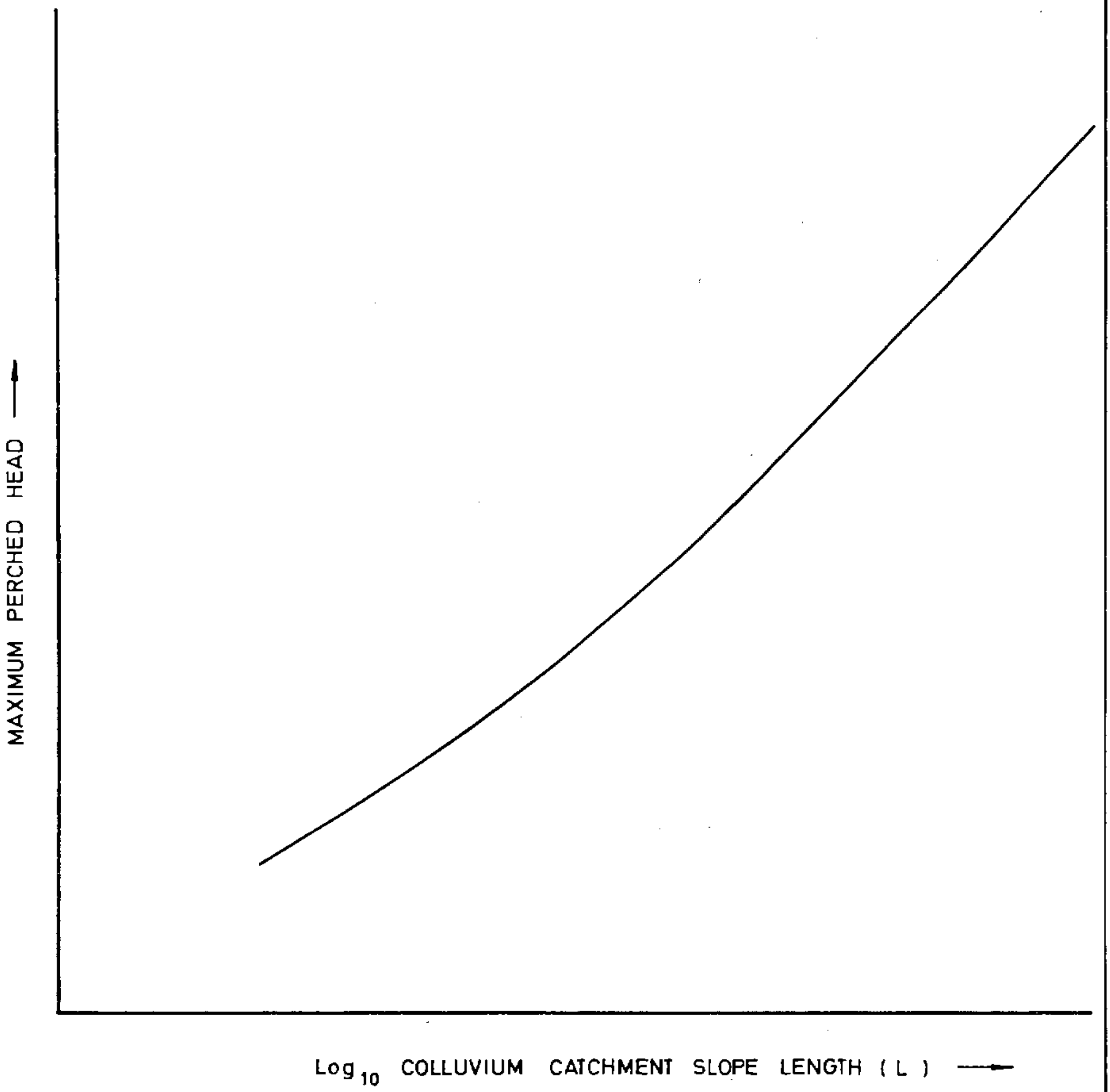
FIGURE 5.6



MID-LEVELS STUDY

MAXIMUM PERCHED HEAD

FIGURE 5.7



MID-LEVELS STUDY

6. CONCLUSIONS

Rainfalls in 1979 were higher than average. Return periods of the maximum observed rainfalls were less than 5 years for durations up to 4 days and between 7-12 years for longer durations. 1980 was extremely dry and all durations of rainfall had return periods below two years.

Direct storm runoff averaged 34% of rainfall and was very variable. For the estimation of annual recharge to groundwater from rainfall soil moisture deficits should be considered. Soil moisture deficits are likely to have only a small effect on the recharge from major storms.

Assessed permeabilities of the various aquifer materials are given in Table 3.2.

Soil suctions in colluvium in the upper slopes are reduced to zero to depths of at least 10 m below an exposed ground surface. Positive pore pressure throughout colluvium of up to $5KN/m^2$ have been observed after storm events in 1980.

There are two interrelated groundwater systems, a main water table in the bedrock and decomposed rock aquifers, and perched water tables in colluvium.

The main water table is discontinuous across the contact between the volcanics and granite aquifers.

The main water table is recharged by rainfall and, in the lower slopes, by leakage from services.

The base of the major perched water tables is the surface of in situ material where there is a discontinuity in permeability. Local perched water tables can occur elsewhere.

Perched water tables are recharged by rainfall, from the main water table in some areas, and by leakage from services.

All four types of water bearing service (fresh and flushing water mains, sewers and storm drains) leak and provide recharge to the groundwater systems. Leakage can give significant rises in piezometric level, particularly in the perched water table.

Groundwater drainage by horizontal wells and tunnels locally reduce the seasonal and storm responses of the main water table as well as drawing down groundwater levels.

SOIL PROPERTIES

CONTENTS

<u>FIGURES</u>

1. INTRODUCTION

This report presents some of the results of the laboratory studies of soil properties carried out as part of the Mid-levels Study. The objective of the laboratory studies was to determine the shearing resistance of the materials that make up the slopes. The aim was to obtain global parameters to use on large slips over the whole study area and it was never intended to test the large number of specimens which would be required to produce design parameters on a lot by lot basis. Therefore the results presented in this report should not be used for design unless supplemented and confirmed by additional testing.

Shear strength parameters quoted in the report are best-fit lines using a least squares fitting technique on all the data available. These values are not necessarily the same as those used for stability analyses in the study where further interpretation has been used to obtain parameters.

Deep failures are uncommon in Hong Kong and so attention was focused on tests in the low stress range appropriate to shallow slips. Exact in situ stresses are unknown but it was considered that overburden stresses of up to 200 kN/m^2 were reasonable.

Colluvium was, from previous experience, expected to be the weakest of the materials present. Consequently sampling and testing concentrated initially on colluvium. Similar tests to those performed on colluvium were later performed on samples of the other materials providing a comprehensive set of results on which to base stability analyses.

In addition to the standard strain controlled triaxial tests, stress controlled tests were carried out to investigate the effect of increasing pore water pressure to failure.

The results of triaxial tests, together with complementary classification tests are recorded in this report.

2. MATERIAL TYPES

2.1 Geological Classification

The geology of the Mid-levels is described in detail in the Geology Report. Volcanic rocks of the Repulse Bay formation have been uplifted and in places metamorphosed by intrusion of the Hong Kong granite. Both rock types have weathered near the ground surface leaving a layer of decomposed rock and soil on the slopes. Erosion has transported in situ material downslope resulting in the formation of a mantle of slope debris material (colluvium). Subsequent construction of roads and buildings on the slopes has included the use of imported fill behind retaining walls and beneath platforms.

Based on this geological background the soils have been grouped into the following three material types for the purposes of this report:

a) colluvium
b) decomposed volcanic rocks
c) decomposed granite rocks

2.2 Colluvium Classification

The varied nature of the colluvium encountered in the Mid-levels led to a classification system which, by a series of letters and numbers, describes the following characteristics:

a) parent rock type
b) degree of weathering of rock component
c) percentage boulders
d) dominant grain size of the matrix

As a result colluvium specimens have been given, by inspection, a classification code, e.g. VDIT (see Geology Report).

In addition there is some evidence to suggest that the colluvium has accumulated during three separate periods and in the Geology Report the hypothesis of 3 classes of colluvium is put forward. Class 1 is the oldest and class 3 the youngest.

Where possible in this report colluvium has been subdivided according to class and in tables and summaries the classification code is shown as well.

2.3 Materials Sampled

Samples of soils and rocks were recovered during the Mid-levels Study site investigations. They comprised core samples from drill holes and tube and block samples from trial pits and caissons. Soil and highly to slightly decomposed rock samples were taken from these sources for the testing programme.

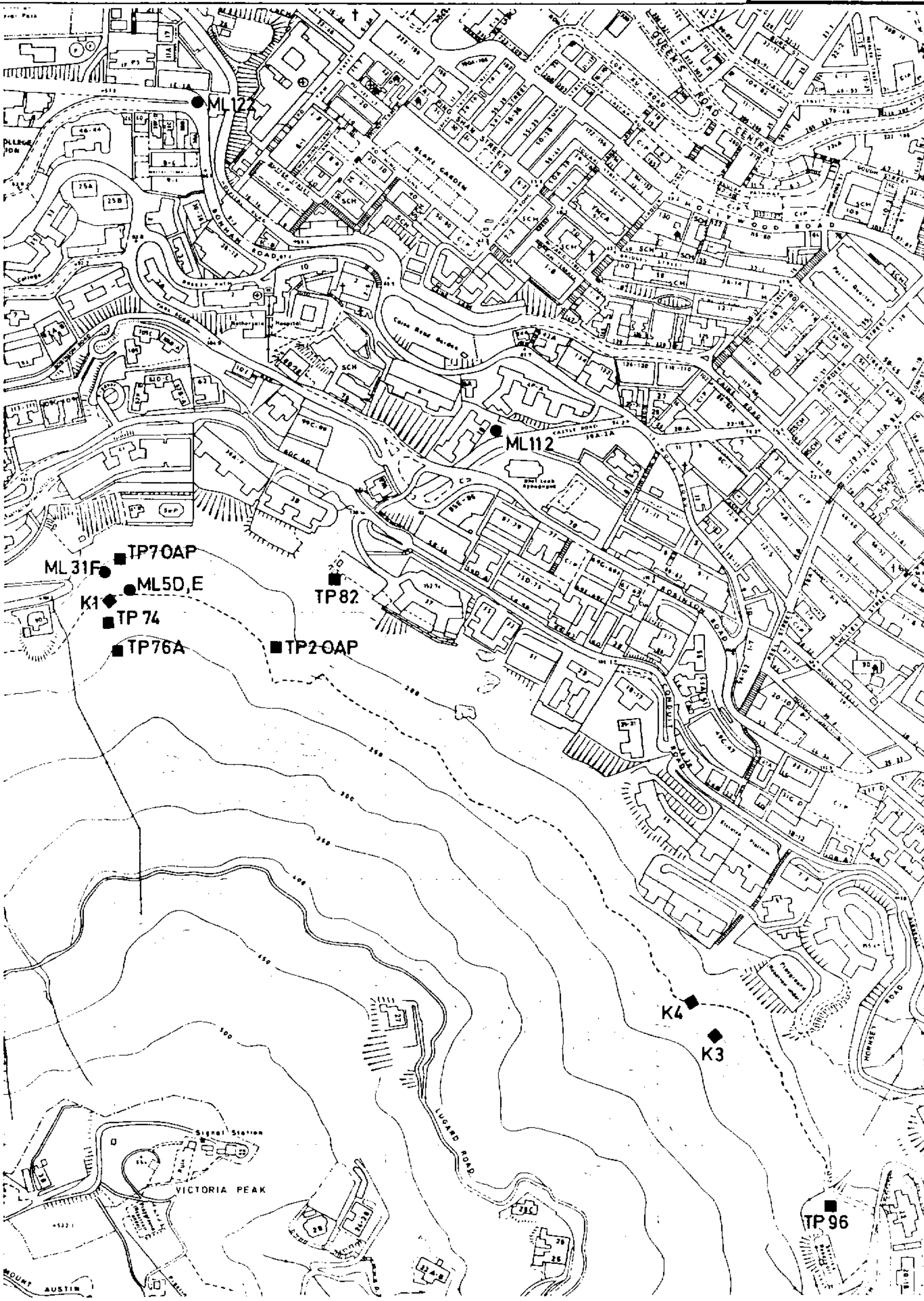
Each site investigation station was given an identifying number, prefixed by ML for drillholes, TP for trial pits and K for caissons. Thus every specimen tested was referenced by its sampling location and its depth below ground surface.

The locations of the site investigation stations are shown in Figures 2.1 to 2.6.

The presence of cobbles and boulders of less decomposed rock within colluvium made sampling and preparation of test specimens difficult. The specimens tested were therefore those containing only particles of medium gravel size and smaller, i.e. the colluvium matrix.

PLAN OF COLLUVIUM SAMPLING LOCATIONS GLENEALY, SEYMOUR AND CENTRAL AREAS

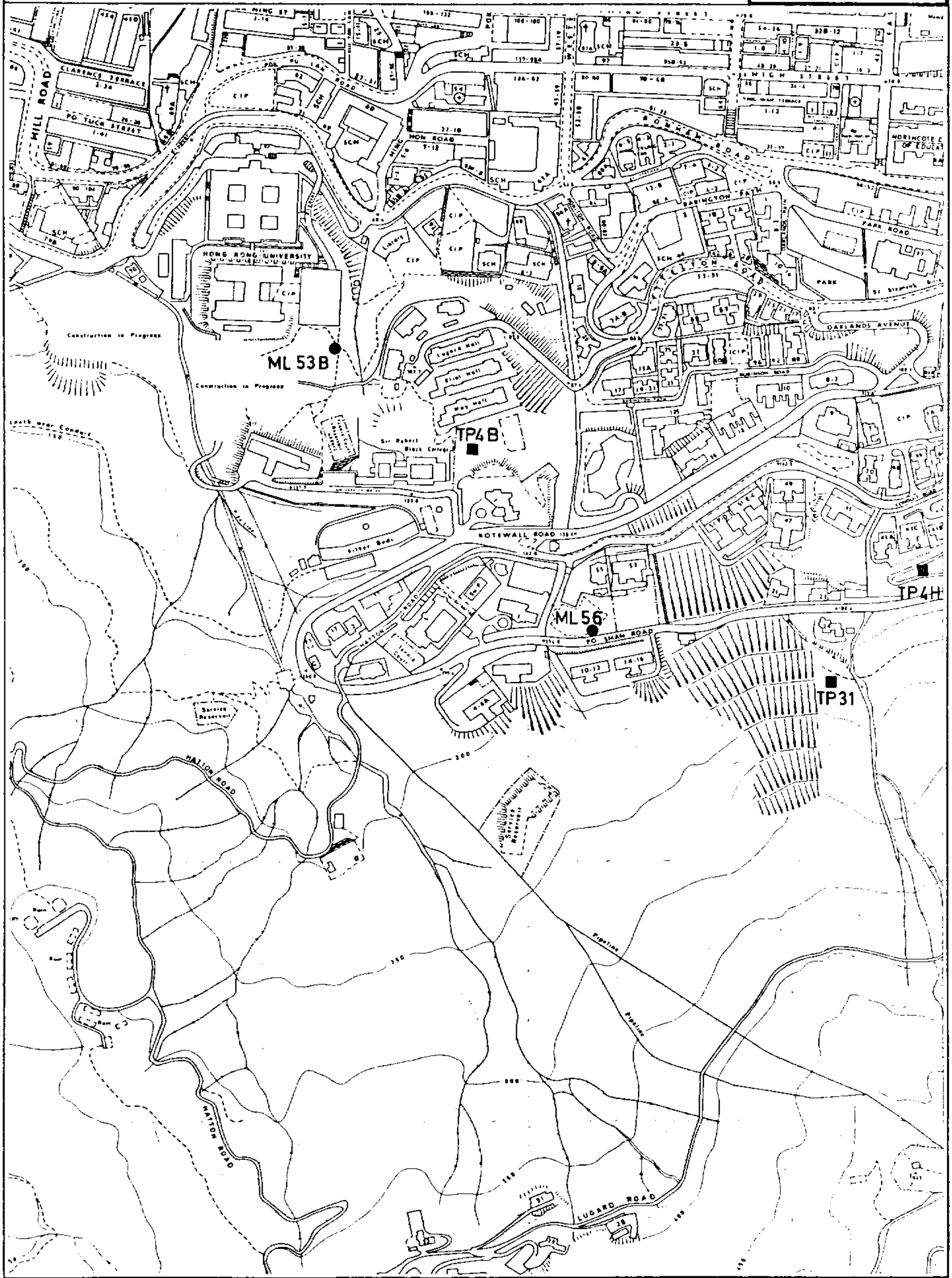
FIGURE 2.1



MID-LEVELS STUDY

PLAN OF COLLUVIUM SAMPLING LOCATIONS
PO SHAN AND UNIVERSITY AREAS

FIGURE 2.2



MID-LEVELS STUDY

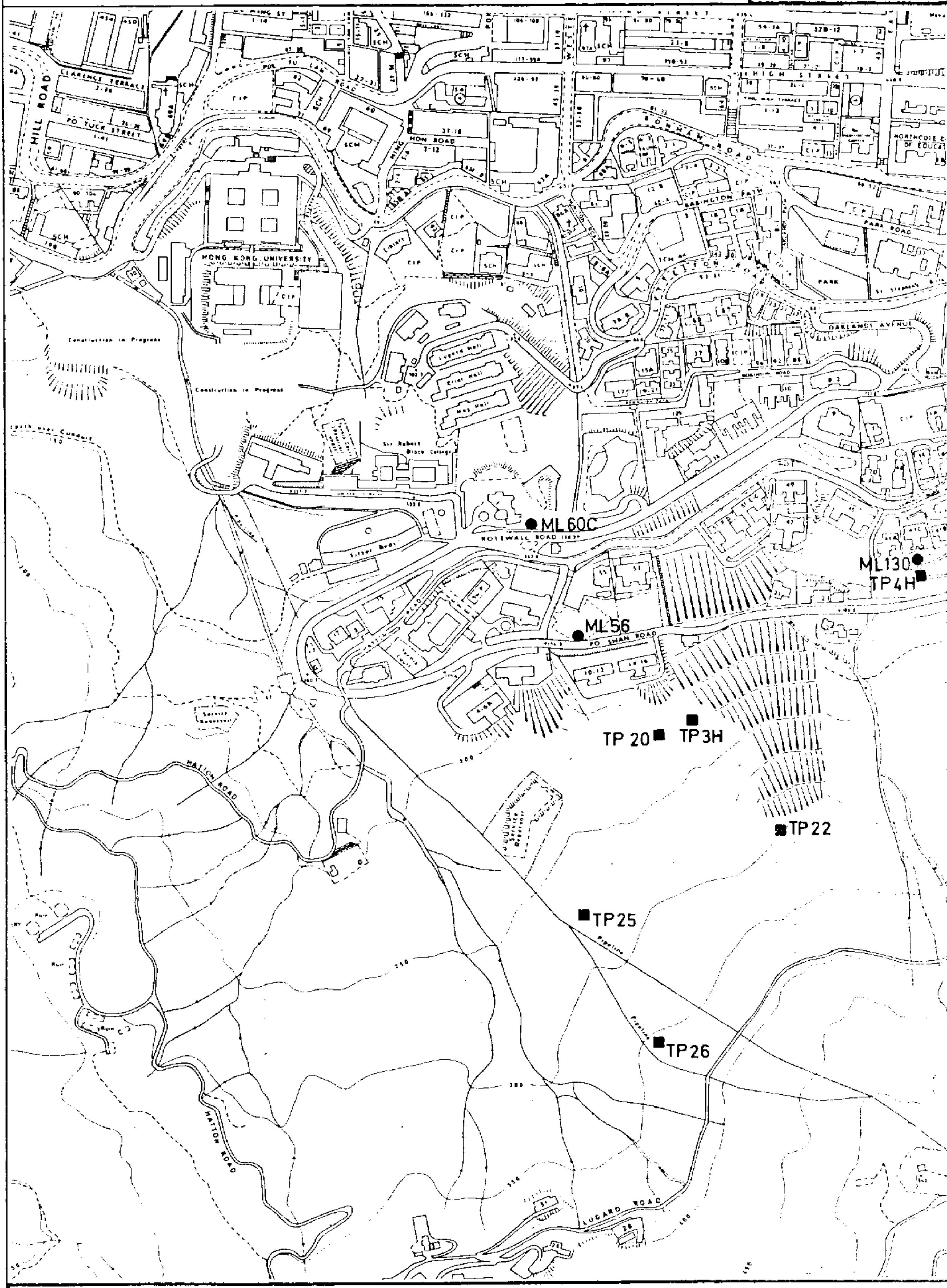
PLAN OF D.V. SAMPLING LOCATIONS
GLENEALY, SEYMOUR AND CENTRAL AREAS

FIGURE 2.3

MID-LEVELS STUDY

PLAN OF D.V. SAMPLING LOCATIONS
PO SHAN AND UNIVERSITY AREAS

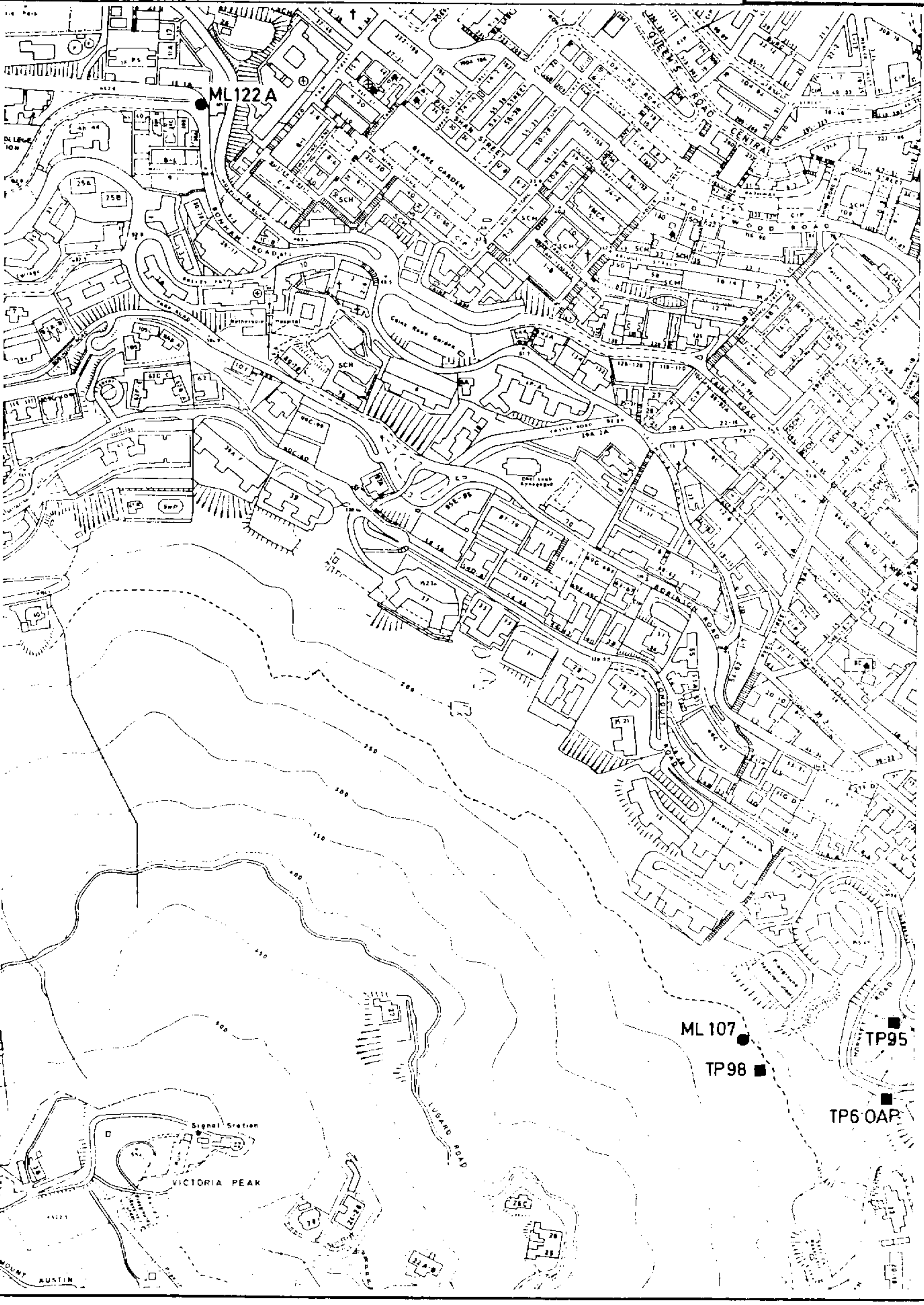
FIGURE 2.4



MID-LEVELS STUDY

PLAN OF C.D.G. SAMPLING LOCATIONS
GLENEALY, SEYMOUR AND CENTRAL AREAS

FIGURE 2.5



MID-LEVELS STUDY

PLAN OF C.D.G. SAMPLING LOCATIONS
PO SHAN AND UNVERSITY AREAS

FIGURE 2.6

MID-LEVELS STUDY

3. CLASSIFICATION TESTS

3.1 Tests Performed

The following classification tests were performed on samples:

a) determination of initial moisture content
b) determination of initial dry density
c) determination of initial degree of saturation
d) determination of particle size distribution

In addition the specific gravity of the soil particles was determined for most of the laboratory specimens.

Stereo pair photographs (1X repro ratio, i.e. negatives are to natural scale and 4R prints have 4X magnification) were taken of selected samples to assist in identification of material types and variations likely to influence the properties of the soils.

The moisture content, initial dry density and initial degree of saturation were calculated from measurements taken prior to shear strength testing of the specimens. Allowance should therefore be made for the following sampling disturbances which may have caused the properties to differ from those of the material in situ.

a) change in moisture content and degree of saturation due to sample drying out during the sampling process and during storage

b) change in moisture content and degree of saturation, caused by infiltration tests, of samples taken from caissons

c) change in moisture content and degree of saturation of drill hole samples caused by absorption of flushing water during drilling

d) change in dry density caused by sampling disturbance and stress relief swelling.

Particle size distribution (PSD) analyses were originally carried out using the standard routine at the GCO Materials Laboratory; i.e. using the oven dried triaxial specimen upon completion of the triaxial test. The dry specimen was broken down using pestle and mortar before riffling and wet sieving in accordance with BS 1377:1975. In order to reduce the harshness of this method a dispersion only technique (with no prior oven drying or grinding with a pestle and mortar and using sodium hexametaphosphate as dispersing agent) was developed and employed on later tests.

3.2 Results

3.2.1 Colluvium

The results of classification tests performed on specimens of colluvium are summarised in Figures 3.1 to 3.4. The data are recorded in Appendix A and data from supplementary PSD tests are given in Appendix B.

The initial moisture content shows no discernible trend with depth (Figure 3.1) with values scattered about the median value of 20%. Recorded dry densities range from 13.0 kN/m^3 to 17.1 kN/m^3, the lower values corresponding to shallow samples. The degrees of saturation of shallow samples show more scatter than do those of the deeper samples. This is to be expected, being caused by near surface evaporation and drying, and by drying of the soil during trial pit excavation. Samples taken from below 4 m indicate a typical degree of saturation of 70% to 100%.

Insufficient data are available on class 1 and class 3 colluvium to warrant a comparison of the initial conditions of the different classes of colluvium.

PSD results have been summarised on a soil classification chart. In Figure 3.2 the results of PSD tests carried out on colluvium triaxial specimens or specimen trimmings are shown. The majority of the results fall in the clayey-silty sand area. The amount of scatter of the results appears to be independent of the method used to determine PSD.

In Figures 3.3 and 3.4 the results of the PSD tests carried out on bulk samples of colluvium from trial pits are shown. In Figure 3.3 the results from Glenealy, Seymour and Central areas are shown whilst those from Po Shan and University areas are shown in Figure 3.4. All tests were performed using the dispersion only technique. A comparison between the results from the two groups of areas is shown below.

	clay	silt	sand & medium gravel
Glenealy, Seymour and Central	15% - 30%	10% - 35%	40% - 70%
Po Shan and University	2% - 40%	15% - 60%	15% - 80%

Overall, the samples from Glenealy, Seymour and Central areas had less variation in clay content and a lower silt content than the samples from Po Shan and University areas.

In Figures 3.3 and 3.4 the class of each colluvium sample has been shown. No obvious grouping of results from similar classes exists in either of the figures. The PSD of the matrix is not thought to be a sensitive indicator of colluvium class since the classification is based upon degree of decomposition of boulders, 'stiffness' of matrix etc. (see Geology Report).

3.2.2 Decomposed volcanics

Figure 3.5 shows in summary form the initial conditions of the decomposed volcanic rock samples tested. The results are similar to those shown for colluvium in that the moisture content is scattered about a median of 19% and dry densities range from 12.8 kN/m^3 to 17.1 kN/m^3.

The PSD tests on decomposed volcanics are summarised in Figures 3.6 and 3.7. In Figure 3.6 the results of tests performed on triaxial specimens are shown. The majority of the results indicate clay contents of less than 20% with approximately half the total number containing less than 10% clay size particles. Of those above 20% all but two come from one trial pit, TP4H, in which the colluvium/in situ material interface was diffuse and dipping (reference 10). Consequently it is likely that block sample 1 contained colluvium and decomposed volcanic rock. Figure 3.7 shows the PSD results obtained from bulk samples collected in the Po Shan and University areas. The predominant feature is the low proportion of clay size particles, again typically less than 10%.

3.2.3 Decomposed granite

The initial conditions of decomposed granite samples are shown in Figure 3.8. Moisture contents range from 10% to 33% with a steady increase with depth. This trend is also evident when comparing the initial degree of saturation of the samples. Since they do not come from a single borehole or trial pit the trend does not indicate the location of groundwater. However it does illustrate near surface drying.

At depths down to 4 m there is considerable variation in the dry densities of samples recorded. Samples with dry densities below 13.0 kN/m^3 were recovered from TP98 where the completely decomposed granite (CDG) is more decomposed than the CDG found nearby (ML107 and TP95). This difference in dry densities also manifests itself in the variation in shear strength of the CDG samples. Below 4 m the dry densities recorded were less variable. The apparent decrease in dry density with depth is not thought to be representative but a function of the limited number of locations from which CDG samples were obtained.

PSD tests on decomposed granite have been performed on triaxial specimens and some bulk samples from the University area. The results are summarised in Figure 3.9. The data is limited but indicates the samples to be composed predominantly of sand and gravel size particles with a small (up to 30%) proportion of silt and clay size particles.

3.2.4 Specific gravity (SG)

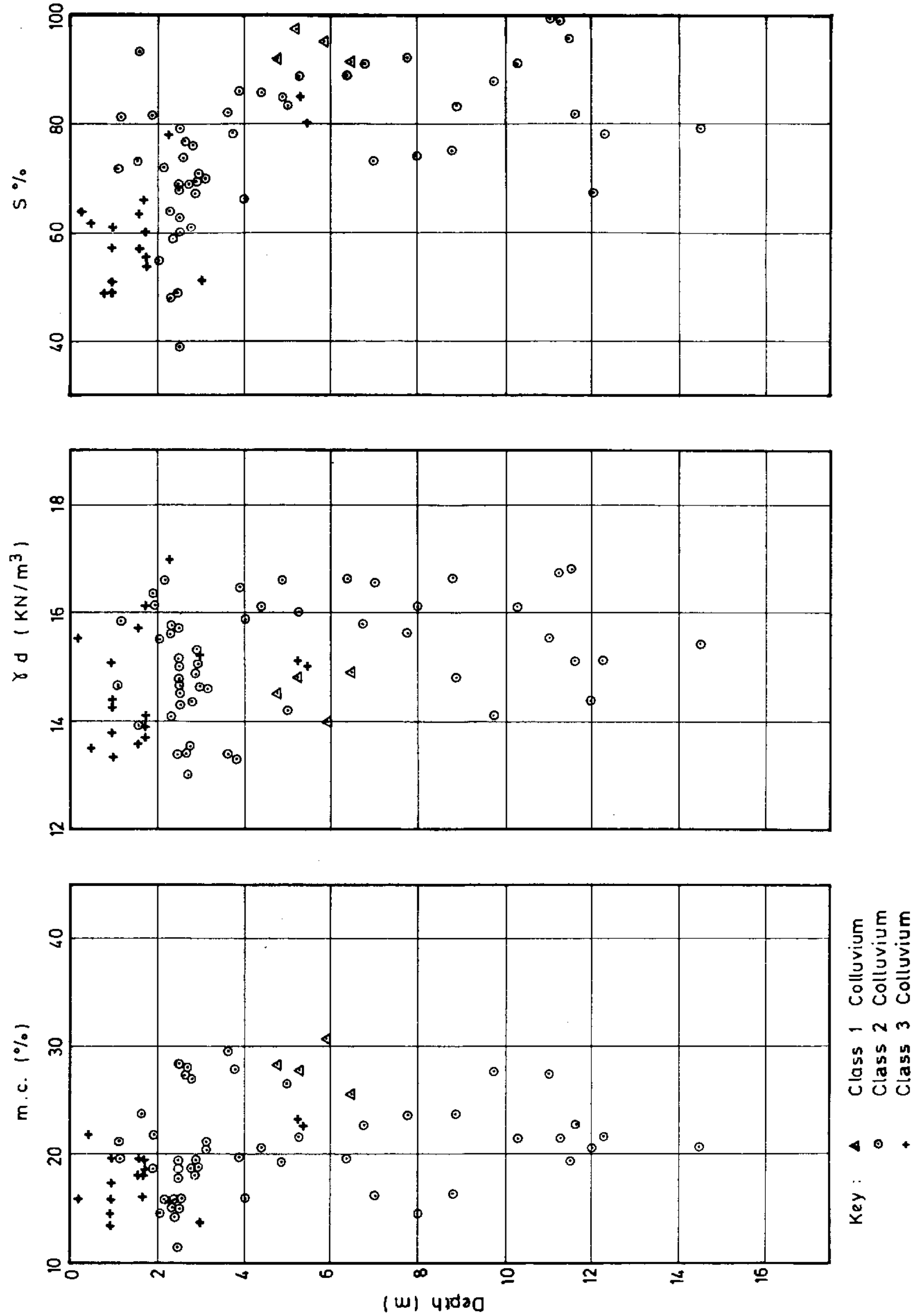
A fraction of the oven dried triaxial specimen was taken from most specimens after testing to enable a measurement of the specific gravity of the particles.

The results are shown in summary form in Figure 3.10. The distributions indicate that it is reasonable to assume the following values if no other data are available.

Material	SG
colluvium	2.68
decomposed volcanics	2.65 - 2.68
decomposed granites	2.65

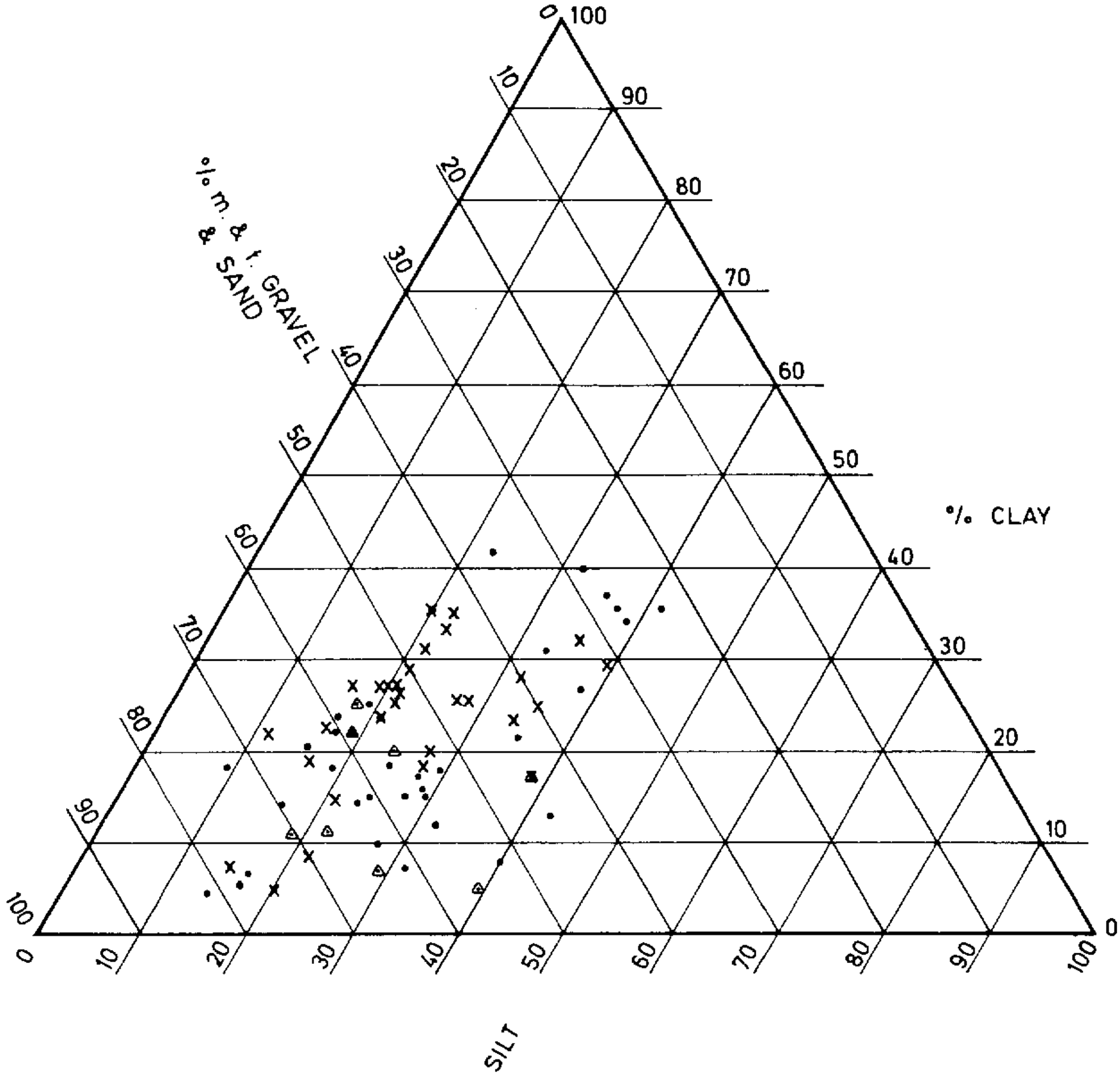
INITIAL CONDITIONS - COLLUVIUM SAMPLES

FIGURE 3.1



MID-LEVELS STUDY

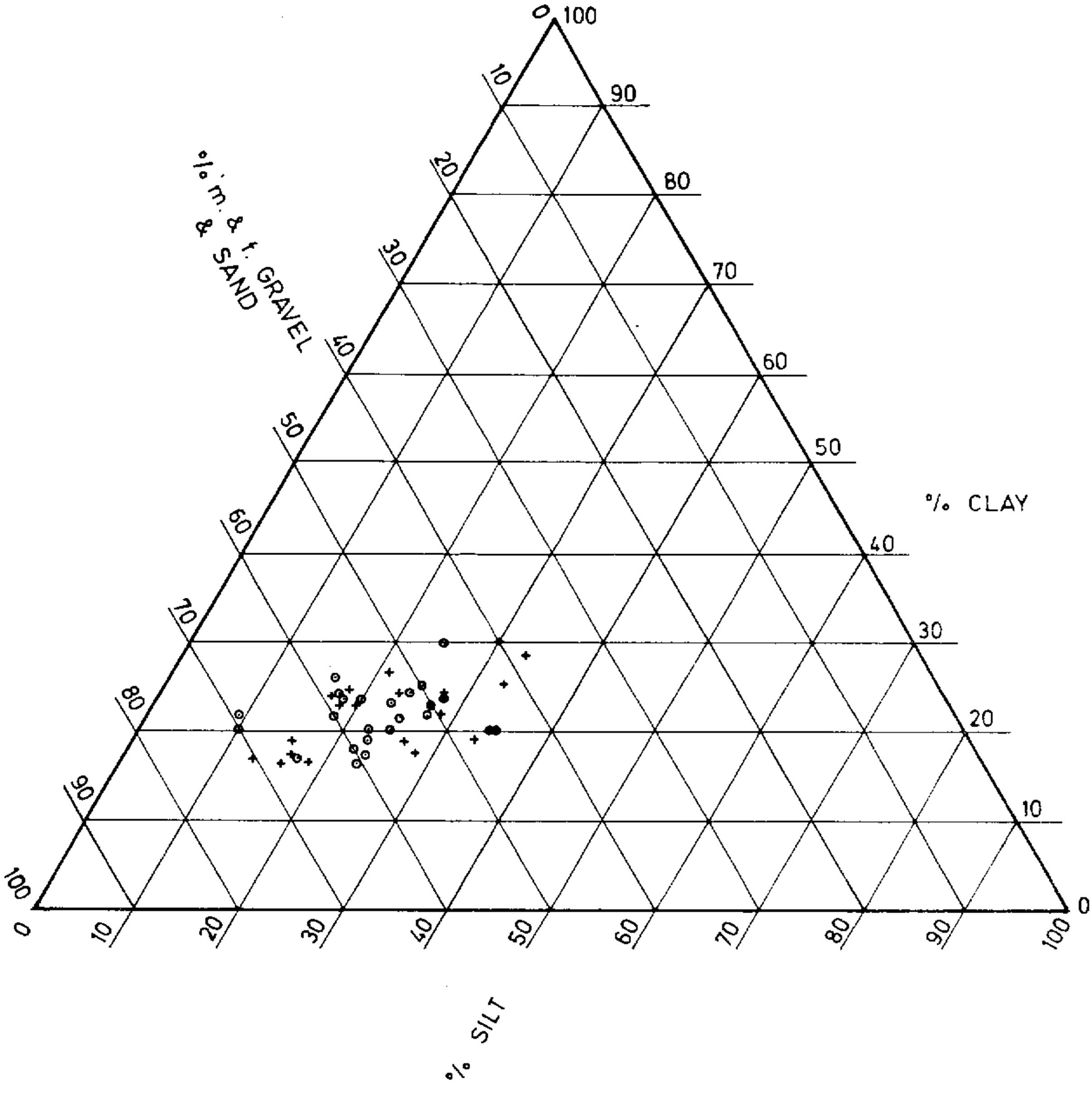
P.S.D. SUMMARY - LABORATORY SAMPLES OF COLLUVIUM | FIGURE 3.2



Key:
x Oven dried method
△ Air dried method
• Dispersion only method

MID-LEVELS STUDY

P.S.D. SUMMARY - BULK SAMPLES OF COLLUVIUM GLENEALY, SEYMOUR AND CENTRAL AREAS	FIGURE 3.3

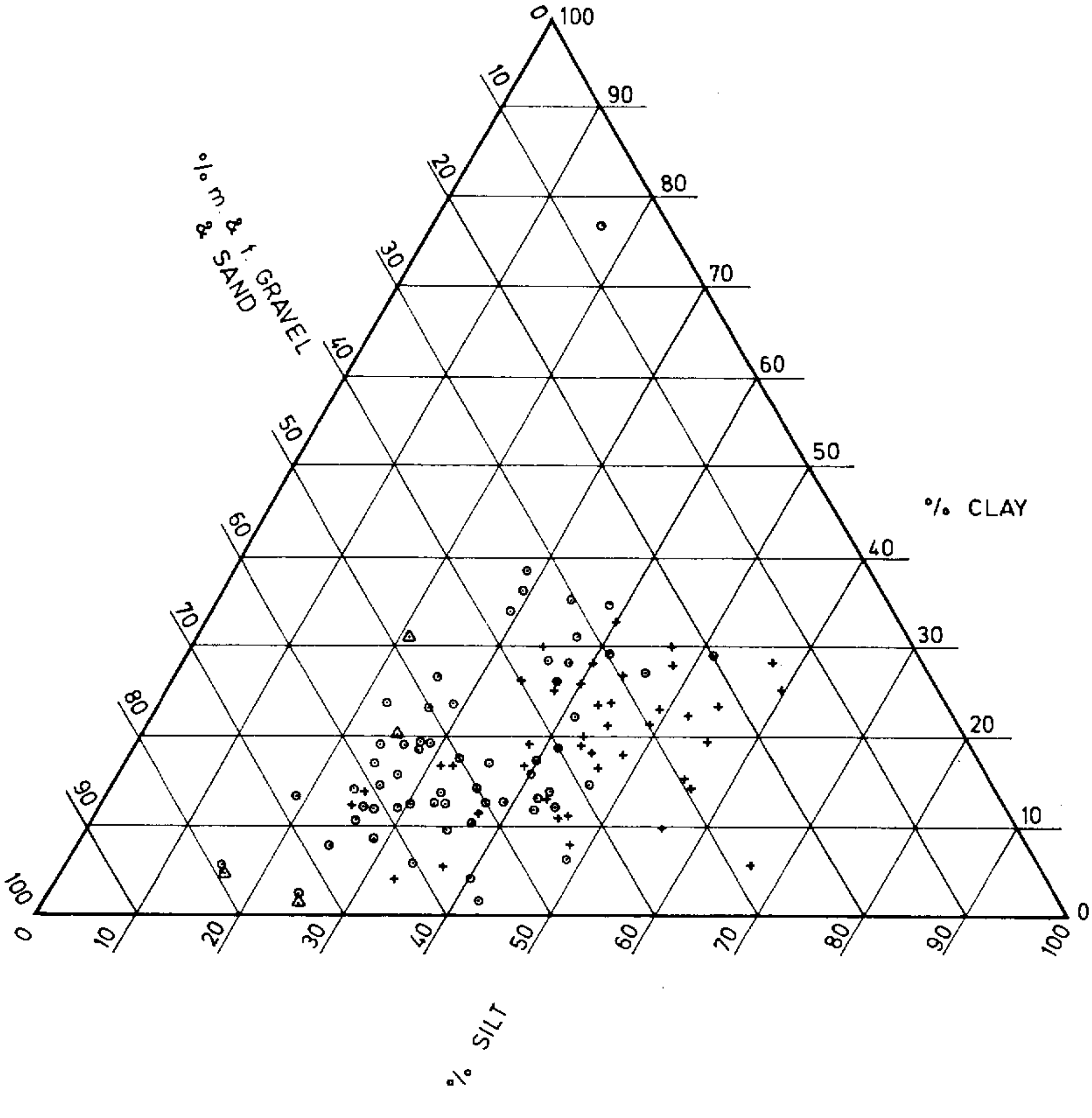


KEY

- ⊙ Class 2 Colluvium
- \+ Class 3 Colluvium

MID-LEVELS STUDY

P.S.D. SUMMARY-	BULK SAMPLES OF COLLUVIUM PO SHAN AND UNIVERSITY AREAS	FIGURE 3.4



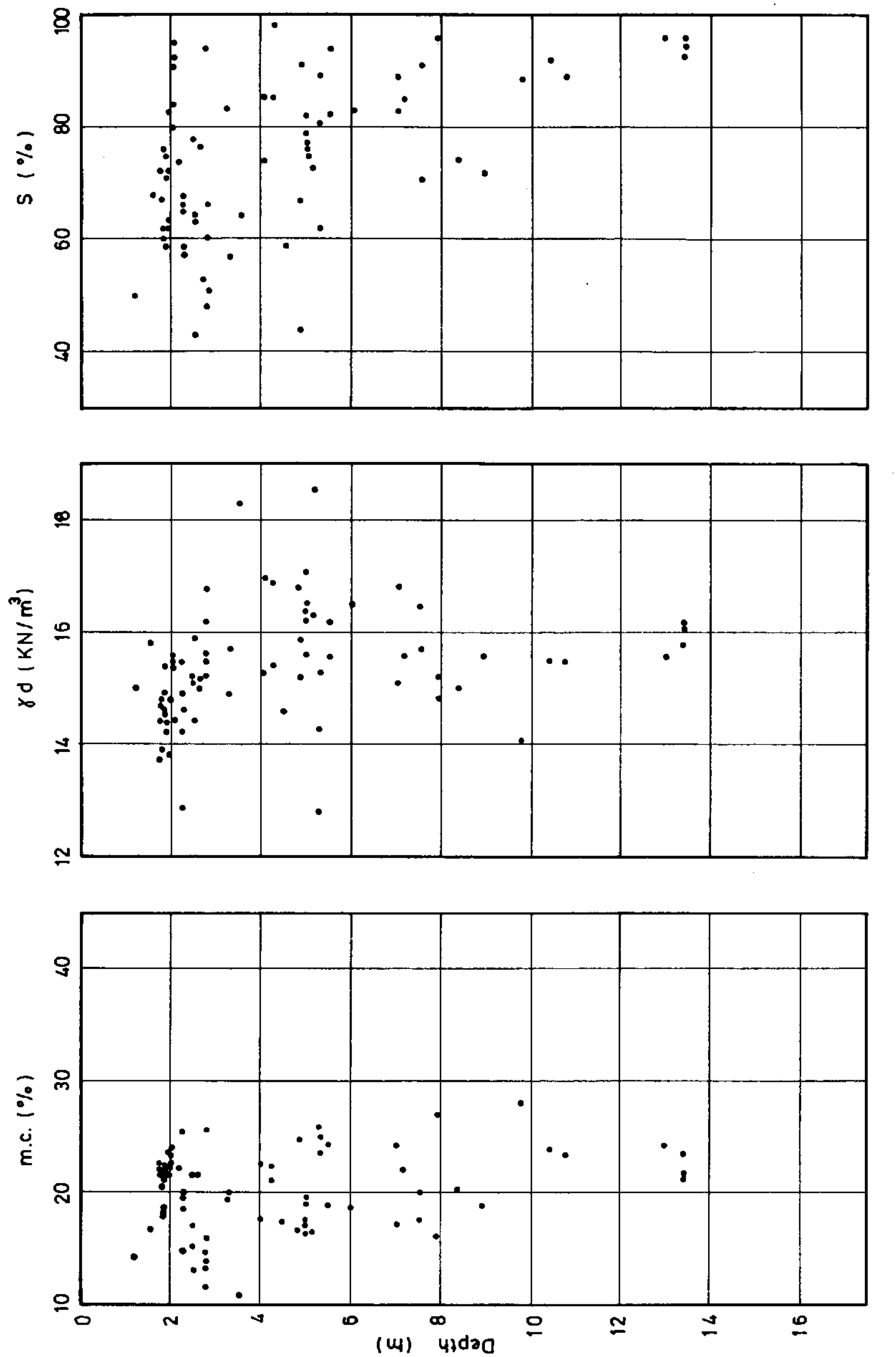
KEY:

△	Class 1	Colluvium
⊙	Class 2	Colluvium
+	Class 3	Colluvium

MID-LEVELS STUDY

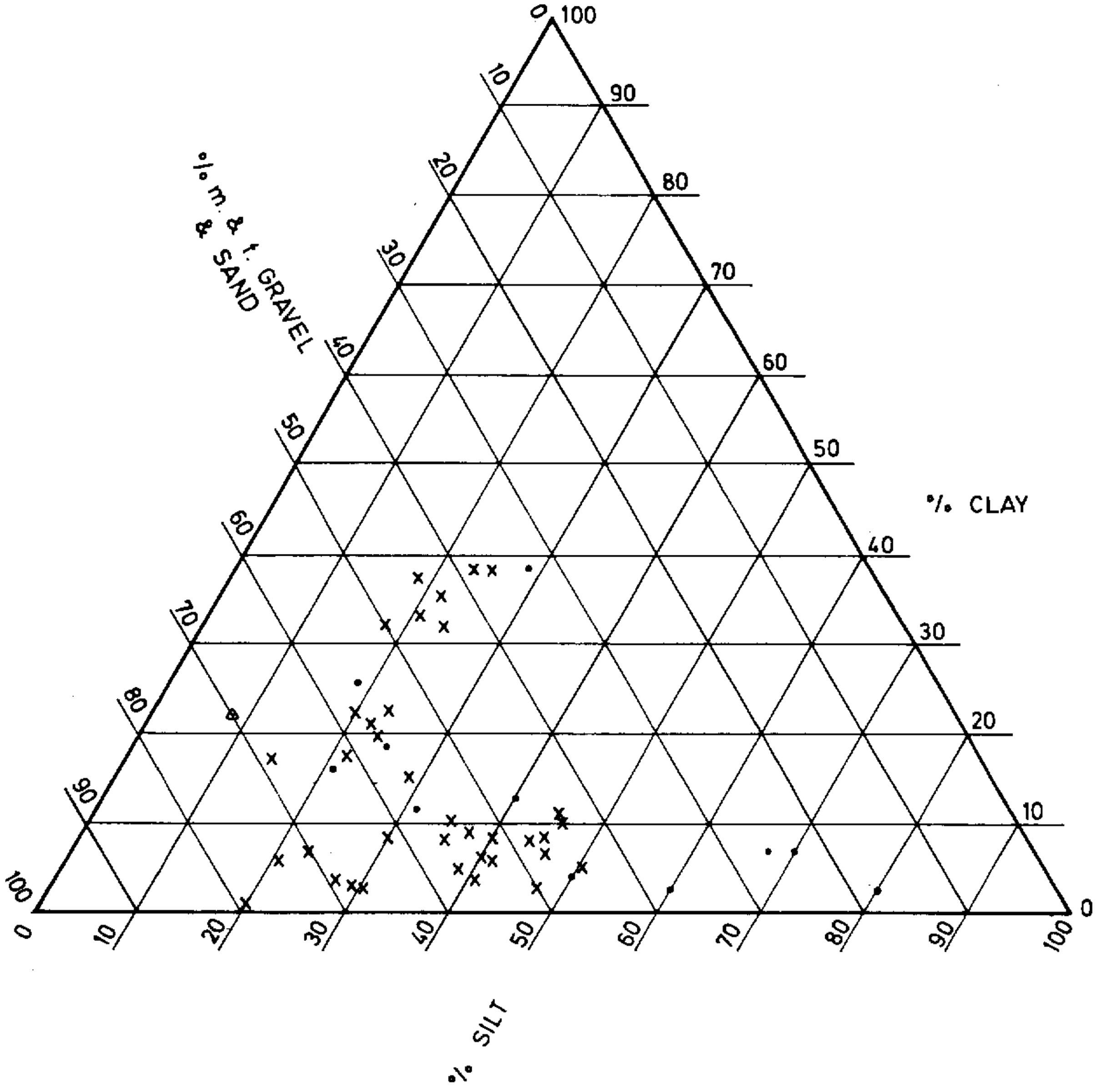
INITIAL CONDITIONS - D.V. SAMPLES

FIGURE 3.5



MID-LEVELS STUDY

P.S.D. SUMMARY - LABORATORY SAMPLES OF D.V. | FIGURE 3.6



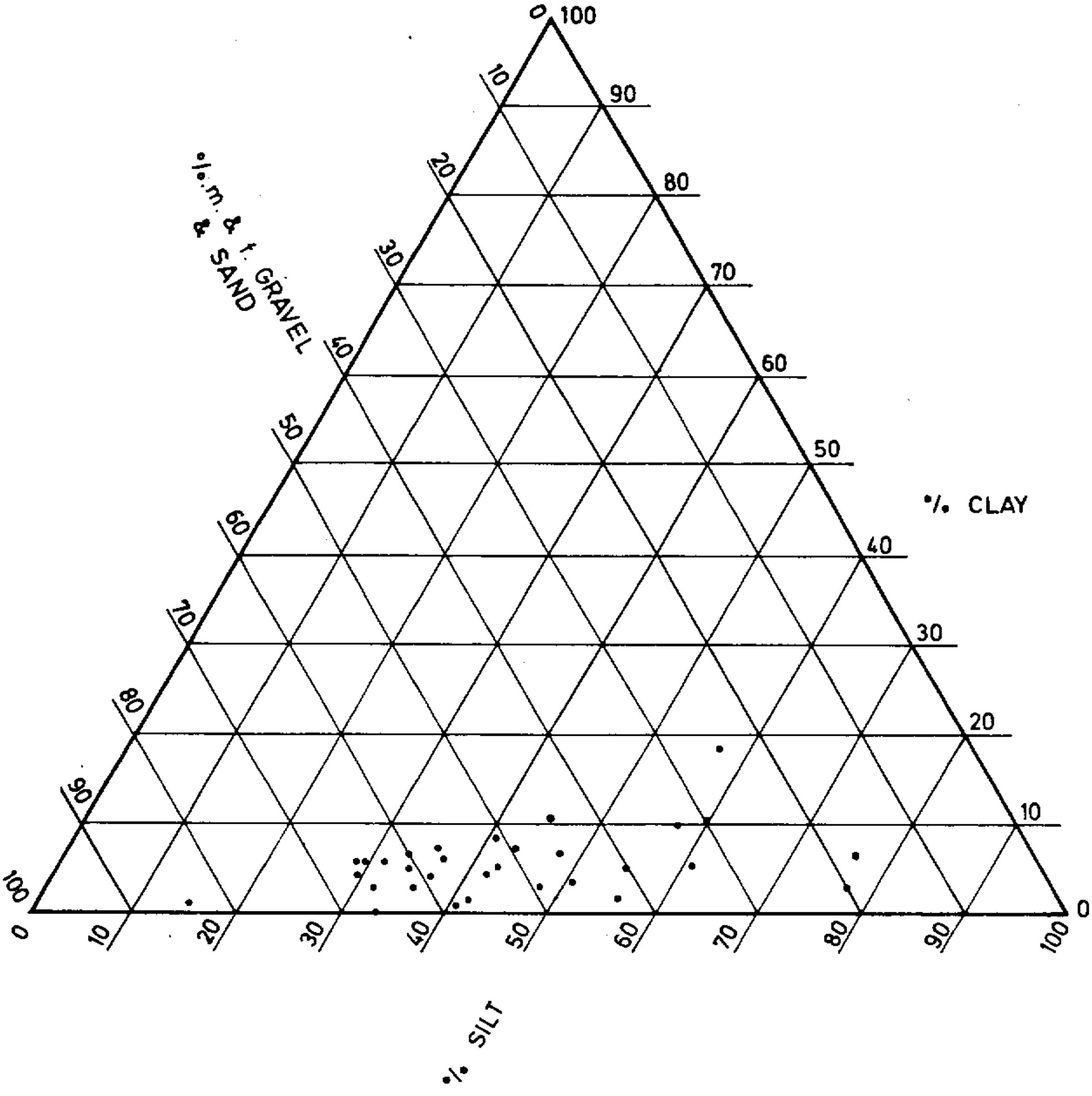
Key : × Oven dried method

△ Air dried method

• Dispersion only method

MID-LEVELS STUDY

P.S.D SUMMARY –	BULK SAMPLES OF D.V. PO SHAN AND UNIVERSITY AREAS	FIGURE 3.7

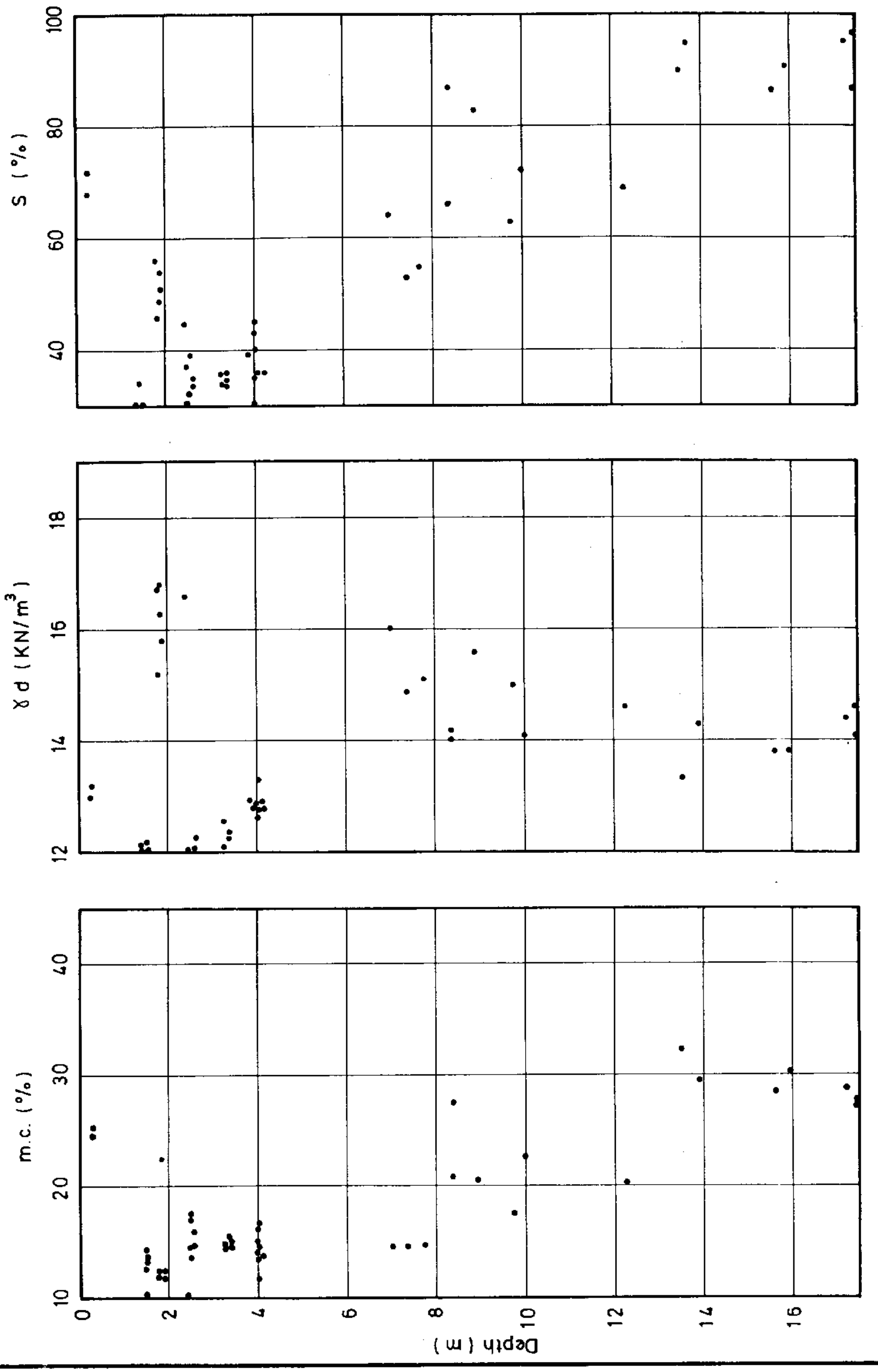


Note: Dispersion only P.S.D. method used.

MID-LEVELS STUDY

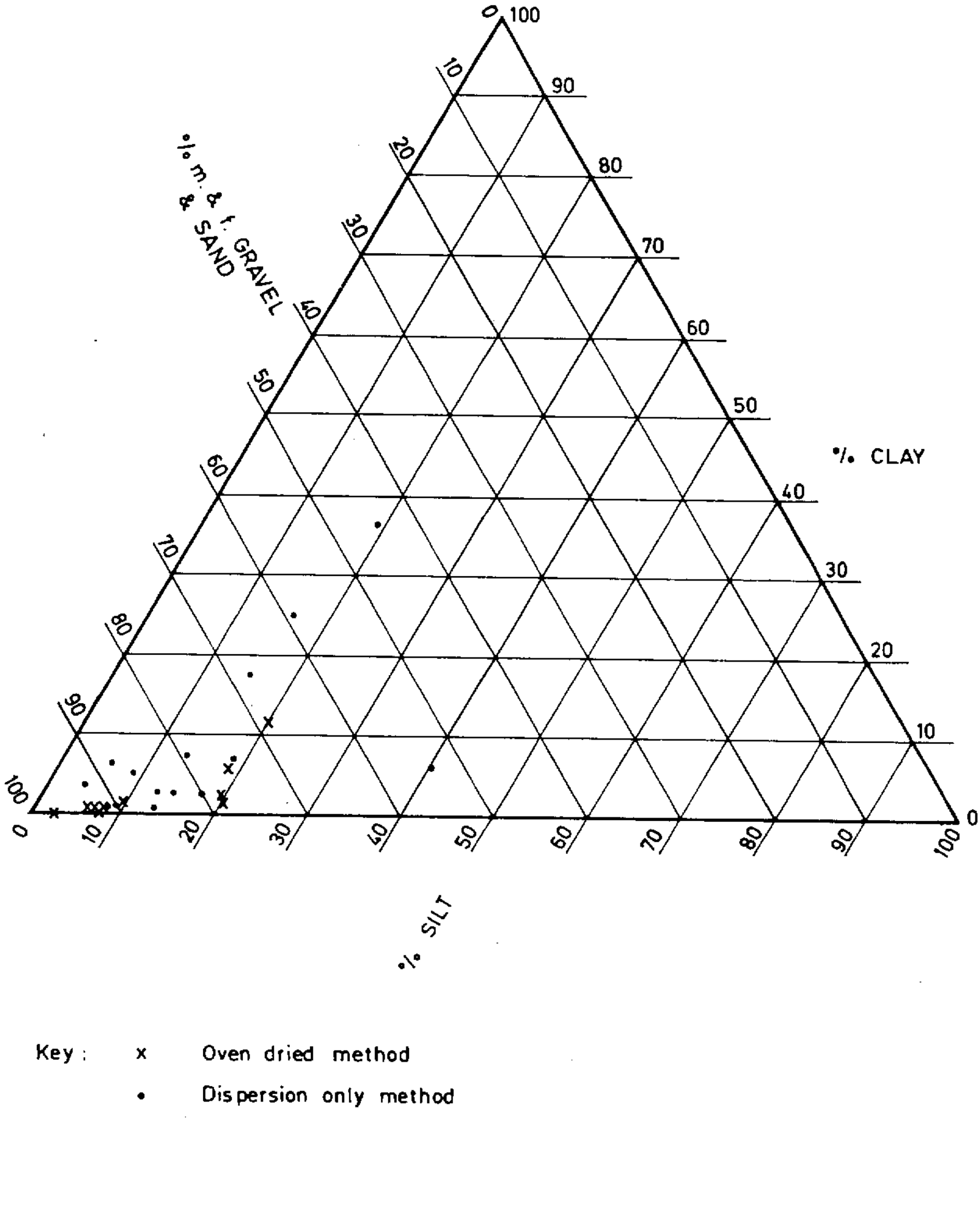
INITIAL CONDITIONS – C. D. G. SAMPLES

FIGURE 3.8



MID-LEVELS STUDY

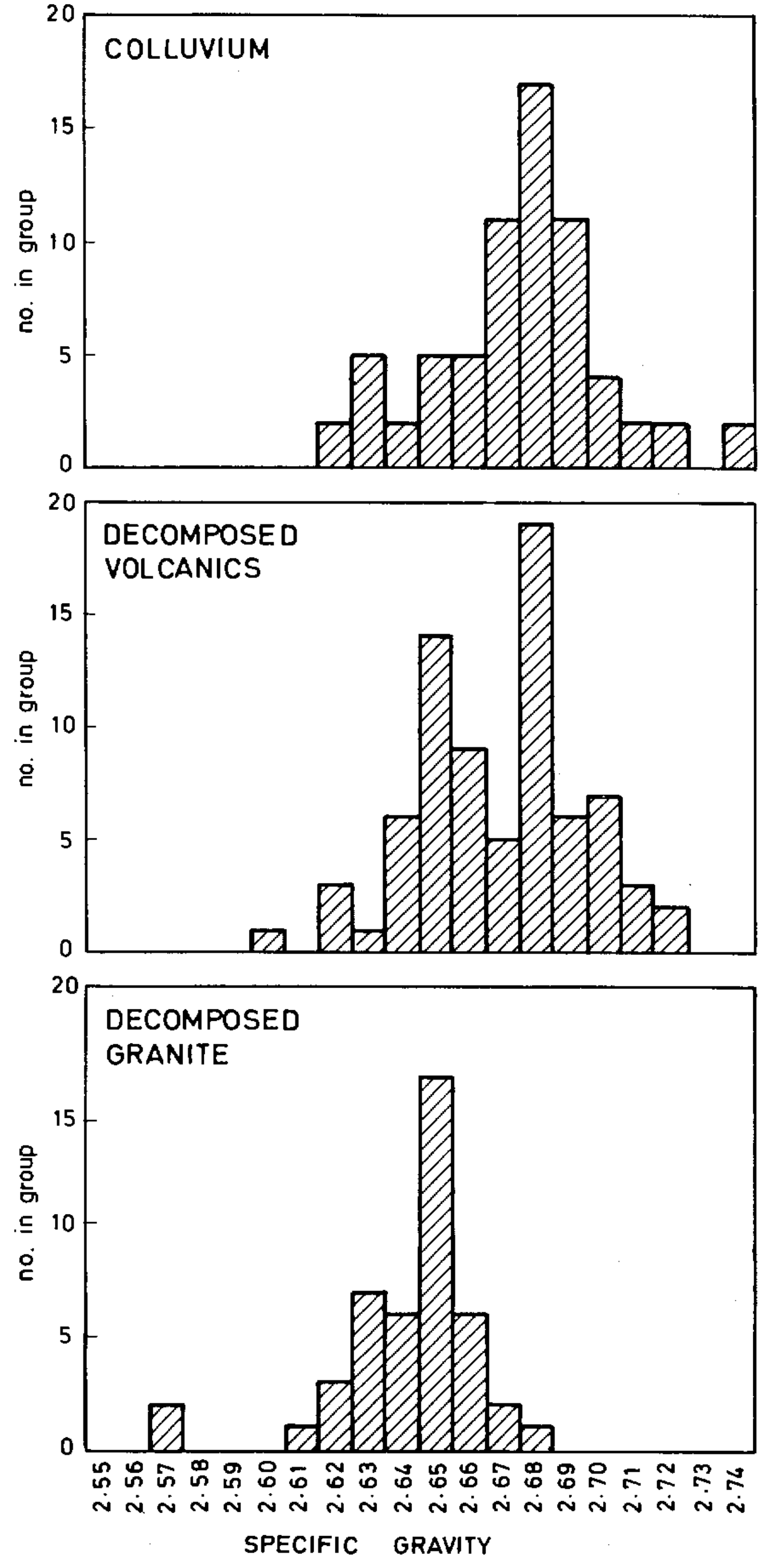
P.S.D. SUMMARY - LABORATORY AND BULK SAMPLES OF C.D.G.	FIGURE 3.9



MID-LEVELS STUDY

DISTRIBUTION OF RECORDED SPECIFIC GRAVITIES

FIGURE 3.10



MID-LEVELS STUDY

4. SHEAR STRENGTH TEST METHODS

4.1 Introduction

The stability of the slopes in Hong Kong is an effective stress stability problem. Consequently testing and analysis has to be in terms of effective stresses rather than total stresses.

The types of tests performed are outlined below and a summary of the test results is given in Appendix A.

4.2 Triaxial Tests to Determine Saturated Shear Strength Parameters

4.2.1 The consolidated undrained test (CU)

Specimens were back pressure saturated maintaining a minimum difference between cell pressure and back pressure of 2 kN/m^2. Saturation was checked by measuring the pore pressure parameter B. If $B \geqslant 0.97$ saturation was taken to be complete and, for calculation purposes, it was assumed no further air remained in the voids. The specimen was next consolidated, by raising the cell pressure, to the desired test effective stress and then sheared undrained at a rate of less than 2% per hour whilst measuring pore water pressure.

A number of two stage tests were performed in which the first shearing stage was stopped at maximum stress ratio ($\sigma'1 / \sigma'3$) or 2% axial strain (whichever occurred first), the axial load removed and the specimen reconsolidated to a higher all round stress before shearing again.

The outcome of such two stage tests was that full shear strength was not always mobilised in the first stage whilst the higher than insitu stresses selected for the second stage caused the specimens to exhibit compressive type failure when sheared.

4.2.2 The consolidated drained test (CD)

The consolidated drained test was the same as the CU test except that the specimen was sheared drained and volume change measured.

Only two of these tests were performed as the CD test was superseded by the wetted drained test.

4.2.3 The consolidated percolated undrained test (CPU)

The consolidated percolated undrained test with pore pressure measurement was adopted to avoid reducing the effective stress during saturation to a low value as in the CU test.

Rather than saturate and then consolidate, as in the CU test, the specimen was consolidated and then saturated. In this way the stress cycling was reduced by maintaining a realistic effective stress during saturation. Back pressure saturation was again employed but this time preceded by percolation to increase the initial moisture content and reduce the time taken to achieve saturation. Measurement of B was still used to indicate attainment of a satisfactory degree of saturation, but, in view of the increased stiffness of the soil skeleton at the higher effective stresses, saturation was halted when $B \geq 0.90$ provided the inflow of water in the final stage was small compared with previous stages. To improve the reliability of calculations the volume change of the specimen was monitored by measuring the flow of water to and from the cell. All tests were single stage.

Attempts to monitor the degree of saturation throughout the test did produce some unreasonable calculated results (i.e. $S > 100\%$) but generally they were reasonable and gave a useful indication as to the progress of the saturation stage. The accuracy of the calculation is dependent upon the accuracy to which measurements of volume change and dimensions are taken, and the value of specific gravity used.

4.2.4 The wetted drained test (WD)

The wetted drained test was performed in order to simulate more closely the field saturation process. The specimen was consolidated to an all round total stress approximately equal to the in situ overburden stress and then water was percolated through by means of a small head difference between burettes, one connected to each end of the sample. Percolation was taken to be complete, and hence field 'saturation' achieved, when the change in inflow and outflow volumes became equal. The specimen was then sheared drained to atmospheric pressure whilst measuring inflow and/or outflow of water . The total volume change of the specimen was measured via the cell fluid. The degree of saturation after percolation, where calculated was typically 85% or greater.

4.2.5 The dead load test (DL)

The dead load test is a stress controlled test that closely simulates the behaviour during failure of soil slopes caused by rising groundwater. This is achieved by maintaining a system of constant total anisotropic stresses on a specimen and increasing the pore water pressure to cause failure. The required initial stresses were calculated assuming an infinite slope failure plane at the depth of the sample.

The specimens were consolidated anisotropically, but not under K_o conditions, then percolated and finally sheared by increasing the pore water pressure. The rate of increase of pore water pressure was controlled by monitoring the pore pressure at the other end of the specimen and adjusting the rate of pore pressure change so that the difference between the pressure at each end of the specimen was less than 2 kPa. It was necessary to use a hanger system for the axial load such that the deviator stress did not vary unacceptably.

By monitoring, using transducers, pore water pressure at both ends of the specimens, axial load, axial strain and volumetric strain it was possible to follow the behaviour of the specimens throughout shearing. Since the stress path of the shearing stage of the test is horizontal the low stress range can be investigated using this test. However, it is not possible to go beyond the tension cut-off (i.e. $\sigma'_3 = 0$).

Sheared at their approximate insitu stresses specimens failed with constant or decreasing pore water pressure as inflow of water continued. This indicates dilative failure rather than compressive.

5. SHEAR STRENGTH TEST RESULTS

5.1 Results of Triaxial Tests to Determine Saturated Shear Strength

5.1.1 Analysis of results

The results of each of the triaxial tests have been analysed and relevant details recorded in the summary tables in Appendix A.

Since the field conditions are expected to be 'drained', as opposed to 'undrained', failure has been defined as occurring when the maximum obliquity (of the stress path) has been reached. (see Figure 5.1)

i.e. failure = $\frac{q^1}{p^1}$ max

where $q' = \frac{1}{2}(\sigma'_1 - \sigma_3')$, $p' = \frac{1}{2}(\sigma_1' + \sigma_3')$

This is equivalent to maximum stress ratio, (σ_1'/σ_3') max

The mode of failure has been determined from the slope of the stress path at failure.

The summary tables also show other details of the triaxial test such as the dry density and, where measured, the degree of saturation and pore pressure parameter B at the start of the shearing stage.

5.1.2 Colluvium

The failure stresses from triaxial tests to determine the saturated shear strength of colluvium are shown in Figure 5.2. A distinction is made between the type of test performed and also the class of colluvium. From the results presented there is no discernible difference in shear strength beween the classes, all lying within the same scatter. The few results from class 1 colluvium do however tend towards the higher values. The class 1 colluvium specimens contained highly to completely weathered granite boulders as well as matrix material which would account for the higher shear strength values as all other tests were carried out on specimens of matrix only.

A 'least squares' line for the failure stresses yields the following shear strength equation:

$$\tau(kN/m^2) = 4.4 + \sigma_n' \cdot \tan 34.3°$$

Overall, DL tests gave shear strengths slightly above those from other types of test whilst the CU tests were generally lower and it is of interest to note that DL tests alone yield the shear strength equation

$$\tau(kN/m^2) = 9 + \sigma_n' \cdot \tan 35°$$

One of the DL test results, that from a specimen from ML53B is surprisingly high. The pore water pressure exceeded the cell pressure during this test and the results may be unreliable. Although the seven WD tests performed on colluvium showed a wide scatter of results those on class 2 colluvium showed higher shear strengths than those on class 3 colluvium.

5.1.3 Decomposed volcanics

Failure stresses from triaxial tests on decomposed volcanics are shown in Figure 5.3.

The results are scattered reflecting varying degrees of decomposition of the specimens. The 'least squares' straight line yields the following shear strength equation

$$(\mathrm{kN/m^2}) = 7.4 + \sigma_n' \cdot \tan 35.7^\circ$$

A comparison of test types is difficult since all show the same magnitude of scatter. However the majority of DL tests gave results that tended towards the upper limit of the range although the 'least squares' line yields the equation

$$(\mathrm{kN/m^2}) = 13 + \sigma_n' \cdot \tan 32^\circ$$

5.1.4 Decomposed granite

The failure stresses of the saturated tests on decomposed granite are shown in Figure 5.4.

The scatter in the results is larger than that of the results for the other material types and reflects the greater variability in decomposed granite. The 'least squares' line for all the points shown yields the shear strength equation

$$(\mathrm{kN/m^2}) = 5.3 + \sigma_n' \cdot \tan 41.5^\circ$$

The variation in the results can be explained by consideration of the degree of decomposition of the feldspars of the decomposed granite specimens tested. Those from ML107, ML53B and TP95 were least decomposed with the feldspars still generally intact. Samples from TP98 and TP6 OAP contained feldspars which were softer and more decomposed. Some discolouration had taken place where feldspars had decomposed to clay minerals, mainly halloysite (see Geology Report). However the majority of the feldspars in the samples from ML122A had formed clay minerals but retained the original granite structure. The variation in initial dry density of decomposed granite samples is likewise related to the state of decomposition of the feldspars. However, no relationship between dry density and shear strength has been found from the limited testing.

By separating the decomposed granite results according to the observed degree of feldspar decomposition and for each group assessing the best line to fit the failure stresses the following shear strength equations have been obtained.

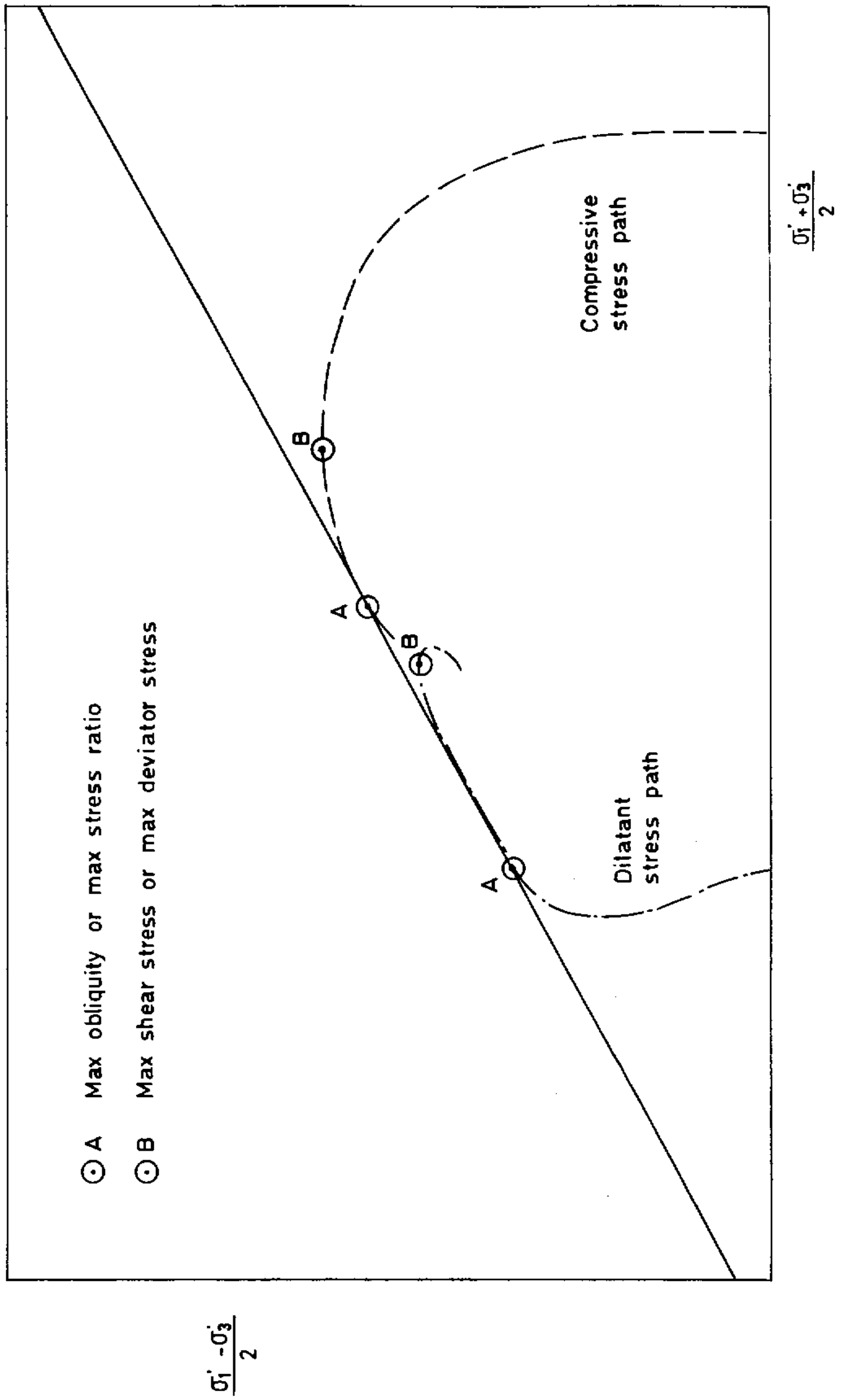
Location	Equation	
ML107/TP95/ ML53B	$\tau\ (kN/m^2) = 14 + \sigma_n' \cdot \tan 44°$	increasing
		degree of
TP98/TP6 OAP	$\tau\ (kN/m^2) = 12 + \sigma_n' \cdot \tan 37.5°$	decomposition of feldspars
ML122A	$\tau\ (kN/m^2) = \sigma_n' \cdot \tan 37°$	↓

For comparison the failure lines corresponding to these shear strength equations are shown in Figure 5.5.

DL tests again gave results that were higher than those from other tests. Those performed on specimens from TP98 reached the tension cut off (i.e. $\sigma_3' = 0$) and had to be failed by increasing the deviator stress. This demonstrates the existence of some shear strength even when $\sigma_3' = 0$.

DEFINITION OF TERMS USED FOR STRESS PATHS

FIGURE 5.1

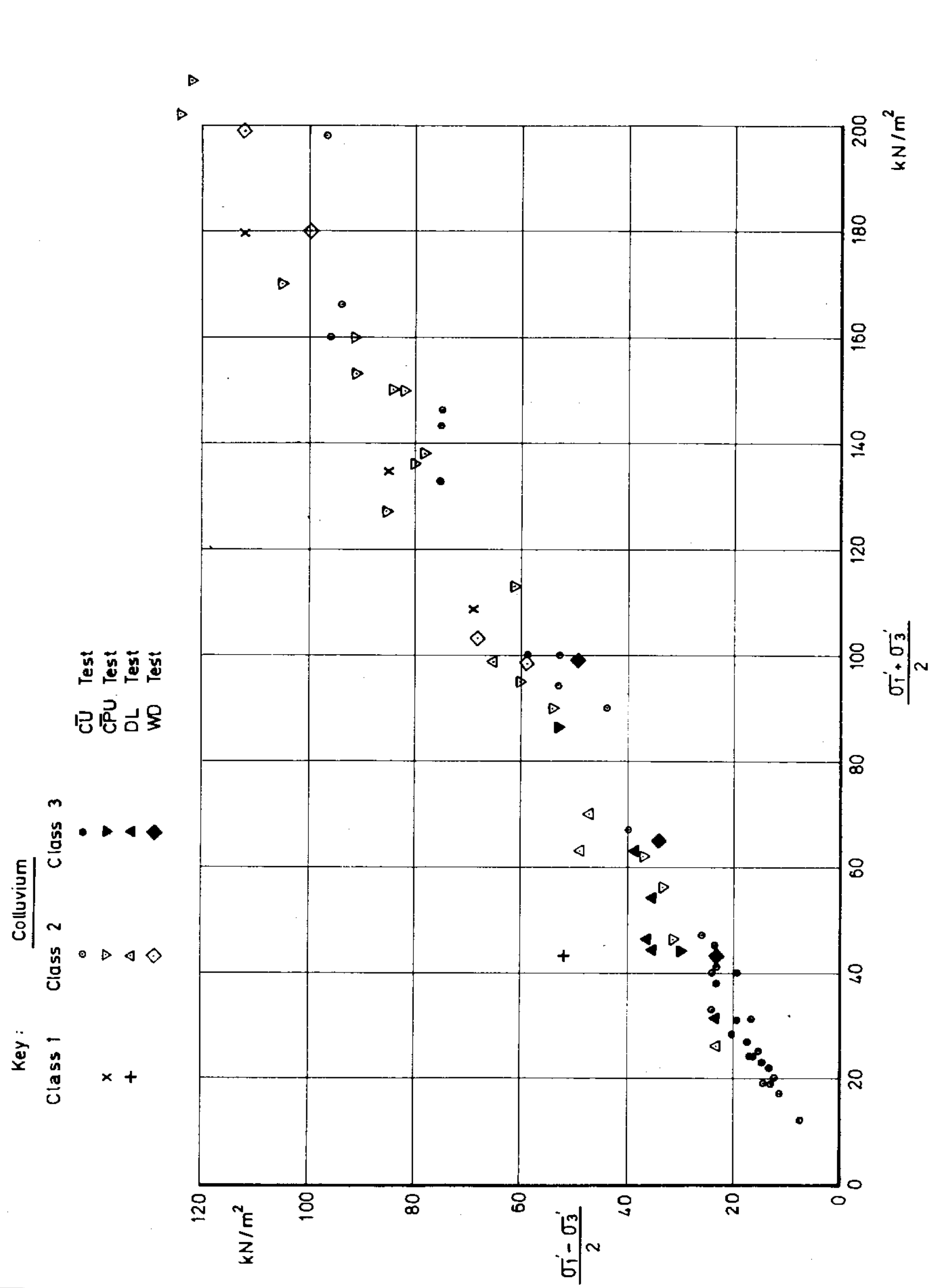


MID-LEVELS STUDY

SUMMARY OF TRIAXIAL TEST FAILURE STRESSES

COLLUVIUM

FIGURE 5.2

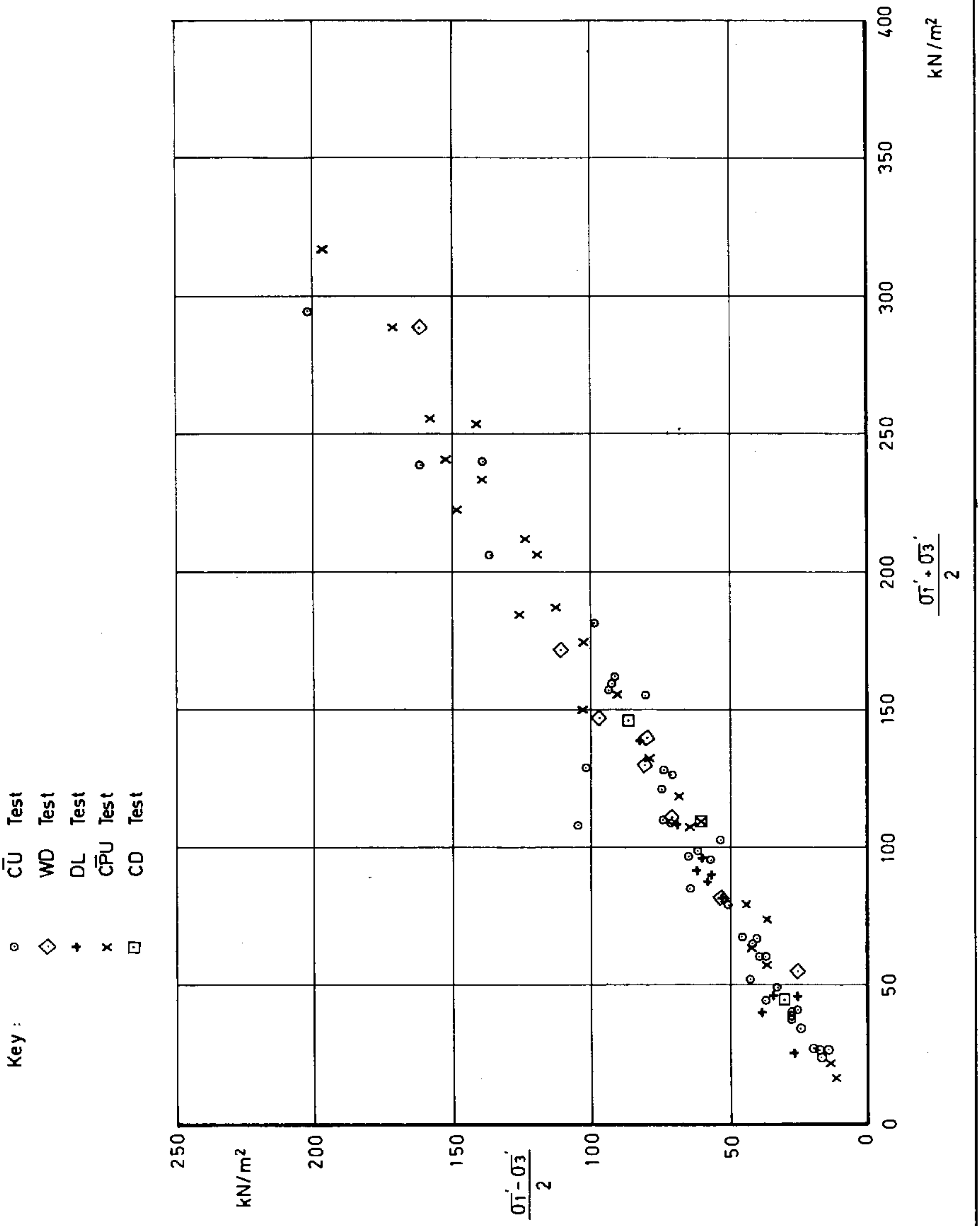


MID-LEVELS STUDY

SUMMARY OF TRIAXIAL TEST FAILURE STRESSES

DECOMPOSED VOLCANICS

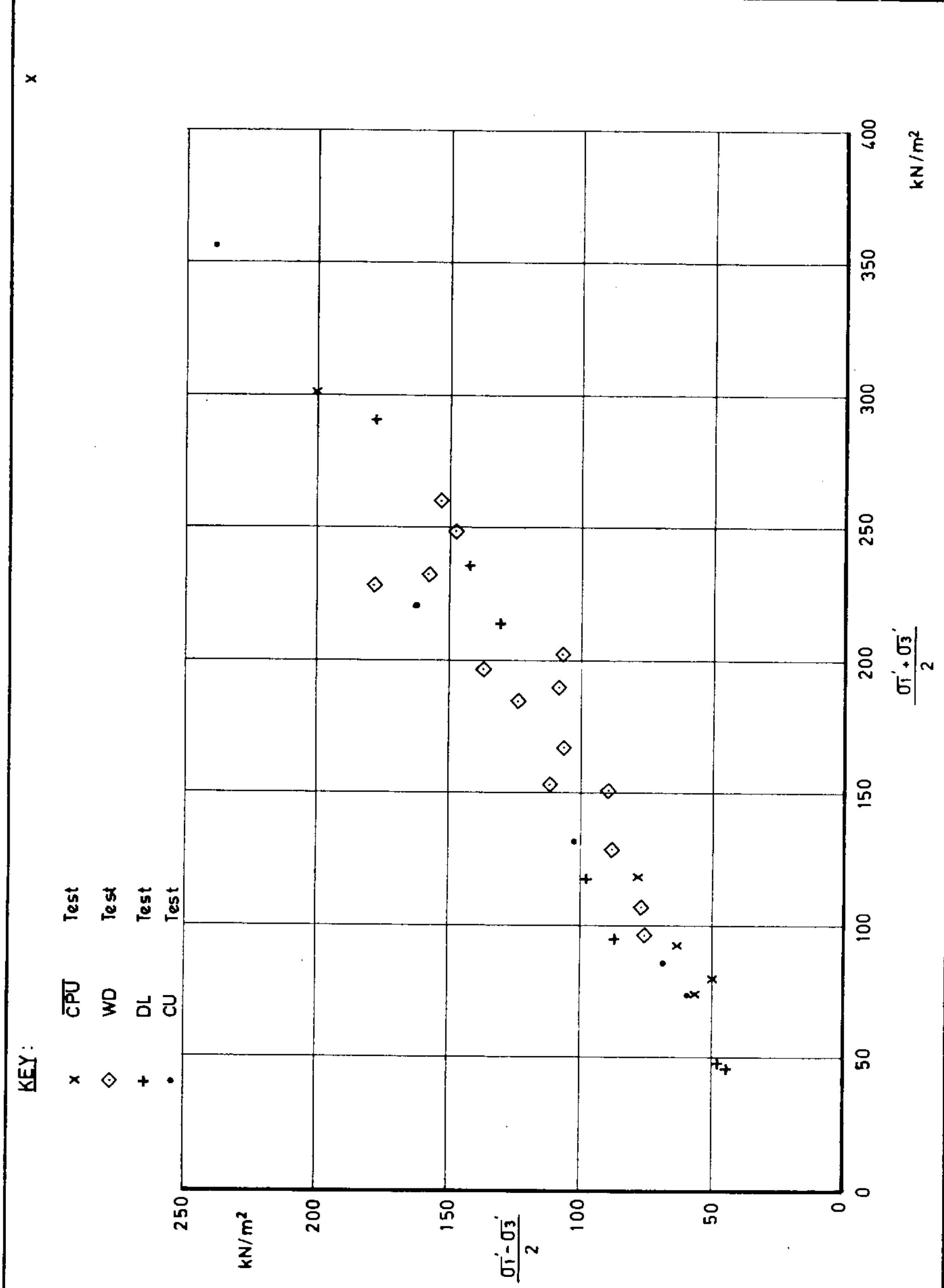
FIGURE 5.3



MID-LEVELS STUDY

SUMMARY OF TRIAXIAL TEST FAILURE STRESSES - C.D.G.

FIGURE 5.4

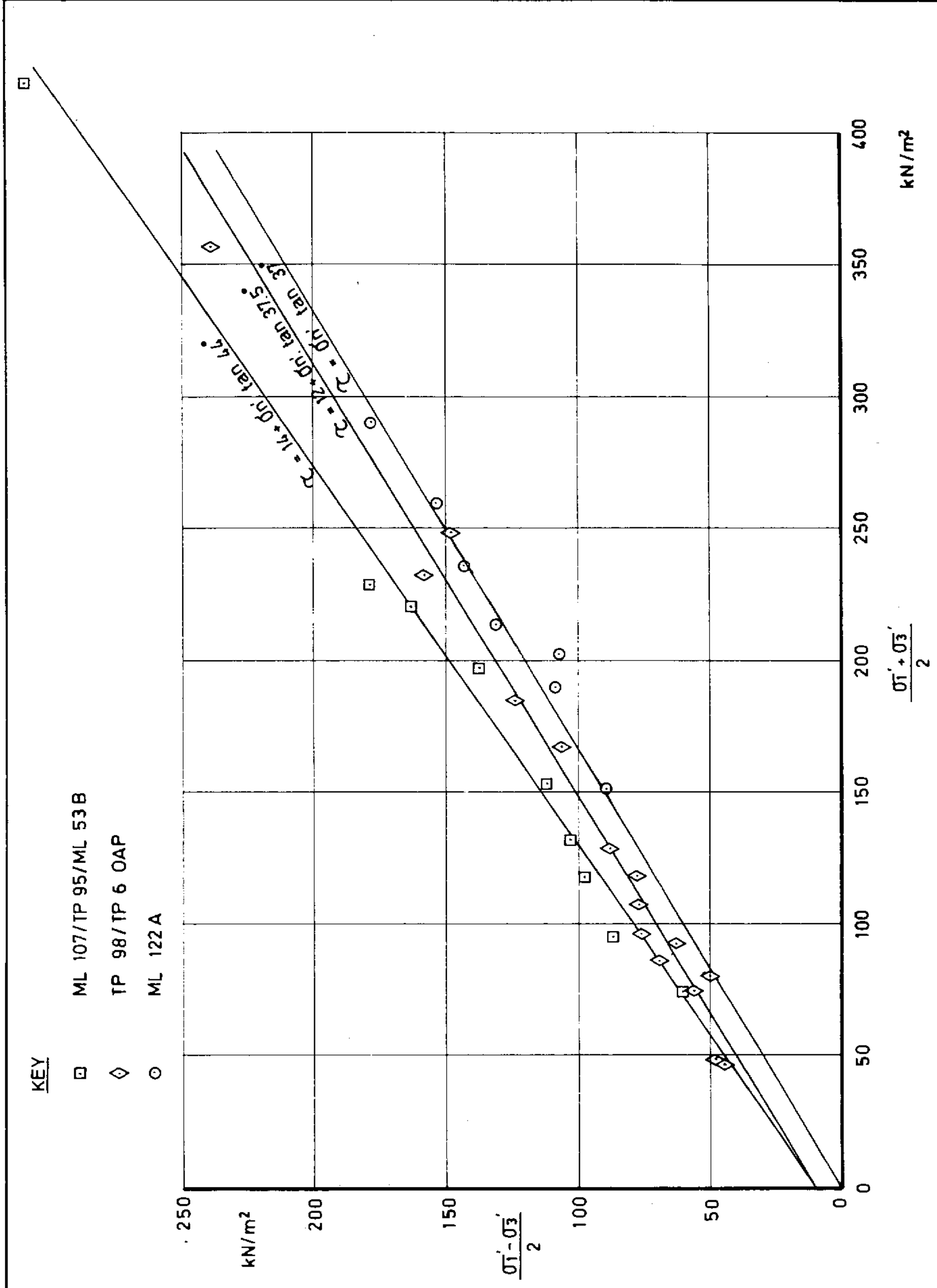


MID-LEVELS STUDY

SUMMARY OF TRIAXIAL TEST FAILURE STRESSES.

C.D.G.
(by feldspar weathering)

FIGURE 5.5



MID-LEVELS STUDY

APPENDIX A

SUMMARY OF TRIAXIAL TEST RESULTS

SUMMARY OF LABORATORY TEST RESULTS

TABLE A1.1

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c. %	γ_d kN/m^3	S %	PSD type	% clay	% silt	% sand	% gravel	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press kN/m^2	(†) app dev kN/m^2	(††) $-u_w$ kN/m^2	γ_d kN/m^3	S calc %	B	AT FAILURE p'_f kN/m^2	q'_f kN/m^2	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP2 OAP	T1	0.3	COLLUVIUM/VC4GT/3	16.0	15.5	64						$\overline{CU}$	76	15	-	-	-	-	.98	23	14	.609	1.9	D	1.9	1ST STAGE
												$\overline{CU}$	76	250	-	-	-	-	-	133	75	.564	11.9	C/D	2.8	2ND STAGE STRESS LEVEL TOO HIGH
TP2 OAP	T2	0.5	COLLUVIUM/VC4GT/3	22.0	13.5	62						WD	76	50	-	-	-	79	-	99	49	.495	8.9	C	8.9	VOL. CHANGE DURING PERCOLATION NOT RECORDED
TP4B	1/1	2.3-2.6	COLLUVIUM/VD1T/2	17.9	14.5	60	h	29	39	27	5	$\overline{CU}$	76	15	-	-	14.5	-	1.0	20*	12*	.600	0.7	D	0.7	1ST STAGE FAILURE REACHED?
												$\overline{CU}$	76	100	-	-	15.4	-	-	45	23	.511	3.4	C	1.4	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	1/2	2.3-2.6	COLLUVIUM/VD1T/2	16.0	15.7	63	h	-	55	20	25	$\overline{CU}$	76	15	-	-	16.6	-	.99	19	13	.684	1.8	D	19.1	
TP4B	1/3	2.3-2.6	COLLUVIUM/VD1T/2	16.0	15.6	64	h	24	34	25	17	$\overline{CU}$	76	200	-	-	15.9	-	.97	100	59	.590	5.4	C	1.9	STRESS LEVEL TOO HIGH
TP4B	1/4	2.3-2.6	COLLUVIUM/VD1T/2	18.6	15.1	68	h	25	26	23	24	$\overline{CU}$	76	60	-	-	15.5	-	.99	47*	26*	.553*	1.6	C	1.6	1ST STAGE *FAILURE NOT REACHED
												$\overline{CU}$	76	200	-	-	16.4	-	-	94	53	.564	11.0	C	2.1	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	1/7	2.3-2.6	COLLUVIUM/VD1T/2	19.1	15.0	69	h	32	36	26	6	$\overline{CU}$	76	30	-	-	14.7	-	.99	31*	16*	.516*	0.5	C	0.5	1ST STAGE *FAILURE NOT REACHED
												$\overline{CU}$	76	150	-	-	15.6	-	-	90	44	.489	3.4	C	1.7	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	1/A	2.3-2.6	COLLUVIUM/VD1T/2	15.0	14.3	49						$\overline{CU}$	76	15	-	-	15.0	-	.99	12	7	.583	1.9	C	1.9	1ST STAGE HORIZONTAL SPECIMEN
												$\overline{CU}$	76	100	-	-	15.8	-	-	41	23	.561	4.8	C	1.2	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	1/B	2.3-2.6	COLLUVIUM/VD1T/2	11.4	14.7	39						$\overline{CU}$	76	30	-	-	15.8	-	.98	27*	17*	.630*	1.5	C	1.5	1ST STAGE HORIZONTAL SPECIMEN *FAILURE NOT REACHED
												$\overline{CU}$	76	150	-	-	16.7	-	-	67	40	.597	8.7	C	2.4	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	1/C	2.3-2.6	COLLUVIUM/VD1T/2	15.1	14.1	48						$\overline{CU}$	76	60	-	-	15.5	-	.98	40	19	.475	1.6	C	1.6	1ST STAGE HORIZONTAL SPECIMEN
												$\overline{CU}$	76	200	-	-	16.3	-	-	100	53	.530	9.2	C	2.1	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	2/4	2.85-3.05	COLLUVIUM/VD1T/2	18.7	15.0	69						$\overline{CU}$	75	15	-	-	15.3	-	.97	25	15	.600	1.3	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	360	-	-	16.2	-	-	188	97	.516	4.8	C	1.7	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	2/5	2.85-3.05	COLLUVIUM/VD1T/2	18.3	15.3	70						$\overline{CU}$	76	60	-	-	15.3	-	.97	40	24	.600	2.2	C	2.2	1ST STAGE

KEY:

P.S.D. type
- h – oven dried
- a – air dried
- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure

ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive (A > 1/3 at failure)
- D = dilatant (A ≤ 1/3 at failure)

failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A1.2

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS: m.c. %	γ_d kN/m^3	S %	PSD: type	% clay	% silt	% sand	% M gravel	TRIAXIAL: Test	BEFORE SHEARING: dia. mm	Eff. cell. press. kN/m^2	(†) app. dev. kN/m^2	(††) $-u_w$ kN/m^2	γ_d kN/m^3	S calc %	B	AT FAILURE: p'_f kN/m^2	q'_f kN/m^2	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP4B	2/5	2.85-3.05	COLLUVIUM/VD1T/2									$\overline{CU}$	76	350	-	-	16.0	-	*	231	115	.498	6.9	C	1.9	2ND STAGE STRESS LEVEL TOO HIGH
TP4B	2/7	2.85-3.05	COLLUVIUM/VD1T/2	19.2	14.9	69						$\overline{CU}$	76	30	-	-	14.6	-	.97	24*	16*	.678	2.0	C	2.0	1ST STAGE *FAILURE NOT REACHED
												$\overline{CU}$	76	310	-	-	16.0	-	-	143	75	.524	9.4	C	1.8	2ND STAGE STAGE STRESS LEVEL TOO HIGH
TP4B	2/8	2.85-3.05	COLLUVIUM/VD1T/2	18.5	14.3	61						$\overline{CU}$	76	310	-	-	16.1	-	.99	146	75	.514	7.1	C	2.4	STRESS LEVEL TOO HIGH
TP4B	3/4	3.05-3.35	COLLUVIUM/VD1T/2	21.0	14.6	70	h	34	36	25	5															
TP4B	3/9	3.05-3.35	COLLUVIUM/VD1T/2	-	-	-	s	22	35	30	13	DL	76	98	44	-	-	-	-	26	23	.885	2.5	D	-	
TP4H	T1	1.1	COLLUVIUM/VC1T/2	21.2	14.6	71	h	25	22	48	5	$\overline{CU}$	76	15	-	-	14.9	-	.98	19	14	.737	1.31	D	3.1	1ST STAGE
												$\overline{CU}$	76	200	-	-	15.7	-	-	166	94	.566	6.1	C/D	11.1	2ND STAGE STRESS LEVEL TOO HIGH
TP7 OAP	2/2	1.6m	COLLUVIUM/VC2GT/2	24	13.9	73						$\overline{CU}$	76	15	-	-	13.3	-	.98	17	11	.647	2.5	D	2.5	1ST STAGE FAILURE NOT REACHED
												$\overline{CU}$	76	200	-	-	14.4	-	-	94	53	.564	6.2	C	2.1	2ND STAGE STRESS LEVEL TOO HGIH
TP31	D/1	2.5-2.8	COLLUVIUM/VD1T/2	27.6	13.4	77	s	40	32	23	5	WD	76	40	-	-	14.0	-	-	99	59	.596	14.3	C	14.3	
TP31	D/2	2.5-2.8	COLLUVIUM/VD1T/2	27.3	13.5	78	s	36	42	17	5															
TP31	D/3	2.5-2.8	COLLUVIUM/VD1T/2	28.0	13.0	74																				
TP31	D/4	2.5-2.8	COLLUVIUM/VD1T/2	28.1	13.4	79	s	37	35	23	5															
TP31	2Y2	3.5-3.8	COLLUVIUM/VD1T/2	27.9	13.3	78	s	33	39	24	4															
TP31	2Y3	3.5-3.8	COLLUVIUM/VD1T/2	29.2	13.4	82	s	35	37	23	5															
TP74	1/1	1.7-2.0	COLLUVIUM/VC2GT/3	18.6	13.7	54						$\overline{CU}$	76	30	-	-	15.0	-	.93	22	16	.727	14.2	C	0.7	
TP74	1/4	1.7-2.0	COLLUVIUM/VC2GT/3	19.3	14.1	60						$\overline{CU}$	76	60	-	-	14.7	-	.90	31	19	.613	7.7	C	1.4	
TP74	1/7	1.7-2.0	COLLUVIUM/VC2GT/3	18.0	13.9	55	h	25	27	30	18	WD	76	37	-	-	-	-	*	65	34	.523	13.1	C	13.1	
TP74	1/8	1.7-2.0	COLLUVIUM/VC2GT/3	19.7	13.6	57	h	27	16	35	22	$\overline{CU}$	76	45	-	-	14.8	-	.98	22	13	.591	6.1	C	0.9	

KEY:

P.S.D. type
- h – oven dried
- a – air dried
- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

- ε_f = axial strain at failure
- ε_m = axial strain at max dev. stress

Mode (of failure)
- C – compressive (A > ⅓ at failure)
- D – dilatant (A ≤ ⅓ at failure)
- failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A1.3

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c. %	γ_d kN/m³	S %	PSD type	PSD clay %	PSD silt %	PSD sand %	PSD M gravel %	TRIAXIAL Test	dia. mm	BEFORE SHEARING Eff. cell press. kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP74	1/9	1.7-2.0	COLLUVIUM/VC2GT/3	18.1	13.7	53	a	22	35	29	14	$\overline{\text{CU}}$	76	90	-	-	14.7	-	.92	38	23	.605	11.0	C	1.7	
TP74	T3	1.8	COLLUVIUM/VC2GT/3	16.0	16.1	66	h	24	21	32	23	DL	76	65	40	-	18.1	91	-	31	23	.742	0.5	D	-	
TP76A	T3	3.0	COLLUVIUM/VC3GT/3	13.9	15.2	51						DL	76	65	70	-	16.6	98	-	54	35	.648	1.7	D	-	
TP82	4/2	0.8-1.1	COLLUVIUM/VC1ST/3	13.1	15.1	49						$\overline{\text{CPU}}$	76	40	-	-	15.9	-	.92	44	30	.682	1.10	C	0.6	
TP82	4/3	0.8-1.1	COLLUVIUM/VC1ST/3	14.3	13.3	39	h	31	22	30	17	WD	76	20	-	-	14.0	82	-	43	23	.535	3.0	C	3.0	
TP82	4/7	0.8-1.1	COLLUVIUM/VC1ST/3	19.8	13.8	61	h	29	21	31	19	$\overline{\text{CPU}}$	76	20	-	-	15.0	-	1.00	28	20	.714	0.6	C	0.6	INITIAL GAUGE PWP RECORDED IS 10kN/m² TOO HIGH CORRECTED P' SHOWN
TP82	4/8	0.8-1.1	COLLUVIUM/VC1ST/3	17.1	14.4	57	a	15	26	42	17	DL	76	70	71*	-	15.8	97	-	46	36*	.783	1.0	C/D	-	PERFORMED WITHOUT LOADCELL *SUSPECT DUE TO RAM FRICTION
TP82	4/9	0.8-1.1	COLLUVIUM/VC1ST/3	15.6	14.3	51						DL	76	70	76	-	15.7	90	-	63	38	.603	0.8	C	-	
TP96	T2	2.4m	COLLUVIUM/VBC1GT/2	14.8	15.7	59																				
ML5D		4.7-5.0	COLLUVIUM/VBD3GT/2	19.0	16.6	85	h	9	21	30	40	$\overline{\text{CPU}}$	102	80	-	-	17.4	-	.90	127	85	.669	1.4	D	15.2	MEMBRANE SPLIT DURING TEST TEST RE-RUN(SHOWN)
ML5E		4.9-5.1	COLLUVIUM/VBD3GT/2	26.6	14.2	84	h	33	23	30	14	WD	104	90	-	-	15.1	92	-	199	112	.563	11.9	C	11.9	
ML31F		1.8-2.0	COLLUVIUM/VC1GT/2	18.7	16.3	82						$\overline{\text{CU}}$	102	15	-	-	16.7	-	.99	33	24	.727	1.1	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{\text{CU}}$	102	250	-	-	17.3	-	-	181	113	.624	3.9	C/D	-	2ND STAGE MAX. SHEAR STRESS NOT REACHED STRESS LEVEL TOO HIGH
ML31F		1.06-1.28	COLLUVIUM/VC1GT/2	19.9	15.8	81						$\overline{\text{CU}}$	102	15	-	-	16.5	-	.97	24	16	.667	2.2	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{\text{CU}}$	102	288	-	-	17.4	-	-	160	96	.600	13.1	C/D	-	2ND STAGE MAX. SHEAR STRESS NOT REACHED STRESS LEVEL TOO HIGH
ML53B	7	4.9-5.2	COLLUVIUM/VGD4T/1	28.1	14.5	92	a	18	25	35	22	$\overline{\text{CPU}}$	103	75	-	-	15.0	101	.92	135	85	.630	1.9	D	11.8	
ML53B	8	5.2-5.8	COLLUVIUM/VGD4T/1	27.7	14.8	97						DL	103	50	105	-	15.6	105	-	43*	52	1.21	0.7	-	-	*PWP > CELL PRESSURE
ML53B	9	5.9-6.3	COLLUVIUM/VGD4T/1	30.5	14.0	95						$\overline{\text{CPU}}$	103	110	-	-	14.5	99	.92	109	69	.633	1.7	D	15.3	
ML53B	10	6.6-6.8	COLLUVIUM/VGD4T/1	25.3	14.9	91						$\overline{\text{CPU}}$	103	150	-	-	15.6	101	.92	180	112	.622	3.3	D	18.1	
ML56	1	1.8-2.0	COLLUVIUM/VC1ST/2	22.0	16.1	93	a	15	29	33	23	$\overline{\text{CPU}}$	102	35	-	-	16.7	104	-	56	33	.589	2.3	D	8.3	

KEY:

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- $\overline{\text{CU}}$ - consolidated undrained with pwp
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- CD - consolidated drained
- WD - percolated drained
- DL - dead load
- SC - suction conditioned
- CWC - constant water content

Symbols
- γ_d - unit dry weight
- S - degree of saturation
- B - pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure

ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive (A > ⅓ at failure)
- D = dilatant (A ≤ ⅓ at failure)
- failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A1.4

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	clay %	silt %	sand %	gravel %	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
ML56	6	3.9-4.1	COLLUVIUM/VC1ST/2	16	15.9	66						CPU	104	70	-	-	16.5	92	-	62	37	.597	9.9	C	9.9	
ML112		6.4	COLLUVIUM/VBD1GT/2	19.5	16.6	89						CPU	102	120	-	-	-	-	.92	136	80	.588	2.7	D	-	MAX. SHEAR STRESS NOT REACHED
ML112		6.7	EOLLUVIUM/VBD1GT/2	22.9	15.8	91	h	18	27	38	17	CPU	103	100	-	-	-	100	.94	95	60	.632	2.86	D	13.8	
ML112		7.75	COLLUVIUM/VBD1GT/2	23.4	15.6	92	h	20	27	41	12	CPU	103	150	-	-	16.3	-	.96	138	78	.565	2.5	D	5.0	
ML112		8.90	COLLUVIUM/VCD23ST/2	23.6	14.8	83	h	19	16	38	27	CPU	103	175	-	-	15.6	-	.90	153	91	.595	4.5	D	11.9	
ML112		9.4-9.9	COLLUVIUM/VCD23ST/2	27.8	14.1	88	h	26	22	36	16															
ML112		11.6	COLLUVIUM/VCD23ST/2	22.8	15.1	82	h	15	21	41	23	CPU	102	200	-	-	16.4	-	.80	150	84	.560	4.0	C	3.4	
ML112		11.7-12.3	COLLUVIUM/VCD23ST/2	20.5	14.4	67	h	8	14	62	16															
ML112		12.1-12.3	COLLUVIUM/VCD23ST/2	21.8	15.1	78	h	5	20	38	37															
ML112		14.4-14.6	COLLUVIUM/VCD23ST/2	20.9	15.4	79	h	15	22	45	18	CPU	103	225	-	-	16.2	-	.88	202	124	.614	3.4	C/D	6.7	
ML122	-	7.0	COLLUVIUM/VCD23ST/2	16.1	16.5	73	s	10	28	24	25	DL	103	55	98	-	17.2	97	-	63*	49	.778	1.2	-	-	SUSPECT VOLUME CHANGE READINGS *ASSESED TRUE VALUE
ML122	12	8.0	COLLUVIUM/VCD23ST/2	14.4	17.1	72						DL	103	74	124	-	17.7	90	-	99	65	.657	1.2	-	-	
ML122	14	8.8	COLLUVIUM/VCD23ST/2	16.2	16.6	75	s	14	32	39	15	DL	103.5	55	96	-	17.1	87	-	70	47	.671	1.5	-	-	
ML122		10.2-10.4	COLLUVIUM/VCD23ST/2	21.5	16.1	91	s	17	27	39	17	CPU	101.	175	-	-	17.2	99.8	1.0	160	91	.569	5.0	D	12.8	
ML122	18	11.0	COLLUVIUM/VCD23ST/2	27.1	15.5	103	s	8	31	44	17	CPU	108	198	-	-	16.7	117	.90	150	82	.547	4.5	D	14.1	
ML122	18	11.12-11.32	COLLUVIUM/VCD23ST/2	21.2	16.7	99	s	16	28	41	15	WD	103	80	-	-	17.6	104	-	180	100	.556	7.1	C	7.15	
ML122	19	11.48	COLLVUIUM/VCD23ST/2	19.5	16.8	96	s	8	40	47	5	CPU	103	200	-	-	17.8	112	.98	208	122	.587	3.8	D	12.9	COBBLE IN SPECIMEN
K1	Ta	2.16	COLLUVIUM/VBD4GT/2	15.9	16.6	71	h	27	19	41	13	WD	76	35	-	-	17.4	96.5	-	103	68	.660	2.7	D	2.7	
K1	Tb	2.1	COLLUVIUM/VBD4GT/2	14.7	15.5	55	h	23	16	26	35	CPU	76	35	-	-	16.1	97	.98	46	31	.674	4.0	D	19.6	
K1	Td	3.9	COLLUVIUM/VBD4GT/2	19.6	16.4	86	h	35	0	57	8	CPU	76	80	-	-	16.4	-	.94	90	54	.600	2.1	D	9.2	INSUFFICIENT VOL. CHANGE READINGS TO CALC S

KEY:

P.S.D. type
- h – oven dried
- a – air dried
- s – dispersion

Test
- CU – consolidated undrained with pwp
- CPU – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive (A > 1/3 at failure)
- D = dilatant (A ≤ 1/3 at failure)

failure = max. obliquity

(†) applied deviator stress for dead load test
(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A1.5

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	PSD clay %	PSD silt %	PSD sand %	PSD M gravel %	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
K1	Te	4.4	COLLUVIUM/VBD4GT/2	20.5	16.1	86	h	27	21	34	18	$\overline{CPU}$	76	90	-	-	16.0	-	.96	113	61	.540	5.3	D	13.2	INSUFFICIENT VOL. CHANGE READINGS TO CALC S
K1	T9	5.26	COLLUVIUM/VBD4GT/2	21.6	16.0	89	h	27	20	41	12	$\overline{CPU}$	76	105	-	-	16.7	-	1.00	170	105	.618	0.5	D	9.7	INSUFFICIENT VOL. CHANGE READINGS TO CALC S
K4	T2	2.3	COLLVUIUM/VB3GT/3	15.9	17.0	78	h	22	11	19	48	DL	76	71	70	-	13.3	90	-	44	35	.795	0.3	-	-	
K4	T6	5.3	COLLUVIUM/VB3GT/3	23.2	15.1	85	h	25	35	32	8															
K3	T7	5.3	COLLUVIUM/VB3GT/3	22.7	15.0	80	h	28	32	28	12	$\overline{CPU}$	76	100	-	-	16.9	-	.98	86	53	.616	3.1	D	16.1	

KEY: P.S.D. type
- h – oven dried
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- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive ($A > \frac{1}{3}$ at failure)
- D = dilatant ($A \leq \frac{1}{3}$ at failure)
- failure = max. obliquity

(†) applied deviator stress for dead load test
(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A2.1

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c. %	γ_d kN/m³	S %	PSD type	% clay	% silt	% sand	% M gravel	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP3H	1/1	1.8-2.2	D.V.	21.1	14.9	75	h	6	42	46	6	$\overline{CU}$	76	15	-	-	15.3	-	.97	38	27	.706	1.1	D	-	MAX. SHEAR STRESS NOT REACHED
TP3H	1/2	1.8-2.2	D.V.	22.0	14.4	71	h	8	45	42	5	$\overline{CU}$	76	200	-	-	15.8	-	.98	154	79	.515	9.1	C/D	9.8	STRESS LEVEL TOO HIGH
TP3H	1/3	1.8-2.2	D.V.	17.8	14.5	59	h	3	47	46	4	$\overline{CU}$	51	15	-	-	14.6	-	.99	40	27	.666	3.8	C/D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	51	100	-	-	15.0	-	-	121	73	.606	3.3	C/D	-	2ND STAGE MAX. SHEAR STRESS NOT REACHED
TP3H	1/6	1.8-2.2	D.V.	21.7	15.4	83	h	5	51	40	4	$\overline{CU}$	76	60	-	-	15.1	-	.99	98	61	.620	2.1	C/D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	200	-	-	15.9	-	-	240	138	.574	4.1	C/D	5.3	2ND STAGE STRESS LEVEL TOO HIGH
TP3H	1/9	1.8-2.2	D.V.	21.5	14.8	76	h	7	48	42	5	$\overline{CU}$	76	30	-	-	14.8	-	.97	41	25	.592	2.9	C/D	4.3	1ST STAGE
												$\overline{CU}$	76	150	-	-	15.5	-	-	126	70	.552	4.8	C/D	5.0	2ND STAGE
TP3H	2/1	2.2-2.6	D.V.	21.5	15.1	78	h	11	35	37	17	$\overline{CU}$	76	200	-	-	16.1	-	.98	128	72	.557	8.8	C	10.1	
TP3H	2/2	2.2-2.6	D.V.	21.5	15.0	77	h	8	40	44	8	$\overline{CU}$	76	15	-	-	13.0	-	.99	23	16	.685	0.7	D	1.7	
TP3H	2/4	2.2-2.6	D.V.	17.0	15.2	63	h	6	40	49	5	$\overline{CU}$	75	15	-	-	15.7	-	.99	26	15	.577	0.8	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	75	100	-	-	16.2	-	-	95	56	.589	4.3	C	3.4	2ND STAGE
TP3H	2/7	2.2-2.6	D.V.	15.0	15.9	64	h	3	30	59	8	$\overline{CU}$	76	60	-	-	16.5	-	.97	85*	64*	.753*	-	C/D	-	1ST STAGE *FAILURE NOT REACHED
												$\overline{CU}$	76	200	-	-	16.9	-	-	238	161	.676	2.1	D	-	2ND STAGE STRESS LEVEL TOO HIGH MAX. SHEAR STRESS ~~NOT REACHED~~
TP3H	3/1	1.8-2.2	D.V.	24.0	15.4	92	h	12	46	38	4	$\overline{CU}$	76	200	-	-	15.9	-	.98	157	93	.589	6.3	C	10.3	
TP3H	3/2	1.8-2.2	D.V.	22.4	16.0	95	h	8	44	43	5	$\overline{CU}$	76	15	-	-	15.5	-	.97	39	27	.685	1.3	D	-	MAX. SHEAR STRESS NOT REACHED
TP3H	3/3	1.8-2.2	D.V.	23.0	15.6	91	h	12	31	50	7	$\overline{CU}$	76	15	-	-	15.6	-	.98	49	32	.652	1.5	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	150	-	-	16.0	-	-	158	92	.582	4.7	C/D	-	2ND STAGE MAX. SHEAR STRESS NOT REACHED
TP3H	3/4	1.8-2.2	D.V.	1.9*	15.5	8						DL	76	84	122	-	16.2	81	-	88	58	.659	1.7	D	-	*DRIED DURING STORAGE
TP3H	3/5	1.8-2.2	D.V.	1.6*	14.8	6	s	13	40	44	3	DL	76	67	72	-	15.5	82	-	47	34	.723	0.3	D	-	*DRIED DURING STORAGE

KEY:

P.S.D. type
- h – oven dried
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- $\overline{CU}$ – consolidated undrained with pwp
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Symbols
- γ_d – unit dry weight
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- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

- ε_f – axial strain at failure
- ε_m – axial strain at max dev. stress

Mode (of failure)
- C – compressive (A > ⅓ at failure)
- D – dilatant (A ≤ ⅓ at failure)
- failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A2.2.

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m^3	S %	PSD type	clay %	silt %	sand %	gravel %	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m^2	(†) app. dev. kN/m^2	(††) $-u_w$ kN/m^2	γ_d kN/m^3	S calc %	B	AT FAILURE p'_f kN/m^2	q'_f kN/m^2	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP3H	3/6	1.8-2.2	D.V.	22.0	15.5	80	h	8	35	49	8	$\overline{CU}$	76	30	-	-	15.7	-	.98	61	36	.598	2.3	D	-	1ST STAGE MAX. SHEAR SHEAR NOT PEACHED
												$\overline{CU}$	76	150	-	-	16.2	-	-	162	91	.562	5.2	C/D	5.2	2ND STAGE STRESS LEVEL TOO HIGH
TP3H	3/9	1.8-2.2	D.V.	23.9	14.8	84	h	10	46	36	8	$\overline{CU}$	76	60	-	-	15.5	-	.97	67	40	.598	2.9	C/D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	200	-	-	16.1	-	-	181	98	.541	4.2	C/D	4.9	2ND STAGE STRESS LEVLE TOO HIGH
TP3H	T1	2.2	D.V.	25.2	15.2	94	-	-	-	-	-															
TP4H	1/1	1.7-2.0	D.V.	22.2	14.4	72	h	36	22	39	3	$\overline{CU}$	76	30	-	-	14.6	-	.98	61	48	.787	1.1	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	100	-	-	15.3	-	-	110	73	.664	2.5	C	1.0	2ND STAGE
TP4H	1/2	1.7-2.0	D.V.	22.3	14.2	71	h	33	21	34	12	$\overline{CU}$	76	150	-	-	14.8	-	.97	67	45	.672	4.2	C	4.2	
TP4H	1/4	1.7-2.0	D.V.	18.0	14.6	60	h	22	20	40	18	$\overline{CU}$	76	15	-	-	15.0	-	.98	45	36	.806	0.4	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	150	-	-	-	-	-	109	71	.651	2.5	C	0.9	2ND STAGE
TP4H	1/6	1.7-2.0	D.V.	22.7	13.7	67	h	37	18	42	7	$\overline{CU}$	76	15	-	-	14.0	-	.98	27	19	.713	2.1	C	4.5	
TP4H	1/7	1.7-2.0	D.V.	20.4	13.9	62	h	38	25	26	11	CD	76	15	-	-	14.4	-	.98	45	30	.667	3.2	D	3.2	
TP4H	1/8	1.7-2.0	D.V.	18.6	14.6	62	h	32	18	30	20	CD	76	60	-	-	14.9	-	.99	146	86	.589	9.9	C	9.9	
TP4H	1/9	1.7-2.0	D.V.	21.2	13.8	63	h	32	24	37	7	$\overline{CU}$	76	60	-	-	14.6	-	.98	65*	42*	.643	-	-	-	TEST STOPPED AT 1.5% AXIAL STRAIN. * FAILURE NOT REACHED
TP4H	2/1	2.2-2.5	D.V.	19.3	14.9	68	h	19	23	53	5	$\overline{CU}$	76	15	-	-	15.0	-	.98	52	42	.794	1.0	D	3.3	
TP4H	2/2	2.2-2.5	D.V.	25.3	12.9	66	h	38	23	34	5	$\overline{CU}$	76	200	-	-	14.3	-	.97	102	53	.520	3.1	C	2.3	STRESS LEVEL TOO HIGH
TP4H	2/3	2.2-2.5	D.V.	18.3	14.2	58	h	23	23	49	5	$\overline{CU}$	76	31	-	-	14.6	-	.98	34	24	.715	1.3	C	1.3	1ST STAGE
												$\overline{CU}$	76	150	-	-	15.1	-	-	97	64	.664	7.7	C/D	10.3	2ND STAGE
TP4H	2/4	2.2-2.5	D.V.	19.6	14.6	65	h	21	22	55	2	$\overline{CU}$	76	15	-	-	14.9	-	.97	38	27	.705	0.5	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	100	-	-	15.2	-	-	79	50	.632	6.9	C	1.2	2ND STAGE

KEY: P.S.D. type

- h – oven dried
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- s – dispersion

Test

- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols

- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure

ε_m = axial strain at max dev. stress

Mode (of failure)

C = compressive (A > ⅓ at failure)

D = dilatant (A ≤ ⅓ at failure)

failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A2.3

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	PSD clay %	PSD silt %	PSD sand %	PSD gravel %	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'	ε_f %	Mode	ε_m %	Comments
TP4H	2/7	2.2-2.5	D.V.	14.6	15.5	57	h	3	29	61	7	$\overline{CU}$	76	60	-	-	16.4	-	.97	129	102	.793	1.7	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	76	200	-	-	16.8	-	-	492	341	.693	3.5	D	7.6	2ND STAGE STRESS LEVEL TOO HIGH
TP4H	3/1	3.0-3.3	D.V.	19.1	15.9	83	h	0	13	51	36	$\overline{CU}$	52	15	-	-	15.8	-	1.00	108	104	.969	0.6	D	-	COBBLE IN SAMPLE MAX. SHEAR STRESS NOT REACHED
TP4H	3/4	3.0-3.3	D.V.	19.1	14.9	57	h	1	20	66	13	$\overline{CU}$	50	182	-	-	15.7	-	.99	293	201	.686	3.3	D	4.9	STRESS LEVEL TOO HIGH
TP7 OAP	T1	3.5	D.V.	10.8	18.3	64	a	21	9	33	17	$\overline{CU}$	75	15	-	-	-	-	.97	26	18	.709	2.2	D	-	1ST STAGE MAX. SHEAR STRESS NOT REACHED
												$\overline{CU}$	75	200	-	-	-	-	-	206	136	.659	7.1	D	11.3	2ND STAGE
TP20	T27	4.0	D.V.	17.6	16.0	74						DL	73	55	52	-	16.1	96	-	36	26	.722	0.5	-	-	
TP20	T34	5.0	D.V.	16.5	16.3	73						$\overline{CPU}$	76	90	-	-	17.2	100	.98	134	79	.590	3.4	D	11.8	
TP20	T35	5.0	D.V.	19.0	16.2	82																				
TP20	T46	5.5	D.V.	18.7	16.2	82						WD	73	30	-	-	17.0	97	-	82	52	.634	4.0	D	4.0	
TP20	T42	6.0	D.V.	18.5	16.5	83						$\overline{CPU}$	73	120	-	-	17.3	98	.92	175	103	.589	5.4	D	20.1	
TP22	T45	7.0	D.V.	17.0	16.8	83						DL	74	85	135		17.5	100	-	92	64	.696	2.1	D	-	
TP22	T46	7.0	D.V.	24.2	15.1	89						DL	73	91	147	-	16.2	100	-	109	69	.633	3.8	D	-	FAILURE OCCURRED AFTER 110 HOURS
TP22	T49	7.5	D.V.	20.0	16.5	91																				
TP22	T50	7.5	D.V.	17.4	15.7	71						WD	74	50	-	-	16.3	95	-	130	80	.615	3.2	D	3.2	
TP25	T25	4.3	D.V.	22.6	15.4	85						WD	77	60	-	-	15.7	91	-	139	79	.568	4.0	D	4.0	
TP25	T26	4.3	D.V.	21.0	16.9	98	a	3	60	29	8															
TP25	T29	4.9	D.V.	24.6	15.2	91	s	3	80	15	2	$\overline{CPU}$	76	80	-	-	*15.8	*110	.96	156	91	.583	2.9	D	12.0	*SUSPECT DUE TO CELL LEAKAGE
TP25	T30	4.0	D.V.	22.5	15.3	85			28	27	7	DL	76	85	124	-	15.7	99	-	96	60	.625	0.5	-	-	
TP25	31/1	4.9-5.1	D.V.	16.2	17.1	79	a	38				$\overline{CPU}$	76	100	-	-	18.1	108	.94	222	148	.667	1.3	D	12.7	

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SC – suction conditioned
CWC – constant water content

Symbols
γ_d – unit dry weight
S – degree of saturation
B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
$p'_f = \frac{1}{2}(\sigma_1' + \sigma_3')$ at failure
$q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress
Mode (of failure)
C = compressive (A > ⅓ at failure)
D = dilatant (A ≤ ⅓ at failure)
failure = max obliquity

(†) applied deviator stress for dead load test
(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A2.4

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS: m.c %	γ_d kN/m³	S %	PSD: type	clay %	silt %	sand %	gravel %	TRIAXIAL: Test	Before shearing: dia. mm	Eff. cell press. kN/m²	(+) app. dev. kN/m²	(++) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	At failure: p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP25	31/2	4.9-5.1	D.V.	19.8	15.6	77	s	7	70	21	2	WD	76	50	-	-	16.0	83	-	147	97	.660	1.1	D	1.1	
TP25	31/3	4.9-5.1	D.V.	17.0	16.4	75	s	7	67	23	3	$\overline{CPU}$	76	120	-	-	*17.3	*107	.96	256	158	.617	2.0	D	-	MAX. SHEAR STRESS NOT REACHED *SUSPECT DUE TO CELL LEAKAGE
TP25	31/4	4.9-5.1	D.V.	17.4	16.5	76																				
TP25	T34	5.3	D.V.	23.5	15.3	89						DL	76	80	102	-	15.9	98	-	82	52	.634	0.9	-	-	
TP25	T35	5.5	D.V.	24.1	15.6	94						$\overline{CPU}$	76	60	-	-	16.1	108	.92	*74	*36	*.486	1.1	D	10.3	*JOINT FAILURE
TP26	T22	5.2	D.V.	25.0	12.8	62						$\overline{CPU}$	73	50	-	-	13.4	106	.96	64	42	.656	3.5	D	17.7	
TP26	T23	5.2	D.V.	25.7	14.3	81						$\overline{CPU}$	74	80	-	-	14.7	97	.92	80	44	.550	4.2	D	14.7	
TP74	T2	4.8	D.V.	9.4	16.8	44						DL	76	80	116	-	17.8	89	-	90	57	.633	7.8	D	-	
TP74	2/1	2.5-2.8	D.V.	15.8	15.5	60	h	15	28	35	22															
TP74	2/3	2.5-2.8	D.V.	13.1	15.6	51	h	17	22	45	16	CD	76	*50	-	-	16.3	79	-	110	60	.545	4.4	D	4.4	*PRE-CONSOLIDATED TO 200 kPa
TP74	2/4	2.5-2.8	D.V.	14.7	15.2	53						$\overline{CPU}$	76	75	-	-	-	-	.98	*58	36	.621	2.0	C	1.7	
TP74	2/5	2.5-2.8	D.V.	14.0	16.8	66	a	16	31	40	23	DL	75	59	76	-	16.1	93	-	41	38	.927	0.1	D	-	
TP74	2/6	2.5-2.8	D.V.	11.3	16.2	48	s	25	18	43	14	DL	75	117	171	-	-	-	-	138	82	.594	4.9	C	-	
TP74	2/7	2.5-2.8	D.V.	-	-	-						DL	76	100	54	-	-	-	-	*26	*26	1.0	0.1	D	-	SUSPECT DUE TO RAM FRICTION AND 6 kPa PWP DIFFERENCE ACROSS SPECIMEN
ML5D		7.8-7.95	D.V.	16.0	15.2	58	h	4	27	62	7	$\overline{CPU}$	102	120	-	-	16.0	81	1.0	118	68	.578	9.0	C	9.0	
ML5D		8.3-8.5	D.V.	20.5	15.0	74	h	5	38	51	5	$\overline{CPU}$	103	140	-	-	15.7	85	.96	110	60	.545	10.8	C/D	12.8	
ML5D		8.7-8.9	D.V.	18.6	15.6	72	h	9	37	47	7	CWC	104	*163	-	-3	17.0	88	-	*359	196	.546	13.1	D	13.1	*SUCTION ADDED WATER CAME OUT THROUGH AIR LINE DURING SHEAR
ML5E		7.0-7.2	D.V.	22.1	15.6	85	h	17	14	25	31	WD	103	125	-	-	16.8	96	-	288	161	.559	9.9	C	9.9	
ML56		10.4-11.02	D.V.	23.7	15.5	92	h	4	41	50	5	$\overline{CPU}$	104	180	-	-	16.5	104	.97	289	171	.592	5.2	D	7.5	
ML56		10.4-11.02	D.V.	23.3	15.5	89						$\overline{CPU}$	104	130	-	-	*16.9	*112	1.0	206	118	.573	3.8	D	7.1	*UNREALIABLE CELL VOLUME CHANGE MEASUREMENTS

KEY:

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Symbols
- γ_d - unit dry weight
- S - degree of saturation
- B - pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

- ε_f = axial strain at failure
- ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive (A > 1/3 at failure)
- D = dilatant (A ≤ 1/3 at failure)
- failure = max. obliquity

(+) applied deviator stress for dead load test

(++) negative pwp in cwc or ac test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A2.5

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c. %	γ_d kN/m³	S %	PSD type	clay %	silt %	sand %	gravel %	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m²	(†) app. dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p_f' kN/m²	q_f' kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
ML56		12.64-13.24	D.V.	24.4	15.6	96	h	6	21	63	10	$\overline{CPU}$	104	250	-	-	*18.4	*142	.94	316	197	.623	4.6	D	7.0	*SUSPECT DUE TO LEAKAGE THROUGH MEMBRANE
ML56		13.25-13.65	D.V.	21.7	16.2	96						$\overline{CPU}$	104	150	-	-	*17.3	*107	1.0	241	151	.627	5.0	D	10.1	*SUSPECT DUE TO CELL LEAKAGE
ML56		13.65-13.85	D.V.	21.6	16.1	93						$\overline{CPU}$	103	220	-	-	17.3	103	1.0	253	141	.557	6.2	D	9.5	
ML56		13.85-14.45	D.V.	23.4	15.8	95	h	3	30	53	14	$\overline{CPU}$	104	210	-	-	16.5	96	.98	233	139	.597	4.0	D	8.7	
ML60C		7.8-8.08	D.V.	27.0	14.8	96						$\overline{CPU}$	103	140	-	-	15.6	105	.90	187	113	.603	4.5	D	9.5	
ML60C		8.9-9.09	D.V.	27.9	14.1	89	a	4	50	43	3	$\overline{CPU}$	105	150	-	-	15.1	93	1.0	210	123	.586	5.7	D	8.0	
ML130		4.25-4.8	D.V.	17.1	14.6	59						$\overline{CPU}$	103	90	-	-	15.4	105	.90	150	103	.687	5.8	D	18.1	CONSOL DURING SAT AS B.P. DRIFTED. PWD DROPPED TO-82 kPa IN SHEAR
ML130		2.0	D.V.	22.1	14.4	74						$\overline{CPU}$	104	40	-	-	14.9	94	.90	184	125	.680	4.0	D	11.4	PORE PRESSURE REDUCED TO -103 kPa DURING SHEAR
K2	Ta	1.17	D.V.	14.0	15.0	50	h	7	23	35	35	WD	76	30	-	-	15.5	76	-	55	25	.455	12.5	C	12.5	
K2	Tb	1.6	D.V.	16.7	15.8	68	h	8	30	51	11															
K2	B/1	1.6	D.V.	*5.5	13.7	*16	s	18	24	47	11	DL	76	62	42	-	15.6	71	-	**46	**25	.543	0.2	C	-	*DRIED DURING STORAGE **FAILED BETWEEN READINGS NEAREST KNOWN STRESSES SHOWN
K2	B/1	1.6	D.V.	*6.7	14.5	*22	s	18	24	56	2	$\overline{CPU}$	76	30	-	-	15.4	95	.96	22	13	.591	3.8	C	0.7	*DRIED DURING STORAGE
K2	BC/1	2.5	D.V.	13.0	14.4	43	s	12	31	49	8	$\overline{CPU}$	76	25	-	-	15.7	107	.97	17	11	.647	12.8	C	0.8	CONSOLIDATED DURING SATURATION DUE TO DRIFTING BACK PRESSURE
K2	T1	4.97	D.V.	16.2	15.9	67						$\overline{CPU}$	76	100	-	-	*16.9	*114	.98	108	64	.593	4.7	D	9.1	*SUSPECT DUE TO CELL LEAKAGE

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Symbols

γ_d - unit dry weight
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B - pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
$p_f' = \frac{1}{2}(\sigma_1' + \sigma_3')$ at failure
$q_f' = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress
Mode (of failure)
C = compressive (A > ⅓ at failure)
D = dilatant (A ≤ ⅓ at failure)
failure = max. obliquity

(†) applied deviator stress for dead load test
(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A3.1

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	% clay	% silt	% sand	% M gravel	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m²	(+) app. dev. kN/m²	(++) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p_f' kN/m²	q_f' kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP95	1/1	1.8-2.1	C.D.G.	11.9	15.8	49						WD	76	40	-	-	-	-	-	152	112	.736	7.9	D	7.9	
TP95	1/2	1.8-2.1	C.D.G.	12.4	16.3	54						$\overline{CU}$	76	30	-	-	17.5	-	.97	73	59	.808	2.9	D	17.4	
TP95	1/5	1.8-2.1	C.D.G.	12.4	15.2	46																				
TP95	1/6	1.8-2.1	C.D.G.	11.8	16.7	56						$\overline{CU}$	76	60	-	-	18.0	-	.97	131	103	.786	2.7	D	14.4	
TP95	1/8	1.8-2.1	C.D.G.	22.3	16.4	51						$\overline{CU}$	76	90	-	-	17.7	-	.97	220	162	.739	5.7	D	14.8	
TP95	2/3	2.3-2.6	C.D.G.	9.5	16.6	45																				
TP98	1/1	1.5-1.8	C.D.G.	12.6	11.8	28						$\overline{CPU}$	76	90	-	-	12.9	-	.97	117	78	.667	1.4	D	12.4	
TP98	1/4	1.5-1.8	C.D.G.	10.2	12.2	24						$\overline{CPU}$	76	60	-	-	12.2	-	.97	92	63	.685	2.0	D	10.0	
TP98	1/5	1.5-1.8	C.D.G.	13.1	11.7	29						$\overline{CPU}$	76	40	-	-	12.9	-	.98	80	55	.688	1.0	D	10.1	
TP98	1/7	1.5-1.8	C.D.G.	13.4	12.0	30	h	2	20	51	27	$\overline{CPU}$	76	25	-	-	12.9	-	.98	74	56	.757	1.7	D	12.3	
TP98	1/8	1.5-1.8	C.D.G.	14.5	12.2	34																				
TP98	3/1	2.3-2.6	C.D.G.	17.0	11.8	37	h	2	8	47	43															
TP98	3/2	2.3-2.6	C.D.G	17.3	12.0	39	h	2	9	62	27	WD	76	20	-	-	12.5	82	-	96	76	.792	1.0	D	1.0	
TP98	3/5	2.3-2.6	C.D.G.	6.9	10.8	13	h	12	20	46	22	DL	76	63	64	-	**11.8	**60	-	*45	*45	-	-	-	-	DRIED DURING STORAGE *FAILED BY INCREASING LOAD **NOT RELIABLE
TP98	3/6	2.3-2.6	C.D.G	14.8	11.6	32	h	6	18	60	16															
TP98	3/7	2.3-2.6	C.D.G.	13.7	11.7	30	h	1	7	45	47	WD	76	100	-	-	12.9	83	-	248	148	.597	8.3	D	8.3	
TP98	3/8	2.3-2.6	C.D.G.	14.8	12.3	35	h	0	3	44	53	WD	76	60	-	-	13.0	76	-	184	124	.674	6.9	D	6.9	
TP98	3/9	2.3-2.6	C.D.G.	16.0	11.5	34	h	0	7	49	44	WD	76	40	-	-	12.2	77	-	128	88	.688	6.7	D	6.7	
TP98	4/1	3.2-3.5	C.D.G.	15.3	12.2	36						WD	73	30	-	-	12.9	86	-	107	77	.720	2.2	D	2.2	
TP98	4/2	3.2-3.5	C.D.G.	14.6	12.3	35																				

KEY:

P.S.D. type
- h – oven dried
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- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p_f' = \frac{1}{2}(\sigma_1' + \sigma_3')$ at failure
- $q_f' = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

- ε_f – axial strain at failure
- ε_m – axial strain at max dev. stress

Mode (of failure)
- C – compressive (A > ⅓ at failure)
- D – dilatant (A ≤ ⅓ at failure)
- failure = max. obliquity

(+) applied deviator stress for dead load test

(++) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A3.2

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	% clay	% silt	% sand	% M gravel	TRIAXIAL Test	BEFORE SHEARING dia mm	Eff. cell press. kN/m²	(†) app dev. kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'	ε_f %	Mode	ε_m %	Comments
TP98	4/3	3.2-3.5	C.D.G.	15.3	11.9	34	h	3	19	55	23	WD	76	60	-	-	12.7	79	-	166	106	.639	8.9	D	8.9	
TP98	4/4	3.2-3.5	C.D.G.	14.5	12.6	36																				
TP98	4/5	3.2-3.5	C.D.G.				h	1	7	58	34	DL	76	66	84	-	12.6	82	-	47*	47*	-	**	-	-	**LESS THAN 0.1% *FAILED BY INCREASING LOAD
TP98	4/6	3.2-3.5	C.D.G.	14.9	12.1	34	s	36	19	33	12															
TP98	5/1	3.9-4.2	C.D.G.	15.2	12.9	40	h	2	5	54	39	WD	76	75	-		13.8	83	-	232	157	.677	6.2	D	6.2	
TP98	5/2	3.9-4.2	C.D.G.	16.2	13.3	45	h	2	6	56	36															
TP98	5/4	3.9-4.2	C.D.G.	16.7	12.8	43																				
TP98	5/5	3.9-4.2	C.D.G.	11.6	12.8	30																				
TP98	5/6	3.9-4.2	C.D.G.	14.1	12.9	36	s	-	10	53	37															
TP98	5/7	3.9-4.2	C.D.G.	14.9	12.9	39	s	2	8	63	27															
TP98	5/8	3.9-4.2	C.D.G.	13.9	12.8	35																				
ML107	-	6.44-7.05	C.D.G.	14.6	16.0	64	s	-	12	56	32	WD	104	125	-	-	16.8	79	-	754	629	.834	3.6	D	3.6	
ML107	-	7.4	C.D.G.	14.8	14.9	54	s	1	14	50	35	WD	104	125	-	-	15.7	54	-	504	379	.752	9.1	D	9.1	
ML107	-	7.7	C.D.G.	14.9	15.1	55	s	-	7	41	52	WD	104	130	-	-	15.8	85	-	492	362	.736	9.1	D	9.1	
ML107	-	8.4	C.D.G.	20.9	14.0	66	s	-	8	50	42	WD	102	60	-	-	14.5	90	-	197	137	.695	9.6	D	9.6	
ML107	-	8.9	C.D.G.	20.5	15.6	83						WD	102	50	-	-	16.2	101	-	228	178	.781	8.4	D	8.4	
ML107	-	9.7	C.D.G.	17.4	15.0	63	s	3	17	52	28	$\overline{CPU}$	103	160	-	-	15.8	102	.93	418	308	.737	7.6	D	13.6	
ML107	-	12.05-12.25	C.D.G.	20.4	14.6	69	s	1	9	51	39	$\overline{CPU}$	103	200	-	-	15.5	93	1.00	297	200	.673	7.8	D	-	*MAX. SHEAR STRESS NOT REACHED
ML122A	-	13.5	C.D.G.	32.1	13.3	90						DL	74	102	263	-	14.6	102	-	214	131	.612	0.9	D	-	
ML122A	-	13.84-13.99	C.D.G.	29.3	14.3	95						WD	74	80	-	-	15.2	95	-	189	109	.576	8.1	D	8.1	

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- CD – consolidated drained
- WD – percolated drained
- DL – dead load
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- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress
Mode (of failure)
C = compressive (A > 1/3 at failure)
D = dilatant (A ≤ 1/3 at failure)
failure = max. obliquity

(†) applied deviator stress for dead load test
(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A3.3

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS: m.c (%)	γ_d (kN/m³)	S (%)	PSD: type	% clay	% silt	% sand	% M gravel	TRIAXIAL: Test	BEFORE SHEARING: dia. (mm)	Eff. cell press. (kN/m²)	(†) app. dev. (kN/m²)	(††) $-u_w$ (kN/m²)	γ_d (kN/m³)	S calc (%)	B	AT FAILURE: p'_f (kN/m²)	q'_f (kN/m²)	q'/p'_f	ε_f (%)	Mode	ε_m (%)	Comments
ML122A	-	15.5	C.D.G.	28.5	13.8	86.4						DL	73	123	289	-	15.2	92	-	236	141	.597	1.9	-	-	
ML122A	-	15.85-16.0	C.D.G.	30.3	13.8	91						WD	73	95	-	-	14.9	98	-	202	107	.529	10.8	D	10.8	
ML122A	-	17.0-17.3	C.D.G.	28.8	14.4	95						WD	73	107	-	-	15.5	101	-	259	152	.587	6.1	D	6.1	
ML122A	-	17.5	C.D.G.	27.9	14.1	87	a	7	40	45	8	DL	74	137	356	-	16.6	89	-	290	177	.610	0.4	-		
ML122A	-	17.9-18.0	C.D.G.	28.7	14.6	97						WD	73	60	-	-	15.2	100	-	149	90	.604	4.7	D	4.7	
ML53B	-	8.4	C.D.G.	27.4	14.2	87	s	7	18	42	33	DL	102	89	170	-	14.8	96	-	94	87	.926	0.5	D	-	
ML53B	-	9.4-10.0	C.D.G.	22.9	14.1	72						DL	103	90	207	-	15.0	85	-	117	98	.838	2.8	D	-	
TP6 OAP	1/1	0.3	C.D.G.	24.6	13.0	68						$\overline{CU}$	76	200	-	-	-	-	.98	356	238	.669	4.2	D	10.0	
TP6 OAP	1/3	0.3	C.D.G.	25.3	13.2	72						$\overline{CU}$	76	15	-	-	-	-	.97	85	69	.812	0.9	D	-	MAX. SHEAR STRESS NOT REACHED

KEY:

P.S.D. type
- h – oven dried
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- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
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- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

- ε_f – axial strain at failure
- ε_m – axial strain at max dev. stress

Mode (of failure)
- C – compressive (A > ⅓ at failure)
- D – dilatant (A ≤ ⅓ at failure)
- failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

SUMMARY OF LABORATORY TEST RESULTS

TABLE A4.1

Bore-hole or trial pit	Block no / Spec	Depth (m)	Material description	INITIAL CONDITIONS m.c %	γ_d kN/m³	S %	PSD type	PSD % clay	PSD % silt	PSD % sand	PSD % M gravel	TRIAXIAL Test	BEFORE SHEARING dia. mm	Eff. cell press. kN/m²	(†) app. dev kN/m²	(††) $-u_w$ kN/m²	γ_d kN/m³	S calc %	B	AT FAILURE p'_f kN/m²	q'_f kN/m²	q'/p'_f	ε_f %	Mode	ε_m %	Comments
TP85	1/1	3.0	FILL	30.7	13.3	85						DL	76	60	45	-	13.9	95	-	31	23	.742	0.3	D	-	
TP86	1/3	3.0	FILL	32.9	13.4	93						WD	76	15	-	-	13.8	97	-	30	15	.500	1.0	*D	1.0	*SMALL RATE OF DILATION
TP86	1/5	3.0	FILL	31.8	13.0	84						WD	76	30	-	-	13.3	86	-	63	34	.540	4.4	*-	4.4	*REACHED CONSTANT VOLUME
TP86	1/6	3.0	FILL	31.6	13.1	85						WD	76	20	-	-	13.5	95	-	61	41	.674	1.5	D	1.5	
TP86	1/7	3.0	FILL	29.6	13.3	82						DL	76	50	112	-	14.2	94	-	88	54	.614	1.9	D	-	

KEY:

P.S.D. type
- h – oven dried
- a – air dried
- s – dispersion

Test
- $\overline{CU}$ – consolidated undrained with pwp
- $\overline{CPU}$ – consolidated, percolated, undrained with pwp
- CD – consolidated drained
- WD – percolated drained
- DL – dead load
- SC – suction conditioned
- CWC – constant water content

Symbols
- γ_d – unit dry weight
- S – degree of saturation
- B – pore pressure parameter ($\frac{\Delta u}{\Delta \sigma_3}$)
- $p'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ at failure
- $q'_f = \frac{1}{2}(\sigma_1 - \sigma_3)$ at failure

ε_f = axial strain at failure
ε_m = axial strain at max dev. stress

Mode (of failure)
- C = compressive (A > ⅓ at failure)
- D = dilatant (A ≤ ⅓ at failure)

failure = max. obliquity

(†) applied deviator stress for dead load test

(††) negative pwp in cwc or sc test

APPENDIX B

SUMMARY OF PSD RESULTS

TABLE B1	PSD results from Laboratory samples of colluvium (All areas)
TABLE B2	PSD results from bulk samples of colluvium (Po Shan and University Areas)
TABLE B3	PSD results from bulk samples of colluvium (Glenealy, Seymour and Central Areas)
TABLE B4	PSD results from bulk samples of DV (Po Shan and University Areas)
TABLE B5	PSD results from bulk samples of CDG (Po Shan and University Areas)

KEY TO 'METHOD' ABBREVIATION

H - oven dried method

A - air dried method

S - dispersion only method

Note: Unless otherwise shown all PSD tests were done using the dispersion only method.

PSD RESULTS FROM LAB. SAMPLES OF COLLUVIUM

TABLE B1.1

LOCATION	SAMPLE No.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %	METHOD
K_1	Na	3.5	VBD4GT	2	47	26	9	18	S
K_1	Nb	3.9	VBD4GT	2	17	39	19	25	S
K_1	Td	2.16	VBD4GT	2	37	26	16	21	S
K_3	T_2	2.35	VBC3GT	3	17	36	29	18	S
K_3	N_1	2.1	VBC3GT	3	5	31	33	31	S
K_4	T_1	2.3	VB3GT	3	5	39	42	14	S
K_4	T_4	4.5	VB3GT	3	20	30	30	20	S
K_4	T_5	4.5	VB3GT	3	43	20	19	18	S
K_4	T_7	5.3	VB3GT	3	7	29	38	26	S
ML5D		4.7 - 5.0	VBD3GT	2	30	30	18	22	S
ML5E		4.8 - 5.4	VBD3GT	2	30	29	19	22	S
ML112		6.5	VBD1/2GT	2	27	37	29	7	A
ML112		6.6 - 6.8	VBD1/2GT	2	23	33	39	5	A
ML112		7.6	VCD2/3ST	2	10	35	38	17	A
ML112		7.75	VCD2/3ST	2	10	47	23	20	A
ML112		8.9	VCD2/3ST	2	18	42	18	22	A
ML112		9.2	VCD2/3ST	2	22	48	19	11	A
ML112		9.4	VCD2/3ST	2	14	43	18	25	A
ML112		9.4 - 9.98	VCD2/3ST	2	21	39	24	16	S
ML112		11.6	VCD2/3ST	2	17	49	22	12	A
ML112		11.8 - 12.4	VCD2/3ST	2	17	64	14	5	S
ML112		12.1 - 12.3	VCD2/3ST	2	20	66	16	8	S
ML112		14.4 - 14.6	VCD2/3ST	2	35	42	16	7	S
ML122	18	10.83 - 11.38	VCD2/3ST	2	25	37	23	15	S
TP4B	B3-7	3.35	VD2T		7	26	?	?	S
TP4H		1.5	VC1T	2	19	23	22	36	H
TP77	N_1	1.1	VC2/3GT	2	4	31	23	42	S
TP82	B4-6	0.8	VB1ST	3	13	32	20	35	S
TP96	B1-1	3.3	VBC1GT	2	35	35	16	14	S
TP96	B1-2	3.3	VBC1GT	2	25	35	17	23	S

PO SHAN & UNIVERSITY AREAS

PSD OF BULK SAMPLES - COLLUVIUM

TABLE B2.1

LOCATION	SAMPLE No.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %
TP20	3	0.9	VBC2ST	2	9	34	44	13
TP20	6	1.4	VBC2ST	2	26	28	32	14
TP20	9	1.1	VBC2ST	2	16	32	35	17
TP20	17	2.5	VBC2ST	2	4	38	43	15
TP20	18	2.5	VBC2ST	2	5	39	42	14
TP22	4	1.5	VBC2ST	3	9	25	56	10
TP22	5	2.0	VBC2ST	3	6	24	49	21
TP22	11	2.5	VBC2ST	3	3	35	43	19
TP22	14	3.0	VBC2ST	3	15	29	39	17
TP22	25	4.0	VBC2ST	2	9	39	39	13
TP22	28	4.5	VBC2ST	2	5	50	35	10
TP22	32	5.0	VBC2ST	2	19	34	34	13
TP23	2	0.5	VC2T	3	7	34	47	12
TP23	6	1.1	VC2T	3	7	21	49	23
TP23A	2	0.5	VC2T	3	15	23	37	25
TP23A	4	1.1	VC2T	3	11	16	40	33
TP23A	6	1.5	VC2T	3	15	19	40	26
TP23A	12	2.5	VC2T	3	2	22	48	28
TP23A	15	5.0	VC2T	3	5	25	43	27
TP23A	19	3.3	VC2T	3	13	23	34	30
TP23A	21	3.5	VC2T	3	3	31	45	21
TP24	1	0.5	VCD3ST	3	27	24	32	17
TP24	4	1.0	VCD3ST	3	19	18	37	26
TP24	7	1.7	VCD3ST	3	21	19	34	26
TP24	11	2.2	VCD3ST	3	21	23	37	19
TP24	15	3.0	VCD3ST	3	39	22	25	14
TP24	18	3.8	VCD3ST	3	33	30	24	13
TP24	21	4.0	VCD3ST	3	20	43	32	5
TP24	24	4.2	VCD3ST	3	19	38	37	6
TP25	7	2.1	VBC2T	3	22	18	41	19
TP25	11	2.6	VBC2T	3	15	18	43	24
TP25	15	3.5	VBC2T	3	3	14	67	6
TP25	20	3.7	VBC2T	3	15	29	48	8

PO SHAN & UNIVERSITY AREAS

PSD OF BULK SAMPLES - COLLUVIUM

TABLE B2.2

LOCATION	SAMPLE No.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %
TP26	6	2.3	VBC2ST	3	6	17	47	30
TP26	10	2.9	VBC2ST	3	14	18	44	24
TP26	14	3.2	VBC2ST	3	6	16	55	23
TP26	18	3.8	VBC2ST	3	7	22	55	16
TP26	24	5.3	VBC2ST	3	9	25	48	18
TP26	27	5.6	VBC2ST	3	18	19	46	16
TP28	1	1.5	VC1T	3	29	22	37	12
TP28	4	1.8	VC1T	3	35	17	31	17
TP29	1	1.0	VC2T	3	22	16	44	18
TP29	9	2.5	VC2T	3	5	39	42	14
TP29	13	3.0	VC2T	3	2	27	56	15
TP30	6	2.1	VCD2T	3	13	19	40	28
TP30A	2	1.2	VCD2T	3	19	24	43	14
TP31	2	1.0	VBC2T	2	6	28	28	38
TP31	3	1.0	VBC2T	2	4	28	37	31
TP31	14	2.5	VD1T	2	9	29	29	33
TP31	18	3.0	VD1T	2	7	30	37	26
TP31	22	3.5	VD1T	2	6	33	37	34
TP31	25	4.0	VD1T	2	6	38	40	16
TP31	30	4.5	VD1T	2	7	45	37	11
TP31	32	4.8	VD1T	2	13	37	37	13
TP31	35	2.0	VD1T	2	4	33	35	28
TP31	39	5.0	VD1T	2	5	40	42	13
TP31	43	5.6	VD1T	2	8	48	40	4
TP33	2	5.0	VBC2T	3	13	21	46	12
TP33	4	1.2	VBC2T	3	8	16	53	23
TP33	9	1.8	VBC2T	3	2	14	60	24
TP33	10	2.3	VBC2T	3	1	15	57	27
TP33	13	2.9	VBC2T	3	5	21	55	19
TP33	16	3.4	VBC2T	3	14	23	43	20
TP35	9	3.0	VC2ST	2	32	23	27	18
TP35	11	3.5	VC2ST	2	13	22	29	36
TP35	13	4.0	VC2ST	2	14	17	4	35
TP35	16	4.5	VC2ST	2	17	38	26	19
TP35	20	5.0	VC2ST	2	14	21	37	28
TP35	23	5.65	VC2ST	2	5	25	41	29

PO SHAN & UNIVERSITY AREAS

PSD OF BULK SAMPLES - COLLUVIUM

TABLE B2.3

LOCATION	SAMPLE No.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %
TP36	4	0.5	VD1ST	2	5	35	41	19
TP36	7	1.0	VC2ST	2	4	24	45	27
TP36	11	1.5	VD1ST	2	2	18	51	29
TP36	14	2.0	VD1ST	2	5	32	41	22
TP36	17	2.5	VD1ST	2	5	34	46	15
TP36	20	3.0	VD1ST	2	19	44	28	9
TP36	24	3.5	VD1ST	2	9	34	40	17
TP36	29	4.0	VD1ST	2	22	37	29	12
C2	2	2.2	VBC4CT	2	27	40	25	8
C2	6	5.2	VC1ST	2	17	55	25	3
C5	6(a)	4.6	VC1ST	2	7	37	49	7
C6	3	2.7	VBC4CT	2	27	33	33	7
C6	3(a)	3.4	VBC4CT	2	0	8	16	76
C6	6	4.6	VBC4CT	2	18	32	36	14
C6	8	6.5	VC1ST	2	13	42	42	3
C9	3	2.1	VBC4CT	2	13	46	27	14
C10	2	1.0	VBC2T	2	23	38	26	13
C11	3	2.6	VBC2T	2	22	31	28	19
C12		6.5	VBC2T	2	12	45	30	13
C12	3	2.2	VBC2T	2	27	33	20	20
C12	3(a)	3.2	VBC2T	2	18	30	26	26
C12	5	3.7	VBC2T	2	16	34	27	23
C12	6	4.2	VBC2T	2	15	42	27	16
C12	6	4.9	VBC2T	2	16	40	25	19
C13		6.0	VBC2T	2	16	47	26	11
C13	3	2.4	VBC2T	2	28	26	22	24
C13	6	4.4	VBC2T	2	18	43	26	13
C13	8	6.1	VBC2T	2	16	43	24	17
C15	10	8.0	VBC2T	2	12	35	29	24
Ud.t.	S2		VD2ST	1	23	25	21	31
UC6	S1	1.4	VD2ST	1	17	37	26	20
UC16	S16	3.4	VD4ST	1	6	43	16	5
UC17	S15	4.2	VD4ST	1	28	44	25	3
UC26	S9	2.5	VDC3ST	2	30	34	19	17
UC29	S13	3.4	VDC3ST	2	37	24	25	14
UC34	S6	4.3	VDC3ST	2	19	31	33	17
UC39	S12	2.5	VDC2ST	2	51	30	14	5

GLENEALY, SEYMOUR AND CENTRAL AREAS

PSD OF BULK SAMPLES - COLLUVIUM

PSD SUMMARY

TABLE B3.1

LOCATION	SPEC.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %
TP74		0.5	VC2GT	2	26	29	23	22
TP74		0.5	VC2GT	2	18	32	25	25
TP74		1.0	VC2GT	2	34	31	18	17
TP74		1.0	VC2GT	2	30	30	23	17
TP74		1.5	VC2GT	2	6	46	26	22
TP74		1.5	VC2GT	2	9	36	25	30
TP76A		0.5	VC2GT	3	26	31	18	25
TP76A		1.0	VC2GT	3	39	21	16	24
TP76A		2.0	VC2GT	3	24	28	21	27
TP76A		3.0	VC3GT	3	27	33	18	22
TP76A		4.0	VC3GT	3	24	34	20	22
TP78		0.5	VB2T	3	22	38	18	22
TP78		1.0	VB2T	3	28	27	26	19
TP78		1.0	VB2T	3	29	23	26	22
TP78		1.5	VB2T	3	22	28	27	23
TP78		1.5	VB2T	3	28	25	23	24
TP78		2.0	VB2T	3	27	28	28	17
TP78A		0.5	VC1GT	2	33	27	19	21
TP78A		1.0	VC1GT	2	28	29	17	26
TP78A		1.0	VC1GT	2	41	19	17	23
TP78A		1.0	VC1GT	2	30	22	24	24
TP78A		2.0	VC1GT	2	27	24	27	22
TP82		0.5	VC1ST	3	33	35	16	16
TP82		1.0	VC1ST	3	30	36	17	17
TP82		1.5	VC1ST	3	38	33	13	16
TP82		2.0	VC1ST	3	27	38	19	16
TP90		0.5	VBC2/3GT	3	23	19	33	25
TP90		1.0	VBC2/3GT	3	17	20	34	29
TP90		1.5	VBC2/3GT	3	30	18	33	19
TP90		2.0	VBC2/3GT	3	24	26	29	21
TP90		3.0	VBC2/3GT	3	25	21	34	20
TP90		3.5	VBC2/3GT	3	25	20	35	20

GLENEALY, SEYMOUR AND CENTRAL AREAS

PSD OF BULK SAMPLES - COLLUVIUM

PSD SUMMARY

TABLE B3.2

LOCATION	SPEC.	DEPTH (M)	TYPE	CLASS	GRAVEL %	SAND %	SILT %	CLAY %
TP96		0.5	VBC1GT	2	25	35	22	18
TP96		0.5	VBC1GT	2	21	38	24	17
TP96		1.0	VBC1GT	2	35	2	24	21
TP96		2.0	VBC1GT	2	23	25	22	20
TP96		2.0	VBC1GT	2	25	34	22	19
TP96		3.2	VBC1GT	2	26	33	18	23
TP98A		0.5	VBC2/3GT	3	36	29	16	19
TP98A		1.0	VBC2/3GT	3	25	25	30	30
TP99		0.5	VC1GT	3	20	0	16	24
TP99		1.0	VC1GT	3	26	34	23	17
TP7H		0.5	VC3T	2	26	29	25	20
TP7H		1.0	VC3T	2	32	23	23	22
TP7H		1.5	VC3T	2	34	23	20	23
TP70AP		1.0	VC2GT	2	40	30	9	21
TP70AP		1.5	VC2GT	2	44	26	10	20

PO SHAN AND UNIVERSTIY AREAS

PSD OF BULK SAMPLES - DV

TABLE B4.1

LOCATION	SAMPLE NO.	DEPTH (M)	GRAVEL %	SAND %	SILT %	CLAY %
TP20	20	3.0	19	34	42	5
TP20	29	4.0	3	64	29	4
TP20	36	5.0	10	46	36	8
TP20	45	6.0	4	47	43	6
TP20	48	5.5	3	42	48	7
TP20	52	6.5	12	37	48	3
TP20	54	7.0	3	48	41	8
TP22	36	5.5	3	42	51	4
TP22	40	6.0	25	41	28	6
TP22	41	6.5	6	45	43	6
TP22	48	7.0	4	52	42	2
TP22	52	7.5	8	41	44	7
TP22	56	8.0	22	43	32	3
TP22	60	8.7	18	43	36	3
TP25	24	4.3	2	27	60	11
TP25	28	4.9	1	19	77	3
TP25	32	5.5	1	17	76	6
TP27		2.5	36	26	32	6
TP27	11	3.0	5	28	61	6
TP27	13	3.5	3	22	57	18
TP29	17	3.5	7	35	56	2
TP29	21	4.2	10	34	45	11
TP36	33	4.5	16	49	29	6
TP36	37	5.0	21	38	37	4
C1	7	6.2	6	27	57	10
C3	7	5.6	20	51	34	5
C3	9	7.8	6	78	15	1
C4	7	5.4	10	30	55	5
C9	4	12.6	17	43	33	7
C12	11	8.5	10	48	41	1
C12	14	9.9	14	44	36	6
C15	10	9.8	11	55	34	0

PO SHAN AND UNIVERSTIY AREAS

PSD OF BULK SAMPLES - CDG

TABLE B5.1

LOCATION	SAMPLE NO.	DEPTH (M)	GRAVEL %	SAND %	SILT %	CLAY %	METHOD
Udt	S3		42	17	16	25	S
UC16	S17	3.4	50	40	9	1	S
UC27	S14	7.2	46	36	15	3	S
UC34	S7	4.3	58	34	5	3	S
UC34	S8	4.7	36	43	13	8	S
UC37	S10	9.9	35	49	13	3	S
UC37	S4	6.8	47	40	6	7	S
UC40	S5	4.3	43	24	15	18	S
UC41	S11	6.0	40	46	9	5	S

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