SITE CHARACTERISATION STUDY - PHASES 1 AND 2

GEO REPORT No. 71

N.P. Koor

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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Prepared by:

Geotechnical Engineering Office,
Civil Engineering Department,
Civil Engineering Building,
101 Princess Margaret Road,
Homantin, Kowloon,
Hong Kong.

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. A charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
July 1999

FOREWORD

This report presents the results of Phases 1 and 2 of the Site Characterisation Study carried out by the GEO in response to Professor Morgenstern's recommendation (d) made in his report on the Kwun Lung Lau landslide in 1994. The objective of the study was to determine the applicability of non-invasive geophysical methods to the identification of some important features which affect slope and retaining wall stability in Hong Kong.

The study was managed by Mr N.P.Koor. It involved him carrying out an initial literature review, preparing contract documents, designing and supervising the two phases of site trials, managing six contractors, and designing and supervising ground investigations at four of the trial sites.

A Working Group was established in order to obtain a consensus on the form and direction of the study from the geotechnical community in Hong Kong. The Working Group comprised professionals representing the Geotechnical Engineering Office (GEO), the Hong Kong Institution of Engineers (HKIE) and local universities. The members were Messrs R.P.Martin (GEO), Hugh H.H.Choy (GEO), N.P.Koor (GEO), K.C.Yeo (GEO), A.C.O.Li (GEO), S.Parry (GEO), J.L.P.Ho (GEO), J.Morris (GEO), P.R.Sayer (HKIE), K.Morton (HKIE), L.S.Chan (University of Hong Hong (UHK)) and X.S.Li (Hong Kong University of Science and Technology (HKUST)).

Dr L.S.Chan of the Deparment of Earth Sciences of the University of Hong Kong provided valuable advice and direction on the technical aspects of the site trials, geophysical results and interpretations.

Technical support for site supervision of the Phase 1 site trials was provided by Messrs K.C.Chan and P.C.Cheng. Technical support for site supervision of the Phase 2 field trials and subsequent ground investigation was provided by Mr K.W.Cheung, and Engineering Geology Graduates Ms A.Y.S.Liu, Ms W.H.Y.Law and Mr J.C.F.Wong. Technical support in procuring the ground investigation was provided by Mr K.W.Cheung with professional advice and support from Mr M.J.Shaw. All the Figures, Tables and Plates contained in the report were produced by Messrs K.C.Chan and P.C.Cheng. All of these contributions are gratefully acknowledged.

(R.P.Martin)
Chief Geotechnical Engineer/Planning

ABSTRACT

In response to recommendation (d) contained in Volume 1 of the report on the Kwun Lung Lau Landslide of 23 July 1994 (Hong Kong Government, 1994), the Planning Division of the GEO initiated a Site Characterisation Study on the use of non-invasive geophysical techniques to investigate masonry walls and man-made slopes in Hong Kong. Seven non-invasive geophysical methods have been assessed through two phases of field trials to determine their applicability to the identification of features which affect slope and retaining wall stability in Hong Kong. Selection of the seven methods was based on a literature review of international and local land-based geophysical practice. The selected methods were ground penetrating radar (GPR), shallow seismic reflection, spectral analysis of surface waves (SASW), resistivity imaging (RI), self potential, electromagnetic methods (EM) and thermal imaging. Two other methods, seismic refraction and sonic method, were tested at individual sites by two contractors but were not stipulated in the contract requirements.

The field trials were carried out by six contractors for Phase 1 and five out of the original six for Phase 2. The contractors were selected based on their technical expertise and track record in geophysical investigations for civil engineering works. The participating contractors for the Phase 1 field trials were Guandong South China EGD Co. (GSC), Bachy Soletanche Group (BS), Meinhardt Works (MW), Golder Associates (HK) Ltd (GA), Fugro Geotechnical Services (HK) Ltd. (FGS) and the Institute of Geophysical and Geochemical Exploration (IGGE). The Phase 2 contractors were the same as for Phase 1 apart from Meinhardt Works who decided not to participate.

The Phase 1 field trials, which tested all seven methods were made at four sites consisting of two pre-war masonry walls, one cut slope and one fill slope. All four sites had some existing ground investigation data which were provided to each of the contractors prior to the trials for purpose of calibrating the geophysical results. Based on observations of the fieldwork and the geophysical results, the techniques were divided into three groups, as follows:

Group 1. Techniques which produced some promising results and may be applied widely to sites in Hong Kong. These are Ground Penetrating Radar and Resistivity Imaging.

Group 2. Techniques which are affected by environmental noise and limited by characteristics inherent in the technique, but might be usefully applied at specific sites. These are Electromagnetic Conductivity (frequency domain) and Spectral Analysis of Surface Waves.

Group 3. Techniques which did not provide any useful information and do not warrant further consideration. These are High Resolution Seismic Reflection, Self Potential, Electromagnetic Conductivity (time domain) and Thermal Imaging.

In Phase 2 the Group 1 and 2 techniques were further tested at four new sites again consisting of, two pre-war masonry walls, one cut slope and one fill slope. The Phase 2 sites had no existing ground investigation data so as to allow the Working Group to assess if the geophysical methods alone can be reliable and accurate without any pre-conceptions of the site conditions. Preliminary interpretations were made by each of the contractors based solely on the geophysical results. Subsequent conventional ground investigations by GEO were then carried out at the sites and the results were given to each of the contractors for re-interpretation of their geophysical results.

The Phase 2 trials concluded that the overall quality of the raw data and interpretations varied significantly with the expertise of the individual geophysics teams. Certain contractors were able to demonstrate that a combined geophysical investigation utilising GPR and RI can locate the back of masonry walls if they are less than 3m thick and define zones of elevated moisture content and voids with reasonable accuracy. Only limited success was achieved in determining the location of corestones and the thickness of loose fill at the two slope sites.

In the light of the conclusions from the field trials, it is recommended that due to the inconsistent raw data and interpretations it would not be advisable at present to let a term contract specifically for non-invasive geophysical techniques for site characterisation in Hong Kong, or to require the use of such techniques in LPM studies. Also, further trials of the four or other techniques are not warranted. However, since some of the results were encouraging it is recommended that further research work on developing GPR and RI for masonry wall investigations is carried out through a local University with support from GEO. The research should focus on ways to enhance the resolution of the two techniques and also to develop a site and data processing methodology which would ensure more consistent results from contractors.

CONTENTS

1. INTRODUCTION

The collapse of an old masonry wall in Kwun Lung Lau on Hong Kong Island in 1994 was triggered by saturation of the soil behind the wall. Subsequent investigations showed that the main cause of the landslide was defective water-carrying services which saturated the slope. The masonry wall was thinner than indicated on available records and aggravated the failure (Hong Kong Government, 1994). Recommendation (d) contained in Volume 1 of the Report on the Kwun Lung Lau Landslide of 23 July 1994 (Hong Kong Government, 1994) stated that:

> "The GEO should undertake and support elsewhere in Hong Kong, research into improved means of site characterisation focused on factors that affect slope stability in Hong Kong. The Writer does not think that the development of slope warning systems for the conditions found in Hong Kong is promising. The critical features are small and numerous and instability often develops in an abrupt manner. However, there are a number of new developments in geophysics such as radar and non-contact resistivity that might be found useful in discovering subsurface defects and enhanced moisture zones."

In response to this recommendation, the Planning Division of the GEO initiated a Site Characterisation Study on the use of non-invasive geophysical techniques to investigate masonry walls and man-made slopes in Hong Kong. The project brief for the study is reproduced in Appendix A which was agreed by Prof. Morgenstern on 27 February 1995. Seven non-invasive geophysical methods have been assessed through two phases of field trials to determine their applicability to the identification of features which affect slope stability in Hong Kong. The methods were selected based on a literature review of international and local land-based geophysical practice. This literature review, which describes briefly the theory of each of the techniques together with an extensive reference list, is presented in Appendix B. The seven techniques assessed were; ground penetrating radar (GPR), shallow seismic reflection (SSR), spectral analysis of surface waves (SASW), resistivity imaging (RI), self potential (SP), electromagnetic methods (EM) and thermal imaging (TI).

The field trials were carried out at eight sites (Figure 1). Phase 1 field trials were made at sites with some existing ground investigation (GI) data (sites A, B, C and D) and were used as a calibration exercise to identify potentially applicable methods. Phase 2 field trials focused on four techniques selected from Phase 1 which were tested at four sites with no existing ground investigation data (sites E, F, G and H). In Phase 2 preliminary interpretations were made by each of the participating contractors based solely on the geophysical results. Subsequent conventional ground investigation by GEO was then carried out and the results were given to the contractors for re-interpretation of their results. The techniques were assessed by comparing the preliminary interpretations made by the contractors with the results of the ground investigations, and by taking into account the consistency of the raw data and interpretations presented by the different contractors.

Six contractors were selected based on their technical expertise and track record in geophysical investigations for civil engineering works. Profiles of each of the contractors, together with the techniques and equipment tested, are presented in Appendix C. The participating contractors for the Phase 1 field trials were; Guandong South China EGD Co. (GSC), Bachy Soletanche Group (BS), Meinhardt Works (MW), Golder Associates (HK) (GA), Fugro Geotechnical Services (HK) Ltd. (FGS) and the Institute of Geophysical and Geochemical Exploration (IGGE). Meinhardt Works voluntarily withdrew from the Phase 2 trials. The personnel and equipment used in Phase 2 were essentially the same as for Phase 1.

This report summarises the results of the Phase 1 and 2 field trials and makes recommendations for further work. Sub-surface features that can affect slope or retaining wall stability in Hong Kong and potentially identifiable with geophysical methods are discussed in Section Two. The two phases of the field trials are summarised in Sections Three and Four. Conclusions and recommendations for further work are presented in Section Five.

Interpretative reports on both phases have been produced by each contractor and are referenced in Section Six and Appendix C (Section C.3). Each report describes the geophysical methods used, field procedures, methods of analysis and presents the contractors interpretation of the results. These reports are contained in the Geotechnical Information Unit (GIU) of the Civil Engineering Library and should be read in conjunction with this report. Ground investigation reports for the eight trial sites are also contained in the GIU.

The interpretations contained in the contractors final reports were based on the geophysical information obtained, the GI data supplied by GEO, and the expertise of the contractor. The contractors were required to submit draft reports to the Working Group for comments and an interim review. The intention of this review was not to question the accuracy of the interpretations made but to ensure that each interpretation was justified and supported in the report. This procedure also helped to obtain a measure of the relative quality of the work of each contractor

2. APPLICATION OF GEOPHYSICS TO SLOPES AND WALLS

2.1 Introduction

Geophysical methods are most successful where there are strong contrasts in the physical properties between the target feature and its surroundings. The material properties frequently used in geophysical investigations are elasticity, electrical conductivity, density, magnetic susceptibility and electrical polarisation potential. Some of the most important features which can affect the stability of man-made slopes and walls are listed below and discussed in the subsequent sections along with a description of old masonry walls in Hong Kong.

(i) Geometry and structural condition of retaining structures which support a slope or platform.

(ii) Strength and spatial disposition of the soil and rock which form a slope or platform.

(iii) Discontinuities within a slope such as faults, shear zones, joints, hydrothermally altered zones and dykes.

(iv) Ground water.

(v) Zones of enhanced moisture content within a slope or behind a retaining structure.

(vi) Voids behind hard surface coverings.

2.2 Retaining Structures in Hong Kong

Two distinct groups of walls may be identified in Hong Kong, namely, pre-war and post-war (Jukes et al 1986). Post-war retaining structures are mainly cantilever, counterfort and gravity structures of mass or reinforced concrete construction. In general they have been designed and constructed in accordance with engineering principles and are not discussed further in this report.

Pre-war walls are largely gravity structures of masonry or mass concrete construction. Records of the design of pre-war walls generally do not exist, and therefore the condition and geometry of these walls require investigation if stability is to be determined. Shelton (1980) and Chan (1996) have studied the construction and stability of pre-war masonry retaining walls in Hong Kong. Chan (1996) describes three main types of masonry wall in Hong Kong, namely tied face walls, stone rubble walls and stone pitching (Figure 2).

Tied face walls were normally adopted to retain cuttings and were most popular in the 1840s. Typically, the front layer of blocks are well squared and bonded by thin layers of lime/sand mortar. The rear blocks in contact with the cut face are dry-packed and are tied to the front face by headers.

Stone rubble walls cover a wide range of wall types from random rubble walls to dressed block walls with horizontal tie beams (Figure 2). However, they all have a similar cored structure consisting of a front layer of dressed or dry-packed blocks, a rear vertical layer of commonly dry packed, rough slabby blocks and a core of excavated rock fill ranging in size from gravel to boulders. Variations include cores stabilised with lime, tied rubble walls with rectangular granite headers and rubble walls with concrete or lime stabilised soil beams.

Stone pitching is a single layer of bonded rock blocks, normally 300mm thick, laid onto a slope surface to prevent infiltration and erosion. The blocks are normally laid on a layer of no-fines concrete or crushed rock which acts as a drainage layer (Figure 2).

The three aspects of a masonry wall that are crucial to the assessment of its stability are its thickness, internal structure, and seepage conditions (Chan, 1996). Investigation techniques commonly adopted in Hong Kong to determine the geometry of masonry walls include weephole probing, coring using standard concrete coring equipment or modified triple tube core barrels, and hand-excavated trial pits behind or at the toe of the wall. Weephole probing has many drawbacks. The weepholes may not extend completely through the wall section or may become blocked or collapsed. The often sinuous nature of the weepholes makes probing difficult and the potential to probe beyond the back of the wall if internal erosion around the weephole has taken place will result in an over-estimation of wall thickness. Due to the heterogeneous nature of the building materials used to construct masonry walls and their complex structure, recovery using single core barrels is often very poor, rendering interpretation difficult. Hand-excavated trial pits are a satisfactory method of investigation but have time, cost, safety and access implications which often make them unattractive during the preliminary stages of an investigation. Obstructions due to services, large rock boulders and restricted access space often limit the depth of trial pit excavation to only a few meters.

Physical contrasts in density and resistivity may exist between the masonry wall and the back-fill materials behind the wall such that the interface may be identified by several geophysical techniques. If the wall contains voids or zones of increased moisture content then these areas may also produce geophysical anomalies especially in terms of electrical properties such as resistivity or conductivity.

2.3 Natural and Man-made Slopes

Although geophysical techniques cannot directly determine the *in situ* shear strength of a particular soil or rock, they are potentially useful in mapping the distribution of soil or rock of similar strength and stiffness between physical investigation points since these properties will manifest themselves as variations in density and elasticity. The mapping of fill bodies is an example where geophysics may be useful.

Adversely-oriented discontinuities, including relict features in soils, may control slope stability. Such features are often difficult to locate using conventional ground investigation techniques. Geophysical techniques are potentially suitable to detect open or infilled discontinuities since contrasts in conductivity and density may exist between the discontinuity and the surrounding intact rock mass. The resolution of many geophysical techniques however, is probably insufficient for detailed discontinuity mapping.

Geological features which may influence the local pore pressure distribution in a slope may also be identified using geophysical methods. Massey & Pang (1988) describe three types of feature which can influence pore pressure distribution in a slope as follows; soil zones or rock bands having substantially different permeability from the surrounding soil, relict discontinuities and relatively impermeable dykes and veins, and pipes and other subsurface erosion features. All of these features may modify electrical, elastic and density properties of the rock and soil to some extent.

2.4 Physical Anomalies Behind Slopes or Walls

Zones of high moisture content behind masonry walls or hard slope surface coverings may indicate an ongoing deterioration of the feature, for example due to a leaking water main or other water-bearing utility pipe, or preferential ground water flow along a discontinuity or non-water bearing utility. An increase in moisture content will affect the electrical properties of the host medium and should therefore be identifiable with certain electrical techniques.

Voids can develop behind hard slope surface covers due to internal erosion by flowing water or settlement of loose fill. The detection of a void may indicate a leaking water main behind the feature, or preferential ground water flow along a discontinuity, and may be a natural progression from a zone of elevated moisture content (Figure 3). Voids may produce significant contrasts in electrical, density or elastic properties that can be detected with certain geophysical techniques.

3. PHASE 1 FIELD TRIALS

3.1 Introduction

The Phase 1 Field Trials were made at four sites by six geophysical contractors between 5 December 1995 and 12 February 1996. Selection criteria for the four sites were: (i) the existence of good quality ground investigation data, (ii) the features were typical to Hong Kong, (iii) ease of access and, (iv) located within the urban environment. This section briefly describes the important aspects of each site, presents a summary of the results and the recommendations for the Phase 2 field trials. A detailed description of each site is presented in Interim Report No. 2 (Koor, 1996). The locations of the trial sites are shown in Figure 1.

3.2 Phase 1 Field Trial Sites

3.2.1 Trial Site A - Kennedy Town Police Quarters. Feature No. 11SW-A/C256

Site A is a north-west facing "L-shaped" soil cut slope (Plate 1) with both chunam and shotcrete surface protection located between Block A and Block B at Kennedy Town Police Quarters (Figure 4). The slope decreases in height from about 10.8m in the north-east to about 8.2m in the south and is inclined to between 50° and 60°. A 1m-wide berm runs at about mid-height throughout the full length of the slope. During the trials two wet areas existed along the south western half of a concrete platform at the crest of the slope (Figure 4 and Plate 1). They were caused by outflow from drainage pipes from Block A. Seepage from weepholes through the shotcrete was evident on the slope below these two areas (Figure 4). A transformer room is located at the north eastern end of Block A (Figure 4). The geophysical trials were made along four traverses TA01 to TA04 which were chosen to coincide with existing borehole and chunam strips, information from which is shown on Figure 4.

A total of 7 drill holes and 3 chunam strips were excavated previously at the site between 23 January and 12 February 1988 under Term Contract GC/85/02 (Bachy Soletanche, 1987[a]) and between 18 to 19 March 1988 under Term Contract GC/85/02 (Bachy Soletanche 1987[b]). A typical geological section through the slope is shown in Figure 5. A layer of concrete 100mm to 150mm thick covers the platform at the crest of the slope. Below the concrete slab is a layer of fill 1.5m to 5.5m thick. A reinforced concrete layer between 200mm and 700mm thick was encountered at the base of the fill in the north-east. A wedge of colluvium up to 3.5m thick underlies the fill in the south western half of the slope. Tuff was encountered in varying states of weathering below the fill and colluvium.

3.2.2 Trial Site B - Eliot Hall, Hong Kong University. Feature No. 11SW-A/R703

Site B is a 6.4m high vertical, 100m long, east-west trending, dressed block retaining wall with horizontal tie beams (Plate 2). It is located between May Hall and Eliot Hall at the University of Hong Kong, Pok Fu Lam (Figure 6). The retaining wall has a facing of bonded, squared fresh to slightly decomposed tuff blocks (typically 400mm by 350mm) with three, 300mm-thick horizontal concrete beams equally spaced up the wall. Clay pipe weepholes, 75mm in diameter are equally spaced up the wall. The wall is supported along its length by concrete flying buttresses spaced at about 10m. The wall supports a 7m-wide building platform for May Hall which is covered with a concrete slab in poor condition. The geophysical trials were made along five traverses TB01 to TB05 located to coincide with existing drillhole, corehole and trial pits, information from which is shown on Figure 6.

Three vertical drillholes, three horizontal and three inclined coreholes and three trial pits were excavated at the site between 8 and 15 September 1993 under Term Contract GE/93/06 (Vibro (H.K) Ltd, 1993). As shown in Figure 7, a fill layer 3m to 4m thick overlies 2m to 4m of colluvium, which in turn overlies completely decomposed tuff. The core of the wall is composed of cobbles and boulders of weathered tuff. The thickness of the wall increases from about 1.3m at the top to between 1.6m and 2.6m at the base of the wall.

3.2.3 Trial Site C - Sir Ellis Kadoorie School. Feature No. 11SE-A/R39

Site C is a 7.5m to 8m high sub-vertical, 31.8m long, south-west facing, dressed block retaining wall with horizontal tie beams (Plate 3). It is located behind the Sir Ellis Kadoorie School (Figure 8). The wall is inclined at 65° to the horizontal and is faced with bonded, square (400mm by 380mm) slightly decomposed granite blocks. Three 200mm-thick concrete tie beams run horizontally along the wall. Drainage through the wall is provided by six levels of 80mm-diameter clay pipe weepholes. A 30° vegetated slope exists above the crest of the wall. A 2.5m wide passage way separates the foot of the wall from the school building. The geophysical trials were made along three traverses located to coincide with existing drillhole and coreholes TC01 to TC03, information from which is shown on Figure 8.

Two vertical and one sub-horizontal drillhole, one sub-horizontal corehole and two trial pits were excavated at the site between 30 October to 8 November 1995 under Term Contract GE/95/06 (Bachy Soletanche, 1995[a]). A section through the retaining wall is presented on Figure 9. The ground behind the retaining wall comprises about 3.5m of fill overlying completely decomposed granite. The wall is about 2m wide at its base, thinning to about 0.5m wide at the top.

3.2.4 Trial Site D - Cape Collinson Crematorium. Feature No. 11SE-D/F19

Site D is a 16m-high north-facing, fill slope with an average inclination of 37° (Plate 4). It is located on the northern side of the Cape Collinson Crematorium (Figure 10). At the crest of the slope is a 80m-long, 9m to 15m wide platform covered with grass and trees (Plate 5). The surface of the fill slope is covered with dense vegetation of mainly shrubs and small trees. The surface materials at the top half of the slope consist of very loose fill with an abundance of broken glass. The lower part of the slope surface is composed of cobble and boulder-sized angular fragments of rock fill. Geophysical trials were made along two traverses TD01 and TD02 (Figure 10).

A total of 4 drillholes and 15 trial pits have been excavated previously. Six trial pits to between 1.3m and 2.2m below ground level were excavated by on 11 and 12 May 1987 under Contract 197 of 1985 (Gammon (Hong Kong) Ltd., 1987). Four drillholes to between 7.58m and 12.3m and 9 trial pits to between 1.3m and 3.2m deep were excavated between 18 November and 2 December 1995 under Term Contract GE/95/06 (Bachy Soletanche, 1995[b]). The location of existing ground investigation points are shown in Figure 10. A geological section is presented in Figure 11. The fill attains a maximum thickness of 5.7m. It thins down-slope to 1m to 1.5m at drill holes DH3 and DH4 and was absent in trial pit TP3. The fill is generally described as gravel and cobbles of weak to moderately strong angular tuff. Down slope the colluvium appears to be about 1m thick in trial pits TP1, TP2 and TP3. The colluvium is underlain by moderately to slightly decomposed tuff.

3.3 Phase 1 Results

3.3.1 Introduction

The Phase 1 field work was observed full time by personnel from Planning Division (GEO). Details of the contractors, including joint venture partners and key personnel, equipment, and breakdown of site works progress, are contained in Appendix C.

All of the contractors experienced difficulty working on the fill slope at Site D due to the loose surface materials and the abundance of glass fragments in the fill on the upper portion. Prior to the commencement of the field trials all visible glass fragments were removed. However some buried glass fragments worked to the surface as the contractors moved about the slope, imposing some hazard to the field work. Therefore surveys were only carried out from wooden walkways erected either side of the traverse from where they could be done in safety (Plate 4). Most contractors also found that the bamboo scaffolding erected for wall access tended to interfere with the deployment of the large low-frequency GPR and EM antennas. In hindsight it would have been better to provide moveable ladders rather than a fixed access.

3.3.2 Assessment of Results

The assessment was based on field and environmental constraints, limitations intrinsic to the method, degree of subjectivity in interpretation, and consistency of results among contractors in comparison with the geotechnical information. The detailed assessment of each of the geophysical methods tested is presented in Appendix D. Sections 3.3.3 to 3.3.10 summarise the main results from the Phase 1 field trials.

3.3.3 Shallow Seismic Reflection (SSR)

The high-resolution seismic reflection method is relatively simple and quick to carry out in the field and has good penetration depth. Sophisticated data processing is required to manipulate the raw data. Interpretation appears subjective, resulting in inconsistent results. Ground roll, seismic refraction events, environmental noise and back scatter are the major limitations of the method. No useful data were obtained by any of the contractors using this method. The feasibility of using the reflection method for site characterisation in Hong Kong is doubtful. Further trials were not warranted.

3.3.4 Spectral Analysis of Surface Waves (SASW)

The SASW method was not adversely affected by noise during the site trials but depth of penetration and consistency of results was significantly reduced by site constraints, complex ground conditions and voids below surface slabs. The depth of penetration was good where space permitted and resolution of 0.1m was achieved. There are some ambiguities and inconsistencies in the interpretations, especially with regard to the masonry wall traverses. The usefulness of the SASW method to determine wall thickness was limited and this aspect was not pursued specifically in Phase 2. However, the technique has an advantage over many other geophysical techniques because it measures shear wave velocity which can be related directly to geotechnical parameters. The technique was therefore tested further in Phase 2.

3.3.5 Resistivity Imaging (RI)

Generally consistent results were obtained despite the use of different electrode configurations and equipment. The RI method is quick and only limited data processing is required to produce an apparent resistivity pseudosection. Further modelling is required to produce true resistivity versus depth profiles. Along short traverses, the depth of penetration is limited to about 2m and data for the end portions of the traverse are difficult to obtain. The method can also be affected by the presence of metallic objects and electrical installations. Where a sufficiently long traverse allows, the technique has had some success in determining elevated moisture zones and the geometry of retaining walls. This method was tested further in Phase 2.

3.3.6 Self Potential (SP)

The method has severe limitations due to the interference of metallic objects and hard surface covering which prevent direct contact with the soil. None of the contractors managed to produce any useful interpretations. Further trials of this technique were not warranted.

3.3.7 Electromagnetic Methods (FDEM & TDEM)

The frequency-domain electromagnetic (FDEM) method is quick and easy to apply and surveys can be made by a single geophysicist. Its major limitation is the large anomalies caused by surface and near-surface metallic objects. Surveys are made on a grid to produce iso-conductivity maps. None of the contractors produced any useful interpretations. However, at sites free from metallic interference, the method may still be useful as a quick reconnaissance tool to identify shallow conductivity contrasts such as air-filled voids. Further trials of this method were therefore made in Phase 2.

The time-domain electromagnetic (TDEM) method using small coincident loop coil configurations is quick and requires one to two personnel to carry out a traverse. It has a good penetration range but the resolution is dependent on the sampling rate. The results suggest that the resolution was inadequate for the top 5m of the profile. The technique is also severely affected by near-surface metallic objects which produce large anomalies. No useful information was obtained by either of the contractors that practised the method in the field trials. Use of this method for site characterisation purposes in Hong Kong is not recommended unless the technology can be developed to allow higher resolution in the top 5m to 10m.

3.3.8 Ground Penetrating Radar (GPR)

Ground penetrating radar (GPR) surveys at all four sites could be made by two personnel in a few hours. The equipment is light and robust, especially the SIR 2 and pulseEKKO 1000 systems. Large, low-frequency antennas were however difficult to manoeuvre along sub-vertical and vertical traverses by one person. Investigation depths of 10m were reported using the 100MHz antenna. Resolution to 50mm was achieved with the 900MHz and 1GHz antennas. The most significant noise resulted from surface metallic objects along the traverse, although air wave interference to the 100MHz antenna was noted at Site C by GA. The electromagnetic wave velocity used by the contractors varied by as much as 50%. Uncertainty in determination of wave velocity is the most significant limitation to the method since it affects the depth-to-reflector calculations. GPR produced some reasonably accurate images of concrete slab thickness, voids below slabs, utilities and the thickness of retaining walls. The success of the technique does however appear to be contractor-dependent. Better results were obtained when the contractors carried out pre-data acquisition calibration. Different processing software used by different contractors, however, have produced highly variable radargrams for the same traverses. This technique was tested further in Phase 2.

3.3.9 Thermal Imaging (TI)

The method is quick and easy to carry out by one person. Limited processing is required. There are constraints of time, location and weather which limit its applicability. The results from BS and GA were ambiguous and not consistent. Further trials of this technique were not warranted.

3.3.10 Others

Seismic refraction was carried out by GA at all of the sites (see Appendix C). The technique was successful at Site D along TD01 where a simple two-layer case was modelled. The results identified the top of weathered tuff within 1m but could not differentiate between fill and colluvium. The method is useful only when the sub-surface layers are relatively uniform and sub-parallel to the surface. In view of the generally non-uniform ground conditions in slopes and retaining walls in Hong Kong, further trials were not warranted.

The sonic method in the form of PUNDIT testing of masonry blocks and concrete slabs was performed by BS. Seismic velocities were determined for some of the materials tested but otherwise this method was of little use and was not considered further.

3.4 Phase 1 Conclusions and Recommendations

(a) Phase 1 field trials were successfully completed between 5 December 1995 and 12 February 1996. The importance of selecting a competent contractor to carry out non-invasive engineering geophysical surveys was demonstrated.

(b) The results obtained by different contractors in some cases differed even when the same equipment was used.

(c) Based on observations of the fieldwork and the results, the geophysical techniques were divided into the three groups, with Groups 1 and 2 being tested further in Phase 2:

Group 1. Techniques which produced some promising results and may be applied widely to sites in Hong Kong. These are Ground Penetrating Radar and Resistivity Imaging.

Group 2. Techniques which could be severely affected by noise and limited by characteristics inherent in the technique, but might be usefully applied at specific sites. These are Electromagnetic Conductivity (frequency domain) and Spectral Analysis of Surface Waves.

Group 3. Techniques which did not provide any useful information and did not warrant further consideration in the Site Characterisation Study. These are High Resolution Seismic Reflection, Self Potential, Electromagnetic Conductivity (time domain) and Thermal Imaging methods.

4. PHASE 2 FIELD TRIALS

4.1 Introduction

The Phase 2 Field Trials were carried out by the five contractors at four sites between 1 October and 1 November 1996. Preliminary interpretative reports based on the geophysical results were submitted to GEO in late November 1996. Each contractor then gave a presentation of their results to GEO in early December 1996. Ground investigation (GI) carried out under the GEO term contract at the four sites commenced on 11 December 1996 and was completed on 11 January 1997. Factual reports on the GI were received in GEO on 4 February 1997 and were submitted to each of the geophysical contractors. Final interpretative reports based on the GI results were submitted by all five contractors by June 1997. This section of the report briefly describes each site, presents the results of the preliminary interpretations and briefly describes the GI results. Section 5.5 describes the comparisons between the preliminary interpretations and the GI data, final interpretations and conclusions made. Details of each of the contractors' personnel, techniques and equipment used are shown in Appendix C.

4.2 Phase 2 Field Trial Sites

4.2.1 Introduction

The initial criterion for selection of the Phase 2 sites was that they should be currently in the Landslide Preventive Measures (LPM) work programme with the GI planned for December 1996. This was desirable for the following reasons: the geophysical results could be integrated into the proposed ground investigation, a high density of GI points would be possible, topographic plans of the sites would be available and land matters and access would be easier to resolve. In the event, due to logistical reasons only one of the trial sites (Site G) was a pre-GI LPM site. Two masonry walls, one cut slope and one loose fill slope were finally selected. The two walls and the cut slope had no existing GI data. The fill slope (Site H) was included in the LPM 1995/96 Programme and had been investigated by Ove Arup and Partners under Agreement No. CE 45/94. Existing GI data therefore existed for the fill slope but were not released to the contractors prior to the geophysical field trials.

4.2.2 Trial Site E - Wall North of Stubbs Villa off Stubbs Road - Wall No. 11SW-D/R208

Site E is a 62m long dressed block masonry wall which reduces in height from 11m in the east to 0m in the west and is inclined at about 80° (Plate 6). It forms part of the retaining wall and cut slope complex which supports the building platform for Stubbs Villa located to the south of the wall (Figure 12). At the toe and crest of the wall is a 3.5m to 4m wide ramp which accesses Stubbs Road to the east. The masonry wall facing is composed of bonded, square (450 by 450mm), slightly decomposed granite blocks. The mortar bonding between each block was in poor condition at the time of the trials, showing signs of spalling, erosion and weathering, and was friable in places.

The masonry wall has three, 300mm thick horizontal concrete tie beams at +38.21mPD, +41.12mPD and +44.24mPD. The upper and lower tie beams are pinched out by the top and toe of the wall (Figure 12). Two rows of clay pipe weepholes with an internal diameter of 85mm and an outside diameter of 100mm are located in between the tie beams (Figure 12). Probing shows that the weepholes are generally about 4m deep in the eastern half of the wall with a few anomalous depths of 5m and above (Figure 12). The weephole depths reduce westwards to about 3.3m and then 2.5m in the central portion of the wall. The weephole depths then increase further west to greater than 5m close to the low end of the wall. Some of the weepholes showed evidence of persistent seepage such as algal growth around the hole and the build-up of calcite deposits below the mouth (Figure 12).

Several large trees grow at the top and part-way up the face, with aerial root systems covering large sections of the wall. A 100mm-diameter Towngas pipe runs vertically up the face of the wall and several manhole covers were located on the ramps (Figure 12).

4.2.3 Trial Site F - Wall adjacent to Hollywood Road Police Quarters Block B - Wall No. 11SW- A/R124

Site F is a vertical 41m long tied rubble masonry wall between 3.5m and 5m high (Plate 7) located north east of the Hollywood Road Police Quarters Block B (Figure 13). At the base of the wall is a playground and basket-ball court at a level of about +38.60mPD. A 5m-wide concrete covered platform at about +43.70mPD separates the wall from the quarters building. A masonry stair case links the playground to the platform (Figure 13). The change in height of the wall from 5m to 3.5m coincides with a reduction in the level at the top of the wall to +42.15mPD. The wall facing consists of irregular blocks of slightly decomposed granite and tuff. The individual blocks vary in length between 1m and 2m and are generally 0.5m to 0.75m wide. At certain levels in the wall there appear to be header blocks spaced horizontally at about 1.5m which are more regular in shape and size at about 400mm square and are mostly composed of granite (Plate 8). These are probably long rectangular blocks which tie the wall together in a semi-systematic pattern as described by Chan (1996). The facing blocks are bonded with cement mortar which was in poor condition at the time of the trials and was seen to be missing in places, revealing voids within the wall.

The wall has two levels of weepholes in the higher part (Figure 13). The weepholes are composed of clay pipe with an internal diameter of 85mm and a outside diameter of 100mm. The depths from weephole probing are shown on Figure 13 and range between 1.5m and 2.15m. The weepholes are inclined at about 17° to the horizontal. Several trees were growing along the top of the wall and in one place the root system has lifted the granite coping (Figure 13). Along the outer edge of the coping is a 3m high chain link fence. A series of manholes are equally spaced along the 5m wide platform at the top of the wall.

4.2.4 Trial Site G - Cut Slope opposite to Lions Morning Hill School, Blue Pool Road - Part of Slope No. 11SE-C/C51

Site G is a north-east facing, 20m high, 55° road-side cut soil slope (Plate 9) adjacent to Blue Pool Road (Figure 14). The slope is 40m wide at road level and has two 1.2m wide berms at +144mPD and +149.5mPD. Blue Pool Road falls from south east to north west from about +140.6mPD to +136.1mPD along the length of the slope. The slope is covered with chunam which at the time of the trials was cracked and spalled in several places. Prior to the field trials the site had vegetation growing through the chunam mostly at the south eastern end. In the spalled areas the exposed slope forming material was extremely weak, medium-pinkish brown, coarse-grained completely decomposed granite. The slope is bounded to the south by a vegetated natural slope. Surface drainage is provided by a series of 300mm U-channels along the toe and on each berm which feed into 300mm and 450mm wide stepped channel running along the north western boundary of the slope. A 250mm-wide inclined drainage channel with weepholes drains into the catch-pit at the base of the slope at the north-western end. A Towngas utility box is located at the north western end of the slope on the first berm and a series of pipes run down the slope from the box into the pavement at the toe (Figure 14). A metallic pipe filled with concrete protrudes from the lower slope at the south-eastern end (Figure 14). Further north-west of the pipe is a rock outcrop of moderately decomposed granite which protrudes through the chunam.

4.2.5 Trial Site H - Fill Slope adjacent to Sze Yu House, Choi Wan Estate - Part of Fill Slope No. 11NE - A/F125

Site H is situated in the north part of Choi Wan Estate, Kowloon. It is bounded by a small playground with a small carpark to the east, Clear Water Bay Road to the south, St. Joseph Primary School and a small shed to the west, and slope 11NE-A/F79 to the north (Figure 15). The slope has a maximum height of 7m, is 31m long and inclined at 26° (Plate 10). Prior to the field trials the slope was thickly vegetated with trees and small shrubs. A metal fence runs along the crest of the slope separating it from the playground. A 900mm-wide concrete drainage channel runs along the toe the slope and is covered by a concrete and cast iron grating over its full length. A 250mm U-channel is located at the crest of the slope. A twin 1650mm-diameter culvert and a 450mm diameter sewer pipe passes the slope to the east and are located about 6m to 7m below ground level (Figure 15).

The slope was previously registered as slope 11NE-A/F125 for which there are two Stage 1 Study Reports prepared in April 1993 and October 1994. Both reports characterise the feature as high consequence and further studies were required.

The slope was investigated by Ove Arup and Partners under LPM Contract No. GE/96/05 in August 1995. The results of the investigation are contained in the Stage 3 Study Report S3R14/96 (Ove Arup and Partners, 1996). The ground investigation was carried out under GEO Term Contract GE/93/11. It comprised two vertical boreholes, four trial pits and four GCO probe holes. *In situ* density tests were also carried out in the trial pits to determine the field density of the fill. The locations of the investigation points are shown on Figure 15 and the results are contained in the contractors report (Bachy Soletanche, 1995[c]). The *in situ* density tests and laboratory compaction results indicate that the fill is generally loose, with the degree of compaction ranging between 49% to 83% with an average of 71%. The geology at the site is described in Section 4.4 together with the more recent GI information.

4.3 Field Trials

4.3.1 Introduction

Based on the results and recommendations from the Phase 1 field trials, GPR, RI, FDEM and SASW were evaluated further at the four sites described above. GPR and RI were tested at all four sites, FDEM at the two slope sites and SASW at the fill slope only. In order to allow comparisons to be made between the contractors, predetermined traverses were selected at each site for testing the geophysical methods. The traverses are shown on Figures 12 to 15 inclusive. Extra traverse made by each of the contractors are described in their Final Interpretative Reports (see Section 6.).

This Section summarises the conclusions from the preliminary Phase 2 field trial results based on the suitability of each technique in terms of consistency of results, resolution, depth of penetration and the effects of noise. Typical radargrams, resistivity sections, iso-conductivity maps and shear wave velocity profiles are presented and discussed in Appendix E, which support the conclusions made in this section. Only selected results which best illustrate the conclusions are presented in Appendix E. For a full set of results the reader should consult the contractors final interpretative reports.

The GEO provided access at each sites. At the two wall sites, bamboo scaffolding was erected at 0.75m from the wall face to allow easy passage of the large GPR antennas up and down the walls. At the cut slope, bamboo scaffolding was also used but major structural elements had to be laid directly onto the slope surface to ensure stability. Although the grid was 0.75m from the slope surface, the elements on the slope surface were a major obstacle to the smooth running of the GPR antennas, especially for horizontal traverses. Vegetation growing out from the chunam was also removed prior to the trials. At the fill slope, vegetation clearance was made to provide a relatively un-obstructed slope surface apart from two groups of trees (Figure 15). No other access was provided at the fill slope.

The field trials were attended full time by GEO staff so that observations and records could be made of the works completed each day, the different methodologies and equipment used, and any difficulties experienced by each contractor.

4.3.2 Preliminary Field Trial Results

Based on the results of the Preliminary Interpretative Reports the following conclusions were made:

(a) Many of the GPR radargrams produced by different contractors appeared not to show the same information.

(b) The 100MHz GPR antenna appeared to be affected by air wave interference which limited its usefulness.

(c) The 500MHz GPR antenna has a maximum two way travel time of 50ns which effectively limits its penetration to a maximum depth of approximately 2.5m (based on a bulk electromagnetic wave velocity of 0.1m/ns).

(d) Some of the contractors interpreted gain controlled artefacts on the radargrams as reflectors.

(e) The general forms of the inverted RI sections produced by each contractor were generally consistent, but the details of these sections were variable. Such variations were due to some extent to the different contouring packages used.

(f) Several anomalies observed on the iso-conductivity maps for Sites G and H were caused by surface metal artefacts such as the Towngas pipes at Site G and the cast iron toe drainage gully at Site H.

(g) Different processing methodologies were used by the three contractors who presented SASW results in the Preliminary Interpretative Report. Consequently no consistent results were obtained for this method.

4.4 Ground Investigation

4.4.1 Introduction

Ground investigations were carried out by Enpack (Hong Kong) Limited under Contract No. GE/95/03. Full time site supervision of the GI was made by professional and trainee staff of GEO. The ground investigation design was based on the contractors' preliminary interpretations so that identified anomalies could be investigated. The investigation techniques used at each site were horizontal drillholes and trial pits at sites E and F, chunam strips and horizontal coreholes at site G and trial pits at site H. To ensure high quality core was recovered from the horizontal drillholes, triple tube HMLC core barrels were used in conjunction with an air-foam flushing medium. Generally good quality core was obtained with core recovery greater than 90% being achieved.

4.4.2 Trial Site E

Four horizontal drillholes, one vertical drillhole and two trial pits were excavated at the site at locations shown on Figure 12. Details of the drillhole and trial pit logs and core and trial pit photographs can be found in the contractors report (Enpack, 1997[a]) and are therefore not repeated in this report.

Interpreted cross-sections through the masonry wall based on the ground investigation data are presented on Figures 16 and 17. The section lines coincide with the two predetermined vertical wall traverses TE01 and TE02 (Figure 12) and are discussed below.

The wall height at Section "1E - 1E" is 9.5m. The wall is 0.95m wide at the top as exposed in TPE2. At about +45mPD the wall increases in thickness to 3.8m and to about 4.1m at +38.5mPD (Figure 16). Two lines of mass concrete haunching exposed in TPE2 coincide with service manhole covers and appear to be sitting on-top of the wall. The granite foundation for the upper retaining wall was also exposed in TPE2 and appears to sit directly on the wall, however this was not confirmed on site due to lack of space for excavation between the service haunching and the foundation blocks. The back face of the wall is approximately vertical, the increase in thickness due to the inclination of the front face of the wall. The wall thickness of 4m estimated from the weephole probing agrees well with the GI results.

The wall is composed of slightly decomposed cobbles and boulders of granite bonded together with an orangish brown to brown medium sand mortar. Some of the block interfaces have an orangish brown weathering rind up to 5mm thick. The vertical drillhole (DHE1) encountered four layers of mass concrete within the wall. The upper three coincide with the horizontal tie beams. The lower layer is 400mm thick and sits directly on completely decomposed granite and is interpreted to be the wall foundation (Figure 16).

The top and bottom tie beams appear to have been constructed perpendicular to the front face of the wall and hence slope back into the wall, whilst the middle beam dips out of the wall at a shallow angle. At +43.4mPD a fragment of a weephole was encountered in drillhole DHE1. This suggests that the weepholes at this level are inclined at about 20° to the horizontal dipping out of the wall. This does not agree with the probing results which indicated that the weepholes were essentially horizontal. This disparity could be due to the weephole fragment being displaced within the core-run during the drilling or extraction process.

The wall height at Section "2E - 2E" is 5.3m. The top of the wall is 0.68m wide as exposed in trial pit TPE1. The wall section at "2E - 2E" is more complex than at "1E - 1E" and appears to be a composite structure with the retaining wall above (Figure 17). The interpretation is that the wall is about 2.1m thick at about +42.0mPD and increases to about 2.5m thick at the toe. The back face is approximately vertical, the increase in thickness being due to the inclination of the front face. The wall is composed of slightly decomposed cobble and boulders of granite bonded together with a reddish brown and pinkish brown sand mortar. Behind the back face is a layer of fill which ranged from a sandy coarse angular gravel of strong granite in drillhole HDHE1 to a soft brown very sandy clay in drillhole HDHE2. This fill layer ranges in thickness between 0.8m and 1.5m. Behind the filled zone another layer of granite boulders and cobbles bonded together with pinkish brown mortar was encountered in both the drillholes. This layer is between 1.9m to 2.4m thick and is assumed to be part of the foundation of the upper wall. The back face of the foundation is vertical, behind which is a 0.9m thick fill layer composed of orange brown and pinkish brown mottled orange and white slightly clayey sand. Behind the fill, both drillholes encountered completely decomposed granite. The weephole probing confirm the composite nature of this section of the wall. Weepholes adjacent to the section line are either 2.5m deep, i.e. extend to the back of the masonry wall, or are greater than 5m deep and presumably extend through the masonry wall and the upper wall foundation and drain ground water from behind the foundation.

4.4.3 Trial Site F

Four horizontal drillholes and two trial pits were excavated at locations shown on Figure 13. Details of the drillhole and trial pit logs and core and trial pit photographs can be found in the contractors report (Enpack, 1997[b]).

An interpreted cross-section and horizontal section through the masonry wall ("1F - 1F" and "2F - 2F") are presented on Figures 18 and 19 respectively. The section lines coincide with the two predetermined vertical wall traverses TF01 and TF02 (Figure 13) and are discussed below.

At section "F1 - F1" the wall is 2.2m thick at the top as exposed in trial pit TPF1 and increases in thickness to 3.15m. at about +4.5mPD. A stepped profile to the wall has been assumed but a gradual increase in thickness of the wall below +41.5mPD could also be possible as indicated by the double-dotted line on Figure 18. The upper part of the wall has a 0.2m thick facing of dark grey fresh tuff. Behind the facing the wall is composed of grey honeycombed mass concrete, the aggregate consisting of strong angular granite gravel (drillhole HDHF2). The lower part of the wall has a 0.4m thick facing of dark grey fresh tuff. Behind the facing is 0.22m of grey mass concrete, the rest of the wall being composed of cobbles and boulders of tuff and granite at different grades of weathering. The matrix between the cobbles and boulders generally consists of silty fine to coarse sand (drillhole HDHF4). Both trial pits encountered an old dry granite sewer running parallel to the wall, see Figure 18 & 19. Orangish brown mass concrete encountered in drillhole HDHF2 has been interpreted as the mass concrete foundation to the sewer (Figure 18).

Figure 19 shows the different wall thickness at different elevations. At +40mPD the wall ranges between 3.15m to 3.3m thick along the 5m high section and reduces to 1.5m thick at the 3m high section. The wall at this level appears to be composed of cobble and boulder sized rock blocks of tuff and granite at varying grades of decomposition. The backfill behind the wall at this level is variable, ranging from a stiff reddish brown silt clay at drillhole HDHF1 to mixtures of gravel and soft clay a drillhole HDHF3 (Figure 18). The wall is about 2.1m thick at +41.5mPD along the 5m high section.

Weephole probing at the site prior to the ground investigation indicated that the wall was 2m thick in the area of the weepholes which corresponds well with the GI data (drillhole HDHF2).

4.4.4 Trial Site G

Four horizontal coreholes and four chunam strips were excavated at the locations shown on Figure 14. Details of the corehole and chunam strip logs and core photographs can be found in the contractors' report (Enpack, 1997[c]).

The chunam stripping and horizontal coreholes indicate that the cut slope is very homogeneous being composed almost entirely of extremely weak to very weak, reddish pinkish brown, mottled black and white completely to highly decomposed, medium grained granite with closely-to medium-spaced, smooth planar extremely narrow kaolin-and manganese infilled joints (Plate 11). The western end of the lower slope is composed of a dense brown clayey silty sand fill with angular gravel. Some zones of less weathered granite were noted especially along the lower part of the slope adjacent to the protruding corestone (Plate 12). Quartz veins up to 30mm thick were also encountered (Plate 13). The slope above the upper berm is composed mostly of residual soil with zones of less weathered granite.

4.4.5 Trial Site H

Five trial pits were excavated at the locations shown on Figure 15. Details of the ground investigation including trial pit logs and photographs can be found in the contractors' report.

A cross-section ("H1-H1") through the slope is shown on Figure 20. The fill is about 8.3m thick at the crest of the slope and decreases to about 1m thick at the toe. The fill overlies completely decomposed granite. Generally the top 1m to 2m of fill is a dense sand with angular rock, concrete and brick fragments up to boulder size. Below this fine-grained layer the fill becomes loose and extremely variable with layers of tuff boulders, fence posts, chain link mesh, sand and soft to firm clay. Trial pits TPH1 to TPH1C all encountered mass or reinforced concrete at about +27mPD (Figure 20). In trial pit TPH1B the concrete layer was horizontal and appeared to be the remains of a floor slab. The ground water level measured in standpipe piezometers installed in drillholes VBH-1 and VBH-2 and as encountered in trial pit TPH2 was at +24mPD.

4.5 Comparison of Preliminary Interpretation with Ground Investigation Data

4.5.1 Introduction

Figures 21 to 23 summarise the preliminary interpretations made by each contractor at all four sites and compares them with the preferred interpretations based on the GI data. Each figure is discussed in turn. A detailed assessment of the preliminary field trial interpretations with accompanying results can be found in Appendix E.

4.5.2 Trial Sites E and F

Summary information for the two masonry wall sites is presented on Figure 21. The wall geometry determined from the GI is presented along the left hand side of the figure.

GSC and IGGE both interpreted the wall along traverse TE01 to be stepped, with increases in wall thickness coinciding with each tie beam. This model appears to be based on a preconceived idea of what the wall should look like rather than from information obtained from the geophysical results. As shown in Appendix E, some of the boundaries between different geological or man-made materials have been determined from poor-quality data. GA suggested that the wall is 3m thick and that the back face is parallel to the front of the wall. The three zones identified by GA were not identifiable from the GI results. Also, the zones of enhanced moisture identified by GA mainly from the RI results could not be confirmed from the GI (see Appendix E). FGS's interpretation suggested that the wall is only 1m thick and shows some reflectors at 2m, 6m and 9m in from the wall face. Most of these reflectors have been interpreted from GPR data and are likely to be gain-controlled artefacts rather than true reflectors (see Appendix E). BS presented data in the form of different zones of reflection coefficient and continuous reflectors based only on the GPR results with no interpretation of the wall geometry. The wall thickness was estimated to be 4m from the weephole probing. This is more accurate than any of the preliminary geophysical interpretations and agrees well with the GI data.

GSC assumed a stepped profile for the section along TE02 with the wall being 4m thick at the base, stepping to 2m thick above the tie beam. IGGE interpreted the wall as having a fairly constant thickness of 2m at the base, thinning to about 1.5m at the top. This agrees fairly well with the interpreted thickness of the wall from the GI information; however the composite nature of the structure was not identified. GA's interpretation the wall as being a constant 3m thick and composed of three zones does not agree with the GI data. FGS and BS's interpretations for TEO2 are similar to TE01. Weephole probing adjacent to TE02 gave variable results, with some weepholes being about 2.5m deep whilst others are greater than 5m deep. For this section weephole probing did not yield any better estimates of wall thickness than the geophysics.

At Site F none of the contractors picked up the increase in base thickness of the wall from 2.1m to 3.2m at TF01 (Figure 21). GSC interpreted the wall as being 2m thick with fill behind whilst IGGE suggested that the wall was 1m thick with fill behind. GA interpreted the wall as 2m thick with a zone of decreased void content in the upper 3m. This zone of decrease in voids coincides well with the area of wall composed of mass concrete rather than boulders and cobbles of partially weathered rock (see Appendix E). The zone of stronger reflections in GA's interpretation also appears to coincide with the base of the dry sewer foundation. Both FGS and BS marked various reflectors without attaching any interpretation. For this interpretation, an estimate of wall thickness from weephole probing would have been as good as any of the geophysical interpretations; however some of the interpretations did provide information regarding the spatial distribution of voids within the wall which obviously could not be determined from weephole probing or conventional GI alone. Along the lower part of the wall the GI confirmed the interpretations made by GA, BS, IGGE and GSC that the wall reduced in thickness to about 1.5m (Figure 19).

In summary, the preliminary interpretations made by each of the contractors at the two wall sites are inconsistent, with interpreted wall thickness ranging from 1m to 4m for TE01, 1m to 4m for TE02 and 1m to 2m for TF03. At Site E the results from weephole probing to determine wall thickness were better than or at least as good as the geophysics. At Site F the results from weephole probing to determine wall thickness were as good as the geophysics but the geophysics provided more information regarding the spatial distribution of voids within the wall.

4.5.3 Trial Site G

Interpretations from GPR radargrams constructed from data obtained using 500MHz antennas along TG03 at Site G are presented in the top three diagrams on Figure 22. A single reflector parallel to the slope surface at 12ns is presented by IGGE. BS also present a single reflector parallel to the slope surface at 9ns. They also show a series of discrete reflectors which are interpreted as less weathered corestones within the weathered rock mass. GA also present discrete reflectors interpreted as corestones, and possibly discontinuities. The location and depth (two-way travel time) of the discrete reflectors presented by BS and GA are not coincident. Interpretations made from radargrams produced using 100MHz and 35MHz antennas by GSC and FGS respectively are presented on the bottom two diagrams on Figure 21. It should be noted that the two-way travel time axis is increased to 200ns for the 100MHz and 300ns for the 35MHz antenna. Three reflectors parallel to the slope surface have been interpreted by GSC from the 100MHz data. The reflectors at 120ns and 200ns are both considered to be artefacts in the data and not true reflectors. FGS present anomalous zones, discrete deep reflectors and a continuous shallow reflector, all of which are considered to be artefacts in the data and not true reflectors.

It is apparent from the data presented above that reflectors within the weathered rock mass identified on radargrams by each of the contractors at Site G are not consistent. It is considered that some of the contractors have interpreted artefacts as true reflectors.

4.5.4 Trial Site H

Preliminary interpretations made by each contractor along traverse THO1 are compared to the GI information on Figure 23. GSC, IGGE, GA and BS all show a boundary parallel to the slope surface at about 1m, interpreted as the interface between fill and the underlying geology. However, from the GI data it appears that the interface is probably the boundary between dense sand fill and underlying loose fill below. The zone of “no GPR penetration” identified by GA coincides approximately with the large concrete boulder identified within the fill during the GI.

4.6 Final Interpretation

Final interpretative reports based on the preliminary geophysical results and the GI information have been produced by all five contractors (Bachy Soletanche, 1997, Fugro Geotechnical Services (HK) Ltd., 1997, Guandong South China EGTD, 1997, Institute of Geophysical and Geochemical Exploration, 1997 and Golder Associates Inc., 1997). Each contractor was able to refine his preliminary interpretation to a certain extent. However, it is considered that no additional major conclusions (over and above those made from the preliminary interpretative reports) into the usefulness of these methods for site characterisation in Hong Kong was gained from the final reports.

5. CONCLUSIONS AND RECOMMENDATIONS

The main conclusions from Phases 1 and 2 of the Site Characterisation Study are:

i) The overall quality of the raw data and interpretations varied significantly according to the expertise of the individual geophysics teams.

ii) Certain contractors were able to demonstrate that a combination of ground penetrating radar and resistivity imaging can determine the back of masonry walls if less than 3m thick and locate zones of elevated moisture content and voids with reasonable accuracy.

iii) Limited success was achieved in determining the location of corestones and the thickness of loose fill at the two Phase 2 slope sites.

It is recommended that:

(a) Due to the inconsistent raw data and interpretations it would not be advisable at present to let a term contract specifically for non-invasive geophysical techniques for the determination of masonry wall thickness in Hong Kong, or to require the use of such techniques in LPM studies of walls.

(b) Further trials at present of the four selected methods (namely ground penetrating radar, resistivity imaging, electromagnetic conductivity and spectral analysis of surface waves) or other geophysical methods are not warranted.

(c) Since some of the results were encouraging it is recommended that further research on developing GPR and RI for masonry wall investigations is carried out through a local university with support from GEO. The research work should focus on ways to enhance the resolution of the two techniques and also to develop a site and data processing methodology which would ensure more consistent results from different contractors. Also, local expertise in the two methods could be developed in this way.

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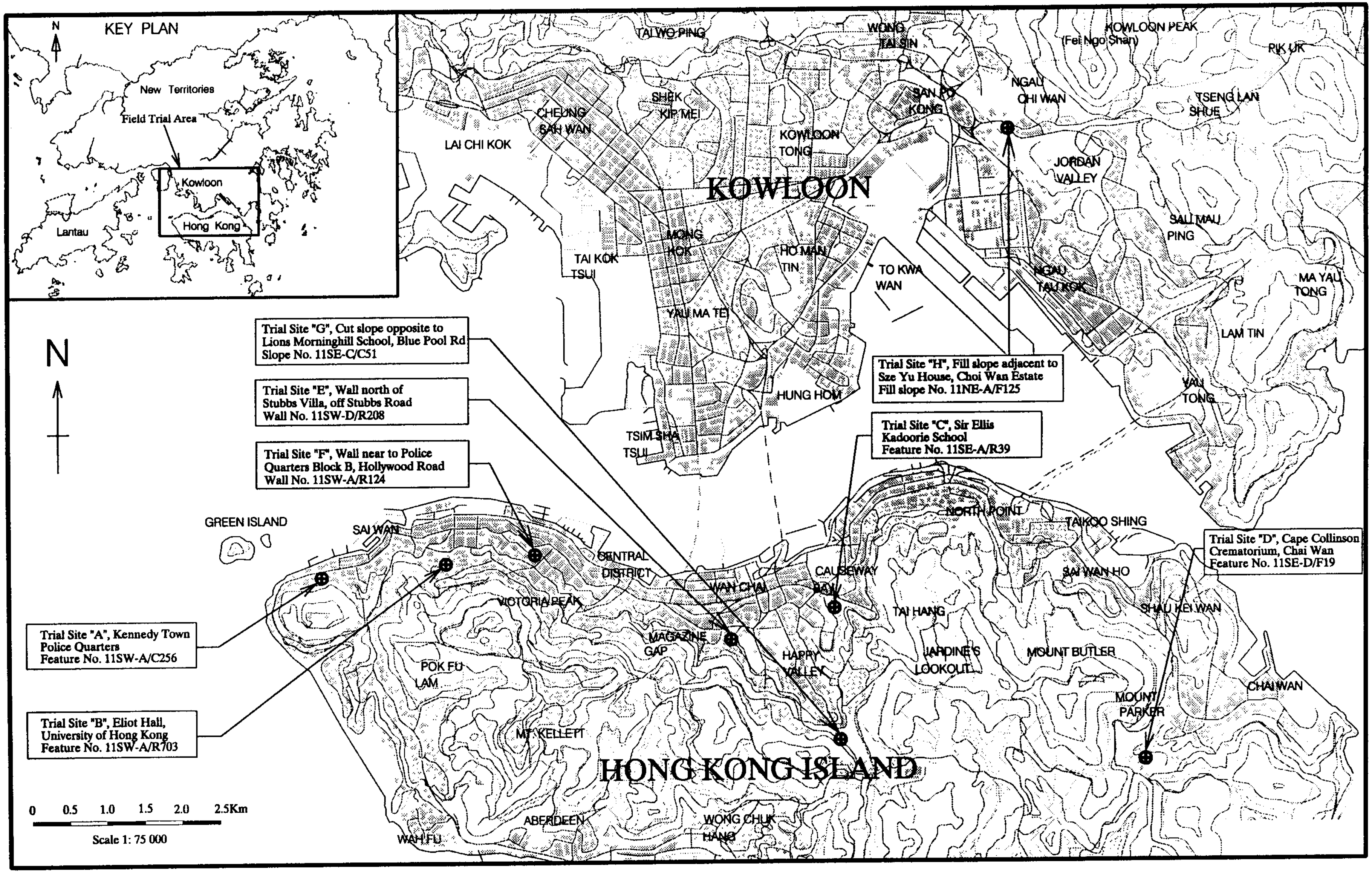

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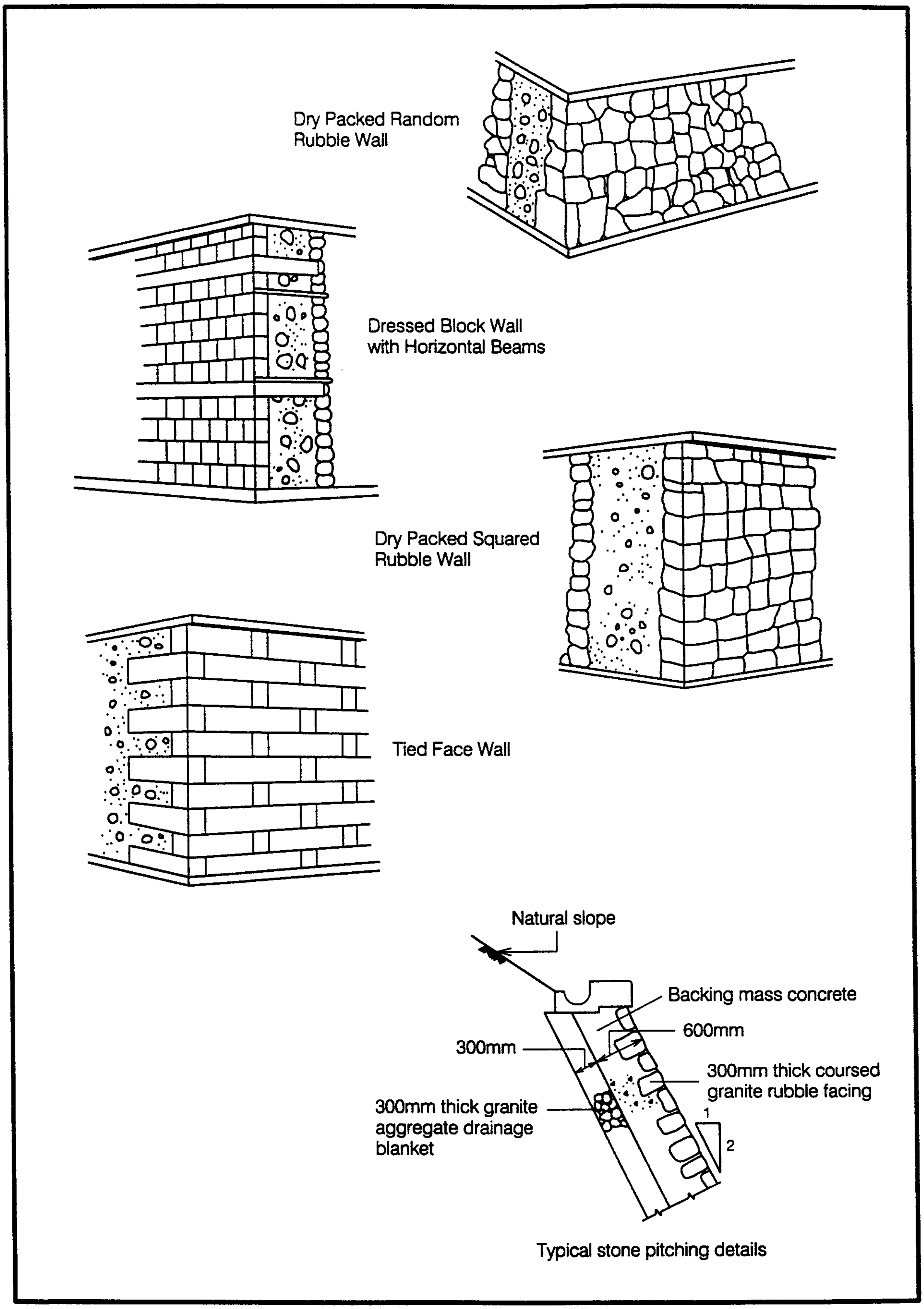

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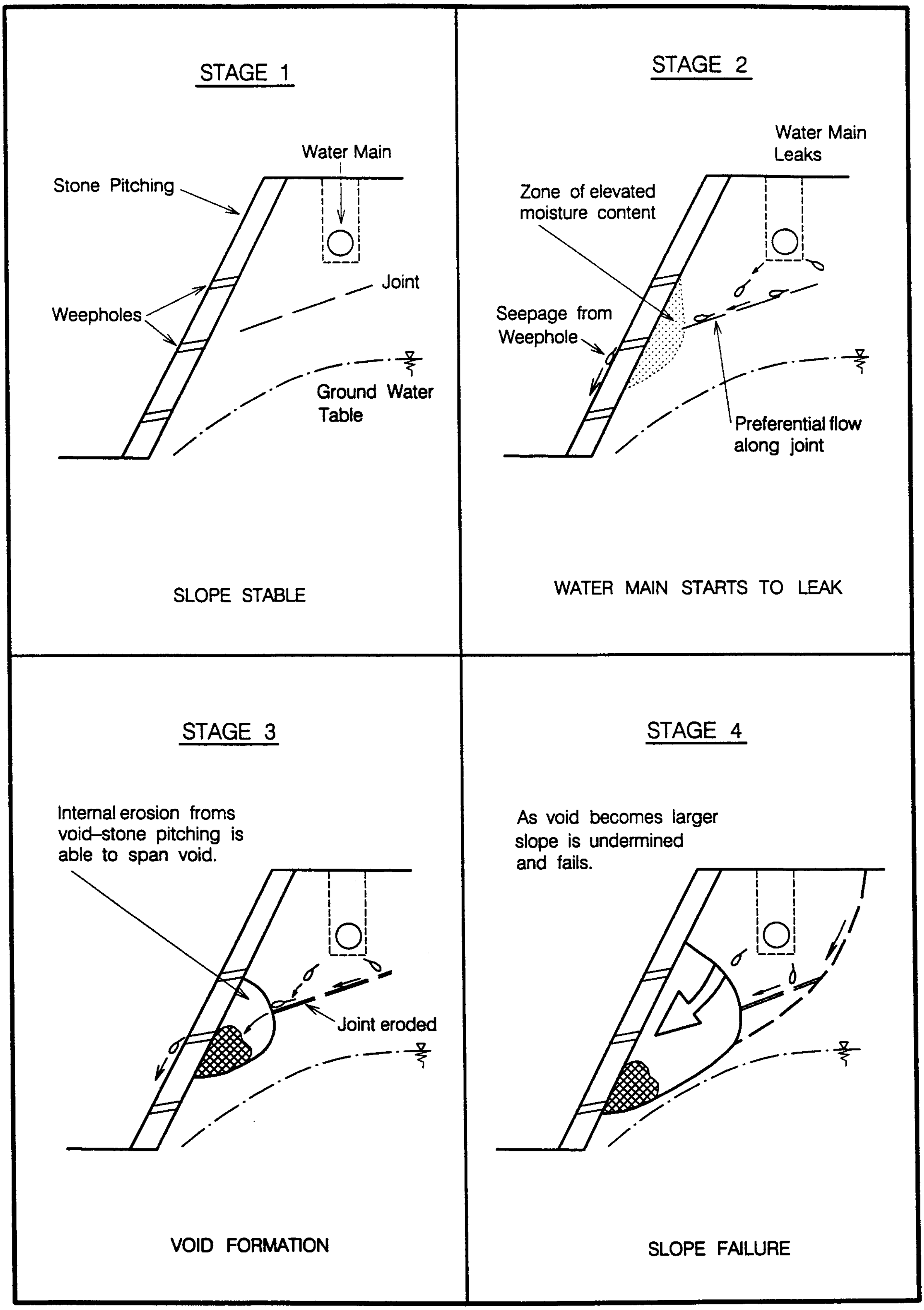

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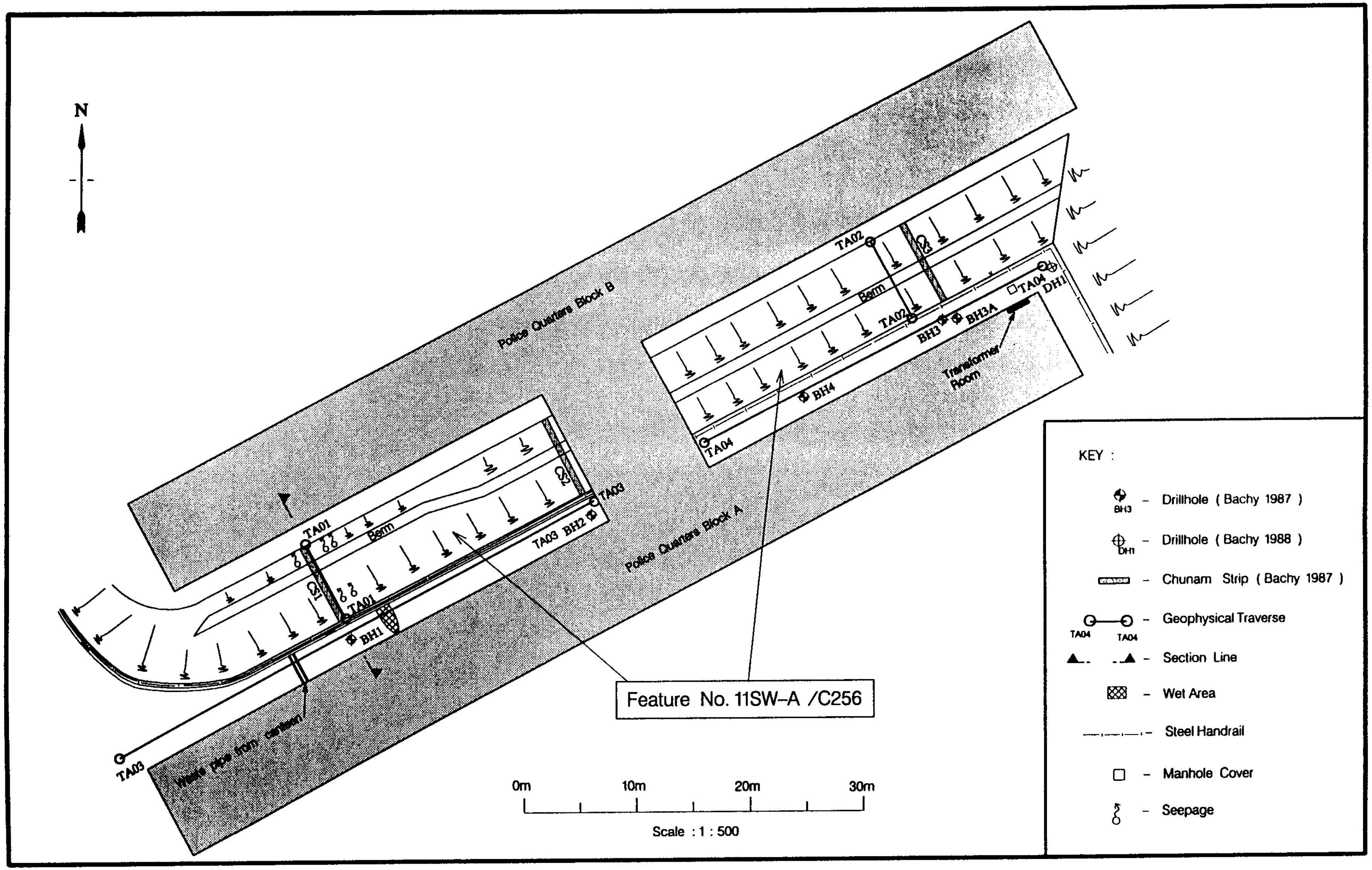

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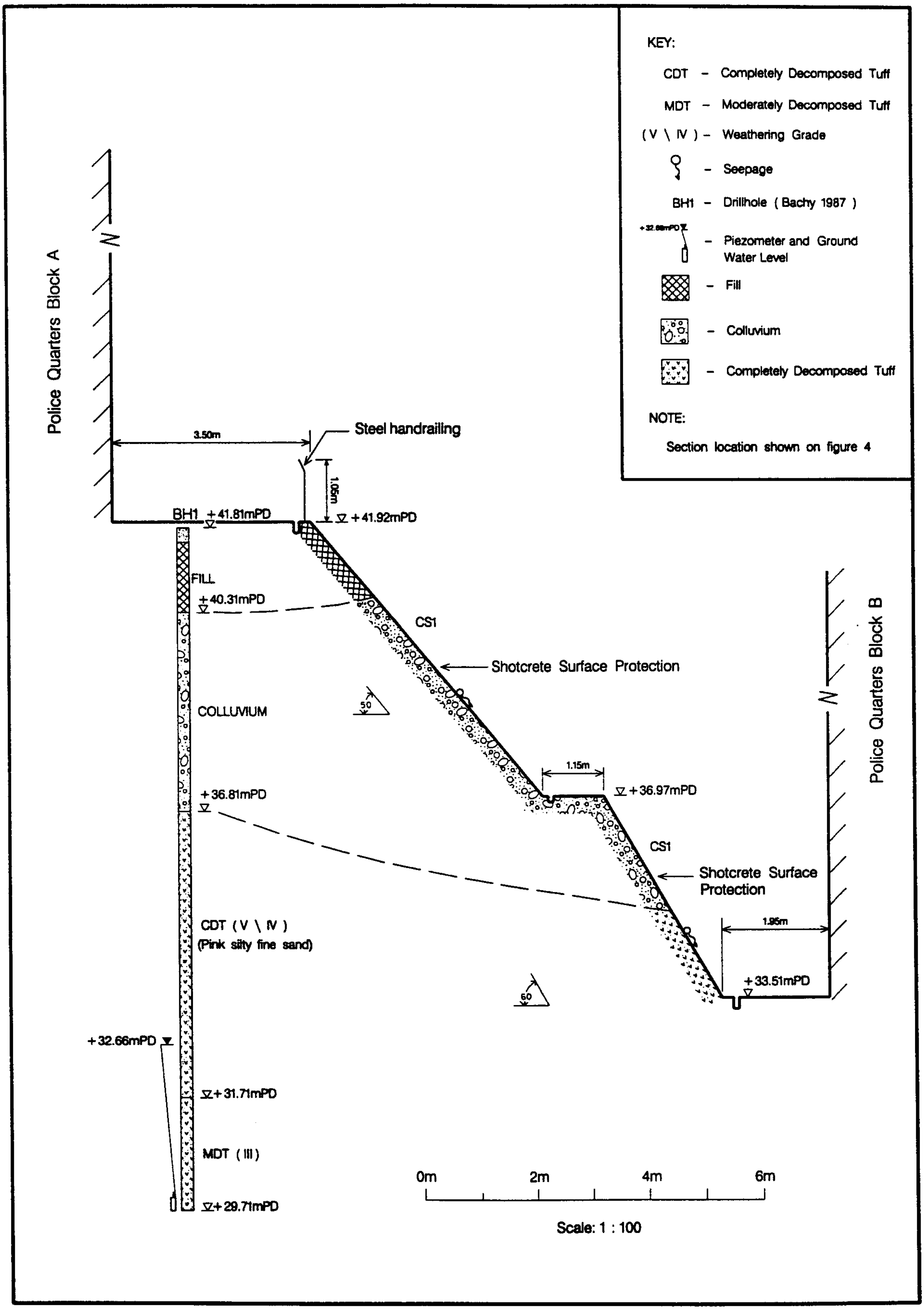

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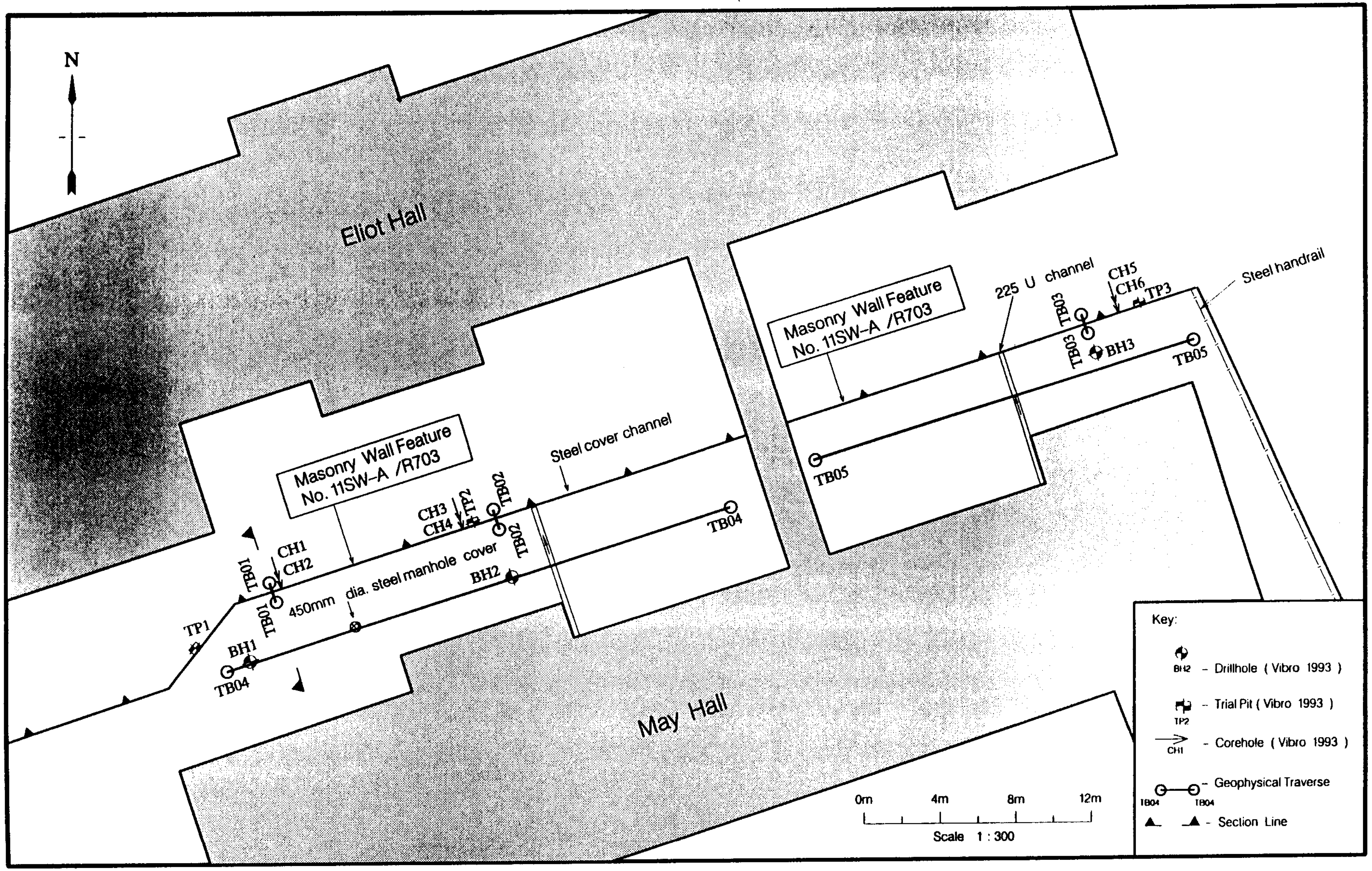

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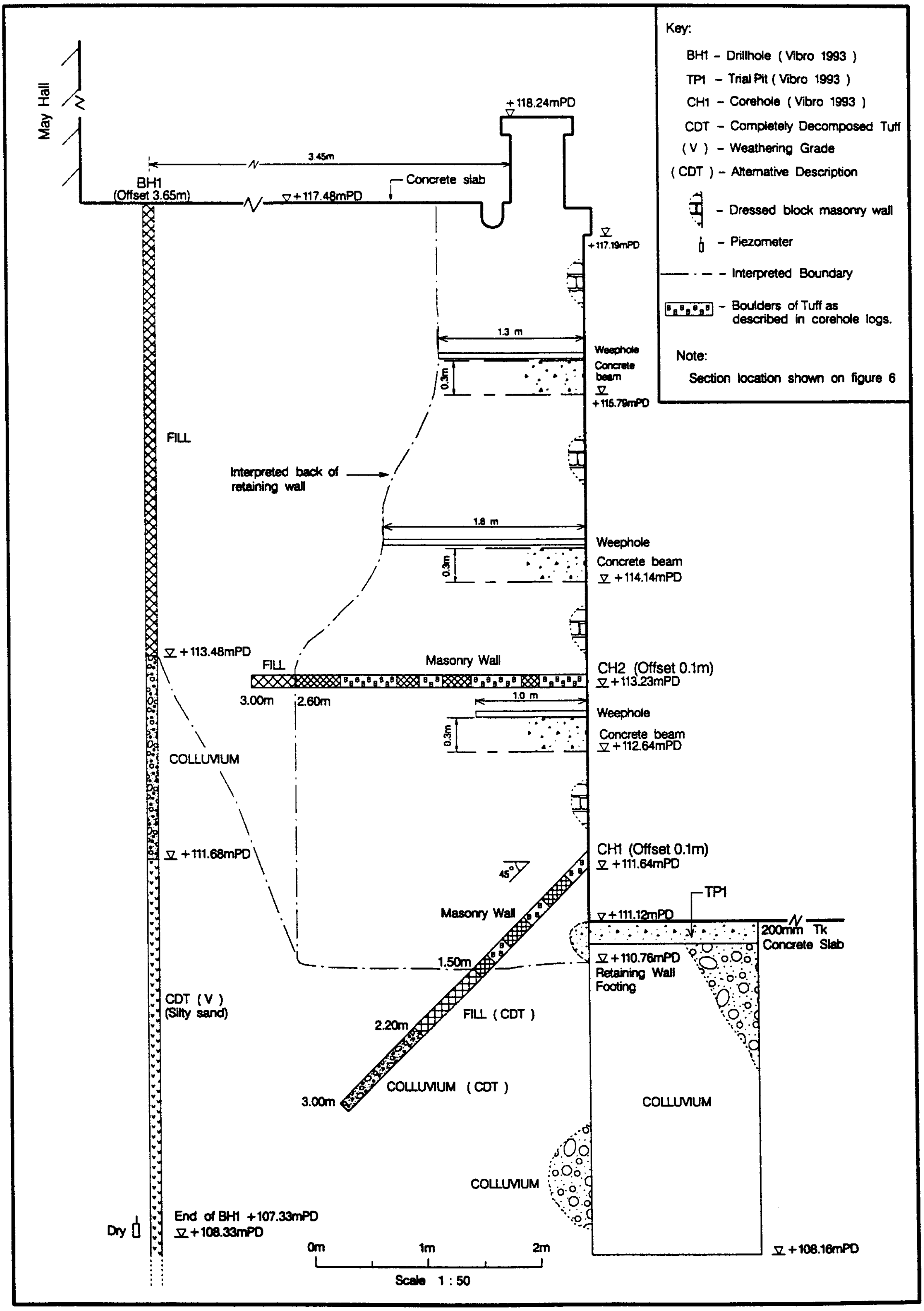

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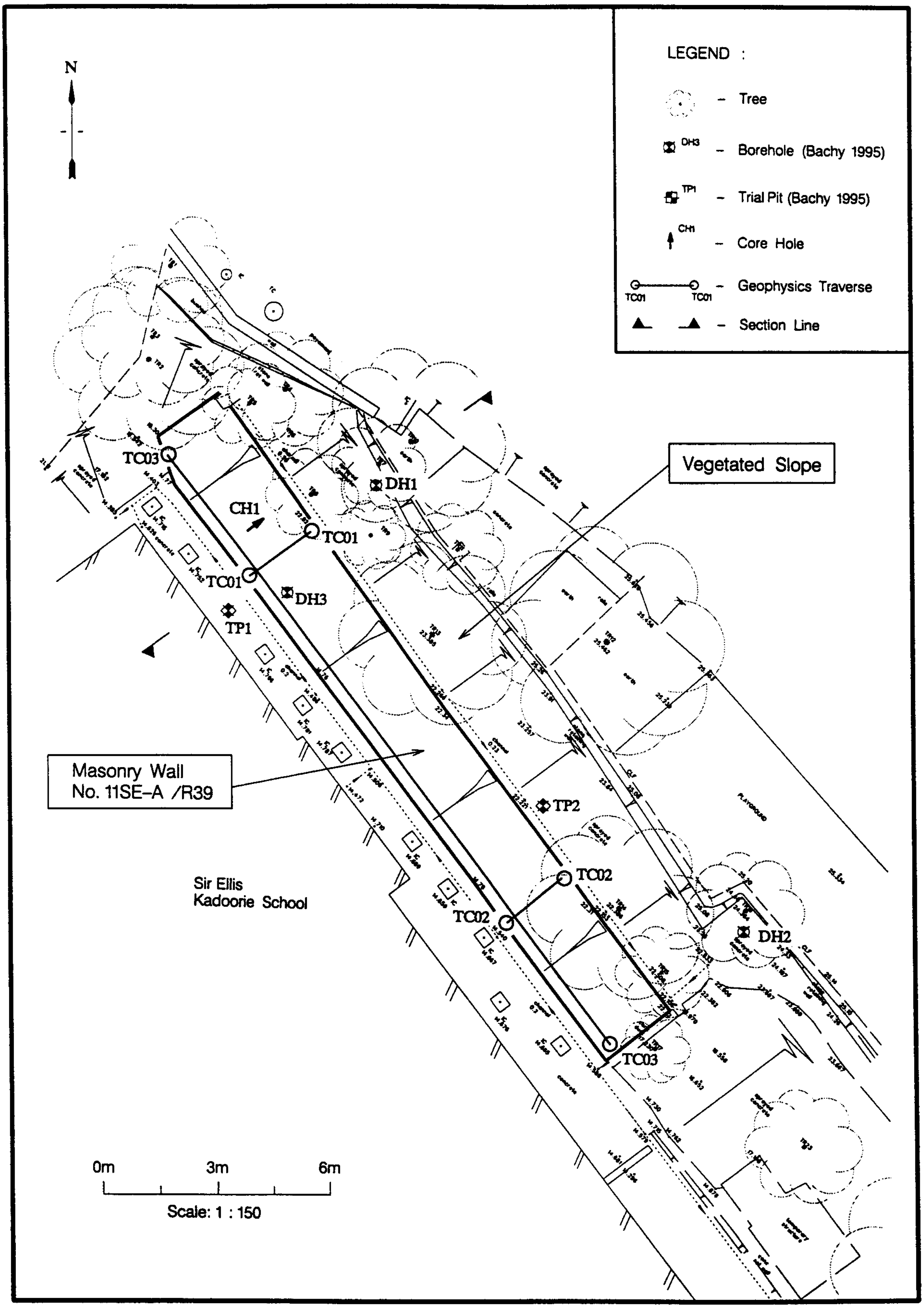

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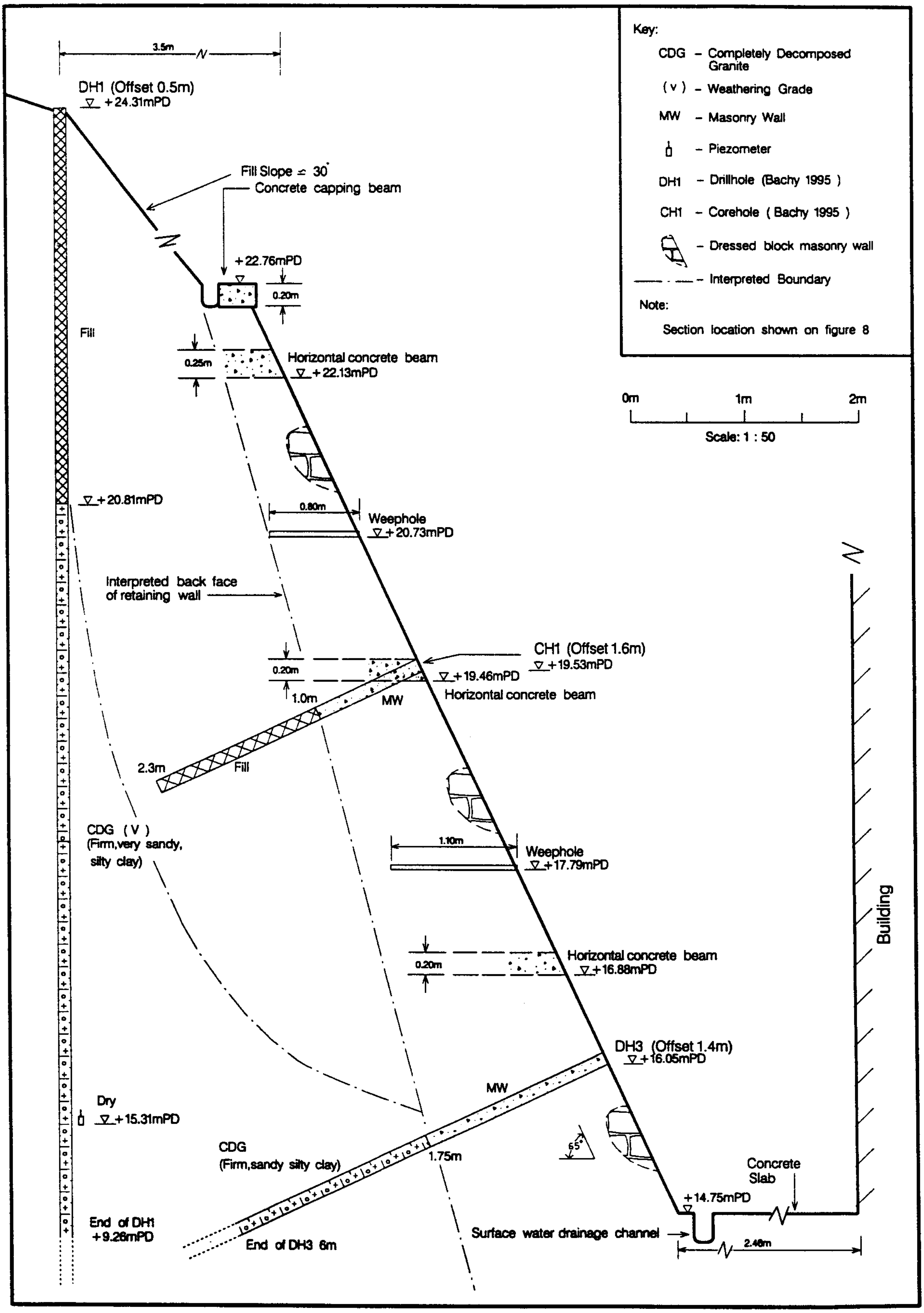

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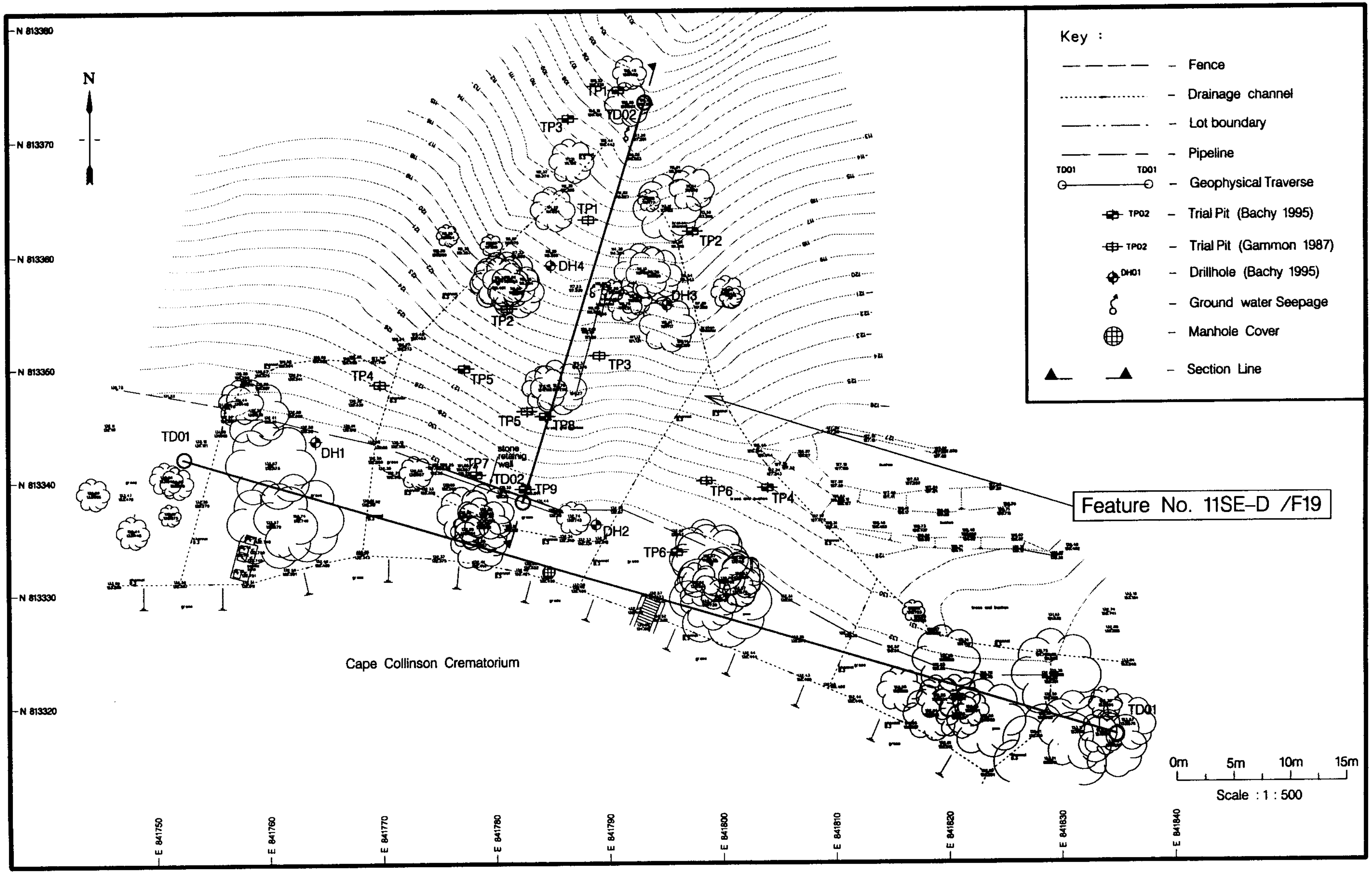

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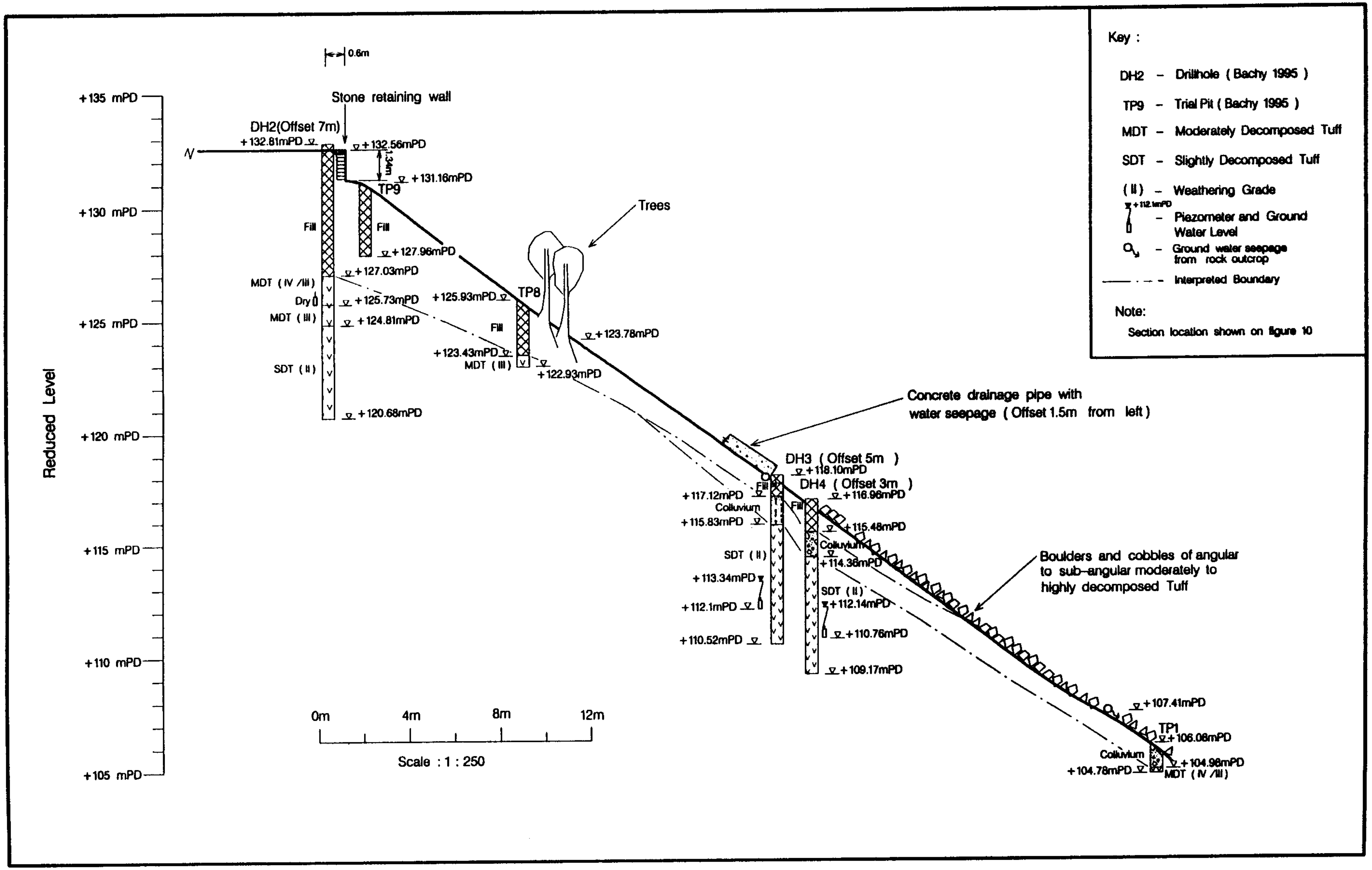

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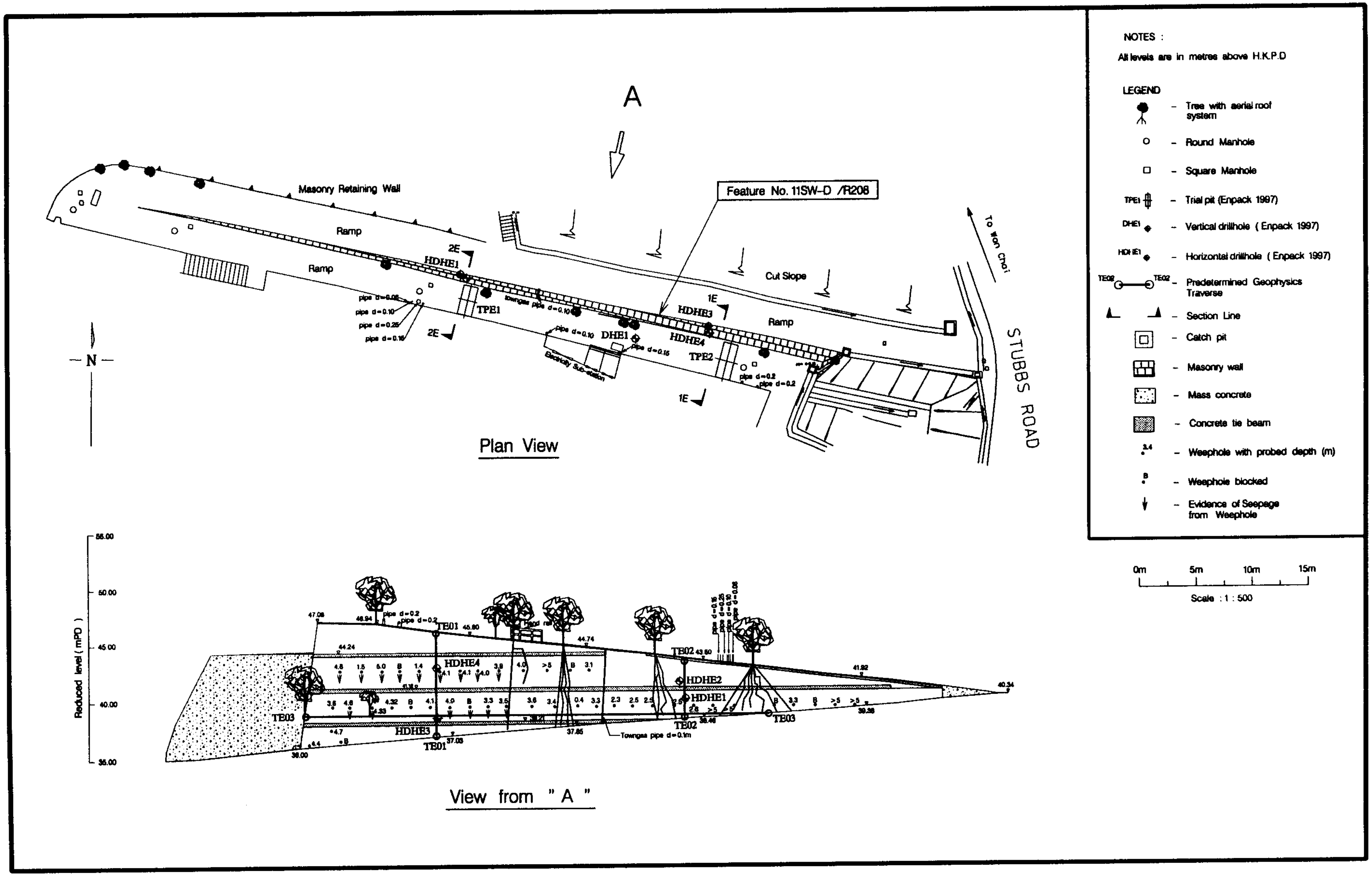

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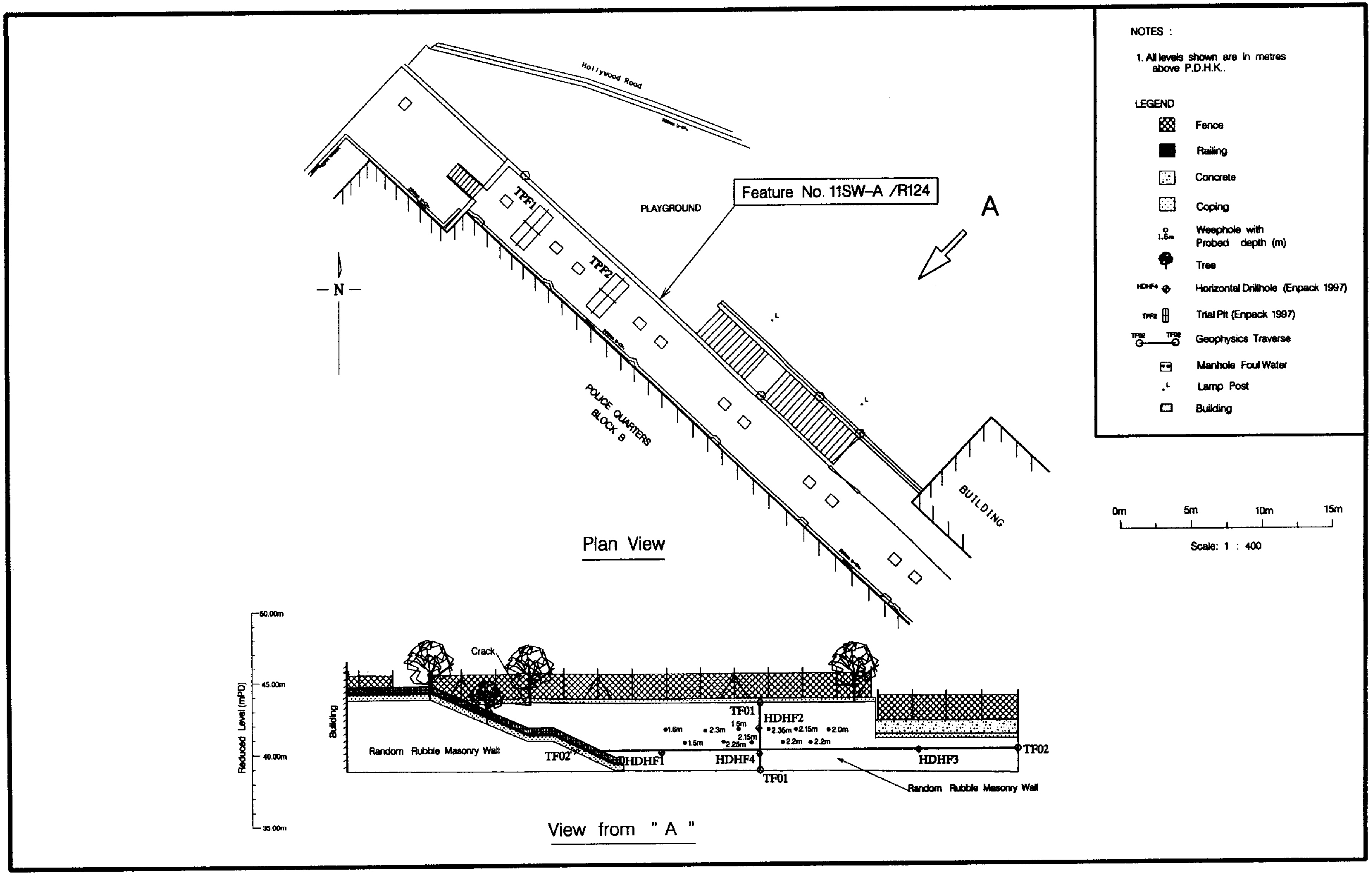

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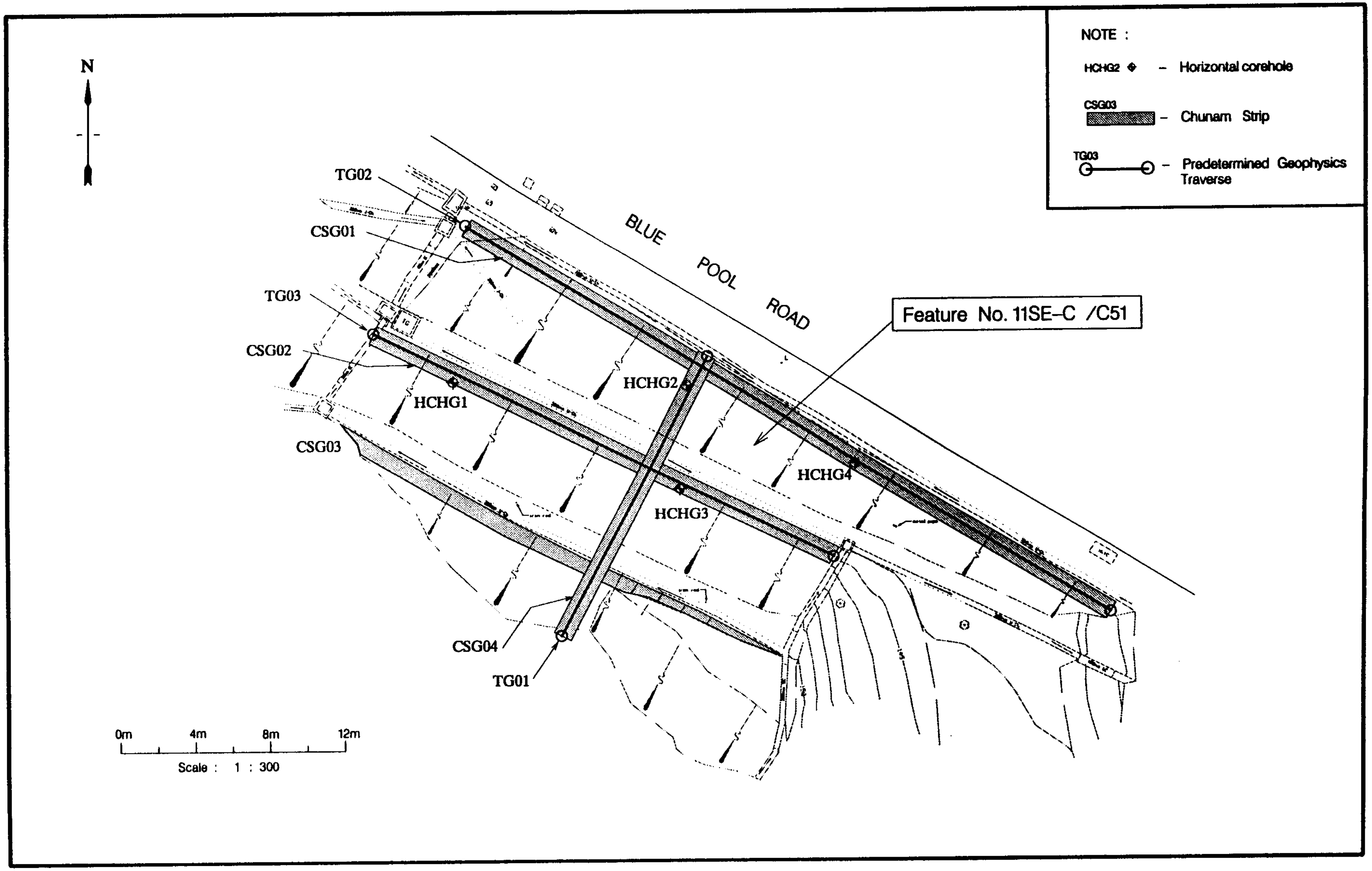

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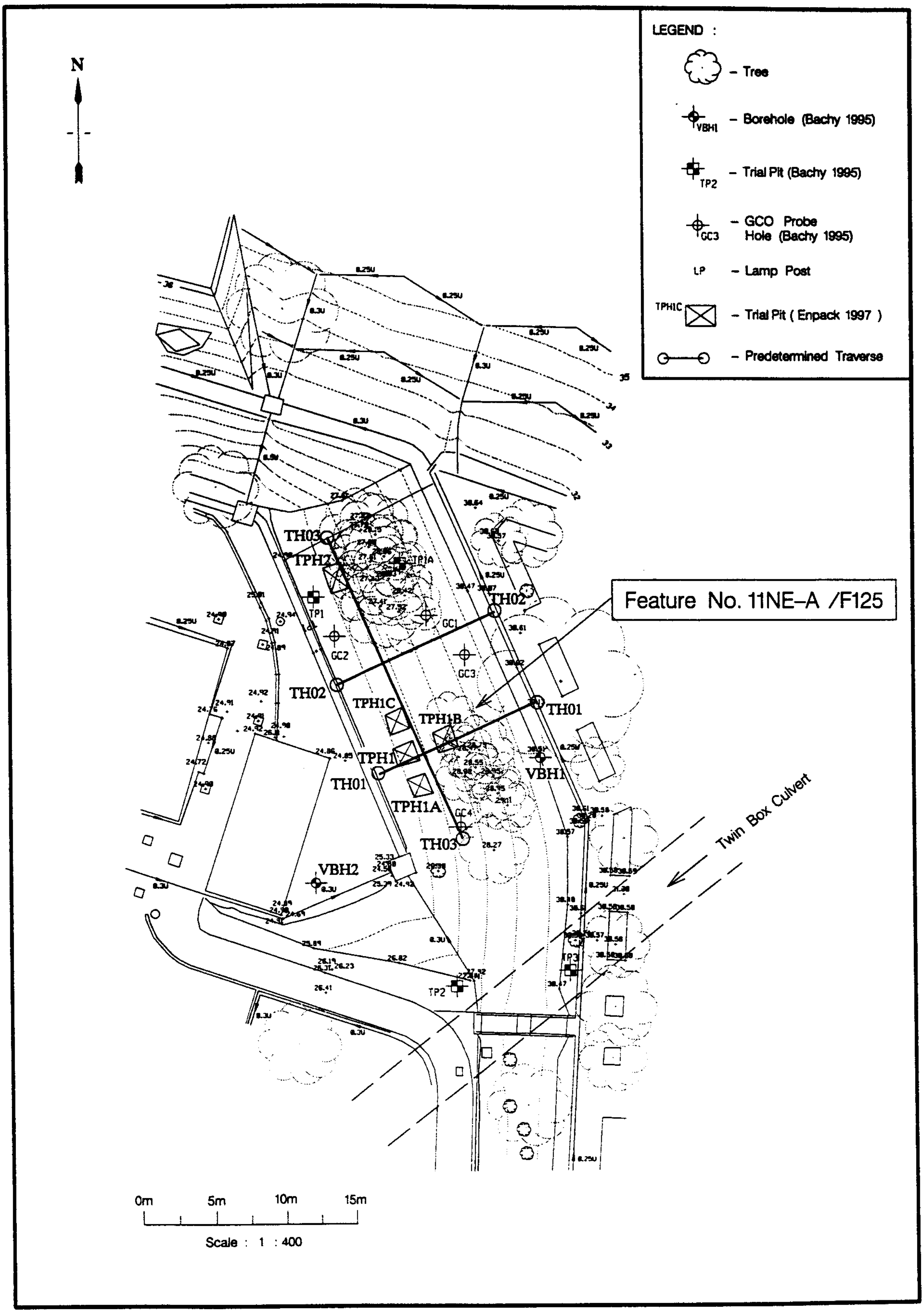

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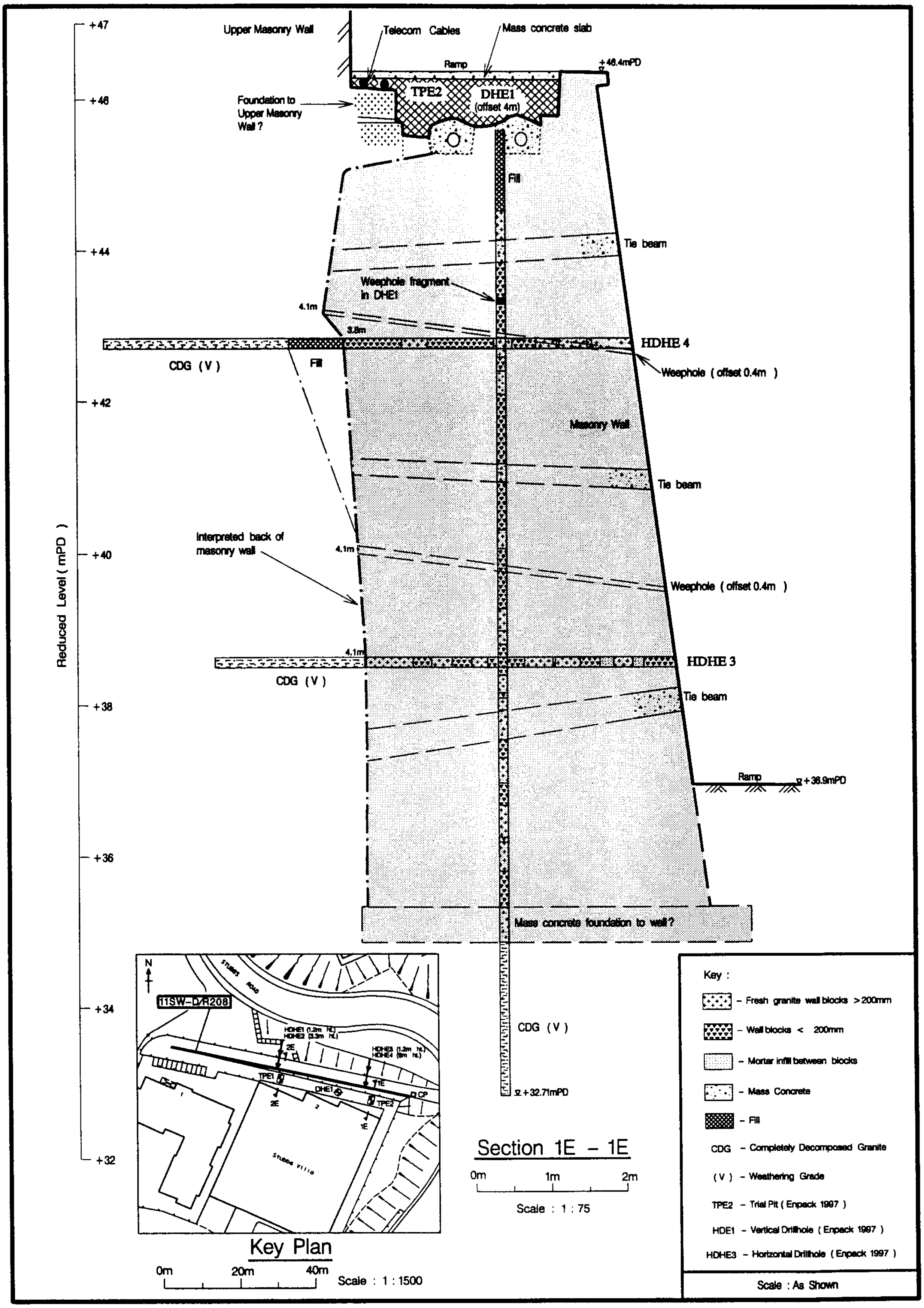

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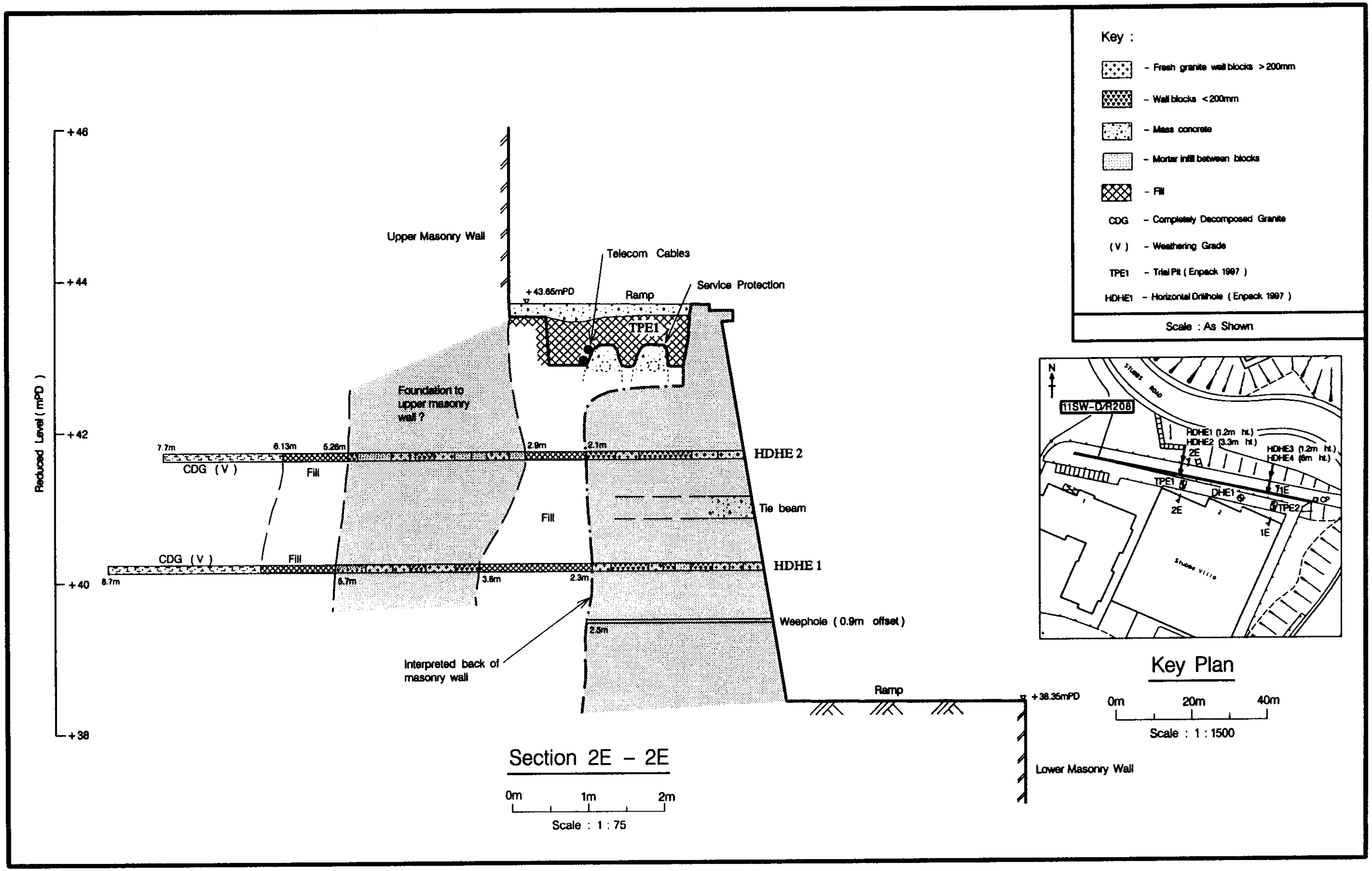

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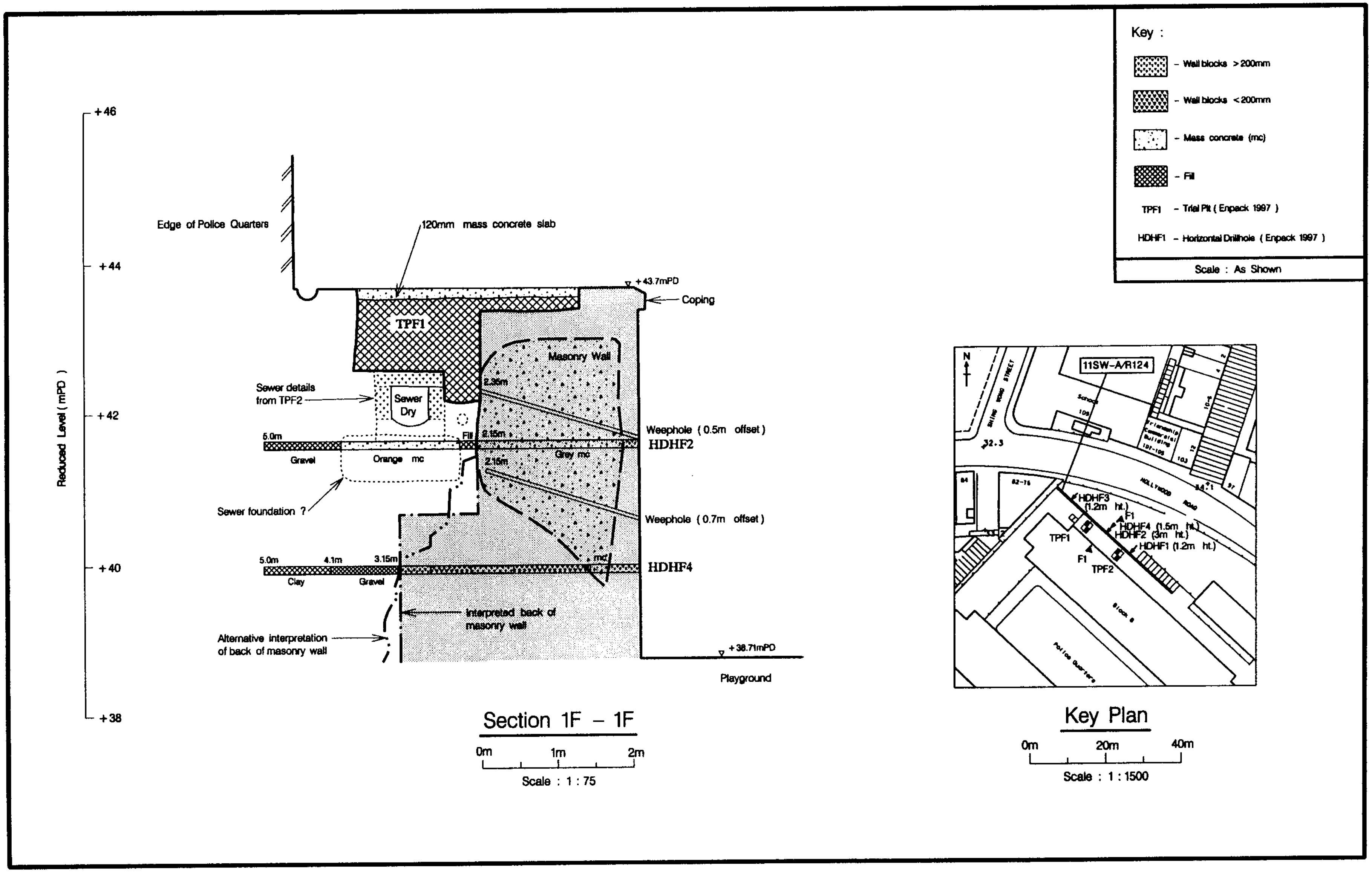

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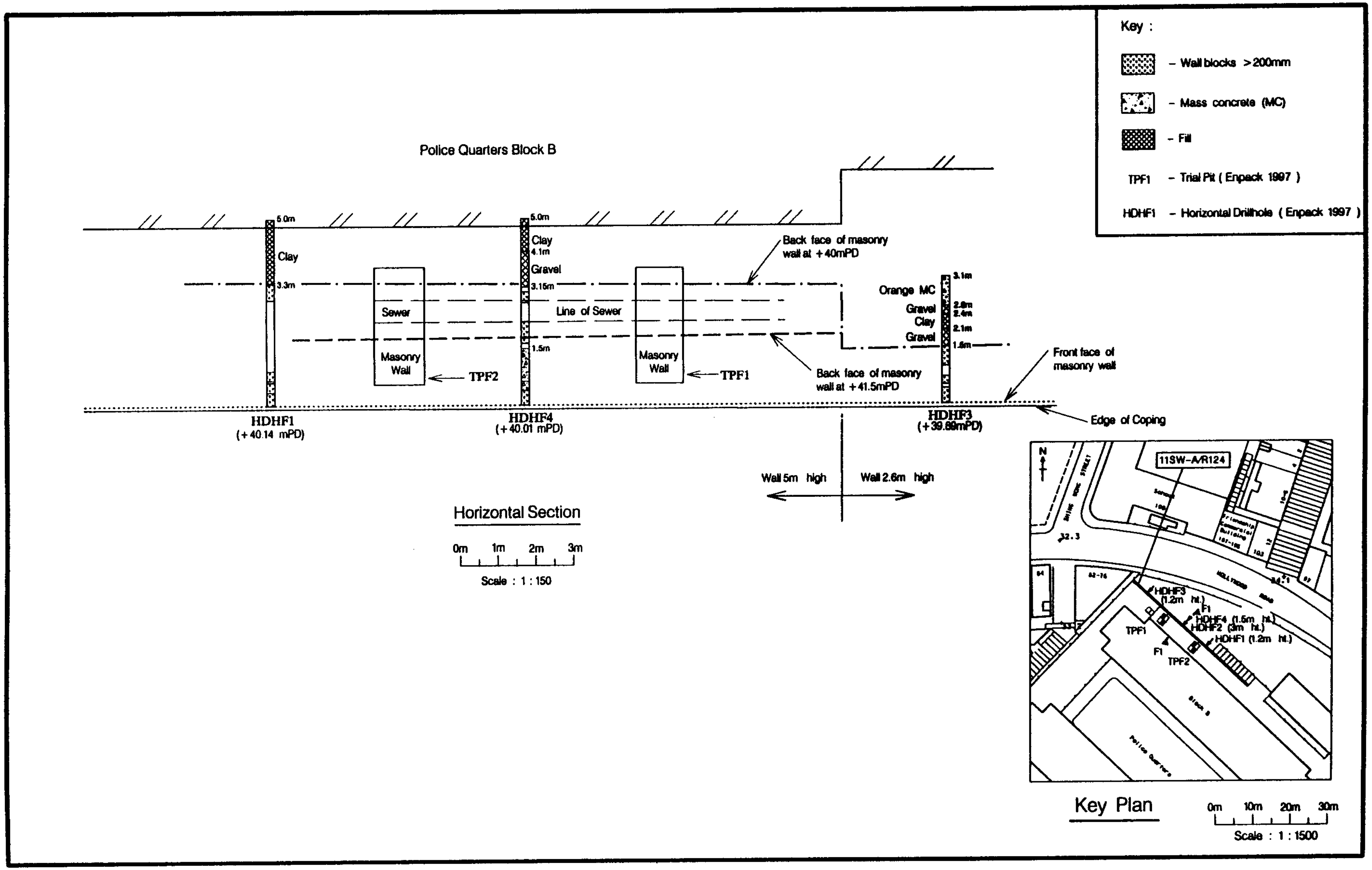

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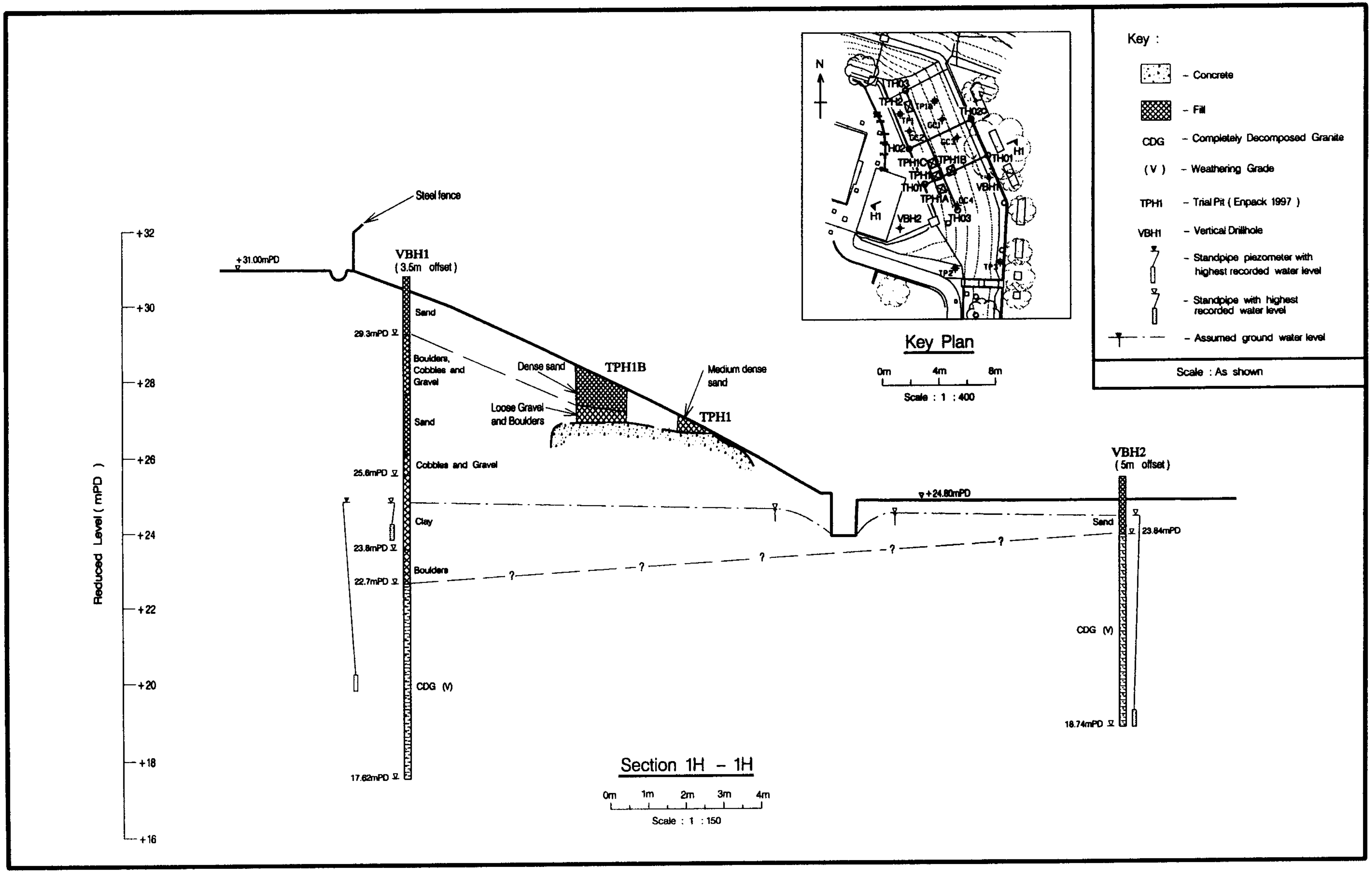

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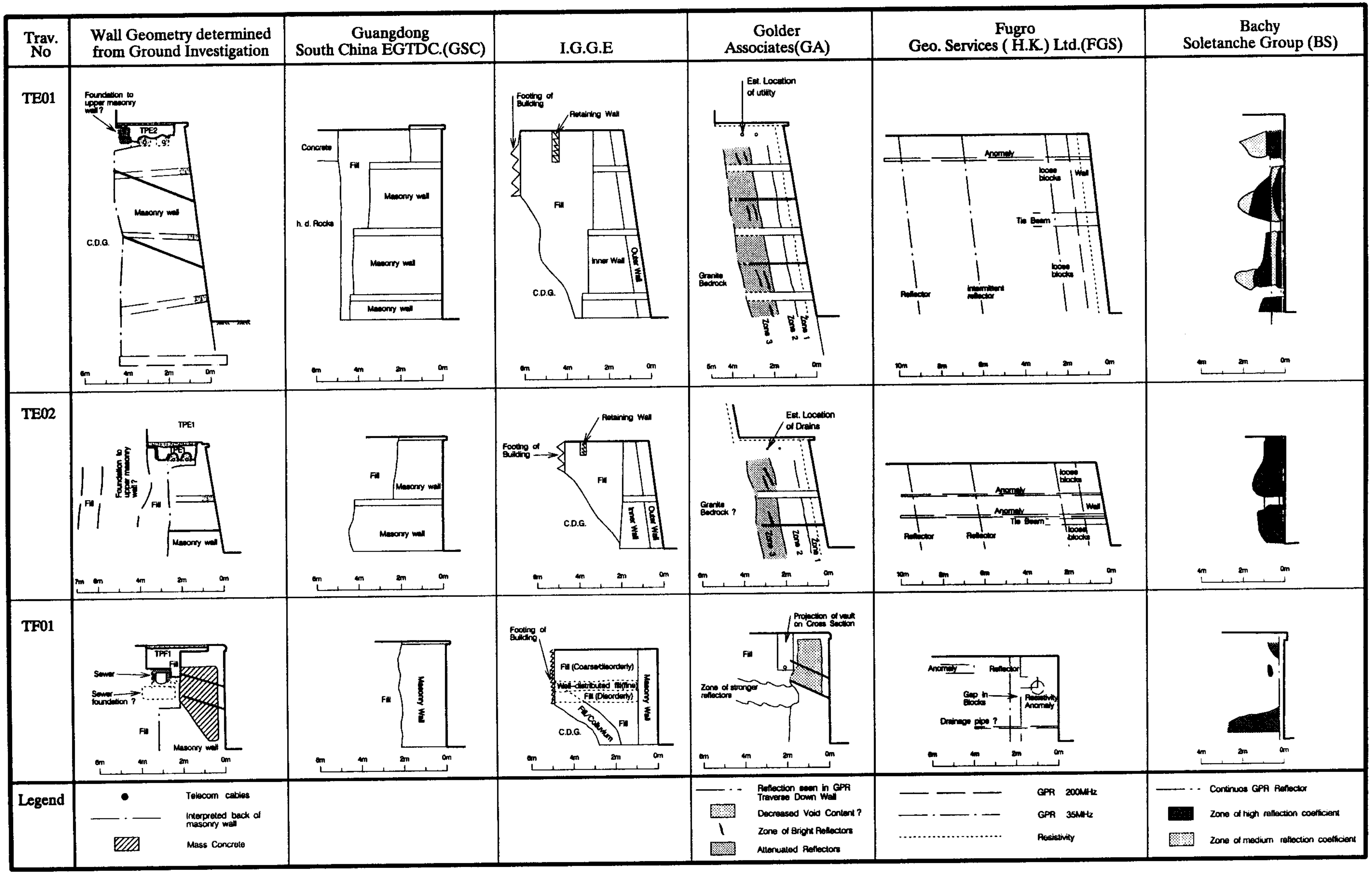

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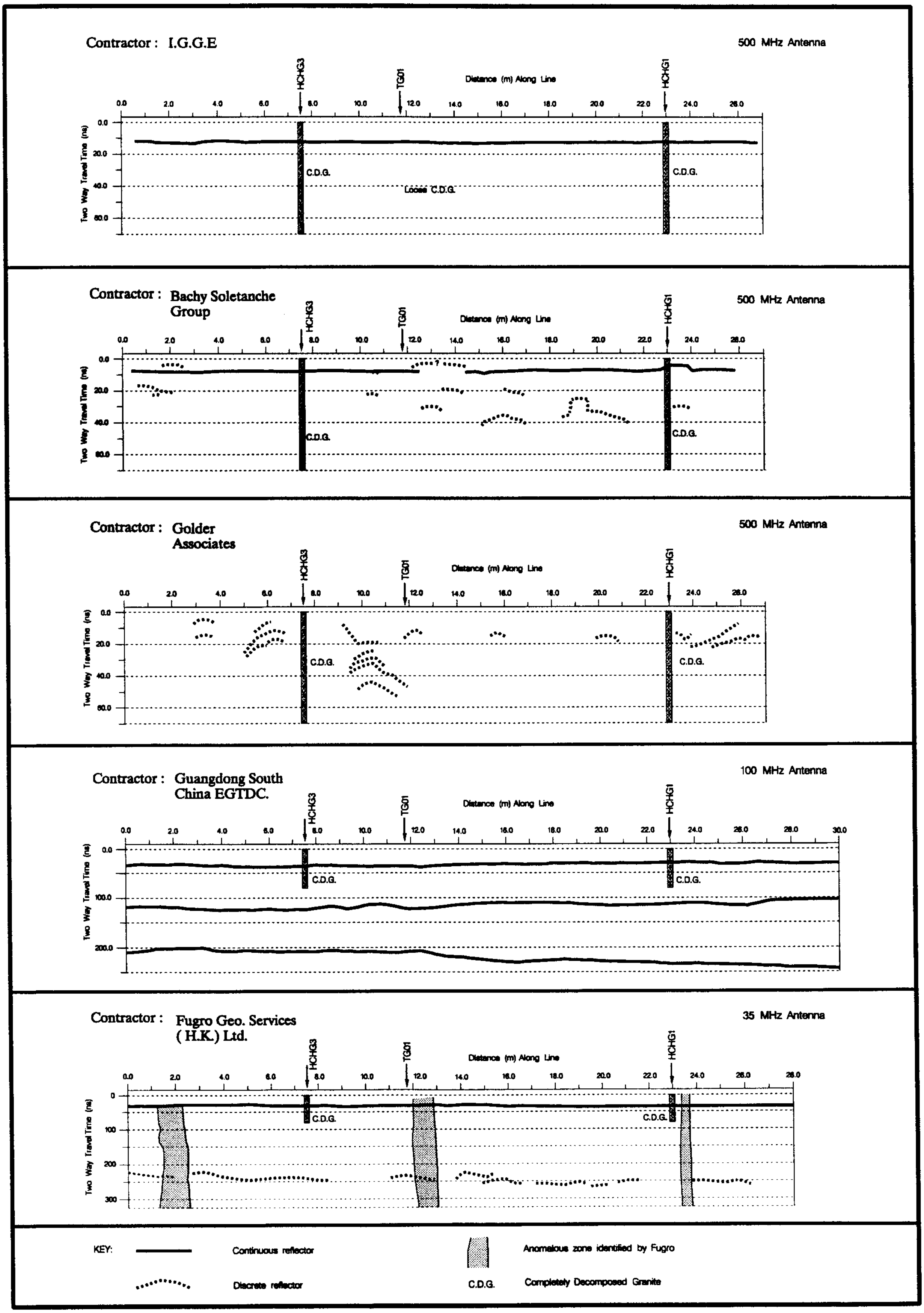

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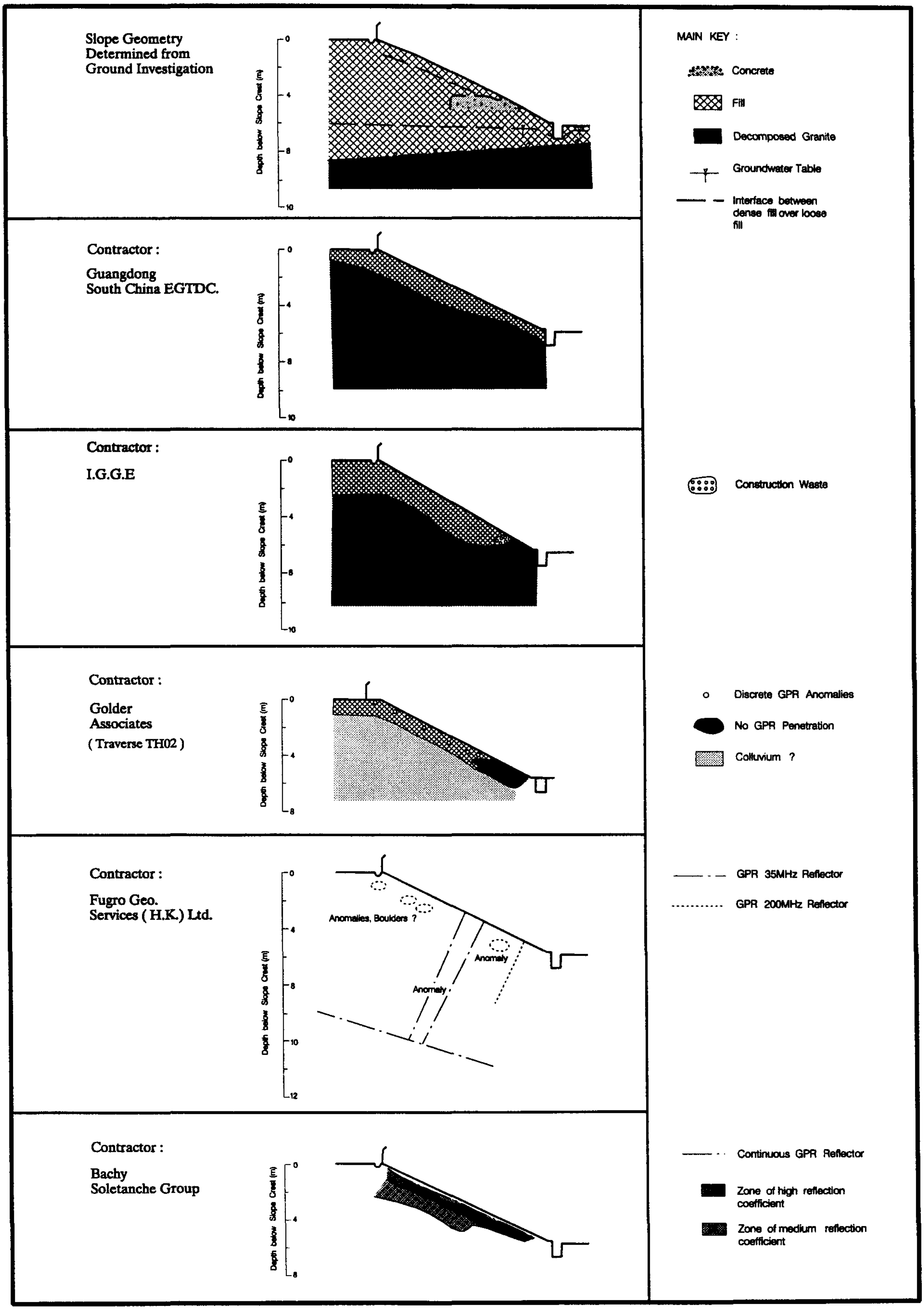

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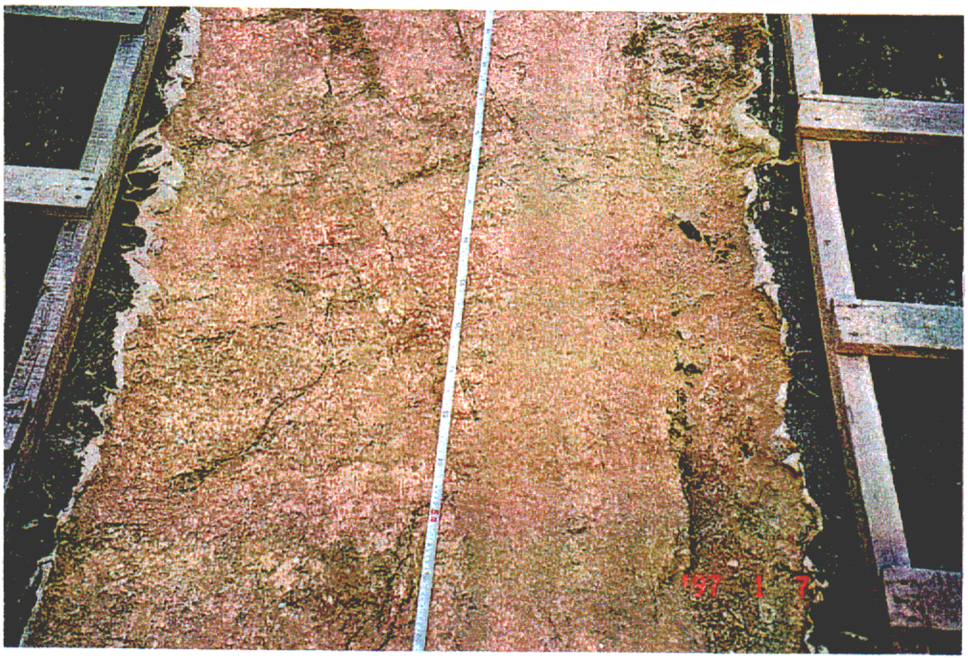

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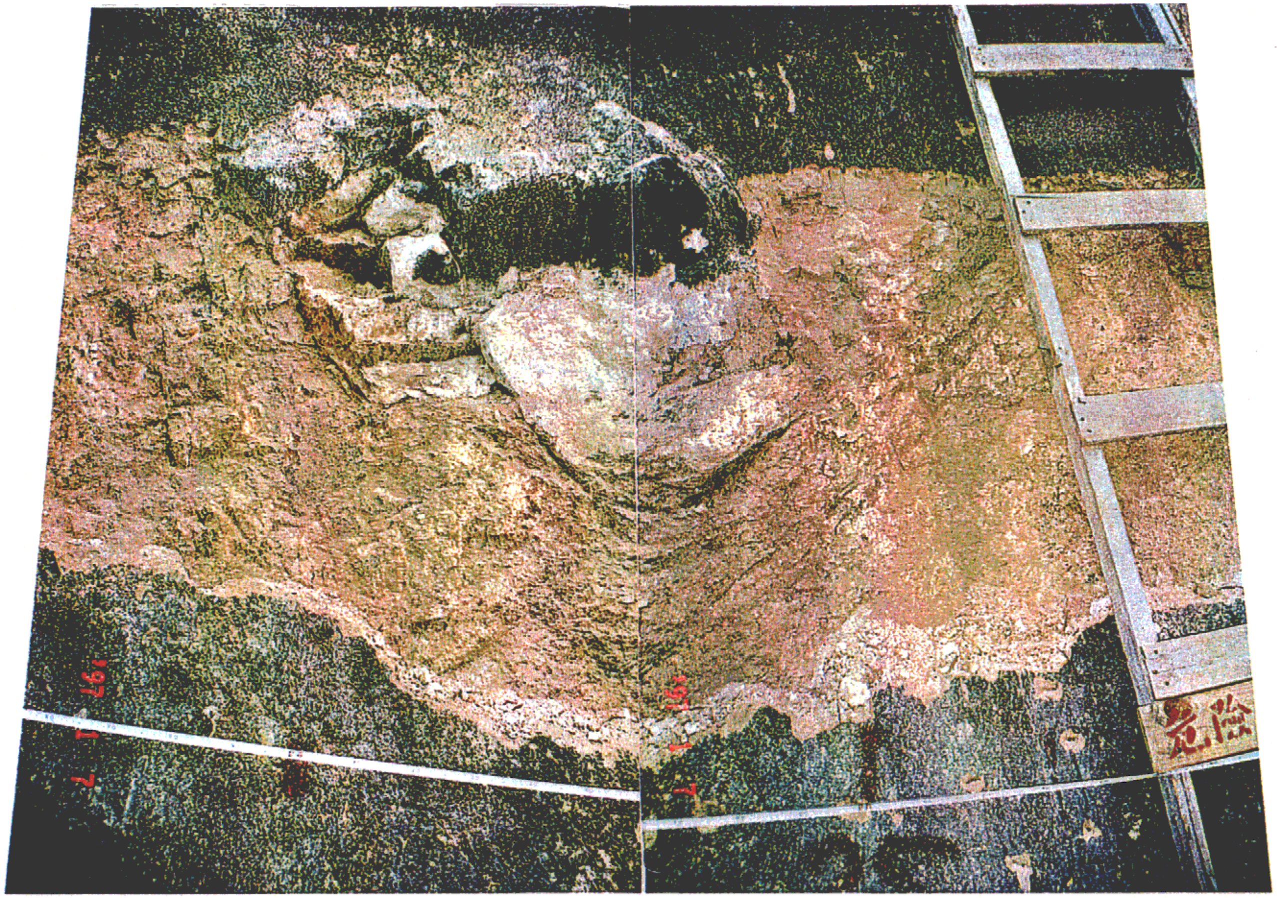

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