GROUNDWATER DISTURBANCE AND SURFACE SETTLEMENTS AROUND A DE-WATERED SAND AND GRAVEL QUARRY

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ABSTRACT

This study has examined the causes of apparently recent ground and structure movements around a de-watered sand and gravel quarry in Hoveringham, Nottinghamshire. The effects of de-watering on the sand and gravel aquifer itself and the response of groundwater levels around the quarry were also examined.

An initial literature study demonstrated the current uncertainty of the effects of de-watering on shallow sand and gravel aquifers. It was clear that spatial variation of permeability and "suffosion" (particle movement due to laterally flowing water) were given little or no consideration at the design stage. This study has focused on these two mechanisms as potential causes of the apparent ground movement in Hoveringham.

A geological desk study on the area of interest was followed by structural defect investigations on the structures of Hoveringham village close to the de-watered quarry in order to classify the type of defects existing. The results from the structural investigations suggested a correlation between proximity to the de-watered void and the occurrence of ground defects due to settlement.

Soil samples taken from very close to the quarry face during de-watering showed a significant lack of fine particles compared to samples taken from similar locations prior to de-watering.

A full scale pumping out test was used to examine the drawdown and settlement response of a section of the aquifer showing a large range of electromagnetic conductivities. The results showed that no significant variation in the distances of influence occurred within the zones showing different conductivities and that no significant ground settlement occurred during the test. This latter result suggested that even if suffosion was occurring remote from the pumped source, it had not led to significant ground settlement.

Numerical modelling of various de-watering scenarios were then carried out both in plan ("quasi three-dimensional") and axi-symmetric fashion. The objective of these studies was to examine the effects of spatial permeability variation on groundwater levels around the quarry and to examine groundwater velocities (termed "unit fluxes" in the thesis) under conditions of vertical permeability variation. Derived unit fluxes were compared to values published by previous researchers for which suffosion had first been observed. The results from the quasi three-dimensional analyses showed that even under extreme boundary and permeability conditions, the resulting drawdown was unlikely to cause structurally significant settlements. The axi-symmetric investigations showed that other than very close to the quarry wall, unit fluxes were unlikely to reach levels at which suffosion would occur.

The original contribution to knowledge contained in this study is in two parts. The first is the in-depth case study into the local effects of groundwater abstraction in the Hoveringham area, the conclusion from which is that groundwater withdrawal is highly unlikely to have caused structurally significant settlement within Hoveringham village. The second is the conclusion that in shallow alluvial aquifers subjected to de-watering operations on a similar scale to that studied, the previously little understood mechanism of suffosion is unlikely to cause any ground settlement. Indeed, the results suggest the mechanism is unlikely to occur at all other than very close to the de-watered zone.

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CHAPTER ONE
INTRODUCTION

1.1 Introduction

This research project is a study into groundwater disturbance and settlement caused by pumping. The source of pumping studied was a large sand and gravel quarry near Hoveringham in Nottinghamshire. The precise location of the site studied is shown in Figure 1.1.

Sand and gravel is an important aggregate for the construction industry and river floodplains have long been exploited for the mineral. One floodplain extensively quarried has been the Trent Valley. An inspection of the geological map of the Hoveringham area where many infilled pits are evident confirms this. Hoveringham is located within Ordnance Survey Grid Reference square SK 6947.

An investigation into the history of mineral extraction within the Trent Valley reveals that extraction has been occurring since 1946. However, until quite recently wet working of strips of land was the preferred procedure. This technique involved excavating a small strip of land using a dragline to obtain the mineral from the open void which would be flooded with groundwater. Once worked out, the water filled void would be backfilled and excavation would move onto the next strip. In some cases, dry working of the strips took place. This meant a small scale pumping operation was required to keep the strip dry during the extraction process.

In recent years, it has proved a quicker and more efficient method of aggregate exploitation to carry out dry extraction of extensive areas. This inevitably means groundwater removal operations on a large scale in order to keep the quarry dry. One disadvantage of this is that large scale pumping can significantly affect the surrounding groundwater regime and water table levels. Research into the effects of all de-watering operations has been carried out since the early 1900`s but the subject of the effects of de-watering operations on shallow, unconfined aquifers is one where much uncertainty still exists.

Changing the groundwater regime by extracting water and lowering the water table can have adverse effects such as:

1. Ground settlements caused by increases in effective stress. If severe enough these settlements can be structurally damaging.

2. If "piping" or "suffosion" (both mechanisms by which fine particles are displaced by flowing water) take place on a large enough scale, ground settlements can occur.

3. Streams and water courses can become depleted and lowered.

4. Water levels in nearby wells can be lowered.

5. Vegetation surrounding the de-watered area can be deprived of groundwater.

6. Archaeological artefacts can lose their integrity if the water table is lowered from above to below them.

1.2 Areas of uncertainty surrounding industrial de-watering activity.

The Department of the Environment commissioned a report entitled, "Low Level Restoration of Sand and Gravel Workings Restored to Agriculture with Permanent Pumping - A Review of the Existing Data and Experience on the Legal and Technical Aspects" (1988). Even though this report was commissioned to study effects of large scale pumping operations required for a different end purpose than that being studied for this report (namely restoration to agriculture rather than pumping facilitating extraction), the technical points studied and researched are exactly the same (i.e; the effects of extensively de-watering mineral deposits for extraction). The Sand and Gravel Association (17/8/93, pers.comm) and the British Aggregate Construction Minerals Industries (29/9/93, pers.comm) both confirmed that in their opinion the state of the art on de-watering problems in and around sand and gravel quarries was summarised in this Department of the Environment report. Some important relevant quotes from the report are included in Figure 1.2

The report discusses a desk study of published geological and hydrogeological data, and a literature search carried out by the British Library on Line Search Centre, which included Enviroline, Georef and Geoarchive databases. The report concludes from this study that,

> "information is not apparently available on de-watering problems in sand and gravel workings or sand and gravel pits restored to low level."

Thirty five local authorities were contacted as part of the report. The report states,

> "Many authorities comment that there is inadequate information available on the effect on the local groundwater table with the result that it is difficult for them either to allay fears that may be expressed to them or to quantify the extent of the possible problem.

Also contacted were the Regional Water Authorities, the Internal Drainage Boards, Planning Authorities, and The Ministry of Agriculture Fisheries and Food (M.A.F.F.). The report states that,

> "concern was expressed by these bodies regarding the lack of information on the extent and effects of the depression of the water table by mineral workings."

The report also points out that limited information on the extent of the drawdown and distribution of information was available from sites that have been monitored. It points out that,

> "the theoretical extent of the drawdown effect of de-watering is difficult to calculate as it is dependent on the hydraulic properties of the aquifer which may vary significantly over short distances in sands and gravels."

The report points out that,

> "while attempts can be made to apply classical calculations these are based on the behaviour of water in a homogeneous medium ..."

To this end, the report states that the geology and hydrogeology of a particular site must be investigated thoroughly before workings commence. The report suggests, "a simple model could be made to predict drawdown around de-watered sites."

This Department of the Environment report suggests that little conclusive research into de-watering problems associated with sand and gravel workings has been carried out. Even if relevant research has been carried out, few of the findings have been communicated to industry.

Thames Water (1979) carried out a computer model study of local land drainage problems in the Lower Colne valley between Slough and the River Thames. The authority was not obliged by law to carry out this study but initiated it because public concern was growing about drainage problems caused by the high concentration of gravel workings and old infilled quarries.

Objectives of the study were threefold:

1. To understand the existing groundwater system including infilling effects.

2. To predict the effects of proposed mineral extraction schemes.

3. To lay down guidelines for future development.

A "regional model of groundwater flow" was developed for the study by the Department of Mathematics at Reading University. This model found that streams and water filled pits in the area played a crucial role in governing ground water levels. This is a fact that was not fully appreciated beforehand.

This study emphasised the need to move on from any "one equation approach" to the investigation of the effects of de-watering. It showed the benefits of using hydrogeological information in conjunction with a calibrated and continuously updated model.

A good example of a "local problem" which involved de-watering of a sand and gravel quarry is illustrated in the newspaper article in Figure 1.3 taken from the Weekend Telegraph of July 31st, 1993. An orchard owner in Huntingdon had noticed a distinct deterioration in his trees following nearby large scale pumping operations. In such cases, it is for the orchard owner to prove the liability of the quarry operator. This was an impossible task for him both technically and financially. The operator is not obliged to disprove liability.

Prior to the research detailed in this report, structural surveys were carried out within the village of Hoveringham by Page (1992, 1993). Two reports summarising results from the surveys are included in Appendices 1.1 and 1.2. The information in these reports shows that prior to the start of this research project, there existed substantial debate about the causal mechanisms for some of the observed structural defects in Hoveringham Village. This research project represents a detailed study into these causal mechanisms and the effects on the surrounding water table of de-watering a large sand and gravel quarry. One subject given particular attention is the potential for ground settlement caused by the removal of finer particles by groundwater flowing laterally. The research has given strong indications as to whether the mechanisms studied were the causes of structural defects in the village.

1.3 The Research Programme

The work detailed in this report represents an attempt to advance knowledge of the effects of de-watering in shallow, unconfined aquifers and to investigate de-watering as one possible cause of ground and structure movements in the Hoveringham area.

The research programme was necessarily multi-disciplinary. Initially, preliminary geological investigations were done to get an overview of the area and the founding soil of the structures under investigation. This was followed up with structural investigations and structural monitoring to qualify and quantify as many of the structural defects as possible. After this, both laboratory and large scale field tests were carried out in order to examine the soil and its behaviour under seepage conditions. The mechanism of "suffosion" (fine particle movement induced by lateral water flow) was given particular attention. Geophysics and a large scale pumping out test formed the bulk of the field investigations.

Numerical modelling was used to assess the likely zone of influence of de-watering in the Hoveringham area and to examine interstitial flow velocities. These velocities were used to assess the likelihood of fine particle movement. Numerical models of the Hoveringham de-watering operation were formed both in plan (to form a "pseudo-three-dimensional" model) and in cross section (axi-symmetric fashion).

1.4 Outline of the work carried out

Chapter Two is a literature review of subject areas related to the research. The multi-disciplinary nature of the work is reflected by the wide range of subject areas covered.

Chapter Three provides an overview of the geology of Hoveringham and the surrounding area and presents the findings from a geological desk study.

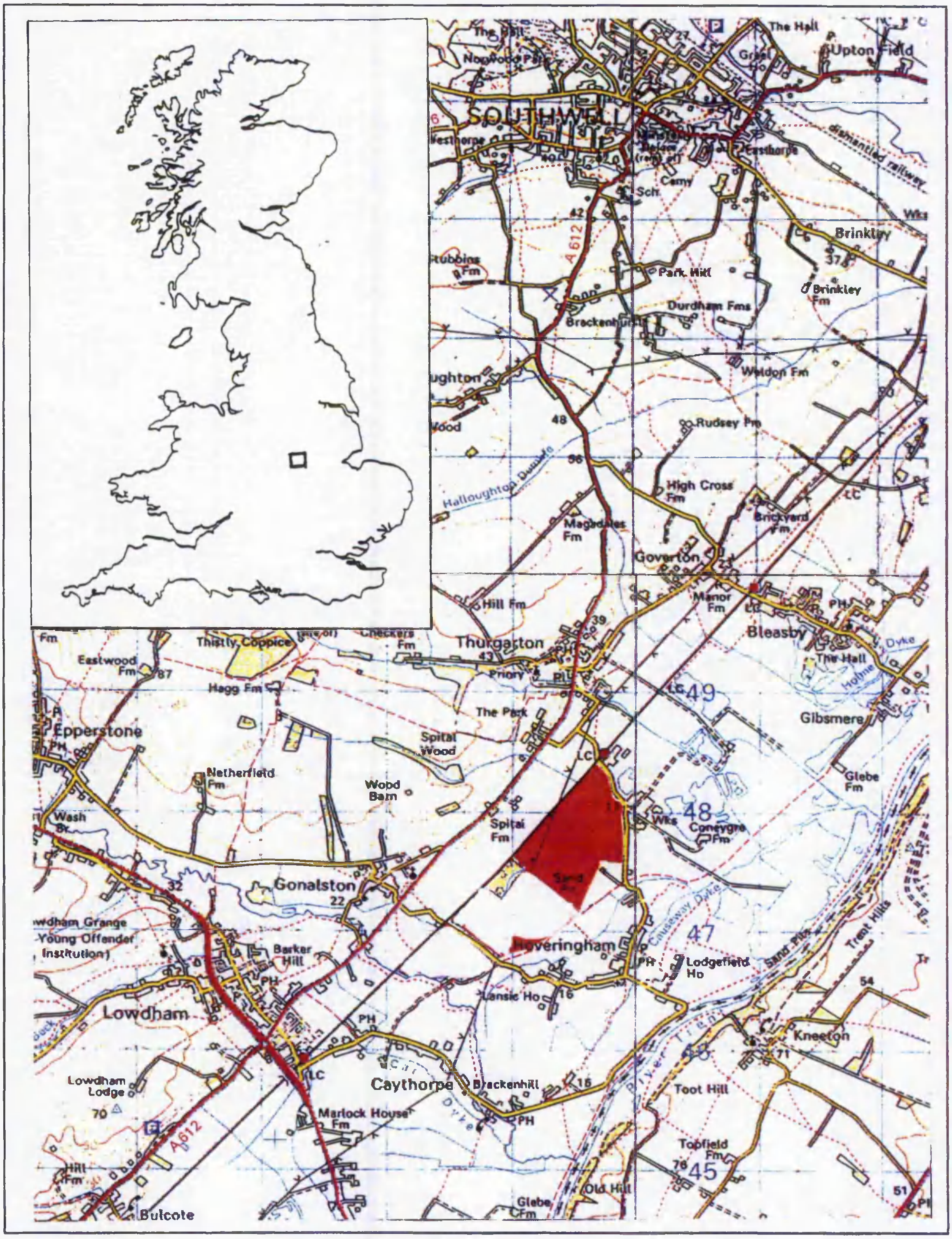
Chapter Four discusses results and findings of the structural investigations. Many structural inspections of properties in Hoveringham were undertaken in order to qualify and quantify the distribution and type of defects existing within the possible zone of influence of the de-watering.

Chapter Five presents evidence for and against the hypothesis that lateral flow of water towards the Hoveringham quarry caused the displacement of finer particles in the in-situ soil. In this chapter the Kenney and Lau (1985) method is introduced. Using this technique in conjunction with particle size distributions for soil samples obtained prior to extraction, it was possible to quantify the susceptibility of the Hoveringham soil to fine particle movement under lateral flow of water. Samples were also obtained from the quarry face during de-watering operations. Analyses of these samples suggested finer particles had been lost at the locations where the hydraulic gradients had been highest. Analyses detailed in Chapter Five show it is not possible that contamination with topsoil during sampling of the pre-extraction samples could have been the cause of the differences in the particle size distributions of the two sets of samples. The conclusions from the Chapter are that fine particle movement is only likely to have occurred close to the quarry and is unlikely to have caused structurally damaging settlements.

Chapter Six discusses the full scale pumping out test used to study the unconfined aquifer during a de-watering operation. The potential for varying distances of influence around a single pumped source were of interest as were any ground settlements due to effective stress increases or fine particle movement. Initially geophysics was used as a wide ranging, low cost, site investigation technique. It will be seen that findings from the geophysics corresponded well to published geological maps of the area. The geophysical method employed was ground conductivity. This was used to investigate a region containing both clean sand and gravel and cohesive alluvium. This area was then subjected to a two week continuous pumping out test. This was an attempt to simulate quarry de-watering and examine the aquifer for any variation in drawdowns and settlements around the pumped source.

Chapter Seven summarises the numerical modelling work carried out. The WTABLE programme, Bromhead (1981), was used in pseudo three dimensional mode to investigate the behaviour of the unconfined aquifer in the Hoveringham area. Initially the model was calibrated against data obtained from a previous site investigation of the de-watered quarry and surrounding areas. Following a parametric study to investigate the sensitivity of variables such as recharge to the aquifer, infiltration, and permeability, worst case scenarios were investigated to examine the effects on the distances of influence around the de-watered quarry. These worst case scenarios included very low infiltration, zero recharge to the aquifer (both situations which might arise during a dry summer) and high permeabilities. The results and conclusions from these worst case scenarios are discussed in Chapter Seven.

The finite element programme FLONET developed by Bromhead (1981), was used to model the de-watering operations in Hoveringham quarry in axi-symmetric fashion. The model was calibrated in the same way as WTABLE. FLONET was used in particular to examine the effects on interstitial flow velocities of layers of high permeability material existing within the sand and gravel aquifer. The flow velocities derived were compared with those required to cause particle disturbance ("suffosion") within the aquifer.



Scale 1:50,000

Figure 1.1

Location Of The Quarry Studied

(After Tarmac Quarry Products Ltd. 1993)

"The absence of data on the extent of drawdown resulting from the long term de-watering of mineral workings has been identified as a problem."

"**1.3.1** Data acquisition has included a desk study of published geological and hydrogeological data to identify the principal areas where the restoration of sand and gravel is likely to result in low level restoration. A literature search carried out by the British Library on Line Search Centre, which included Enviroline, Georef, and Geoarchive, produced a list of 70 titles and abstract from Europe and the United States. However, none of these were relevant to this study. Information is not apparently available on de-watering problems in sand and gravel workings or sand and gravel pits restored to low level."

"Concern has been expressed by the Mineral Planning Authorities regarding the potential impact on the stability of adjacent properties of lowering the groundwater table."

"**2.3.4** No insurmountable technical difficulties with this form of development were identified. Many authorities comment that there is inadequate information available on the effect on the local groundwater table with the result that it is difficult for them either to allay fears that may be expressed to them or to quantify the extent of the possible problem."

"**3.4.2** From the limited information that is available from the sites that have been monitored it is apparent that the lateral extent of drawdown is highly variable with measurable effects recorded between tens of metres and hundreds of metres from the edge of the site."

"**3.4.3** The theoretical extent of the drawdown effect of de-watering a site is difficult to calculate as it is dependent on the hydraulic properties of the aquifer, which in the case of glacial sands and gravels, or river terrace gravels, vary significantly over short distances both horizontally and vertically as a result of the heterogeneous nature of the material. While attempts can be made to apply classical calculations these are based on the behaviour of water in a homogeneous medium resulting from de-watering a relatively small diameter borehole."

Figure 1.2

Extracts From "Low Level Restoration Of Sand And Gravel Workings Restored To Agriculture With Permanent Pumping".(Department Of The Environment. 1988)

Blight of the living dead

Trees dying of water starvation are one symptom of a planning disease that has no cure, says George Hill

The fruit on Howard Dolby's pear and plum trees is not filling out as it should at this time of year. The leaves are pale and curled up, many branches are dead, and what fruit has appeared is dry and stalky. This is the third year that this part of Dolby's fruit farm near Huntingdon, in John Major's constituency has failed to yield a worthwhile crop.

There used to be wonderful crops here, he says. We have had no problems of this kind on the rest of the farm. But the closer you get to that great hole in the ground, the more you can see that the trees are suffering from lack of water. Even the poplars planted as windbreaks are dying.

"That great hole" is a raw cavity hundreds of yards across and 30ft deep, carved out of the land next to the orchards of Dolby's Hyland Fruits by Earith sand and gravel quarry. He claims that the quarry is drawing groundwater away from the land on which the trees grow. But it appears he would have no remedy in law even if that could be proved.

Dolby complains that he is a victim of a legal quirk that is causing widespread disquiet. In planning terms it is the curse of the living dead. Planning permissions from an earlier generation that have slept for decades in the files of local government and landowners can still rise up to haunt today's farmers, householders and conservationists.

The Government plans to announce proposals later this summer for changes in the law. Its dilemma is that the only means of laying the living dead to rest may be to sacrifice the principle that holders of property rights - valid planning commissions in this context - should be compensated in full if the state takes those rights away.

Significant operations at Earith quarry, which is run by ARC Central, started in 1989 under planning permission granted in 1963. To reach the 250,000 tons of sand and gravel it extracts every year, ARC pumps out thousands of gallons of groundwater. The trees suddenly began to fail in 1991, Dolby says. The plums shrivelled like dried prunes and the pears just dropped on the floor two weeks before they were due to be picked.

Cause and effect are seldom straightforward in disputes over water supply. Independent consultants commissioned by Dolby found that the water table was 20 feet below ground level with indications that it had been higher, and said the fall was highly likely to have been caused by the gravel extraction. But the National Rivers Authority (N.R.A.), after its own tests, blamed the recent dry summers. ARC has offered to allow surplus water to be piped to the fruit trees and this proposal may solve Dolby's problems. But the offer has been made entirely as a concession. Even if the quarry were demonstrably drawing water away, Dolby would have no legal right to demand relief or compensation.

Yet if ARC sought planning permission for a quarry today, the local authority would not grant it unless it was satisfied that the proposal complied with strict rules laid down by the N.R.A. about changes to the water table.

Clive Thompson, chief planning officer with Cambridgeshire County Council says, ARC is entitled to work under the terms of the permission as granted and the Council has no powers to impose further conditions now.

Strictly, Thompson is wrong, but in practice he is right. Planning authorities do have the power to rewrite permissions. But the compensation terms usually laid down by statute usually make the cost prohibitive.

Many permissions granted in the post war years are still valid. Rising demand for stone and gravel, and growing opposition to the granting of new extraction rights, make them valuable commodities.

Some of the sites under threat are the cream of our natural heritage, says Charles Couzens, conservation officer for the Royal Society for Nature Conservation. Land in National Parks, designated Sites of Special Scientific Interest (SSSI's) and areas of outstanding natural beauty are gradually being eroded by quarrying, under permissions granted before the introduction of environmental legislation.

Old permissions do not only affect groundwater. They tend to set no limit on the kind or extent of abstraction allowed, impose no controls on noise or traffic nuisance, and make no stipulations about reinstatement after quarrying ends.

In Northamptonshire, at least 15 per cent of the county is covered by old permissions. Two SSSI's in South Yorkshire - the unique lowland peat bogs of Thorne and Hatfield Moors (which support merlins, nightjars, and sundews) - are threatened by planning permissions covering 2500 hectares permitting peat abstraction. Three-quarters of Kirby Moor in Cumbria, another SSSI, is covered by an old slate quarrying permission.

The Government has twice tried to impose new rules. A date has now been set at which permissions set before 1992 will expire. But that date is not until 2042.

Figure 1.3

Daily Telegraph Article From Saturday July 31st 1993 Describing A Local Problem Caused By Ground Water Abstraction.

CHAPTER TWO

LITERATURE REVIEW OF SUBJECTS RELATED TO THE RESEARCH

2.1 Introduction.

Sections 2.2 and 2.3 summarise current techniques used in structural defect surveying and de-watering operation design. Case studies of ground and structure movements caused by industrial de-watering activity are summarised in Section 2.4. This section also demonstrates the lack of case study data about the de-watering of shallow unconfined aquifers around sand and gravel quarries. Section 2.5 introduces spatial variation of permeability as a phenomenon which can be responsible for the occurrence of differential distances of influence around a de-watered zone. Section 2.6 summarises in-situ and laboratory based soil property tests while concentrating on those associated with permeability determination. Section 2.7 describes numerical methods, including finite element techniques, which can be used to analyse de-watering and seepage in unconfined aquifers. In this section the WTABLE and FLONET finite element programs which were used during this research are introduced. Section 2.8 describes mechanisms by which seeping water can cause sub-surface or surface disturbance. In particular the mechanism of "suffosion" (fine particle displacement by seeping water) which formed a study area for this research is introduced.

2.2. Structural Movements and Defects, Structural Defect Surveying And Structural Monitoring.

This research project considered not only ground and foundation movements but also the effects of those foundation movements on the structure itself. As early as 1947, Meyerhof recognised that the effects of interaction between a superstructure and its foundation could cause some redistribution of stress and hence departure from an analysis assuming complete restraint at the structure/ foundation junctions. King and Chandrasekaran (1974) summarised the most important computer modelling work which had been carried out into soil and structure interaction up to that point.

Littlejohn (1974) carried out full scale in-situ tests of movements of brick walls due to mining subsidence and describes the foundation movements relative to the wall movements. In relation to crack formation and behaviour, Littlejohn states that where failure occurred as a result of tensile strains, initial cracking invariably resulted from a breakdown of the brick/mortar bond. After this critical tensile strain had been surpassed, the crack usually operated as a hinged joint enabling the structure to operate as a composite unit. Littlejohn concluded that the degree of damage is often a function of ground strain, which can cause cracking at very small angular distortions.

Further to the idea that cracking usually appears after a certain critical tensile strain has been surpassed, Pryke (1974) makes reference to the fact that this critical strain can be different for structures of varying ages. He states that the timber framing and soft mortar used in houses built before 1900 can accommodate substantial movement without cracking. Houses built in the 20th century and particularly after the 1950's will be more brittle and will crack with less differential foundation movement.

Structural defects and defect surveying of low rise residential properties was given particular attention as nearly all the properties surveyed in Hoveringham village were of this type. Information on current practice concerning the three subjects in the heading was initially obtained from unpublished course booklets from the B.Eng (Hons) Civil and Structural Engineering degree course, The Nottingham Trent University (1992).

Page (1984 and 1992) discusses methods for structural surveying of low rise buildings as well as the diagnosis of observed defects. Page (1992 and 1993) also discusses specific defects observed in the village of Hoveringham. Data contained in these reports was used for preliminary defect diagnosis purposes for some structures in the village.

Example structural survey case studies and defect diagnoses were obtained from White (1992), Wintersgill (1993); and Chapman (1994) (all unpublished theses, The Nottingham Trent University).

Attewell and Taylor (1984), and Burland and Burbridge (1985) provide information relating to structural behaviour and movements of buildings founded on sands and gravels.

Nineteen separate British Reasearch Establishment Digests dealing with structural defects and their diagnoses for low rise buildings were studied to ascertain the current state of the art. These digests are fully referenced at the rear of the thesis.

Information from the documents quoted above was backed up by attendance at the "One-Day Seminar for Professional Surveyors - Structural Movement in Residential Property", by Leonard Murray and Associates, Chartered Civil Engineers (21 September 1993). Current practices were explained during the seminar.

Based on the sources of information summarising the current state of the art of defect surveying and diagnosis discussed above, a "Structural Inspection Document" was created for use while surveying the properties of Hoveringham Village. The document and results from the surveys are discussed in Chapter Four. The document took the engineer through the survey, first by use of a question and answer session with the property owner, then an overview of the general property and its immediate surroundings followed by detailed internal and external examinations.

2.3 Current Design Practices And "State Of The Art" Theory Used For De-Watering Operations And Associated Problems.

The CIRIA (Construction Industry Research and Information Association) report 113, "Control of Groundwater for Temporary Works" by Somerville (1986) contains current initial guidelines used by contractors for the design of de-watering works. Examination of this report shows that currently, the radius of influence around a de-watered zone is estimated using the equation below which is quoted on page 18 of the report. The permeability value used in the equation is usually the average from a number of field tests. This equation obviously takes no account of any spatial variation of permeability which may exist around a de-watered zone.

$$R = Ch\sqrt{k}$$

Where:	R	=	estimated distance of influence (m)
	C	=	a factor of approximately 3000 for radial flow
	h	=	drawdown of the water table
	k	=	peremeability (m/s)

The classical equations and techniques which can be used to analyse steady or unsteady state flow of water in confined or unconfined aquifers are fully summarised by Krusemann and De-Ridder (1983). British Standard 6316 : 1992, "Code of Practice for Test Pumping of Water Wells", Preene and Roberts (1993), and Hamill and Bell (1986) all describe good practice and important factors to bear in mind when designing and carrying out a pumping test.

Roberts (1988) argues that analytical support available for the design of current, highly complex de-watering projects is inadequate. Roberts identifies specific problem areas as being accuracy of aquifer parameters, effects of anisotropy, inhomogeneity, partial penetration, physical cut-offs, the location of the boundary of influence and the use of superposition in shallow unconfined aquifers.

The extracts from the Department of the Environment document in Figure 1.2 also highlight some apparent inadequacies in the way de-watering operations are currently designed and monitored.

Powers (1981) produced an excellent review of important practical aspects related to de-watering and groundwater control. Sladen and Bitcheno (1988) emphasised the need for a thorough feasibility study and site investigation prior to a de-watering operation. This, when properly carried out, allows the volumes and quality of water to be disposed of to be estimated along with the effects of the de-watering operation on the surrounding area. They illustrate their assertions with reference to a case study.

The purpose of and techniques for well and aquifer development in unconfined aquifers are described by Clark (1988) and Driscoll (1989). The purposes of development are broadly twofold:

1. To repair damage done to the formation by the drilling operation so that the natural hydraulic properties are restored.
2. To alter the basic physical characteristics of the aquifer near the borehole so that water will flow more freely to the well.

Driscoll (1989) describes "overpumping", "backwashing" and "mechanical surging" as development techniques suitable for wells operating at relatively low heads. Backwashing, or "rawhiding", was the well development technique chosen for the pumping test described in Chapter Six. This involved switching the pump on and off quickly to lift a column of water which was then allowed to fall back into the well. This process was repeated a number of times to develop the well.

2.4 Ground And Structure Movements Caused By Industrial De-Watering Activity.

In the literature many examples of ground movements due to de-watering operations are found. Some deal with drawdowns of greater than 100 metres. Some cases described below are world famous such as Mexico City and Venice. Much has been written concerning groundwater withdrawal problems in both places. However, on examining the literature, there is an apparent lack of information on problems associated with the de-watering of shallow unconfined alluvial aquifers. Poland (1981 and 1984) discusses case studies of settlements due to groundwater withdrawal in the United States. Many of these case studies involve settlements of the order of metres caused by extensive and prolonged water withdrawal for supply. Water table drawdowns were up to 140 metres. Marsal (1959) presents a summary of the situation as it then was in Mexico City where prolonged water withdrawal for supply had caused metres of settlements. It was considered that consolidation of clay deposits between the surface and 70 metres depth was the chief cause of the displacements. Mishra *et al.* (1993) discuss the case of subsidence caused by groundwater withdrawal from the unconsolidated alluvial deposits of the Indo-gangetic basin. These deposits were up to 400 metres thick. They discovered a very small present rate of settlement not critical for local structures. However, after applying their "quasi-three-dimensional model approach" to the system they were able to determine allowable safe upper limits for the magnitude of the pumping operations.

Bouwer (1978) summarises locations world-wide which have suffered large settlements due to extensive groundwater abstraction. A brief summary of these locations is given in Table 2.1.

Lin *et al.* (1991) discuss subsidence due to deep aquifer compaction in Taiwan. This compaction occurred after extensive groundwater extraction for use in pisciculture (fish farming). The magnitude of the settlement was over 2 metres in some places. Mulder *et al.* (1994) discuss a study carried out in the Netherlands to determine present and likely future damage to structures which has and could occur due to the pumping dry of lakes for land reclamation. They state that "persistent pumping activities have caused substantial land subsidence in areas surrounding these polders (pumped areas)."

In all the above cited cases, the magnitude of the drawdown of the water table or piezometric head was very high. Powrie (1994) discusses case studies in which drawdowns of 4 to 6 metres were seen as a potential threat to nearby structures. In one case study, a wellpoint de-watering system was used to achieve a piezometric drawdown of approximately 4 metres in glacial sands and gravels overlain by approximately 4 metres of peat and soft alluvial clay (the water table was originally 0.3 metres below ground level.). After three weeks of pumping, owners of structures 500 metres away began to complain of structural damage and the de-watering system was switched off. It was thought that consolidation of the soft alluvial clays was to blame for the settlements.

2.5 The Geology Of River Floodplains And Spatial Variation Of Permeability.

2.5.1 What Is Spatial Variation Of Permeability?

Spatial variation of permeability is a phenomenon which can lead to differential distances of influence around a de-watered zone. It can arise when geologically varying materials, landforms or features are in close proximity to each other. This can happen in a constantly changing environment such as a river floodlplain where flow velocity, direction, channel gradient and natural floods can all cause deposition and erosion characteristics to change over geologically short time periods.

2.5.2 The Literature Examined.

The literature was examined specifically with a view to answering the following two questions:

1. Which geological environments predominantly display the phenomenon of spatial variation of permeability?

2. Could spatial variation of permeability lead to the distance of influence and drawdowns around a de-watered zone being significantly different to those predicted using current design practices?

The literature tends to be in two distinct categories. The first covers physical or field studies related to permeability variation and the second deals with mathematical models which have attempted to quantify the phenomenon.

The existence of the phenomenon of spatial variation of permeability has been known for a long time. Johnson and Hughes (1948) stated that flow through consolidated oil bearing sands may be 25 - 30% less in one direction than another. They discuss the fact that geologists then recognised the existence of directional trends in the permeabilities of sands, but at that time no published data existed on the magnitude and significance of any variations. They therefore carried out a series of laboratory investigations into directional permeability variation within consolidated sands. They came to the conclusion that directional permeability trends occurred less in sands of higher permeability. Johnson and Hughes state that in the case of a river deposit, the grains tend to become orientated at right angles to the river currents carrying them. This would lead to an increase in the permeability across the floodplain (Figure 2.1). Should this phenomenon occur, then any direction of increased permeability may be very different to simply across the floodplain depending on how the river meandered, braided, and flooded historically. Other complicating issues which modify depositional environments are solution, cementation and compaction.

Chester (1982) discusses the various fluvioglacial landforms that can be formed in depositional environments. Some important possible landforms are:

- Kames: Isolated mounds of sand and gravel laid down in a fissure or hole in a melting ice sheet.
- Kame Terrace: A bank of sand and gravel deposited in water ponded between an ice mass and a valley side.
- Outwash: Extensive, often flat, sheets of sand and gravel laid down in front of an ice sheet or valley glacier.
- Eskers: Elongated ridges of fluvioglacial deposits consisting largely of sand and gravel in channels or tunnels in the ice.

The above are examples of the type of feature which can be expected in a fluvioglacial zone such as the Trent Valley. The various landforms, depending on their extent and composition, could lead to permeability variation within the floodplain.

Tarmac Quarry Products Ltd (1993) describe the variation in geology in the Trent Valley. Small breaks of slope in the valley floor are described as coinciding with the junction between the Holme Pierrepont River Terrace Gravels and alluvium. These materials are described as being the two main types of superficial deposits existing below the topsoil. The Terrace Gravel is described as a sand and gravel with varying silt quantities with the alluvium being described as a firm grey-brown clay. Nottinghamshire County Council (1993) describe the alluvial sands and gravels of the Trent Valley as thin unconsolidated deposits which were spread across the valley floor by flooding rivers in the recent geological past. The most important period of deposition is described as having occurred 11,000 years ago towards the end of the last Ice Age.

Further specific geological details relating to the Trent Valley can be found in Sections 3.2, 3.3 and 3.4.

Edge and Sills (1989) carried out laboratory studies which attempted to simulate the way layered sediment beds are formed. They state that a genuinely uniform naturally occurring sediment is very uncommon.

Mansur and Kaufman (1980) discuss water deposited sands and gravels. They describe how levees form at the river banks during periods of high water and state all rivers that flood contain extremely variable material on their floodplains. They describe river terrace formation and discuss the variability of geological landforms which have been created by the Colorado and Mississippi Rivers. Petts *et al.* (1989) discuss the "Historical change of large alluvial rivers." They concentrate on the rivers of western Europe but state that many smaller scale rivers in the United Kingdom, including the Trent, have mimicked some of these changes. They comment on recent variations induced by man as well as geological time scale changes. They discuss river terrace formation and variation in sediments due to glacial retreat. An important point made is that man has greatly influenced fluvial hydro-systems over the past 5000 years. Deforestation along river banks and land drainage schemes have increased occurrences of flooding in some areas. They demonstrated how maps can be used to identify the changes that have occurred in river channel position even in the short time maps have been available. The diagram in Figure 2.2 illustrates some of the geological features which can exist within a river floodplain.

In his paper "Field Study of Macrodispersion in a Heterogeneous Aquifer", Boggs (1990) used data from aquifer tests, borehole flowmeters, double packer tests, slug tests and a laboratory permeameter to examine the variability of permeability within a heterogeneous alluvial aquifer. Boggs studied the dispersion of tracers away from an input source and he states that hydraulic conductivity can vary significantly within an aquifer. He states that reasonable resources must go into quantifying any variation if a transport model is to be used. Boggs found that over a horizontal distance of as little as 280 metres, permeability could vary from 10^{-3} m/s to 10^{-6} m/s.

2.6 In-Situ And Laboratory Based Soil Property Tests.

Data from permeability determination tests for sand and gravel material, whether expressly obtained for this research, or obtained from previous site investigations of the study area, formed an important part of the work. This section describes some techniques discovered within the literature. Time, finance, and available resources were all factors which precluded the use of some tests described within this section and determined the final choice of tests used.

Head (1981), describes the two types of laboratory test available for the direct measurement of permeability. They are:

1. The Constant Head Test: This test is applicable to soils of high permeability such as sands.
2. The Falling Head Test: This test is for soils of intermediate and low permeability such as silts and clays.

Head describes the Constant Head Permeability Cell (or "Permeameter") which is used for tests on sands. A larger version is used for tests on gravels.

The two techniques described above are known as DIRECT methods of permeability determination. Head also describes two INDIRECT methods which both require the calculation of a particle size distribution for the soil being tested. These two methods are:

1. The Hazen method.
2. The Kozeny method.

Hazen developed his method empirically in 1892. This method only indicates the order of magnitude of permeability and is based solely on particle size data. A particle size analysis carried out on a truly representative sample using a wet sieving procedure is required before the Hazen formula can be used. The formula is:

$$k = C_1(D_{10})^2$$

Where: k is permeability in centimetres per second,
D_{10} is the sieve size in cm which 10% of the sample passed,
C_1 is a factor of approximately 100.

The use of the Hazen method on soil samples as part of this research is described in Chapter Six.

The equation proposed by Kozeny in 1927 is more complex and relates permeability to particle size, porosity, angularity of particles, specific surface and viscosity of water. Again, a wet sieve procedure is carried out after which the angularity of grains is assessed using a microscope. Mean specific gravity is determined and used to calculate other required parameters which are then used in an equation.

Murray (1995) presents a review of the many published analytical methods of predicting the permeability of granular materials. Murray states that laboratory tests on recompacted granular materials are not normally reproducible to an accuracy greater than one order of magnitude, and that it is useful to back laboratory tests up with analytical methods. The following eight factors are reported as influencing the permeability of granular materials:

Properties of the Particles

1. size and range of particles
2. shape of particles (e.g. rounded, angular, flaky, lenticular)
3. texture of particles

Dispersion of the Particles

4. density of compaction
5. particle configuration (e.g. orientation, alignment, segregation)

Pore Space Characteristics

6. pore size distribution and shape
7. tortuosity of flow (measure of length of flow path)

Properties of the Permeant

8. viscosity and unit weight of permeant

The many analytical methods presented by Murray all take into account at least some of the factors given above. The equations presented are grouped into empirical formulae, semi-empirical formulae and theoretical hydraulic radius models.

Loudon (1952) presents an earlier review of several published formulae relating the permeability of granular materials to their geometrical properties. Loudon describes Hazen's formula as "quite useful due to its simplicity" and states that Kozeny's formula is the best of those quoted in his paper.

Craig (1989), describes the following in-situ methods for determination of permeability:

1. The well pumping test - after Vreedenburgh (1936).
2. The constant and variable head borehole tests - after Hvorslev (1951).
3. In-situ seepage velocity measurements - after (1951).

Craig describes the well pumping test as suitable for coarse grained soil strata. The method involves continuous pumping from a well which penetrates to the bottom of the stratum and observing the water levels in a number of adjacent boreholes. These boreholes must be located along radial lines away from the well (at least two boreholes per line) and water levels in the boreholes are measured during steady state conditions. Figure 2.3 shows a typical section through a pumped well and two boreholes.

British Standards Institution Code of Practice 6316:1992, "Code of Practice for Test Pumping of Water Wells", describes the correct procedures to follow for the design and execution of a well pumping test. The use of a well pumping test as part of this research project is described in Chapter Six.

For the constant head borehole test, water is allowed to flow under constant head into or out of the stratum through the bottom of a borehole. The important parameters are the difference in head between natural water table level and borehole water level, flow rate, and borehole diameter. An equation using these three parameters is used to calculate permeability. Both Mishra *et al.* (1993) and Stephens (1992) describe the theory and current research developments relating to constant head borehole permeameters.

During the variable head borehole test, the hole is filled with water and water level measurements are taken with time as the water in the borehole seeps into the aquifer. A relationship between borehole diameter and change of head with time is used to estimate permeability.

In situ seepage velocity measurements can involve excavating trial pits a certain distance apart and making observations as seepage takes place from one pit to the other. Dye can be inserted into the water at the upstream trial pit and the time taken for this dye to appear in the downstream trial pit is used to estimate the permeability. Burgess (1983) reviewed in-situ permeability tests for soil and rock and produced the summary information shown in Table 2.2. This table describes the applicability and limitations of the various methods.

2.7 Numerical Modelling.

2.7.1 Required Capabilities Of The Numerical Model And User.

In the literature, many authors describe the application of finite element and finite difference methods to the problem of seepage in unconfined aquifers. The capabilities of the numerical analysis techniques chosen for this project had to be two-fold:

1. The ability to present groundwater level contours in plan around a de-watered zone under conditions of spatial variability of aquifer permeability.

2. The ability to estimate unit flux values (seepage velocities) at discrete locations within the seepage domain.

In his paper "Required knowledge for finite element software use", Macleod (1991), presents a useful review of the limits of knowledge required by users of finite element software. He points out that, "within the information explosion there seems little point in gaining knowledge of that which will not be used."

Macleod defines the steps in the finite element method as:

1. Define the constitutive relationships.
2. Define the element relationships
3. Form the system model.
4. Solve.
5. Back Substitute.

In the case of seepage, a constitutive relationship would be the Dupuit assumption and related equations. The Dupuit Assumption is that all hydraulic gradients (and therefore flow velocities) throughout the depth of the aquifer are equal to the gradient at the phreatic surface. This will therefore lead to a progressively larger overestimation of hydraulic gradient for points lower down the aquifer.

Macleod states that a user should understand the relationship on which a finite element method is based and know its limitations and assumptions.

Macleod states that a "Catch 22" problem sometimes arises when users ask, "How well do my results model the behaviour of the system I am attempting to simulate?" To answer this, the user needs to understand the system behaviour, which is the very thing he is attempting to model. Macleod states the two ways to solve this apparent anomaly are with the use of a "Checking Critique" and "Parameter Studies". It will be seen how both these processes were applied to the finite element models used in this research in Chapter Seven. A parameter study was carried out and the "Checking Critique" is referred to as the "Calibration" in Chapter Seven where model results were compared to data measured in the field.

Hamill and Bell (1986), in their book, "Groundwater Resource Development", state that,

> "Any groundwater flow model is only as good as the data upon which it is based ... the more data that is available, the better the model in general".

They stress that during construction and preparation of a model, two distinct phases are often recognised. These are "Calibration" and "Verification". These two stages are similar to the "Checking Critique" and "Parameter Studies" described above by Macleod. They present twelve requirements for a model if it is to effectively simulate the aquifer. These are:

1. The extent of the aquifer and the location and nature of any aquifer boundaries.
2. The flow of water into and out of the aquifer.
3. The rest water levels in the aquifer.
4. Variations in the thickness and depth of the aquifer and any confining strata.
5. The spatial variation of the coefficients of transmissivity and storage.
6. Data recorded during the pump testing of wells, such as the discharge of the well and the drawdown recorded at various points in the aquifer.
7. Water level fluctuations in the aquifer over a number of years.
8. The rate of infiltration to the aquifer in the recharge area during the same period.
9. The pumping schedules for the same period.
10. River base flows.
11. Spring locations and spring flows.

12. General background information regarding the hydrogeology of the region such as areas of interconnection between surface and groundwater, interflow between aquifers, artificial recharge and so on.

The types of pre-analysis data required were also summarised by Jones (1994).

Peck (1988) presents an account of the types of factors to be taken into account while modelling groundwater behaviour. These included such things as:

- The causes of errors
- Intended objectives possible
- The effects of aquifer variability.

Peck presents, in flow chart form, the "Development sequence for a prediction model using numerical simulation." This flow chart is shown in Figure 2.4. While using a pre-supplied computer package obviously only the part of the table after the first "Yes" is relevant. It will be seen in Chapter Seven how this process described by Peck was followed during the finite element investigations for this research project.

2.7.2 Types Of Model Available.

Many types of modelling programs are available to study flow in unconfined aquifers. Four of these models are described in this section.

Jones (1994) describes the use of a program called SEFTRANS to model groundwater protection zones in the East Yorkshire chalk aquifer. SEFTRANS is a Fortran program which operates under MS-DOS. The hydrogeology of an unconfined cretaceous chalk aquifer was studied. The purpose was to determine whether agricultural nitrates would pollute water supplies drawn from boreholes.

SEFTRANS was used to model the aquifer in two dimensions. It was recognised by Jones that within chalk, flow is predominantly along fissures and to counter this, the aquifer was modelled as a "continuum", the bulk permeability for which was fixed at many times higher than the matrix permeability. From the steady state model, flow trajectories and groundwater protection zones were found.

A finite element model currently widely used in industry to study groundwater problems is "C.V.M." (standing for Curved Valley Model). This model has been developed by Oxford Geotechnica Ltd. to suit the needs of industry. CVM can operate in one or two dimensions and can be used to model such problems as:

1. Evaluation of the radius of influence from a pumped well.
2. Calculation of the effective permeability of a composite landfill liner.
3. Calculation of the effective permeability of a damaged landfill liner.
4. Estimation of concentrations of groundwater beneath landfill.
5. The simulation of leachate wells.
6. Contamination impact on pumping wells.

Two finite element programs were used during this research project. They are called "WTABLE" and "FLONET" and were developed by Professor E. Bromhead of Kingston University. Bromhead (1985) describes the application of modelling packages to dam and reservoir problems. Their usage and the theory upon which they are based are described in detail in Chapter Seven. The mesh of elements used in WTABLE represents the plan of the seepage domain and the output from a WTABLE run is the elevation of the groundwater table. WTABLE can model seepage in a confined or unconfined aquifer, and can deal with unsteady flows. FLONET is a finite element program for confined or unconfined seepage in a plane or axi-symmetric section through the seepage domain. FLONET can be likened to a numerical model for producing "flownets". It deals with the same vertical plane through the aquifer that a sketched flownet would. The results from an analysis comprise the distribution of pressures and seepage velocities in the domain.

Bromhead (1985) also describes the application of modelling packages to dam and reservoir problems.

2.8 Mechanisms Of Soil And Ground Disturbance By Lateral Subsurface Flow Of Water.

This subject was investigated as a possible cause of surface subsidence which could have led to structural damage. Structurally damaging settlements due to effective stress increase caused by large scale water table lowering were shown to be highly improbable for areas more than a few tens of metres from the de-watered void. This led to particle disturbance due to lateral flow of water being considered as another possible cause of settlement.

The literature gives three mechanisms by which particles can be displaced by laterally flowing water:

1. Piping
2. Suffosion
3. Entrainment

Piping is the gradual erosion of material along a permeable conduit. An example of this might be where a small amount of seepage begins to occur through an earth and rockfill dam or embankment. The prolonged exploitation by water of this seepage path can lead to erosion as fine particles are transported away by the flowing water. This removal of finer particles creates larger voids which in turn leads to greater seepage and in this way the cycle of erosion goes on.

Skempton and Brogan (1994) carried out tests on sandy gravels to assess their susceptibility to piping. They defined samples that suffered particle washout at relatively low hydraulic gradients as "internally unstable". They concluded the reason for washout from these samples was that the major part of the overburden load was carried on a framework of gravel particles, leaving the sand under relatively small pressures. Skempton and Brogan state that a simple assessment of stability may be made by dividing a material into its fine and coarse components. If these components fulfil the filter rule below suggested by Terzaghi (1939), the material as a whole will be stable and self filtering. If they do not, the material will be unstable.

Terzaghi's Filter Rule:

$$f = D'_{15} / d'_{85} \leq 4$$

Where: f is a dimensionless index defined as less than or equal to four for stable soils.

and, D'_{15} and d'_{85} are the 15% and 85 % by mass passing sizes for the coarse and fine components respectively.

A summary of the basic properties of sand and gravel filters is provided by Sherard *et al* (1984).

Suffosion is the migration of fine particles through the voids between larger particles either to be carried away with the water, or to percolate into adjacent voids. Two examples of suffosion are given in Figures 2.5(a) and 2.5(b) (after Head, 1981). A fine grained soil "A" is shown overlying a coarse grained soil "B" in Figure 2.5(a). Water flowing downwards can carry fine particles from soil "A" into the relatively large voids in soil "B". A soil consisting of fine particles contained in a matrix of coarser particles is shown in Figure 2.5(b). Water emerging from this soil may carry fine particles away with it if the velocity of flow is great enough. Suffosion of fine particles can occur not only from or into materials which have been placed or compacted as fill but also from one naturally occurring deposit to another if they have never been subjected to significant hydraulic gradients in the past, and a movement of water is artificially imposed. Suffosion has been extensively researched in the laboratory by Kenney and Lau (1984, 1985 and 1986), Kenney, Lau and Clute (1984), and Kenney *et al* (1985).

Entrainment is described by Reineck and Singh (1973). This is the term used to describe the process of a particle being picked up by a flowing stream and carried along in suspension. The main difference between entrainment as opposed to piping and suffosion is that entrainment involves an open flowing body of water such as a stream. Entrainment does not refer to any subsurface process.

The above definitions indicate which of the above processes are of relevance to this study. Piping is a process which could only occur at the open seepage face of a de-watered void. If this process had been occurring at Hoveringham quarry, it would be of interest in the sense that it would show that fine particles had the potential to be moved within the aquifer. It would not be relevant to our study of settlement mechanisms many tens of metres from the void. Entrainment, referring only to particle transport within open, flowing water bodies, would not be relevant either. Suffosion was the mechanism studied for this investigation, referring as it does to particle movement by interstitial (intergranular) water flow within a body of granular material. This was studied as a possible mechanism whereby particles (which would also have to be subject to some effective stress) might be moved a certain distance by the lateral movement of water and thus cause surface settlement.

Reference to the work of Kenney and Lau enabled a quantitative study to be carried out of suffosion in relation to the de-watering at Hoveringham Quarry. These two researchers developed a method of analysing the particle size distribution of a granular material indicating whether particles would be likely to move through the void network of the material should they become initially disturbed. Kenney and Lau called this an assessment of “stability”. They also defined minimum interstitial water velocities (“unit fluxes”) at which they found particle movement to occur. The “stability assessment” and “unit flux” areas of Kenney and Lau’s work have both been applied to soil samples as part of this research project and this work is described in Chapter Five.

Kenney and Lau described having subjected the soil sample they tested to "severe" seepage conditions. "Severe" implied a Reynold's Number of flow of 10 or greater. Estimating the Reynold's Number of flow within a granular media is not as easy however, as within or around a body of well defined dimensions such as say a pipe or a pile. Also, Reynold's Numbers derived from analyses of flow within different scenarios such as within pipes, around piles, or within granular media are not directly inter-relatable. In Chapter Five, it will be seen how the Reynold's Number of flow adjacent to the quarry wall was estimated as approximately one. However, this depended on a correct estimation of the effective particle diameter (see Appendix 5.1), which in turn depends on the confidence in having taken a representative sample of the material. It must therefore be appreciated that methods of estimating the Reynold's Number within a granular media provide only an approximate index giving an indication of the severity of flow. Further details of the techniques employed by Kenney and Lau are given in Chapter Five.

2.9 Summary

Section 2.2 describes how , with reference to state of the art techniques, a "Structural Inspection Document" was devised for surveying the properties in Hoveringham Village. This document enabled the classification and diagnosis of defects within the structures surveyed. Section 2.3 highlights some inadequacies in current de-watering operation design and makes the point that spatial variation of permeability is given no consideration. Chapters Six and Seven both have sections summarising how spatial variation of permeability around a de-watered zone was examined as part of the research. Section 2.4 illustrates how ground subsidence due to water table drawdown, usually involving extensive drawdowns in deep aquifers, has been well documented world-wide. However, little information was found concerning ground or structure movements due to de-watering of shallow unconfined alluvial aquifers. Furthermore, no literature was found which examined the mechanism of suffosion as a potential cause of ground settlement in such an aquifer. Chapters Five, Six, and Seven have sections which discuss and examine suffosion as a potential cause of ground settlement around Hoveringham Quarry.

<table>
<tr><th>Location</th><th>Magnitude of Subsidence</th><th>Extent of Drawdown</th><th>Time Period</th><th>Reason for Abstraction</th><th>Ground Conditions if Known</th></tr>
<tr><td>Venice</td><td>15 cm</td><td>-</td><td>1930 - 1973</td><td>Industrial pumping</td><td></td></tr>
<tr><td>Mexico City</td><td>up to 8 metres</td><td>-</td><td>began 1938</td><td>Water supply</td><td>Thick, compressible deposits of sand and silt</td></tr>
<tr><td>Tokyo / Osaka</td><td>3 to 4m max</td><td>-</td><td>1928-1934</td><td>Water supply</td><td></td></tr>
<tr><td>Taipei Basin</td><td>up to 1m max</td><td>-</td><td></td><td>Supply / Industry</td><td></td></tr>
<tr><td>London</td><td>6 to 18cm</td><td>-</td><td>1865 - 1931</td><td>Water supply</td><td>Thick clay overlies pumped chalk aquifer</td></tr>
<tr><td>Baton Rouge, Louisiana</td><td>0.3m</td><td>60m total (5cm/10m drop in head)</td><td>since 1890</td><td>Industrial pumping</td><td rowspan="3">Subsidence occurring in coastal plain and shallow marine deposits</td></tr>
<tr><td>Houston, Texas</td><td>1.5m max</td><td>60m approx</td><td>before 1969</td><td>-</td></tr>
<tr><td>Baytown, Texas</td><td>up to 2.7m</td><td>60m approx</td><td>before 1976</td><td>-</td></tr>
<tr><td>San Joaquin, California</td><td>up to 8.5m</td><td>up to 100m</td><td>before 1969</td><td>Irrigation</td><td rowspan="3">Susidence occurring in alluvial fan and floodplain deposits</td></tr>
<tr><td>Santa Clara, California</td><td>4m subsidence observed</td><td></td><td>before 1969</td><td>Irrigation</td></tr>
<tr><td>central Arizona</td><td>2.3m</td><td>46m</td><td>before 1975</td><td>-</td></tr>
<tr><td>Wairakei, New Zealand</td><td>4m max</td><td>0.4m per year and ongoing</td><td>since 1956</td><td>Hot water from geothermal fields for heating, power</td><td></td></tr>
</table>

Table 2.1

Locations World-Wide To Have Suffered Large Settlements Caused By Groundwater Abstraction (After Bouwer, 1978)

	APPLICABILITY					
Type of test	Gravel	Sand	Clay-layered	Clay-structured	Weathered Rock	Fresh Rock
<u>Well tests</u>						
Pump Out Test-unconfined aquifer	*	*	o	o	x	x
Pump Out Test-confined aquifer	*	*	o	o	x	x
<u>Borehole tests</u>						
Constant Head - open	x	*	*	*	x	x
Constant Head - packer	x	x	*	*	*	*
Variable Head - open	*	x	o	o	x	x
Variable Head - packer	x	x	x	*	*	*
<u>Static test</u>						
Open	*	*	o	o	x	x
Packer	x	x	*	*	*	*

Key:

o Of limited use or with special precautions
x Useful but not always applicable
* Recommended test procedure

<u>Table 2.2</u>

<u>In-Situ Permeability Test Methods And Their Applicability.</u>

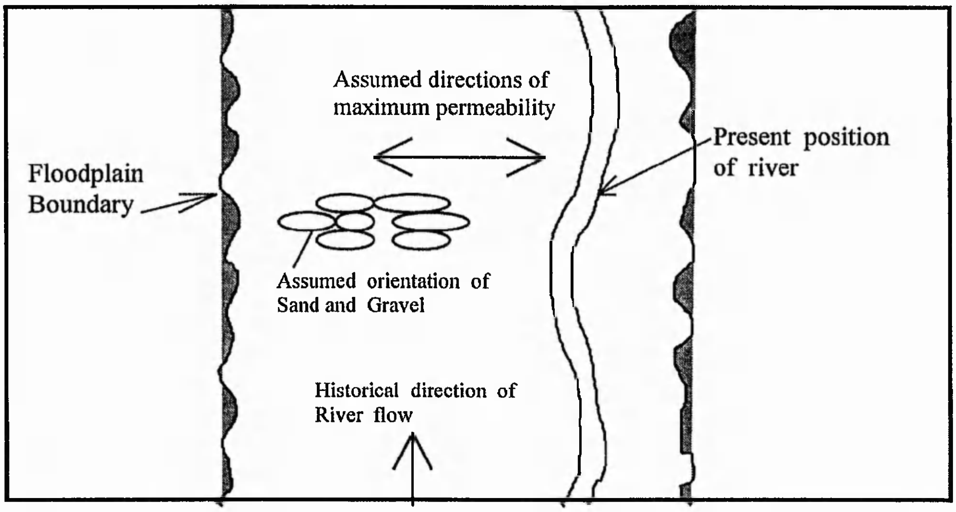


Figure 2.1

A Mechanism Which Can Lead To Directional Permeability Variation Within A River Floodplain.

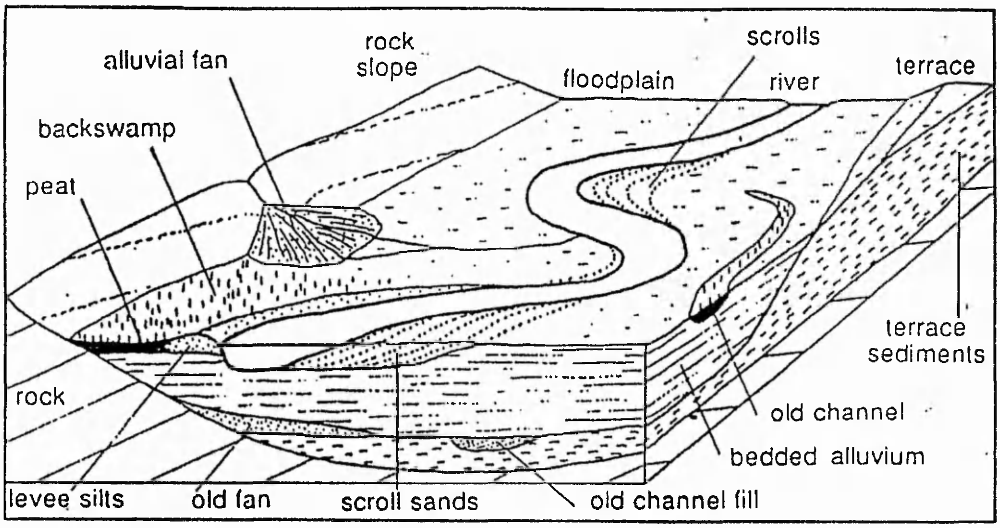


Figure 2.2

Various Geological Features Which Can Exist Within A River Floodplain

(After Petts, 1989)

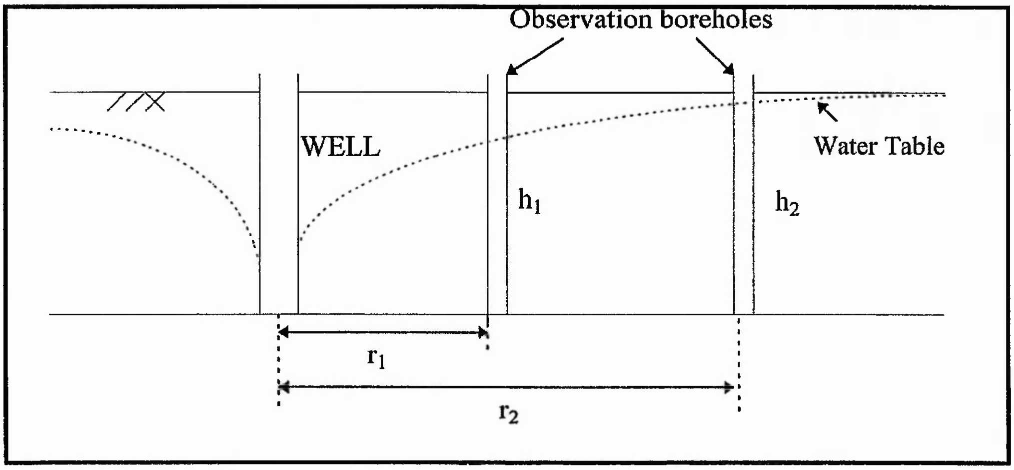


Figure 2.3

Cross Section Through The Well And Two Boreholes During A Well Pumping Out Test.

(After Craig, 1987)

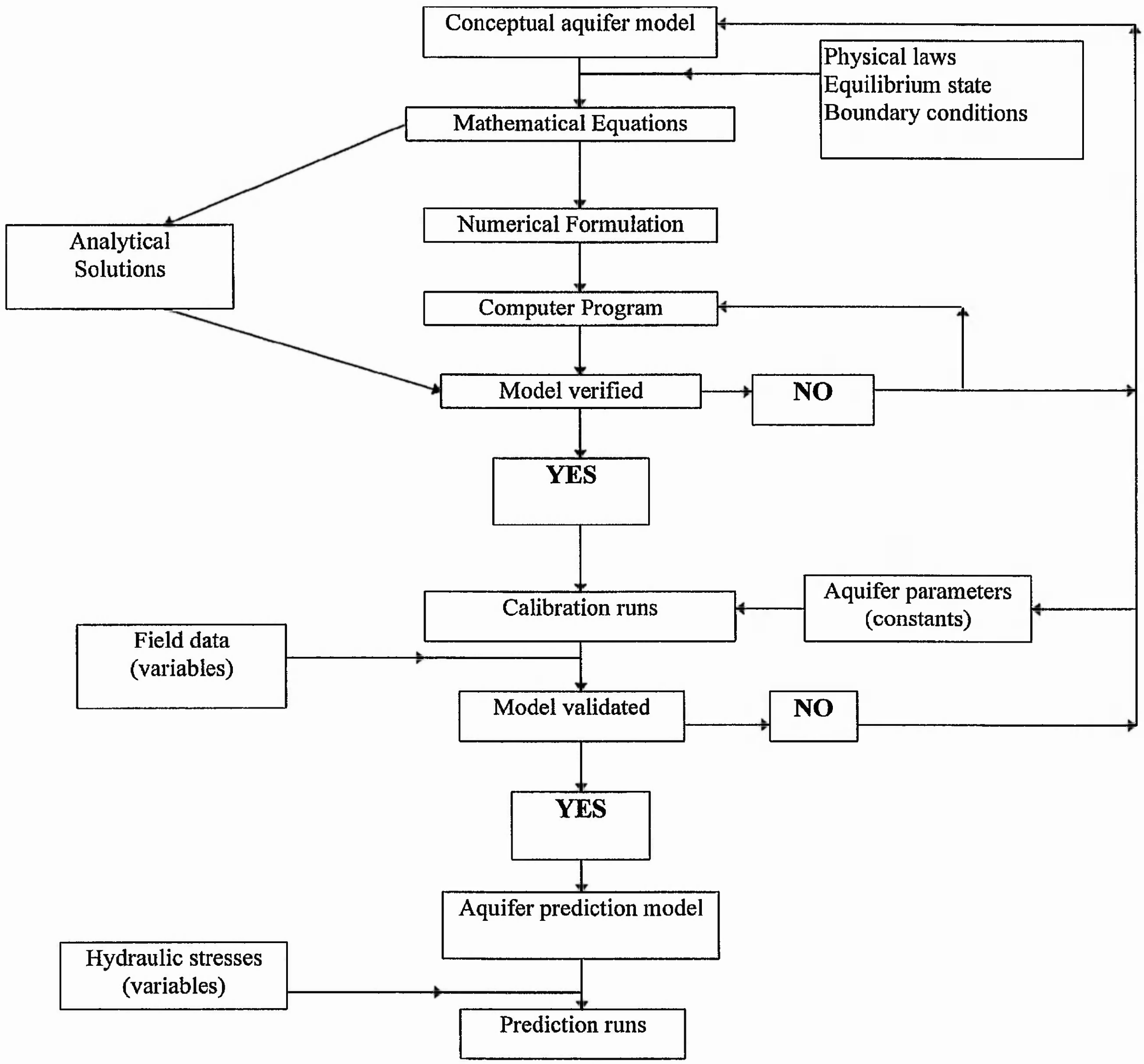
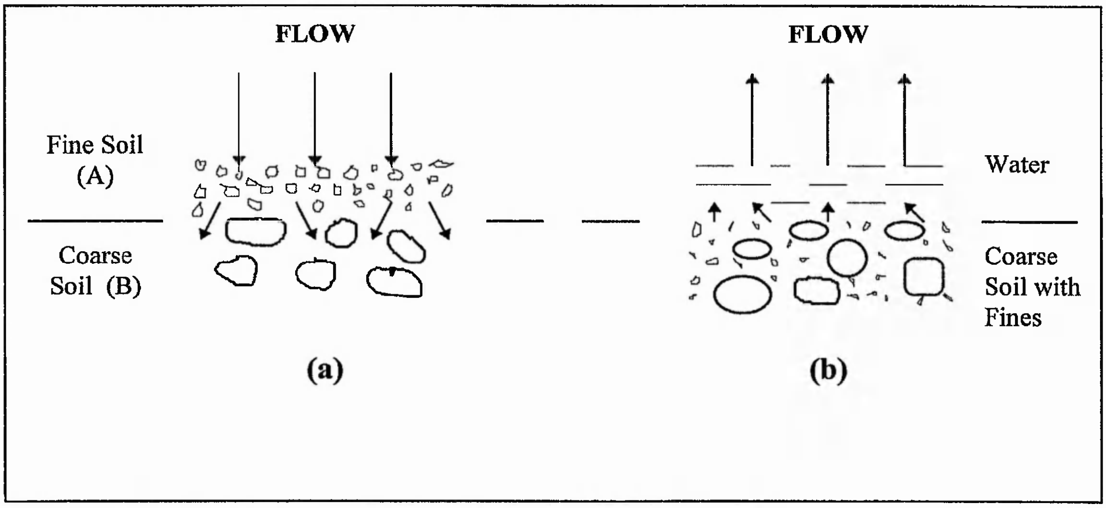


Figure 2.4

Development Sequence For A Prediction Model Using Numerical Simulation.

(After Peck.1988)



Figures 2.5(A) And 2.5(B)

Mechanisms Of Suffosion

(After Head, 1981)

CHAPTER THREE

FINDINGS FROM A GEOLOGICAL AND ENVIRONMENTAL DESK STUDY OF HOVERINGHAM AND ITS SURROUNDING AREAS.

3.1 Introduction

The following areas were investigated for this desk study:

1. Previous Site Investigations, Reports and Plan Maps.
2. Previous Desk Studies.
3. Geological and Ordnance Survey Maps.
4. Mining Record Maps.
5. Verbal Information from Local People or Professionals within the industry.
6. Aerial Photographs.

By far the most fruitful sources of information were previous site investigations, desk studies, plan maps and reports. These gave valuable information as to sand and gravel depth and quality, overburden thickness, floodplain width, and past quarried and infilled areas. Geological and Ordnance Survey maps were also useful as further confirmation of findings reported in the previous site investigations. Written confirmation from British Coal (17/2/93) and active and abandoned coal seam plans confirmed mining could not be the cause of any past, present or future ground settlements in the area of interest. Verbal information and aerial photographs were both less useful avenues of investigation.

This research project dealt with some sensitive environmental issues which are also discussed here. It will be seen that some environmental and technical issues are inter-related. A good example is the use of pulverised fuel ash as an infill material which could have a significant effect on the hydrogeological regime by lowering the permeability of large regions across the aquifer.

3.2 Previous Site Investigation: A Hydrogeological Investigation Of The Area Surrounding Hoveringham Sand And Gravel Quarry

3.2.1 Objectives And Data Sources

This in depth site investigation of the area was undertaken by Tarmac Quarry Products Ltd (1993). Site work was underataken during late 1992 and early 1993. The study objectives were to gain an understanding of the present hydrological and hydrogeological regime and assess the environmental effects of a proposed quarry on the area. The following formed data sources for the report:

Publications of the British Geological Survey (B.G.S)

Published geological maps at scales of 1:50,000 and 1:10,000
Regional Geology Memoirs: "Central England" and "Pennines and Adjacent Areas"
Technical Report WA/90/1 - "Nottingham: A Geological Background for Planning and Development"

Field Investigations

Stream flow monitoring
Falling head and constant head in-situ permeability tests
Installation of 28 piezometers to determine groundwater levels
Mineral assessment borehole drilling
Static cone penetrometer testing

Institute of Hydrology (I.O.H)

Well records register

National Rivers Authority (N.R.A)

Rainfall data
Historical flooding data
Licensed surface and groundwater abstractors
Discussions with pollution control officers, hydrogeologists and development co-ordinators

Newark Area Internal Drainage Board

Discussions to assess feasibility of re-aligning ditches

Natural Environment Research Council

"Hydrogeology of the Bunter Sandstone in Nottinghamshire."

3.2.2 Site Location

The study area was Hoveringham sand and gravel quarry and its surroundings approximately 1.5 kilometres to the north of Hoveringham village and 15 kilometres to the south-west of Newark. The site is shown in the extract from the O.S. 1:50,000 sheet number 129 (Nottingham and Loughborough) in Figure 3.1.(after Tarmac Quarry Products Ltd,1993)

3.2.3 Topography And Hydrology

3.2.3.1 Topography

The study area is described as lying within the uniformly flat valley of the River Trent which trends from south-west to north-east, averaging 18 metres above ordnance datum (A.O.D). The valley floor slopes gently towards the River Trent and down the long profile of the valley with a gradient of approximately 1 in 1,400. Small breaks of slope in the valley floor generally correlate well to the junctions of terrace gravels and alluvium. The valley floor width is described as approximately 2.5 kilometres, with sideslopes rising sharply to about 70m A.O.D. on either side.

3.2.3.2 Rainfall

Data was obtained from the two nearby rainfall gauging stations at Southwell and Burton Joyce Waterworks. A summary of the long term monthly average rainfall for the area on the basis of this data appears in Table 3.1.(after Tarmac Quarry Products Ltd, 1993).

3.2.3.3 Watercourses

Several manmade and natural watercourses are described as existing in the vicinity. The locations of these are shown in Figure 3.2. The largest watercourse is the River Trent which is generally over to the eastern most part of the valley throughout the area of interest. At its closest point, the river approaches to within 1.1 kilometres of the quarry.

3.2.3.4 Limits Of The Floodplain

The limits of the floodplain of the River Trent were obtained from the N.R.A. Flood Defence and are shown in Figure 3.3. The Figure also shows how nearly all the structures in Hoveringham village have been sited off the floodplain to decrease the likelihood of suffering flood damage.

3.2.4 Geology

3.2.4.1 Regional Geology

The sand and gravel mineral worked at Hoveringham is described as part of the continuous belt of Trent River Gravels which extends for over 80 km from Burton Upon Trent to Dunham Bridge in the north-east. The mineral overlies Permo-Triassic Marls of the Mercia Mudstone Group which in turn are underlain by the Sherwood Sandstone Group. Young deposits of alluvium (fine silts and clays) rest on the sand and gravel but these are limited in extent.

3.2.4.2 Local Geology

The report presents the geological sequence which outcropped in Hoveringham Quarry. The sequence is shown in Table 3.2.(after Tarmac Quarry Products Ltd, 1993)

3.2.4.3 Detailed Geology

Both the deep Sherwood Sandstone and the Mercia Mudstone underlying the sands and gravels are described in the report. They will not be described here as their relevance to this study is limited. It will be sufficient to note that the impermeable and extensive nature of the Mercia Mudstones means that they cause the underlying Sherwood Sandstone to exist as a confined aquifer. The sand and gravel superficial deposits exist as an unconfined aquifer above the Mercia Mudstones which prevent any hydraulic continuity between the two aquifers. A generalised section through the Mercia Mudstone Group is shown in Figure 3.4.

More than 100 mineral evaluation and groundwater investigation boreholes were drilled by Tarmac Quarry Products Ltd. The locations of these holes and a geological cross section based on them are shown in the plan map of Appendix 3.1.

The cross section shows the superficial deposits across the floodplain to be extremely uniformly distributed. The main characteristics of the deposit are clay or silty sand and gravel on the surface to a depth of 1 to 1.5 metres. These are underlain by a consistent 6 or 7 metres of the sand and gravel mineral. The almost flat upper surface of the "Marl" occurs at a depth of 7 to 9 metres below ground level. The "Marl" (previously known as "Keuper Marl" now commonly known as "Weathered Mercia Mudstone") occurs as a low permeability stiff clay where it outcrops at the floodplain boundaries and becomes a Mudstone at depth.

3.2.5 Hydrogeology

3.2.5.1 The Hydrogeological Regime

The report states that the hydrogeological regime at Hoveringham can be divided into three sections:

1. The Trent River Gravels.
2. The Mercia Mudstone Group.
3. The Sherwood Sandstone.

The Trent River Gravels are described as a "laterally extensive but vertically limited aquifer which is considered to be unconfined". Permeability tests indicated relatively uniform aquifer parameters existed. Recharge to this aquifer was considered to derive from the three sources listed below in order of decreasing quantity of recharge.

1. Rainfall.
2. Runoff from the valley sides.
3. Leakage from water courses.

3.2.5.2 Permeability Of The Sands And Gravels

As part of the investigations for the report, both field and laboratory based testing was undertaken to estimate the permeability of the sands and gravels. Falling head and constant head borehole tests were carried out. Hazen type analysis of particle size distributions (P.S.D.'s) was used to assess the sand and gravel permeability in the laboratory. The following average values were found from these tests:

Falling Head Tests (in situ)	-	7.7×10^{-5} m/s
Constant Head Tests (in-situ)	-	2.3×10^{-5} m/s
Hazen P.S.D analyses	-	5.8×10^{-4} m/s

3.2.6 Potential For Settlement Due To De-Watering

The report states that ground settlement due to de-watering of the sands and gravels can arise from the following two mechanisms:

1. Effective stress increase due to water table lowering.

2. Removal of fine particles by flowing water.

The report states that Mechanism 1 should cause negligible ground settlements under nearby structures due to the distance of the structures from the proposed quarry and the resulting small water table drawdown beneath them. Mechanism 2 is described as being unlikely to occur as the sands and gravels consist of a broad and continuous range of particle sizes helping the material to act as a "natural filter" preventing particle migration. Both the above mechanisms were studied in more detail as part of this research project.

It is noticeable in the report that Tarmac Quarry Products Ltd did not carry out any plate loading tests which could have helped in settlement estimation by providing compressibility data. It is recognised that such testing would have been a desirable part of the research for this thesis but this was precluded due to time and budget constraints.

The report did not consider spatial variation of permeability as a mechanism which could lead to varying distances of influence around a de-watered zone. Spatial variation of permeability is described in Chapter Two and its possible effects on the de-watered zone were studied as part of the finite element investigations described in Chapter Seven. The author feels that the above report's conclusions on the potential for settlements due to fine particle movement by flowing water were based on insufficient research. This mechanism (called "suffosion") was studied in much greater detail for this research project and the conclusions from this work are detailed in Chapter Five.

3.3 Geological And Plan Maps

3.3.1 British Geological Survey Walkover Map

A plan map summarising findings of a walkover survey to study the surface deposits in the Hoveringham area was obtained from the British Geological Survey. The study for the map was commissioned in 1989 by the Natural Environment Research Council. A section of the map appears in Figure 3.5. Considering Hoveringham Village on the Trent floodplain, the plan shows extensive areas of sand and gravel with only topsoil cover with lower cohesive alluvium infilled channels in between these areas. The cohesive alluvium is defined as "Red-Brown Clay" and is shown to occur from ground level to a depth of 0.5m to 1m under which lies sand and gravel. Figure 3.5 also shows how nearly all of the buildings in the village are founded on the higher sand and gravel terraces where a cohesive alluvium cover does not exist. This was presumably to decrease the likelihood of flood damage, as well as to avoid the problems which can be caused by clay shrinkage under a foundation.

3.3.2 "State Of Workings" Plan Map, Tarmac Roadstone Eastern

The plan map in Figure 3.6 prepared by Tarmac Roadstone Eastern in 1981 and amended in 1983 shows the "State of Workings" related to mineral extraction in the Hoveringham area. Extraction began in the north-east of this map and progressed in a south-westerly direction with extraction and infill rates probably being virtually identical. The map shows how an area of approximately 4km^2 has been worked out and infilled with pulverised fuel ash ("p.f.a."). Historically, extraction would have been by the "strip" method, a strip of land being worked and then immediately infilled. It can be seen that previously worked out areas are mostly well away from settlements. As the workings have approached the settlement of Hoveringham and large scale pumping of extensive dry worked quarries has taken over as the preferred working method, mineral extraction has become a much more sensitive local issue. It can also be seen from the plan map that further south the occurrence of large worked out areas which are not infilled or have been left as lagoons increases. This is probably a result of the local source of p.f.a., Staythorpe Power Station, closing.

This has added to the sensitivity of the extraction workings as the local people have realised that without a ready form of infill they could become an "island community".

3.3.3 The 1:10,000 Scale Ordnance Survey Map

The 1:10,000 scale Ordnance Survey plan map in Figure 3.7 clearly defines the boundaries of the Trent floodplain, the size of the gravel quarry, worked out but water filled areas and the boundaries of the village in the area of interest. To the north the floodplain boundary is approximately defined by the A612 road which is also effectively where the Mercia Mudstone outcrops. The river itself forms the southern boundary over virtually the whole extent of the plain. The river is on the eastern boundary of the floodplain well away from the extraction area and so would not form a ready source of recharge water to the section of the aquifer subject to water table drawdown around the quarry. What is also evident is the very large size of the present quarry compared to the other worked out areas.

3.3.4 Plan Map Of Hoveringham Quarry By Tarmac Quarry Products Ltd

The plan map of Figure 3.8 shows in detail the phased processes in the area which was being worked during 1993 and a proposed future extraction area. The 1993 workings are labelled "Existing Quarry" and "Ash Lagoons". The structures of Hoveringham Village are also clearly marked. The nearest structures to the quarry are approximately three hundred metres away. The village then extends away from the worked quarry in a half moon shape, the furthest structures being over a kilometre away. One important finding emerging from this map is that it shows that the first areas to be p.f.a. infilled were between the structures and the large "Existing Quarry" area. This could have been beneficial in terms of reducing the effects of drawdown towards the structures as the large quarry was de-watered.

3.4 Previous Desk Studies

3.4.1 Hydrogeological Desk Study by Wimpey Environmental

In August 1992, Wimpey Environmental prepared a "Hydrogeological Desk Study of a Proposed Extension to Hoveringham Sand and Gravel Quarry, Nottinghamshire". The objective behind this report was to assess the hydrogeological implications of a proposal by Tarmac Roadstone (Eastern) Ltd to extract sand and gravel from land located to the west of Hoveringham Village.

The proposed extension was to the west of the quarry studied for this research. The location of this proposed extension is shown in Figure 3.9. The boundary of influence of the de-watering drawn around the site in Figure 3.9 was calculated assuming an aquifer permeability of 1×10^{-4} m/s and a total drawdown of 10 metres.

The report points out that as the proposal was to de-water a large void which would then be worked in the dry, possible subsequent localised problems could be:

- Settlement of buildings.
- Derogation of quality of agricultural land and reduction of crop yields.
- Derogation of licensed groundwater abstractions.
- Reduction of flows in local streams.

The report also states that p.f.a. backfill could cause water logging due to its very low permeability. If domestic or other waste were used as a backfill in the event of a p.f.a. shortage, possible resulting problems would be:

- Pollution of groundwater within the shallow gravel aquifer.
- Pollution of surface waters in ditches, becks and the River Trent.

The report summarises the geology of the area with reference to The British Geological Survey 1:50,000 scale map. The report describes the site as underlain by Recent and

Pleistocene Alluvium overlying River Terrace Gravels. Bedrock is described as Permo-Triassic Flag Sandstones and Marls which are collectively known as Waterstones and are part of the Mercia Mudstone Group. The gently sloping upland areas bordering the floodplain are described as Weathered Mercia Mudstone. The report states the sand and gravel deposit is on average 7.5 metres thick extending to a depth of up to 11 metres. This is a slightly greater depth of sand and gravel than that described in Section 3.2.4.3. The bedrock consists of 55 metres of Waterstones which in turn are underlain by over 100 metres of Sherwood Sandstone.

A study carried out by A.H.S.Waters and Partners of Birmingham in July 1961 is quoted from which the depth to groundwater in the deposit was found to be 1.1m to 1.9m.

The report states that possible "operational" or short term effects of the de-watering could be:

- Drying of the soil in the unsaturated or vadose zone with its subsequent adverse effect on plant growth.
- A reduction in surface runoff to streams and ditches in the area.
- A reduction in the overall load bearing capacity of the substrata, leading to ground settlement.
- High groundwater seepage close to the excavation could cause clay and silt particles to be drawn out from the sand and gravel leading to a reduction in the strength characteristics of the sub soil. (This is in fact a reference to the phenomenon of "suffosion" discussed in other sections of this thesis)

3.4.2 The Hoveringham Parish Council Submission To Nottinghamshire County Council Prepared By Wimpey Environmental Ltd (November 1993)

In the Hydrological and Hydrogeological Issues section of the above report information is given on total and effective rainfall, both monthly and annually. Effective rainfall is that proportion of rainfall which is available for surface drainage flows and groundwater recharge by infiltration through to the water table after evapotranspiration has taken place. The data was derived from two local rainfall gauging stations. The data is given in Table 3.3. and shows the typical UK situation where effective rainfall is very low or even zero in the Summer. This information was used during selection of infiltration values for the finite element analyses described in Chapter Seven.

In the "Review of Environmental Statement : Geology" section of the report, it states that minor sandstone aquifers were encountered within the Mudstone strata during borehole investigations which were reported as being artesian. The report goes on to say that if a minor artesian aquifer were close to the Mudstone/River Terrace Gravel interface and the artesian pressure released during quarrying, settlement of the upper Mudstone strata might occur. However, no evidence was cited for the existence of such "minor sandstone aquifers". It is the author's opinion that this is a mechanism of settlement which is highly unlikely to occur.

In the "Review of Environmental Statement : Excavation" section of the report, the possible consequences of the two main restoration options are discussed. These are leaving the excavation as a water filled feature and infilling with p.f.a. According to the report, environmental and technical factors which are the subject of doubt are:

1. The chemical properties of the groundwater within the mass of the p.f.a. fill. Is it a "non-contaminating source"?

2. What will be the in-situ permeability of the p.f.a. infill and what effect will this have on groundwater levels in the surrounding area?

3.5 Summary

The main findings from the desk study were:

- The unconfined aquifer in the Hoveringham area is between 5 and 10 metres thick and occurs as sand and gravel terraces in between which are lower alluvium filled channels.
- The unconfined water table is encountered at a depth of approximately 1 metre below ground level.
- Below the unconfined aquifer lies Weathered Mercia Mudstone preventing any lower, confined water reaching the surface.
- The floodplain boundaries are as shown in Figure 3.3.
- The lower and upper limits of permeability of the sands and gravels can be taken as 10^{-5} m/s and 10^{-3} m/s respectively.
- Spatial variation of permeability was not taken into account when the initial design of the de-watering operation for Hoveringham quarry was carried out.
- Fine particle movement due to lateral flow of water was not subjected to sufficient analysis to rule it out as a potential cause of ground settlement.

Average rainfall in millimetres for the month shown											
Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
52	45	51	51	53	54	48	59	54	46	59	58

Table 3.1

Long Term Average Monthly Rainfall Data For A Period Of Approximatey 50 Years.

(After Tarmac Quarry Products Ltd. 1993)

Formation / Unit	Thickness	Lithology
Overburden/Alluvium	1.4m ave	Topsoil, subsoil. Sand with silt; sand with gravel and silt.
Trent River Gravels	6.1m ave	Medium to coarse grained sand and gravel comprising grains of quartz, quartzite, flint and occasional localised coal fragments
Unconformity	--------	---
Mercia Mudstone Group		Red-brown and grey mudstones with thin beds of dolomitic sandstone (skerries). Gypsiferous beds occur within the upper part of the formation.

Table 3.2

Geological Sequence Recorded In Hoveringham Quarry

(After Tarmac Quarry Products Ltd. 1993)

Month	Long Term Average Rainfall (mm)	Effective Rain (mm)
January	63	50
February	52	37
March	56	28
April	53	15
May	54	7
June	61	1
July	57	4
August	63	1
September	58	3
October	59	9
November	63	22
December	68	45
YEAR	**707**	**224**

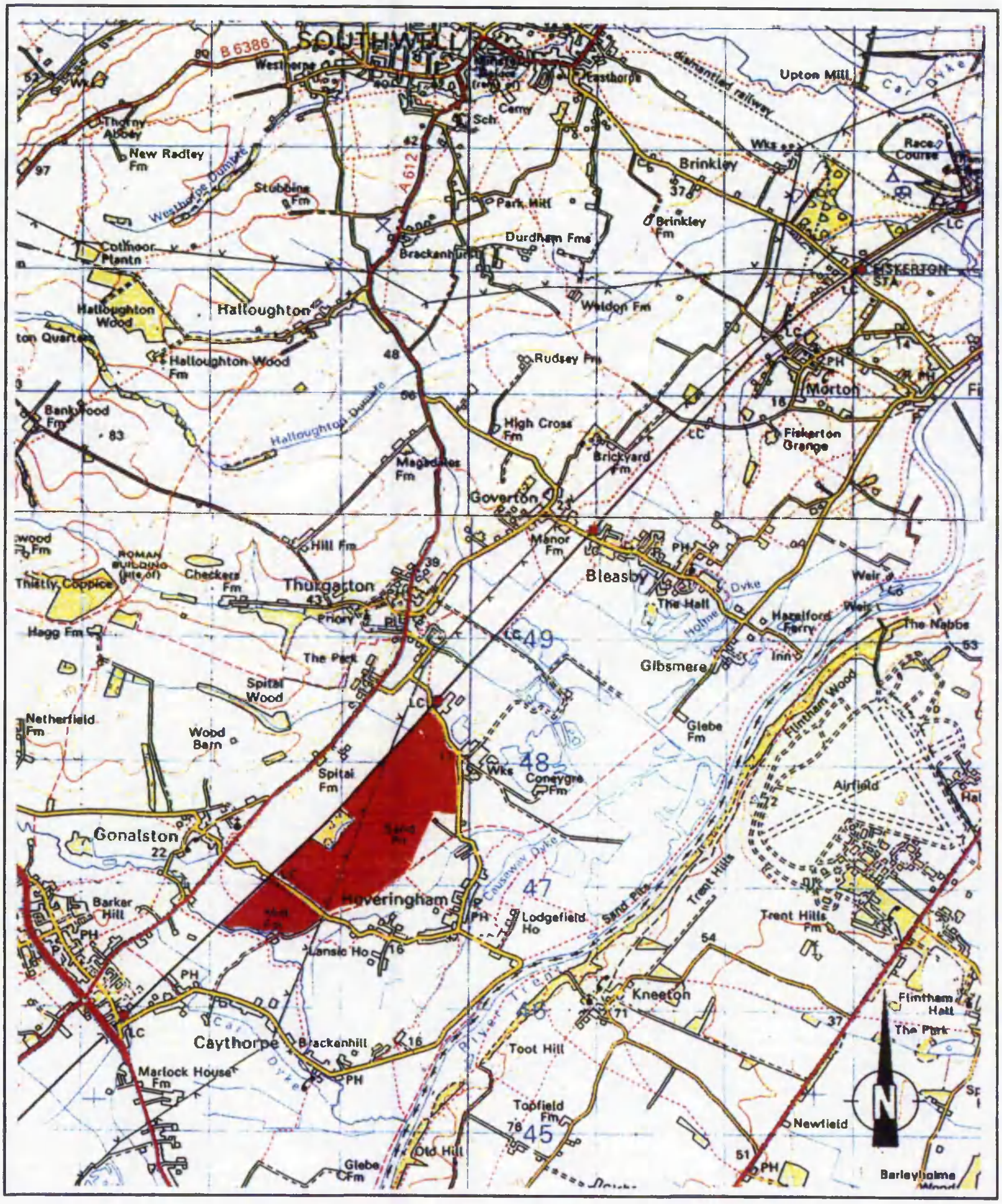
Table 3.3

Meteorological Office : Long-Term Average And Effective Rainfall Data For Nottingham From 1961 - 1992.

(After Hoveringham Parish Council. 1993)

Note to Table 3.3:

It will be seen that the values in Tables 3.1 and 3.3 differ. The most probable reason for this is that the averages for the two tables refer to different total time periods and different overall catchment areas.

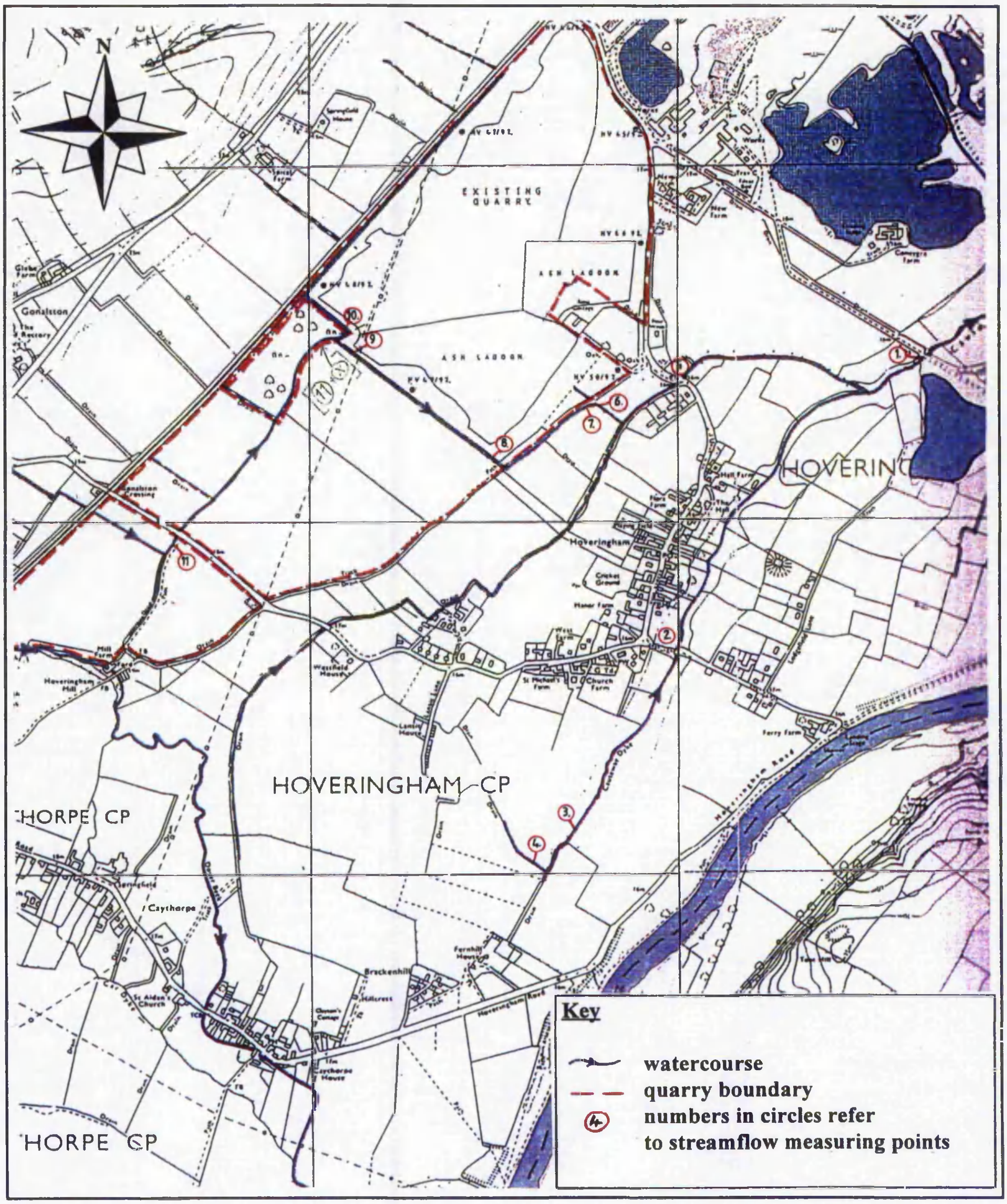


Scale: 1:50,000

Figure 3.1

Location Plan For Hoveringham Quarry

(after Tarmac Quarry Products Ltd, 1993)

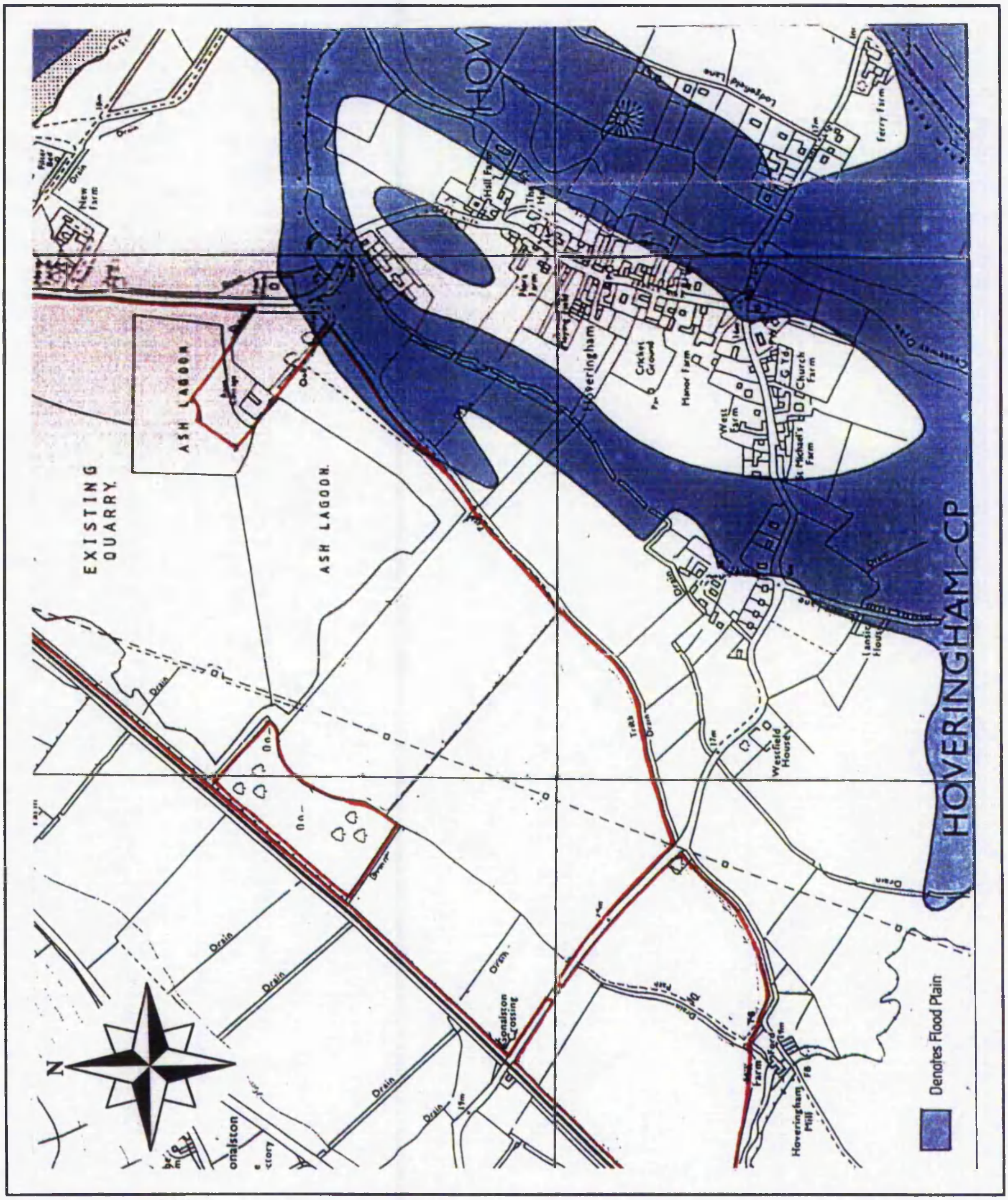


Scale 1:18,000

Figure 3.2

Watercourses In The Vicinity Of Hoveringham Village

(after Tarmac Quarry Products Ltd. 1993)



Scale 1:12,000

Figure 3.3
Limits Of The Trent Floodplain In The Hoveringham Area
(after Tarmac Quarry Products Ltd. 1993)

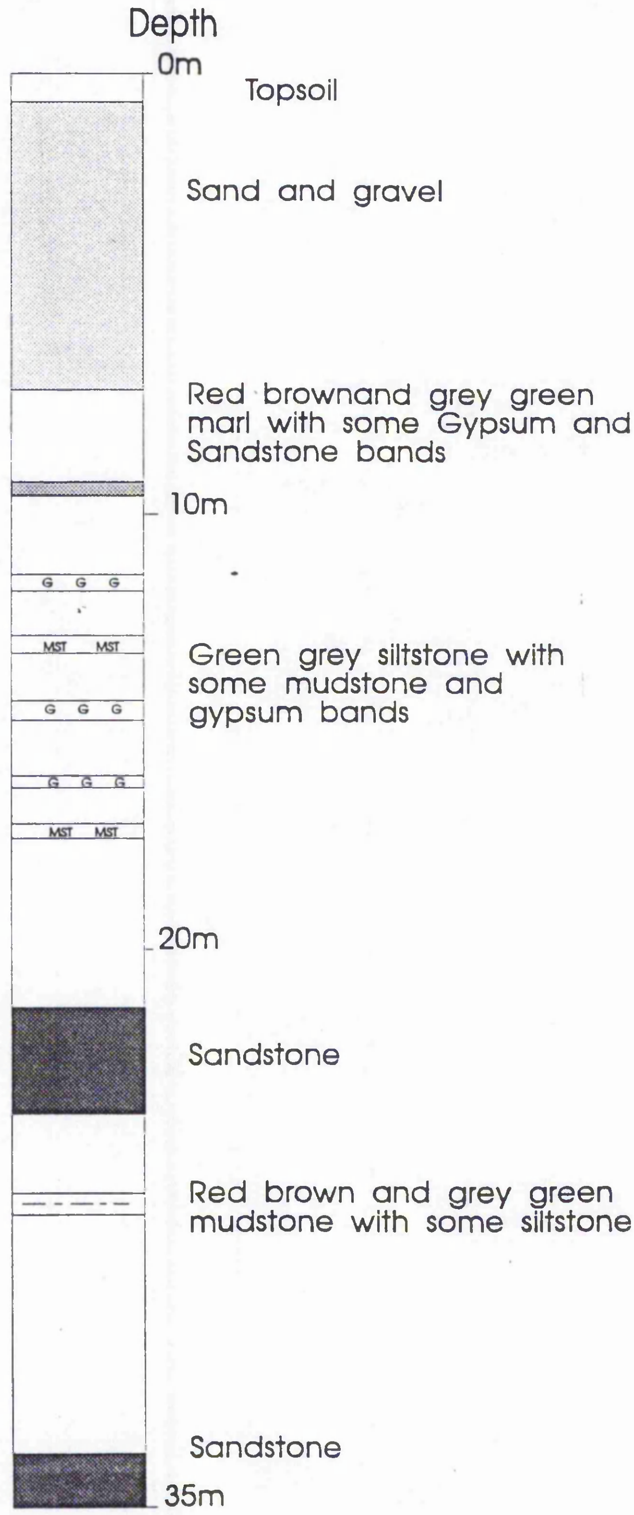
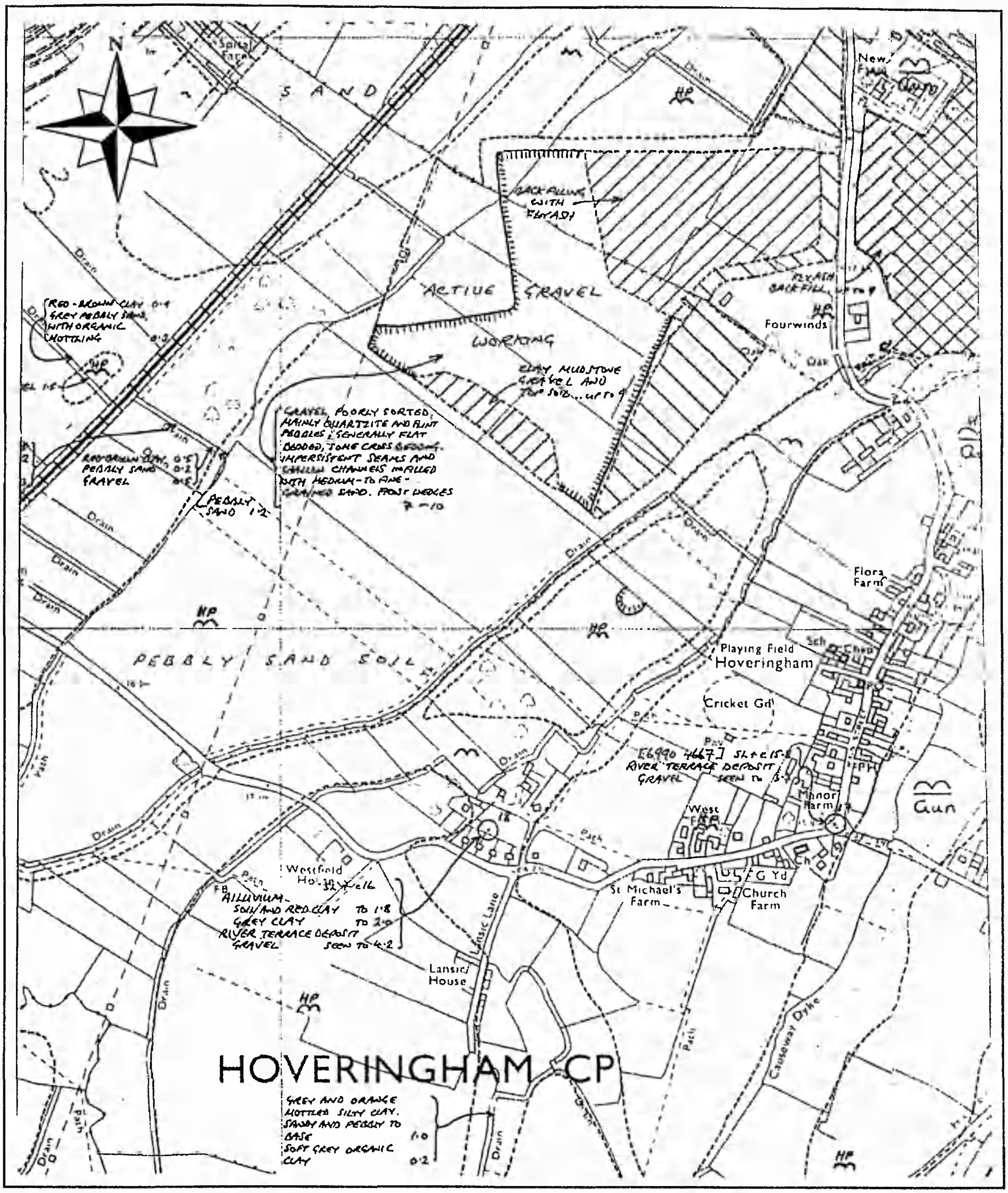


Figure 3.4

Generalised Section Through The Mercia Mudstone Group

(After Tarmac Quarry Products Ltd, 1993)



Scale 1:10.000

Figure 3.5
An Extract From The British Geological Survey Walkover Map Of The Hoveringham Area

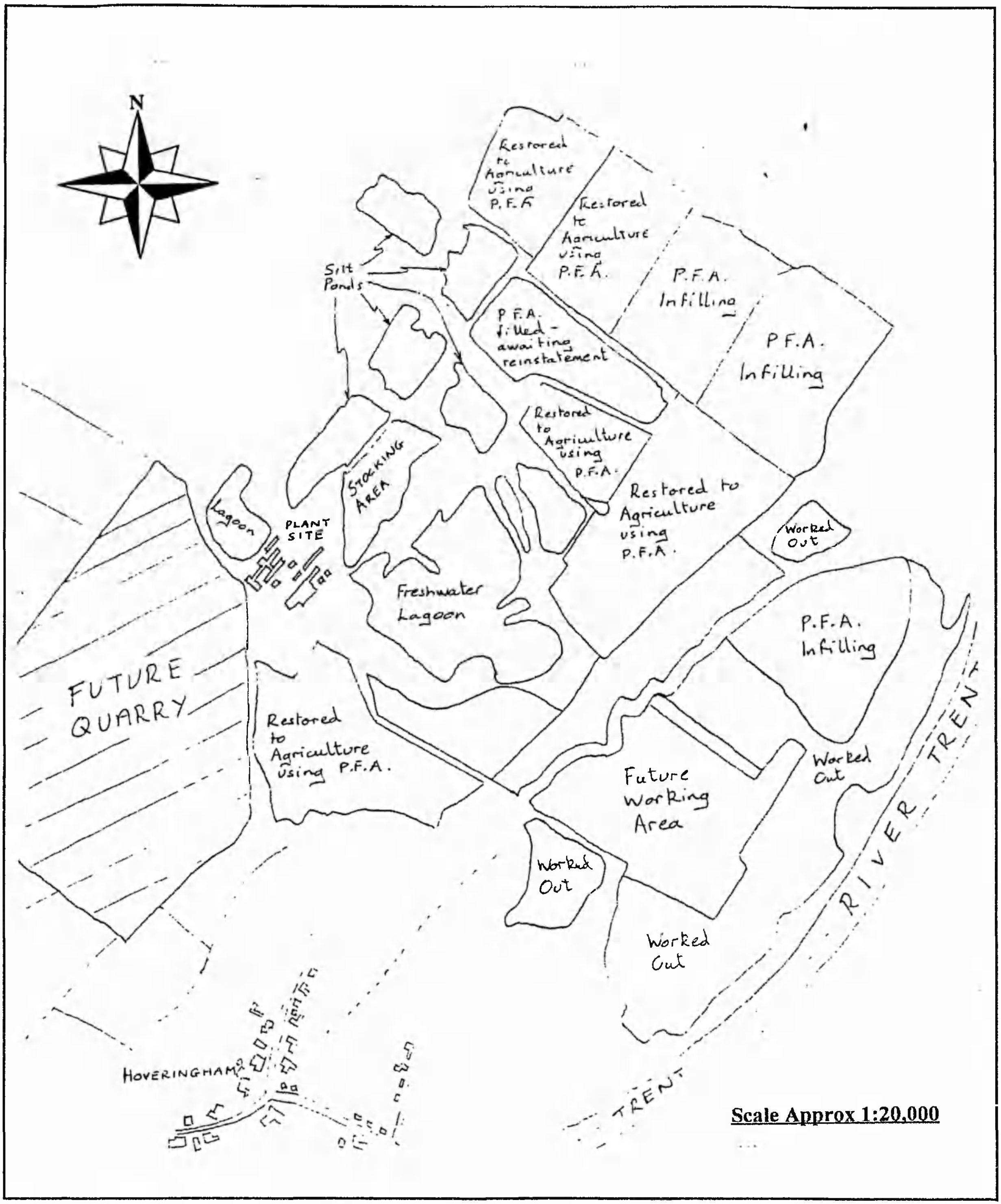


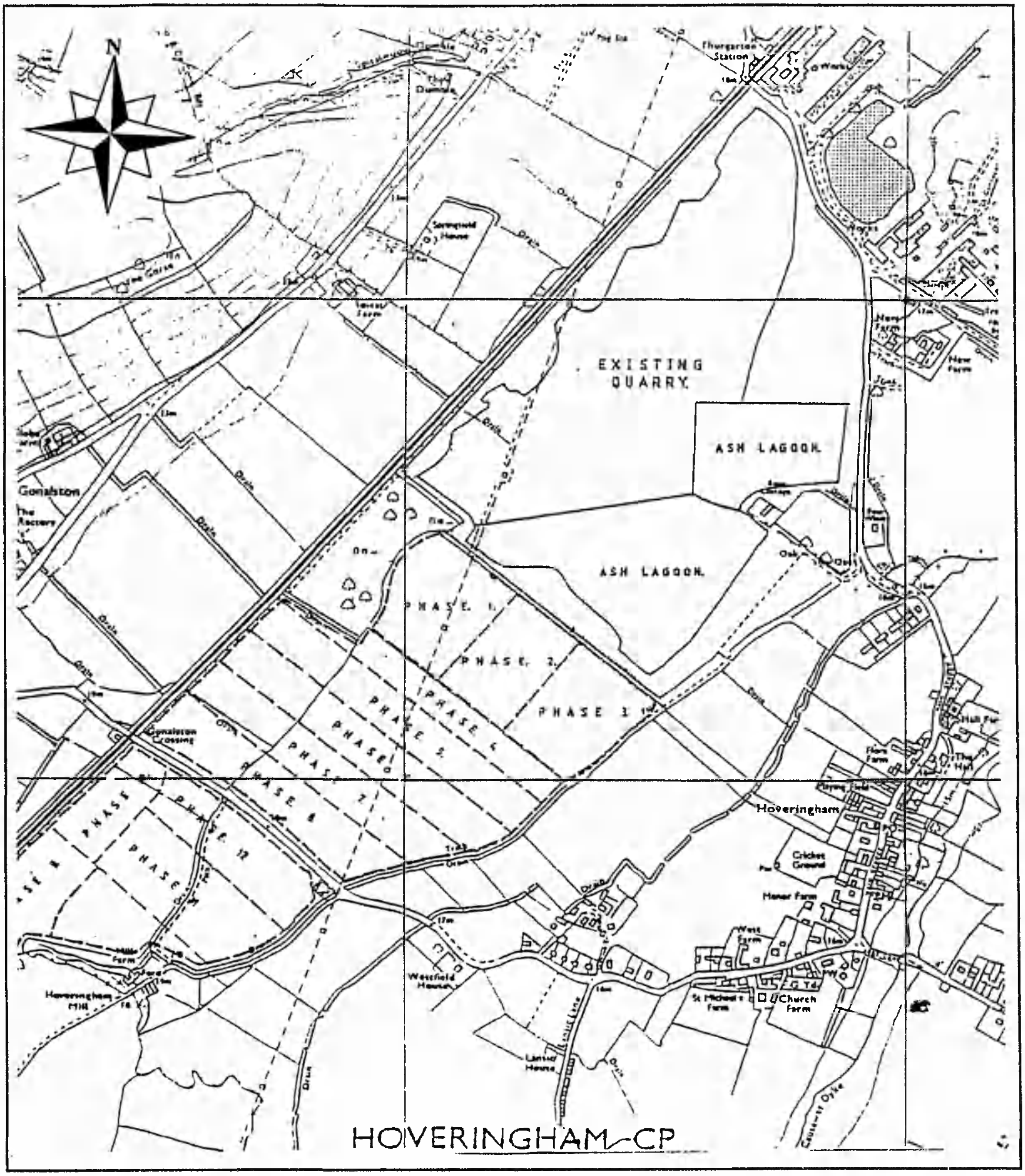
Figure 3.6

The State Of Workings Around Hoveringham Quarry In 1983

(after Tarmac Quarry Products Ltd. 1993)

THIS FIGURE IS AN OVERSIZE MAP LOCATED IN THE REAR WALLET

Figure 3.7
An Extract From The 1:10,000 Scale Ordnance Survey Plan Map Of The Hoveringham Area



Scale Approx 1:13.000

Figure 3.8
Phased Processes In The Quarry Worked In 1993 And
A Proposed Future Extraction Area
(after Tarmac Quarry Products Ltd. 1993)

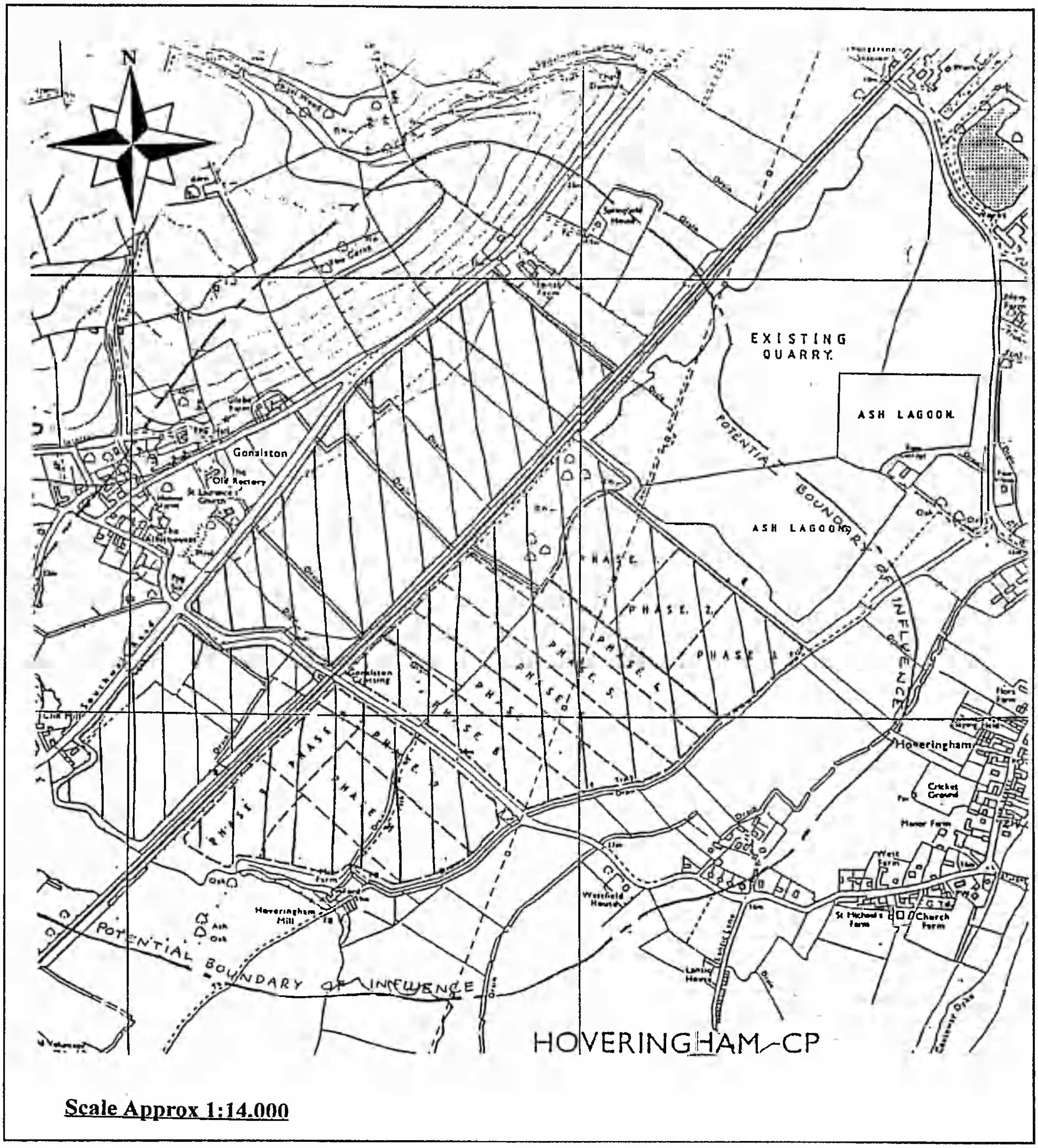


Figure 3.9
Location Of A Proposed De-Watered Extension To Hoveringham Quarry With Estimated Boundary Of Influence Sketched On (after Wimpey Environmental Ltd. 1992)

CHAPTER FOUR

STRUCTURAL DEFECT INVESTIGATIONS OF THE STRUCTURES IN HOVERINGHAM VILLAGE.

4.1 Introduction

The original hypothesis at the beginning of the research was that one possible mechanism which had lead to ground and structure settlements in the village of Hoveringham was extensive pumping of groundwater from a nearby sand and gravel quarry.

It was decided that it would be necessary to conduct a rudimentary structural inspection of as many properties as possible in an attempt to classify the type, severity and distribution of defects within the village. The sources of information referred to on how to conduct structural inspections, the observation and diagnosis of defects and how to record them are set out in Chapter Two.

4.2 Structural surveys of properties in Hoveringham carried out prior to commencement of the research

The report dated 28th July 1992 in Appendix 1.1 (after Page, 1992) summarises structural surveys of properties in Hoveringham village which were carried out prior to commencement of the research. The report states that nine properties were surveyed, all of which had suffered structural damage due to ground subsidence. In five properties the settlement was attributed either solely or to a combination of:

- A faulty drainage system
- Partial cellar construction and sloping ground
- Clay shrinkage
- Poor construction details

In four of the properties discussed in the report, the settlement is reported to be apparently at least partly attributable to groundwater movement or lowering. The location of these four properties in Hoveringham village can be seen on the plan map in Figure 4.1. The properties were:

Flora Farm

Movement in the property, rear garden, and yard was reported. The occupants stated that the appearance and worsening of the cracks in the property and ground disturbance coincided with the commencement and ongoing of the groundwater abstraction from the nearby quarry.

Four Winds

Page states that cracking in the property had occurred since 1986 when a previous survey was carried out. He concludes that because the structure is founded in the gravel the most likely cause of settlement is groundwater abstraction associated with the nearby gravel quarry.

The Willows (or "Tankards")

This is one of the nearest properties to the gravel workings in Hoveringham. Differential settlement at the rear of the structure and subsidence of the rear garden were thought to be wholly or partly attributable to groundwater movement.

The Church

No single obvious cause for the settlement observed at the church was apparent. Groundwater movement was thought to be one possible cause.

4.3 The Structural Inspections carried out as part of this research

There are approximately 120 separate residences in the village of Hoveringham. There is a large range of structure type, age and standard of construction. There are semi-detached properties, a stately home (Hoveringham Hall), bungalows and old 2 and 3 storey properties.

The letter in Figure 4.2 was sent to properties in Hoveringham requesting permission to do a structural inspection. Letters were sent to 93 properties in Hoveringham village. 44 replies were received from owners willing to allow an inspection. This reply rate was obviously indicative of a large degree of interest within the village in the subject being studied.

The objective of the structural inspections was to identify a structure's age, standard of construction, the history of any observed defects and to examine the structure for any evidence of recent or historic ground movement. Particular attention was given to any defects which had apparently arisen due to ground settlement.

4.4 The structural inspection document

The structural inspections were carried out using a "Structural Inspection Document" report sheet. This pro-forma was devised based on the sources of information quoted in Chapter Two. The document was split into various sections. A brief discussion of the contents and purpose of each section follows:

4.4.1 Part One: Preliminary Questions

The questions in Part One of the document were asked of the owners on arrival at the property. They were:

Qu 1: How long have the residents lived there?

Qu 2: What is the age of the structure?

Qu 3: How deep are the foundations?

Qu 4: Have you observed any cracks in the building and if so have you observed any "recent" (last five years) widening of them?

Qu 5: Have any extensions been made to the property?

Qu 6: Are there any large trees near the property and if so what type?

Qu 7: Have any trees been felled on the property recently?

Qu 8: Is the structure cavity walled? (If the residents were not sure the wall thickness was measured)

Qu 9: Are there or have there been any broken drains or downpipes on the property?

Part One was designed to get as much of the history and details of the structure from the owners as they knew. Owners may not possess technical knowledge but after all, they were the people who spent most time in the structure and could possibly provide useful information on past events. Answers to question four had to be treated with caution because the times when a crack first appears and the first time that the owners notice it can be two completely different times. This was especially important to bear in mind during the surveys given the Hoveringham residents now "growing awareness" of the possibility of fairly recent ground settlements having occurred within their village. Questions five to nine were designed to bring to light any of the more common mechanisms of ground settlement before the visual inspection was carried out, so that any cracks observed could be examined in the light of this knowledge.

4.4.2 Part Two: Survey Particulars

The sub-sections in this part were:

- Structure type and use.
- Construction details (Frame Type? Wall Type? Cavity Walled? Rendered? Roof Covering?
- Weather Conditions.

This was a general information section. The "structure type and use" information would be useful in seeing if similar structures were suffering similar defects.

4.4.3 Part Three: Preliminary Inspection Notes

Information recorded in this section included the type of ground, the topography, and details of any nearby watercourses. Owners were also asked whether their land was quick or slow to drain after rainfall. The answer to this question gave a strong clue as to whether the underlying ground was granular or cohesive in nature.

4.4.4 Part Four: Detailed External Examination

This was the most important section of the inspection document and the section on which most time was spent at each structure. Factors taken into account during the examination were:

- Trees (type and height were recorded) and their proximity to the structure
- Drainage systems, grates, inspection chambers.
- Disturbance to ground.
- Structural disturbance to neighbouring properties.

The section was to include sketch plans and elevations as necessary to show clearly any cracking, leaning, bulging, bowing or settlement and the movement quantified wherever possible. Details of any existing staining were recorded if any was observed.

4.4.5 Part Five: Detailed Internal Examination

This part was split into three sub-sections. The sections and information recorded in each were:

1. Roof Void or Loft

    - joist sizes and spans
    - purlin and rafter sizes and spans
    - condition of structural bearings

- general condition (dampness, water ingress, missing covering etc..)

2. Floor Areas Storey by Storey

- Floor condition (slope, springiness)
- Wall condition (dampness, crack details, loss of plumb, bulge etc..)
- Ceiling condition (dampness, sagging etc..)
- Serviceability of doors, windows etc..

3. Cellar / Sub Floor Area

- Extent of cellar - partial?
- Dampness, fungal attack, decay etc..
- Type and presence of damp proof course.

The internal examination of the property was carried out in the light of what had been observed during the external examination. Internal continuity of any external cracks was recorded. Defects such as plaster cracks, rising damp, or sagging and warping of floors obviously due to rot were not of direct relevance to the research and so were not always recorded. It was only possible or convenient to the owner to inspect the roof space in a minority of cases and not every property had a cellar. Where "partial cellars" existed (i.e, a cellar extended under only part of the structure) this was noted as this type of cellar can lead to differential settlement.

4.5 Findings from the inspections

The findings from the structural inspections are set out in Table 4.1 and summarised on the colour coded plan map of Figure 4.1. The property name is given in the table after which follows a brief description of any relevant information or defects recorded. Where terms such as "slight" are used to qualify the observed defects, these are as defined in Table 4.3 (after the Building Research Establishment, 1981). The most likely causes of any cracks in the structures are given. The road in Hoveringham village on which the property is situated is given in the table. Road and property names are marked on the plan map. Where the terms "left" and "right" or "front" and "rear" are used in Table 4.1 to describe relative positions of elevations of the structure, these refer to the sides of the building observed while standing in the road facing the structure. The more concise information in Table 4.2 and the map summarise the findings by breaking down the structures into two distinct categories:

1. Those where apparent settlement exists for which a ready explanation did not exist or where the history of the settlement was unknown. One possible explanation for the settlement recorded at these properties was the de-watering of the sand and gravel quarry.

2. Those where no settlement was observed or was observed but was attributable to factors other than de-watering.

Where a recent structural survey was made available for a particular property, the conclusions of the report were usually adopted and have been given in Table 4.1. Any other conclusions concerning the inspection of a particular property are the author's.

It will be noticed that the cricket ground is one area marked on Figure 4.1 as potentially having suffered settlement from de-watering. This was because "furrows" had reportedly begun to appear and deepen on the ground since 1990. As well as possibly being related to a historical farming method, de-watering was not discounted as a possible cause or contributor to this reported movement.

It should be noted that the modern building regulations for the Hoveringham area are set out by Newark and Sherwood District Council. These require that all new structures be founded on sand and gravel strata below any looser, less competent, surface soils. It is believed that concrete strip foundations adhering to these guidelines were used for all low rise residential structures built in Hoveringham after 1970. The photographs in Figures 4.3 to 4.11 show some of the types of defects observed during the inspections. The pictures show some defects which could be at least partly attributable to de-watering and others which were almost certainly explainable by other mechanisms. Figures 4.3 and 4.4 show Trentham House, a structure believed to be founded in the gravel. An apparent settlement crack, widening at the top, was observed on the eastern elevation of the property. Figures 4.5 and 4.6 show a section of Hoveringham Church to have suffered settlement. The left hand wall in Figure 4.5 was leaning substantially. Evidence for progressive settlement of this corner existed as cracks previously twice repointed had reopened. Figure 4.6 shows a closer view. The cracking had travelled around the window and progressed higher up the wall.

Figure 4.7 shows an extension at Hillsboro House on Boat Lane. Often when extensions are built onto properties, they move relative to their parent structure due to having been constructed on less consolidated ground. In this extension at Hillsboro House on Boat Lane, there had been movement relative to the parent structure which according to the owners had apparently begun "around the time of commencement of the de-watering in 1985". However, the extension was built in 1979 therefore predating the de-watering by at least six years. Therefore, given the two pieces of conflicting evidence in this case, the defect was classified as "potentially at least partly due to de-watering". The type of crack depicted in Figure 4.8 is often observed above lintels with "brick on end" details as shown. Usually this type of cracking is structurally unimportant being due to nothing more than lintel sag accompanied by a drop of the bricks on end. However, this particular crack, observed at Hillsboro House was attributed only to eroded mortar as the lintel and bricks on end appeared quite sound.

At Three Pines on Boat Lane, shown in Figures 4.9 and 4.10, a crack widening upwards was observed beneath one of the long span central windows and the one storey porch seen on the right in Figure 4.9 had settled by up to 50mm relative to the parent structure. A previous structural survey which involved demec monitoring of the crack reported little or no progressive movement and described the deformation of the porch as "due to poor construction... and a poorly founded shallow concrete slab." Figure 4.11 shows how simple explanations can exist for apparently severe shear distortions and associated cracking of a wall. This outbuilding at Church Farm House, Gonalston Lane had certainly suffered structurally under the action of a large force but a quick discussion with the property owner soon identified the source of this force - cattle pushing against the wall!

4.6 Structural Monitoring

Precise level and demec gauge monitoring of five properties in Hoveringham village was carried out for periods all less than one year between November 1992 and December 1993. Four of the five properties were observed to have suffered settlement in the past. They were:

- Hoveringham Hall
- Hoveringham Church
- Flora Farm
- Little Orchard

The other structure was "Robsal" situated off the Main Street. This was a very new two storey residence built to modern building regulation standards and was therefore founded in the gravel. The objectives behind the monitoring were to be :

1. To investigate structures known to have suffered movement for signs of progressive settlement.

2. To compare the behaviour of Robsal with the other four properties. If all five settled in similar fashion it would be strong evidence for a single common mechanism - possibly de-watering.

However, the pumps in the de-watered quarry were switched off shortly after commencement of monitoring, which meant the water table would now return to its original level if it had been affected below the village. Any fines movement due to lateral water flow would also cease as a settlement inducing mechanism. Would a recovery of the ground level be observed as water levels rose? This was unlikely due to recovery in gravels being generally low, coupled with the small drawdown likely to have existed at the distance of the structures from the quarry. It was also recognised that a substantial area of the worked quarry in between the village and the de-watered zone had been recently p.f.a. filled. This was likely to have reduced the drawdown effect below the village, given the low permeability of p.f.a. This apparent "shielding" effect of the p.f.a. against the effects of drawdown was described in Chapter Three.

It was perhaps unsurprising that the monitoring of the structures detected little more than seasonal movements of the structures. It would appear that monitoring had begun "after the event". All structural monitoring was discontinued approximately one year after its commencement.

4.7 Conclusions

The structural inspections showed that a wide variety of structures and defects existed in Hoveringham village. Many structures were shown to have suffered either historic or recent settlement. In some cases, obvious explanations for the settlement were apparent. Where there was no obvious explanation, the settlement was classified as "potentially at least partly attributable to de-watering". On this basis the structures were classified and Figure 4.1 prepared. A visual inspection of this figure appears to indicate that more defects potentially attributable to de-watering exist as the de-watered zone is approached. This conclusion was backed up by calculating the average distance away from the de-watered zone for the two classification groups as given in Table 4.2. Properties with settlement "potentially due to de-watering" and those with none or easily explainable settlement had average distances from the de-watered zone of 480m and 580m respectively.

Preliminary indications had therefore been given that structurally significant settlements due to de-watering of a nearby quarry had occurred. The results were however, far from conclusive being based sometimes on "owners knowledge" or a diagnosis for a defect based on an assumed "most likely" cause. The other areas of research discussed in this thesis were used in conjunction with the findings in this chapter to give firmer indications whether de-watering could be used to explain the structural movements.

Ref No	Property Name	Property Location	Defects recorded / Notes
1	The White House	Main Street	No significant settlement apparent. Resident reported noticing two recent cracks but these were not caused by settlement
2	3 Bradley's Lane	Bradley's Lane	Built 1990. Concrete strip foundations onto the gravel. Plaster shrinkage cracks observed inside structure. Cracking observed above arches of front windows and patio doors - sagging of arch most likely explanation. Two cracked areas of breeze blocks in the garage were noted: • Below window - window frame thermal expansion most likely cause. • Apparently very old crack high in garage wall - most likely due to movement early in the structure's life.
3	4 Bradley's Lane	Bradley's Lane	Built 1990. Concrete strip foundations onto the gravel. No settlement cracks observed. No lean of walls recorded.
4	2 Bradley's Lane	Bradley's Lane	Built 1990. Concrete strip foundations onto the gravel. No settlement cracks observed. No lean of walls recorded.
5	Trentham House	Boat Lane	Main structure built 1850. Front elevation was extensively repointed in 1991 - cause of cracks requiring repointing unknown. This repointing was in good state of repair in Sept 1993 according to a survey carried out then. Settlement had occurred since as a repointed area had reopened on the right elevation. Structure has cellar under the oldest parts but not under the newer parts.
6	Rose Bank	Boat Lane	Built 1890's. Structure founded at shallow depth straight onto sand and gravel below. No significant cracking or leaning of walls recorded. Cracking occurred above rear left window due to kitchen window widening - was repointed and has not re-opened.
7	Hillsboro House	Boat Lane	Original structure built 1720's - various extensions built on since. One storey kitchen extension built 14 years ago - this extension has moved away from house leaving large crack at junction.
8	The Old Forge House	Boat Lane	Original structure built 1864 - two extensions built on since then. Partial cellar exists extending under only original section of the house. Poor details include straight junctions between original sections and extensions. Blown facings and eroded mortar exists (aesthetic problems only) <u>Information from a previous structural survey:</u> Main front: "entirely satisfactory" ; "Hairline settlement cracks exist under the right hand ground floor and over left hand ground floor windows" ; "the old two storey section is noticeably down towards the rear." Observed settlements appear to be explainable by mechanisms other than de-watering.

Table 4.1
Summary of findings and conclusions drawn from the structural inspections
(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
9	Four Winds	Thurgarton Lane	Built 1952. Evidence of historical differential settlement exists. Structural survey by Leonard Murray and Associates, 1987 stated "cracks had re-opened no more than would be expected due to seasonal thermal and moisture content variations" - a later structural survey (Page 1992) showed that settlement had occurred since 1986 -see section 4.2 above. One possible cause for this settlement was groundwater disturbance.
10	3 Pines	Boat Lane	Built 1960 - two storey house. Notes from structural survey by Howard Ward Associates: • Brickwork shows little sign of structural movement. • Crack under rear lounge window is undoubtedly long standing in nature and there exists no positive evidence of current or recent movement. • The cracking in the single storey rear porch is clearly due to poor construction, the brickwork having been built off a poorly founded shallow concrete slab. • Demec gauge monitoring has shown little or no progressive movement in the property
11	Walnut Cottage	Gonalston Lane	Original section most likely 17th century. Numerous extensions built on over the years. There are many trees at various distances from the property. Whole of the outside of the house is rendered or finished - this may have "disguised " some movement of the house. Poor details and points of interest include: • A soakaway drain used to discharge adjacent to the to the west wall. • Roof spread has probably been induced by replacement of roof tiles with heavier ones. A large fresh crack exists inside in the bathroom at ground level - most likely a settlement crack - possible explanations include the existence of a "sumac" tree very close to the property, the existence of an oil tank (2mx1mx1m) just outside this wall, a defective drainage system the siting and route of which is indicated by two manholes.
12	West Cottage	Gonalston Lane	Built 1790. Foundation most likely very shallow and built directly, onto the underlying sand and gravel. Whole property appears generally sound. The 15 year old repointing is in generally good condition. The only significant defect observed in the property is cracking around the new, south facing window frame. Crack pattern is not suggestive of settlement - most likely cause is disturbance to the brickwork during frame installation or frame thermal expansion .
13	Landsdyke	Gonalston Lane	Built 1969 - bungalow. Foundation is a concrete raft approx 1m deep founded directly onto the sand and gravel below. Owners report no recent cracking or movement - they decorate 'regularly' so this testimony probably more reliable than the average house owner's. All external brickwork apparently sound.

Table 4.1 (cont...)

Summary of findings and conclusions drawn from the structural inspections

(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
14	Church Farm House	Gonalston Lane	Original structure approx 300 years old (local builder's information to the owners) A significant section is Victorian. Most of the observed defects are likely to be due to age, poor details or poor standards of workmanship. Examples include old weathered brick facings and mortar and apparent sulphate attack of the brickwork due to defective guttering. An apparently severe defect was the 30mm lateral displacement of a large section of the brickwork in an out building - this is in fact due to cattle pushing against the wall!
15	The Gables	Gonalston Lane	Built 1980. Should therefore have been built as the modern building regulations for the area stipulate. Whole structure sound - no evidence of settlement.
16	Riverlyn House	Gonalston Lane	Original house built 17th century. Other sections have been added to the property at different times in the past. One corner was underpinned in the past due to settlement having taken place - the cause of this settlement is not readily apparent. The settlement occurred in a younger section. A drainage system with a history of blockages exists but this is not near the subsided area. No trees nearby. Repointed areas exist above the kitchen. Most likely cause of these cracks is the removal of a lower structural wall to facilitate the extension construction. Substantial roof and floor joist sag exists in some of the older parts of the property - this manifests itself as bowing floors. Defects suggestive of an apparently large amount of movement exist inside the house. However, it seems the most likely cause is that a once exterior wall, now interior, suffered from stress redistribution when the extensions were built on.
17	Rose Cottage	Main Street	Built in the 17th century. The structure was originally two separate cottages, since integrated. Extensions have been made to the property in the last ten years. Various areas of minor cracks exist which are most likely due to plaster shrinkage or thermal expansion of doors and window frames. No evidence of ground or differential settlement was found.
18	Ivy House	Main Street	Oldest part of property was built in 1722. It appears an extra section was added in the early 1900's. The owners believe there have been numerous modifications to the structure's interior throughout its life. Minor defects include blown brick facings, eroded mortar, roof line rippling (12 years ago, new roof beams were installed presumably as a remedial measure to this), and a poor damp proof course. No evidence of settlement was found.

Table 4.1 (cont...)
Summary of findings and conclusions drawn from the structural inspections
(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
19	3 Gonalston Lane	Gonalston Lane	Built 1937. Semi-detached property. This is one of 8 properties built in the area which is designated "alluvium" (soft red-brown clay) on the geological maps of the area NOT on the Holme Pierrepoint sand and gravel terrace areas. Verbal information from the occupants indicated that the foundations for the main part of the property are on the clay but that the foundations of a 1991 extension are on the sand and gravel. Occupants report no apparent worsening of existing cracking since they moved in 1990.
20	1 Gonalston Lane	Gonalston Lane	Settlement crack observed at the front, south west corner of the house is progressive - was repointed and has since reopened. On the rear north east elevation an apparently large force has acted as cracks exist which pass through two courses of blue bricks. Foundations are very likely to be on clay.
21	West Lodge	Gonalston Lane	Original structure was built in 1890. The original structure was 3 storey. A 1 storey lounge and 2 storey kitchen extension were added in 1969. Movement appears to be associated with the 1 storey lounge extension. Sagging of a section of brickwork exists above what was very likely to have been a section of exterior structural wall which was knocked out to extend the lounge. There is evidence (crack pattern and wall out of plumb) that the lounge extension has settled relative to the rest of the property. The crack at the base of the extension (widening upwards) travels through two blue bricks, suggesting a strong force has acted.
22	Lark Rise	Gonalston Lane	Built 1971 - bungalow. Concrete strip or raft foundations bearing directly onto the sand and gravel below. In 1991, the brickwork mortar for the whole of the property was checked and re-pointed as necessary - no settlement cracks exist in the mortar or brickwork suggesting structure has been stable for at least two years.
23	Rose Cottage	Thurgarton Lane	Main body of the property is about 250 years old. A 1 and 2 storey extension were both added in 1971. A small entrance hall / shower was added in 1991. Slight damage (3mm - 6mm cracks) exists in the property in locations suggestive of differential settlement. In the past money was awarded by an insurance company to repair the damage due to "land movement". On both the south-east and south-west sides of the property, repointing was carried out prior to 1988. Cracks in the brickwork have appeared since then suggesting that progressive movement is taking place. Possible historic ground movement due to groundwater abstraction given the proximity of this structure to the gravel workings.

Table 4.1 (cont...)
Summary of findings and conclusions drawn from the structural inspections
(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
24	The Anchorage	Gonalston Lane	Built in 1968 - bungalow. Foundations likely to bear directly onto sand and gravel subsoil. Four areas of "slight" damage exist: • right elevation (east facing) - crack pattern suggests window frame thermal expansion to blame. • Rear elevation (north facing) - crack pattern suggests window frame thermal expansion to blame. Top right of patio doors on rear elevation - cracks exist within an extension built in 1991 - likely cause is movement of extension relative to rest of the property - precise cause unknown.
25	Bay Cottage	Gonalston Lane	Original part of structure built 1870. In 1981, a two storey extension was built onto the north and east faces of the property. A large garage outbuilding (named "the piggery" by the owners) also exists next to the main structure. In the older main body of the property no settlement was evident. The connection between the main structure and the extension appeared sound. Three areas of cracking were observed in the "piggery": • In upper left of south elevation running underneath the eaves. The most likely cause is roof movement. • Two other areas of cracking exist at an apparent joint between two sections of the building. One of these cracks was filled 5 years ago and has reopened suggesting progressive movement. The movement appears to be settlement of the smaller rear section. If these two distinct sections of the piggery were built at different times, this may have caused some initial relative movement but cannot explain the apparent movement in the last 5 years.
26	The Spinney	Gonalston Lane	Built 1966 - bungalow No evidence of ground settlement found. All brickwork and mortar apparently sound.
27	Hunter's Moon	Gonalston Lane	Built 1966 - bungalow. An R.I.C.S. report done in 1987 stated that, "there are no signs of movement to the main structure save extremely minor mortar cracks." On the east facing side of the property, cracks exist which can be described as "very slight" to "slight" damage after reference to BRE Digest 251.(see Table 4.3) One area of repointing underneath the large window of the right elevation has reopened. The reopening is minor and is most likely due to thermal movements.

Table 4.1 (cont...)

Summary of findings and conclusions drawn from the structural inspections

(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
28	Lodgefield House	Lodgefield Lane	Original section of the property built in the early 1800's. Various extensions have been added in the past but nothing has been added for the last 23 years. A structural report done on the property by Elliott and Brown Consulting Civil and Structural Engineers stated that the foundations were,"a mix of shallow and very shallow foundations with differing spread dimensions." A brief damage summary was given as follows: • Extensive cracking / slopes exist to external walls, windows and door openings. • Internally, plaster cracking, sagging ceilings and lintels were found. • Levels taken by Elliott and Brown suggested that 50mm of differential settlement had taken place in the past. They suggested that the problems may be active now. • Leaking drains have affected the founding soils in the past. • Varying depths of foundations coupled with the poor detail of straight joints between older and newer areas have no doubt contributed to the observed defects. • The report states,"The distress observed is generally compatible with the structure's age and most of it can be related to long term localised settlements and minor structural inadequacies common to building practice at the time of construction."
29	Meadow-field	Main Street	Built 1896. Foundations are very shallow commensurate with the structure's age. The residents have lived in the property since 1990. They reported that the outside hand pumped well worked when they arrived but now does not. They reported cracks which had been repointed and had reopened since. A structural engineer's report suggested that faulty gutters discharging excess water onto the ground near to the structure was one likely cause of movement. The engineer reported faulty drains after performing drain pressure tests. The possibility exists that water table disturbance caused the well to dry up and disturbed the stability of the drains.
30	Vine Cottage	Main Street	Built late 1700's. Partial cellar exists under oldest parts of property. A breeze block extension exists which does not have a cellar. This extension is 4m x 5m in plan and is two storey. The main defects to note on the property were: • Blown facings and missing mortar on the oldest parts of the property. • Dropped bricks in "bricks on end" detail above windows accompanied by associated cracks of the brickwork sections above. • Brickwork decayed, in poor state of repair and also showing signs of sulphate attack at low levels on the front elevation. • There exists evidence of settlement having occurred at the right hand corner of the front elevation.

Table 4.1 (cont...)

Summary of findings and conclusions drawn from the structural inspections

(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
31	The End	Main Street	Built 1896. Viewed from the front there exists a very substantial lean (of the order of a few degrees) of the right hand wall of the property. A carport was turned into the living room on this side of the property and the foundations were made deeper to a depth of approximately 2 metres according to the occupant. The "substantial lean" mentioned above was apparently present before the amendments were made to the property. The occupant reports he believes the "lean" to have been the same for the last 7 years. Sagging sections of the roof line are evident. This is most likely historical and unlikely to be progressive as roof cross ties have been installed at third points across the length of the roof. No evidence of recent ground movement exists on the property even though historic movement is evident.
32	The Forge	Main Street	Main structure built in 1870's. One outbuilding exists - an older one storey, masonry structure. An area of cracking exists in the main structure suggestive of settlement. The resident suggests this crack has widened in the last 3 years. The outbuilding has three cracks apparently caused by settlement in three parallel walls. Faulty guttering on the apparent "downside" of the settlement could be to blame. Heavy roof tiles exist on this outbuilding which do not appear to be the originals. It is possible that some roof spread occurred because of this, causing or worsening the observed crack pattern.
33	2 Lansic Cottages	Post Office Yard	Built 1840. The property is a semi-detached two storey cottage. Foundations are likely to be extremely shallow. Only serious defect is the large deflection of an upstairs ceiling caused by a roof timber dropping from the point at which it was nailed to a joist (this was mentioned in 1984 in a house buyer's report)
34	1 Linden Cottages	Post Office Yard	Built approx 200 years ago. Foundations are likely to be very shallow. No evidence of recent ground movement exists at the property. Some minor defects exist which can be related to the age of the structure.
35	The Old School	Main Street	Original section of the property was built in 1875 - a major two storey extension has been added in the last 6 years. No evidence of recent ground settlement exists at this property.
36	Brook House	Brookfield Drive	Built 1908. Foundations likely to be very shallow. Property is a detached, two storey house. On the right elevation, a crack exists which travels the full height of the wall, widening at the top, suggesting settlement. The resident suggests there has been a widening of the crack in the last year. Other cracks on the property which were repointed in the last few years have also reopened.

Table 4.1 (cont...)

Summary of findings and conclusions drawn from the structural inspections

(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
37	Inglewood	Brookfield Drive	Built 1910. The property is a two storey, semi-detached house of brick construction. The foundations are likely to be very shallow. The residents complained that the "whole house seemed to be expanding". Widening door frames were evident, gaps now existing between doors and frames. This is probably what gave the resident the impression of "expansion". Evidence of driveway settlement found although all exterior walls were shown to be vertical on the spirit level
38	1 Manor Cottages	Main Street	Built 1900. The property is a two-storey, semi-detached, masonry structure. No evidence for ground settlement exists on the property.
39	The Willows (Tankards)	Brookfield Drive	Built 1910. The property is a two-storey, detached, masonry structure. Three main areas of structural defects exist on the property: • Cracking below the rear kitchen extension window which has apparently appeared in the last year even though the extension was itself built 16 years ago. • Cracking running from the top of the bay window adjacent to the extension to the bottom of the window above. • The grate frame in the driveway has dropped approximately 20mm Groundwater disturbance is one possible cause of the first two defects described above.
40	Hov'ham Church	Gonalston Lane	Settlement observed at the south-west corner of the structure which is apparently progressive. Possible explanations include: • Collapsed grave(s) at that corner - bone fragments retrieved from a trial hole at this corner were identified by staff at a local hospital as coming from a human finger and rib. • Faulty drainage and guttering system. • De-watering effects not to be discounted as a direct source of settlement or as a cause of disruption to the drainage system even though the distance from the quarry is approximately 700 metres.
41	Hov'ham Hall	Main Street	Many types of defects existing due to age, extensions or settlement. De-watering not to be discounted as one possible cause of some settlement
42	Little Orchard	Main Street	The north east corner of the property had suffered differential settlement and a very large crack existed below a first floor window which correlated with this. A drain system ran close to this wall and its integrity was unknown. No large trees existed nearby. De-watering as a direct or indirect cause of settlement will not be discounted.
43	Flora Farm	Main Street	The occupant reported an apparent worsening of differential settlement cracks on the eastern elevation since 1991. Settlement and deforming of the driveway and adjacent grassed area was apparent which again had reportedly worsened since 1991.

Table 4.1 (cont...)
Summary of findings and conclusions drawn from the structural inspections
(continued over)

Ref No	Property Name	Property Location	Defects recorded / Notes
44	Riversdale	Brookfield Drive	Built 1900. The property is a two-storey, detached, masonry structure. Foundation depth is probably very shallow. Significant cracking of the left elevation of the property accompanied by 0.5 degrees of lean were suggestive of movement having taken place at the north-west corner of the property. The concrete patio at this corner also showed signs of settlement. The two manhole covers at this corner having remained at the same level were protruding from the patio surface. There is evidence for this apparent movement being progressive in nature, the residents having complained of a worsening of the cracks from 1991 - 1993 and a crack inside the property having reopened after decoration. Groundwater disturbance is one possible cause of some of the observed defects.

Table 4.1 (cont...)
Summary of findings and conclusions drawn from the structural inspections

Properties with settlement - de-watering as one possible cause	Approx distance from quarry	Properties without settlement or with settlement due to known causes	Approx distance from quarry
5. Trentham House (Boat Lane)	850m	1. The White House	525m
7. Hillsboro' House (Boat Lane)	900m	2. 3 Bradley's Lane	575m
9. 21. Four Winds (see App 1)	150m	3. 4 Bradley's Lane	575m
16. Riverlyn House (Gonalston Lane)	575m	4. 2 Bradley's Lane	575m
19. 3 Gonalston Lane	575m	6. Rose Bank (Boat Lane)	875m
20. 1 Gonalston Lane	575m	8. Old Forge House (Boat Lane)	875m
21. West Lodge (Gonalston Lane)	575m	10. Three Pines (Boat Lane)	825m
23. Rose Cottage (Thurgarton Lane)	550m	11. Walnut Cottage (Gonalston Lane)	675m
24. The Anchorage (Gonalston Lane)	550m	12. West Cottage (Gonalston Lane)	575m
25. Bay Cottage (Gonalston Lane)	575m	13. Landsdyke (Gonalston Lane)	600m
28. Lodgefield House (Lodgefield Lane)	800m	14. Church Farm House (Gonalston Lane)	600m
29. Meadowfield (Main Street)	425m	15. The Gables	580m
32. The Forge (Main Street)	360m	17. Rose Cottage (Main Street)	400m
36. Brook House (Brookfield Drive)	200m	18. Ivy House (Main Street)	575m
37. Inglewood (Brookfield Drive)	175m	22. Lark Rise (Gonalston Lane)	550m
39. The Willows(Brookfield Dv) "Tankards" on the map	160m	26. The Spinney	575m
40. Hoveringham Church (Gonalston Lane)	650m	27. Hunter's Moon (Gonalston Lane)	600m
41. Hoveringham Hall (Main Street)	650m	30. Vine Cottage (Main Street)	525m
42. Little Orchard (Main Street)	400m	31. The End (Main Street)	400m
43. Flora Farm (Main Street)	375m	33. 2 Lansic Cottages	425m
44. Riversdale (Brookfield Drive)	160m	34. 1 Linden Cottages	475m
		35. The Old School	400m
		38. 1 Manor Cottages	550m
	Ave =480m		**Ave=580m**

N.B: The reference numbers appearing before each property name are those given in Table 4.1

Table 4.2

Structures Classified Into Those Having Suffered Settlement For Which De-Watering Was One Possible Cause And Those For Which Any Observed Defects Were Explainable By Other Mechanisms.

Category of damage	Description of typical damage	Approximate crack width (mm)
0	Hairline cracks of less than about 0.1mm	Up to 0.1
1	Fine cracks which can be easily treated during decoration. Perhaps isolated slight fracturing in the building. Cracks rarely visible in external brickwork.	Up to 1
2	Cracks easily filled- redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally but some repointing may be required to ensure weathertightness. Doors and windows may stick slightly.	Up to 5
3	The cracks require some opening up and can be patched by a mason.Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking, service pipes may fracture, weather tightness often impaired.	5 to 15 (or a number of cracks up to 3)
4	Extensive repair work required involving breaking out and replacing sections of walls, especially over doors and windows. Window and door frames distorted and floors sloping noticeably. Walls leaning or bulging noticeably - some loss of bearing in beams. Service pipes disrupted.	15 to 25 but also depends on number of cracks
5	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing and walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	Usually greater than 25 but depends on number of cracks

Table 4.3

Classification Of Visible Damage To Walls.

(after The Building Research Establishment, 1981)

The Nottingham Trent University
Faculty of Environmental Studies
Department of Civil and Structural Engineering
Burton Street
Nottingham
NG1 4BU

Dear Occupant,

I am a member of the academic staff at The Nottingham Trent University, Dept of Civil and Structural Engineering currently undertaking studies of the ground and structures within the Hoveringham area. My studies form part of a research project to investigate the effects on the ground in the Hoveringham area of the nearby Tarmac gravel extraction pits.

I am writing to you because your property comes within the area of the village in which I am particularly interested, and to ask if you would allow me to undertake a structural survey of your property. This "structural survey" would entail us making detailed examinations of both the inside and outside of your property. We would record the position and severity of any cracks as well as any walls "out of plumb". This examination would take approximately one hour.

There would of course be an advantage to you from our structural survey, should you allow us to undertake one. You would find out free of charge from a qualified civil engineer, whether your house was damaged or not and the likely future stability of your property. This information may be very useful to show any prospective future buyers of your property, should you ever wish to sell.

I look forward to hearing from you shortly as to whether or not you would be agreeable in this matter. I enclose a postage paid envelope for your reply. Alternatively, feel free to telephone me any time at The University on Nottingham 418418.

Yours faithfully,

Mr Ian Froggatt B.Eng(Hons)

Figure 4.2

The Letter Sent To Hoveringham Residents Requesting Permission To Do A Structural Inspection.

Figure 4.3 - Trentham House

Figure 4.4

Trentham House - Apparent Settlement Crack Widening At The Top

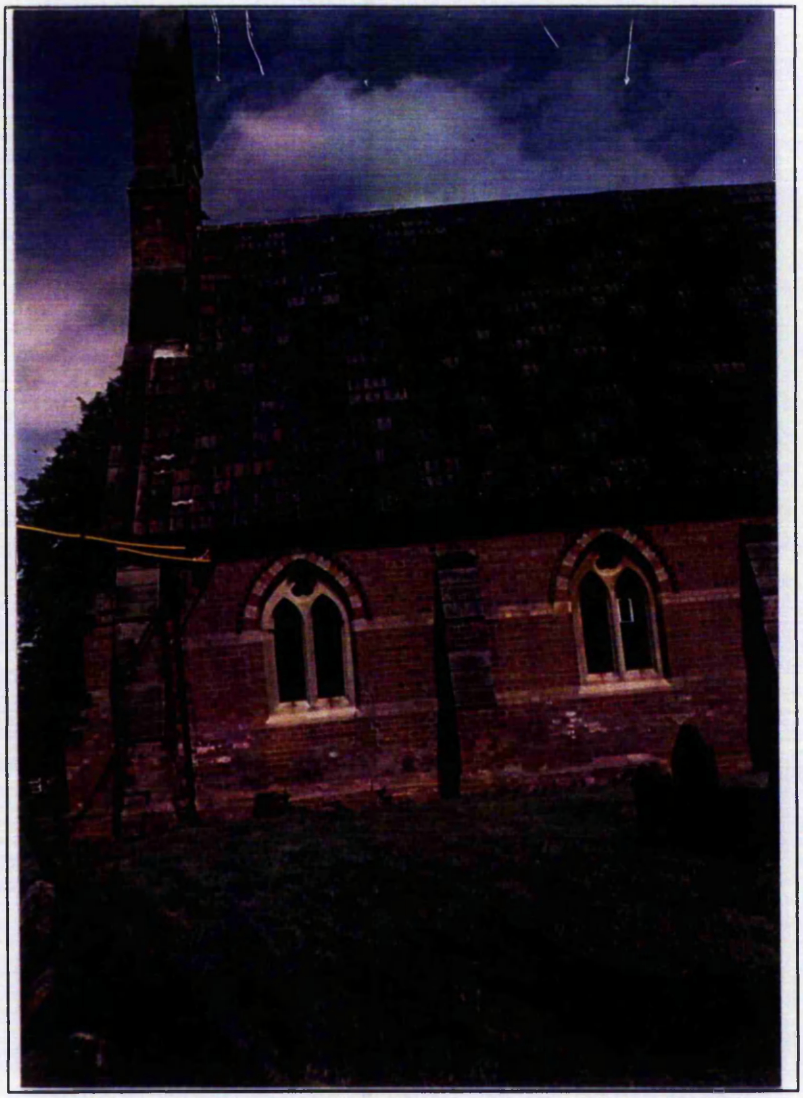


Figure 4.5

Hoveringham Church - Rear Elevation

Figure 4.6 - Hoveringham Church. Close up of corner suffering settlement.

Figure 4.7 - An Extension At Hillsboro House, Boat Lane

Figure 4.8

Eroded Mortar Above A Lintel At Hillsoro House.

Figure 4.9 - Three Pines, Boat Lane - Rear Elevation.

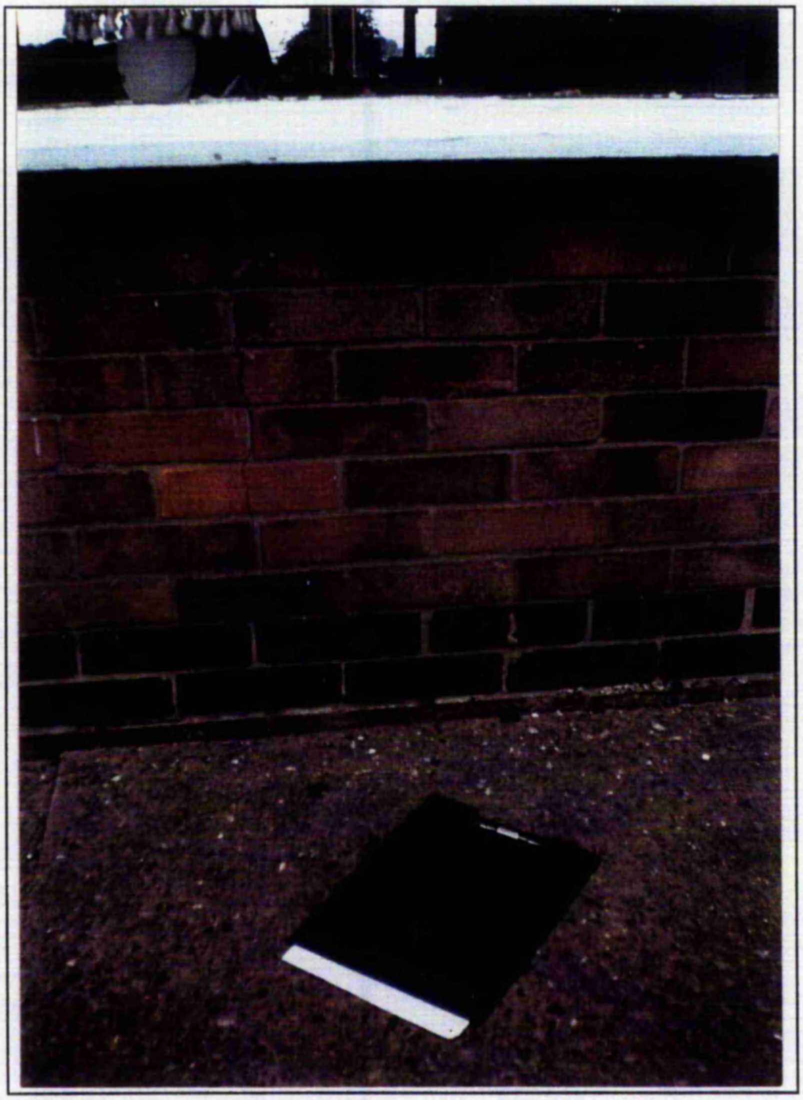


Figure 4.10

Three Pines - Cracking Beneath Long Span Rear Window

Figure 4.11

Church Farm House, Gonalston Lane

Cracking Due To Cattle Pushing Against A Wall

CHAPTER FIVE

INVESTIGATIONS OF LATERAL FLOW OF WATER AS A POTENTIAL CAUSE OF PARTICLE DISTURBANCE AND GROUND SETTLEMENT

5.1 Introduction

Kenney and Lau (1984) presented a method for determining the "internal stability" of a granular material by analysing the particle size distribution of the material. Their method arose from the need to determine whether a granular material would lose particles under conditions of seepage if it were used as a filter in a dam. If particle loss occurs the material becomes a less effective filter. The method (described below) was developed empirically by analysing the results of many tests. Kenney and Lau used "severe" transport conditions during their tests. "Severe" conditions meant a Reynold's number of flow within the granular material of 10 or greater occasionally accompanied by light vibration.

Calculation 5.1 on the following two pages shows that the seepage conditions at the face of the quarry corresponded to a Reynold's number of approximately 1. This is well below the Reynold's numbers employed by Kenney and Lau during their tests. This is also well below the generally accepted laminar / transitional flow boundary number of 10 at which particles are assumed to become susceptible to disturbance by flowing water. Therefore if the material sampled in the Hoveringham area of the Trent floodplain were shown to be "stable" under Kenney and Lau's criteria then it would be very unlikely for any fine particles to be moved within the aquifer if the highest Reynold's number of flow were likely to be no greater than one. It should however be borne in mind that the method employed for Calculation 1 assumes a completely homogeneous media. It is possible that stratification within the aquifer could have caused a local increase in Reynold's Number, thus increasing the possibility that fine particles will be moved.

CALCULATION 5.1

Estimation of Reynold's Number of flow which existed at the quarry face during de-watering.

This calculation was carried out using the particle size distribution curve of Sample 1, Location 5 (see later for exact details of sample location).

The method uses a tabular format for the calculation of Effective Particle Diameter (D) which is summarised in Appendix 5.1.

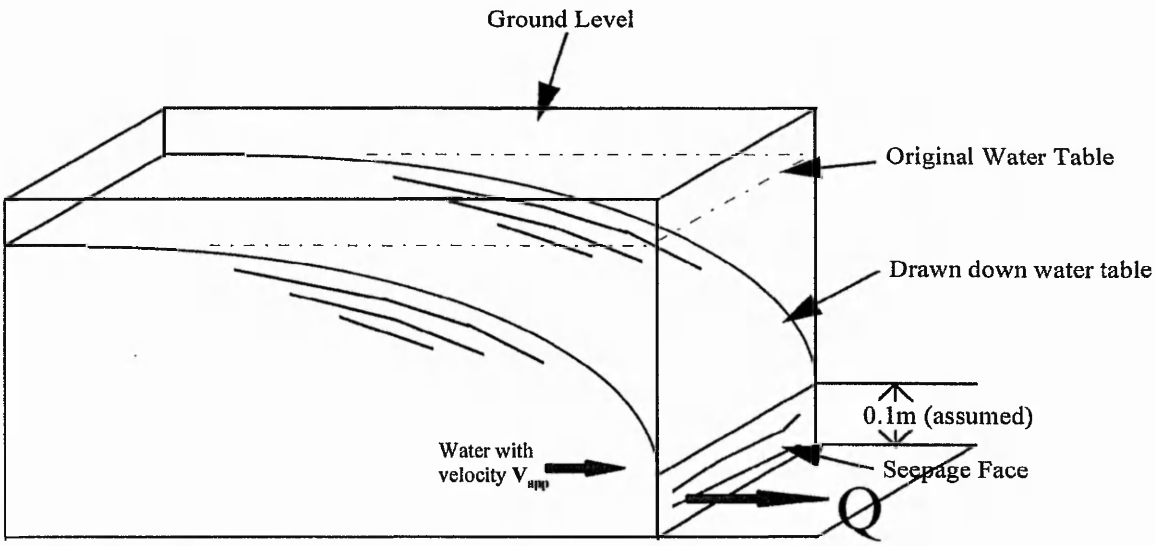
The Effective Particle Diameter, D, was calculated as 0.9

The equation for the calculation is:

$$\Re = \frac{V_{app}\, D\, \rho_w}{\mu}$$

where:	$\Re$	=	*Reynold's Number*
	V_{app}	=	*approach velocity of water calculated using simple continuity equation (see over)*
	D	=	*Effective Particle Diameter for the grading curve analysed*
	ρ_w	=	*Density of water (1000 kg/m^3)*
	μ	=	*Viscosity of water (1.3x10^{-3} m/s at 10^{o}C)*

Calculation of "V_{app}" - approach velocity of water at the quarry face:



Section through the de-watered aquifer at the quarry face

Using the continuity equation: $V_{app} = Q / A = 0.001 m/s$

where: A = *Area of seepage face* = $0.1m^2$ *(for a 1m strip)*

Q = *Flow rate through seepage face* = $0.1x10^{-3}m^3/s$

Referring to the diagram above, the values assumed for "A" and "Q" were thought to be conservative, thus giving rise to a conservative estimate of Reynold's Number. The flow rate through the seepage face was calculated assuming a total daily flow rate of 1 million gallons per day and a length of seepage face of 600 metres.

Conclusion from Calculation One:

From the above equation, the Reynold's No at the quarry face is estimated as approximately 0.7 which is much less than the generally assumed laminar/transitional flow boundary number of 10 at which particles are generally assumed to become susceptible to movement from the flowing water.

All of Kenney and Lau's tests were performed on "relatively dense specimens" using "severe" transport conditions. They state the three factors which affected grading stability were:

1. Particle Gradation
2. Relative Density
3. Transport Conditions

In the tests conducted by Kenney and Lau, factors 2 and 3 were purposely not varied therefore the factor most likely to have been responsible for differences in behaviour of the tested samples was particle gradation.

Kenney and Lau's work (1984 and 1985) has shown that in regard to stability of grading, as long as the materials are composed of particles coarser than silt size and the Reynold's number of flow within the material is less than 10 coupled with light vibration, the **absolute** sizes of the particles are of little importance in comparison to the shape of the grading curve. Soil samples taken by the operator of Hoveringham Quarry (Tarmac Quarry Products Ltd, 1993) were shown to contain some degree of silt sized particles and therefore the Kenney and Lau method cannot be considered to be relevant at the very lowest sizes of the grading curve. However the method was useful as a first indication of whether particles larger than silt were likely to suffer disturbance from the groundwater flowing to the quarry.

The method proposed by Kenney and Lau for analysing grading curves relies on determining the amount of particles between a given size "D" and a size "4D". It is assumed that a particle of size "D" can pass through the voids within a matrix of particles of size 4D or greater. Therefore, if a particle of size "D" is caused to move by the seepage of water, and there is a deficiency of material between size "D" and "4D", it is possible for the particle to move.

It is important to remember that for the case of de-watering in the Trent floodplain, it is hypothesised that removal of finer particles by seepage may be the cause of ground settlements larger than those predicted by effective stress increase alone. For this to have been the case, it would seem sensible to assume that a disturbed particle would have to have been supporting some amount of effective stress which would give it an added resistance to movement.

The chart used by Kenney and Lau (1985) to assess grading stability is shown in Figure 5.1 The shaded diagonal region is the boundary between stable and unstable gradings. In the chart, "H" (y axis) is "mass fraction between particle sizes of D and 4D" obtained from the particle size distribution curve. This is therefore a measure of how much material exists within the granular matrix which may "seal up" voids through which a disturbed particle may otherwise pass. "F" is the total mass fraction of the material smaller than the particle size "D", also obtained from the grading curve.

5.2 Discussion of analyses of samples.

5.2.1 Samples taken by Tarmac Quarry Products Ltd.

Prior to commencement of quarrying, Tarmac Quarry Products Ltd (1993) took disturbed bulk samples of material retrieved from augered holes which penetrated the full depth of the superficial deposits. Particle size distributions of the samples were presented as part of the planning application for the quarry. A typical full grading of the sand and gravel quarried in the Hoveringham area is given in Figure 5.6. The method presented by Kenney and Lau for the determination of internal stability of a soil was applied to the samples as part of this research project. The results from the analyses are summarised in Table 5.1. The sample reference letters in the table are the author's but the borehole references are those given by Tarmac Quarry Products Ltd. Of the 21 samples tested, 2 had shape curves which stayed completely within the stable zone of the Kenney and Lau chart and 17 entered the "unstable" zone. For particles of medium sand or larger, all the samples were found to be stable.

5.2.2 *Samples taken from the face of Pit R to examine the fines content*

5.2.2.1 *Objective*

The investigations detailed in Section 5.2.1 showed that the sand and gravel in the Hoveringham area could suffer fine particle loss under certain seepage conditions. What was now needed was a practical investigation of whether this had occurred. On 12th November 1993, samples of sand and gravel were taken from the face of the quarry. The locations from where the samples were taken are shown in Figure 5.2. The particle size distributions of the samples were examined to see if they exhibited any evidence of particles of fine sand size or less having been removed. The highest hydraulic gradients during the pumping operation and therefore the greatest potential for fine particle movement would have existed at the quarry walls. If these samples showed a fine particle content very similar to samples taken prior to gravel extraction (ie, prior to pumping and groundwater movement) then it would be a reasonable conclusion to draw that nowhere else in the aquifer would finer particles have been caused to move. It should be noted here that the Kenney and Lau method has not been applied to these samples for the very fact that the hypothesis being tested is that some proportion of the material within these samples has been removed.

5.2.2.2 *Method used to obtain the samples.*

The samples were taken at a time when the water level in the de-watered quarry was approximately 1.7 metres below the level of the natural water table. Two samples were taken from each of the five locations shown in Figure 5.2. The samples were taken by digging approximately 400mm into the quarry walls in order to ensure the material sampled had been unaffected by surface weathering. Figure 5.3 shows the holes in the face of the quarry at one location from the back of which the samples were taken. A conical section metal cup of approximately 2 litres in volume was used to obtain the samples. The metal cup was ground into the cleaned off face at the back of the 400mm deep holes. Care was taken not to let material fall into the cup from the roof of the hole which could possibly have been of different composition and have contaminated the sample. Samples were taken from just above the water level in the quarry and also approximately 0.5m above that. It was not possible to take more samples in the 1.7 metres height of unsaturated soil without completely collapsing the vertical face.

5.2.2.3 Levels of the samples relative to the original water level.

Table 5.2 shows the levels from which the samples were taken. It can be seen that all the samples came from between 1.7m and 0.5m below the original water level. The fact that the water was rising in the gravel quarry meant that it was impossible to take samples from any greater depths. However, a reasonable assumption is that at lower levels, the hydraulic gradients which existed were greater and so if it was shown that samples from these higher levels had apparently suffered movement of finer particles, then any similar material at a lower level would have been affected in a similar manner.

5.2.2.4 Relative location of the samples taken by Tarmac Quarry Products Ltd and those retrieved as part of this research.

The “quarry wall” samples were taken close to where Tarmac Quarry Products Ltd took their samples. It may be assumed that the material from the quarry walls is very similar to the material Tarmac Quarry Products Ltd sampled. This is because the extent of the quarry was not governed by a change in nature of the river deposit but only by permissions, access and workable quarry size (Tarmac Quarry Products Ltd Geologist, pers.comm; February 1994).

5.2.2.5 Fine particle content of the two sets of samples.

Tables 5.2 and 5.3 give the percentages by mass smaller than 0.2mm in size determined for the two sets of samples. (this is actually the top of the "FINE SAND" range on a particle size distribution chart). The ranges of particle sizes are plotted together in Figure 5.4 to give a clearer representation of the difference between the two sets of samples. It can be seen that the upper and lower boundary lines all reach the 100% point at a particle size of approximately 5mm. This is because the particle size distributions examined were for the "sand fraction only" of the samples. Tarmac Quarry Products Ltd presented particle size distributions for the "sand fraction only" because they used the Hazen method of permeability estimation on their samples. The Hazen method generally overestimates permeabilities and if applied to the sand fraction a better estimate of permeability for the material is obtained. Therefore any loss of fine particles should be clearer than if the particle size distribution from the whole sample were used. Tables 5.2 and 5.3 show that the samples from the quarry walls all contain a very low percentage of fine sand size particles relative to the samples taken by Tarmac Quarry Products Ltd. The range of percentages of material less than 0.2mm in size is 1.7 to 6.8. The samples taken prior to quarrying and de-watering contain from 7 to 28 percent of material less than this size.

5.3 Possible reasons for the differences in the ranges.

5.3.1 Could the actual material sampled have been different?

The quarry boundary was not governed by geological variations but by other factors such as the railway line to the north, permissions on the land and workable quarry size. The particle size distributions upper and lower bounds in Figure 5.4 show that the two sets of samples originated from very similar material. Tables 5.2 and 5.3 show that the depths of the samples from the quarry walls were generally shallower than those taken by Tarmac Quarry Products however there is considerable overlap.

5.3.2 Sampling methods.

The method by which the samples were taken from the quarry walls has been described. The samples taken by Tarmac Quarry Products were retrieved using a 200mm hollow stem auger. Let us examine the possibility that this variation in sampling method can be used to explain the differences between the two sets of samples. When material is brought to the surface by the augers, it is possible that materials from different depths become mixed together. Therefore, there is a possible contamination problem when sampling using the hollow stem auger system. One possible explanation for the apparent increased content of finer particles in the samples presented by Tarmac Quarry Products Ltd is that the surface soil (say the top 1 metre) contained a large percentage of finer particles. If this topsoil became mixed with cleaner sands and gravels from lower down during the augering process, an apparently larger fines content of the soil would be the result.

The hypothesis that samples became mixed with material of a high fines content from shallow depths during the augering process was tested by "combining" particle size distributions in specified ratios of the supposedly lower, cleaner, material and the alleged contaminant. The calculation for this (Calculation 5.2) is set out at the rear of the chapter. The calculation shows that in order for the Tarmac samples to be created via “clean gravel” mixing with “silty contaminant”, a degree of contamination of 3 parts topsoil contaminant to 1 part lower clean gravel would have been required. The contaminant required was a silty sand. The particle size distribution for the contaminant material derived from the process set out below is given in Figure 5.5. It should be noted that no material with a particle size distribution approaching that required for the contaminant has been observed on the site and therefore that this degree of contamination is highly unlikely ever to have occurred. It may therefore be concluded that topsoil contamination during sampling is not the reason for the differences in fine particle content between the two sets of samples.

5.3.3 Seepage of water through the face of the quarry.

The level of the water table is one significant variable between the two sets of samples. The samples taken from the quarry walls were of soil through which the water table had been lowered and through which water had seeped in the past. This seepage is one possible mechanism which may have removed finer particles from the soil. The Kenney and Lau method was not applied to the pit wall samples for the very fact that it was hypothesised that some proportion of material had been removed.

5.3.4 Mass of the samples.

The mass of the samples taken from the quarry walls was governed by the size of the sampling cup which was used. The total masses varied from 1 to 1.5 kilograms. The maximum size of particle within these samples was 28mm for six samples, 37.5mm for three samples and 20mm for one sample. For the maximum sizes of particle quoted above, British Standard code number 1377:Part 2:1990 recommends the dry mass to be taken for the sample to be truly representative of the soil. These dry masses are set out in Table 5.4.

It can be seen that the masses of the "quarry wall" samples were below those recommended by BS 1377 if one considers the maximum particle size. However, the particle size distributions which were drawn for the comparison of fine particle content were drawn for the material less than 5mm in size. BS 1377 recommends that a sample size of 0.2kg be taken if the maximum size of particle within a sample is 6mm. The actual masses of sample taken were much larger than 0.2kg and therefore the distributions would all be accurate at the sizes less than 5mm. The comparisons of fine particle contents are therefore valid. Masses of the samples are therefore unlikely to be the reason for the variation between the two sets.

5.4 Samples taken during Well and Standpipe Installation for the Pump Test.

During standpipe installation for the pumping test described in Chapter Six, 10 bulk soil samples were taken from various locations within the test site. The location of the field used for the pump test is as shown in Figures 6.4 and 6.10. Six of the samples were taken from material brought to the surface using the Nottingham Trent University's 150mm hollow stem auger system. This system is very similar to the 200mm hollow stem auger system which was employed by Tarmac Quarry Products Ltd to retrieve their own samples during standpipe installation. The other 4 samples retrieved from the test site were taken at various depths from the hole into which the well was installed. This 250mm diameter hole was driven using a percussion boring rig. The percentage of fine particles found in the samples are shown in Table 5.5.

The previously discussed method of grading stability determination presented by Kenney and Lau (1985) was applied to all 10 samples. Two of the samples were found to be stable under these criteria, namely samples 4 and 9. Sample 10 was on the borderline between stable and unstable. The remaining 7 samples were found to be unstable.

The percentages by mass less than 0.2mm on the particle size distributions are shown in the table. It can be seen that the samples taken by the percussion boring method contain consistently less material below this size than the samples retrieved by the hollow stem auger system. Reasons for the difference probably lie in the sampling methods. When sampling using a percussion boring rig, the shell is withdrawn from the ground and it is at this moment that water can flow through the material in the shell thus removing finer particles. This does not occur with the hollow stem auger system and it is very probably this process which can be used to explain the consistently lower proportion of finer particles in the samples retrieved using the percussion boring shoe.

The samples taken from the site used for the pump test were taken after the quarry had been extensively de-watered for a period of years, but before the pump used in the test was switched on. If the "sand fraction only" distributions of the samples taken by Tarmac Quarry Products Ltd are compared with samples from the pump test field, it can be seen that both sets of distributions are very similar in character. Both sets show a large amount of material in the Medium Sand range with the percentage of material less than 0.2 millimetres being of the order of 10 - 25%. This would appear to be evidence that pumping from the quarry did not remove fine particles from the samples assessed.

The pump test itself subjected this section of the aquifer to undoubtedly higher hydraulic gradients than it would have been subject to during the de-watering of the quarry. However, the periodic examinations of the discharge water indicated that no finer particles were being removed from the aquifer. This was despite the fact that the pump seemed to be in a region of the aquifer which would be classified as unstable given Kenney and Lau's criteria.

5.5 Summary

The results presented in this chapter originated from the assertion that effective stress increase alone could not have been the reason for structurally significant settlements due to de-watering having occurred in Hoveringham village given its distance from the de-watered void. Fine particle movement by flowing water ("suffosion") was examined as another potential settlement mechanism. In section 5.1 the "Kenney and Lau method" was introduced. This method can be used to indicate if a given material might suffer suffosion under any given seepage conditions.

The seepage conditions employed by Kenney and Lau in the derivation of their method were more severe than would be likely to occur in nature under such conditions as flow towards a de-watered void in an unconfined aquifer. Indeed, for the case study, the Reynold's number of flow at the quarry face during the conditions of maximum drawdown was estimated as approximately one. This was significantly lower than Reynold's numbers typically employed by Kenney and Lau in their tests which were generally greater than ten. It can be concluded that should the natural material in the Hoveringham study area be shown to be "stable" under the Kenney and Lau criteria (i.e, not possessed of the ability to lose finer particles) this material is unlikely to experience movement of finer particles in nature.

Section 5.2.1 describes how the Kenney and Lau method was applied to 21 soil samples taken in the case study area prior to de-watering. Four were found to be "stable" and 17 were found to be "unstable" (possessed of the ability to lose finer particles). This was not proof that finer particle movement had occurred in the case study area only that the majority of the material possessed the potential to lose finer particles under certain seepage conditions.

In section 5.2.2 it was discussed how soil samples were taken from the quarry face to investigate whether the seeping water had removed finer particles at this location. This was the ideal place to sample from because at the quarry face hydraulic gradients would have been greatest during de-watering. If samples from the quarry wall were shown not to be deficient in finer particles and it was less likely that material further away from the quarry face and into the aquifer had lost particles, then there would be no strong case for fine particle movement to have occurred on a wide scale within the aquifer (or on a wide enough scale to have caused structurally significant settlements).

In Figure 5.4 it was shown that compared to a set of soil samples taken prior to de-watering, the set of samples taken from the quarry wall were significantly deficient in particles of the lower medium and fine sand sizes. On examination of a number of possible reasons for this difference, it was apparent that seeping water was the most likely cause for the lack of finer particles in the samples obtained from the quarry wall.

In section 5.4, it was discussed how soil samples were taken after de-watering of the quarry had begun. These samples were found to contain similar percentages by mass of finer particles (less than 0.2mm in size) to the set of samples taken prior to de-watering.

5.6 Conclusion

The analyses detailed in this chapter have shown that under certain seepage conditions, finer particles within the sand and gravel material in the Hoveringham area could be displaced. The Reynold's Number of flow which existed close to the quarry wall was estimated and shown to be well below the values at which Kenney and Lau first noticed particle movement during their tests. Samples taken before and after de-watering in the quarry were assessed and it was found that the samples taken after de-watering were deficient in fine particles compared to those taken prior to de-watering. Sample contamination during sampling, sampling method, sample levels, location of the samples, and mass of the samples were all ruled out as a cause of the differences in fine particle content of the two sets of samples. It was concluded that seepage of water through the samples was the most likely cause of the difference between the two sets of samples.

It was also described how samples taken approximately 100 metres from the de-watered quarry were shown to have similar percentages of finer particles to those taken prior to de-watering.

From the results described above it may be concluded that even though the material in the aquifer was shown to have the potential to lose finer particles under certain seepage conditions, it is highly unlikely that particles were displaced other than very close to the quarry wall. This is very likely to be due to the relatively low Reynold's Numbers which would have existed away from the quarry wall compared with those that would have been required to cause fine particle movement.

It is therefore also highly unlikely that fine particle movement occurred beneath the structures of Hoveringham village and caused structurally damaging settlements

CALCULATION 5.2

Type and amount of topsoil contaminant required during sampling in order to create the "pre-site investigation material"

In the following calculation, the particle size grading from "Sample 1, Location 5" taken from the pit wall during the time of maximum water table drawdown has been used. What has been calculated is the grading of fines rich topsoil contaminant required, along with the degree of contamination that would have been required during sampling, in order to create the sample "HV 34/92" taken by Tarmac Quarry Products.

The steps in the calculation were as follows:

1. A total of 500 grams of final sample was assumed. A degree of contamination was assumed and the respective masses of the contaminant and "Sample 1, Location 5" soil were computed.

2. The grading of the "Sample 1, Location 5" soil was known and so the "masses passing" each individual particle size on the distribution chart could be calculated.

3. This meant that at each stage a remaining mass of topsoil contaminant could be calculated. This mass had to get smaller as size decreased otherwise the degree of contaminant was too low.

4. If the degree of contamination was shown to be too low, a higher degree was assumed and the whole process repeated until all masses in the "Topsoil Contaminant" column decreased in sequence as required.

Iteration 1 - Degree of Contamination assumed: 2:1

(Topsoil contaminant:Clean gravel).

For a total mass of sampled material of 500 grams, the assumed ratio gives:

Mass of contaminant = 167g

Mass of clean gravel = 333g

Size of Particle	CONTAMINANT mass & % passing	Sample 1 Loc'n 5 mass passing	Sample HV 34/92 mass passing
(mm)	(mass / %)	(% x g = g)	(% x g = g)
5	167g / 100%	100% x 333 = 333	100%x 500 = 500
3.35	184g / XXXX	87.5% x 333 = 291	95% x 500 = 475
2.00	Increase amount of	75.6% x 333 = 126	89% x 500 = 445
1.18	contaminant !!!	65.4% x 333 = 109	86% x 500 = 430
0.600	-	48.5% x 333 = 81	76% x 500 = 380
0.425	-	28.9% x 333 = 48	58% x 500 = 290
0.300	-	12.5% x 333 = 21	39% x 500 = 195
0.212	-	2.85% x 333 = 4.7	30% x 500 = 150
0.150	-	1.1% x 333 = 1.8	24% x 500 = 120

Notes to Iteration 1:

More topsoil contaminant is required for the HV 34/92 distribution to be feasible. The degree of contamination will be increased in Iteration 2.

Iteration 2 - Degree of contamination assumed: 1:1

For a total mass of sampled material of 500 grams, the assumed ratio gives:

Mass of contaminant = 250g

Mass of clean gravel = 250g

Size of Particle	CONTAMINANT mass & % passing	Sample 1 Loc'n 5 mass passing	Sample HV 34/92 mass passing
(mm)	(mass / %)	(% x g = g)	(% x g = g)
5	250g / 100%	100 x 250 = 250	100%x 500 = 500
3.35	257g / XXXX	87.5 x 218 = 219	95% x 500 = 475
2.00	Increase amount of	175.6	89
1.18	contaminant !!!	65.4	86
0.600	-	48.5	76
0.425	-	28.9	58
0.300	-	12.5	39
0.212	-	2.8	30
0.150	-	1.1	24

Notes to Iteration 2:

More topsoil contaminant is required for the HV 34/92 distribution to be feasible. The degree of contamination will be increased in Iteration 3.

Iteration 3 - Degree of contamination assumed: 1:2

For a total mass of sampled material of 500 grams, the assumed ratio gives:

Mass of contaminant = 333g

Mass of clean gravel = 167g

Size of Particle	CONTAMINANT mass & % passing	Sample 1 Loc'n 5 mass passing	Sample HV 34/92 mass passing
(mm)	(mass / %)	(% x g = g)	(% x g = g)
5	333g / 100%	100% x 167 = 167	100%x 500 = 500
3.35	329g / 99%	87.5% x 167 = 146	95% x 500 = 475
2.00	319g / 96%	75.6% x 167 = 126	89% x 500 = 445
1.18	320g / 96%	65.4% x 167 = 109	86% x 500 = 430
0.600	299g / 90%	48.5% x 167 = 81	76% x 500 = 380
0.425	242g / 73%	28.9% x 167 = 48	58% x 500 = 290
0.300	174g / 52%	12.5% x 167 = 21	39% x 500 = 195
0.212	145g / 44%	2.8% x 167 = 4.7	30% x 500 = 150
0.150	118g / 35%	1.1% x 167 = 1.8	24% x 500 = 120

Notes to Iteration 3:

The figures in **bold** in the table show that the degree of contamination assumed is very slightly below the minimum degree of contamination required for the HV 34/92 distribution to be feasible. In iteration 4, a higher degree of contamination was assumed.

Iteration 4 - Degree of contamination assumed: 1:3

For a total mass of sampled material of 500 grams, the assumed ratio gives:

Mass of contaminant = 375g

Mass of clean gravel = 125g

Size of Particle	CONTAMINANT mass & % passing	Sample 1 Loc'n 5 mass passing	Sample HV 34/92 mass passing
(mm)	(mass / %)	(% x g = g)	(% x g = g)
5	375g / 100%	100% x 125 = 125	100%x 500 = 500
3.35	366g / 98%	87.5% x 125 = 109	95% x 500 = 475
2.00	350g / 93%	75.6% x 125 = 95	89% x 500 = 445
1.18	348g / 93%	65.4% x 125 = 82	86% x 500 = 430
0.600	319g / 85%	48.5% x 125 = 61	76% x 500 = 380
0.425	254g / 68%	28.9% x 125 = 36	58% x 500 = 290
0.300	180g / 48%	12.5% x 125 = 16	39% x 500 = 195
0.212	147g / 39%	2.8% x 125 = 3.5	30% x 500 = 150
0.150	119g / 32%	1.1% x 125 = 1.4	24% x 500 = 120

Conclusion

It can be seen from Iteration 4 that the distribution for HV 34 /92 is obtained. The degree of contamination is therefore beyond the minimum required. It can be seen from iterations 3 & 4 that the correct degree of contamination is between 1 part clean gravel:2 parts topsoil contaminant and 1 part clean gravel:3 parts topsoil contaminant. From its grading, the contaminant is classified as a silty sand.

Sample Reference	Depth Below Ground Level (m)	Borehole Reference	Notes
A	0.8-4.0	HV 36/92	Stable down to 0.075mm size
B	4.0-8.3	HV 36/92	as for A above
C (2 no samples)	0.6-4.0	HV 35/92	Unstable in the FINE SAND zone
D	3.0-5.0	HV 34/92	Unstable in the FINE SAND zone
E	5.0-8.8	HV 34/92	Unstable in the FINE SAND zone
F	4.0-7.5	HV 33/92	Stable sample
G	0.5-4.0	HV 37/92	Unstable in the FINE SAND zone
H	4.0-7.3	HV 37/92	Unstable in the FINE SAND zone
I	1.0-4.0	HV 38/92	Unstable in the FINE SAND zone
J	4.0-7.3	HV 38/92	Unstable in the FINE SAND zone
K	2.0-4.0	HV 39/92	Stable sample
L	4.0-7.5	HV 39/92	Unstable in the FINE SAND zone
M	1.5-4.0	HV 40/92	Unstable in the FINE SAND zone
N	1.5-4.0	HV 41/92	Unstable in the FINE SAND zone
O	4.0-6.5	HV 41/92	borderline of unstable for FINE SAND
P	3.0-4.0	HV 43/92	Unstable in the FINE SAND zone
Q	4.0-5.1	HV 43/92	Unstable in the FINE SAND zone
R	4.0-8.2	HV 40/92	Unstable in the FINE SAND zone
S	1.5-4.0	HV 42/92	Unstable in the FINE SAND zone
T	4.0-6.0	HV 42/92	Unstable in the FINE SAND zone

Table 5.1

Summary Of Sample Stability Investigations On The Samples Taken By Tarmac Quarry Products Limited (1993)

Location No	Sample No	Approx Depth of sample below original ground water level	% by mass passing 0.2mm sieve size
1	1	1.4m	3.9
1	2	0.8m	2.5
2	1	1.5m	4.7
2	2	0.6m	6.8
3	1	1.7m	1.8
3	2	1.3m	1.7
4	1	1.5m	3.9
4	2	0.5m	4.1
5	1	1.4m	2.8
5	2	0.7m	3.6

Table 5.2

Approximate Depths Below Original Ground Water Level And Percentages Determined At The Top Of The Fine Sand Range For Samples Taken From The Quarry Walls.

Note to Table 5.2

All the above samples were taken from between 0.5m and 1.7m below the level of the original water table.

Locations within the quarry from where samples were taken are shown on Figure 5.2.

Sample reference	Approx depth of sample below original ground water level	% passing 0.2mm sieve size
A	0-3m	10
B	1.0-7.3m	25
C	0-3.0m	10/8 (2 samples)
D	2.0-4.0m	28
E	4.0-7.8m	12
F	0-3.0m	19/20 (2 samples)
G	0-3.0m	19
H	3.0-6.3m	15
I	0-3.0m	19
J	3.0-6.3m	19
K	1.0-3.0m	15
L	3.0-6.5m	22
M	0.5-3.0m	23
N	0.5-3.0m	15
O	3.0-5.5m	12
P	2.0-3.0m	16
Q	3.0-4.1m	9
R	3.0-7.2m	10
S	0.5-3.0m	11
T	3.0-5.0m	7

Table 5.3
Approximate Depths Below Original Ground Water Level And Percentages Determined At The Top Of The Fine Sand Range For Samples Taken By Tarmac Quarry Products Prior To De-Watering.

Largest significant particle size within the sample (mm)	Dry mass of sample required (kg)
6	0.2
20	2
28	3.5
37.5	8

Table 5.4

Dry Masses Of Samples Recommended For Maximum Particle Size Within The Sample (From B.S.1377:Part 2:1990)

Sample Identification	Origination of sample	Sampling method employed	Approx % passing 0.2mm sieve
2	Wellhole - depth 3m	percussion	6
3	Wellhole - depth 4m	percussion	4
4	Wellhole - depth 5m	percussion	9
5	Wellhole - depth 5.5m	percussion	10
6	Standpipe E	auger	24
7	Standpipe F	auger	22
8	Standpipe J	auger	15
9	Standpipe K	auger	18
10	Standpipe L	auger	15
11	Standpipe P	auger	19

Table 5.5

Fine Particle Content Of Samples Retrieved During Well And Standpipe Installation For The Pumping Out Test.

Notes to Table 5.5:

- Sample No.1 was merely a topsoil sample and therefore was not subjected to particle size analysis.
- "Percussion" implies a 250mm borehole formed using a percussion boring rig. "Auger" implies a 150mm hollow stem auger system.
- The quoted percentages at the top of the fine sand range were taken from particle size analyses of the material less than 5mm in size.

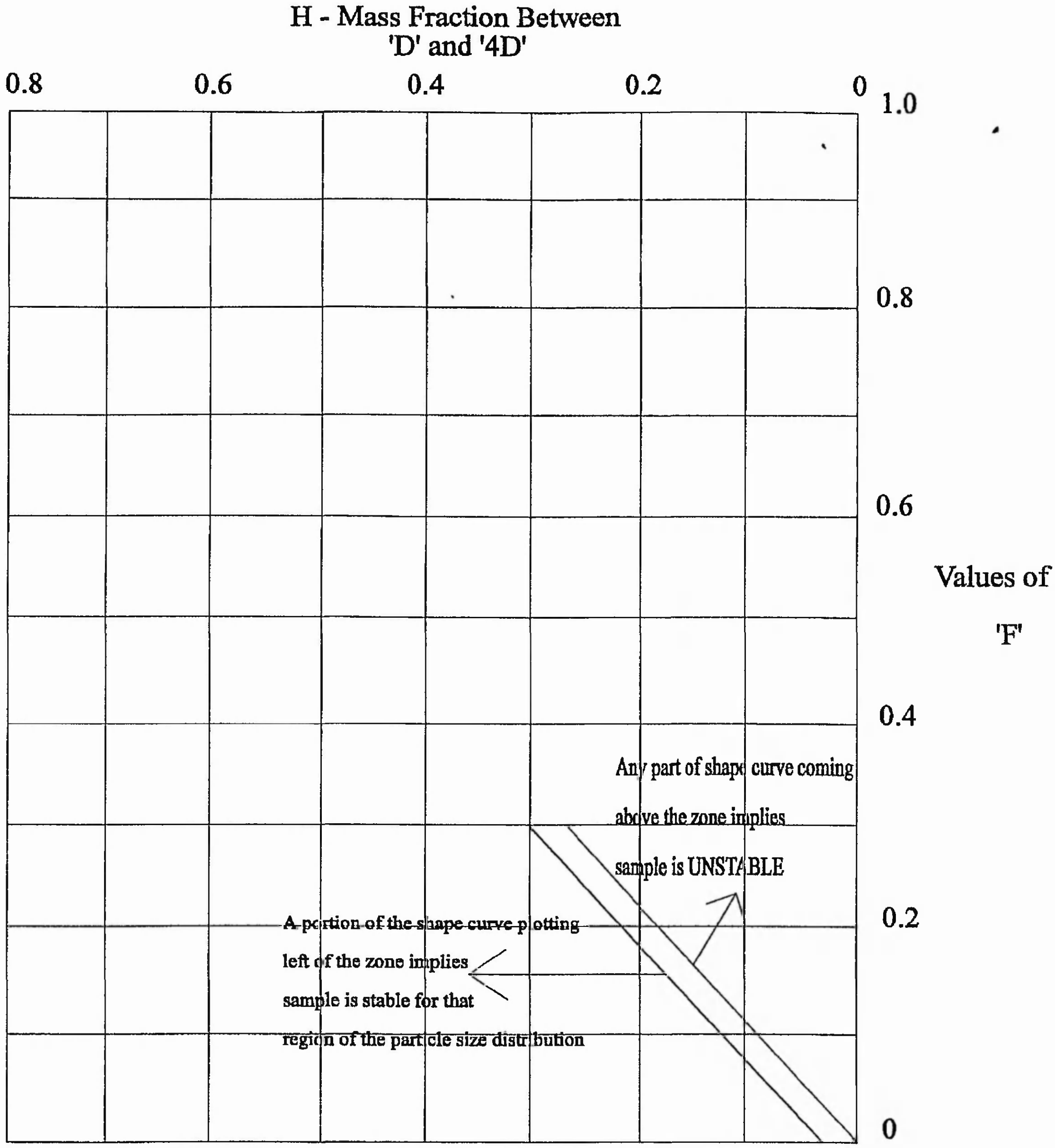


Figure 5.1

Chart for assessing grading stability

of a granular material

(Kenney and Lau , 1984)

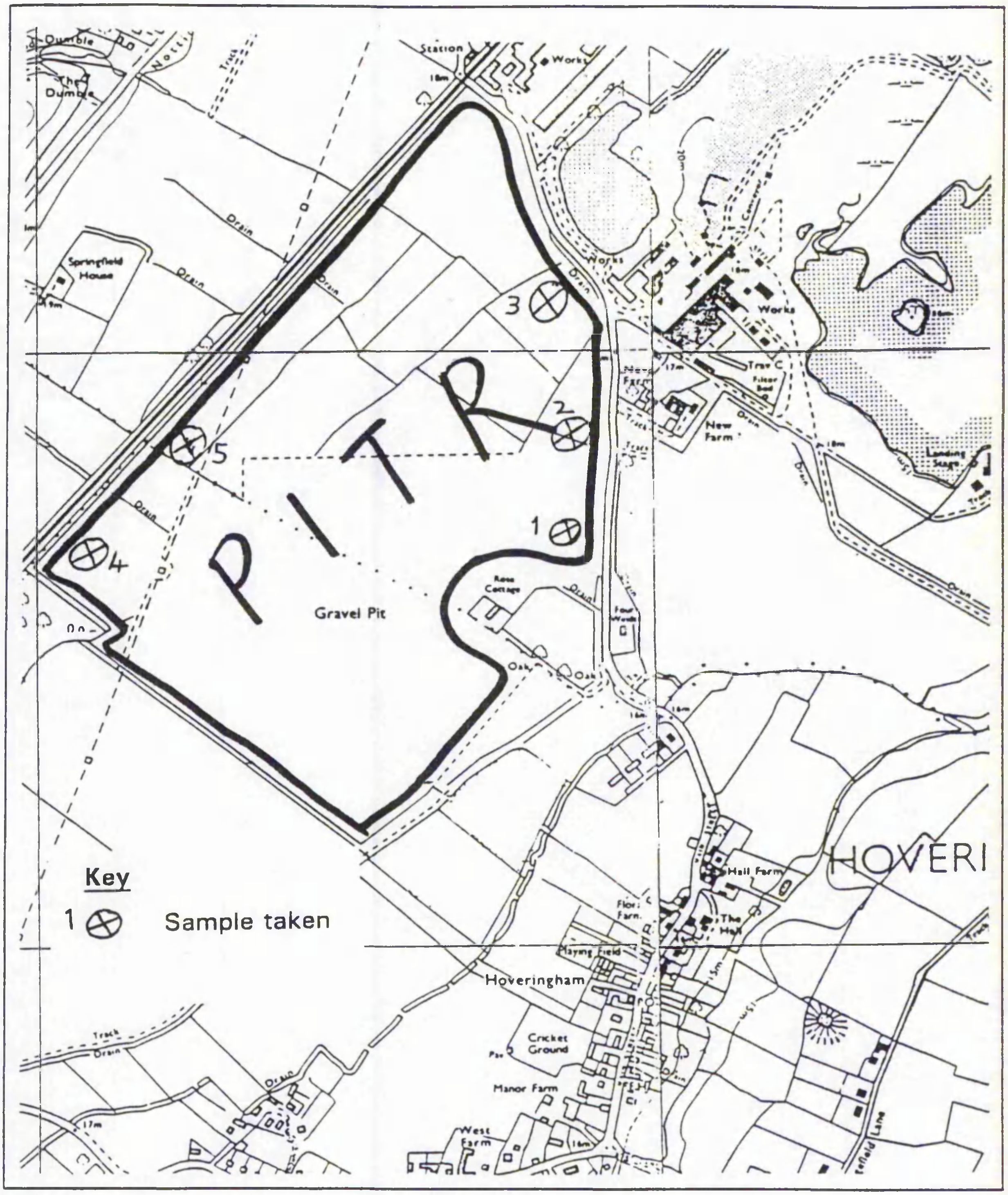


Figure 5.2

Locations From Where Samples Were Taken From The Quarry Wall To Assess Fine Particle Content.

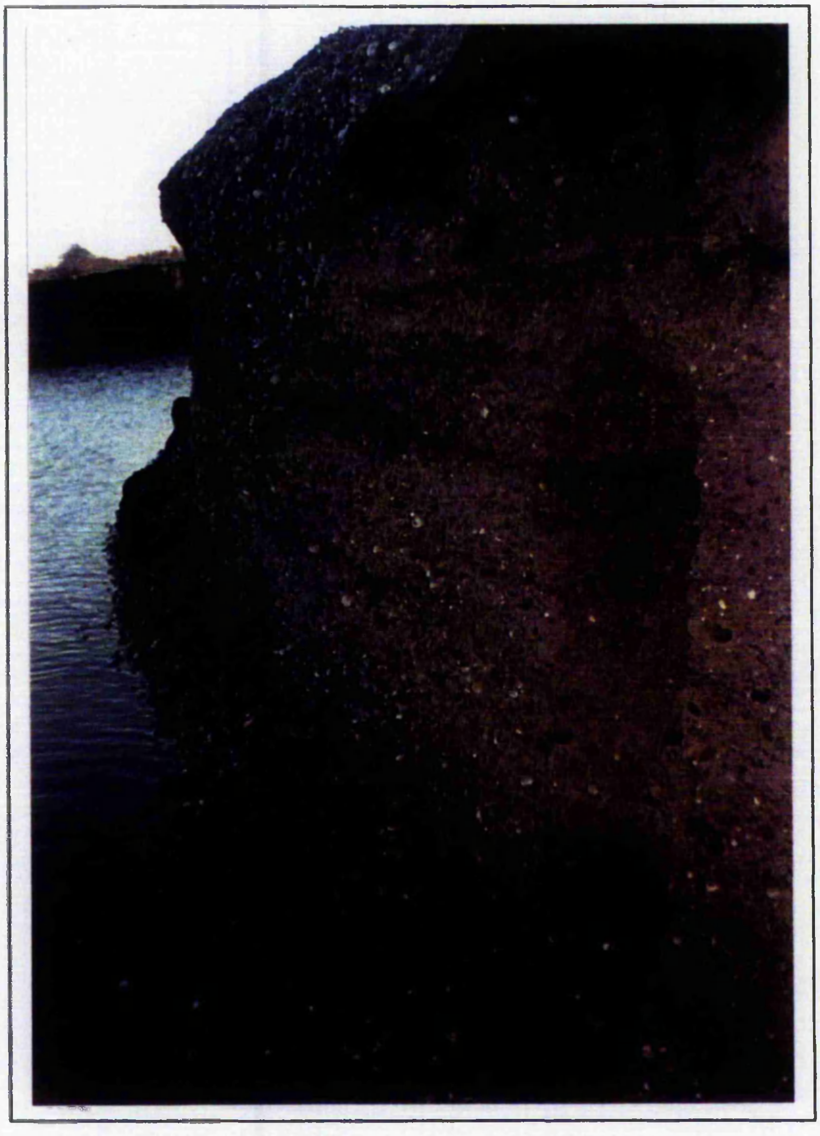


Figure 5.3

A Hole Dug Into The Quarry Wall To Retrieve

A Sand And Gravel Sample

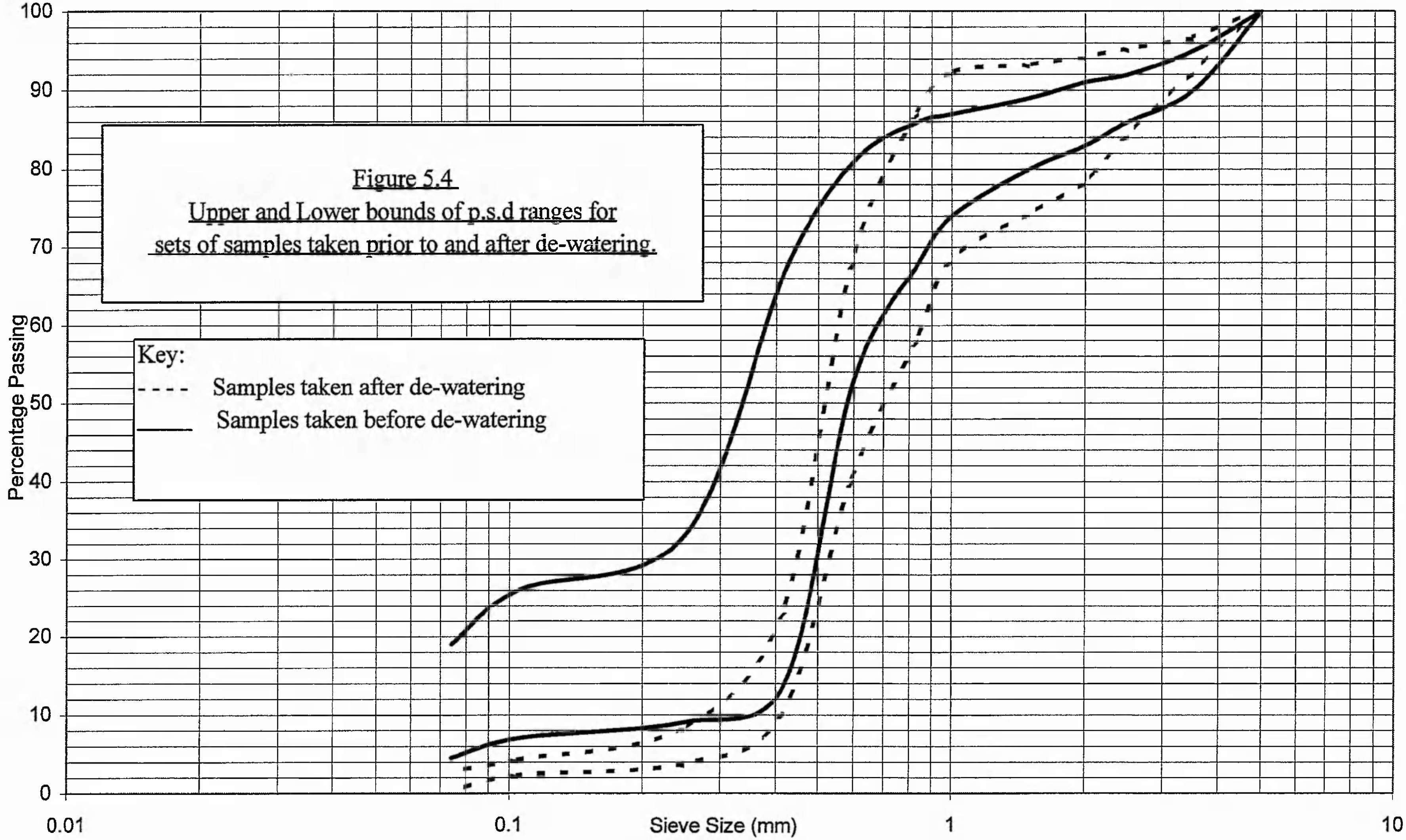


Figure 5.4
Upper and Lower bounds of p.s.d ranges for sets of samples taken prior to and after de-watering.

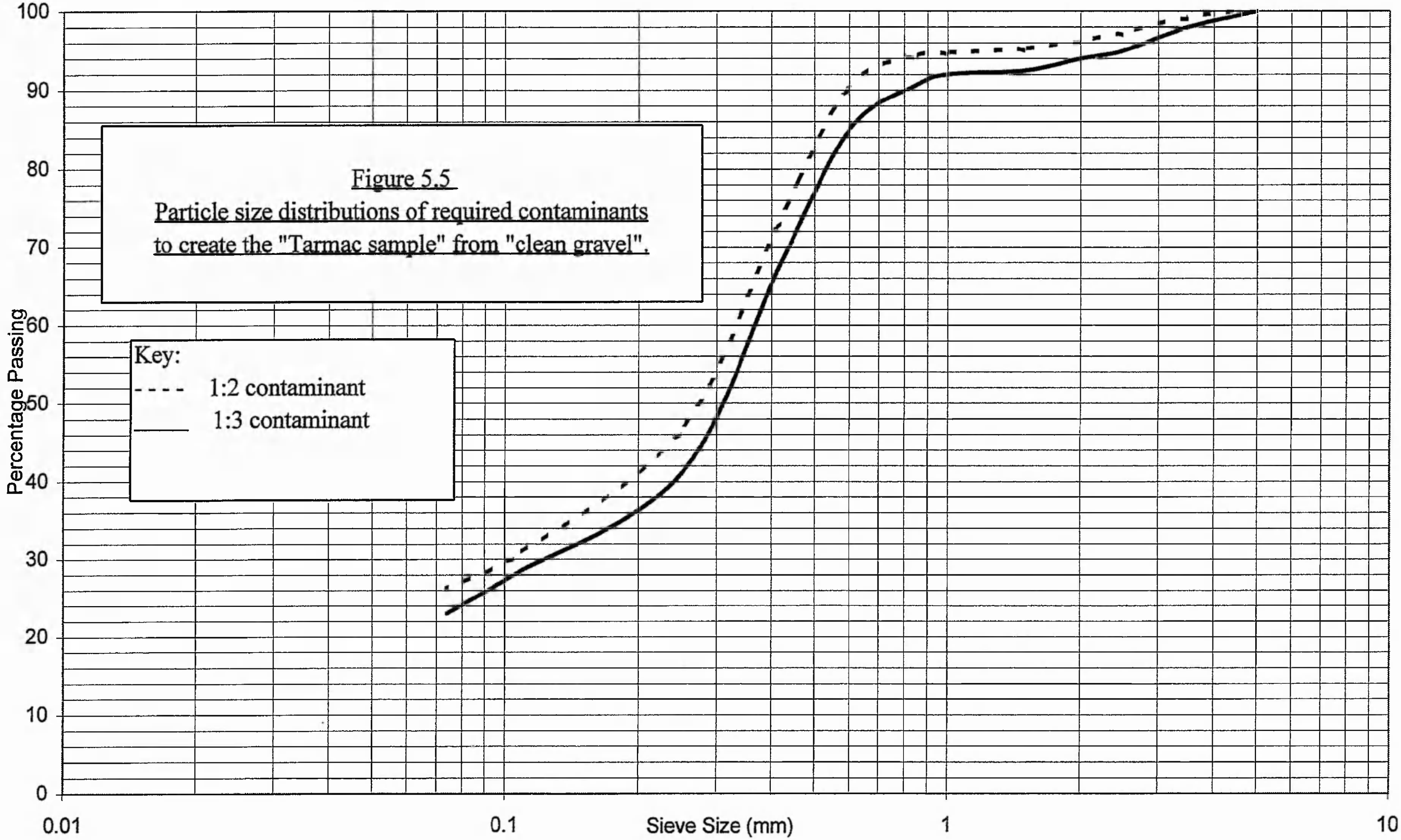


Figure 5.5
Particle size distributions of required contaminants to create the "Tarmac sample" from "clean gravel".

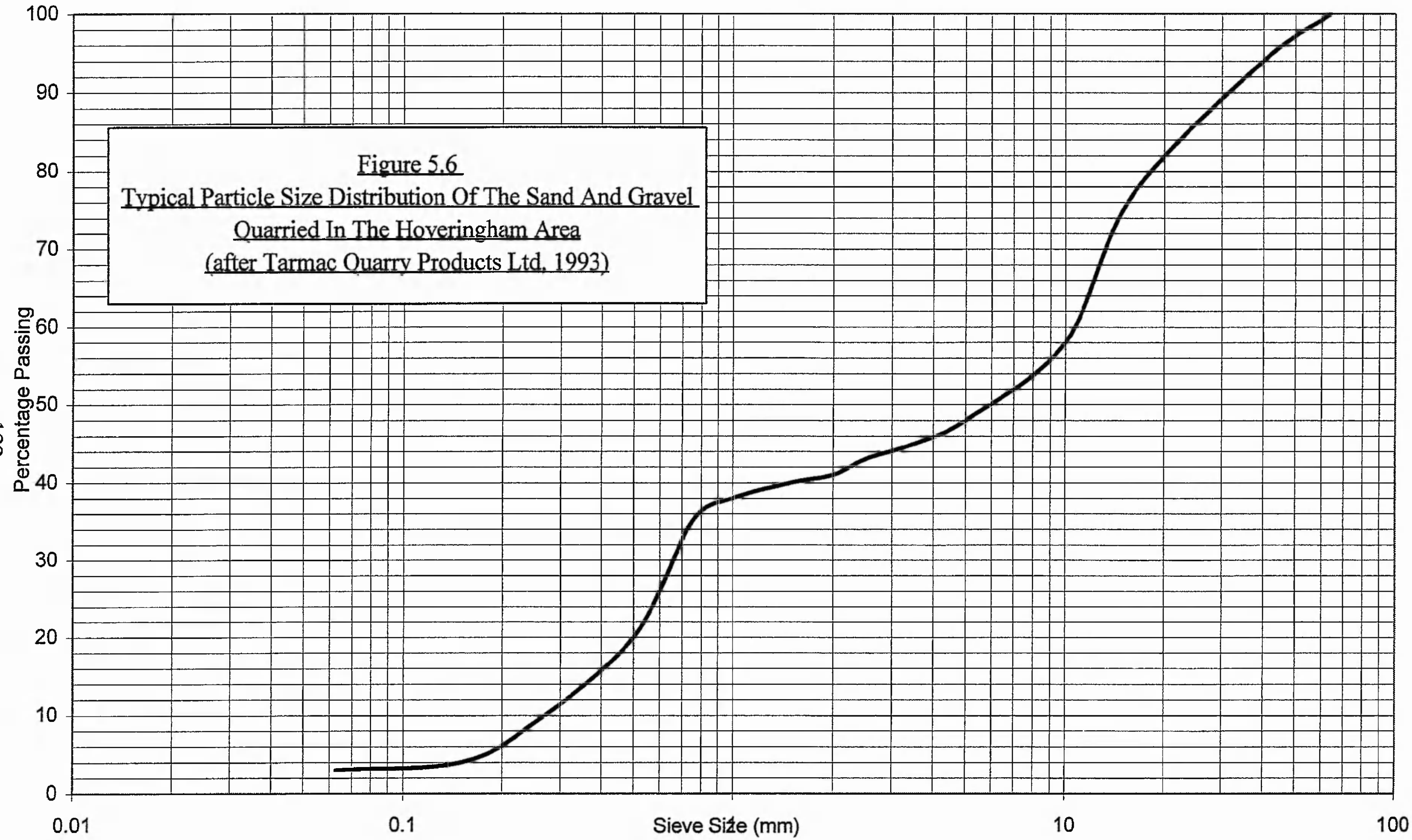


Figure 5.6
Typical Particle Size Distribution Of The Sand And Gravel Quarried In The Hoveringham Area
(after Tarmac Quarry Products Ltd, 1993)

CHAPTER SIX
GEOPHYSICS AND THE PUMP TEST

6.1 Introduction

Chapter Four indicated that a direct correlation may exist at Hoveringham between the occurrence of structural defects due to settlement and proximity to the de-watered quarry. In Chapter Five the mechanism of "suffosion" was examined as a potential cause of settlement. The conclusion from Chapter Five was that it is highly unlikely that suffosion had occurred beneath the structures in the village and lead to settlement.

Spatial variation of permeability within an aquifer was introduced in section 2.5 as a phenomenon which could lead to differential distances of influence around a de-watered zone. The literature examined tended to be only qualitative and rarely quantitative about the phenomenon. It is recognised that spatial variation of permeability exists but no evidence of previous research was found which investigated how variable the distances of influence could be within the de-watered zone of an unconfined alluvial aquifer.

The planning application for Hoveringham quarry was granted on the basis of calculations of the radius of influence based on the technique described in CIRIA Report 113 (see Section 2.3). The equation used was:

$$R = C h \sqrt{k}$$

where:	**R**	=	Radius of influence (metres)
	h	=	drawdown of the water table (metres)
	k	=	permeability of the aquifer (metres/second)
	C	=	a factor equal to 3000 for radial flow to pumped wells

Using the above equation the radius of influence calculated for the whole of the de-watered zone around the quarry was approximately 230m.

It can therefore be seen that no account was taken of the possible existence and consequences of spatial variation of permeability within the section of the aquifer to be de-watered at Hoveringham.

The work described in this chapter aimed to identify, quantify and examine the consequences during de-watering of any spatial variation of permeability existing within the floodplain around the quarry and to examine the aquifer for evidence of suffosion. To do this, geophysics was used in conjunction with a full scale pumping out test.

6.2 Why Geophysics And A Pump Test?

For reliable in-situ permeability determination, the constant or variable head borehole tests are normally used. Carrying out these tests as part of this research was ruled out by financial constraints. It was decided that a full scale pumping out test would be more appropriate to mimic the quarry de-watering process while simultaneously testing a large region of the aquifer. Standpipes and temporary bench marks placed around the central well could be used to investigate varying drawdown and settlement behaviour. In order to investigate spatial variation of permeability, the pump test needed to be located in a section of the aquifer where geological variation existed. How was any geological variation to be observed and quantified? The answer was geophysics. With the limited funds available, large areas of land could be investigated using geophysics as a low cost site investigation technique. The pump test could then be sited in a geologically variable area.

6.3 Choice Of Geophysical Techniques And The Method Adopted

Within the Trent Valley aquifer around Hoveringham, permission to use large amounts of land had been granted subject to the provision that our impact on the land was "reasonable". Many geophysical methods were available which would cause minimal or no disruption. There is currently no geophysical method which can directly measure permeability. Geophysical methods can only give measurements which have a non-linear relation to this soil property. However a technique which was capable of differentiating between materials which exhibited varying drawdown and settlement characteristics during the pump test was desirable. The various techniques considered were:

6.3.1 Seismic Methods

Seismic methods of ground investigation, either using explosives or a ground impacting sledgehammer are usually used to define boundaries between materials. Seismic methods rely on the different velocities of shockwaves within different materials and are better employed for deeper surveys than were required at Hoveringham. The unconfined alluvial aquifer in Hoveringham consists of 5-10 metres of sand and gravel which is underlain by Weathered Mercia Mudstone. Problems would be encountered using seismic methods in such a shallow aquifer as the air and ground interference signals would be received by the geophones almost at the same time as the signal from the strata of interest because of the short signal travel times involved. Indeed, it is known that at the present time Dr Ian Hill of Leicester University is conducting research into the usage of shallow seismic methods within shallow sandy strata. It is obvious then, that at this point in time, no tried and tested seismic method exists for the investigation of variations within a shallow sandy strata.

6.3.2 Resistivity

Resistivity methods measure the apparent resistivity of the ground in ohm metres. This was considered as a possible method but eventually rejected for the following reasons:

- The electrodes (metal prongs) would form poor contact with the in-situ granular material. It was felt that the ratio of contacting to non-contacting surface area of electrode would have been unacceptable especially as much of the land available consisted of ploughed fields, the surfaces of which had become hard and broken during the hot summer at the time of the investigations. It was unlikely that even 20% electrode-ground contact would be obtained within ground of this type.

- It was felt that the rate of progress would be too slow bearing in mind the amount of cable moving that would be involved.

- A quicker, cheaper and apparently better method existed anyway. This was ground conductivity (see below).

6.3.3 Gravity Methods

Differences in rock density produce small changes in gravity fields which can be measured using portable instruments known as gravity meters or gravimeters. Differences in elevation (although not a problem at Hoveringham) produce minute variations in the measured quantity for which readings have to be corrected. The method is best suited to detecting buried structures which produce significant variations in the measured gravitational field. This method would have been inappropriate for looking at geological variations within an aquifer consisting of material of similar density.

6.3.4 Magnetic Methods

Significant magnetic effects are produced by only a very small number of minerals. The main application of this method is the detection of magnetic variations within deep rocks. The method is not suitable for studying void ratio, permeability, or geological variations within shallow aquifers. The magnetic method may have helped detect which regions of the aquifer contained the greatest content of finer particles (the finer particles being likely to consist of the more magnetic minerals). Although this may have indicated which areas were likely to experience permeability and void ratio lows, there was no evidence within any examined literature which suggested this would be the case.

6.3.5 Electromagnetic Conductivity Methods

Electromagnetic conductivity meters create an electromagnetic field in the ground at the transmitter end of the meter, the intensity of which is then measured by the receiver at the opposite end of the meter. The property measured is the mean conductivity of the hemisphere of ground in between the receiver and transmitter.

Instrumentation available to examine electromagnetic conductivity of the ground includes the EM31 (one man) and EM34/3 (two man) ground conductivity meters. Zalasiewicz *et al* (1985) and Auton (1991), both describe the use of these instruments to map shallow glacial sediments. In particular, Auton describes the mapping of sand and gravel deposits. An EM31 meter is shown in Figure 6.1 and an EM34/3 is shown in Figures 6.2 and 6.3. Both these types of instrument were used during the research. The EM31 can give depths of investigation of 3m and 6m. The EM34/3 apparatus can give a depth of investigation of 7.5m with both coils held horizontally (Figure 6.2) and 15m with both coils held vertically (Figure 6.3). Both methods are quick and easy to use. Large areas of land can be covered in very reasonable amounts of time and both instruments are relatively low cost.

Conductivity being simply the inverse of resistivity, it was thought probable that conductivity methods would be able to detect areas of higher fines content in the same way. Areas of higher clay content typically show up as conductivity highs because clays tend to contain higher proportions of the more conducting minerals. For these reasons, ground conductivity was the geophysical method used for the research. One slight disadvantage with this method is that "one number" is obtained representing the bulb of ground in between the two ends of the instrument. This method will not therefore give an indication of whether or not stratification exists within the "bulb".

6.4 The Geophysical Investigations And Results

6.4.1 Methodology

The sections of land in Hoveringham which were investigated using either an EM31 or an EM34/3 ground conductivity meter are shown in Figure 6.4 which is an oversize plan located at the rear of the thesis. Each point on the grids corresponds to where a reading of apparent ground conductivity was made. Readings were taken in both the shallow and deeper reading modes for the particular instrument at each point on the grids. Fields 1, 2 and 3 in Figure 6.4 were investigated using the EM34/3 probe and Fields 4 and 5 were traversed with the EM31. While it is recognised that differing depths of penetration are given for the two probes they both measure the same property and overall variation was the most important factor. All the investigations were carried out during April and May 1994.

6.4.2 Measurement Station Spacing

The gap between each measurement station was determined by the size of feature to be observed. Geological variation within the Trent valley floodplain would depend on factors such as velocity and location of the river channel at anytime, type of bedload deposited, and flood events among other factors. Some features typically found in such an environment are:

1. Old, infilled, ox-bow lakes.
2. Alluvial fans from tributary streams.
3. River terraces.
4. Old, alluvium filled channels between river terraces.
5. Interbedded scroll sands formed on the insides of bends in the meandering channel.

A 30 or 40 metre spacing of measurement station was considered. Such a spacing would have allowed large areas of the aquifer to be traversed but would have been too coarse to observe most of the geological features listed above. In the end a 10 metre spacing between readings taken with the EM31 and a 20 metre spacing for EM34/3 readings was decided upon. The reason for the difference for the two methods was that the EM34/3 penetrated deeper and readings were quicker with the EM31. These spacings were large enough to allow coverage of reasonable areas of land in the time available and thought close enough to allow detection of the geological features.

6.4.3 Results From The Geophysical Investigations

Contour plots of variation of electromagnetic conductivity within the sections of land investigated are given in Figures 6.5 to 6.9. The contours are plotted for the instruments used in the mode for which it read deepest. This was of more interest than the reading for the shallower mode as it was the deeper modes which encompassed the greatest depth of the aquifer investigated.

The plot for Field 1 in Figure 6.5 shows a gentle variation of conductivity across the area with the absolute value remaining relatively low. In Figure 3.5 it can be seen that Field 1 is entirely within a region marked as "pale brown sandy soils with abundant bunter and flint pebbles". There was no surface covering of cohesive alluvium which could have lead to increased conductivity values. This field was also located in an area of generally higher ground levels which meant the water table would have been deeper than for lower lying land. All these factors combined to provide the relatively low measured conductivities.

The plot of variation of conductivity for Field 2 is given in Figure 6.6. It can be seen from Figure 3.5 that Field 2 is located mostly within a region marked as "river terrace gravel" with the eastern edge of the field beginning to encroach into a region containing cohesive alluvial material at the surface. Similarly to Field 1, conductivity values vary little across the site with absolute magnitudes being only marginally higher. To the east of the field, conductivity values are seen to increase due to the cohesive alluvium.

The plot of variation of conductivity for Field 3 is given in Figure 6.7. This field was adjacent to Field 2 and the conductivity response is similar accordingly. The "river terrace gravels" give rise to lower conductivities than the alluvium which is encountered in the north of the field.

The plot of variation of conductivity for Field 4 is given in Figure 6.8. Located in an area of generally lower ground levels than Field 1 but also within a sand and gravel terrace area, the magnitude of the measured conductivities were generally higher. It is likely that the higher values were caused by the water table being closer to the surface in this area.

Field 5 (contour plot in Figure 6.9) is shown in Figure 3.5 to contain a boundary between cohesive alluvium and terrace gravels. This boundary was evident from breaks of slope in the field and from the findings of hand augered holes prior to carrying out the pump test. It can be seen in Figure 6.9 how the electromagnetic conductivity contours define the two contrasting materials. As the gravels become cleaner towards the south west, a drop in conductivity is noticed. The cohesive alluvium which thickens towards the north east gives rise to increasingly higher conductivity values possibly due to the close proximity and higher contact area of the particles of this material relative to the cleaner gravels in the south west.

The results from the geophysics, Field 5 in particular, showed that electromagnetic conductivity was capable of distinguishing between different materials which occurred within the floodplain as well as being able to detect variation within these materials. It is important to note that the water table was found to be at constant depth across the whole of the site and so this could not be a factor which affected the measured conductivities. The pump test would be carried out to look for variation in drawdown and settlement behaviour of materials of varying conductivity. If variation was observed it would then be possible to estimate how very large areas of the aquifer behaved during the large scale de-watering of the quarry.

6.5 The Pump Test - Objectives, Location, Design And Estimated Preliminary Values

6.5.1 Objectives Of The Pump Test

The objectives of the pump test were:

1. To subject a section of the unconfined aquifer of varying geophysical response to groundwater abstraction for sufficient time to achieve steady state drawdowns.

2. To examine the de-watered zone for variations in drawdown and settlement response including progressive settlements.

3. To examine the surface for evidence of settlement severe enough to cause structural damage. Such magnitude of settlement, should it have occurred, would have been strong evidence that finer particles were being moved by flowing water. Progressive settlement of this nature has rarely been researched or reported in the past.

4. To relate the behaviour of materials in the test area to how materials would behave generally in the aquifer as a whole.

5. To examine the permeability of the test area for spatial variation.

6.5.2 The Choice Of "Field 5" As The Location Of The Pump Test

Field 5 had been identified (using the E.M 31) as a region where a large range of ground conductivities existed. The variation in conductivity had been caused by the existence of a channel of Holme Pierrepoint sand and gravel river terrace deposit surrounded by a crescent of alluvium (red-brown or grey silt and clay). This region of the aquifer presented an opportunity to carry out a "calibration exercise" of the behaviour of the different materials under conditions of water table drawdown.

Other reasons for choosing "Field 5" as the location of the pump test were:

- The field was far enough away from the village for noise produced by the pump to be reduced to insignificant levels by surrounding the pump with straw bales.

- It was likely that services existed in the second choice field, “Field 4” which was also too close to the village. Searches showed that no services existed in Field 5.

- Field 5 was a section of the aquifer within which considerable flow of water, drawdown, and possibly settlements should have occurred if properties within the village were to have been affected by the quarry de-watering. The results from the pump test would indicate if such phenomena may have occurred. It may be argued that any mechanisms of particle movement leading to settlements may already have been activated and would not be reactivated by the pumping. However, a pumped well in the centre of Field 5 would undoubtedly cause higher hydraulic gradients to exist within this section of the aquifer than had existed during the quarry de-watering. If the quarry de-watering had caused settlements and removal of finer particles, it could be reasonably expected that the pump test would.

- Land to the north of the railway line and also to the west of Field 5 would have been useful sites for geophysical investigations. It was obviously desirable to study as much of the section of the floodplain of interest as possible. This land is owned by Trinity College, Cambridge who were allowing Tarmac Roadstone Ltd to extract gravel. Tarmac were approached in spring 1994 for permission to carry out geophysical investigations of this land. Permission was denied and so this again reduced the amount of land available.

- The type and time of growth of crop in Field 5 was more convenient than that of other fields. Within other possible pump test locations, maize was to be grown. Maize is a tall cereal plant which would have caused drill rig movement problems and probably have made sighting during levelling impossible.

- Access to Field 5 presented no problems. Local landowners were very willing to allow access across fields or driveways.

- Field 5 was surrounded on all 4 sides by streams which could be assumed as boundaries of influence.

Financial constraints and farmer's permissions dictated that the pump test had to take place between the 22^{nd} July and the 14^{th} August 1994. Funding was available for approximately three weeks of continuous pumping.

6.5.3 Three Preliminary Hand Augered Boreholes

To determine the depth to the unconfined water table and to examine surface materials three holes were hand augered in the locations shown in Figure 6.10 in Field 5. The locations represent areas of higher, intermediate, and lower conductivities.

Findings from the three holes were as follows:

BOREHOLE 1

Approximate conductivity at location: 30 millimhos/m

Topsoil:	Clayey topsoil to about 500mm
Lower soil:	Becoming more sandy but staying slightly cohesive
Water :	Encountered first at a depth of 1.2m
Seepage:	Water seeped into the hole over the first half an hour of the hole being open to give a final depth to water of approximately 1m
Other details:	Cleaner gravel was encountered at the base of the borehole

BOREHOLE 2

Approximate conductivity at location: 20 millimhos/m

Topsoil / lower soil:	To depth of 1.2m, soil was sandy and slightly pebbly
Water :	Encountered first at approximate depth of 1m. Depth to the water table stayed the same over the next half an hour

Other details: Cleaner gravels were encountered at the base of the hole

BOREHOLE 3

Approximate conductivity at location: 60 millimhos/m

Topsoil: To 500mm, topsoil was very dark and very cohesive (possibly organic as indicated by a distinctive pungent aroma!)

Lower soil: Soil to approximately 1m depth became very cohesive and had to be removed in thick "corkscrew" type curls from the hand auger

Water : Encountered first at a depth of 1.2m

Seepage: Water seeped into the hole to increase the depth to 1m over the first half an hour

Findings from the holes correlated well with Figure 3.5. Holes 1 and 2 contained sandy, less cohesive material than borehole 3 which was located within dark, cohesive soil. The varying permeabilities of the surface materials was indicated by the rates at which water seeped back into the holes. Without being able to quantify the variation, the order of decreasing permeability was assumed to be;

BOREHOLE 2

BOREHOLE 1

BOREHOLE 3

The depth to the water table across the whole site was taken as approximately 1 metre, ground level at each hole being approximately the same.

6.5.4 Detailed Design Of The Test

British Standard 6316 : 1992, "Code of Practice for Test Pumping of Water Wells" was referred to during the design of the test. Not all of this Code was relevant to the type of test to be conducted. Some of the relevant items and clauses from the Code which were adhered to were:

- "Typical Pumping Test Procedure" - see Figure 6.11
- Clause 10.8 Disposal of pumped discharge.

- Clause 10.9 Noise
- Clause 11 Design of the test - this section states that for discharge rates up to $500m^3$ / day the minimum duration of a constant discharge test should be 24 hours. The final discharge rate used for the pump test was approximately $350m^3$ / day but the test was run for 14 days because progressive settlement mechanisms were of interest.
- Clause 12.2 Number and purpose of observation wells (standpipes)
- Clause 12.3 Depth of observation wells. This clause states that ideally observation wells should be constructed to the same stratigraphic level as the test well.
- Clause 13.5 Measurement of pumped discharge.
- Clause 19.1 Barometric pressure
- Clause 19.2 Rainfall
- Clause 25.3 Measurement of discharge. This clause states that when the discharge is small, measurements of time taken to fill a container of known volume can be used. This was the method adopted during the pump test.

The method of well development used was "backwashing" as described in Section 2.3. The pump was switched on and off five times as quickly as the mechanics of the suction pump would allow. This should have caused a column of water to repeatedly travel up and down thus "surging" water in and out of the well, screen, and the natural material immediately adjacent to the well. This process attempts to "clean" the material adjacent to the well removing any "mud" or "cake" built up during hole driving in order to return this important part of the aquifer at least to its original hydraulic conductivity or even surpass it.

6.5.5 Plant And Well Type Used

A GP100M Self Priming Univac Pumpset with an 8 metre suction hose was used to create the drawdown of the water table. The pump was positioned within Field 5 such that it should have caused significant water table drawdown within materials showing a large variation of conductivities. The primary pump, a spare, and straw bales for noise attenuation are shown in

Figure 6.12. A view across Field 5 with the pump at the centre is shown in Figure 6.13. The tracks in the picture through the crop were where the drillrig was towed in order to install standpipes. One of the closer structures in the village can be seen in the background. The water was discharged into one of the streams surrounding Field 5 via 90 metres of header pipe. The pumped well itself was 150mm in diameter and was installed within a 7 metre deep, 250mm diameter borehole. The wellscreen was made of high density polyethylene, the bottom 6 metres of which contained 25mm x 1.5mm vertical slots to give an open area of 16%. The filter around the screen was 50mm thick and consisted of 6mm pea gravel. The top one metre of the wellscreen was plain tubing, being installed above the level of the natural water table. From the borehole log taken on installation of the well shown in Figure 6.14 it can be seen that the base of the sand and gravel material was encountered at a depth of 5.50 metres below ground level. Therefore, the bottom 1.5 metres of slotted screen was actually installed into the Weathered Mercia Mudstone below. This was done in order to achieve the maximum possible drawdown.

The slot size of 25mm x 1.5mm was chosen after referring to particle size distribution charts and the findings from Chapter Five. If any fine particles were to be displaced by moving water within the soil they were likely to be smaller than 1.5mm and would enter the well through the screen to be removed by the pump which was purposely chosen for its ability to remove small solids as well as water. A 6mm pea gravel filter was chosen precisely because particles smaller than 1.5mm would be able to pass through it.

6.5.6 The Standpipes Used During The Test

25 standpipes were installed around the pumped well in the pattern shown in Figure 6.15. This array ensured that groundwater information was obtained from all regions of interest (ie regions of varying conductivity) and from varying radial distances away from the pump. The author is shown reading one of the standpipes in Figure 6.16. In the figure a temporary bench mark used to examine the settlement of the aquifer can also be seen next to the standpipe. Standpipe "V" was used to check the drawdown adjacent to the well. The standpipes were installed using a hollow stem auger system. Once the required depth had been reached, the standpipe was inserted into the hollow stem and water added to achieve a positive head

within the augers. The positive head prevented material rising up the inside of the augers which may have blocked the standpipe or caused it to jam inside the casing.

The standpipes themselves consisted of coupled 3 metre lengths of 21.5mm internal diameter U.P.V.C. plastic tubing with holes drilled at regular intervals. Holes were drilled at approximately 50mm spacing to ensure that groundwater would enter all along the length of the tubing. Holes were not drilled at a smaller spacing in order to keep the entrance of finer particles on installation to a minimum.

On boring the holes for the standpipes, the base of the sand and gravel material was encountered at an average of 6.0 metres below ground level. It was attempted to install the standpipes a depth of approximately 0.5 meters into the Weathered Mercia Mudstone below the sand and gravel. This was to try and prevent fine sand from rising up the inside of the standpipe and secondly to provide a firm base that the standpipe could be pushed into as an additional precaution against it rising up the inside of the hollow stemmed augers. The eventual success achieved in installing the standpipes was mixed. On measuring the open depths of the standpipes, 9 of them had open depths of 5 metres or more, 8 had open depths from 3 to 5 metres, another 5 had open depths less than 3 metres, and 3 were completely blocked. It was fortunate that the majority of open depths were sufficient to record the final drawdowns in the standpipe. A few of the standpipes actually rose up the hollow stems on withdrawal of the augers and consequently had to be cut back. It may therefore be the case that these standpipes were open for their full lengths but were installed shallower than intended.

6.5.7 *Mercia Mudstone Levels And Aquifer Thicknesses Observed During Standpipe Installations*

Table 6.1 shows the thickness of sand and gravel and the level at the top of the Mercia Mudstone bedrock recorded during the standpipe installations. Ground level across the site varied by no more than 0.6m difference between any two measured levels. The greatest aquifer thicknesses of approximately seven metres were observed in the east and north east of the site. It was unfortunate that the chosen location for the well was where the mudstone was at its highest, the aquifer being only 5 - 5.5m thick at this point. This would obviously limit the maximum drawdown which would be achieved during the test to probably no more than 4m.

6.5.8 Estimates Of Important Parameters Calculated Prior To The Pump Test

6.5.8.1 Drawdowns and Settlements

It was important to know if the pump test gave rise to drawdowns or settlements which contrasted significantly with those predicted from state of the art theory used by de-watering contractors. Therefore preliminary estimates of these parameters were made prior to commencement of the pump test. Settlements and drawdowns were estimated using the methods given in CIRIA Report 113 (1986) and are summarised in Table 6.2.

In order to calculate theoretical drawdown and settlement values, an assumption of the bulk permeability for the aquifer had to be made. Chapter Three discusses various studies which investigated this parameter. Tarmac Quarry Products Ltd adopted a value of 15 metres/day (1.74×10^{-4} m/s) based on borehole and laboratory tests (see section 3.2.5.2). This value was adopted for the calculations discussed below.

The drawdown expected was set at 3 metres for the calculations. In the calculation of theoretical settlements, a value for the bearing capacity of the sand and gravel strata had to be assumed. Values representative of sand and gravel strata are given in BS 8004:1986 and summarised by Craig (1989). Maximum and minimum values of 600 kN/m^2 and 200 kN/m^2 respectively were adopted.

The radius of influence was estimated using both the tabulated values and the equation given in Section 2.3. The two estimates were 131 metres and 119 metres respectively.

6.5.8.2 Time To Achieve Maximum Drawdown

It was important to know that the maximum drawdown would be achieved during the pump test in the time available and that a significant time could be spent pumping at "steady state". Using a method given by Roberts (1988), for an assumed permeability of 1.7×10^{-4}m/s and a well penetration of 5.5 metres, the time calculated to achieve the required drawdown was estimated as approximately 8 hours.

6.5.8.3 Expected Flowrate

The flow rate to be expected was estimated using the Dupuit Forchheimer equation given below:

$$Q = \frac{\pi k(h_e^2 - h_w^2)}{\ln\left(\frac{r_e}{r_w}\right)}$$

where assumed values were:

k	=	permeability = 1.7×10^{-4}m/s
h_e	=	height of natural water table above base of sand and gravel = 4.4m
h_w	=	height of water in the well above base of sand and gravel = 1.4m
r_e	=	radius of influence = 230m (see Section 6.1)
r_w	=	radius of well = 0.125m

The flowrate estimated was 1.2 litres per second.

6.6 The Pump Test - Results and Discussion

6.6.1 Conditions At Steady State Pumping

Pumping began at 1.00p.m. on the 27th July 1994. The pumping proceeded 24 hours a day for 17 days. The pump was switched off on the 14th August 1994. A dip meter was used to check the final drawdown within the well and the drawdown within the aquifer 1.5m from the well was measured using standpipe V. The final conditions at steady state during the test are summarised in Figure 6.17. It will be seen that the final drawdown achieved was only 3 meters due to the well being located on the high point in the Mudstone discussed above. The short term drawdown characteristics of standpipe V are shown in Figure 6.18. It can be seen that the final time to maximum drawdown was about six hours. This agreed favourably with the calculation described in section 6.5.8.2.

6.6.2 The Readings From The Standpipes

The water levels recorded for all operating standpipes during the pump test are given in the plots of Figures 6.19 to 6.29. Standpipes A, G, and T became blocked very soon after installation and were not read during the pump test therefore no plots appear for these standpipes. The water levels are all plotted relative to an arbitrary local datum. It will be seen from the plots that standpipe readings were taken for a number of days prior to commencement of the test to establish "at rest" water table levels in the aquifer.

6.6.3 Final Drawdowns Achieved Within Standpipes Located At Equal Distances From The Well.

Tables 6.3 to 6.8 give the maximum drawdowns which occurred within each standpipe, the actual distance from the pumped source and the apparent conductivity recorded at the standpipe location.

It can be seen that the standpipes located at approximately 5 metres and 15 metres from the well were located within materials of very similar conductivities. There was no significant difference in the recorded maximum drawdowns for the standpipes when comparing those at similar distances.

There were four standpipes located at approximately 25 metres from the well. Standpipes U and W were located within materials of apparent conductivities of 43 and 33 millimhos/metre respectively. The respective maximum drawdowns measured for these standpipes were 0.73 and 0.57 metres. It appears therefore that the opposite effect to that expected was observed for these two standpipes. Also, if the conductivities of the material in between standpipe and well are examined for standpipes U and W, it can be seen that the material between well and standpipe U has the greatest conductivity. It was expected that this would also have encouraged a reduced drawdown in standpipe U but this was not the case. Standpipes K and L, although located within materials of significantly different conductivity to standpipe U, exhibited no significant difference in maximum drawdown. In fact, maximum drawdowns recorded were all within 22% of each other for the standpipes at 25 metres from the well.

There were four standpipes located at approximately 35 metres from the well. This time, the standpipes located within the materials of lowest conductivity (standpipes C and H) exhibited the greatest drawdowns of 0.57m and 0.53m respectively. Also, the material in between the pumped well and these two standpipes was of a lower conductivity than that between the well and the other two standpipes (N and Q). Therefore, the standpipe information at 35 metres from the well would appear to comply with what was originally expected. However, the maximum drawdowns recorded were all within 18% of each other.

Six functioning standpipes were located at 60 metres from the well. It can be seen that the minimum drawdown of 0.27m was recorded in standpipe S which was installed in the location of maximum recorded apparent conductivity. The material in between standpipe S and well was also of relatively high apparent conductivity. The material in between pump and standpipe B to the south, which showed the highest drawdown, was of the lowest conductivity for all the standpipes at 60 metres away. However, contrasting to this, standpipes J and R, showed two of the highest and lowest drawdowns respectively. For both, material in between well and standpipe was of relatively low conductivity.

6.6.4 *Final Radii Of Influence In Different Directions From The Pump*

Groundwater level information should be obtained from no fewer than 3 standpipes in a line to define the shape of a cone of depression. Standpipes were installed for the pump test along 8 main lines radiating outward from the pump. Along three of these lines, 3 or more standpipes were installed. Along 4 lines, only 2 standpipes existed but the known water level at the well gave the third data point.

Diagrams of the final drawn down water table were produced for 7 lines radiating outward from the pump and these are presented in Figures 6.30 to 6.36. Figure 6.37 combines the drawdown plots for all directions for ease of comparison. The 7 lines were on directions of:

1. North
2. North West
3. West
4. South West
5. South
6. South East
7. North East

On examination of the diagrams of the drawn down water table it can be seen that the water levels rose very sharply within the first 20 metres away from the pump. Beyond this the slope of the phreatic surface became shallower. Typically, the drawn down water table was no more than 0.5 metres below its original level beyond 60 metres away from the pump.

On examination of the location of the standpipes in relation to the various regions of conductivity, the lines of standpipes to the south, south-east and north-east passed through the regions of highest conductivity and the line west extended through the lowest conductivity zone. It may appear from the combined plot in Figure 6.37 that the line west passing through a region of lower conductivity has given rise to a relatively more "drawn out" cone of depression relative to the other directions. However this is much more likely to be due to the fact that the nearest standpipe to the well along this line was at a larger distance than for the other lines, the plot therefore having been artificially extended. It is considered likely that the cone of depression would have been measured as steeper had a standpipe been placed closer to the well along this line.

The lines south-west, south-east, and north-east of the pump contained most data points and it is therefore these lines that provided the best indications of how the water table reacted within the different zones. It can be seen that only minor variations of the phreatic surface existed within the varying zones. The sharp rise in the phreatic surface close to the well for lines south-west and north-east is caused by the reading from the two close standpipes, F and E. It would be reasonable to expect that a similar sharp rise existed for the other lines, however in the absence of close standpipes only the available data has been plotted.

6.6.5 Permeabilities Calculated For Radial Standpipe Lines.

Permeabilities for the observed water levels along the various standpipe lines were calculated to see if any spatial variation of permeability existed. The measurements taken seemed to imply very similar hydrogeological characteristics across the whole site. This was tested mathematically using the Dupuit-Forcheimer equation given in Section 6.5.8.3 rearranged as given below.

$$k = \frac{Q \ln (r_2 / r_1)}{\Pi (h_2^2 - h_1^2)}$$

Where h_1 and h_2 represent the heads of water recorded above datum at distances of r_1 and r_2 from the pumped source.

Using the equation in conjunction with the data from the different lines of standpipes, a mass permeability value was estimated for each direction away from the pump. Table 6.9

shows the results of the calculations. The calculations summarised in the table assumed that the flowrates for each direction were the same. Given that the radii of influence were very similar in each direction, this was a reasonable assumption. It can be seen that all the calculated permeabilities are of the same order of magnitude varying from $3x10^{-4}$ m/s for standpipes P and Q on the south-east line to $7.9x10^{-4}$ m/s for standpipes E and D on the north-east line.

The original hypothesis was that material of higher conductivity would exhibit less drawdown and be generally less permeable than the lower conductivity regions. The lines of standpipes which extended into the materials of higher conductivity were south, south-east and north-east. For the south-east line the range of calculated permeabilities was from $3.0x10^{-4}$m/s (for standpipes P and Q) to $4.6x10^{-4}$m/s (for standpipes Q and R). For the north-east line the range of calculated permeabilities was from $4x10^{-4}$m/s (for combinations of standpipes of D and C, D and B, C and B) to $7.9x10^{-4}$m/s (for standpipes E and D). Of all the pairs the two combinations of standpipes which were in the materials of highest conductivity were C and B, and Q and R, located at regions of conductivity of 33 and 40, and 41 and 52 millimhos/m respectively. Their permeabilities came towards the middle of the range of all calculated values.

The lines of standpipes which extended into the materials of lowest conductivity were south and south-west. For the south-west line the range of calculated permeabilities was from $4.2x10^{-4}$ m/s (for standpipes H and I) and $5.1x10^{-4}$ m/s (for standpipes F and H). For the south line, the calculated permeability was $4x10^{-4}$m/s. The combination of standpipes which were in the material of lowest conductivities was H and I, located in material of conductivities of 28 and 26 millimhos/m respectively.

No significant variation exists in the calculated permeability values for the regions of variable conductivity.

6.6.6 Examination Of Well Performance

During the pump test, the maximum flow rate measured was 3.9 l/s. It is believed that this flowrate was the maximum that could have been achieved for the following reasons.

- The flow at the outlet end of the header pipe was not steady but "pulsed". This implies that the pump was sucking air from inside the well for some of the time and therefore had "spare" capacity. Groundwater was therefore entering the well at the maximum achievable rate for the well size used.

- Roberts (1988) states that flow to a well in the unconfined state is not always governed by the pump rate. More often than not, the well itself or the aquifer in the immediate vicinity of the well appears to have a finite maximum capacity. Within such an aquifer the limiting value of hydraulic gradient which can exist is 1. An examination of the water levels in the standpipes closest to the well shows that the hydraulic gradient close to the well was very close to unity. This is further evidence that the aquifer was being de-watered at the maximum possible rate.

6.6.7 Anomalies In The Graphs Of Drawdown With Time For The Standpipes.

Abrupt rises and falls in the water levels in most standpipes are evident on the 31st July and the 5th August. The reasons for the anomalies and their significance are now described. The shapes of the graphs of standpipe water levels are also discussed below. Important occurrences during the duration of the pump test which may have significantly affected the level of water in the standpipes were (in chronological order);

1. Day 1 (19/7/94) - Pump switched on.
2. Days 7/8 (26 & 27/7/94) - approximately 8mm rain in 24 hour period.
3. Day 11 (30/7/94) - Pump switched off for 2 hours from 10a.m. to 12p.m.
4. Days 11/12 (30 & 31/7/94) - approximately 10mm rain in 24 hour period.
5. Day 12 (31/7/94) - atmospheric pressure low.
6. Day 13 (1/8/94) - pump revs increased to maximise pump rate.

7. Days 14/15 (2 & 3/8/94) - approximately 12mm rain in 24 hour period.
8. Day 15 (3/8/94) - pump suction hose realigned on exit from well in an attempt to increase flows/efficiency.

Therefore, the three things that could have significantly affected water levels in the standpipes as well as steady pumping from the well were:

- Rainfall variations.
- Pressure variations.
- Pump performance variations.

Meteorological information supplied by the Met Office for the period of the pump test is shown in Table 6.10.

It seemed reasonable to assume that rainfall would cause a recharge of groundwater to the aquifer and therefore a rise in the level of water in the standpipes. An indication can be obtained of how much standpipe levels are affected by rainfall and pressure changes if standpipe levels are examined for the days before the pump was installed on 27th July. Generally, the trend prior to pumping was a slightly increasing water level interrupted by a small downward anomaly on the 24th July. Examination of the pressure and rainfall data prior to pumping shows that during the period of generally higher pressure from 21st to 23rd July the water levels within the standpipes were measurably lower than the water levels during days 5 to 7. The range of variation of level in the standpipes was 70 to 100mm during a period of pressure variation of approximately 11.5 millibars. Atmospheric pressure variation therefore caused measurable changes in standpipe levels. This may explain why after the theoretical time to achieve the drawdown had elapsed, the water levels within the standpipes showed a generally downward trend. It can be seen that on the 31st July, the atmospheric pressure began to increase from approximately 1006 millibars reaching 1025 millibars on the 7th August. It is during this period that all water levels show a lowering trend. Conversely, after the 8th August, during a period of falling atmospheric pressure, all standpipes showed a slight rise in water level. Even so, it can also be seen that any change in standpipe water levels caused by atmospheric changes did not affect the clear identification of the effects of pumping on water levels.

There is a slight upward anomaly in all the standpipe graphs on 31st July. The size of the jump varies from approximately 240 millimetres and 350 millimetres in standpipes E and V respectively (which are the two closest standpipes to the pump) to almost undetectable in some of the furthest standpipes such as standpipe J. This was caused by the switching off of the pump for 2 hours. These anomalies in the standpipe graphs indicate that the aquifer was able to respond very quickly to any change in pumping conditions.

Another trend, this time indicating a drop in water level is evident half way through 5th August. At first glance, this may appear confusing given this was after a fairly heavy rainfall event and also coincided with a temporary atmospheric pressure decrease. The most likely cause of these anomalies is that the suction hose was lowered slightly down the well in an attempt to increase drawdown and pump efficiency. It appears that this is exactly what was achieved because a flow rate measurement the following morning showed an increase from the previous 3.3 l/s to 3.9 l/s. This could not be sustained however because the suction hose became so close to the bottom of the well that it sucked itself onto the bottom. The hose was therefore raised slightly to avoid this and the water levels were observed to rise back to their previous levels accordingly.

The rainfall events appear not to have affected standpipe levels significantly.

6.6.8 Settlements Of The Temporary Bench Marks During The Test

Temporary bench marks were installed in the pump test field and surveyed at regular intervals during the test using precise levelling techniques. The bench marks were installed in the locations shown in Figure 6.15. It will be seen that, apart from at a few locations, bench marks were placed adjacent to standpipes. A total of 25 bench marks were installed into the test field. Each bench mark consisted of a 1 metre length of 68 millimetre diameter U.P.V.C tubing filled with concrete into which was fixed a 0.5m length of steel reinforcement bar with a domed top to receive a levelling staff. Backfill was placed and compacted around each bench mark in an attempt to prevent lateral movement but not so tightly as to provide a firm connection between any topsoil material that may have suffered movements due to rainfall or thermal effects. A temporary bench mark can be seen in Figure 6.16 next to the standpipe.

Plots of movements of the bench marks are presented in Figures 6.38 to 6.62. It will be seen that most of the temporary bench marks show an initial settlement of the order of 1 or 2mm between 27th July and 30th July after the initial drawdown had taken place followed by a cease in the settlement after the 31st July, in turn followed by a rise in level after the 3rd August. The cease in settlement and the rise in level both occurred after significant rainfall events. It is possible that topsoil material swelled after these rainfall events and carried the bench marks up. Movement of the main site reference bench mark (a concrete culvert on one of the streams surrounding the test field) did not occur during the monitoring period and so cannot be the cause of the observed movements. This reference was surveyed separately at regular intervals during the test relative to a road 400m away and no significant movement was observed.

6.7 Conclusions

It has been described how a full scale pumping out test was carried out in the floodplain at Hoveringham in an area of varying geophysical response. The test area was instrumented with standpipes and temporary bench marks to look for evidence of spatial variation of permeability, varying radii of influence and progressive settlement during pumping possibly caused by suffosion.

The drawdowns and radii of influence measured in areas of varying geophysical response were found not to differ in a manner significant enough to suggest that material within the aquifer as a whole would show variable drawdown behaviour if subjected to long term full scale groundwater disturbance such as would occur during quarry de-watering. Permeabilities calculated for lines radiating at different directions from the pumped source through regions of varying geophysical response were of the same order of magnitude.

Settlement monitoring during the test was inconclusive as to true ground movements. It is thought possible that topsoil swelling caused the bench marks to rise slightly. However the results do show that no large scale settlements were measured on the site during the test. This implies no large scale settlement due to suffosion or effective stress increase occurred.

Standpipe reference	Ground level* at standpipe (m)	Thickness of sand and gravel (m)	Level at top of Mercia Mudstone (m)
A	18.489	5.7	no data
B	18.415	7.1	11.3
C	18.619	6.9	11.7
D	18.677	6.9	11.8
E	18.718	5.0	13.7
F	18.714	5.5	13.2
G	18.684	no data	no data
H	18.647	5.5	13.1
I	18.704	5.5	13.2
J	18.704	6.0	12.7
K	18.645	no data	no data
L	18.653	no data	no data
M	18.349	6.4	11.9
N	18.565	6.3	12.2
O	18.732	5.8	12.9
P	18.626	5.9	12.7
Q	18.492	5.5	13.0
R	18.229	5.0	13.2
S	18.126	5.4	12.7
T	18.350	no data	no data
U	18.485	no data	no data
V	18.722	6.5	12.2
W	18.704	6.5	12.2
X	18.694	no data	no data
Y	18.657	no data	no data

* *All levels are relative to an arbitrary local datum.*

Table 6.1

Thickness of unconfined aquifer and levels at the top of the Mercia Mudstone observed during standpipe installations.

Radial distance from well (m)	Estimated Drawdown (m)	Maximum Estimated Settlement (mm)	Minimum Estimated Settlement (mm)
5	2.28	5.6	1.9
15	1.35	3.3	1.1
25	1.17	2.9	1.0
35	0.93	2.3	0.8
50	0.63	1.5	0.5
60	0.54	1.3	0.4

Table 6.2

Preliminary Estimates Of Drawdowns And Settlements At Various Distances From The Pumped Well.

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
E	5.74	17.63	16.44	1.19	30
F	4.45	17.63	16.43	1.20	28

Table 6.3

Standpipes At Approximately 5m From Pump

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
D	14.04	17.63	16.70	0.93	30
G	blocked	-	-	-	-
O	12.34	17.65	16.74	0.91	32
P	13.69	17.62	16.68	0.94	33

Table 6.4

Standpipes At Approximately 15m From Pump

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
K	19.87	17.61	16.90	0.71	30
L	23.74	17.66	16.99	0.67	34
U	26.02	17.60	16.87	0.73	43
W	24.09	17.61	17.04	0.57	33

Table 6.5

Standpipes At Approximately 25m From Pump

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
C	36.51	17.62	17.05	0.57	33
H	33.12	17.64	17.11	0.53	28
N	35.99	17.70	17.23	0.47	40
Q	37.31	17.52	17.05	0.47	41

Table 6.6

Standpipes At Approximately 35m From Pump

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
M	48.03	17.69	17.33	0.36	36
X	49.37	17.70	17.37	0.33	40

Table 6.7

Standpipes At Approximately 50m From Pump

Standpipe	Actual distance from well (m)	Maximum water level (m)	Minimum water level (m)	Maximum drawdown (m)	Conductivity at location (mmho's)
B	60.59	17.62	17.26	0.36	40
I	60.84	17.64	17.31	0.33	26
J	56.82	17.58	17.22	0.36	33
R	58.91	17.42	17.13	0.29	52
S	59.71	17.40	17.13	0.27	68
T	blocked				54
Y	57.24	17.55	17.26	0.29	26

Table 6.8

Standpipes Located At Approximately 60m From Pump

Line	Standpipe combination	Respective distances from well (m)	Respective water levels (m)	Calculated permeabilities (m/s)
N/W	O & N	15 & 35	3.49 , 3.93	$3.2x10^{-4}$
West	W & X	25 & 50	3.83 , 4.07	$4.5x10^{-4}$
S/W	F & H	5 & 35	3.20 , 3.87	$5.1x10^{-4}$
	F & I	5 & 60	3.20 , 4.07	$4.9x10^{-4}$
	H & I	35 & 60	3.87 , 4.07	$4.2x10^{-4}$
South	K & J	25 & 60	3.69 , 4.04	$4.0x10^{-4}$
S/E	P & Q	15 & 35	3.46 , 3.93	$3.0x10^{-4}$
	P & R	15 & 60	3.46 , 4.11	$3.5x10^{-4}$
	Q & R	35 & 60	3.93 , 4.11	$4.6x10^{-4}$
N/E	E & D	5 & 15	3.21 , 3.47	$7.9x10^{-4}$
	E & C	5 & 35	3.21 , 3.83	$5.5x10^{-4}$
	E & B	5 & 60	3.21 , 4.04	$5.1x10^{-4}$
	D & C	15 & 35	3.47 , 3.83	$4.0x10^{-4}$
	D & B	15 & 60	3.47 , 4.04	$4.0x10^{-4}$
	C & B	35 & 60	3.83 , 4.04	$4.0x10^{-4}$

Table 6.9

Permeabilities Calculated For Different Lines And Combinations Of Standpipes.

Date (1994)	Night min temp (C)	Day max temp (C)	Rainfall (mm)	Pressure at 0900h (mb)	Pressure at 1500h (mb)
20 July	9.6	27.1	trace	1020.7	1018.5
21 July	13.3	26.6	trace	1017.8	1017.0
22 July	15.2	25.1	0.0	1021.7	1021.6
23 July	12.7	25.2	trace	1022.8	1019.7
24 July	14.8	25.3	0.2	1014.9	1011.6
25 July	13.2	27.8	trace	1013.0	1011.2
26 July	15.9	21.7	6.4	1012.8	1013.8
27 July	16.8	21.8	1.4	1016.5	1017.4
28 July	11.6	21.5	0.0	1018.5	1018.3
29 July	10.6	25.1	0.0	1018.1	1015.2
30 July	13.7	28.1	0.8	1012.1	1008.0
31 July	17.1	25.9	9.6	1007.7	1006.4
1 Aug	13.2	23.9	0.0	1010.0	1009.6
2 Aug	12.2	21.5	3.2	1015.5	1016.0
3 Aug	16.9	25.6	8.8	1018.5	1016.7
4 Aug	16.5	23.5	0.4	1014.0	1015.1
5 Aug	16.1	24	0.0	1019.1	1019.4
6 Aug	12.3	22.5	0.0	1024.8	1023.7
7 Aug	12.7	19.7	0.0	1025.2	1024.0
8 Aug	12.6	21.8	0.0	1023.9	1022.1
9 Aug	10.1	19.1	0.0	1020.5	1016.2
10 Aug	13.3	17.5	0.4	1009.4	1008.8
11 Aug	13.6	18.9	trace	1008.3	1010.7
12 Aug	9.9	17.4	0.0	1015.9	1016.2
13 Aug	9.1	19.8	0.0	1022.0	1021.8
14 Aug	7.2	18.1	0.0	1022.8	1022.4
15 Aug	9.5	22.1	0.0	1025.3	1022.8

Table 6.10

Meteorological Data For The Pump Test Period

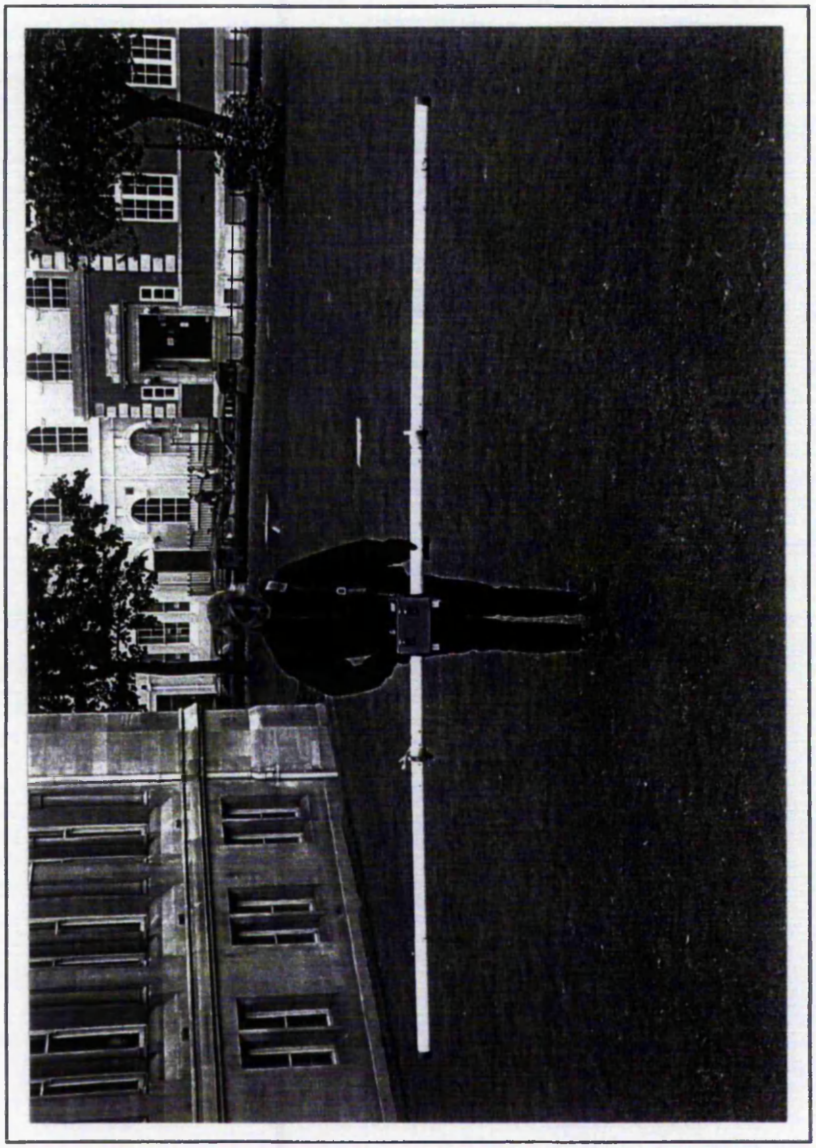


Figure 6.1

The EM 31 Ground Conductivity Meter

Figure 6.2

An EM 34/3 Ground Conductivity Meter Being Used In The Horizontal Co-planar Mode

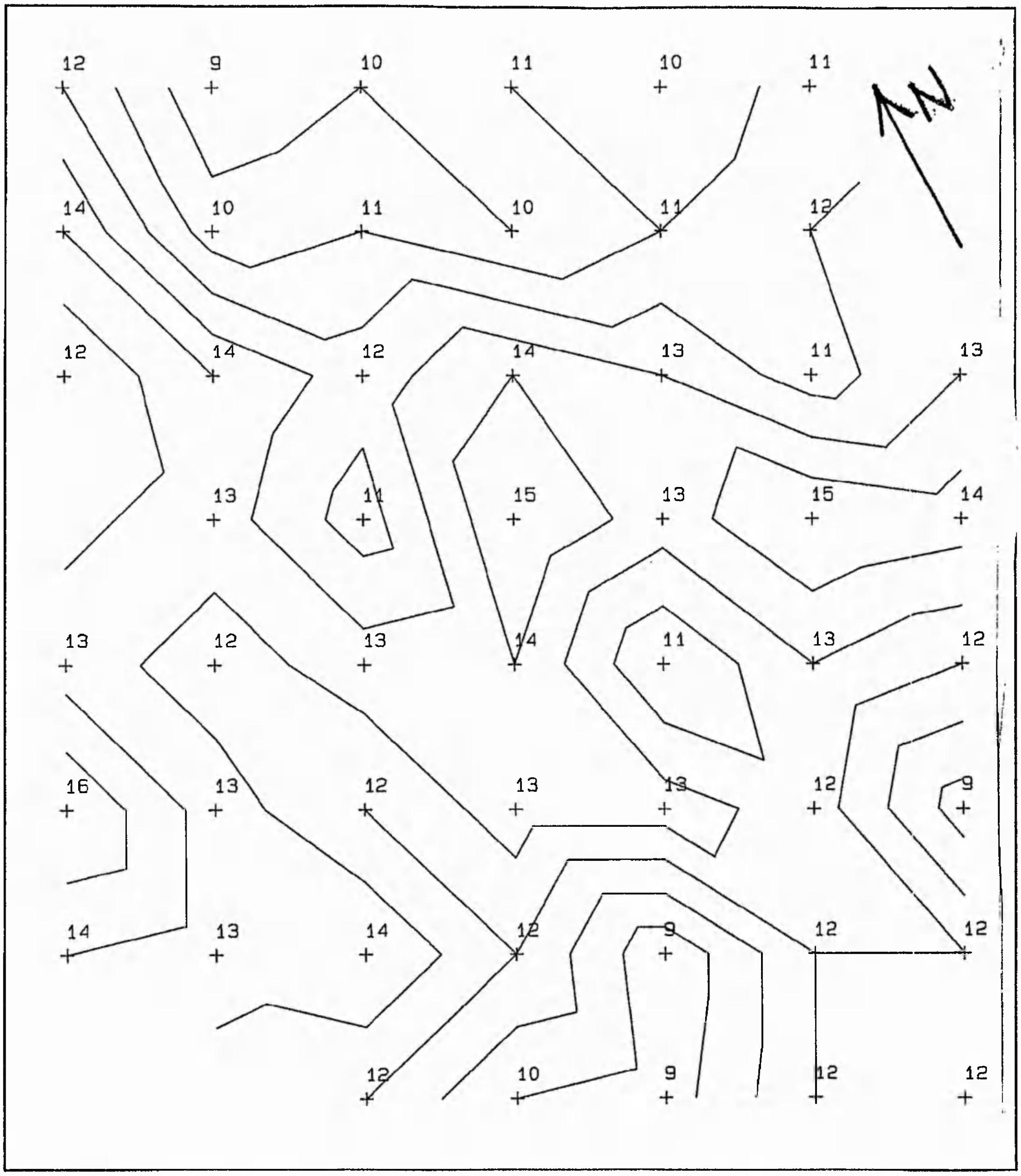
Figure 6.3

An EM 34/3 Ground Conductivity Meter Being Used In

The Vertical Co-planar Mode

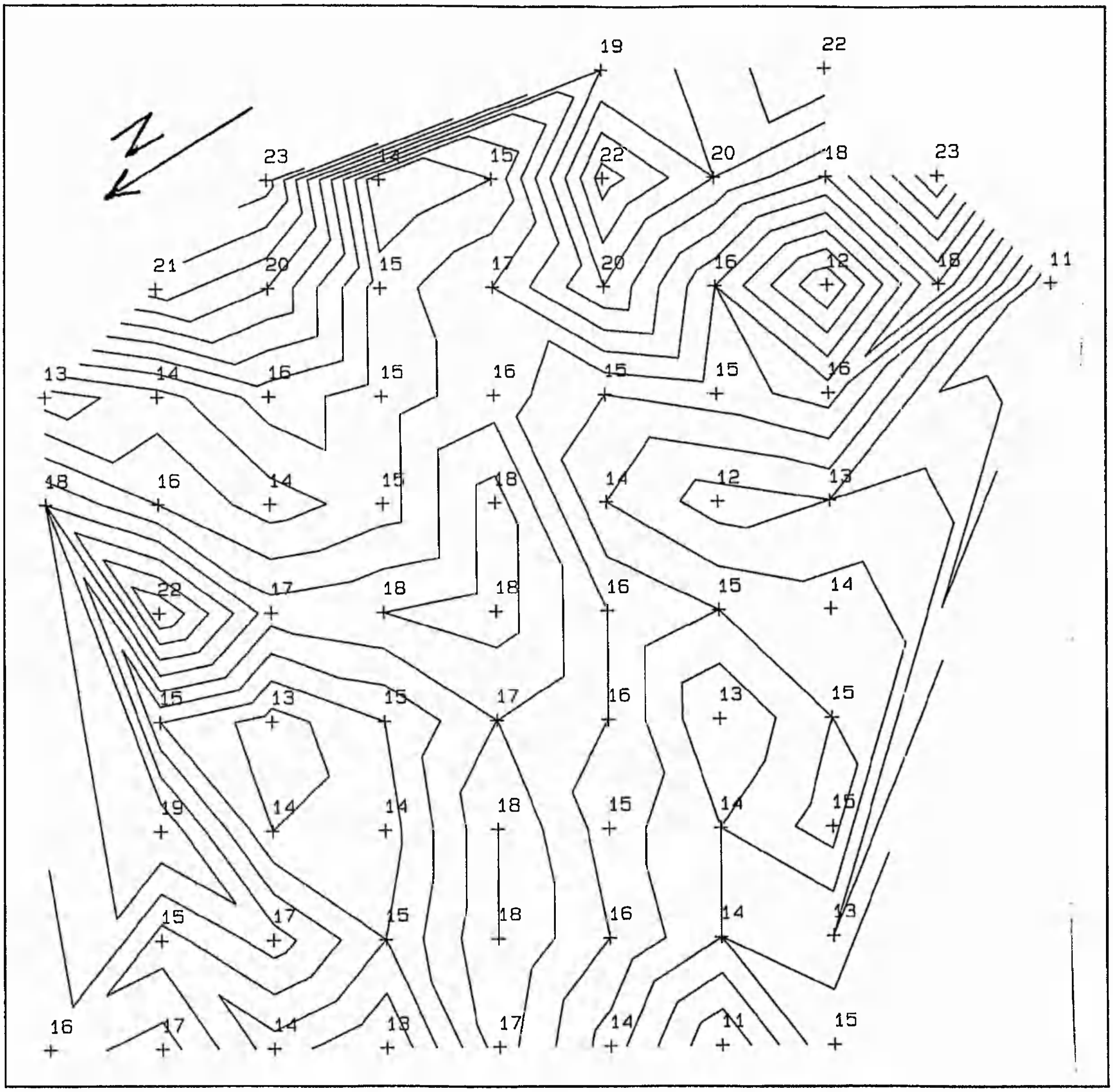
FIGURE 6.4 IS AN OVERSIZE PLAN

LOCATED AT THE REAR OF THE THESIS



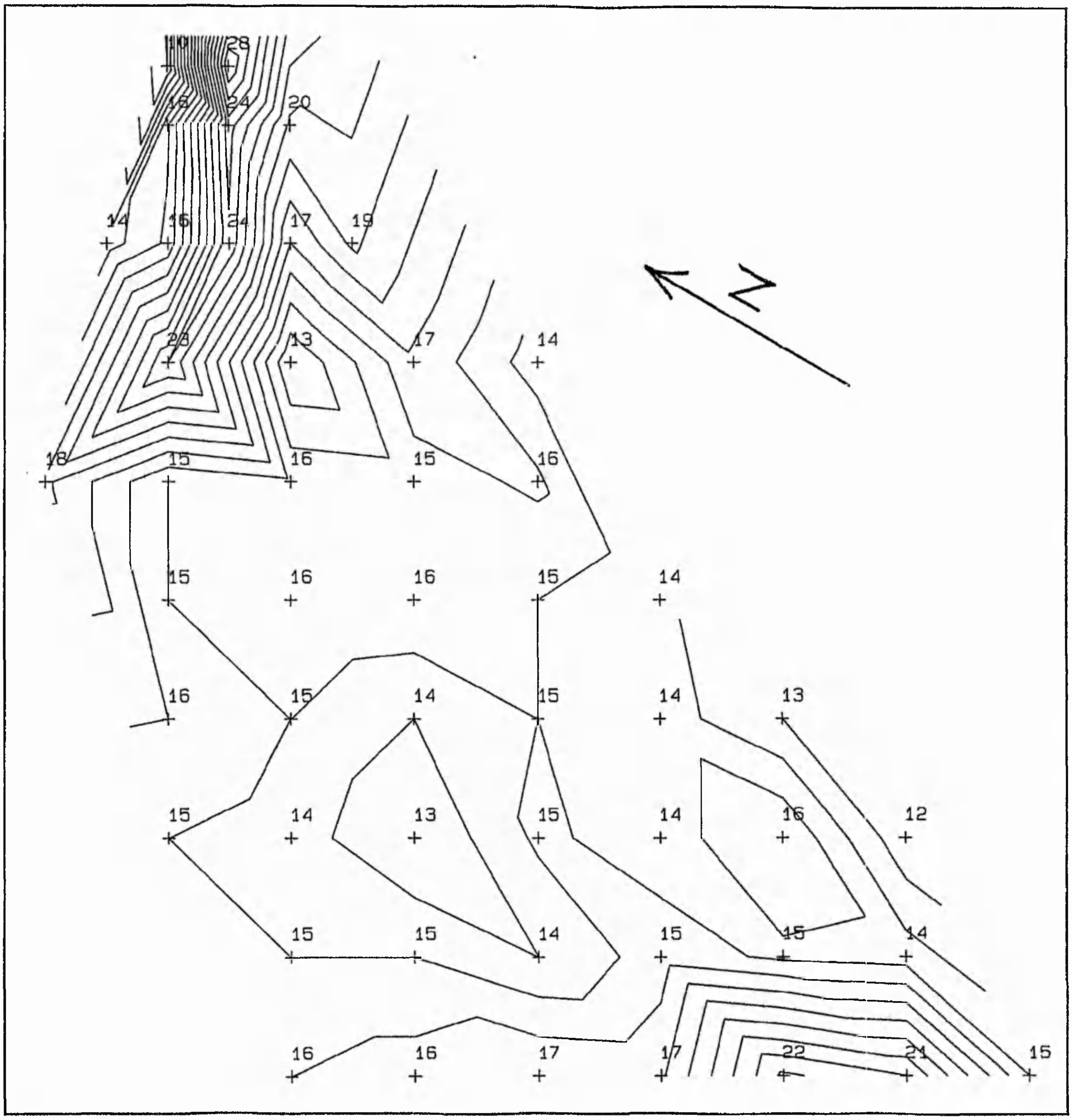
N.B. The plotting package used has caused slight inaccuracies in contour positions but the resulting approximate contours were still valuable for interpretation purposes.

Figure 6.5
Spot Readings And Conductivity Contours At 1 Millimho Intervals For Field 1



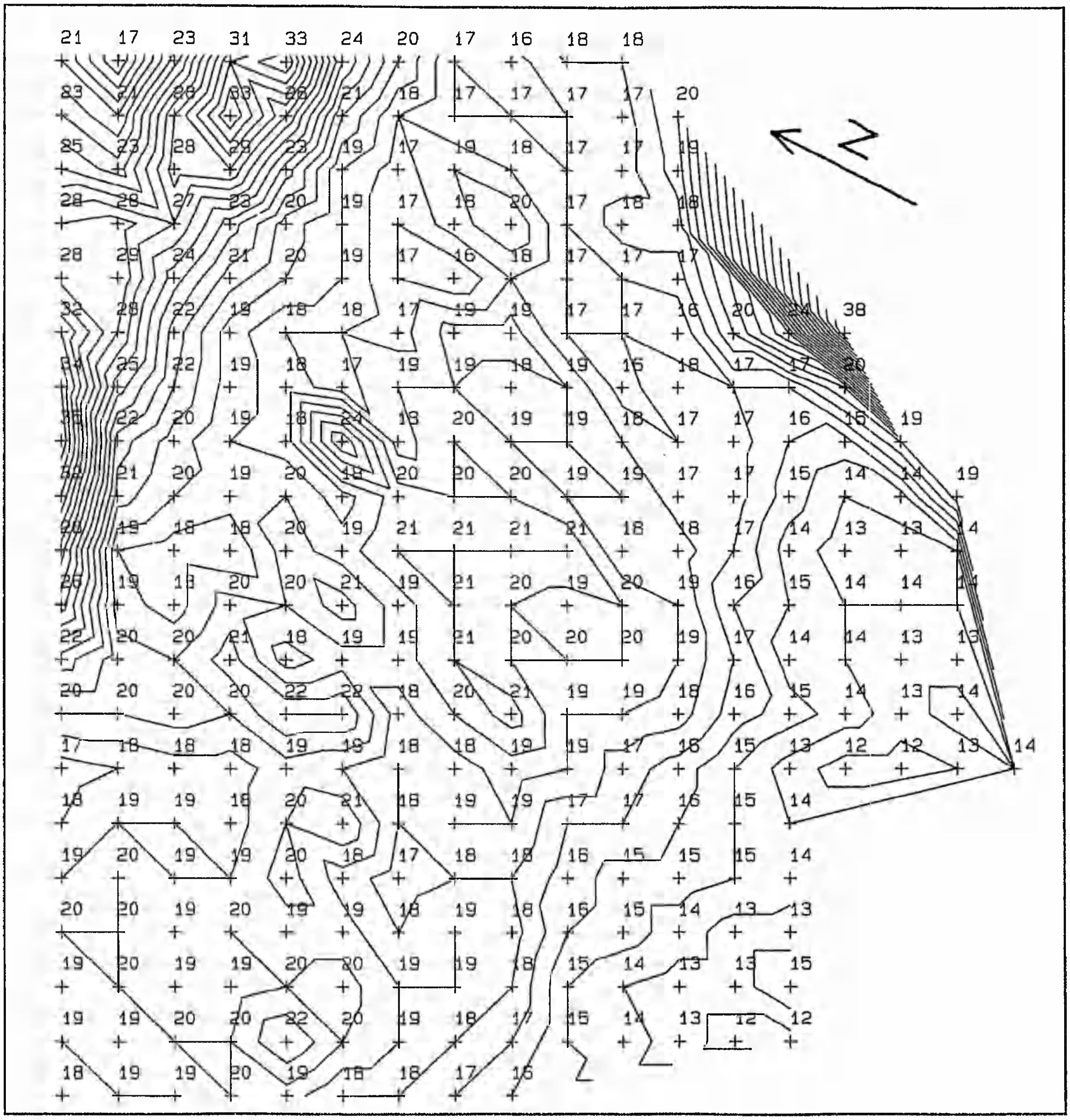
N.B. The plotting package used has caused slight inaccuracies in contour positions but the resulting approximate contours were still valuable for interpretation purposes.

Figure 6.6
Spot Readings And Conductivity Contours At 1 Millimho Intervals For Field 2



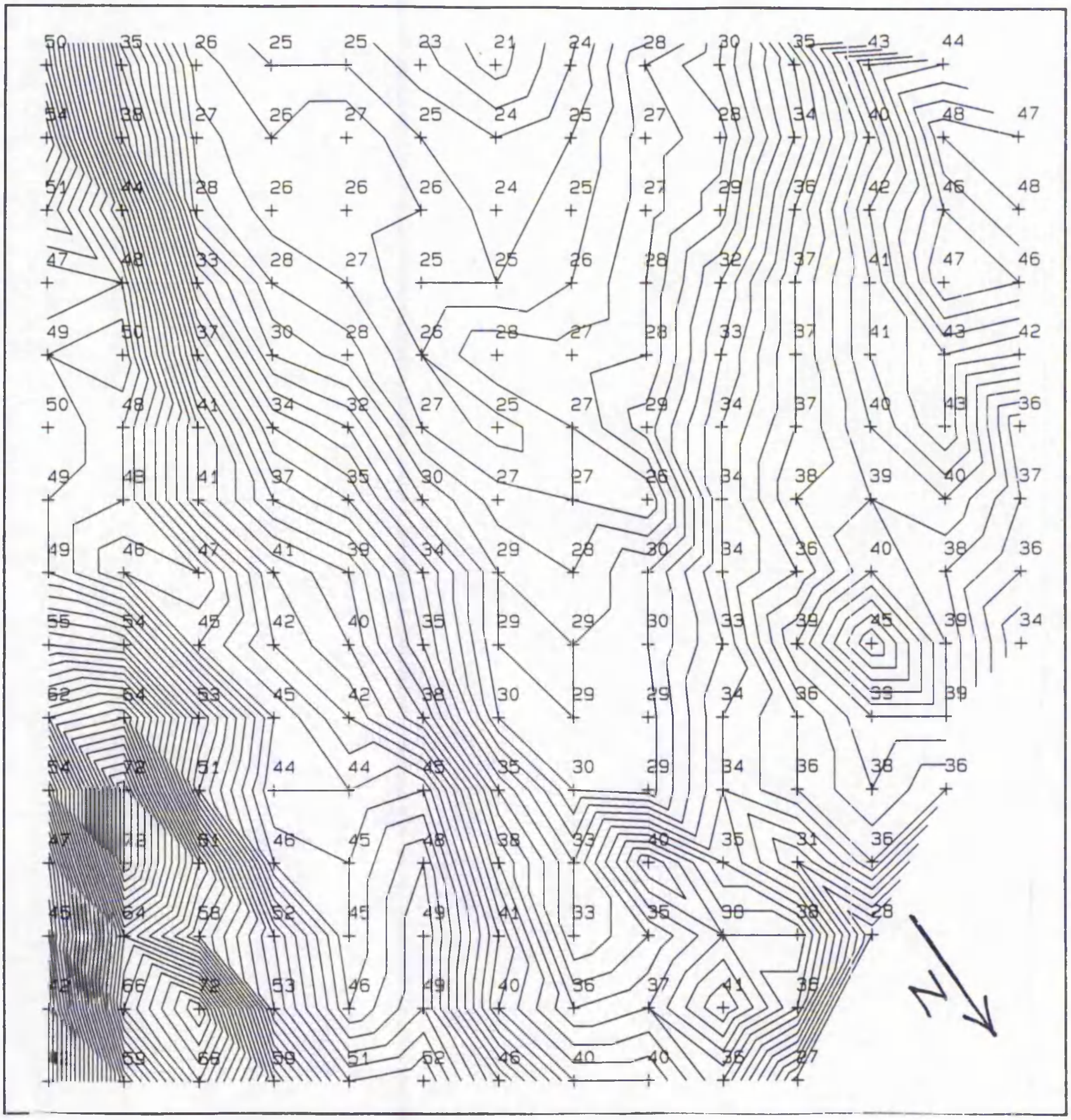
N.B. The plotting package used has caused slight inaccuracies in contour positions but the resulting approximate contours were still valuable for interpretation purposes.

Figure 6.7
Spot Readings And Conductivity Contours At 1 Millimho Intervals For Field 3



N.B. The plotting package used has caused slight inaccuracies in contour positions but the resulting approximate contours were still valuable for interpretation purposes.

Figure 6.8
Spot Readings And Conductivity Contours At 1 Millimho Intervals For Field 4



N.B. The plotting package used has caused slight inaccuracies in contour positions but the resulting approximate contours were still valuable for interpretation purposes.

Figure 6.9

Spot Readings And Conductivity Contours At 1 Millimho Intervals For Field 5

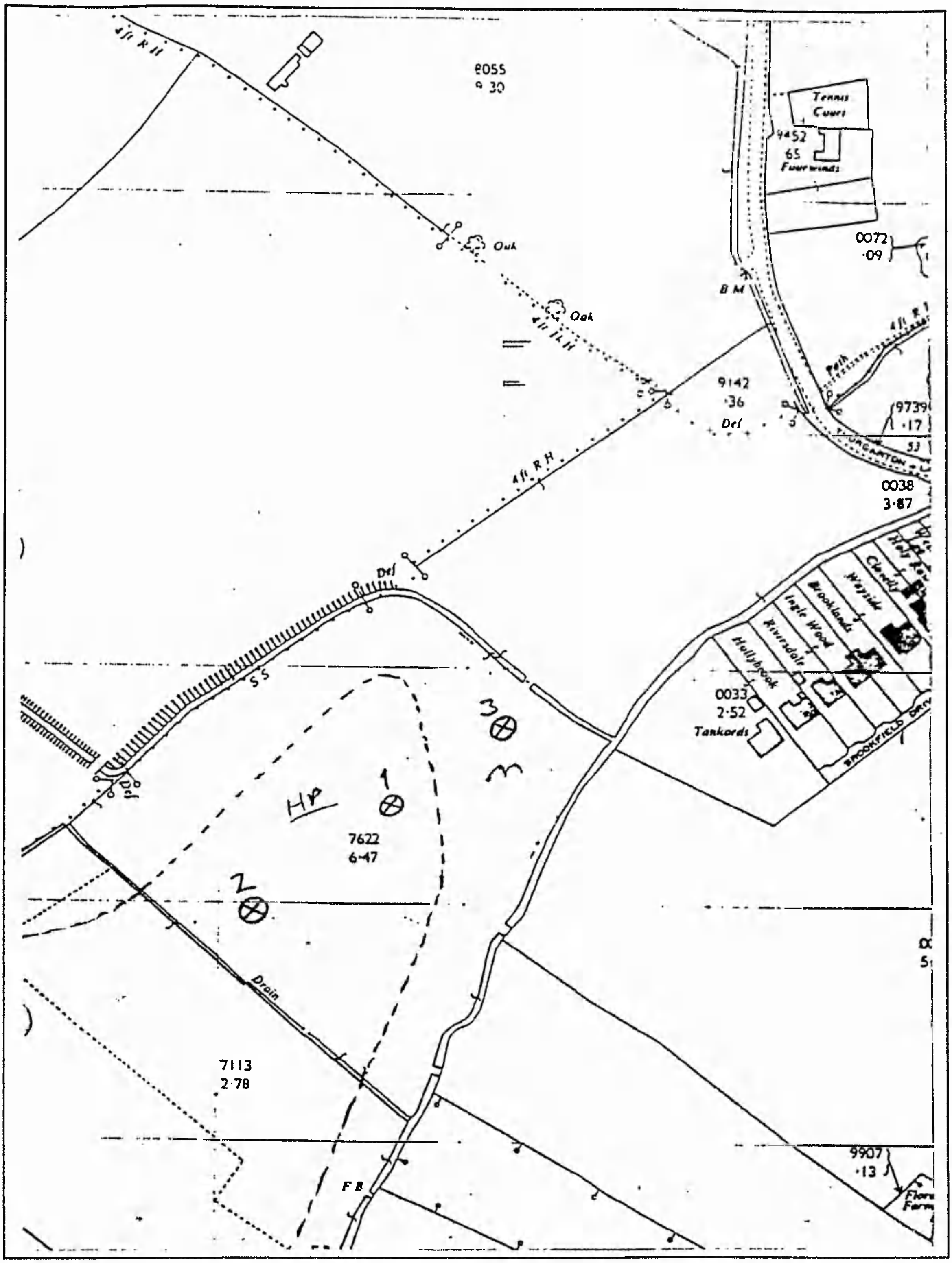


Figure 6.10

Position Of The Three Preliminary Hand Augered Holes For The Pump Test

PROCEDURES TO BE CARRIED OUT DURING A TYPICAL PUMPING OUT TEST

Objectives and initial appraisal

- *Objective*
- *Study of all available information*

Planning

- *Pre-test Planning*
- *Determine likely discharge*
- *Select Equipment*

Observations

- *Pre-test observations*
- *Observations during tests*
- *Post test observations*

Supervision

- *Test supervision*

Tests

- *Equipment test*
- *Step test*
- *Constant discharge or constant drawdown test*
- *Recovery test*
- *Special tests*

Conclusion

- *Presentation of data*
- *Analysis and interpretation (outside scope of the code)*

Figure 6.11

Typical Pumping Test Procedure

(after British Standards Institution Code of Practice 6316 : 1992 "Code of Practice for Test Pumping of Water Wells")

Figure 6.12

The Pump Used During The Test And The Bales Used For Noise Attenuation

Figure 6.13

Long View Across The Pump Test Field Showing Pump, Bales And Hoveringham Village In The Background

Depth Range Below Ground Level (m)	Strata Description
0.0 - 0.45	Soft Brown Fine to Coarse Slightly Clayey Sand
0.45 - 1.2	Soft Light Brown Silty Sand
1.2 - 1.9 ------------Groundwater------------	Purple Grey Silty Sand (Groundwater Level 1.4 m.b.g.l)
1.9 - 2.5	Very Gravelly Fine Sand
2.5 - 5.5	Fine to Medium Gravels
5.5 - 7.0	Reddish Brown Grey Mercia Mudstone

Figure 6.14

The Borehole Log From The Wellhole

Figure 6.15

The Standpipe And Temporary Bench Mark Array Used During The Pump Test

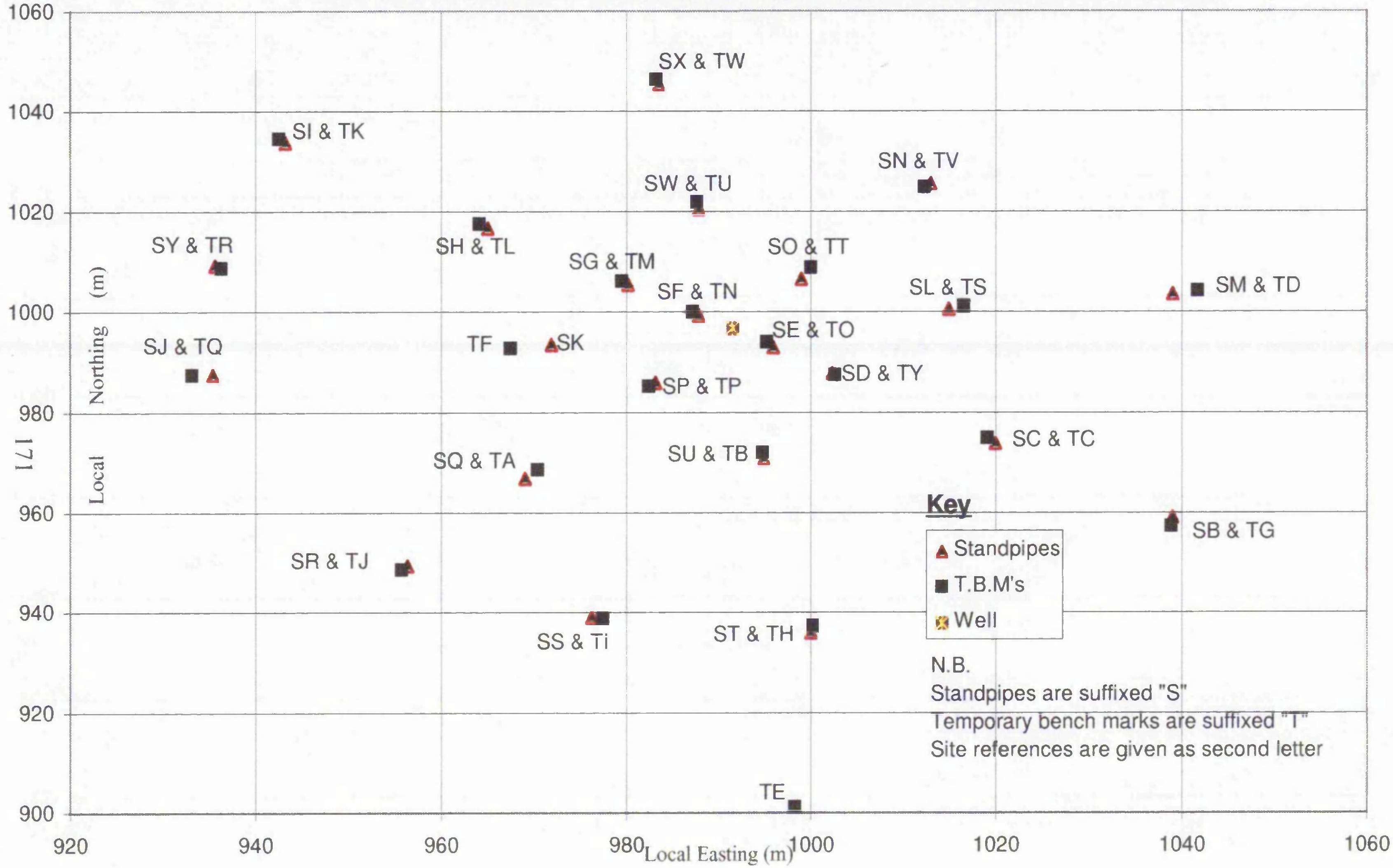


Figure 6.16

A Standpipe Being Read With A Temporary Bench Mark In The Foreground

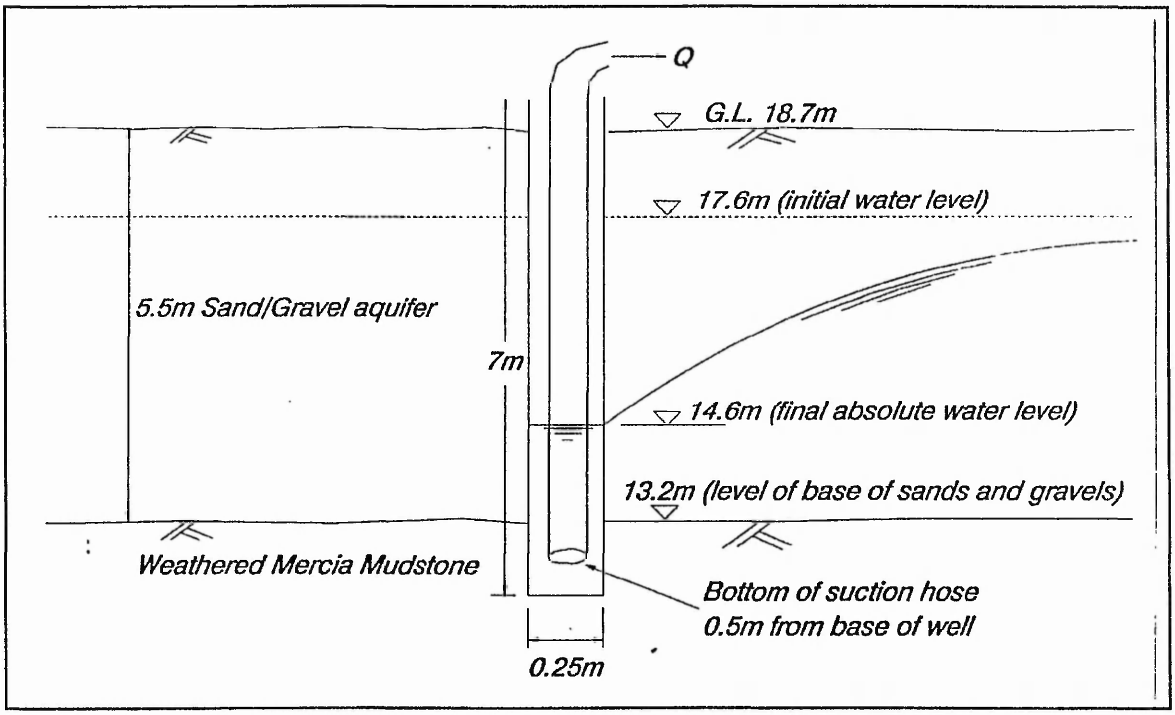


Figure 6.17

Conditions At Steady State During The Pump Test

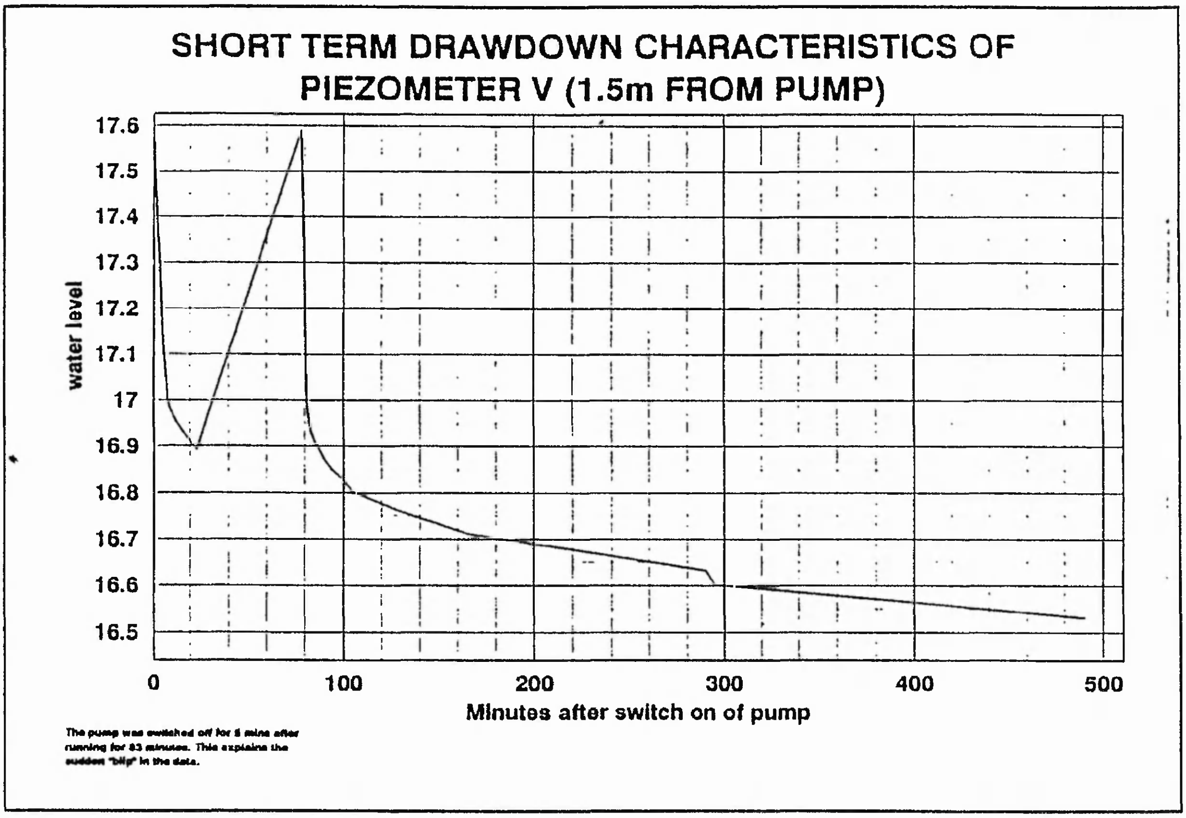
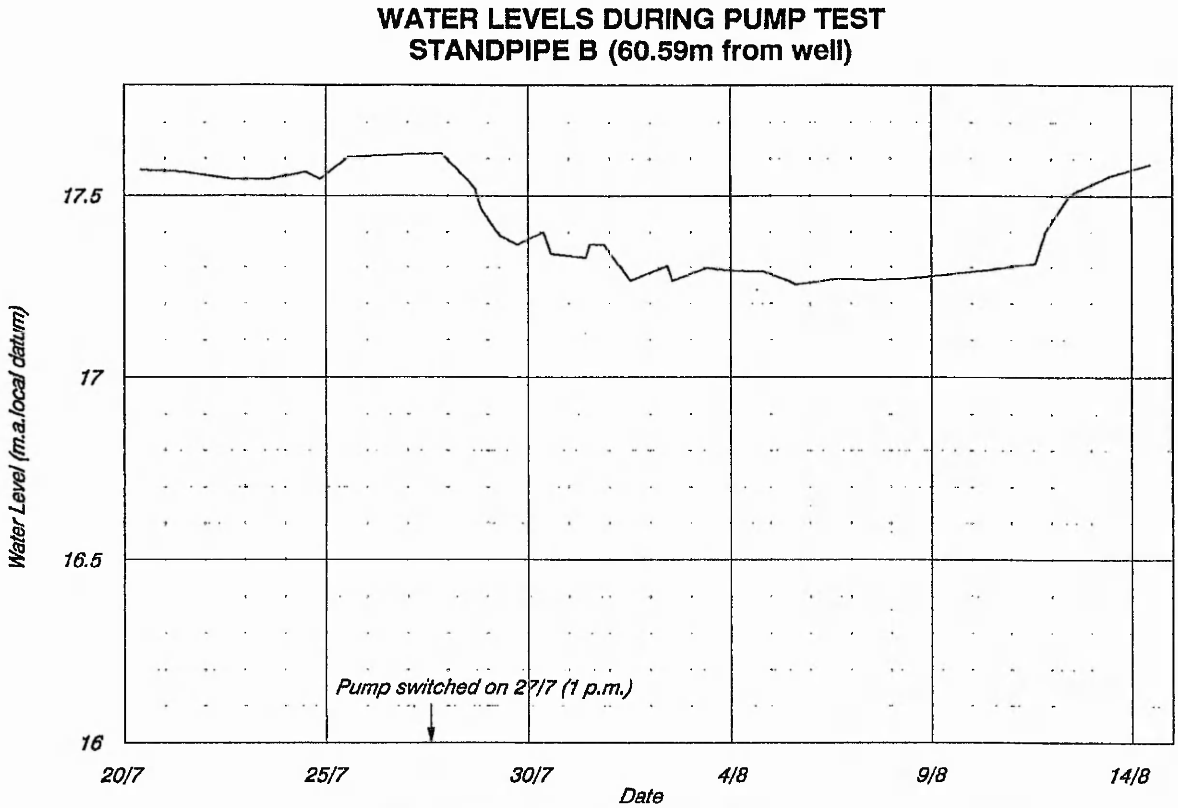
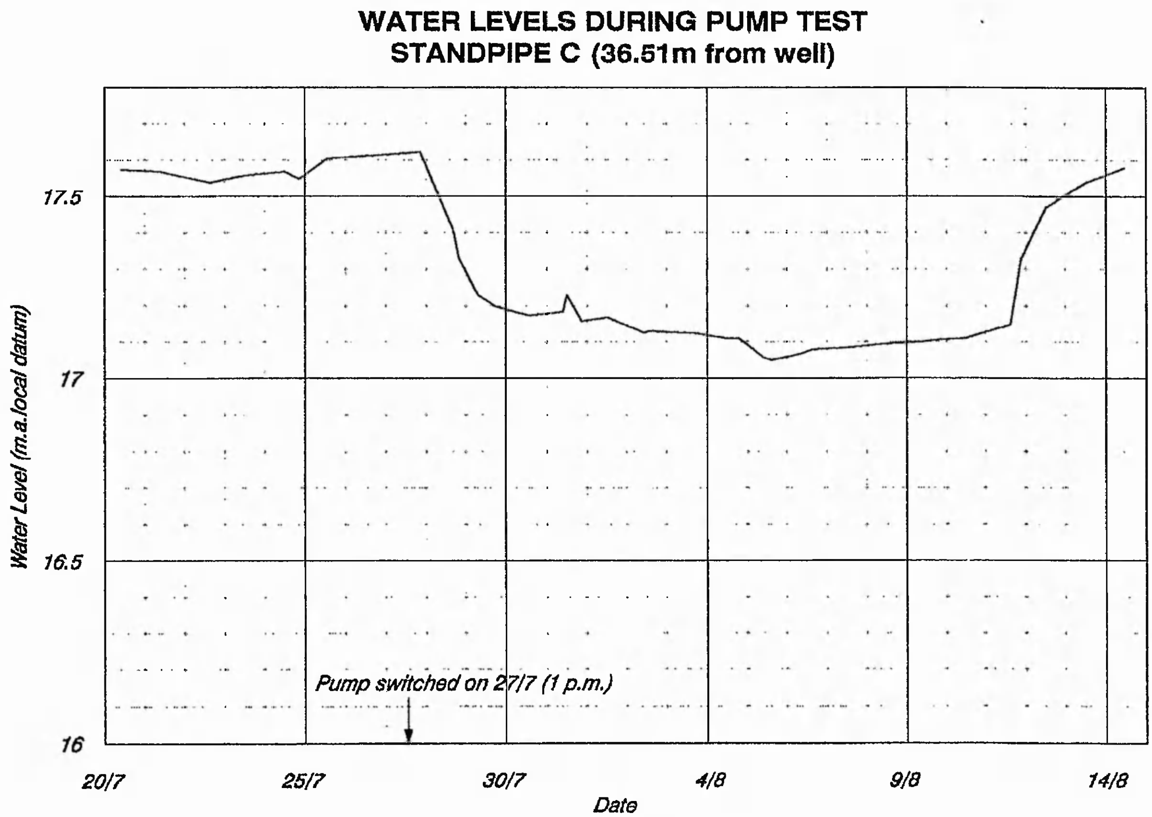

SHORT TERM DRAWDOWN CHARACTERISTICS OF PIEZOMETER V (1.5m FROM PUMP)
water level
17.6
17.5
17.4
17.3
17.2
17.1
17
16.9
16.8
16.7
16.6
16.5
0
100
200
300
400
500
Minutes after switch on of pump
The pump was switched off for 5 mins after running for 83 minutes. This explains the sudden "blip" in the data.

Figure 6.18

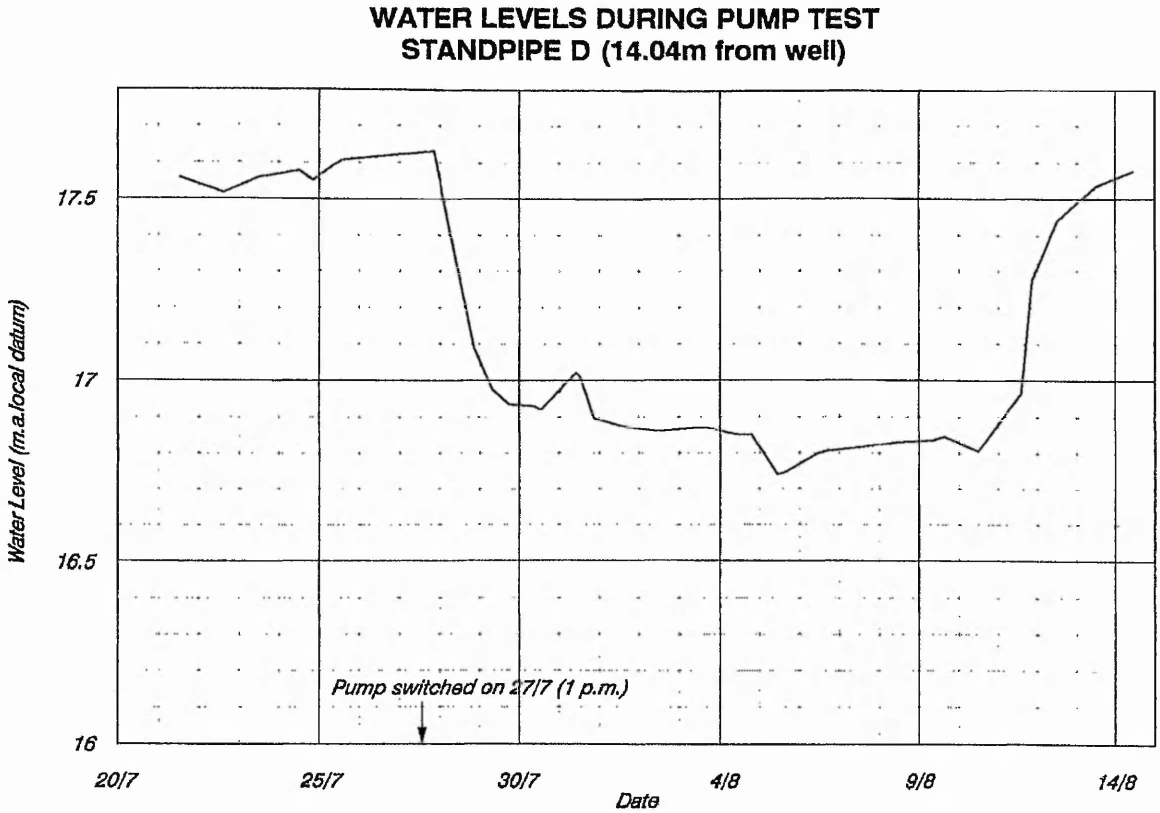


WATER LEVELS DURING PUMP TEST
STANDPIPE B (60.59m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date



WATER LEVELS DURING PUMP TEST
STANDPIPE C (36.51m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.19


WATER LEVELS DURING PUMP TEST
STANDPIPE D (14.04m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

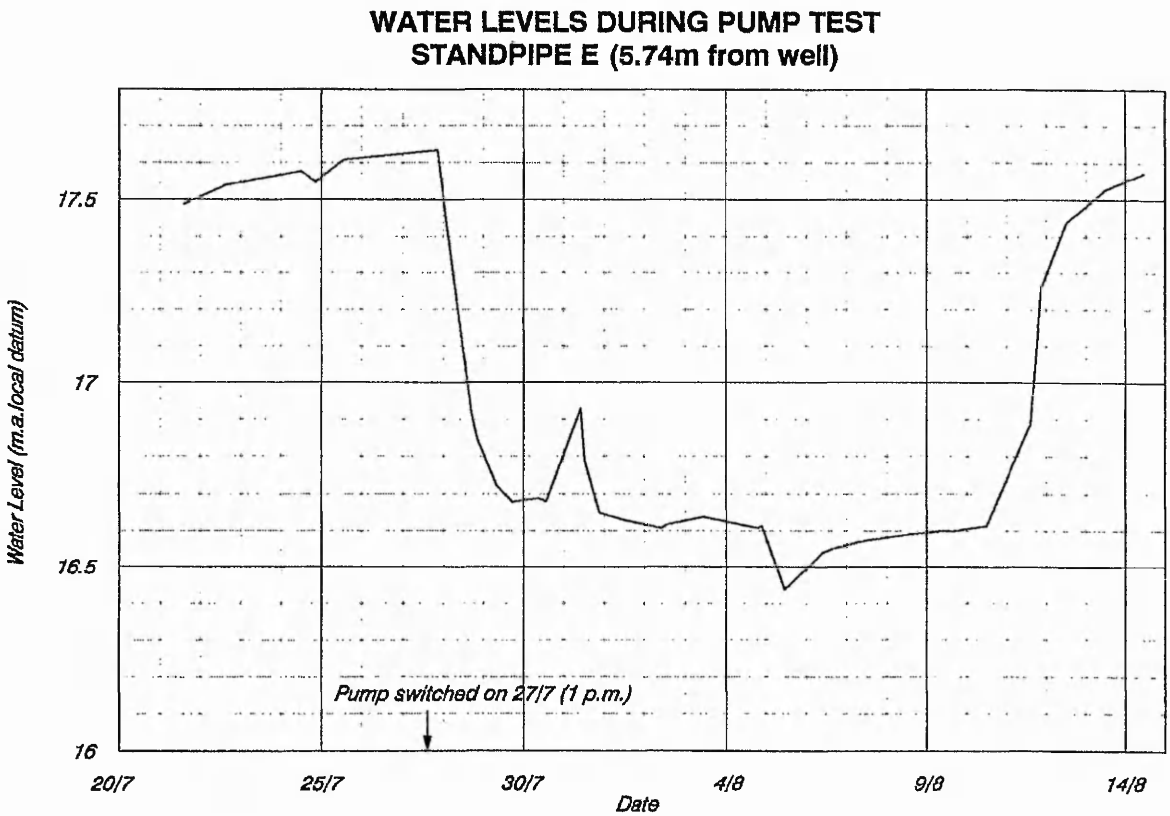
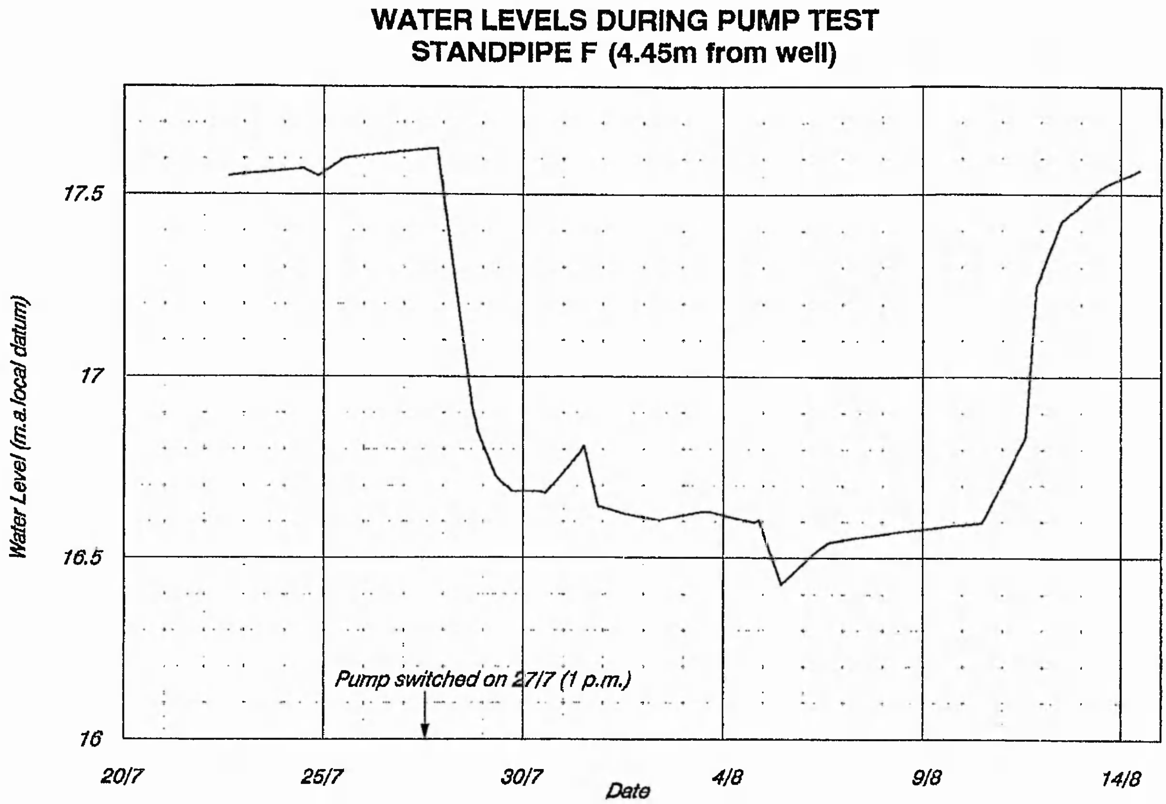

WATER LEVELS DURING PUMP TEST
STANDPIPE E (5.74m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.20


WATER LEVELS DURING PUMP TEST
STANDPIPE F (4.45m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

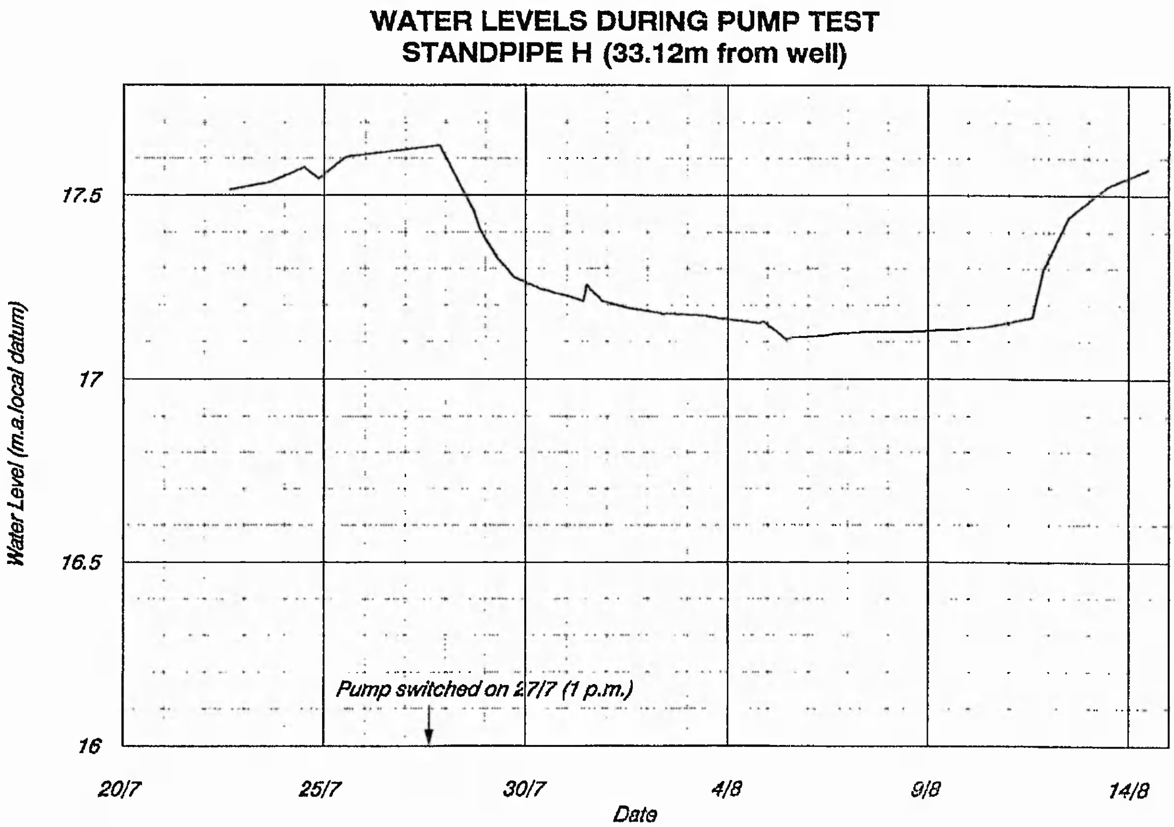
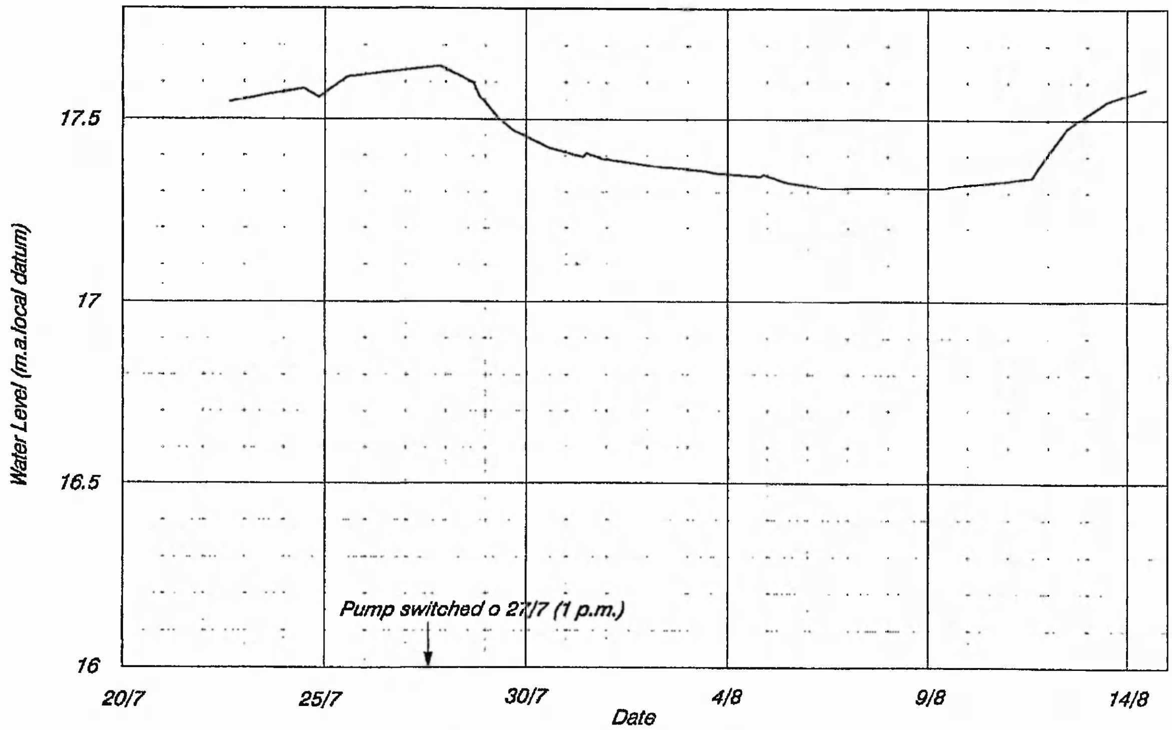

WATER LEVELS DURING PUMP TEST
STANDPIPE H (33.12m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.21

WATER LEVELS DURING PUMP TEST
STANDPIPE I (60.84m from well)



WATER LEVELS DURING PUMP TEST
STANDPIPE J (56.82m from well)

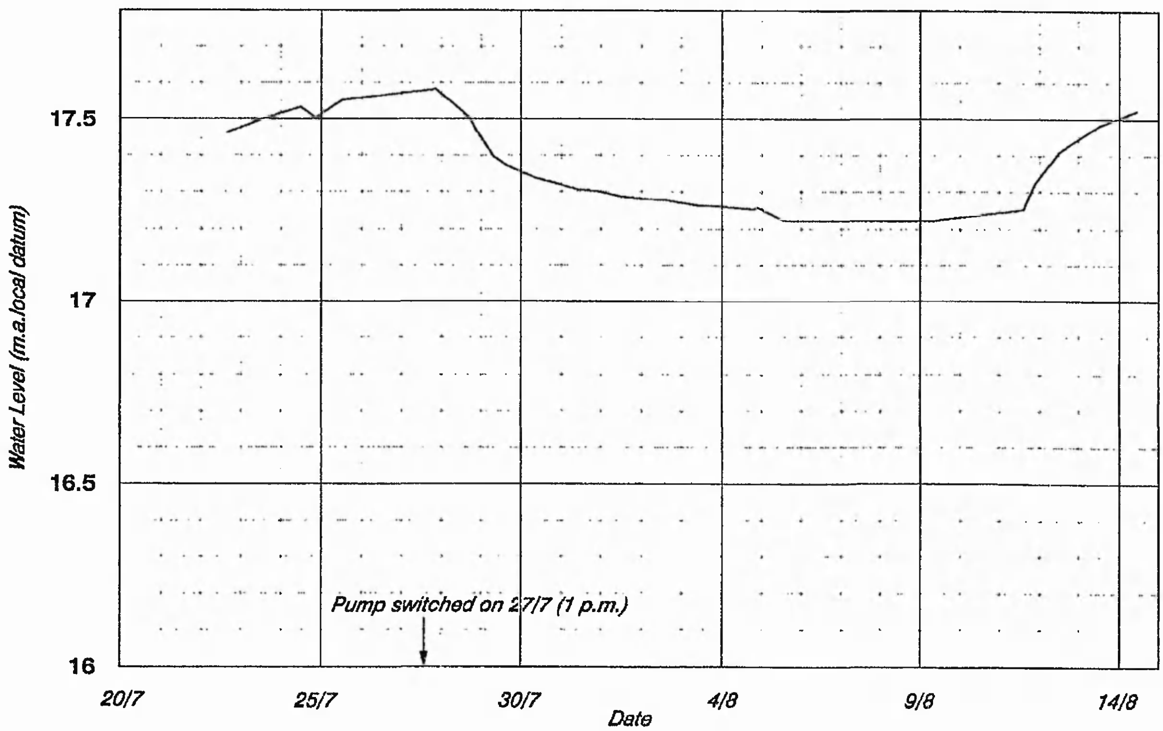
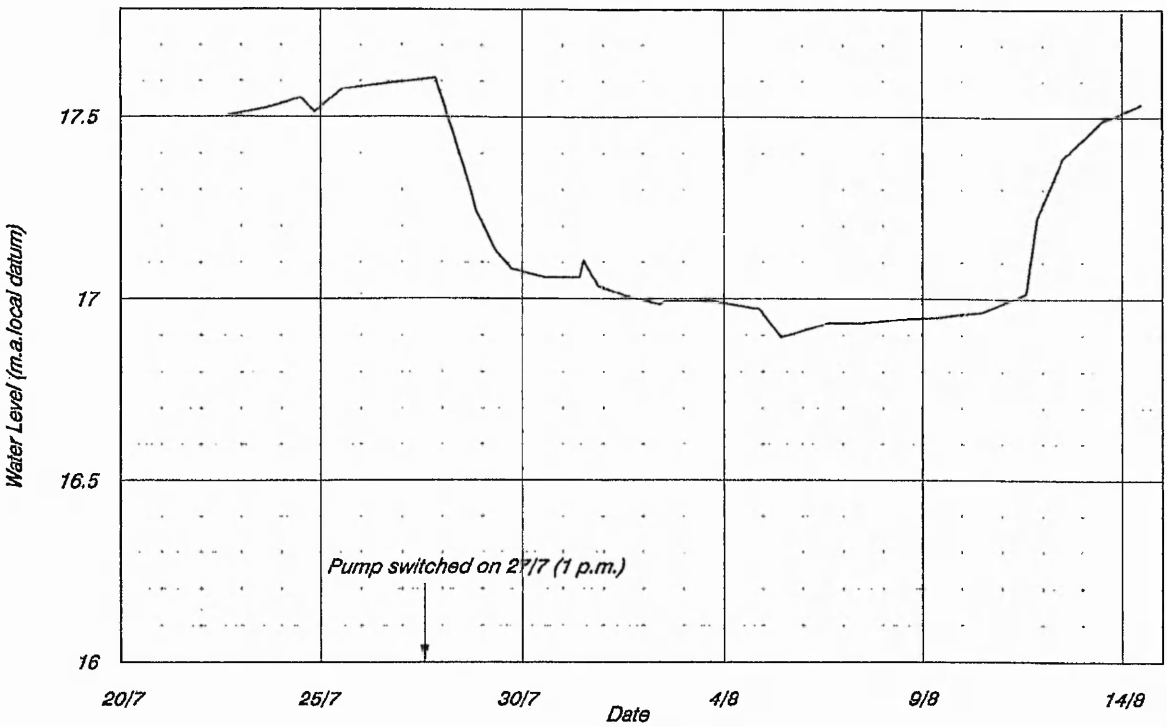


Figure 6.22

WATER LEVELS DURING PUMP TEST
STANDPIPE K (19.87m from well)



WATER LEVELS DURING PUMP TEST
STANDPIPE L (23.74m from well)

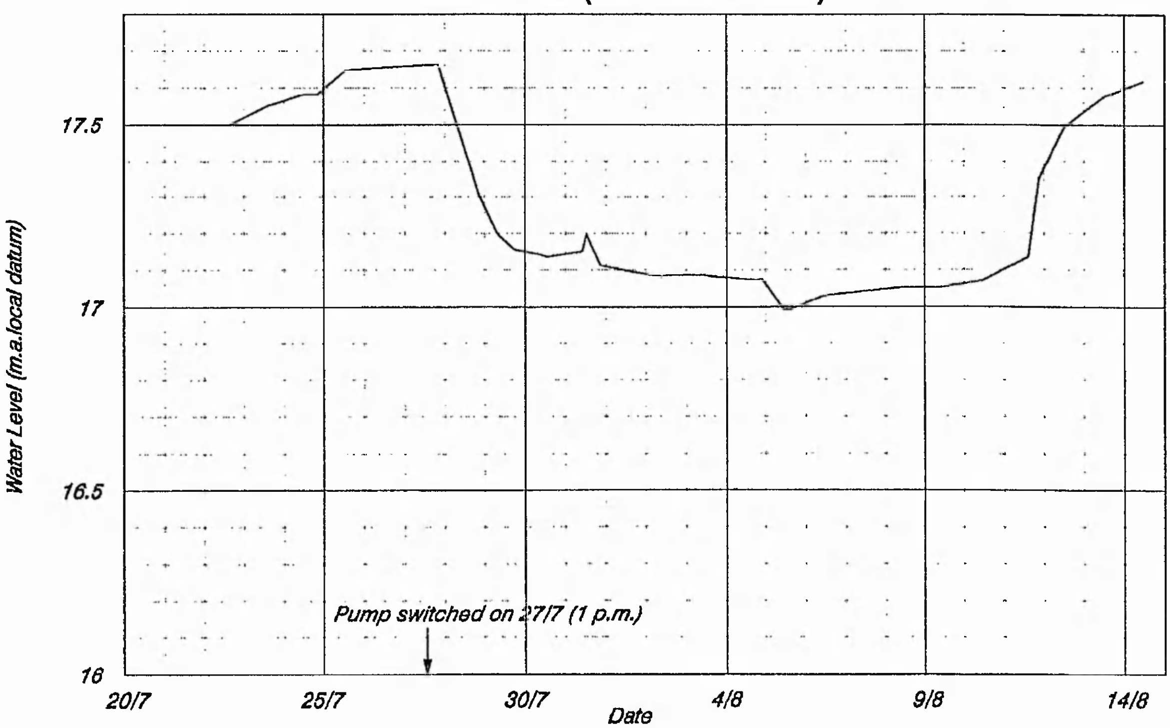
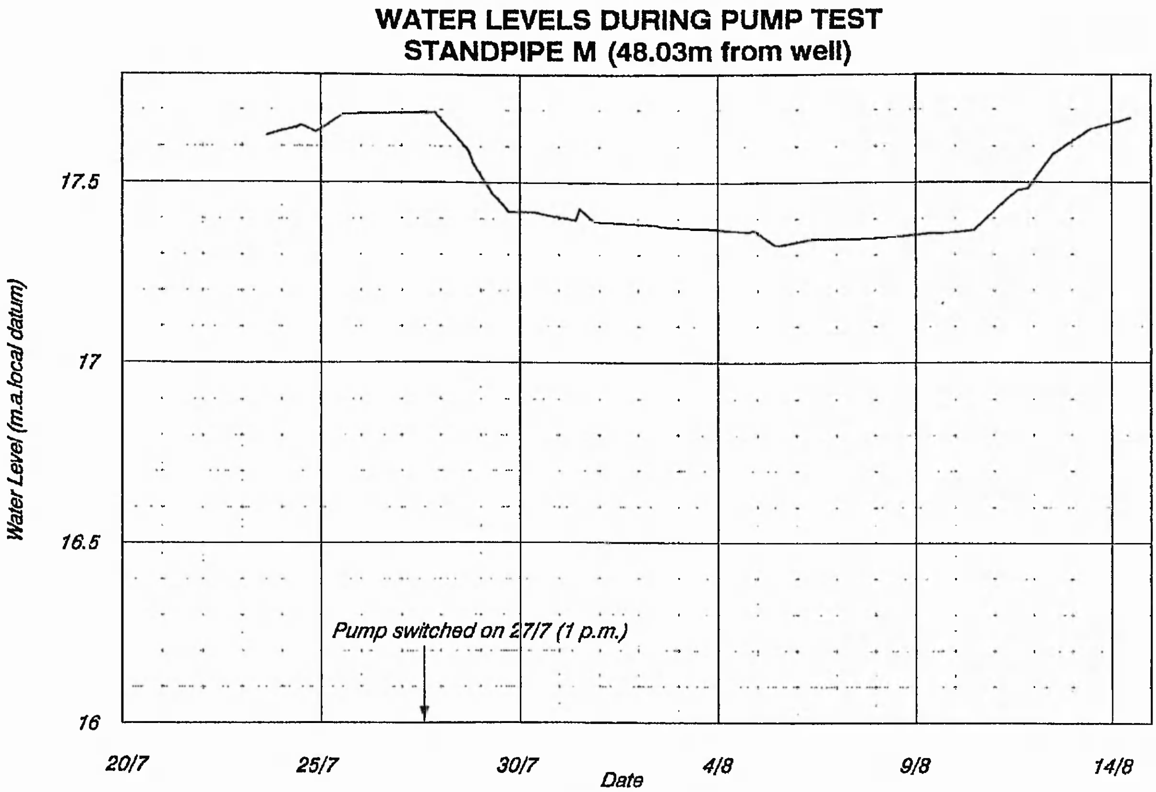


Figure 6.23


WATER LEVELS DURING PUMP TEST
STANDPIPE M (48.03m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
Date
4/8
9/8
14/8

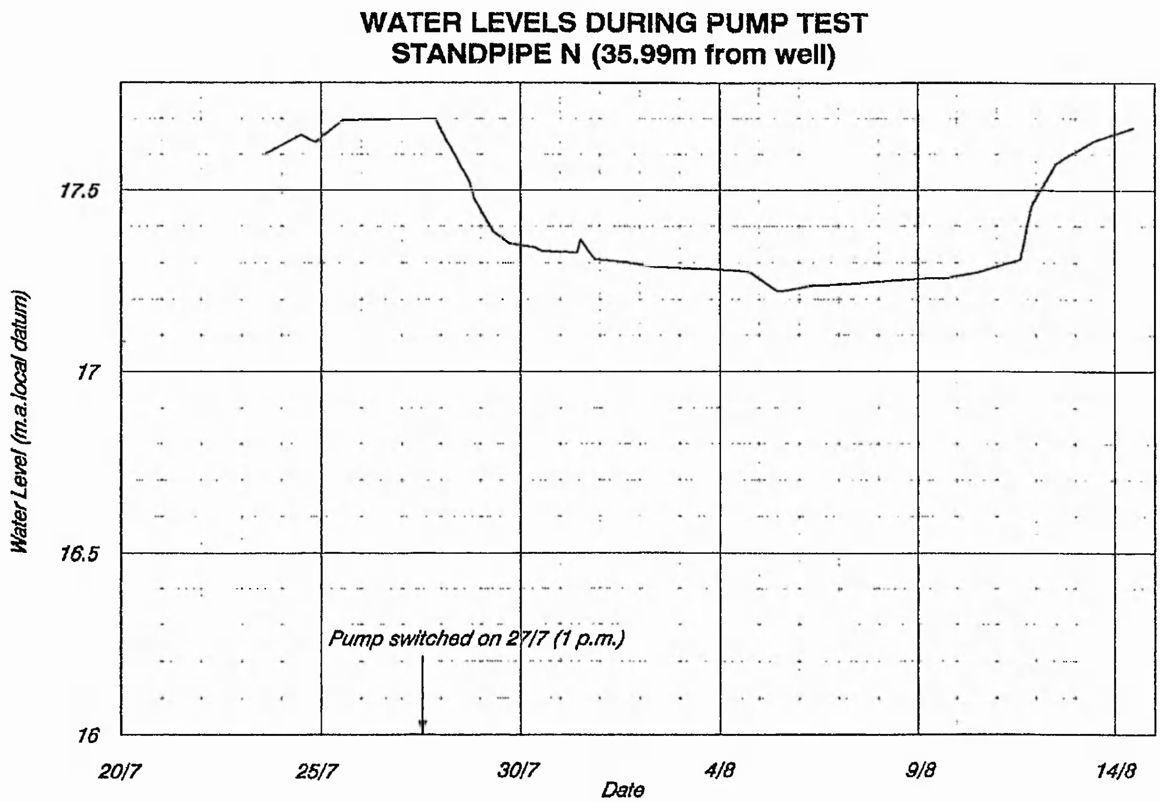
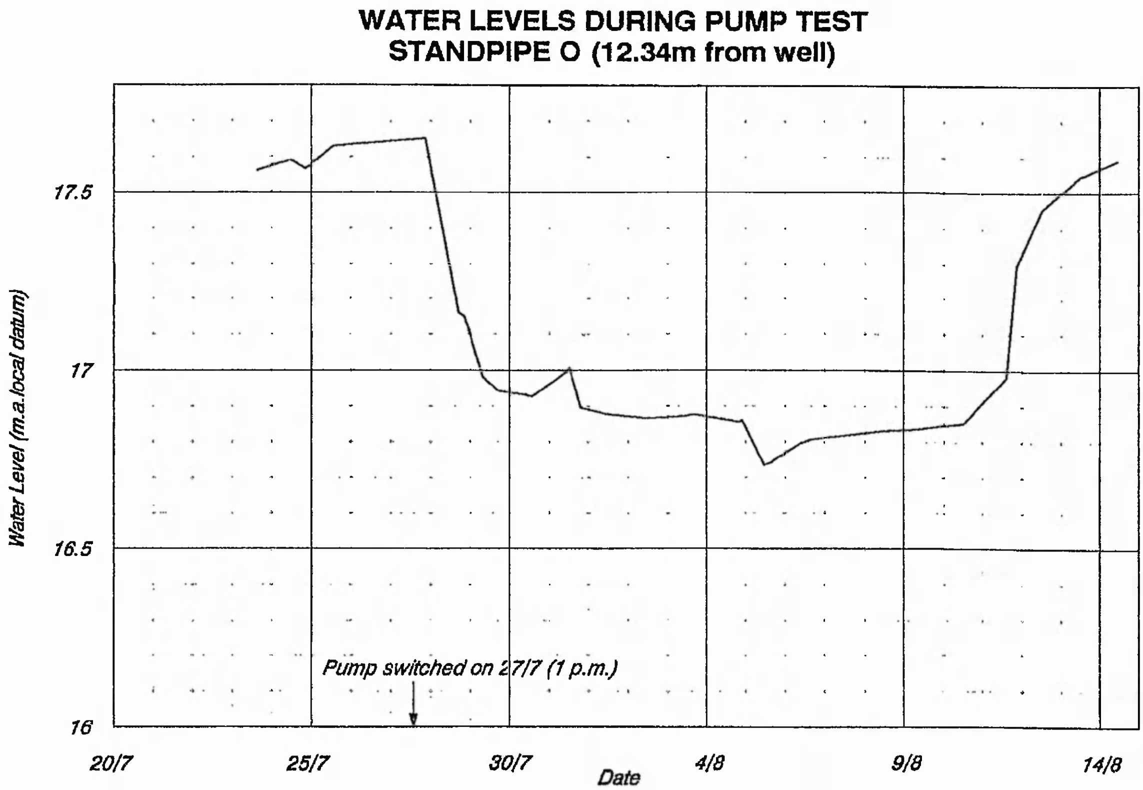

WATER LEVELS DURING PUMP TEST
STANDPIPE N (35.99m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
Date
4/8
9/8
14/8

Figure 6.24


WATER LEVELS DURING PUMP TEST
STANDPIPE O (12.34m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

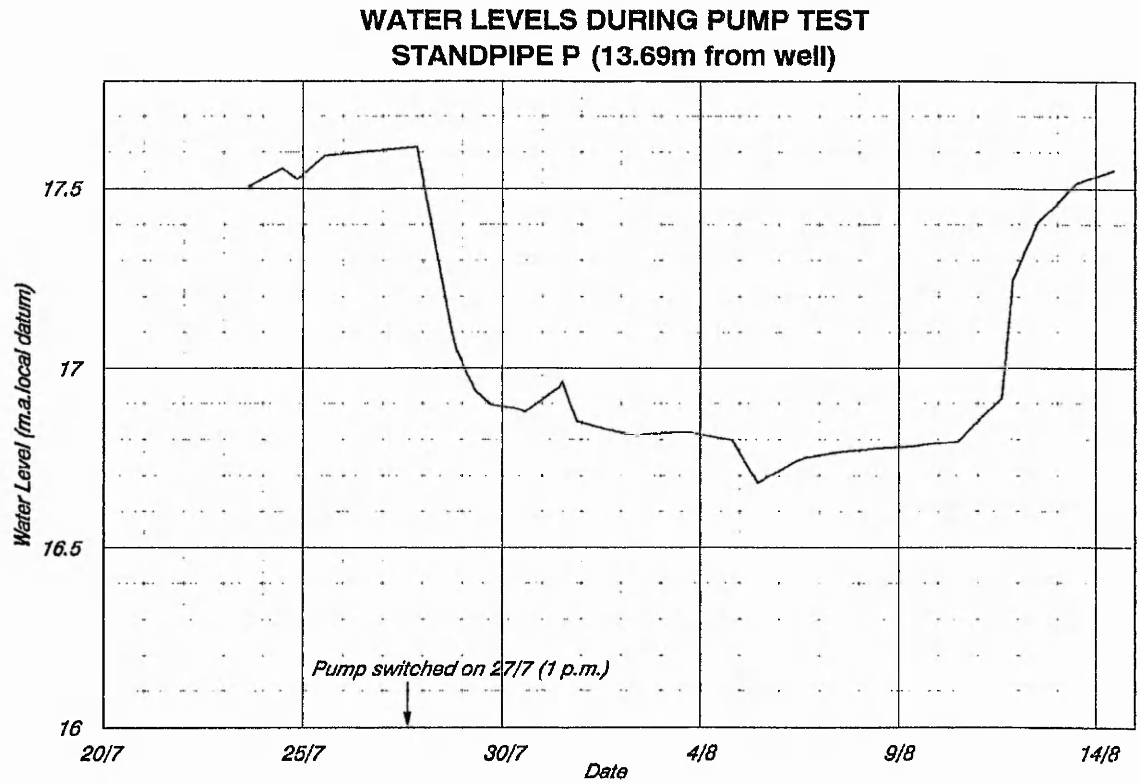
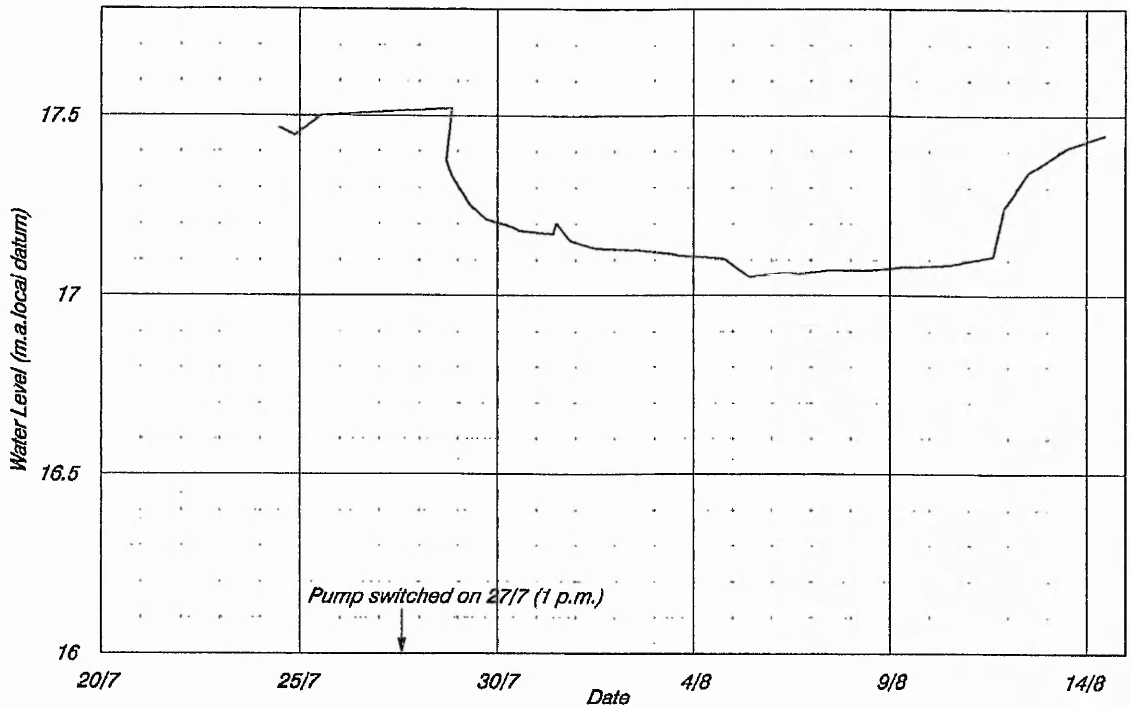

WATER LEVELS DURING PUMP TEST
STANDPIPE P (13.69m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.25

WATER LEVELS DURING PUMP TEST
STANDPIPE Q (37.31m from well)



WATER LEVELS DURING PUMP TEST
STANDPIPE R (58.91m from well)

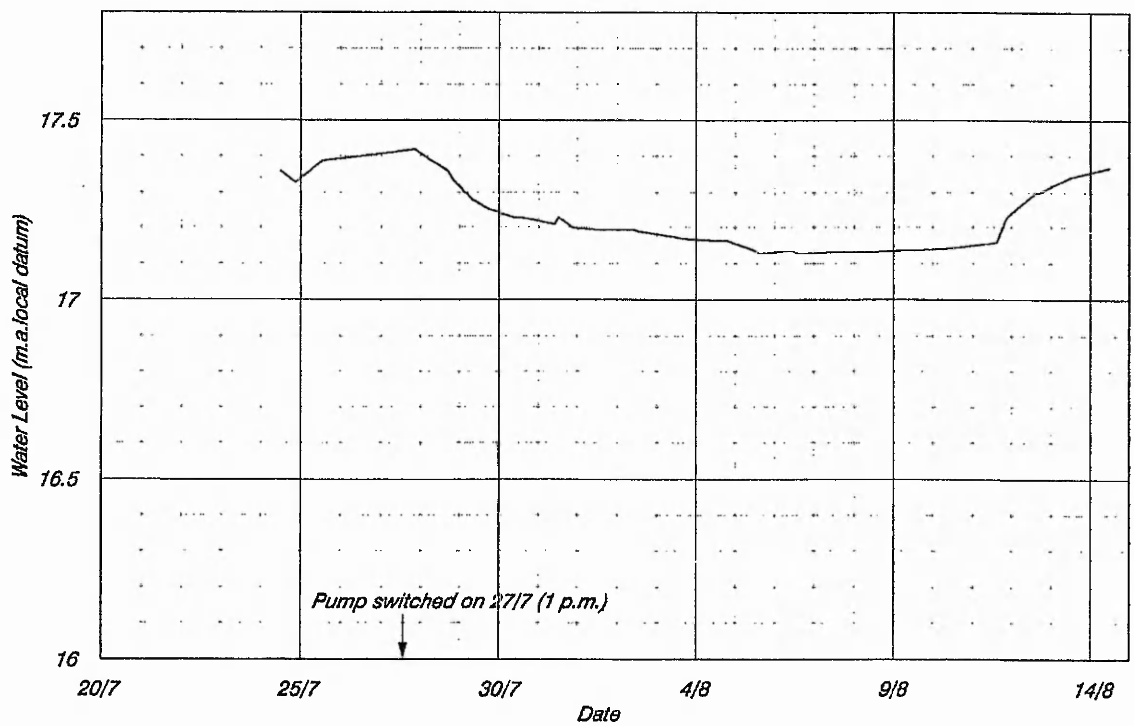
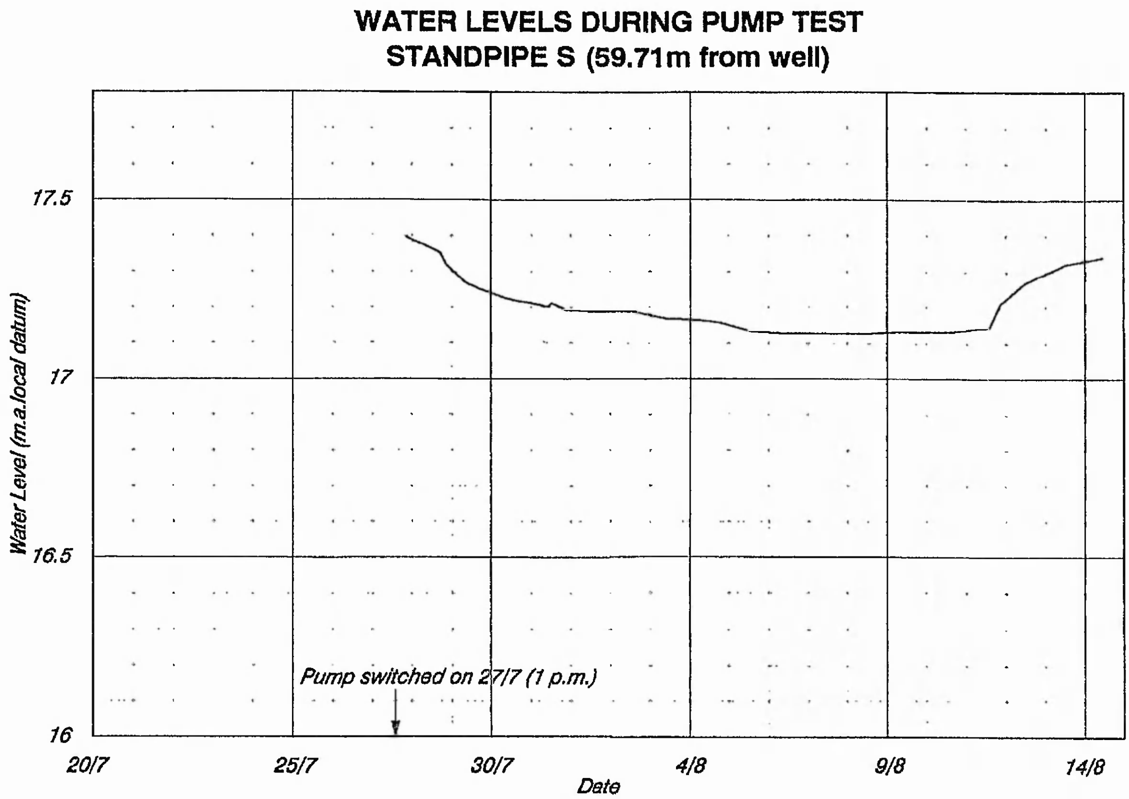
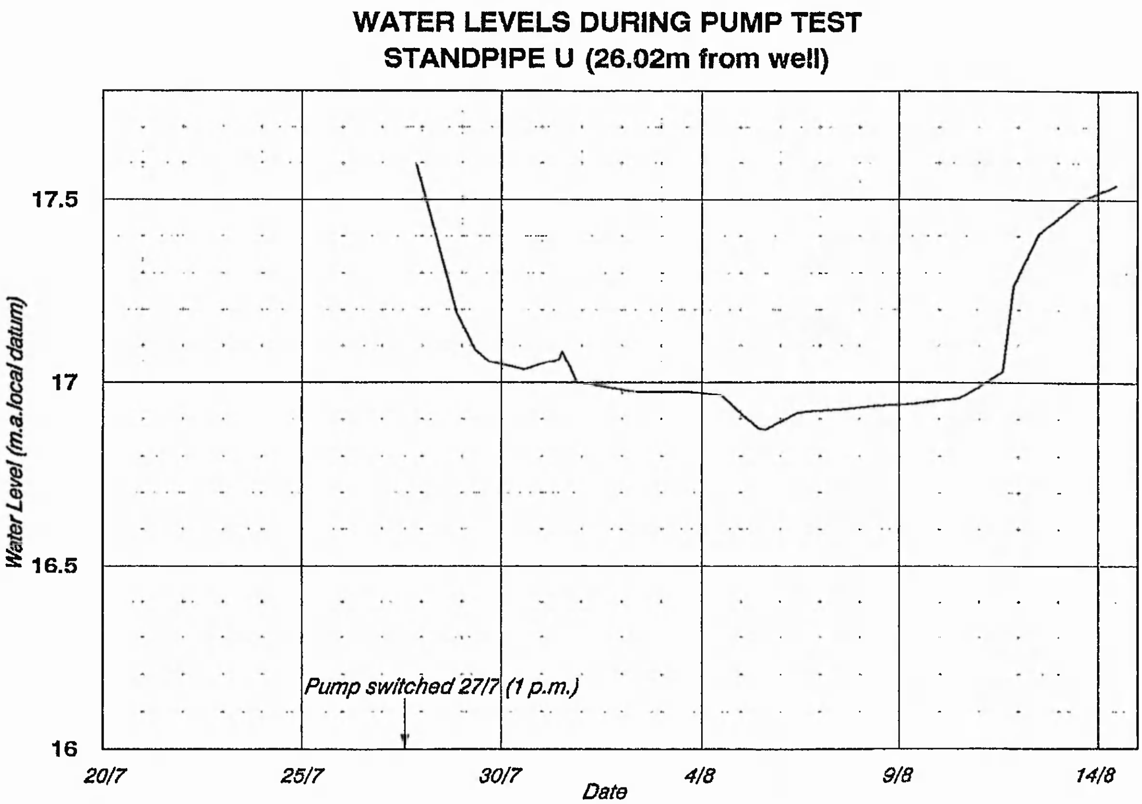


Figure 6.26



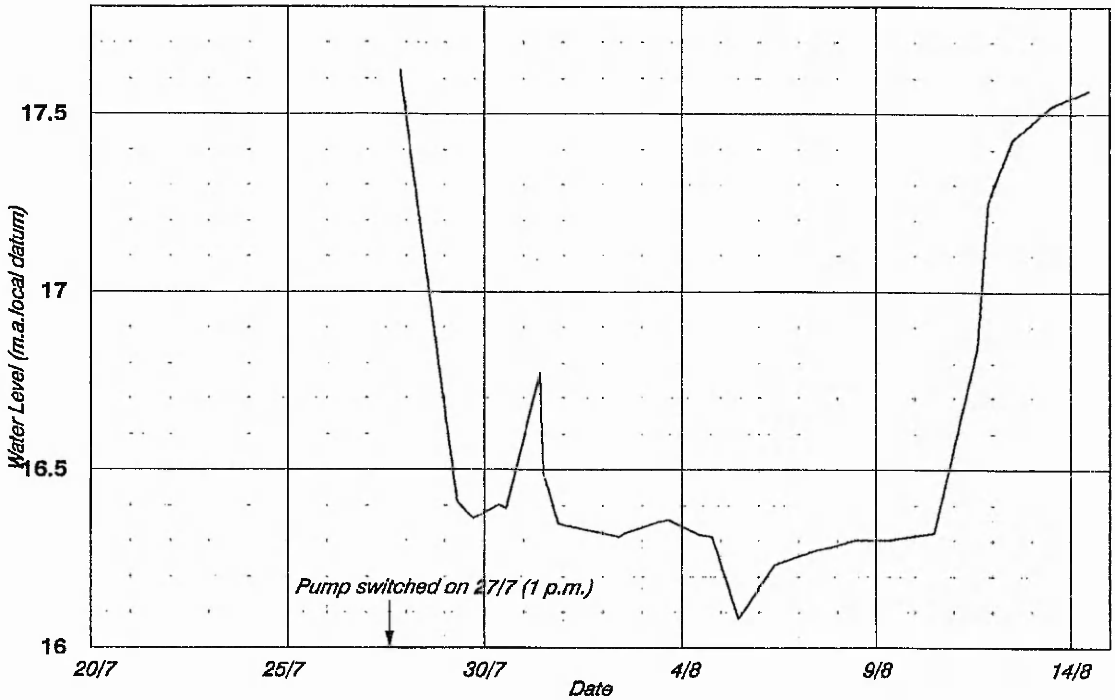
WATER LEVELS DURING PUMP TEST
STANDPIPE S (59.71m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date



WATER LEVELS DURING PUMP TEST
STANDPIPE U (26.02m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.27

WATER LEVELS DURING PUMP TEST
STANDPIPE V (1.41m from well)



WATER LEVELS DURING PUMP TEST
STANDPIPE W (24.09m from well)

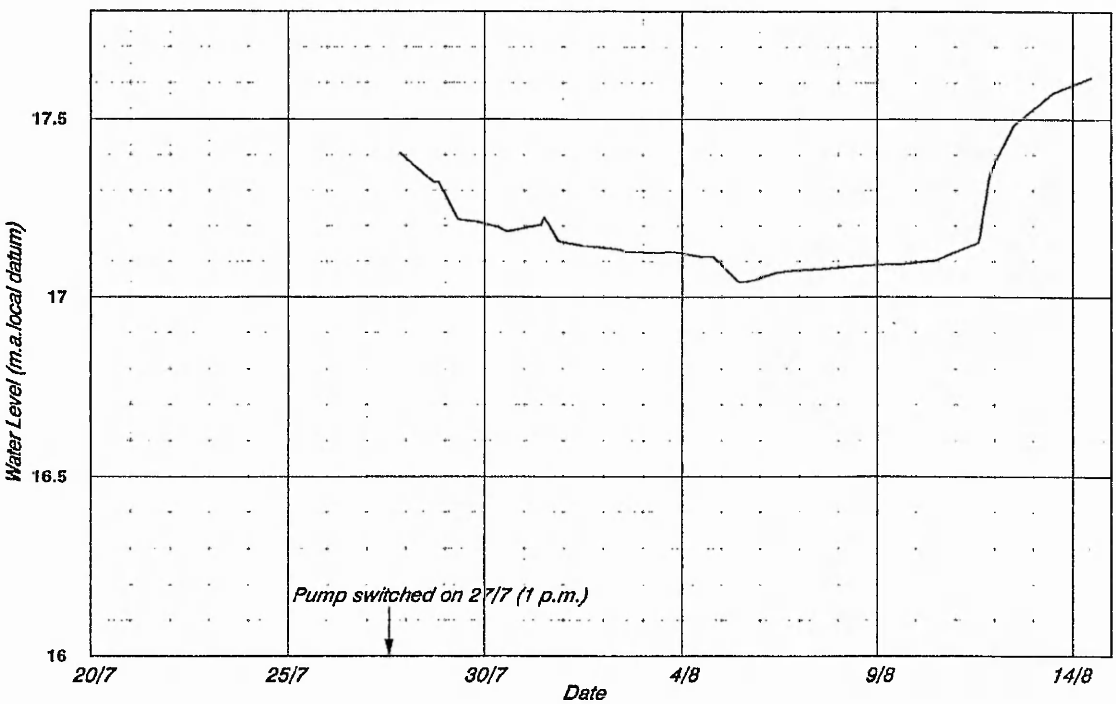
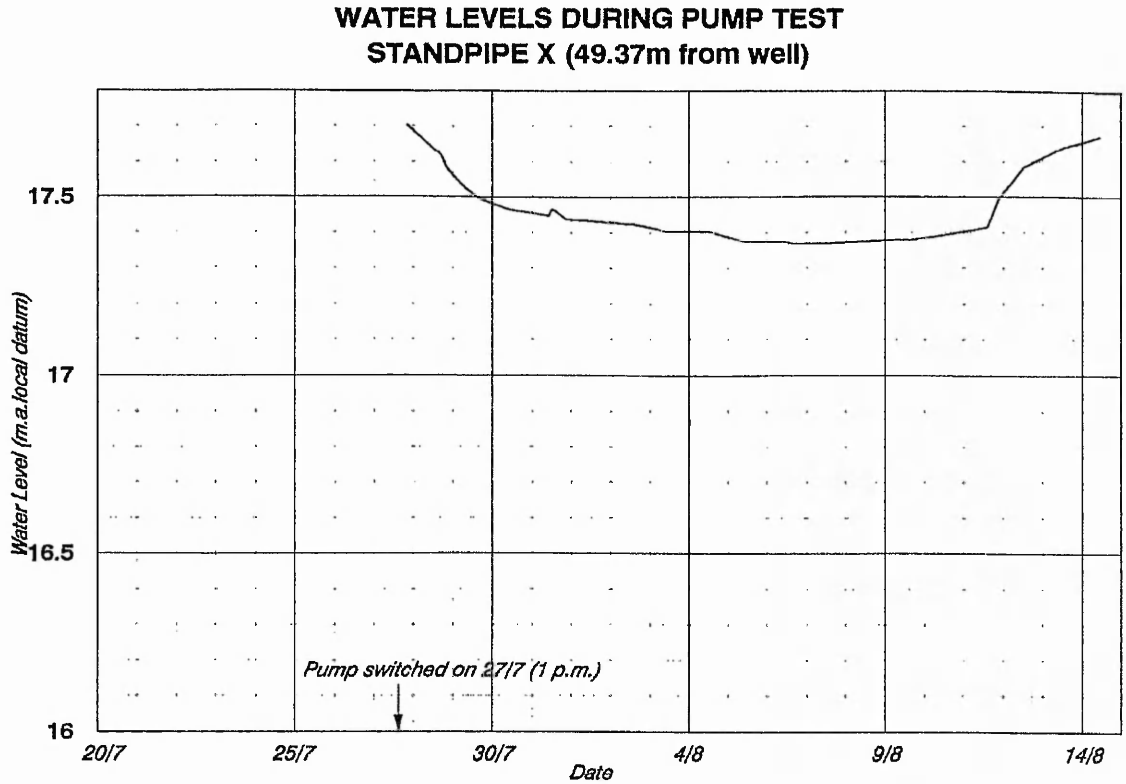


Figure 6.28


WATER LEVELS DURING PUMP TEST
STANDPIPE X (49.37m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

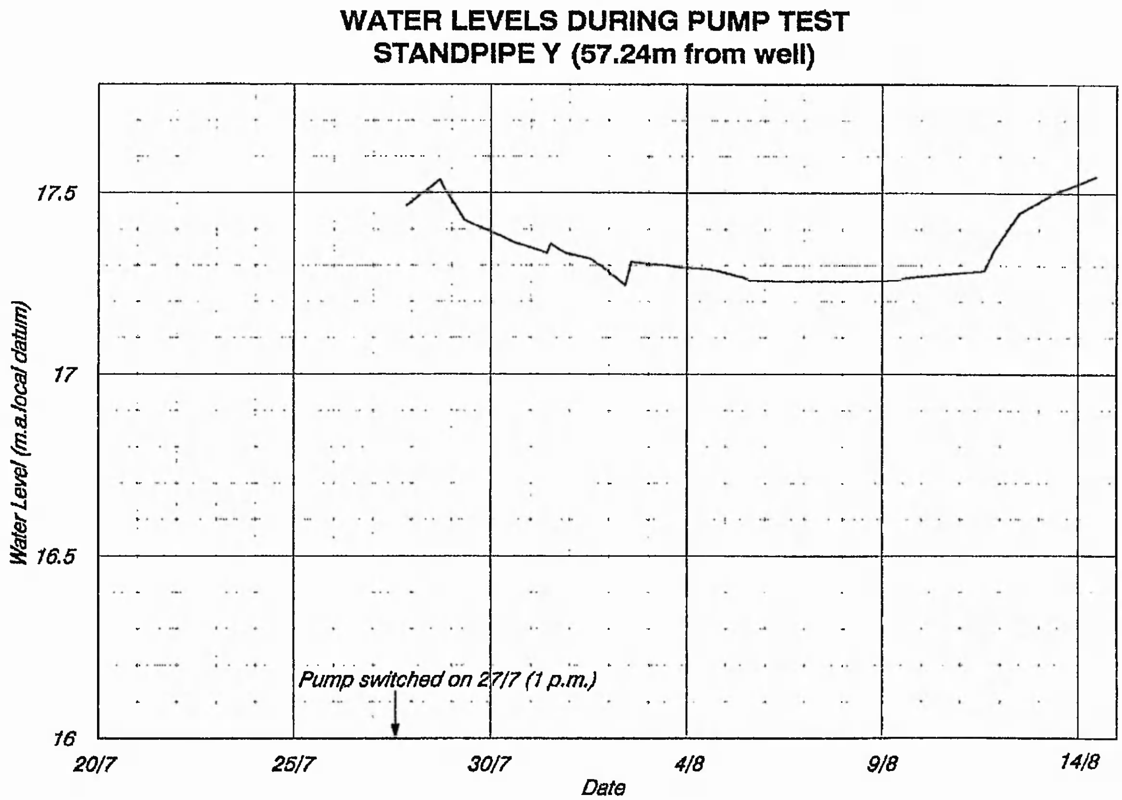

WATER LEVELS DURING PUMP TEST
STANDPIPE Y (57.24m from well)
Water Level (m.a.local datum)
17.5
17
16.5
16
Pump switched on 27/7 (1 p.m.)
20/7
25/7
30/7
4/8
9/8
14/8
Date

Figure 6.29

Figure 6.30
Cone of Depression North from Pump

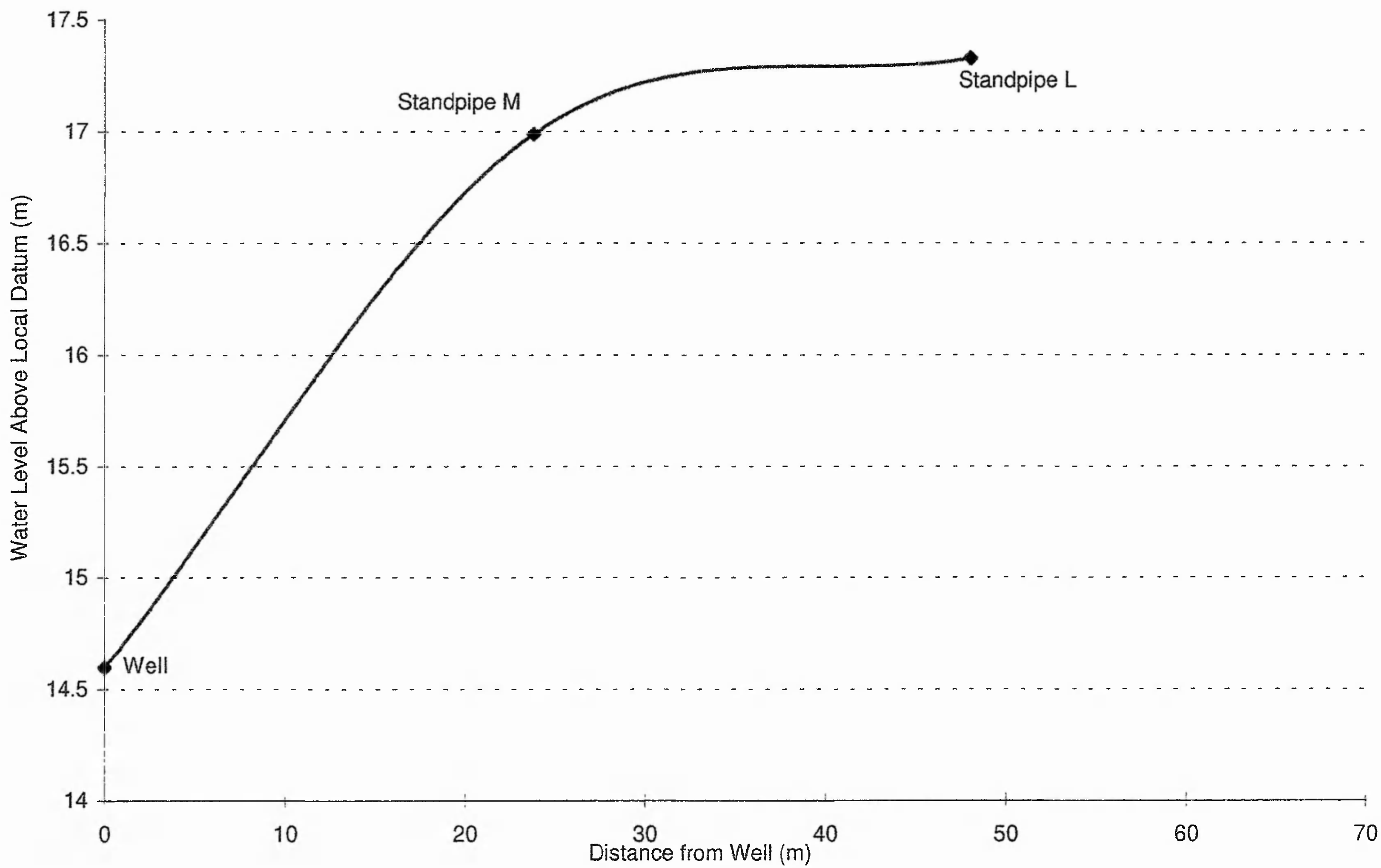


Figure 6.31
Cone of Depression NorthWest from Pump

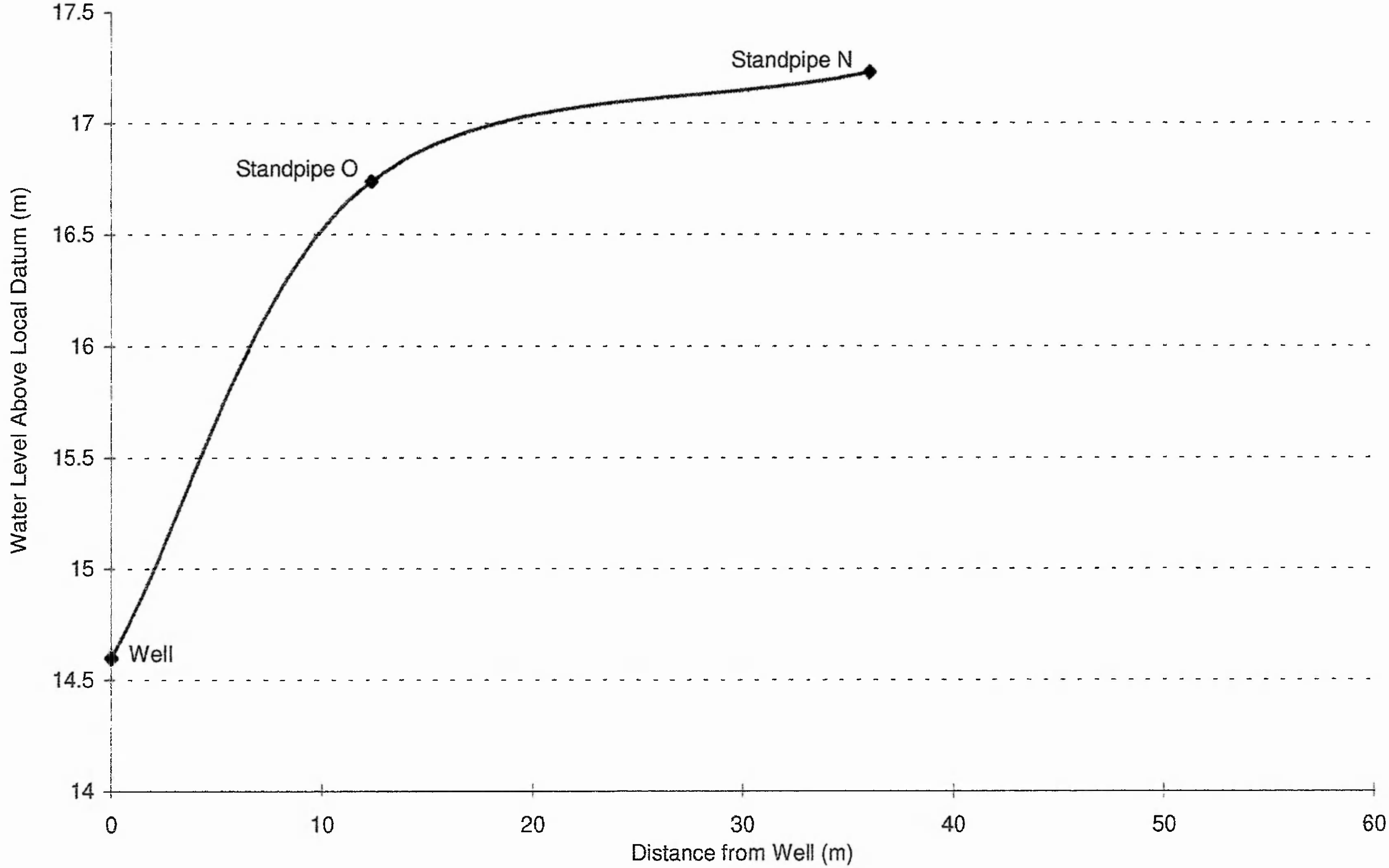


Figure 6.32
Cone of Depression West from Pump

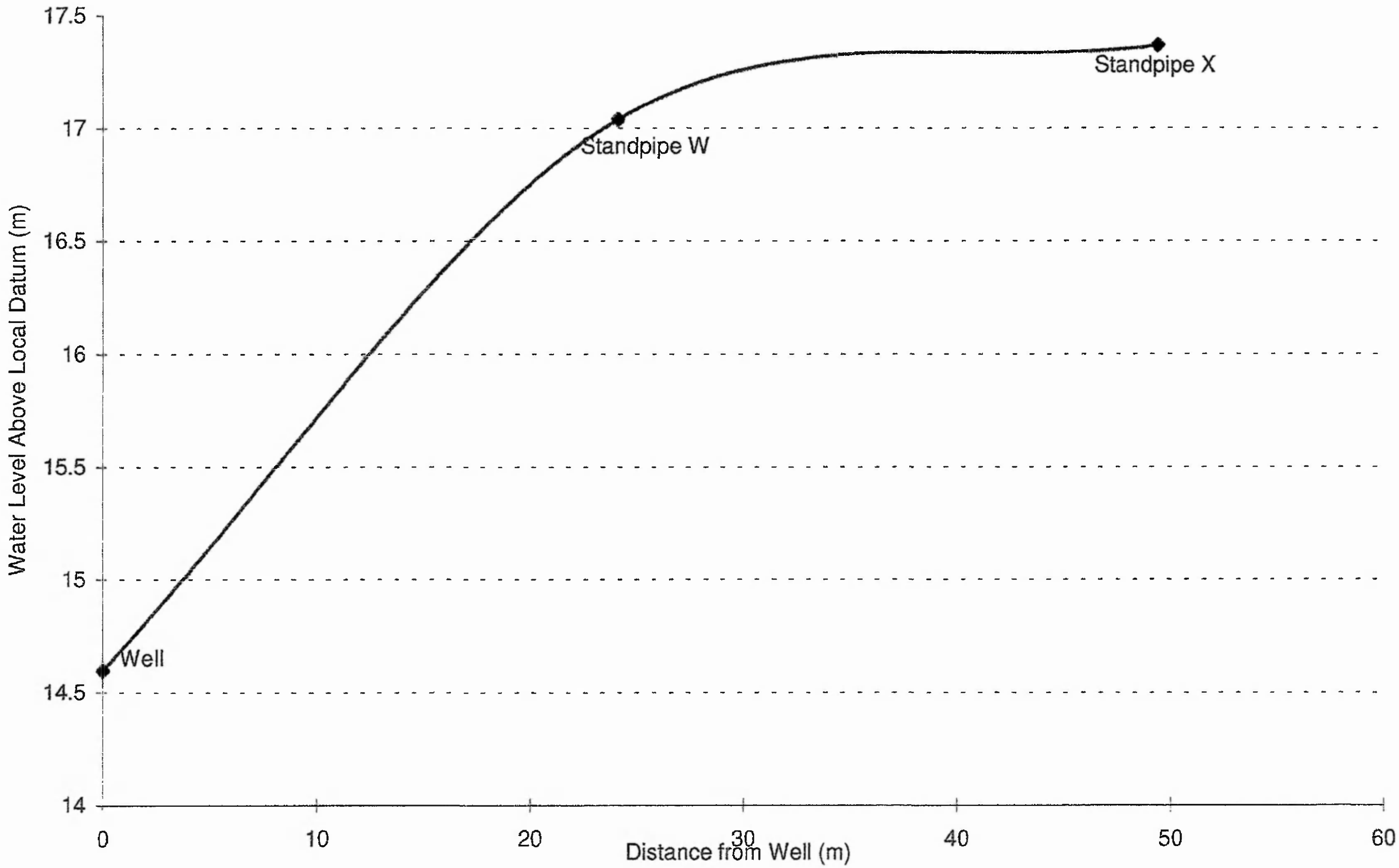


Figure 6.33
Cone of depression South West from Pump

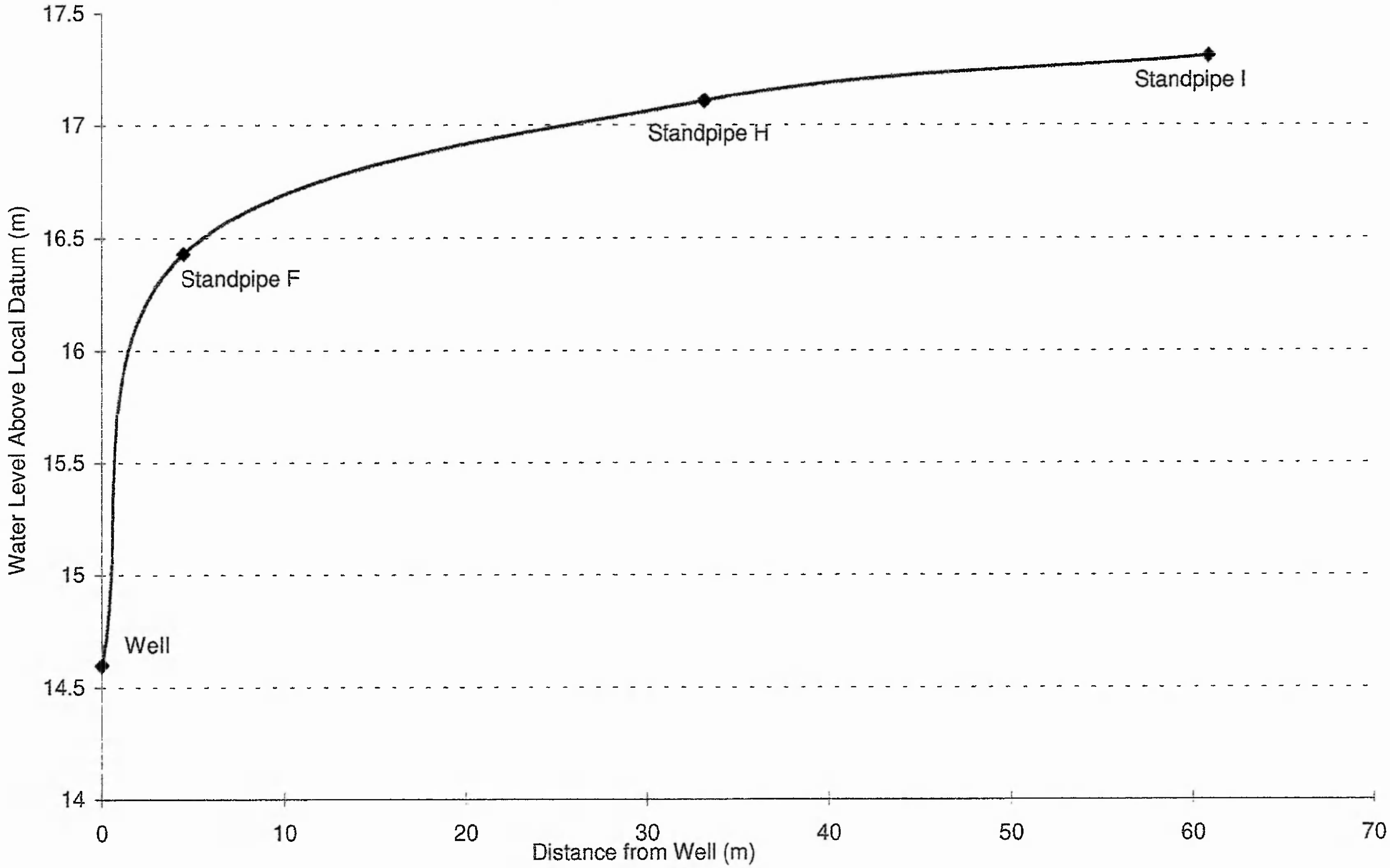


Figure 6.34
Cone of Depression South from Pump

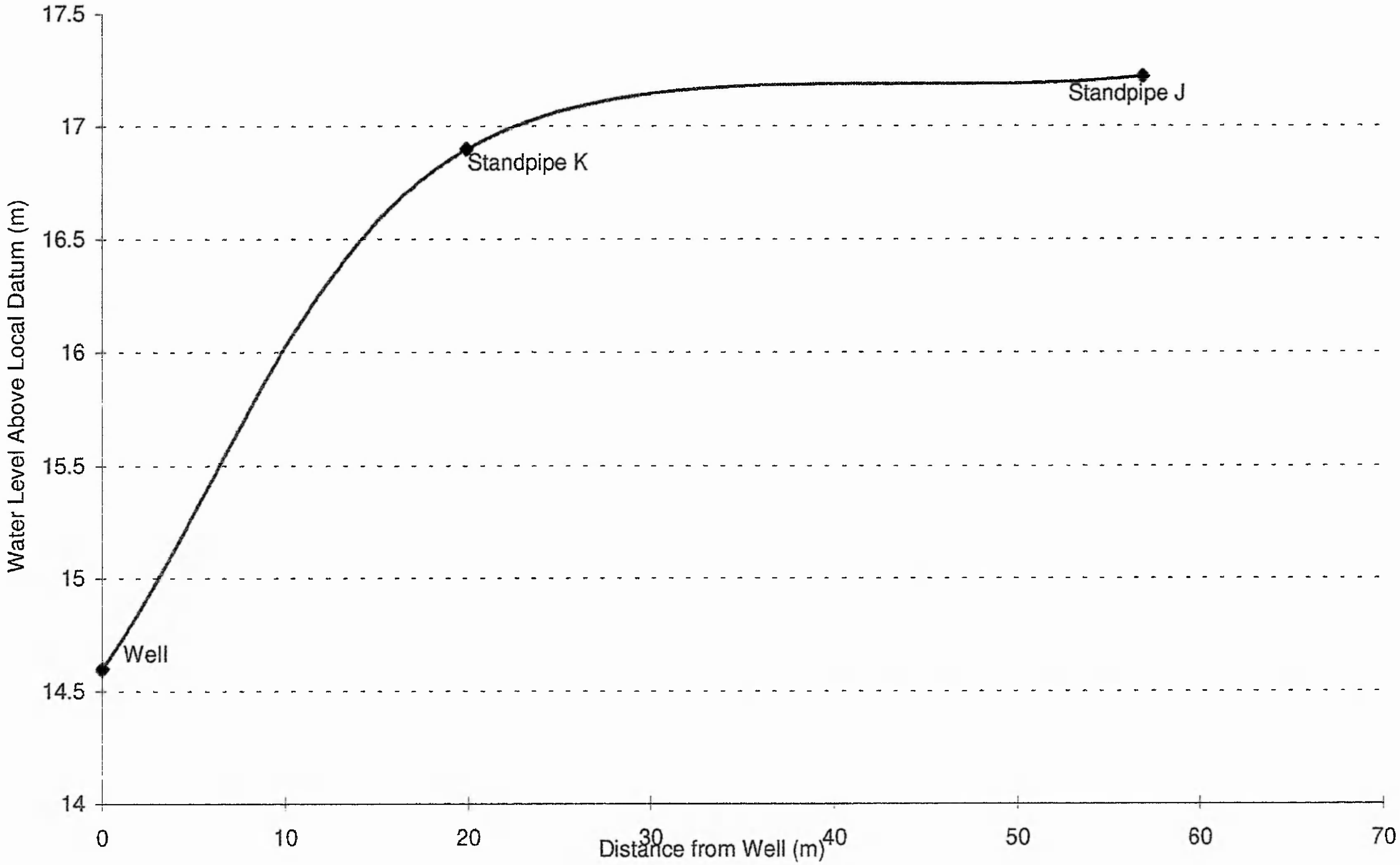


Figure 6.35
Cone of Depression South East from Pump

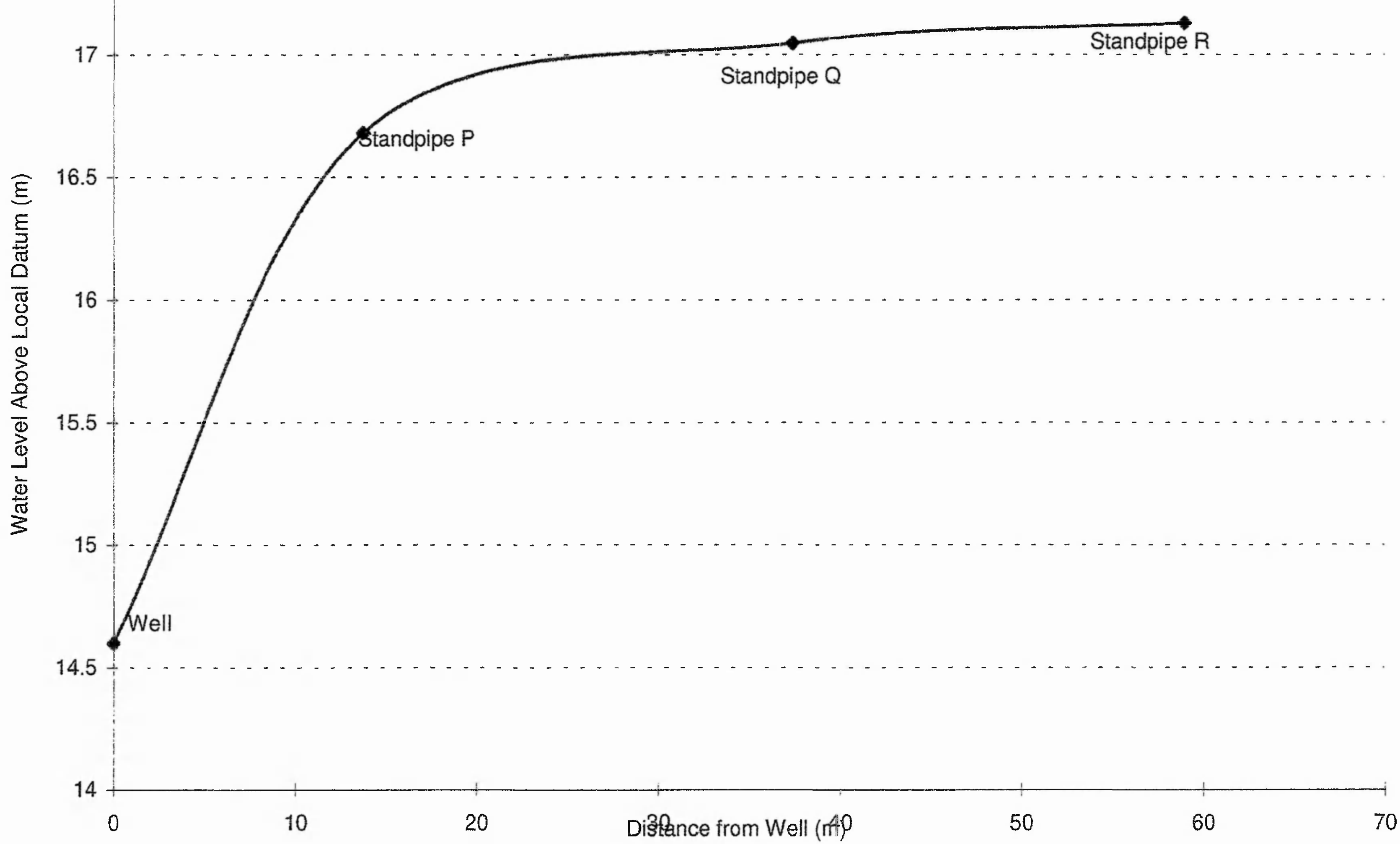


Figure 6.36
Cone of Depression North East from Pump

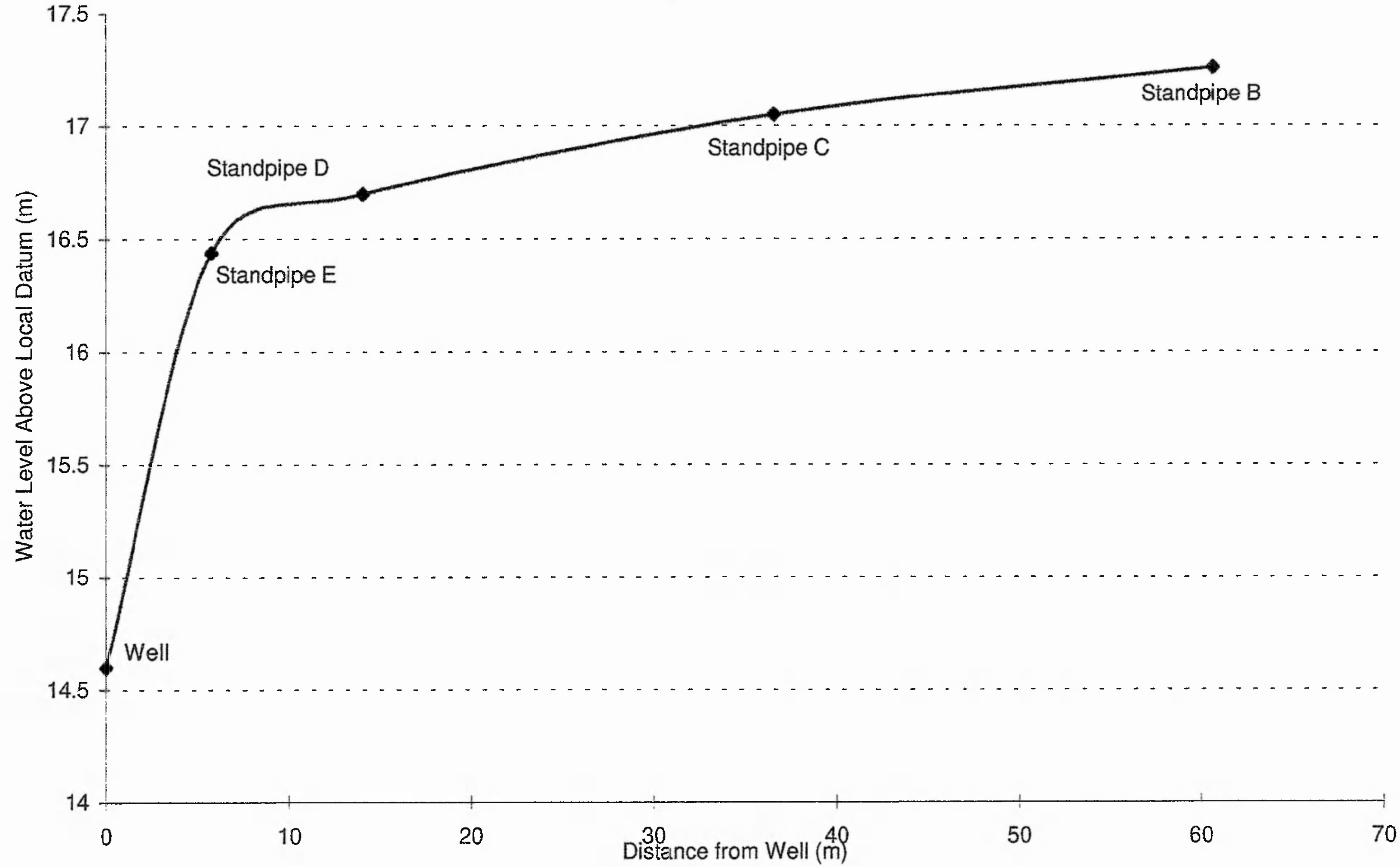


Figure 6.37
Combined Cones Of Depression For The Various Directions From The Pump

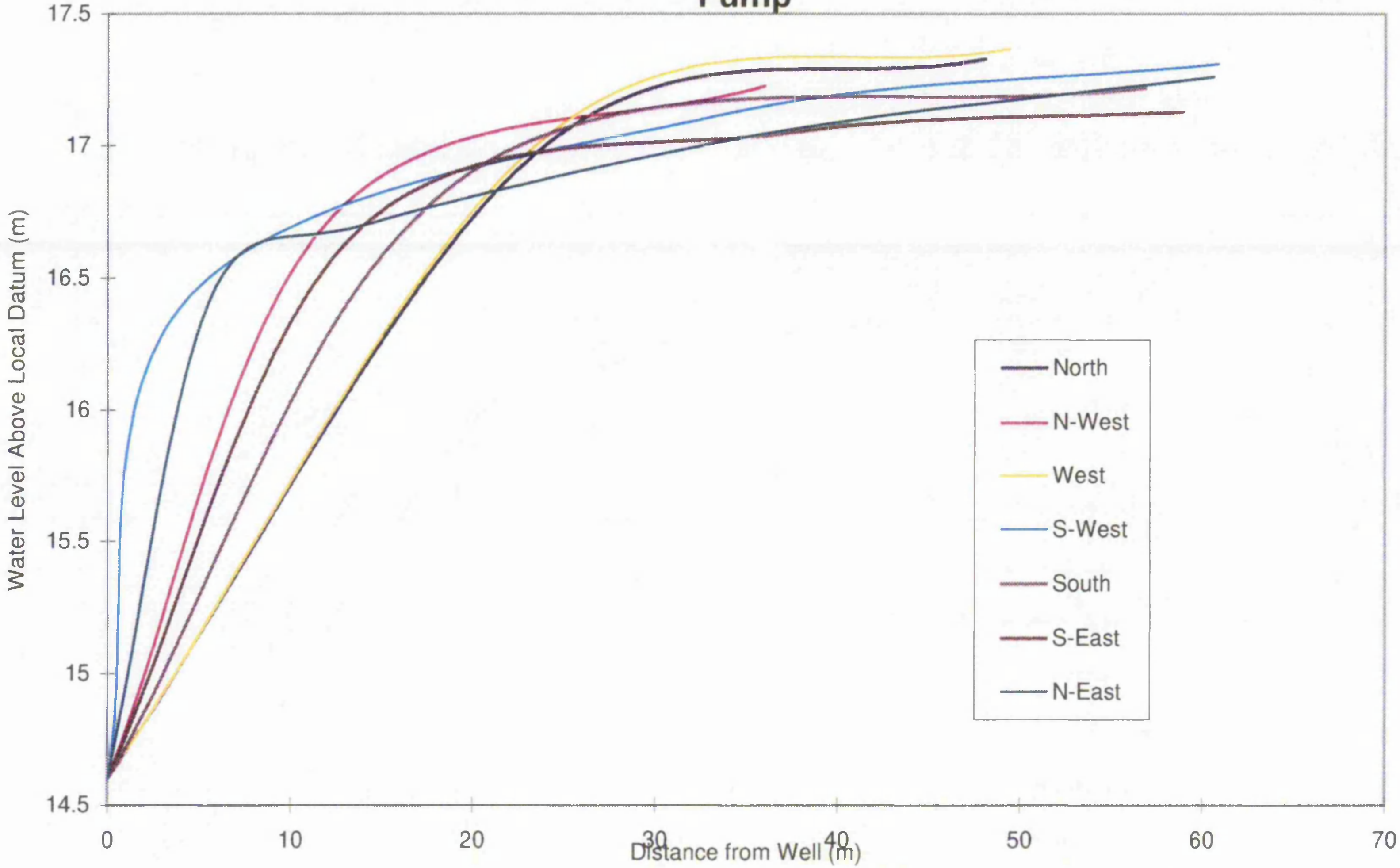


Figure 6.38
Movement of TBM A during the Pump Test

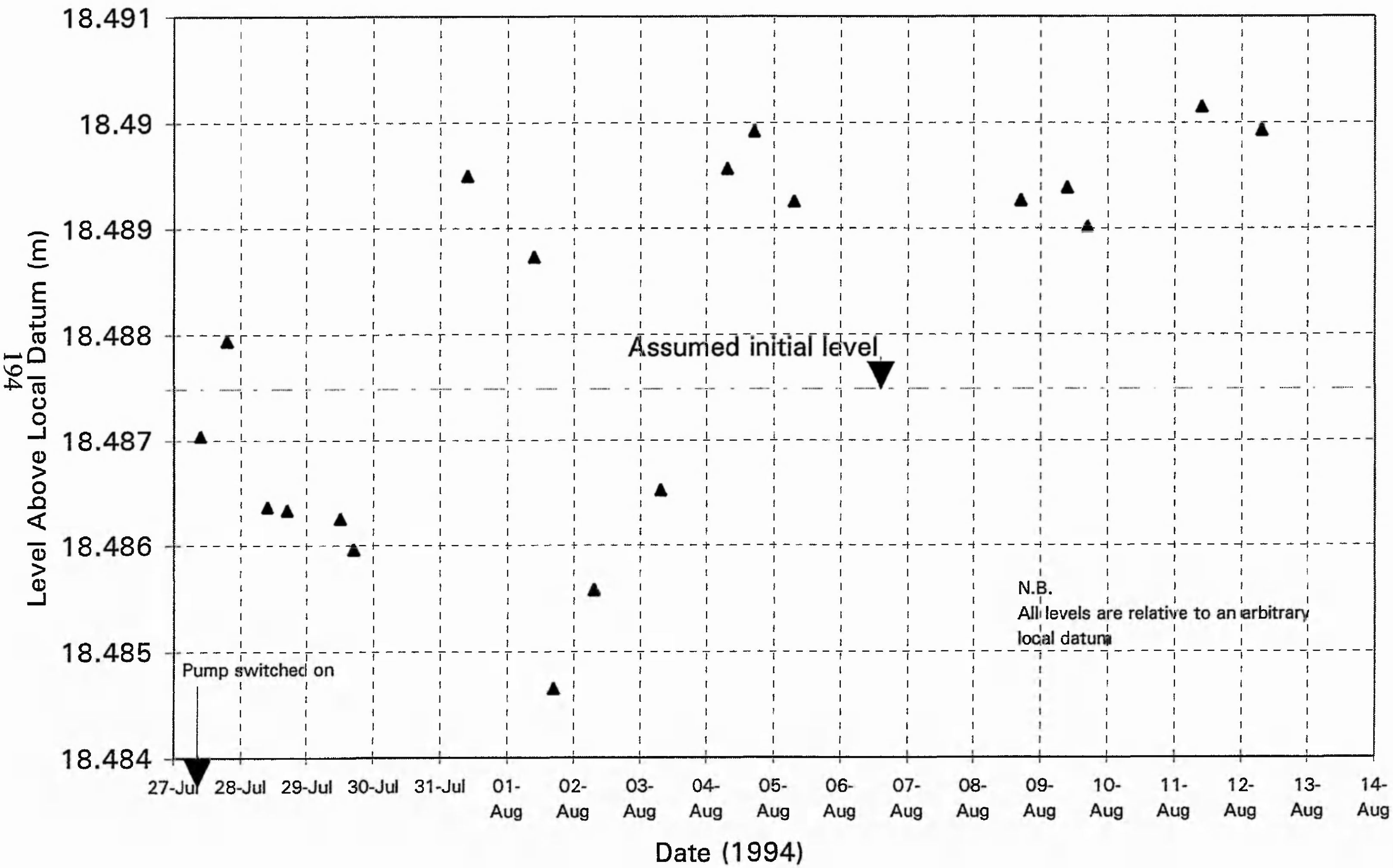


Figure 6.39
Movement of TBM B during the Pump Test

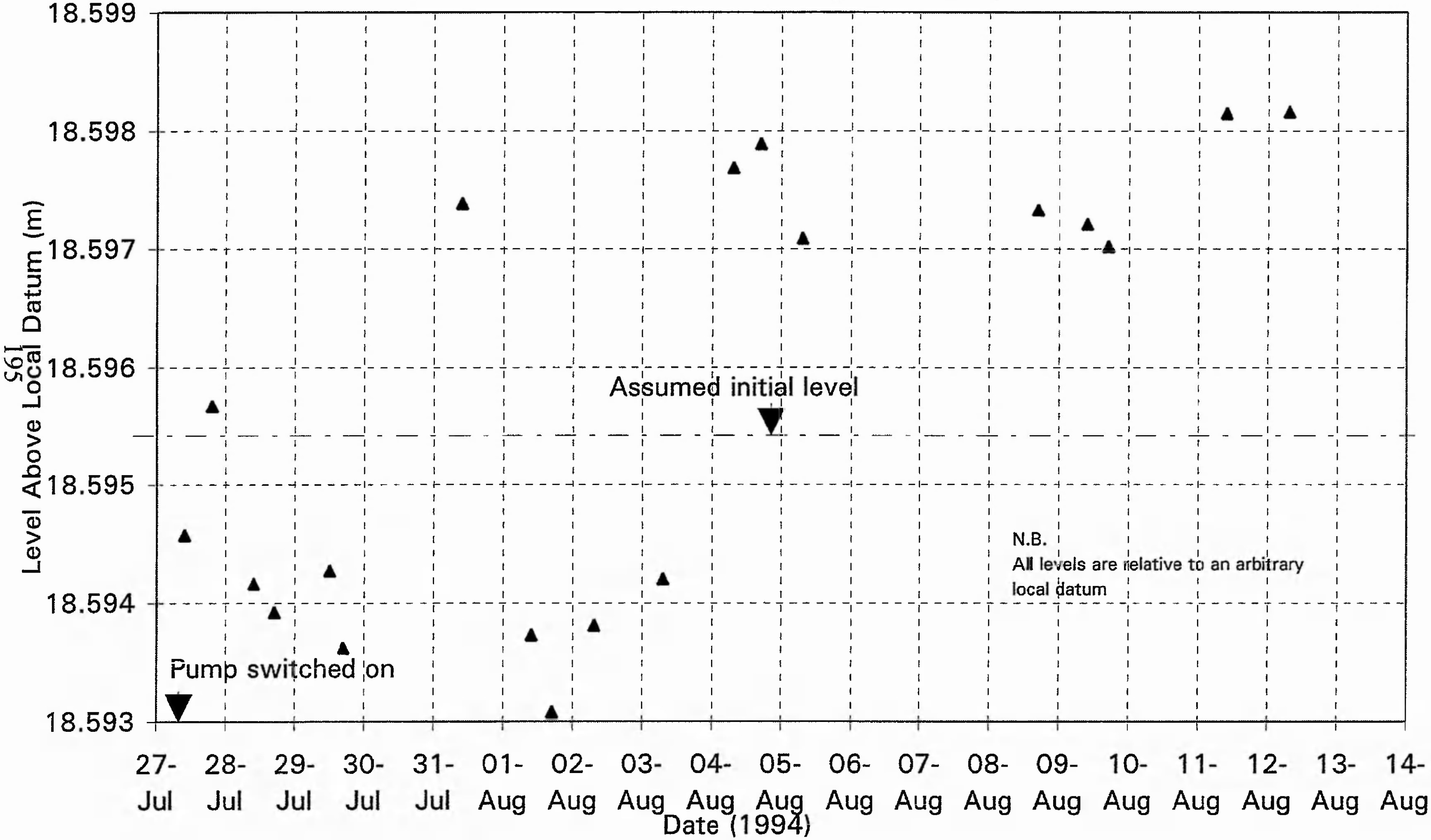


Figure 6.40
Movement of TBM C during the Pump Test

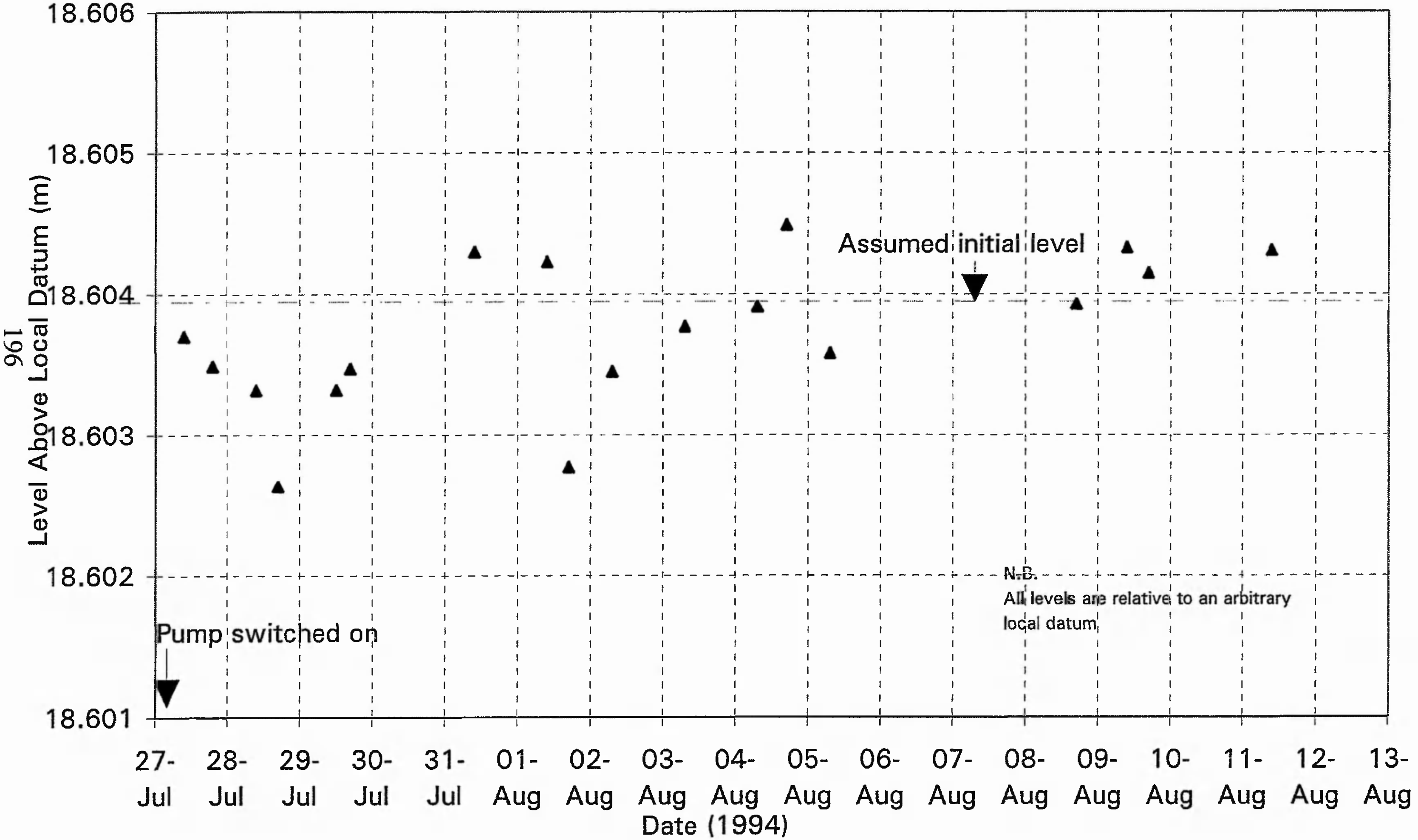


Figure 6.41
Movement of TBM D during the Pump Test

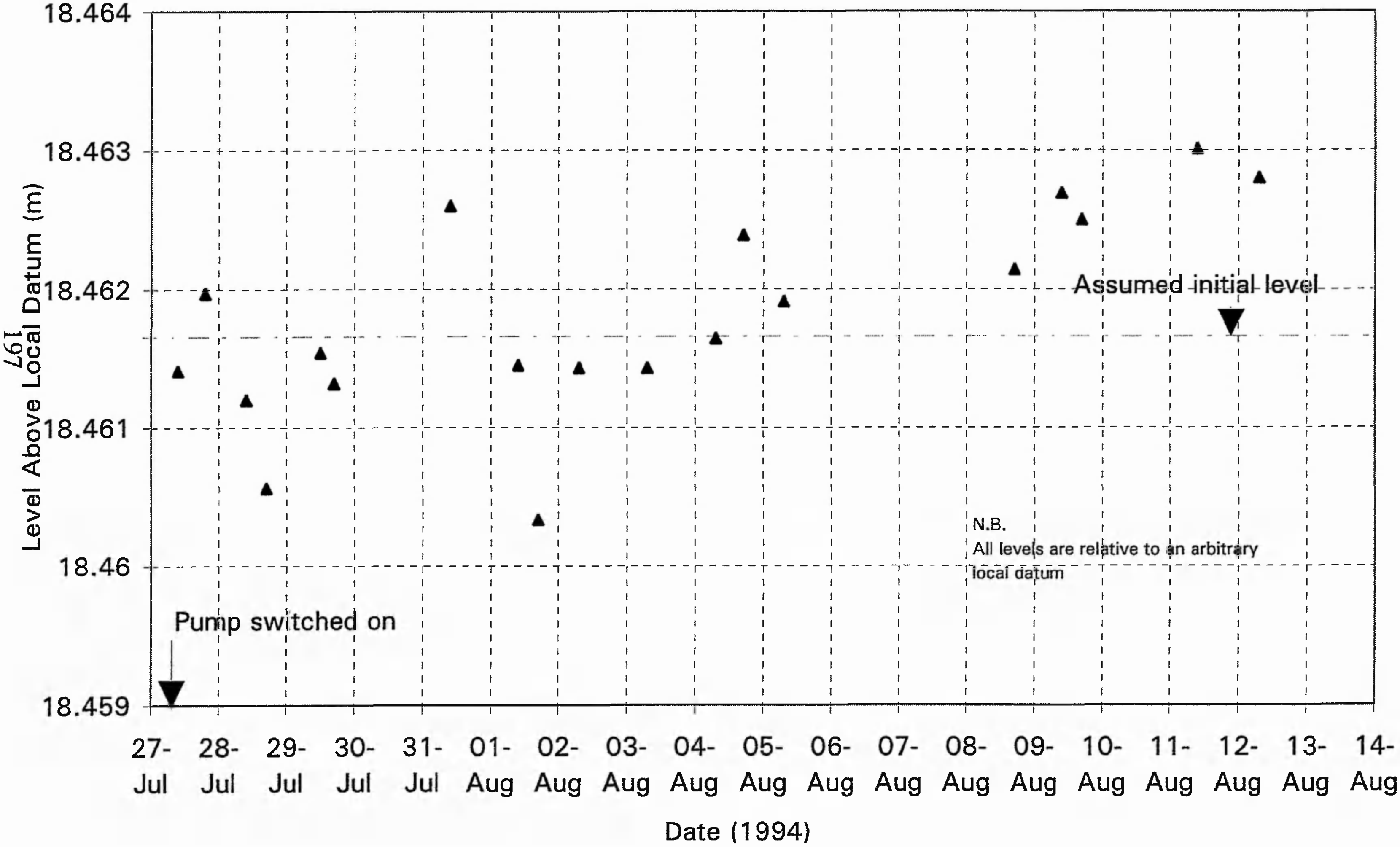


Figure 6.42
Movement of TBM E during the Pump Test

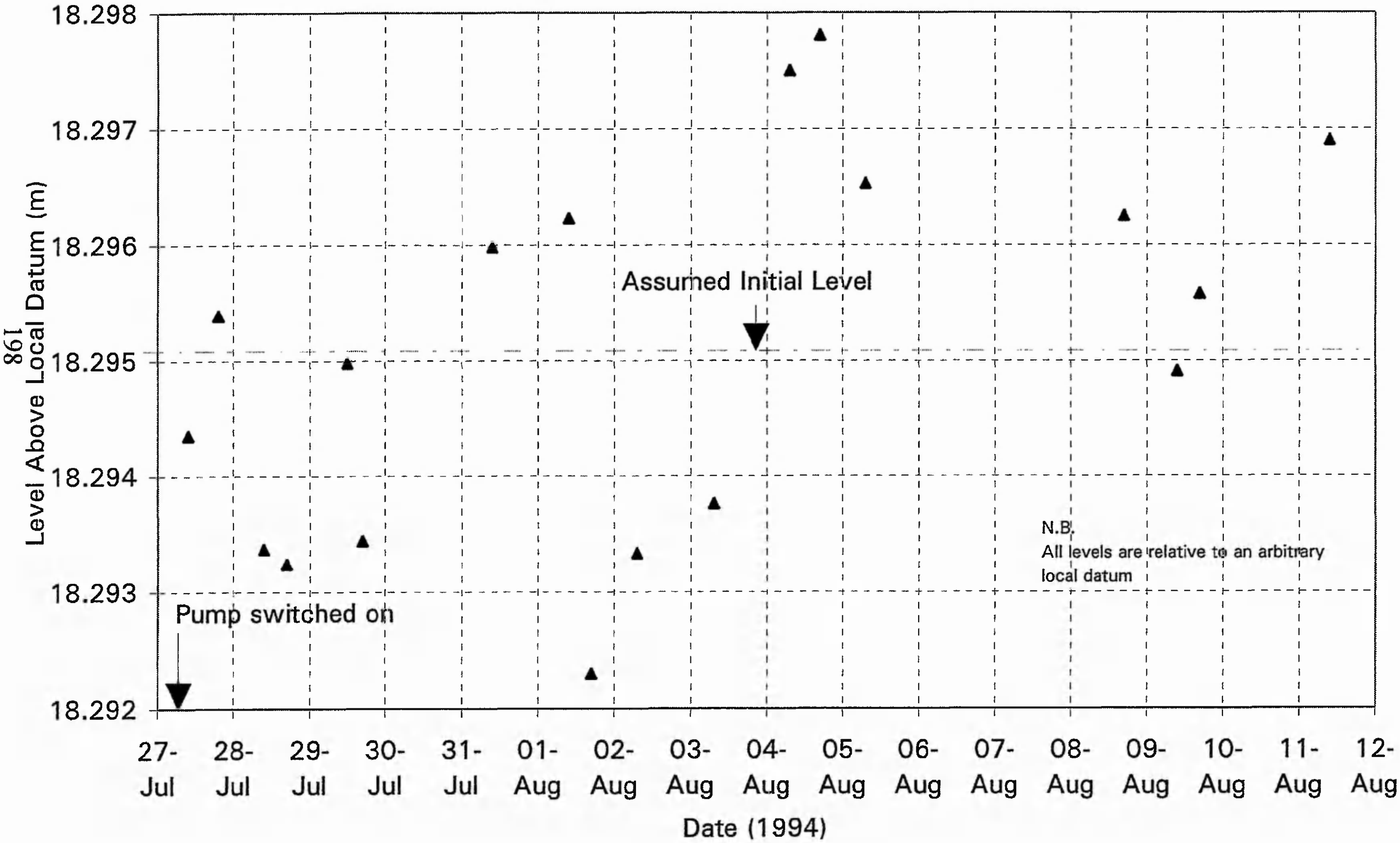


Figure 6.43
Movement of TBM F during the Pump Test

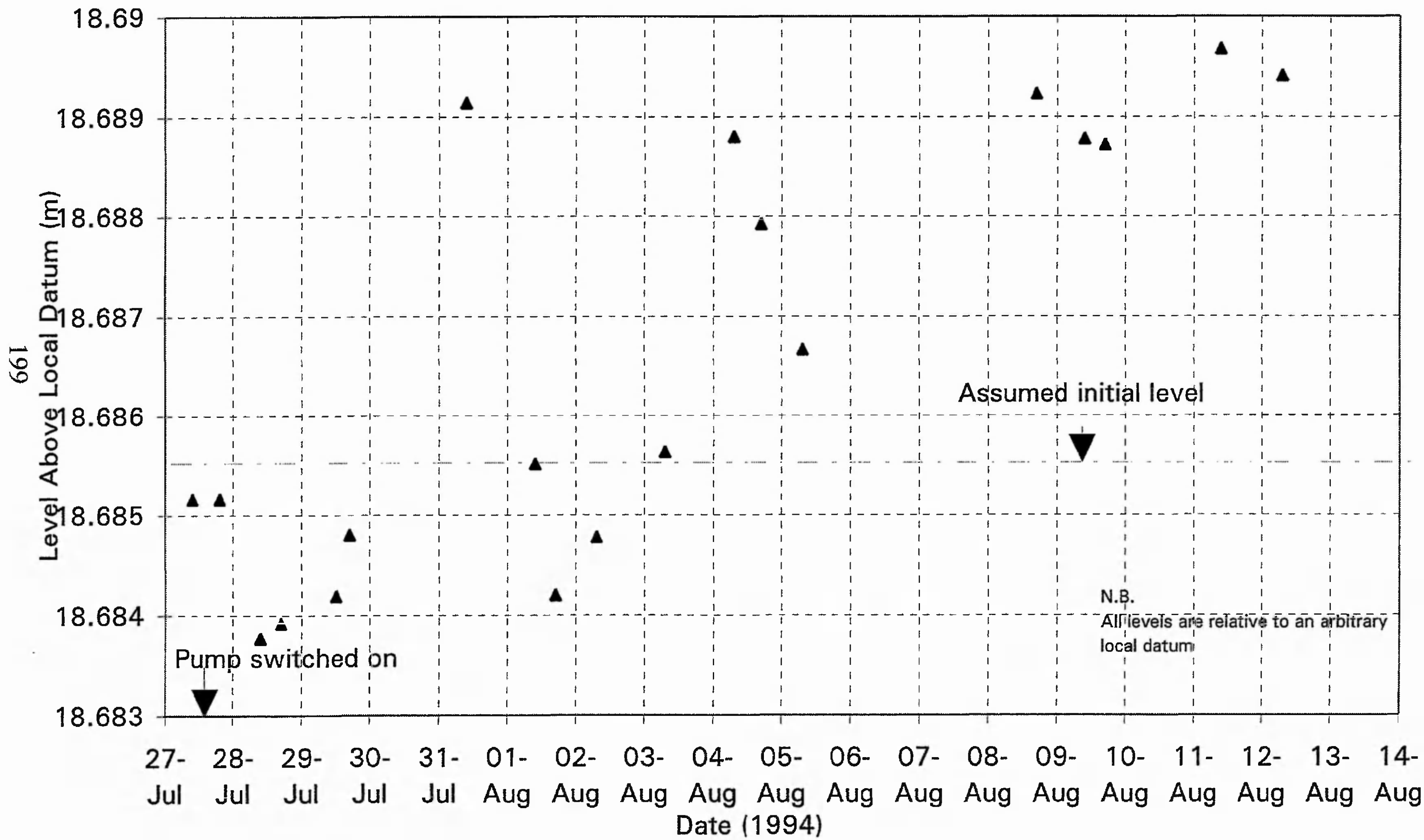


Figure 6.44
Movement of TBM G during the Pump Test

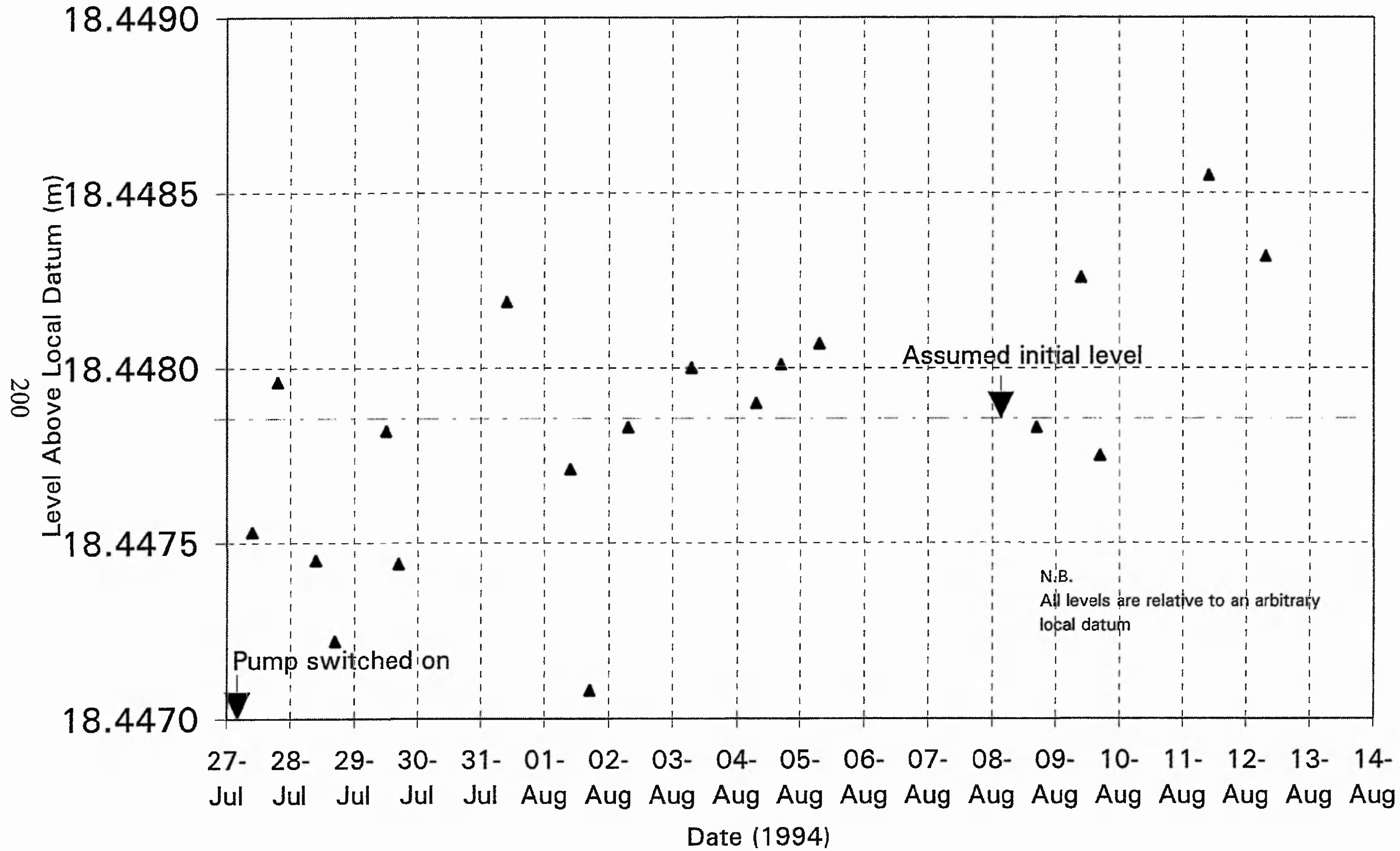


Figure 6.45
Movement of TBM H during the Pump Test

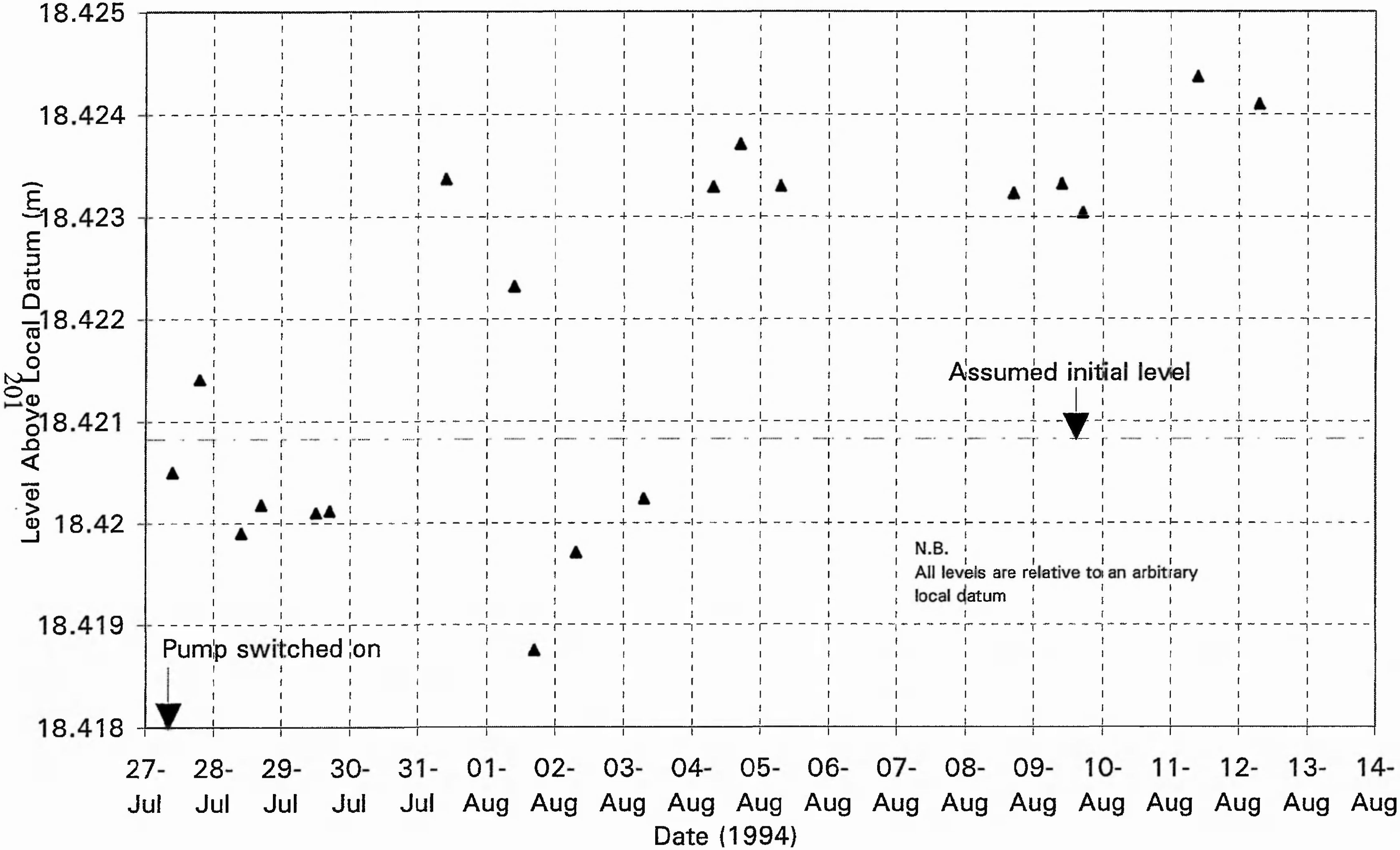


Figure 6.46
Movement of TBM I during the Pump Test

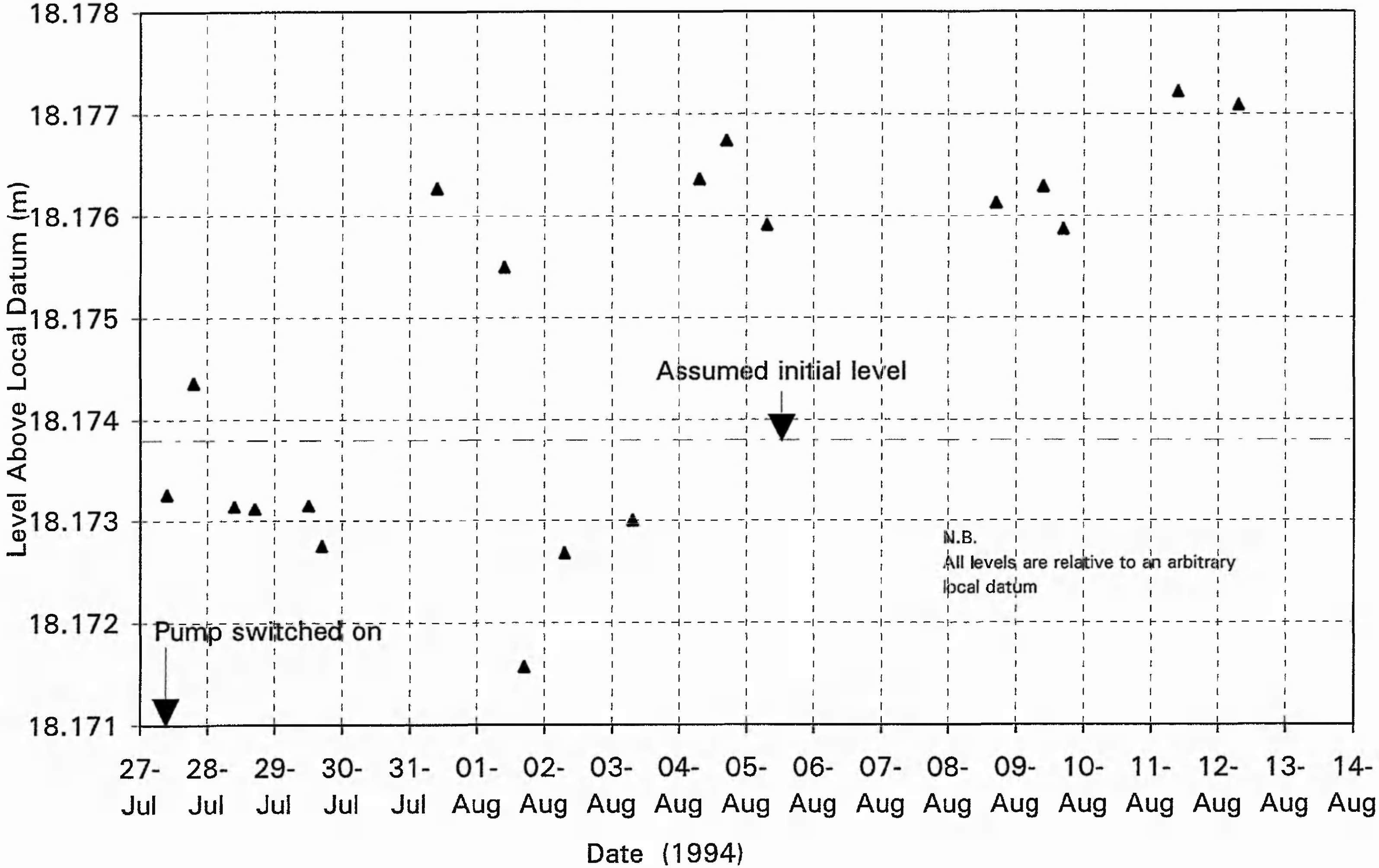


Figure 6.47
Movement of TBM J during the Pump Test

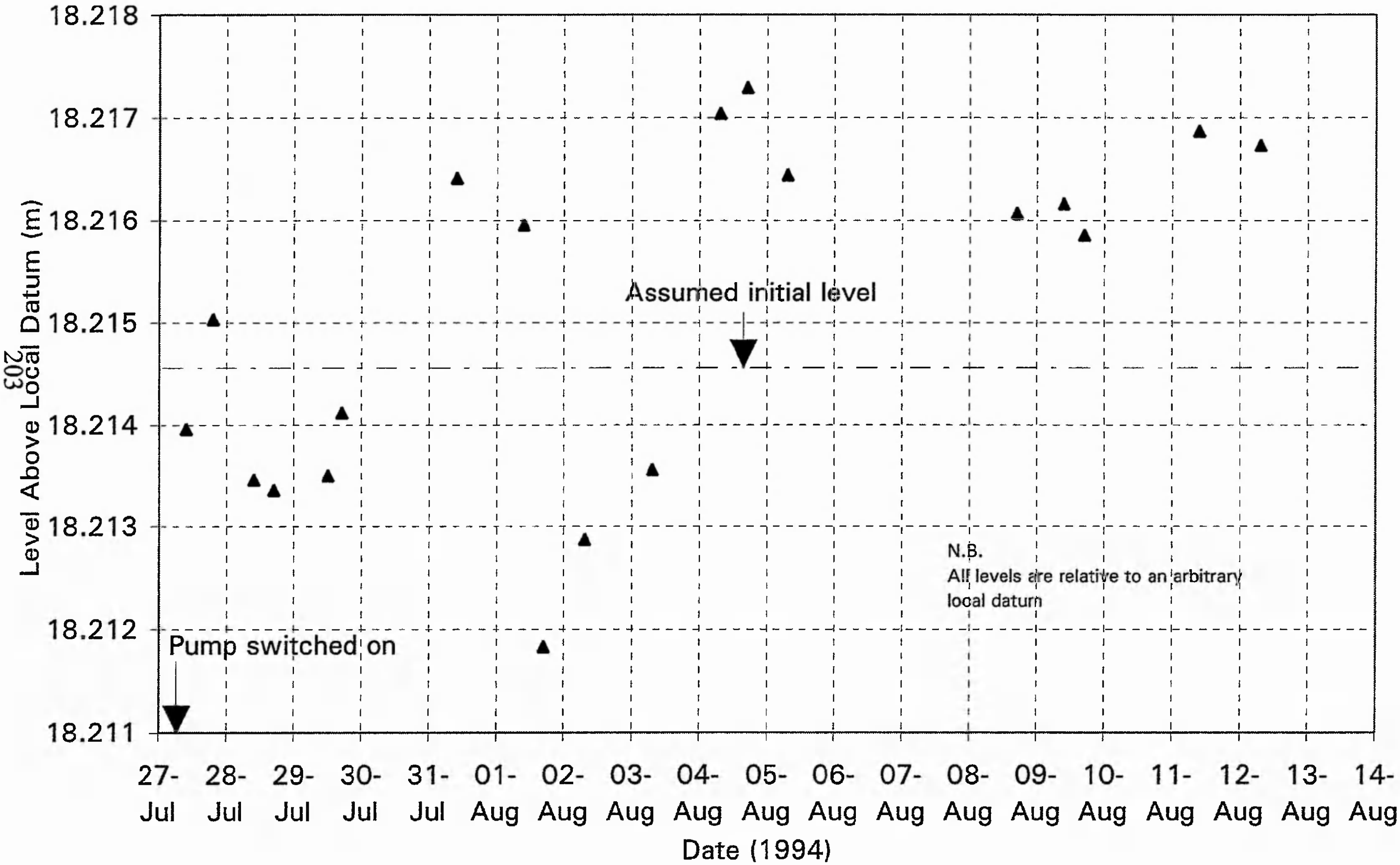


Figure 6.48

Movement of TBM K during the Pump Test

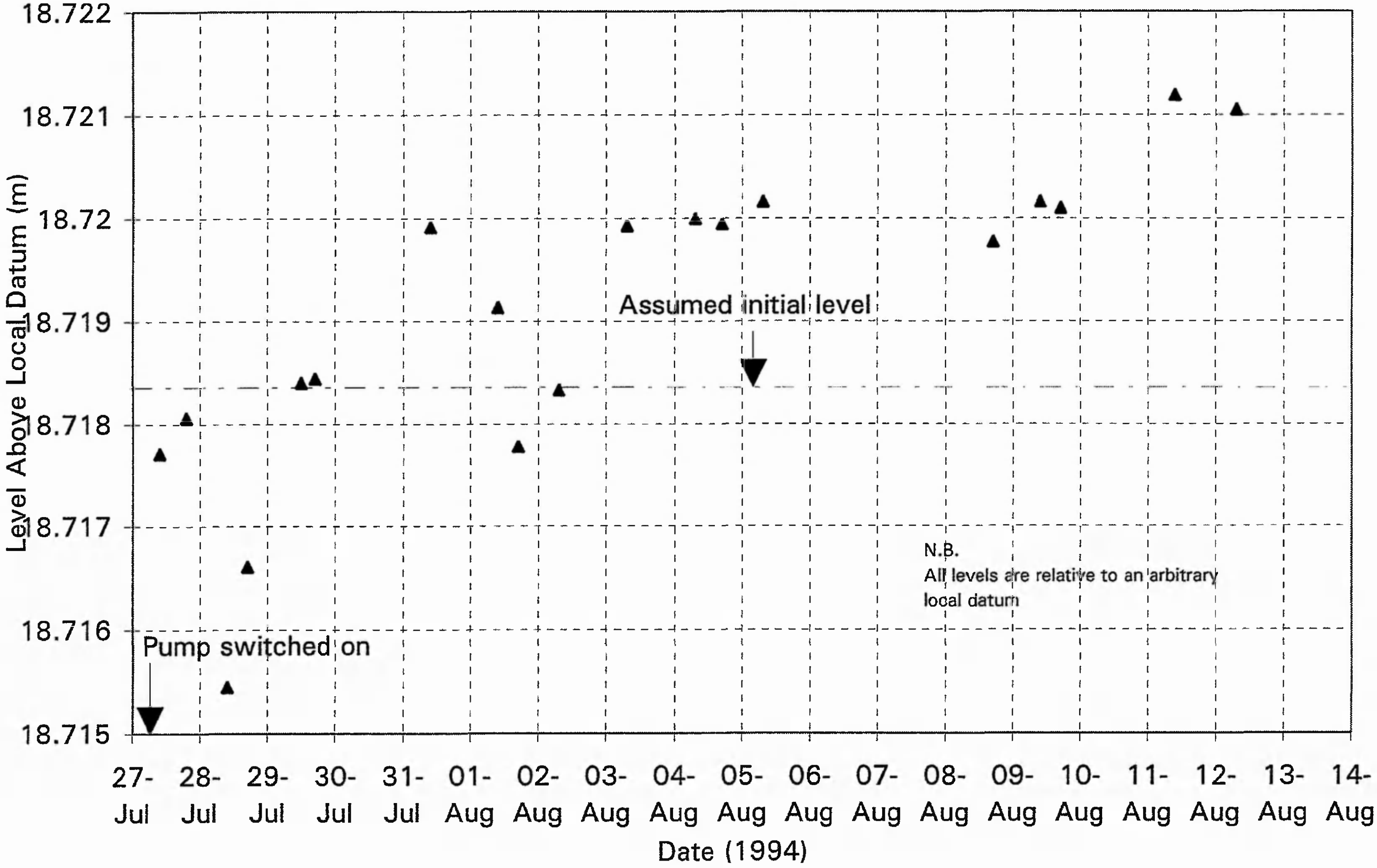


Figure 6.49
Movement of TBM L during the Pump Test

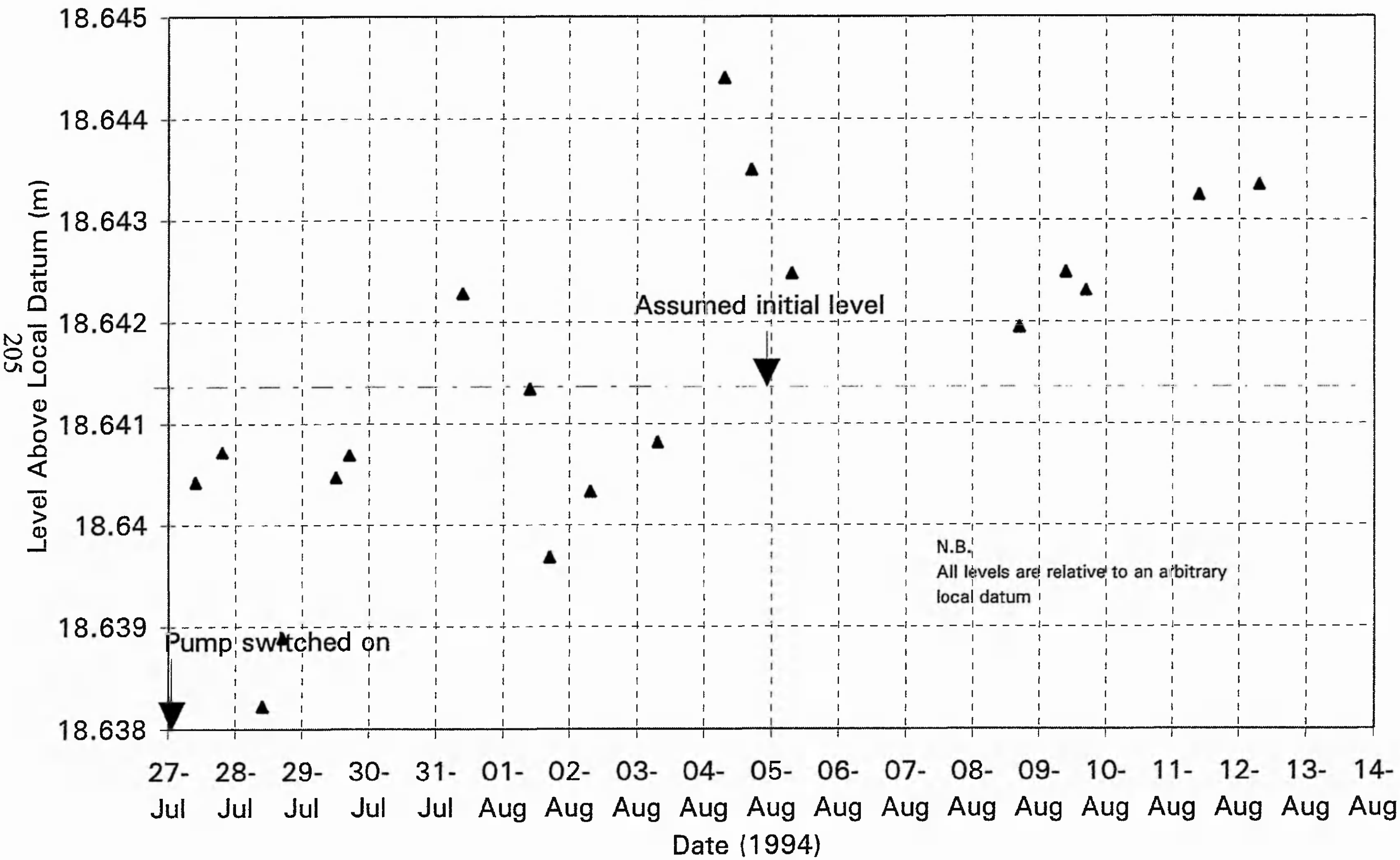


Figure 6.50
Movement of TBM M during the Pump Test

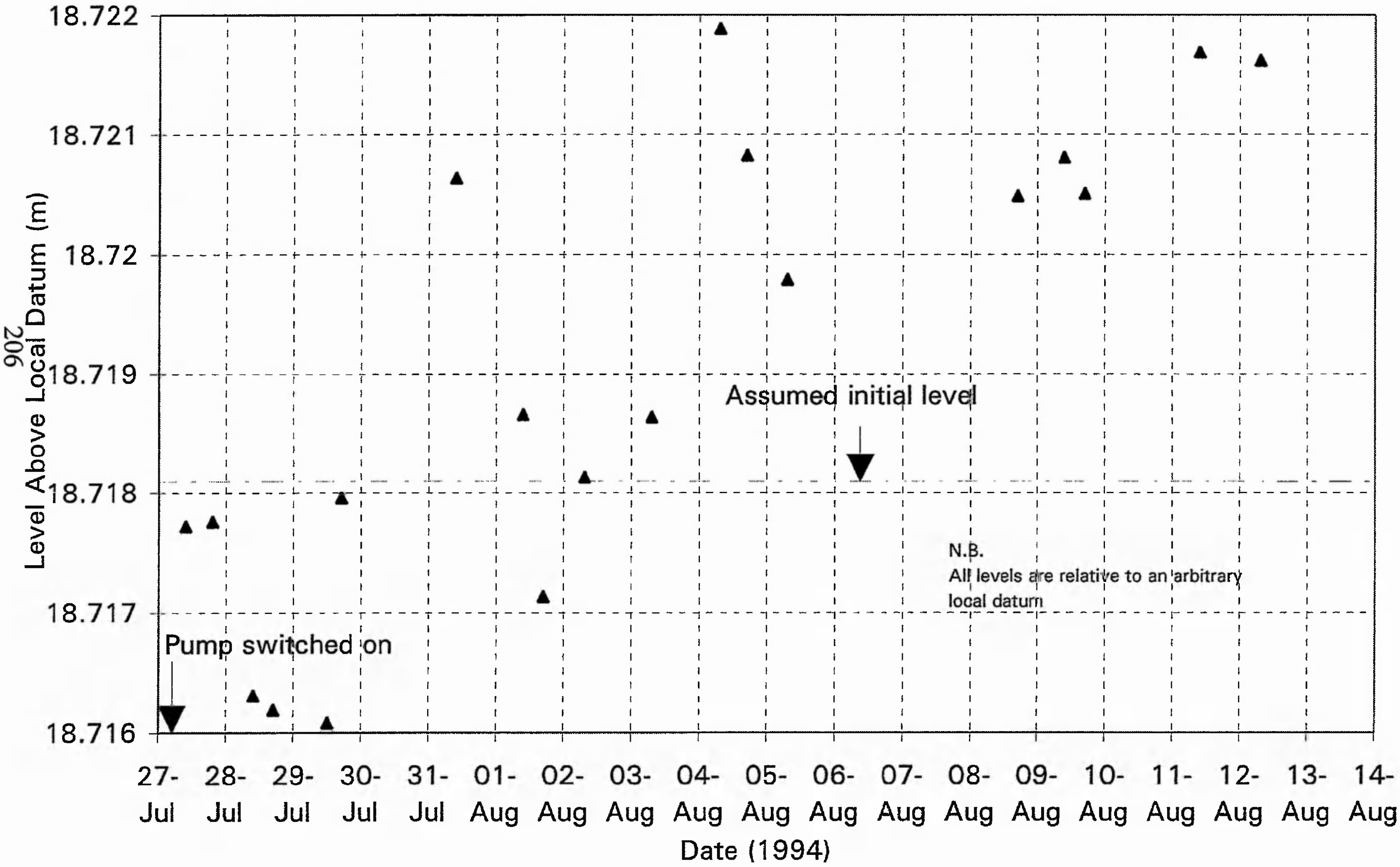


Figure 6.51
Movement of TBM N during the Pump Test

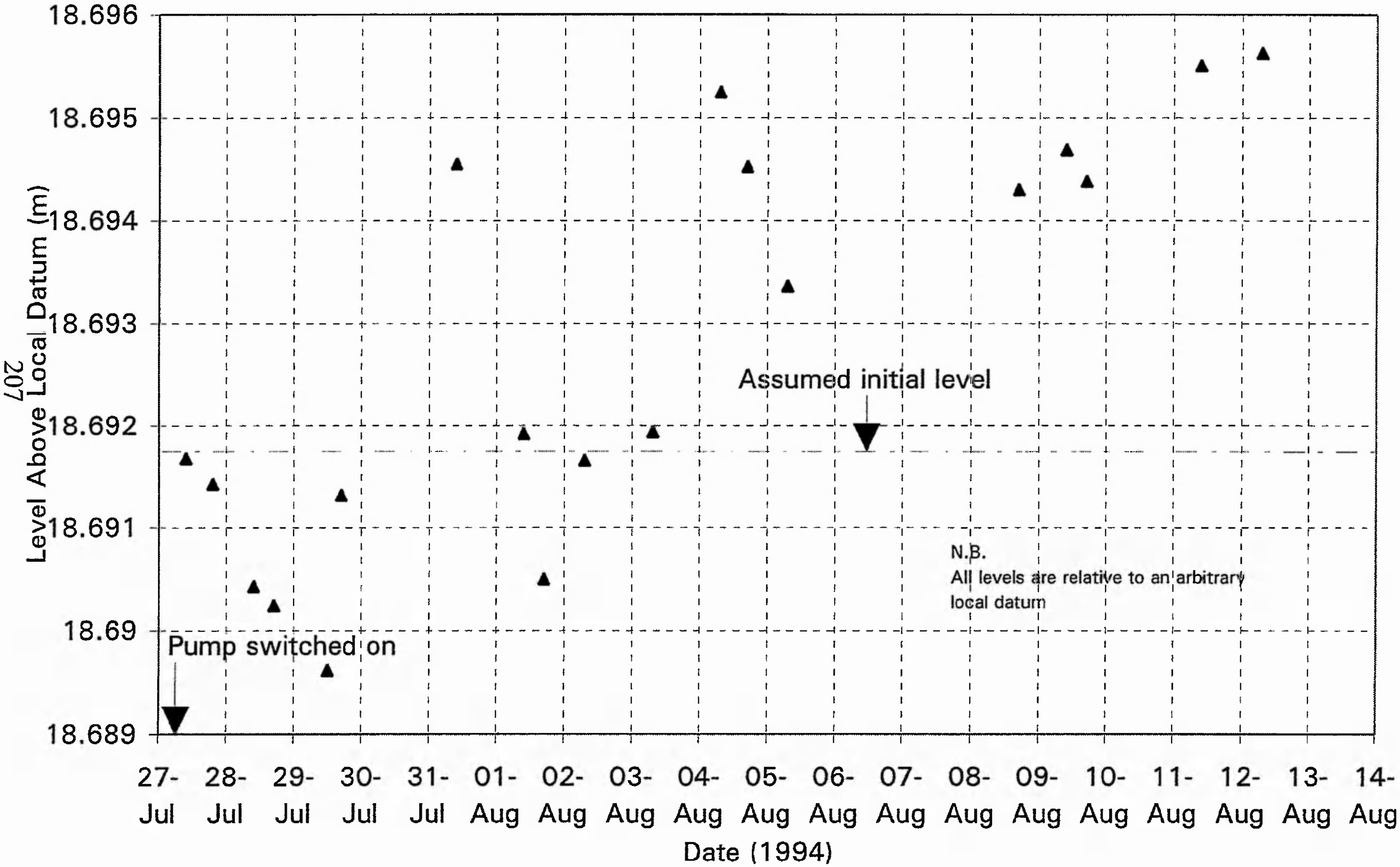


Figure 6.52
Movement of TBM O during the Pump Test

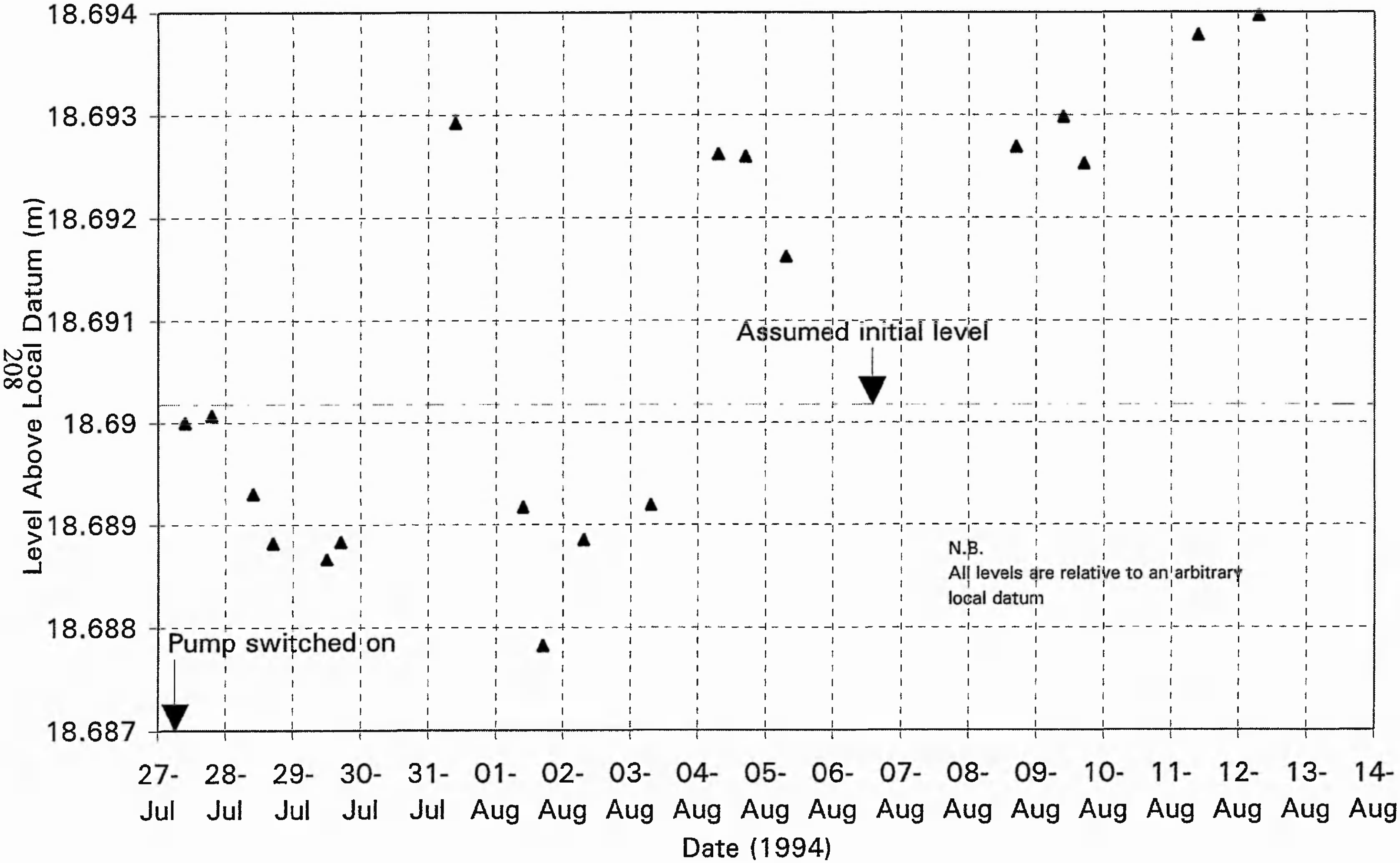


Figure 6.53
Movement of TBM P during the Pump Test

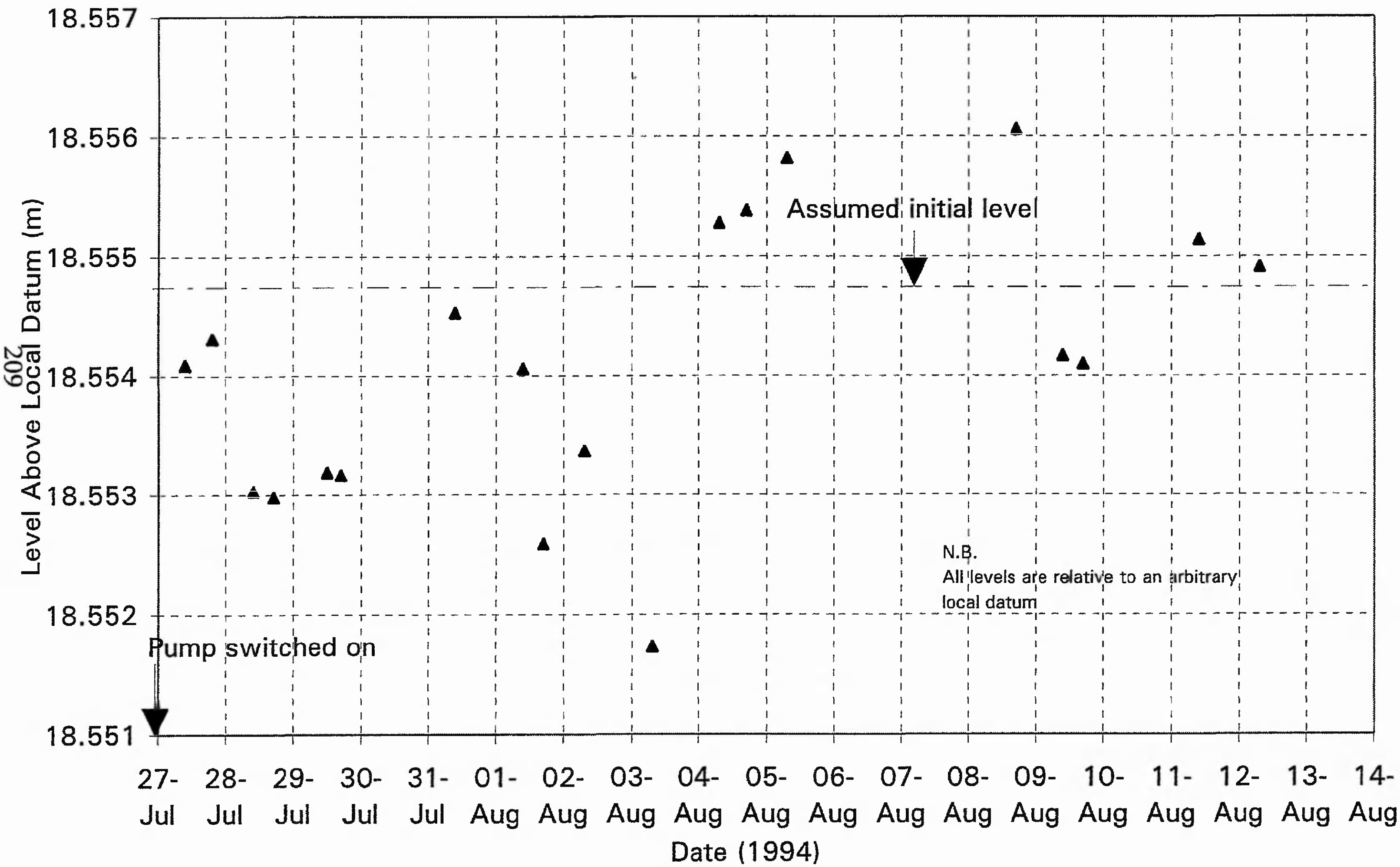


Figure 6.54
Movement of TBM Q during the Pump Test

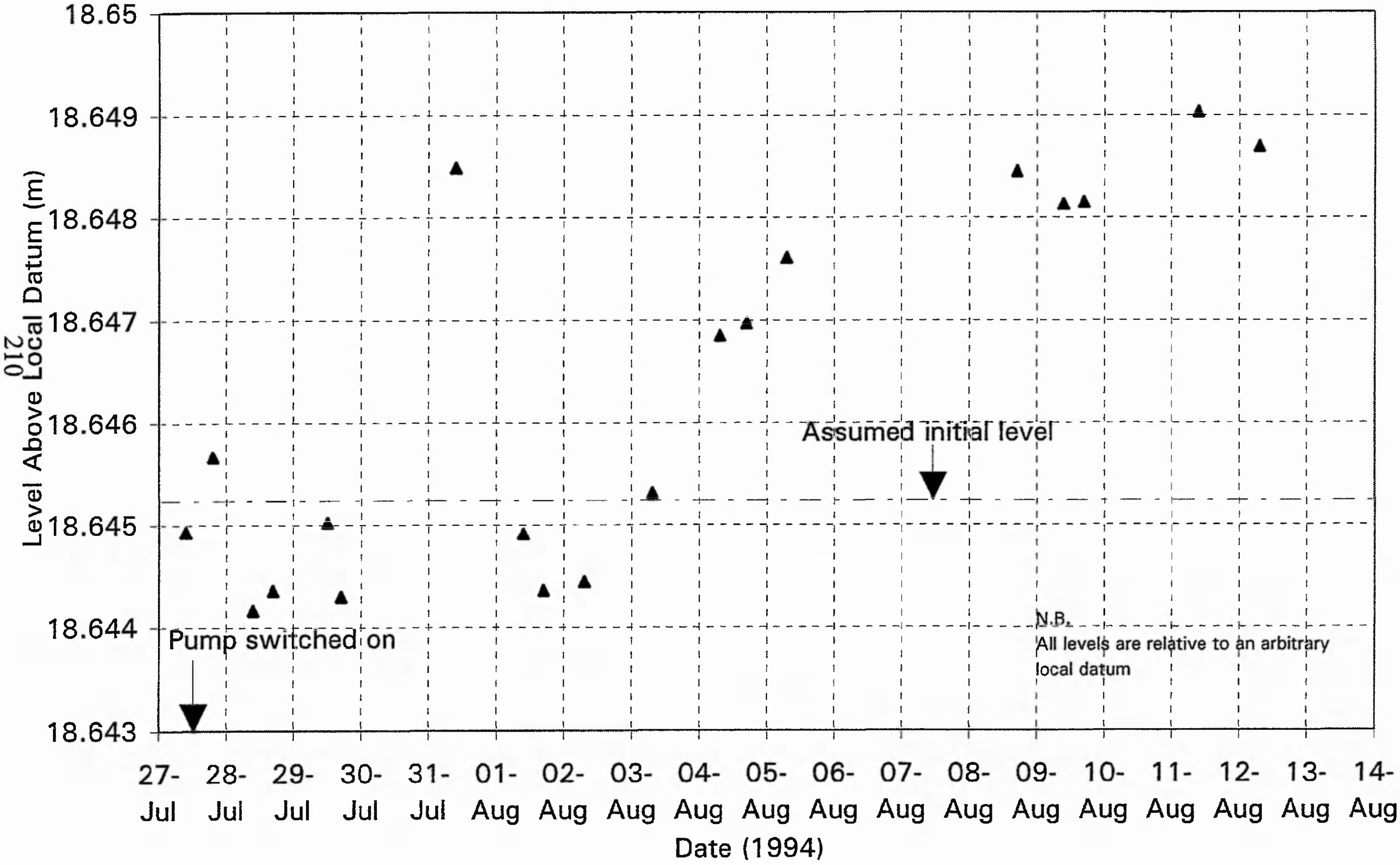


Figure 6.55
Movement of TBM R during the Pump Test

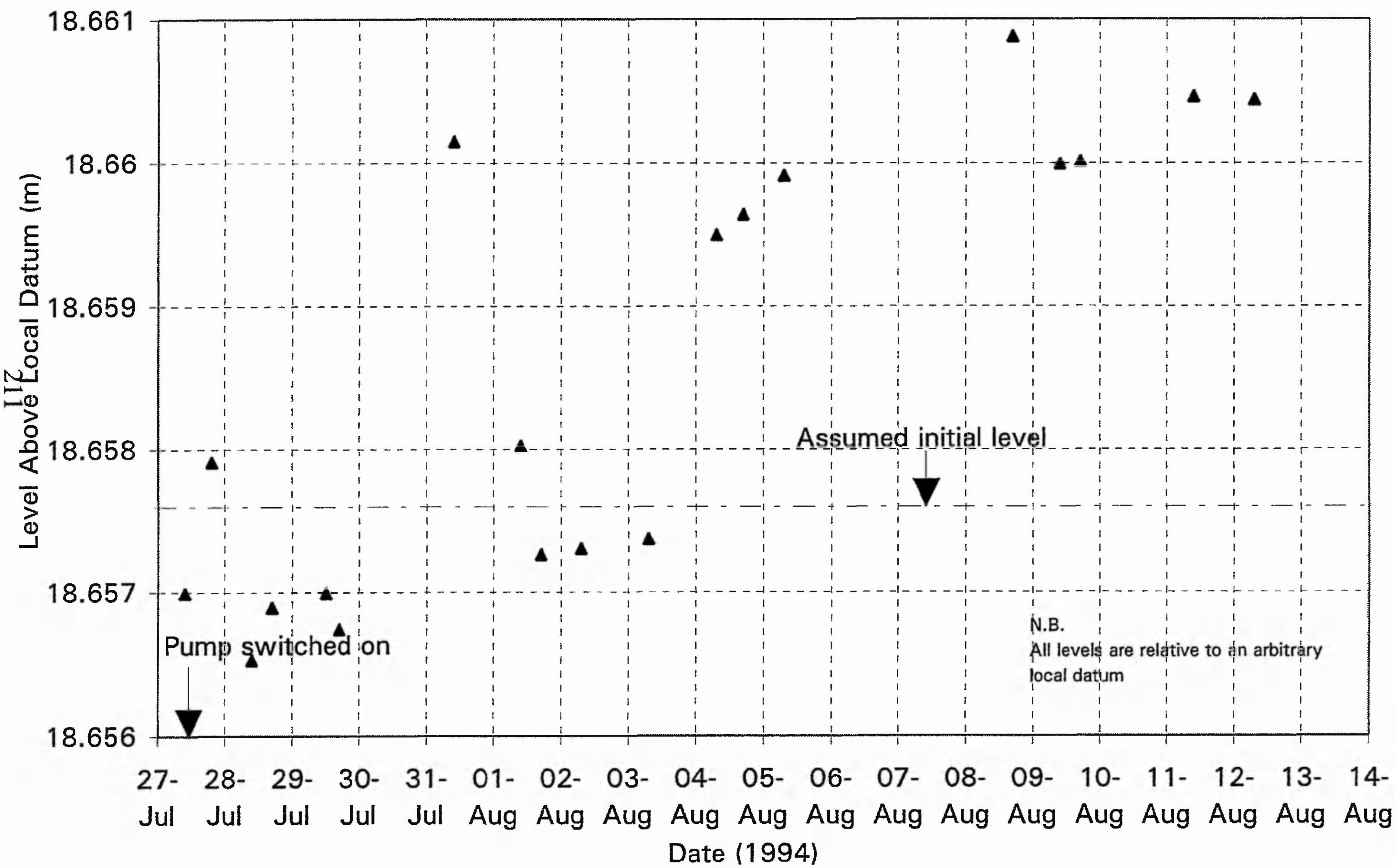


Figure 6.56
Movement of TBM S during the Pump Test

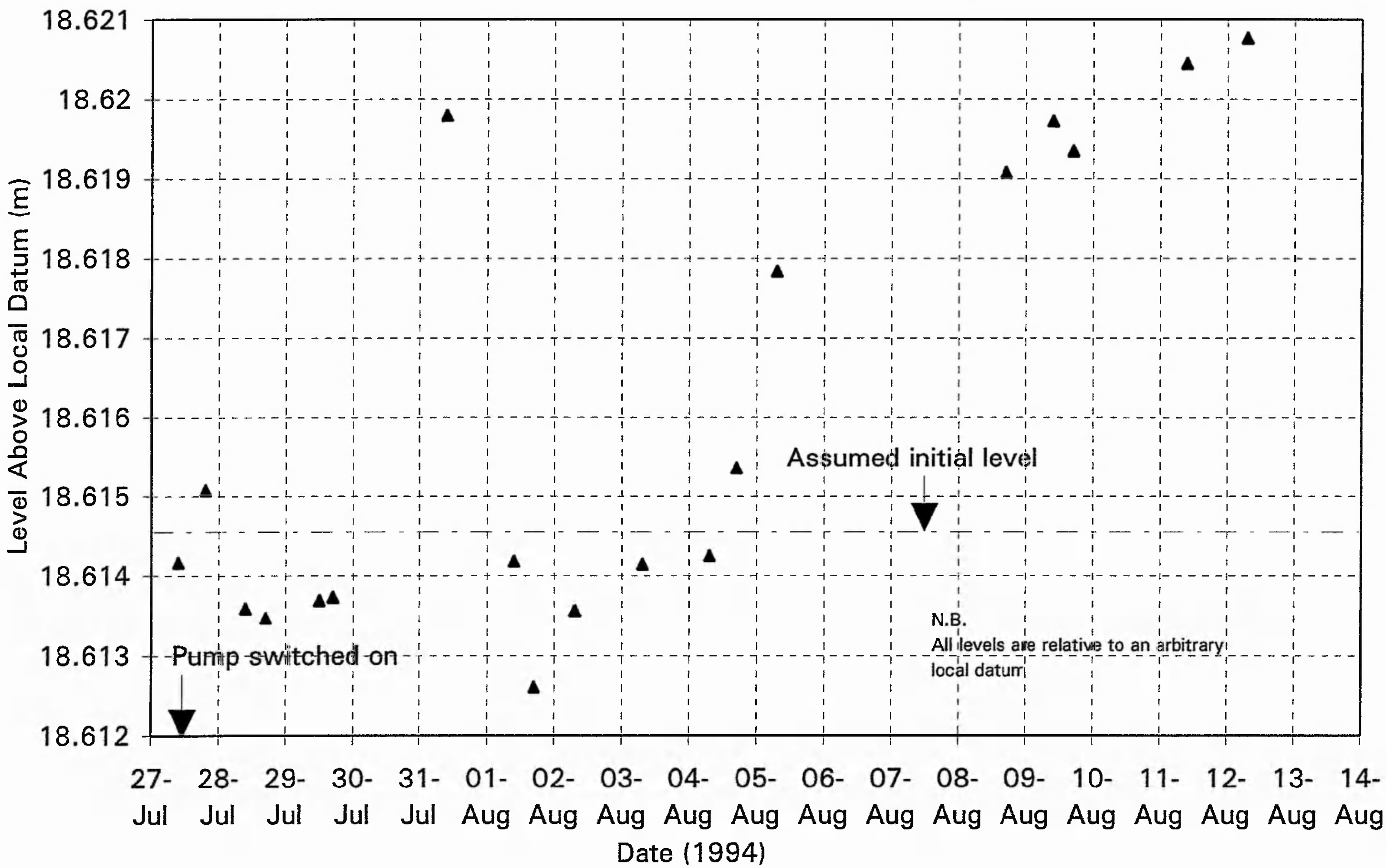


Figure 6.57
Movement of TBM T during the Pump Test

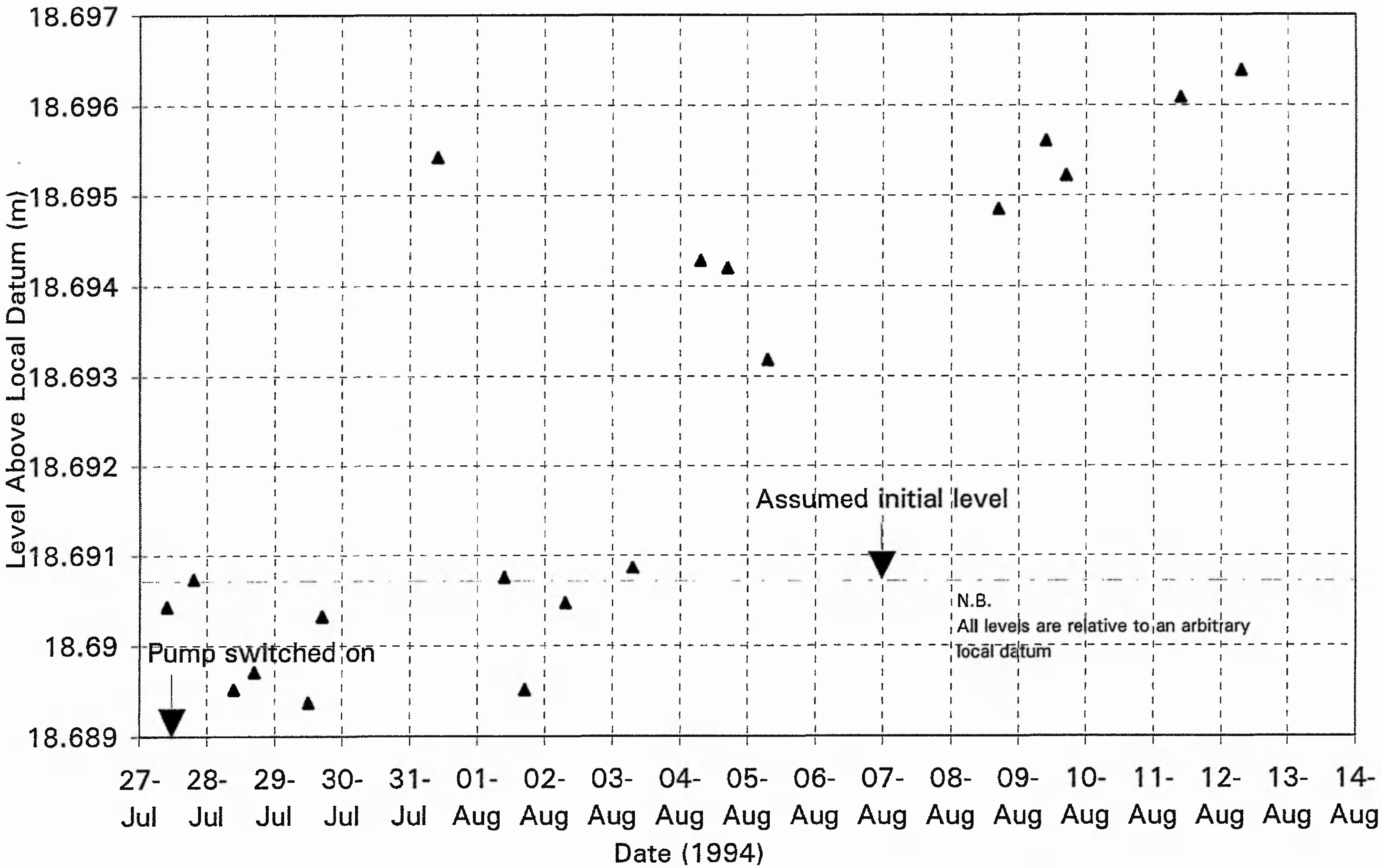


Figure 6.58
Movement of TBM U during the Pump Test

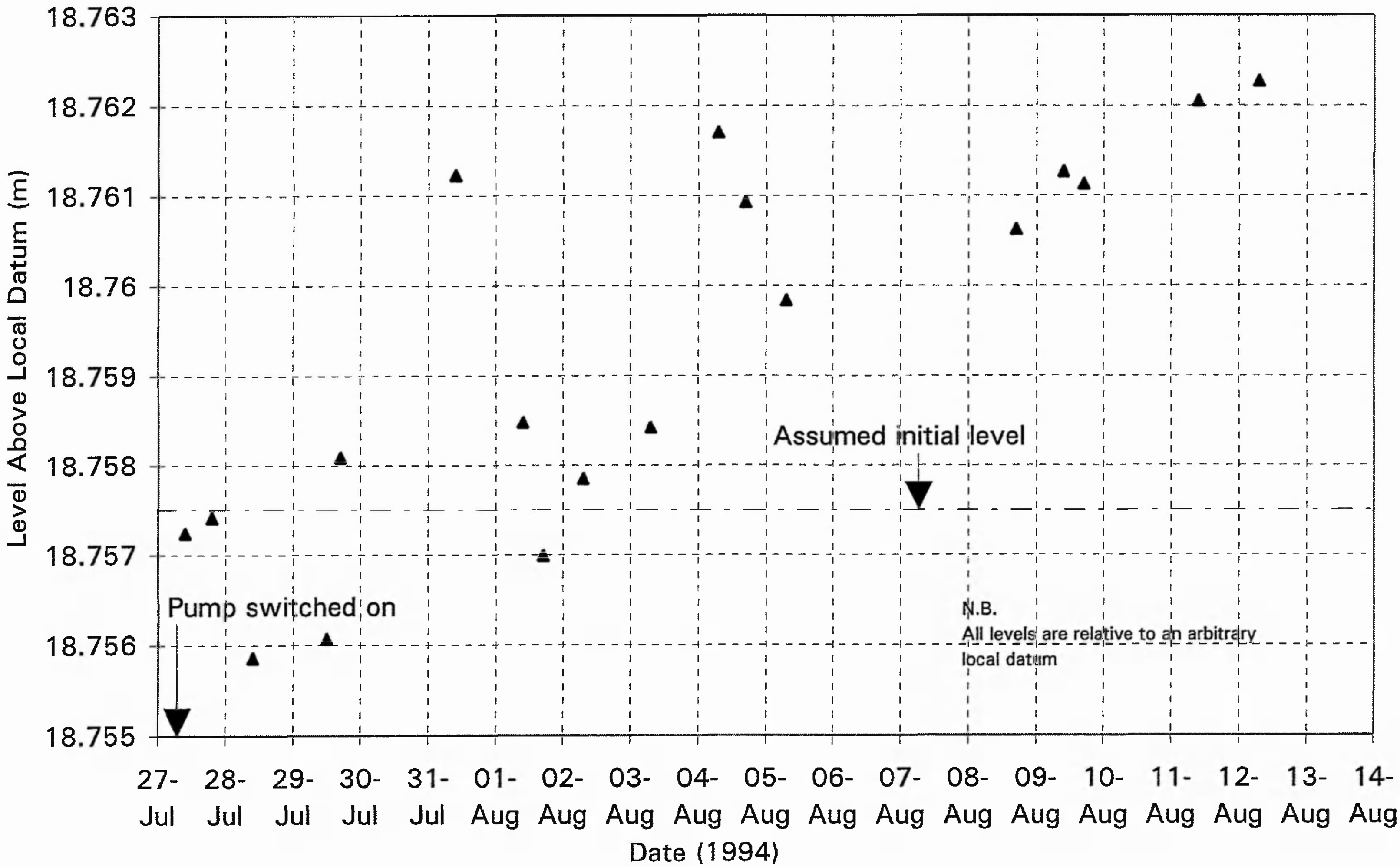


Figure 6.59
Movement of TBM V during the Pump Test

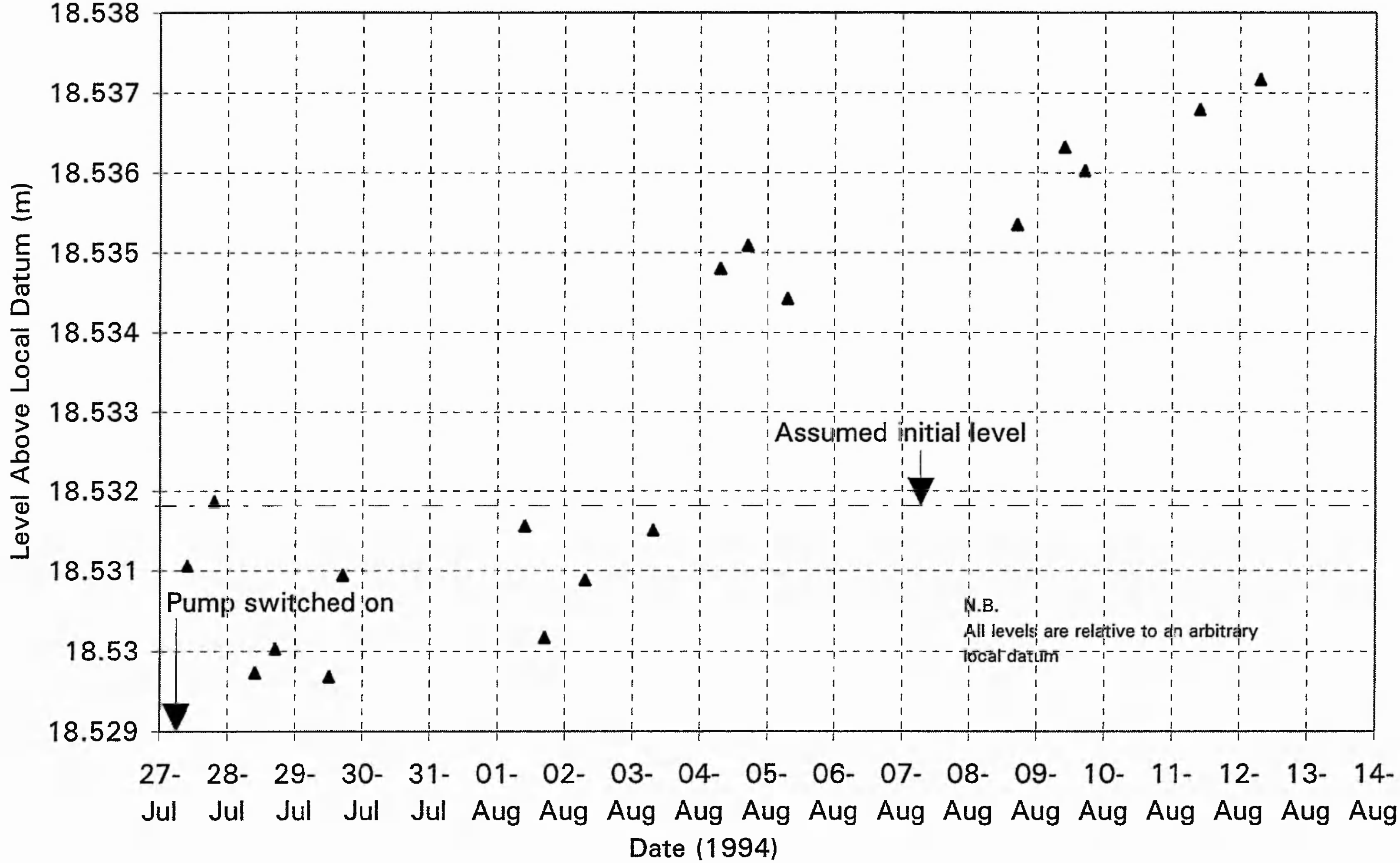


Figure 6.60
Movement of TBM W during the Pump Test

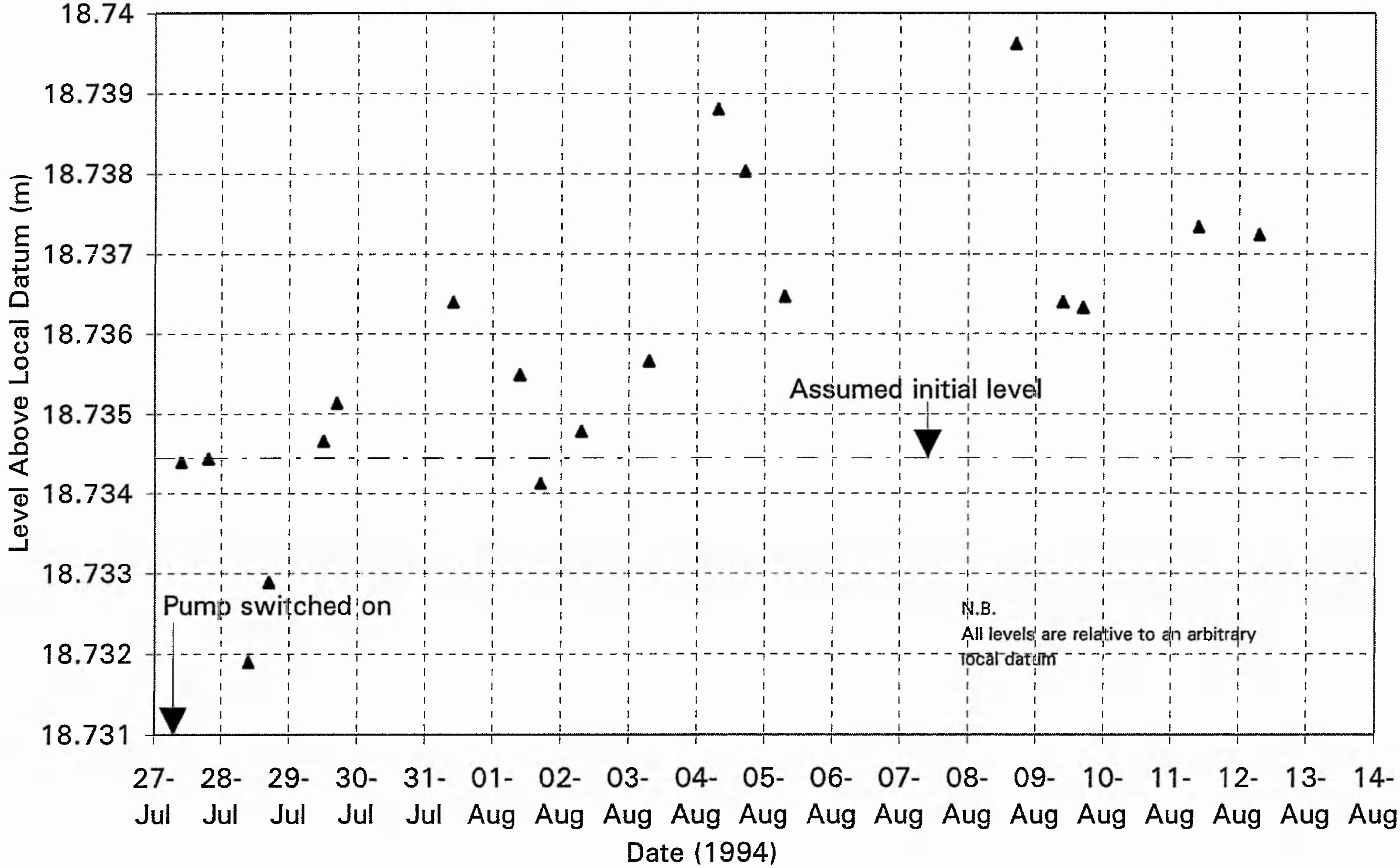


Figure 6.61
Movement of TBM X during the Pump Test

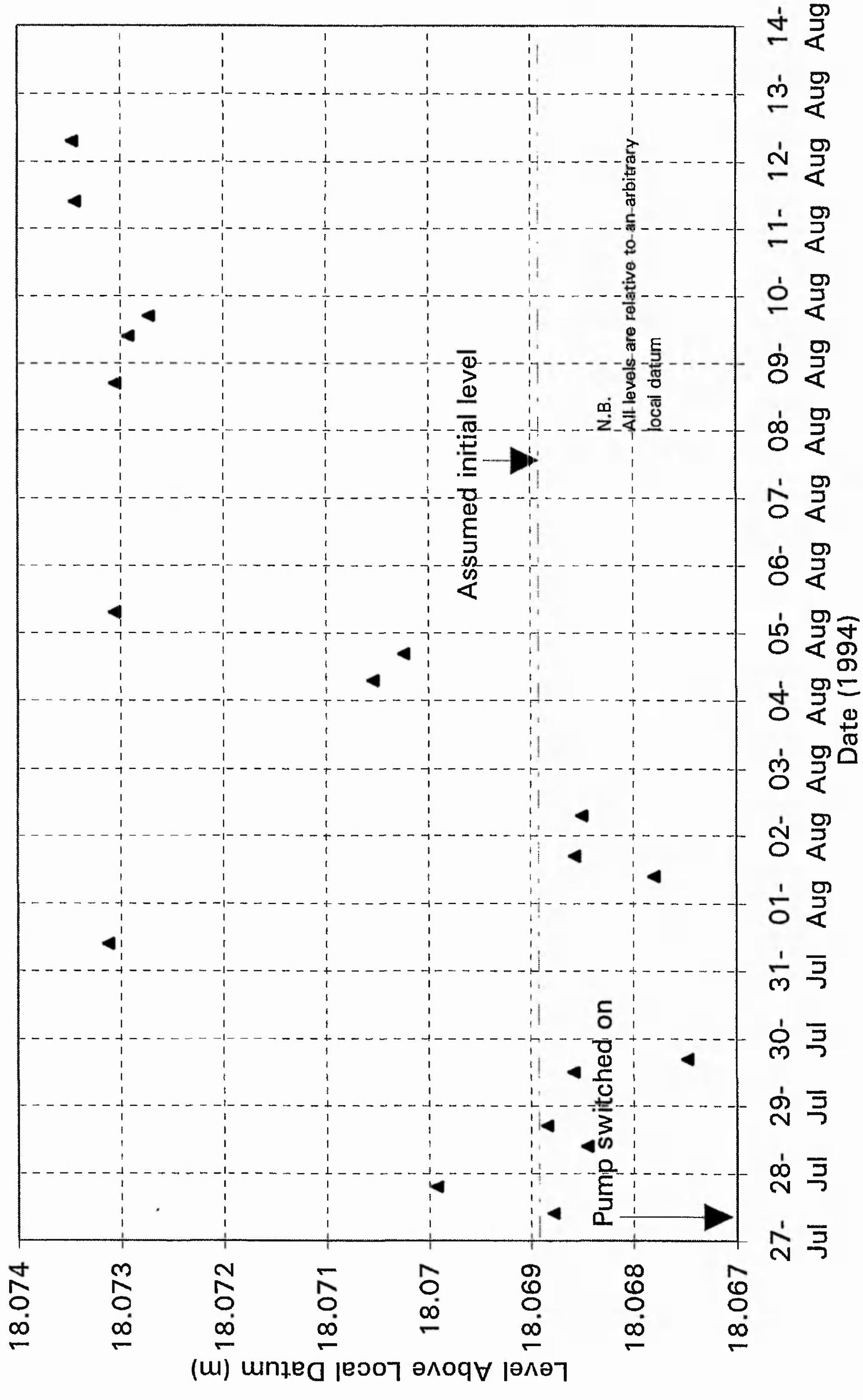
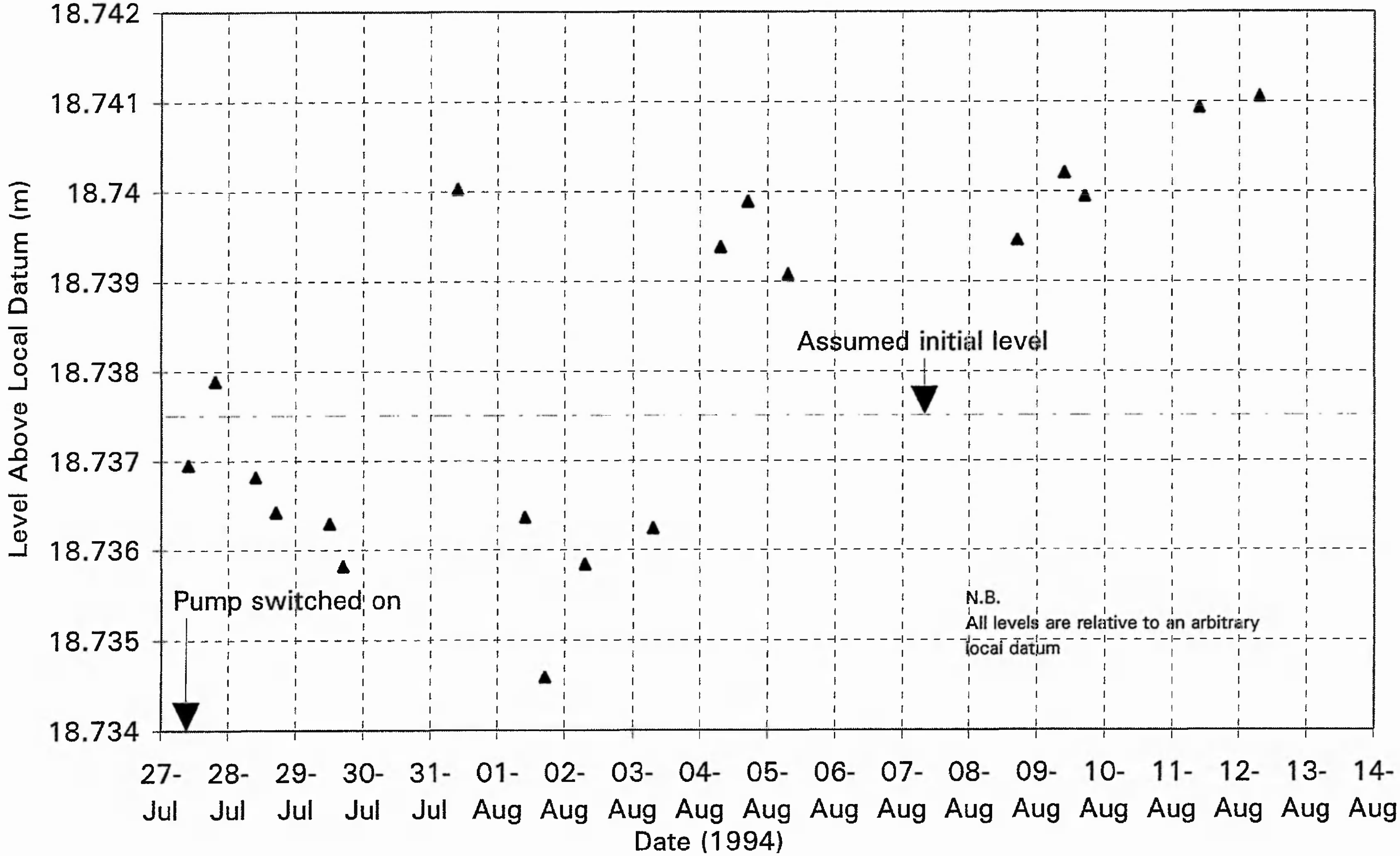


Figure 6.62

Movement of TBM Y during the Pump Test



CHAPTER SEVEN

NUMERICAL MODELLING

7.1 Introduction

This chapter summarises numerical modelling investigations of various de-watering scenarios within the unconfined aquifer around Hoveringham village. Two distinct types of modelling method were employed. In the first, the finite element method was used to model aquifer de-watering in a "quasi three dimensional" fashion. A seepage domain was defined in plan and the third dimension was given by the derived height of the water table or piezometric surface. The program used for this type of analysis was called WTABLE. The second method also incorporated the finite element method and modelled cross sections through the de-watered aquifer in axi-symmetric fashion. The program used for this type of analysis was called FLONET. Both programs were developed and are described by Bromhead (1981) so there was compatibility between the two methods in terms of data and boundary condition specification.

7.2 Quasi Three Dimensional Investigations

7.2.1 How WTABLE Works

Using WTABLE, a de-watering problem was modelled by breaking down the area of interest into a number of rectangular "elements". These elements represented the region of the aquifer to be taken into account during any given de-watering scenario. Collectively the elements are known as the "mesh" and go together to make up the "seepage domain". The corners of each element in the mesh are known as "nodes", each of which is assigned an initial water level or aquifer thickness (for confined problems) which the program uses as the initial condition state for the analysis.

It is only possible to specify strata permeabilities which are constant with depth using

WTABLE. FLONET can deal with permeabilities which vary with depth. How FLONET was used to investigate whether a "layered" soil structure has different consequences for groundwater velocity and fine particle movement is discussed in section 7.3.

Boundary conditions which can be defined when using WTABLE are shown in Figure 7.1 and are:

- **An impermeable boundary** - this is modelled by only including elements in the permeable part of the seepage domain.
- **A boundary along which there is a known pressure head** - the magnitudes of the known pressure heads are input with the edge node co-ordinate data.
- **A free surface** - this is usually found by the program using an iterative procedure after initial conditions have been specified.
- **A discharge boundary** - a segment of the seepage domain across which a given discharge is specified.
- **Infiltration** can be specified over all or sections of the seepage domain to model effective rainfall.
- **A concentrated sink or source** (such as a well or a contaminant inflow source) can be modelled by specifying an inflow or outflow at a particular node.
- **The physical boundary of the seepage domain** is modelled by simply coming to the edge of the mesh.

The output from a WTABLE analysis is the elevation of the groundwater table or piezometric surface. The program can model seepage in the confined or unconfined state and can deal with unsteady flows.

For the investigations described in this report, WTABLE was only used in the unconfined mode to model drawdown and seepage within the shallow, alluvial deposits of the Trent Valley.

7.2.2 *Objectives Of The Quasi Three Dimensional Analyses*

The objectives behind the quasi-three-dimensional investigations were:

- To calibrate the WTABLE program and parameters used against data recorded during the pump test and the groundwater contour maps presented by the quarry operator. This would test the accuracy of assumed parameters prior to including them within the main modelling analyses. Any parameters found to be significantly in error would be revised during the calibration and the more suitable values adopted for use during the cases discussed below.

- To model the pumping test discussed in Chapter 5 investigating permeability variations and groundwater velocities (Reynold's No's of flow) required to cause any observed variations in drawdowns or settlements.

- To model de-watering of only the "Existing Quarry" area shown in Figure 7.2 examining drawdowns, distances of influence, and groundwater velocities (Reynold's Numbers of flow) for various combinations of feasible permeability and infiltration values. The models would be done both with and without the specification of very low permeability zones to simulate the P.F.A infilled areas.

- To model de-watering of only the "Ash Lagoon" areas shown in Figure 7.2 examining drawdowns, distances of influence, and groundwater velocities (Reynold's Numbers of flow) for various combinations of representative permeability and infiltration values. The models would be done both with and without the specification of very low permeability zones to simulate the P.F.A infilled areas (marked "Ash Lagoon" in the Figure).

- To investigate the effects of specifying high permeability channels within the aquifer on drawdowns, distances of influence and velocities of flow (Reynold's Numbers of Flow).

- To model "worst case scenarios" in terms of water table drawdowns underneath Hoveringham village for each of the combinations of de-watered quarry area discussed above. Highest permeability values for the aquifer would be drawn from previous site investigation information in order to create these "worse case scenarios".

7.2.3 Parameters Specific To Hoveringham De-Watering Problems And Their Specification Within The Models

7.2.3.1 The Mercia Mudstone Boundary To The North-West

It has already been discussed (in sections 3.3.3 and 3.4.1) how the outcrop of the Mercia Mudstone defines the North-Western boundary of the floodplain gravels. A series of sensitivity analysis runs were done with the purpose of investigating the effects of very low permeability northern edge elements (to simulate the Mercia Mudstone boundary) and also very low permeability eastern and western edge elements. The purpose was to determine whether the shape of the derived groundwater drawdown contours were significantly different with and without low permeability elements. It was noticed that using similar de-watering conditions, the groundwater contours came out very similar whether low permeability elements were specified or not. This indicated that WTABLE would not attempt to "draw water from outside of the mesh" and that the permeability of edge elements furthest from the de-watered point would have little effect on the results of the analysis for the internal elements.

The Mercia Mudstone boundary to the north-west of the quarry was therefore specified as a "physical boundary of the seepage domain". In practice this was simply the edge of the mesh.

7.2.3.2 Downstream And Upstream Continuations Of The Aquifer

No conditions were specified for the upstream and downstream boundaries. They were simply fixed far enough away from any de-watered area for infiltration to keep the groundwater contours completely inside the mesh. It was found that upstream and downstream continuations of the aquifer could not be modelled by specifying water levels at the eastern and western edges of the mesh as this simply had the undesired effect of "stretching" the groundwater contours out to meet this boundary. Only at lower infiltration values would the groundwater contours reach the upstream and downstream boundaries which only then would be given fixed water table levels. This seemed reasonable as in theory, beyond the last contour, negligible flow towards the de-watered void should occur. The respective distances at which western and eastern boundaries were fixed from the quarry edge were 1900 metres and 1400 metres.

7.2.3.3 Recharge From Local Watercourses And The Northern Mercia Mudstone Boundary

Preliminary calculations and the groundwater contour maps from the planning application suggested it was unlikely that the distance of influence away from the de-watered quarry had reached the River Trent in the south. Preliminary attempts were made to model the river as a fixed water level boundary but as discussed above this had the unwanted effect of 'expanding' the distance of influence towards the river as the program began to reduce water levels from immediately inside the fixed river level. The river could not be specified as a recharge boundary as the rate of recharge was unknown. It was therefore decided not to specify the river at all until (under very low infiltration) the distance of influence reached the southern boundary of the model.

Many streams, ditches and irrigation channels cross the area of interest. Some of these water courses divert runoff from the northern mudstone bank to other more major streams in the area. Many of the channels appeared to have been clay lined to decrease leakage back into the aquifer. This would seem sensible especially in the case of the runoff channels leading from the clay bank, as their primary purpose would be to keep excess water out of the floodplain aquifer.

In order to be able to model aquifer to stream continuity or recharge from the mudstone bank, it was necessary to answer the following two questions:

- Does any water incident on the clay bank as rainfall actually find its way into the floodplain aquifer? (i.e. become recharge water)

- Is there any degree of hydraulic connection between watercourses and the unconfined aquifer on the floodplain?

Normally, finding answers to these questions might mean extensive amounts of field investigations of stream flow rates. However, it was fortunate that in their most recent planning application (see section 3.2), the quarry operating contractor included results of stream flow rate measurements with which it was possible to gauge any degree of hydraulic connection between streams, ditches and aquifer.

In order to calculate whether any recharge to the aquifer occurred from the clay bank, catchment analyses were done using 1:50,000 contour maps of Nottinghamshire and other maps included within the above mentioned planning application. Conclusions from these calculations were as follows:

1) Negligible recharge to the aquifer occurs from the streams, watercourses and ditches on the floodplain. (i.e. for the purpose of the finite element investigations, zero connection between watercourses and aquifer was assumed)

2) If drainage ditches were not diverting water from the northern bank and evapotranspiration is assumed at 10% then the recharge to the aquifer from the clay

bank is estimated as 325 m^3/m run per year. For an evapotranspiration rate of 50% and no diversion, recharge is 177 m^3/m run per year. However, a worst case scenario for the de-watering (and apparently the most likely one as the drainage ditches appeared to be in good working order) is that no recharge occurs from the clay bank to the aquifer. In any case, various combinations of the above scenarios were investigated.

7.2.3.4 Quarry De-Watering And Water Table Lowering Method

WTABLE has two facilities for simulating pumping or piezometric level lowering operations. Either a low value for the water table or piezometric level can be specified at chosen nodes (perhaps quarry edge nodes) or a pumped discharge rate can be specified from nodes or lines of nodes to simulate individual wells or wellpoints.

In the case of the de-watering operations at Hoveringham quarry, both the depth of water table lowering and the pump rate used are approximately known. They are respectively 6 to 6.5 metres, and 50 litres per second. It was thought better to specify low water table levels at quarry edge nodes within the model rather than extraction rates as the level of the water always had to be kept low by the contractor during times of extraction and it was not known whether the pump rate would vary during wet or dry periods. In essence, the fixed low water table level was the key parameter for the quarry operating contractor.

Sensitivity analyses were done in which the difference in level between bedrock and low water level was varied from 0.2m -1.0m, while the maximum water level was kept at 7.0m. There were no significant differences in the shape of the groundwater contours or the distances of influence away from the de-watered zone on any side. It was concluded from these runs that so long as the maximum water level drawdown is fixed, and the difference in level from bedrock to low water level is varied between reasonable values, the groundwater contours and distances of influence calculated would show no significant variation.

7.2.3.5 Rainfall And Effective Rainfall (Infiltration)

The terms "rainfall" and "effective rainfall" refer respectively to the total amount of rain incident on the ground and the proportion of that rain which then finds its way to the main body of groundwater. Differences between the two amounts are due to evapotranspiration, satisfaction of soil moisture deficit and runoff to streams. The relative importance of the above three factors depends chiefly on local climate, type of soil and topography. Investigations were carried out to estimate these quantities for the Hoveringham area. The conclusions were:

- Total annual rainfall for the area is approximately 650mm.

- 40% of rainfall is lost through evapotranspiration for the geology and climate of the study area.

- It is assumed that zero runoff to streams and watercourses occurs in the study area.

- The annual average effective rainfall (or "infiltration") rate for the study area is 390mm/year.

Using WTABLE it was important to specify the correct infiltration value as this parameter significantly affected the distance of influence. Sensitivity analyses showed that using half the annual infiltration for a de-watered zone in the centre of a mesh increased the distance of influence by 30%. The distance of influence increased by a greater amount than this for lower infiltrations on the side of a de-watered zone which was near to a boundary. Due to the findings from boundary condition sensitivity analyses, infiltration was used as the main variable to change distances of influence (i.e. simulating wetter or dryer conditions) and boundary conditions were only specified if groundwater contours reached the mesh boundary.

7.2.3.6 Range Of Aquifer Permeability Values

The three main sources from which permeability values were drawn to represent the range of feasible values were published literature, the planning application submitted by the quarry operating contractor and the pumping out test discussed in Chapter Six.

British Standards Institution Code of Practice No.8004:1986, "Code of Practice for Foundations", gives ranges of values of permeability for various materials as shown in Table 7.1. For the type of floodplain deposits for which the Trent Valley is quarried, upper and lower feasible permeability limits respectively are 5×10^{-2}m/s and 1×10^{-5}m/s.

The planning application quoted permeability values determined from in-situ Rising and Falling Head Tests, and Hazen (particle size distribution) analyses. Rising head test values varied from 3.8×10^{-5}m/s to 1.2×10^{-4}m/s. Falling head test values varied from 9×10^{-6}m/s to 3.3×10^{-5}m/s. Hazen values ranged from 2.4×10^{-4}m/s to 8.9×10^{-4}m/s.

Values calculated from the pump test based on levels of water in pairs of standpipes ranged from 3.0×10^{-4}m/s to 7.9×10^{-4}m/s.

7.2.3.7 Size Of De-Watered Quarry

Information relating to the size of, order of extraction within, pumping from, and order of infill within the quarry were gained both from local people and the contractor's planning application. This information is summarised in Figure 7.2. The order of extraction in the quarry was area R1 first, then area R2, and then area R3. It appears that areas R1 and R2 were open and quarried at approximately the same time and as a worst case scenario for the structures within Hoveringham village, areas R2 and R1 were de-watered to the maximum extent simultaneously. Area R3 is the largest but it is also known that not long after extraction had commenced in R3, areas R1 and R2 had been backfilled with pulverised fuel ash.

Therefore, the two main cases used in the WTABLE runs were:

1. Areas R1 and R2 de-watered simultaneously to the maximum extent

2. Area R3 de-watered to the maximum extent while areas R2 and R1 were specified as being "filled with pulverised fuel ash" (i.e. given very low permeabilities) or specified as unquarried sites.

7.2.3.8 The P.F.A Filled Areas

The permeability of pulverised fuel ash is much lower than the in-situ sand and gravel which it replaces when extraction quarries are infilled. The permeability of pulverised fuel ash is similar to that of clay, therefore any areas specified as being infilled with this material were given a permeability of $1x10^{-9}$m/s.

7.2.3.9 Depth Of De-Watering

In their planning application, the quarry operating contractors stated that the depth of de-watering was 6 metres. Levelling carried out of the quarry water level at the time of maximum drawdown supported this figure. Therefore, 6 metres was the depth of de-watering used within the WTABLE runs unless stated otherwise.

7.2.3.10 Aquifer Thickness

Within WTABLE for the unconfined condition, there is the provision for specifying bedrock level and lowest and highest phreatic surface levels. The program is therefore not concerned with the depth of any underlying impermeable layer, topsoil, or vadose zones which may exist above the phreatic surface. During the analyses, the locally level bedrock was assigned an elevation of zero metres, the lower water level at 1.0 metre and the regional natural water level at 7.0 or 7.5 metres depending on the case analysed.

7.2.4 Calibration Of The WTABLE Program

7.2.4.1 Purpose And Objective

As part of their planning application submitted for an extension to the existing quarry, the quarry operating contractor in Hoveringham presented contours of ground water levels for 1 April 1993 and 14 December 1992. These contour plots are shown in Figures 7.3 and 7.4. These groundwater contours were drawn by interpolating between values taken in existing piezometers located from the quarry edge to 1000 metres away from the de-watered void. In all, values from 19 piezometers were used as a basis on which to draw the contours. As can be seen, most of the piezometers were located to the north-west and south-west of the de-watered void. Case study data of the de-watering of the quarry was therefore available which could be used to examine the accuracy of the previously discussed derived infiltration and permeability values and the boundary assumptions.

The objective behind the calibration runs was to attempt to model the conditions (including P.F.A infilled areas) at the time the contours were drawn in Figures 7.3 and 7.4 and see how close the finite element model values were to these "real life" results. Values for parameters could be adjusted slightly or completely revised depending on the variation between the two cases.

7.2.4.2 Local Ground Conditions At The Time Groundwater Contours Were Drawn

It can also be seen from Figures 7.3 and 7.4 that to the south of the "existing quarry", at the time the contours were drawn, two older extraction areas had already been infilled with P.F.A, these areas being labelled "Ash Lagoon". It is reasonable to expect the P.F.A infilled areas to have protected areas behind them from the effects of drawdown within the quarry. Although no piezometers were placed in the areas apparently "shielded" from the effects of de-watering, the groundwater contours appear to confirm that some protection was afforded. No other large scale modification has apparently been made to these areas for which the contours were drawn and so all other permeability values specified were reasonable for floodplain deposits.

7.2.4.3 The Mesh Used And Parameters Assumed

The mesh used is shown in Figure 7.5. The de-watered zone was designed so that it matched as closely as possible with the actual size of the existing quarry, the de-watering being simulated by specifying a 1 metre water level all around the quarry edge. The P.F.A infilled areas were specified as elements in a row of permeability 1×10^{-9}m/s. The parameters used in the calibration runs were not designed to simulate worst case conditions as was done with other runs but were specifically designed to simulate as closely as possible the conditions in Figures 7.3 and 7.4. Only by doing this could the accuracy of the finite element model be judged and parameter values be justified for any subsequent de-watering case. The main varied parameters were infiltration and permeability. Infiltration values were varied from the derived annual average value of 390 mm/year to one quarter of this value in an attempt to simulate very dry conditions. Permeability values were drawn from the sources discussed above in section 7.2.3.6.

7.2.4.4 Results From The Calibration Runs

The Results from the calibration runs are summarised in Table 7.2. When examining the results the main area of interest is to the north-west of the de-watered quarry. No piezometer water levels or permeability values were given in the site investigation report for the areas to the north-east and south-east of the quarry. The only area which appeared to be unshielded by P.F.A for which there existed piezometer and groundwater contour data was the area to the north-west.

7.2.4.5 Conclusions From The Calibration Runs

The best match to the case study groundwater contour values were given by the runs which used the Falling and Rising Head permeability test values. The best match to the case study contour values lies somewhere between full and half annual infiltration for these two methods of permeability specification.

Either the permeabilities calculated using the Falling and Rising Head methods are lower than the true field values or too high an annual infiltration value has been assumed in the runs. The answer is most likely to be some combination of the two. In any case Falling and Rising Head values appeared to be reasonably representative and were a good guide of values about which to vary permeabilities in the finite element runs. It may also be concluded that if the annual infiltration value assumed of 390 mm/year was slightly too high then to vary values from this to one quarter of this value should have covered all possible infiltration values.

The values of permeability derived from Hazen particle size distribution analyses, when used in WTABLE with full annual infiltration gave a large overestimation of the distance of influence to the north-west of the quarry. The maximum permeability value assumed was $8x10^{-4}$m/s. This would suggest that a permeability value of $1x10^{-3}$m/s is the maximum that should be chosen in the finite element program (to be representative of a worst case for distance of influence from the quarry).

7.2.5 Modelling Of The Full Scale Pumping Out Test Discussed In Chapter Six

7.2.5.1 Objective

The objective behind the WTABLE analyses discussed in this section was to model the pump test discussed in chapter six. Field data had been collected from the pump test on drawdowns at various distances from the well, permeabilities calculated, and distances of influence had been measured. The pump test provided data collected in the field against which the derived parameters and assumptions being used in WTABLE could be judged.

7.2.5.2 The Mesh Used and Parameters Assumed

The mesh which was used to model the pump test is shown in Figure 7.6. This 190m x 190m mesh was approximately the size of the de-watered zone in the pump test field. To simulate the well, a 3.1m water level was specified at a central node, and a 6.1m initial water level specified for all other nodes.

7.2.5.3 Results From The Runs Modelling The Pump Test

Initial runs were carried out in which a 6.1 metre water level was specified at the mesh boundaries, a 3.1 metre water level specified at the "pump location", and infiltration fixed at zero while permeability values were varied from 1×10^{-2}m/s to 1×10^{-8}m/s. Groundwater contours were in exactly the same position for all permeability values. Obviously, if two fixed water levels were specified in the absence of infiltration, WTABLE would present groundwater contours in the same place no matter what permeability was specified.

When derived annual average infiltration was then specified for runs with fixed boundary water levels and permeabilities of 10^{-6}m/s, 10^{-8}m/s, and 1.7×10^{-4}m/s the following results were observed:

1) For permeabilities of 10^{-6}m/s and 10^{-8}m/s, some groundwater levels in the grid rose to above the boundary levels of 6.1 metres. This was due to too high an infiltration value specified for material of this permeability.

2) For a permeability of 1.7×10^{-4}m/s, the values of drawdown in the grid with and without infiltration differed by insignificant amounts. The values of drawdown and groundwater contour plots for the respective runs with and without infiltration for a

permeability of 1.7×10^{-4}m/s are given in Table 7.3 and Figures 7.7 and 7.8. The infiltration specified probably causes no significant change in groundwater levels because on a grid of the size used, the volume of water due to the infiltration is not large enough to cause a change in water level. (Whereas in the runs to be discussed which modelled much larger areas, infiltration volumes caused significant changes in water levels.)

7.2.5.4 Conclusions From The Runs Modelling The Pump Test.

On comparison with field data from the pump test, WTABLE overestimated drawdowns close to the pump. At distances greater than approximately 35 metres from the de-watered source, the agreement improved with WTABLE still overestimating field values by 30% on average. If WTABLE were to model other problems with similar overestimations this would be advantageous as it could be assumed that the program was deriving "worse case" values.

7.2.6 Modelling Of Full Scale De-Watering Scenarios

The runs modelling full scale de-watering scenarios were of three types. The mesh used was common to all the three sets and is shown in Figure 7.5. It can be seen that for the purpose of examining the effects on the groundwater in areas close to the quarry in greater detail and in particular in between quarry and village, the nodes are closer spaced to give smaller elements in these areas. The mesh was the approximate width of the Trent Valley with the river (if ever specified) to the south. The mesh was made to extend east and west for a large distance to simulate the continuations of the aquifer upstream and downstream. The number of elements specified for this mesh (384) was close to the capacity with which WTABLE could work.

Specifically, the 3 sets of runs were:

1. **"HOVA" runs:**

   It has been described how the main quarry in Hoveringham was divided into three main extraction areas. In the "HOVA" set of runs, the two "Ash Lagoon" areas were specified as de-watered zones, with a one metre water level all around their edge, while the "Existing Quarry" area was specified as a 'greenfield site' not yet quarried. This was then, in terms of drawdown in the village and proximity of the village to the de-watering operation, a worst case scenario. Other important fixed and varied parameters are discussed in the following section, and given in Tables 7.4 and 7.5.

2. **"HOVB" runs.**

   The area in Figure 7.2 labelled as "Existing Quarry" was de-watered for this set of runs. The "Ash Lagoon" areas were either specified as unquarried sites of appropriate permeability for gravel areas or as regions of very low permeability to simulate P.F.A infill. Parameter variation adopted for these analyses is fully summarised in Tables 7.6 and 7.7.

3. **The "Channel" set of runs.**

   The purpose of these runs was to investigate the effects on drawdowns, distances of influence and groundwater velocities of specifying channels of high permeability within the main body of the aquifer. Parameter variation for these runs is fully summarised in Tables 7.8 and 7.9.

7.2.6.1 The "HOVA" Runs

Fixed and varied parameters used for these runs of WTABLE are given in Tables 7.4 and 7.5. It should be borne in mind that the "HOVA" set of runs were designed to simulate worst case scenarios for the de-watering underneath the structures of Hoveringham village. Each of the different runs of the set are now briefly discussed in turn. The contours of groundwater levels given by WTABLE are presented for each of the runs in Figures 7.10(a) to 7.10(i).

HOVA1 - Figure 7.10(a)

In the absence of anything specified at any mesh boundary, a distance of influence away from the quarry of approximately 80 metres was observed on all sides of the de-watered zone.

> (For all the runs now discussed (HOVA2 - HOVA9), the permeability of elements in the mesh was as shown in Figure 7.9. A permeability of $1x10^{-3}$m/s was given to the shaded elements as the highest permeability that could reasonably be assumed for alluvial sands and gravels. This was also the highest permeability calculated by the de-watering contractor in his permeability tests and the highest permeability calculated from the pump test results.)

HOVA2 - Figure 7.10(b)

The specification of high permeability elements caused most of the village to come within the area of influence of the de-watering and the drawdown at the base of the clay bank to be approximately two metres. Under the main body of the village, the drawdown was approximately 0.5 metres, and Brookfield Drive approximately 3 metres.

HOVA3 - Figure 7.10(c)

In this run the infiltration rate specified was so low (half yearly average) that when combined with the high permeability elements, this caused the distance of influence to extend beyond the boundaries of the mesh. In HOVA6 the river was specified as a "flow boundary segment" supplying enough water to keep all groundwater contours within the southern boundary.

HOVA4 - Figure 7.10(d)

This run was the same as HOVA2 except for the specification of recharge from the northern clay bank. The maximum derived recharge value of $325m^3$ per metre run per year was specified. Groundwater drawdown contours decreased both to the north and south of the de-watered zone.

HOVA6 - Figure 7.10(f)

With the permeability distribution as in Figure 7.10, infiltration set at half yearly average, and zero clay bank recharge, it is considered that the conditions specified in this run were the worst which could have occurred during the de-watering of the quarry in terms of distances of influence and drawdowns. Infiltration was set very low at half yearly average, no mudstone bank recharge was applied and permeability was set at maximum values. As a result, the distances of influence were very large with the drawdown under the main part of the village being 1 to 2 metres.

HOVA5 & HOVA7 - Figures 7.10(e) and 7.10(g)

HOVA7 was the same as HOVA5 except for the specification of the river as a flow boundary segment as the contours of water levels had reached the southern boundary in HOVA5. It can be seen in HOVA7 that even under the very severe conditions of half annual infiltration (to simulate a dry summer), high permeability elements, and worst case quarry de-watering that the drawdown under the main part of the village is still only 1 to 2 metres.

HOVA8 & HOVA9 - Figures 7.10(h) and 7.10(i)

These runs were to examine the effects of specifying a low permeability alluvial infilled channel in between quarry and village. HOVA8 shows that for annual average infiltration, high permeability areas beneath the village and between quarry and village and for worse case quarry de-watering, that the specification of such a channel greatly reduces the effects of the de-watering, reducing the drawdowns under the village to zero. It can be seen from the geological map of the area that such channels exist but are unlikely to extend the full depth of the aquifer as specified here.

7.2.6.2 The "HOVB" Runs.

The "Existing Quarry" in Figure 7.2 was de-watered for this set of runs. Runs were carried out with and without the specification of very low permeability "P.F.A. filled" areas to simulate the two "Ash Lagoons". To generate "worst case" results again in terms of distances of influence and drawdown under the village, the permeability distribution for elements was as shown in Figure 7.9. Parameter variation for these runs was as summarised in Tables 7.6 and 7.7.

HOVB1 - Figure 7.10 (j)

No drawdown occurred under the main body of the village. However, the drawdown was extensive at the base of the weathered Mercia Mudstone bank to the north. A large "U" shaped depression in the water table approximately 1.5 kilometres wide with a maximum drawdown of approximately 3.5 metres occurred at the bank base. The distances of influence to the east and west were approximately 200 and 350 metres respectively, obviously having been affected by the specification of average and highest possible permeability values in these directions.

HOVB2 - Figure 7.10(k)

Conditions for this run were the same as for HOVB1 except that infiltration was specified as half the annual average. Due to the fact that without it, the groundwater contours extended beyond the southern boundary, the river was modelled as a "distributed flow boundary" of sufficient flow rate to keep the 7 metre groundwater level contour within this boundary. Also, apart from quarry size and location, this run is identical to HOVA6. It can be seen that the total distance of influence was very large towards the southern boundary (obviously caused by a "worst case" combination of high permeability elements and very low infiltration) but drawdown under the main body of the village was still only minimal at 0.5-1 metre and under Brookfield Drive was approximately 2 metres. The comments made concerning conditions within the HOVA6 run are also pertinent to this run and will not be repeated here.

HOVB3 & HOVB4 - Figure 7.10(l) and 7.10(m)

Runs HOVB3 and HOVB4 were the same as HOVB1 and HOVB2 respectively, except for the specification of very low permeability ($1x10^{-9}$m/s) areas to the south of the de-watered zone to simulate P.F.A. infilled areas. It can be seen that even at the very low infiltration value, the "P.F.A. infilled areas" provided an effective barrier to the effects of drawdown, the contours failing to extend past the edge of the "P.F.A. zones" in both cases. However, it can be seen that the drawdown at the base of the weathered Mercia Mudstone bank was large in both cases being a maximum of 3 metres and 4.5 metres for HOVB3 and HOVB4 respectively.

7.2.6.3 The "Channel" Runs

The high permeability channels specified within these runs all lead directly from the quarry towards Hoveringham Village. This direction was arbitrarily chosen in order to model a worst case scenario for the structures within the village.

The permeability of the main aquifer was specified as $1.7x10^{-4}$m/s and that of the high permeability channels as $1x10^{-3}$m/s. These values were the average and highest permeability values respectively found from the de-watering contractor's tests. Other parameters specified and how they were varied (if at all) are given in Tables 7.8 and 7.9 The actual conditions specified in these runs can be thought of as worst possible case scenarios. Their purpose was to investigate how (if at all) the groundwater level contours would be affected by any areas of significantly higher permeability than the surrounding aquifer. The results from the runs were examined for evidence of significantly increased drawdowns and distances of influence above those suggested by the analysis based on the assumed "bulk average" permeability value. The contours of groundwater levels given by WTABLE are presented for each of the runs in Figures 7.10(n) to 7.10(v).

Results from the "Channel" runs were examined for effects which might significantly affect the following:

1) Effective stress increase leading to settlement.
2) Fine particle movement leading to settlement or otherwise.
3) Nearby wells - might they be depleted or lowered?
4) Would trees or vegetation have their groundwater supply significantly depleted?

Channel1 to Channel4 - Figures 7.10(n) to 7.10(q)

A channel 100 metres wide of permeability $1x10^{-3}$m/s of varying length leading south from the southern edge of the "quarry" was specified and the effect on the groundwater contours was examined.

For a channel 400 metres long (Channel1) it can be seen that the distance of influence has increased by over four times in comparison with results from a model with constant permeability of $1.7x10^{-4}$m/s to a distance of approximately 400 metres. An important point to make from run Channel1 is that within the "bulge" of contours at the drawdown at the location which was previously at the limit of the zone of influence (i.e. approximately 90 metres from the quarry) it can be seen that the drawdown is now approximately 4 metres. This obviously represents a significant increase in drawdown at what was previously considered to be the edge of the zone of influence.

In the 2nd, 3rd, and 4th runs of the "Channel" series, high permeability channels of lengths 1600, 1000, 600 metres respectively were specified. The respective maximum distances of influence from the southern boundary for these runs were approximately 310, 320, and 350 metres respectively.

Channel5 and Channel6 - Figures 7.10(r) & 7.10(s)

In Channel5, a high permeability channel was specified twice as wide as in Channel1. All other conditions were similar between the two runs. The result was that the final distance of influence increased slightly by approximately 15% but generally at any given location, the final amount of drawdown increased only slightly. Very similar variations to those described above were observed between runs Channel3 and Channel6. The total distance of influence and drawdowns again increasing only slightly for a channel of double the width.

Channel 7, 8, and 9 - Figures 7.10(t), (u) & (v)

Within runs 7,8, and 9 of the "Channel" series, infiltration was specified as half the annual average. Distances of influence were observed to increase as expected with this decreased infiltration value and the contours of groundwater levels were observed to "bulge out" close to the high permeability channel generally giving higher drawdowns at any given distance from the de-watered zone as compared to areas away from the high permeability channels.

7.2.7 Discussion Of Implications Of Results From All Quasi Three Dimensional Analyses

7.2.7.1 Drawdowns And Distances of influence.

In the HOVA set of runs, the drawdowns and distances of influence were not critical when the average bulk permeability used by the de-watering contractor in the Hoveringham area was specified for the whole of the mesh. In HOVA1, the distance of influence was approximately 80 metres. When higher permeability ($1x10^{-3}$m/s) zones were specified in HOVA2, the drawdowns and distances of influence increased significantly. For average infiltration values but all other conditions similar, the distance of influence increased from 80 metres to just over 800 metres, extending beyond the village. In HOVA6 the distance of influence extended all the way to the river at the southern boundary.

WTABLE showed that the potential exists for significantly larger drawdowns and distances of influence to exist than would be calculated using "bulk average" values in an analysis. However, the important question to ask at this stage is, “was there sufficient time during any dry period for the large radii of influence given by WTABLE to develop”.

Roberts (1988) goes someway to qualifying this problem by stating that drawdowns and distances of influence tend to be achieved rapidly in soils of "higher" permeability. Indeed, Roberts presents a method for determining the time to achieve the drawdown for a given de-watering scenario, but presents no method for determining the time to achieve full radius of influence. The author is unaware of any work which quantifies the time to achieve a given distance of influence and therefore this is a limitation to the WTABLE results.

7.2.7.2 Effective Stress Increase

In some of the WTABLE runs discussed above, the drawdowns and distances of influence appear to be very large compared to those assuming average values. However, such increase in drawdowns or distances of influence need not necessarily mean structurally significant settlements will occur. Obviously, drawdowns of the magnitude given from some runs would be undesirable beneath structures, but if a severe drawdown of 4.5 metres was assumed beneath some of the structures nearest to the quarry, the total maximum settlement becomes 11 millimetres using the equation below from CIRIA report 113:

$$Settlement(mm) = 490\frac{h_o}{q_a}$$

where: h_o = Drawdown of the water table.

q_a = Allowable bearing capacity estimated from in-situ standard penetration test value (N).

This value of 11 millimetres was calculated assuming a very low bearing capacity of 200 kN/m^2 for the sand and gravel strata. This amount of settlement would not in itself represent a problem even if the whole of the settlement were to be differential settlement. A more realistic scenario is that any structure on ground with a total settlement calculated at 11 millimetres is only likely to suffer a fraction of this value as differential settlement.

7.2.7.3 *Movement Of Fine Particles*

The granular soil in the Hoveringham area has been shown to have the potential to lose finer particles under certain severe conditions of seepage (see Chapter Five). Evidence was found to suggest that finer particles had been removed by flowing water at the quarry face where the most severe conditions of hydraulic gradient and groundwater velocity existed. In Chapter Five it was discussed how Kenney and Lau estimated the potential for granular material to lose fine particles. They found that a "unit flux" of at least 6.8×10^{-3} m/s was required to cause movement of these particles. Essentially unit flux is the same as flow rate per unit area of cross section through an aquifer and can be calculated using the following equation:

$$\textbf{Unit Flux} = \mathbf{k \times i}$$

Where: k = aquifer permeability

i = hydraulic gradient within the aquifer

On examining Figures 7.10(a) to 7.10(v) it can be seen that in the area of the structures of Hoveringham Village, WTABLE derived hydraulic gradients of approximately 1 in 100. This gradient, when combined in the equation with a high aquifer permeability of, say, 1×10^{-3} m/s would give a unit flux of only 1×10^{-5} m/s.

The WTABLE results therefore suggest that it is highly unlikely that fine particle movement occurred beneath Hoveringham Village.

7.2.7.4 *P.F.A. Infilled Areas*

The results of runs HOVB3 and HOVB4 suggest that should P.F.A. filled areas exist adjacent to a de-watered zone then the effects of drawdown become negligible beyond the filled area, even at very low infiltration rates. The results suggest that P.F.A. filled areas form a more effective barrier against de-watering effects than cohesive alluvium infilled channels. It is extremely likely that effects of this sort occurred during the recent de-watering of the quarry as it is known that P.F.A. filled areas exist and that they extend the full depth of the aquifer (being a replacement for fully worked out gravel areas). This finding has important consequences for the structural monitoring investigation discussed in Chapter Four. It was stated that no significant progressive settlement was noticed. The monitoring was taking place at a time when P.F.A. infilled zones existed as well as when de-watering had ceased. If the de-watering had been causing any mechanism of progressive settlement, it would surely have stopped in the region of the village with the creation of P.F.A. infilled areas.

7.2.7.5 Weathered Mercia Mudstone Bank Recharge

Runs HOVA2 and HOVA4 show that recharge from the weathered Mercia Mudstone boundary, should it occur, would have a large influence on drawdowns at the base of this bank. For these two runs the maximum drawdown at the base of the bank decreased from just over two metres to virtually zero. However, as discussed previously, it is thought highly unlikely that recharge occurs at the magnitude specified in these runs.

7.2.7.6 Alluvium Infilled Channels

Runs HOVA8 and HOVA9 showed the effects that cohesive alluvium filled channels would have if they extended the full depth of the aquifer. The effects of de-watering would become negligible beyond the channel even when very close to the de-watered area. However, the borehole data suggest it is unlikely such infilled channels extend the full aquifer depth and so it is unlikely that the effects shown in HOVA8 and HOVA9 occurred during de-watering.

7.2.7.7 *High Permeability Channels*

The "channel" set of runs suggested that should significantly higher permeability material exist in zones extending from a de-watered area, drawdowns and distances of influence may be larger than would be calculated if average permeability values were assumed. Should the geological conditions specified in some of the "channel" runs occur in nature and a de-watering contractor were to take eight permeability tests, seven giving a permeability of 10^{-7}m/s and one giving a permeability of 10^{-3}m/s, the average permeability would be calculated as 1.3×10^{-4}m/s. Furthermore, if the equation from section 6.1 were to be used to estimate the distance of influence of the de-watering then the average value for the distance of influence would be approximately half that for if the highest measured permeability value were used on its own.

Some of the effects noticed in the "channel" runs, although occurring under unlikely geological conditions, show that high permeability areas can distort the groundwater contours away from an "assumed average" pattern and should be taken into account in any attempted prediction of how the de-watered zone will be affected.

7.2.7.8 *Clay Lens Compression*

The settlements at the ground surface could have become more severe if the de-watering had lowered the water table from above to below a lens of clay material. The consolidation of such material could cause settlements larger than the 11 millimetres mentioned above, although again there would have to be a significant amount of differential settlement for structural damage to occur. Given the ground investigation borehole data presented by the contractor in his planning application, the borehole findings from the pump test, and the degree of drawdown required at large distances from the de-watered zone, this would seem to be a highly unlikely mechanism of achieving high ground settlements.

7.3 Axi-Symmetric Cross Sectional Investigations

7.3.1 Introduction.

During the previously described pseudo-three-dimensional investigations the permeability was always constant with depth. However, an aquifer may also be layered (or stratified) with layers varying in permeability. Stratification usually leads to an aquifer exhibiting the highest permeabilities in the horizontal direction. Evidence that the Trent Valley sands and gravels are stratified came from an inspection of the exposed quarry face during the time of maximum water table drawdown and the different materials brought to the surface in sequence by the augers during standpipe installation prior to the pump test described in Chapter Six. The following sections describe how the FLONET finite element program was used to model various dewatering scenarios in which permeability was varied with depth.

7.3.2 The "FLONET" Program

FLONET is a finite element program capable of modelling confined or unconfined (free surface) seepage in a plane or axi-symmetric section through the seepage domain. The results from a FLONET run comprise the distribution of pressures and velocities throughout the seepage domain. FLONET can be likened to a numerical method for producing "flow nets". It deals with the same vertical plane section through the seepage domain that a sketched flow net would. FLONET only performs steady seepage analyses but can handle anisotropic and non-homogeneous permeabilities. FLONET not only allowed the investigation of seepage characteristics within a stratified aquifer, it also allowed the examination of flow conditions within any given layer. It this way it was possible to investigate how conditions within the stratified aquifer varied from those within a completely homogeneous media.

7.3.3 How FLONET Was Employed

Roberts (1988) showed, via elemental analysis of water at a free surface, that the maximum hydraulic gradient that can exist within an unconfined aquifer is unity. This result was worked out independently of the permeability variation within the aquifer and applies to both transient (free surface moving) and steady seepage conditions. This result implies that there is an upper limit to the velocity and flow rate per square metre which can be achieved within any given de-watered unconfined aquifer. It may also be concluded that, for any given drawdown, the maximum unit flux achievable will decrease if the permeability decreases.

Kenney and Lau (1984) (see Chapter Five) called the flowrate per unit area of cross section within a granular material subject to seepage the "unit flux". The lower limit of unit flux which they found to cause disturbance of the finer particles (or "suffosion") within granular material was 6.8×10^{-3} m/s. Unit fluxes derived from the FLONET analyses were compared to this value. FLONET quotes unit flux values in both horizontal and vertical directions. The unit flux values referred to in this section are the resultant of these two components. Unit flux values discussed are therefore the maximum values which occurred in any location and direction within the seepage domain.

FLONET runs were carried out with layer thickness and permeability variations which modelled bulk permeability values quoted in Chapter Three taken from a previous site investigation of the area. Those values calculated from the pump test discussed in Chapter Six were also used.

7.3.4 Estimate Of Unit Fluxes Existing During De-watering

The Darcy assumption can be combined with the findings of Roberts discussed above to obtain a method of estimating the maximum unit flux within any de-watered unconfined aquifer.

The Darcy Equation is: $\quad Q \quad = \quad A\,k\,i$

where:

Q = total flow rate in m^3 per second

A = area of x-section through the aquifer (m^2)

k = permeability of the aquifer in m^2/s

i = hydraulic gradient (dimensionless)

From this, "Unit Flux" can be defined as;

$$\textbf{Unit Flux} = Q/A = ki$$

Therefore, it can be seen that within a de-watered unconfined aquifer the maximum unit flux value expected is equal to the permeability assuming an upper limit of unity on the hydraulic gradient. It is known that as distance increases away from a surface of seepage (say a quarry face), the hydraulic gradient reduces quickly as the phreatic surface flattens. A corresponding reduction in unit flux values will also occur as distance increases away from a seepage face.

7.3.5 The Mesh Used, Modelling Methodology And An Example Analysis

The mesh used to represent the seepage domain is shown in Figure 7.11. A 6 metre head was fixed on the left hand side of the mesh to represent the height of the natural water table above the base of the aquifer at the distance of influence from the de-watered quarry. The length of the mesh represented the distance of influence of the de-watering. This distance was fixed at 50 metres. It is appreciated that this 50 metre length is less than any quoted distances of influence from other areas of the research. This was done purposely in an attempt to make the derived hydraulic gradients at any distance from the "de-watered quarry" higher than those that would have existed during de-watering. In this way, FLONET would be deriving "worst case" unit flux values. The de-watered quarry was modelled as a line of nodes on the right hand edge of the grid with a fixed head of zero. A total of 21 "free surface nodes" at the top of the grid defined the phreatic surface. FLONET adjusted the position of these free surface nodes to match the new calculated position of the phreatic surface.

An example of a deformed mesh after a FLONET run is given in Figure 7.12. It can be seen that the mesh is distorted according to the position of the phreatic surface and the height of the surface of seepage. It will be noticed that FLONET's distortion of the mesh meant that the new deformed mesh no longer exactly represented the geological conditions the mesh was initially designed to simulate. However, this was considered advantageous as the element heights (layer thicknesses) were reduced to a greater degree as one approached the "de-watered quarry" , thus increasing unit flux values within them above those that would have been derived had element heights remained constant. This was another factor, along with the specified distance of influence, working to give worse case values of unit flux.

7.3.6 The Use Of FLONET To Model The Pump Test.

In order to compare an analysis with measured field data as a calibration exercise, the direction south away from the pump was modelled. Standpipes were positioned south from the pump at 15m, 35m, and 60m away. Water levels were also deduced at 25m and 5m away via interpolation on a plot of the cone of depression and the water levels at these distances away were compared with the FLONET results. The mesh used for the analysis is shown in Figure 7.13. The grid was 4.4m high as this was the static height of water in the well prior to pumping and was 80m long as this was the estimated distance of influence south from the pump. The permeability specified for the analyses was 3.7×10^{-4}m/s (the average of values calculated from the pump test data for direction south).

Table 7.10 summarises the drawdowns observed during the pump test (both from standpipe measurements and interpolation between standpipes) and from a FLONET analysis for various distances away from the pumped source. It can be seen that FLONET gave a large overestimation of drawdowns when the two sets of results are compared. It is possible that local geological variation in the pump test field caused some variation with the flonet results however, this was another reason to believe that worse case unit flux values would be derived.

The total flow measured during the pump test at steady state was 3.9 litres per second. This value was used to estimate unit flux values which existed during the pump test at 3.75m and 26.25m away from the pumped source. Table 7.11 summarises both these calculated values and values of unit flux given by FLONET at 3.75m and 26.25m from the pump for an axi-symmetric analysis. The following observations can be made:

- For a distance of 3.75m away from the pumped source, FLONET underestimates the unit flux values, the value being one third of calculated field values.

- For a distance of 26.25m from the pump, FLONET overestimated the flux by a factor of 1.7 when compared to the estimated field value.

7.3.7 Discussion Of Results From The Pump Test Model

The finite element model overestimated field drawdowns. The overestimation was greater closer to the pumped source. It may be expected that FLONET will derive greater drawdowns than existed in the field.

The model appeared to overestimate unit fluxes at distances greater than about 20 metres from a pumped source. However, if the model's "worst case" unit flux values were not of a magnitude likely to cause movement of fine particles, one would be able to conclude that it would be very unlikely that particles would be disturbed during a de-watering operation.

7.3.8 Main FLONET Runs / Results Discussion.

7.3.8.1 Types Of Main Run Used

Figure 7.14 shows the permeability variations used in all the main analysis runs. The first type of run had equal permeability in horizontal and vertical directions over the whole seepage domain (mesh). The highest conceivable permeability value was used in run 1A to obtain worst case unit flux values. More realistic situations were modelled by runs 1B and 1C. Horizontal permeabilities were made orders of magnitude larger than the vertical permeabilities to reflect the way these two properties usually vary. Permeability values were varied over a sensible range up to highest conceivable values.

Runs 2A to 2F involved a layer of high hydraulic conductivity material being specified within a surrounding aquifer of significantly lower hydraulic conductivity. Only one layer of higher permeability was specified. In runs 2C, 2D, and 2F, the high permeability layer was not continuous throughout the length of the seepage domain to see if this significantly affected unit fluxes within that layer. Runs 3A to 3D included high hydraulic conductivity layers specified either at the top of or at the base of the aquifer. Run "Layers4" had two adjacent layers of high permeability specified which ran the full length of the mesh.

In the layered systems the bulk aquifer permeability for each model was calculated. It will be seen in the sections that follow that some of the bulk permeability values which the layered systems represented were purposely made very high compared to the aquifer. Consequences of this for unit flux values are described in the following sections.

7.3.8.2 The "Layers1" Series Of Models

Run "Layers1A":

Horizontal (k_x) and vertical (k_y) hydraulic conductivity values were both assigned the value of 1×10^{-3}m/s for all elements. Conditions were therefore homogeneous throughout the seepage domain with the hydraulic conductivity having been set at the upper limit for sand and gravel material. As expected, the maximum unit flux value anywhere occurred adjacent to the quarry wall. At 0.52×10^{-3}m/s, this maximum value was approximately one order of magnitude lower than the value for which Kenney and Lau first observed particle movement.

Run "Layers1B"

Aquifer Properties: $k_x = 1x10^{-3}$m/s ; $k_y = 1x10^{-5}$m/s, for ALL elements.

The maximum derived unit flux value was $0.17x10^{-3}$m/s, again adjacent to the "quarry wall". This value was 40 times less than values quoted by Kenney and Lau to cause suffosion.

Run "Layers1C"

Aquifer Properties: $k_x = 1x10^{-3}$m/s ; $k_y = 1x10^{-7}$m/s, for ALL elements.

The maximum unit flux value calculated was $0.1x10^{-3}$m/s, approximately 70 times less than the value required for suffosion according to Kenney and Lau. No runs were done with the vertical permeability reduced further as it was now obvious that this would simply cause a further reduction in unit flux values.

7.3.8.3 The "Layers2" Series Of Models

Runs "Layers2A" to "Layers2F" involved the specification of a high permeability layer within a surrounding aquifer of lower permeability. Table 7.12 summarises the permeability combinations used along with the resultant bulk permeability values the layered systems modelled. Unit flux values are tabulated for locations adjacent to the quarry wall and 5 metres back from it. In all cases the maximum unit flux value observed for any run is contained in the table. It can be seen from Figure 7.14 that not all higher permeability layers traversed the full width of the seepage domain. This was designed to test the effects of the presence of such a discontinuous geological feature within the aquifer.

The unit flux value at which Kenney and Lau first observed particle movement to occur was $6.8x10^{-3}$ m/s. It can be seen from Table 7.12 that for runs with a bulk permeability value representative of the aquifer of interest (namely runs 2A to 2D) that unit flux values are well below this value apart from at the “quarry face” within the high permeability layer. Unit flux values within the high permeability layer were of the order of or greater than Kenney and Lau values for runs 2E and 2F. However, the bulk permeability values of the seepage domains for these runs were too high to be representative of the Trent Valley unconfined aquifer.

7.3.8.4 The "Layers3" Series Of Models And Run "Layers4"

The "Layers3" series of runs were designed to test the effects of specifying high permeability layers at the very top and bottom of the seepage domain. The high permeability layers at the bottom were designed to investigate whether this had an "underdraining" effect on the material above. Any material close to the surface of seepage (i.e. at the top of the mesh) would be in the region where hydraulic gradients were a maximum. The layers at the top of the seepage domain were examined for increased unit fluxes due to this effect.

Runs "Layers3A" and "Layers3C":

The model used for "layers3A" was essentially the same as that used for "Layers2B" but with the high permeability layer displaced to the bottom of the seepage domain. For this run, the bulk permeability value was $6.8\text{x}10^{-4}$ m/s. On examining unit fluxes for the two runs, the values were found to be virtually identical at similar locations within the seepage domain. The specification of the high permeability layer at the base of the seepage domain had caused no significant change in unit flux values. For run "Layers3A", other than adjacent to the quarry wall, unit flux values never became as high as those values quoted by Kenney and Lau for suffosion. On examining unit flux values for run "Layers3C", again the positioning (top of the seepage domain) of the high permeability layer was found to produce no significant change in unit flux values.

Runs "Layers3B" and "Layers3D":

The bulk permeability modelled by these two runs, $6.7\text{x}10^{-3}$ m/s, was slightly higher than the upper limit of permeability required to reasonably model the aquifer. Derived unit flux values were therefore conservative. Within the high permeability layer for both runs, unit flux values increased from $6\text{x}10^{-3}$ m/s (approximately equal to Kenney and Lau values) at the distance of influence to $29\text{x}10^{-3}$ m/s (approximately four times Kenney and Lau values) in “quarry wall” elements.

Run "Layers4".

The geological conditions for this run can be seen in Figure 7.14. The bulk permeability was $7.4x10^{-4}$m/s. This was a sensible value to represent the aquifer of interest. Unit flux values were well below Kenney and Lau values everywhere within the seepage domain apart from at the "quarry wall" within the highest permeability layer. The unit flux value at this point was approximately equal to Kenney and Lau values.

7.3.9 Conclusions From All FLONET Runs

From the "Layers1" set of runs, even for a high permeability value for the whole of the aquifer ($1x10^{-3}$m/s), the unit flux values were approximately one order of magnitude smaller than the values required to start suffosion according to Kenney and Lau.

In the "Layers2" set of runs, a thin layer of high permeability material was specified within a surrounding aquifer of lower permeability. The bulk permeability which the aquifer conditions represented were estimated using equations for non-homogeneous soil conditions and compared with a typical value for the Trent Valley unconfined aquifer. Calculated values were compared with those quoted in a previous site investigation of the area and values calculated from the pump test described in Chapter Six.

Runs B, C and D of the "Layers2" set modelled bulk permeabilities representative of the aquifer of interest. The highest unit flux observed for these three runs was in run 2B within the high permeability layer at the quarry face. This unit flux was slightly lower than the required value to begin suffosion.

Two runs were carried out ("Layers2E" and "Layers2F") for which the calculated bulk permeability ($7x10^{-3}$ m/s) was higher than the upper limit of representative values for the aquifer. In fact, this value was almost one order of magnitude too large. Within the high permeability layer for these runs, high unit flux values occurred. These high unit fluxes were either approximately equal to or greater than values quoted by Kenney and Lau. Right at the quarry edge, a unit flux value of $0.472x10^{-1}$m/s occurred, some four times larger than values quoted by Kenney and Lau for the start of suffosion.

For the "Layers3" set of runs, when the bulk permeability of the aquifer was representative of the Trent Valley unconfined aquifer, unit fluxes were not as high as those quoted by Kenney and Lau. When conditions were specified which were representative of the Trent Valley unconfined aquifer, unit flux values did not reach those quoted by Kenney and Lau for the start of suffosion other than very close to the quarry face.

Unit flux values which may have caused particles to move were observed at distances greater than 5 metres from the quarry wall only when the "bulk" permeability of the seepage domain was made too high to reasonably represent the Trent Valley unconfined aquifer.

The results of the FLONET analyses described above imply it is highly unlikely that movement of fine particles occurred within the Trent Valley sand and gravel aquifer due to the de-watering of Hoveringham quarry other than very close to the quarry face. Furthermore, it can also be considered highly unlikely that suffosion was a cause of structural movement within the village of Hoveringham given the relatively large distances of the structures from the quarry.

Permeability range (m/s)	**$1 - 10^{-1}$**	**$10^{-1} - 10^{-4}$**	**$10^{-4} - 10^{-7}$**	**$10^{-7} - 10^{-10}$**
Material description	Clean Gravels	Clean sands and sand / gravel mixtures	Very fine sands, silts and clay-silt laminates	Unfissured clays and clay-silts (>20% clay)
		Desiccated and fissured clays		

Table 7.1

Coefficients Of Permeability Ranges For Different Types Of Material

(After BS 8004:1986))

Calibration run	**Permeability variation based on**	**Infiltration (% annual average)**	**General agreement with case study data**
1	Falling Head Tests	100	Radius of influence approximately 0.5 times case study values
2	Falling Head Tests	50	Radius of influence approximately 1.5 times case study values
3	Falling Head Tests	25	Large overestimation of radius of influence
4	Rising Head Tests	100	Radius of influence approximately 0.5 times case study values
5	Rising Head Tests	50	Overestimation of radius of influence
6	Rising Head Tests	25	Large overestimation of radius of influence
7	Hazen Analyses	100	Large overestimation of radius of influence
8	Hazen Analyses	50	Very large overestimation of radius of influence

Table 7.2

Summary Of The Results From The Calibration Runs.

Distance from well (m)	Drawdown for run with infiltration (m)	Drawdown for run without infiltration (m)	Pump test range of drawdowns (m)
5	2.3	2.3	1.20
15	1.3	1.3	0.91 - 0.94
25	0.9	0.9	0.57 - 0.73
35	0.7	0.7	0.47 - 0.57
50	0.45	0.46	0.33 - 0.36
60	0.30	0.35	0.27 - 0.36

Table 7.3

Values Of Drawdown From WTABLE Runs Modelling The Pump Test With And Without Infiltration Compared To Range Of Actual Drawdowns Measured In The Field.

Parameter Common To All "HOVA" Runs	Varied Parameters
No stream/aquifer connection.	Aquifer permeabilities varied as detailed in Table 7.5
"Ash lagoon" areas in Figure 7.2 de-watered	Infiltration rate.
Water table lowering method - 1.0m water level fixed at "quarry edge".	Clay bank recharge.
No P.F.A. filled areas specified.	River Trent - specified as a "flow boundary segment" when distance of influence reached southern boundary.
Drawdown depth fixed at 6.0m.	
Aquifer depth - constant.	

Table 7.4

Common and varied parameters within all "HOVA" runs.

"HOVA" run	Permeability variation	Infiltration rate (mm/year)	Clay bank recharge	River Trent
A1	All elements: $k=1.7x10^{-4}m/s$	390	NO	NO
A2	As in figure 7.9	390	NO	NO
A3	As in figure 7.9	195	NO	NO
A4	As in figure 7.9	390	YES	NO
A5	As in figure 7.9	195	YES	NO
A6	As in figure 7.9	195	NO	Yes.As a"flow boundary".
A7	As in figure 7.9	195	YES	Yes.As a"flow boundary".
A8	As run A2 but low k zone specified between pit and village to simulate cohesive alluvium channel	390	NO	NO
A9	As A8	195	NO	NO

Table 7.5

Parameters Used For The "HOVA" Set Of Runs.

Notes to table 7.5.

1. Yearly average infiltration is 390 mm/year.
2. Clay bank recharge, when specified, was $11x10^{-6}m^3$ per metre run per second.

Parameter common to all "HOVB" runs	Varied parameters
No stream/aquifer connection.	Material permeabilities (see Table7.7)
"Existing quarry" area in Figure 7.2 de-watered.	Infiltration rate.
Water table lowering method - 1.0m water level fixed at "quarry edge".	River Trent - specified as a "flow boundary segment" when distance of influence reached southern boundary.
Drawdown depth fixed at 6.0m.	P.F.A. filled areas (for runs B3 and B4).
Aquifer depth constant.	
Zero northern clay bank recharge.	

Table 7.6

Common and varied parameters for all "HOVB" runs.

"HOVB" run	Permeability variation	Infiltration rate (mm/year)	River Trent	"P.F.A. filled areas".
B1	As in figure 7.9	390	NO	NO
B2	As in figure 7.9	195	YES	NO
B3	As in Figure7.9 except for "cohesive alluvium filled channels" specified between pit and village.	390	NO	YES
B4	As for B3	390	YES	YES

Table 7.7

Varied parameters for the "HOVB" set of runs.

Parameter Common To All "Channel" Runs	Varied Parameters
No stream/aquifer connection.	Width / length of high permeability channel.
"Ash Lagoon" areas "A" &"B" in Figure 7.1 de-watered (as "HOVA" set of runs).	Infiltration rate.
Water table lowering method - 1.0m water level fixed at "quarry edge".	
No "P.F.A. filled areas" specified.	
Drawdown depth fixed at 6.0m.	
Aquifer depth - constant.	
Permeability of main aquifer and high permeability channel.	
Zero northern clay bank recharge.	
No "River Trent" specified.	

Table 7.8

Common and varied parameters within all "Channel" runs.

RUN	Width / length of high permeability channel (m/m)	Infiltration rate (mm/year)
1	100 / 400	390
2	100 / 1600	390
3	100 / 1000	390
4	100 / 600	390
5	200 / 400	390
6	200 / 1000	390
7	100 / 400	195
8	100 / 1600	195
9	100 / 1000	195

Table 7.9

Varied parameters for the "Channel" set of runs

Source of data	Derived water levels from FLONET analyses and observation (O) or interpolation (I) from pump test data at given distances from the pump.				
	60m	35m	25m	15m	5m
FLONET/axi-s	2.20m	1.42m	1.14m	0.85m	0.48m
Pump Test	4.11m	3.93m	3.70m	3.46m	2.70m

Table 7.10
Field Drawdowns And Values Derived From The Finite Element Model For The Pump Test

Source of data	Unit fluxes derived / calculated ($m^3/sec/m^2$)	
	3.75m from pump	26.25m from pump
FLONET/axi-s	$0.25x10^{-4}$	$0.10x10^{-4}$
Calculation	$0.72x10^{-4}$	$0.06x10^{-4}$

Table 7.11
Summary Of Unit Flux Values From Flonet And Estimated By Calculation From The Pump Test Data.

Run I.D.	Bulk Permeability (m/s)	Unit Flux at given distance and location from the quarry wall (m/s)			
		5m from wall in high permeability layer	5m from wall not in high permeability layer	Quarry face in high permeability layer	Quarry face not in high permeability layer
2A	6.7×10^{-5}	0.17×10^{-3}	0.17×10^{-7}	0.47×10^{-3}	0.46×10^{-7}
2B	6.8×10^{-4}	0.17×10^{-2}	0.17×10^{-5}	0.47×10^{-2}	0.47×10^{-5}
2C	6.8×10^{-4}	0.23×10^{-5}	0.23×10^{-5}	0.64×10^{-5}	0.62×10^{-5}
2D	6.8×10^{-4}	0.64×10^{-4}	0.87×10^{-7}	0.77×10^{-5}	0.75×10^{-5}
2E	6.8×10^{-3}	0.17×10^{-1}	0.17×10^{-4}	0.47×10^{-1}	0.46×10^{-4}
2F	6.8×10^{-3}	0.26×10^{-4}	0.26×10^{-4}	0.70×10^{-4}	0.68×10^{-4}

N.B: Kenney and Lau first observed suffosion at a unit flux value of 6.8×10^{-3} m/s

Table 7.12
Bulk Aquifer Permeabilities And Derived Unit Flux Values for The "Layers 2" Series Of Models

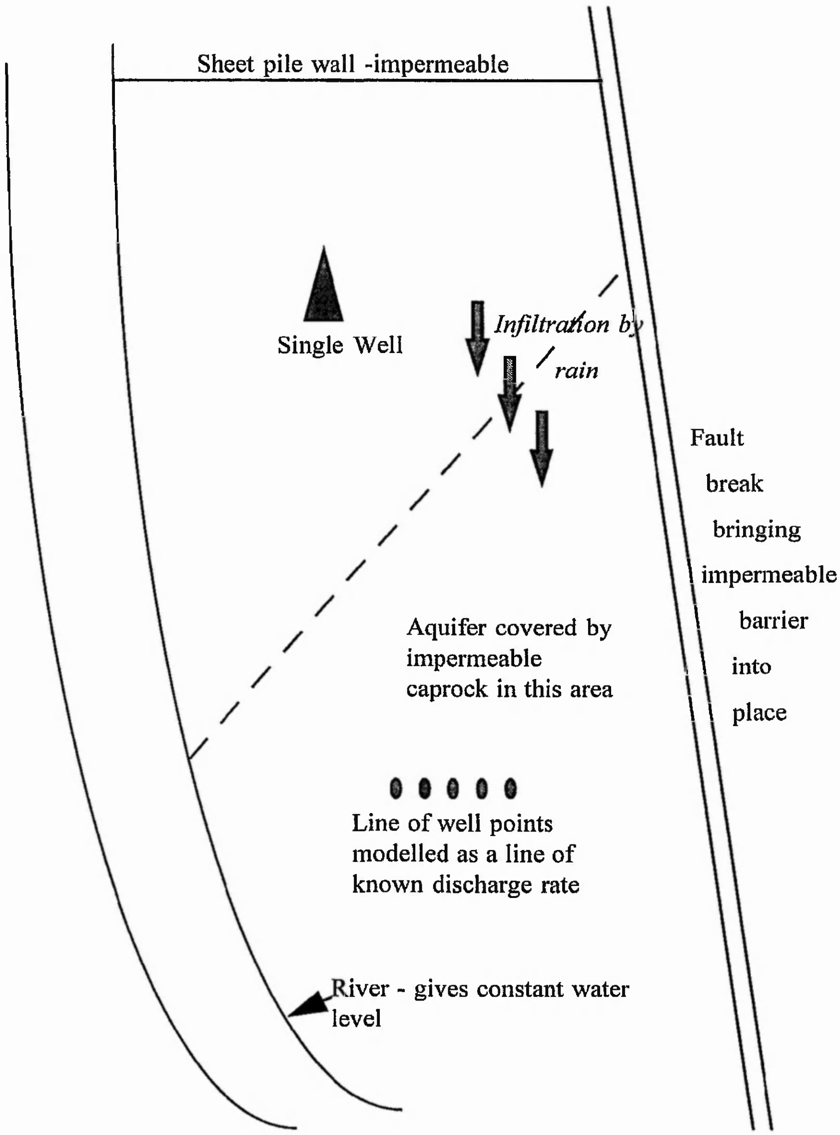
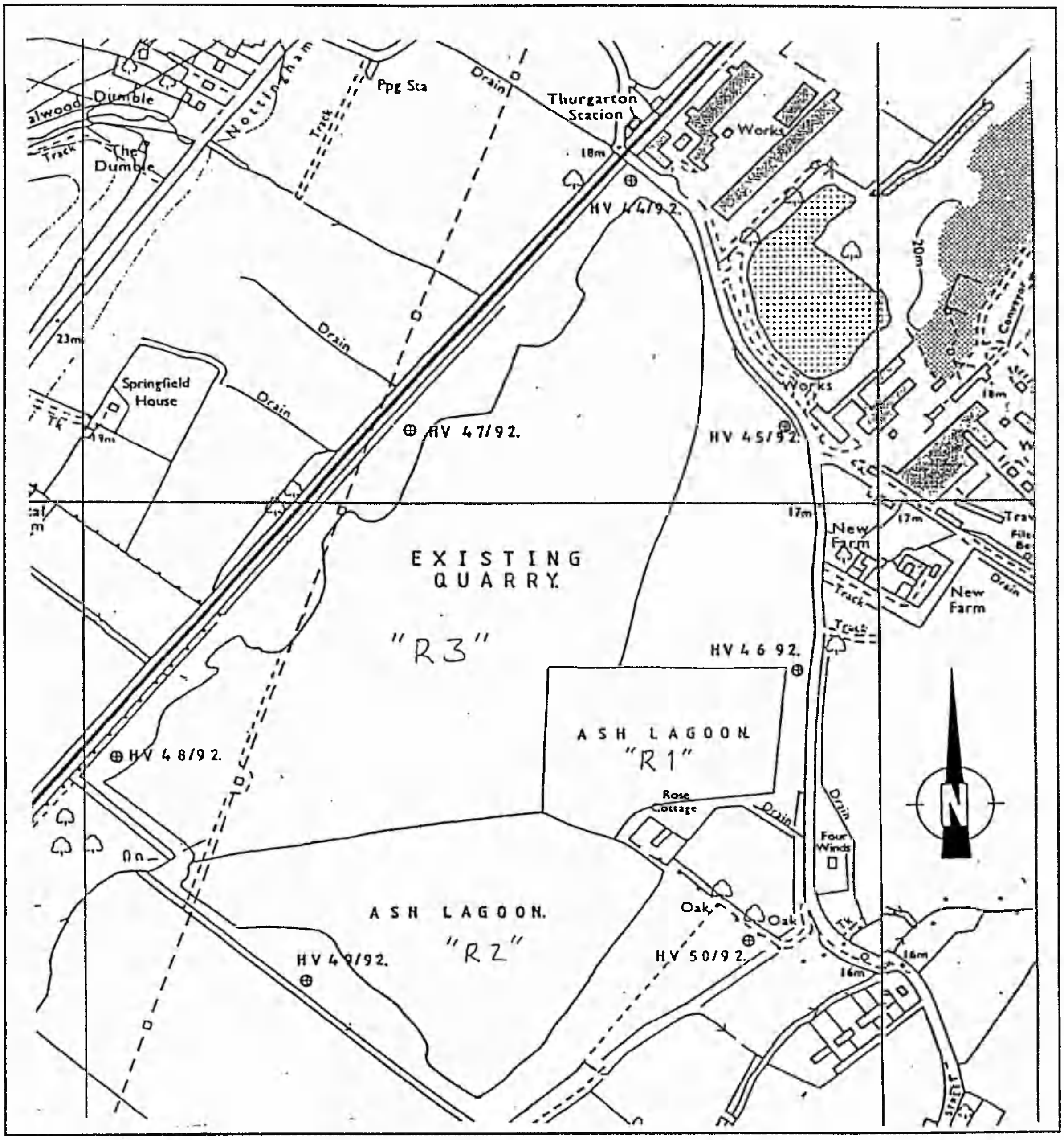


Figure 7.1

Boundary Conditions Available In WTABLE



Scale 1:8000

Figure 7.2

Worked Zones In The Quarry

(after Tarmac Quarry Products Ltd, 1993)

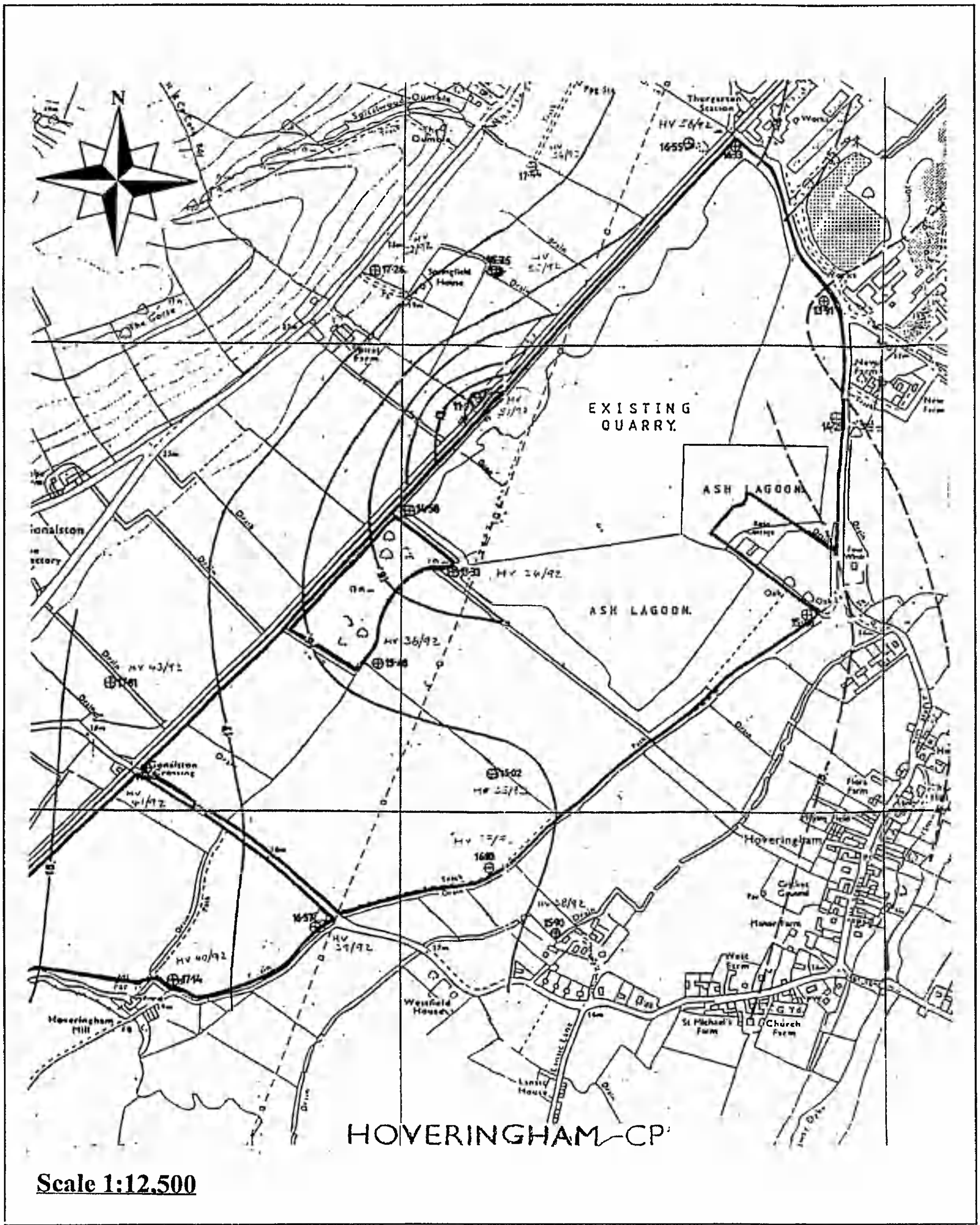


Figure 7.3

Groundwater Level Contours Around The Quarry For 14th December 1992

(after Tarmac Quarry Products Ltd. 1993)

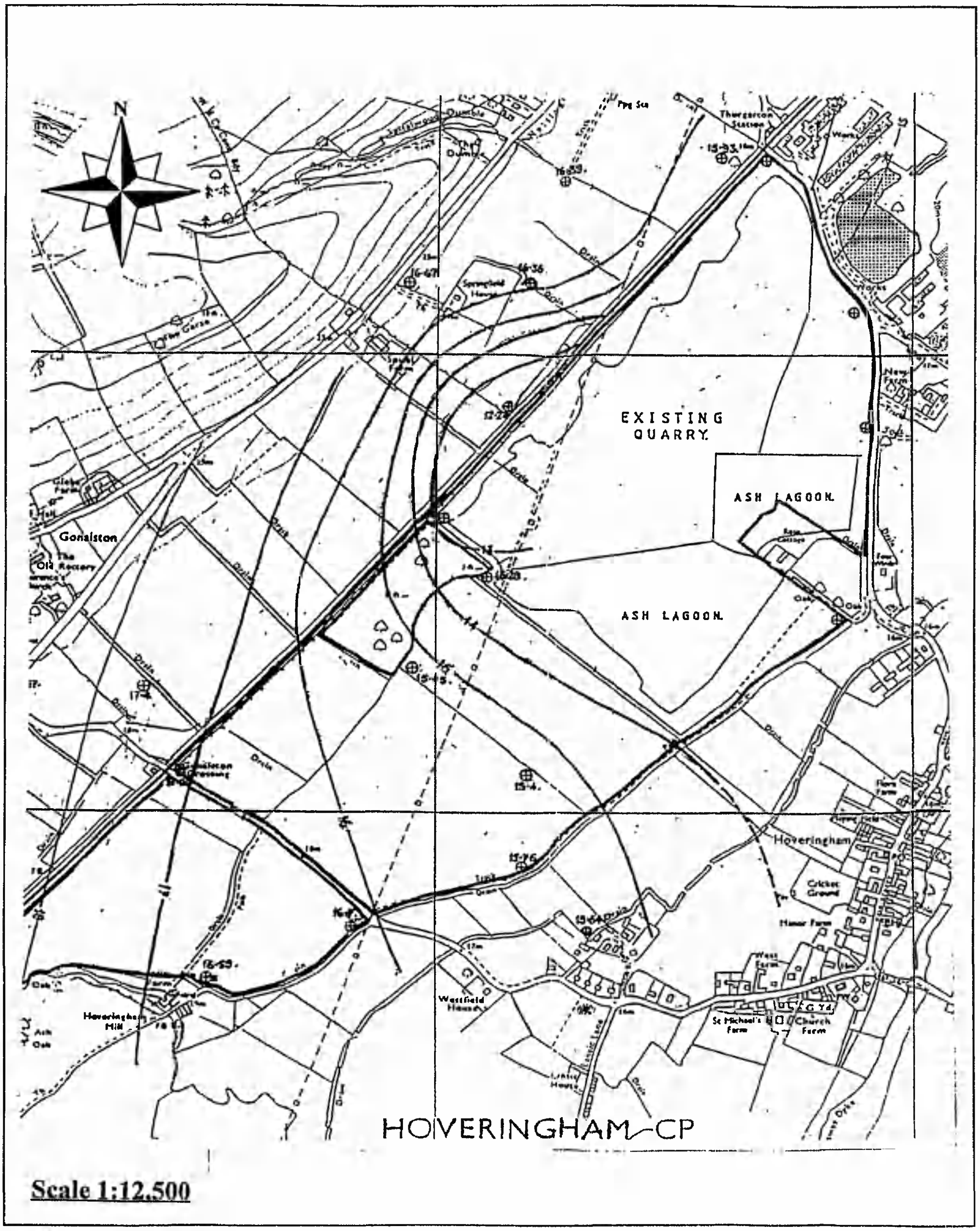


Figure 7.4

Groundwater Level Contours Around The Quarry For 1st April 1993

(after Tarmac Quarry Products Ltd. 1993)

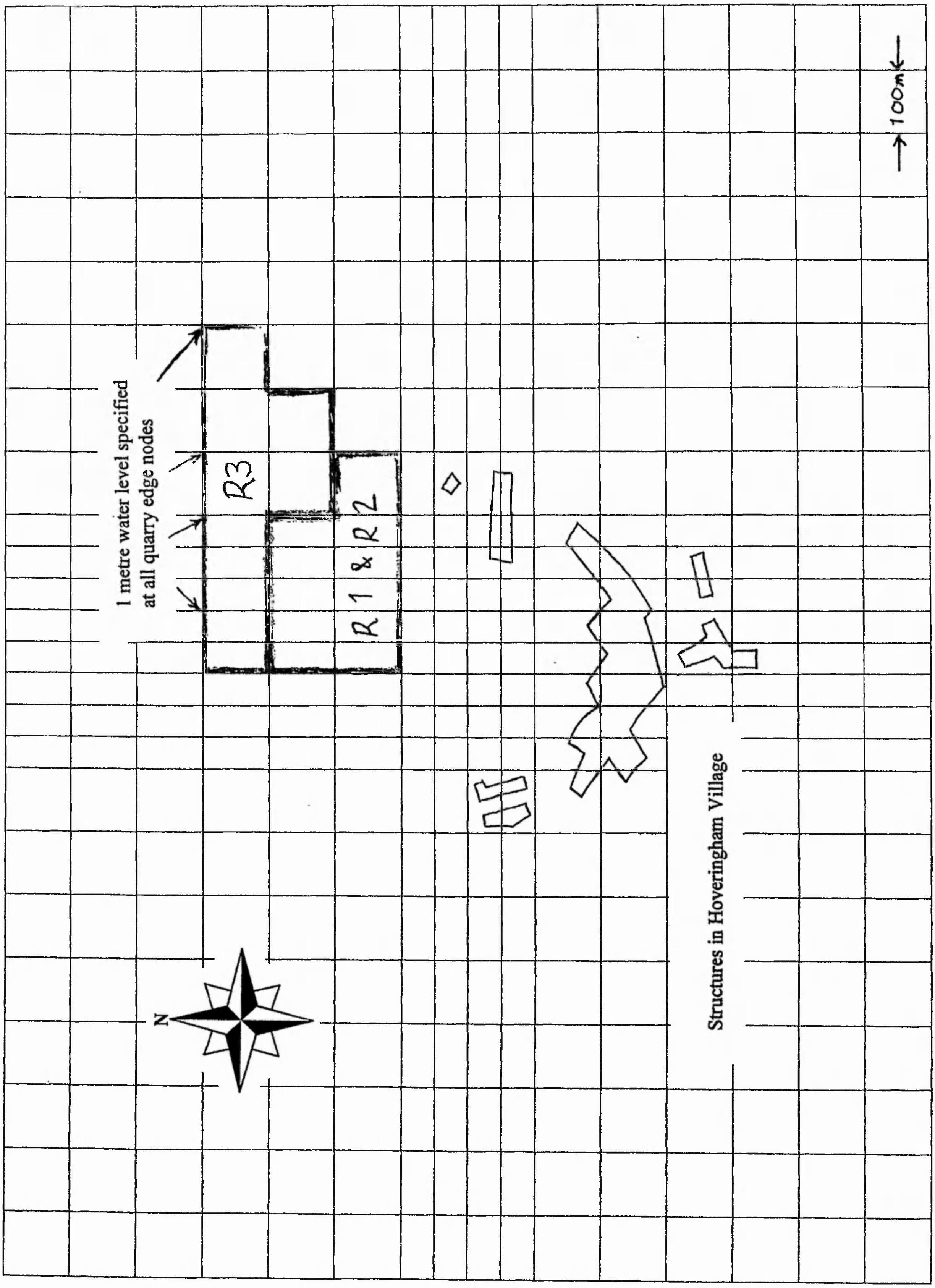


Figure 7.5

The Mesh Used During The Calibration Runs (and main analysis runs)

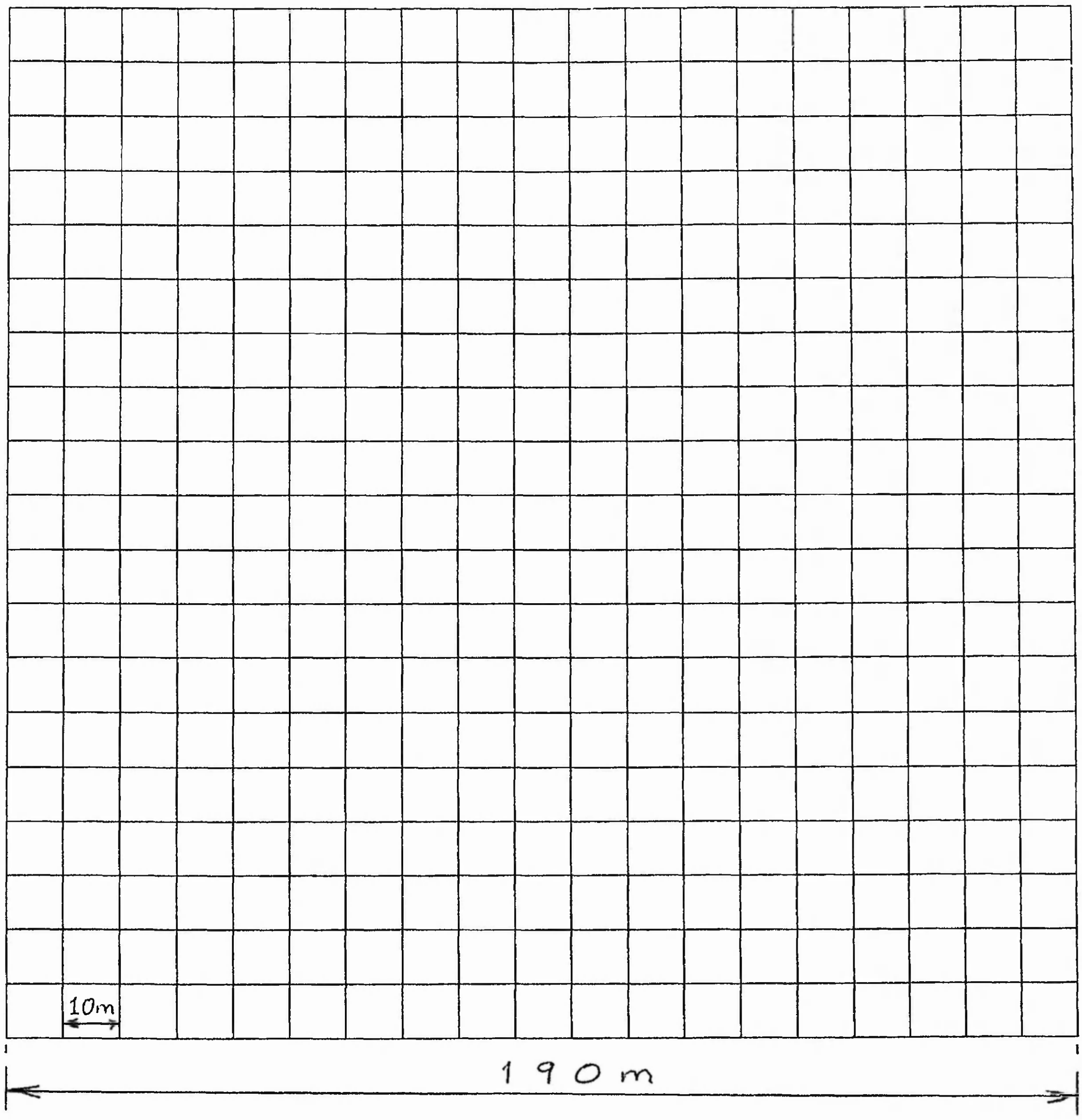
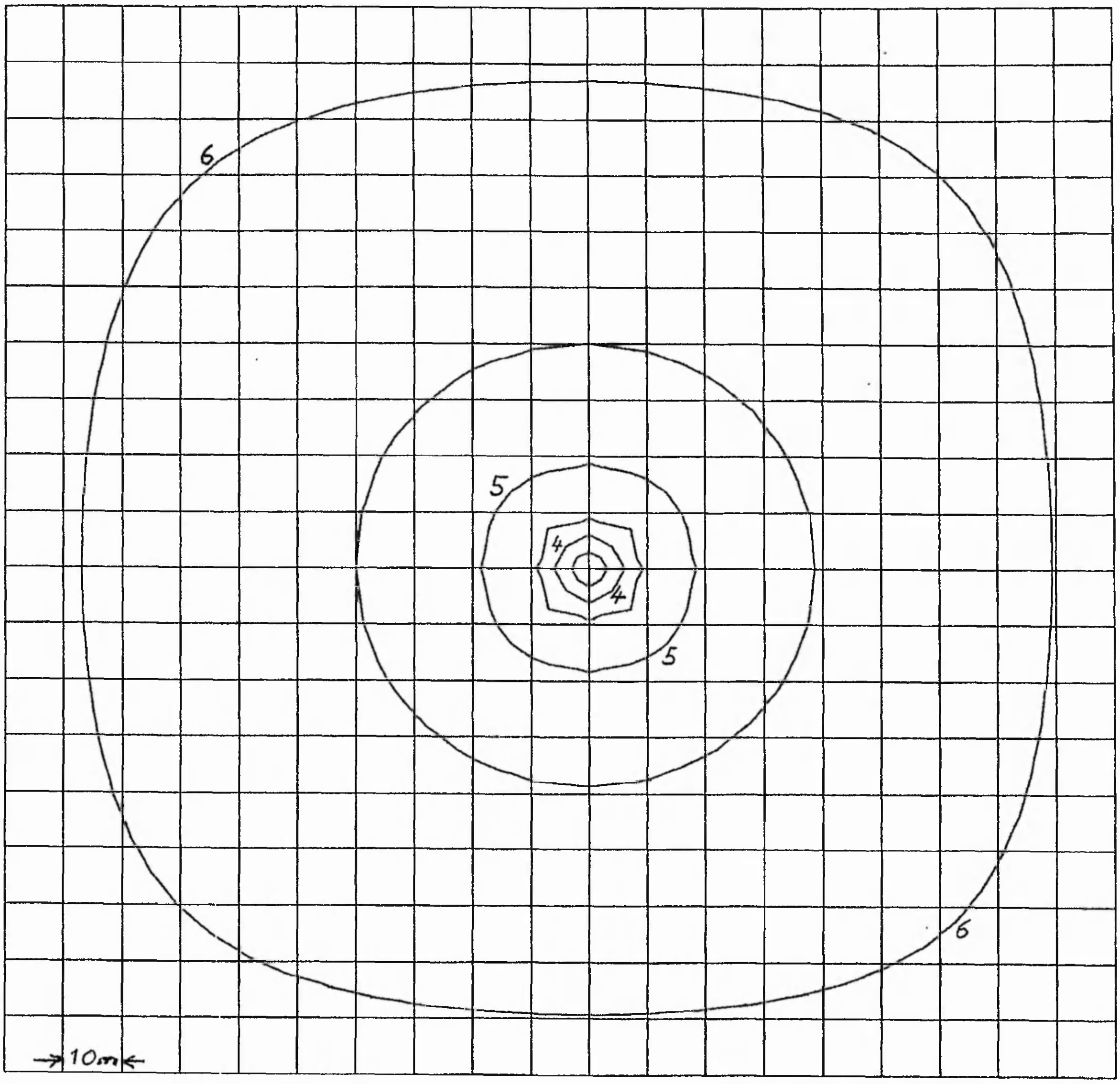


Figure 7.6

The Mesh Used To Model The Pump Test

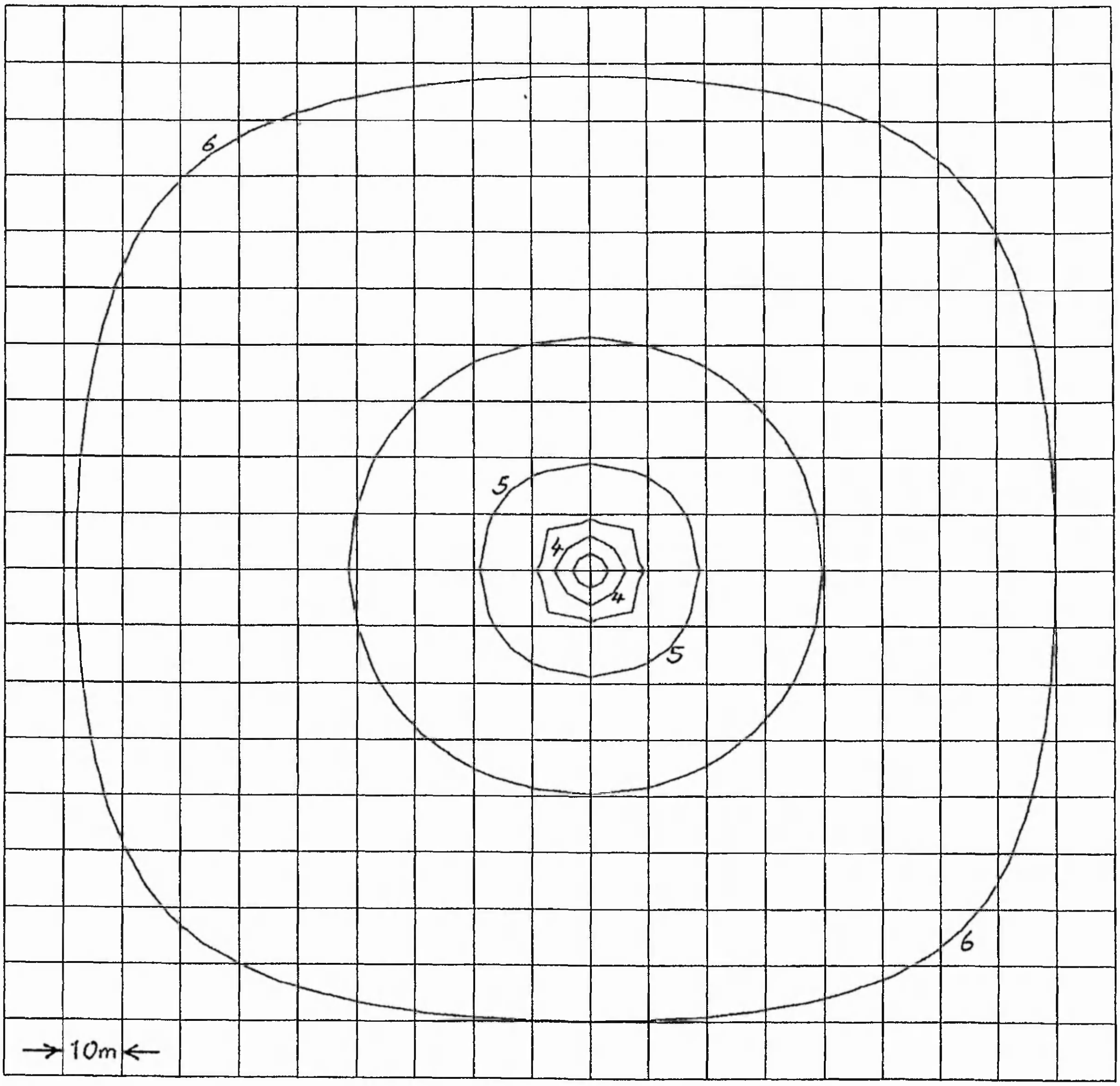


N.B:

Groundwater level contours are at intervals of 0.5m.
3.1m head specified at the central node.
6.1m head specified at the mesh boundary.

Figure 7.7

Results From Modelling Of The Pump Test With Infiltration



N.B:

Groundwater level contours are at intervals of 0.5m.
3.1m head specified at the central node.
6.1m head specified at the mesh boundary.

Figure 7.8

Results From Modelling Of The Pump Test Without Infiltration

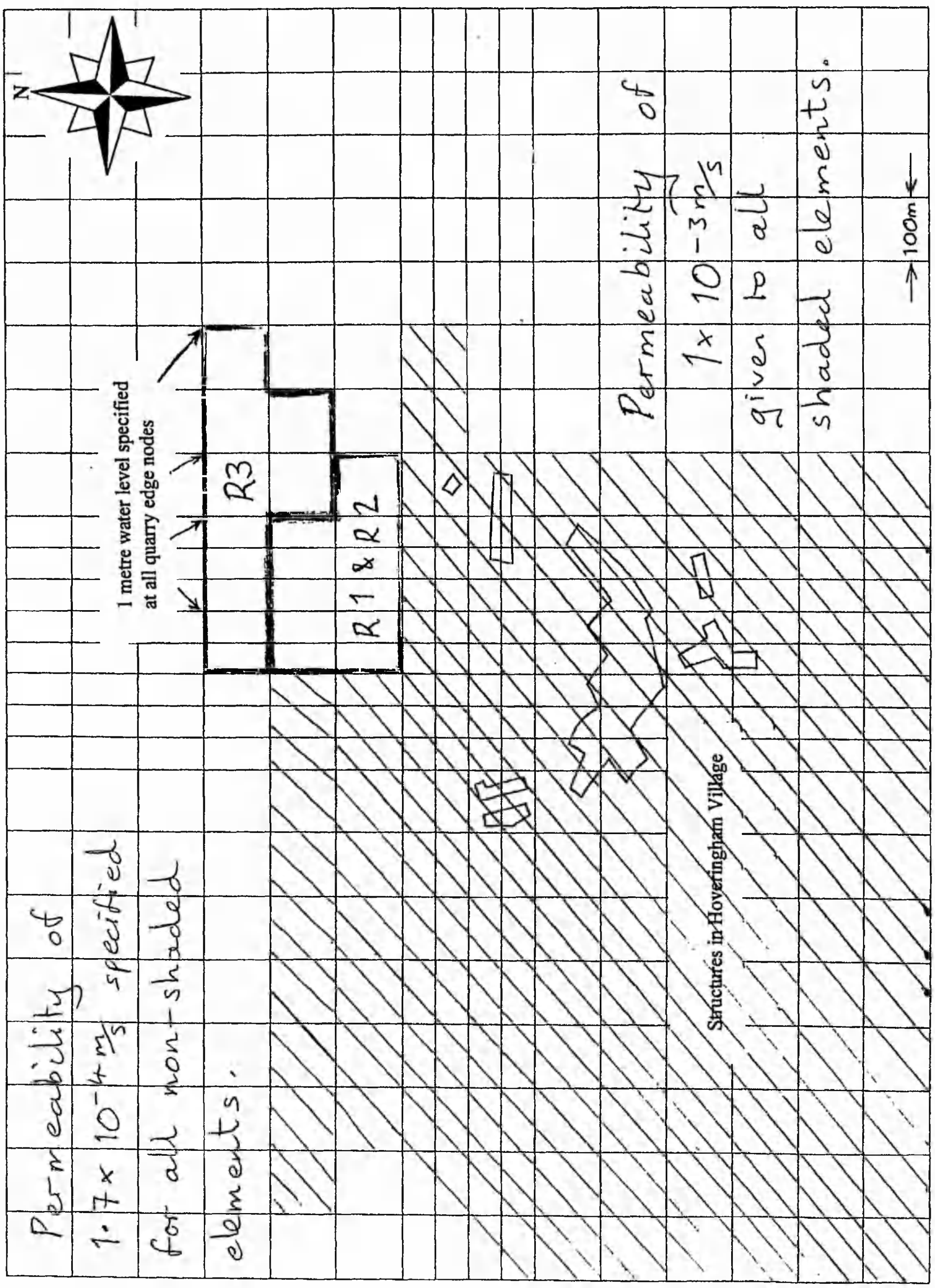


Figure 7.9

Permeability Distribution Used For Elements In Runs HOVA2-HOVA9

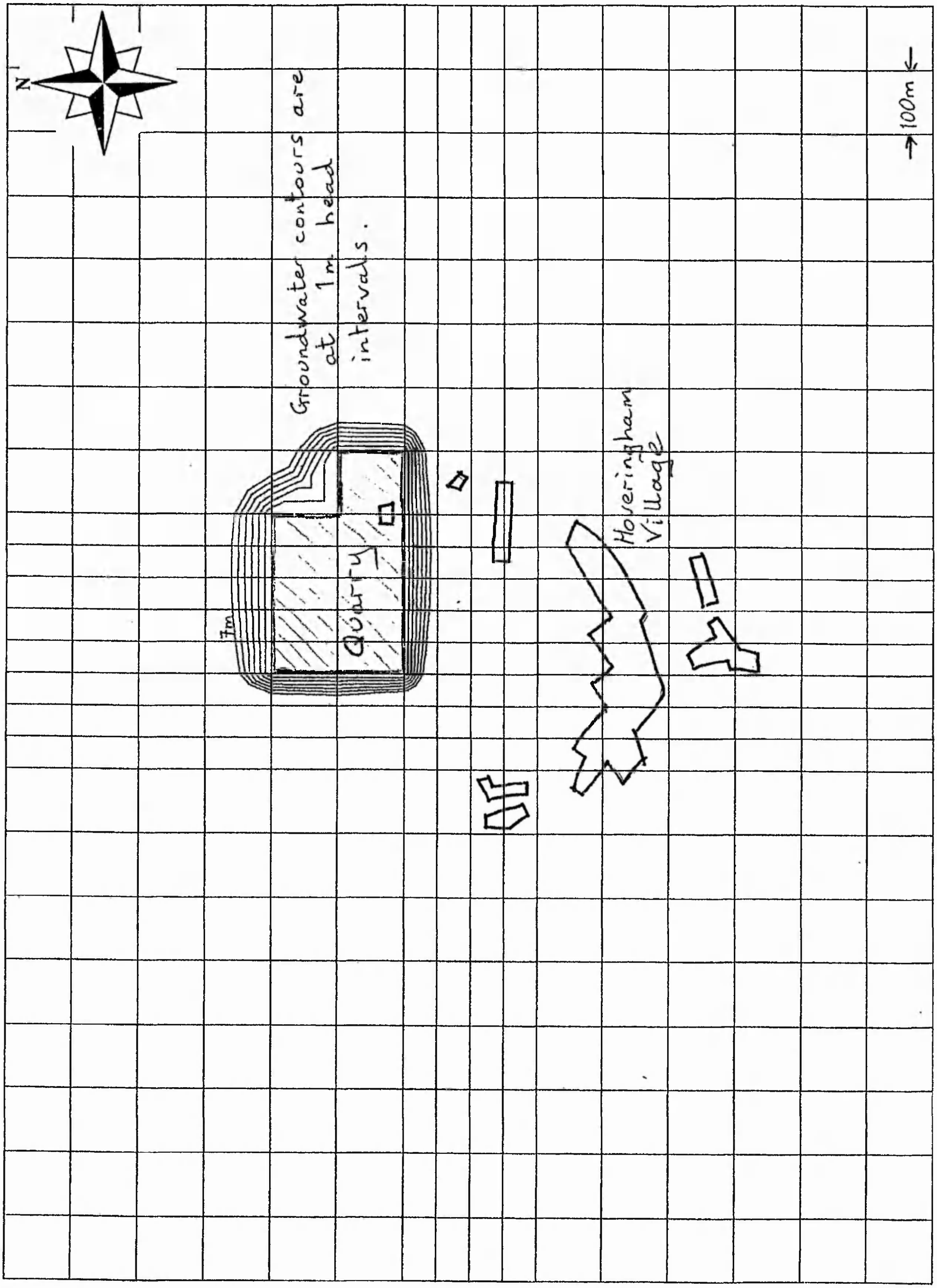


Figure 7.10 (a) - Plot For HovA1

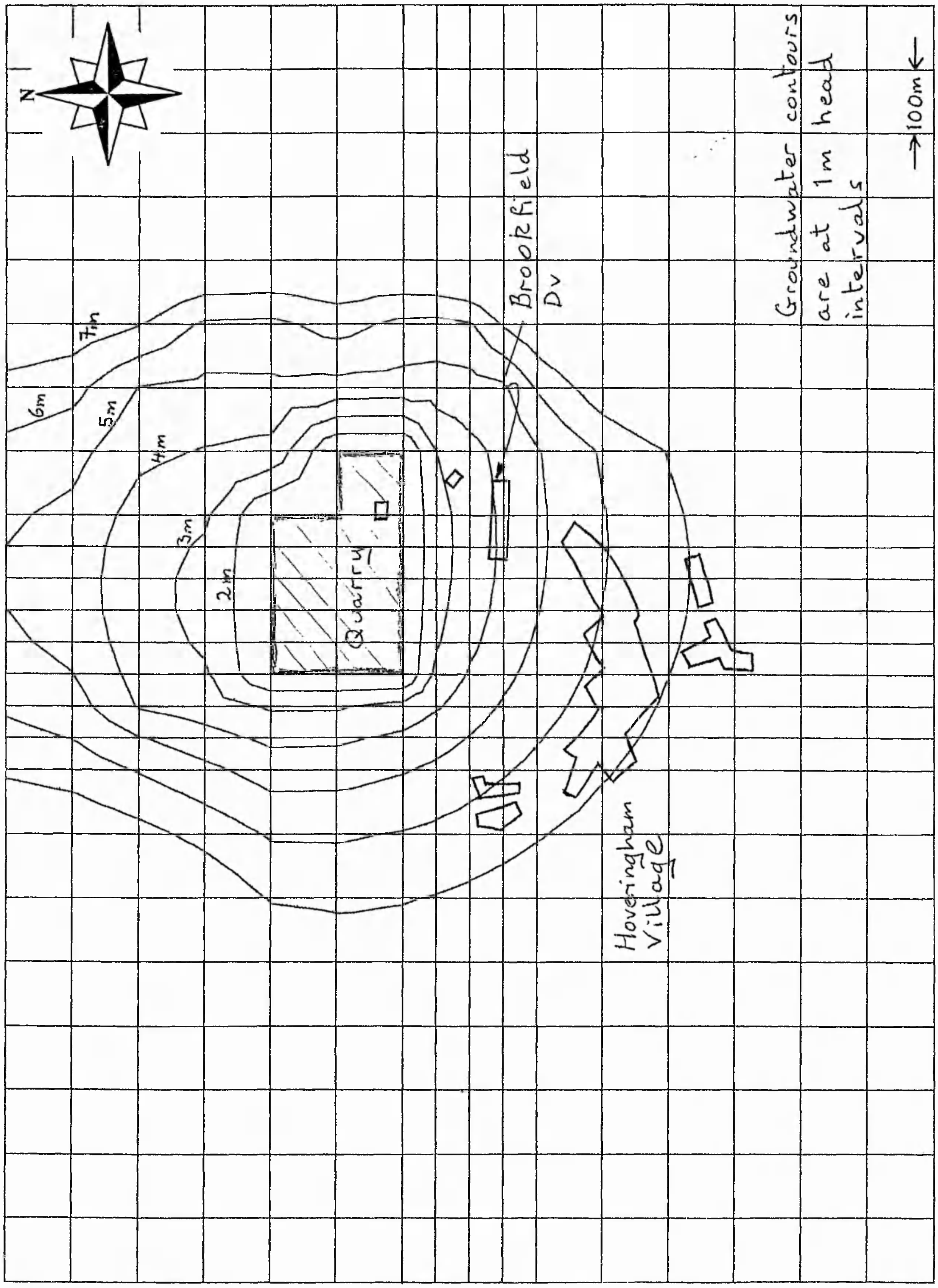


Figure 7.10 (b) - Plot For HovA2

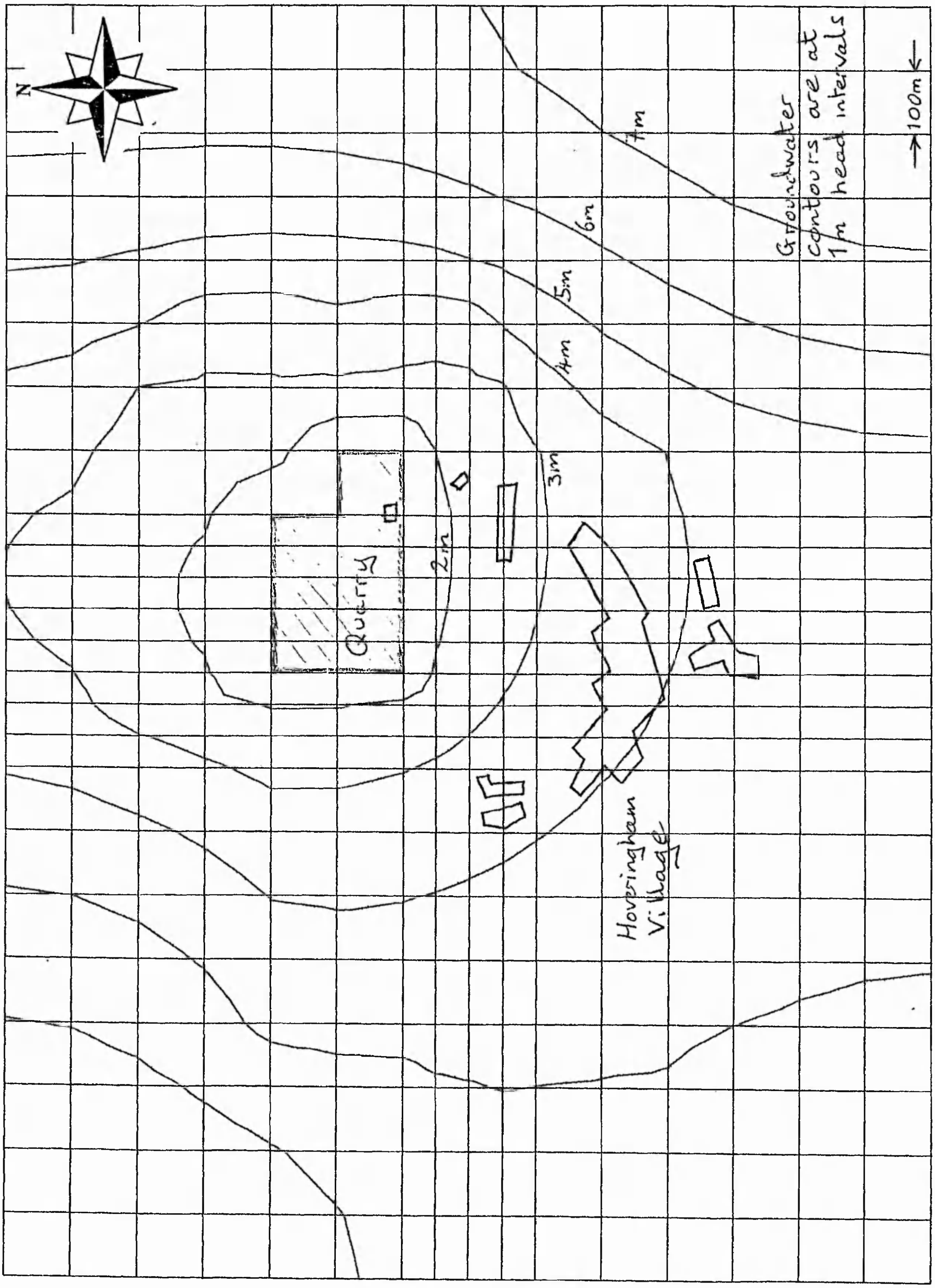


Figure 7.10 (c) - Plot For HovA3

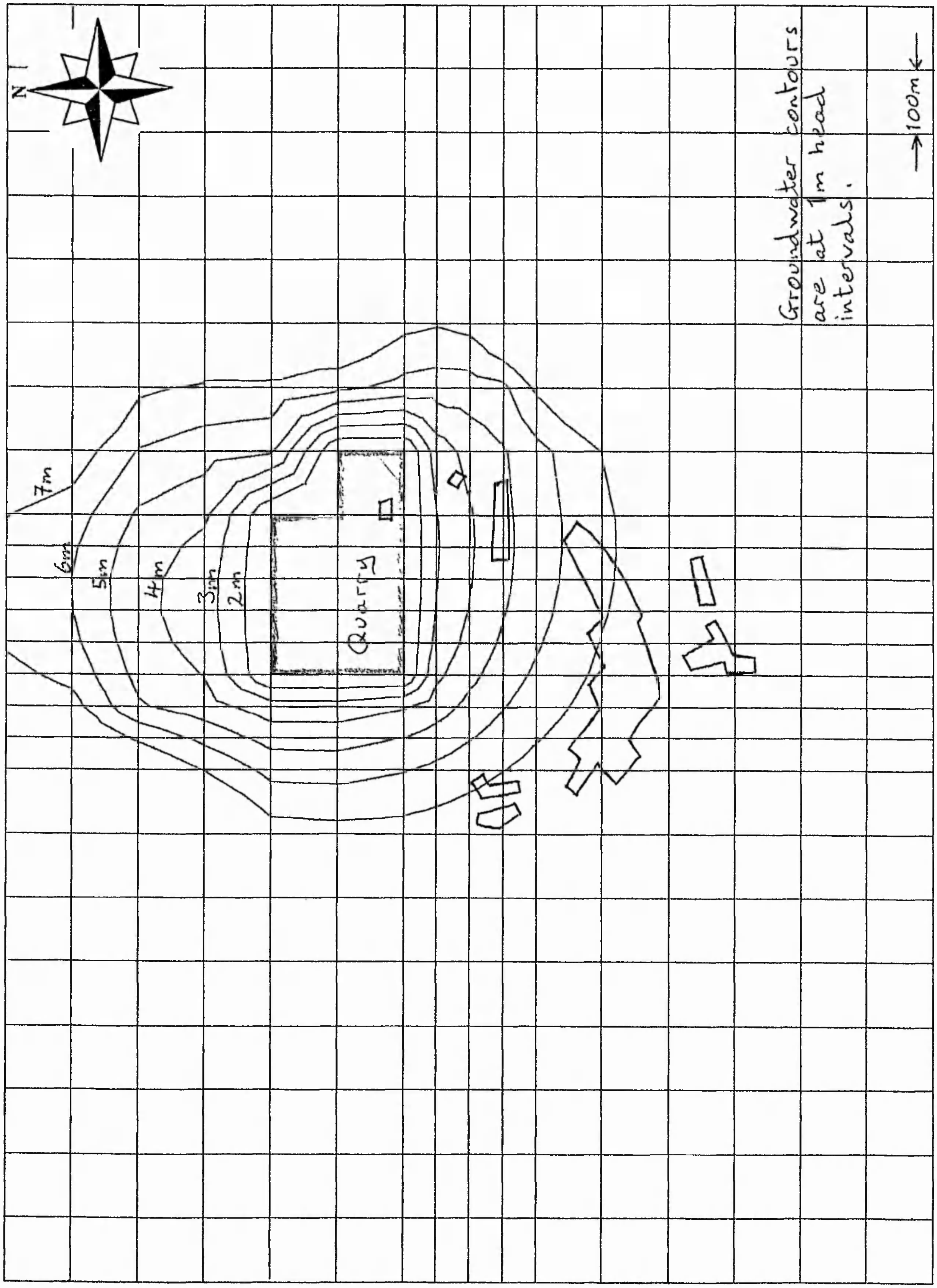


Figure 7.10 (d) - Plot For HovA4

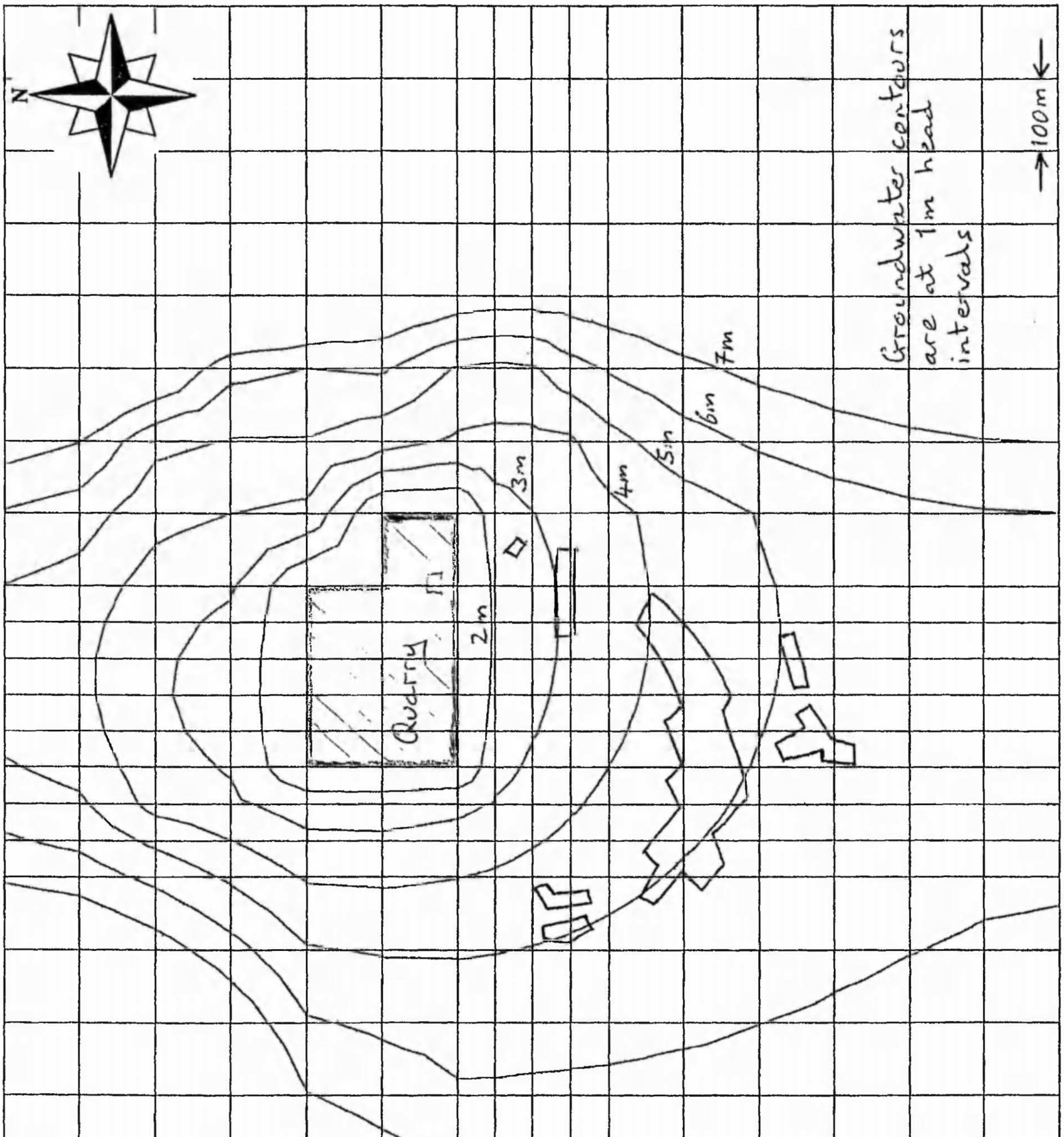


Figure 7.10 (e) - Plot For HovA5

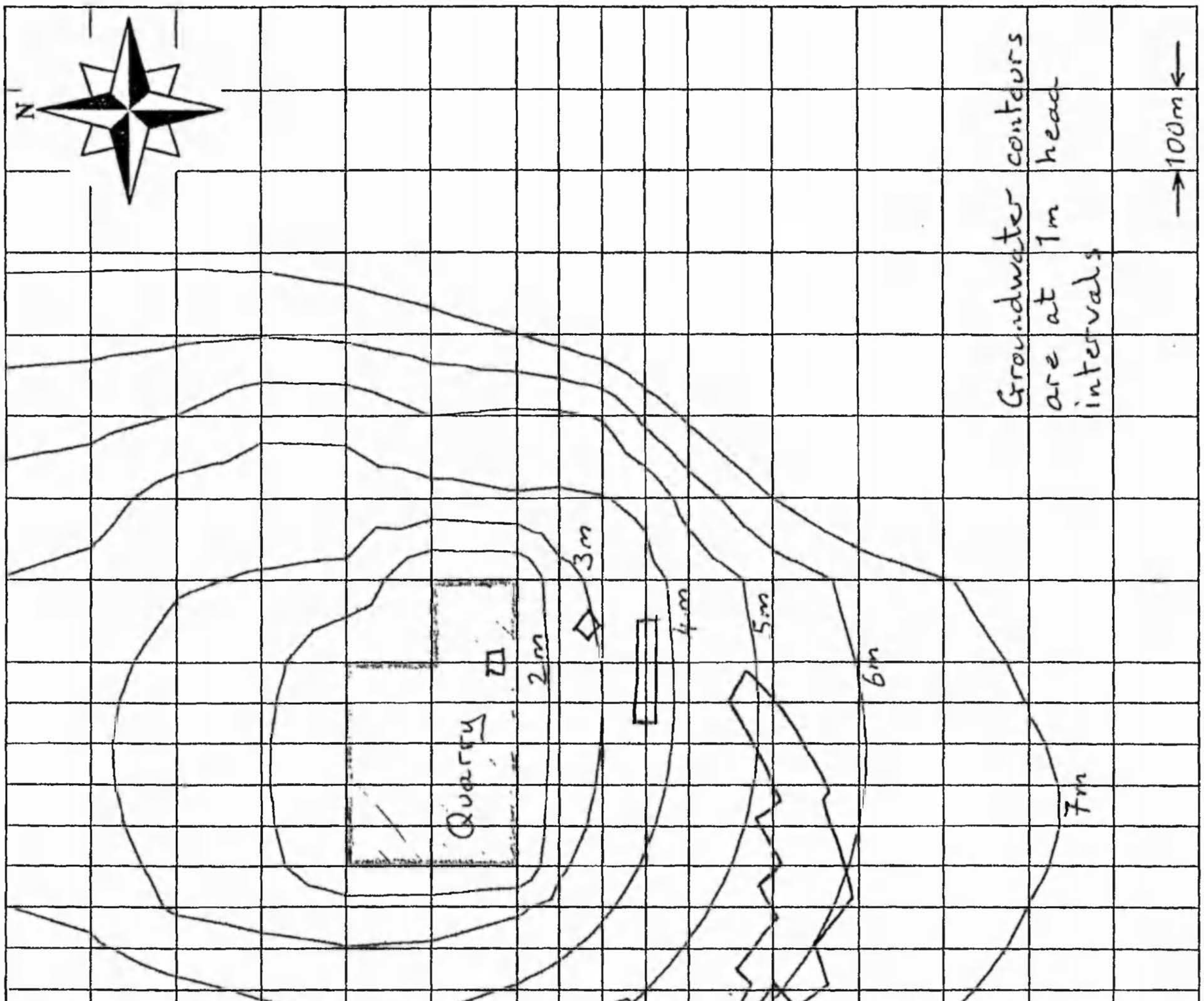


Figure 7.10 (f) - Plot For HovA6

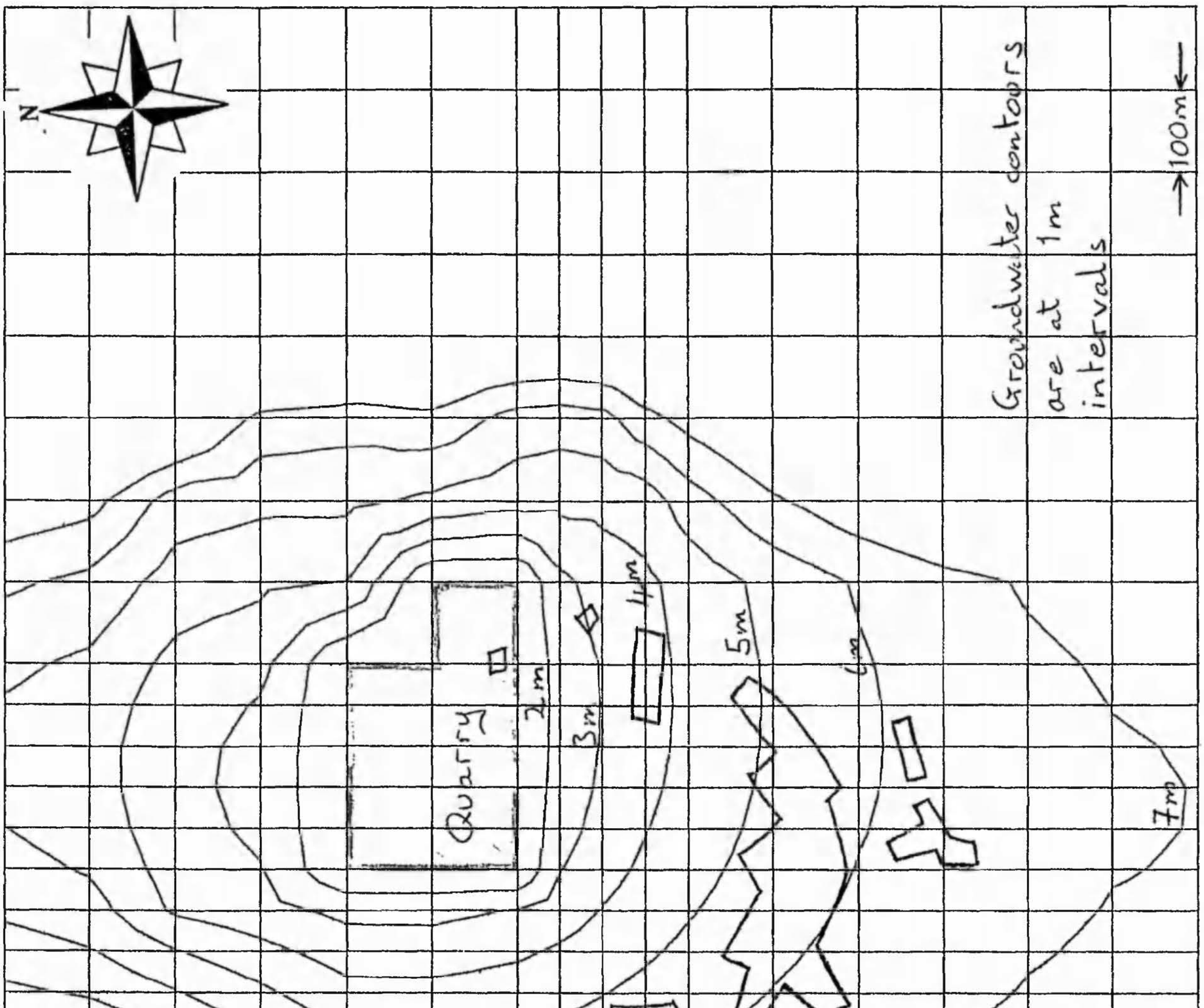


Figure 7.10 (g) - Plot For HovA7

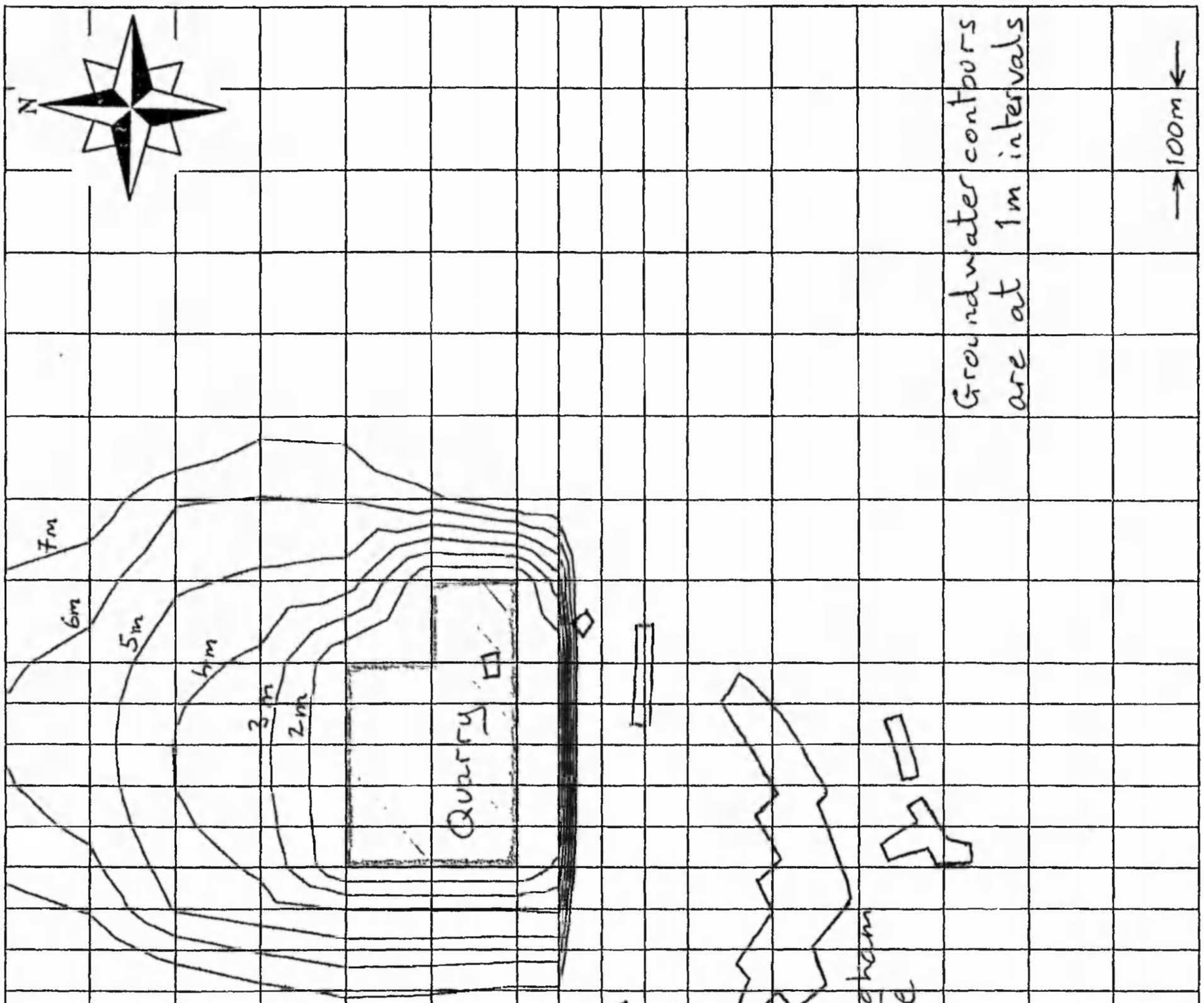


Figure 7.10 (ii) - Plot For HovAS

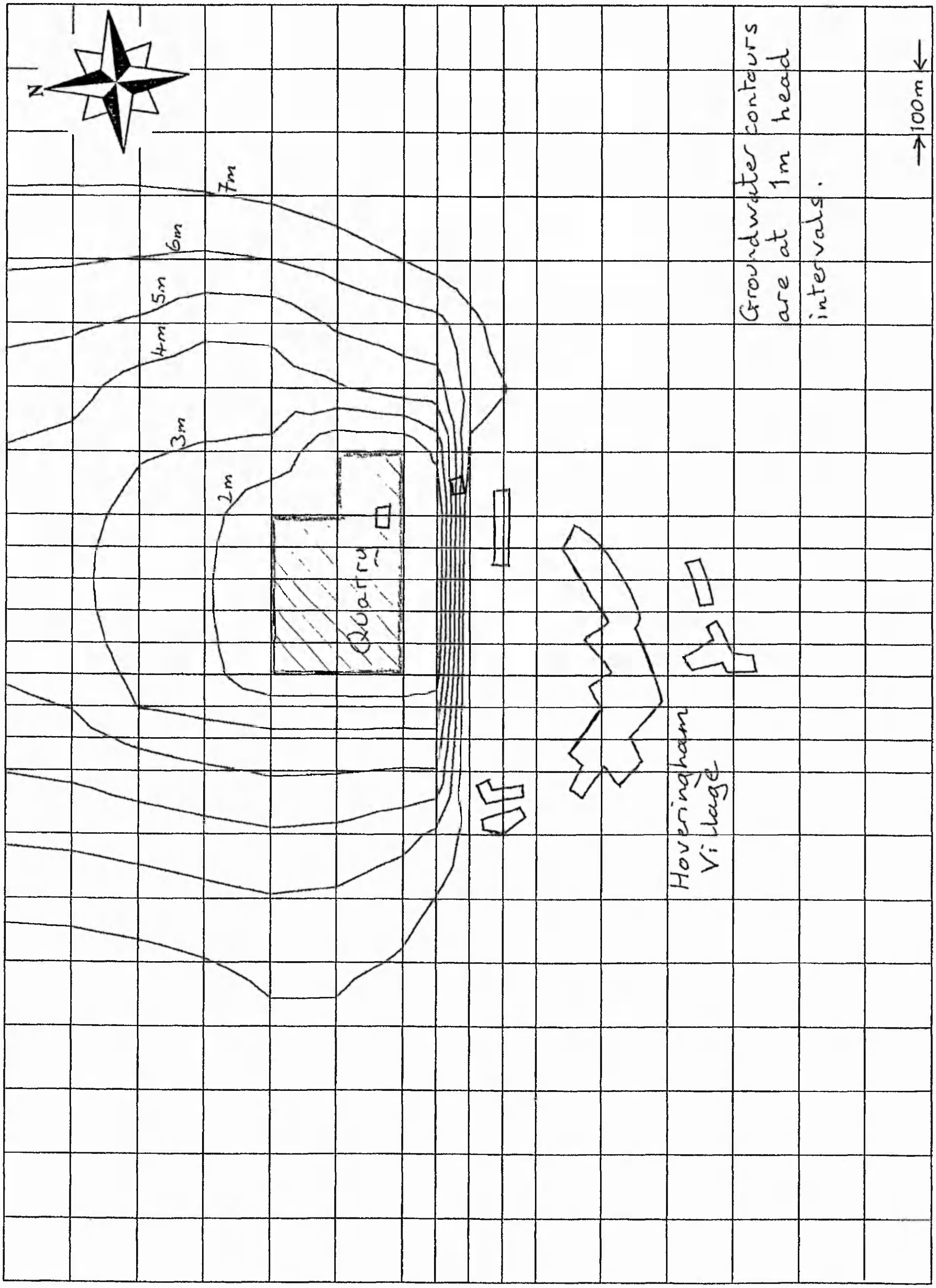


Figure 7.10 (i) - Plot For HovA9

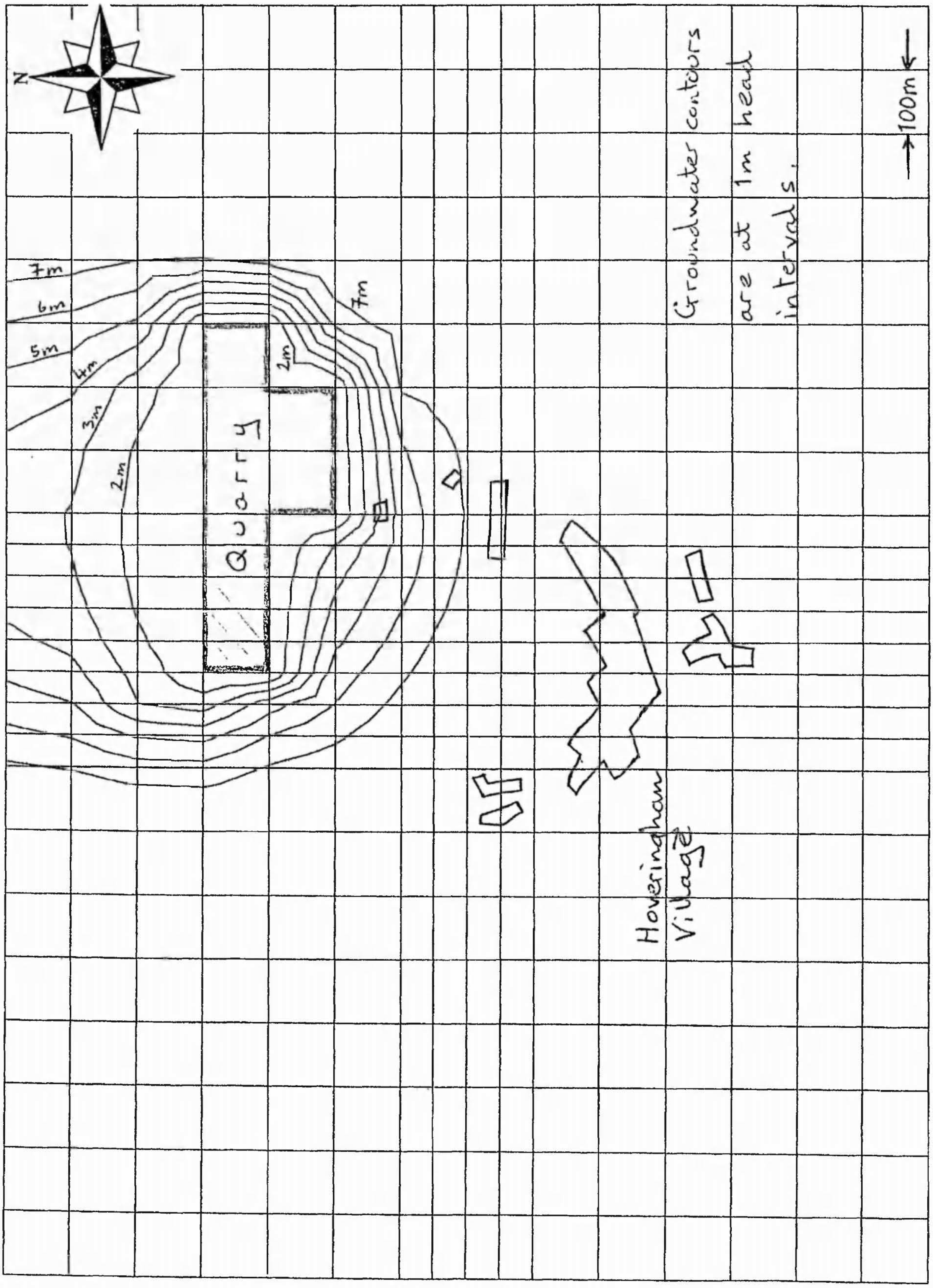


Figure 7.10 (i) - Plot For HovB1

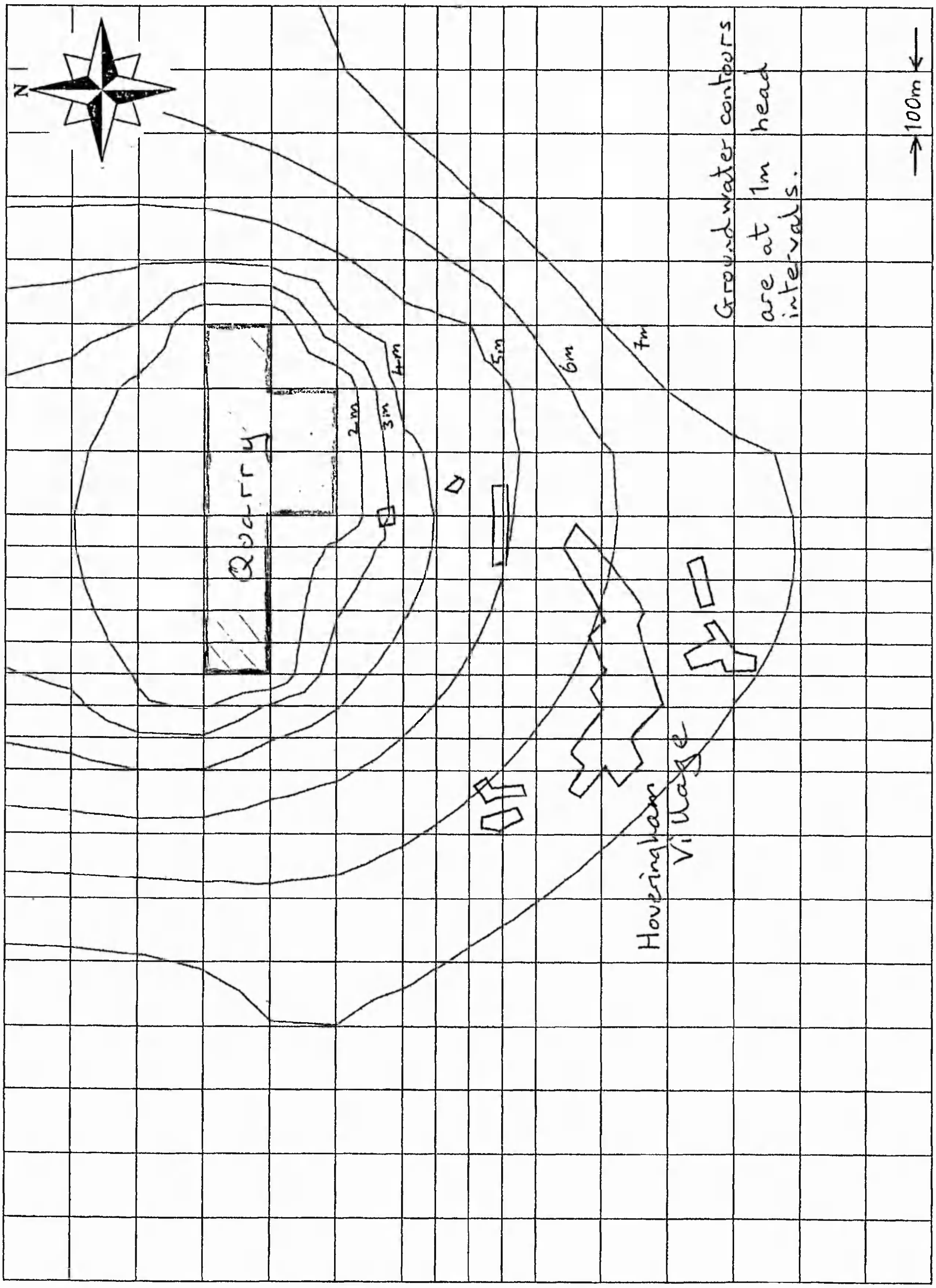


Figure 7.10 (k) - Plot For HovB2

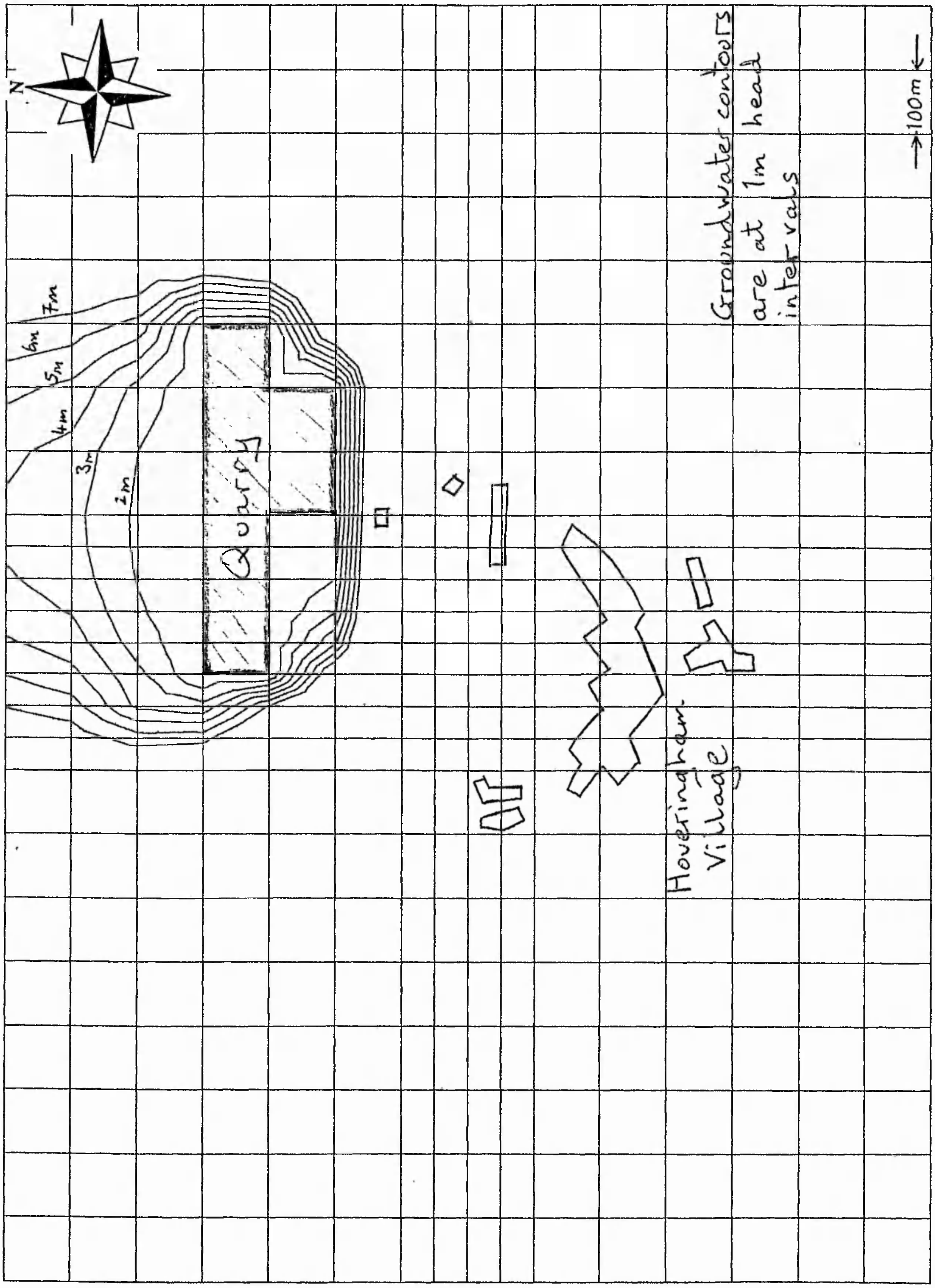


Figure 7.10 (l) - Plot For HovB3

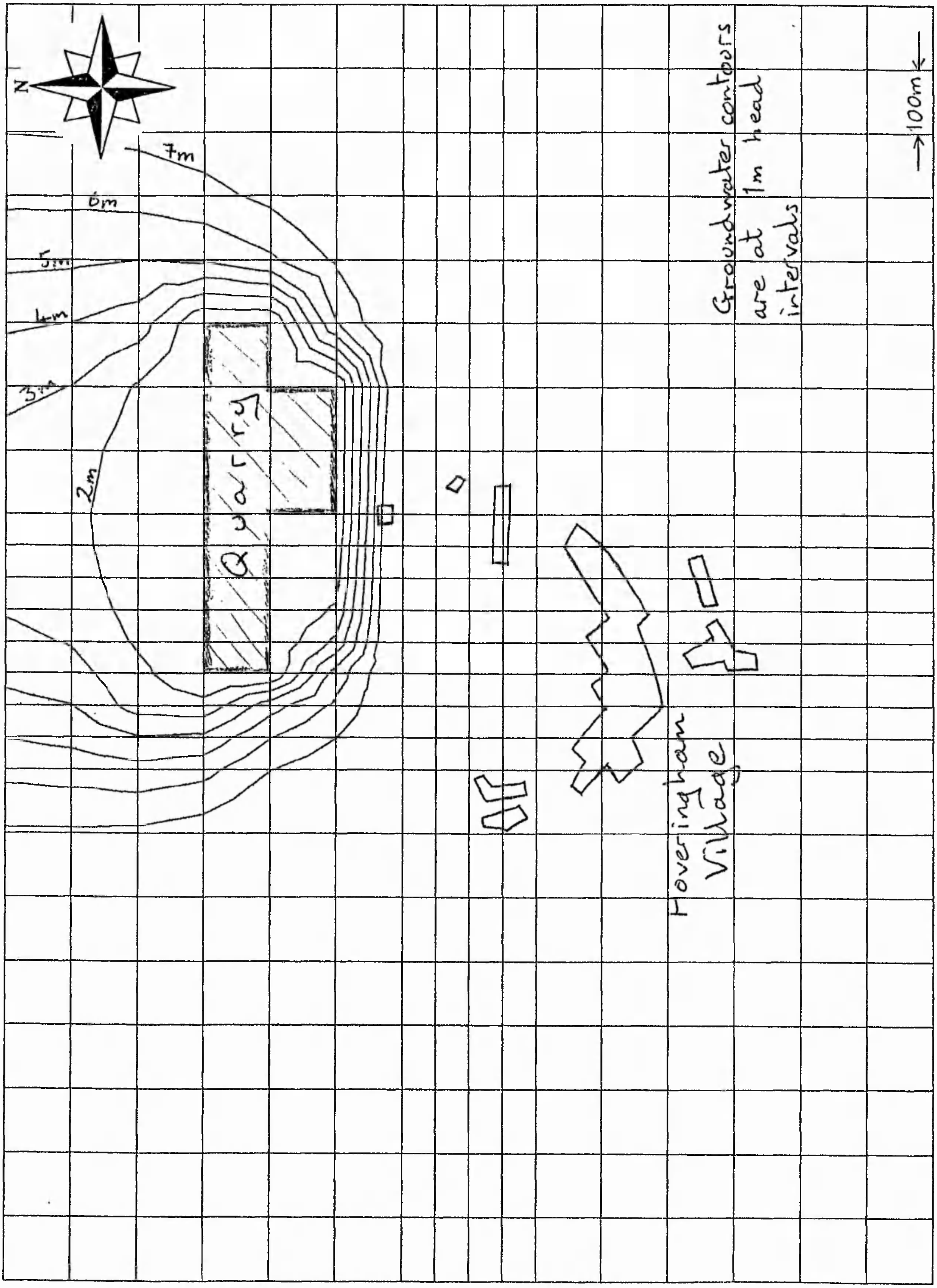


Figure 7.10 (m) - Plot For HovB4

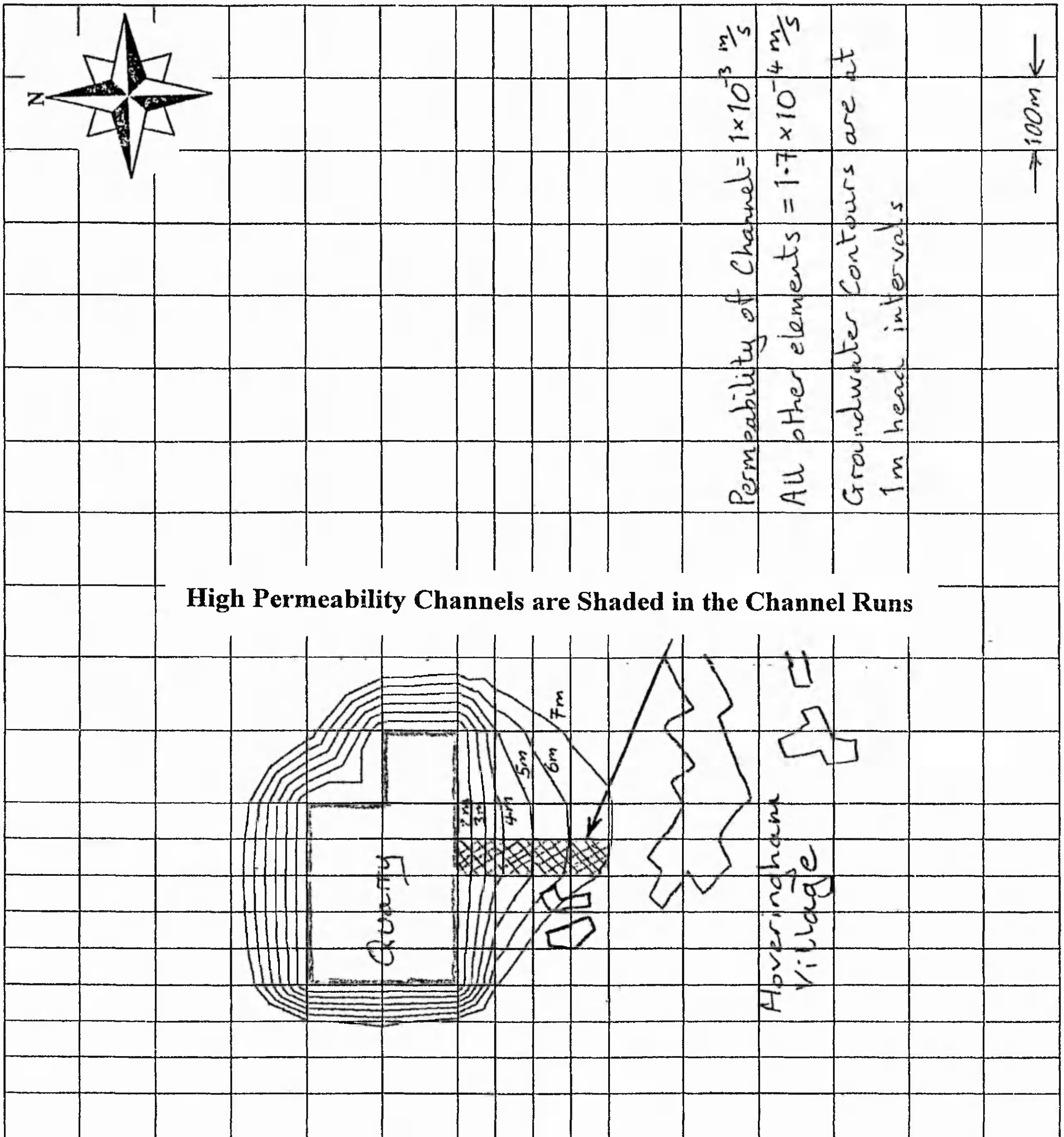


Figure 7.10 (n) - Plot for Channel1

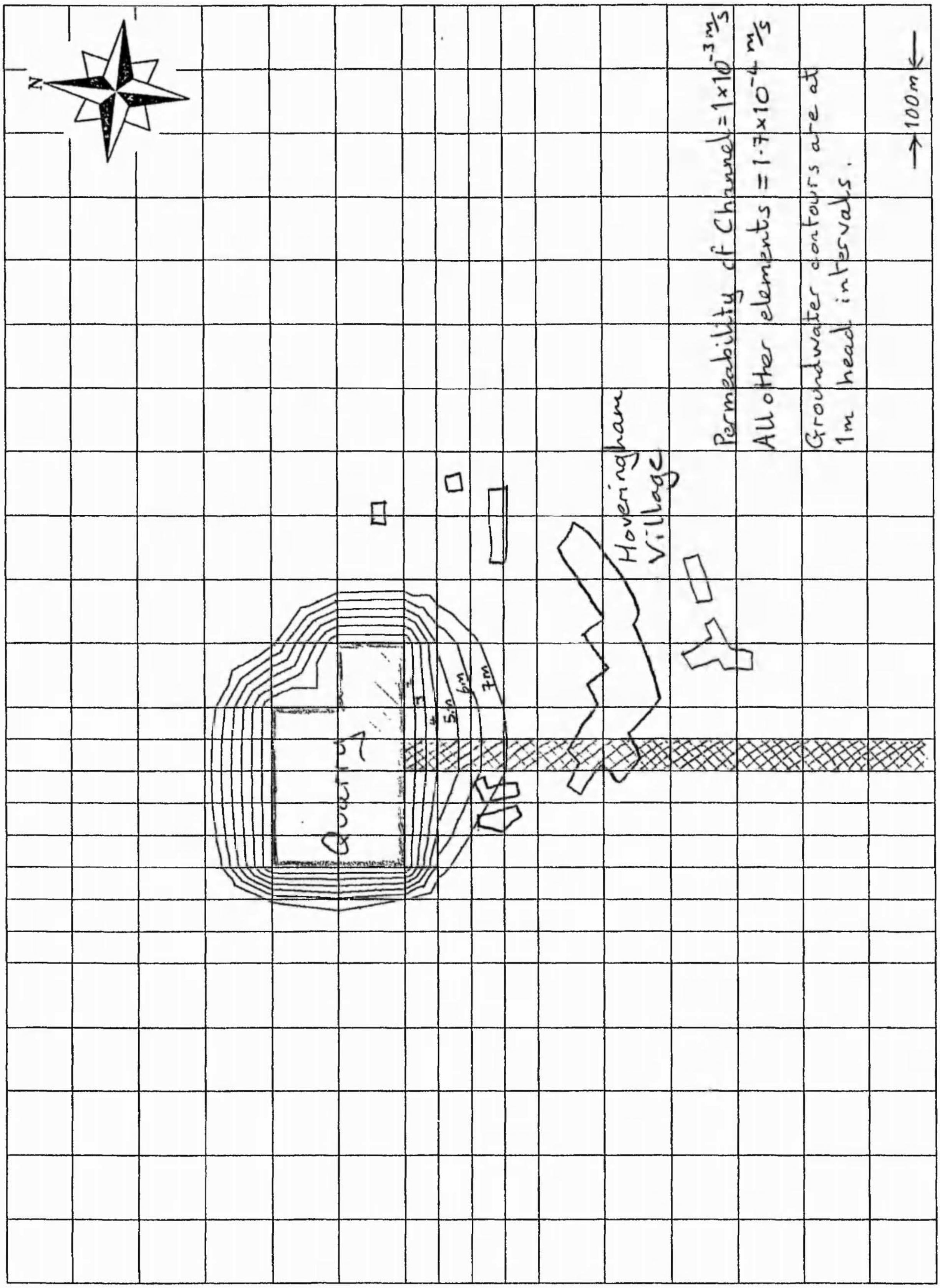


Figure 7.1ft (a) - Plot for Channel2

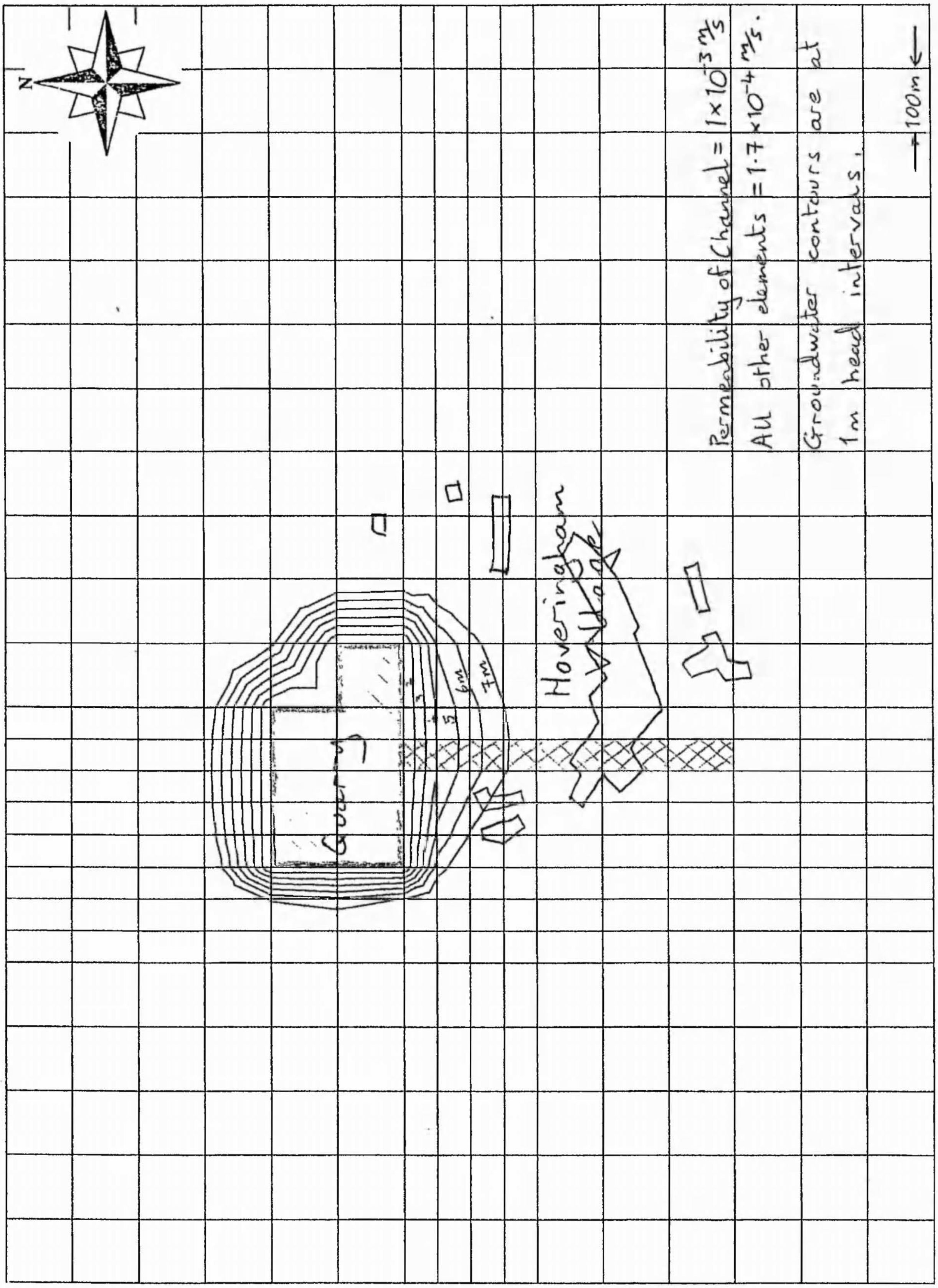


Figure 7.10 (p) - Plot for Channel3

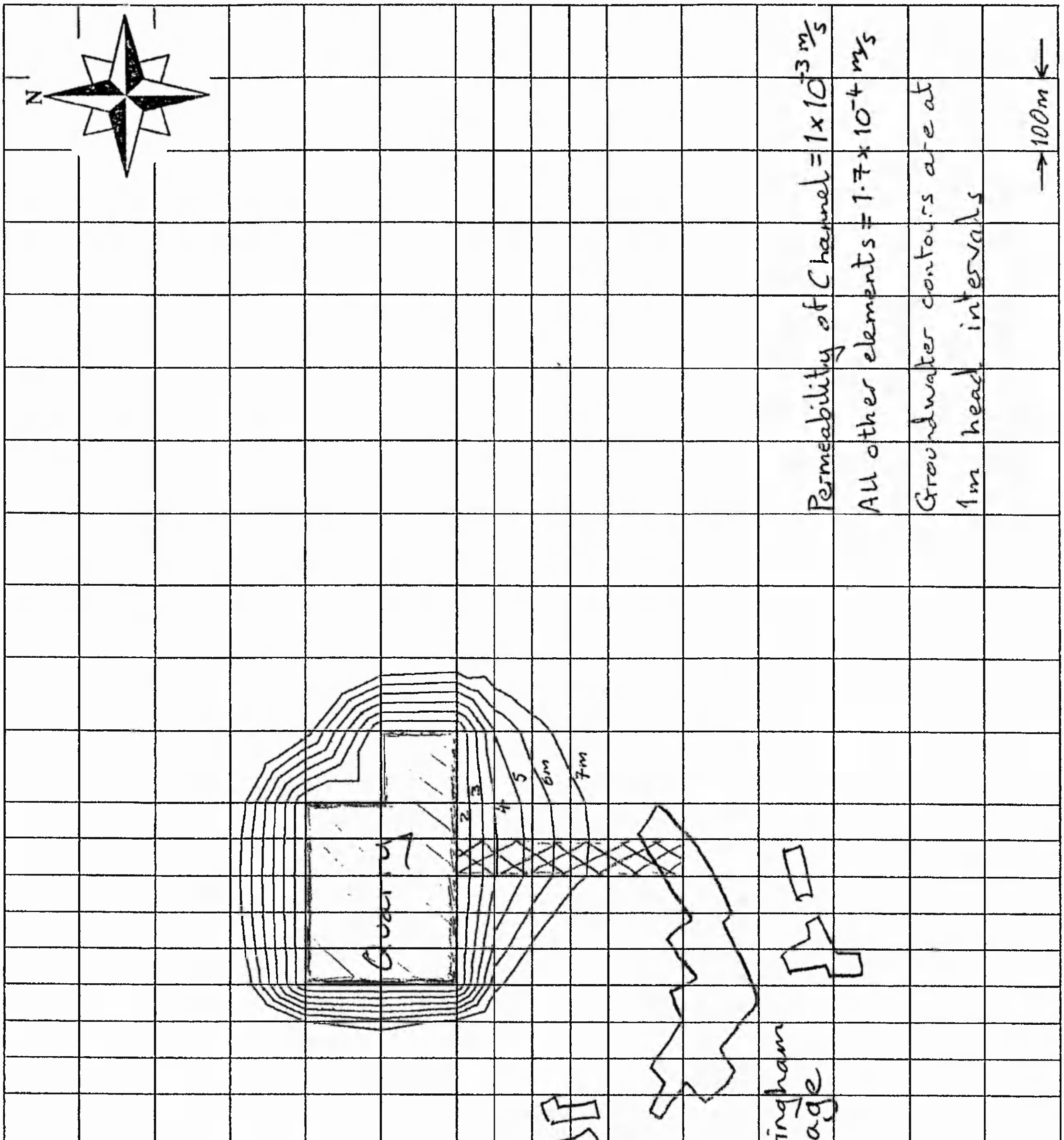


Figure 7.10 (q) - Plot for Channel4

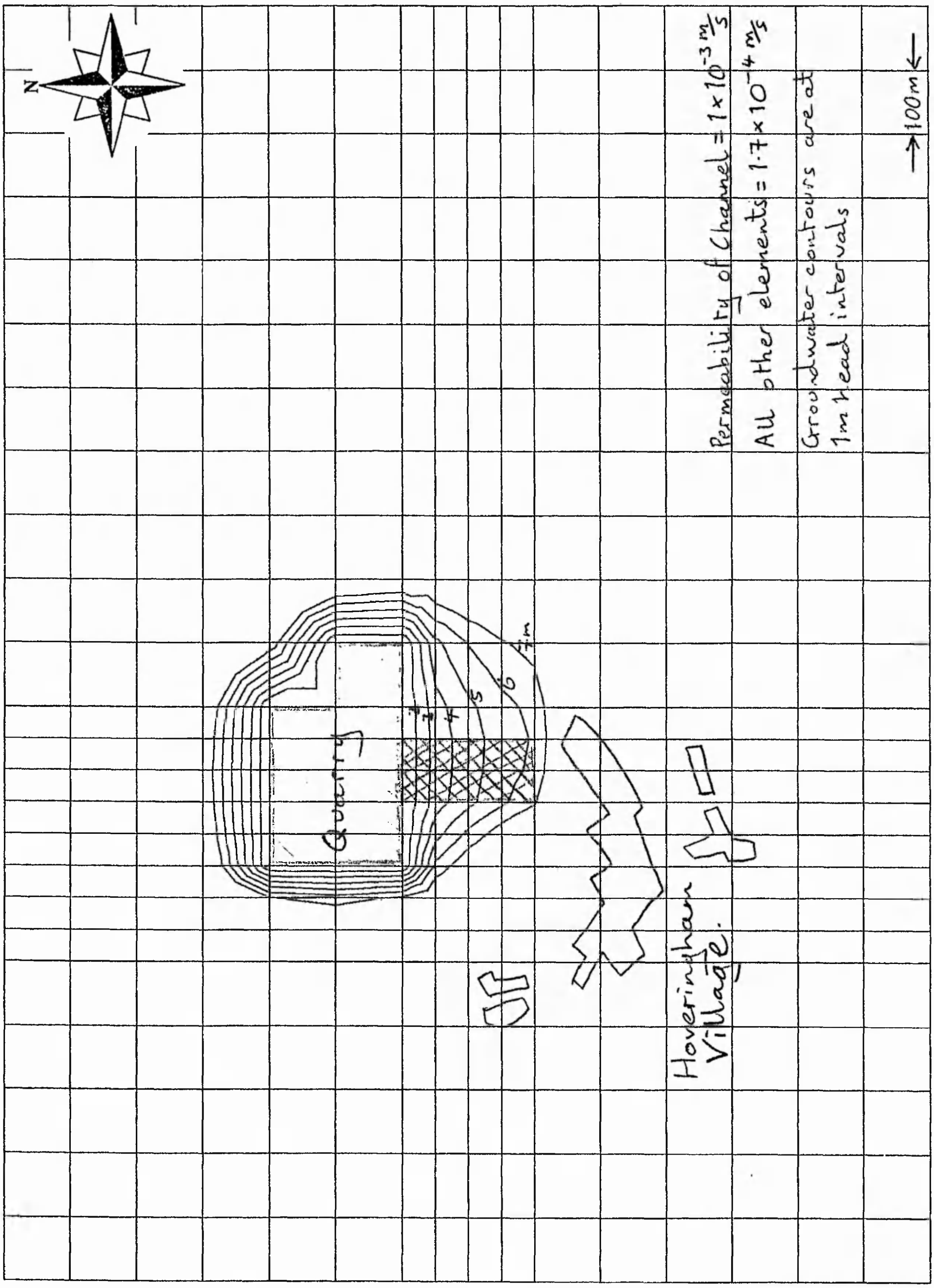


Figure 7.10 (r) - Plot for Channel5

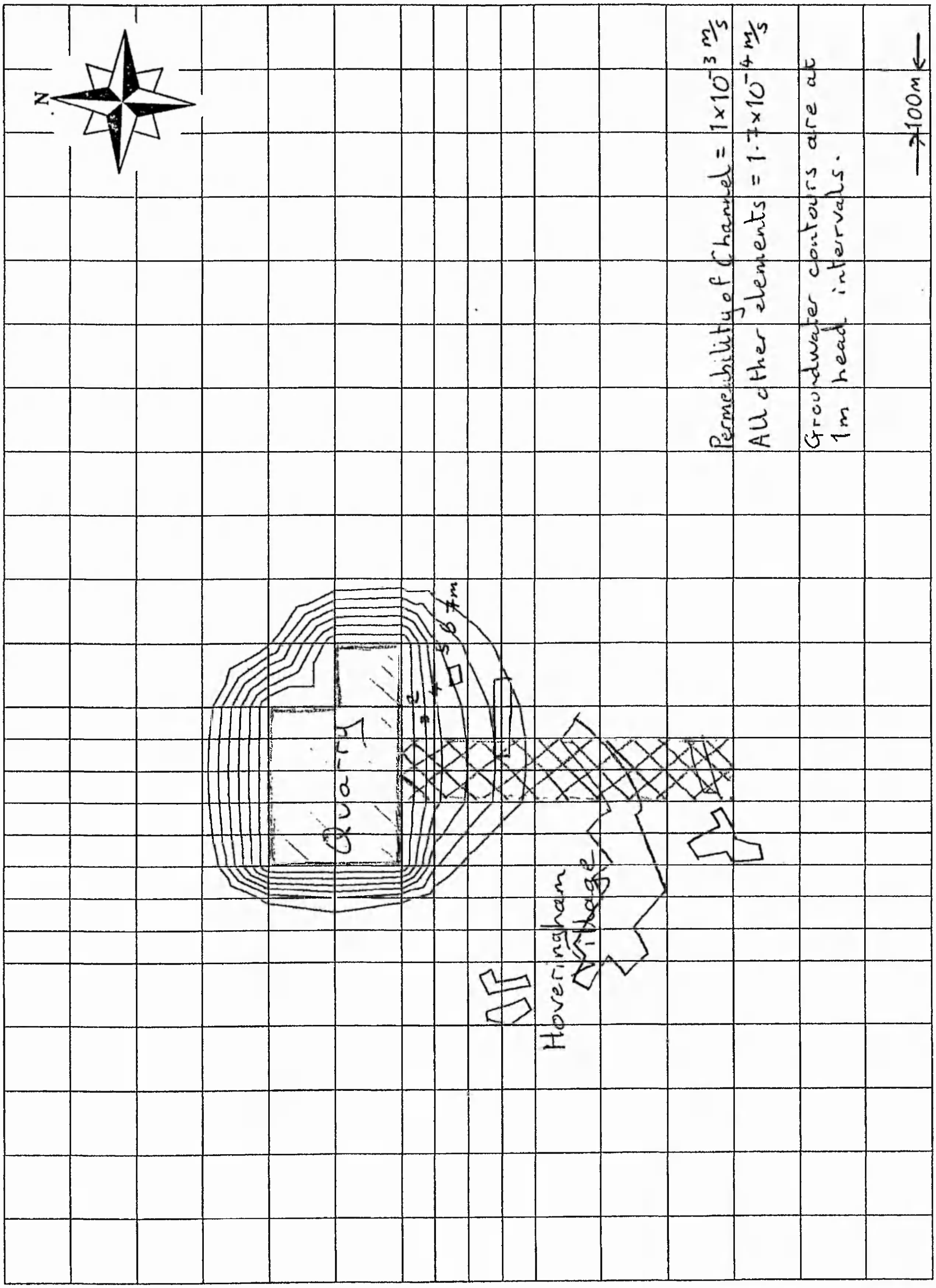


Figure 7.10 (s) - Plot for Channel6

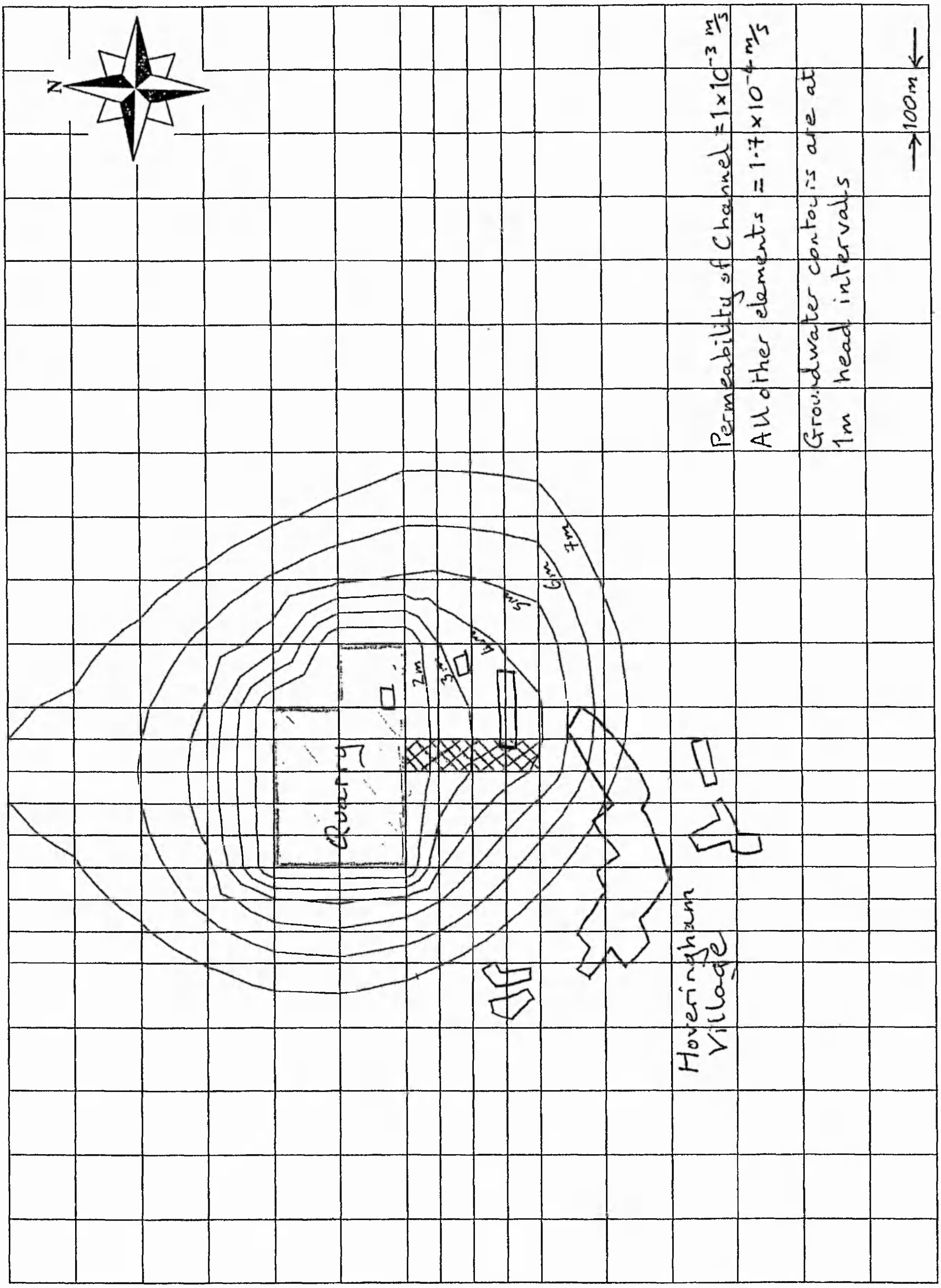


Figure 7.10 (t) - Plot for Channel7

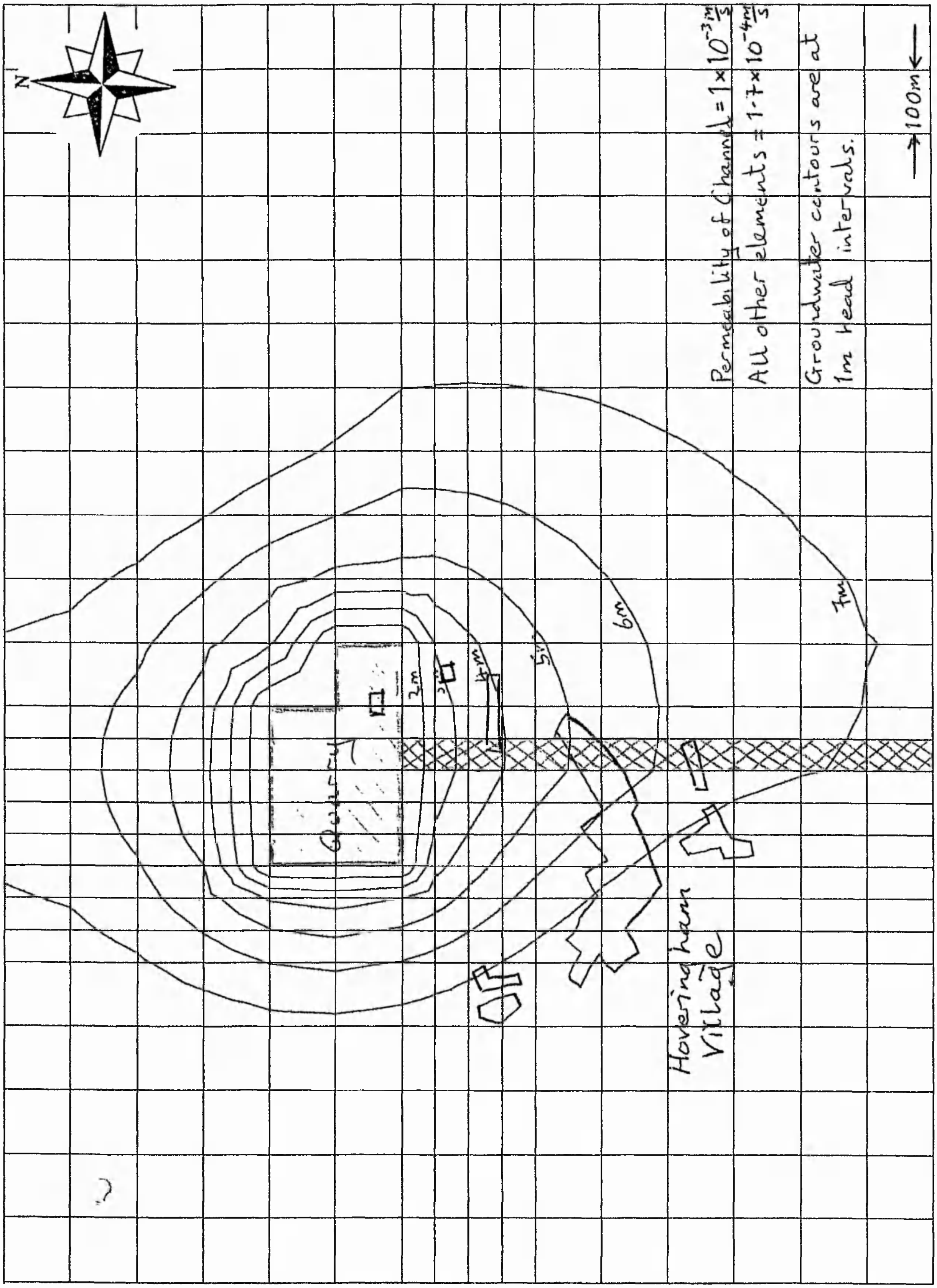


Figure 7.10 (u) - Plot for Channel8

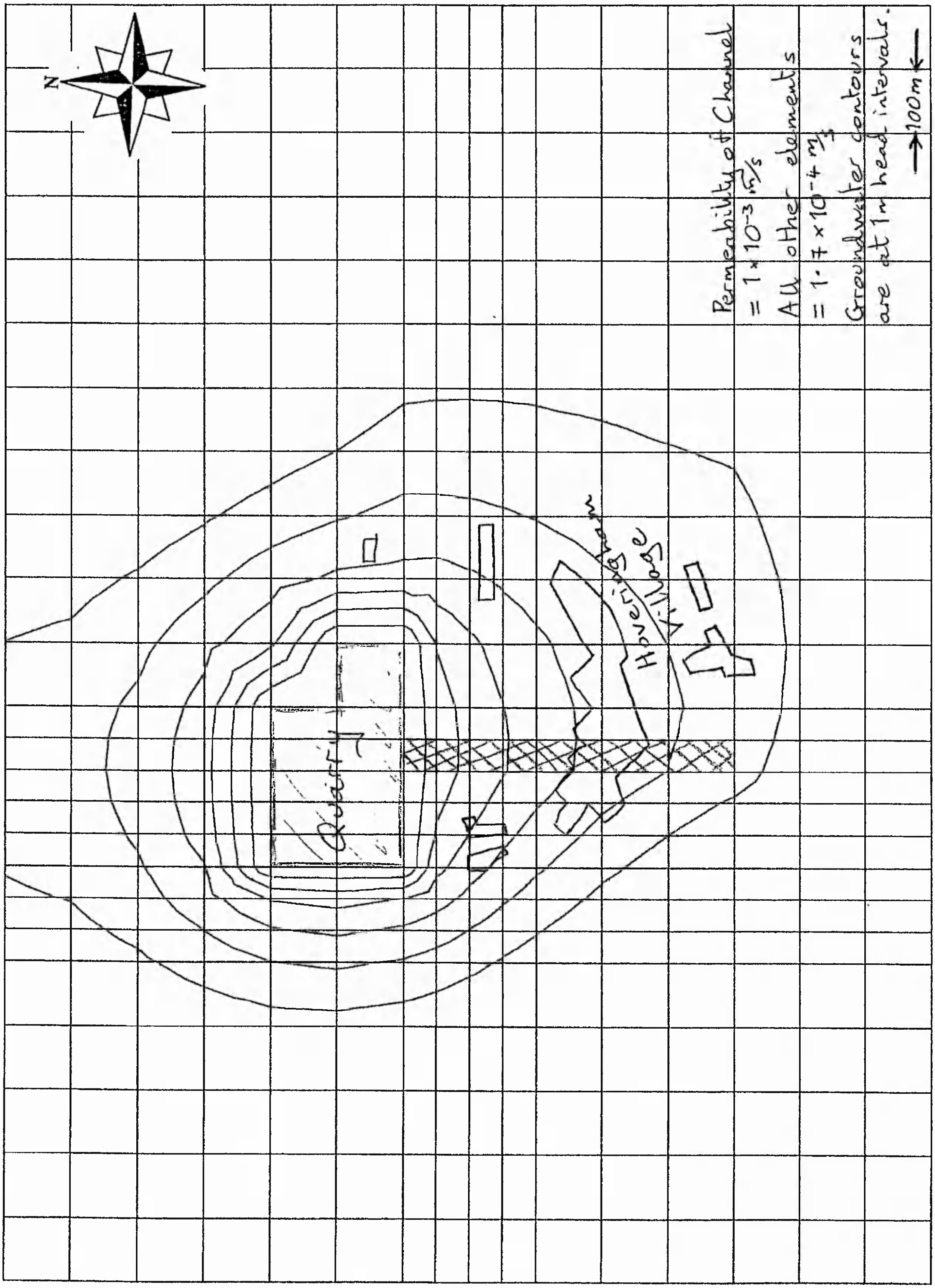


Figure 7.10 (v) - Plot for Channel9

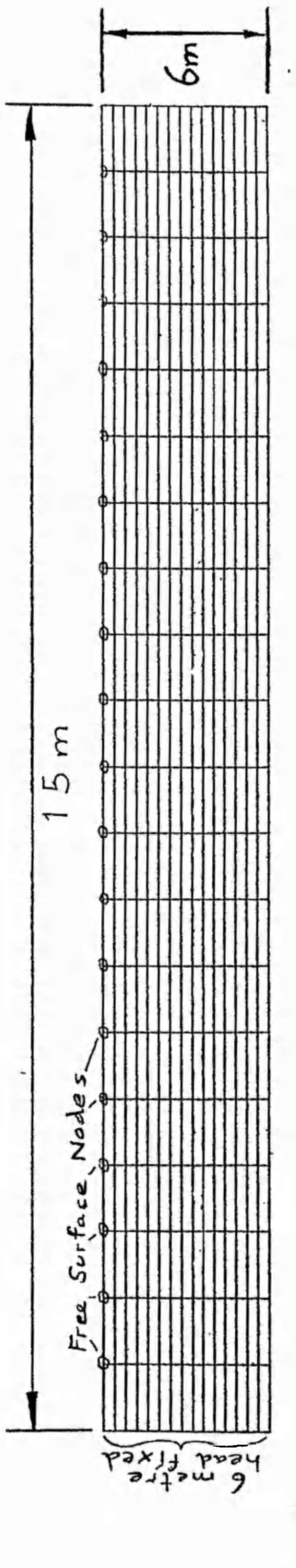


Figure 7.11

The Mesh Specified For The FLONET Analyses

Note:
1. Phreatic surface nodes have been shifted downwards.
2. Equipotentials are drawn at 0.5m intervals.
3. The mesh shown is for run "Layers4".

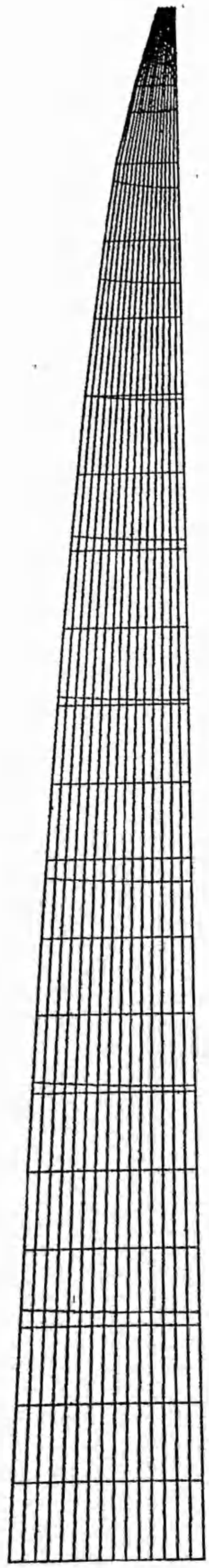


Figure 7.12

Example Of A Deformed Mesh After A FLONET Run

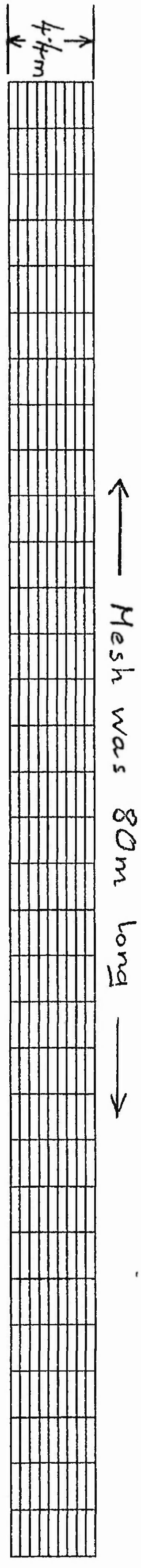


Figure 7.13

The Mesh Used To Model The Pump Test With FLONET

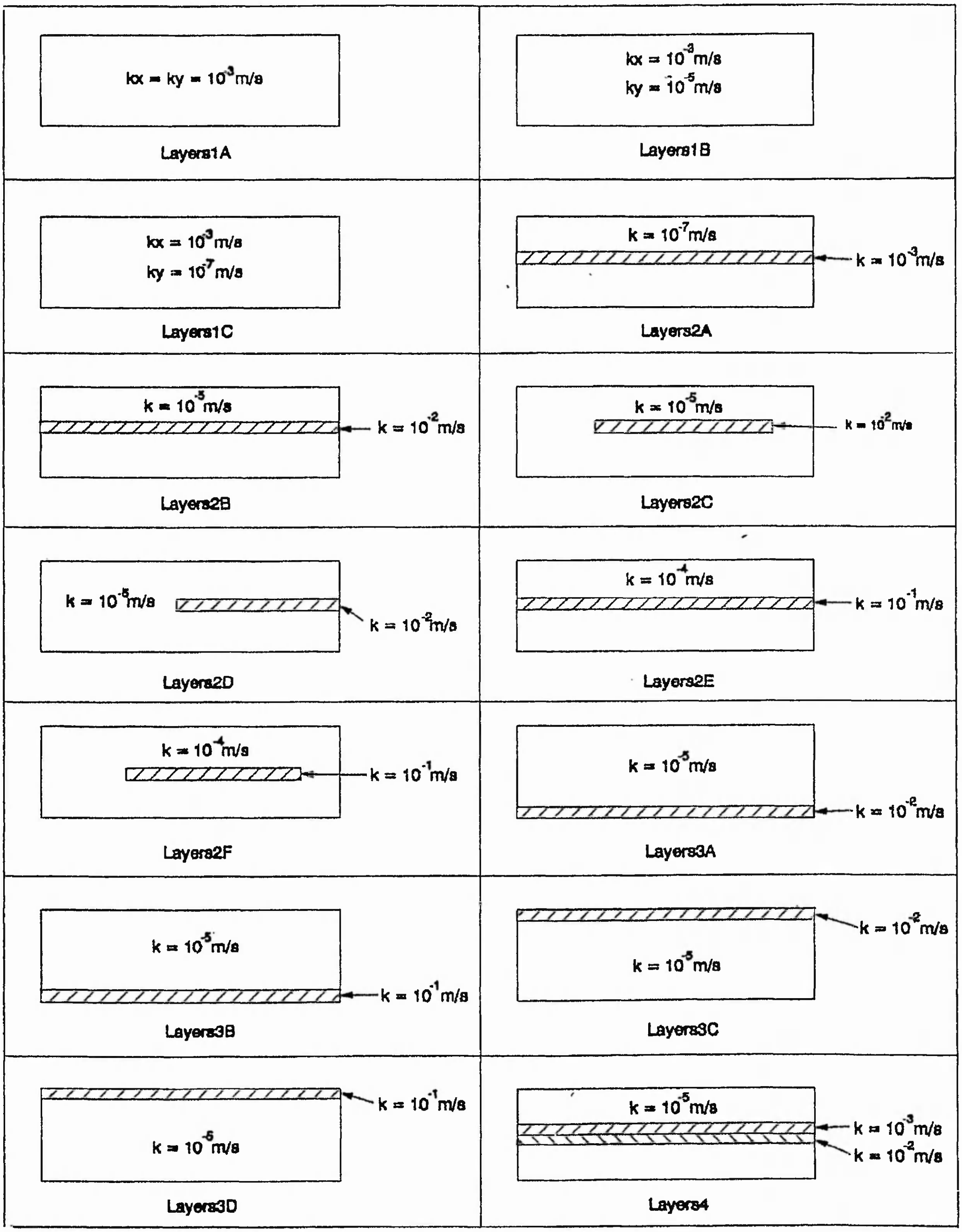


Figure 7.14

Permeability Variation Used During The FLONET Analyses

CHAPTER EIGHT
CONCLUSIONS

8.1 Overall Summary

This research project originated because de-watering of a sand and gravel quarry in the Hoveringham area of Nottinghamshire was identified as one possible cause of apparent ground and structure movements.

In Chapter One the possible adverse effects of de-watering were discussed. Some areas of uncertainty surrounding industrial de-watering activity were also introduced. The lack of information on de-watering in sand and gravel aquifers and its related effects on groundwater disturbance and related ground settlement was identified as a source of concern.

The aim of the literature search summarised in Chapter Two was to investigate whether any research had been carried out into de-watering and related ground and settlement problems in shallow alluvial aquifers. Spatial variation of permeability was introduced as one phenomenon which can cause variation in the distances of influence and drawdowns around a de-watered zone. It was also possible that spatial variations of permeability could lead to larger effective stress increases and therefore settlements than would be calculated based on a homogeneous aquifer. The various geological features which can arise in a floodplain and lead to directional permeability trends were summarised. Suffosion was introduced as a possible mechanism which could have lead to particle movement due to the influence of laterally flowing water and therefore could have lead to ground settlements larger than those predicted by the usual drawdown/effective stress methods. Although Kenney & Lau had studied suffosion in the laboratory this mechanism had apparently not been considered previously as a settlement mechanism during de-watering of alluvial aquifer.

What was apparent was that neither spatial variation of permeability nor suffosion had been taken into account in detail during the planning stage or any post de-watering analysis of Hoveringham Quarry. It can therefore be seen that as well as having considered conventional mechanisms, much of the work discussed in Chapters Four to Seven is aimed at investigating the effects in the Hoveringham area of spatial variation of permeability and suffosion.

Chapter Four studied the distribution and type of defects existing in Hoveringham. A wide distribution of structures and defects were observed and Figure 4.1 summarises the results. Where settlement of a structure was observed but there was no readily apparent explanation, this was classified as "potentially at least partly attributable to de-watering".

A visual inspection of Figure 4.1 appeared to indicate that more defects possibly due to de-watering existed as one approached the de-watered zone. This was backed up statistically by calculating the average distance from the quarry of structures with and without possibly de-watering related defects. These average distances were 480m and 580m respectively. This piece of evidence was not by itself conclusive and needed to be backed up with more detailed, targeted research into the effects of the de-watering on the ground itself. Chapters Five, Six and Seven were an attempt to provide these more targeted studies.

Chapter Five used the Kenney and Lau method to examine the likelihood of suffosion having occurred due to laterally flowing water around the de-watered quarry. It was recognised that not only would widespread suffosion have been required to cause structurally damaging settlements but also large scale movement of particles under effective stress would have been required. In Chapter Five it was shown that 17 of 21 soil samples from the Hoveringham area were classified as "unstable" under the Kenney and Lau criteria (possessed of the potential to suffer suffosion). However, it was also stated that the seepage conditions in the aquifer generally were much less severe than the seepage conditions employed by Kenney and Lau in their laboratory tests. Particle size analyses of soil samples taken at the quarry face during de-watering showed an apparent lack of finer particles compared to samples from a similar location prior to de-watering. This suggested suffosion may have taken place at the quarry face despite the apparently mild seepage conditions. Sample contamination during sampling, sampling method, sample levels, sample locations and sample mass were all ruled out as a possible cause of the apparent lack of fine particles in the "post de-watering" set of samples. It was concluded that seepage of water through the samples was the most likely cause of the difference between the two sets of samples. What was needed now as a way of determining whether suffosion (and differential distances of influence) had occurred on a large scale around the de-watered quarry and lead to settlement. The geophysics and pump test discussed in Chapter Six attempted to research these problems.

The geophysics was intended as a low cost site investigation technique for detecting geological variability within the shallow floodplain aquifer in order to find the optimum site for the pump test. The pump test field was instrumented with standpipes and temporary bench marks to examine the drawdown and settlement response. The measured drawdowns during the test did not show sufficient variation to suggest that variable drawdown behaviour would occur in the aquifer as a whole during long-term quarry de-watering. Permeabilities calculated for standpipe lines radiating at different directions from the pumped source through regions of varying geophysical response were of the same order of magnitude. Even though it is thought that topsoil swelling during the test may have masked the true ground movements, no large scale settlements were recorded. This was an important finding as the pump test would undoubtedly have subjected that portion of the aquifer to higher hydraulic gradients than existed during de-watering of the quarry. The results therefore showed that no large scale settlement due to suffosion or effective stress increase occurred during the test. This does not of course rule out the possibility that suffosion occurred but did not induce settlements.

Chapter Seven presents the results from numerical modelling techniques used to investigate various de-watering scenarios around Hoveringham Quarry. Spatial permeability variation was investigated using a quasi-three dimensional approach and permeability variation with depth was studied using an axi-symmetric technique.

Important conclusions from the quasi-three dimensional investigations were:

- The investigations suggested the potential for larger drawdowns and distances of influence to exist in an aquifer of varying permeability compared to results based on a "bulk average" analysis.

- Even if larger drawdowns occurred than may have been derived in a planning application or pre-extraction analysis, these would be unlikely to cause structurally damaging effective stress increases within an aquifer such as the Trent Floodplain.

- Fine particle movement (suffosion) was highly unlikely to have occurred beneath Hoveringham village given the hydraulic gradients and unit fluxes that existed.

- Large P.F.A. infilled areas existing adjacent to a de-watered zone would be likely to reduce the effects of de-watering to negligible beyond the filled areas.

The pump test results had already suggested that suffosion had not occurred on a wide scale but the axi-symmetric investigations were aimed at attempting to quantify the unit fluxes which existed during de-watering under various permeability/depth scenarios. These unit fluxes were compared to Kenney and Lau values for which suffosion was first observed. The results from the analyses suggested that suffosion had not occurred other than very close to the quarry face.

8.2 Final Conclusions

The evidence presented in this thesis suggests that structurally damaging settlements within the village of Hoveringham did not occur due to the de-watering of Hoveringham quarry either from effective stress increase or suffosion. However, it was shown that greater disruption to the surrounding groundwater may have occurred than was initially anticipated by the quarry planning application. Therefore, with reference to the factors discussed in Section 1.1, it is concluded that a similar de-watering operation may have greater impact than initially foreseen on the following:

- Water levels in wells.
- Groundwater supplies to vegetation or crops.
- The integrity of archaeological artefacts.

8.3 Recommendations

It is recommended that more account be taken of aquifer permeability variations in any initial planning applications for a de-watering operation and the consequences of a "bulk average" analysis examined.

A more "observational" approach to de-watering projects would undoubtedly be useful in helping to avoid disputes similar to that which arose at Hoveringham given the current climate of uncertainty surrounding de-watering effects. This could mean continuous monitoring of settlement markers and standpipes surrounding the de-watered zone.

<u>REFERENCES</u>

Attewell, P.B. & Taylor, R.K. (1984). Ground movements and their effects on structures. *Surrey University Press, Surrey.*

Auton, C.A. (1992). The utility of conducting surveying and resistivity sounding in evaluating sand and gravel deposits and mapping drift sequences in north-east Scotland. *Engineering Geology, 32, pp.11-28.*

Boggs, J.M. (1990). Field study of macro-dispersion in a heterogeneous aquifer. 2. Observations of hydraulic conductivity variability. *Proceedings of the International Conference Workshop on Transport and Mass Exchange Processes in Sand and Gravel Aquifers, Part 1 - Field Study Of Macrodispersion.*

Bouwer, H. (1978). Groundwater Hydrology., *McGraw-Hill, Tokyo, Kogakusha, London*

Reineck, H.E., and Singh, I.B. (1973). Depositional Sedimentary Environments. *Springer-Verlag, Berlin / Heidelberg / New York. ISBN 3-540-06115-0. 439pp.*

Building Research Establishment (1980). Digest No. 63 - Soils and Foundations : 1. *7pp*

Building Research Establishment (1980). Digest No. 64 - Soils and Foundations : 2. *8pp*

Building Research Establishment (1980). Digest No. 67 - Soils and Foundations : 3. *6pp*

Building Research Establishment (1980). Digest No. 240- Low rise buildings on shrinkable clay soils, Part 1. *6pp*

Building Research Establishment (1990). Digest No. 241- Low rise buildings on shrinkable clay soils, Part 2. *4pp*

Building Research Establishment (1980). Digest No. 242 - Low rise buildings on shrinkable clay soils, Part 3. *4pp*

Building Research Establishment (1990). Digest No. 251 - Assessment of damage in low rise buildings. *8pp*

Building Research Establishment (1990). Digest No. 274 - Fill Part 1 : Classification & load carrying characteristics. *4pp*

Building Research Establishment (1983). Digest No. 275 - Fill Part 2 :Site Investigation, Ground Improvement and Foundation Design. *8pp*

Building Research Establishment (1983). Digest No. 276 - Hardcore. *4pp*

Building Research Establishment (1987). Digest No. 298 - The influence of trees on house foundations in clay soils. *8pp*

Building Research Establishment (1985). Digest No. 299 - Dry Rot - its recognition and control. *8pp*

Building Research Establishment (1986). Digest No. 313 Mini-piling for low rise buildings. *8pp*

Building Research Establishment (1987). Digest No. 318 - Site investigation for low rise buildings : desk studies. *8pp*

Building Research Establishment (1987). Digest No. 322 - Site investigation for low rise buildings : procurement. *8pp*

Building Research Establishment (1989). Digest No.343 - Simple measuring and monitoring of movement in low rise buildings, Part 1-Cracks. *8pp*

Building Research Establishment (1989). Digest No. 344 - Simple measuring and monitoring of movement in low rise buildings, Part 2 - Settlement, heave, and out of plumb. *4pp*

Building Research Establishment (1989). Digest No. 348 - Site investigation for low rise buildings : the walkover survey. *8pp*

Building Research Establishment (1990). Digest No. 352 - Underpinning. *6pp*

Bromhead, E.N. (1981). "WTABLE" and "FLONET" finite element computer programs user manuals. *40pp*

British Standard 1377 (1990). Methods of Tests for Soils for Civil Engineering Purposes - Part 2. Classification Tests. British Standards Institution, London.

British Standard 6316 (1983). Code of Practice for Test Pumping of Water Wells. British Standards Institution, London.

British Standard 8004 (1986). Code of Practice for Foundations. British Standards Institution, London.

Bromhead, E.N., et al. (1985). Groundwater modelling by micro-computer - applications to dam and reservoir slope stability. *Groundwater in Engineering Geology - 21st Regional Conference of the Engineering Group of the Geological Society. pp.323-335*

Burgess, P.J. (1983). In-situ permeability testing in soil and rock. *From: In-situ Testing for Geotechnical Investigations, Meehan et.al (eds). pp.111-124.AA Balkema, Rotterdam.*

Burland, J.B. and Burbridge, M.C. (1985). Settlement of foundations on sand and gravel. *Proceedings of the Institution of Civil Engineers, Part 1, 1985, 78, Dec., 1325-1381.*

Chapman, M.C. (1994). Structural Appraisal of Low Rise Properties. *Unpublished final year thesis, The Nottingham Trent University, Department of Civil and Structural Engineering. 98pp*

Chester, D.K. (1982). Predicting the quality of sand and gravel deposits in areas of fluvioglacial deposition. *University of Liverpool, Dept of Geography, Research Paper No.10. Liverpool University Press.*

Clark, L.J. (1988). The field guide to water wells and boreholes. *Geological Society Handbook 1st Edition, Open University Press, Milton Keynes / Halsted Press, New York / Toronto.180pp*

Craig, R.F. (1987). Soil Mechanics. *4th Edition. Van Nostrand Rheinhold (International) Co.Ltd, London.410pp.*

Department of the Environment (1988). Low Level Restoration Of Sand And Gravel Workings Restored To Agriculture With Permanent Pumping - A review of the existing data and experience on the legal and technical aspects. *prepared for the Department of the Environment by M.J.Carter and Associates. HMSO London.*

Driscoll, F.G. (1989). Groundwater and Wells. *(2nd Edition) Johnson Filtration Systems Inc, St.Paul, Minnesota 55112. 1089pp.*

Edge, M.J. & Sills, G.C. (1989). The development of layered sediment beds in the laboratory as an illustration of possible field processes. *Quarterly Journal of Engineering Geology, London, Vol.22, No.4, pp.271-279.*

Hamill, L. and Bell, F.G. (1986). Groundwater Resource Development. Butterworths, London. *344pp.*

Head, K.H. (1982). Manual of Soil Laboratory Testing Volumes 1 & 2 - *Pentech Press, London / Plymouth. 748pp.*

Hoveringham Parish Council. (1993) - Submission To Nottinghamshire County Council. *Prepared By Wimpey Environmental Ltd (November 1993)*

Hvorslev, M.J. (1951). Time lag and soil permeability in groundwater observations. *Bulletin No.36, Waterways Experimental Station, U.S.Corps of Engineers, Vicksburg, Mississippi.*

Johnson, W.E. & Hughes, R.V. (1948). Directional permeability measurements and their significance. *Proceedings of the Twelfth Annual Secondary Recovery Conference, Pennsylvania State College, Pa., Oct 14-16, 1948. pp.17-25.*

Jones, S.P. (1994). Modelling Groundwater Protection Zones In The East Yorkshire Chalk Aquifer. *Unpublished M.Sc. thesis, Department of Earth Sciences, University of Leeds.*

Kenney, T.C. and Lau, D. (1984). Stability of particle grading of compacted granular filters. *Department of Civil Engineering, University of Toronto, Toronto, Ontario., Internal Technical Report 84-06, 48p.*

Kenney, T.C. and Lau, D. (1985). Internal stability of granular filters. *Canadian Geotechnical Journal, Vol 22, part2, pp.215-225.*

Kenney, T.C. and Lau, D. (1986). Internal stability of granular filters-reply. *Canadian Geotechnical Journal, Vol 23, part2 pp.420-423.*

Kenney, T.C., Lau, D., and Clute, G. (1984). Filter tests on 235mm diameter specimens of granular materials. *Department of Civil Engineering, University of Toronto, Toronto, Ontario.,Internal Technical Report 84-07, 105p.*

Kenney *et al.* (1985). Controlling constriction sizes of granular filters. *Canadian Geotechnical Journal, Vol 22, part1, pp.32-43.*

King, G.J.W., & Chanresakaran, V.S. (1974). An assessment of the effects of interaction between a structure and its foundation. *from the Proceedings of the Symposium on Settlement of Structures, organised by the British Geotechnical Society, The Lady Mitchell Hall, Cambridge, April 1974. pp368-383. Pentech Press, London.*

Krusemann, G.P. and De-Ridder, N.A. (1983). Analysis and evaluation of pumping test data, 3rd Edition. *Bulletin No.11 International Institute for Land Reclamation and Improvement, Wagenigne, Wageningen, The Netherlands, 200pp.*

Leonard Murray and Associates, Chartered Civil Engineers. (1993). "Structural Movement in Residential Property" - One day seminar for professional structural and building surveyors. *Unpublished course paper, 42pp*

Lin, P.S., Lin, M.H., & Wang, W.J. (1991). A theoretical prediction of land subsidence due to pumping. *In: Neotectonics and Resources (1992) Edited by Mervin Jones and John Cosgrove. Belhaven Press.*

Littlejohn, G.S. (1974). Observations on brick walls subjected to mining subsidence. *from the Proceedings of the Symposium on Settlement of Structures, organised by the British Geotechnical Society, The Lady Mitchell Hall, Cambridge, April 1974. pp384-393. Pentech Press, London.*

Loudon, A.G. (1952). The computation of permeability from simple soil tests. *Geotechnique Vol.3, No.4, pp165-183.*

MacLeod, I.A. (1991). Required knowledge for finite element software use. *From : Finite Element Analysis, Chapter 18, pp.236-241. Belhaven, London.*

Mansur, C.I. & Kaufman, R.I. (1962). Foundation Engineering - De-watering. *Leonards G.A. (editor). McGraw-Hill Book Company, New York. 290pp.*

Marsal, R.J. (1959). Foundation Problems in Mexico City. *Proceedings 1st Panamerican Conference on Soil Mechanics and Foundation Engineering, pp.1071-1094.*

Meyerhof, G.G. (1947). The settlement analysis of building frames. *The Structural Engineer. Vol.25, pp.369-409.*

Mishra, S.K., Singh, R.P., & Chandra, S. (1993). Prediction of Subsidence in the Indo-gangetic basin carried by groundwater withdrawal. *Engineering Geology,33 (1993), pp.227-239. Elsevier Science Publishers B.V., Amsterdam.*

Mishra et.al. (1993). Design of a constant head well permeameter for shallow, saturated, high hydraulic conductivity soils. *Engineering Geology,34 (1993), pp.117-129. Elsevier Science Publishers B.V., Amsterdam.*

de Mulder, E.F.J. *et.al.* (1994). Environmental impact assessment on land reclamation projects in The Netherlands: A case history. *Engineering Geology. Vol No.37, pp15-23.*

Murray, E.J. (1995). Prediction of Permeability of Granular Materials. *Proceedings of the Fourth International UNBAR4 Symposium,"Unbound Aggregates in Roads" , July 17-19, held at The University of Nottingham. edited by Andrew Dawson and Ron Jones. Published by The Dept of Civil Engineering, The University of Nottingham. pp.61-74 .*

Nottinghamshire County Council (1992). Nottinghamshire Minerals Local Plan - Consultative Draft". *68pp*

Page, R.C.J. (1984). Inspection and Analysis of Structural Defects in Low Rise Buildings. *Proceedings of the Symposium on Structural Defects in Low Rise Buildings, Trent Polytechnic. pp5-20.*

Page, R.C.J. (1992) Structural Survey of Properties in Hoveringham. *Unpublished Structural survey summary report from The Nottingham Trent University. 2pp*

Page, R.C.J. (1992). The diagnosis and prognosis of structural defects in traditionally built residential buildings. *Proceedings CIB W70 Symposium, Rotterdam. Volume 5, Section 7.2.2.*

Page, R.C.J. (1993) Structural Survey of Properties in Hoveringham. *Unpublished Structural survey summary report from The Nottingham Trent University. 2pp*

Peck, A. *et.al.* (1988). Consequences of spatial variability in aquifer properties and data limitations for groundwater modelling practice. *IAHS Publication No.175 - A Contribution to the International Hydrological Programme of UNESCO (project 2.4d).*

Petts, G.E., Moller, H, & Roux, A.L. (1989). Historical change of large alluvial rivers in western europe. *Wiley Publishers, London.*

Poland, J.F. (1981). Subsidence in the United States due to Groundwater Withdrawal. *Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers. Vol. 107, No. IR2, June, 1981. pp276-294*

Poland, J.F(ed). (1984). Guidebook to studies of land subsidence due to groundwater withdrawal. *Prepared for the International Hydrological Programme Working Group 8.4, Unesco Studies and Reports in Hydrology, 40.*

Powers, J.P. (1981). Construction De-watering. *John Wiley & Sons, New York. pp484.*

Powrie, W. (1994). Ground movements due to construction de-watering. *In: Groundwater Problems in Urban Areas.* Thomas Telford, London.

Preene, M. & Roberts, T.O.L. (1993) The application of pumping tests to the design of construction dewatering systems. *In: Groundwater Problems in Urban Areas.* Thomas Telford, London.

Pryke, J.F.S. (1974). Differential foundation movement of domestic buildings in south-east England. Distribution, investigation, causes and remedies. *from the Proceedings of the Symposium on Settlement of Structures, organised by the British Geotechnical Society, The Lady Mitchell Hall, Cambridge, April 1974. pp.403-419. Pentech Press, London.*

Roberts, T.O.L. (1988). Seepage in Shallow Unconfined Aquifers. Permeability limits for gravity drainage. *PhD Thesis at University of London, Kings College (unpublished). 185pp.*

Sherard, J.L., Dunnigan, L.P., & Talbot, J.R. (1984). Basic properties of sand and gravel filters. *Jl Geotech Eng Div ASCE 110 No GT6 and 111No GT12 1467-1472.*

Skempton, A.W., & Brogan, J.M. (1994). Experiments on piping in sandy gravels. *Geotechnique Vol.44, No.3, pp.449-460.*

Sladen, P.J. & Bitcheno, P.J. (1988). Quarry De-watering - Testing, monitoring and modelling. *Technical paper published by Knight Piesold and Partners, Ashford, Kent.*

Somerville, S.H. (1986). Control of groundwater for temporary works. *Report 113 of the Construction Industry Research and Information Association. 88pp.*

Stephens, D.B. (1992). Application of the borehole permeameter. *Chapter Four in:Advances in Measurements of Soil Physical Properties: Bringing Theory into Practice. SSSA Special Publication No.30. Soil Science Society of America, 677 S.Segoe Rd., Madison, WI 53771, U.S.A.*

Tarmac Quarry Products Ltd. (1993). A Hydrogeological Investigation Of The Area Surrounding Hoveringham Sand And Gravel Quarry. *Unpublished Ground Investigation and Desk Study Summary Report.*

Terzaghi, K. (1939). Soil Mechanics: a new chapter in engineering science. *Journal of the Institution of Civil Engineers, Vol.12, pp.106-141.*

Thames Water International, Environmental Section, Groundwater Development Course 1991 by Scott Wilson Kirkpatrick Consulting Engineers. *Case Study One (1979) - Lower Colne Valley (Thames Water Authority).*

Vreedenburgh, C.G.F. (1936). On the steady flow of water percolating through soils with homogeneous-anisotropic permeability. *Proceedings of the 1st International Conference SMFE, Cambridge, Massachusetts, Volume 1.*

White, J.R. (1992). Structural defects in low rise buildings. *Unpublished final year thesis, The Nottingham Trent University, Department of Civil and Structural Engineering.94pp*

Wimpey Environmental. (1992). A Hydrogeological Desk Study of a Proposed Extension to Hoveringham Sand and Gravel Quarry, Nottinghamshire. *Unpublished report prepared for "Hoveringham Against Local Tipping" action group.*

Wintersgill, G.C. (1993). Stuctural Defects in Buildings. *Unpublished final year thesis, The Nottingham Trent University, Department of Civil and Structural Engineering.98pp.*

Zalasiewicz, J.A., Mathers, S.J., & Cornwell, J.D. (1985). The application of ground conductivity measurements to geological mapping. *Quarterly Journal of Engineering Geology, London, 1985, Vol 18, No.2. pp.139-148.*

User-oriented and Cost Effective Management, Maintenance and Modernization of Building Facilities

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Assessing the Long-Term Structural Integrity of Residential Buildings

R C J Page, K Roper and I J Froggatt

Abstract

Since the beginning of this decade, the cost of insurance claims in the United Kingdom associated with structural damage caused by subsidence has averaged over £300 million each year. It is anticipated that this figure will rise dramatically over the next few years when the full effect of the driest summer for 350 years is realised. This claims profile has inevitably led to an increase in the cost of insurance premiums.

At present, insurance premiums are rated using part of the postcode for an area from information provided by the British Geological Survey, known as The Geo-Hazard Susceptibility Package (GHASP). However, this system of rating only typically reduces a region to about 2000 dwellings. Such a region is still a large geographical area, hence the risk of structural damage occurring to a building may vary considerably within the region.

When considering property protection insurance, large sums of money are involved and claims can be costly to insurance companies. Therefore, any reduction in the magnitude of the insured risk should reduce the cost of insurance against subsidence damage.

This paper describes the proposed development of an expert system for two particularly high risk areas of the East Midlands region based on the experience of the authors in carrying out an in depth study of subsidence damage in a rural village in Nottinghamshire. It is believed that such a system will enable an accurate assessment of the risk of structural defects occurring in the life of a particular building in a specific location to be made. Such a system should therefore prove to be invaluable to both insurers and investors and should, in addition, facilitate the investigation of any defects occurring.

Origins of the Expert System

In 1992, the authors were asked to investigate a number of structural defects which, it was reported, had all occurred recently in a small rural village situated close to the River Trent in Nottinghamshire. Extensive quarrying work to extract sand and gravel had been carried out in the area for the previous fifty years but the present workings were situated close to the village and involved the water table being lowered by as much as 10 metres.

The geology of the village was relatively uniform and simple with the aquifer (Sand the Gravel) overlying, in a relatively horizontal plane, the impermeable Mercia Mudstone. The Sand and Gravel Beds varied in thickness generally between 6 and 10 metres. Alluvium up to a maximum depth of 2.5 metres lay over the Sand and Gravel Beds over part of the site of the village and this included some clay.

There was obvious evidence of recent ground movement having occurred within the area of the village and it was suspected that this was settlement resulting from the draw down of the natural ground water table caused by the quarry dewatering operation and by changes to the local hydrological regime brought about by the re-routing of the existing surface water drainage system and the quarry pumped drainage.

The resulting lateral flow of water through the sand and gravel aquifer would have been accompanied by the migration of fines with resulting settlement of the ground. It was considered probable that the loss of fine material would occur if there was lenticular zonal distribution of almost single-sized component materials forming the deposit, which is not uncommon in river terrace ground deposits. It was considered possible that some highly permeable paths could exist through which migration of fines could occur.

Our initial investigations confirmed that there was firm evidence of recent ground settlement in the area and the structural damage observed in several properties appeared to be consistent with such ground movement.

In order to investigate this theory, a thorough investigation was carried out. This involved a

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very detailed investigation of the geology of the area, an investigation of the variations in the grading of the sand and gravel deposits and structural surveys of as many buildings as was possible.

A detailed geological map of the area was drawn up using information from the relatively large number of boreholes that had been sunk as part of the original investigations carried out by the quarrying company, supplemented by our own trial holes and boreholes. Additional information was obtained from a local builder who had kept detailed records of the ground when he had excavated for foundations at numerous locations throughout the village.

Samples of the sand and gravel were analysed to identify any areas of potentially high permeability.

Letters were sent to the owners of all the 130 properties in the village explaining the reasons for the investigation and offering a free structural survey. This produced an excellent response and 70% of the village properties were eventually surveyed.

All the information obtained was stored on a database together with details of any other factors which may have affected the structural integrity of any property, such as the position and size of trees, defective sewerage systems or local areas of filled ground. From this information it was possible to assess the likelihood of any property having been affected by the quarrying operations and this facilitated the analysis of the properties from the structural survey information.

The Expert System

Our experience in investigating the structural defects of the buildings of Hoveringham led us to the realisation that what we had developed was an expert system for the area from which we were able to make an accurate assessment of the risk of structural defects occurring in the life of a particular building in a specific location. In addition, the system facilitated the investigation of the structural defects occurring in any building within the area.

If such a system was to be developed nationally, for areas where there was a greater than normal risk of subsidence occurring, properties could be individually rated for insurance purposes as opposed to being grouped together over a larger area. Furthermore, it would provide purchasers and insurers with an accurate assessment of the future threat of subsidence damage occurring.

We decided, therefore, to develop a similar system for a small 'high risk' area of Nottinghamshire, and devise a risk assessment model. It was thought that by undertaking a detailed analysis of a small area the effectiveness of the model may be tested and its capability of being used on a larger scale assessed.

The area chosen for this initial study was in West Bridgford, a suburb of Nottingham lying to the south-east of the city. West Bridgford is founded on Weathered Mercia Mudstone, with pockets of Sand and River Gravels and occasional lenses of peat. Cases of subsidence damage are widespread and the authors already possess extensive knowledge of the area. An area covering approximately 200 buildings, divided into two separate areas, was selected to take advantage of the variety of different house types and construction periods.

Methodology

Specifically designed large-scale maps showing the known ground conditions for the risk assessment area are being prepared in conjunction with British Geological Survey. Borehole data from British Geological Survey and from Local Authority records, consulting engineers, contractors, and from our own previous investigations has been recorded and this is being supplemented with further site borehole tests.

Further data is being obtained from the Local Authority, both from details of remedial works carried out on properties and from the local knowledge of the Building Control Officers, and from three local contractors specialising in structural repair work.

It is hoped that insurance companies will co-operate by releasing details of subsidence claims carried out within the area. Additional information is being obtained from surveys of individual properties. Based on the information already obtained, letters are being sent to the owners of selected properties seeking information of any remedial works carried out on their property with respect to subsidence damage. Where no remedial works have been undertaken a free visual structural survey is being offered in return for the use of the information gathered. The response to date has been similar to that of the survey carried out in Hoveringham.

Sewer plans have been obtained from the Local Authority and private sewers and the positions of drains and soakaways have been plotted on these. Any known information regarding repaired or defective lengths of sewer is being recorded.

The positions of and size of all trees or vegetation likely to affect the structural integrity of buildings or the integrity of the sewerage system are being recorded.

A computer database has been created to store the information gathered. The 'Paradox for Windows' package has been chosen for its ability to sort data into categories which will aid in providing a risk assessment model for individual properties which will assess the risk of subsidence in relation to the details inputted. From an analysis of the model an appraisal of the risk from subsidence that this particular area of West Bridgford may encounter will be developed. The model will be tested by inputting information from 50 case studies previously selected from 200 case studies being acquired from the Subsidence Claims Advisory Bureau to test their ability to incorporate the risk assessment model and be compatible with current policies used by insurance companies. This information will then be used to determine whether there is a potential for the national adoption of the risk assessment model.

Requirements for Reliability Evaluation

Evaluation of risk and reliability in ground engineering is a multi-dimensional task. Often the assessment of risk and reliability involves a number of components or elements which together form a system. Each system is likely to have several modes of failure.

Methods for the estimation of system reliability have been developed by structural engineers for several decades. However, relatively little has been achieved in the field of geotechnical engineering.

The risk engineering process has been identified as follows (Godfrey, 1993):

- identification of hazards;
- understanding the causes and sources of hazard;
- assessing the consequences which might arise as a result of the hazard occurring;
- assessing the probability of the hazard occurring;
- developing precautions to minimise the risk or mitigate the consequences; and
- assessing residual risk and its tolerability.

The principles sources of hazards in geotechnical engineering have been described under the following headings (Boyd, 1993):

- geological uncertainty;
- limitations of methods of analysis; and
- lack of awareness.

It must be recognised that even at the current state of the art, there may well be hazards for which the causes and mechanisms are still not understood and the black hole where we are not aware of our lack of knowledge must be eliminated.

We believe that we have identified the vast majority of the hazards that can cause subsidence damage to buildings and have documented the causes and sources of each hazard and the consequences of it occurring, (Page and Murray, 1996). We further believe that with the information stored in our database we will be able to assess the probability of a hazard occurring and will be able to advise on the precautions that should be taken to minimise that risk, or mitigate the consequences, and accurately assess the seriousness of the residual risk remaining.

Conclusions

The expert system we have devised and are now developing fulfils, so we believe, the requirements for reliability evaluation. We believe that such a system will prove invaluable to both insurers and investors and will, in addition, facilitate the investigation of any structural defects in properties within the area covered by the system.

The predicted merits of such a risk register, for areas where there is a greater than normal risk of subsidence occurring, have already been discussed. However, such a system may produce new problems in that owners of properties found as being susceptible to subsidence damage or those that have already suffered subsidence damage may find it difficult to obtain insurance cover. If risk assessment schemes are implemented, new insurance policies will have to be devised, or existing policies amended, to prevent discrimination against owners whose properties are assessed as having a high risk of subsidence damage.

Research has shown that a saving of around £50 million could be made each year if a proper programme of preventative maintenance were to be implemented by homeowners, (Page, 1996). The expert system we have described would identify the essential maintenance each homeowner should carry out.

Finally, there is the question that if such systems are to be developed, who should finance their development. The obvious answer is probably the insurers, but it may well be that local authorities, British Geological Survey and possibly the Building Research Establishment should have important parts to play. To date, most interest has been shown by Consultants who see the possibility of commercial exploitation of such systems.

We would estimate that such a system would, once fully developed, cost in the region of £30 per property. We believe its value to insurers and investors would be considerably greater than this figure.

References

Godfrey, P. 'The holistic approach to risk engineering in the construction industry', 1993.

Boyd, RD. 'Application of an engineering system for the control of risk in ground engineering', Thomas Telford, London, 1993.

Page, RCJ and Murray, PL. 'Analysis of structural defects occurring in residential buildings in the East Midlands region', Structural Survey, 1996. Awaiting Publication.

Page, RCJ. 'Long term structural stability of buildings - where maintenance can be cost-effective', CIBW70 Helsinki 1996 Symposium, 1996.

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APPENDIX 1.1

STRUCTURAL REPORT ON HOVERINGHAM PROPERTIES
DATED 28th JULY 1992

28 July 1992

The Nottingham Trent University
Faculty of Environmental Studies
Department of Civil and Structural Engineering
Head. professor Roger Hawkins
MSc PhD CEng FICE FIHT

F.a.o
Mr. Mike Butler
Chairman H.A.L.T
'Rosebank'
Boat Lane
Hoveringham
Nottinghamshire

Dear Mr Butler,

STRUCTURAL SURVEY OF PROPERTIES IN HOVERINGHAM

I refer to your letter of 16 June and would report that I have now surveyed nine properties in Hoveringham which have suffered structural damage and, although my investigations are far from complete, I understand that an interim report of my findings to date would be useful.

Our geological survey of the area has shown that there is generally a layer of silty clay (Trent Alluvium), varying in thickness between one and two metres overlying the gravel beds which in turn vary in thickness between six and nine metres. The original water table level in the area was close to the gravel floor at a depth of eight to nine metres below ground level but there is considerable evidence that significant alterations to the level of the water table have occurred in recent years.

The evidence available, would suggest that the underlying cause of these changes in the water table levels is not necessarily associated with the semi-drought conditions that have existed in the area for the past three years. The extensive sand and gravel extraction that has been taking place in recent years has necessitated substantial alterations to the natural drainage network. In some workings, where the sand and gravel has been extracted below the natural level of the water table extensive pumping has been carried out with consequent localised lowering of the water table level.

As you are aware, I intend to excavate trial holes and sink some boreholes in the near future to further my investigation and, in certain cases, verify evidence that has been supplied by the occupants of the houses as to the depth of foundations and the thickness of the clay layer. This information will, I hope, verify some of the conclusions I have drawn to date and establish the cause of some movements that are not, as yet, obvious.

All nine properties I have surveyed to date have suffered structural damage due to ground subsidence. The extent of the damage varies considerably from very slight to severe.

In one property, the damage appears to have been caused by a faulty sewerage system but the remaining eight properties exhibit damage which could be consistent with either clay shrinkage or the effect of variations in the level of the water table or by a combination of the two. In one property the structural movement that has occurred has been accentuated by partial cellar construction and the fact that the rear of the property was constructed on sloping ground. In six properties there has been movement of an extension relative to the original building and in most cases the majority of the damage has resulted from poor construction rather than substantial movement.

I believe that four of the properties surveyed are founded in the clay layer and that the main part of the structural movement that has occurred is associated with clay shrinkage which cannot be related directly to changes in the level of the water table, although it is possible that there has been some structural movement from this cause in two of the houses.

There are, however, four buildings which I believe are founded in the gravel. This assumption is based on my own observations or on evidence supplied by the occupants based on work they have themselves carried out usually in excavating trenches for water or drainage pipes or for garden landscaping.

In the case of one property, Flora Farm, there is evidence of substantial ground movement in the rear garden and yard, the degree of subsidence increasing in the direction of the deep gravel pit which was recently excavated in the rear of the property. There was, in addition, evidence of substantial recent movement in the house and the occupants detailed recollection of the appearance of the cracks and the settlement of the land at the rear of the building was consistent with the variations in the level of the water-table which would have resulted from the gravel extraction. Certainly, it was reported that the cracks began appearing before the recent semi-drought which has been responsible for the settlement of so many houses which are founded on clay.

I had personally carried out a survey of 'Four Winds' in 1986 and since that time structural cracking has occurred. Since it is again reported that the building is founded on the sand and gravel and not in the clay layer the most likely mechanism for movement must be the changes in the level of the water table associated with the gravel workings which were carried out recently in the close proximity.

'The Willows' was one of the nearest properties to the recent workings and exhibited slight differential settlement in its rear elevation but the owner reported substantial settlement of the rear garden, the settlement again increasing in magnitude towards the workings.

The settlement of the southwest corner of the church, which I would assume is founded in the gravels, requires further investigation as there is no obvious mechanism for the movement other than that associated with variations in the level of the water table in the gravel beneath it.

When the water table is lowered by artificial means such as pumping or changing the drainage pattern of an area locally, differential settlement of the ground can occur due to the 'leeching out' of fines suspended in the water which will be moving laterally. Generally, this effect will decrease with distance from the source. The area affected will depend on the amounts of water being removed which in the case of recent workings appears to be immense.

Consequently, on the basis of my investigation to date, there would appear to be four properties which have suffered structural damage as a result of ground movement which it would appear has been caused by the artificial lowering of the water table in the gravel workings. This has resulted in a lateral flow of water through the gravel for a prolonged period and a reduction of the level of the water table over a large area around the workings. Such a large scale movement of water would be certain to remove find material from the gravel beds and result in subsidence of the ground above. There is also a possibility that the same mechanism has been responsible for a part of the structural damage apparent in some of the other properties surveyed.

Yours sincerely,

R C J Page
Principal Lecturer

APPENDIX 1.2

STRUCTURAL REPORT ON HOVERINGHAM PROPERTIES DATED 6th JANUARY 1993

6 January 1993

The Nottingham Trent University
Faculty of Environmental Studies
Department of Civil and Structural Engineering
Head. professor Roger Hawkins
MSc PhD CEng FICE FIHT

F.a.o
Mr. Mike Butler
Chairman H.A.L.T
'Rosebank'
Boat Lane
Hoveringham
Nottinghamshire

Dear Mr Butler,

STRUCTURAL SURVEY OF PROPERTIES IN HOVERINGHAM

As you are aware, we have been investigating structural defects in the buildings of Hoveringham village since late June 1992 and have now surveyed all the buildings listed by you as having suffered structural damage. We have, in addition, examined other buildings exhibiting evidence of structural movement. As a result of our investigations to date, we have identified other properties that we would like to look at and have approached the owners for permission to do so. We hope, eventually, to look at most of the properties in the village.

From the properties surveyed to date we have selected three for structural monitoring and have installed precise levelling stations and demec strain gauges and have commenced recording movements. We intend to extend our structural monitoring activities to include further selected properties as suitable buildings are identified.

A detailed investigation of the geology of the area has been carried out and we have supplemented information obtained from borehole records held by the Institute of Geological Sciences at Keyworth with our own boreholes. We have also obtained useful information from builders who have been carrying out construction works at various locations in the village. We shall, in due course, carry out further investigations.

In my letter of 28 July 1992 I stated that there was generally a layer of silty clay, varying in thickness between one and two metres overlying the gravel beds. We have now discovered that in parts of the village there is in fact no overlying clay, the gravel being encountered at a very shallow depth beneath a thin layer of top soil. Several of the buildings exhibiting substantial structural movement are therefore now known to be founded in the gravel and would therefore, not be affected by volume changes occurring in the clay.

From the information available to us at present, we believe that the majority of structural movement that has occurred is associated with one of two clauses: the changes of volume of the clay upon which some houses appear to have been built and the variations in the water table level in the gravel beds. The first cause is easily identified and is of little interest to yourself. The second cause, which appears to be the most common, since the majority of the structurally damaged buildings we have looked at to date appear to be founded in the gravels, requires greater consideration.

There are two possible causes for the observed structural movement associated with variations in the water table level in the Gravel Beds. Firstly, variations in water table levels both natural and artificially induced usually result in stresses being built up within the ground which can cause differential movement to occur. However, it is normal in such cases for those movements to be fairly constant over a given area and this does not normally result in substantial differential settlement of buildings. Differential settlement of buildings usually occurs due to secondary affects, the most common being the ground movement causing drains to fracture and the resulting leakage causes local differential settlement.

The second cause of differential settlement is the removal of fines from the gravel by lateral migration of the water. This can result in substantial local variations in movement as the gravel deposits usually contain lenses of ground with higher concentrations of fines.

Gravel has been extracted in this area for many years and recent workings have resulted in an artificial lowering of the water table to the west of the main village. The 'draw down' effect of this would undoubtedly have affected the water-table under the properties we have surveyed and could account for the recent structural damage that has occurred. It is obvious that some of the damage has been caused by drains which have fractured as a result of the settlement that has occurred. However, there is considerable evidence of structural movement having occurred which does not appear to have been caused solely by the removal of 'clean' water and this has resulted in our belief that damage is being caused by the migration of fines from the gravel by the lateral movement of water through the gravel strata caused by the artificial lowering of the water table.

Very little research has been carried out to date on the migration of fines by this type of mechanism and we are presently studying similar occurrences in other locations of the country. Obviously movement of fines will be greatest closest to the source of the artificial lowering of the water table as the hydraulic gradient, which will dictate the rate of lateral flow, will decrease with distance and the distance of the majority of the damaged properties is relatively great. However, the properties lie between the River Trent and the workings and the topping-up from the river may possibly have resulted in an increase in hydraulic gradient. Certainly, the analysis of effluent from the workings that have been taken by the National Rivers Authority reveals a substantial fines content and this does indicate that more than clear water is being removed from the area.

My reason for trying to distinguish between these two different mechanisms is that the existing law appears to exempt from liability those causing damage by the removal of 'clean' water from the ground whilst anyone causing damage by removing material other than 'clean' water is liable for any damage caused.

Obviously, it is too early for us to be able to be conclusive and it will be several years before we are able to establish the causal mechanism or mechanisms for the structural damage we have observed to some of the properties of Hoveringham village.

Certainly, there appears to be substantial evidence that damage to buildings has occurred due to variations in the water table levels. Although there has been a natural reduction in water table levels in the past three years due to a near-drought situation, the history of some of the damage we have surveyed would strongly suggest that it is related to the artificial lowering of the water table caused by the local gravel extraction works. We are, at present, unable to say how much of this damage has been caused by the extraction of 'clean' water from under the buildings and how much from the removal of fines in the resulting lateral flow of water. However, we would record that we have observed structural damage which we cannot associate with the removal of 'clean' water and can, at present, only explain its occurrence by the 'fines migration' mechanism and it is for this reason that we are given very serious consideration to this causal mechanism in our investigations.

Yours sincerely,

R C J Page
Principal Lecturer

APPENDIX 5.1

CALCULATION OF "D" (EFFECTIVE PARTICLE DIAMETER)

FOR SAMPLE 1, LOCATION 5.

APPENDIX 5.1

Calculation of "d" (effective particle diameter) for sample 1, location 5.

Size of particle (mm)	% smaller than size	Fraction in adjacent sieve sizes(%)(P_i)	Mean diameter of adjacent sieves(m)(d_i)	P_i /100 d_i (P_i as %)
50	98.7	1.3	0.0433	0.3
37.5	95.4	3.3	0.0324	1.02
28	95.4	2.1	0.0237	0.89
20	93.3	6.6	0.0167	3.95
14	86.7	7.8	0.0118	6.61
10	78.9	17.3	0.0079	21.90
6.3	61.6	6.0	0.0056	10.71
5	55.6	8.9	0.0041	21.71
3.35	46.7	8.2	0.0026	31.68
2	38.5	4.9	0.0015	31.54
1.18	33.6	6.5	0.0008	81.25
0.6	27.1	10.8	0.0005	216.0
0.425	16.3	9.0	3.8x10-4	236.84
0.3	7.3	3.6	2.5x10-4	144
0.212	3.7	1.3	1.8x10-4	72.22
0.15	2.4	1.4	9.7x10-5	144.33
0.063	1	-	-	

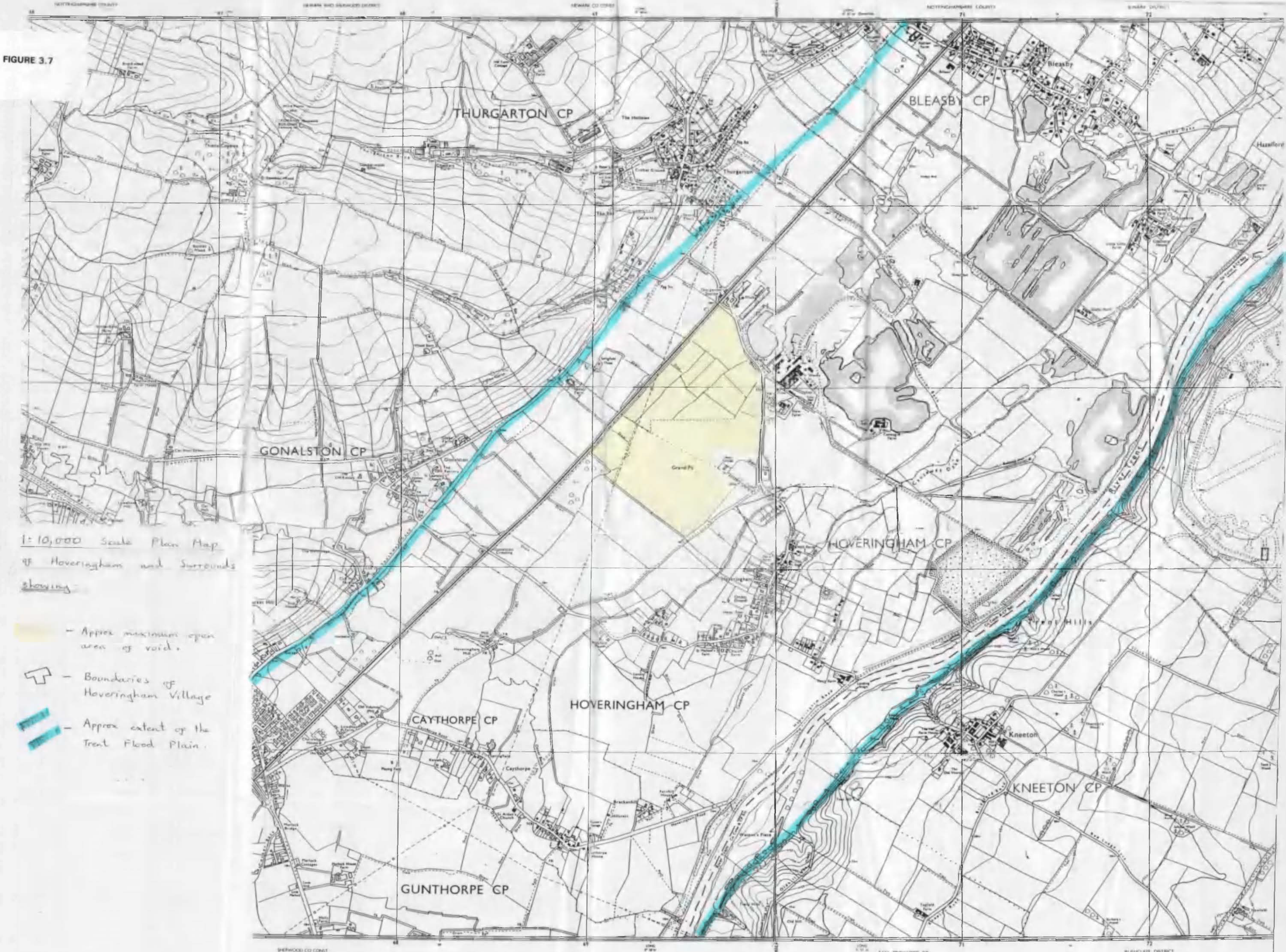
TOTAL = 1025

Effective Particle Diameter = 1/1025 = 0.98mm

Figure 4.1

Colour Coded Plan Map - Results of Structural Inspections

FIGURE 3.7



1:10,000 Scale Plan Map of Hoveringham and Surrounds showing:

- Approx maximum open area of void.
- Boundaries of Hoveringham Village
- Approx extent of the Trent Flood Plain.

FIGURE 6.4

MAP SHOWING REGIONS OF LAND INVESTIGATED USING GEOPHYSICS

LEGEND

Application area

Boreholes

APPENDIX 3.1

Tarmac
Quarry Products

Tarmac Roadstone Limited

HOVERINGHAM QUARRY

Geological Plan and Cross Sections

DRAWN BY	SCALE
R.B.S. /MLA/S.R.	1:2500
DATE	DRAWING NO