UNIVERSITY OF SOUTHAMPTON

THE USE OF HORIZONTAL WELLS FOR LEACHATE AND GAS CONTROL IN LANDFILLS

by

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ABSTRACT

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As the pore pressure in a landfill rises, there is an increased likelihood that leachate will be driven through the site liner and contaminate groundwater in surrounding formations. This dissertation examines the use of directionally drilled horizontal wells for leachate and gas control purposes.

The main findings of the research are

- an understanding of the practical difficulties of horizontal well installation, the theoretical reasons for them and ways in which they can be overcome
- an indication of the required characteristics of the well screen (material, strength, slot width and open area)
- an understanding of the way in which pumping from a horizontal well affects pore pressures in a landfill, which is heavily influenced by the anisotropic permeability of the waste
- the importance of measuring leachate heads/pore pressures using piezometers with a discrete response zone, as a result of the spatial variability of head within the landfill
- the desirability of extracting leachate from the lower horizons, if that is where leachate head/pore pressure control is required
- the importance of establishing the barometric influence on piezometer readings and fluid extraction rates
- the ability of horizontal wells to extract large volumes of landfill gas, even from the notionally saturated zone.

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TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX E 294

LIST OF TABLES

NOTATION

av	subscript indicating the average value of a parameter
BE	barometric efficiency
C_u	uniformity coefficient
D_{10}	largest particle size in smallest 10% etc. of particles by mass
D_{10f}	largest particle size in smallest 10% etc. of particles by mass in a filter material
d	well screen diameter
E'_0	one-dimensional stiffness modulus
F	friction
F	tensile load required to cause plastic deformation
fin	subscript denoting final
g	acceleration due to Earth's gravity (= 9.81m/s^2)
h	depth below leachate level
h	drawdown
h_{exit}	height of exit point above horizontal section of the well
h_{piezo}	height of water column in piezometer above horizontal section of the well
ini	subscript denoting initial
k	hydraulic conductivity
k_h	horizontal hydraulic conductivity
k_v	vertical hydraulic conductivity
N	normal load
N	well screen weight per unit length
P	pulling force

P	barometric pressure
P_{ult}	ultimate tensile strength of well screen
P_y	yield strength of well screen
Q	flow rate
Q_{G}	gas flow rate
Q_{L}	leachate flow rate
r	radius of sphere (gas bubble)
T	thickness of waste horizon
t	well screen wall thickness
t	time elapsed
u	pore pressure
V	velocity of flow across the well screen
V	volume
V_{s}	settling velocity (or rise velocity of bubble)
W	width of influence of well
w	moisture content (oven drying method)
w_{obs}	moisture content (observed)
z	depth below surface
α	coefficient with a value between 0 and 1
γ	unit weight
$\gamma_{leachate}$	unit weight of leachate
Δ	prefix denoting increment
μ	coefficient of friction
μ	dynamic viscosity of medium (leachate)
ρ	bulk density

ρ	density of well screen material
ρ	settlement
ρ_1	density of sphere (gas bubble)
ρ_2	density of medium
ρ_{dry}	dry density
$\rho_{leachate}$	density of leachate
$\rho_{reduced}$	density of leachate mixed with gas
ρ_{sat}	saturated density
σ_v	total vertical stress
σ'_v	vertical effective stress
φ	angle of interface friction

CHAPTER 1

Introduction

1.1 Statement of Problem

It is now recognised that the potential for a landfill to pollute the environment will remain for many decades after the tipping of waste has ceased. In order to surrender a site licence in the UK, a landfill operator must demonstrate that the condition of the land is such that it is unlikely to cause pollution of the environment or harm to human health.

One of the main pathways by which a landfill might pollute the environment is the contamination of surrounding land and groundwater by leachate leaking through the base and/or sides. To prevent such leakage, the Environment Agency will often stipulate that the leachate level within a landfill must be maintained below a certain elevation. Modern sites and new landfill cells are constructed with effective provisions for leachate control that may include geotextile liners and basal drainage layers. Traditional methods of leachate control at older sites generally rely on retro-fitted vertical wells, but may meet with limited success because yields are frequently very low [Powrie & Beaven, 1999].

As a result of this apparent ineffectiveness of vertical leachate extraction wells, alternative solutions are being sought. This research has addressed, as its main focus, the installation and performance of retro-fitted directionally drilled horizontal wells to control leachate (and gas) in landfills.

1.2 Horizontal Directional Drilling Overview

Horizontal directional drilling techniques were developed in the oil exploration industry, and have become commonplace in civil engineering since the early 1990's. The majority of projects using the technology are pipeline and cable installations. Examples of directional drilling rigs are shown in Figure 1.1.

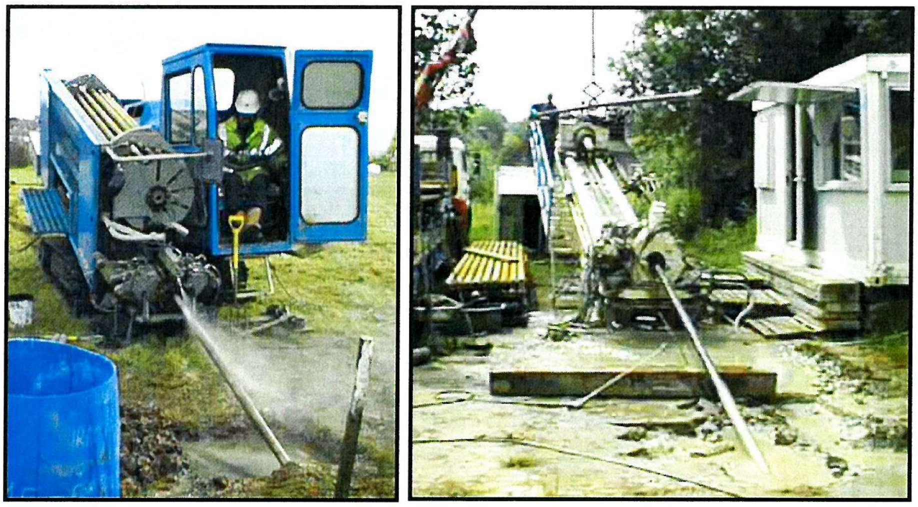

Figure 1.1 Directional Drilling Rigs; (a) 'mini-rig', (b) 'maxi-rig' [Metallic Tile 2000 & River Maas 1999]

Directional drilling is most commonly used in situations where conventional trenching is prevented by an obstruction such as a watercourse, or where it would cause disruption to surface activities such as traffic flow. Examples include;

1) The installation of a gas supply line below the River Nene and an adjacent railway in Lincolnshire. By using directional drilling the need for a pipe bridge over the river was avoided and rail services were able to continue uninterrupted during installation [Horizontal Drilling International, 1998].

2) The installation of a fibre optic telecommunications cable underneath the Canal Tessera in Venice. This replaced a cable previously laid on the canal bed which had been damaged by dredging operations [Gassler, 1999].

In the US and Western Europe, directional drilling is being used for an increasing number of environmental applications. Examples include;

1) A horizontal well was completed in a shallow sand and gravel aquifer to supply potable water to the city of Des Moines, Iowa, (Figure 1.2). As the aquifer was shallow, 47 vertical wells would have been necessary to expose an equivalent length of well screen to the aquifer [Maxwell, 1998].

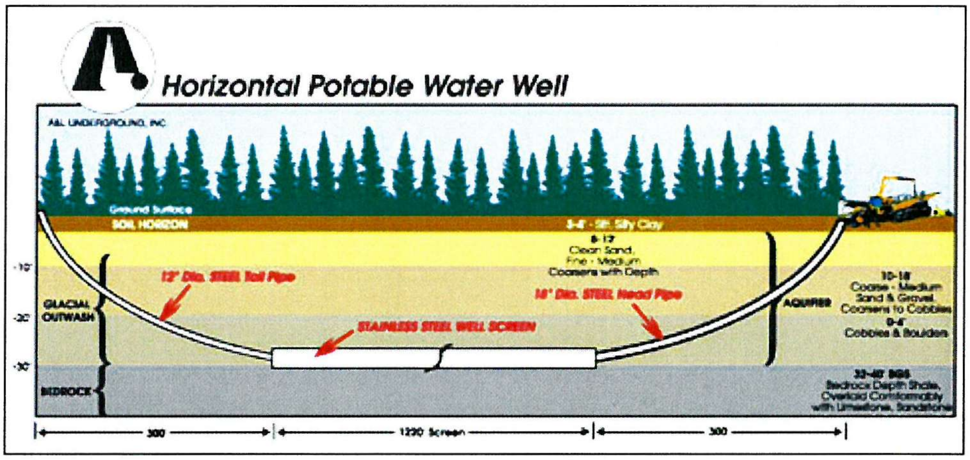

Figure 1.2 Schematic diagram of horizontal water well in a shallow sand and gravel aquifer, Des Moines, Iowa. [Source, A&L Underground Inc.]

2) The installation of horizontal wells for Air Sparging and SVE (Soil Vapour Extraction) to volatilise and remove spilt jet fuel at JFK Airport, New York. More than fifty wells were drilled under the runways, terminal buildings and apron areas of the airport whilst flights continued. Vertical wells were not chosen as they required both raised well heads and a greater number of installations to remediate the same area [Wilson, 1996a].

In the UK, implementation of the technology for environmental applications has been slow. The only significant project apart from this research took place in Autumn 1999 in Cardiff. Four horizontal wells were installed to control groundwater levels raised by the closure of the Cardiff Bay Barrage [Clarke, 2000]. This project is discussed further in section 2.3.

1.3 Objectives

Worldwide, a number of horizontal wells have been installed beneath or around landfill sites to remediate leachate plumes [Longbore, 1997; Petrowsky, 1999]. However, no reference could be found to wells installed within the body of a municipal solid waste landfill. Directional drilling has been successful in many types of ground conditions from hard rock to soft clay but the extreme heterogeneity of municipal solid waste (MSW) presents many uncertainties for the drilling process. For example, in one area the waste may consist of paper and slurry whilst in an adjacent area there may be concrete and steel construction debris (Figure 1.3). Another major difference between natural ground and MSW is that natural formations do not contain materials with tensile strength, whereas MSW contains, for example, steel rope and fabrics which may become wrapped up in the drilling tools. These uncertainties may be some of the reasons why directional drilling within waste had not been previously attempted. This research project was therefore initiated to determine whether horizontal wells could be installed within a landfill and to assess their effectiveness at leachate control. The research has focused on a series of trial well installations followed by a subsequent performance assessment based on results obtained from a large network of monitoring points.

Figure 1.3 Heterogeneity of waste, (a) slurry, (b) discarded washing machine, (c) concrete and steel [Rainham 1998 – 2000]

As the research continued, references were found to five previous projects in which it was attempted to install horizontal wells within the body of a landfill using directional drilling [Yach 1997, Bruxvoort et al 1998, Longbore 2001(a, b), Wolfe 2001]. Each of these projects met with varying degrees of success but none was purely research based. The results have not been widely disseminated and where they have been published they lack comprehensive analysis and so have been of limited use to waste management companies wishing to adopt the technology. An investigation of the advantages and disadvantages of the installation and operation of horizontal wells was required together with a thorough understanding of the mechanisms involved.

References to previous horizontal well installations are mostly publicity material written by the drilling contractors themselves and consequently lack discussion on how the wells performed once the drillers had left the site. For this project the network of instrumentation installed for monitoring the performance of the wells was extensive and the results and their analysis constitute a major part of this dissertation.

Conventional horizontal drainage systems requiring the excavation of a trench become less practical as the depth of waste becomes greater. Figure 1.4 shows a trench being installed where a large volume of waste has had to be removed to gain access to the lower horizons. The health and safety issues associated with excavating waste are considerable, indeed the operators in the photograph are wearing chemical splash suits, gas masks and restraining harnesses. Installation of horizontal wells using directional drilling reduces the hazards faced by operators.

Figure 1.4 Excavation of trench in waste for installation of drainage system [Rainham Phase 1, Jan 2001].

Until recently landfills have often been conceived as receptors for the dry-tomb encapsulation of waste. The emphasis is now shifting toward the concept of a *flushing bio-reactor* where, by repeated flushing of water through the waste, the rate of decomposition may be increased to achieve stability within one generation [Powrie & Beaven 1999, Powrie & Robinson 2000]. This obviously relies on effective mechanisms for the drainage and recirculation of leachate. If horizontal wells are to play a role in this, a sound understanding of their hydrogeological and geotechnical interactions with the waste is essential.

1.4 Outline of Dissertation

<u>Chapter 2: Background Information</u>

The reasons why horizontal wells were thought to represent a more effective solution for leachate control than vertical wells are discussed. A detailed description of the directional drilling technique is presented and then illustrated using the Cardiff Bay horizontal wells as a case study.

<u>Chapter 3: Site Investigation</u>

A brief description and history of Rainham Landfill site is given, justifying its selection as the site for the field trials. The results of the extensive desk study and invasive site investigation are presented. The classification of waste recovered from landfill is also discussed.

<u>Chapter 4: Well Installation Field trials</u>

The field trials are discussed in detail, indicating how the well design and installation techniques were refined after each trial. The main findings of the trials, together with a series of recommendations for future trials, are presented at the end of the chapter.

Chapter 5: Monitoring Instrumentation

Before the Rainham wells were brought into operation, an extensive monitoring network was installed to assess their performance. The instrumentation used is described and evaluated.

Chapter 6: Results and Discussion

The results from operation of the horizontal wells are presented and well performance is assessed in terms of leachate flow, gas flow, pore pressure reduction and waste compression.

Chapter 7: Conclusions

The main findings of the research are summarised and recommendations for future work are discussed.

CHAPTER 2

Background Information

It was essential to gain a thorough understanding of the directional drilling process before any adaptations could be made to facilitate the installation of horizontal wells in waste. The directional drilling process is described in this chapter. The Cardiff Bay case study is then used to illustrate an interesting application of the technology and one which had a significant influence on the subsequent landfill trials.

2.1 Comparison of Vertical and Horizontal Wells

It was considered that horizontal wells installed in landfill might offer a more effective solution for leachate control than conventional vertical wells for the following reasons.

1) At large drawdowns, vertical wells may suffer from a reduction in the wetted area through which flow can occur [Powrie & Beaven, 1999]. Horizontal wells, owing to their position near the base of the waste, should not become exposed to unsaturated material until fluid extraction is at an advanced stage.

2) The landfill may be compartmentalised as a consequence of depositing low permeability cover material over the waste at the end of each day. The greater length of a horizontal well makes its overall performance less susceptible to installation through isolated

compartments which may have low hydraulic conductivities. However, if the entire horizon in which the horizontal well is situated is of low permeability then flow rates will be reduced. If the objective of the well is to reduce pore pressures as opposed to extracting large volumes of leachate then compartmentalisation may not be a problem.

3) Owing to its greater length, a single horizontal well may exhibit an effect over the same volume of waste as a number of vertical wells, particularly if the waste mass is shallow. For general land remediation projects it has been suggested that one horizontal well can replace between 10 to 50 vertical wells depending on the dimensions of the contaminant plume [STAR Environmental Inc., 1998].

4) The installation costs of a horizontal well are significantly greater than for a vertical well but the overall cost of the remediation project is a more important consideration. [STAR Environmental Inc., 1998]. Subsequent infrastructure, maintenance and pumping costs should be lower with horizontal wells as fewer wells are required. Indeed, by targeting the specific horizons concerned, directionally drilled wells can extract water and remove contaminants without the unnecessary removal and treatment of water from unaffected horizons.

5) Dense vertical well-fields on landfills delay tipping operations by restricting the movement of vehicles over the site, particularly when the wells are connected by a network of pipes for transporting leachate and gas. In addition, vertical wells are frequently damaged through being struck by vehicles. As tipping continues, vertical wells must be continually extended upwards and the connecting pipelines replaced. The well in Figure 2.1 is protected by a concrete ring but the repeated removal and replacement of concrete rings as the waste level rises is awkward and time consuming. The emerging end(s) of a horizontal well can be located away from tipping locations or even beyond the landfill boundary.

Figure 2.1 Large plant must work around vertical wells (and piezometers) and collisions are common. Here the piezometer tubing is protected by a concrete ring [Rainham 1999].

6) The settlement of waste can result in vertical and lateral movements which frequently damage vertical wells. Watts & Charles (1999) state that most settlement of a landfill is usually associated with one-dimensional compression although the variability of the waste will usually mean that differential settlements are relatively large. Figure 2.2 is a photo taken from a borehole camera and shows how two sections of vertical well casing have become offset due to lateral movement of the waste. Horizontal wells can be installed entirely within the lower, older waste horizons where movements are smaller owing to the proximity of the site base.

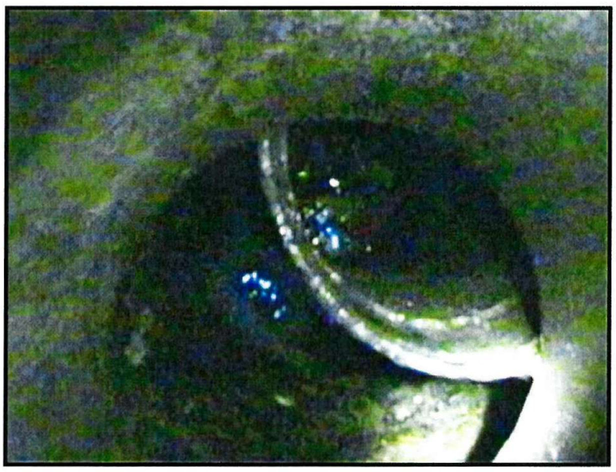

Figure 2.2 Vertical well casing offset as a result of lateral movement of the waste mass [Ockendon Landfill, Essex, 2000]

7) Other systems of 'horizontal' drainage such as tyre trenches, gravel filled trenches* and basal drainage layers have proved effective at dewatering waste and maintaining low leachate levels [Cleanaway, 2001b].

2.2 The Directional Drilling Method

During this project four horizontal wells were installed in two landfills by directional drilling. An account of the basic technique, known as back-reaming, is given below. Chapter 4 discusses the installation of the four wells and gives details of the adaptations made to the basic technique to overcome the particular problems of directional drilling in a landfill.

Before drilling commences the desired profile and dimensions of the borehole must be calculated. These include target depth, required length, borehole diameter and the position of the entry and exit points. The profile is dictated by:

- the type and diameter of the installation required (e.g. cable, large pipeline),
- the width of the obstacle being crossed (e.g. wide river, narrow rail tracks)
- the ground conditions that are likely to be encountered, and
- any obstructions that might be present such as existing services or building foundations.

The drilling rig provides three main forces:

- thrust to the drilling assembly to advance the drill bit through the formation,
- torque to rotate the drill string to reduce skin friction, and
- tensile force to pull the drill string, back-reamer and casing (or cable/well screen) through the borehole.

The size of the drilling rig will be determined by the scale and dimensions of the project. Rigs are categorised according to their maximum pullback capacity and range from *mini-rigs* with <200 kN capacity, through *midi-rigs* of 200-800 kN capacity, to *maxi-rigs* with up to

* Cleanaway Limited have excavated large trenches back-filled with vehicle tyres for the purpose of leachate control. Cleanaway have also installed 9km of 8m deep gravel trenches through an area of ageing industrial waste using a specialised trenching machine. Both these trench types have proved effective in the control of leachate levels [Cleanaway, 2001b].

4000 kN capacity. Photographs of a mini and maxi rig are reproduced in Figure 1.1. The main components of a directional drilling rig are indicated in Figure 2.3.

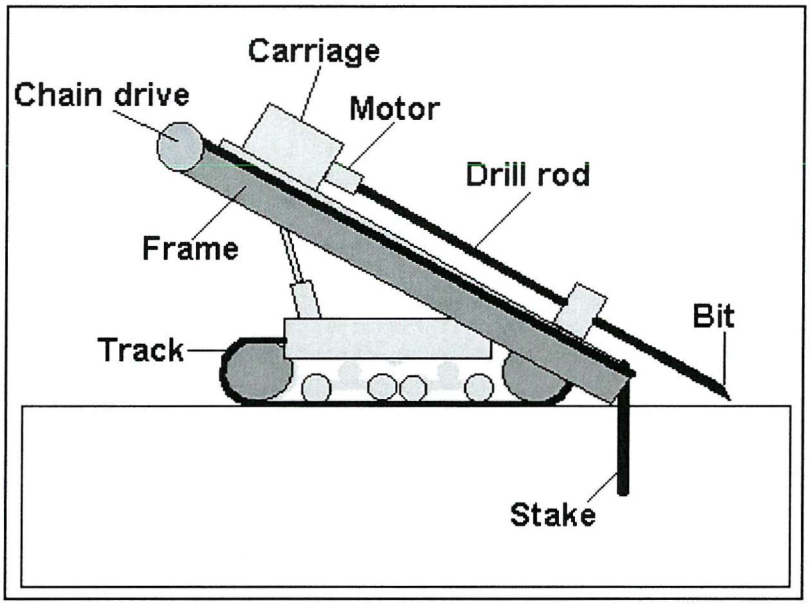

Figure 2.3 Schematic diagram of directional drilling rig [Source, HDDwell.com]

The dimensions of the drill rods will be determined by the project specifications and possibly constrained by the size of the rig used. Rods range from 25mm to 300mm in diameter and can be 2 to 9m long. Drill rods are hollow to allow fluids to pass from the drilling rig down to the drill head. Drilling rigs must be anchored to the ground by staking or attachment to a buried weight. This provides a reaction force to the thrust or pull-back.

2.2.1 Pilot Drilling

The first stage of any installation is to drill the pilot borehole as indicated in Figure 2.4. This is started at a shallow angle, typically 5° to 30°, to the ground surface. After drilling to the target depth, the drill head is reoriented to drill in the horizontal direction. This is often done gradually so as not to exceed the *minimum bend radius*[*] of the drill rods or the casing which will follow. Once the horizontal section is complete, the drill bit is reoriented upward and returns to the ground surface.

[*] The *minimum bend radius* is the shortest radius of the circle defined if the rods were drilled at maximum curvature.

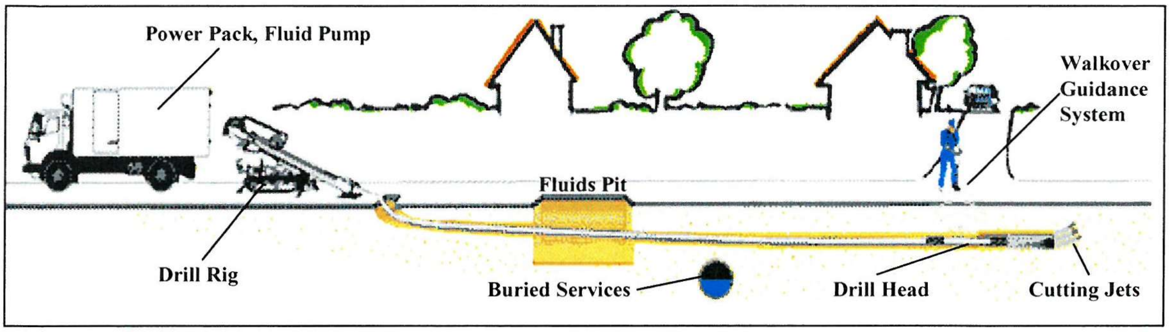

Figure 2.4 Drilling of pilot borehole [Source, HDDwell.com]

2.2.2 Drilling Fluids

Throughout the drilling process fluids are pumped through the drill rods to the drill bit. Water is the main component of drilling fluid. In most cases bentonite or polymer is added to improve the drilling fluid properties. When bentonite (a montmorillonite clay) is mixed with water it readily disperses to form a colloidal suspension which exhibits thixotropic properties, i.e. it gels when left undisturbed but becomes fluid when agitated [Craig, 1992]. Drilling fluid has a number of functions:

- to cut the formation by jetting or to provide fluid pressure to drive a tri-cone reamer,
- to lubricate and cool the drilling assembly,
- to convey cuttings in suspension from the borehole,
- to prevent continual migration of fluid into the formation through formation of a *filter cake*, and
- to provide a fluid pressure to prevent collapse of the borehole walls.

Fluids flowing out of the borehole are collected in a fluids pit. From here the suspended cuttings are filtered out and the fluids recycled. This recycling system uses less water and fluid additives and a smaller volume of returning fluids must be disposed of. This has obvious cost savings.

The critical properties of the drilling fluid are flow velocity, gel strength, density and the ability to control fluid migration.

Velocity and Gel Strength

Flow velocity and gel strength are the primary factors that determine the ability of the drilling fluid to remove cuttings from around the drill bit and transfer them through the borehole. The gel strength of the fluid determines whether cuttings will be held in suspension or will settle out during a hiatus in fluid flow.

Density and Fluid Migration Control

According to Milligan (2000), the first requirement of the slurry (drilling fluid) is that it should form a filter cake of consolidated and gelled bentonite (or polymer) against and within the soil. This then becomes a low permeability membrane able to transmit fluid pressure in excess of groundwater pressure, into effective stress between soil particles and hence stabilise the borehole walls, Figure 2.5. To do this the fluid must not be able to penetrate too readily into the formation; in open soils a fluid may simply dissipate into the ground without transmitting any significant stresses into the soil, causing large consumption of fluids and poor borehole support.

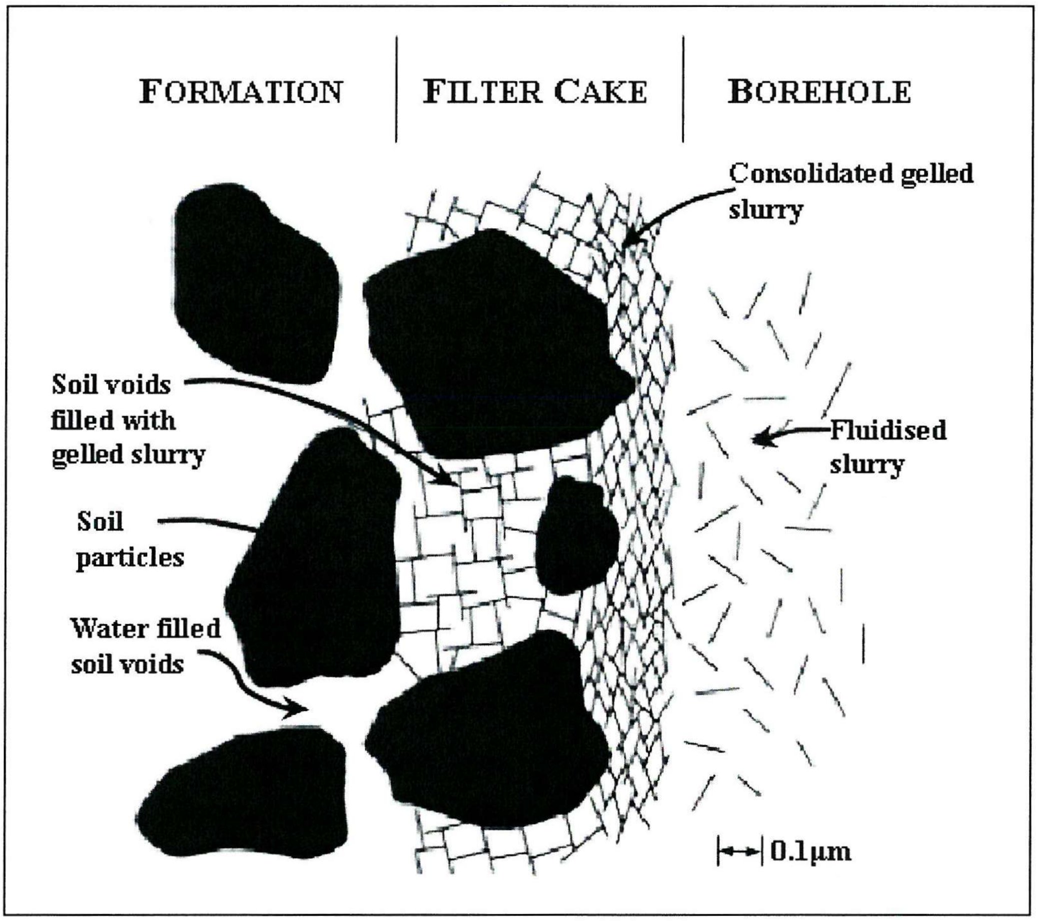

Figure 2.5 Formation of a filter cake, from Washbourne (1986).

The thickness of the filter cake is a function of the hydraulic conductivity of the formation and the density of the drilling fluids. The filter cake formed in, say, a coarse sand formation will be thicker than in a fine sand formation as a result of the larger pore spaces that require sealing. If the grain size of the formation is too large a filter cake may not form and the fluids will continually migrate out into the formation. This is a particular problem for installations within highly permeable formations such as gravel.

Ideally, for the installation of horizontal wells, the filter cake will form quickly during construction of the borehole, yet will be readily removed during well development. The blocking of the pores with bentonite particles or polymer causes a localised reduction to the hydraulic conductivity of the formation. In recognition of the likely detrimental effects of bentonite based drilling fluids on the hydraulic conductivity of the surrounding ground, specialised biodegradable drilling fluids have been developed for projects where formation permeability must be maintained, e.g. the installation of horizontal wells.

Drilling fluids that migrate into the formation may change the pH, redox potential or ionic strength of the groundwater. This may cause dissolution or precipitation of minerals. The minerals most likely to dissolve or precipitate (depending on groundwater and drilling fluid chemistry) are the oxides and hydroxides of Fe and Mg, and carbonates including aragonite and dolomite. The precipitation of minerals will reduce the permeability of the formation (HDDwell.com, 2001a).

2.2.3 Guidance Systems and Steering

At the front of the drill head is the drill bit. Drill bits come in two main types. The more common has a bevelled edge as in Figure 2.6. Nozzles on the bevelled edge create jets of drilling fluid which provide the cutting action. The asymmetrical angle of the bevel causes the drill head to change direction when thrust forward. However, if the drill head is thrust and rotated simultaneously a straight course will be maintained as the bevelled edge and jets have no preferred orientation.

Figure 2.6 Drill bit with bevelled edge.

The second type of drill bit is the tri-cone reamer, Figure 2.7. The angle required for steering is provided by a short piece of drill rod (*bent sub)* located behind the drill bit that incorporates a 3°, 4° or 5° bend. The asymmetry of the bent sub causes the drill head to change direction when thrust forward. If thrust and rotation are applied simultaneously a straight course will be maintained.

Figure 2.7 Tri-cone drill bit. The angle of the bent sub can be seen [DrillTec, Rainham 2000].

It is crucial to know the rotational position (*roll*) of the bevelled edge or bent sub when making a steering correction. This is measured by a roll sensor that uses a twelve position fluid switch and gives a *clock-face* read out. The inclination (*pitch*) of the drill head is also measured using a fluid switch [Gallucci, 2001]. Depth is the third parameter required to guide a directional borehole. With the *walkover system* a signal is transmitted from the sonde to the ground surface and a hand held receiver is placed where this signal is strongest, i.e. directly above the drill head (Figure 2.8). The signal strength is converted into a reading of depth below ground level. Pitch and roll information are also transmitted with this signal. The signal strength limits the walkover system to depths of less than 10m.

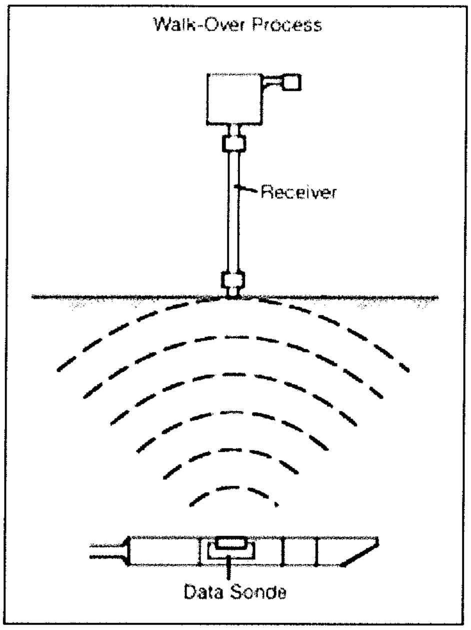

Figure 2.8 Operating principle of the walkover guidance method; a sonde in the drill head sends signals indicating depth, pitch and roll to a surface receiver [Source, HDDwell.com].

A *wireline* system is used at greater depths. This system calculates the pitch and roll using triaxial magnetometers and accelerometers housed within a non-metallic drill head. The orthogonal magnetometers measure the three components of the earth's magnetic field and determine the angle made by the long axis of the drilling head with this field. The accelerometers determine the angle between the long axis of the head and the earth's gravitational field. From these angles the azimuth, pitch and roll may be determined. The data are transmitted along a wire running through the drill rods to a computer at the surface. Here they are combined with 'distance drilled' data to determine the location and orientation of the drill head in three dimensions. The AccuNav® system, transmits data from the drill head to the surface via an electromagnetic signal sent through the actual drill rods [Maurer Technology, 2002]. Connection of new wire and problems with wireline breaks are avoided. AccuNav® is essentially 'wireless', but is not to be confused with the true wireless walkover system.

Although wireline systems allow drilling at greater depths than walkover, they have two principal disadvantages:

1. Any errors are propagated such that a small error near the start of a borehole can result in an increasing deviation between the actual and theoretical locations of the drill head as drilling continues.

2. Interference to the earth's magnetic field originates from a variety of sources including pipelines, railway tracks, overhead power lines and steel reinforcement in concrete. In these common circumstances, the system may lose accuracy.

A second type of wireline system, TruTracker™, overcomes these problems by using a secondary electromagnetic survey system in conjunction with the magnetometer-accelerometer guidance instrument. A cable (surface coil) is laid on the ground in a rectangular configuration with the long axis centred over the borehole path and the 3D Cartesian coordinates of the coil corners are surveyed. A DC current is applied to the coil to induce an electromagnetic field of known intensity and size into the subsurface around the drilling head (Figure 2.9) and a 'shot' of the magnetometer values is taken. These values are influenced by a) the field generated by the surface coil, b) the earth's magnetic field, and c) fields from any sources of interference. A reverse current is then applied to the coil and a second shot is taken. The second shot is influenced by the same fields except that the field generated by the coil has been reversed. The surface computer then subtracts the values of the second shot from those of the first to eliminate the effect of the earth's magnetic field and any interference. The data, including the coordinates of the coil corners, can then be used to determine the three dimensional location and orientation of the drill head. With this system each calculation of the drill head position is independent of the last and so errors are not propagated.

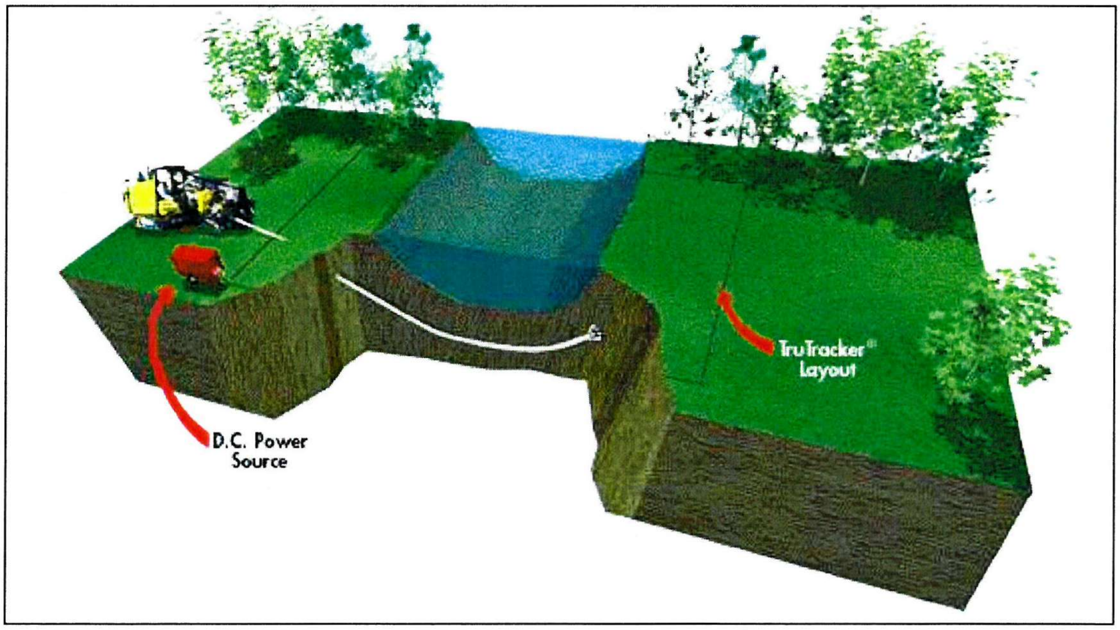

Figure 2.9 Lay out of wire grid for TruTracker™ guidance system [Source, Tensor Energy Products]

The vertical depth and lateral position provided by the TruTracker™ system is accurate to ± 2% of depth provided that a field of sufficient intensity is produced at the drill head [Waters, 2001].

2.2.4 Back-reaming and Casing Installation

The pilot borehole is complete once the drill head exits the ground. The drill head is then removed and the *back-reamer*, *swivel* and *casing* are attached (Figure 2.10). During the back-reaming process, as the assembly is pulled through the borehole, drilling fluid is pumped to the reamer (Figure 2.11).

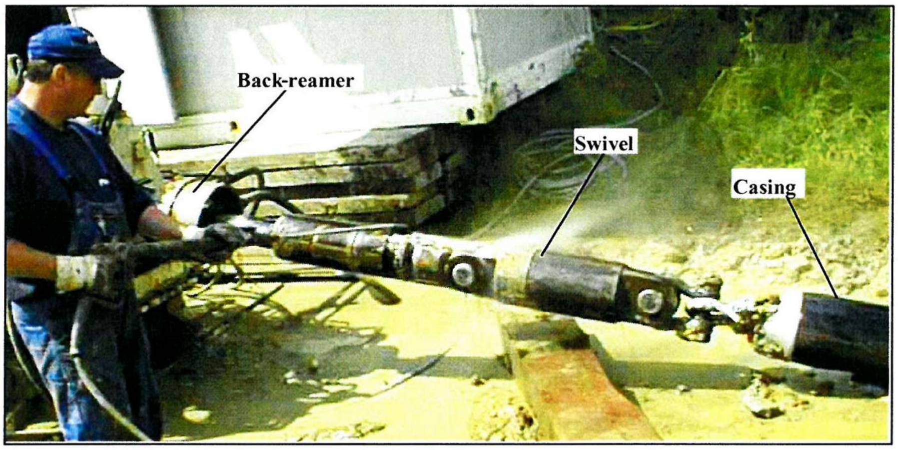

Figure 2.10 Back-reaming assembly showing back-reamer, swivel and casing [Flowtex installation under River Maas, Netherlands 1999]

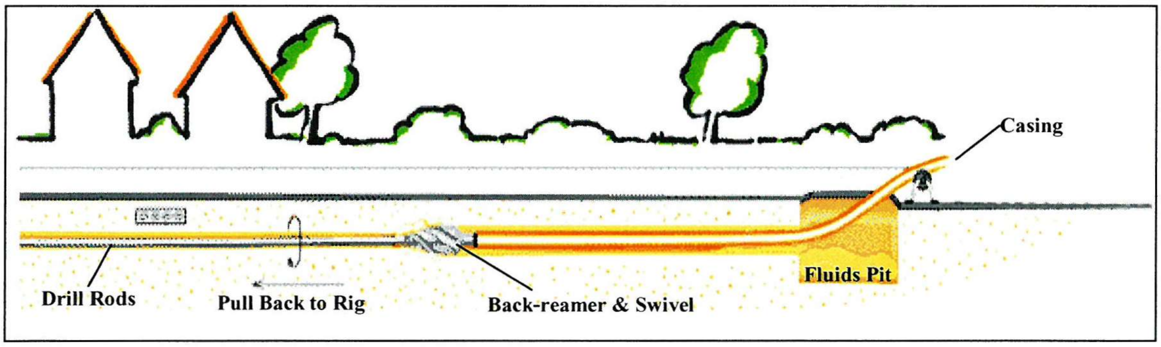

Figure 2.11 Back-reaming and casing installation [Source, HDDwell.com]

In theory the effect of the back-reamer is to cut the formation and increase the diameter of the borehole enough to allow installation of the casing. It is important that the back-reamer mixes the cuttings with the drilling fluid to produce a flowable slurry which is discharged from the borehole to create a void for the casing. Back-reamers come in a variety of styles and can be selected to suit the ground conditions. The purpose of the swivel is to allow the back-reamer to rotate without rotating the casing. Depending on ground conditions and the required diameter of the borehole a number of pre-reams may be carried out prior to casing installation. During pre-reaming, back-reamers of increasing size are used to gradually enlarge the diameter of the borehole. As no well screen is being pulled in during a pre-ream, a continuous drill string is maintained through the borehole by the addition of drill rods at the exit end. Pre-reaming is often unnecessary but is advisable for projects where the casing has a low tensile strength, for example PVC and slotted well screen [Wampler, 1997]. On completion, a horizontal well will require development.

The problems associated with well screen of low tensile strength may be overcome by use of a carrier casing. Before installation the well screen is placed inside a steel carrier casing so that during pullback all tensile forces are applied to the casing and not to the screen. This allows the design of the well screen to focus on fluid extraction properties rather than the strength needed for installation. Following installation, the carrier casing is pulled out, exposing the well screen to the formation. Table 2.1 shows the tensile strength values for selected well materials.

Table 2.1 Tensile strength of selected well materials.

Material	Tensile strength (MPa)	Manufacturer
Carbon steel pipe (carrier casing)	240 - 500	U.S. Steel
Stainless steel well screen (Vee-wire)	34	Johnson
HDPE pipe	22* - 34**	CPChem
PVC well screen (Vee-wire)	14	Johnson

All pipe diameters in the table are 102mm. *Yield strength ** Ultimate tensile strength.

Tensile strength will vary with wall thickness and diameter. However, the tensile strength of the Vee-wire screen is dependent on the area of the rods that run the length of the well screen (Figure 2.12). Conventional slotted well screen will retain a greater tensile strength if slotted longitudinally as opposed to radially. For vertical well installations the well screen will be subject to column loading from the weight of the well screen above. However, column loading is not a consideration for horizontal wells installed using the back-reaming technique.

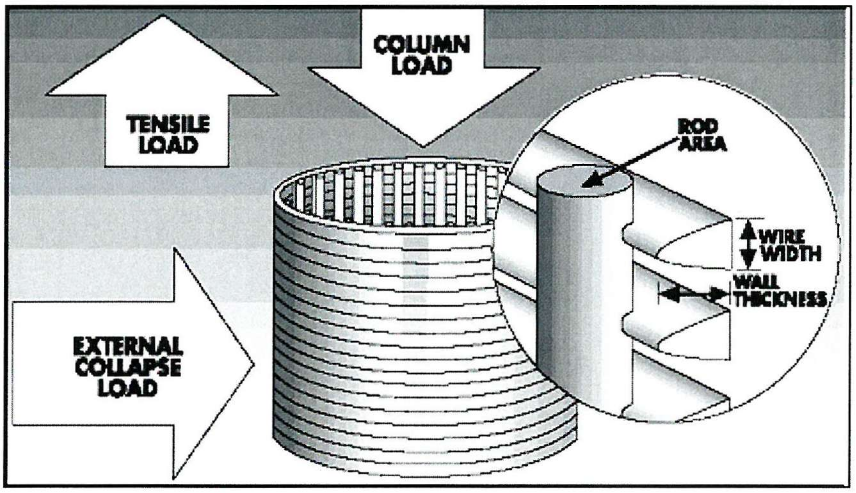

Figure 2.12 Vee-wire well screen. Tensile strength is proportional to rod area [Source, Johnson]

A well screen may become smeared with cuttings and drilling fluid as it slides along the borehole walls if no carrier casing is used. This will reduce the open area available for flow and cause higher entrance velocities at the slots that remain open. These higher velocities may entrain coarser particles which may then enter or block the well screen. Smearing is reduced when a carrier casing is used as the well screen is not in contact with the borehole walls during installation. In addition, on removal of the carrier casing, the borehole walls will (eventually) collapse into the void and onto the well screen, this loosening may increase the local hydraulic conductivity and improve well performance.

2.3 Horizontal Wells Case Study – Cardiff Bay

The Taff & Ely river estuary in Cardiff Bay has a tidal range of 14m which causes large areas of mud flats to be exposed at low tide. As part of the redevelopment of Cardiff Bay, a barrage 1.1km in length has been constructed to enclose the estuary and create a freshwater lake by maintaining the water level at the equivalent of high tide (Figure 2.13). As intended, the lake has become the focal point for housing, retail and leisure developments.

Figure 2.13 The Cardiff Barrage encloses the Taff & Ely estuary [Source, Cardiff Harbour Authority]

As fluvioglacial gravel deposits in the surrounding area are hydraulically connected to the freshwater lake, the local groundwater levels have risen. In order to prevent the flooding of basements and damage to foundations, an extensive groundwater control scheme was installed around South Cardiff. This scheme included the first directionally drilled horizontal wells in the UK. A single well was installed as a trial in May 1998 with a further three wells installed in Autumn 1999. The specifications of these four wells are outlined in Table 2.2, followed by a discussion of the success and failure of these specifications.

Table 2.2 Installation details of horizontal wells in Cardiff Bay.

ITEM	TRIAL WELL	WELLS 2, 3 & 4
Drilling contractor	Allen Watson	Stockton Pipelines
Date of installation	May 1998	Autumn 1999
Geological sequence	Made Ground, Alluvium, Fluvioglacial Gravels (target aquifer), Mercia Mudstone	Made Ground, Alluvium, Fluvioglacial Gravels (target aquifer), Mercia Mudstone
Drilling rig size	Midi Rig (approx. 400kN)	Well 2, Midi Rig - 400kN Wells 3 & 4, Maxi Rig - 1000kN
Drill rod diameter	100mm	88mm
Guidance system	Wireline	Walkover then wireline
Drilling fluid additive	Bentonite	Bentonite
Entry angle	Various attempts, 18° successful	18°
Borehole length	134m	160m, 130m, 130m
Max. depth below ground	14.5m	14.5m
Carrier casing diameter	250mm	200mm
Carrier casing material	Steel	Steel
Well screen material	Stainless steel wire wrap on a perforated steel pipe base	Stainless steel wire wrap on a perforated steel pipe base
Well screen diameter	140mm OD, 125mm ID	140mm OD, 125mm ID
Well screen slot width	0.5mm	0.5mm
Length of well screen in gravel aquifer	54m	70m
Well development methods	Surging, dosing with SAPP*, clearance pumping	Surging, jetting, jetting with SAPP, clearance pumping
Well yield post development	14 l/s	5 to 10 l/s depending on tide
Well yield June 2001 (barrage closed)	4 l/s	6 l/s

Entry Angle and Bend Radius

* SAPP Sodium Acid Pyrophosphate (deflocculant chemical for breaking up clay particles).

The intention was to install a 100m long well in the 6-7m thick gravel aquifer. It was difficult to steer the drill bit in the gravel layer as it had a tendency to rise upwards. As a result, the first attempt, with an entry angle of 10°, exited at the top of the gravel layer with only a 31m penetration of the gravels. On the second attempt the entry angle was increased to 20°. This proved to be an overcompensation and caused the drill bit to exit at the base of the gravel layer. After two further failed attempts at 16° and 19°, the optimum entry angle was found to be 18° and a 54m length of well screen was installed in the gravel layer. This was short of the original target but considered to be the greatest length obtainable given the constraints of maximum total borehole length and the minimum bend radius of the rods. The importance of selecting the correct entry angle was well illustrated in the trial.

For the installation of wells 2, 3 and 4 the drill rod diameter was reduced from 100mm to 88mm and the carrier casing from 250mm to 200mm. This permitted a shorter bend radius which in turn allowed a greater length of well screen to be installed within the gravel layer.

<u>Rig Size</u>

A 400kN capacity rig was used for both the trial well and well 2. Although this midi-rig was of adequate size, both rigs ran at near full capacity during pullback. Had an obstruction been encountered, capacity may have been exceeded. To avoid potential problems a larger 1000kN rig was used for wells 3 and 4.

<u>Guidance System</u>

A wireline system was used successfully to guide the trial borehole. As explained in section 2.2.3 the wireline system is more complicated, time consuming and therefore more costly to operate than the walkover system so the guidance of well 2 began with walkover. However, this was subject to interference from the saline groundwater and the drilling had to recommence with a wireline system [Clarke, 2000].

<u>Drilling Fluid, Well Screen and Well Development</u>

The drilling fluid additive used in the trial was bentonite. Due to the five attempts made at pilot drilling, 256m^3 of drilling fluid were used and lost to the formation. No drilling fluid returned from the borehole suggesting that the pore spaces in the gravel were too large, enabling fluids to migrate away from the borehole and preventing the formation of a filter cake. The bentonite concentration used was 37kg/m^3, meaning that a total of 9.5tons of bentonite were introduced into the formation. It is to be expected that this amount of clay would cause a significant reduction to the hydraulic conductivity in the vicinity of the well. As a result the well development process was prolonged and involved nine cycles of surging, chemical dosing and clearance pumping. The effects of development on well yield were marked and are indicated in Figure 2.14.

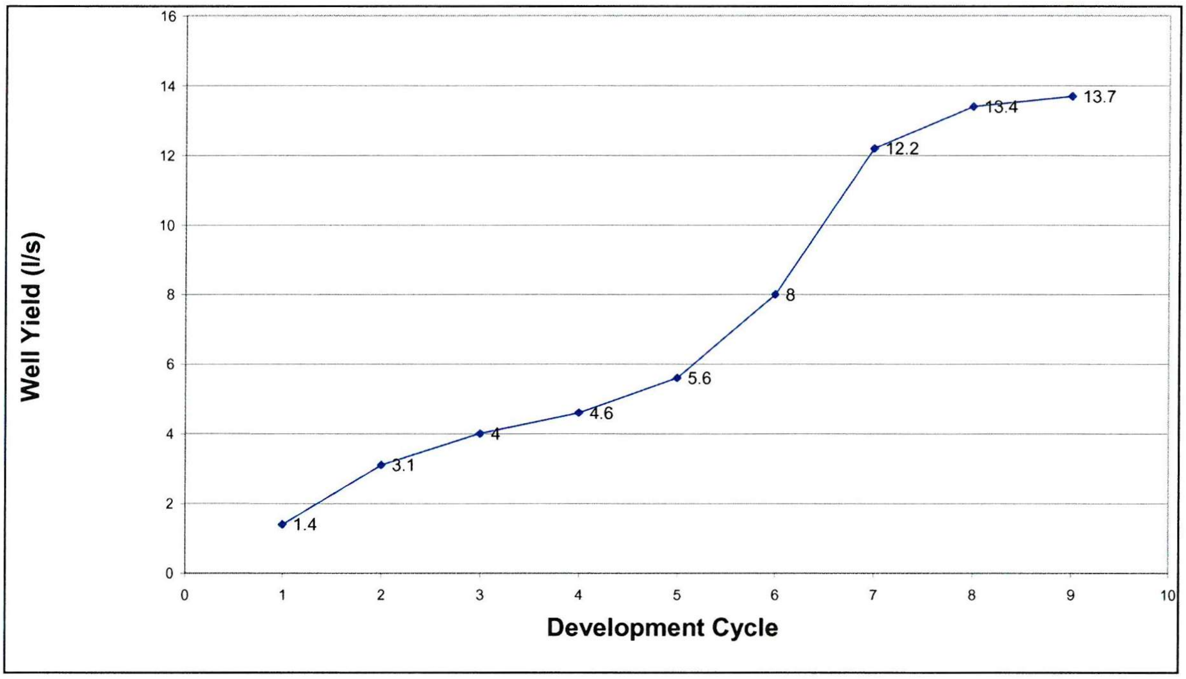

Figure 2.14 Sustainable yield of trial horizontal well after each development cycle [Source, CBDC 1998]

Before work began on wells 2, 3 and 4, the well development contractor [WJ Groundwater Ltd, 1998] advised Stockton Pipelines to make the following changes to the drilling and installation plan:

- The slot width of the well screen used in the trial (0.5mm) was rather fine for a sandy-gravel aquifer and that a slot width of 2 or even 3mm might be suitable and give higher flow rates.
- The use of bentonite in the drilling fluid should be avoided, in favour of biodegradable alternatives if possible.
- The volume of drilling fluid should be minimised as the 260m^3 used in the trial may have penetrated the formation by more than 0.5m along the entire length of the well. Even high energy development techniques of jetting and surging would be unlikely to have an effect over this radius.

WJ Groundwater Ltd considered that attention to these three points would 'have an infinitely greater beneficial effect on well efficiency' than any subsequent well development. However, these changes were not implemented. No flow was obtained from wells 2, 3 and 4 when suction pumping first began. Well development, which included chemical dosing, surging, jetting and clearance pumping, increased the flow rates to 8 to 10 l/s although these values were still only 50% of the target.

Following installation, development and testing of the horizontal wells, the barrage was closed, impounding freshwater in the bay and preventing tidal fluctuations. The groundwater control scheme which included the four horizontal wells was brought into operation to maintain groundwater at its pre-impoundment level. The performance of the wells is summarised in the Cardiff County Council Harbour Authority Report [CCCHA, 2001]. Three of the wells have exhibited a gradual decline in flow from an average of 10 l/s in December 1999 to an average of 4.5 l/s in June 2001. The decline in flow rate is matched by a steady rise in groundwater levels of between 0.7m and 2.6m, Figure 2.15. The fourth well, located in a different area of Cardiff Bay from the other three wells, has maintained a flow rate of between 7-8 l/s. CCTV inspections in May 2001 revealed silt and bacterial deposits in the three wells with declining flow rates. The fourth well was free from these deposits. All wells were jetted to displace any deposits and after further CCTV inspection the wells were found to be clean. However no increase in flow rate was subsequently recorded and rates continued to decline.

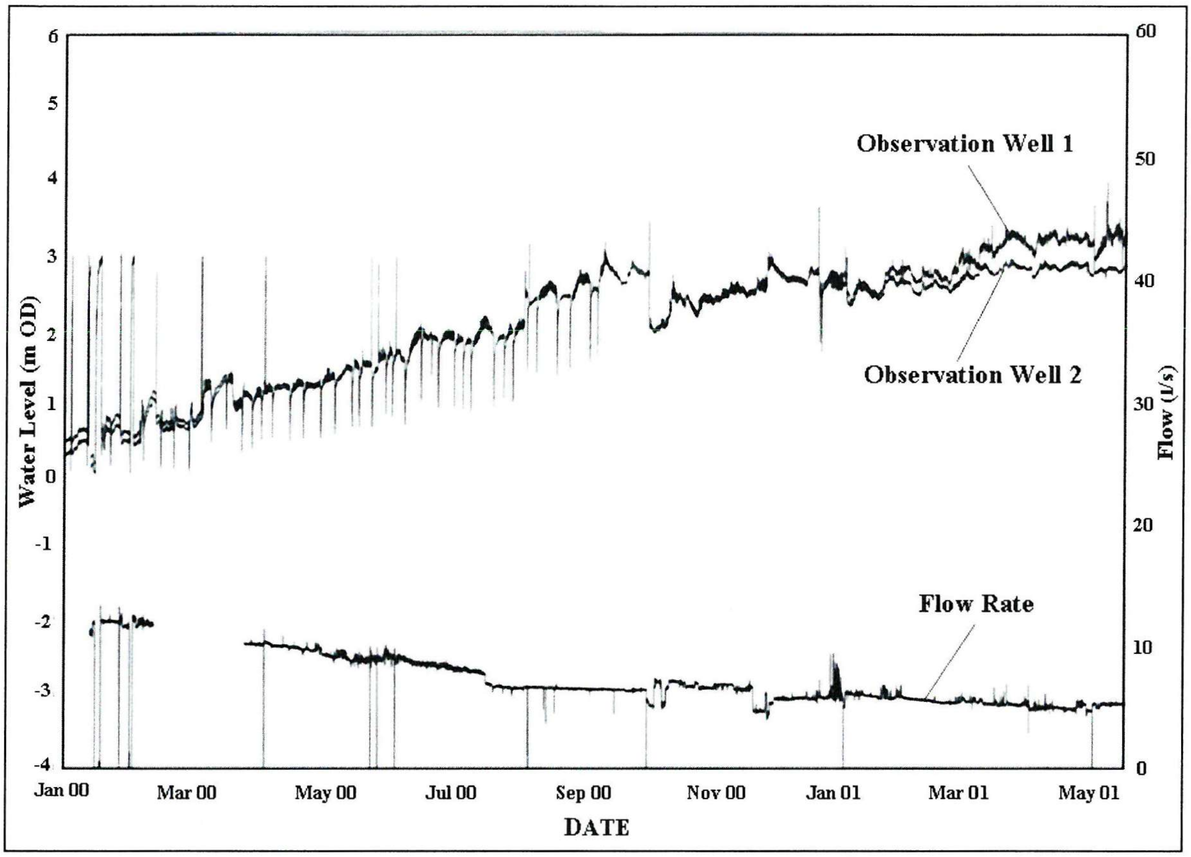

Figure 2.15 Flow from Horizontal Well (location Recreation Ground South, Cardiff Bay). Note gradual decline in flow rate matched by gradual rise in groundwater levels. [Source, CCCHA 2001]

It is possible that although jetting cleaned the inside of the wells, the design of the well screen may have severely inhibited the effect of jetting on the surrounding gravel formation. The well screen was a Vee-wire® design mounted on a perforated pipe base*, Figure 2.16. Whilst the Vee-wire has a large and evenly distributed open area (17% for 0.5mm slots), by mounting it on a pipe base with infrequent perforations the jets can only pass directly into the formation in a limited number of places. Where jets do not pass directly into the formation their energy will be quickly dissipated by the pipe and wire. It seems likely that although the wells were clean after jetting, the silt and bacterial deposits remained in the surrounding gravel, impeding flow to the well. The fourth well that did not exhibit a decline in flow rate was clean on first inspection suggesting that the surrounding formation was also free from silt and bacterial deposits. This fourth well is located in a different area of Cardiff Bay where the silt volume in the gravel layer may be less.

* Vee-wire well screen is described in detail in section 4.3.3.

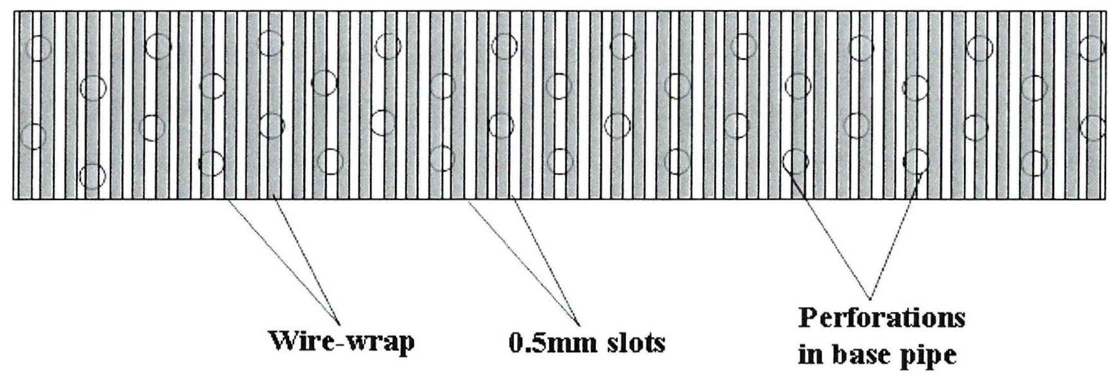

Figure 2.16 Vee-wire well screen mounted on a perforated pipe base as used in Cardiff Bay. The total area, where slots and perforations overlie is small, severely impeding the action of the jets.

Nevertheless, groundwater levels in the vicinity of the horizontal wells remained below pre-impoundment levels in April 2002, and in this respect the wells can be described as effective. However, if flow rates continue to decline, levels may rise above pre-impoundment levels within the next two to three years.

The wells are operated with surface mounted suction pumps. As the invert of each well is approximately 14m below ground level the wells can never be fully dewatered using suction. Greater flow rates may be achieved by using submersible pumps and lowering the water level to the invert. Increasing the hydraulic gradient and flow rate in this way may also release some fines that have become lodged in the pore spaces of the gravel.

It is unclear why well development immediately after installation should have caused such a marked increase in flow rate while jetting of the wells after 18 months of pumping had no apparent effect. Three possible reasons are given below.

1) Although not inspected with CCTV, it is likely that the volume of sediment lying within the well post-installation was enough to impede flow, so that its removal would have caused a flow increase. The volume of sediment within the well after 18 months was likely to have been less and may not have impeded flow.
2) Much of the sediment that limited post-installation flow would have been bentonite slurry (from the drilling fluid). The chemical deflocculant 'SAPP' may have been effective in breaking down the clay particles so that they could be removed in suspension in the flow. Chemical dosing was not used during the jetting 18 months later.

3) The pore spaces in the gravel formation that were initially blocked may have been close to the well where jetting and surging were effective. Over the next 18 months, pore spaces much further from the well may have become blocked as fine material was carried toward the well from more distant areas. The effect of jetting is likely to be limited to a radius of only 0.5m [WJ Groundwater Ltd, 1998] and so these blocked spaces were unaffected.

It seems that (a) the excessive use of bentonite drilling fluid, (b) a high silt fraction in the gravel formation, and (c) well screen that does not allow effective development, were the principal factors limiting the performance of the Cardiff Bay wells.

CHAPTER 3

Site Investigation

This chapter introduces Rainham landfill site as the chosen location for the drilling trials. At an early stage it was important to identify the hazards that the directional drilling process might encounter so a desk study and invasive site investigation were carried out, the findings of which are discussed below. The system used for logging and classification of the waste arising from the site investigation boreholes is also discussed.

3.1 Introduction to Rainham Landfill Site

3.1.1 Site Location & History

The well installation trials took place at Rainham Landfill in Essex. The following section provides a general introduction to the site focusing on aspects relevant to the project. The landfill is situated in East London on the north bank of the River Thames, 6km northwest of the M25 Dartford Crossing, Figure 3.1. The site is operated by Cleanaway Limited and receives waste principally from East and Central London with an annual input of approximately 1.7million tons. The waste type is Municipal Solid Waste (MSW) which includes domestic, commercial and (non-hazardous) industrial wastes. Since 1973, Rainham has not accepted liquid or hazardous wastes. The landfill has been expanded considerably in recent years and now covers approx. 177 hectares (1.77km^2), making it one of the largest operational UK landfills. Leachate from Rainham is pumped to the main sewer network for treatment at the local sewage treatment works. A landfill gas collection and utilisation system was installed at the site in 2000 with a generating capacity of 17MW.

Landfilling at the Rainham site has a long history with the earliest recorded filling taking place prior to 1860 directly onto the marsh land. Between 1945 and 1973 a mixture of industrial and domestic waste was accepted. Leachate from areas of the site where these materials were deposited is now heavily contaminated with red list substances*.

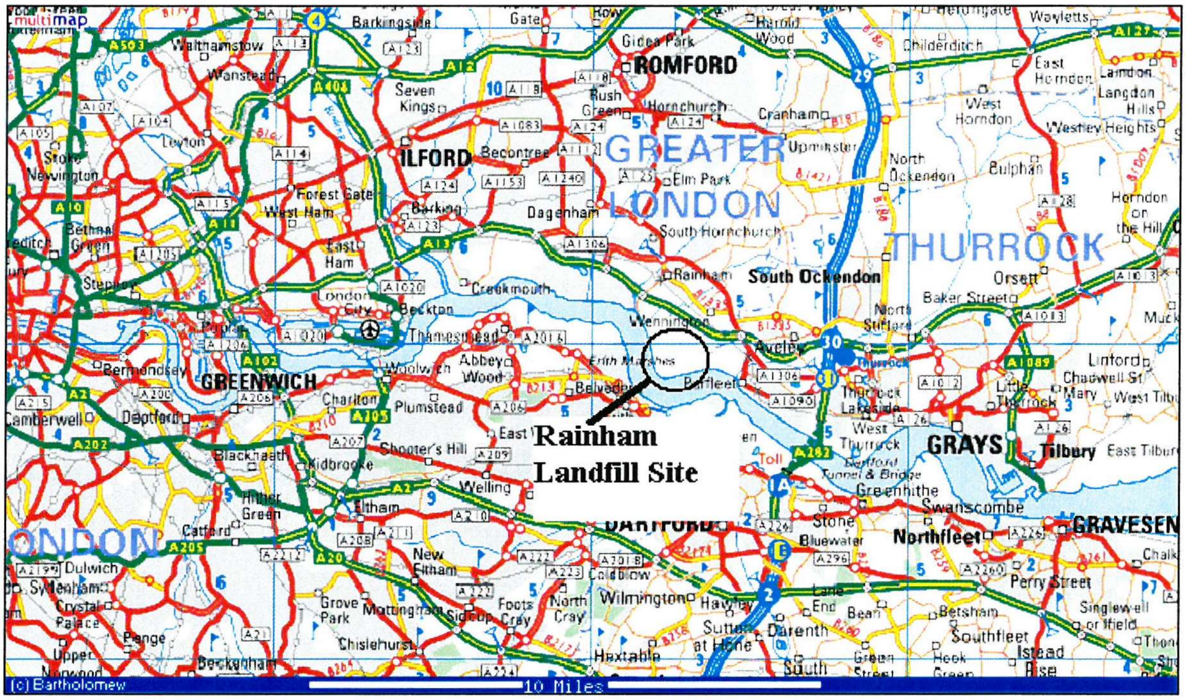

Figure 3.1 Location of Rainham Site, East London, 6km NW of the Dartford Crossing [Multimap.com].

* The UK has a Red List of 23 of the most dangerous substances which have been selected for priority pollution control. The substances are toxic, do not or are very slow to degrade in water, and are likely to accumulate in living organisms [DEFRA, 1998].

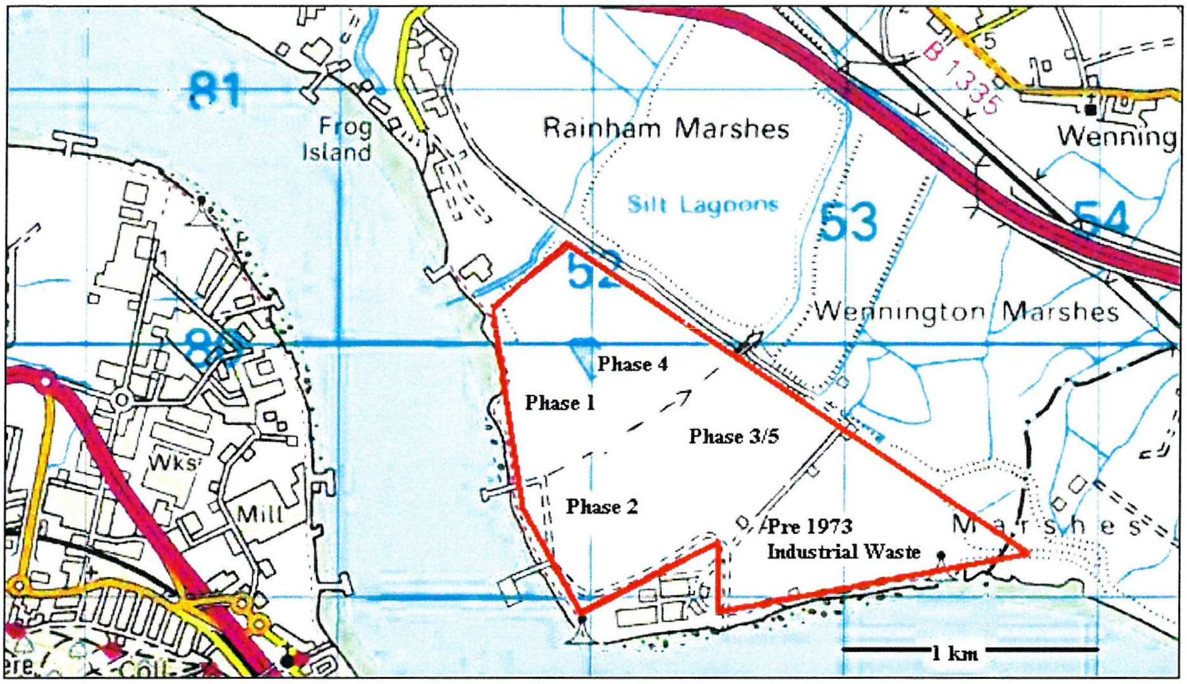

Figure 3.2 Rainham Landfill Site outlined in red. Position of Phases 1 - 5 are marked [Multimap.com].

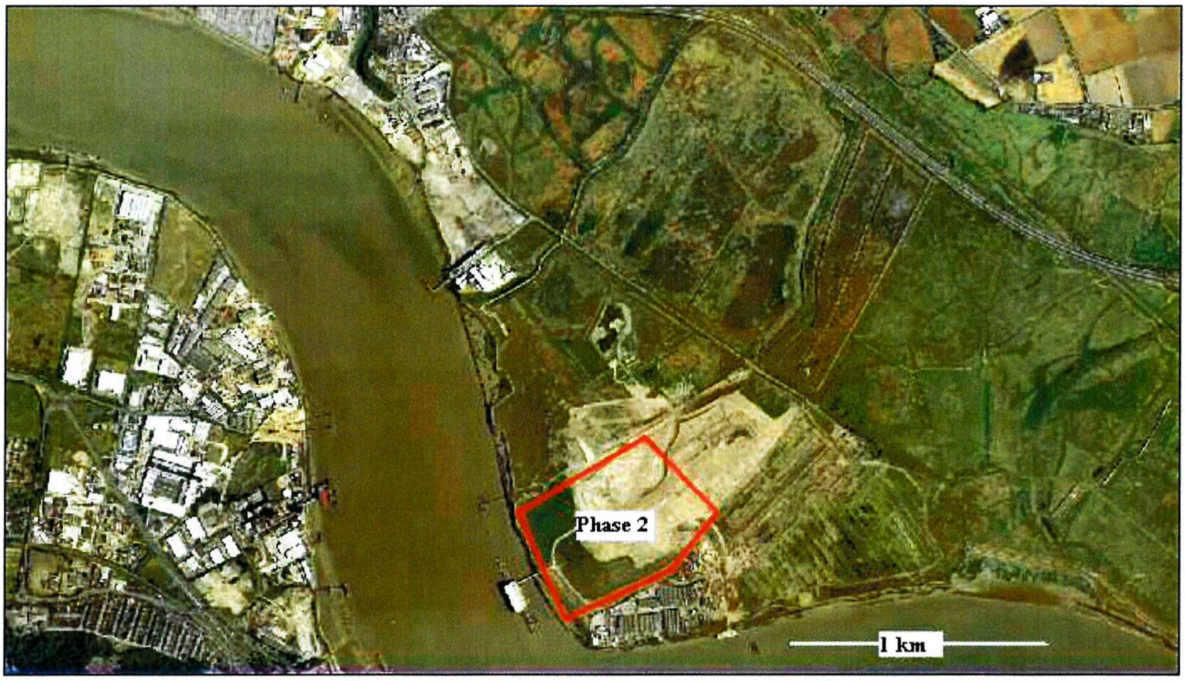

Figure 3.3 Aerial photo of Rainham landfill showing Phase 2, where field trials took place [Multimap.com]

3.1.2 Site Geology & Hydrogeology

From 1968 tipping has occurred in what were formerly Port of London Authority silt lagoons used for the drying and deposition of dredgings from the River Thames, Figure 3.4. This silt or 'hydraulic fill' is a heterogeneous material although it is composed mostly of grey clayey SILT. Boreholes penetrating the silt have found it to be of variable thickness between 1m and 3m. At Phases 1 to 4 (Figure 3.2) waste was tipped directly onto the silt with no additional geotextile or clay liners, nor any drainage systems for leachate control. In line with more stringent environmental legislation, recent phases have been engineered prior to the tipping of waste. The field trials for this project have all taken place within Phase 2 (Figure 3.3, and Appendix E). Geraghty & Miller International Inc. (1994) estimated the hydraulic conductivity of the waste in Phase 2 at 1×10^{-5} m/s. A schematic geological profile of the Phase 2 area is shown in Figure 3.5.

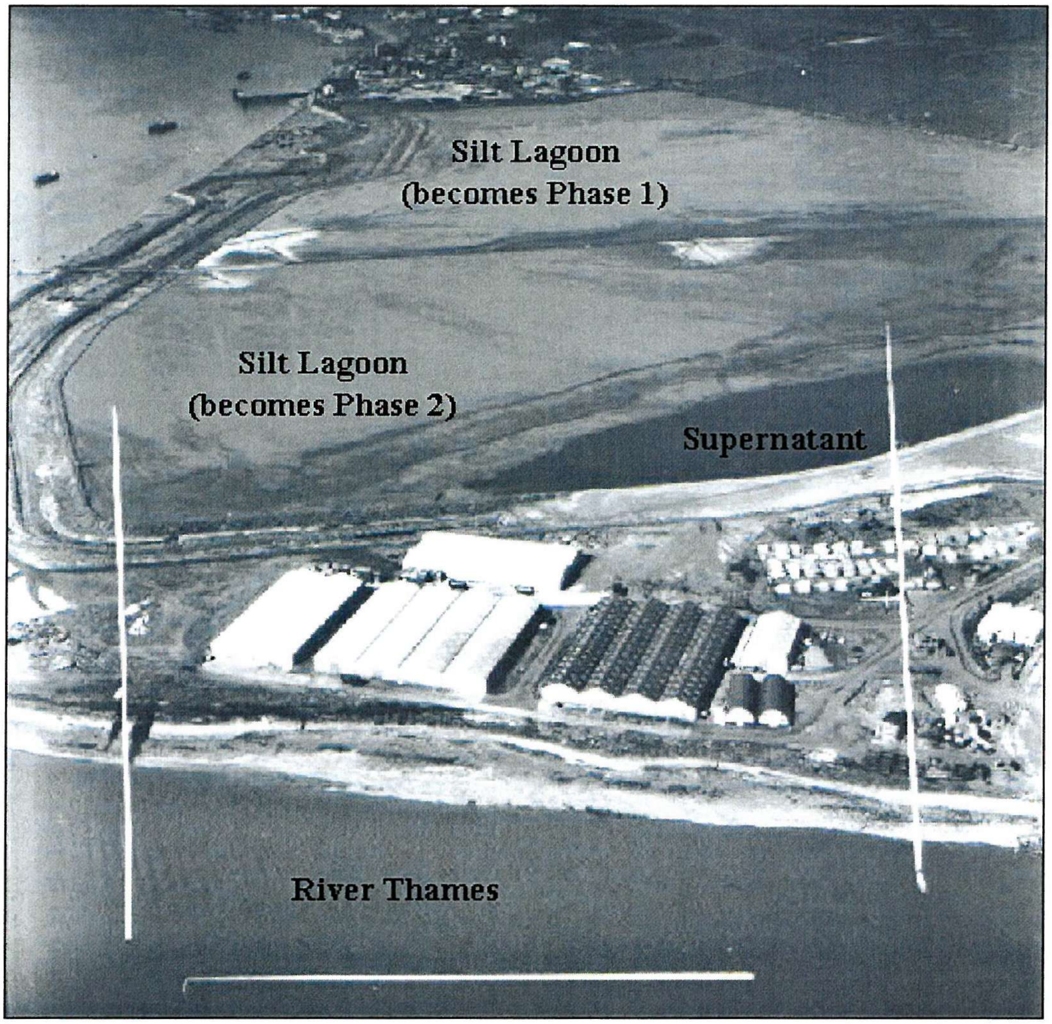

Figure 3.4 Silt Lagoons at Rainham, circa 1970, before the tipping of waste on Phases 1 & 2 began.

Rainham landfill comprises a series of waste domes currently contained within large bunds that formerly surrounded the silt lagoons [Cleanaway, 1997]. The 'landfill' is predominantly an above ground facility and would therefore be more accurately termed a 'landraise'. Phase

2 is approximately rectangular and covers an area of 35 hectares (625m × 560m). The top of Phase 2 forms a plateau at 31.5mOD and the silt/waste interface is between 0.5 to 1.5mOD. A vertical expansion of Phase 2 up to 40mOD is proposed providing leachate head reduction targets are met. The waste volume in Phase 2 is approximately 7.2million m^3 (as of February 2002).

Underlying the silt are Holocene alluvial clays typically 6 to 8m thick with hydraulic conductivity values ranging between 10^{-8} and 10^{-9}m/s [Cleanaway, 1997]. These values represent the unconsolidated state and it is considered that these values reduce to 10^{-9} to 10^{-12}m/s as waste is placed and the clay consolidates [Cleanaway, 1997]. The clays contain up to four lenses of peat which can make up to 50% of the thickness of the deposit, although the peat thickness is generally less than 20% of the total. Below the alluvial clays are Quaternary flood plain gravels (Thames Gravels) with a sand fraction of variable grain size. The gravels have a thickness of between 4.6 and 8m. In the southern area of the site, where Phase 2 is situated, the gravels unconformably overlie the Cretaceous Chalk. The chalk is between 70m and 165m thick [Golder Associates, 1995].

The gravels have hydraulic conductivities ranging between 1.4×10^{-3} and 7.3×10^{-5}m/s. An extensive fracture system dominates groundwater flow in the chalk and it is assumed the interstitial component of flow is negligible. Hydraulic conductivities for the chalk beneath the site are estimated at 6×10^{-5} m/s [Jones, 1992]. The gravel and chalk are considered to be aquifers.

The gravels and chalk outcrop in the bed of the River Thames and are therefore in hydraulic continuity with tidal waters as well as each other. According to the laboratory and field determined values for hydraulic conductivity of the silt and alluvial clay these units may be considered aquicludes. As a result of waste deposition, pore water pressures in the silt and peat are currently higher than those in the overlying waste (Figure 3.5). The resulting hydraulic gradient therefore promotes an upward flow of pore water into the waste as it dissipates from the silt. Consequently there is no potential for the downward migration of leachate into the gravel and chalk aquifers [Cleanaway, 2001b]. However, pore water pressures in the silts and alluvial clays are also higher than those in the underlying gravels, resulting in a second hydraulic gradient promoting downward flow of pore water as it dissipates from the clay.

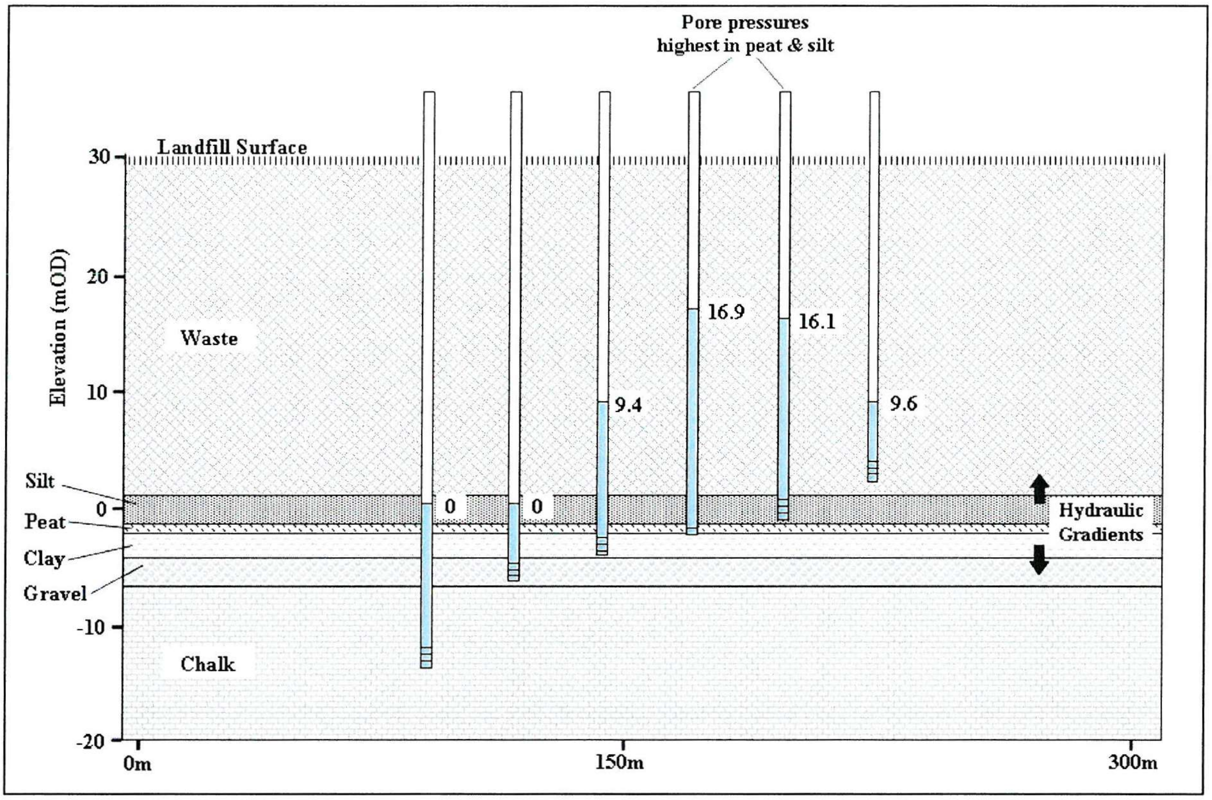

Figure 3.5 Geological cross section of Rainham at Phase 2, showing pore pressures in different horizons [Data from Cleanaway, 2001].

The tipping of waste increased the total vertical stress; this was initially borne by an increase in the pore water pressure of the silt and clay. Water is gradually seeping out from the pores owing to the hydraulic gradients upwards into the waste and downwards into the gravels. As a result the silt and clay are consolidating, through which process the effective stress is increasing. The rate at which pore water can dissipate is related to the hydraulic conductivity of the soil and reduces with reduced hydraulic conductivity [Powrie, 1997]. The periodic addition of waste to the landfill since 1973 has meant that total vertical stress has repeatedly increased and so pore water pressures in the silt and clay have not yet reached equilibrium. Figure 3.6 shows the pore pressure changes in the silt and clay under Phase 3, from 1997 to 2001. The rapid increases coincide with the deposition of each waste 'lift' (section 3.2.2). Pore pressures begin to dissipate between each lift but do not reach equilibrium before further loading occurs. The last lift was deposited in August 1999 and the prolonged dissipation of pore pressures since that time can be clearly seen.

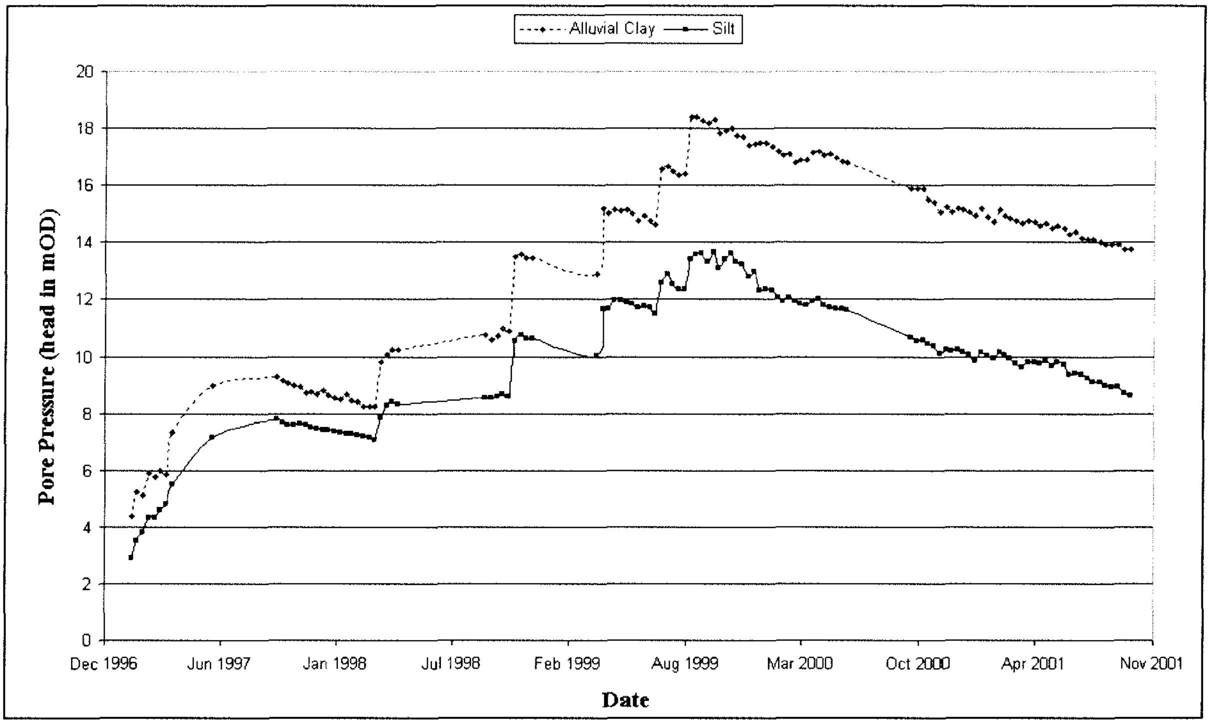

Figure 3.6 Pore pressure changes in Alluvial Clay and Silt at Rainham Phase 3, [Data from Cleanaway].

However, in areas of the landfill where (a) tipping has ceased, (b) total vertical stresses from the waste overburden are low and (c) pore pressures have had time to dissipate, the pore pressures in the waste may be greater than those in the silt and clay. Thus creating the potential for the downward migration of leachate [Cleanaway 2001a]. At some point in the future these conditions may also become established under the central Phase 2 area. To reduce any future hydraulic gradient that would promote the downward migration of leachate, the Environment Agency have stipulated that leachate heads must be reduced to 2m above the silt/waste interface. In some areas of Phase 2 leachate heads are currently 14m above the interface. This has provided the incentive for research into more effective methods of leachate level reduction and control.

3.1.3 Selection of Site for Field Trials

Field trials were conducted at Rainham Landfill Site for the reasons listed below.

1) The site has elevated leachate heads which need to be reduced, and as with many older landfill sites Rainham has no pre-engineered basal drainage system for leachate control.

2) Previous operation of vertical well fields had met with limited success and alternatives were required.
3) Other forms of 'horizontal drainage' such as excavated tyre trenches*, fin drains and deep trenches†, had proved successful at Rainham but these methods were not suitable for Phase 2 due to the depth of waste that had already been deposited.
4) Rainham is substantially a 'landraise' rather than a landfill and this would permit a more simple horizontal drilling profile.
5) As the site remains active, machinery, plant and other equipment were available to assist with site works.
6) The site operators, Cleanaway, made many resources available including their laboratories, databases and historical files. Cleanaway also offered assistance with a range of various practical aspects including construction works, health and safety planning and data collection.

3.2 Site Investigation

3.2.1 Aims of Site Investigation

The principal aim of the project was to investigate the effectiveness of horizontal wells in landfill. The proposed method of investigation was to install and test two full scale horizontal wells through the waste. These would be continuous across Phase 2 (i.e. having two surfacing ends), being 550m and 400m in length. Before attempting to install the horizontal wells a site investigation was conducted. The aims of the site investigation were threefold.

1) To produce a detailed description of the waste composition and properties in order to establish the conditions directional drilling would encounter.
2) To obtain information that would assist with the design specifications for the wells including;
   a. well screen material (steel, HDPE, PVC, GRP‡),
   b. perforation diameter (or slot width) from particle size distribution analyses,

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* Tyre Trenches - trenches were excavated in shallow areas of Phase 3 and backfilled with tyres to provide a flow pathway.

† Deep Trenches - a large specialist trenching machine was used to install horizontal drains and gravel backfill up to 8m deep through an area of pre-1973 industrial waste.

‡ GRP – Glass Reinforced Plastic

c. well length and diameter, and
d. borehole route, with particular reference to the position of the site base.

3) To assist with the interpretation of results obtained from test pumping the horizontal wells once installed.

3.2.2 Desk Study

For the desk study, many reports of previous projects and investigations at Rainham were examined. Much of the historical and geological information in section 3.1 was gathered from these sources. Significant findings included:

1) The existing landfill is licensed to accept solid household, commercial and non-industrial wastes.
2) Between 1968 and 1979, Phase 2 was filled with predominantly pulverised domestic waste to approximately 8.8 to 9mOD [Cleanaway 2001c]. This marked the final level of the original planning application. Between 1988 and 1997, filling continued with an estimated 2 million tons of (non-pulverised) waste being deposited [Cleanaway, 1997]. Two final stages of fill occurred in 1998 & 1999, the latter after the site investigation was complete, bringing the landfill to 31.5mOD at the highest point.
3) As the waste deposited during the first stage of tipping was pulverised, the number of large items within the layer that may obstruct directional drilling, should be limited.
4) As at most landfills, tipping at Rainham occurred in layers or 'lifts'. Waste was deposited at the tipping face and compressed using compaction plant weighing up to 54 tons (Figure 3.8). During the filling of Phase 2, waste was deposited at a rate of approximately 1,400tons/day and the tipping face advanced across the phase in layers 3m in height, Figure 3.7, Figure 3.9 & Figure 3.10. At the end of each day a thin layer of inert material such as soil or silt (daily cover) was spread over the fresh waste to limit vermin and odour problems. This layering may have important implications for the anisotropic hydraulic conductivity discussed in section 6.8.5. However, the daily cover does not form an unbroken layer over the waste as can be seen from Figure 3.11.

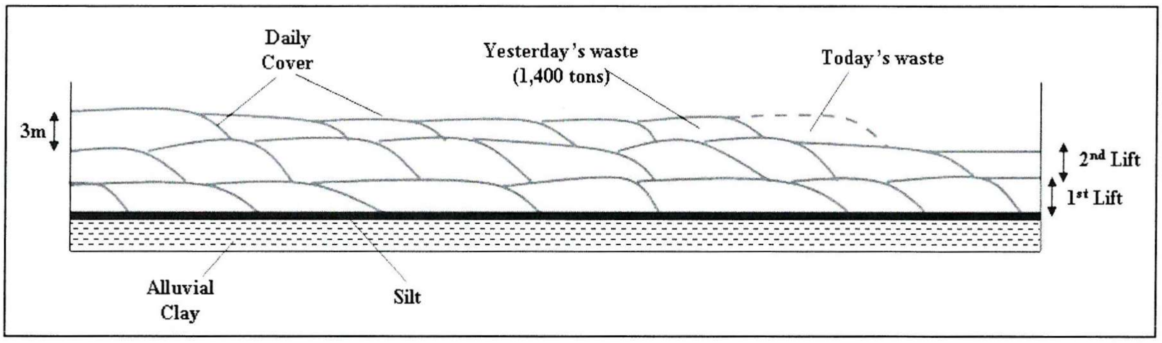


Figure 3.7 Schematic cross section of Rainham Phase 2, showing how waste was deposited in cells each covered with inert cover material at the end of each day. The cells progressed across the site in 'lifts'.

Figure 3.8 Landfill Compactor weighing 54 tons [Rainham, 1998]

Figure 3.9 Tipping in layers. The most recent layer advances across the phase [Rainham, 1998].

Figure 3.10 The tipping face is advanced over the previous layer of daily cover [Rainham, 2000].

Figure 3.11 The layer of daily cover is rarely complete and unbroken but will have significant effects on the permeability of the waste [Rainham, 2000].

5) Many of the vertical wells installed for leachate control terminate within 0.5m of the basal silt layer. In many cases the underlying silt migrated into the wells possibly as a result of the creation of large hydraulic gradients due to the rapid reduction in well leachate level when pumping commenced. This silt migration has caused repeated blockages and caused significant delays to site-wide leachate extraction efforts [Cleanaway 2001c]. Ensuring the horizontal wells were kept clear of the silt/waste interface was therefore considered to be of high importance.

The desk study was a valuable exercise but gave little information on the properties and composition of the waste in-situ, what changes it had undergone since deposition and how its properties might affect the drilling process and subsequent operation of the wells. It was therefore decided that the site investigation should be expanded to include the drilling of a series of vertical boreholes which would be accurately logged.

3.2.3 Need for a Descriptive Standard

Before drilling of the investigation boreholes commenced, a system for describing the recovered waste samples was required. Although substantial research has been carried out on the characterisation of waste streams disposed to landfill, there was no definitive standard or protocol to provide either

- a framework for the accurate and consistent description of degraded, co-mixed waste samples arising during the site investigation of landfills, or
- a materials classification for degraded, co-mixed waste materials.

Indeed, most existing drilling logs from Rainham described the recovered materials using only vague terms such a 'municipal solid waste' or 'made ground', with occasional reference to specific materials such as paper or plastic. Other researchers have taken samples from boreholes drilled in waste and attempted a limited description but have failed to reference other studies [Gabar & Valero 1995, Townsend *et al* 1996 and Zornberg *et al* 1999]. Boland *et al*, (1998) reported the heterogeneity of waste samples arising from five boreholes in a MSW landfill. Of 102 samples collected from the boreholes no two samples were considered the same but had nevertheless been sorted into 24 material categories. However, no detailed descriptions were given and the method of classification was not clearly described.

To design landfill engineering works that are safe and economical it is vital to understand the physical properties of the waste materials that will be encountered. For example, one UK waste management company, Shanks, have used drilling logs combined with geophysical resistivity surveys to identify the location, shape and composition of unsaturated zones within the waste at Brogborough Landfill Site in Bedfordshire. Recirculation of leachate into these dry zones has reduced leachate treatment costs and promoted more rapid decomposition of the landfill [Painter, 2000].

The lack of a standard makes it difficult if not impossible to compare or relate experimental results from different researchers. It was clear that a definitive standard was required. The site investigation for the horizontal wells provided an ideal opportunity to produce and test such a standard.

3.2.4 Derivation of a Standard

For logging and analysis of the waste, core samples would be required. The continuous flight auger drilling technique would therefore be unsuitable and so the barrel auger drilling technique was selected. For practical purposes it was decided that each core brought to the surface would be logged as a separate sample. If significant variations occurred over the depth of one sample this would be noted and logged accordingly.

A checklist was drawn up of all the parameters that could be logged whilst on site. To ensure results were comparable, the checklist included written procedures for obtaining measurements such as moisture content. The parameters included in the checklist are listed below. A full explanation of each is given in Appendix A.

1) Location
2) Depth
3) Temperature
4) Moisture Content
    (a) by observation
    (b) by oven drying
5) Density
6) Age
7) Water Strikes and Water (Leachate) Level
8) Waste Description
    (a) the *Main Contents*
    (b) the *Matrix**

* The term 'matrix' is defined in section 3.3.5.

9) Degree of Decomposition
10) Photographs

The checklist was first used during the installation of four vertical leachate extraction wells at Cleanaway's Pluckley Landfill Site in Kent. This was intended as a trial run of the system before it was used for the Rainham site investigation. A number of minor revisions and one key revision was made before use at Rainham. The key revision was to replace the *checklist* with a *logging sheet.* A sample logging sheet is included as Appendix B. The sheet provided space to record observations by completing the blanks or by ticking boxes, which speeded up the process of logging and simplified the subsequent collation of data. One logging sheet was used for each waste core. In addition, the trial run at Pluckley allowed sampling techniques to be refined, for example, the use of only matrix material for laboratory determination of moisture content.

Following further refinement of the system after use at Rainham and other landfill sites, the proposals for a descriptive standard were published in October 2001 - Board *et al*, 2001. The paper is attached as Appendix C. In addition to the classification of waste, the paper discusses the use of geophysical resistivity surveys to identify wet and dry zones within the waste where leachate can be drawn from and recycled to.

3.3 Site Investigation Boreholes

3.3.1 General Information

Following completion of the four vertical wells at Pluckley the barrel auger rig, Figure 3.12, was relocated to Rainham in October 1998 to drill the site investigation boreholes. The intention was to drill twelve equally spaced boreholes along the proposed route of each of the two horizontal wells. However, the rotational speed of the rig proved to be inadequate and progress was slow. In the lower, more saturated horizons the boreholes repeatedly collapsed and significant volumes of leachate and fines flowed into the void.

Cleanaway have also noted the difficulty of drilling boreholes to the silt/waste interface without sloughing of the borehole walls [Cleanaway 2001c]. The repeated borehole collapse had an adverse effect on the accuracy and usefulness of the data obtained through logging. In particular, density calculations were affected as the assumption that barrel diameter was equal to borehole diameter became largely invalid.

Figure 3.12 Barrel auger drilling rig used for site investigation boreholes. Platform scales for measuring sample weight can be seen in the foreground.

It was intended that each borehole would be drilled to the silt/waste interface. The crossing of this interface was indicated by a sudden fall of the drill stem due to the lower resistance from the silt. A sample of silt in the barrel auger confirmed that the interface had been reached. Not all boreholes reached the base of the site. In the first borehole an impassable obstruction thought to be concrete was encountered at 7.2mOD, and in the seventh and eighth boreholes continual wall collapse in the saturated horizons prevented completion. The location of the site investigation boreholes, numbered H01 to H08, are marked on the plan in Appendix E.

After six weeks of drilling, eight boreholes had been attempted and only five had reached the silt/waste interface. As a result, it was decided that this stage of the site investigation should be abandoned. As 162 samples had already been recovered it was considered that a fair representation of the waste composition of Phase 2 had been gathered. All eight boreholes were drilled in the north-eastern area of Phase 2 along the path of one of the two proposed wells (later to become HW1). A number of vertical wells already existed in the vicinity of the second proposed horizontal well (the western end of Phase 2) and consultation of borehole logs revealed sufficient information on the position of the silt/waste interface in that area.

Piezometers were installed in six of the eight site investigation boreholes (numbered H03 to H08) to gather background information on pore pressures.

3.3.2 Position of Silt/Waste Interface

Locating the silt/waste interface was perhaps the most important aspect of the site investigation as installation of the horizontal wells in or through the silt would firstly compromise the integrity of the site containment layer and secondly impede the performance of the horizontal wells. However, it was also important to install the wells close to the base to maximise potential leachate head reductions.

From the site investigation boreholes, the interface between the waste and the silt appeared to be located between 0.5 and 1mOD. In one instance the interface was recorded at 1.8mOD, although this appeared to be an anomaly. Cleanaway borehole logs from vertical wells drilled at the western end of Phase 2 located the silt/waste interface at 0 to 1.5mOD in that area.

It was therefore decided that the horizontal wells should be installed no lower than 1.5mOD, and to allow room for error the target height of the horizontal sections was set at 2mOD.

3.3.3 Temperature

Temperature profiles were taken from only four of the eight site investigation boreholes as instrument failure prevented the taking of readings from the remaining four. The profiles are shown in Figure 3.13 to Figure 3.16. A general description follows, referring to the letters (a.) to (d.) on the charts.

a. In the upper 1 to 3m, temperatures remain low at around 15 to 30°C. Little heat is produced in these layers as decomposition may still be in the early stages. In addition heat may be lost to the landfill surface.

b. Over the next 2 to 5m, temperatures increase from 30 to 35°C. The waste at these depths has been buried for a greater length of time and so the accelerating rate of decomposition may give rise to the additional heat.

c. Higher temperatures (35 to 50°C) are sustained over the middle horizons of the landfill where decomposition is most intense. Individual temperature readings from these horizons reached 60°C in some cases.

d. The waste in the lower horizons, particularly that dating to 1975, may have passed the peak rate of decomposition and as such the temperatures gradually decline from 50 to 21°C. Some heat may also be transferred away through the site base.

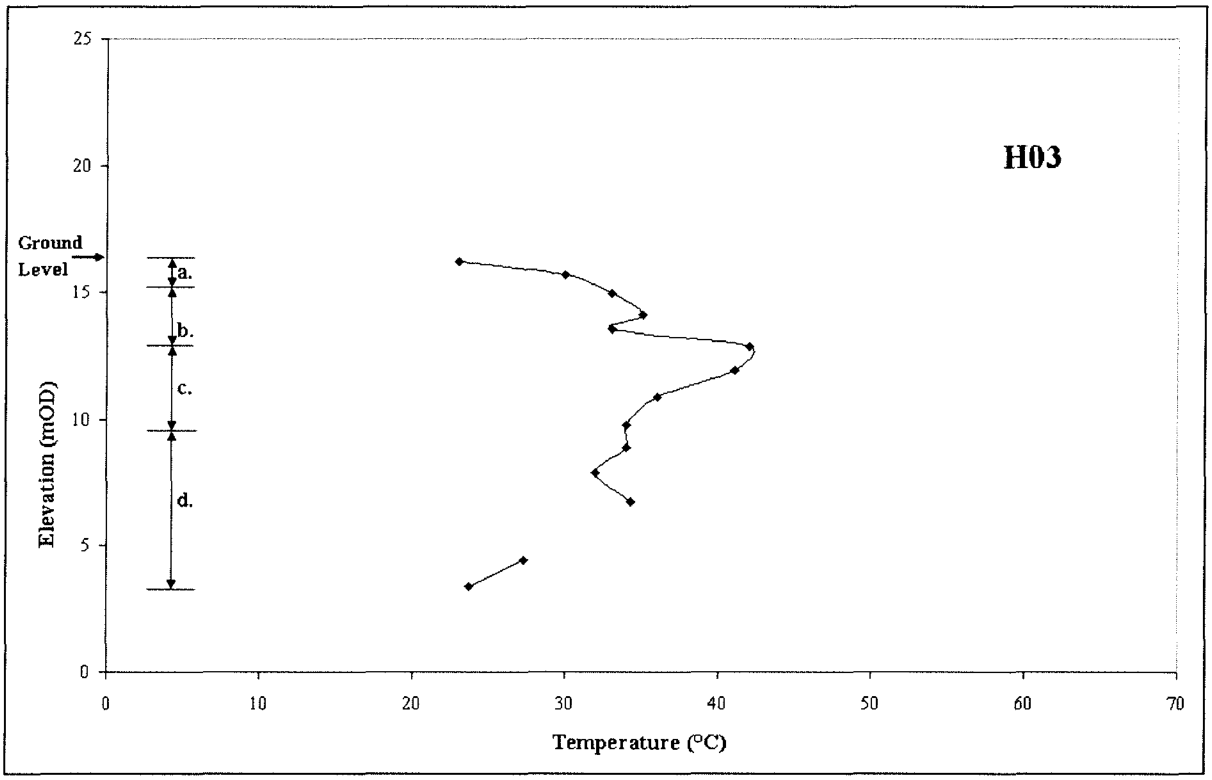


Figure 3.13 Temperature profile of borehole H03.

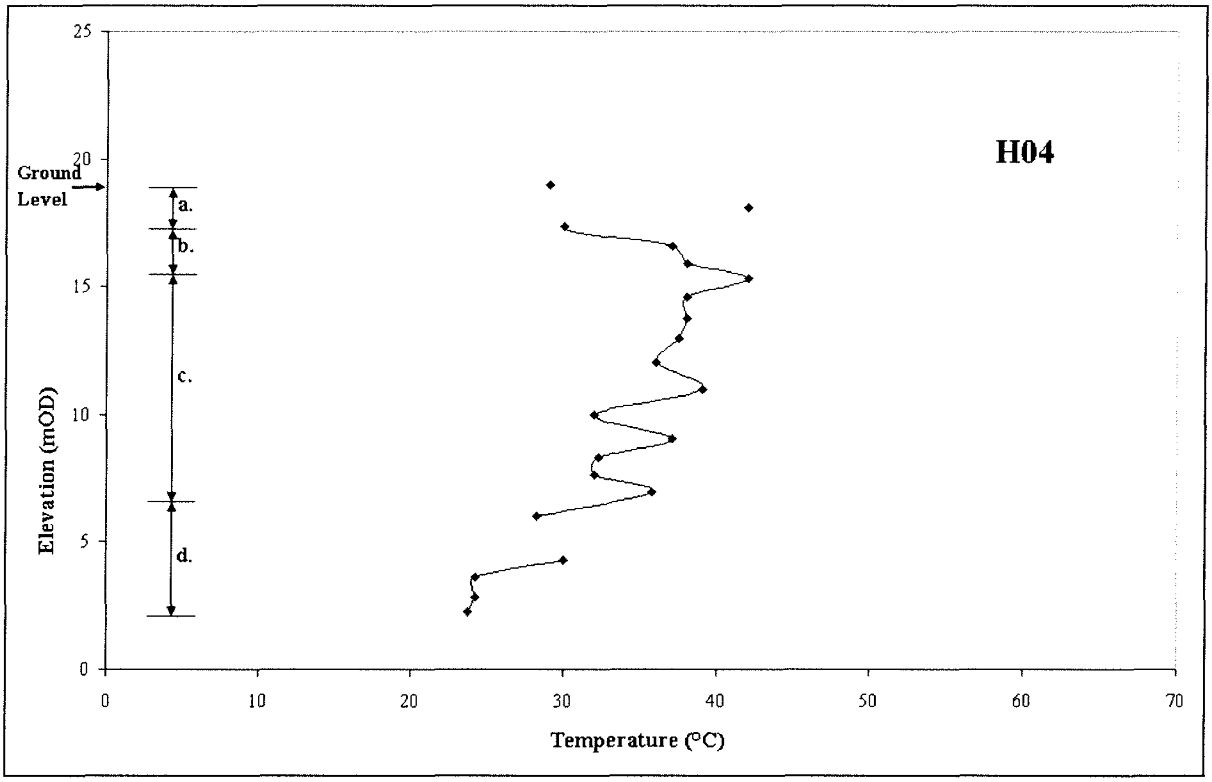


Figure 3.14 Temperature profile of borehole H04.

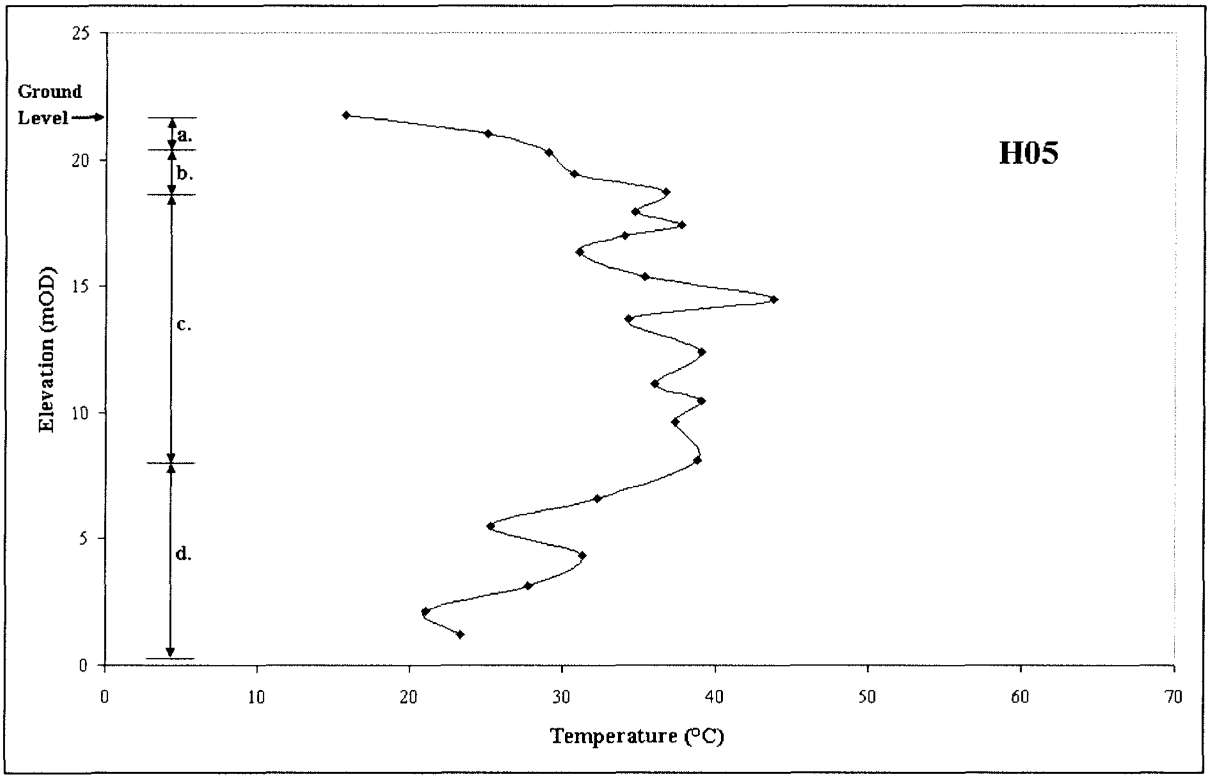


Figure 3.15 Temperature profile of borehole H05.

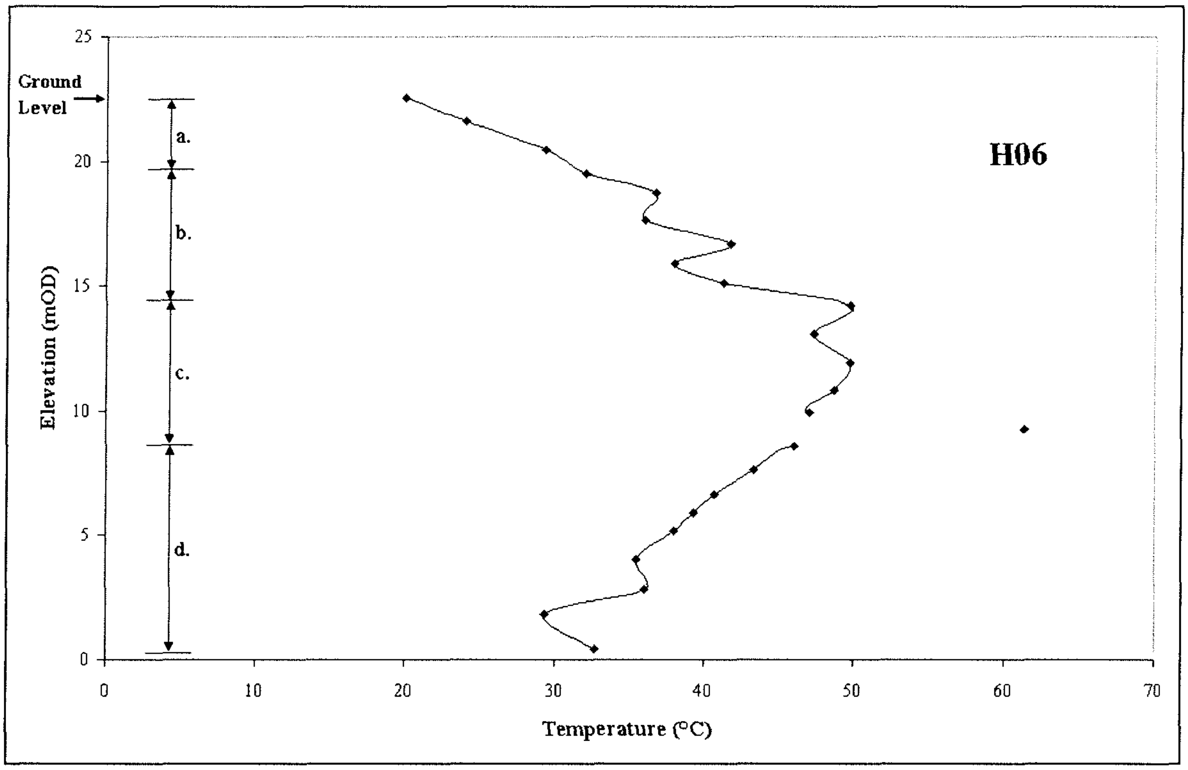


Figure 3.16 Temperature profile of borehole H06.

The main section of the proposed horizontal wells would be installed in waste with a temperature of 20 to 30°C, whilst the entry and exit sections may encounter temperatures of 50°C or even 60°C. The effects of these elevated temperatures on drilling equipment and drilling fluids (such as fluid viscosity) was identified as an issue requiring further investigation.

3.3.4 Age of Waste & Degree of Decomposition

In 32% of the recovered waste samples it was possible to determine the approximate date of deposition from newspapers and other items within the waste. Using these data, Figure 3.17 shows how Phase 2 had been built up from six discrete stages of waste deposition. During time periods not covered on the graph, tipping of waste was occurring on different areas of the landfill site.

A seventh stage of waste deposition took place in the Spring/Summer of 1999, after the site investigation had been completed. The seventh stage raised the landfill to 31.5mOD.

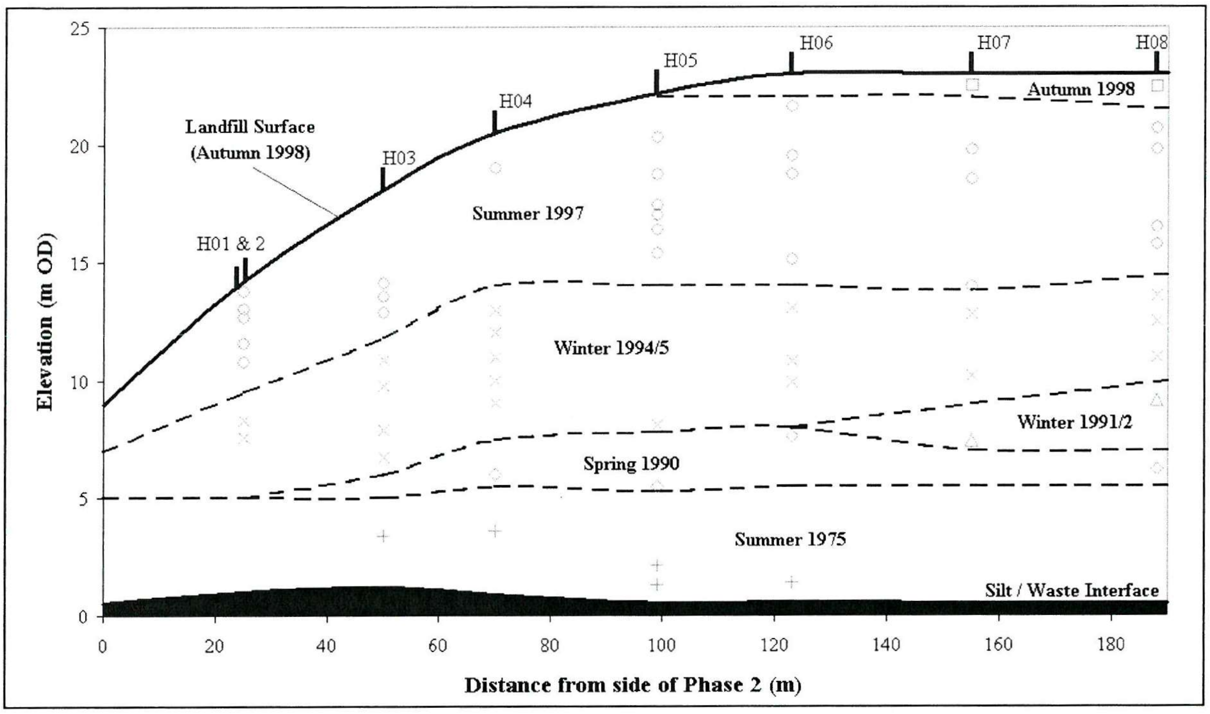


Figure 3.17 Stages of waste deposition in cross section through the site investigation boreholes.

Using the 5 point scale outlined in Section 3.2.4, the degree of decomposition was observed generally to increase with depth in each of the site investigation boreholes although the variability between successive waste cores was large (as much as 3.5 points). Figure 3.18 compares the age of each waste core with the degree of decomposition. As expected, there is a clear progression in the degree of decomposition with increasing age between the 1998 waste and the 1990 waste. However, there is little change between the waste from 1990 and 1975 suggesting that most decomposition of the 1990 waste had already occurred.

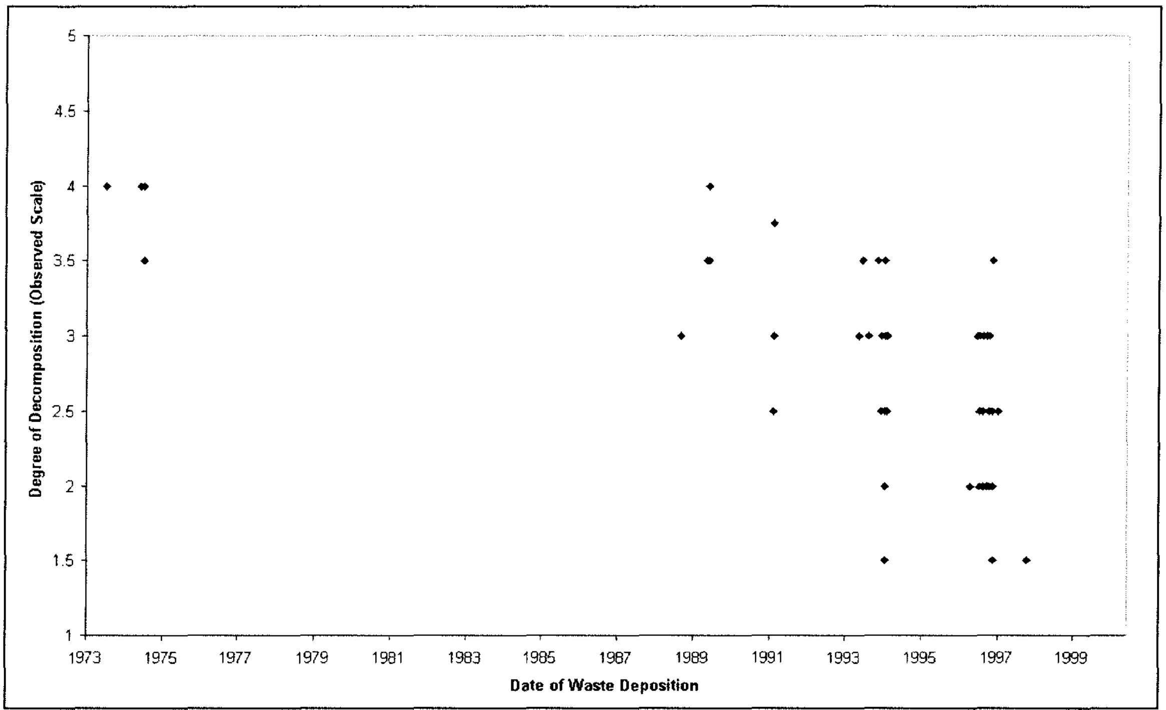


Figure 3.18 Degree of Decomposition (qualitative) plotted against Age of Waste.

3.3.5 Composition of the Waste

As expected the composition of the 162 recovered waste cores was extremely heterogeneous, with no two cores containing the same materials in similar proportions. The following three photographs demonstrate the heterogeneity of the waste samples, Figure 3.19 to Figure 3.21.

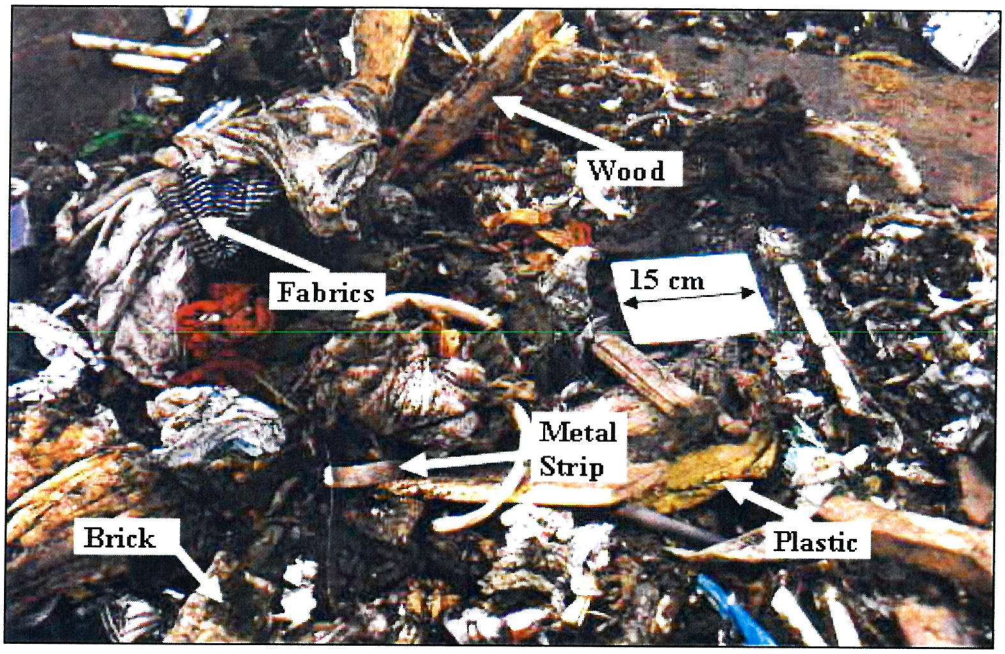


Figure 3.19 Waste in unsaturated horizons easy to identify, small percentage of matrix material, very heterogeneous in composition, retains original colours.

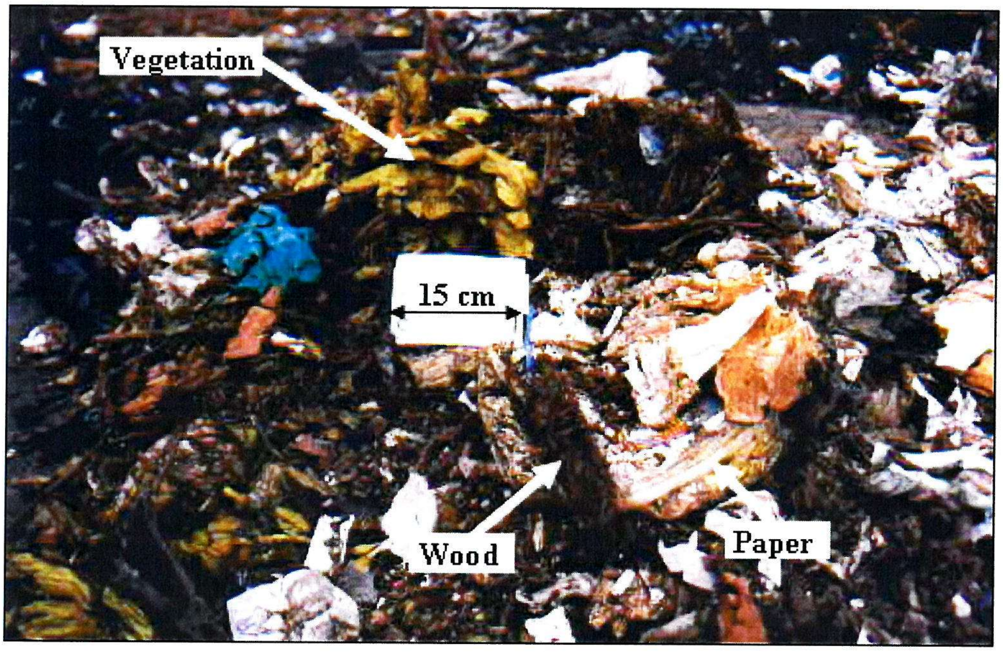


Figure 3.20 Heterogeneity of Waste (unsaturated horizon).

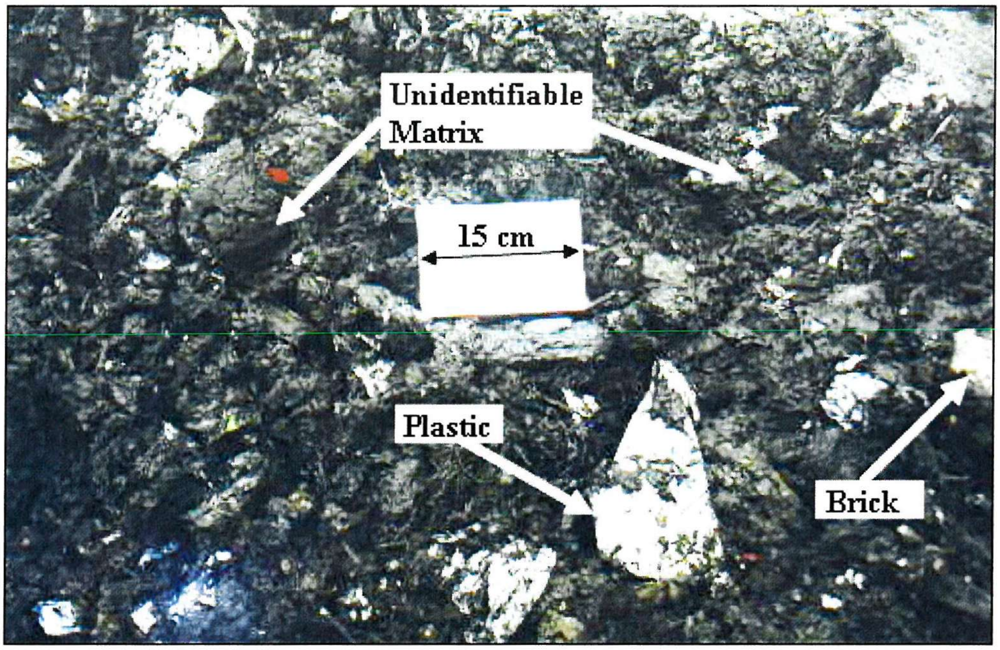


Figure 3.21 Waste in saturated horizons difficult to identify, high percentage of matrix material, more uniform in composition and colour.

For ease of description the waste was divided into two principal components, the *main contents* and the *matrix.*

Main Contents

Plastic

Plastic was present in 88% of waste samples due to a combination of its widespread use and slow rate of decomposition. The vast majority of the plastic appeared to originate from carrier bags, bin liners and the like. Although the resulting thin layers of plastic were not continuous over large areas they may cause a significant reduction to the hydraulic conductivity of the waste by making flow paths considerably more tortuous. More substantial plastic pieces, such as 20mm thick HDPE, were found in only two cores. As the barrel auger had cut through these pieces it was not possible to determine their lateral extent. However, the rarity of such items meant that plastic was not considered to be a potential impediment to the directional drilling process.

Paper

As with plastic, paper was present in large quantities in the majority of recovered cores. In the lowest horizons dating to 1975 paper was found less frequently as a result of decomposition. Paper was not regarded as a potential problem for directional drilling.

Wood

Wood was found in most cores although the amount varied widely as did the size of individual pieces. Some larger pieces of wood were recovered and were thought to be pieces of railway sleepers. Three of these pieces were present in the older, supposedly pulverised waste through which the main section of directional drilling would occur. The progress of the site investigation rig was slowed by these larger wooden pieces and it was considered they might also slow the progress of the directional drilling. Figure 3.22 shows large pieces of wood found on the landfill surface, believed to have similar dimensions to the wood recovered from the older waste.

Figure 3.22 Large wooden blocks are present at all depths within the waste [Rainham 1998].

Vegetation

Vegetation (other than wood) was recorded to indicate how advanced decomposition was. Vegetation occurred in 35% of the samples from the upper 10m of waste while the 1975 waste did not contain any. Vegetation consisted mostly of garden waste such as grass cuttings and leaves.

Glass

Glass was found throughout the landfill in 85% of waste samples. As glass is brittle, fragments were rarely greater than 2cm in diameter.

Rubber

Of the nine material types recorded, large rubber items were the least common, occurring in only 9% of samples. In most cases the rubber originated from a vehicle tyre. Tyres, with their radial wire reinforcement, can withstand high tensile loads and may get twisted around the drilling tools during directional drilling. Indeed, this caused a significant problem during the Metallic Tile field trial discussed in section 4.4. However, no tyres were found in the 1975 waste where the main sections of the horizontal wells were to be installed.

Fabrics

Fabrics occurred at all levels within 85% of boreholes but were notably less frequent in the lower waste (34%). This material type included a broad range of sizes, from rags to old carpets. Fabrics have significant tensile strength and may become wound around the drilling tools impeding the cutting action and restricting steering. Fabrics were found wrapped around the drilling assembly of the site investigation rig on a number of occasions.

Hardcore / Brick

Fragments of brick, stone and concrete were found in 93% of waste cores. Particle sizes ranged from less than 2mm (sand) to complete bricks. In 4% of samples larger pieces were recovered, the largest a 250mm thick piece of concrete, Figure 3.23. The auger eventually cut through this item after three hours. The first site investigation well (H01) was abandoned at 7.2mOD after an obstruction, believed to be concrete, prevented further progress. Such large pieces were rare and were not found in the lower waste horizons below 5.5mOD.

Figure 3.23 A 250mm thick piece of concrete recovered from H06. The edges have been cut by the barrel auger. The white card is 15cm across.

Metal

Although cans and similarly small metal items were common, larger pieces of metal which might obstruct directional drilling were rare. The most notable finds were three 6.5mm thick steel plates, possibly former manhole covers, each greater than the diameter of the barrel auger. The steel plates delayed drilling and caused considerable wear on the drill bit. No large metal items were found in the lower horizons. Prior to the drilling of these investigation boreholes, steel reinforcing bar (as found in concrete) had been highlighted as a potential obstruction to directional drilling, due to its high tensile strength and its ability to twist around the drilling assembly. However, no reinforcing bar was discovered during the investigation. Although its presence cannot be ruled out it is perhaps a lesser threat to directional drilling than first thought. Other large metal items such as whole vehicles and washing machines (Figure 1.3) were identified as potential obstructions to drilling. Although none were encountered during drilling of the site investigation boreholes, such items were observed lying (part buried) around the site.

Matrix

The material surrounding the main contents varied widely in its constituents and properties. By considering all the investigation boreholes together, patterns of how the matrix changed with depth could be observed.

Unsaturated Horizons

In 70% of the samples recovered from the unsaturated horizons the matrix constituted 25 to 50% of the total volume of each core. The matrix was soft, crumbly, very dry and fine enough to blow away as dust. The often fibrous nature suggested organic origins.

Saturated Horizons

The properties of the matrix underwent a marked change as the saturated horizons were reached. The leachate caused the particles to amalgamate to form a highly viscous slurry, black or grey in colour, very soft and often sticky. In 66% of samples the matrix constituted 50 to 75% of the total volume as a result of (a) the increased rate of decomposition due to the presence of water, and (b) the greater age of the waste at depth.

The matrix in the lowest horizons, dating from 1975, was characterised by fine and medium gravel particles (2 to 20mm) of angular stone and glass fragments. This gravel comprised 10 to 20% of the matrix volume. The matrix slurry from the 1975 horizon had an iridescent sheen and a strong organic/oily odour.

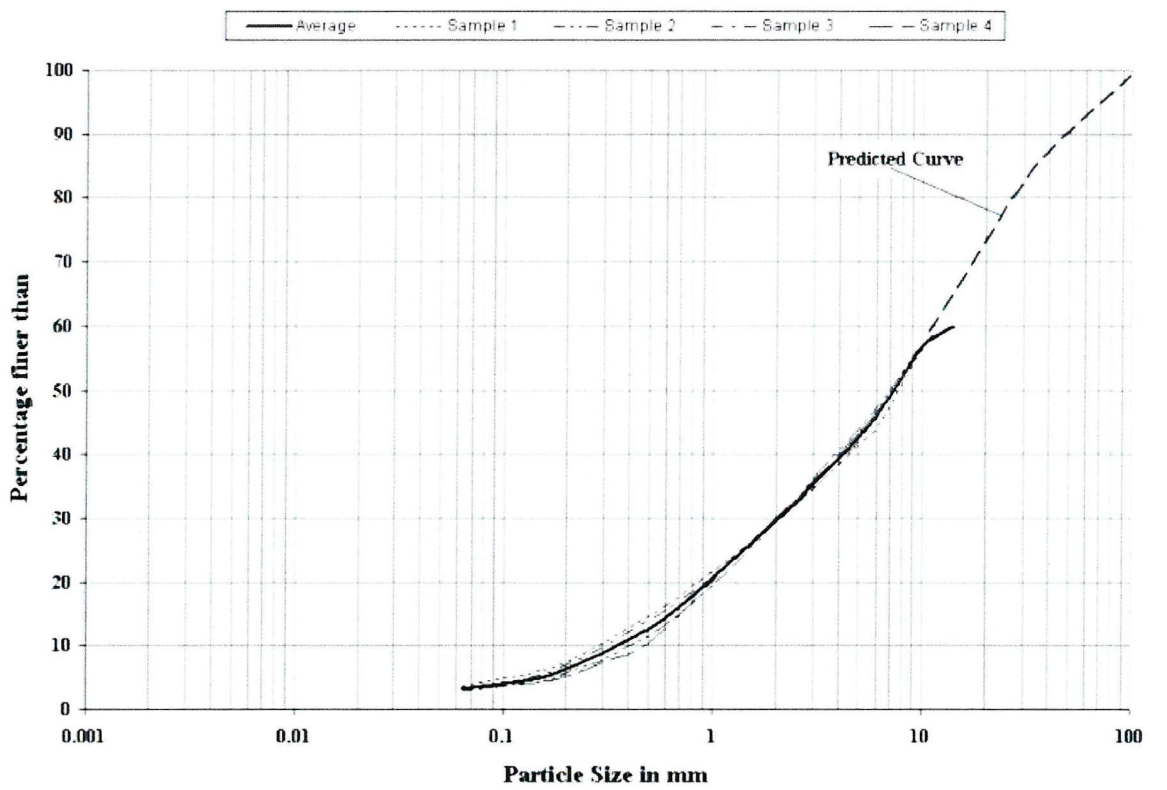
Soil, or at least material that had decomposed to resemble soil, was found in all core samples. In 16% of samples firm, cohesive blocks of clay and silt were also present. These materials were thought to originate from daily cover. The low hydraulic conductivity of the silt and clay has obvious connotations for the overall hydraulic conductivity, however, only in 3% of samples did the clay/silt materials form a continuous band across the 300mm core. Although daily cover is spread in layers the physical action of tipping waste the following day and the effects of differential settlement during decomposition, seem to have caused these layers to break up.

3.3.6 Particle Size Distribution Analysis

The efficiency with which the well screen allows leachate to drain freely from the waste will depend largely on the PSD (particle size distribution) of the waste relative to the well screen slot width. To assist with the selection of the slot width, PSD analyses were carried out on samples taken from the 1975 waste as this was the material within which the screened sections of the horizontal wells would have to operate. The large size and awkward shape of the 'particles' of the *main contents* did not lend themselves to PSD analysis and so only the *matrix* material was used. This was separated out using a 13mm sieve and four 2kg samples were sent to RSA Geotechnics for analysis using the wet sieve method. The fraction larger than 13mm was accidentally returned to the landfill before it could be sorted and weighed and so the full PSD analyses could not be completed. However, the fraction less than 13mm was estimated to be 60% of the total weight. Each of the four samples were analysed separately. The results are presented in Table 3.1 and Figure 3.24.

Table 3.1 Particle Size Distribution of 1975 Rainham Waste Matrix (less than 13mm).

	Size (µm)						**Size (mm)**						
Sieve Size	63	150	212	300	425	600	1.18	2	3.35	5	6.3	10	13
	Percentage by mass passing through sieve												
Sample 1	3.6	6	7.8	10.2	13.2	16.2	23.4	29.4	36.0	41.4	45.0	56.4	60
Sample 2	3.0	4.2	6.0	7.8	10.2	13.2	21.6	29.4	37.8	43.8	48.0	57.0	60
Sample 3	3.0	5.4	7.2	9.6	12.6	15.6	22.8	29.4	36.0	42.0	46.8	57.0	60
Sample 4	3.0	4.2	5.4	7.2	9.0	12.6	22.8	30.0	37.2	43.2	47.4	56.4	60
Average	**3.2**	**5.0**	**6.6**	**8.7**	**11.3**	**14.4**	**22.7**	**29.6**	**36.8**	**42.6**	**46.8**	**56.7**	**60**



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	SILT			SAND			GRAVEL			

Figure 3.24 Particle Size Distribution curves for 1975 Rainham waste. Only finest 60% analysed.

The proportions by soil type are:

Clay & Silt	3.2%
Sand	26.4%
Fine & Medium Gravel	30.5%
Coarse Gravel & larger	40.0%

In addition to the PSD results the following description of the sample was provided, based on BS 5930 [RSA Geotechnics, 1999];

"Made Ground - Dark grey brown slightly clayey slightly silty fine-coarse humic sand (possibly topsoil) mixed with subangular fine-medium gravel sized fragments of ash, brick, mortar, wood, glass, plastic and metal fragments and occasional pockets of partially decayed organic material."

From the soil type proportions and the RSA description it is clear that the material is well graded and very heterogeneous. These PSD results were applied to the design of the well screen, discussed in section 4.3.3.

3.3.7 Moisture Content

The moisture content of each sample was determined using the techniques described in section 3.2.4. Figure 3.25 shows the moisture content results* (by oven drying) from all samples plotted against depth below the surface.

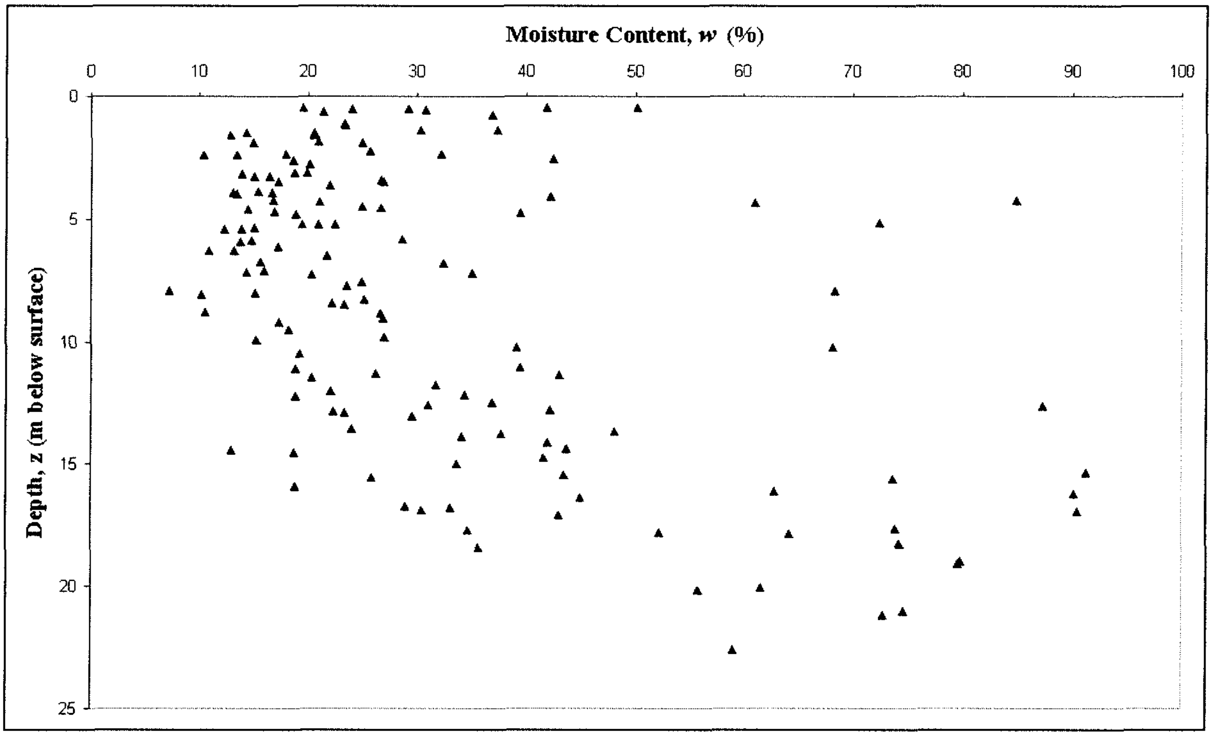


Figure 3.25 Sample moisture content, *w*, with depth, *z*, from site investigation boreholes.

The chart shows a high degree of scatter, the reasons for which include the following.

1. Periodic inflow of free leachate into the borehole particularly in the lower horizons, and thereafter inclusion of this leachate in (or exclusion from) the subsample. The effect of leachate flow into the borehole was exacerbated by the slow rate of drilling.
2. The heterogeneity of the waste meant that some samples may have been unrepresentative.

* Equal to the mass of water divided by the mass of dry solids.

3. Poor testing procedure;
   a. Samples were sealed in plastic bags for transport to the laboratory but were often left for up to four days before testing during which time some drying may have occurred if the seals were not perfect.
   b. Condensation from the waste may have formed in the bag and may have been excluded from the analysis.
4. Actual variations in the moisture content of the waste.

Below the (ill-defined) leachate level, moisture contents were expected to decrease with an increase in depth which was related to an expected decrease in waste porosity [Powrie and Beaven 1999]. However, the chart suggests a general increase in moisture content with depth. This is thought to be a result of free leachate flowing into the borehole and thereafter inclusion in the samples. This was a particular problem in the saturated horizons, hence the higher degree of scatter at depths greater than approximately 15m.

A second set of vertical boreholes were drilled at Rainham in May 2000 using a different drilling rig (section 5.1.1). Each borehole was completed in less than one day and the problems of borehole collapse and ingress of leachate were largely avoided. Samples were taken from eight of these boreholes for moisture content and density analyses. The results, discussed below, were considered to be more accurate than those from the original site investigation. Figure 3.26 shows the moisture content results (by oven drying) plotted against depth.

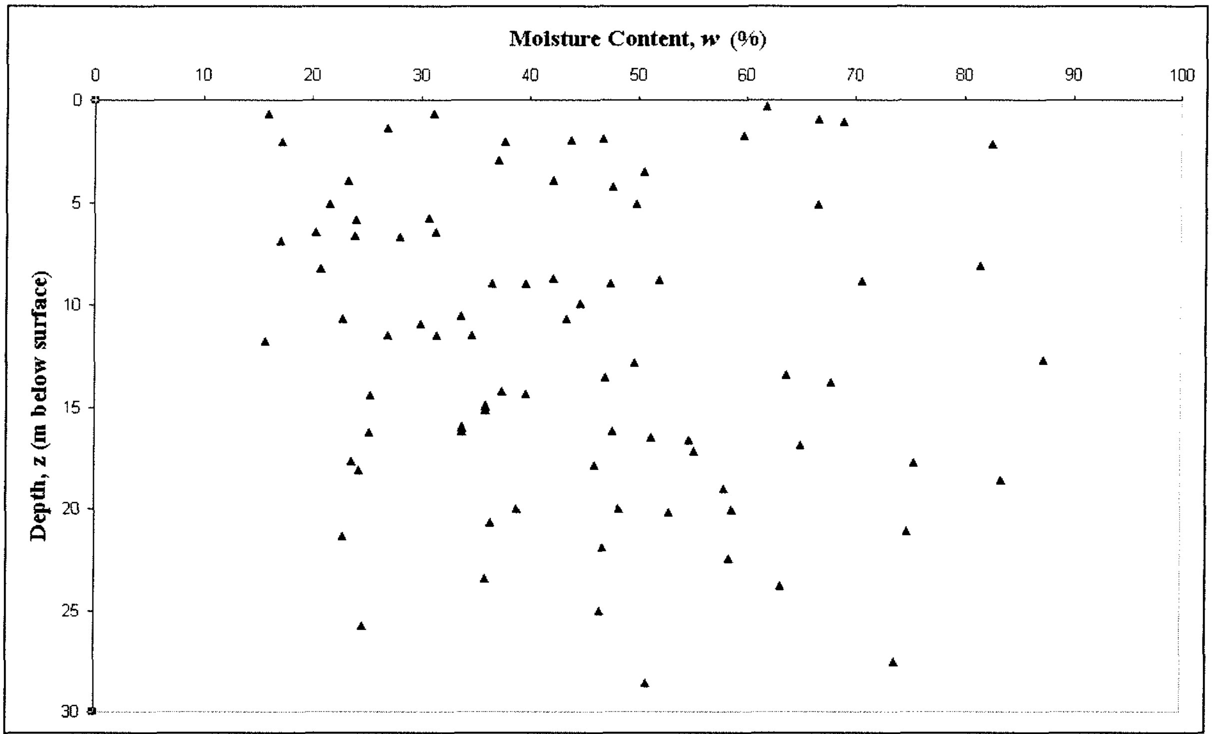


Figure 3.26 Sample moisture content, *w*, with depth, *z*, from May 2000 boreholes.

The average moisture content is 44% although the chart shows a high degree of scatter with no obvious 'leachate level' and therefore no clear division of unsaturated and saturated layers. This is significant, suggesting that moisture is found in many pockets and horizons throughout the landfill.

Within the upper 15m of waste, the moisture content results from the original site investigation boreholes are lower and less scattered than those from the May 2000 boreholes and therefore may appear more reliable. However, the slow rate of drilling during the site investigation reduced the ability to retrieve undisturbed samples. Indeed, it is possible that due to the slow rate of drilling and often high degree of waste compaction into the barrel auger, any free leachate may have been driven from the waste before it could be sampled. Furthermore, (in the lower horizons at least) the sloughing of the borehole walls and ingress of water meant that the site investigation results were regarded as suspect and were rejected in favour of the results from May 2000. It should also be noted that the differences between Figures 3.25 and 3.26 may have been related to (a) the May 2000 boreholes being drilled in three different areas of the site instead of the single area covered during the site investigation, and (b) between the site investigation in Autumn 1998 and May 2000 an additional 7m thick layer of waste had been deposited.

In addition to the quantitative analyses, a visual observation of moisture content was made, w_{obs}, using a 5 point scale (1 = dry, 5 = saturated). Although not as precise as the oven drying method, the w_{obs} value is determined from the entire waste core and does not rely upon a single sub-sample. Figure 3.27 shows these results plotted against depth.

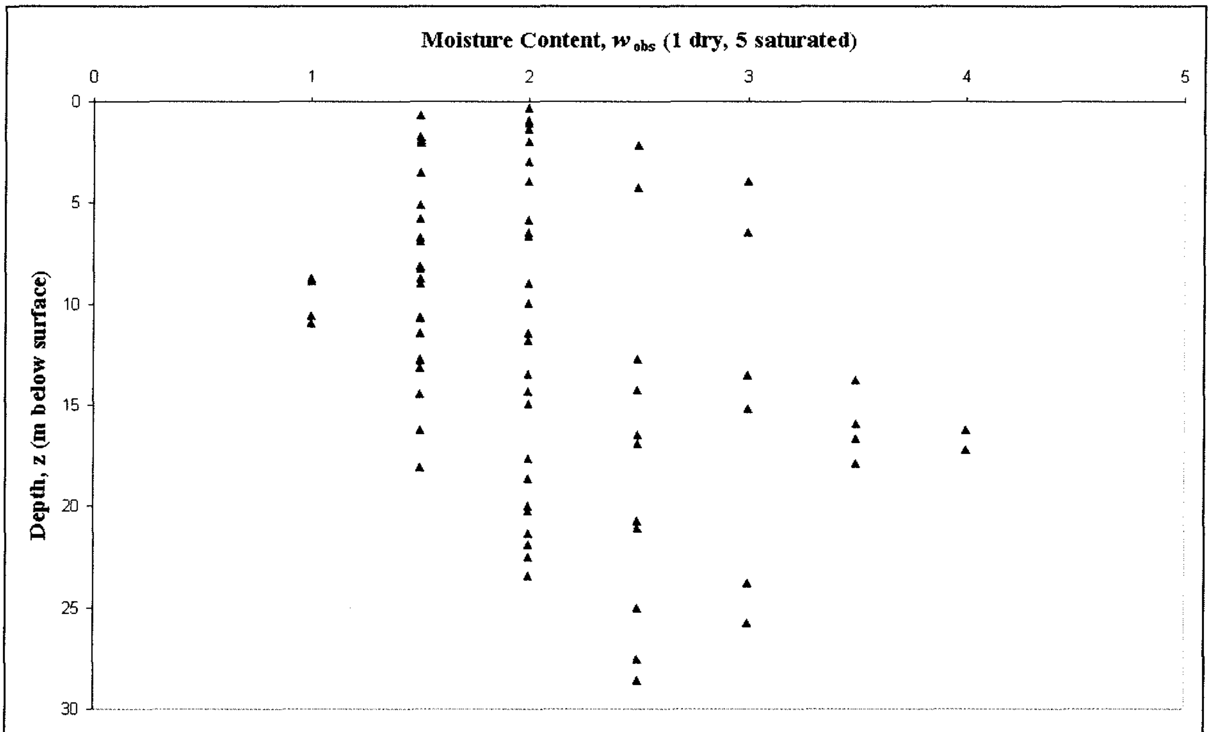


Figure 3.27 Observed moisture content, w_{obs}, with depth, z, from May 2000 boreholes.

The observed values support the measured values in showing a high degree of scatter. Again, there is no clear division of saturated and unsaturated zones nor a clear leachate level.

3.3.8 Waste Density

By weighing each waste core and using the increase in borehole depth to calculate its in situ volume, as described in section 3.2.4, the bulk density of each sample was determined. This was done for both the site investigation boreholes and the boreholes drilled in May 2000. However, the results from the site investigation were poor as borehole collapse seriously affected the accuracy of the volume calculation. The results shown in Figure 3.28 are the calculated bulk density values from the May 2000 boreholes.

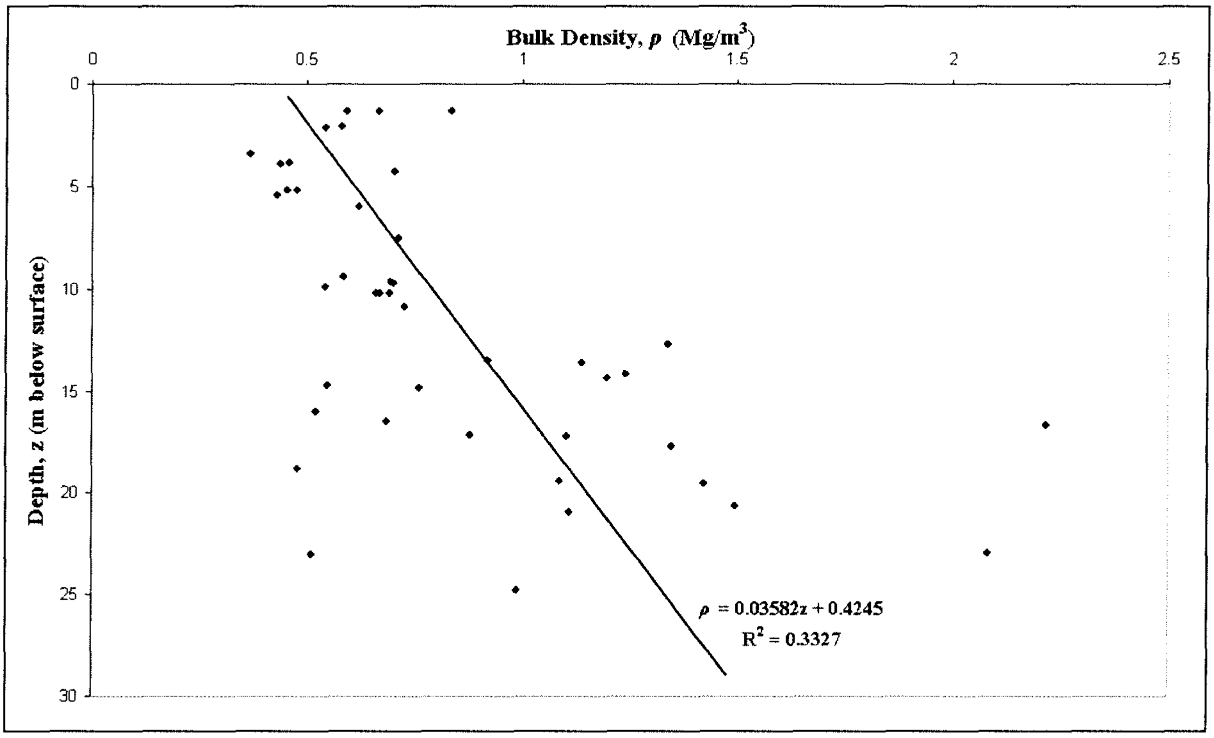


Figure 3.28 Bulk density, ρ, of waste samples with depth, z.

The scatter in these data is possibly a product of the unrefined sampling technique but may be due to actual variations in waste density that are caused by;

a. the heterogeneity of the waste, e.g. a sample containing mostly metal and concrete could be 4 to 10 times more dense than one containing mostly paper and plastic (depending on the moisture content),
b. the loading history, i.e. the effective stress at each point, the duration of exposure to this stress and whether the current stress is the maximum stress that has been applied at each point,
c. densification, i.e. the lighter fractions degrade more readily and so as the waste ages, the particle sizes (and therefore pore spaces) become smaller and density increases.

From the least squares regression line plotted on Figure 3.28, ρ (in Mg/m^3) is found to be related to z (in m) through

$$\rho = 0.03582z + 0.4245 \qquad (3.1)$$

However, the correlation coefficient of the line is low, at 0.33.

Figure 3.29 shows the dry density, ρ_{dry}, calculated through combination of the bulk density, ρ, and the moisture content, w, of the May 2000 waste, using equation (3.2). The dry density is the density that the refuse would have at the same total (gas + liquid) void ratio but zero moisture content.

$$\rho_{dry} = \rho / (1 + w) \tag{3.2}$$

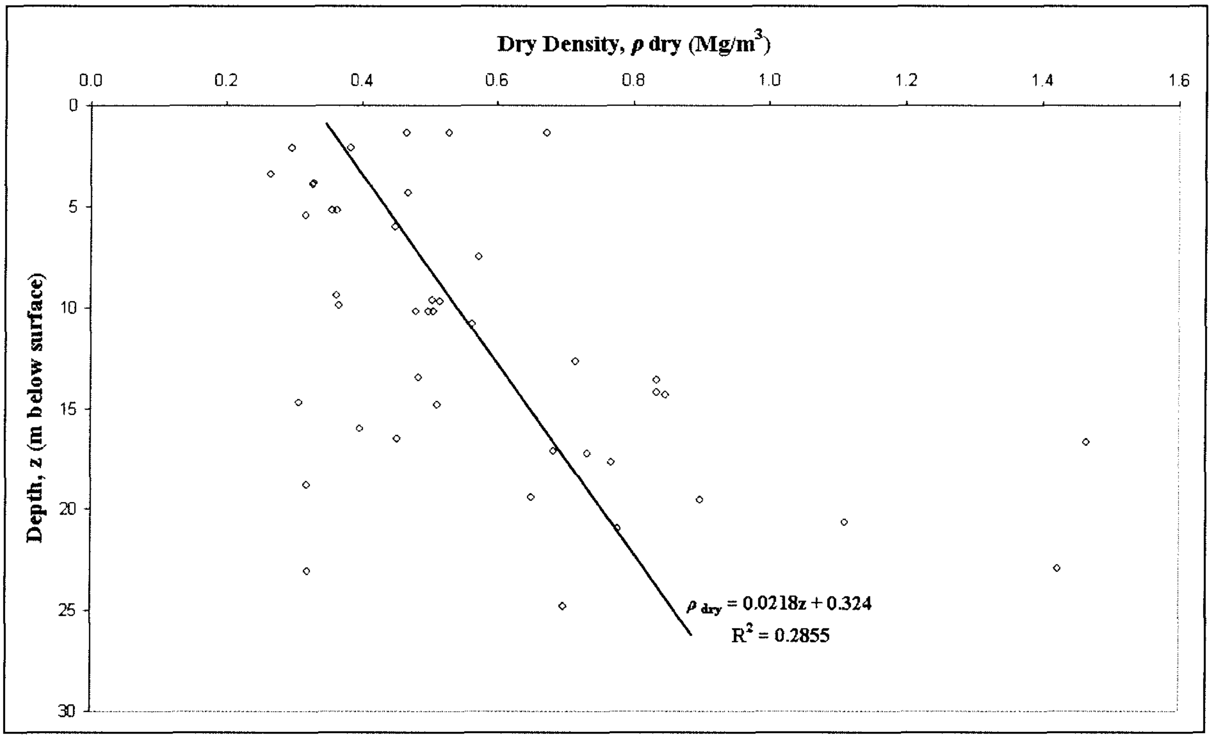


Figure 3.29 Dry density of May 2000 waste samples.

From Figure 3.29, ρ_{dry} is found to be related to z through

$$\rho_{dry} = 0.0218z + 0.324 \tag{3.3}$$

Again, the correlation coefficient is low at 0.29.

From equation (3.1) the values of bulk density range from 0.42 Mg/m^3 at the landfill surface to 1.28 Mg/m^3 at 24m depth. Assuming g = 10m/s^2, these values can be expressed as unit weights; 4.2 to 12.8 kN/m^3. Other reported values are as follows.

Watts & Charles (1999) quote bulk densities for four UK landfills;
Calvert = 8 kN/m^3, Brogborough = 6 kN/m^3, Redditch = 18 kN/m^3, East End = 18 kN/m^3
At Calvert and Brogborough the values refer to the unit weight at the surface following compaction by heavy plant. At Redditch the unit weight values were described as 'typical'

from the 6m deep landfill dating to *c.*1960. At the site in the East End of London the 6.5m deep waste dated to *c.*1935 and again values were described as typical.

Kavazanjian (2001) reports unit weights for two landfills in southern California. The Azusa landfill is 'relatively dry' with moisture contents well below field capacity. The unit weights near the ground surface were between 9 and 10 kN/m^3 but reached 14 kN/m^3 at depth. At the Operating Industries Inc. landfill, unit weights in excess of 20 kN/m^3 were measured. Kavazanjian reported that the highest values corresponded to areas where liquid flowed freely into the borehole and where the waste was heavily degraded. For the Rainham investigations the diameter of the borehole was assumed to be equal to the diameter of the barrel auger; however, where borehole collapse may have occurred this assumption becomes invalid and leads to overestimation of the unit weight. To more accurately determine borehole volume in the California experiments, the volume of the borehole was measured by backfilling with calibrated gravel.

Pestana (2001) states that the unit weight of waste is highly variable and can range from 3.2 to 12 kN/m^3 and that modern, well compacted, landfills and old degraded landfills are typically 10.4 ± 1.6 kN/m^3.

Powrie & Beaven (1999) monitored the change in waste unit weight with increasing effective stress and showed the *saturated* unit weight to range from approximately 0.8 to 1.2 Mg/m^3 between zero effective stress and 600kPa respectively according to equation (3.4). The waste tested had not undergone any major decomposition and the authors suggested that density would increase with age.

$$\rho_{sat} = 0.6691(\sigma'_v)^{0.0899} \tag{3.4}$$

The unit weight values obtained from the May 2000 boreholes, 4.2 to 12.8 kN/m^3, therefore lie within the range suggested in the literature.

The above results show that both the moisture content and the density of the waste is variable. In areas where the waste is dry and of low density then the volume of drilling fluid lost into the waste during installation of the horizontal wells may be considerable. The use of bentonite based fluids is therefore undesirable if the hydraulic conductivity of the waste is to be maintained.

3.3.9 Settlement & Hydraulic Conductivity

Cleanaway (2001a) suggested that if the 1975 waste was saturated when over-tipping commenced then the increase in total vertical stress may have been carried initially by an increase in pore pressure rather than compression of the soil skeleton and a reduction in the size of the pore spaces. Pore pressures would then have decreased over time as consolidation of the waste took place, but could still be above their equilibrium values. However, the age profile in Figure 3.17 shows that the interface between the 1975 and 1990's waste is now at 5.5mOD having settled from 9mOD*, although approximately 2m of this settlement has taken place in the underlying silt, peat and clay [Cleanaway, 2000]. The 1975 waste has therefore settled by 22%, which is in line with the effects of bioconsolidation and self-weight compression of landfills summarised in Watts & Charles (1999). Watts & Charles also state that the settlement of a landfill is generally one-dimensional although the variability of the waste will usually mean that differential settlements are relatively large.

Powrie & Beaven (1999) suggested that aged or degraded wastes might be expected to be of lower hydraulic conductivity than when fresh owing to particle size reduction and density increase. In addition to the clear layering between the 1975 waste and the 1990's waste, the age profile showed a number of layers within the 1990's waste. As each layer has been decomposing for a different period of time and has different loading histories, it is possible that the hydraulic conductivity of each layer will be different.

An estimate of the hydraulic conductivity can be made from the effective stress using the relationship suggested by Powrie and Beaven (1999):

$$k = 2.1(\sigma'_v)^{-2.71} \tag{3.5}$$

where k is the hydraulic conductivity (in m/s) and σ'_v the vertical effective stress (in kPa). The following analyses assumes that the different layers of waste identified at Rainham have uniform properties of hydraulic conductivity which change only with effective stress. In addition, the waste is assumed to be saturated. Powrie and Beaven also suggest a 'worst case' relationship based on the lower hydraulic conductivity values obtained in their tests:

$$k = 17(\sigma'_v)^{-3.26} \tag{3.6}$$

* With the addition of a further 8.5m of waste to the surface of Phase 2 after the site investigation was complete the interface may have settled further.

To complete these analyses it is first necessary to find σ'_v, this is obtained from Terzaghi's equation as follows:

$$\sigma'_v = \sigma_v - u \tag{3.7}$$

where σ_v is the total vertical stress and u is the pore pressure.
u is equal to:

$$u = h \times \gamma_{leachate} \tag{3.8}$$

where h is the depth below the leachate level and $\gamma_{leachate}$ is the unit weight of leachate and is approximately equal to 10 kN/m^3.
σ_v is equal to:

$$\sigma_v = \int_0^z \gamma \, \delta z \tag{3.9}$$

where z is the depth of overburden and γ is the unit weight.

Equation (3.1) gives the relationship between bulk density, ρ, and depth, z. Converting (3.1) into a relationship between γ and z, (assuming g = 10m/s^2) gives

$$\gamma = 0.3582z + 4.2447 \tag{3.10}$$

Integrating (3.10) with respect to z gives;

$$\sigma_v = \int_0^z 0.3582z + 4.2447 \, \delta z \tag{3.11}$$

$$\sigma_v = 0.1791z^2 + 4.2447z + C \tag{3.12}$$

where $z = 0$, $\sigma_v = 0$ and so C = 0. Figure 3.30 shows the curve described by (3.12).

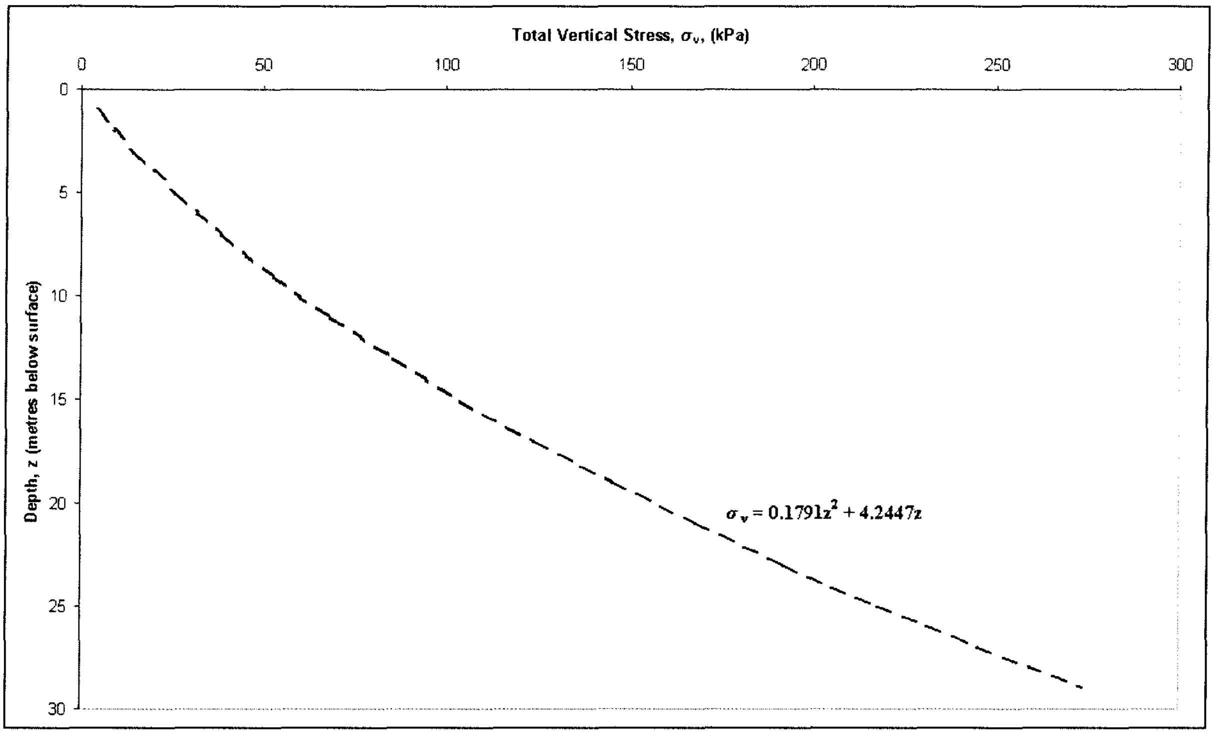


Figure 3.30 Variation in total vertical stress with depth.

At Rainham, the landfill surface is at 31mOD, and the average leachate level at 14mOD. If the waste below 14mOD is assumed to be saturated and the appropriate values are substituted into equations (3.5) to (3.12) the relationship between hydraulic conductivity and elevation within the saturated waste is shown in Figure 3.31.

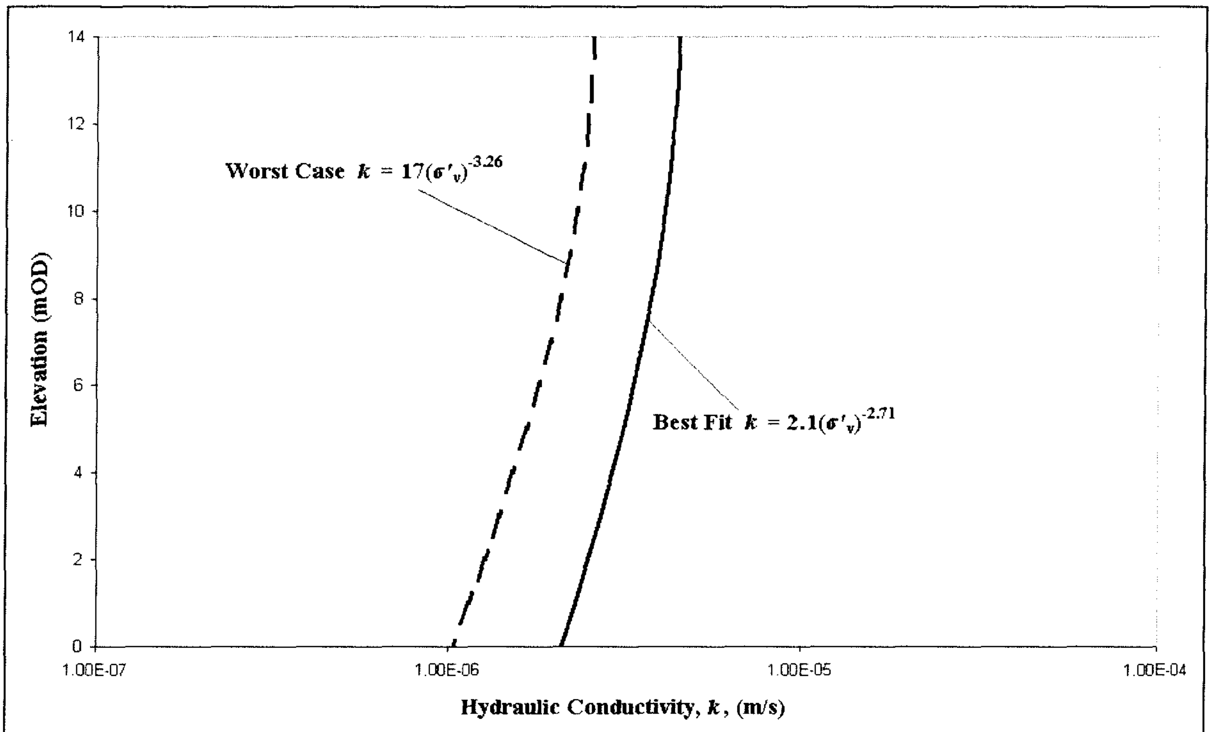


Figure 3.31 Steady-state variation in hydraulic conductivity with elevation, with landfill surface at 31mOD and leachate level at 14mOD, using equations from Powrie and Beaven (1999).

From Figure 3.31 the range of k values reduces from 4.5×10^{-6} m/s at the top of the saturated waste (best fit) to 1×10^{-6} m/s at the base of the landfill (worst case). At 2mOD, the elevation of the proposed horizontal wells, k ranges from 1.3×10^{-6} to 2.5×10^{-6} m/s (worst case and best fit respectively). However, as Powrie and Beaven tested 'fresh' waste, they suggest that aged or degraded wastes might be expected to be of lower hydraulic conductivity owing to particle size reduction and density increase.

These estimates of hydraulic conductivity should be regarded as speculative as they cover only a narrow range of values which, given the heterogeneity and layering of the waste, is perhaps unlikely. Furthermore, the curves on Figure 3.31 rely upon the strength of the relationship between bulk density and depth which has a low correlation coefficient of 0.33. Nevertheless, this analysis does provide an indication of approximate hydraulic conductivity values.

3.4 Site Investigation Conclusions

Of the parameters recorded during the site investigation, the composition of the waste is likely to be the most influential in the success or otherwise of the horizontal well installation process. In addition, the waste composition, particle size distribution and permeability will be major factors affecting the performance of the wells once installed.

The site investigation showed that the main section of each horizontal well is to be installed in the most aged and decomposed waste below 5.5mOD which was deposited in 1975. Although this waste and the waste deposited in the 1990's are generally similar in that they both contain a wide variety of materials, some clear differences have been revealed which are summarised in Table 3.2.

Table 3.2 Comparison between 1975 waste and 1990's waste.

Date of Deposition	1975	1990's
Elevation (mOD)	0.5 to 5.5	5.5 to Surface (max 31.5)
Temperature (°C)	25° to 35°	35° to 60°, lower near surface
Moisture Content (%)	Range 23 to 75. Average 49	Range 16 to 87. Average 43
Bulk Density (Mg/m^3)*	Range 1.39 to 1.52 at base	Range 0.42 at surface to 1.39
Degree of Decomposition	4 / 5	1.5 to 4 / 5
Main Contents *Wood*	Large wooden pieces present	Large wooden pieces present
Paper	Less frequent	More frequent
Rubber	No tyres	Some tyres & tyre fragments
Fabrics	Found in 34% of samples	Found in 85% of samples
Hardcore / Brick	No large pieces	Large pieces in 4% of samples
Metal	No large pieces	3 thick steel plates discovered
Matrix Description	10 to 20% gravel, soil, fibres, organic odour, oily sheen	Soily, fibrous, dry and dusty in unsaturated horizons
Matrix Volume	50-75%	25-50%

Table 3.2 demonstrates that directional drilling in the upper waste horizons is likely to be more difficult due to the presence of larger more problematic particles in this newer waste,

e.g. steel plates and concrete. The main horizontal sections are to be drilled through the 1975 waste although the entry and exit sections will require drilling through the 1990's waste.

The high percentage of slurry like matrix material present in the saturated waste may not provide enough resistance to permit accurate steering of the drill bit. However, accurate drilling is required to avoid the silt/waste interface whilst ensuring the wells remain as deep as possible to maximise the potential for leachate extraction. With the silt/waste interface between 0 and 1.5mOD it was decided that the target borehole elevation should be no lower than 2mOD.

In areas where the waste is dry and of low density then the volume of drilling fluid lost to the waste during installation of the horizontal wells may be considerable. The use of bentonite based fluids is therefore undesirable if the hydraulic conductivity of the waste is to be maintained.

3.4.1 Need for a Directional Drilling Trial

The site investigation aims outlined in Section 3.2.1 were largely met despite the problems experienced with the vertical drilling rig. A considerable insight has been obtained into both the

- composition and properties of the waste in-situ, and
- general characteristics and features of Rainham landfill site such as the stratigraphy, history and groundwater regime.

Although the site investigation raised a number of concerns, none were considered to be of major significance. However, little information was obtained that would assist with design specifications such as drill rig capacity, well diameter and screen material. Therefore, before committing the project funds to a full scale trial it was considered prudent to attempt the installation of a short, 100m long, horizontal well at Rainham. The aim of this trial was to identify the principal difficulties associated with directional drilling in landfill and to provide recommendations for the major design specifications. The small scale trial is discussed in section 4.1.

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* Including the 8.5m depth of waste tipped since the site investigation.

CHAPTER 4

Well Installation Field Trials

Conventional directional drilling techniques were reviewed in Chapter 2. The intention of Chapter 4 is to show how these techniques were modified and improved to overcome the particular problems of horizontal well installation in a landfill. To do this, a series of field trials are discussed that were observed or carried out as part of the current research. These trials demonstrate an ongoing refinement of the drilling technique for landfill applications. Recommendations for future horizontal well installations are included at the end of the chapter.

4.1 First Rainham Field Trial

As identified at the end of Chapter 3, the objectives of the first Rainham field trial were twofold;

1. to identify the principal difficulties associated with directional drilling in landfill, and
2. to provide recommendations for the design of the second (and larger) field trial.

At this stage it was not clear what adaptations to the drilling technique would prove useful for landfill applications. Following a review of other (non-landfill) directional drilling projects the specifications given in Table 4.1 were selected for the trial. These were considered to be representative of the specifications used for similar small scale ground remediation projects.

Table 4.1 Specifications for first Rainham field trial

Drill Rig Size (Pullback Capacity)		Mini Rig (120kN)
Borehole Target Length		100m
Maximum Borehole Depth		10m
Drilling Method		Backreaming
Guidance System		Walkover
Drill Bit Type		Bevelled Edge
Back-Reamer Diameter & Design		250mm, Cutting Reamer
Drilling Fluid		Polymer / Biodegradable
Well Screen	*Material*	Low Carbon Steel
	Diameter	150mm
	Length	40m (centre section of full 100m)
	Perforation Diameter	15mm (Round holes)
	Open Area	$0.27m^2$, 1.5%

An area of Phase 2 was selected where a short well (100m in length) could be installed that would penetrate both the 1990's waste and the 1975 waste and return to the surface whilst avoiding tight bends in the borehole profile. The estimated completion time for the trial was three days. Breheny Civil Engineering Contractors agreed to carry out the works without charge in order to develop their project portfolio. Drilling commenced in March 1999.

4.1.1 Pilot Drilling

Following rig set up, Figure 4.1, a bevelled drill bit 75mm wide was drilled into the waste. The drill rods were 55mm in diameter. The profile of the pilot borehole is shown in Figure 4.2. From a starting elevation of 10.3mOD, the borehole was driven downwards at an angle of 26° to 4mOD and then steered through a shallow curve to become horizontal at 2.5mOD. This was slightly above the target of 2mOD. The horizontal section, which would include the slotted well screen, was intended to be 40m long. However, a hard obstruction was encountered 50m from the entry point and the drill head was deflected upwards so drilling continued in a gradual arc toward the exit point. Further obstructions were encountered between 80 and 90m, where the drill head was twice deflected downwards by hard materials.

Figure 4.1 Mini-rig at Rainham for trial drilling.

The walkover guidance system had a maximum operating depth of 10m, equal to the maximum target depth of the borehole. At depths greater than 8m the depth reading would fluctuate by approx. ±0.25m as both (a) the operating limit of the instrument was reached, and (b) metallic objects in the waste caused interference with the signal.

The polymer based drilling fluid did not flow back to the surface at any time during the pilot drilling. Three possible explanations were put forward to explain this;

i) fluids were absorbed into unsaturated zones which may have been isolated until hydraulically connected by the drilling;

ii) the borehole may have closed around the pilot rods preventing return flow; and

iii) a filter cake may not have been able to form in areas with coarse grain size allowing a continual migration of fluids out into the waste.

As there was no flow of drilling fluids from the borehole no cuttings were removed. The waste must therefore have been compressed into the borehole walls as the drilling assembly passed through. It is doubtful whether the waste materials would have actually been cut and milled as is usual with soil or rock. Indeed, the tensile strength of waste materials such as plastic, fabric and metal may have resisted the cutting action of the jets and so a flowable slurry of waste particles would not have formed.

Despite the obstructions, pilot drilling was generally less problematic than had been anticipated and the 105m long borehole was completed in only four hours.

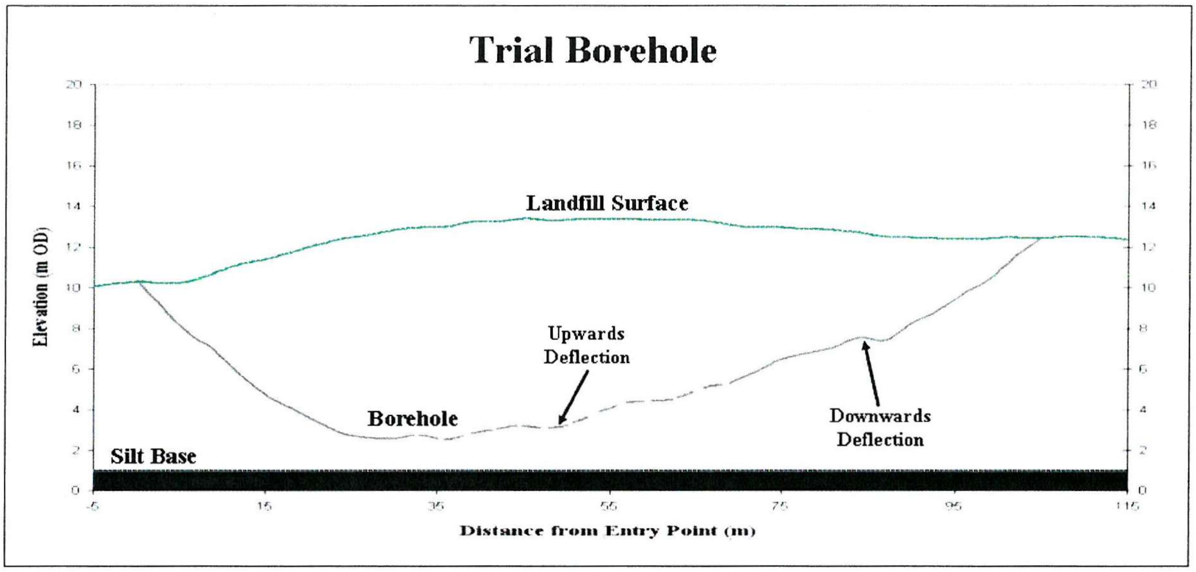

Figure 4.2 Profile of trial borehole, dashed line represents section intended for completion with slotted well screen.

4.1.2 Pre-reaming

4000 Nm of torque was required to rotate the drill rods in the borehole. The drill rig operators advised that this was 'considerably more' torque than usually required after drilling pilot boreholes through natural ground formations. This suggested that the 75mm diameter borehole may have closed or collapsed around the 55mm diameter drill rods. As a result it was decided to enlarge the borehole twice by pre-reaming before final installation of the 150mm well screen.

The first pre-ream used a 100mm cutting reamer combined with a 150mm compaction reamer, Figure 4.3. The cutting reamer was designed to cut the waste to form a larger borehole, with particles flowing to the surface suspended in the drilling fluid. The compaction reamer was designed to compress any remaining loose waste into the borehole walls to limit closure/collapse.

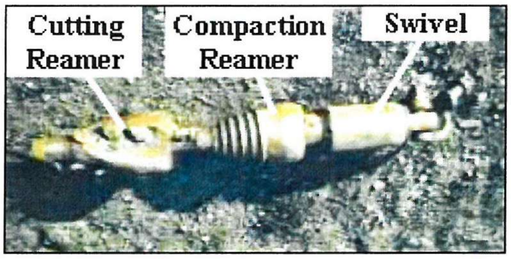

Figure 4.3 Drilling assembly used on first pre-ream; cutting reamer, compaction reamer and swivel.

The pre-ream assembly was pulled and rotated through the borehole without meeting any obstructions although the pulling force required often reached the maximum 120kN capacity of the rig. No fluids or cuttings returned to the surface. On completion of the pre-ream the assembly was found to be wrapped in waste items that had some degree of tensile strength, including plastic bags, banding tape, metal wire, a metal chain and a blanket. The cutting teeth of the reamer were also covered, preventing milling of the waste.

The second pre-ream used the same assembly as the first with an additional 250mm compaction reamer to enlarge the borehole to the diameter required. Due to the inability to cut the waste a larger cutting reamer was not used. The second pre-ream progressed without difficulty (although there were still no drilling fluid returns) until at 50m the drilling assembly encountered an obstruction that halted progress for one hour. Continued pulling and rotation slowly advanced the drilling assembly over a length of 3m before the obstruction was passed. The material that caused the delay was unknown and it was not clear whether it was 3m in length or thickness, or whether a smaller item had been dragged over 3m. On completion of the second pre-ream the drilling assembly was again found to be wrapped in waste materials including plastic bags, metal wire and metal cable.

4.1.3 Well screen installation

The cutting and compaction reamers were removed from the drilling assembly. The screen was positioned at the exit end of the borehole and attached to the drill string. An excavator was used to lift the screen to allow it to pass into the borehole at the same angle as the exit trench, Figure 4.4a.

Figure 4.4 a) Excavator lifts well screen allowing it to pass into the exit trench at the correct angle, b) excavator feeds screen into the borehole to assist the pulling action of the rig.

The first 10m was pulled into the borehole using the drilling rig alone. However, the required pullback force increased rapidly so that after 10m the full 120kN capacity of the rig was reached. The excavator then assisted the rig by feeding the well screen into the borehole, Figure 4.4b. A further 15m was installed in this way before the force required once again exceeded the force applied and the trial had to be abandoned. The 25m of well screen that had already been installed was pulled back out (with difficulty) using a bulldozer. The surface of the well screen revealed deep scratches on all sides indicating that the borehole had possibly closed around it. The following analyses support the hypothesis that borehole closure could have prevented installation.

Analysis 1 – Borehole Remains Open

Milligan and Norris (1996) studied the forces required to move pipes through soil during pipe-jacking operations*. They concluded that pipes sliding in a stable (open) borehole will generate frictional resistance in proportion to their weight. This is illustrated in Figure 4.5. Based on this principle, the pulling force, *P*, required to draw the well screen used at Rainham through an open borehole is analysed below.

* Pipe-jacking – this is a technique for installing underground pipes or ducts in which the pipes are pushed through the ground behind a tunnelling shield using hydraulic jacks reacting against a thrust wall in a jacking or thrust pit [Milligan, 2000].

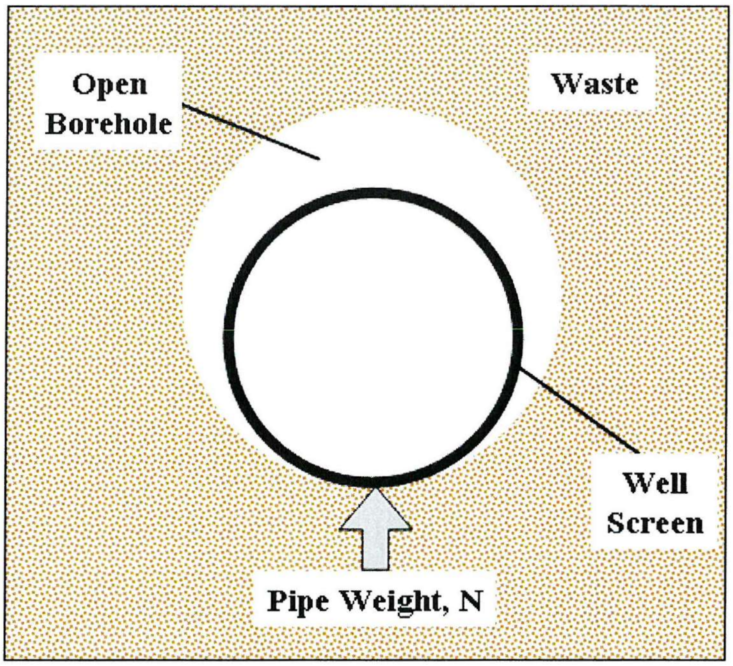

Figure 4.5 End on diagram of well screen in a borehole that remains open.

The relationship between friction, F, and normal load, N, is

$$F = \mu N \tag{4.1}$$

where μ is the coefficient of friction between the load and the surface over which it is sliding. The coefficient of friction can be expressed in terms of the angle of interface friction, φ. This is the angle of inclination of the resultant force on the sliding interface, measured from the normal. Substituting $\mu = \tan \varphi$ and $F = P$ into equation (4.1) to find the pulling force required to make the well screen move,

$$P = N \tan \varphi \times l \tag{4.2}$$

where N is the well screen weight per unit length and l is the well screen length. To measure interface friction between waste and steel, Powrie & Beaven (1999) placed approximately 1 ton of loosely compacted household waste on the smooth metal back of a tipper lorry. The back was slowly raised and the angle at which the waste just started to move was measured. The test was repeated a number of times giving φ values of 25-30°.

The mass of well screen per unit length (N) is approximately

$$N = \pi\, d\, t\, \rho\, g \tag{4.3}$$

where d is the well screen diameter, t is the screen wall thickness, ρ is the density of the well screen material* and g is the gravitational constant.

The Rainham well screen was made from low carbon steel and had dimensions of

$$d = 0.15\text{m}, \qquad t = 0.005\text{m}, \qquad \rho = 7800\ \text{kg/m}^3$$

In the worst case, when $\varphi = 30°$, substitution of the values gives P as 0.104 kN/m and for the full 105m of well screen, P equals 11kN. This value is much smaller than the 120kN maximum pullback capacity of the rig. Had the borehole remained fully open, installation of the well screen should therefore have been successful. It should be noted that $\varphi = 30°$ was obtained from tests on loosely compacted waste, however, if the waste had been more densely compacted the angle of friction may have been higher. Nevertheless, even with, say $\varphi = 45°$, the resulting pulling force required, at 19kN, is still very much lower than the pullback capacity of the rig.

Milligan (2000) states that 'if a borehole is filled with a fluid lubricant, the pipes will become partially or completely buoyant, and the contact forces between pipes and ground will be greatly reduced'*. As a result, Milligan goes on to say that 'pipes at neutral buoyancy in a slurry lubricant of low shear strength may experience almost negligible (frictional) resistance'. With fluid thought to have migrated away from the borehole it is unlikely that the installation would have benefited from this buoyancy effect.

The above analysis neglects the frictional resistance exerted on the drill rods as they are moved through the borehole. Using the rod specifications $d = 0.055$m, $t = 0.01$m, and $\rho = 7800\ \text{kg/m}^3$, then the full 105m length of drill rods would require 8kN of pulling force if the borehole (drilled at a diameter of 75mm) remained open. Although the drill rods are removed as the well screen is installed, the full length of the well screen must be pulled over the landfill surface before it enters the borehole. The combined total (19kN) therefore represents the greatest force required but is considerably less than the 120kN capacity of the rig. However, the full capacity of the rig was at times required to draw the drill string itself through the borehole, much more than the calculated 8kN required had the borehole remained open.

* The effect of the perforations on well screen density were assumed to be negligible as the open area was less than 1.5%, and the well screen itself comprised only 40% of the total pipe length.

Milligan and Norris (1996) observed that misalignment of pipes during pipe-jacking operations can cause localised contacts between the pipe and the borehole walls which add to the frictional resistance. Deviations in borehole route during directional drilling possibly have the same effect as pipe misalignment and increase frictional resistance. This may partly explain why the full rig capacity was periodically required during pre-reaming.

Analysis 2 – Full Borehole Closure

Milligan (2000) states that the primary function of a drilling fluid in unstable ground is to provide a fluid pressure to support the borehole and prevent collapse of the ground onto the pipes. To do this in permeable ground requires the formation of a filter cake at the soil (waste) interface. Filter cake formation has been discussed in section 2.2.2. With fluids dispersing into the waste a filter cake would have been unable to form and the borehole may have collapsed. The pulling force required to draw the Rainham well screen through a closed borehole where friction is acting all round the screen, as in Figure 4.6, is analysed below.

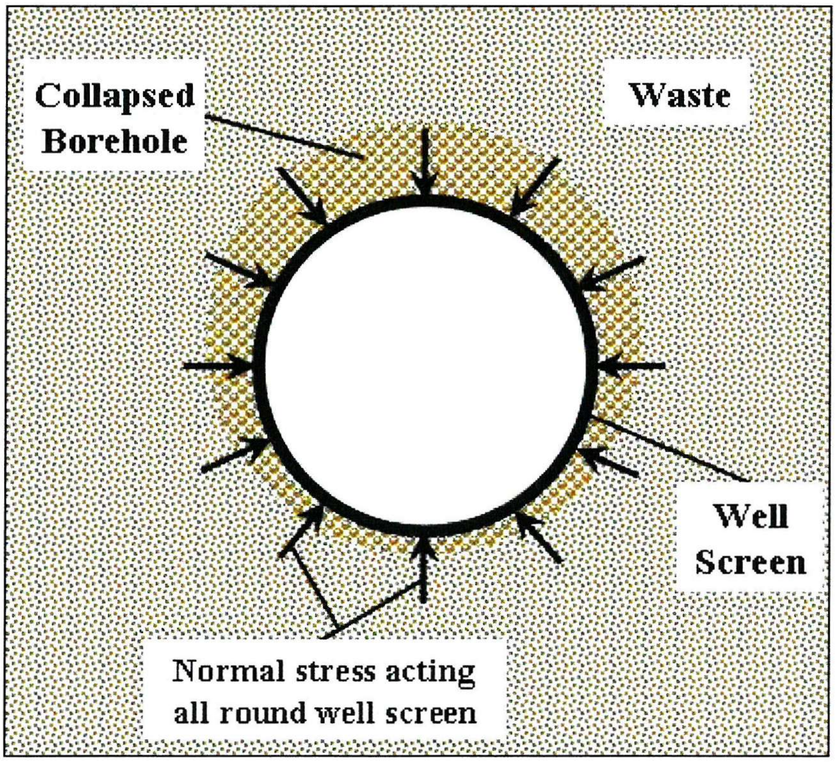

Figure 4.6 End on diagram of well screen in a closed/collapsed borehole.

* The effect will be less for perforated well screen than for an air-filled pipe.

The frictional resistance to movement is related to the average effective stress, σ'_{av}, acting on the well screen, so that

$$P = \pi d\,\sigma'_{av} \tan\varphi \times l \qquad (4.4)$$

σ'_{av} is related to the vertical effective stress σ'_v by

$$\sigma'_{av} = \alpha\,\sigma'_v \qquad (4.5)$$

where α is a coefficient with a value likely to be between 0 and 1.

The values required to calculate the σ'_v value for the Rainham trial are

Ground Level (GL)	= 10.5mOD	
Leachate Level (LL)	= 5mOD	
Borehole Level (BL)	= 2.6mOD	(assuming full length of borehole at this depth)
Total Stress, σ_v	= 45kPa	(based on measured in situ density)
Pore Water Pressure, u	= 24kPa	

From Terzaghi's equation, $\sigma'_v = \sigma_v - u$

$$\sigma'_v = 45 - 24 = 21\text{kPa}$$

Substituting the values into equation (4.4) and taking α to be 0.5

$$P = \pi \times 0.15 \times (0.5 \times 21) \times \tan 30 \times l$$

$$P = 2.86\text{kN/m}$$

For a borehole length of 105m, P = (105 × 2.86) = 300kN. Clearly the 120kN capacity of the mini-rig would be exceeded if the borehole had fully closed.

The pulling force required to move the 55mm diameter drilling rods through a closed borehole is not included in the above analysis. Substituting the appropriate values into equation (4.4), for a 105m length of drill rods

$$P = \pi \times 0.055 \times (0.5 \times 21) \times \tan 30 \times 105\text{m}$$

$$P = 110\text{kN}$$

Therefore pulling the drill rods may have just been possible using the 120kN rig if the borehole had fully closed up.

With such a small spare capacity, it is surprising that even a 10m length of the 150mm diameter well screen was installed with the rig alone. It therefore seems likely that only partial collapse of the borehole had occurred with some sections remaining open. Additional factors reducing the pulling force required may have included:

- conveyance of effective stress around the borehole through arching, and
- reduced stress at each end of the borehole as the depth of the borehole decreased.

In summary, if the borehole had remained open, movement of the drill rods and installation of the screen would not have been difficult. If the borehole had fully closed, movement of the drill rods may have been possible but the rig would not have had spare capacity to install the larger diameter well screen. In actuality, the screen was pulled 10m into the borehole using the rig alone, suggesting that the borehole had undergone partial but not complete closure. It is thought that if the well screen had been of a similar diameter to the drill rods, installation may have been successful. This hypothesis led to the Metallic Tile field trial discussed in section 4.4 in which a small diameter well screen was installed. However, a well screen less than 100mm in diameter will not easily accommodate submersible pumps or well development tools.

It is possible that the degree of closure of the borehole was increased as the walls were not continuously supported. Had the drill rods been of a larger diameter, say 100mm, the walls would have been better supported and closure may have been reduced, possibly enabling the installation of a larger diameter well screen.

The analyses made clear that the issue of borehole closure was of paramount importance and needed to be resolved if the longer installations of the second Rainham trial were to be successful.

4.1.4 Conclusions & Recommendations from the First Rainham Trial

Although the site investigation highlighted the presence of large waste items which might obstruct pilot drilling, few obstructions were actually encountered and none caused major problems. However, waste materials with tensile strength such as wire and fabrics became wrapped around the drilling assembly and prevented effective milling of the waste.

The large variations in density and the presence of slurry-like waste identified during the site investigation did not prove to be a problem for steering of the drilling head. Some deflections were caused by hard items but none were considered to be significant.

Reasons for the absence of fluid returns were thought to include (a) absorption of fluids into unsaturated zones, (b) partial borehole closure preventing return flow, and (c) coarse grain size preventing filter cake formation.

Readings of the assembly temperature, obtained from the guidance sonde, remained at 20°C ± 4°C throughout the pilot drilling, demonstrating that the drilling fluids kept the drilling assembly cool despite the elevated temperatures in the landfill.

The principal difficulty associated with directional drilling in waste materials was identified as borehole closure. This occurred despite the use of compaction reamers in an attempt to compress any loose waste into the borehole walls. Investigations into alternative or modified methods of drilling were required to overcome this problem.

Despite the abandonment of the trial before installation was complete, many lessons for future attempts were learned. These are discussed below.

1. A higher capacity rig would be required. The mini-rig often ran at maximum capacity (120kN) even before well screen installation was attempted. With the proposed wells more than five times longer, a maxi-rig (>800kN) may be required.
2. The back reamer should not be overly aggressive, with numerous protruding teeth, as these are likely to become wrapped in waste materials thereby reducing their cutting potential.
3. The tensile strength of the well screen should ideally be greater than the maximum pullback capacity of the drill rig, thus preventing breakage of the well screen during installation.
4. Partial closure of the borehole prevented installation of the 150mm diameter well screen although movement of the 55mm diameter drill rods was possible. Use of a smaller diameter well screen, and/or the provision of better borehole support (perhaps through the use of larger diameter drill rods) may have resulted in a successful installation. As a result the diameter of the well screen specified for the second trial

was reduced from 150mm to 100mm. According to Price (1985) and the US EPA (2001) this should not cause a significant reduction in well performance. Price [1985] states that *'doubling the diameter of a* (vertical) *well will decrease the drawdown by only about 10% for the same yield'*. The US EPA [2001] state that *'doubling the diameter of the well screen will increase the yield of the well by 10 to 15%'*.

5. The walkover guidance system experienced some loss of accuracy below 8m depth. The proposed wells were to extend up to 29m below ground level, so it is clear that a wireline system would be required.
6. With all fluids remaining in the waste, bentonite based fluid should be avoided as this is likely to irreversibly reduce the hydraulic conductivity of the waste. If fluid does not return and no filter cake can be formed then water alone may be sufficient for cutting and cooling purposes.
7. The perforation design and open area of the well screen, Figure 4.7, was given little consideration. The perforation diameter (or slot width) should be designed using particle size distribution data for the waste and, if possible, the open area should be maximised to reduce well losses.

Figure 4.7 150mm diameter well screen used in the first Rainham trial. 15mm holes, open area of 1.5%.

In addition to the above, a number of minor, but nevertheless important, recommendations arose from the drilling trial. These are listed below.

1. Welding of the well screen sections could not take place during installation due to potentially explosive landfill gas venting from the borehole. Consequently the full 105m length had to be welded in a gas free location away from the landfilled area, and then moved into place. Manoeuvring even 105m of screen across the landfill was difficult; had the well screen been 550m (as proposed for the second Rainham trial)

this would have been impossible. Threaded joints or a similar alternative were therefore recommended.

2. The proposed route of the borehole should be surveyed and clearly staked out, allowing the operator to align the rig correctly before drilling commences. The target depth of the borehole below each stake should be calculated beforehand to act as a guide whilst drilling. Further, the drilling contractor should provide the full coordinates of the completed borehole profile together with the elevation in metres Ordnance Datum (mOD). This information would be crucial for positioning of the piezometers.
3. The leachate level at some point along the well screen may be higher than the entry or exit point of the well leading to artesian conditions. It is therefore important to have end caps available to stem such flow.
4. Finally, issues were raised that would require consideration before attempting the second, larger scale, trial. These included;
    a. space and foundations required for rig set up,
    b. time required for rig set up,
    c. anchorage for rig,
    d. water supply and water storage (if demand outstripped supply),
    e. interference with ongoing site activities, and
    f. health and safety issues such as equipment testing certificates, gas meters, protective clothing etc.

These recommendations proved invaluable when organising and conducting the second Rainham trial discussed in section 4.3.

4.2 Trials in the American Midwest

4.2.1 Introduction

Following completion of the first Rainham trial, reference was found to two separate projects in the American Midwest where directionally drilled horizontal wells had been installed in waste materials for the purpose of leachate control. At the first site, Livingston Landfill in

Illinois, a single well had been installed in 1996 and two further wells in 1999. At the second site, the City of Superior Landfill in Wisconsin, a single well had been installed in 1996. A visit was made to each site in October 1999 to discuss the projects in detail. The specifications for the second Rainham trial were subsequently modified as a result of the failures and successes encountered in the Midwest.

Conditions at Livingston were in many respects similar to those at Rainham. Tipping of waste had commenced on level ground in the early 1970's and a 26m high waste mound had since been created with no system for basal drainage or other infrastructure for leachate control. Elevated leachate levels had prompted the installation of control measures, and horizontal wells were proposed as an alternative to an extensive vertical well field.

At Superior, tipping had also begun in the 1970's with no infrastructure for leachate control in place. The site base was below the surrounding water table and the initial hydraulic gradient between the groundwater and leachate promoted flow into the landfill. However, leachate levels had risen, reversing this hydraulic gradient. This prompted the installation of a horizontal leachate control well with the aim of reestablishing the former inward hydraulic gradient.

4.2.2 Midwest Project Details

The specifications of the Livingston and Superior projects are summarised in Table 4.2. A discussion of the successes and failures of the techniques used and the consequential influence on the specifications for the second Rainham trial, follows.

Table 4.2 Specifications for horizontal wells installed in Midwest landfills

Item	Livingston Well 1	Livingston Wells 2 & 3	Superior Landfill Well 1
Drilling contractor	Hoerr Construction	A&L Underground	Mears
Date of installation	March 1996	April – November 1999	September 1996
Duration of installation	30 days	7 months	4 days

Item	Livingston Well 1	Livingston Wells 2 & 3	Superior Landfill Well 1
Type of waste	Old waste (1978) - higher proportion of soil to MSW & little construction waste	New MSW (1990s), and older waste (1978).	Old waste (1970's) Higher proportion of soil to MSW
Drill rig size	Mini Rig (107kN)	5 rigs from 267kN to 2,224kN	Midi Rig (311kN)
Drill bit type	Tri-cone	Tri-cone	Tri-cone
Guidance system	'Wireless' (AccuNav)	Wireline (TruTracker™)	Wireline (TruTracker™)
Drilling fluid additive	Biodegradable	Biodegradable	Biodegradable
Drilling fluids returns	Some fluids, no cuttings	No fluid returns beyond 60m	No fluid returns beyond 12m
Borehole length	170m	2 × 400m	365m
Max depth below ground	26m	26m	14m
Back-reamer diameter & design	200mm fluted reamer to minimise snagging	200mm fluted reamer to minimise snagging	400mm pre-ream 300mm back-ream barrel reamer.
Carrier casing material & diameter	No carrier casing	No carrier casing	HDPE (SDR 11) 250mm
Well screen material	Perforated HDPE	1st attempt perf. HDPE 2nd attempt perf. Steel	HDPE (SDR 17)
Well screen diameter	1st attempt 150mm, 2nd attempt 100mm	100mm	100mm
Well screen perforation diameter	6.35mm (holes)	9.5mm (holes)	0.5mm (slots)
Open area of well screen (approx.)	0.16%	0.5%	3%
Well development methods	Jetting with bleach solution & overpumping	Not developed	Jetting with water & calcium hypochlorite to breakdown fluids
Well yield post development	$0.27m^3$/day	Not yet pumped	$13m^3$/day
Well yield, longer term	$0.16m^3$/day	Not yet pumped	$4m^3$/day

[Additional sources for Livingston County: PUC 1996, Yach 1997, Friend & Hock 1998, Friend 2001]
[Additional sources for City of Superior: Wilson 1996b, Bruxvoort *et al* 1998, HDDwell.com 2001b]

4.2.2.1 Rig size

The first horizontal wells installed at Livingston and Superior in 1996 were completed using a mini and midi-rig respectively. However, the importance of rig size became apparent during the second Livingston project when five drilling rigs of increasing capacity were deployed in succession before a well was successfully installed. Rigs up to 730kN pullback capacity proved to be insufficient, and it was not until a maxi-rig of 2,224kN was used that the first installation was completed after a total of seven months on site.

Both the Cardiff Bay installations and the first Rainham trial had suggested a maxi-rig would be required for the second Rainham trial. The events at Livingston confirmed this.

4.2.2.2 Site investigation

No site investigation boreholes were drilled prior to the installation of the horizontal wells at either site and as a result little was known about the waste types or the exact position of the basal liner. Tipping records showed that both sites had accepted relatively small amounts of waste during the first years of operation (<250 tons/day), so that the daily cover material (150mm thick clay) made up a significant volume of the waste. It is thought that the clay may have assisted the drilling by (a) offering less resistance to the passage of the drilling tools, and (b) maintaining the temporary stability of the borehole wall.

At both Livingston and Superior the horizontal section of the borehole was drilled to within 1m of the clay liner. However, both project managers subsequently conceded that as the exact position of the site base was not known there was a possibility that sections of the wells may have been installed directly on, or even in, the clay liner [Abichou 1999; King 1999]. Although this would make installation less problematic it would compromise both the performance of the wells and the integrity of the leachate containment barrier. In addition, it was observed that while the elevation of the drilling assembly was monitored throughout the pilot drilling stage, subsequent back-reaming may have cut lower into the waste, bringing the borehole even closer to (or deeper into) the clay liner. As a result of these concerns, the specified elevation for the proposed Rainham wells was raised by a further metre from 2mOD to 3mOD.

4.2.2.3 Drilling tools & guidance systems

The pilot borehole of the first Rainham trial had used a drill bit with a bevelled edge whereas the pilot boreholes of all three Midwestern projects had successfully used tri-cone drill bits. On one occasion an obstruction was encountered during the pilot drilling at Superior. The drill head was retracted 10m and successfully redirected around the obstruction. It was thought that a tri-cone drill bit would be more likely to mill the waste than a bevelled bit and so the tri-cone type was specified for use in the second Rainham trial. The 'fluted' reamer used at Livingston and the 'barrel' reamer used at Superior were both relatively smooth in comparison to other reamers, i.e. the cutting teeth did not project more than 10mm. This smooth design prevented materials such as wire and fabrics becoming wound around the reamer and covering the cutting teeth as had happened during the first Rainham trial.

The first project at Livingston County used the AccuNav® guidance system, the operating principle of which was discussed in section 2.2.3. This relies on the earth's magnetic field to calculate drill head orientation and is therefore subject to interference from a number of sources including metallic objects which are common in waste. The system is also subject to error propagation along the borehole path. The tight tolerance on borehole elevation for the second Livingston project demanded greater accuracy and so the TruTracker™ system was used. TruTracker™ performed well at Livingston (and Superior) and was therefore specified as the guidance system for the second Rainham trial.

4.2.2.4 Drilling fluids

Each of the Midwestern projects was completed using a biodegradable drilling fluid. This was in recognition of the need to avoid the introduction of low permeability bentonite into an already low permeability formation. Permeability values at Livingston were estimated to be 10^{-7} to 10^{-8}m/s [Friend 2001]. Referring to the Superior project, Bruxvoort *et al* (1998) states that; "*The degradation of biodegradable drilling fluid eliminates much of the development effort associated with bentonite based drilling fluids*".

At Livingston and Superior no drilling fluids returned once the pilot boreholes had progressed 60m and 12m respectively. With no fluids (and therefore no cuttings) returning to the surface it is questionable whether the use of drilling fluid additives was of any benefit in terms of borehole support, in which case water alone might have been sufficient to satisfy cutting and cooling requirements.

During the first Livingston trial, fluids periodically returned to the surface but were free of cuttings. This suggests that either the gel strength of the fluid was too low to carry cuttings in suspension or, as in the first Rainham trial, the drilling tools did not cut the waste but simply pushed through it. With no cuttings, the waste would be compressed into the borehole walls from where it could collapse or expand back into the borehole. If the cuttings remained compressed into the borehole wall the local permeability would be reduced.

For the second Rainham trial it was decided that biodegradable fluid additives would be used in an attempt to provide borehole support and to remove cuttings, but if no fluids returned then only water would be used.

4.2.2.5 Well screen installation

In the first Livingston trial, an attempt was made to install a 150mm diameter perforated HDPE well screen into a borehole which had been back-reamed at a diameter of 200mm. During pullback, the tensile load exerted by the drill rig had to be gradually increased to overcome the frictional resistance acting between the borehole walls and the surface of the well screen. However, the pulling swivel snapped before installation was complete. On removal of the HDPE well screen from the exit end of the borehole a 6m section was found to have been plastically deformed (stretched), indicating that the yield strength of the well screen had been exceeded. The tensile load F, required to cause plastic deformation of the 150mm diameter HDPE well screen is approximately

$$F = \pi\, d\, t \times P_y \tag{4.6}$$

where d is the well screen diameter, t is the wall thickness and P_y is the yield strength in Pascals. Assuming a yield strength of 22MPa (Table 2.1), and a wall thickness of 9mm, F equals 93 kN. This was marginally lower than the 107kN pullback capacity of the rig. By substituting the ultimate tensile strength P_{ult} for P_y in equation (4.6) and assuming a P_{ult} value of 34MPa (Table 2.1) the tensile load required to break the well screen would be 144kN. This is greater than the pullback capacity of the rig and may explain why the HDPE stretched but did not break. To reduce the frictional resistance a second successful attempt was made with smaller, 100mm diameter HDPE.

In the second Livingston project, the installation of similar 100mm diameter HDPE well screen was attempted. On this occasion a 267kN pullback capacity rig was used and, not surprisingly, the HDPE snapped. The HDPE was then exchanged for 100mm diameter steel well screen. Using equation (4.6) and values of d = 100mm, t = 15mm and P_{ult} = 500MPa (from Table 2.1), the tensile strength of this screen was approximately 2360kN, much greater than the capacity of the rig and the well screen remained intact. However, partial borehole closure again caused the frictional resistance on the well screen to exceed the capacity of the rig and installation could not be completed. The 267kN rig was exchanged for a succession of three increasingly powerful midi-rigs up to 730kN but it was not until a maxi-rig (2,224kN capacity) was brought to the site some seven months after drilling had commenced that the well screen was successfully installed.

During both Livingston trials, partial borehole closure again proved to be the principal factor preventing or delaying successful installation. This reinforced the conclusion made from the first Rainham trial that the issue of borehole closure must be addressed before a second trial was attempted at Rainham.

In contrast to the Livingston trials, the installation at Superior proved to be rapid and problem free with no evidence of borehole closure. Why Superior should be different is not clear, but may be a combination of the following factors:

- high clay/soil content in waste providing better structure to the borehole wall,
- demolition materials and appliances (e.g. cookers, fridges) were not accepted at the landfill,
- some sections were possibly installed within the clay liner,
- the smooth walled carrier casing may have reduced frictional resistance as perforations/slots in the well screen may catch on protrusions from the borehole wall,
- installation was rapid (3 days), minimising the opportunity for borehole closure, and
- due to a smaller depth of waste the vertical effective stress at the base of the Superior site was 50kPa, compared with 170kPa at Livingston. From equation (4.4), a lower effective stress will reduce the frictional resistance on the well screen during installation.

Table 4.2 applies equation (4.4), which assumes the borehole has closed, to both the Superior and Livingston installations. For comparison, equation (4.2) which assumes the borehole remains open, is also applied.

Table 4.2 To compare pulling forces required at Superior and Livingston assuming (a) full borehole closure and (b) an open borehole.

Factor	Superior	Livingston (2nd Trial)
Screen/casing Diameter (*d*)	0.25m	0.1m
Screen/casing Wall Thickness (*t*)	0.015m	0.015m
Screen/casing Density (*ρ*)	955kg/m^3	7800kg/m^3
Average Effective Stress (σ'_{av})	25kPa	85kPa

Angle of Interface Friction (tan φ)	30°	30°
Borehole Length (l)	365m	400m
Pulling Force Required (P) **Closed Hole** $P = \pi d\,\sigma'_{av} \tan\varphi \times l$	4,140 kN	6,170 kN
Pulling Force Required (P) **Open Hole** $P = \pi d t \rho g \times \tan\varphi \times l$	23 kN	83 kN
Drill Rig Capacity	311 kN	from 267 kN to 2224 kN

Although the pulling forces theoretically required if the borehole had closed far exceed the drill rig capacity used at both sites, the force required at Livingston is 50% greater than Superior. More extensive borehole closure would therefore need to have occurred at Superior before the drill rig capacity was exceeded. Had the boreholes remained open, neither installation should have been difficult.

4.2.2.6 Carrier casing & open area

Well screen can be installed using a carrier casing as at Superior. To do this the well screen is placed inside a larger diameter plain casing (i.e. with no perforations or slots) prior to installation. The casing and screen are then pulled into the borehole together with all tensile forces exerted on the casing and not the screen. Once installed, the screen is held in place whilst the carrier casing is pulled out. Use of a carrier casing has a number of advantages.

- As all tensile forces are exerted on the casing, the tensile strength of the screen can be much lower, allowing the screen design to focus on fluid extraction properties (e.g. a larger open area).
- The casing prevents the smearing of low permeability materials over the slots as the screen is pulled in.
- During removal of the carrier casing, waste may collapse into the void left behind, thereby loosening the material immediately surrounding the well screen and increasing the local permeability (provided cuttings have been removed during the pilot and reaming stages).

The open areas of the well screens used were low; 0.16% & 0.5% at Livingston, and 3% at Superior despite the use of a carrier casing [Friend 1999, King 1999]. The project managers were not concerned that the open area was small, arguing that the low permeability of the waste would be the limiting factor for flow rates so that the open area of the screen would not need to be large. However, Figure 4.8, a photograph of the 0.5% open area Livingston screen, indicates that the perforations are very widely spaced (approx. one perforation per $0.02m^2$). The likelihood of any permeable pathways in the waste matching up with a perforation in the well screen is therefore limited, i.e. well losses will be high. Further, it is probable that many perforations will become covered by sheet like materials such as plastic bags. With a larger open area, or at least a more even distribution of the same open area, permeable channels are more likely to match up with perforations. In addition, jets used during well development would rarely match up with the sparse perforations. For these reasons a well screen with a large and evenly distributed open area was recommended for use in the second Rainham trial. Finally, it was apparent that the project managers for the Midwestern projects had give little thought to the influence of aperture size on the control of fines.

Figure 4.8 100mm diameter steel well screen for the 2nd Livingston project, 9.5mm holes, open area 0.5%.

4.2.2.7 Well development & flow rates

Development of the first Livingston well was by means of a jetting tool attached to the front of the pilot drilling rods. This assembly was passed through the well screen while water was pumped through the rods to the jetting tool. During jetting, leachate, water and drilling fluid were pumped out of the well from the exit end at a rate faster than the jetting tool supplied. This *overpumping* promoted a net flow of diluted leachate from the well, carrying with it drilling fluid and fines from both the well itself and the surrounding waste. After the well had been operational for thirteen weeks a CCTV inspection found possible biofouling around the perforations. Jetting with bleach solution using standard sewer jetting equipment removed the deposits and "a notable improvement in yield was observed" [Friend, 1999], although no quantification was available. For the first nineteen weeks of pumping, the horizontal well yield averaged $0.27m^3$/day but for the following twenty-five weeks only $0.16m^3$/day. However, no significant effects on piezometric levels were observed over this period [Friend & Hock, 1998]. The observation wells were fully screened and may have failed to respond to differential pore pressure reductions at different depths or in discrete horizons. A series of contractual changes following the publication of these results has led to a failure to operate the two wells installed in 1999 to the present date (February 2002).

Development of the Superior well used standard sewer jetting equipment and calcium hypochlorite solution to break up and degrade the drilling fluid. Bruxvoort *et al* (1998) reported that flow rates averaged $13m^3$/day during the first two months of operation and then reduced to a stable $4m^3$/day for the following ten months. Piezometric levels were monitored using fifteen vertical wells. The average reduction in level was 1m after one year of pumping with a 4m reduction observed in one vertical well. As at Livingston, these observation wells were fully screened which may have masked changes in individual horizons. However, the average reduction was enough to regain the former inward hydraulic gradient and so the project objectives were met. Finally, Bruxvoort *et al* (1998) suggested that the influence of a horizontal well operating in MSW can extend up to 30m laterally.

4.2.3 Summary of Recommendations from the Midwest Trials

From the above discussion of the successes and failures of the Midwestern projects a number of recommendations were made for the second Rainham trial. These are summarised below.

1. The need to resolve the issue of borehole closure was again illustrated.
2. Although success may be achieved with a midi-rig it is advisable to use a maxi-rig, i.e. with a minimum pullback capacity of 800kN.
3. To ensure that the horizontal wells are installed wholly within the waste mass and do not encounter the site base, the target elevation should be raised by one metre (to 3mOD).

4. Drilling tools, especially the back-reamer, should be designed so as to minimise the chance of snagging on waste materials that can become tangled around the drilling assembly.
5. The TruTracker™ wireline system appears to be the best option for guiding the pilot borehole.
6. Biodegradable drilling fluid should be used initially to try and convey cuttings from the borehole. However, if no cuttings return then water alone may suffice. Bentonite based fluids should be avoided.
7. HDPE well screen should be avoided as it has low tensile strength. Steel well screen should be used in preference.
8. Use of a casing will reduce both the tensile strength requirements of the well screen and the likelihood of smearing during installation. However, the diameter of the borehole must be larger to accommodate the carrier casing.
9. The diameter of the well screen or carrier casing should be minimised in order to reduce frictional resistance during installation. 100mm well screen is perhaps the smallest diameter that will accommodate submersible pumps and development tools.
10. The open area of the well screen should be maximised and evenly distributed to minimise well losses. Selection of the aperture size should be justified with reference to the particle size distribution of the waste.
11. Thorough well development should be attempted and, where possible, the well screen should be designed to assist development.

4.3 Second Rainham Field Trial

4.3.1 The Overwashing Technique

During the preliminary investigations into directional drilling, a technique called *overwashing* had been discussed with a German drilling contractor named FlowTex (now DrillTec). Overwashing was considered to be more expensive as the drilling equipment it required was less conventional. Examples of wells completed using overwashing were limited, with most drilling companies offering only the backreaming technique as used in all the trials referenced above. However, overwashing offers the chance to limit the degree to which the borehole is able to close. The stages of overwashing are outlined below.

Stage 1 – Pilot Borehole

The drill rods are advanced through the formation using the same drilling assembly, guidance systems and fluids as in the backreaming technique. However, overwashing permits the installation of *blind* wells, i.e. wells that do not continue to surface at an exit point. The pilot borehole can therefore stop where required, for example, in ground remediation projects this might be at the end of a contaminant plume. If desired, the pilot borehole can continue to the surface. Blind wells are most commonly used where there is nowhere for an exit hole, e.g. underneath a building.

Stage 2 – Installation of Overwash Casing

The overwash casing is essentially a solid walled pipe with a cutting shoe at the leading end. The drill rods fit inside the overwash casing and act to guide the casing as it is drilled into the borehole, as in Figure 4.9. As the overwash casing cannot be steered directly, the preceding pilot stage is essential. Drilling fluids are pumped into the annulus between the overwash casing and the rods both for lubrication and to ensure the inside of the casing remains free of cuttings. As the cutting shoe is of a larger diameter than the rest of the casing, a small annulus is created to allow fluids and cuttings to return along the casing exterior (provided the borehole does not close). When the overwash casing has been inserted into the ground the same distance as the drill rods, the overwash stage is complete.

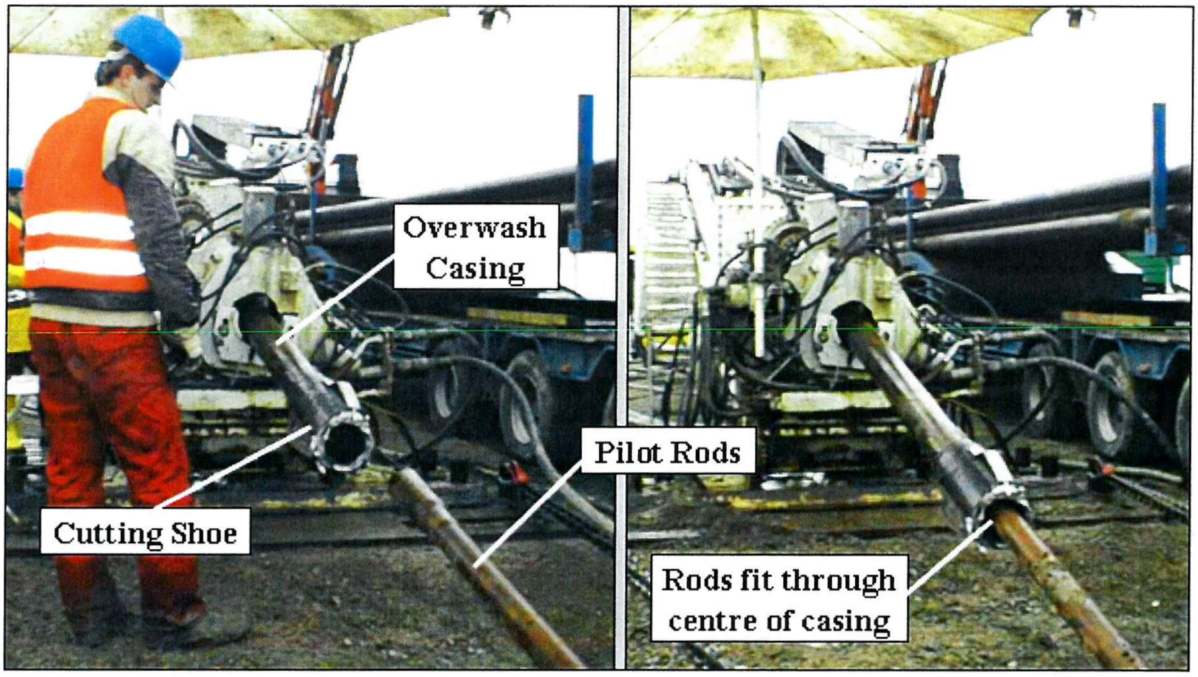

Figure 4.9 a) Drill rods left in ground following pilot drilling, overwash casing is connected to drilling rig. b) overwash casing fits over drill rods and is guided through borehole [DrillTec, Rainham 2000].

Stage 3 – Removal of Drill Rods

The drill rig is disconnected from the overwash casing and reconnected to the protruding end of the drill rods. These rods are then withdrawn leaving the casing in the ground to support the borehole walls. The degree to which the borehole can close is limited to the difference in diameter between the cutting shoe and the overwash casing.

Stage 4 – Installation of Well Screen

The chosen well screen is then pushed into the casing. If the casing remains free of cuttings, stresses should be minimal. A sprung anchor is placed on the leading end of the casing which opens, embedding itself in the surrounding ground, as it passes out of the end of the casing. This prevents the well screen from being pulled from the borehole in the final stage.

Stage 5 – Overwash Casing Removal

Finally, the rig is connected to the overwash casing which is pulled from the borehole, leaving the well screen in place.

Advantages of the overwash method over the more conventional backreaming method are listed below in order of relative importance for the second Rainham trial.

1. Borehole closure is limited to the difference in diameter between the overwash casing and the cutting shoe.
2. If an impassable obstruction is encountered and the borehole must be terminated short of the target length, well screen may still be installed over the length drilled so far.
3. As no significant pulling force is exerted on the well screen at any stage it does not require any great tensile strength, allowing the design to focus on fluid extraction properties.
4. Without the need for an exit section the wells can be reduced in length, creating a considerable cost saving in terms of time and materials.

The only major disadvantage associated with overwashing is that blind-ended wells have, obviously, only one accessible end. This limits the options for well development and installation of pumping systems. Moreover, should the well become blocked or broken near the accessible end, the entire well may be rendered useless. However, overwashing can be used to complete continuous, two-ended, wells by simply continuing the pilot borehole and overwash stage to an exit point at the surface.

4.3.2 Specifications for Design & Installation

For the reasons given above it was decided to adopt the overwashing technique for the second Rainham trial. The specifications for installation and well design are given in Table 4.3. Many of these result directly from the recommendations discussed above, those marked with * are considered below.

Table 4.3 Specifications for second Rainham trial

Drilling contractor*	DrillTec GmbH
Drilling method	Overwashing
Number of wells*	3
Length of each well*	250m
Blind / Continuous*	Blind
Well profile*	Gradual curve down to 3mOD over first 50m, remaining horizontal until 250m

Drill rig size		Maxi Rig (min. 800kN pullback)
Drill bit type		Tri-cone
Guidance system		Wireline (TruTracker™)
Drilling fluid additive		Biodegradable or water alone
Target elevation of horizontal section (equivalent max. depth below ground)		3mOD (28m)
Overwash casing material & diameter*		Steel, 150mm
Well Screen*	*Material*	Stainless steel wire wrap (slotted)
	Diameter	114mm
	Length	200m (first section 50m plain casing)
	Slot Width	1mm, 2mm & 5mm (different each well)
	Open Area (m^2)	$20m^2$, $32m^2$, & $48m^2$ respectively
	Open Area (%)	29%, 44% & 67% respectively

The original proposal was to install two wells, one of 400m length and the second of 550m length. These lengths were determined from the distance required to drill from one side of the Phase 2 waste 'dome' to the other. With overwashing, the option now existed of drilling shorter, blind wells. Although two-ended continuous wells are preferable to blind wells, it was considered that the shorter the length of the well, the greater the chance of a successful installation. Furthermore, the money saved by drilling shorter wells (materials and time) could be used to install a third well. Three wells would provide a better degree of redundancy and/or a significantly more robust data set than two wells.

250m was the minimum length required to reach to below the centre of the Phase 2 dome. The first 50m of each well was designed to curve gradually from the entry point (between 7.2 and 10.3mOD), becoming horizontal at 3mOD. From 50m to 250m the wells were to remain horizontal at 3mOD. In plan view the wells would be straight, although owing to the proximity of the site base any left-right deviation was of little concern in comparison with deviations in elevation. The first 50m sections were to be completed with plain casing. This should allow a natural seal to form between the waste and the well, ensuring fluids did not leak along the casing sides or allow rainwater to infiltrate. From 50m to 250m the boreholes were to be completed with slotted well screen.

The diameter of the overwash casing was minimised to reduce friction in the borehole. However, to permit installation of the 114mm diameter well screen, the casing had to be at least 150mm in diameter. Steel was chosen as the material for the overwash casing to ensure the casing had adequate strength to withstand both installation and withdrawal from all three boreholes. As the casing would not be left in the waste for any significant period of time, corrosion resistance was not a factor and so mild steel was used.

The specific locations for the horizontal wells, shown in Appendix E, were chosen according to three principal criteria:

1. maximising their distance from existing vertical wells, trenches and other fluid extraction systems, including each other,
2. availability of a suitable working area for the drill rig, and
3. the ability to reach areas where leachate levels were highest, so that any influence of the wells would be readily identifiable.

DrillTec were selected as the drilling contractor for the well installations. DrillTec fully acknowledged the research nature of the project and were keen to assist during the design stage.

4.3.3 Well Screen Selection

Selection of a suitable well screen for the first Rainham trial had been difficult. Indeed, as no horizontal well installations had previously been attempted in the UK a suitable off-the-shelf product could not be readily located. In haste to start the first trial, steel pipe with crudely drilled perforations was used (Figure 4.7). The well screen used in the Livingston County trials was also a solid walled steel pipe with crude perforations, Figure 4.8. A US EPA report (2001) states that although pipes perforated with cutting torches or drills may substitute for manufactured well screen, they have the following limitations:

- a wide spacing is required between slots to maintain structural strength;
- perforations may vary in size;
- the percentage open area is usually very low;

- it is difficult if not impossible to cut openings small enough to retain fine sand; and
- the jagged edges around the perforations are more susceptible to corrosion.

In contrast, a great deal of consideration was given to the well screen to be used for the second Rainham trial. The following criteria proposed by Wampler (1997) were used as a basis for selection.

1. The flow restrictions across the well screen should be minimised, e.g. by maximising the open area.

2. The design should resist clogging by sediments, corrosion, mineral deposits and microorganisms. Slot size must be large enough to permit adequate development and not to impede flow, yet small enough to prevent the continual intake of fine grained sediments.

3. The well screen should resist chemical degradation from in situ contaminants.

4. The hydraulic connection with the surrounding aquifer should be optimised. Performance is generally enhanced when a well graded* filter pack, either placed during construction or created during development, fills the gap between the well screen and the borehole walls.

Wampler argues that although these criteria have been largely addressed for the construction of vertical wells, they do not transfer easily to horizontal wells due to the differences in construction techniques. Additional factors that require consideration are the tensile stresses experienced during installation (if using the back-reaming technique) and the constraints of a horizontal configuration on well development methods. In blind wells, compressive strength becomes a factor as the screen must be pushed into the overwash casing. Wampler adds that resistance to crushing by overburden soils (collapse strength) is important if the installation is very shallow or the formation is poorly consolidated (as in landfill).

In many vertical wells the well screen is situated in a borehole of a larger diameter with a graded filter pack filling the annulus which is designed to prevent fines from entering the screen. The function of a filter pack is twofold:

1. To allow water to drain freely out of the formation.

2. To support the formation and prevent it from being eroded by the water which drains out of it [Powrie, 1997].

The filter material gradation should be selected on the basis of the particle size distribution (PSD) of the formation, and the well screen slot size based on the PSD of the filter material. However, in a horizontal well the screen will rest on the borehole invert, precluding installation of a filter pack that fully surrounds the screen. Centralisers can be used to raise the screen off the borehole invert but these must be closely spaced and can cause problems with friction during installation. Since gravity will not move the chosen filter material into the horizontal borehole it must be placed using an injection device or a tremie pipe. This process is made more difficult when centralisers are in place. For these reasons, and the additional problems of borehole closure experienced in the trials, it was decided that the installation of a filter pack would be impractical. However, a number of well screen designs include an integrated filtration system to eliminate the need for an external filter pack. Three such screens were considered and are discussed below.

Schumasoil®. The well screen wall is made from sintered polyethylene resin beads to resemble a natural porous medium (Figure 4.10). The size of the resin beads determines the porosity and permeability. The open area is evenly spread and can be up to 36%. However Schumasoil is generally not very robust, with a maximum tensile strength of only 13.5kN [Schumacher GmbH, 1997]. Despite the protection that would be provided by the overwash casing, such fragile materials were not favoured.

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* Well Graded filter packs include a range of particle sizes as opposed to uniform packs which consist mostly of similar sized particles.

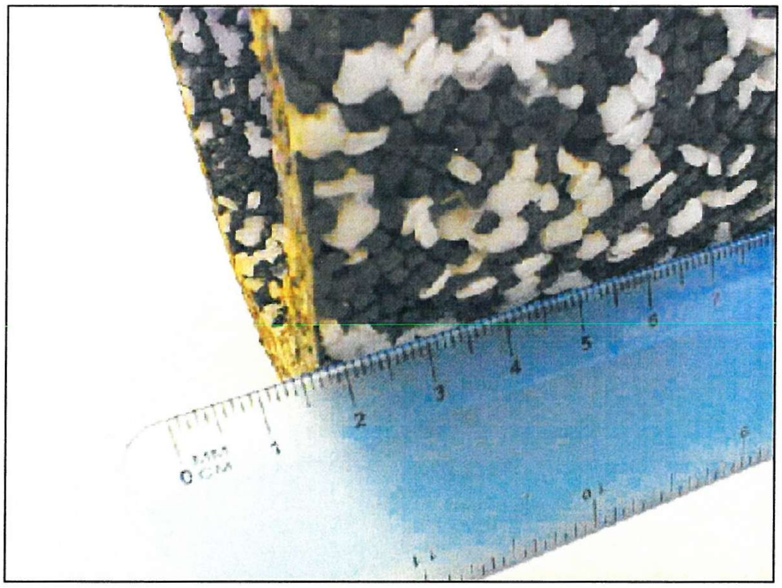

Figure 4.10 Schumasoil. Individual polyethylene resin beads can be seen.

Glass Reinforced Plastic (GRP) with gravel filter. This well screen is constructed of an epoxy bonded gravel, sandwiched between two layers of glass reinforced plastic. The porosity of the well screen is determined by the size of gravel grains used, although the maximum open area of the GRP layers is 10%. The maximum tensile strength is 80kN and compressive strength 30kN. As with Schumasoil, the low strength makes GRP unsuitable.

Stratapac® (and similar) screens use several layers of stainless steel mesh, bonded between inner and outer perforated metal casings, Figure 4.11. The all steel construction is robust and resilient to damage during installation; for a 100mm diameter screen the tensile strength is approximately 900kN, and collapse strength 50MPa [Oiltools International, 1999]. However, it is relatively heavy and stiff and (for this project) prohibitively expensive.

Figure 4.11 Stratapac well screen showing layers of stainless steel mesh and casing [Source, Pall Corporation].

Having ruled out (a) the installation of a filter material and (b) well screens that use an integrated filter system, a 'natural filter pack' remained the only option, i.e. one that is formed from the constituent materials of the waste. Well development would be used to draw the finer material from the surrounding zone into the well, leaving the coarser material in place. Ideally, after the initial stage of normal well operation, the developed natural filter pack would prevent the continued loss of fines.

A number of researchers have provided guidelines for the sizing of filter pack material but few consider aperture size selection for screens installed without filter packs. For a natural filter pack, the Hong Kong Geotechnical Engineering Office (1993) suggested that the *uniformity coefficient*

$$C_u = D_{60}/D_{10} \tag{4.7}$$

should lie within the range

$$4 \leq D_{60}/D_{10} \leq 20 \tag{4.8}$$

The smaller the C_u, the more uniform the material. From the Rainham waste PSD shown in Figure 3.24, the C_u is 36, which lies outside the range given in equation (4.8). The natural filter pack may therefore prove ineffective at preventing the migration of fines. In addition, materials with higher C_u values will have a more tortuous flow paths and therefore lower hydraulic conductivities. Also, waste particles might get continuously smaller as waste degrades.

Somerville (1986) suggests that where the natural soil contains a high proportion of gravel or larger sized particles (70% at Rainham), the filter should be designed on the basis of the grading curve of the portion finer than 19mm. The PSD curve in Figure 4.12. was redrawn from Figure 3.24, so that 19mm represents the D_{100} value.

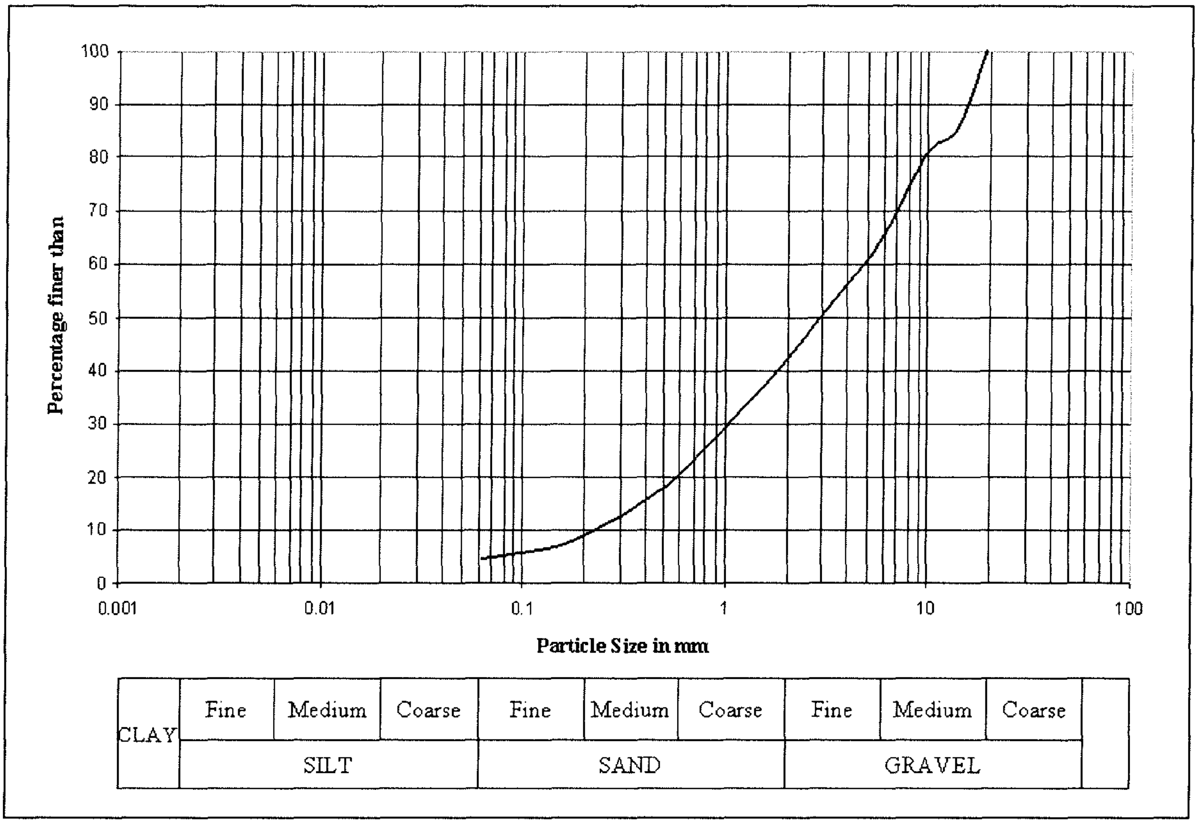

Figure 4.12 PSD for fine material (≤19mm) from Rainham waste.

Applying equation (4.7) to Figure 4.12 gives a C_u of 22.86 which lies close to but still outside the range in equation (4.8), so the migration of some fines into the well seems likely. Somerville (1986) also suggests that if a filter material is to be placed against a slotted well screen, the D_{85f}* should be greater than twice the maximum slot width. From Figure 4.12 the D_{85f} is 12mm and the maximum slot width should therefore be less than 6mm.

Walton (1962) suggested that where natural filter packs are (a) heterogeneous, (b) have a C_u greater than 6, and (c) the aquifer is soft, then the well screen aperture size should be equal to the D_{50} grain size of the filter pack. The Rainham waste fits these criteria and has a D_{50} grain size of 7.3mm (for the full PSD). In addition, Walton recommends that if the zone of the formation to be screened shows a variability due to stratification or other causes, then the D_{50} for the 'finest zone' should be used to determine the slot width. The D_{50} of the fraction less than 19mm, is 3mm (Figure 4.12).

* The D_{85f} is the D_{85} grain size of the filter material, or in this case the natural filter pack.

The above guidelines were intended for application in natural soils, not waste materials. In addition, while the guidelines provided a useful indication for the selection of aperture size there remained a considerable variation in the values.

It was therefore decided that each of the three horizontal wells would be completed with a different slot width, enabling comparisons to be made. In the first instance, a 1mm slot width was chosen as this should prevent the migration of fines suggested by the high C_u value of the waste. The two other aperture sizes selected, 2mm and 5mm, were considered representative of the values calculated above. It was thought that any variations in well performance caused by factors other than those related to slot width, for example local variations in hydraulic conductivity, would be averaged out over the considerable length of the wells (200m of screen).

Of the various well screen types available the product selected was Vee-wire®, a continuous slot screen made by Johnson. This is a 'wire wrap' screen manufactured using a single long wire that is wrapped spirally around an array of longitudinal support rods, leaving a narrow space of specified width between each wrap, thus creating a continuous slot opening (Figure 4.13a). The continuous slot opening, combined with the narrow width of the wire, creates a large percentage open area. The V-shape profile of the wire ensures the outermost part of the slot is the most narrow, so a particle that passes into the slot is unlikely to become lodged as it passes through the slot, as illustrated in Figure 4.13b [Johnson Screens, 2000].

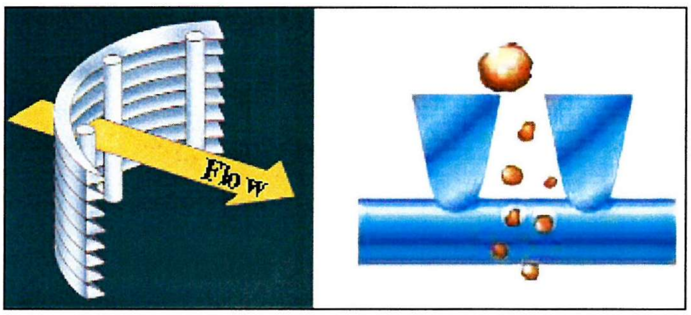

Figure 4.13 a) Wire wrap well screen, b) V-shape profile of wire [Source, Johnson Screens, 2000].

The tensile strength of 114mm diameter Vee-wire is 120kN. The column (or compressive) strength is 60kN and the collapse strength varies with slot width between 1600 kPa for 1mm screen to 248kPa for 5mm screen [Johnson Screens, 1999]. The tensile and compressive strengths were not as great as had been desired, but the use of overwash casing was expected to minimise the stress exerted on the screen. As the screen sections are manufactured in 6m lengths and joined using threaded couplings, the problems associated with welding in the presence of landfill gas and/or manoeuvring long sections of well screen around the landfill, would be avoided.

The wire wrap design offered the highest open area of any well screen [Roscoe Moss Company, 1990]. With reference to soils, Brandon (1986), states that a high open area is important as the *effective* open area is generally one half to one third of the total as a result of blocking by the formation material. It is thought that the presence of sheet like materials in waste, such as plastic bags, may reduce the effective open area further still. The open area of wire wrap screen is distributed evenly (unlike the first Rainham and Livingston trials) and well losses should therefore be reduced.

Wire wrap screen can be manufactured in either 304 or 316-L grade stainless steel, which have different corrosion resistance properties. Williams (1999) conducted a study into the corrosion rates of five steel types commonly used for well screens. In this study three 'coupons' of each steel type were placed in an inactive water production well which had been removed from service due to corrosion problems*. The 'coupons' were tested for weight loss over an eleven month test period. The results are shown in Table 4.4.

Table 4.4 Corrosion rates of five common steel types.

Steel Type	Metal Loss (mg/year)	Corrosion Resistance Factor
316-L stainless	0.0061	× 472
304 stainless	0.0118	× 244
Low alloy (ASTM type 4)	0.3131	× 9
Copper-bearing	0.7438	× 4
Mild	2.8794	× 1

Data reproduced from Williams (1999).

Metal loss from 304 was twice that of 316-L. The higher molybdenum and nickel content and the lower chromium content of 316-L make it substantially more resistant to corrosion than

* It is not clear what substances in the water well were responsible for the corrosion and it should be noted that these may have been very different to those in the landfill.

304. This may prove crucial for well screen longevity during long-term exposure to leachate at temperatures ranging from 20 to 60°C. As a result 316-L grade stainless steel was selected in preference to 304.

4.3.4 Drilling & Installation

At the entry point of each well, foundations of crushed concrete were laid to create a firm, even and dry working surface for the loading crane, drilling rig and operatives. The existing surfaces were unsuitable, consisting of waste overlain by a temporary cap of silt.

DrillTec arrived at Rainham in March 2000 and set up at the entry point for horizontal well number 3 (HW3)*, Figure 4.14 to Figure 4.16. Most of the difficulties with the drilling and installation processes were encountered at HW3, an account of which is given below. Learning from the experiences of HW3, the two remaining wells, HW1 and HW2, were less problematic and only the significant events which occurred during their installation are discussed below.

* The wells are numbered according to their position around the site and not in the order of which they were installed, or the well screen slot width used.

Figure 4.14 Setting up drilling rig and ancillary equipment at HW3.

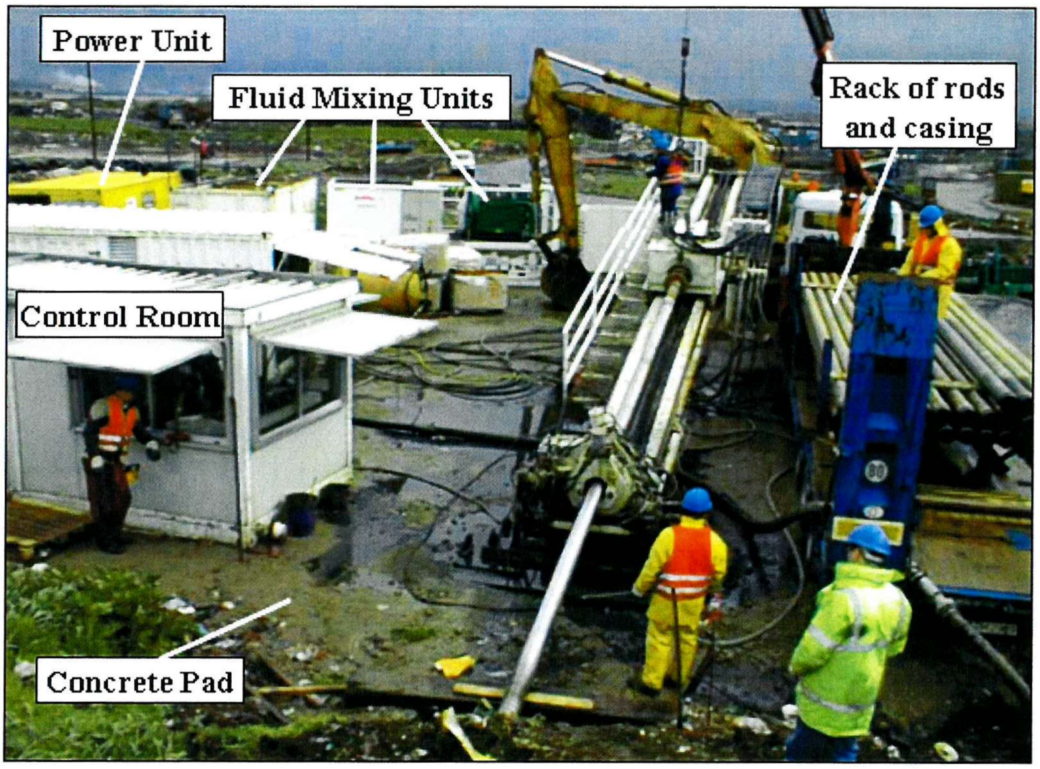

Figure 4.15 Site set up for installation of HW1.

Figure 4.16 1000kN pullback capacity drilling rig at HW3.

4.3.4.1 HW3 Pilot Borehole

Once the rig had been aligned and the TruTracker™ guidance system set up, pilot drilling began with a blunt-end drill bit* at an entry angle of 11°, Figure 4.17a. This bit relied upon an angled high pressure jet of water to cut through the waste, Figure 4.17c. After 27m of drilling an obstruction was encountered that caused the leading 2m of the drill bit to shear off, Figure 4.17d. A tri-cone reamer was obtained, Figure 4.18, and pilot drilling recommenced. At 2.5mOD, after 46m of drilling, soft material was encountered that allowed rapid progression of the drilling assembly. This was thought to be the silt base although at a higher level than predicted. The rods were withdrawn to 16m and an attempt was made to redirect the borehole. However, the drilling assembly refused to steer out of the preferential pathway that had already been created. This difficulty in steering may have been compounded by the soft, slurry-like material present in the saturated horizons as identified in the site investigation. The rods were completely withdrawn and a new borehole was commenced along a slightly different line. Soft sections were again encountered and the initial thought was that the silt

* A tri-cone drill bit had been specified in accordance with earlier recommendations, but this had been overlooked by the drilling contractor.

base had been struck, however, harder sections followed confirming the borehole remained or had moved back into the waste. The pilot borehole proceeded without serious obstruction to the full 250m length. Figure 4.40 plots the profile of HW3, drawn from the TruTracker™ readouts.

Figure 4.17 (a) Blunt-end drill bit with jetting nozzle, (b) aligning drill bit for guidance system, (c) jetting at start of pilot borehole, also range-rods for TruTracker, (d) blunt-end of drill bit sheared off.

Figure 4.18 Tri-cone reamer that replaced the 'blunt-end' drill bit.

The TruTracker™ guidance system performed satisfactorily although the waste created a high level of interference, more so than the operator had previously experienced on other projects. In conventional projects, one coil of wire is usually sufficient to surround a 250m borehole, however, to limit the reduction in current strength, four shorter coils were used. To further increase the intensity of the induced magnetic field, and thereby proportionately reduce the effect of interference, the surface wire coils were double-looped enabling the elevation to be determined with an accuracy of ±0.5m.

Biodegradable drilling fluids were used initially* but after approximately 13m it became apparent that no fluids (or cuttings) would be flushed from the borehole. As indicated in section 4.1.4 it was decided that water alone would be sufficient to drive the tri-cone and to cool and lubricate the drilling assembly. The water used (200 to 800 l/min) was drawn from the River Thames, which is brackish where it passes Rainham. Total volumes are discussed in section 4.3.5.

4.3.4.2 HW3 Overwash Stage

The cutting shoe illustrated in Figure 4.19 was fitted to the overwash casing. This had eleven teeth at the leading end designed to cut through the formation. The assembly was guided into the borehole with the drill rods running centrally through the overwash casing as shown in Figure 4.9. As the casing was drilled into the ground, drilling fluids were flushed into the casing at a rate of 700 to 2400 l/min to keep it free from waste (Figure 4.20). In conventional overwashing projects, the fluids (and cuttings) would be expected to return along the exterior of the casing via the annulus created by the passage of the cutting bit. However, fluids only returned whilst the cutting shoe was within 21m of the entry point and once returns ceased water alone was used. The use of water meant that any residue left by a biodegradable drilling fluid would not have to be removed during well development.

After the casing had been advanced 30m it was discovered that friction between the overwash casing and the internal drill rods had pushed the rods 9m further into the landfill. The only way to retrieve these drill rods was to completely withdraw the overwash casing and excavate a pit (2m deep) to find the end of the rods (Figure 4.21). On removal, the overwash cutting bit was found to be wrapped in fabrics and plastics (Figure 4.22) and all eleven cutting teeth had broken off, leaving only a rough edge to perform the cutting action. Once the drill rods had been located, an additional rod was added so that the end of the string was once again above ground level. A screw-thread coupling was attached to the protruding end of this final rod so that should the problem recur, the string of drill rods could be simply 'fished-out' using other drill rods (Figure 4.24).

Overwashing recommenced and continued without obstruction. Initially, the overwash casing was advanced at a rate of 1m/min. However, progression became increasingly slow and at a distance of 156m had reduced to 0.15m/min. The overwash casing was again withdrawn and the front of the drill bit was found to be very smooth as a result of wear against waste materials (Figure 4.23). Small tungsten carbide pieces were welded to the front of the drill bit to provide a cutting edge before the third overwashing attempt commenced.

The third attempt successfully reached 250m with little difficulty, although progress again became slower with increasing distance (0.08m/min at the end). This slowing may have been a combination of the tungsten carbide wearing smooth and a gradual increase in the thrust and torque required to overcome frictional resistance between the casing and waste. The rig had a thrust force of 320kN and a torque rating of 45,000 Nm, but higher capacities may have enabled more rapid progress.

From early on in the overwash stage, water flowed back up the overwash casing when pumping stopped to add each new length (Figure 4.24 and Figure 4.25). Fluids were only able to pass from the waste into the casing via the 150mm diameter opening at the leading end. This flow was therefore indicative of the high pore pressures in the waste which were no doubt increased by the addition of drilling fluid.

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* The fluid additive used was *Quicktrol* from Baroid Drilling Fluids Inc. who describe the additive as a 'modified natural cellulosic polymer'.

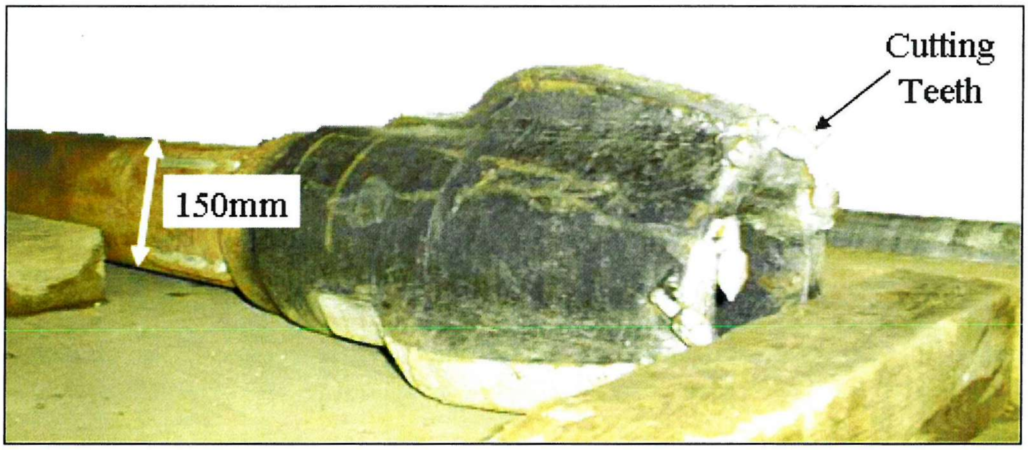

Figure 4.19 Overwash cutting bit, internal diameter 150mm, shows protruding cutting teeth.

Figure 4.20 Overwash bit about to pierce the ground surface, drilling fluids being pumped into the annulus between overwash casing and drill rods.

Figure 4.21 A 2m deep pit was excavated to find the end of the drill rods which had moved during overwashing.

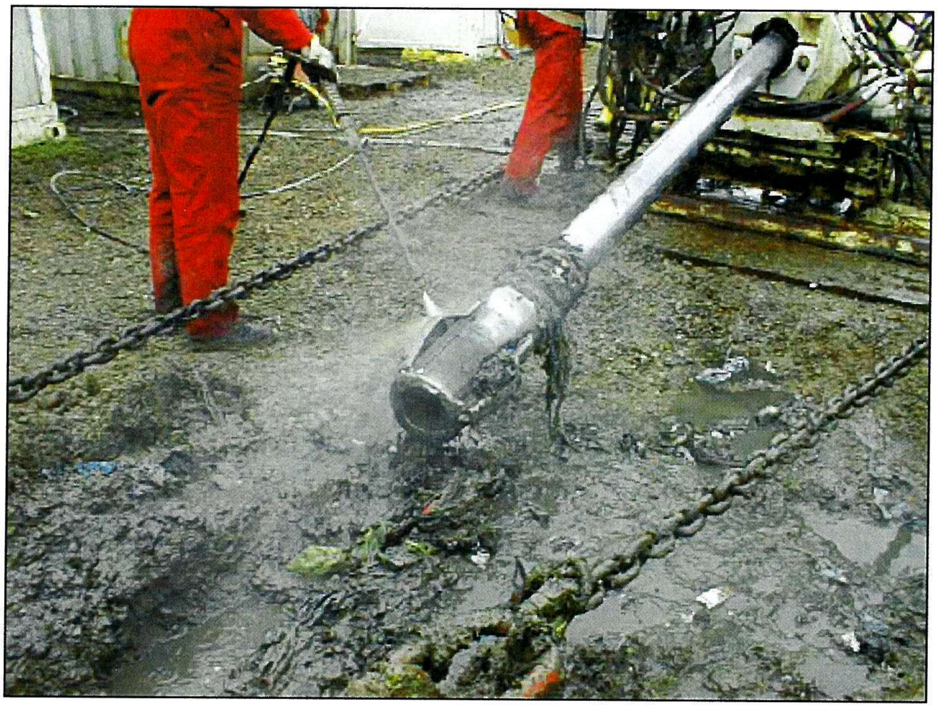

Figure 4.22 Overwash bit wrapped in fabrics and plastics, all cutting teeth had broken off.

Figure 4.23 Overwash bit had been smoothed from friction with hard waste materials (compare to Figure 4.19)

Figure 4.24 Drilling fluids and leachate flowing from overwash casing due to elevated pore pressures.

Figure 4.25 Drilling fluids and leachate flowing from overwash casing and drill rods.

4.3.4.3 HW3 Drill Rod Removal, Installation of Well Screen and Casing Removal

Once the overwash casing was fully installed, the rods were reconnected to the rig and withdrawn from inside the casing. The casing remained in the landfill preventing full borehole closure. The next stage was to install the 114mm diameter Vee-wire well screen*, Figure 4.26. On the end of the first section to be installed a sprung three leafed anchor was fitted, Figure 4.27. Once past the far end of the overwash casing, the anchor would open out to prevent the well screen being pulled from the waste during removal of the overwash casing.

To prevent cross threading and galling* of the stainless steel the 6m sections were carefully screwed together using hand tools and appropriate grease. As each section was added the well screen was fed into the overwash casing (Figure 4.28 to Figure 4.30). After 60m had been installed by hand the increasing friction between the screen and overwash casing meant that the rig was required to push in the well screen. This was done slowly to avoid exceeding the column strength of the screen. As specified, 200m of screen was placed in the most

* The 1mm slot width screen was used for HW3 as it was the only size that had been delivered at that time. Moreover, there were no reasons for assigning particular slot sizes to certain boreholes.

distant, horizontal section of the borehole, followed by 50m of plain stainless steel casing to complete the installation to the surface (Figure 4.31). A cap was fitted to the end of the overflowing well to prevent uncontrolled loss of leachate.

With the well screen in place the final stage was to remove the overwash casing, thereby exposing the screen to the waste. To remove the first 18m, the full 1000kN pullback capacity of the rig was required to overcome the frictional resistance between the casing and the surrounding waste. Milligan and Norris (1996) noted that the force required to move a pipe through soil 'tended to be larger after a rest period and was presumably related to the dissipation of the pore pressures that had been generated during movements'. Once the first 18m of overwash casing had been pulled out, the remaining length was removed with progressive ease so that only 250kN of pullback force was required after 100m had been removed. Despite the three leafed anchor, the well screen initially moved approximately 2m out of the borehole as each 9m length of overwash casing was removed. On each occasion the screen was pushed back in. The length of screen actually within the waste gradually increased and as waste closed around it, the movement each time a length of overwash casing was withdrawn, lessened and finally stopped. Some of the removed lengths of overwash casing were found to have deep gouges, Figure 4.32, probably as a result of hard objects in the waste such as metal or hardcore.

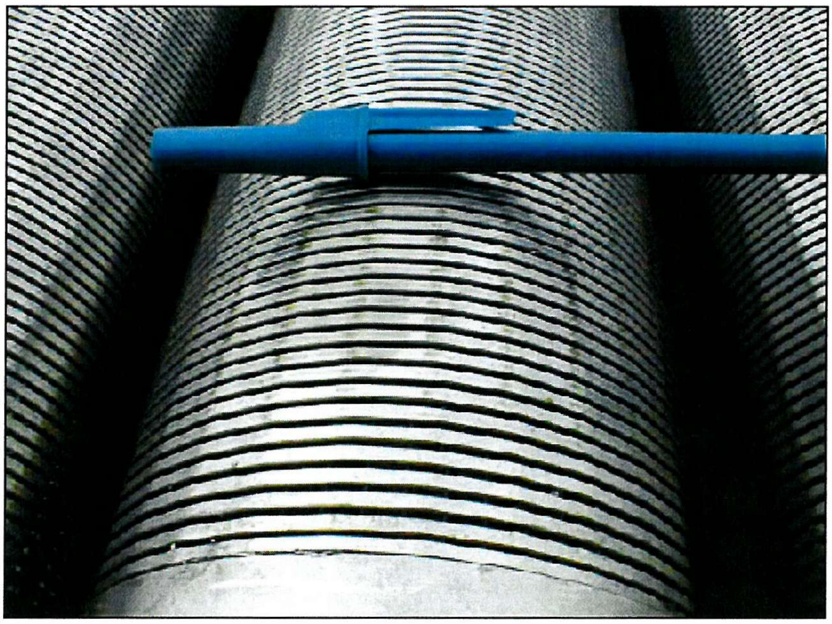

Figure 4.26 1mm slot width Vee-wire well screen. 114mm outside diameter.

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* In metallurgy, galling is a condition in which two rubbing surfaces partially weld due to friction.

Figure 4.27 Sprung anchor fitted to leading end of well screen to fix it in the waste. Leaves are folded in direction of arrows as it is pushed through the overwash casing.

Figure 4.28 Adding a new length of well screen. Remaining lengths of screen in crate on left.

Figure 4.29 Connecting screen sections by hand, applying grease to prevent galling.

Figure 4.30 Well screen being pushed into overwash casing after drill rods were removed.

Figure 4.31 200m of 1mm slot width screen were installed to the far end of the borehole followed by 50m of plain casing to the surface.

Figure 4.32 Gouges found on the 150mm diameter overwash pipe, caused by hard waste materials.

Once the overwash casing was removed, the first horizontal well (HW3) was complete. The installation had taken nine days although some of this time was occupied in setting up and resolving an electrical fault with a generator. The actual drilling and installation time was six days.

4.3.4.4 Installation of HW1

The drilling rig and ancillary equipment were relocated to the entry point of HW1 (Appendix E). Drilling followed the same general procedure as that used for HW3 but incorporated the following improvements from the start:

- use of a tri-cone drill bit (not blunt-end jetting bit),
- the guidance wire coil was double-looped to produce a more intense magnetic field,
- the pilot borehole was drilled 0.5m above the original target elevation of 3mOD to ensure the silt base was avoided,

- a threaded fitting was fixed onto the last drill rod so that the rods could be easily retrieved if they moved further into the landfill during overwashing,
- the teeth on the overwash cutting bit were replaced with a thick covering of tungsten carbide to provide a more effective cutting edge.

The water used for the drilling of HW1 was obtained from the fresh water main as the proximity of the supply made it considerably easier than using water from the Thames.

As with HW3 the rate at which the overwash casing advanced became progressively slower with increasing distance from the entry point. Indeed, the first 50m took just 1 hour, while the final 50m took 3 hours. This is again likely to be a combination of wear on the cutting bit and insufficient thrust and torque.

While the drill rods were being removed and the well screen installed, the overwash casing was periodically pulled out by 0.5m then pushed back in. This served to keep the casing loose in the waste so that it could be easily removed in the next stage, thereby avoiding the initial difficulty in pulling out the overwash casing experienced at HW3.

On removal of the final length of overwash casing from the borehole, the well screen was found to have disappeared into the waste. This was unexpected as the friction between the overwash casing and the screen should cause the screen to be pulled from the borehole (as at HW3) and not pushed further in. It was concluded that the pressure of the water being pumped into the overwash casing had shifted the well screen further into the waste despite (a) the friction acting to pull it in the opposite direction and (b) the presence of undrilled waste at the end of the borehole. Careful excavation (Figure 4.33) uncovered the end of the well screen 6m distant from the rig. To bring the end above ground level, the well cap was removed and a further 6m section of plain well casing added. Whilst the cap was off, leachate poured from the well as the entry point was around 4m below the leachate level* (Figure 4.34). This gave an early indication that the well was likely to overflow until pore pressures were lowered. The slot width used at HW1 was 2mm, Figure 4.35 and the profile of the well is shown in Figure 4.38.

Having learned from the experiences of HW3, installation of HW1 took only three and a half days (excluding set up time).

Figure 4.33 Everyone waits as the waste is excavated to find the end of the well screen.

Figure 4.34 Leachate flowing from HW1 after the cap was removed.

* The entry point of the well was at 10.7mOD, the highest leachate level in the vicinity of HW1 (in the lower waste horizons) at the time of drilling was 14.7mOD. Three other piezometers in close proximity recorded leachate levels of 10.2, 11.1 and 11.7mOD at this time.

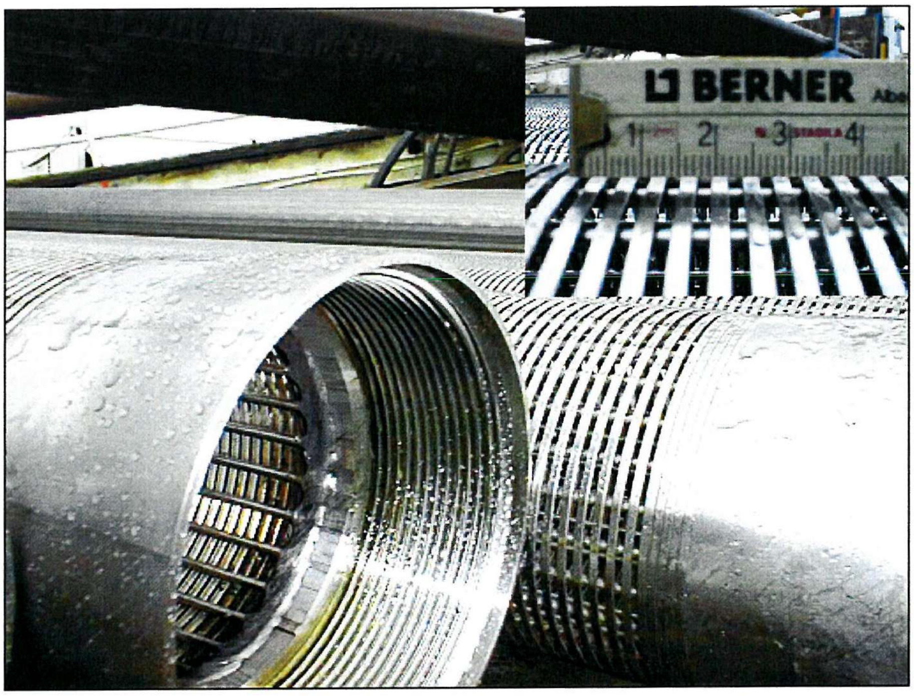

Figure 4.35 2mm slot width Vee-wire well screen used at HW1. Inset shows close up.

4.3.4.5 Installation of HW2

Following completion of HW1, the drilling rig and ancillary equipment were relocated to the entry point of HW2 (Appendix E). Due to proximity, the River Thames was again used for water supply. Drilling of the pilot borehole and the first half of the overwash stage was completed in just 13 hours. However, each length of overwash casing took progressively longer to install and as a consequence, the drill rods began to move gradually further into the waste through friction as they had at HW3. By the time a distance of 236m was reached (14m short of the target length) the near end of the drill string had moved below ground level. To avoid pushing the rods any further it was decided that overwashing would stop at this distance and the well screen would be installed. The threaded fitting on the end of the final drill rod enabled the drill string to be retrieved without excavating a pit such as the one at HW3.

Despite finishing short of the 250m target, the full 200m length of well screen was installed, and the length of plain casing reduced to 36m. This was thought to be enough to allow the waste to form a seal around the casing. The well screen slot width used for HW2 was 5mm

(Figure 4.36). As each 9m section of overwash casing was removed, the well screen shifted out of the borehole by up to 3m, as it had at HW3. On each occasion the well screen had to be pushed back into the borehole, thereby repeatedly applying compressive and tensile loads. This may have been the cause of the problems later encountered with this well, discussed in section 6.2.2.

As with HW3 and HW1 leachate flowed from the well as a result of the leachate head in the waste being higher than the elevation of the well entry point*. However, the leachate was significantly darker in colour (Figure 4.37) than the other two wells. This is thought to have been a result of the larger slot size allowing a greater volume of fines to pass through the well screen.

The experiences of the first two installations enabled HW2 to be completed in less than three days (excluding set up time). The profile of HW2 is shown in Figure 4.39.

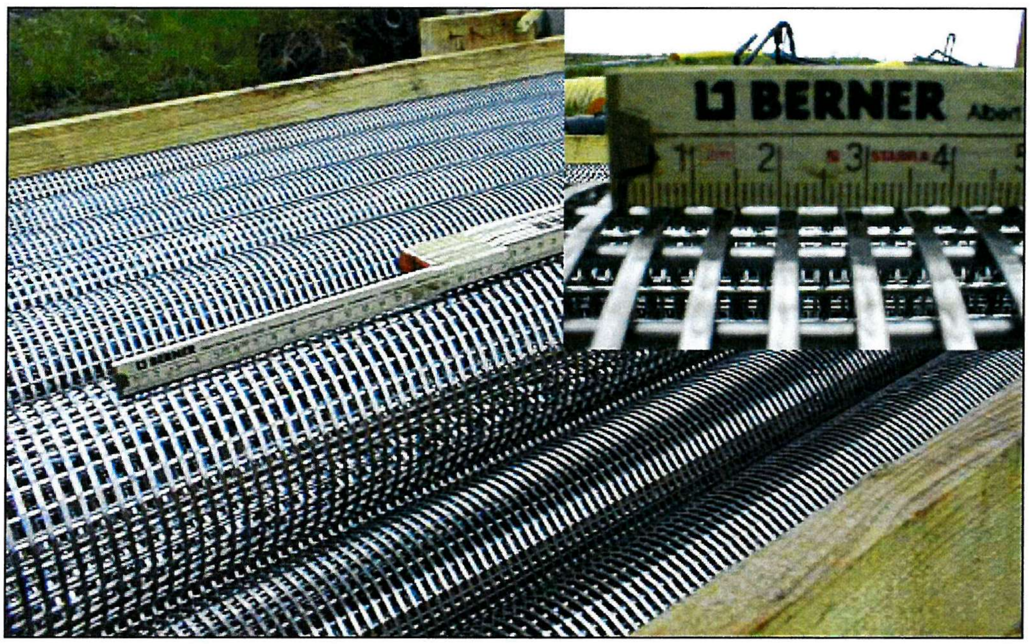

Figure 4.36 5mm slot width Vee-wire well screen used at HW2. Inset shows close up.

* The entry point of the well was at 7.5mOD, leachate levels in the lower waste horizons at a distance of 20m (nearest available measurement) were between 11.1 and 12.8mOD, at least 3.6m higher than the well entry point.

Figure 4.37 Leachate flowing from HW2 on completion. Note it is darker in colour than HW1 and HW3.

4.3.5 Installation Summary

The well installation details are summarised in Table 4.5.

Table 4.5 Summary of well installation details

Order of installation	1st	2nd	3rd
Well I.D. Number	**HW3**	**HW1**	**HW2**
Installation Time (days)	6	3.5	3
Approx. water volume used (m^3)	2000	600	750
Elevation of well opening (mOD)	10	10.8	7.5
Max. elevation of initial leachate head in lower waste horizon (mOD)	14.7	12.8*	12.3*
Av. elevation of horizontal section (mOD)	3.8	3.6	3.5
Total length of well (m)	250	256	236
Length of well screen (m)	200	200	200
Slot width of well screen (mm)	1	2	5
Open area of well screen (%)	29	44	67

Open area of well screen (m^2)	20	32	48

* These values were recorded in piezometers installed two months after installation of the horizontal wells. HW3 and HW1 (and possibly HW2) are thought to have lowered the surrounding leachate levels prior to the installation of the piezometers by redistributing the leachate from areas of high pore pressure to areas of low pore pressure. This is discussed further in section 6.8.4.

The volume of water used to install HW3 was approximately three times greater than for HW1 and HW2. This was a result of repeated attempts at pilot drilling and overwashing.

The TruTracker™ guidance system gave details of the borehole position at intervals of 9m (the length of one drill rod), which included elevation in mOD. The 'away distance' and 'left/right position' data also from the TruTracker™ system were recorded but were not converted into standard grid coordinates by the guidance system operator as requested. These coordinates had to be calculated some weeks later and may therefore have been subject to error. Conversion of the data by the operator as drilling progressed may have been more reliable. The combination of locating errors arising from the TruTracker™ system and the delayed calculation of coordinates has meant that the location of the wells in three dimensions is of limited accuracy. Evidence from pore pressure responses discussed in section 6.8.4 suggests that the location of HW1 is as determined but HW3 may be 4m north-east of its calculated position. It was not possible to install piezometers in close proximity to HW2 and the accuracy of its location cannot be confirmed.

The profiles of the horizontal wells are shown in Figure 4.38 to Figure 4.40.

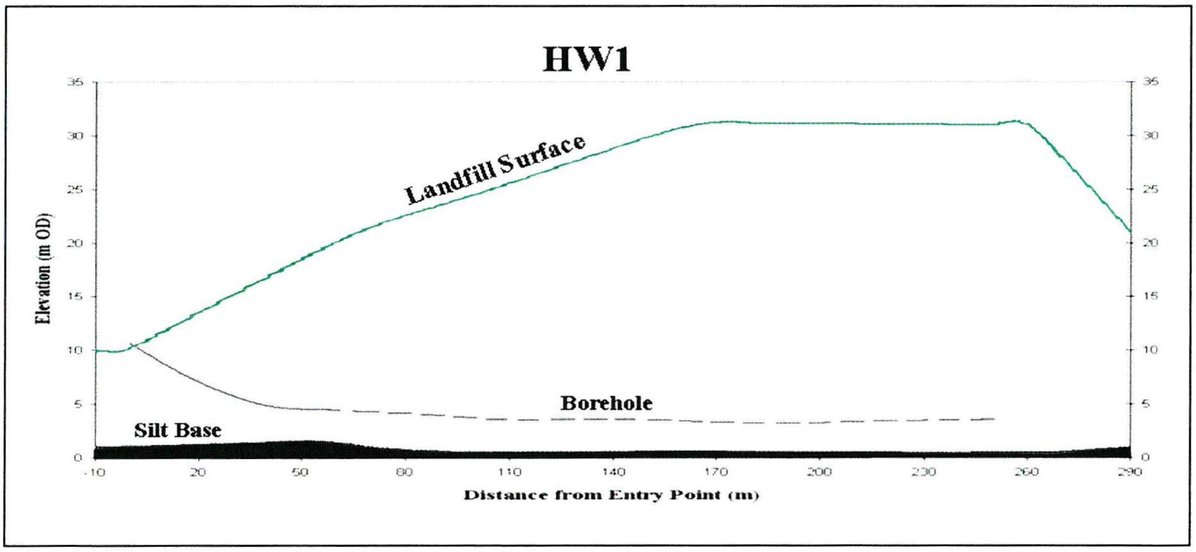

Figure 4.38 Profile of HW1, dashed line represents section completed with slotted well screen.

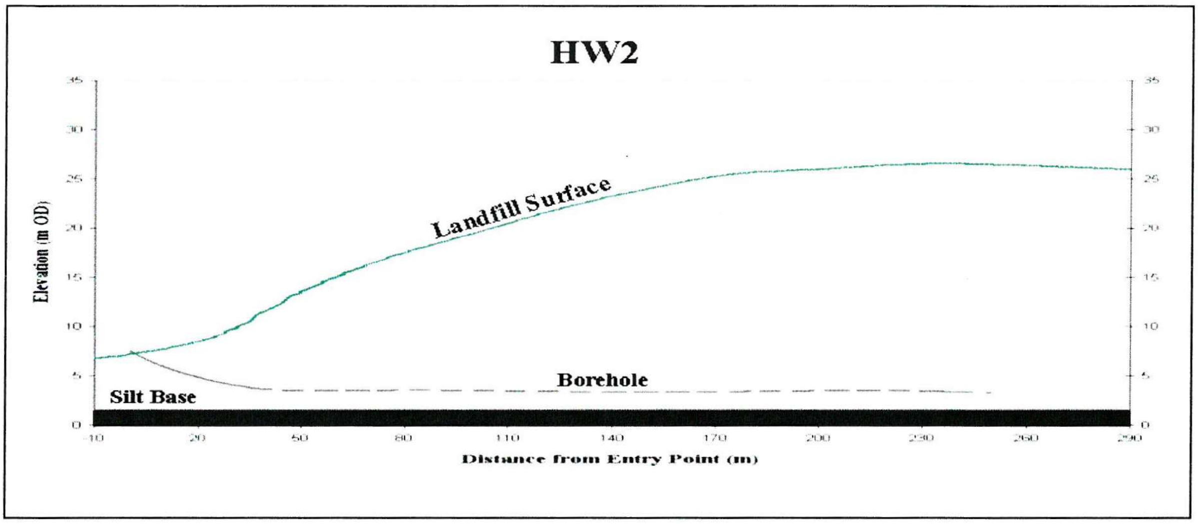

Figure 4.39 Profile of HW2

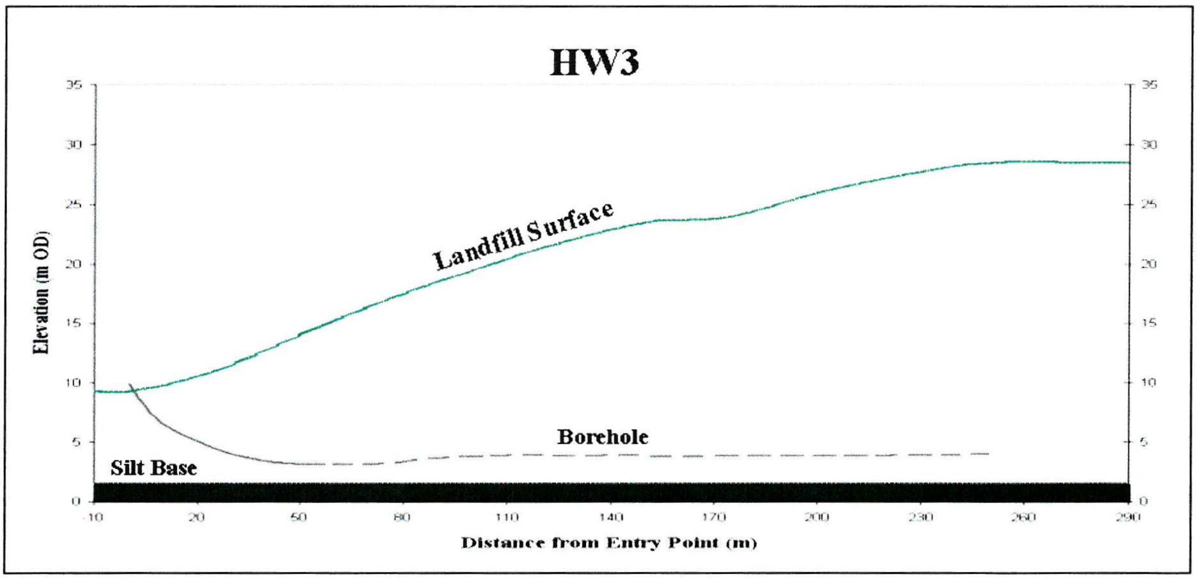

Figure 4.40 Profile of HW3

The wells are shown in plan view in Appendix E. Following installation of HW1, 2 and 3, the next phase of the project was to assess well performance. Detailed planning and design of the monitoring instrumentation and pumping equipment commenced in April 2000. This is discussed in Chapter 5. Recommendations arising from the second Rainham trial are presented in the discussion at the end of this chapter.

4.4 Metallic Tile Field Trial

While the second Rainham trial was being planned, an opportunity arose to install a shallow horizontal well for gas extraction at Metallic Tile landfill site in Staffordshire. The Cleanaway owned site was a closed site and had a permanent cap to prevent both the infiltration of rainwater and the escape of landfill gas. However, migration of gas from the site perimeter had prompted a review of the existing gas control system. The following factors meant that the installation of vertical gas extraction wells was undesirable.

1. The unsaturated horizon was shallow (5-7m deep), meaning that numerous vertical wells would be required to place a significant length of well screen in the unsaturated waste.
2. The barrel auger drilling of vertical boreholes would generate spoil and as the site contained hazardous industrial waste, the spoil would be difficult to re-dispose.
3. Installation of numerous vertical wells and extraction system pipework would cause repeated disturbance to the permanent cap which may compromise its effectiveness as a barrier to the infiltration of rainwater and the escape of gas.

In view of these factors, it was considered that a horizontal extraction well might offer an alternative as it would allow a significant length of well screen to be installed in the shallow unsaturated horizon without the need to repeatedly disturb the cap. Further, if the well could be installed without the production of cuttings (as during the first Rainham trial) the problem of re-disposal of the hazardous waste would be avoided. The site is shown in Figure 4.41 and a schematic cross section of the site is given in Figure 4.42.

The first Rainham trial suggested that installation of a small well screen, similar in diameter to the drill rods, may have been successful. It was argued that the frictional resistance acting on a small diameter screen may be manageable. To test this hypothesis a 100m long well with 62mm diameter well screen was proposed. The installation technique would be similar to that of the first Rainham trial, i.e. backreaming with a small capacity drilling rig. In order to collect gas, the well was to remain within the unsaturated zone at a target depth of 3.5m below the landfill surface.

The aims of the Metallic trial were as follows;

1. to further investigate the viability of small scale directional drilling to install horizontal wells in waste,
2. to reassess the backreaming techniques used in the first Rainham trial, modifying only the diameter of the well screen, to test the hypothesis that borehole closure is the main obstacle to successful installation,

The stages of the drilling and installation are described below and are illustrated in Figure 4.43 to Figure 4.47.

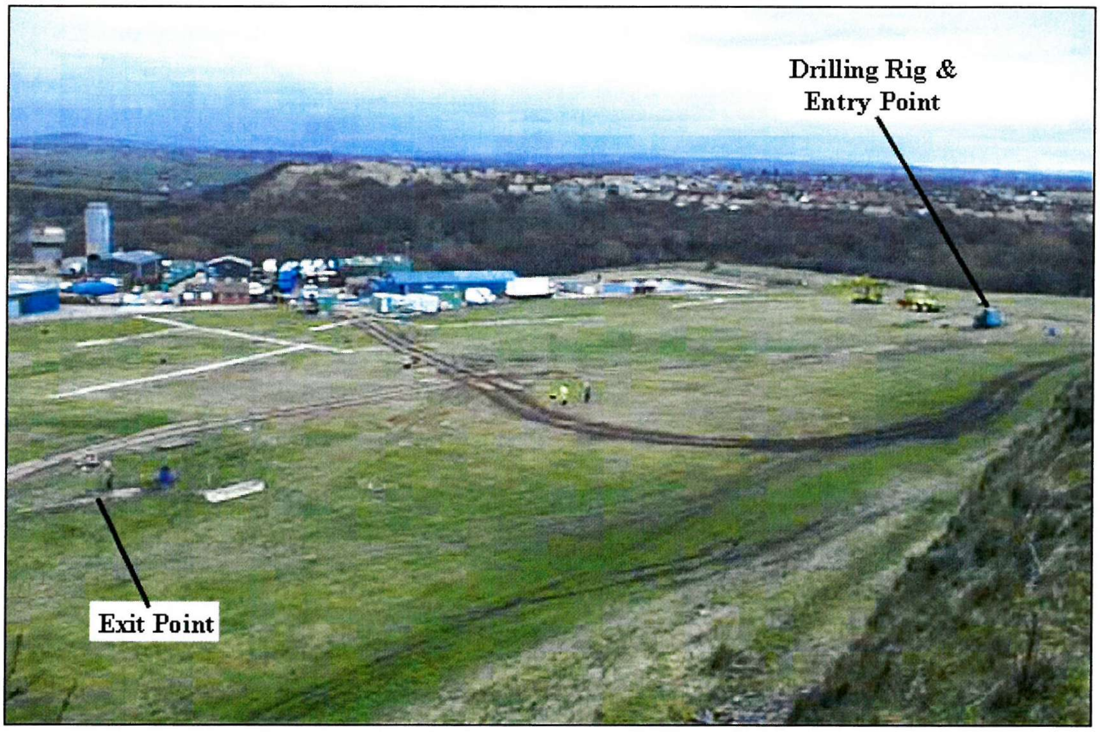

Figure 4.41 Metallic Tile landfill site (grassed area), showing location of drilling rig and exit point (100m apart).

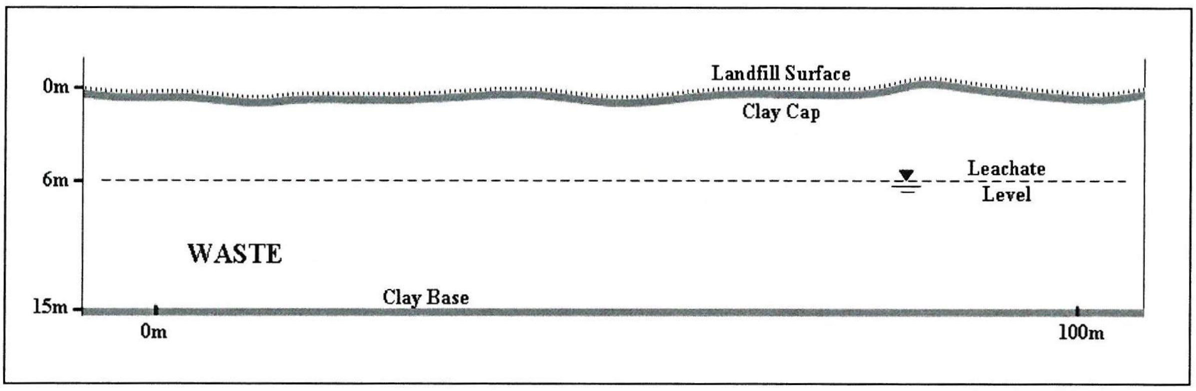

Figure 4.42 Schematic cross section of Metallic Tile landfill.

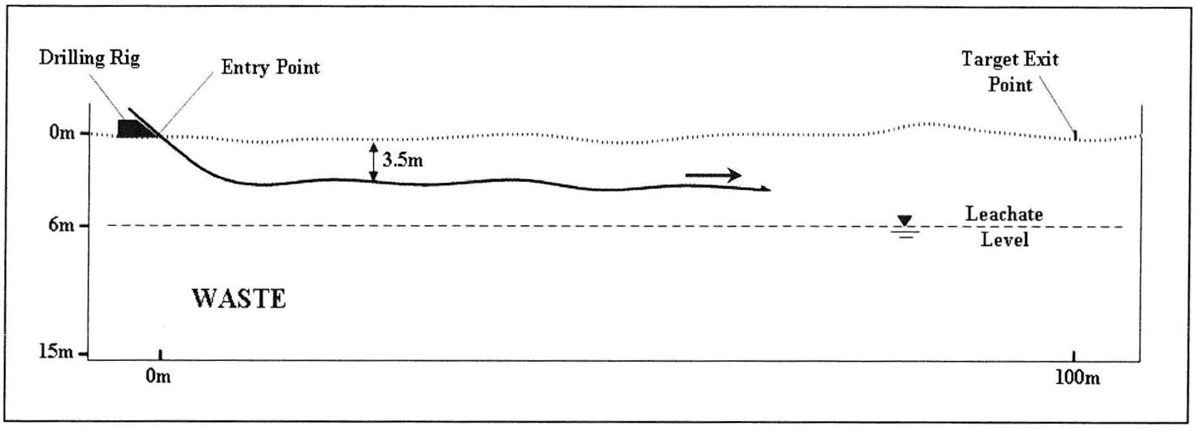

Figure 4.43 Pilot Drilling. Drill rods driven and steered through waste.

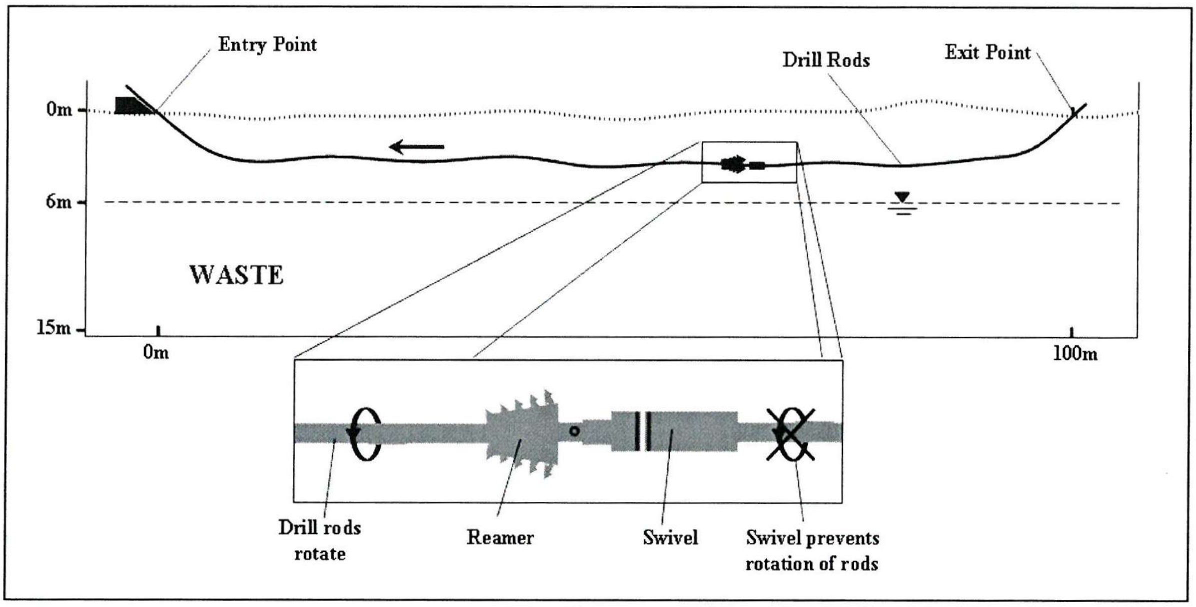

Figure 4.44 Pre-reaming. Drilling assembly pulled through borehole.

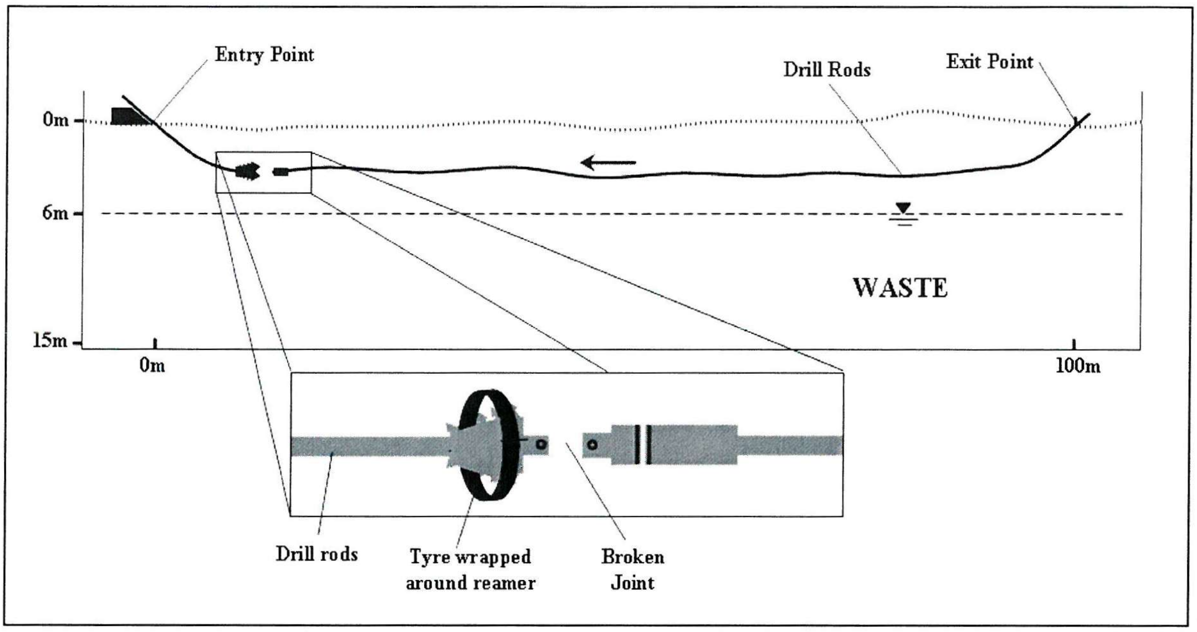

Figure 4.45 Failure during pre-reaming. Reamer entangled with a vehicle tyre, joint to swivel broken.

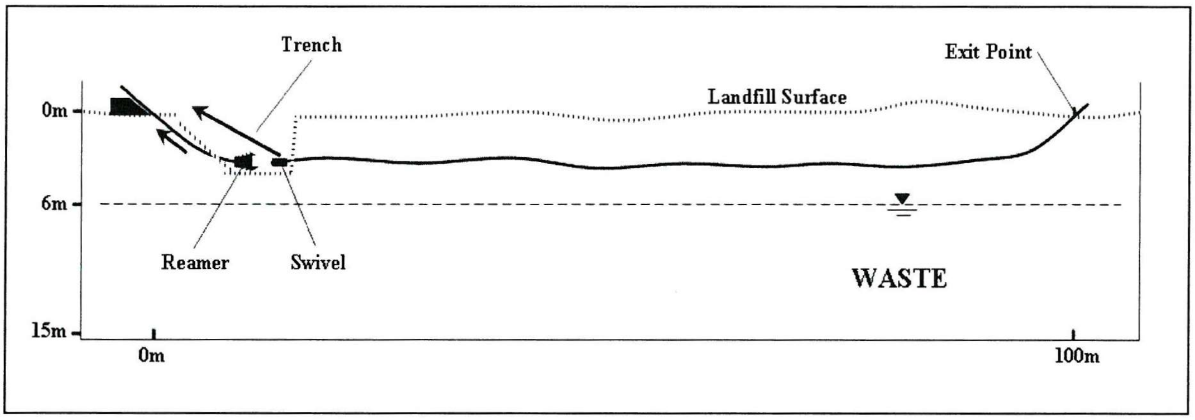

Figure 4.46 Trench excavated to recover drilling assembly. Swivel and drill rods pulled out with excavator.

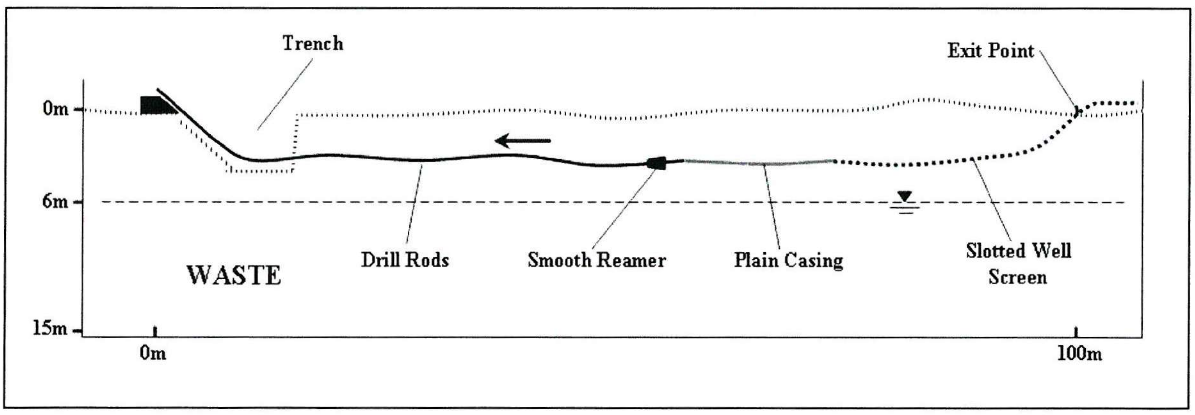

Figure 4.47 Backreaming with smooth reamer and installation of well screen.

4.4.1 Pilot Borehole

Breheny Civil Engineering commenced drilling of the pilot borehole in March 2000 (two weeks prior to the arrival of DrillTec at Rainham). The drilling rig had a pullback capacity of 200kN, more powerful than the 120kN rig used in the first Rainham trial, but still within the 'mini-rig' category. The rig also incorporated a 'dynamic impact unit' which could effectively hammer the drilling assembly into the ground to drive through hard obstructions. The hammer could deliver up to 1000 impacts per minute, momentarily increasing the thrust capacity of the rig from 200kN to 480kN. The rig is shown in Figure 4.48.

The first pilot borehole proceeded to a distance of 21m before a hard obstruction was encountered. Despite the hammer action no further progress could be made. The rods were withdrawn and the 100mm wide bevelled edge drill bit was replaced with a smaller, more pointed version (75mm width) that might better penetrate hard ground. A second pilot borehole was then commenced along a slightly different route. This reached a distance of 27m before hitting an obstruction that caused the drilling assembly to veer down toward the saturated horizon. It proved impossible to redirect the borehole upwards and so the rods were again fully withdrawn and a third borehole was commenced. This twice encountered hard material, at distances of 12m and 66m, but on both occasions the hammer was able to progress the drilling assembly. At 66m the walkover guidance system* registered some interference suggesting that the obstruction may have been metallic.

The third pilot borehole reached the target length of 100m in under 5 hours. Approximately 5.5m^3 of biodegradable drilling fluid (Biobore™) was used in this time and there were no fluid returns. The profile of the borehole is shown in Figure 4.49.

* The maximum target depth of the borehole was 3.5m; the walkover system used at Rainham had performed satisfactorily at this depth and so a wireline system was considered unnecessary.

Figure 4.48 Mini-rig at Metallic Tile, 200kN pullback capacity.

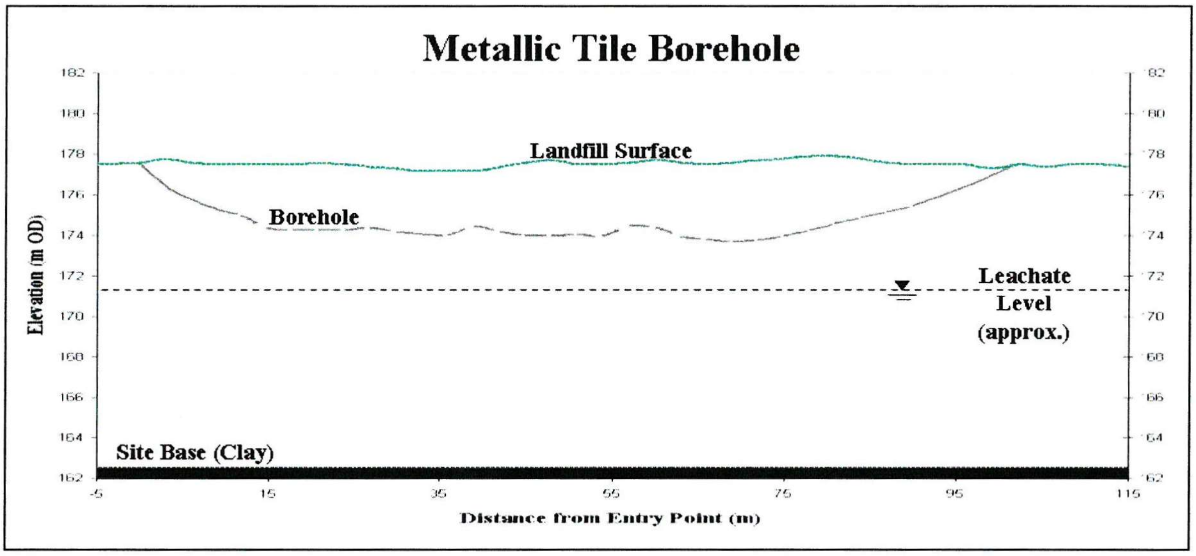

Figure 4.49 Profile of Metallic Tile Gas Extraction Well, dashed section of borehole represents well screen.

4.4.2 Pre-reaming

Approximately 3000 Nm of torque was required to rotate the drill rods in the borehole following completion of the pilot drilling. The drill rig operators advised that this was more than usually required after drilling pilot boreholes of similar dimensions through natural ground formations, suggesting that the 75mm diameter borehole had closed around the 55mm drill rods. It was therefore decided to pre-ream to loosen the waste before attempting installation of the 62mm diameter well screen. The reamer shown in Figure 4.50a was attached at the exit point. A less aggressive reamer had been specified in accordance with the earlier recommendation that drilling tools should not have protruding teeth. However this had been overlooked by the contractor and no other reamer was available.

Pre-reaming (Figure 4.44) commenced and drilling fluids immediately began flowing from the exit hole, although at a much lower rate than they were being pumped in. Approximately $6m^3$ of drilling fluid were used during the pre-ream with $1.5m^3$ collected from the exit hole. The returning fluids contained suspended particles up to sand size, and pieces of paper and plastic (no bigger than approximately 3cm across). Fluids flowed from the exit hole as pre-reaming continued, although at one stage the flow ceased for a ten minute period, presumably whilst a void in the waste was filled.

The reaming assembly incorporated a swivel that allowed the drill rods and reaming tool to rotate while preventing the rods on the exit side from rotating. However, after 20m the rods on the exit side began to rotate, indicating that the swivel was not functioning and had perhaps become wrapped in wire and fabrics. With 15m of the pre-reaming stage left to complete, progress came to a halt. Despite application of pull, thrust, rotation and hammer, the drilling assembly could not be moved either forward or backward. A mini-digger was brought in to excavate through the waste to reveal the obstruction, Figure 4.51. During excavation of the narrow trench, the waste spoil was examined and logged, Figure 4.52. It was found to be well decomposed and black in colour. It contained a high proportion of soil and rubble with some concrete pieces up to 25cm in diameter. There was little paper and plastic and in general the waste resembled the lower horizons at Rainham that had dated to 1975. At a depth of 3m the reamer was uncovered and found to be wrapped in a lorry tyre through the centre of which the pilot bore had passed. A second tyre had also become wrapped around the reamer (Figure 4.45).

It was then discovered that the joint between the reamer and the swivel had broken. This joint was likely to have been weakened as a result of the undue torque to which it was subject when the swivel seized up and had finally broken in the rigorous attempt to free the drilling assembly. The tyres (Figure 4.53) were removed with the excavator. No one could enter the trench to remove the broken swivel and reconnect the assembly due to the presence of landfill gas and the possibility of side wall collapse. The 15m of rods that remained connected to the rig were therefore pulled back to the entry point and in order to remove the

85m of drill rods that remained in the borehole, the trench was extended toward the rig. This created a slope along which the drill rods could be pulled using the excavator* (Figure 4.46). On inspection, the reamer was found to have lost most of its teeth (Figure 4.50b). Together with the entanglement of the reamer in the tyres, this loss of the teeth supported the conclusion that aggressive reamers should be avoided when drilling through waste.

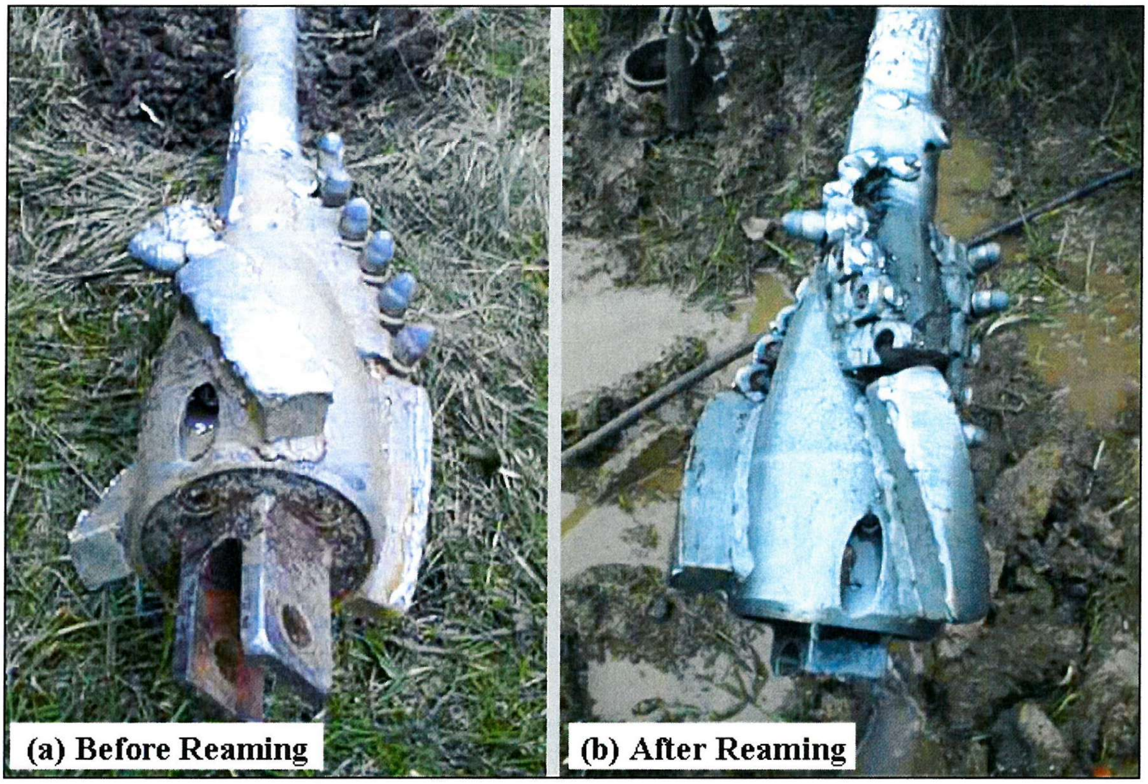

Figure 4.50 Reamer used on pre-ream stage, (a) numerous protruding teeth before start, (b) most teeth detached as passed through waste.

Figure 4.51 Mini digger excavating to discover the obstruction, 15m from the drilling rig.

* The drill rods could have been pulled out from the exit hole but this would have meant losing the borehole.

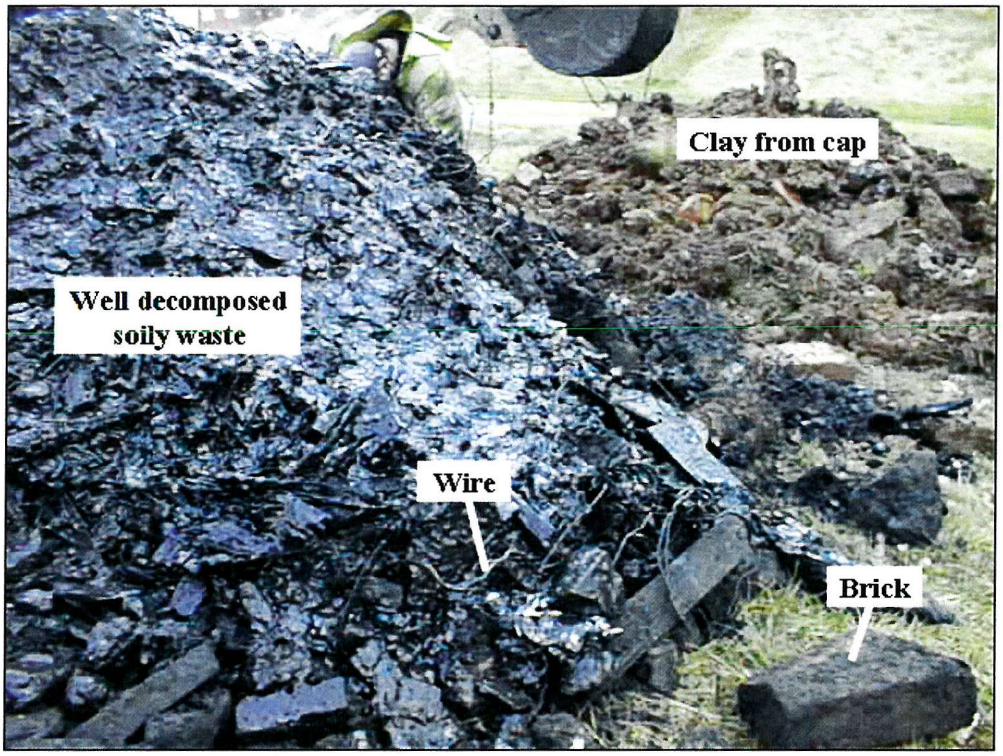

Figure 4.52 Waste removed from trench, well decomposed and black in colour.

Figure 4.53 Lorry tyres that had caused the obstruction during pre-reaming, note how they have been ripped by the action of the aggressive reamer.

4.4.3 Back-reaming and Well Screen Installation

The remaining teeth were removed to create a smooth reamer of 100mm diameter. It was thought this would act to press some of the loose waste particles into the borehole walls, thereby reducing the friction on the following 62mm diameter well screen. This reamer and a new swivel were attached at the exit end and installation of the well screen began (Figure 4.47 and Figure 4.54). The screen (and plain casing) sections were connected with threaded fittings. The first and last 10m of the borehole were completed with plain casing to seal the ends of the well against the clay cap. The middle section was completed with 81m of 0.5mm slot width stainless steel Vee-wire screen (Figure 4.55). This had an open area of 2.63m^2 or 17%. The well screen was pulled into the borehole without incident although the reamer was wrapped in wire from the landfill on removal (Figure 4.56).

The forces required to pull the drilling rods and well screen if the borehole had (a) remained open, or (b) fully closed, were calculated using equations (4.2) and (4.4). The results are given in Table 4.6.

Table 4.6 Pulling forces required for the Metallic Tile installation.

Factor	Drill Rods	Plain Casing & Well Screen
Diameter (d)	0.055m	0.062m
Rod/casing Wall Thickness (t)	0.01m	0.005m (casing)
Rod/casing Density (ρ)	7800kg/m^3	7800kg/m^3 (casing)
Pipe Weight (N)	135N/m	28N/m* (screen) 75N/m (casing)
Average Effective Stress (σ'_{av})	17.5kPa	17.5kPa
Angle of Interface Friction (tan φ)	30°	30°
Length (l)	100m	81m (screen) 19m (casing)
Pulling Force Required (P) **Closed Hole** $P = \pi d\, \sigma'_{av} \tan \varphi \times l$	174 kN	197 kN
Pulling Force Required (P) **Open Hole** $P = N \times \tan \varphi \times l$	8 kN	2.1 kN
Drill Rig Capacity	200 kN	200 kN

* Well screen weight taken from product data sheet [Johnson Screens, 1999], use of the approximation N = $\pi dt\rho g$, was considered inappropriate due to the wire wrap design and the high open area (17%).

The pulling forces actually required for installation were between 20kN and 60kN. These values are significantly higher than the force required to install the screen had the borehole remained open, but are also somewhat less than the force required had the borehole fully closed, suggesting that the borehole had only partly closed. It was also concluded in the first Rainham trial that borehole closure was only partial. The reduction in diameter of the well screen, from 150mm in the first Rainham trial to 62mm in the Metallic trial, reduced the frictional resistance to less than the maximum capacity of the rig.

No fluids flowed from the exit hole during the installation although some fluids collected in the trench that had been excavated during the pre-reaming stage. The fluids in the trench flowed away readily through a single point in its base, suggesting that the overall hydraulic conductivity of the waste may be dominated by a few preferential pathways*. As some fluid flowed from the borehole during the pre-reaming stage, the borehole may have retained some fluid which may have acted as lubrication for the installation of the well screen.

On completion of the well, the ends were capped ready for connection to the gas extraction system (Figure 4.57) and the trench was backfilled. Total installation time, including set up and set down of the equipment, had taken 5 days. Without the delay caused by the tyres and some technical problems with the drilling rig, completion time might have been reduced to 3 days.

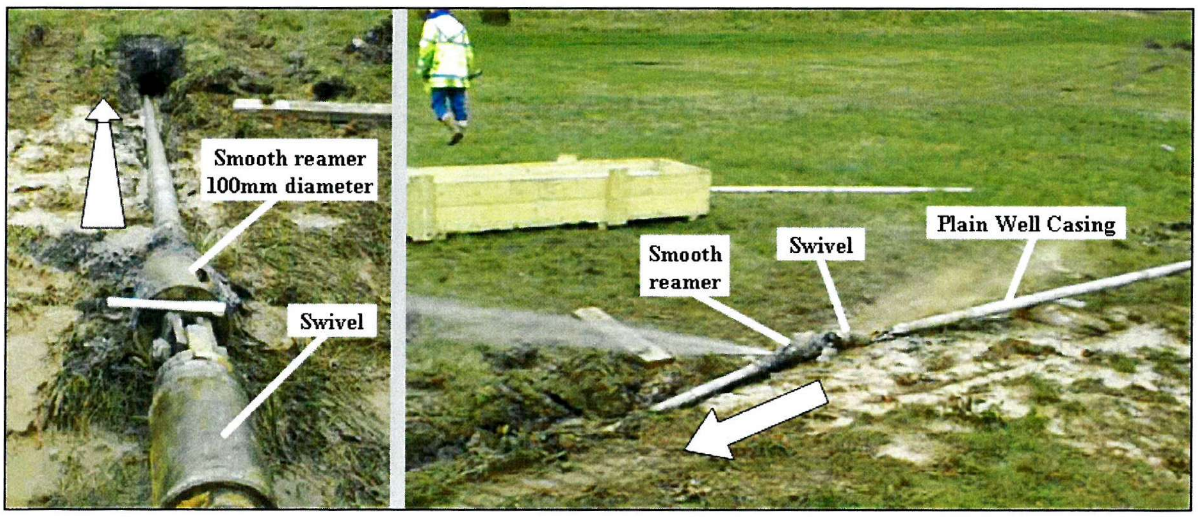

Figure 4.54 Backreaming and casing installation. Reamer, with teeth removed is pulled through borehole ahead of plain casing and well screen.

* Evidence for preferential flow pathways have also been observed at Rainham; continuous streams of landfill gas bubbles can be seen rising out of the ground when wet. The positions of these streams have remained stationary over the two years since they were first noted.

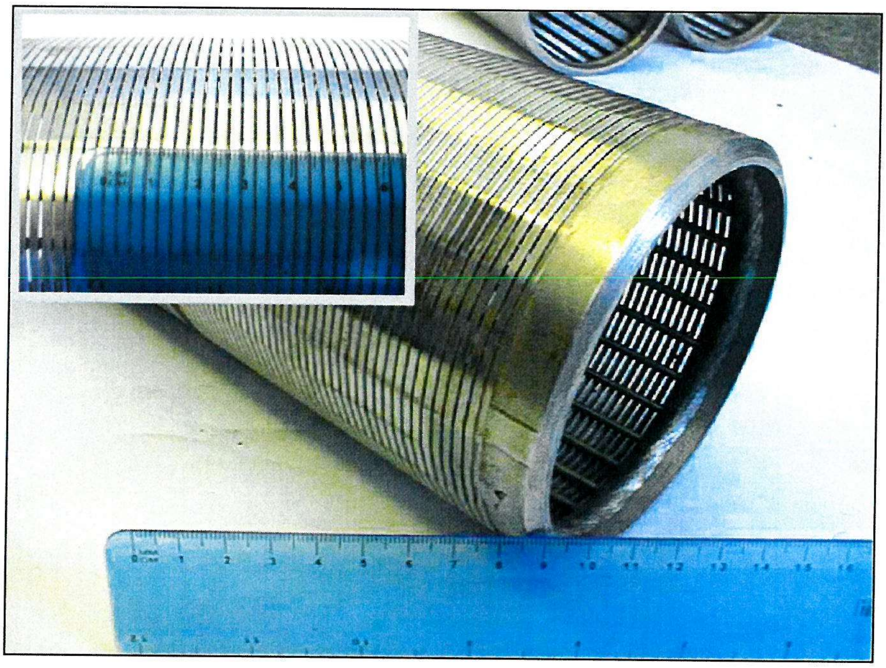

Figure 4.55 Johnson Vee-wire well screen, 0.5mm slot width (photo is of a 100mm diameter sample, screen used at Metallic Tile was 62mm in diameter) .

Figure 4.56 Smooth reamer wrapped in wire after well screen installation.

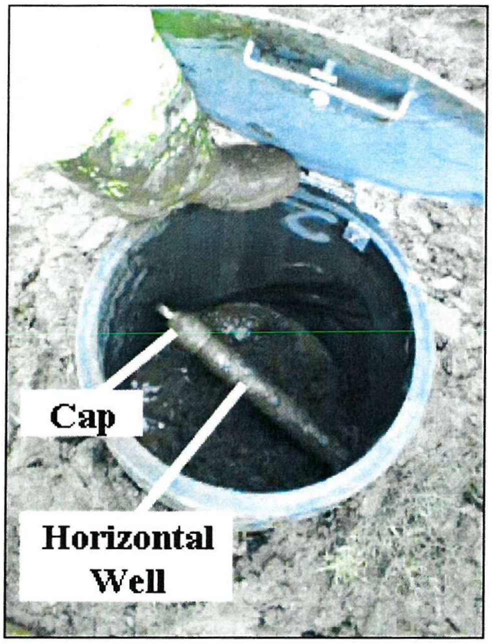

Figure 4.57 Capped end of completed horizontal well, with inspection cover.

Commencement of the second drilling trial at Rainham one week after installation of the Metallic Tile well shifted the focus of the research toward Rainham. Five months after installation, the site operators at Metallic Tile connected the horizontal well to the gas extraction system. This system consisted of an electrically driven fan supplying a flare with gas drawn from both the horizontal well and a series of vertical wells. The layout of the extraction pipework was designed to allow gas to be drawn independently from the horizontal well and the existing vertical wells, thereby enabling flow rates and gas quality from the two systems to be determined separately.

Total gas flow rates from the site were not enough to sustain the flare continuously. If suction was increased, air was drawn into the system via the vertical wells and the flare shut down until the gas concentration had returned to normal. In order to collect good quality data from such a system, a daily presence on site would be required. With the installation and testing of the Rainham wells (in Essex), a daily presence at the site in Staffordshire was not possible and no thorough data have been collected to date (February 2002). An assessment of the performance of the Metallic Tile horizontal well therefore remains outstanding.

4.4.4 Conclusions from Metallic Tile

Attempts to mill the heterogeneous and compressible waste are unlikely to be successful whether using aggressive reamers or smooth reamers. Moreover, reamers with protruding teeth will become entangled in waste materials that have some degree of tensile strength, e.g. wire and fabrics, and should therefore be avoided.

Although collapse of the borehole walls in the loose, uncohesive waste is likely, closure may be only partial and provided the diameter of the well screen is kept small, the frictional resistance may remain below the pullback capacity of the drilling rig.

It should be noted that the Metallic installation was only 3.5m below the ground surface, and the maximum vertical effective stress was only 35kPa (assuming an average bulk unit weight of $10kN/m^3$ and zero pore water pressure). At greater depths, the increase in effective stress may increase the friction between the waste and the well screen to a degree beyond which installation may have failed.

On two occasions hard obstructions were encountered during the pilot drilling. One caused a significant deviation to the route and the other prevented further progress of that particular borehole. Such severe obstructions had not been encountered during the first Rainham trial although the possibility had not been ruled out. However, by withdrawing the drill rods and redirecting the borehole, a route through the waste was made. It therefore seems unlikely that hard obstructions would prevent the eventual completion of a pilot borehole. In addition, the hammer action of the rig proved to be useful in driving the drilling assembly through areas of hard material.

As Metallic Tile had accepted hazardous waste materials it had been hoped that no fluids would return to the surface. However, unlike the first Rainham trial, fluids did return, albeit for only part of the pre-ream stage and at a much lower rate than they were pumped in. Fluid returns cannot therefore be ruled out, although this is still considered unlikely. Moreover, the volume of returning fluids, a total of approximately $1.5m^3$, was considerably less than the volume of waste likely to be created from the installation of a series of vertical wells.

From the success at Metallic Tile it appears that small scale, directional drilling operations can be used to install horizontal wells in waste. However, further trials are required to support this conclusion.

4.5 Discussion and Recommendations

Prior to this research, there had been few attempts made at directional drilling through landfilled waste. The Midwestern trials had approached the installation process in much the same way as would have been done for horizontal wells in natural ground. As a result, these trials met with varying degrees of success. It is now clear that the properties of heterogeneous waste present unique problems to the installation process and that specific adaptations are required to ensure horizontal wells can be completed with confidence. Many of these problems and adaptations were identified during the trials at Rainham and Metallic Tile, and are discussed below.

Conventional backreaming techniques are not suitable for use in waste because the loose, uncohesive nature of the material leads to the collapse/closure of the borehole walls when unsupported. A well screen of a diameter larger than the drill rods will be subject to immense frictional resistance as it is pulled into a partially closed borehole. The research has identified two ways to overcome this problem.

The first, used at Metallic Tile, limits the well screen diameter to a small size similar to that of the drill rods. The frictional resistance on the well screen is kept within the capacity of the rig despite partial closure of the borehole. However, the trial at Metallic Tile installed a well of only 100m at a depth of only 3.5m where effective stresses were small. Whether this technique would work for longer or deeper installations requires further investigation.

The second method, used during the second Rainham trial, is overwashing. The overwash casing limits the degree of borehole closure to the small difference in diameter between the cutting shoe and the casing while the well screen is installed inside. Three wells were completed using overwashing, giving some degree of confidence in the reliability of the technique. However, all three wells were installed at the same landfill site and further trials at other sites are required to assess the reliability of the technique. Milligan (2002) states that 'frictional forces build up rapidly once collapse has occurred and the important thing is to keep the hole open around the pipe, by support from a suitable drilling fluid, a casing, or other means'. Milligan adds that 'it will be very difficult to support a hole using fluid in waste, so that some method employing a casing is the best bet'.

Waste can contain hard materials or objects that can obstruct the path of the pilot borehole. Objects identified in the site investigation included reinforced concrete blocks, wooden railway sleepers and metal items such as washing machines. At Rainham, such objects had caused a number of minor deflections to the pilot borehole. At Metallic Tile, obstructions halted progress on two occasions so that the rods had to be pulled back and redirected. The hammer device fitted to the drilling rig proved to be useful in advancing the drill bit through hard materials. In all the trials, including those in the Midwest, drilling of the pilot borehole was successful and it is therefore considered unlikely that hard materials will prevent completion.

In stark contrast to natural ground, waste can contain materials with a high degree of tensile strength in strands, such as wire, fabric and steel reinforcing bar. As observed, these materials can get tightly wound around the reaming assembly making progress difficult or even impossible. It is apparent that aggressive reaming tools with protruding cutting teeth are more likely to become wound in these materials.

Physical cutting and milling of the waste is unlikely to occur because the waste will generally compress into the borehole walls as the reaming tool passes. Protruding teeth are therefore considered unnecessary as well as problematic. For backreaming, aggressive reamers should be avoided in favour of 'smooth' reamers which push the waste aside. For overwashing, the leading edge of the overwash bit should be coated in a hard material such as tungsten carbide so that it does not become flat and smooth and is able to pierce through the waste during drilling.

For installations using backreaming, a carrier casing will reduce (a) the tensile strength requirements of the well screen, and (b) the likelihood of smearing soil/clay materials over the well screen slots during installation. If the tensile strength requirements are lower, the well screen design can focus on fluid extraction properties, for example, a larger open area. However, as the carrier casing has a larger diameter than the well screen, frictional resistance during installation is more likely to be a problem.

The return of drilling fluids from the borehole occurred for only limited sections of the Metallic Tile and Midwestern trials. Possible reasons for the general absence of returning fluids were identified as:

a) the absorption of fluids into dry or unsaturated zones previously isolated from areas of wet or saturated waste until hydraulically connected by the borehole,
b) waste collapsing around the drill rods, shutting off the pathway for return flow, and
c) areas of waste with coarse grain size allowing the drilling fluid to flow away from the borehole, preventing the formation of a filter cake. With no filter cake fluids were not channeled back along the borehole to the entry point.

If fluids are unlikely to flow out of the borehole, any attempt to cut and mill the waste with aggressive reamers becomes futile as there is no removal mechanism for any cuttings that are created. The lack of fluid returns can be an advantage if redisposal of waste fluid is problematic, such as at hazardous waste landfills. However, evidence from Metallic Tile demonstrates that fluids may return. Nevertheless, waste fluids are likely to be low in volume and, in all probability, of a lower volume than the spoil produced from vertical well installations.

It is clear that as most of the fluid remains within the waste, the use of bentonite based fluid seems unadvisable if the hydraulic conductivity of the waste in the vicinity of the well is not to be reduced. However, a choice remains between biodegradable fluids and water. Water was used during the overwashing at Rainham and the installations were successfully completed. Biodegradable fluid was successfully used at Metallic Tile and, although there is no direct evidence, the fluids may have lubricated both the drill rods and well screen as they passed through the borehole.

A range of rig pullback capacities has been used in the backreaming trials discussed above (from 107kN to 2,224kN). Both success and failure has been experienced with mini and maxi-rigs and, not unexpectedly, this can be loosely linked to the diameter and length of installation, i.e. the smaller the diameter and shorter the length, the lower the pullback capacity required. However, there were many differences between the trials other than the rig capacity and dimensions of the installation. These included waste type, depth of installation, and even whether the borehole remained wholly within the waste horizons. Perhaps the most clear illustration of the importance of adequate rig capacity occurred during the second

Livingston trial. In this case, four increasingly powerful midi-rigs were used unsuccessfully over a seven month period until, finally, a maxi-rig was brought in to complete the installations.

The rig used in the second (overwashing) trial at Rainham had a pullback capacity of 1000kN, all of which was required during removal of the overwash pipes. However, the thrust and torque capacities of the rig are perhaps of greater importance when overwashing. The thrust and torque capacity of the Rainham rig was 320kN and 45,000 Nm respectively. During the overwash stage of each of the three installations, progress slowed considerably as the 250m target was approached. It is considered that a greater thrust and torque capacity may have speeded up the operation.

Walkover guidance systems may be accurate up to depths of approximately 8m. Below this, metallic objects in the waste may cause interference and a loss of accuracy. A wireline system that makes use of a surface induced magnetic field (e.g. TruTracker™) is therefore recommended for deeper installations. Interference from the waste remains high even when using TruTracker™ and so the strength of the magnetic field should be maximised by both double-looping the induction coil and reducing the area it covers.

To maximise the potential for fluid extraction it is desirable to install the horizontal well as close to the base of the landfill as possible. However, the base may be hit if drilling is too close as (a) the route can be deflected by hard materials, (b) the guidance data may be inaccurate, and (c) the base level may naturally fluctuate. From the experiences of the above trials it is recommended that a minimum of 2m is allowed between the site base and the borehole.

The well screen should have threaded fittings as welding in the presence of landfill gas is unsafe. End caps should be available to prevent the uncontrolled flow of leachate and gas from the well on completion. The open area of the screen should be maximised to reduce well losses, particularly as flow through waste may be dominated by a limited number of preferential pathways. Indeed, Brandon (1986) suggests that one half to one third of the total open area may be blocked by the formation material (in soils). The presence of sheet like materials in waste, such as plastic bags, may reduce the effective open area of the well screen still further.

Most of the trials described above have encountered difficulties with installation that have caused the stresses exerted on both the drilling assembly and well screen to exceed the stresses normally encountered when operating in non-waste formations. The use of materials that are relatively fragile, such as HDPE and polyethylene well screen, are therefore to be avoided.

With the three installations at Rainham complete, the next stage was to assess well performance. It was decided that development of the wells should take place after an initial period of testing, allowing any change in well performance to be quantified.

CHAPTER 5

Monitoring Instrumentation

The performance of the horizontal wells was assessed by carrying out long term pumping tests, the analysis of which would rely on an extensive array of monitoring instrumentation which first had to be installed. This chapter describes the instrumentation used and appraises the possible errors from each instrument type.

5.1 Instrument Types

5.1.1 Standpipe Piezometers

Experiments by Hudson *et al* (1999) demonstrated that the ratio of horizontal to vertical hydraulic conductivity (k_h/k_v) of waste increases with applied stress, i.e. the waste becomes more anisotropic. Hudson *et al* reported that the horizontal hydraulic conductivity is always greater than the vertical, and the ratio k_h/k_v increases from approximately 2 at an applied stress of 40kPa to 5 at an applied stress of 600kPa. New research (2002) by Hudson and Beaven suggests that this ratio may be even higher, up to 10 times greater.

In light of this it seemed likely that horizontal flow to the well would predominate over vertical flow and that pore pressures would not vary hydrostatically with depth. It was therefore decided to install standpipe piezometers with discrete response zones to assess the changes in pore pressure. Observation wells, i.e. fully screened piezometers, were ruled out because the leachate head they record may be difficult to interpret or indeed meaningless.

The design of the piezometers is shown in Figure 5.1. Each piezometer consists of a 25mm or 50mm PVC tube with the lowest 0.5m screened to allow the entry of pore fluids. The screened section illustrated in Figure 5.2 had a slot width of 0.4mm. Surrounding the screen was a 1m deep medium/coarse sand filter (grain size 0.5 to 1mm). The 'discrete' monitoring zone measured by each of the piezometers was therefore 1m in height.

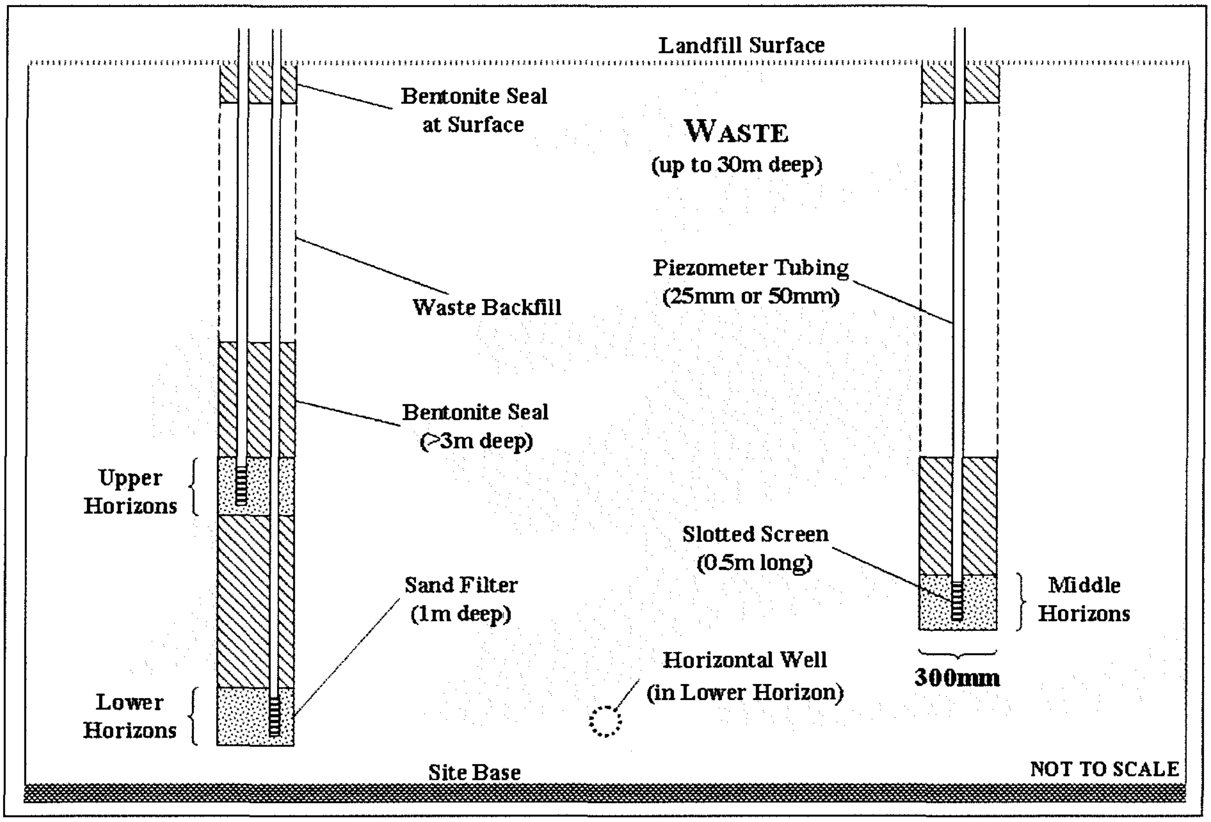

Figure 5.1 Piezometer design (dual installations and single installations).

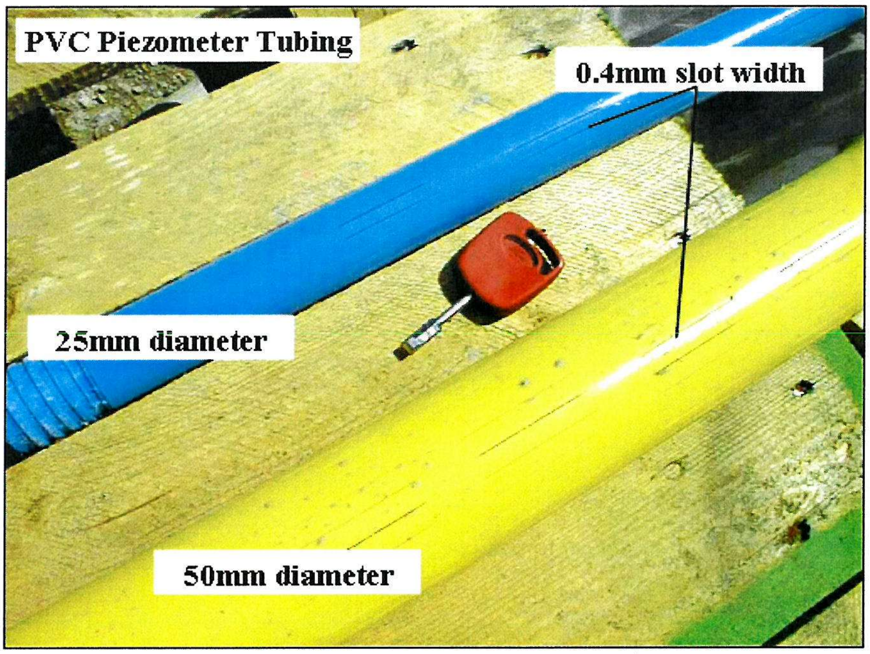

Figure 5.2 Slotted PVC tubing used for piezometers.

The pore pressures in three levels of waste were monitored. The levels were spaced to ensure an adequate depth of bentonite seal (>3m) between the monitoring zones of the dual installations and do not correspond to any layering observed during the site investigation. The levels are referred to as the ‘Lower Horizons’, the ‘Middle Horizons’ and the ‘Upper Horizons’. Piezometers installed in the Lower Horizon have their monitoring zone at approximately the same elevation as the horizontal well to which they are adjacent. Middle and Upper Horizon piezometers are installed at levels of approximately 2.5m and 5.4m above the well respectively. It should be noted that the term ‘Upper Horizon’ refers to a layer generally at one third of the full depth of the waste and does not refer to the actual uppermost waste horizons. The Upper Horizon is also below the leachate level.

Boreholes for the piezometer installations were drilled using a rotary barrel auger of 200mm diameter, Figure 5.3. The repeated collapse of the boreholes that had been drilled during the site investigation led to concerns that the piezometers would not be very clean, i.e. that the sand filter would become mixed (and the slotted screen clogged) with the wet, slurry like waste during installation. To overcome this, the boreholes were lined with a temporary casing to provide wall support and a shell and auger rig was used to bail out the waste slurry (Figure 5.4). The piezometer tubing, sand filter and bentonite seal were installed within the temporary casing, which was then removed.

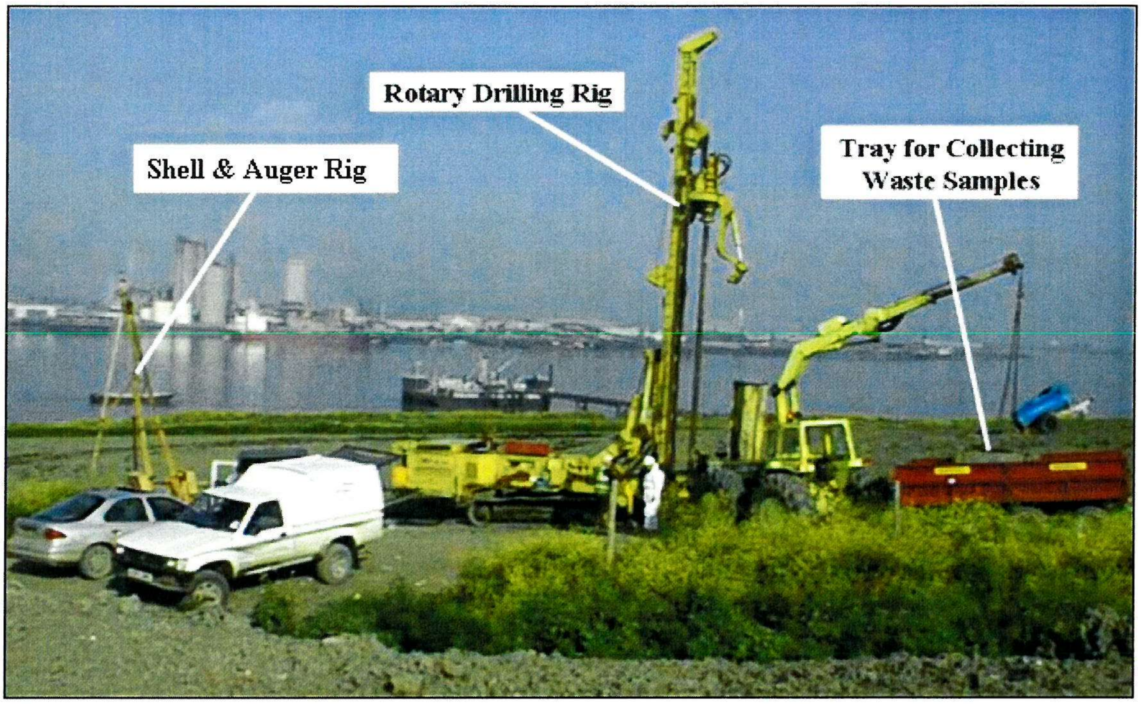

Figure 5.3 Two rig set-up for installing piezometers. Rotary rig to drill borehole, shell & auger rig to clean out temporary casing and complete installation [Rainham, 2000].

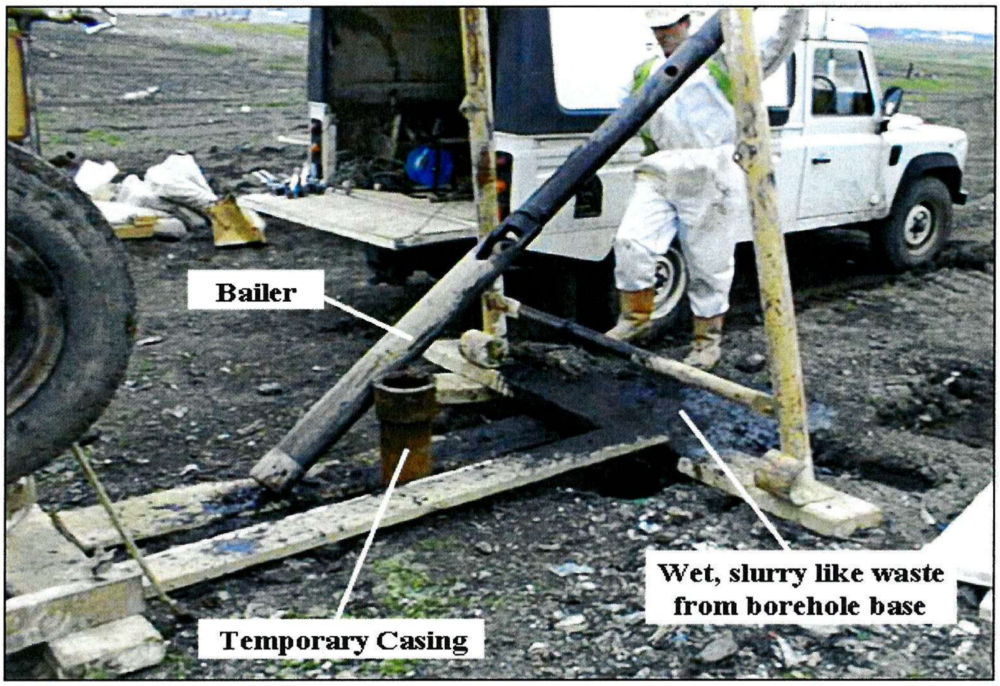

Figure 5.4 Using shell & auger rig to bail out slurry like waste from temporary casing to ensure clean piezometer installations [Rainham, 2000].

To increase the number of monitoring points at minimal cost, many of the boreholes were completed with two piezometers. The two screen sections were separated by at least 3m of bentonite so as to prevent any direct hydraulic connection between them. The effectiveness of the seal is demonstrated by the results presented in Chapter 6, which show that all the pore pressure responses from these dual installations were independent of one another. Piezometers were positioned perpendicular to the line of the horizontal wells, at distances of 0m, 4m, 12m, and 25m so that they were more closely spaced near to the well where variations in drawdown with distance were expected to be greater, although the distance of influence was unknown.

One cross section was instrumented on HW3, and two on HW1. Adverse ground-surface conditions prevented full access to HW2 with the barrel auger rig so that only six piezometers could be installed, each at a distance of 20m from the well. Only a limited understanding of the pore pressure response to pumping from HW2 was therefore obtained.

Figure 5.5 to Figure 5.10 show all the piezometer installations in plan view and cross section, indicating the depth of the monitoring zone and the distance from the adjacent horizontal well in each case. The piezometer locations are also marked on the site plan in Appendix E. The piezometers are referenced using the letters A to Z, with the number of the adjacent horizontal well added as a prefix. Thus piezometer, HW1D monitors Horizontal Well 1, while HW3D monitors Horizontal Well 3.

In addition to the 42 piezometers installed specifically for the assessment of the horizontal wells, 19 other existing monitoring points were used. These included:

- five piezometers that remained intact from the site investigation (reference numbers H03 to H06 and H08),
- three piezometers that were installed to provide background data on pore pressures in the (then proposed) location of HW1 (reference numbers HW1I, HW1O, HW1P),
- eleven vertical wells installed for leachate extraction (when not in use).

The eleven leachate extraction wells were fully screened. With horizontal flow dominating vertical flow as a result of the anisotropic permeability, the pore pressures varied with depth. The leachate levels recorded by fully screened wells may therefore have been misleading and were viewed with caution. Installation details for each piezometer and monitoring point, such as reference numbers and response zone elevations, are given in Appendix D.

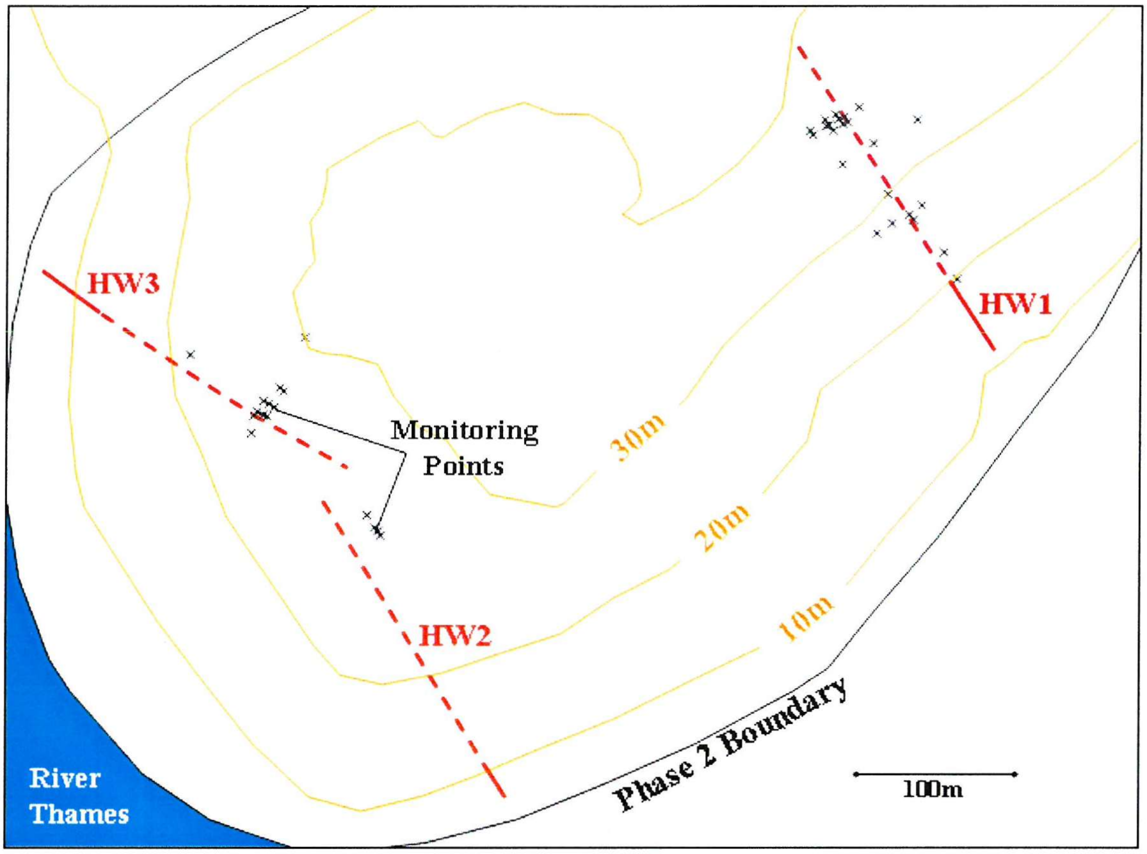

Figure 5.5 Plan view of Rainham Phase 2 showing locations of horizontal wells and monitoring points.

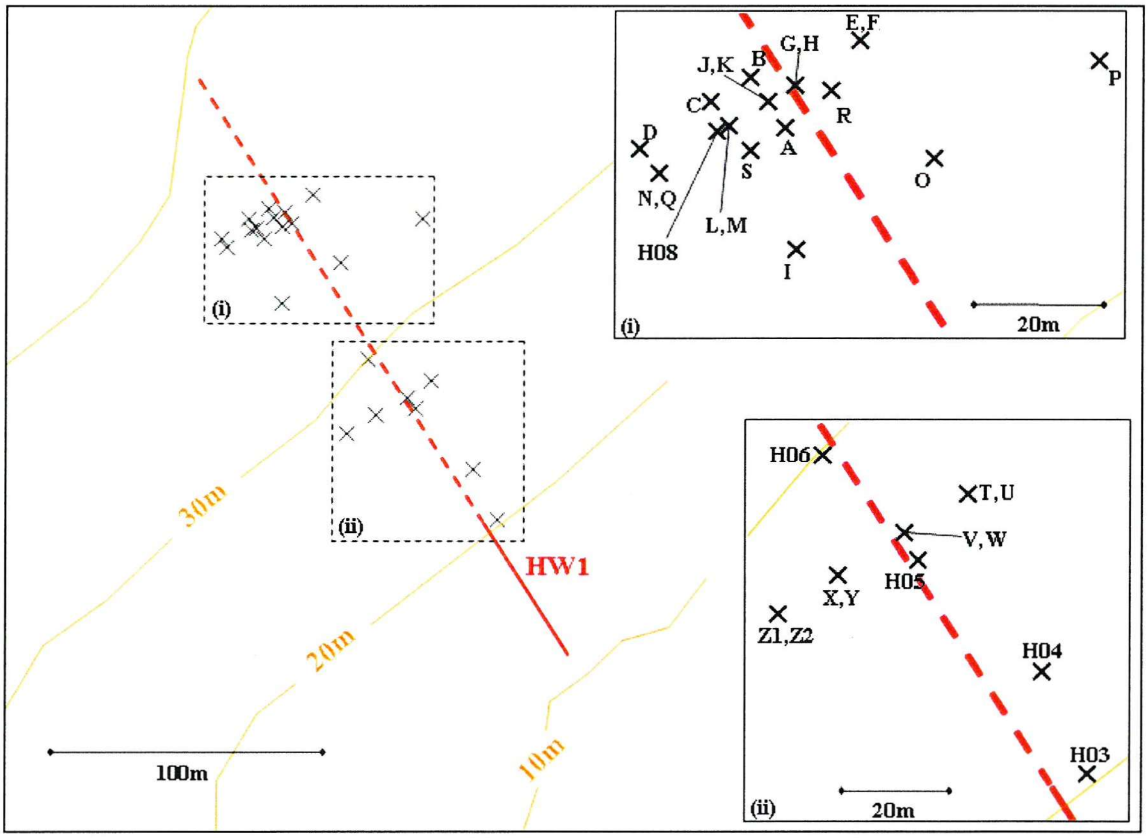

Figure 5.6 Enlarged plan of area around HW1, showing reference numbers of monitoring points.

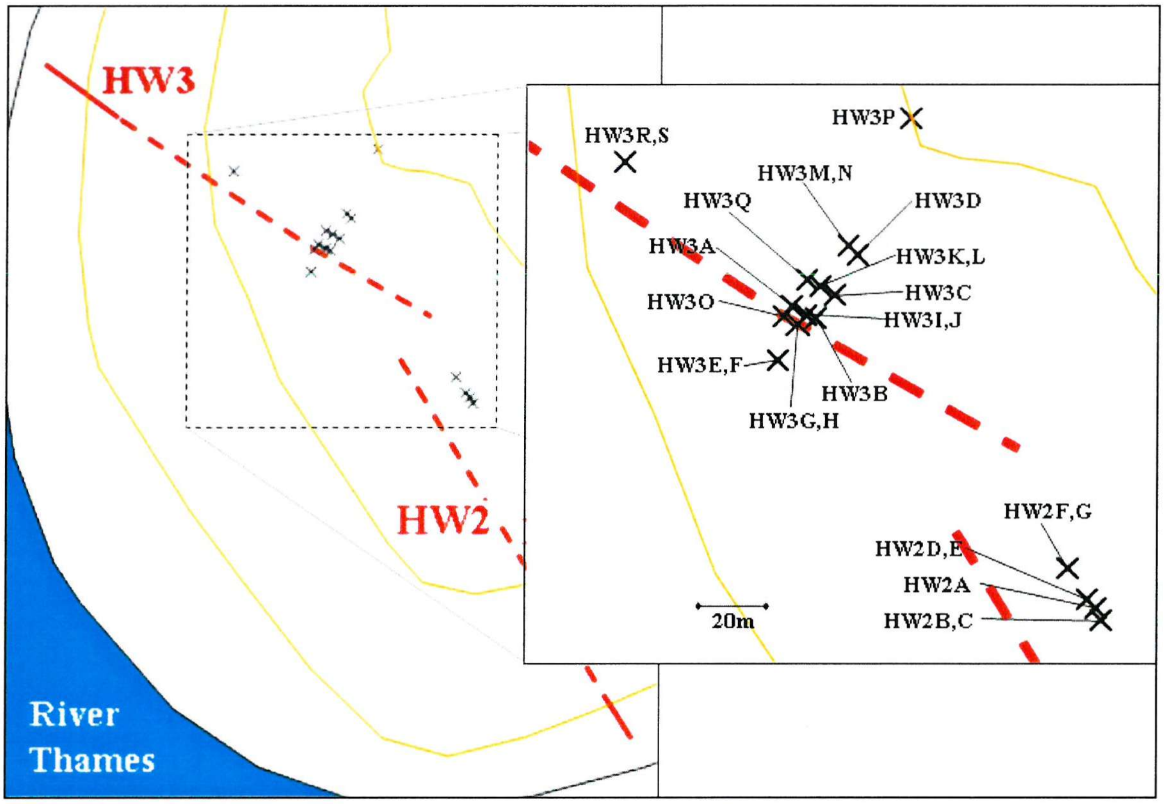

Figure 5.7 Enlarged plan of area around HW2 & HW3, showing reference numbers of monitoring points.

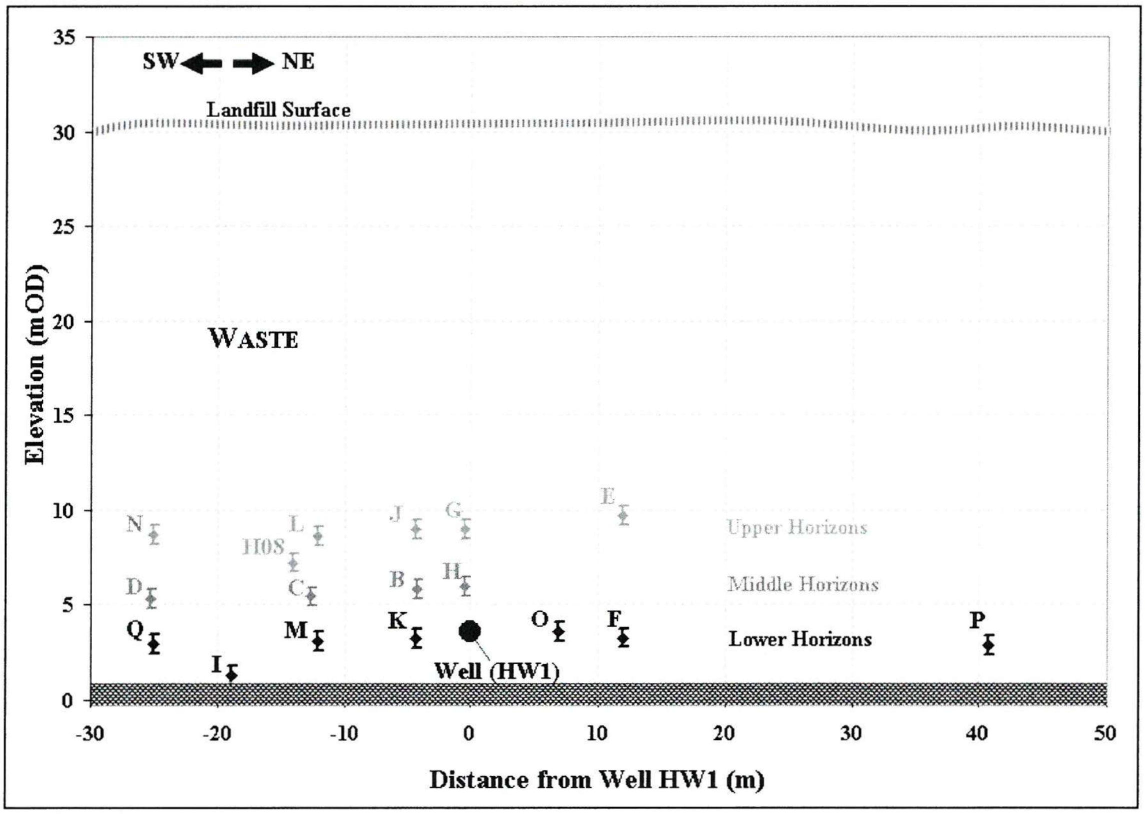

Figure 5.8 Cross Section showing piezometer response zones around HW1 (Line 1). Each zone is 1m in height. Borehole reference numbers have the prefix 'HW1'.

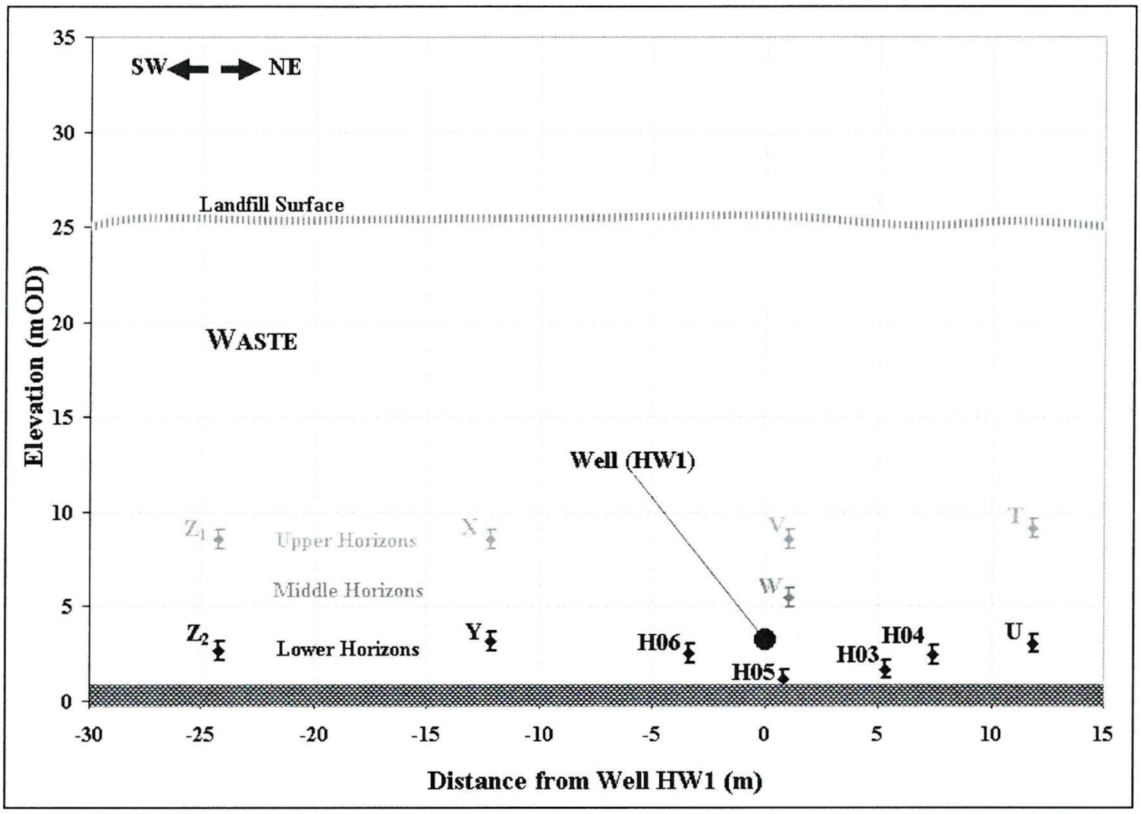

Figure 5.9 Cross Section of Piezometers around HW1 (Line 2). Reference numbers have the prefix 'HW1'

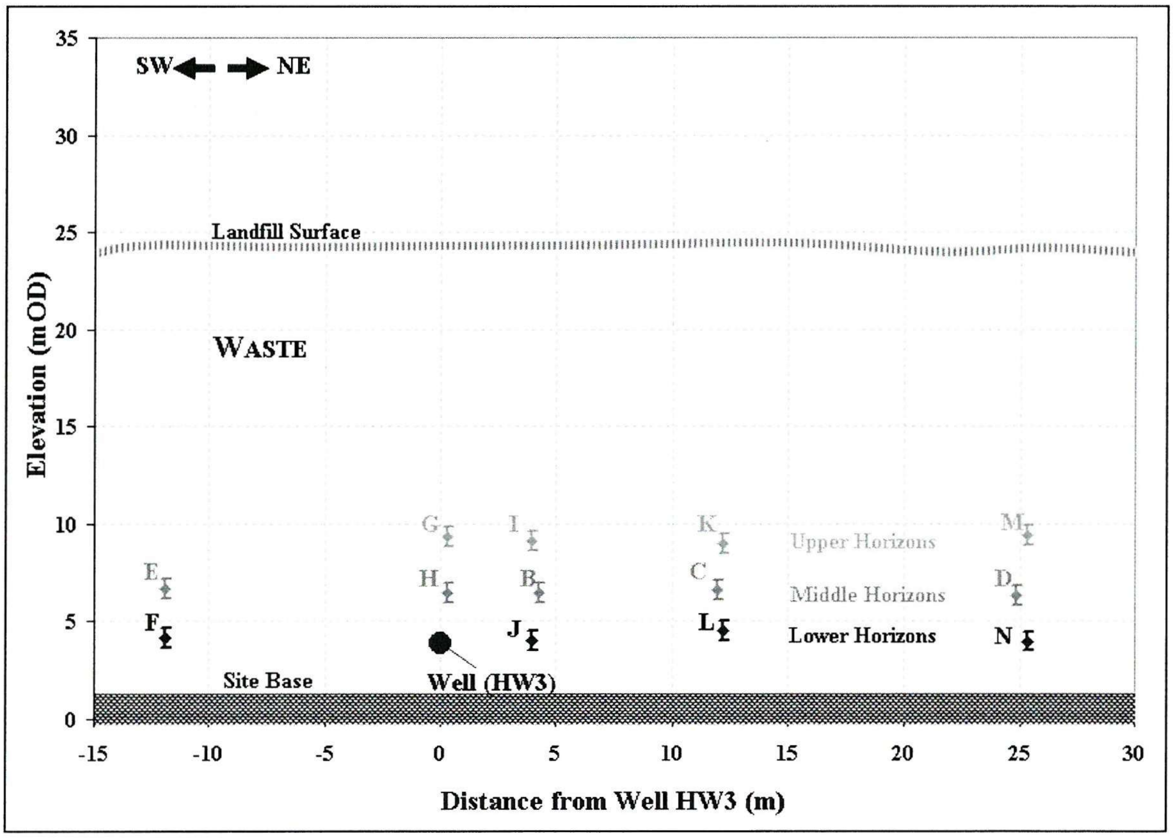

Figure 5.10 Cross Section of Piezometers around HW3. Reference numbers have the prefix 'HW3'

Each of the piezometers was furnished with a vibrating wire pressure transducer (VWPT) to automate the measurement and recording of changes in leachate level. Readings from the instruments were fed to two central dataloggers (one for each of the piezometer clusters around HW1 and HW3), from where they were downloaded to a pc at fortnightly intervals. The intention was to provide a comprehensive record of the changes in leachate level that would have a higher resolution and be more accurate than periodic manual readings. Nevertheless, manual readings were taken using a dip tape to backup the VWPT readings. After two months, it became apparent that the data from the VWPTs were diverging from the manual readings and it was eventually concluded that the instruments were malfunctioning*. Manual readings of the leachate levels continued until March 2002, with the frequency varied as follows;

a) daily (or more frequent) readings after changes to the pumping regime were made, e.g. the start of pumping,
b) weekly, while pumping caused a slow but steady reduction in the leachate levels, and
c) every two to three weeks when little change was evident in the leachate levels.

5.1.1.1 Inherent limitations of the piezometers

As stated above, the piezometers were installed at prescribed distances from each horizontal well. Positioning of these piezometers was therefore dependent on the accurate calculation of the horizontal well locations. As discussed in section 4.3.5 the well locations may only be accurate to within approximately four metres. Data from the piezometers, particularly those close to the wells, should be studied with this limitation in mind.

As the landfill settled, the top of the piezometer access tubes moved downwards, meaning that the reference point from which the leachate level was measured was continually moving. The downward movement of the access tubes averaged 1.12mm/day, whereas the settlement of the landfill surface itself was approximately twice that at a rate of 2.25mm/day. It is not clear whether the movement of the access tubes was a result of tube compression, bowing or whether the entire tube actually moved toward the waste/silt interface. It was not feasible to

* Subsequent investigation suggested that sudden submersion of each VWPT in warm leachate (30° to 55°C) created a thermal shock causing components of the instrument to expand at different rates. This breached the seal surrounding the vibrating wire element causing a gradual loss of vacuum and a drift in the readings.

re-survey the elevation of the reference points each time the piezometers were dipped but surveys were carried out on the following dates:

28/06/00, 06/02/01, 04/03/02

Analysis of the survey data demonstrated that the settlement of the piezometer tubing was approximately linear. Figure 5.11 shows the settlement of HW1U, which represented the worst case scenario in that it exhibited the least linear trend of all 42 piezometers. Nevertheless, it can be seen that the polynomial curve, which may more accurately represent the true settlement rate of the piezometer tubing, does not deviate from the linear by more than 5cm. A sustained error of 5cm or less was regarded as negligible considering that daily fluctuations due to changes in barometric pressure were likely to be at least of this magnitude (barometric effects are discussed further in section 6.9). The linear settlement rate of each piezometer was therefore calculated to enable correction of the manual dip data.

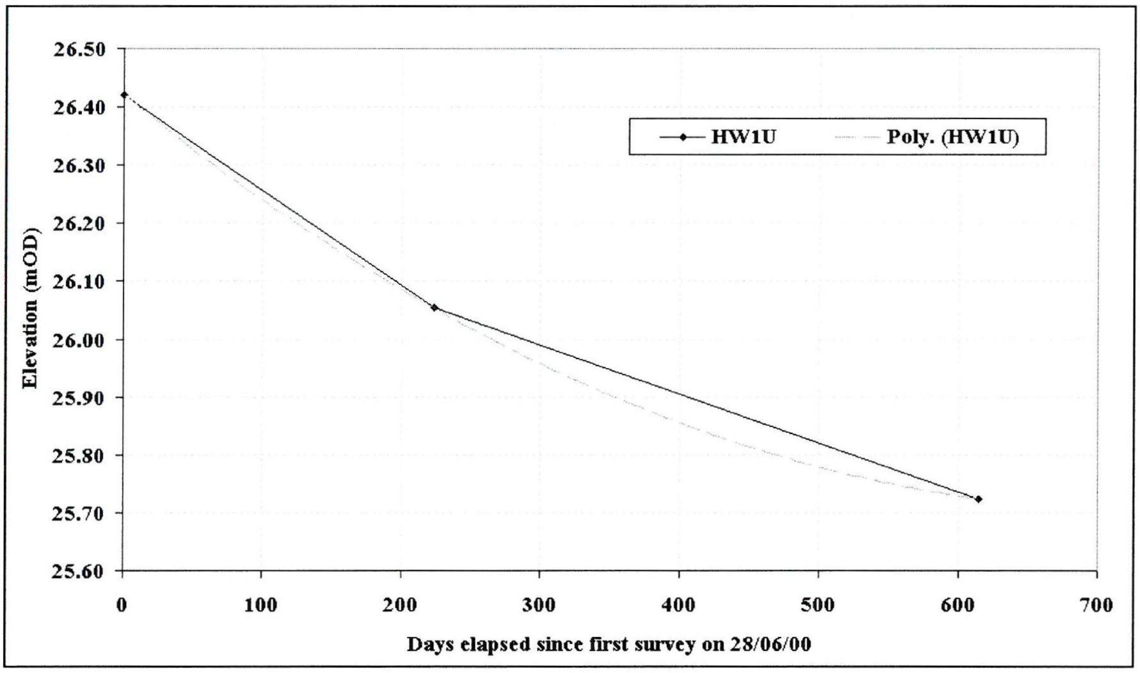

Figure 5.11 Settlement of HW1U piezometer tubing. Settlement rates were assumed to be linear.

Hudson *et al* (1999) noted that samples of waste tested in a large compression cell developed a distinct layering in planes perpendicular to the applied stress. This layering, which gave rise to anisotropic hydraulic conductivity, may develop (a) during the deposition and initial compaction of the waste, and (b) during subsequent loading and settlement. However, Watts and Charles (1999) state that although the settlement of a landfill is generally one-dimensional, the variability of the waste will usually mean that differential settlements are relatively large. It therefore seems probable that the layers may not remain truly horizontal, but become folded to some degree. As a result, the higher hydraulic conductivity pathways may follow these folds. Piezometers installed at identical elevations but some distance apart may therefore lie within different layers/folds and indicate different pore pressure values.

Finally, the gas and foam venting from 5 of the 42 piezometers often prohibited an accurate reading of the leachate level (Figure 5.12).

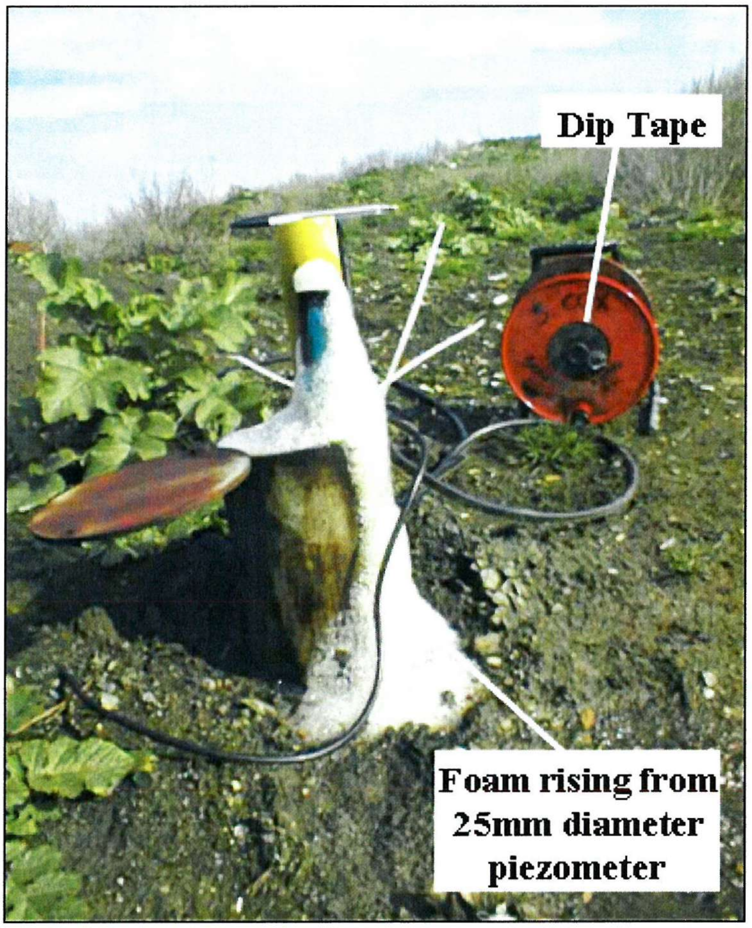

Figure 5.12 Foam rising from piezometer HW3M preventing measurement of the leachate level.

5.1.2 Magnet Extensometers

It was anticipated that the depressurisation of pore fluids as a result of pumping would cause compression of the waste. To quantify the rate and scale of settlement, magnet extensometers were installed. The design of these instruments is illustrated in Figure 5.13.

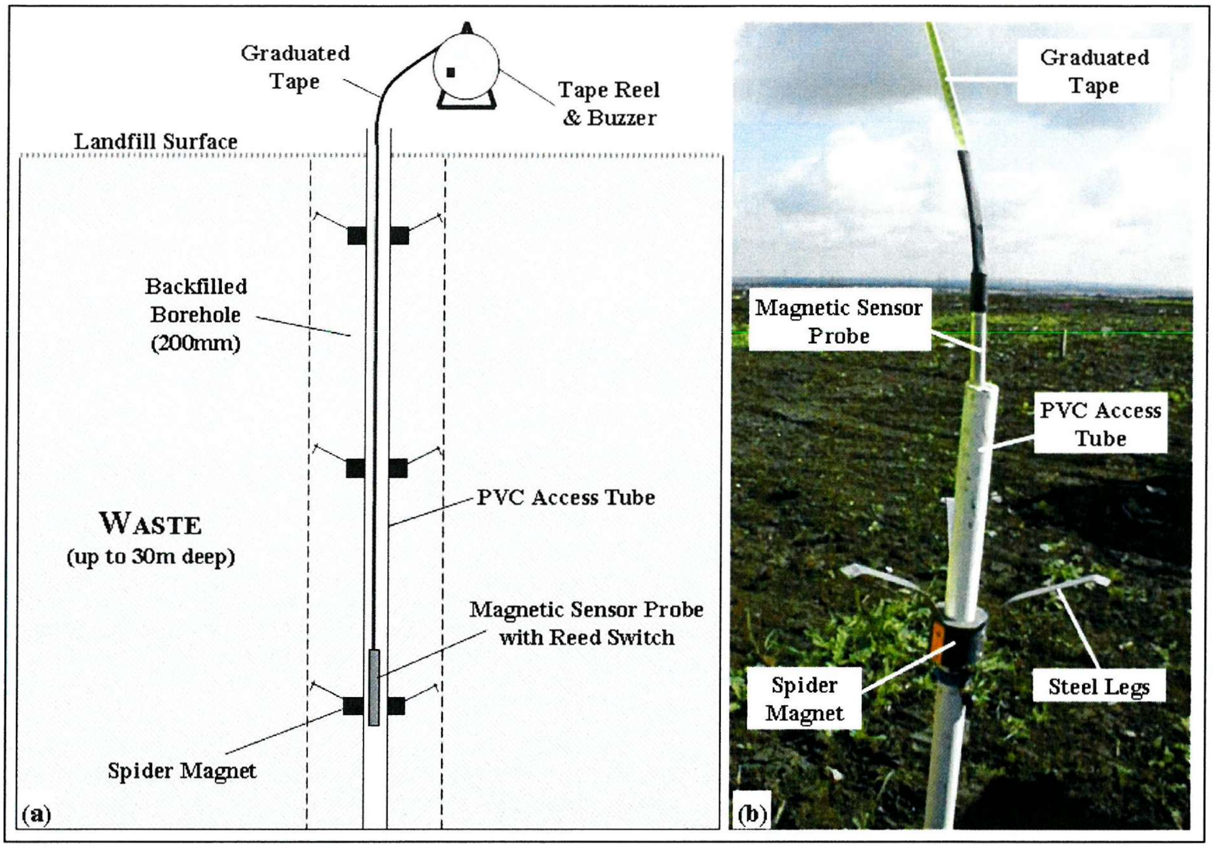

Figure 5.13 (a) Diagram of magnet extensometer installations, (b) photo of component parts.

In general, the system consists of a probe, steel measuring tape, tape reel with built-in light and buzzer, and a number of magnets positioned along the length of an access tube. The spider magnets are coupled to the surrounding waste through their steel legs and move down as settlement occurs. If heave is occurring the magnets will move upwards. Readings are obtained by lowering the probe into the access tube, when the probe enters a magnetic field, a reed switch closes, activating the light and buzzer. The operator then refers to the 1mm graduations on the tape and notes the depth of the magnet. Data from the extensometer can therefore indicate the distribution of settlement with depth as well as the total amount of settlement at the surface.

When the access tube is anchored in stable ground, the depth of each magnet is referenced to a datum magnet that is fixed to the bottom of the access tube. If the bottom of the access tube is not in stable ground, the depths of the magnets must be referenced to the top of the tube, which must be optically surveyed before readings are taken.

Four extensometers were installed, two at HW1 and two at HW3*. In an attempt to assess whether settlement rates changed with distance from each horizontal well, one extensometer was installed directly above the well and the other at a distance of 12m. Magnets were installed with depth spacings of 3 to 5m. Installation details for each extensometer, such as reference numbers and individual magnet elevations, are given in Appendix D.

5.1.2.1 Limitations of the magnet extensometers

As the waste at Rainham is underlain by silt, peat and clay it was not possible to anchor the access tubes in stable ground, so the use of basal datum magnets was not possible. However, optically surveying the top of the access tubes each time readings were taken was impractical. As with the piezometers discussed above, the settlement of the tubes between each survey was assumed to be linear.

The boreholes drilled for access tube installation were backfilled with a fluid mixture of bentonite/cement that would produce a firm solid when set†. This would support the magnets but not be so rigid as to prevent their movement during settlement of the surrounding waste. However, it is unlikely that the mixture could replicate the waste stiffness exactly and so the resulting movements may have either exaggerated or underestimated the true settlement of the waste.

The two extensometers installed directly above the horizontal wells were shortened so that drilling did not breach the well screen. With hindsight it would have be more revealing to drill full length extensometers perhaps 2 or 3m to the side of the horizontal wells so that the settlement at depth could have been more thoroughly assessed.

5.2 Leachate & Gas Extraction Systems

Before pumping from the wells could commence, systems had to be constructed that could extract, separate, measure and discharge the leachate and gas phases. The systems for each of

* The reference numbers are HW1R, HW1S, HW3O and HW3Q. These are marked on Figure 5.6 & Figure 5.7.
† Firm is defined in BS5930 (1981) as 'can be moulded by strong finger pressure'.

the horizontal wells were similar in design and a schematic diagram is given in Figure 5.14. A photograph of the system at HW2 is given in Figure 5.15.

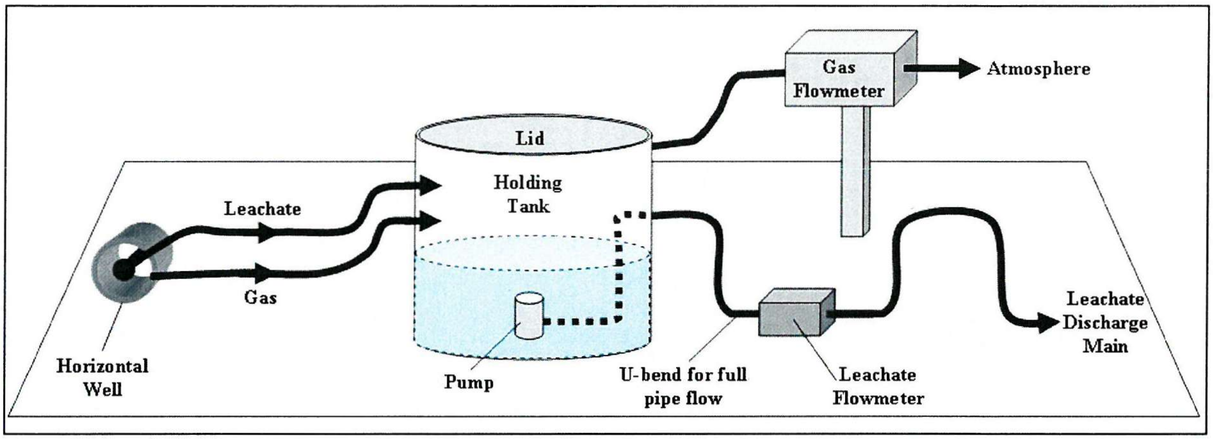

Figure 5.14 System for the collection of leachate and gas and the measurement of flow rates.

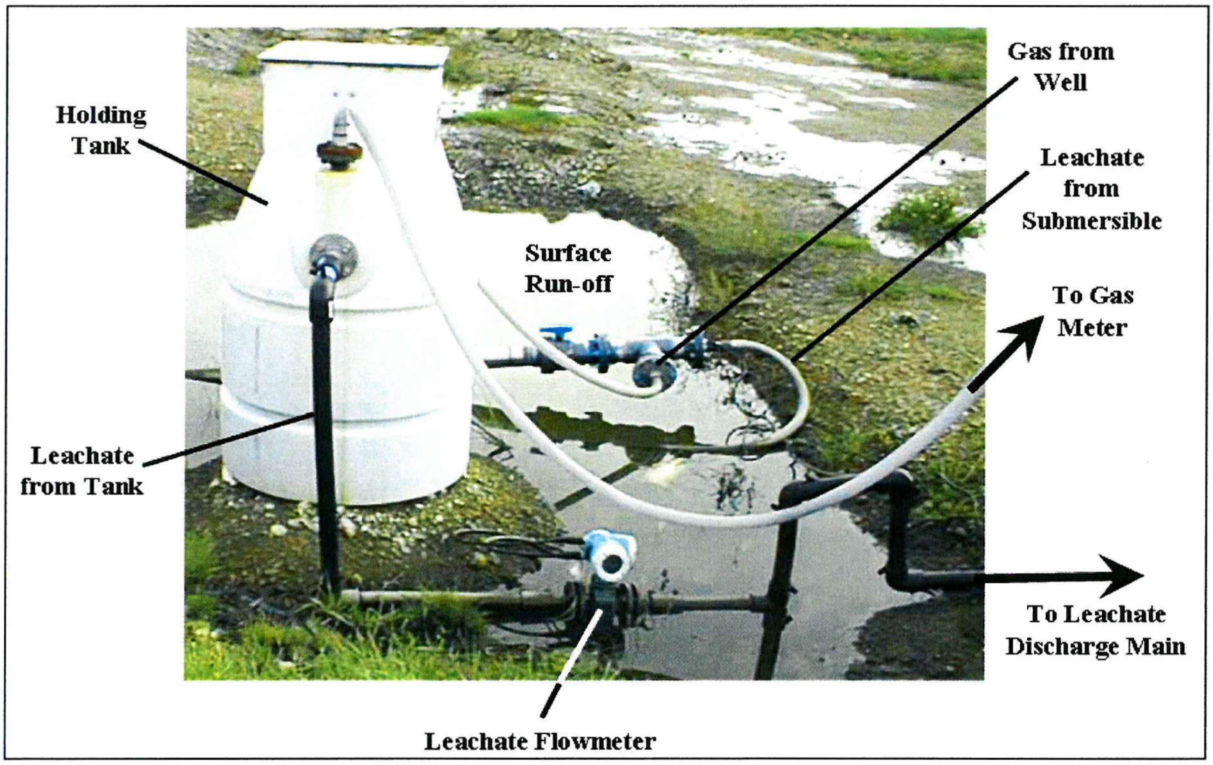

Figure 5.15 Leachate and gas collection system at HW2. The surface water is from heavy rainfall.

The submersible borehole pump, installed at the invert of the well, pushes leachate (and some gas) up through a steel riser pipe to the 'holding tank'. Gas that has travelled to the surface via the annulus between the steel riser and the well casing is channeled into the same holding tank. The purpose of the holding tank is to allow the gas and leachate phases to separate, i.e. any bubbles entrained in the leachate can rise to the liquid surface and rejoin the gas phase.

Any gas dissolved within the leachate as a result of the elevated pressures at depth can come out of solution. Gas escapes from the top of the tank and passes through a flowmeter before final discharge to atmosphere. The gas flowmeter is positioned higher than the holding tank so that any moisture condensing out of the gas will run back into the tank under gravity. When the leachate in the tank reaches a certain level, a float switch activates a pump which draws the leachate from the base of the tank into a 50mm diameter HDPE pipe. A second float switch deactivates the pump when the tank is near empty, and the tank is allowed to refill. Discharge from the tank is therefore periodic, with approximately 300 litres of leachate removed during each cycle. This leachate then passes through an electromagnetic flowmeter. For the flowmeter to provide accurate results the leachate must be flowing at full bore, so it is positioned at the invert of a U-shape in the pipework. The leachate then passes into a discharge main where it combines with leachate from other site extraction systems. The leachate from the whole landfill site eventually passes to the local waste water treatment works where it is treated and then discharged to the River Thames.

Well number HW1, which was artesian on completion, had been capped to prevent uncontrolled flow of leachate and gas while monitoring instrumentation was installed. When the cap was removed after four weeks, gas and leachate were emitted from the well in violent surges. A pressure gauge fitted to the end of the well recorded 0.3 bar, equivalent to 3m head at the capped end. By controlling the elevation at which gas and leachate could flow from the well, and lowering it in stages, a constant-head step-drawdown test was conducted. The four steps used are illustrated in Figure 5.16 and Figure 5.17.

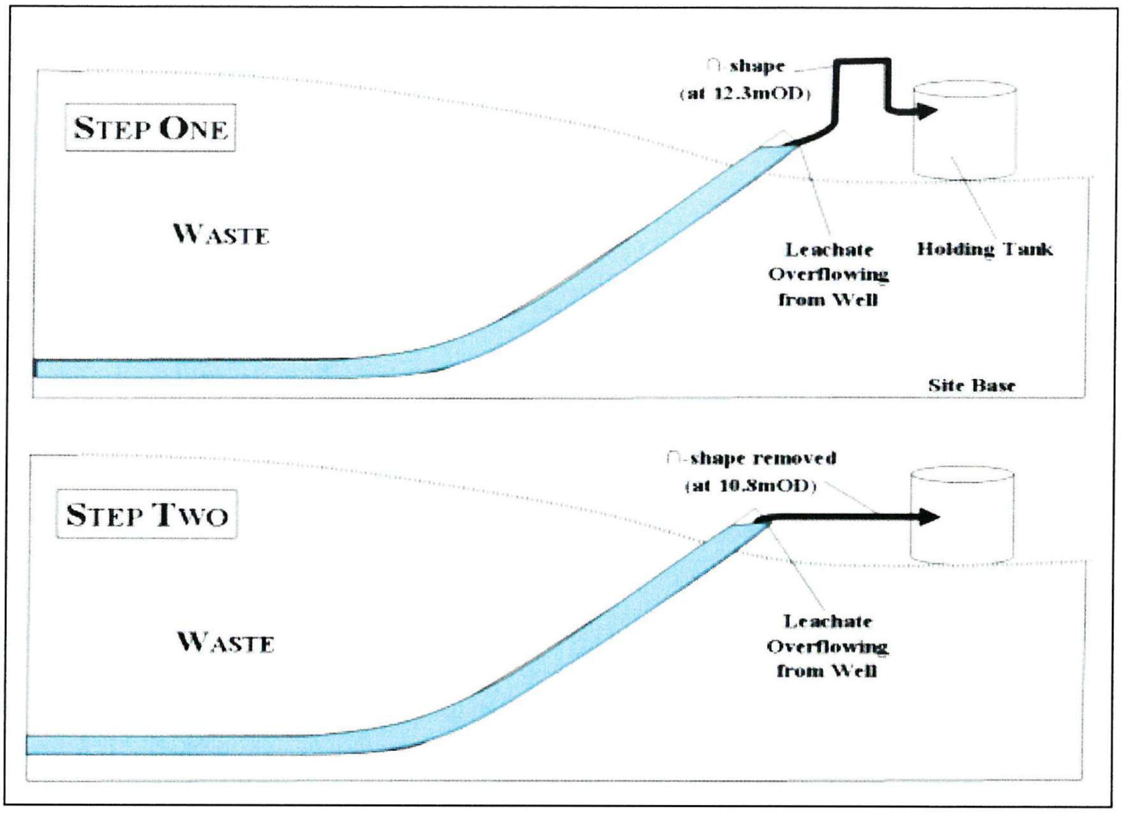

Figure 5.16 Steps One and Two, leachate overflowing, elevation of discharge point controlled.

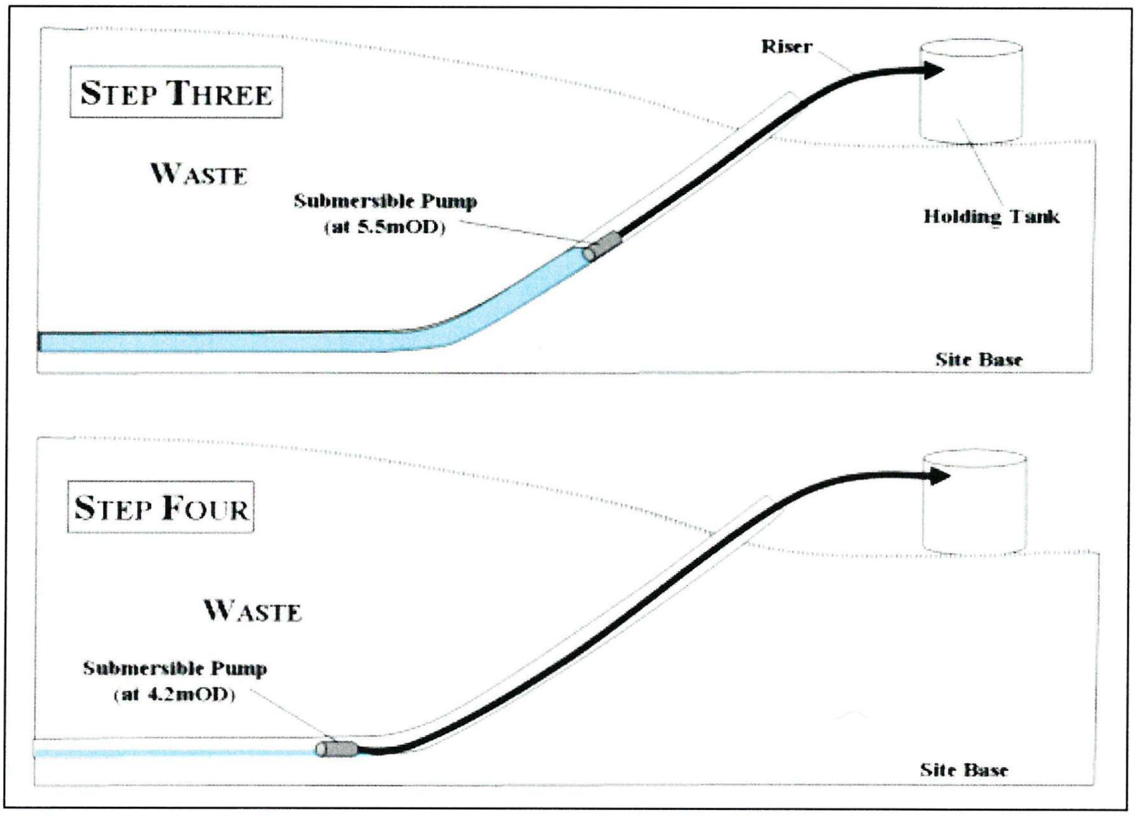

Figure 5.17 Submersible pump in use at two elevations.

- Step One used a 1.5m high ∩-shape in the pipework to raise the level to which the fluids must rise to flow from the well, Figure 5.18. 1.5m was half the pressure head recorded by the pressure gauge.
- Step Two, the ∩-shape was removed and fluids were again collected without the use of a pump.
- Step Three, a submersible borehole pump was installed part way down the rising section of the horizontal well.
- Step Four, the pump was placed in the horizontal section (the lowest part) of the well.

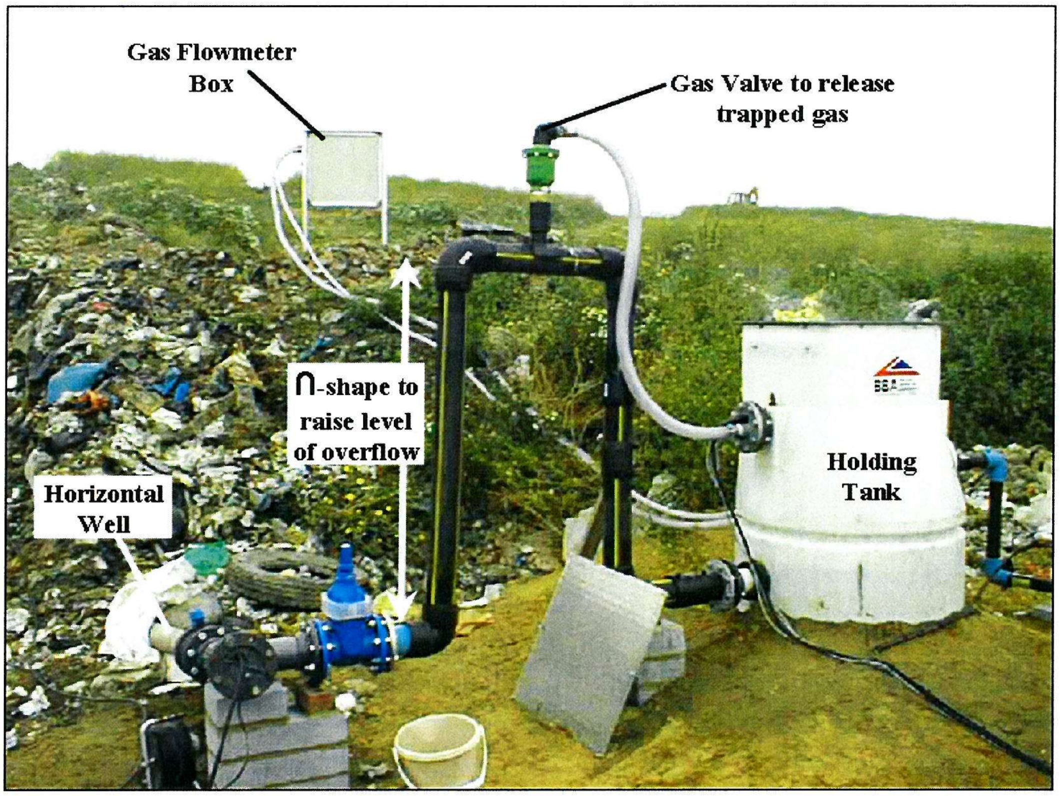

Figure 5.18 Leachate and Gas collection system at HW1 during Step One.

HW2 and HW3 were not overflowing when examined seven weeks after installation and step tests were not performed, instead the submersible pump was installed directly at the lowest part of the well and leachate extracted at the maximum rate.

The elevations of each well, submersible pump and well entry point are summarised in Table 5.1.

Table 5.1 Summary of elevations for each well and pump. All values in mOD.

Well Number	HW1	HW2	HW3
Well top	10.8	7.5	10
Horizontal section (average)	3.6	3.5	3.8
Vertical rise in well (m)	7.2m	4m	6.2m
Step 1 – overflowing	12.3	–	–
Step 2 – overflowing	10.8	–	–
Step 3 - submersible pump	5.5	–	–
Step 4/Constant Head - submersible pump	4.2	3.9	3.3

5.2.1.1 Limitations of the pumping systems

The flowmeter recorded the cumulative leachate volume which, together with the time elapsed between successive readings, was used to calculate leachate flow rates on a regular (usually daily) basis. Although the flow of leachate from each well into the holding tanks was continuous, a volume of approx. $0.3m^3$ had to accumulate in the tank before the pump within the tank would be activated. This ensured flow through the flow meter was at full bore but meant that the discharge from the tank was periodic. As the flowmeter was positioned downstream of the holding tank the fullness (or emptiness) of the tank at the time of reading had an influence on each successive estimate of the flow rate. This was particularly significant when flow rates were low, for example by February 2002 average daily flow rates from HW3 were $1.2m^3$/day, so $\pm$ $0.3m^3$ (depending on whether the tank was full or had just discharged) represented a ±25% variation.

The gas volumes were much higher than anticipated and as the holding tanks had not been designed to contain high gas pressures, a number of leaks occurred around the lid and ports when gas pressure increased during a flow surge. Attempts were made to seal the leaks with epoxy resin but this had only a limited effect.

The gas meters were designed for the measurement of dry natural gas and not moist landfill gas. As a consequence, water that condensed from the gas accumulated in the flowmeter and had to be periodically emptied (approximately once a week). If condensate was allowed to accumulate for too long, gas flow was impeded and more gas escaped (unrecorded) through the leaks in the holding tank. Trace constituents of the gas and condensate were thought to have had a corrosive effect as each flowmeter lasted only 4 to 10 months.

Gas was able to pass through the holding tank and gas flow meter continuously, whereas the discharge of leachate from the holding tank to the flowmeter was periodic. As stated above, each successive daily leachate flow rate calculation was therefore significantly affected by the fullness of the tank at the time of reading, whereas the gas flow rate calculation was not. This made the identification of any flow relationships between the two phases more difficult.

Since Summer 2000 a gas extraction system has been operational at Phase 2 of Rainham landfill. The gas is used for electricity generation with an output of 17MW. The system draws gas from more than 60 wells across the Phase with a suction of approximately 15kPa. In October 2001, one of the horizontal wells (HW1) was connected to this gas extraction system as in Figure 5.19.

Figure 5.19 Gas Extraction Connections at HW1

Gas was taken from the annulus between the well casing and the pump rising main, and was then piped to the electricity generating station. The gas did not pass through a flow meter, so flow rates were determined periodically using an orifice gauge*. This provided only a point reading of the flow rate which then had to be assumed to remain constant until the next flow reading was taken. This was not ideal as flow rates had previously shown daily fluctuations of up to $100m^3$/day and were therefore certainly not constant. In addition, the accuracy of the estimated gas flow rate became lower as the interval between readings increased.

Once the wells were fully operational, the site was visited on a weekly rather than daily basis, so from November 2000 four out of five daily readings were taken by various site operatives†. The operatives took readings routinely at the beginning of each day but often failed to record the specific time. The regularity of the data collection routine meant that the estimated time of reading was likely to have been correct to within 30 minutes in most cases. However, there are likely to have been occasions when the readings were taken at a different time of day making the time estimate incorrect. On a long term basis such inaccuracies were averaged out, but became important in the short term when considering relationships between, for example, flow rate and atmospheric pressure. The use of a fully automated system for recording both gas and leachate flow rates would have resulted in a significant improvement in the accuracy of the data.

* A flow gauge consisting of a thin orifice plate clamped between pipe flanges, with pressure takeoffs drilled into the adjacent pipes. The pressure differential is proportional to the flow rate.
† Readings were not taken at weekends.

CHAPTER 6

Results and Discussion

Following installation of the monitoring instrumentation as described in Chapter 5, operation of the three horizontal wells commenced. This chapter includes sections on leachate flow, gas flow, pore pressure response and waste compression. Using these results a thorough performance assessment of the horizontal wells is made.

6.1 Operation of the Wells

As a result of the overflowing condition, pumping from HW1 began with a constant-head step-drawdown test. The stages of the step test are described and illustrated in detail in section 5.2. The fourth stage of the step test, in which the submersible pump lay in the horizontal section of the well, was extended to become a maximum yield test. Wells HW2 and HW3 were not overflowing and no initial step drawdown tests were conducted. Maximum yield tests were carried out on these wells with the submersible pump positioned in the horizontal (and lowest) section of the well. Leachate extraction from the three horizontal wells is ongoing (June 2002), but the analyses below include data to the 19/02/02 only. Table 6.1 details the leachate volumes and average flow rates for each of the horizontal wells.

Table 6.1 Summary of leachate flows to 19/02/02.

WELL REF. N^{O}.	HW1	HW2	HW3	TOTAL
Slot Width	2mm	5mm	1mm	
Start of Pumping	04/07/00	16/10/00	22/08/00	
Days in Operation (to 19/02/02)	595	491	546	1,632
Volume of Drilling Fluid (Water) Used (m^3)	600	750	2,000	3,350
Total Leachate Volume Extracted (m^3)	4,152	2,377	1,322	7,851
Net Volume Extracted (m^3)	3,552	1,627	-678	4,501
Average Leachate Flow Rate (m^3/day)	7.00	4.84	2.42	4.81

While the table is a useful comparative summary, it should be noted that flow rates from each of the wells have varied significantly since pumping commenced. Nevertheless, the average flow rates from the three wells are substantially different with the greatest yield being from HW1. Although the net volume from HW3 remains negative, it is thought that the water used during drilling dispersed to other site extraction systems before pumping commenced as discussed in section 6.8.4. The changes in flow rate over time for each well are discussed in section 6.3.

6.1.1 A Volumetric Comparison of the Leachate Yield from the Vertical and Horizontal Wells

At the project's inception, horizontal wells were proposed as an alternative to vertical wells for the control of leachate. The following discussion compares typical flow rates from vertical wells at Rainham with the flows from the horizontal wells. The yields are placed in context by considering the annual recharge the site receives from rainfall.

Phase 2 covers an area of 35 hectares and receives 550mm/year of rainfall. The *effective* rainfall, i.e. that which is not lost through evapotranspiration or surface runoff, is 90mm/year [Cleanaway 2001b]. The volume of water that infiltrates the waste is therefore (90mm × 35ha) 31,500m^3/year. The horizontal wells discharge an average of (3 × 4.81m^3/day) 5,300m^3/year, or 17% of the effective rainfall. A new vertical well extraction system commissioned on Phase 2 during summer 2000 consists of 43 wells and has a combined yield of 20,800m^3/year, or 66% of the effective rainfall. However, 44% of this 20,800m^3/year comes from just 5 of the 43 vertical wells. Each of these 5 wells is located where former haul roads are buried and it is believed that the brick/hardcore material used in the construction of the roads is acting as a preferential pathway for flow [Cleanaway 2001b]. These wells are not considered typical and have been excluded from consideration in the following discussion. Additional leachate extraction measures from Phase 2 include perimeter trenches and fin drains. However, it is difficult to quantify their contribution as they also receive leachate from adjoining phases.

In summer 2000 an area of eight hectares along the southern and western flanks of Phase 2 was capped to prevent rainfall recharge in this location. This capped area coincides with the locations of HW2 and HW3.

The flow rate from HW1 was compared with the average flow rate from six vertical wells in its vicinity (uncapped area), and the flow rates from HW2 and HW3 were compared with nine vertical wells in the capped area. The horizontal wells are much longer than the vertical wells and a direct comparison of the flow rates is of limited use. Thus, the flow rates are compared on the basis of flow per m^2 wetted area, with the results summarised in Table 6.2.

Table 6.2 Flow rate comparison of vertical and horizontal wells.

		Average Flow Rate (m^3/day/well)	Flow Rate per m^2 Wetted Area* (m^3/day/m^2)
Uncapped Area	Vertical Wells (6 no.)	1.39	0.25
	HW1	7.00	0.10
Capped Area	Vertical Wells (9 no.)	0.49	0.11
	HW2 (before development)	2.19	0.03
	HW2 (after development)	7.04	0.10
	HW3	2.82	0.04

*** Wetted area is the area of well screen below the leachate level.**

Clearly each horizontal well has a greater yield than any single vertical well. This is not surprising given that the length of each horizontal well is 200m and lies entirely below the leachate level, whereas the vertical wells are at most 31m in length with only 10 to 12m below the leachate level. In both the horizontal and vertical wells the flow rates in the uncapped area are generally greater than those in the capped area, possibly because the infiltration of rainwater is limited by the cap.

When the flow rates are compared on a wetted area basis, the vertical wells yield more leachate than the horizontal, although HW2 (after development) is almost equal. However, it is not sufficient to compare the two well types on the basis of flow rate alone as the effect on pore pressures must also be considered, as is discussed later.

6.2 Well Development

BS6316 (1992) 'The Test Pumping of Water Wells', defines well development as:

'The physical and chemical treatment of a well to achieve minimum resistance to movement of water between well and aquifer.'

Development of the wells did not take place immediately after installation as it did during the Cardiff Bay and Midwestern trials but was carried out in May 2001 after eight to ten months of pumping. This was to allow the performance of the well before and after development to be quantified and compared.

The well development method used the techniques of jetting, surging and suction-pumping. The jetting was designed to dislodge fine materials, bacterial deposits and chemical precipitations that may have become lodged within the slots of the well screen. It was also intended that the jets would pass between the wire slots and dislodge fines and deposits from the surrounding waste to develop a natural filter around the well screen. Unlike the Cardiff Bay wells, the Vee-wire screen was not mounted on a perforated base pipe which would severely restrict the passage of jets into the surrounding formation.

The jetting head was attached to the drill rods of a small directional drilling rig, Figure 6.1. Water was pumped through the drill rods and out through the seven nozzles in the jetting head. Six nozzles directed water at the wall of the well screen and a seventh directed water forwards to break up any blockages in the well. The nozzles were 3mm in diameter.

Figure 6.1 Small directional drilling rig used for well development, showing jetting head in action.

Surging of the wells was carried out using a specially designed head fixed to the drill rods. This surge block included two rubber discs that formed a tight fit with the wall of the well screen (Figure 6.2a). By moving this rapidly backward and forward inside the well, transient increases and decreases of pressure should have been created thereby dislodging fines from the surrounding waste. The surge block also incorporated three jets to loosen material should a blockage be encountered.

A suction tanker (Figure 6.2b) was used to remove leachate and fines from the well. Given the length of the well, simply placing suction over the end may have resulted in a considerable loss of vacuum. To avoid this, a long suction hose was inserted into the well and moved to the distance required. The original intention was to conduct the suction pumping at the same time as jetting so that any loosened fines could be removed before they had chance to re-settle. To do this the suction hose would have to be less than 25mm in diameter due to the limited space within the 104mm I.D. well screen, and would also have to be fixed to the drill rods so that suction occurred in the correct place.

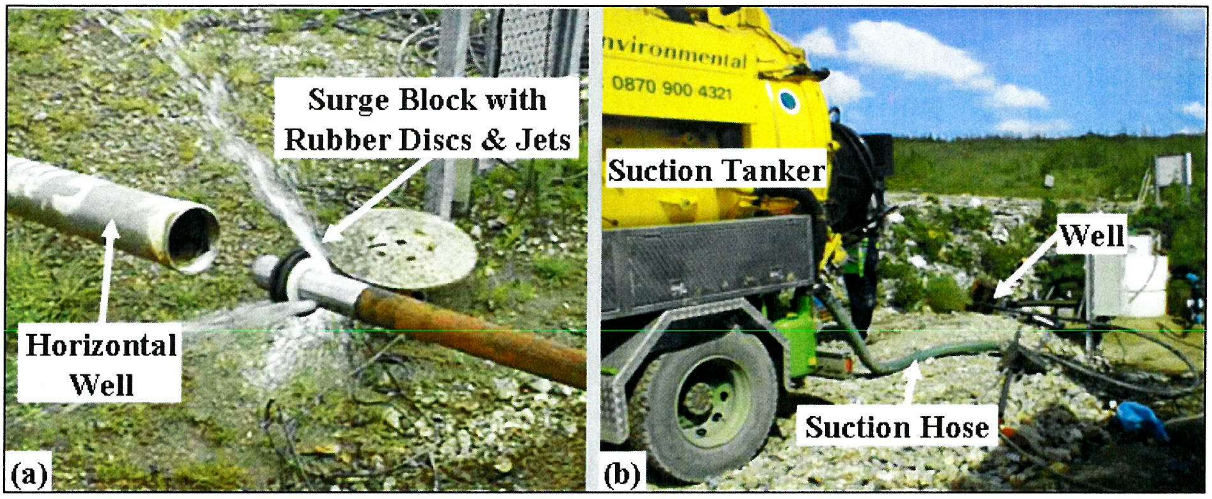

Figure 6.2 (a) Surge block with twin rubber discs to ensure tight fit in well and jets to loosen material if a blockage is encountered, (b) Suction tanker and hose used to draw leachate and fines from the well.

6.2.1 Development of HW3 (1mm slot width)

Well development began with the jetting of HW3. The jetting head was pushed (and rotated) through the well at a rate of approximately 1m per 30 seconds. The water pressure recorded by the drilling rig was in the order of 1000kPa, although the gauge was of low sensitivity and near the minimum level that would register. None of the water jetted into the well (approx. $4m^3$) flowed back out the well. Simultaneous suction-pumping with the 25mm hose was not carried out during the first jetting run in case any obstructions or constrictions were encountered. The jetting head passed through the full 250m length of the well indicating that the well remained intact and without blockages.

Once the first jetting run was complete, suction-pumping began using the 25mm hose. The hose was moved backwards and forwards within the well by hand to collect any sediment that had been dislodged during jetting. After 30 minutes the suction tanker overheated. The 25mm diameter hose was replaced with a 50mm hose so that there would be less resistance to flow and the problem with overheating did not recur. However, the use of the larger 50mm hose now prevented simultaneous use with the jetting assembly inside the 104mm diameter well.

Suction pumping appeared to be effective as fines could be heard* passing through the hose to the tanker. The leachate and gas flow was intermittent and surged from the suction hose. The surging effect is thought to be a result of two-phase flow through the hose and is discussed later in this chapter. The violent surges caused transient rises in flow velocity which may have allowed the entrainment and transport of larger sediment particles. When the tanker was full it was emptied into the holding tank (used for the separation of gas and leachate during pumping), and from the tank the water was pumped to the site discharge main. Using this system it was not possible to quantify the volumes of sediment removed. It would have been more informative to have emptied the tanker into a large settling tank so that the water could be drained off and the sediment volume measured.

Following suction-pumping, the surge block was connected to the drill rods and pushed into the well until submerged below the leachate. The rods were pushed backwards and forwards rapidly to create transient increases and decreases of pressure that would dislodge fines from the surrounding waste. To advance the surge block to the end of the well the rods were pushed 2m and pulled back 1m during each cycle. Once the full length of the well had been surged, the assembly was removed and suction-pumping recommenced. It was noted that the hissing sound of the sediment passing through the hose was quieter than before, indicating that less sediment had been dislodged. A sample was taken as the tanker discharged to the holding tank. The one litre sample contained only 20ml of sediment together with floating bits of paper and plastic. The grain size of the sediment collected was less than 1mm, i.e. small enough to pass through intact well screen slots.

The full potential of the surge block was thought not to have been realised for the following reasons;

a) expansion/compression of the gas in the wells during surging, and

b) the relatively slow speed at which the surge block could be moved in and out.

Following surging, a further two cycles of jetting and suction-pumping were carried out before the equipment was moved to HW2.

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* Prior to use in the well, the suction system had been tested in water free of sediment and the hissing sound of particles passing through the hose was not present.

6.2.2 Development of HW2 (5mm slot width)

In August 2000, before pumping had commenced from HW2, a drain rod was pushed into the well to see if it remained clear. The drain rod (approx. 12mm diameter) hit a blockage at 90m from the well entry point. The nature of the blockage was unknown. Attempts to investigate with a borehole camera proved inconclusive as the camera lights were ineffective in the turbid leachate. Despite the blockage, there may still have been hydraulic continuity with the 146m of well beyond. The blockage at 90m was one of the main influences in choosing a drilling rig to perform well development, rather than a conventional sewer jetting system, as a rig would be able to provide more thrust. Immediately before jetting began with the rig (after eight months of pumping) investigation with a drain rod showed that the blockage at 90m remained.

The first stage of well development in May 2001 was jetting. The jetting head was fixed to the drill rods and pushed into the well. At a distance of 36m (the point where the plain well casing joined the slotted well screen) the drill rods met some resistance to pushing and were slowly advanced to 38m. The rods were then pulled back to determine whether the apparent blockage had been cleared but they again became stuck at 36m. The rods were pulled with increasing force to try and free them but the well casing began to pull from the ground (by approx. 0.5m). A thrust block was placed between the well and the rig so that the rods could be pulled with more force without shifting the well casing further. Eventually the drill rods were freed and the jetting head brought back to the surface. On inspection, metal from the Vee-wire well screen was wrapped around the jetting head and two of the brass nozzles had sheared off, (Figure 6.3b). The drain rod was then pushed into the well, which was found to be blocked at 36m. The ease with which the well casing pulled from the ground and the retrieval of Vee-wire suggested that the action of the jetting assembly had severed the well. As with the blockage at 90m, it was not clear whether hydraulic continuity with the rest of the well had been maintained.

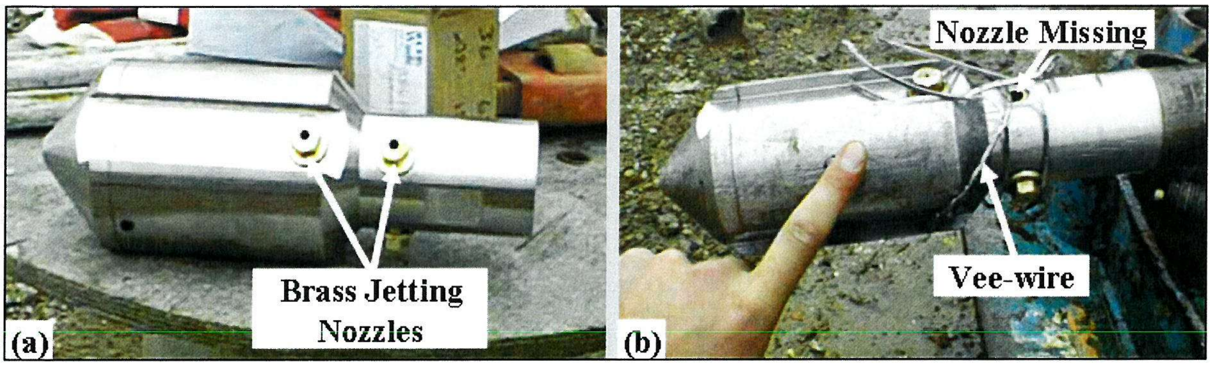

Figure 6.3 (a) Jetting head as new, complete with six brass jetting nozzles, (b) after use in HW2, wrapped in Vee-wire from well screen with brass nozzle sheared off.

With hindsight the design of the jetting head was regarded as poor. The principal fault was the large diameter (95mm), which left only a small annulus in the 104mm diameter well. A second fault was that some of the nozzles stood proud of the main body and were able to catch on the Vee-wire screen.

Despite the damage to the well, suction-pumping was carried out within the first 36m. Sediment could be heard hissing through the suction hose and in 30 minutes 3m^3 of leachate was extracted. In addition, some larger particles were heard rattling through the hose, so after the leachate in the suction tanker had been discharged to the holding tank, the tanker door was opened. Approximately 20 litres of fibrous sludge lay in the base and contained silt, sand and gravel up to 50mm in diameter (the size of the suction hose). Paper, plastic and glass fragments were also present. Particles this size would not have passed through the 5mm slots if they had remained intact.

HW2 had not produced gas during normal pumping, unlike the 300 to 600m^3/day produced by the other two wells. However, after 15 minutes of suction pumping, gas began to vent from the well and surge through the suction hose. Gas flows are discussed in section 6.5.

Despite the damage and possible blockage of the well, the submersible pump was reinstalled to its former position (at a distance of 36m) and the maximum yield test recommenced.

6.2.3 Development of HW1 (2mm slot width)

The jetting head was not used in HW1 for fear of damaging a second well. With the rubber discs removed the surge block could be used as a jetting tool, although the direction of the jets was 45° to the drill rod, not 90°, (Figure 6.2).

Jetting of the well with the modified surge block continued until a solid obstruction was encountered at 210m from the entry point. Despite prolonged jetting at this point to try and dislodge the blockage, the drill rods could not be pushed any further. It seems likely that the blockage was more substantial than just sediment in the well, and may have been a collapse/constriction of the well screen itself.

Following jetting, suction-pumping was carried out. The suction hose was pushed to the blockage at 210m then pulled slowly through the well at a rate of approximately 1.5m/min so that dislodged sediments could be collected. In 2.5 hours, 3m^3 of leachate containing sediment had been extracted. A second cycle of jetting and suction-pumping was then carried out. On this occasion, the suction hose was notably more difficult to push as the length increased and on this occasion could not be pushed further than 180m. The suction was activated and sand could be heard travelling through the hose. As suction continued it was possible to push the hose gradually further into the well, suggesting that the suction was clearing sediment that the jetting had dislodged from the surrounding waste. However, the blockage at 210m remained.

A further two cycles of jetting and suction-pumping were performed. After the end of each of the third and fourth cycles the 3 to 4m^3 of leachate that had been extracted was drained slowly from the tanker so that the sediment would be retained. Approximately 25 litres of sediment was left in the tanker on each occasion. The sediment was described as follows.

A dark brown/black fine-coarse humic sand, including fragments of brick, glass and plastic. Pockets of matted fibres. Oily aroma.

A particle size distribution analysis of the sediment was conducted and gave the following results.

Table 6.3 Particle Size Distribution of sediment removed from HW1 during well development.

	Size (µm)						Size (mm)			
Sieve Size	63	150	212	300	425	600	1.18	2	5	8
	Percentage by mass passing through sieve									
Sample 1	5.8	13.9	20.4	29.8	46.4	63.2	83.1	90.6	95.9	97.7
Sample 2	1.8	5.9	11.1	19.7	32.3	43.2	61.9	71.0	82.2	84.9
Average	**3.8**	**9.9**	**15.8**	**24.7**	**39.3**	**53.2**	**72.5**	**80.8**	**89.1**	**91.3**

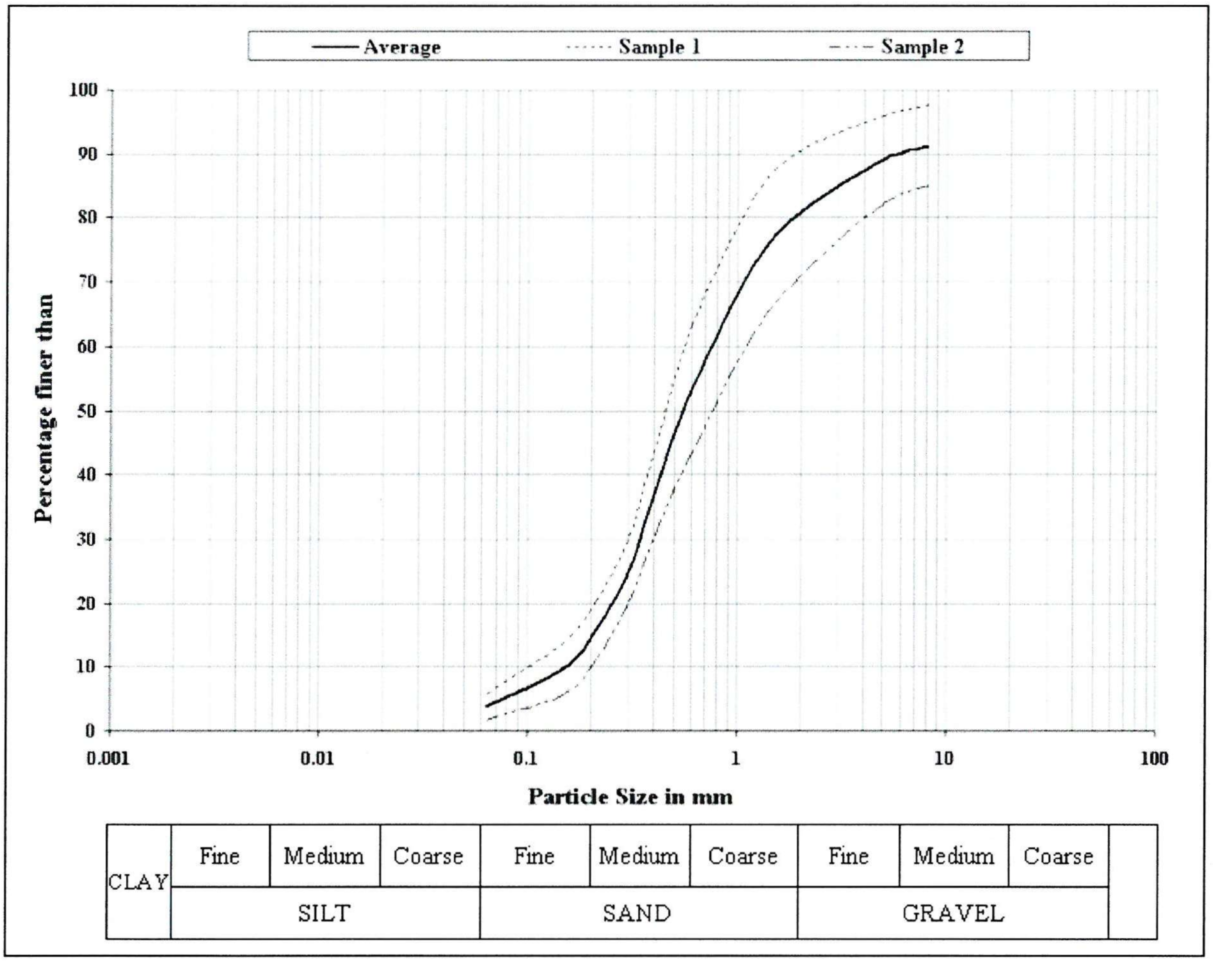

Figure 6.4 Particle Size Distribution curves for sediment collected from HW1 during well development.

The curve illustrates that approximately 70% of the sample is medium to coarse sand with only 10% fine sand or smaller. It is probable that much of the clay and silt was carried in suspension from the suction tanker while it was slowly drained. There is a significant fraction (20%) greater than the 2mm slot width of the HW1 well screen. Some of these larger particles may have passed through the slots longitudinally but others were clearly too large. It therefore seems possible that some of the wires forming the well screen had become bent or broken allowing larger particles to pass through.

Three weeks after development, on the 18/06/01, pumping recommenced with a new submersible pump installed in the same position as the old one (80m into the well at 4.2mOD). Pumping continues to date (June 2002).

6.2.4 Well Development – Discussion & Recommendations

One of the main reasons for selecting a directional drilling rig to carry out the well development in preference to more conventional sewer-jetting tanker units was that it would enable difficult obstructions in the well to be pushed through with some force. With hindsight this was perhaps a mistake for the following reasons:

a) HW3 did not contain any blockages.

b) A small jetting head may have reached the blockage at 90m in HW2, whereas the larger jetting head only reached 36m before damaging the well screen.

c) Despite the thrust available from the rig, the blockage in HW1 (at 210m) was not overcome.

d) A subsequent review of high pressure water jetting techniques suggested that it may have been possible to apply very high pressure jets to cut through blockages [The Water Jetting Association, 2001; Waterjet Technology Inc., 2001]. This remains a possibility at least for HW1.

A second reason for using a directional drilling rig was to enable the suction hose to be fixed just behind the jetting head for immediate collection of dislodged fines. However, the resistance to flow in the 25mm suction hose proved to be too great for the suction tanker and a larger 50mm diameter hose had to be used. The result was that jetting and suction could not occur simultaneously.

It now seems feasible that both jetting and suction pumping could have been achieved using only a sewer-jetting tanker unit. Furthermore, the jetting hose of such a unit may have been of a smaller diameter, allowing the simultaneous use of the suction hose. However, such units are not designed for simultaneous jetting and suction and two separate tankers would have been required. Jetting with a tanker unit would have been quicker than using the drill rig, principally because the process would not be delayed by the repeated addition (and removal) of each drill rod.

The function of the jets was to (a) penetrate into the waste to dislodge fines, and (b) cut through sediment blockages within the pipe. The Water Jetting Association (2001) stress the importance of nozzle design, system pressure and jet velocity for these applications. A more detailed consideration of these factors is required in future.

The vertical distance between the horizontal section and the open end of each well was 7.2m, 4m and 6.2m for HW1, HW2 and HW3 respectively. The two larger values are close to the practical limit for vacuum lifting. The removal of leachate and fines may therefore have only been possible because the gas had reduced the density of the fluid. Therefore the use of the suction method on similar well installations in non-gassing landfills might not be successful.

The domed topography of the Rainham site dictated that a continuous (two-ended) horizontal well would need to be greater than 400m in length. As a result, shorter, blind-end wells were installed. With HW1 blocked at 210m and HW2 blocked at both 36m and 90m, the advantages of a second accessible end for both development and pumping became clear.

Although there is no direct evidence it is thought that the damage of HW2 at the joint between the plain casing and the well screen may be related to the lack of strength of the well screen under axial load. The damage may have occurred during the final stage of well installation when, during removal of each 9m section of overwash casing, the well screen repeatedly moved out of the borehole by up to 3m. On each occasion the well screen had to be pushed back in and this repeated application of tensile and compressive force may have caused the screen to fail at the joint with the stronger plain casing (at 36m).

6.3 Leachate Flow

6.3.1 HW1

Figure 6.5 shows the leachate flow rate as a function of time for HW1. The graph also shows the cumulative leachate volume extracted. The well screen slot width of HW1 is 2mm. The average flow rate between July 2000 and February 2002 was $7m^3$/day.

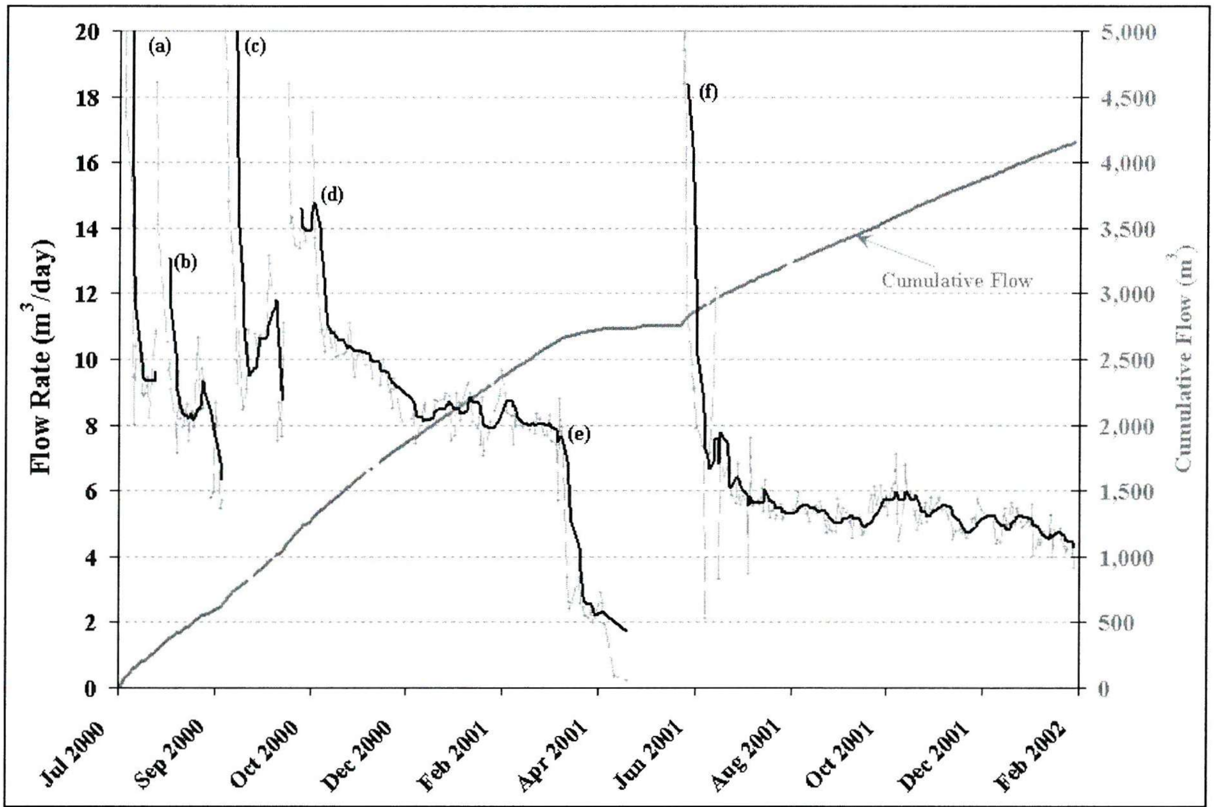

Figure 6.5 HW1 daily leachate flow rate and cumulative leachate volume. The seven day rolling mean* of the daily flow rate (bold line) is also shown to average out the often large daily fluctuations.

The letters (a) to (f) on Figure 6.5 mark the following events.

(a) Step 1, leachate flowing from the well under artesian pressure with the outflow point at an elevation of 12.3mOD†. Start 04/07/00. Finish 25/07/00.

(b) Step 2, leachate flow still under artesian pressure with the outflow point at 10.8mOD. Start 25/07/00. Finish 05/09/00.

(c) Step 3, submersible pump at 5.5mOD. Start 05/09/00. Finish 13/10/00.

* The rolling mean is the average flow rate of the current day, the preceding three days and the following three days, i.e. one week average.

† A pressure gauge mounted on the end of the well recorded 0.3 bar pressure, equivalent to 3m head. The outflow point of the well was raised by 1.5m to control the flow for the first step.

(d) Step 4, submersible pump lowered to invert of well at 4.2mOD. Two weeks into Step 4 the electricity supply to the pump failed for two days, causing a second peak in flow when the pump switched back on. Start 13/10/00. Finish 09/04/01 (when pump failed).

(e) Pump failure, 09/04/01, and well development, May 2001.

(f) Pump restart, 19/06/01.

It was thought that by controlling the elevation at which gas and leachate could flow from the well and lowering it in stages, a constant-head step-drawdown test could be conducted. Step 1 ran for three weeks, Step 2 for six weeks and Step 3, using a submersible pump, ran for five weeks. Step 4 then ran for six months until the pump failed. It is clear from Figure 6.5 that the leachate flow rate during Steps 1 to 3 had not reached equilibrium before the next step began. At the time it was observed that the flow volume over some 24 hour periods exceeded that of the previous 24 hours and as flows had apparently ceased to fall it was thought equilibrium had been reached. These fluctuations are now thought to be related to variations in atmospheric pressure, discussed in section 6.7. Furthermore, it would not have been feasible to have allowed more time at each step for true steady state conditions to develop.

Flow rates were initially high (>20m^3/day equivalent) at the start of Steps 1 to 3, but fell to less than 12m^3/day within a week of the start of each step. Piezometric levels in the surrounding waste actually fell below the level of the exit point during Steps 1 and 2 yet leachate continued to flow without a pump. This is illustrated in Figure 6.6, the levels shown represent the actual levels recorded in HW1 and the piezometer HW1U during Step 2.

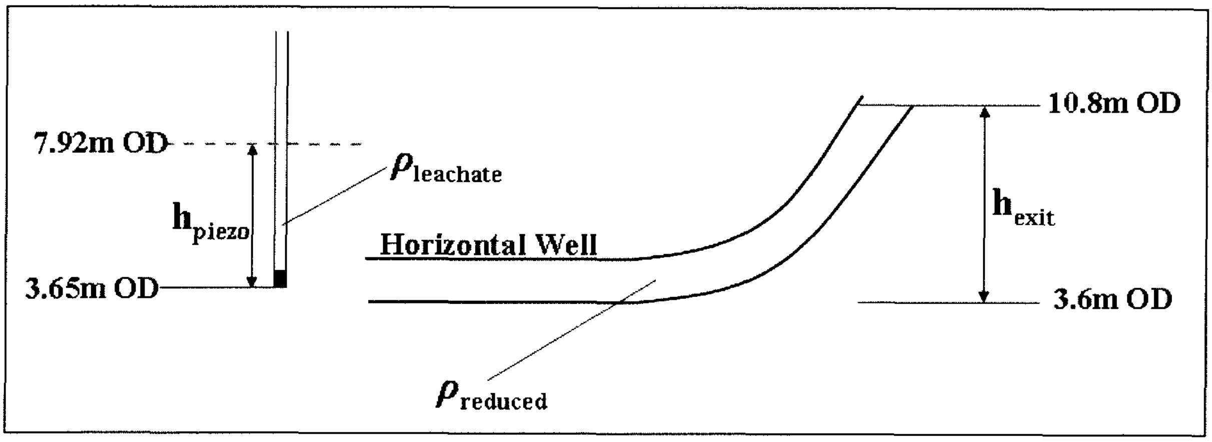

Figure 6.6 Levels recorded in HW1 and piezometer HW1U during Step 2 while artesian flow continued.

For this to be possible the density of the fluid in the horizontal well must have been reduced by the gas entrained within it. This is explained by the following equation.

$$\rho_{\text{leachate}}\ g\ h_{\text{piezo}} > \rho_{\text{reduced}}\ g\ h_{\text{exit}} \tag{6.1}$$

ρ_{leachate} = density of leachate

ρ_{reduced} = density of leachate mixed with gas

h_{piezo} = height of water column in piezometer above horizontal section of well

h_{exit} = height of exit point above horizontal section of well

Provided ρ_{reduced} was small enough, leachate would rise to the exit point even though h_{exit} was greater than h_{piezo}. Using equation (6.1) and the levels from Figure 6.6, the minimum gas composition that would allow artesian flow was calculated (Box 6.1).

Box 6.1 Gas composition required to enable artesian flow.

$\rho_{\text{leachate}} = 1000\text{kg/m}^3$

$h_{\text{piezo}} = (7.92 - 3.65) = 4.32\text{m}$

$h_{\text{exit}} = (10.8 - 3.6) = 7.2\text{m}$

$\rho_{\text{reduced}} = ?$

$1000 \times g \times 4.27 > \rho_{\text{reduced}} \times g \times 7.2$

$\rho_{\text{reduced}} < 593\text{kg/m}^3$

Gas must therefore have formed at least 41% (by volume) of the fluid mixture in the horizontal well to allow artesian flow under these conditions. As discussed later the actual gas to leachate volume ratio discharged from HW1 was 52:1 (or 98% gas by volume), so the reduction in fluid density is a plausible explanation for the continued artesian flow. However, the 98% value was obtained by measurements made at atmospheric pressure. Had the gas volume been measured at, say, twice atmospheric pressure (equivalent to a water depth of 10m and therefore a greater pressure than that in the well) the gas percentage would have halved to 49%. Nevertheless, this value remains high enough to explain the artesian flow.

The duration of Step 4 was extended so that it became a maximum yield test. The flow rate between October 2000 and April 2001 fell gradually from approximately 11m^3/day to 8m^3/day, representing an average reduction of 115 litres/week over this period. In April 2001 the submersible pump failed and overflowing conditions were re-established (hence the small volume of flow after point (e) on Figure 6.5). At this time it was thought that the gradual reduction in flow may have been due to clogging of both the well screen and surrounding waste with bacterial deposits and/or fines [Rowe, 1996], so before a replacement pump was installed well development was carried out as discussed above.

When pumping recommenced in June 2001 the recharge of leachate that had occurred while the pump had been out of commission, caused the initial flow rate to be high (>20m^3/day). However, after one week the flow rate had fallen back to its pre-development value and it appeared that well development had made no significant difference. There are three possible explanations for this;

1. The well development techniques were ineffective, or only temporarily effective.
2. The wells were already ‘developed’ as a result of prolonged pumping.
3. The decline in flow rate was unrelated to any clogging of the well but represented a continued and gradual approach toward equilibrium over a prolonged time-scale consistent with the low permeability of the waste.

Between June 2001 and February 2002 the flow rate decreased gradually from 7m^3/day to 4.2m^3/day, representing an average reduction of 86 litres/week over this period. This average weekly reduction was less than the 115 litres/week reduction for the previous period, suggesting that the flow rate was approaching equilibrium. However, as demonstrated later, leachate levels around HW1 had stabilised by February 2002 and the hydraulic gradient was therefore constant, yet Figure 6.5 shows that flows from HW1 continued to decline. This suggests a gradual reduction in hydraulic conductivity, possibly related to clogging of the well screen.

6.3.2 HW2

Figure 6.7 shows the leachate flow as a function of time for HW2 leachate. The cumulative volume of leachate extracted is also shown. The well screen slot width of HW2 is 5mm.

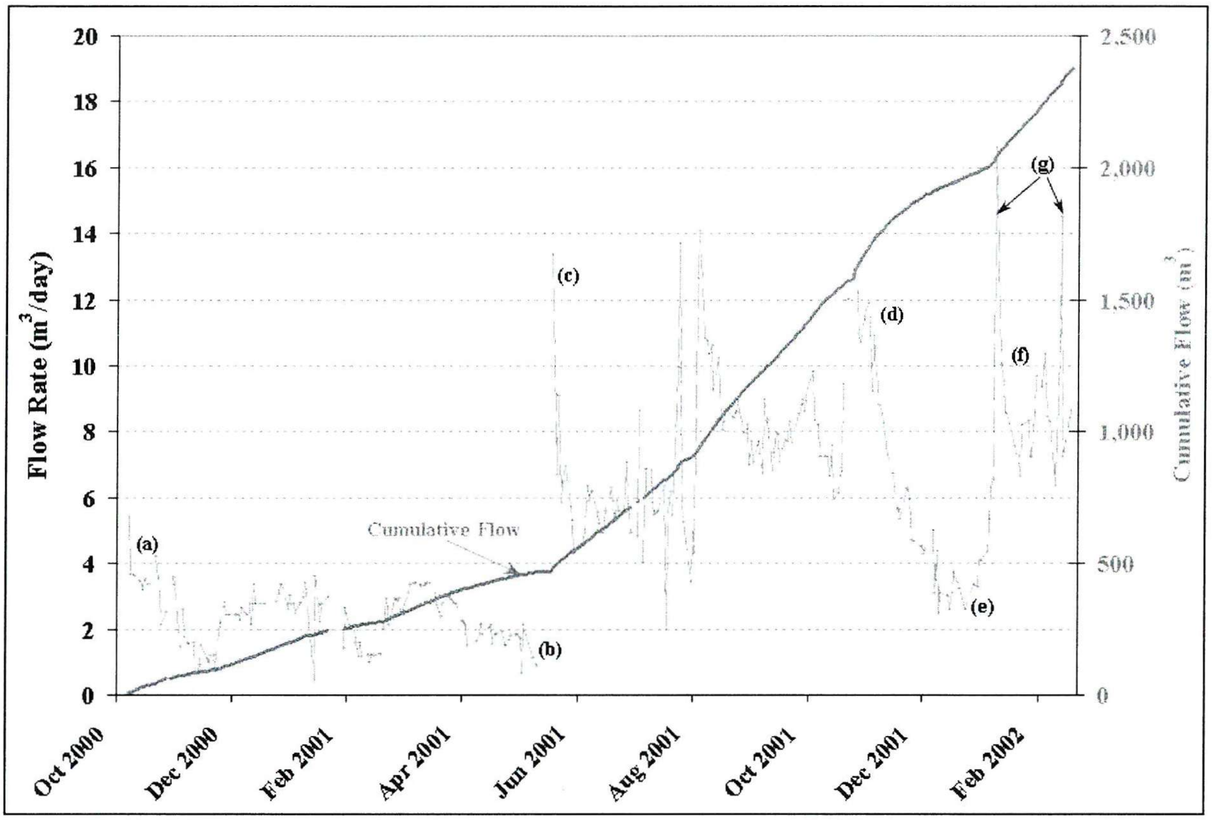

Figure 6.7 HW2 daily leachate flow rate and cumulative leachate volume.

The letters (a) to (g) on Figure 6.7 mark the following events.

(a) Start of pumping 16/10/00, submersible pump at 3.9mOD.

(b) Pump stopped to conduct well development 18/05/01, (damaged sustained at 36m from outflow point).

(c) Pumping restarted despite damage to well 25/05/01, flow rate much higher than before.

(d) – (e) – (f) Long term fluctuations in flow.

(g) Short lived peaks in flow.

During and immediately after the installation of HW2 the well was artesian with a flow rate of approximately 0.7 l/s (equivalent to 60m^3/day). The well was capped to prevent uncontrolled discharge while piezometers and pumping systems were installed. When the cap (at an elevation of 7.5mOD) was removed there was no longer artesian flow. This suggested that it may have been the injection of drilling fluid that had raised pore pressures enough to cause flow but that these pressures had since dissipated. The leachate level was measured as

6.2mOD*. As there was less than a 3m height difference between the leachate level and the horizontal section of the well (at 3.3mOD) it was decided that a constant head step test, like that attempted at HW1, would produce no useful data. On installation of the submersible pump to commence a maximum yield test, it was discovered that the pump could not be pushed further than 36m into the well. This was 0.6m higher than the lowest (horizontal) section but pumping began nevertheless. The extent of the damage at 36m prior to well development was unknown at this time but a drain rod and 50mm diameter hose† were able to pass this point without difficulty. It was during later well development that more severe damage was caused.

As the graph illustrates, the flow rate from HW2 has undergone significant changes and it is difficult to identify clear patterns or predict what the flow rate might be in future. Over the six weeks after pumping commenced in October 2000, flow rates gradually declined from 4 to $1m^3$/day. Flows then fluctuated between 1 and $3m^3$/day until May 2001 when the pump was removed to conduct well development.

It should be noted that unlike HW1 (and HW3) the well did not produce any gas. Given the large gas volumes produced by the other two wells the reason for the total absence of gas at HW2 was unclear and is discussed in section 6.5. The flow of leachate from HW1 and HW3 surged, while flow from HW2 did not, supporting the idea that the surging was an effect of the two phase flow of leachate and gas.

As described above, HW2 was severely damaged during well development and appeared to be blocked where the plain casing joined the well screen. Nevertheless the pump was reinstalled to see if any leachate would flow and surprisingly the flow rate was actually higher than before at $6m^3$/day. After May 2001 the flow rate underwent both long-term and short term fluctuations and averaged $7m^3$/day. Therefore the waste blocking the damaged well at 36m (the junction between the plain and the slotted screen) could not have represented a hydraulic barrier. The 200m of well screen beyond the blockage presumably continued to function to some extent, providing a preferential pathway for leachate to flow

* To measure the leachate level in the horizontal wells the dip tape was tied to a drain rod and pushed into the well. The distance to the leachate was recorded. The leachate level was then calculated from the drilling logs which give borehole elevation against distance.

† When the drain rod did not pass further than the blockage at 90m a 50mm HDPE hose was pushed into the well to try and shift the blockage but failed. The 50mm hose did not experience any difficulty passing 36m.

toward the pump. It is possible that the suction-pumping that was carried out with the end of the hose at 36m, removed enough of the blocking material so that the permeability was high enough not to impede flow.

The low flow rates recorded prior to well development may have been due to a problem with the pump. Although it was noted that the flows of 1 to $3m^3$/day were low, this was thought to be the maximum yield of the well. The sudden and sustained rise in flow rate to $6m^3$/day after the well was damaged brought the previous flows into question. Indeed, on removal of the pump from the well to enable development, sediment was found partly blocking the pump intake area, which as shown in (Figure 6.8), was small. This was washed clean before re-installation.

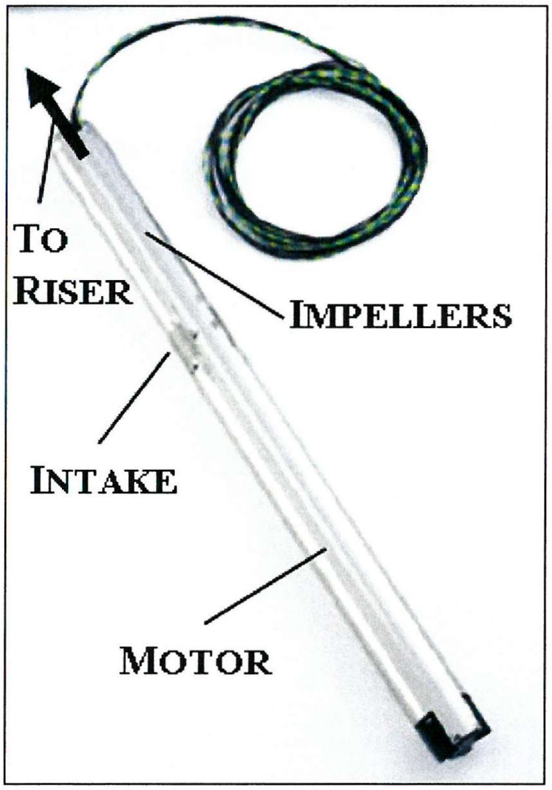

Figure 6.8 The submersible pump (Grundfos SQE), note small intake area. [Source, Grundfos 2000].

While a partial blockage of the pump may explain the low flow rates prior to well development, it cannot explain the large scale fluctuations that occurred afterwards. The possibility that pumping from nearby vertical wells might have been responsible for variations in the yield of the horizontal well was therefore investigated.

Within 50m of HW2 there are three vertical leachate extraction wells (numbers 322, 332 & 341). Between Summer 1998 and Autumn 2000 these wells had been non-operational, but the site operators commenced pumping from the wells between 17/11/00 and 28/12/00. The flow rate changes of the three vertical wells were closely correlated with one another as a result of alterations to and servicing of the compressed air system which ran all the vertical well pumps. The combined flow rate of the three wells is compared with the flow rate of HW2 in Figure 6.7.

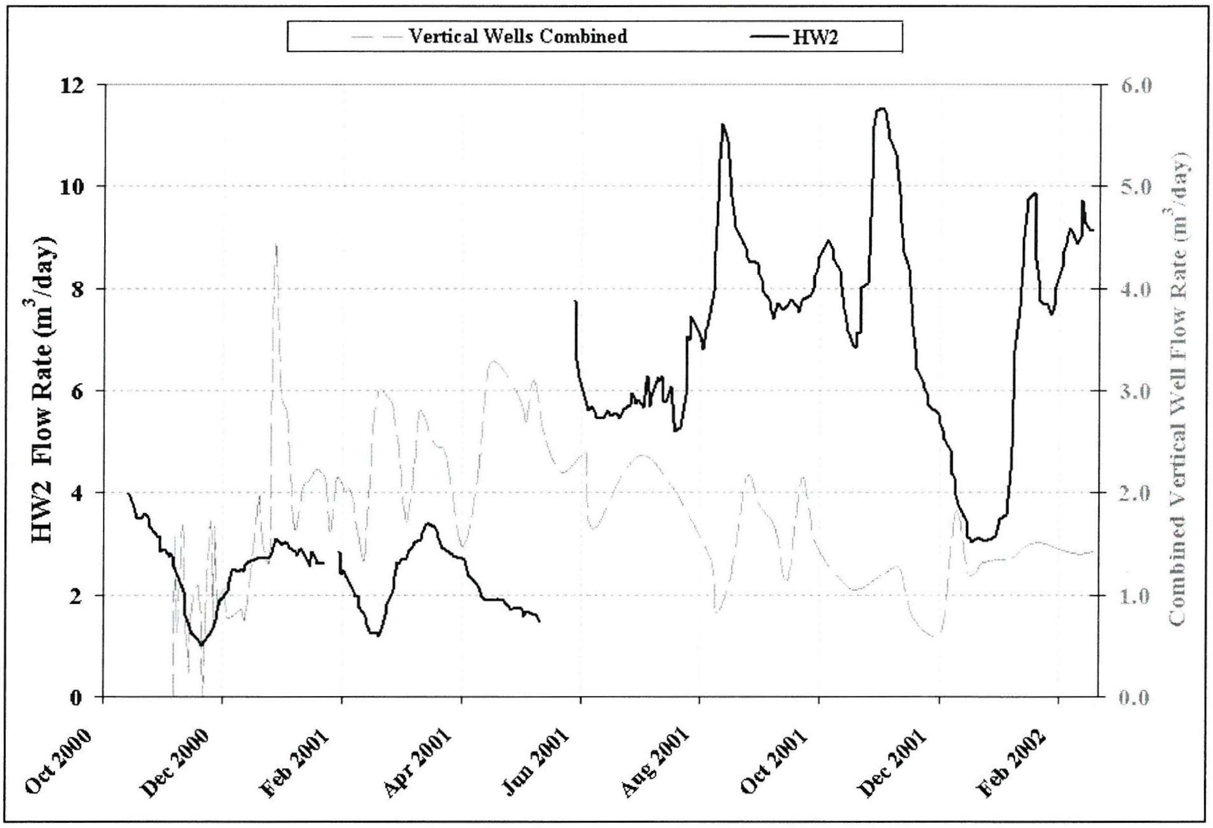

Figure 6.9 Comparison of daily flow rates of HW2 and three proximal vertical wells.

No consistent correlation is apparent between the flow rates of HW2 and the vertical wells and it is concluded that pumping from the vertical wells had no significant influence on the performance of HW2.

It was noted that following periods of heavy rainfall, surface runoff from the southern flank of Phase 2 collected around the exit point of HW2 (Figure 6.10) as construction of the drilling rig foundations had made it the lowest point in the area. It was thought that the water may have seeped into the waste along the exterior of the well casing, giving rise to the fluctuations in flow rate. The flow rate of HW2 was compared with the incident rainfall since well development, Figure 6.11.

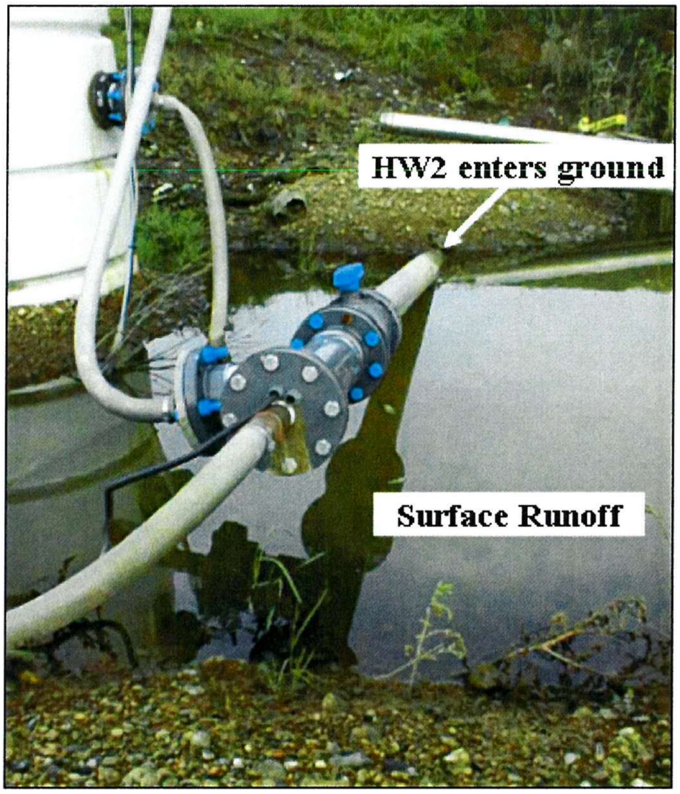

Figure 6.10 Surface runoff collects around the surfacing end of HW2.

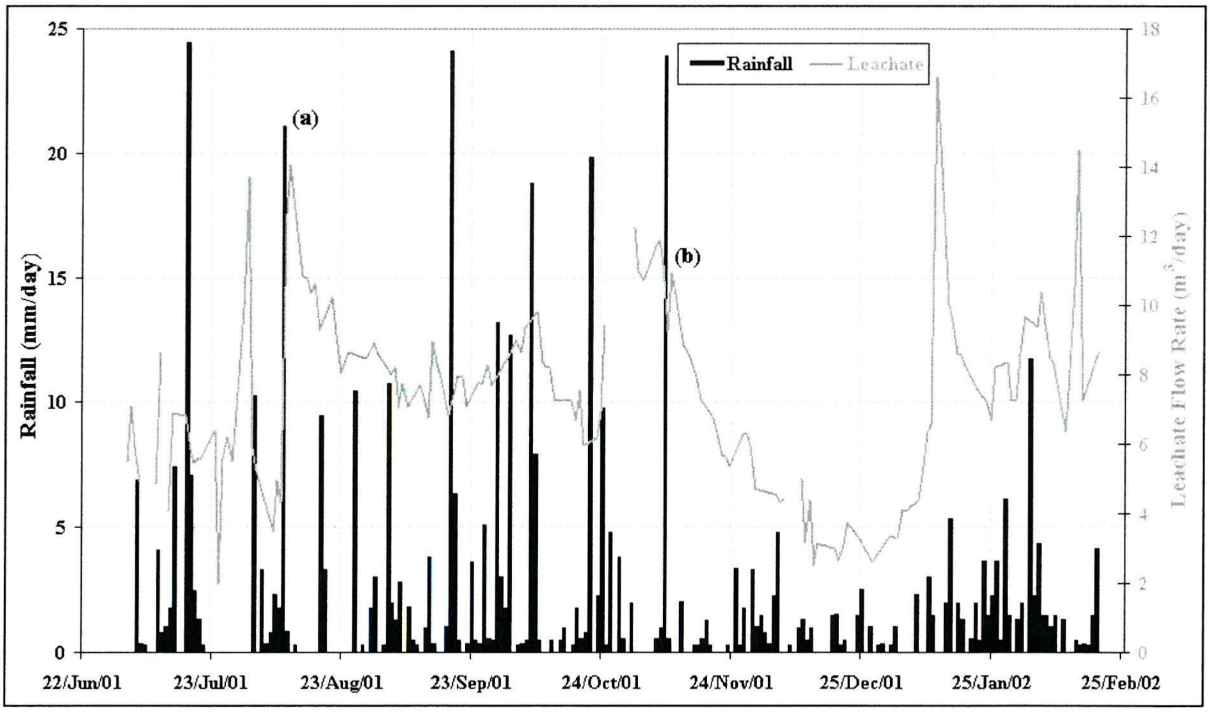

Figure 6.11 Rainfall and HW2 leachate flow rate since well development

Although the chart shows a number of corresponding peaks such as points (a) and (b), the chart reveals no general correlation between rainfall and leachate flow rate. The 36m length of plain casing between the surface and the well screen must have therefore formed an adequate seal with the surrounding waste and prevented the direct infiltration of rainwater along the casing exterior. Further investigation, possibly using a tracer test, is required as the rainfall may take several days to seep into the waste.

From Figure 6.7 it is apparent that many of the decreases in flow were gradual yet many of the increases were rapid. This suggests that a gradual build up of sediment may have been occurring within the well or around the pump intake which was then followed by a rapid clearance of this blockage. However, with the exception of cleaning the pump during well development, a mechanism to explain such sudden clearances is not obvious. Pending further investigation a suitable explanation to account for the large scale fluctuations in the flow rate of HW2 remains outstanding.

6.3.3 HW3

Figure 6.12 shows the leachate flow rate as a function of time for HW3. The cumulative volume of leachate extracted is also shown. The well screen slot width of HW3 is 1mm.

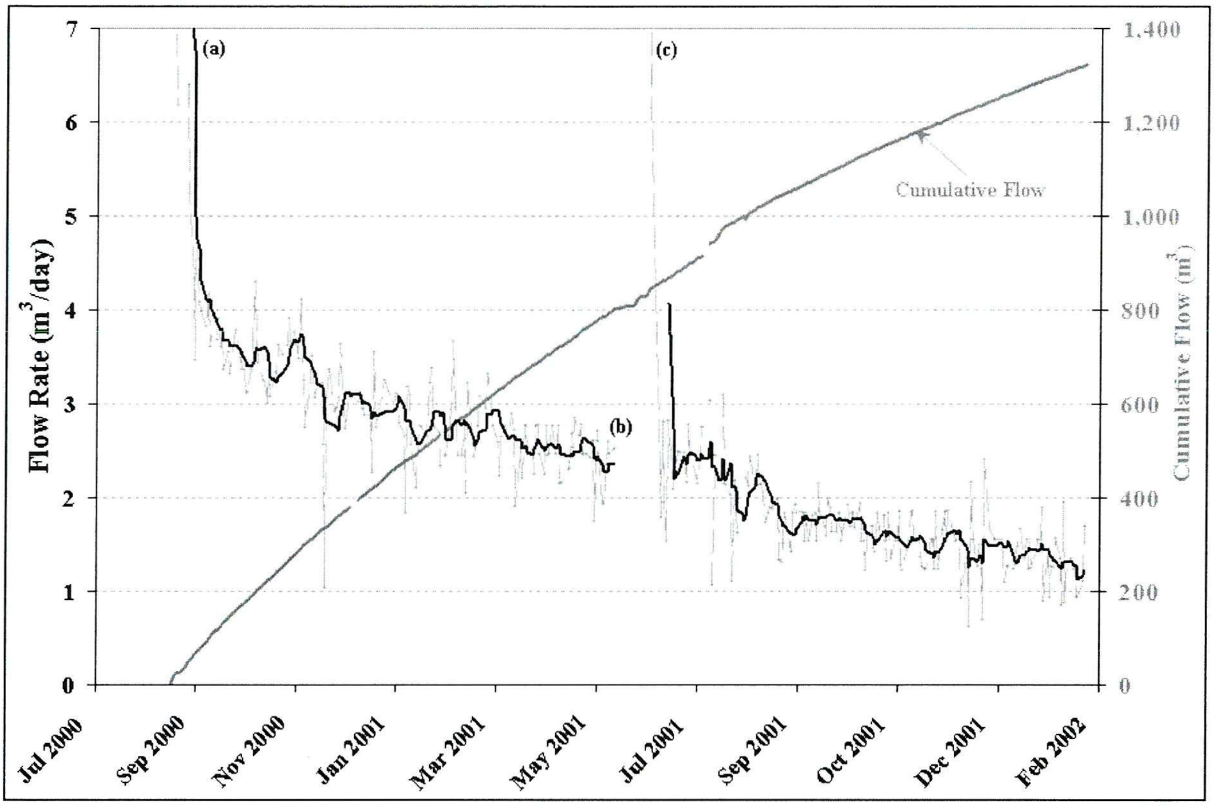

Figure 6.12 HW3 daily leachate flow rate and cumulative leachate volume. The seven day rolling mean of the daily flow rate (bold line) is also shown.

The letters (a) to (c) on Figure 6.12 mark the following events.

(a) Start of pumping 22/08/00, submersible pump at 3.3mOD.

(b) Pump stopped to conduct well development, 14/05/01.

(c) Pumping restarted, 25/05/01.

As with the other two wells, HW3 had been artesian during installation and was capped (at 10mOD) immediately after. When the cap was removed two months later there was no artesian flow and the leachate level was measured at 5.1mOD, just two metres above the horizontal section of the well. As with HW2, it was considered that a constant head step test would not yield any useful data and the submersible pump was installed in the horizontal section. A mean flow rate of 11.4m^3/day was achieved during the first 24 hours of pumping which reduced to 3.5m^3/day after one month. After seven months of pumping the flow had reduced to 2.4m^3/day. When pumping recommenced after well development, the leachate flow rate was again high (6m^3/day) for the first week as a result of the recharge that had occurred during pump shut down. After one week the flow rate had fallen back to its pre-development value and, as with HW1, it appeared that well development had made no long-term difference. By 19/02/02 the flow rate from HW3 did not appear to be approaching steady state.

Disregarding the data from the first month (and for the first week after well development) the flow rate, Q in m^3/day, follows a linear decline described by equation (6.2);

$$Q = -0.0045(t) + 3.4552 \tag{6.2}$$

where t is the time elapsed in days since 22/09/00 (i.e. one month after pumping started). This linear trend, plotted on Figure 6.13, has a R^2 value of 0.85.

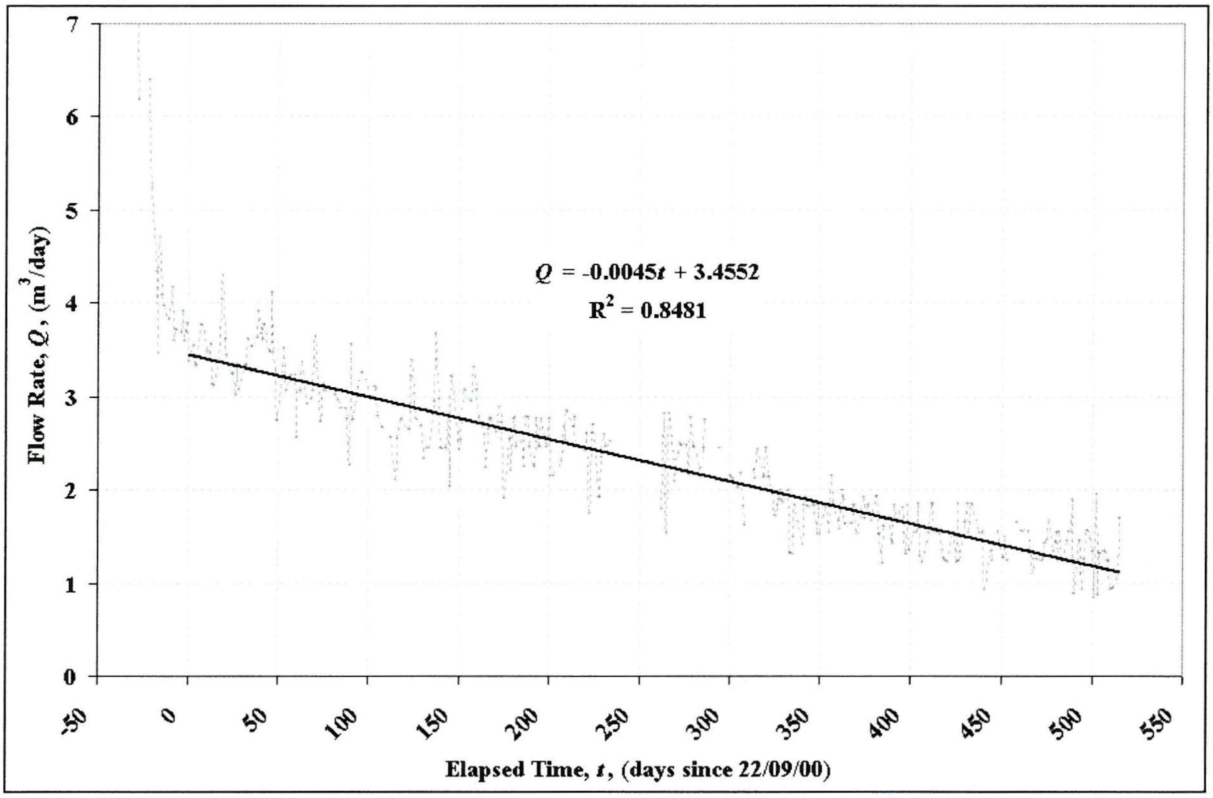

Figure 6.13 HW3 daily leachate flow rate against time, showing linear trend line.

Given the fluctuations in the data caused by (a) changes in atmospheric pressure, and (b) the discontinuous discharge of leachate from the holding tank, the R^2 value of 0.85 gives a high degree of confidence in the trend line.

Figure 6.13 suggests that the flow rate from HW3 was not approaching equilibrium by the end of the monitoring period. However, the hydraulic gradient around HW3 (discussed in section 6.8.4) had stabilised by this time. As with HW1, this suggests that the hydraulic conductivity may have been reducing, possibly as a result of well screen clogging.

6.3.4 Well Screen Slot Width

As there had been some uncertainty concerning the optimum value, each of the three wells was completed with a different slot width. The sizes were 1mm, 2mm and 5mm. It was thought that a comparison of flow rates and pore pressure changes in the surrounding waste might provide an indication of the optimum slot width. While Figure 6.5, Figure 6.7 and Figure 6.12 illustrate a wide variation in flow rates between the wells, there is no direct evidence to suggest this was attributable to the differences in slot width.

As the leachate heads in the horizontal wells were significantly different when the wells were commissioned, a comparison was made of the flow rate, Q, of each well per unit drawdown, h, (measured in the well) to investigate whether there was a relationship between slot width and leachate flow rate (Table 6.4). However, the analysis showed no relationship between slot width and the Q/h value.

Table 6.4 Flow rate per unit drawdown.

Well Number	HW1	HW2	HW3
Slot Width	2mm	5mm	1mm
Q - Av. Flow Rate (m^3/day)	7.00	4.84	2.42
h - leachate drawdown in well (m)	8.1	2.3	1.8
Flow rate per unit drawdown (Q/h)	0.86	2.10	1.34

It was apparent that there were a number of other factors affecting each well individually that prevented a direct comparison of the slot width. These factors are listed below.

1. At the start of pumping, leachate levels in the wells were 12.3, 6.2 and 5.1mOD for HW1, HW2 and HW3 respectively (equivalent to a pressure head of 8.5m, 2.6m and 1.5m above the horizontal section of each well).
2. The design of the well screen was such that as the slot width increased, the percentage open area also increased (1mm = 29%, 2mm = 44%, and 5mm = 67%). It would have

been more useful to compare different slot widths using well screen with an overall open area that remained constant.

3. Proximity of vertical wells and other leachate control systems to the horizontal wells.
4. Thickness of the landfill cap above each well. A temporary silt cap (150mm thick) had been laid on the surface above HW1. However, a more substantial and permanent cap lay over HW2 and HW3, consisting of a silt regulating layer (300mm), geomembrane, drainage blanket and a final silt layer (1000mm).
5. HW1 was possibly blocked at 210m, HW2 was damaged at 36m and possibly blocked at 90m. These blockages may have had a significant affect on flow rates.
6. The gas flow rate, which ranged from 360m3/day (HW1) to zero (HW2), may also have influenced the leachate flow rate.

The leachate flow rates recorded were therefore thought to be a function of a number of factors, including; (a) hydraulic conductivity of the surrounding waste, (b) rainfall recharge rates, (c) change in leachate head, (d) percentage open area, (e) slot width, and (f) gas flow rate.

6.3.5 Flow Velocity

The velocity of the flow, V, across the well screen was calculated using

$$V = Q/A \tag{6.3}$$

where Q is the discharge rate and A is the area through which flow occurs. The results for each well are presented in Table 6.5. The table also includes calculations for reduced open areas according to Brandon (1986) who states that the *effective* open area of a well screen in a conventional soil is generally one half to one third of the total as a result of blocking by the formation material.

Table 6.5 Flow velocity across well screen.

WELL N$^{O.}$	HW1	HW2	HW3
Slot Width	2mm	5mm	1mm
Q - Av. Flow Rate (m^3/day)	7.00	4.84	2.42

Open Area (%)	44	67	29
A - Open Area (m^2)	31.5	48.0	20.8
V – Velocity (m/s) with 100% effective open area	2.6×10^{-6}	1.2×10^{-6}	1.4×10^{-6}
V – Velocity (m/s) with 50% effective open area	5.1×10^{-6}	2.3×10^{-6}	2.7×10^{-6}
V – Velocity (m/s) with 33% effective open area	7.7×10^{-6}	3.5×10^{-6}	4.0×10^{-6}

All the calculated velocities, including the highest at 7.7×10^{-6} m/s, are very low and are unlikely to have resulted in the significant entrainment of particles.

However, it seems unlikely, given the heterogeneous composition of the waste, that flow velocities will be uniform along the length of the well. Flow of leachate (and gas) may well be dominated by flow through preferential pathways and not interstitial flow. This is supported by observations of gas venting from discrete points in the landfill surface as illustrated in Figure 6.14. In addition, where trenches have been cut into the waste, leachate can be seen flowing from discrete locations rather than seeping generally from the waste mass. If flow is predominately through preferential pathways, fluid velocities within them will be significantly higher. Where these velocities are periodically increased by the violent discharge of gas, the entrainment of sediment particles may become possible.

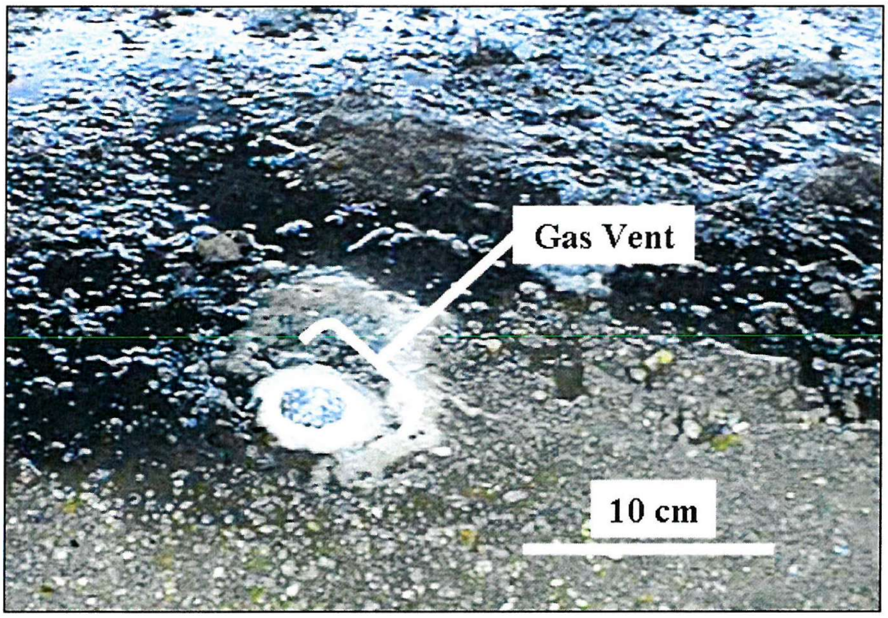

Figure 6.14 Photograph of landfill surface, gas can be see venting from discrete pathways in waste [Rainham 2001]

If flow is primarily through preferential pathways it becomes important to use a screen with an evenly distributed open area to maximise the chance of flow pathways coinciding with an open part of the well screen.

6.4 Gas Flow

Since HW1 was first opened to collect leachate under artesian pressure, landfill gas has also vented from the well. In October 2001, HW1 was connected to an active gas extraction system which drew gas from the well under suction and supplied it to an electricity generating station on site.

Gas also vented from HW3 but only when leachate was being actively pumped. No gas vented from HW2. The gas entered the holding tank along with the leachate before passing through a flowmeter that discharged the gas to atmosphere*. Table 6.6 details the gas volumes and average flow rates that the horizontal wells produced.

* After HW1 was connected to the active gas extraction system, gas no longer passed into the holding tank.

Table 6.6 Summary of gas flows to 19/02/02.

WELL REF. N^{O}.	HW1	HW2	HW3	TOTAL
Start of Pumping	04/07/00	16/10/00	22/08/00	
Days in Operation (to 19/02/02)	595	491	546	1,632
Total Gas Volume Extracted (m^3)	214,240	0	113,024	327,264
Average Gas Flow Rate (m^3/day)	360	0	207	567

Unlike the leachate flow rate, gas flow from both HW1 and HW3 remained reasonably constant throughout the duration of the pumping period. However, there was a significant variation on a short term (daily/weekly) basis that was thought to be related to changes in atmospheric pressure and is discussed in section 6.7. The absence of gas flow from HW2 was notable considering the large volumes discharged by the other two wells. The reasons for this are not clear but possible explanations are discussed later.

Samples of gas were taken from HW1 and HW3 periodically throughout the pumping period. The gas composition (in terms of percentage by volume) was consistently;

Methane:	60-62%
Carbon Dioxide:	35-38%
Nitrogen:	2-3%

A host of NMOCs (non-methane organic compounds), such as ethane, were also present in trace amounts. The gas composition is regarded as typical of landfills in the fourth phase of bacterial decomposition, Figure 6.15, where the composition and production rates of gas remain relatively constant, typically for around 20 years [Crawford & Smith, 1984].

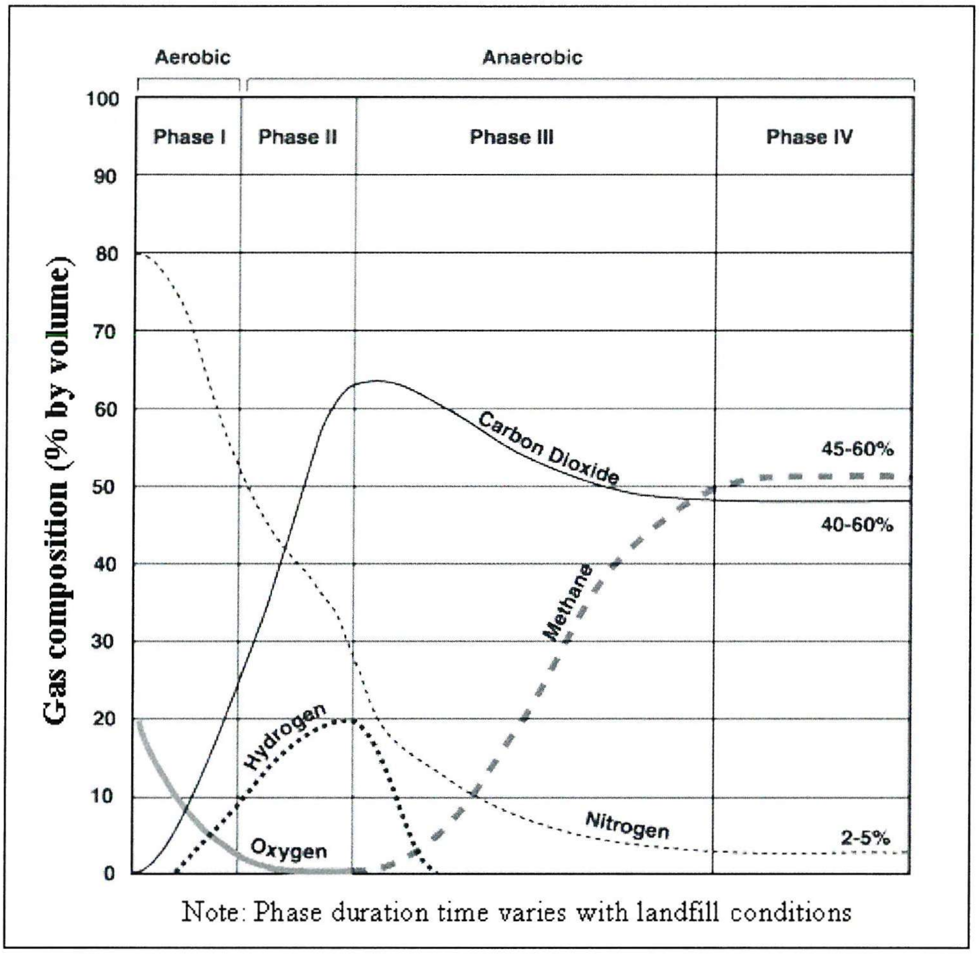

Figure 6.15 Production phases of typical landfill gas [Source, US EPA 1995].

The large volumes and high quality of the gas (i.e. it is not diluted with air) make the horizontal wells ideal for energy generation. The energy potential of the gas is calculated in Box 6.2 below.

Box 6.2 Energy potential of gas from HW1 & HW3

The average combined gas flow rate from HW1 and HW3 is $567m^3/day$.

The calorific value of $1m^3$ of landfill gas is around $18.84MJ/m^3$, based on gas with a 50% methane content [Dhussa & Varshney, 2000]. For the Rainham horizontal wells, methane concentration was 60% and the calorific value was therefore $22.6MJ/m^3$.

$$567m^3/day = 6.56\times10^{-3}\ m^3/s$$

$$22.6 \times (6.56\times10^{-3}) \times 1000 = \mathbf{148kJ/s} \text{ (or kW)}$$

The efficiency of the conversion process to electricity is around 35% [Hains, 2001] and so the gas can realistically be expected to generate approximately 52kW.

6.4.1 HW1

Figure 6.16 shows the gas flow rate as a function of time for HW1 (2mm slot width). The graph also shows the cumulative volume of gas extracted. The average flow rate between July 2000 and February 2002 was 360m^3/day.

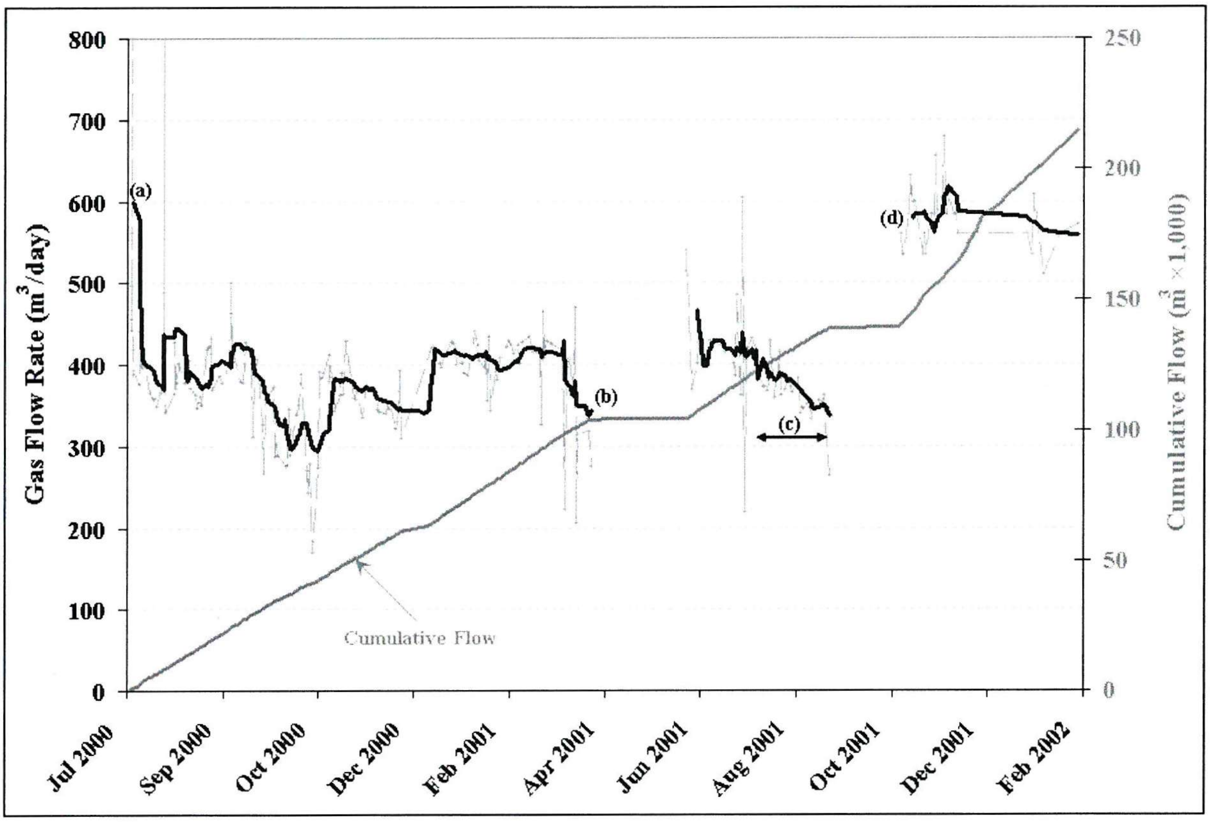

Figure 6.16 HW1 daily gas flow rate and cumulative gas volume. The seven day rolling mean of the daily flow rate (bold line) is also shown.

The letters (a) to (d) on Figure 6.16 mark the following events.

(a) Start of pumping 04/07/00.

(b) Pump failure 09/04/01, well shut off. Well development, May 2001.

(c) Failure of gas flowmeter 17/09/01.

(d) HW1 connected to active gas extraction system, 30/10/01.

When the well was first opened and pumping commenced (point (a)), the gas flow rate was greater than $600m^3$/day but had reduced to $400m^3$/day over two days. For the next nine months the flow rate fluctuated between 420 and $300m^3$/day. On 09/04/01 the submersible pump failed and the holding tank overfilled causing the gas flow rate to drop to point (b), which marks the point when the well was temporarily shut off. Well development was carried out in May 2001 and pumping recommenced on 19/06/01 with a replacement submersible pump. The gas flow continued at approximately $420m^3$/day until the gas meter failed (point (c)).

On 30/10/01 the well was connected to the gas extraction network to remove gas through suction rather than passive venting. The suction applied was approximately 17kPa. The extraction network also drew gas from vertical wells on Phase 2 and other phases of the site and supplied it to an on-site generating station with a capacity of 17MW. The increase in gas flow rate as a result of active suction can be clearly seen on Figure 6.16. The flow rate averaged $580m^3$/day after active suction began. It should be noted that the gas no longer passed through the gas meter and flow rate measurements were taken periodically using an orifice gauge*. However, this method provided only point readings of the flow rate. To calculate the cumulative volume it was assumed that the flow rate remained constant between readings. Given the significant daily fluctuations previously observed, estimating the volume in this way was not ideal, particularly when there were long periods of time between readings.

The active extraction of gas from vertical wells can often result in the dilution of landfill gas with air drawn from the surface which is undesirable for electricity generation and flaring. The intake of air is believed to be a result of the proximity of the vertical well screen to the landfill surface. However, following connection to the gas extraction system, the quality (i.e. methane concentration) of the gas from HW1 did not deteriorate and remained around 60%. It is thought that as the well screen lay 50m away from the exit point and underneath approximately 28m of waste, it was impossible for air to be drawn in despite the suction imposed. This geometry, which allows a long screen section to be positioned well away from the landfill surface, is considered a distinct advantage of horizontal wells over vertical wells for gas extraction purposes.

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* The pressure differential across the orifice is used in conjunction with the diameter of the orifice to calculate the current flow rate.

6.4.2 HW3

Figure 6.17 shows the gas flow rate as a function of time for HW3 (1mm slot width). The graph also shows the cumulative volume of gas extracted. The average flow rate between August 2000 and February 2002 was 207m^3/day.

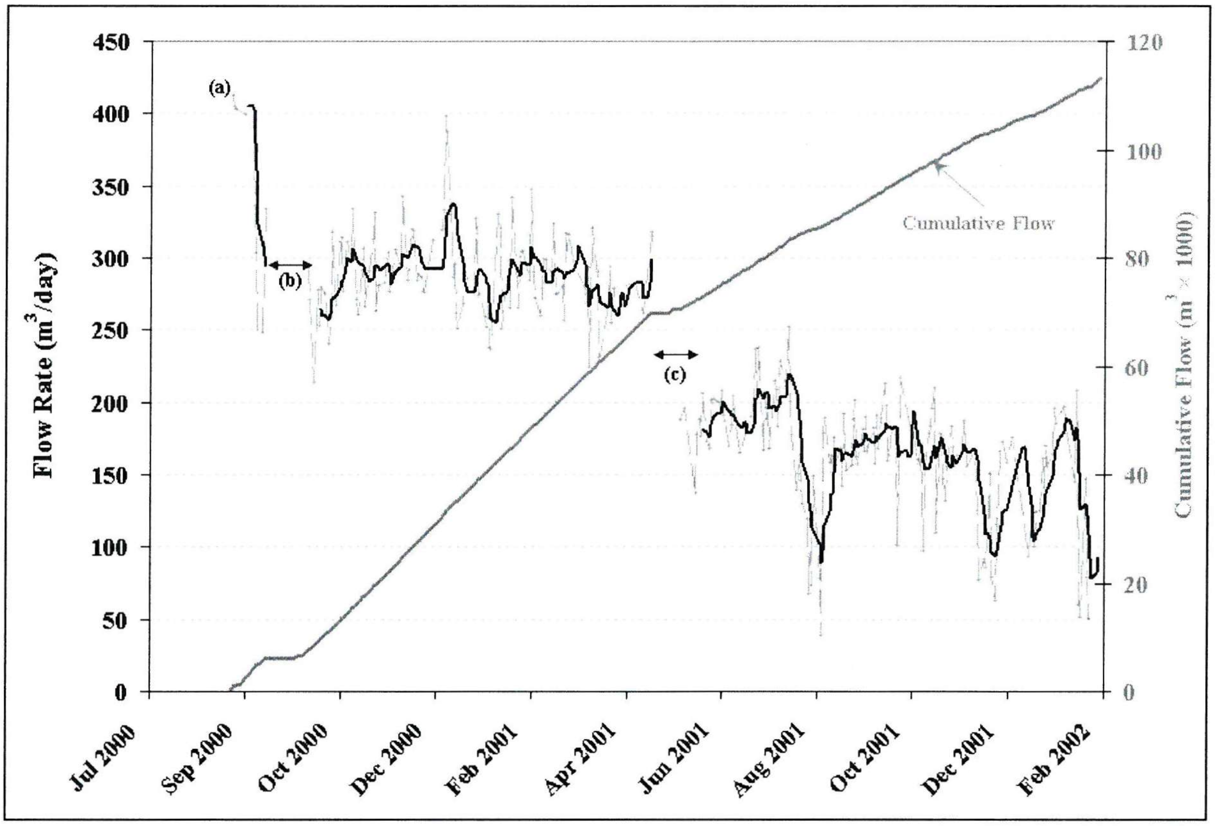

Figure 6.17 HW3 daily gas flow rate and cumulative gas volume. The seven day rolling mean of the daily flow rate (bold line) is also shown.

The letters (a) to (c) on Figure 6.17 mark the following events.

(a) Start of pumping 22/08/00.

(b) Gas meter stopped - full of condensate 14/09/00 to 02/10/00*,

(c) Well development, 14/05/01 to 01/06/01.

When pumping began the gas flow exceeded 400m^3/day but after two weeks had reduced to 300m^3/day before the flowmeter filled with condensate and stopped. Measurements restarted once the condensate had been emptied and for the next seven months the gas flow rate

* Following this event, condensate was removed on a weekly basis.

fluctuated between 250 and 350m^3/day. Despite the large fluctuations on a daily/weekly basis, the uniform gradient of the cumulative volume curve over this period indicates the long term consistency of flow, averaging 289m^3/day.

Well development took place in May 2001, during which time gas flow stopped. HW3 did not produce gas unless the submersible pump was in use. When pumping of leachate restarted on 01/06/01 the flow of gas recommenced but was 38% lower than before at 180m^3/day. From June 2001 to February 2002 the flow rate averaged 164m^3/day and included some significant longer term fluctuations (such as the fall and partial recovery of flow in August 2001). The reason why the gas flow rate was both lower and more variable after well development is not clear.

6.5 Relationship of Gas and Leachate Flow

HW2 did not produce any gas and HW3 produced gas only when leachate was being pumped. However, HW1 produced gas at all times regardless of whether a pump was engaged or not. These three scenarios are illustrated in Figure 6.18.

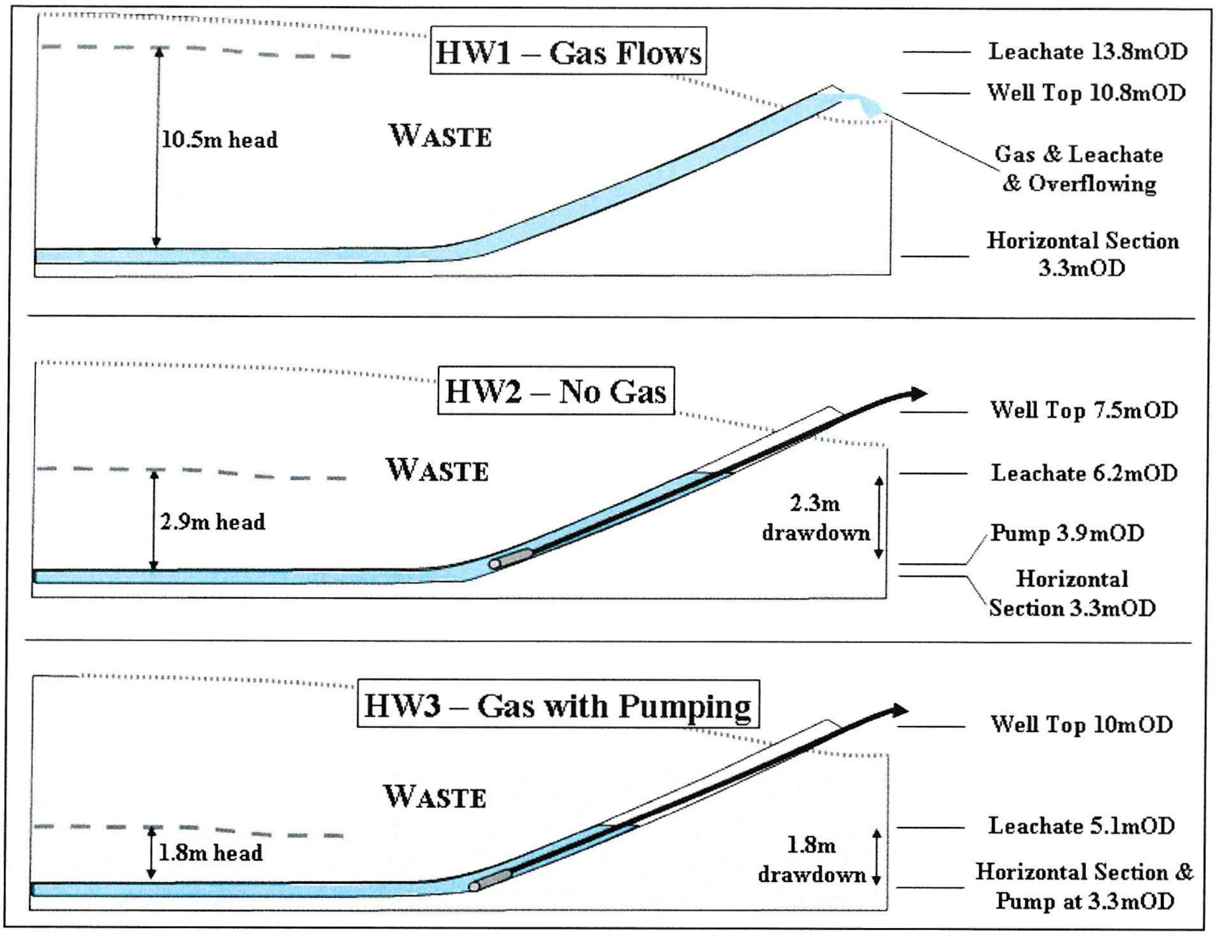

Figure 6.18 Behaviour of the three horizontal wells with regard to gas discharge.

On each occasion when the submersible pump in HW3 was switched off, the gas flow stopped a few hours later. From Figure 6.18 the fall in head at HW3 was 1.8m and it was thought that this reduction in pressure allowed gas to migrate into the well. However the fall in head at HW2 was greater, at 2.3m, yet this did not stimulate the flow of gas. It is a possibility that the gas volume and pressure in the waste around HW2 was less than at HW3, so that a further reduction in leachate head would have been required for gas to flow. Indeed, during the development of HW2, when the suction hose was placed at 36m, gas began to surge from the well. During this event, suction may have lowered the leachate head 0.6m further than the submersible*, which was perhaps enough to stimulate the flow of gas.

* Due to the restriction in the well screen, the pump in HW2 could not be installed in the lowest (horizontal) section of the well, Figure 6.18.

6.5.1 Gas - Leachate Ratios

As the wells were installed near the base of the site, in what was thought to be the saturated zone, it was not clear where the gas was coming from. One possibility was that the gas had been dissolved within the leachate and had come out of solution during the pressure reduction as the leachate was raised to the surface. To investigate this, the mass and volume ratios of gas to leachate were calculated as shown in Box 6.3.

Box 6.3 Gas to Leachate Ratios for HW1 & HW3

HW1 discharged 214,240m^3 of gas and 4,152m^3 of leachate between July 2000 and February 2002. The gas to leachate volumetric ratio was therefore 52:1 (or 98% gas).

From the gas analyses described above, the composition of the gas was approximately 62% CH_4 and 38% CO_2 by volume (the 2-3% nitrogen and other trace compounds have been disregarded).

The mass ratio was calculated using the following density values:
CH_4, Density 0.68kg/m^3 (at 15°C and 1 atm).
CO_2, Density 1.80kg/m^3 (at 15°C and 1 atm).
Leachate Density approximately equal to 1000kg/m^3
Gas discharged by mass:
214,240m^3 × 62% × 0.68kg/m^3 = 90 tons of CH_4
214,240m^3 × 38% × 1.80kg/m^3 = 147 tons of CO_2
The gas to leachate mass ratio was therefore 1:18 (or 95%).

Using the solubility values* of CH_4 (40mg/l) and CO_2 (1700mg/l), the maximum mass that could dissolve would be;
4,152m^3 × 40mg/l = 0.17 tons of CH_4
4,152m^3 × 1700mg/l = 7.06 tons of CO_2

Clearly the mass that could have come out of solution if the leachate was fully saturated with CH_4 and CO_2 was only a fraction of the mass that was discharged.

* In groundwater at STP.

HW3 A similar analysis was performed for HW3. This well discharged 113,024m^3 of gas and 1,322m^3 of leachate between August 2000 and February 2002, giving a gas to leachate volumetric ratio of 85:1 (or 99% gas). The composition of the gas was similar (62% CH_4 and 38% CO_2). The gas discharged by mass was therefore;

CH_4 = 48 tons, CO_2 = 77 tons

whereas the maximum dissolvable amount was:

CH_4 = 0.05 tons, CO_2 = 2.25 tons

Based on these analyses the possibility that the gas came out of solution from the leachate was rejected. A second possibility, that the source of the gas was from ongoing degradation in the older wastes local to each well, is explored in the next section.

6.5.2 Ongoing Gas Generation

The recharge of leachate is thought to have been dependent on the infiltration rate of rainwater and this may have been limited by the low vertical hydraulic conductivity. With limited recharge, the leachate flow rates of HW1 and HW3 gradually declined (Figure 6.5 and Figure 6.12). However, the gas flow rates of both wells were consistently high and there appeared to be no exhaustion of supply. It is thought that the waste continued to generate gas, and the rate of generation was the principal control on flow rate. Indeed Crawford & Smith (1984) state that during 'Phase IV' of landfill decomposition (Figure 6.15), gas is produced at a stable rate for typically 20 years but may continue longer if greater amounts of organics are present in the waste.

The following calculations were carried out to see what volume of waste would be required to generate the gas volume recorded and to estimate the zone of influence of the well. Caine and Bristow (2000) observed that MSW at a landfill in Brogborough, Bedfordshire, produced gas at a rate of 5 to 8m^3/ton/year and that test cells operated at the site produced gas at 9 to 14m^3/ton/year. As the waste in the lower horizons at Rainham dated back to the mid-1970's and was likely to have passed its peak rate of gas production, Caine and Bristow's lowest value (5m^3/ton/year) was used in the following calculations (Box 6.4 and 6.5).

Box 6.4 Volume of waste required to generate gas for HW1 & HW3

Average gas flow rate (HW1)	=	$360m^3$/day	= $131,400m^3$/year
Average gas flow rate (HW3)	=	$207m^3$/day	= $75,555m^3$/year
Gas generation rate (1970's waste)	=	$5m^3$/ton/year	
Density of waste*	=	1 ton/m^3	

131,400 / 5	=	26,280 tons of waste required to generate gas for HW1, at $10kN/m^3$ this occupies a volume, V, of **$26,280m^3$**
75,555 / 5	=	15,111 tons of waste required to generate gas for HW3, at $10kN/m^3$, this occupies a volume, V, of **$15,111m^3$**

It is demonstrated in section 6.8 that the pore pressure response to the operation of the horizontal wells was markedly stratified, most likely as a result of the anisotropic hydraulic conductivity that favours flow in the horizontal direction. If it assumed that the gas is being collected from a horizon of a certain thickness, T, along the full length of the well screen, L, then the width of influence, W, can then be calculated simply.

Box 6.5 Width of influence required to generate gas for HW1 & HW3

HW1	$V = 26,280\ m^3$	$L = 200m$	
If	$T = 1m$,	$W = 132m$, or 66m either side of the well	
If	$T = 2m$,	$W = 66m$, or 33m	" "
If	$T = 3m$,	$W = 44m$, or 22m	" "
HW3	$V = 15,111m^3$	$L = 200m$	
If	$T = 1m$,	$W = 76m$, or 38m either side of the well	
If	$T = 2m$,	$W = 38m$, or 19m	" "
If	$T = 3m$,	$W = 25m$, or 13m	" "

Even when $T = 1m$, the distance from which HW1 must draw gas is only 66m on either side of the well. Observations in section 6.8 show clear reductions in pore pressure at distances of 40m from HW1 and suggest that the influence of the well may extend more than 50m. These calculations show that the local generation of gas is sufficient to produce the gas volumes discharged from HW1 and HW3 and the width of influence need not be that great.

* Taken as an approximate average from the values presented in section 3.3.8.

6.5.3 Biochemical Influence

Ruskin (1982) showed that it is possible for the degradation of the waste, and therefore gas production, to cease as a result of pH inhibition. In experimental studies, Bogner and Spokas (1995), El-Fadel (1999) and Chan *et al* (2002) demonstrated that recirculation of leachate could create conditions more conducive to gas production. Reasons given were the removal of toxic compounds, the increased availability of nutrients and a reduction in acidity. The movement of leachate through the waste during pumping may have increased the production of gas as it did during these recirculation experiments. While the effect of horizontal wells in collecting gas is thought to be a primarily physical one, there may well be secondary biochemical advantages associated with increased leachate flow on gas production.

6.6 Gas Flow Mechanisms

The flow of gas from the well had an important effect on the nature of the leachate flow. Both HW1 and HW3 discharged leachate in periodic surges whereas HW2 (which did not discharge gas) exhibited a smooth, continuous flow of leachate.

It is thought that there were two distinct stages involved in the flow of gas to the exit point of the well, firstly the movement of gas from the waste to the well screen, and secondly the two-phase flow of gas and leachate along the well.

6.6.1 Flow to the Well

The movement of gas through porous media is driven by two forces, associated with pressure gradients (convection) and of concentration gradients (diffusion) [Spokas & Bogner 1995]. It is thought that the principal mechanism driving the movement of gas to the horizontal wells was the pressure gradient created by the pressure reduction around the well.

Gas bubbles are initially held within the waste pores by surface tension acting between the waste particles and the film of leachate surrounding the bubbles. As the pressure in a gas bubble increases through the addition of gas molecules from ongoing decomposition, the surface tension forces are overcome and the bubble begins to move through the pore spaces in the waste in the direction of the pressure gradient. The moving gas bubble then begins to coalesce with other bubbles in its path and so gathers momentum (comparable to rain drops running down a window pane). Eventually the gas bubble, which is by now more of a string of connected bubbles, reaches the well screen and discharges into the well. The trail of pores through which the bubble passed is now empty of gas, except for small bubbles which may have separated from the main bubble, and the gradual process of accumulation must begin again. On a microscopic scale, this gradual build up and rapid discharge of gas is periodic. However, over the 200m length of the well screen these small discharges of gas are averaged out and produce a continuous flow of gas into the well.

6.6.2 Two-phase Flow Along the Well

Extensive research has been undertaken within the petroleum industry into the flow patterns created when two or more fluid phases (e.g. oil and gas) are passing through a horizontal pipe. Multiphase flow is described by El-Sayed *et al* (2001) as "the concurrent flow of two or more phases (liquid, solid or gas) in which motion influences the interface between the phases". The flow regime is a qualitative description of the phase distribution in the pipe and the four principal factors controlling this are (a) gas and liquid flow rates, (b) phase densities, (c) pipe inclination angle, and (d) pipe diameter [Baker Jardine Co., 2002]. The three types of flow regime identified for horizontal pipes are described as segregated, intermittent and distributive [Lake *et al* 2002, El-Sayed et al 2001]. Segregated flow occurs when both liquid and gas flow rates are low so that the liquid flows along the base of the pipe and the gas along the top, with little interaction between the two. As the gas velocity increases in proportion to the liquid velocity, waves form at the phase boundary and if these waves become large enough to fill the pipe, intermittent or 'slug flow' develops. Slug flow is described by Lake *et al* (2002) as "liquid slugs alternating with high velocity bubbles of gas that almost fill the entire pipe" (Figure 6.19). Distributive flow describes the flow regime when gas bubbles are entrained in the flow of liquid or conversely, when liquid droplets are entrained in the flow of gas. The later regime, described as 'mist flow' occurs at high gas flow rates and low liquid flow rates.

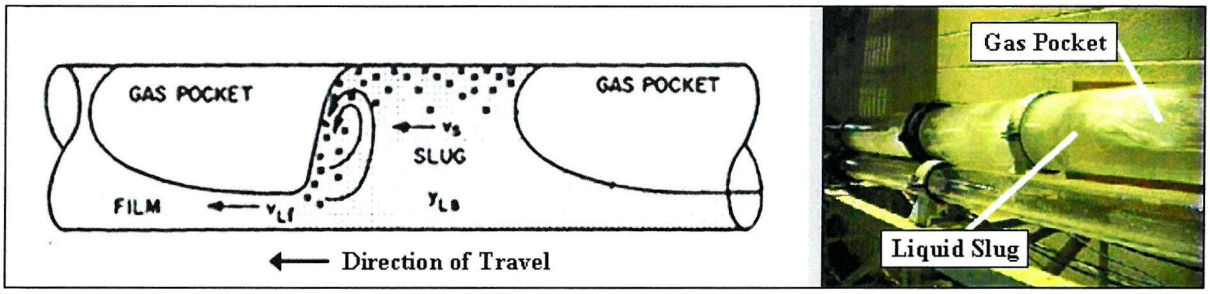

Figure 6.19 (a) Diagram of slug flow showing gas pockets with intermittent slugs of liquid, (b) photograph of laboratory investigations of two phase flow (slug flow), reproduced from Lake *et al*, 2002.

It was not possible to calculate the actual velocity of each fluid phase as it passed along the Rainham wells as the gas/leachate division of the cross-sectional area of the well screen was unknown. However, the average gas and leachate flow rates from each well, (Table 6.7), show that the gas flow rate was 50 to 85 times greater than the leachate flow rate, suggesting that the velocity of the gas was much greater than the velocity of the leachate. This may have caused waves to form at the phase boundary which became large enough to fill the pipe so that fluids were released intermittently, producing slug flow.

Table 6.7 Comparison of Gas & Leachate Flow Rates.

WELL REF. N^{O}.	HW1	HW2	HW3
Average Gas Flow Rate (m^3/day)	360	0	207
Average Leachate Flow Rate (m^3/day)	7.00	4.84	2.42
Gas to Leachate Flow Rate Ratio	51/1	-	86/1

There is another form of slug flow referred to as 'terrain-induced slugging' [De Henau & Raithby, 1995] that may have contributed to the intermittent flow of fluids from the horizontal wells. De Henau and Raithby observed a cyclic flow behaviour in oil and gas pipelines with several uphill and downhill sections. They argued that the oil accumulates in the low points of the pipeline forming liquid bridges that are blown from one pipeline section to the next due to gas pressure, thereby giving rise to large fluctuations in the fluid outflow rate. Although on a smaller scale, fluctuations in level did exist in the horizontal sections of each well and were of a magnitude greater than one well diameter (110mm). These fluctuations may have given rise to an accumulation of leachate in the low points which was cyclically removed due to gas pressure build up and release. It was considered that these fluctuations might be avoided in future installations by drilling the boreholes with a steady inclination. To ensure the well screen remains near the base of the waste (to effect the greatest influence on pore pressures), the maximum difference in level between each end of the screen would need to be no more than approximately 2m, giving an inclination of 1 in 100 (for a similar 200m long screen). However, it would be very difficult to accurately drill at such a shallow inclination through heterogeneous waste materials and it seems that small fluctuations are inevitable.

As mentioned above, the inclination angle of the pipe can also influence the flow regime. As the fluids passed up the 50m long inclined section of each well, the gas bubbles in the leachate would have risen through the leachate due to buoyancy. Following Stokes' law for the velocity of spherical particles in a fluid (equation 6.4), the larger gas bubbles would have risen more quickly than the smaller bubbles.

$$V_s = (2\ g\ r^2)(\rho_1 - \rho_2)\ /\ 9\mu \tag{6.4}$$

Where

V_s	= settling velocity (or rise velocity of bubble)
r	= radius of sphere (bubble)
ρ_1	= density of sphere (bubble)
ρ_2	= density of medium (leachate)
μ	= dynamic viscosity of medium (leachate)

As the larger bubbles rose they would have increased exponentially in size and velocity as a result of the pressure reduction and through coalescence with slower, smaller bubbles. In this way gas bubbles may have become very large, occupying the full bore of the well, and discharged as an almost continuous phase with only intermittent surges of leachate.

Although it is beyond the scope of this project to investigate further the exact mechanisms or categorisation of the flow regime it seems likely that the following factors were responsible for the surging of gas and leachate from the horizontal wells;

(a) differences in velocity between the gas and leachate phases,
(b) fluctuations in the level of the well allowing the accumulation of leachate, and
(c) bubble coalescence and expansion in the inclined section of each well.

Unsteady flow conditions are undesirable in the oil industry as they place excess stresses on pipe lines and create an intermittent supply [De Henau & Raithby, 1995]. However, the gas and leachate from the horizontal wells at Rainham passed into a holding tank where the unsteady flow was smoothed out before fluid volumes were measured and so intermittent flow was not a problem in this respect.

6.7 Influence of Atmospheric Pressure

Gas migrates from landfill sites through convection at a rate governed by the difference in gas pressure between the interior of the landfill and the external atmospheric pressure. This effect is well known and is documented in a number of papers [Cripps, 1996; Christopherson and Kjeldsen, 2001]. The relationship between atmospheric pressure and the rate of gas migration is inverse such that a fall in atmospheric pressure induces a greater rate of gas migration and vice versa [Spokas and Bogner, 1995; CIWMB 1997].

It has already been identified that although the gas flow rates from HW1 and HW3 were relatively stable in the long term, there were significant fluctuations on a daily basis. It was thought that these fluctuations may have been related to changes in atmospheric pressure. To test this, barometric readings* and gas flow rates from the two gassing wells were plotted against time. Figure 6.20 shows the results from HW3 during the period September 2000 to April 2001 when gas flow data were continuous and few interruptions to pumping occurred.

* Atmospheric pressure data were obtained from a datalogged barometer installed on the landfill surface over HW3. To verify the data they were compared with readings obtained from the site weather station and were found to be in close agreement.

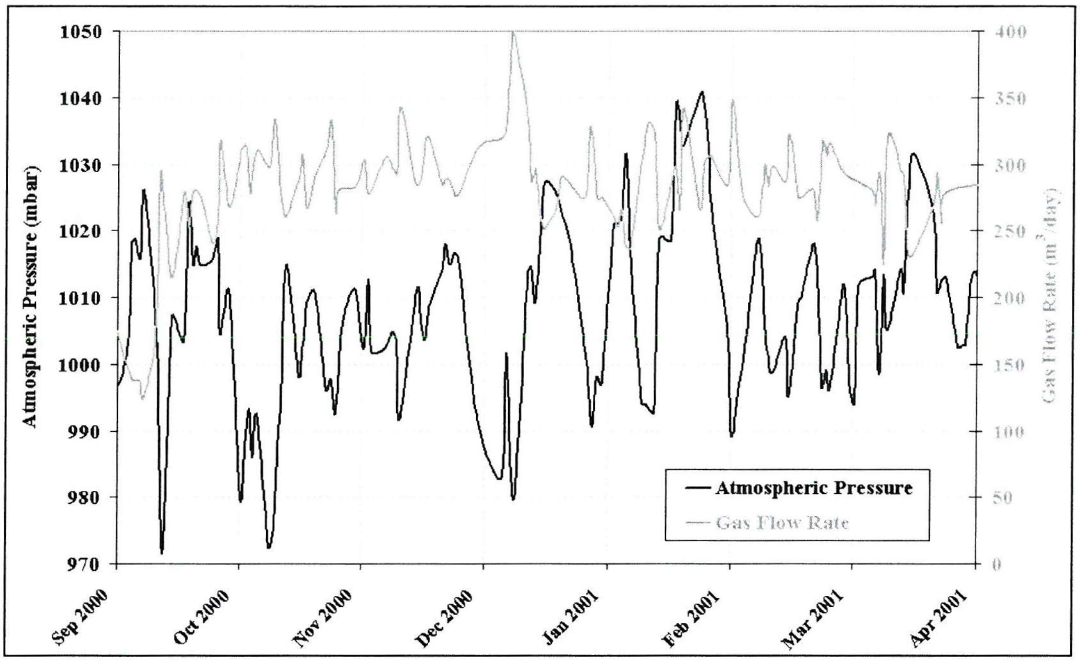

Figure 6.20 HW3 atmospheric pressure and gas flow over time. An inverse relationship is apparent.

It is clear from Figure 6.20 that gas flow rates were inversely related to fluctuations in atmospheric pressure. In order to quantify the relationship the gas flow rates from the same period were plotted as a function of the changes in pressure, Figure 6.21. The points on Figure 6.21 represent the change in atmospheric pressure between the larger peaks and troughs. Only the data relating to the larger variations in pressure were used so that small fluctuations that related to other factors would not mask any correlation. These other factors may have included both variations in the rate at which gas was generated and incorrect estimates of the time at which the gas flow readings were taken.

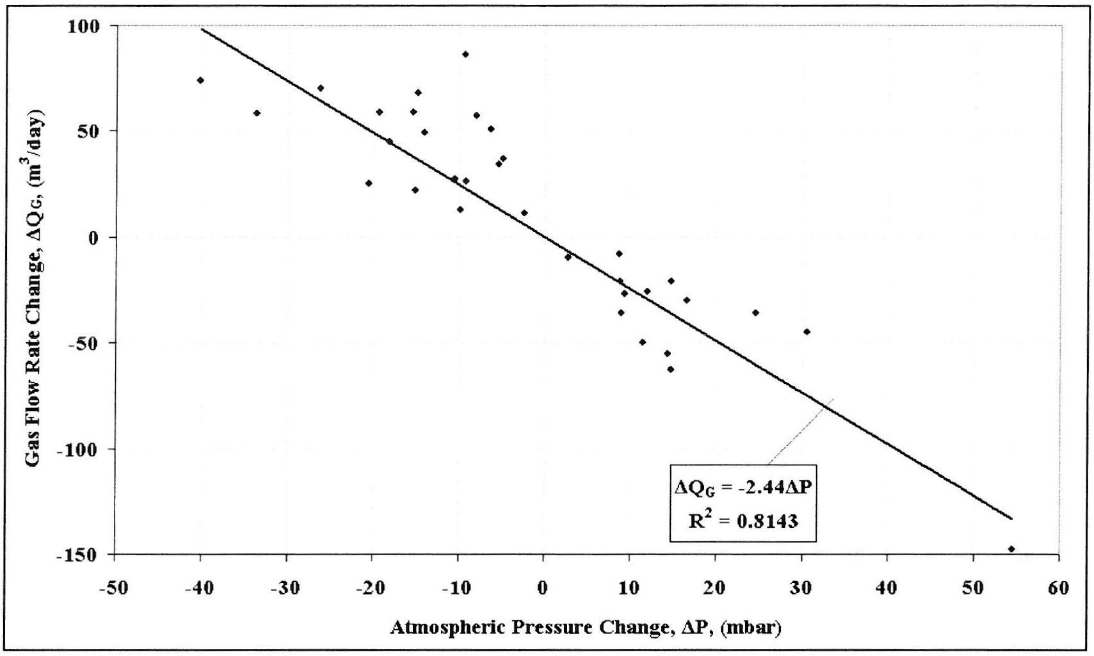

Figure 6.21 HW3 gas flow rate change as a function of atmospheric pressure fluctuations during the period September 2000 to April 2001.

The relationship between atmospheric pressure and gas flow rate from HW3 is approximately linear and is described by

$$\Delta Q_G = -2.44\Delta P \tag{6.5}$$

Where ΔP is the change in atmospheric pressure (in mbars) and ΔQ_G is the corresponding change in gas flow rate (in m^3/day).

Gas flows from HW1 were also compared with atmospheric pressure changes over time. However, the initial data were dominated by the changing flow regime during the step tests and there then followed a number of occasions when the gas meter blocked due to condensation. Furthermore, when the well was connected to the gas suction system in October 2001, flow rate readings were only point values and the cumulative value had to be estimated. This lack of continuity in the data was not significant for long term analysis of flow rates but the data were not robust enough for studying relationships with short term fluctuations in atmospheric pressure.

With atmospheric pressure clearly influencing the gas flow rate of HW3 (and possibly HW1), further analysis went on to compare atmospheric pressure with leachate flow rates for all three wells. Figure 6.22 to Figure 6.27 show (a) leachate flow rate and atmospheric pressure over time, and (b) the change in leachate flow rate as a function of the change in atmospheric pressure. For each well, data from the time period with the least apparent interference to the pumping regime were used. It should be noted that a number of the small fluctuations in flow rate resulted from the temporary storage of leachate in the holding tank.

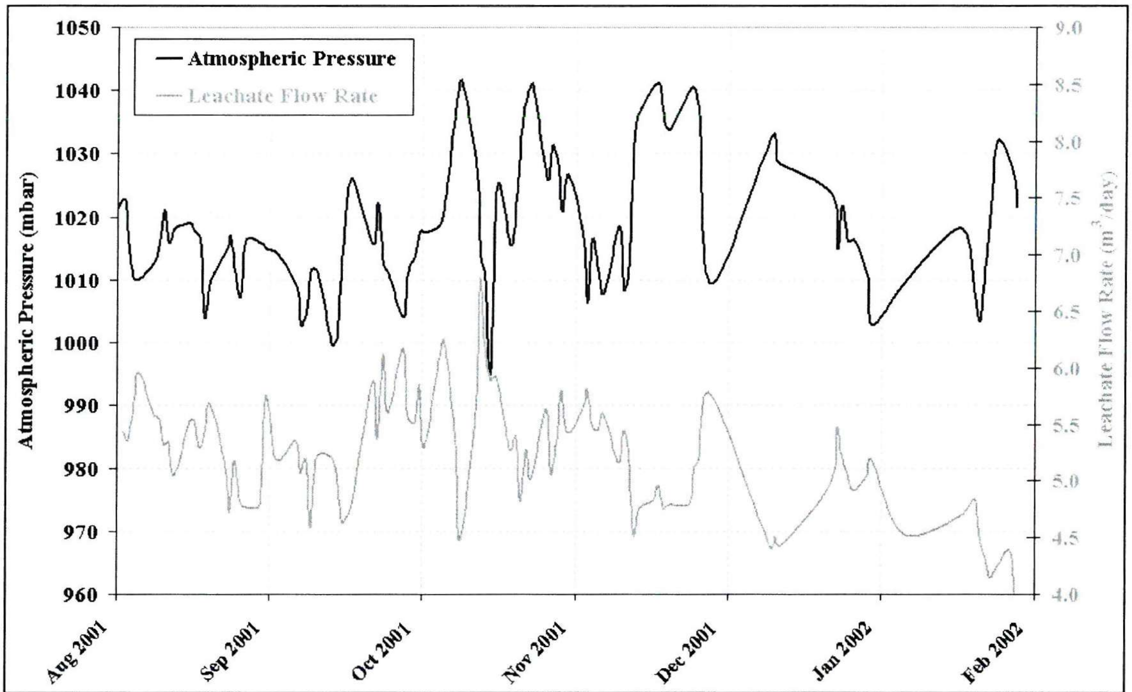

Figure 6.22 HW1 atmospheric pressure and leachate flow over time. An inverse relationship is apparent.

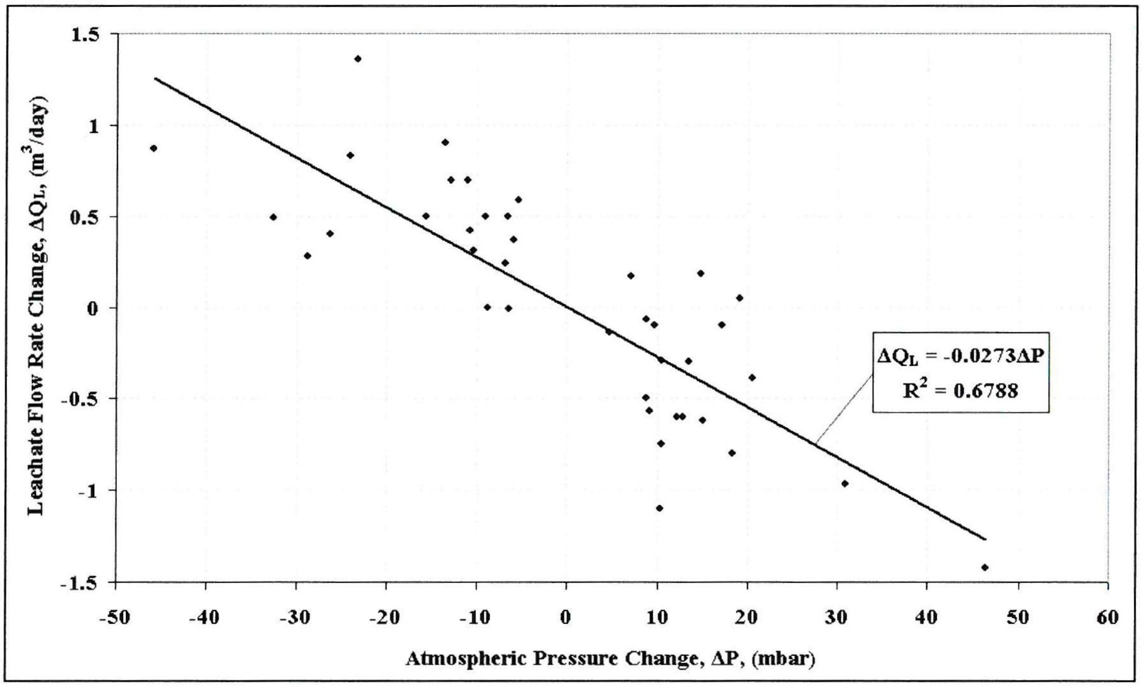

Figure 6.23 HW1 leachate flow rate change as a function of atmospheric pressure fluctuations during the period August 2001 to February 2002.

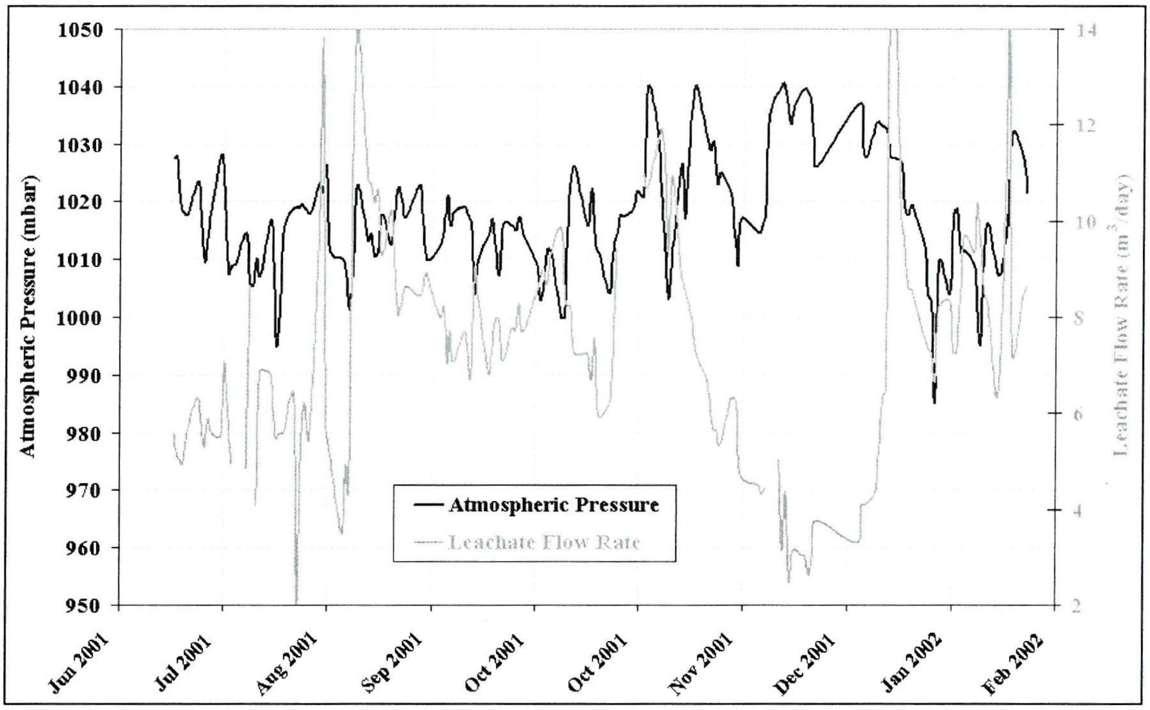

Figure 6.24 HW2 atmospheric pressure and leachate flow over time. No relationship is apparent.

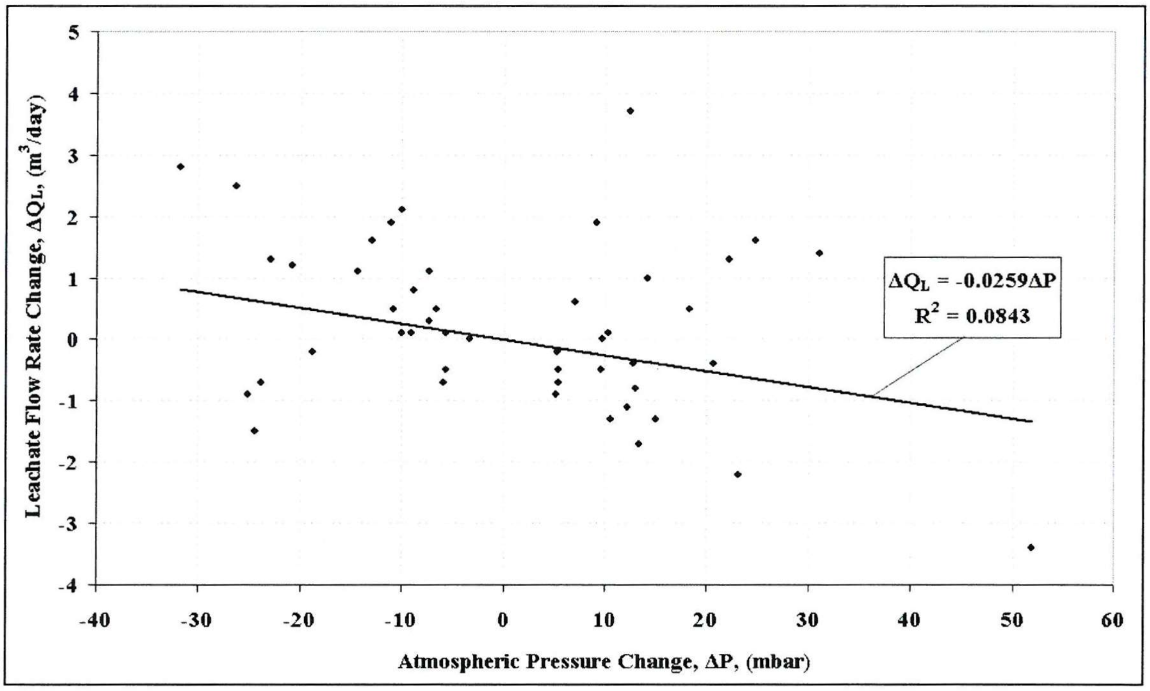

Figure 6.25 HW2 leachate flow rate change as a function of atmospheric pressure fluctuations during the period June 2001 to February 2002. The data are scattered suggesting there is no relationship.

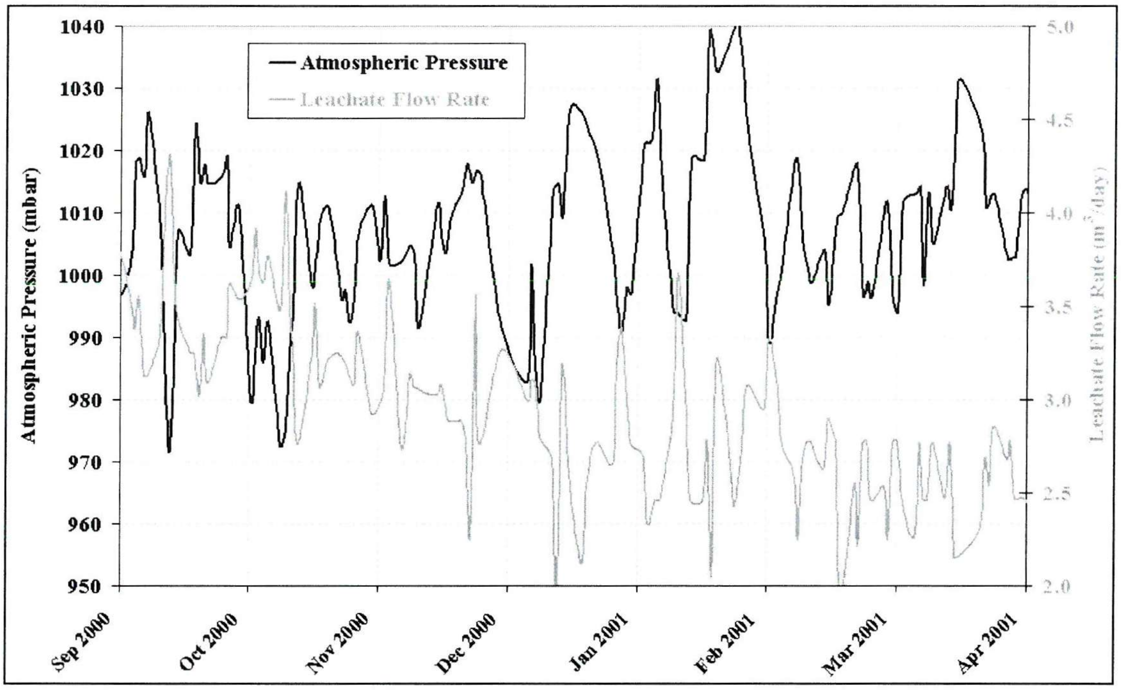

Figure 6.26 HW3 atmospheric pressure and leachate flow over time. An inverse relationship is apparent.

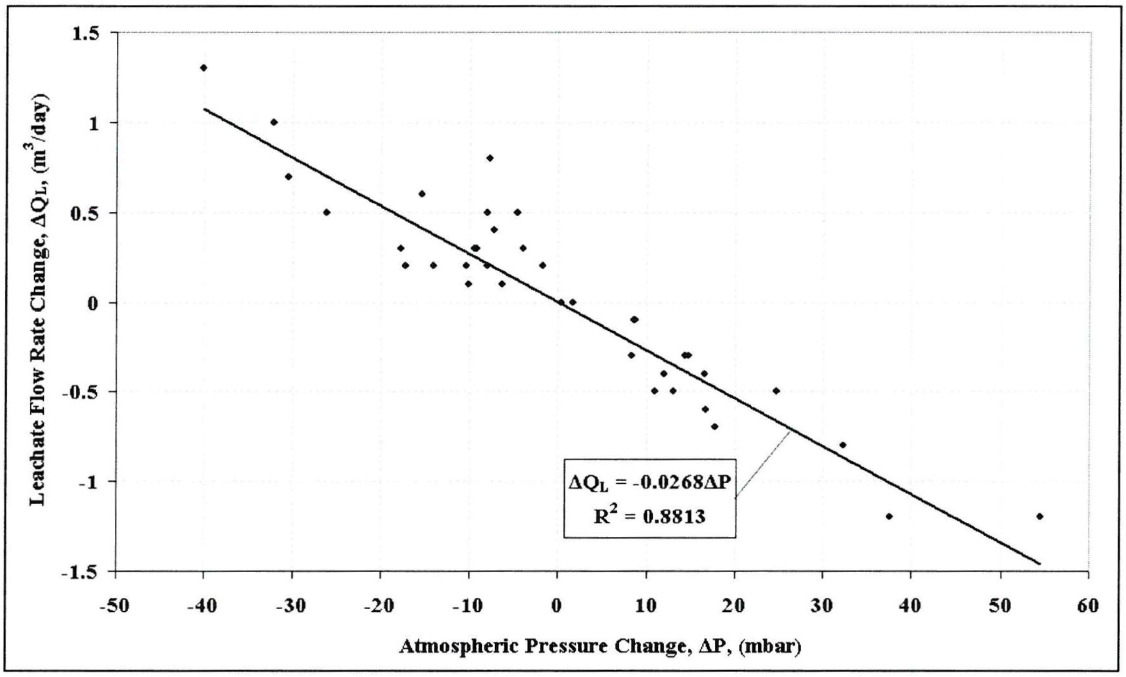

Figure 6.27 HW3 leachate flow rate change as a function of atmospheric pressure fluctuations during the period September 2000 to April 2001.

Leachate flow rates from both HW1 and HW3 showed an inverse linear relationship with atmospheric pressure described by the following equations;

HW1 $$\Delta Q_L = -0.0273\Delta P \tag{6.6}$$

HW3 $$\Delta Q_L = -0.0268\Delta P \tag{6.7}$$

where ΔQ_L is the change in leachate flow rate (in m^3/day). The regression lines have R^2 values of 0.68 and 0.88 for HW1 and HW3 respectively. Conversely, leachate from HW2 did not show any relationship with changes in atmospheric pressure and this was confirmed by both the wide scatter on Figure 6.25, and the low R^2 value of the regression line (0.08). As no gas vented from HW2 the lack of any relationship between atmospheric pressure and leachate flow suggests that the correlations seen in HW1 and HW3 may have been an indirect result of the relationship with the gas. Why leachate flow should increase with gas flow is not clear but may simply be due to the gas forcing leachate through the waste pores at an accelerated rate in its movement toward the well.

6.8 Pore Pressure Changes

Having considered the gas and leachate yields from the horizontal wells, the effects of fluid extraction on pore pressures are now examined. As detailed in Chapter 5, fifty standpipe piezometers where used to monitor changes in pore pressure.

Figure 6.28 to Figure 6.34, show the leachate head response in the waste both over time and in cross section. The layout and monitoring depths of the piezometers have been described in Chapter 5 and a summary of the details for each piezometer is given in Appendix D.

Two lines (cross sections) of piezometers monitored HW1 and a single line monitored HW3. No cross section of piezometers monitored HW2. The data from each cross section are reproduced on separate Figures*.

* The data from HW1O is reproduced as a separate Figure so that the stages of the pumping regime can be more clearly seen. HW1O was installed in the lower waste horizon at a distance of 7m from the well and the pore pressure response is considered typical of the piezometers in the lower horizon.

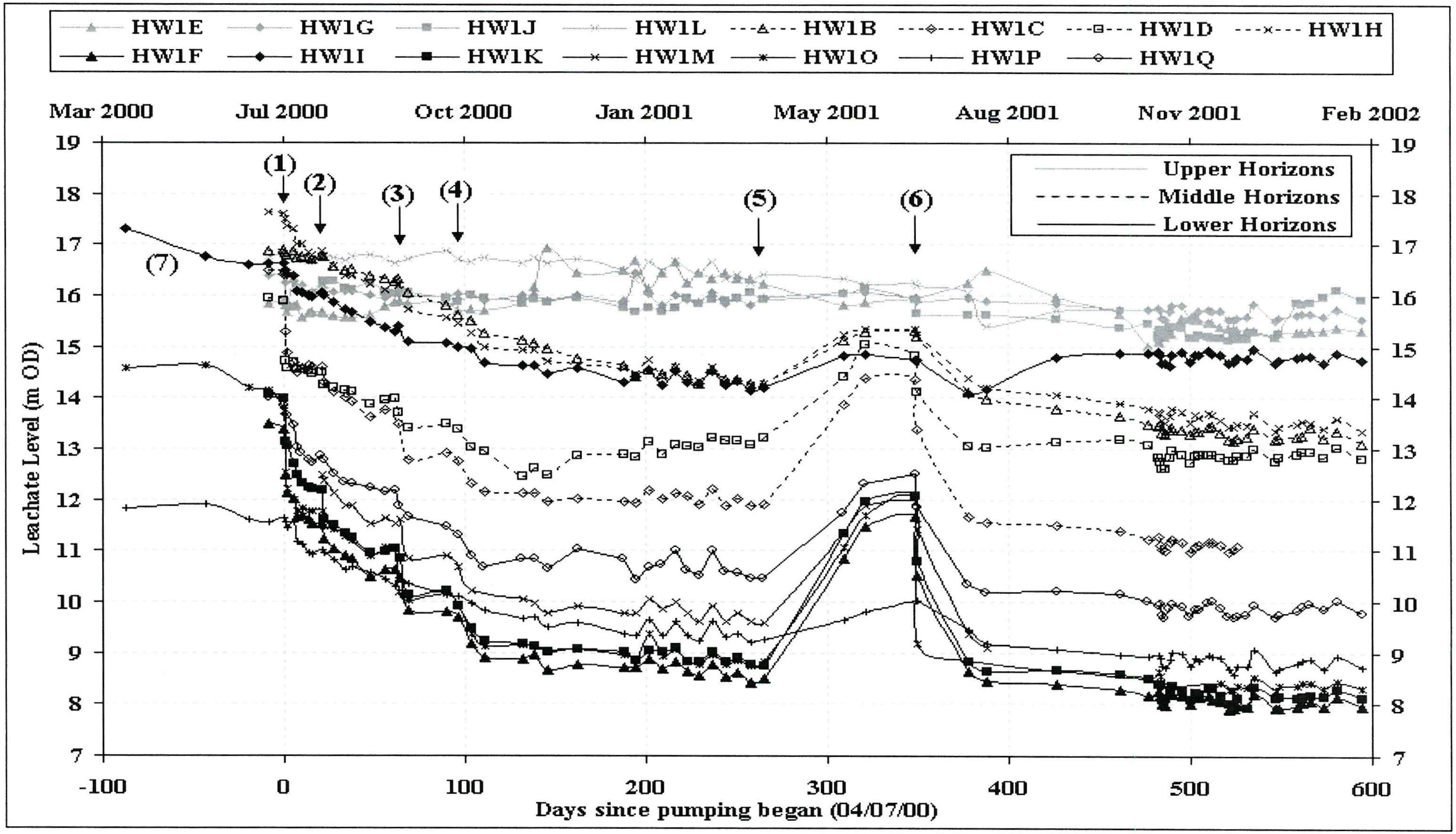

Figure 6.28 Changes in leachate level in response to pumping from HW1 (Line 1 - landfill surface is at 31mOD).

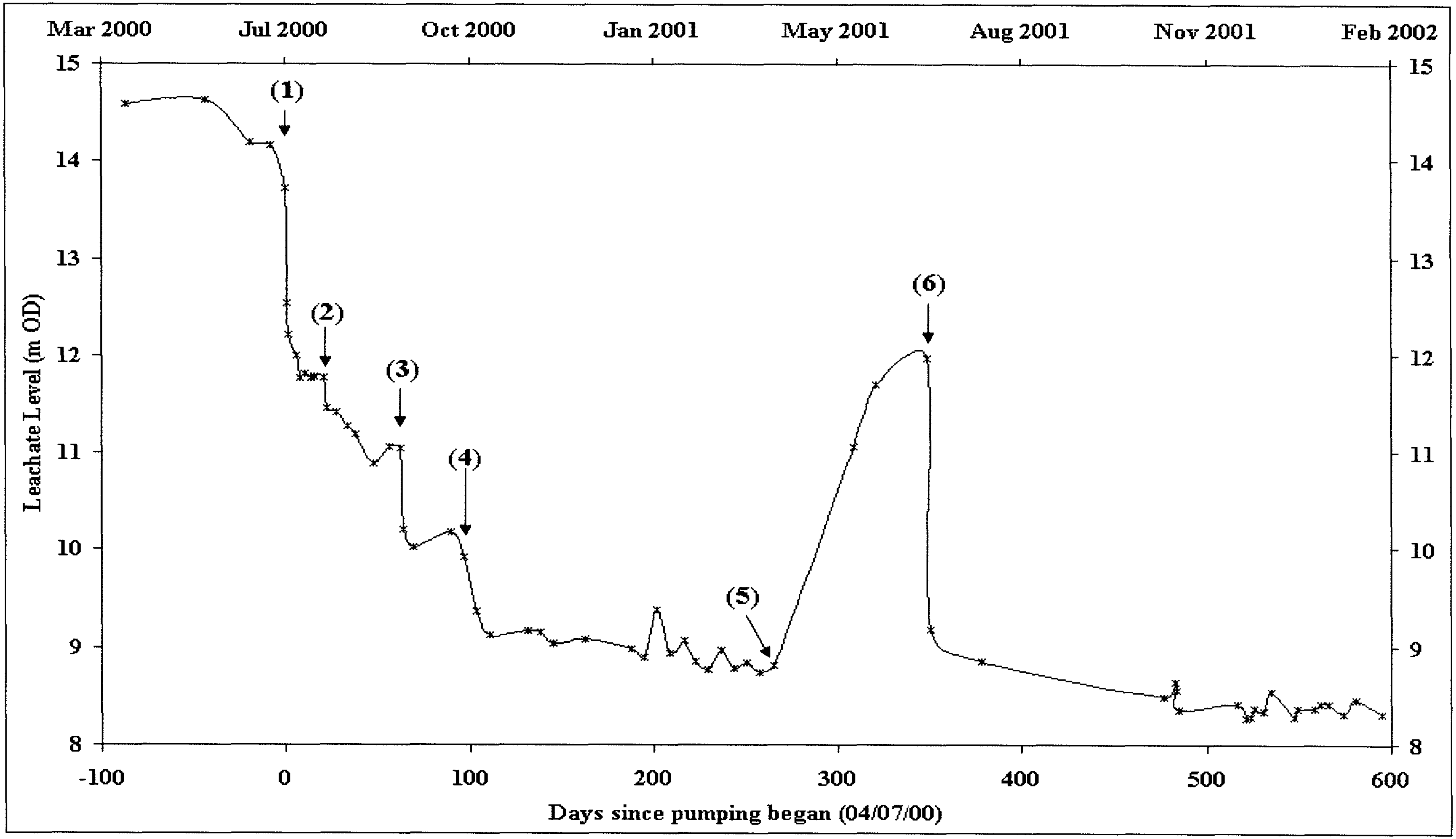

Figure 6.29 Changes in leachate level in piezometer HW1O (Line 1, Lower Horizon, 7m from HW1).

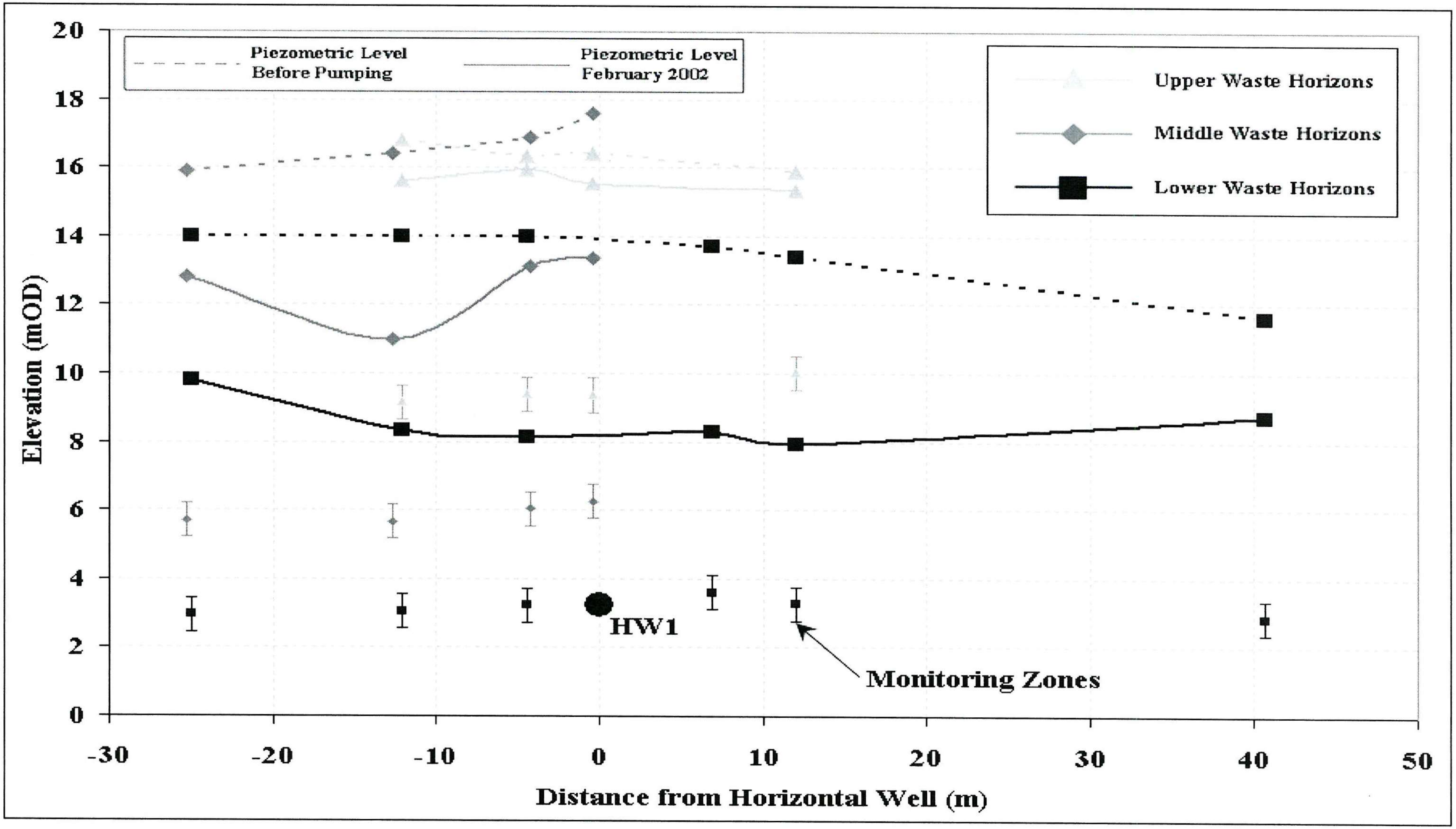

Figure 6.30 Cross section of piezometric levels over HW1 (Line 1). Surface of landfill is 31mOD. The dashed lines represent the pre-pumping level and the solid lines the reduced level after nearly 600 days of pumping.

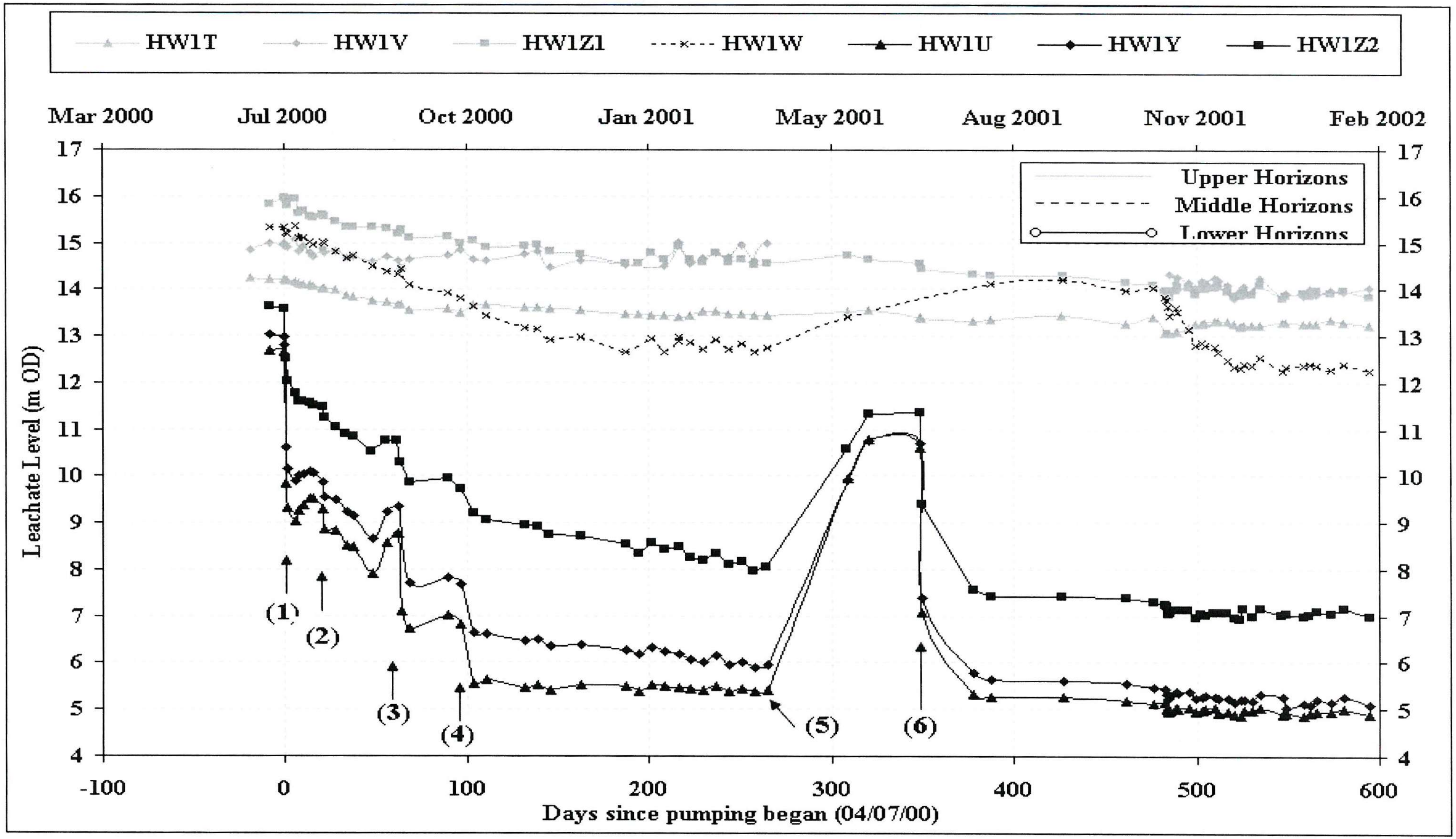

Figure 6.31 Changes in leachate level in response to pumping from HW1 (Line 2 - landfill surface is at 26mOD).

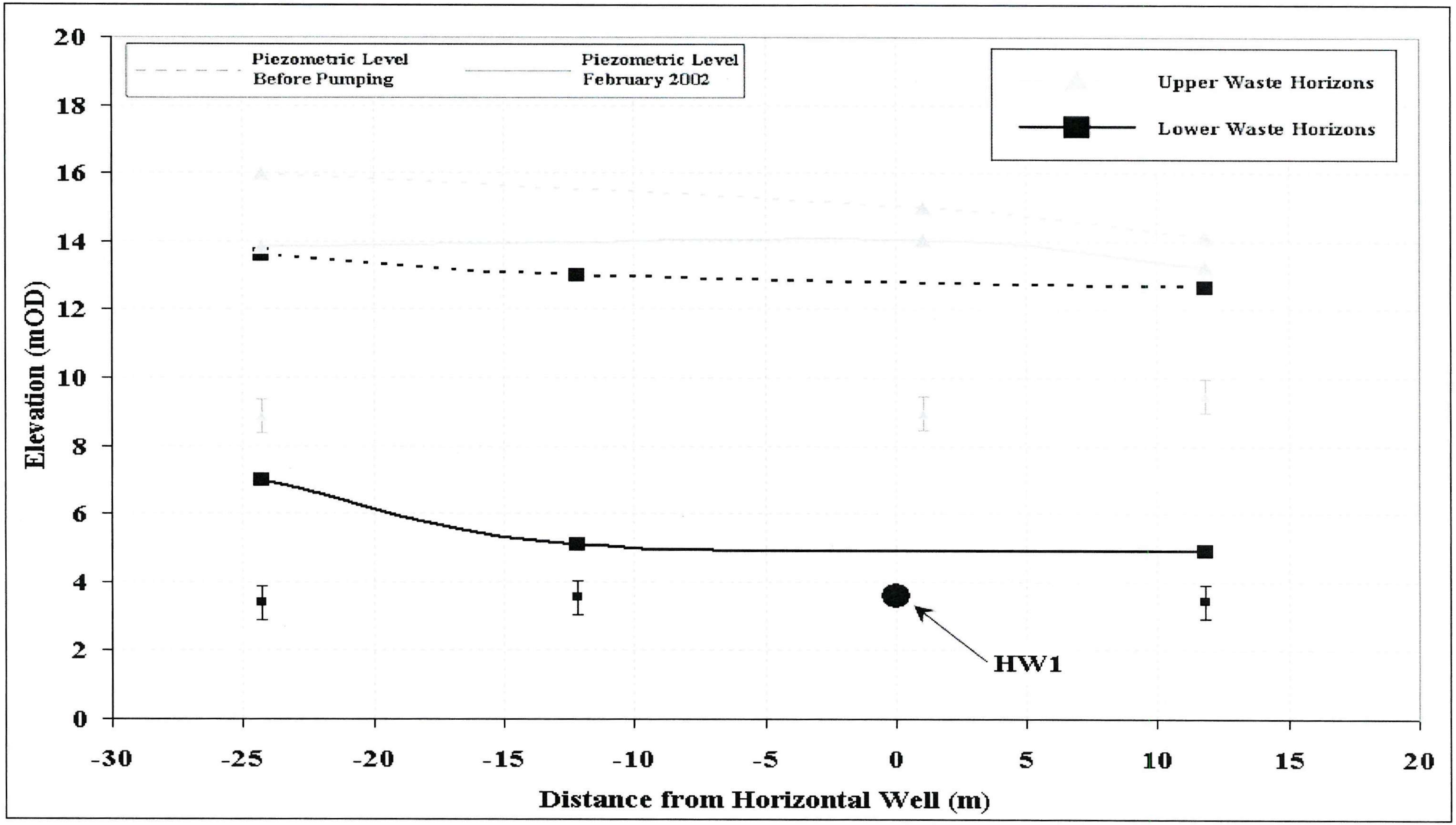

Figure 6.32 Cross section of piezometric levels over HW1 (Line 2). Surface of landfill is at 26mOD. The dashed lines represent the pre-pumping level and the solid lines the reduced level after nearly 600 days of pumping.

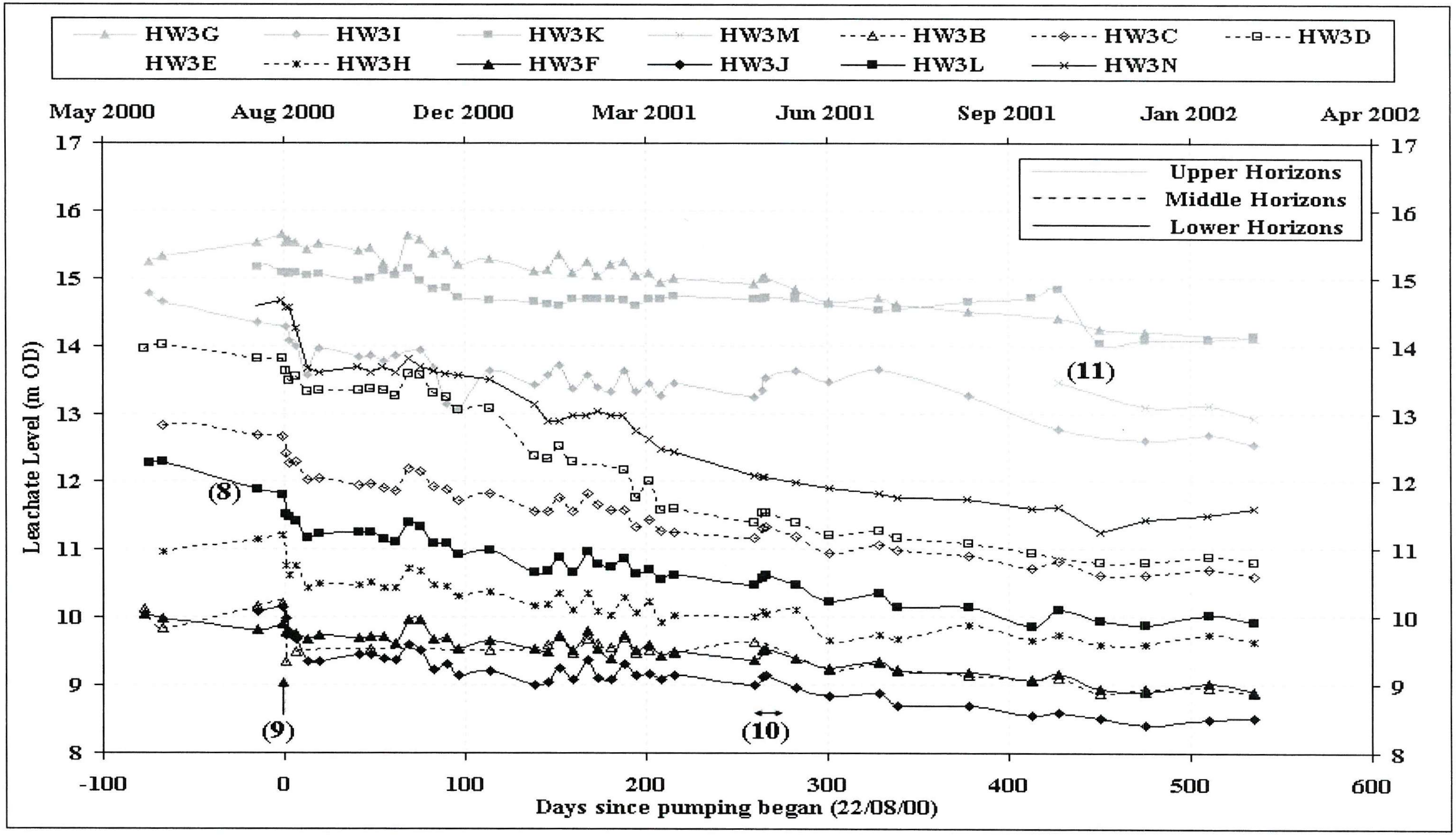

Figure 6.33 Changes in leachate level in response to pumping from HW3 (landfill surface is at 24 to 26mOD).

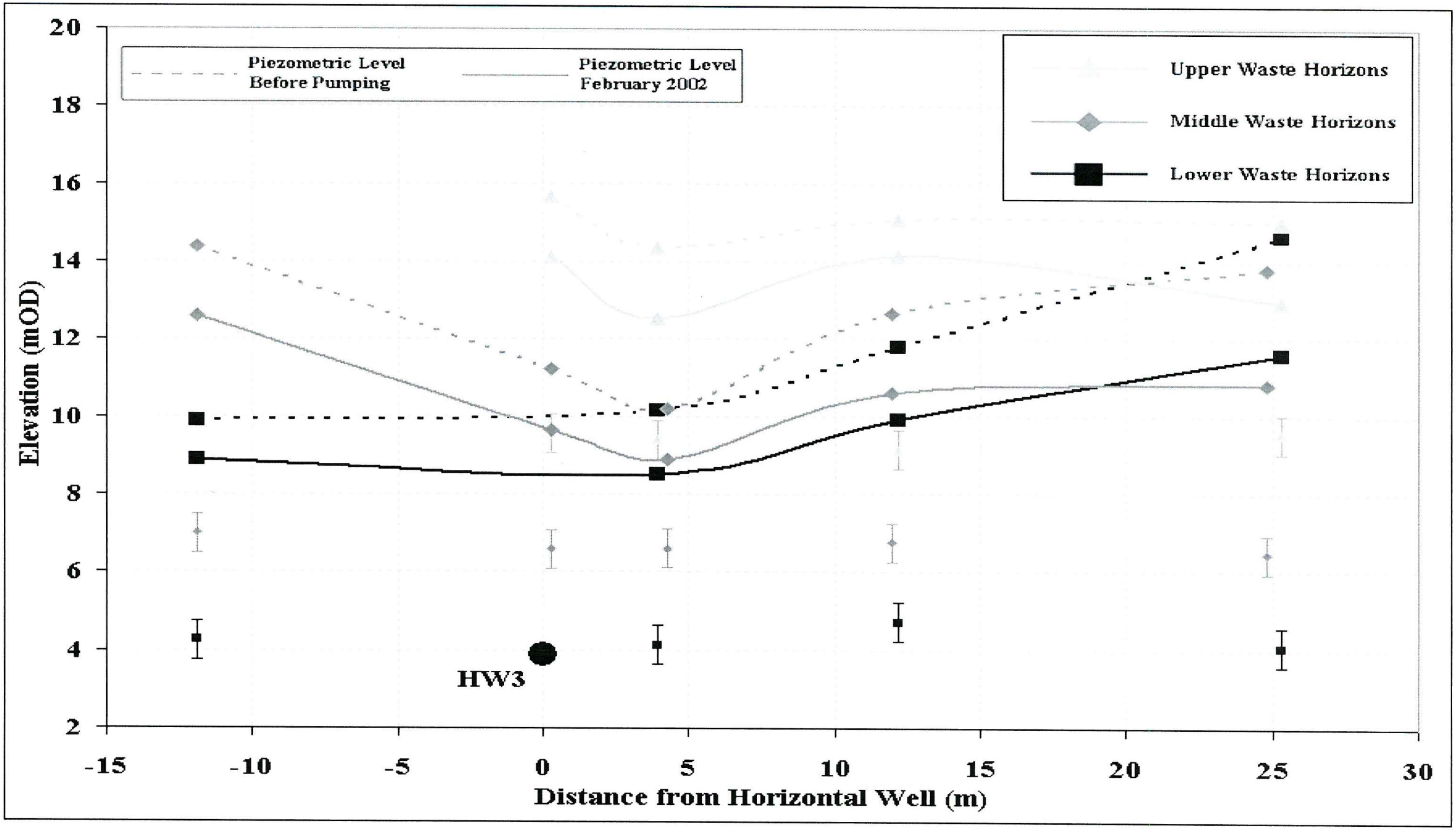

Figure 6.34 Cross section of piezometric levels over HW3. Surface of landfill is at 24 to 26mOD. The dashed lines represent the pre-pumping level and the solid lines the reduced level after 535 days of pumping.

The numbers (1) to (12) on the above Figures mark the following events:

(1) Step 1, start of leachate and gas extraction under artesian conditions on 4th July 2000. (outflow of well at 12.3mOD).

(2) Step 2, outflow lowered to 10.8mOD on 25th July 2000.

(3) Step 3, submersible pump installed at 5.5mOD on 5th September 2000.

(4) Step 4, submersible pump lowered to 4.2mOD on 13th October 2000.

(5) Pump failure on 9th April 2001, followed by well development in May 2001.

(6) Pump restart on 19th June 2001 (at 4.2mOD).

(7) Piezometer HW1I exhibits higher pore pressures than the piezometers in the Lower Horizon both prior to and during pumping. HW1I was installed in advance of the horizontal well at a depth some 2m lower than intended and silt was identified in the cuttings. It is thought that the HW1I data represent pore pressures in the silt horizon and not the Lower waste Horizon.

(8) Piezometers around HW3 showed a fall in leachate level of up to 0.5m in the 10 weeks prior to pumping. This may be linked to a redistribution of pore water by the horizontal well as discussed in section 6.8.4.

(9) Start of pumping from HW3 using a submersible pump installed in the horizontal section of the well at 3.3mOD on 22nd August 2000.

(10) Pump stopped and removed to conduct well development, 14th May 2001.

(11) After installation, piezometer HW3M had continuously foamed (Figure 5.12) which prevented readings being taken. In October 2001 the foaming ceased, presumably when the pumping reduced gas pressures below a critical level.

The main trends identifiable from Figure 6.28 to Figure 6.34 are discussed below.

6.8.1 Vertical Hydraulic Gradients

Based on the data from Line 1 (which is the most comprehensive), Figure 6.35 compares the observed pore pressures in each horizon with the hydrostatic case. The chart assumes that pore pressures above the Upper Horizon reduce hydrostatically with increasing elevation until the phreatic surface is reached.

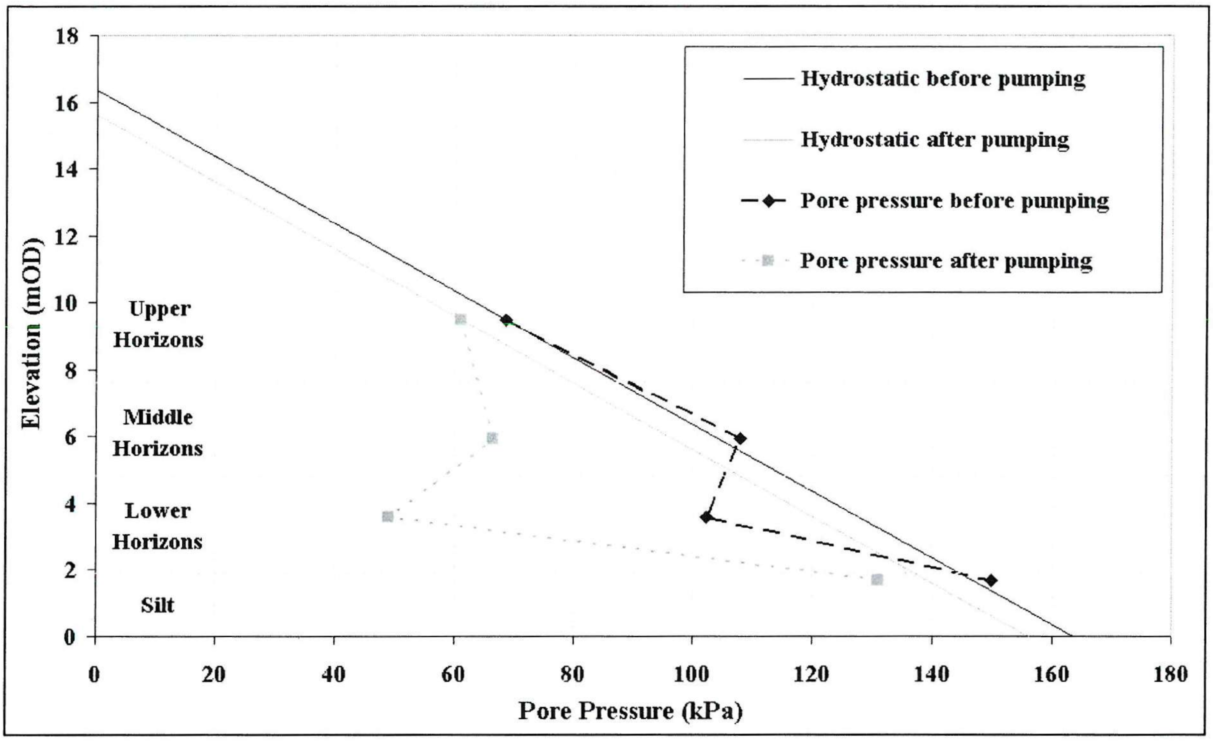

Figure 6.35 Comparison of the average observed pore pressures in each horizon with the ideal hydrostatic case (data from HW1, Line 1). The hydrostatic lines are based upon the pore pressures recorded in the Upper Horizon.

Pore pressures between the Upper and Middle waste horizons increased hydrostatically with depth before pumping commenced, but pore pressures in the Lower Waste were subhydrostatic. The Upper and Middle waste horizons were therefore underdrained by the Lower waste. It is thought that this hydraulic gradient may have developed as the result of the redistribution of pore pressures along the length of the well before pumping commenced. Most of the piezometers were installed a few weeks after the horizontal well and only provided information on the effect of pumping. However, piezometers HW1I, HW1O and HW1P did exist and provided some evidence for a reduction in leachate level related to well installation. Piezometer HW1O in particular (Figure 6.29), lying only 7m from HW1, recorded a 0.8m fall in leachate level before pumping commenced. A similar redistribution of pore pressures prior to pumping was thought to have occurred at HW3, evidence for which is presented in section 6.8.4.

Despite the subhydrostatic pore pressures in the Lower waste, the pressures in the basal silt were greater than hydrostatic (based on data from HW1I which had been installed in the silt layer). This suggested that dissipation of pore water from the fine grained silt was ongoing which is likely to have been the result of sequential loading of waste throughout the 1990's.

This was consistent with the dissipation of the elevated pore pressures identified in the silt during the site investigation (Figure 3.6). Figure 6.35 also shows the pore pressure distribution after 600 days of pumping. A downward gradient had been created from the Upper to the Middle horizon and the existing gradient between the Middle and Lower horizon increased. Some pore pressure dissipation had occurred in the underlying silt but the upward hydraulic gradient here too increased.

6.8.2 Discrete Piezometers

There was a large difference in the initial pore pressures and the response to pumping in each horizon. These observations were identified through the use of piezometers with discrete monitoring zones. Had the piezometers been screened over their entire length a somewhat meaningless pore pressure value would have been recorded and these significant differences would not have been observed. This is an important finding as the majority of pore pressure readings made by landfill operators are made using fully screened observation wells and not discrete piezometers. This implies that at some landfill sites the pore pressure at the base may be higher, or lower, than thought.

6.8.3 Changes in Leachate Level Over Time (HW1)

The divergence of leachate levels in the three horizons is clear from Figure 6.28. The largest reductions occurred in the Lower Horizon (where the well was situated) and the smallest response was recorded in the Upper Horizon. The overall reductions in head are summarised in Table 6.8.

Table 6.8 Summary of the total reductions in average leachate level as a result of pumping from HW1.

	Line 1			Line 2		
HORIZON	**Leachate Level Day 0 (mOD)**	**Leachate Level, Day 595 (mOD)**	**Average Reduction (m)**	**Leachate Level Day 0 (mOD)**	**Leachate Level, Day 595 (mOD)**	**Average Reduction (m)**
Upper	16.42	15.63	**0.79**	15.10	13.85	**1.25**
Middle	16.70	12.59	**4.13**	-	-	-
Lower	13.42	8.60	**4.82**	13.07	5.66	**7.41**
Silt	16.64	14.76	**1.88**	-	-	-

The reduction in piezometric level was clearly more pronounced in Line 2. This suggested that the pressure within the well was greater with increasing distance from the outflow point/submersible pump, so that the pressure gradient driving fluid flow from the waste to the well was reduced. The hydraulic gradient acting along the well, between Line 1 and Line 2 is calculated in Box 6.6 using data from Table 6.8 (Lower Horizon).

Box 6.6 Estimate of hydraulic gradient along HW1 caused by pumping

Day 0	**Line 1**		**Line 2**
Leachate Level	13.42 mOD		13.07 mOD
Elevation of HW1	3.25 mOD		3.58 mOD
h	10.17m		9.49m
Δh		0.68m	
d (distance between Line 1 & 2)		81m	
Initial gradient, Δh/d =		**0.0084**	
Day 595	**Line 1**		**Line 2**
Leachate Level	8.60 mOD		5.66 mOD
Elevation of HW1	3.25 mOD		3.58 mOD
h	5.35m		2.08m
Δh		3.27m	
d (distance between Line 1 & 2)		81m	
Gradient after Pumping, Δh/d =		**0.04**	

It can be seen that while a small hydraulic gradient existed between Line 1 and Line 2 before pumping commenced (which is consistent with the view that some redistribution of leachate occurred before pumping), the effect of pumping was to increase the gradient by a factor of nearly five. This pressure build up along the well may be expected in long lengths of pipe although the effects of terrain induced slugging may also have contributed. The implication is that continuous wells (two-ended) that are pumped from both ends may be more effective than blind-ended wells.

When HW1 was opened to allow the artesian flow of leachate and gas, the initial drop in leachate level in the Lower Horizon was rapid. The largest falls were recorded in piezometers

HW1U and HW1Y which fell 2.8 and 2.4m in the first 17 hours respectively (this was around 33% of the total head reduction achieved). Both HW1U and HW1Y were located on Line 2 (the Line closest to the outflow point) at a distance of 12m either side of the horizontal well. Other piezometers in the Lower Horizon recorded head reductions of between 0.1 and 1.2m in the first 17 hours depending on their proximity to the well. Further sharp reductions in leachate level occurred when Steps 2, 3 and 4 commenced which are clearly illustrated in Figure 6.29.

When extraction of leachate and gas ceased for 80 days in April-June 2001 the leachate heads in the Lower Horizons started to recover. Those piezometers which had shown the largest and most rapid reductions in leachate head also showed the largest and most rapid recoveries.

Leachate head reductions in the Lower Horizon had ceased by Day 500 (mid-November 2001) although the leachate flow rate from the well was continuing to fall at the end of the data collection period (Day 595). The total reductions were between 2.8 and 7.9m dependent on the proximity of the piezometer to the well.

The pattern of leachate head reduction in the Middle Horizon was similar to that in the Lower Horizon, but the changes were of a smaller magnitude. As with the Lower Horizon, reductions had ceased by Day 500. The total reductions in head were between 3.1m and 5.4m.

The magnitude of the pore pressure change in the Upper Horizon was much smaller than that of the Middle Horizon and there were no clearly defined responses to events such as a change in the pumping regime during the step tests. When pumping ceased for 80 days following pump failure a slight rise in leachate level was recorded, averaging 0.2m. Reductions in the Upper Horizons had also ceased by Day 500. Total reductions were small in comparison with the Middle and Lower horizons, being only 0.5 to 2.1m.

Throughout the data collection period, there were daily fluctuations in leachate level in all three horizons that were unrelated to leachate extraction. The fluctuations, of up to 0.4m, were thought to be primarily related to changes in atmospheric pressure. This is discussed in section 6.9.

The cross sections also showed large differences between the final piezometric levels in each waste horizon. The Lower Horizon (at both Lines 1 and 2) showed a broad zone of depressurisation, with uniform head reductions up to 12m on either side of the well. At Line 1 the piezometric surface between 12 and 25m rose at a shallow gradient of 0.114, and at Line 2 the gradient was 0.155. It was apparent that the distance of influence had been underestimated when designing the piezometer layout and there were no data on the position of the piezometric surface beyond 25m (with the exception of HW1P at 41m, but this was influenced by a drainage trench in Phase 3). To estimate the distance of influence, the gradient between 12m and 25m was assumed to continue uniformly to the point where the lowered piezometric surface would have met the original. For Line 1 this gave a distance of influence of 62m and for Line 2, 66m. These approximations were conservative as the gradients were likely to have become shallower with increasing distance from the well. However, 62 to 66m seemed reasonable given that no influence was recorded in existing vertical wells located 100m from HW1.

6.8.4 Changes in Leachate Level Over Time (HW3)

The leachate head reductions around HW3 were not as large as those around HW1 and the trends were less clear. The depth of the response zone (i.e. Upper, Middle or Lower Horizon) remained the principal factor in determining pore pressure change but the proximity of the well was of greater importance here than at HW1. To understand the distribution of pore pressures around HW3 and their change from pre to post-pumping it is more useful to study the cross section (Figure 6.34) than the change over time. The cross section shows that a valley shaped depression was created in the piezometric surface centred 4m to the north-east of the well (to the right on the chart), suggesting that the calculated position of the well may have been 4m from its actual position. This emphasises the earlier recommendation that guidance data from the drilling operations should be converted into standard grid coordinates by the guidance engineer as drilling operations proceed, and not by a third party at a later date (section 4.3.5).

The cross section also shows that a valley shaped depression was present before pumping commenced. Examination of the leachate level readings in the 77 day background monitoring period (prior to pumping) revealed that five piezometers showed clear reductions in level over this time. As piezometers were not installed until after the horizontal well, the initial pore pressure distribution was not known, so it was not possible to determine whether the depression existed prior to well installation or was a result of it. Some data were available from a disused vertical well situated only 2m from HW3 (well 351). This showed a sharp reduction that coincided with the installation of the horizontal well, as illustrated by Figure 6.36.

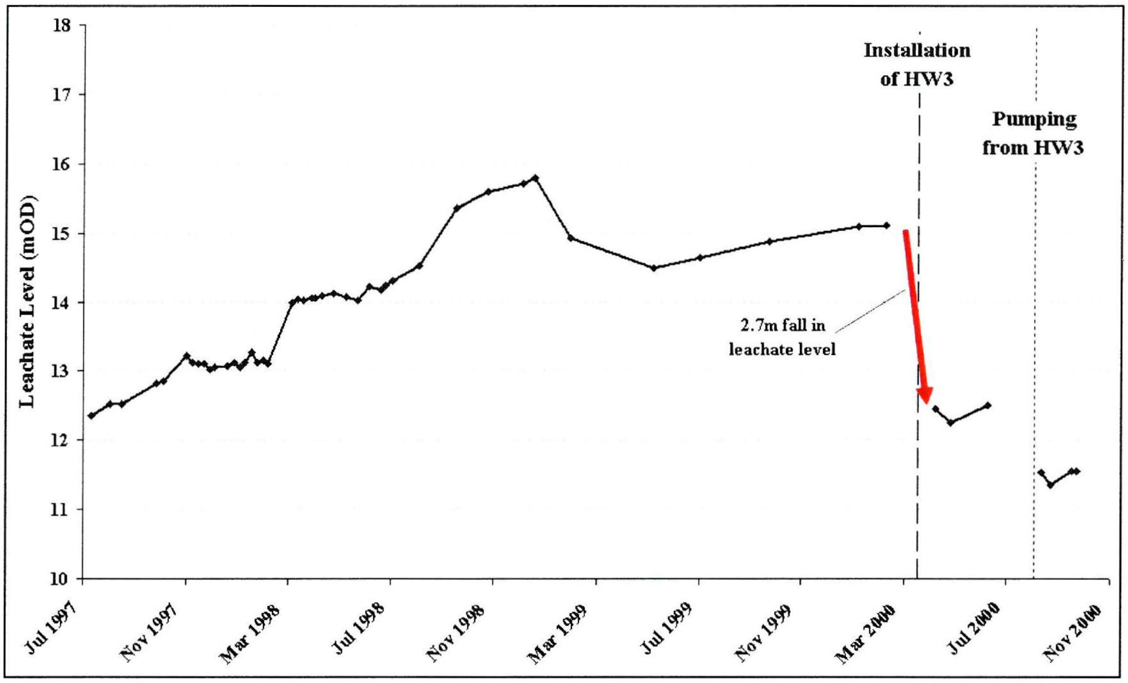

Figure 6.36 Leachate levels in vertical well 351. Arrow marks the 2.7m fall in leachate level associated with the installation of HW3. The preceding long-term rise was caused by the periodic loading of waste.

The sharp fall in leachate level in well 351 and the presence of the valley shaped depression suggests the horizontal well may have affected leachate levels by redistributing leachate from areas of high pore pressure to areas of low pore pressure. This may explain why HW3, which produced leachate under artesian pressure during installation (a head of at least 10mOD), was no longer artesian when inspected eight weeks later when the leachate level in the well had fallen to 5.1mOD.

To quantify the changes in leachate level it was necessary to estimate the level of the piezometric surface prior to well installation. The pre-pumping piezometric surface in the Upper Horizon was flat at a distance from 12 to 25m north-east of the well (to the right on the chart), suggesting that this was unaffected by the redistribution of leachate. Furthermore, at 25m the pore pressure distribution in the three horizons was approximately hydrostatic, so the pre-pumping level at this point, equal to 15mOD, was taken to be the original piezometric level in each horizon at the time of well installation.

Assuming the piezometric surface was flat at this starting level of 15mOD, the cross sections show that the largest reductions in leachate head occurred in the Lower Horizon (6.5m at the centre of the depression) but were almost matched by the reductions in the Middle Horizon (6.1m at the centre). The reductions in the Upper Horizon were significantly less (only 2.5m at the centre).

Inspection of Figure 6.33 reveals that reductions in leachate level had ceased in the Lower and Middle Horizons after 450 days of pumping (the data collection period continued for 535 days). However, leachate level reductions continued in the Upper Horizons at an average rate of 1.5mm/day.

From Figure 6.34, the fall in leachate levels in the Lower Horizon was more pronounced to the south-west (left) than the north-east, yet in the Middle Horizon this situation was reversed, making it difficult to estimate the width of influence of HW3 without further piezometer installations. However, using the gradient of the piezometric surface in the Lower Horizon (to the north-east and between 12m and 25m) the width of influence was calculated as 52m.

6.8.5 Anisotropic Hydraulic Conductivity and Targeted Depressurisation

If the pore pressure at the well is lowered through pumping and the hydraulic conductivity of the waste is uniform in all directions (i.e. $k_h/k_v \approx 1$), flow to well should be approximately radial and equipotentials circular. This assumes the extent of the waste is infinite in all directions, however, the proximity of the silt layer below the well may represent a flow line thus distorting the equipotentials beneath the well. Nevertheless the shape of the equipotentials above and to the side of the well should remain circular and, under steady state flow, the piezometric level at a fixed distance to the side of the well should be the same as the piezometric level at the same distance above the well. Conversely, if the horizontal hydraulic conductivity was greater than the vertical (i.e. k_h/k_v was greater than 1) the equipotentials would become elongated in the horizontal direction. To determine whether the hydraulic conductivity was isotropic or anisotropic the analysis discussed below was carried out.

Figure 6.37* reproduces data taken from around HW1 at the beginning and end of the pumping period. At a vertical distance of 5.5m above HW1 Δh was 1m. Had the hydraulic conductivity been isotropic ($k_h/k_v \approx 1$), Δh should also have been 1m at a horizontal distance of 5.5m from the well. However, Δh in the horizontal direction was much greater, at 8.7m. The resulting pressure heads were 10.4m in the vertical and only 0.7m in the horizontal. This clearly indicated that the hydraulic conductivity was anisotropic, with k_h/k_v being greater than 1. Fluid flow rates would therefore have been greater from the horizontal direction than the vertical. Data from other cross sections around HW1 and HW3 showed a similar trend.

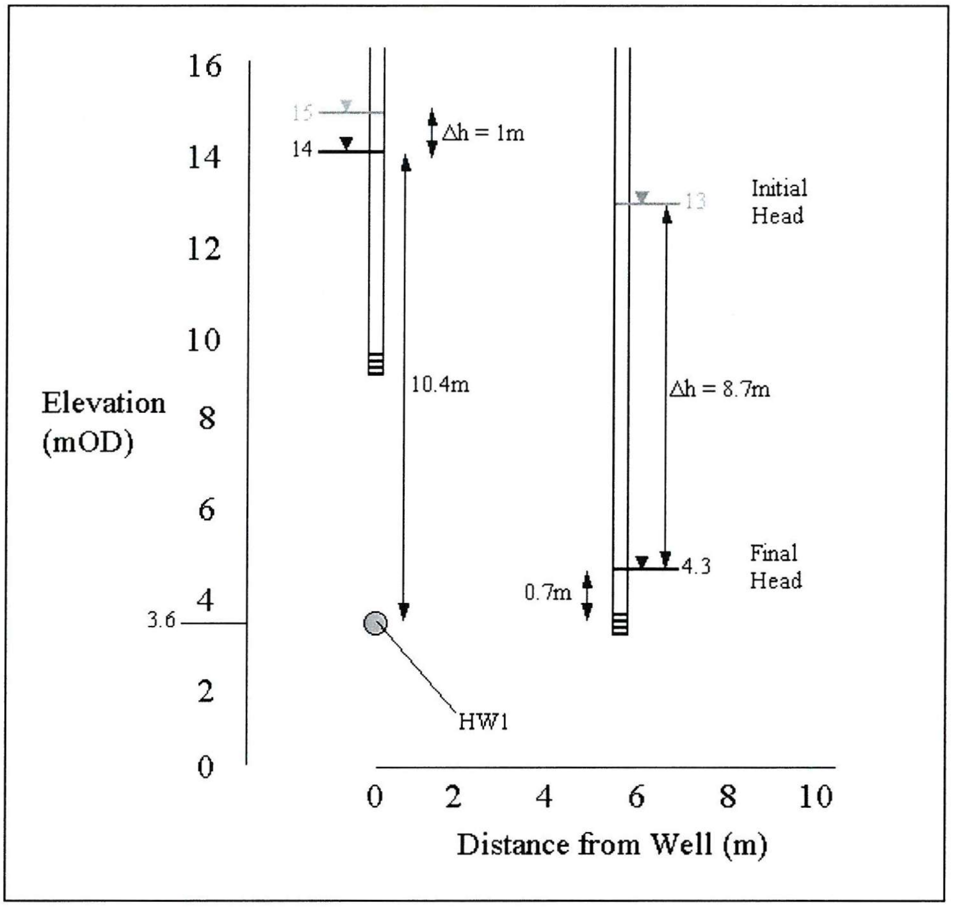

Figure 6.37 Piezometric levels around HW1 at start and end of pumping.

The desk study, section 3.2.2, reported that waste had been deposited in horizontal layers, followed by compaction using heavy machinery weighing up to 54 tons. It is thought that this

* These data are also shown in cross-section on Figure 6.32.

method of deposition, along with self-weight compression, may have been partly responsible for the anisotropic hydraulic conductivity.

This was consistent with the results of Hudson *et al* (1999) who reported experiments on waste in a large scale compression cell. The results of Hudson *et al* (1999) showed that the ratio of k_h/k_v increased with applied stress from approximately 2 at an applied stress of 40kPa to 5 at an applied stress of 600kPa. New research (2002) by Hudson and Beaven suggests that the ratio may be even higher, with k_h five to ten times greater than k_v over a similar stress range.

Pumping from a horizontal well within a strongly anisotropic formation has enabled a significant depressurisation in the horizon in which the well was located while having only a minimal effect on horizons just 5 to 6m above. This ability to target pore pressures in specific horizons may be of great advantage to landfill operators looking to reduce pore pressures in discrete horizons (most likely near the basal layer) while minimising the volume of leachate that must be extracted and treated.

6.8.6 Conclusions from Pore Pressure Changes

The inferences made above from the data of pore pressure change both in cross section and over time are summarised below.

- The pattern of pore pressure change with depth is consistent with anisotropy in the waste. Large scale tests by Hudson *et al* (1999) suggest that the k_h/k_v ratio ranges from 2 to 5 (and possibly 10) and is related to vertical effective stress.

- The anisotropic hydraulic conductivity combined with the horizontal geometry of the well allows the specific targeting of pore pressures in discrete waste horizons. This is potentially useful for landfill operators looking to reduce pore pressures acting on the base of a landfill while minimising the volume of leachate extracted.

- Large differences in pore pressure have been recorded within different waste horizons using piezometers with discrete monitoring zones. If these piezometers had been screened along their entire length the recorded pore pressure would, at best, have been an average value. The use of piezometers with discrete monitoring zones is therefore essential for obtaining a true picture of the pore pressure distribution within a landfill.

- The more pronounced reduction in piezometric level closer to the outflow of HW1 suggests that continuous (two-ended) wells, pumped from both ends, may be more effective than blind-ended wells.

- The lateral extent of the wells influence on pore pressures in the Lower Horizon was estimated to be 50 to 70m.

- There was evidence to suggest that HW1 and HW3 enabled the redistribution of leachate while the wells remained capped before pumping commenced. Most piezometers were installed after the horizontal wells to enable accurate positioning. However, the installation of a number of piezometers, say five, in a broad cross section across the planned route of each well would have provided invaluable information on the pore pressure distribution prior to well installation.

6.9 Influence of Barometric Pressure on Pore Pressures

Price (1985) states that in a confined aquifer, the water level in a standpipe piezometer (essentially a manometer) will be a balance between the pore pressure in the aquifer (in this case, the waste) and the ambient barometric pressure. The relationship is inverse such that a rise in barometric pressure will lower the water level in the piezometer. By comparing the change in barometric pressure (ΔP in mbars) with a corresponding change in piezometric level (Δh in cm) the *barometric efficiency* of the piezometer (*BE* as a %) is given by

$$BE = -100(\Delta h / \Delta P) \qquad (6.8)$$

For example if a rise in pressure of +16mbar causes a change in level of -13cm the efficiency is 81.3%. Barometric efficiency is a measure of how confined an aquifer is, the higher the value the more rigorously confined. Unconfined aquifers have barometric efficiencies of zero, i.e. no measurable change in water level occurs as barometric pressure fluctuates (Price 1985). It should be noted that the change in water level occurs only within the piezometer tubing and does not represent an actual change in the water level of the aquifer.

From the Figures above showing the changes in piezometric level over time, it is apparent that small fluctuations occurred on a short term basis. Few of these fluctuations could be attributed to changes in the pumping regime and it was thought that they may have been linked to fluctuations in barometric pressure. To investigate this, the mean barometric efficiency of each piezometer was calculated over a series of pressure and level readings using equation (6.8). The data from piezometer HW3E are used as an example in Table 6.9.

Table 6.9 Example calculation for barometric efficiency of single piezometer. Data from HW3E.

Date & Time of Reading	Barometric Pressure (mbar)	Piezometric Level (mOD)	Adjusted Level (mOD)	Change in Pressure (mbar)	Change in Level (cm)	Efficiency (%)
23/10/00 10:02	1016	13.86	13.86			
30/10/00 10:45	979.47	14.13	14.16	-36.5	27.1	74.2
13/11/00 10:03	998.18	13.80	13.88	25.5	-21.9	85.9
27/11/00 10:25	1011.5	13.60	13.74	15.0	-18.9	125.9
14/12/00 10:13	1003.9	13.62	13.82	-7.6	1.8	23.8
15/01/01 09:55	1025.7	13.33	13.66	12.0	-5.9	49.2
22/01/01 10:20	1002.5	13.53	13.89	-23.2	20.1	86.6
29/01/01 10:55	1021	13.26	13.65	18.5	-26.9	145.4
06/02/01 10:05	993.9	13.51	13.93	-27.1	25.0	92.1
12/02/01 09:18	1019.2	13.36	13.81	25.3	-14.6	57.7
19/02/01 10:03	1041	13.27	13.75	21.8	-8.7	39.9
26/02/01 09:48	1005.1	13.55	14.05	-35.9	27.3	76.1
05/03/01 10:10	1018.3	13.29	13.82	13.2	-25.7	194.7
12/03/01 10:15	1004	13.27	13.83	-14.3	-1.7	-11.9
19/03/01 09:41	1017.9	13.23	13.81	13.9	-4.7	33.8
26/03/01 10:16	1011.2	13.20	13.82	-6.7	-2.7	-40.3
09/05/01 10:14	1023	13.03	13.82	11.8	-17.4	147.5
14/05/01 09:24	1006.6	13.15	13.96	-16.4	12.5	76.2
01/06/01 12:28	1021	13.08	13.97	16.6	-13.7	82.6
17/07/01 11:19	1004.6	12.84	13.90	-23.3	-3.8	-16.3
27/07/01 14:57	1017.7	12.73	13.83	13.1	-11.0	84.1
					Mean Efficiency	**70.4%**

As pumping of the horizontal wells had caused a general fall in piezometric level over the period of investigation it was necessary to adjust the piezometric readings so that the effects of barometric pressure fluctuations would not be masked by the overriding effects of pumping. To give the 'Adjusted Level' the average gradient of this fall was calculated (0.004m/day in the case of HW3E) and used to estimate what the piezometric level would have been had pumping not occurred. For example, the recorded piezometric level in HW3E on the 12/02/01 was 13.36mOD (112 days after the start of the data set on 23/10/00). The adjusted piezometric level was calculated using

$$(\text{Days Elapsed} \times \text{Gradient}) + \text{Recorded Level} = \text{Adjusted Level}$$
$$(112 \times 0.004) + 13.36 = 13.81$$

The pressure and level data from Table 6.9 are plotted against time in Figure 6.38 where the inverse relationship is visible and the general reduction in piezometric level that resulted from pumping is also clear. The change in barometric pressure (ΔP) as a function of the change in leachate level (Δh) is shown in Figure 6.39 and the inverse relationship can clearly be seen. The data collected from the first two months of pumping were excluded as, despite adjustments, barometric effects were masked by a comparatively rapid and non-linear fall in leachate level.

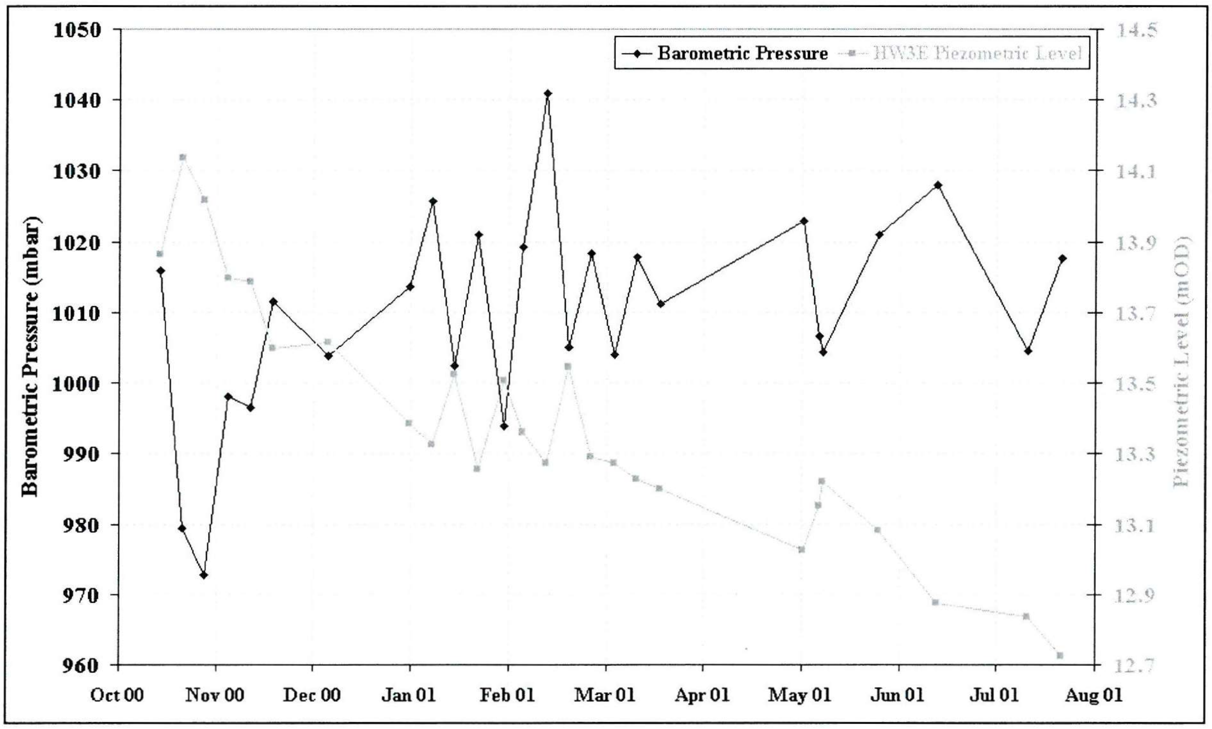

Figure 6.38 Inverse relationship between short term barometric fluctuations and piezometric level while long term reductions in piezometric level continue due to pumping. Data from HW3E.

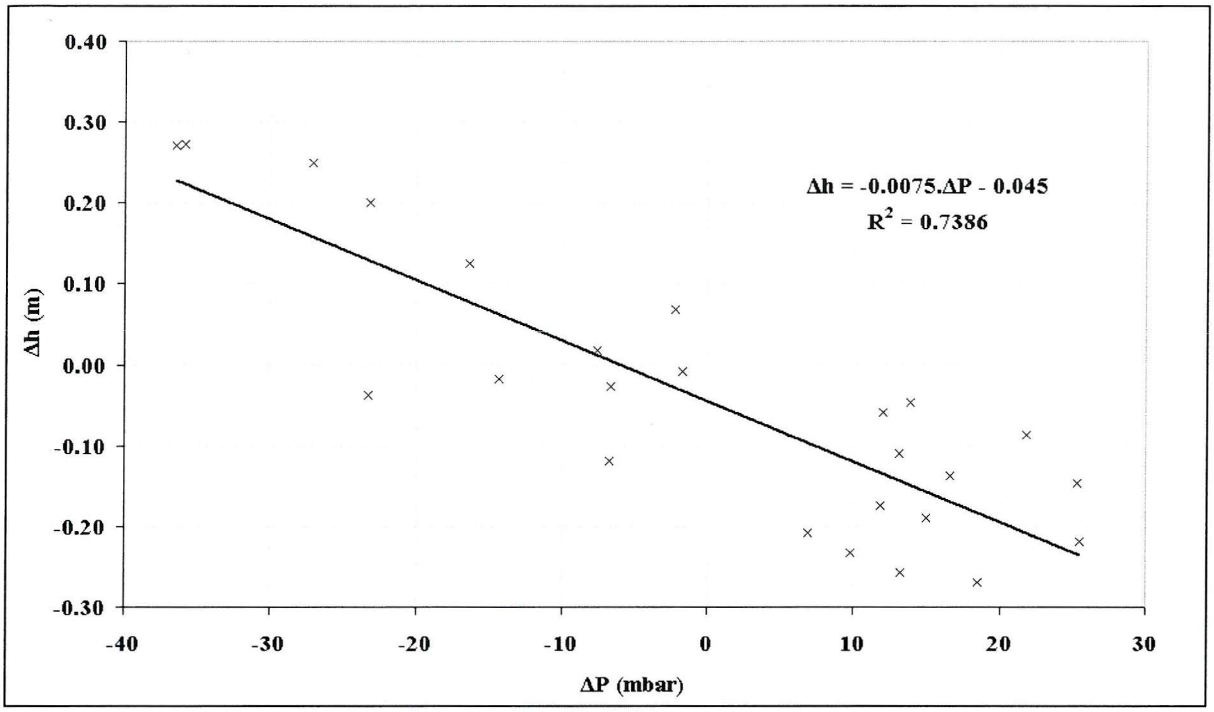

Figure 6.39 HW3E change in leachate level (Δh) as a function of change in barometric pressure (ΔP).

Table 6.10 gives the calculated efficiency of each piezometer and Table 6.11 provides a summary of the data.

Table 6.10 Barometric Efficiency of each Piezometer

Piezometer Reference	Horizon	Barometric Efficiency %
H03	Lower	105.2
H04	Lower	104.1
HW1P	Lower	101.5
HW1W	Middle	97.4
HW1M	Lower	95.8
HW3F	Lower	93.7
HW1L	Upper	93.4
HW1V	Upper	93.0
HW1G	Upper	91.0
HW1Z2	Lower	90.8
HW1F	Middle	88.1
H08	Upper	84.8
HW3L	Lower	84.5
HW1I	Silt	82.7
HW1Q	Lower	80.3
HW3J	Lower	79.9
HW3D	Middle	77.4
HW3I	Upper	76.9
HW1H	Middle	74.9
HW1O	Lower	74.6
H05	Lower	73.8
HW3C	Middle	73.2
HW3R	Upper	70.9
HW3E	Middle	70.4
HW3G	Upper	69.7
HW1C	Middle	68.7
HW3H	Middle	66.9
HW1Z1	Upper	66.8
HW1K	Lower	62.4
HW1B	Middle	62.3
HW3S	Lower	60.7
H06	Lower	59.6
HW1Y	Lower	57.5
HW3B	Middle	52.8
HW1D	Middle	50.5
HW1U	Lower	45.6
HW1J	Upper	32.8
HW3K	Upper	19.0
HW3N	Lower	10.0
HW1T	Upper	6.5
HW1E	Upper	-3.5
Mean Efficiency		**69.4%**

Table 6.11 Summary of mean barometric efficiencies.

Location / Horizon	Number of Piezometers	Mean Efficiency (%)	Standard Deviation
All Piezometers	41	69.4	26
HW1 Piezometers	27	71.9	27.5
HW3 Piezometers	14	64.7	23.5
Upper Horizon	12	58.5	35.2
Middle Horizon	11	71.1	13.8
Lower Horizon	18	75.7	24.5

The mean efficiency of all piezometers was 69.4% suggesting that the waste was reasonably but not rigorously confined. The standard deviation (SD) of all piezometers was 26, indicating a fairly broad range of values about the mean. Indeed, from Table 6.10, there were five piezometers with values over 95% demonstrating near complete confinement yet there were six piezometers with values less than 50%, two of which were near zero and indicated completely unconfined conditions.

Table 6.11 divides the data into (a) location and (b) horizon. It can be seen that piezometers around HW1 had a higher efficiency than those around HW3 which may have been the result of the shallower waste depth at HW3 being less effective at confining the horizons below. The Lower Horizons exhibited more rigorously confined conditions than the Middle Horizons which in turn were more rigorously confined than the Upper Horizons. This may also have been related to the depth of the overlying waste and its ability to act as a confining layer. If this theory is correct the SD value for each horizon should decrease with depth. Indeed, the Upper Horizon exhibits the highest SD value at 35 although the lowest SD value represents the Middle Horizons and not the Lower Horizons.

The calculations above were based on occasional readings of piezometric level. However, barometric pressure, and therefore piezometric levels, were changing continuously. The efficiency calculation would therefore have benefited from a continuous assessment of both barometric pressure and piezometric level. Such data were available from a pressure transducer that had been installed in one of the site investigation wells (H05) shortly after its installation. Data were collected at half hour intervals between December 1998 and March

1999*. These data offered an opportunity to examine barometric efficiency more thoroughly and without the additional influence of pumping. Figure 6.40 shows the pressure and level changes over time in H05.

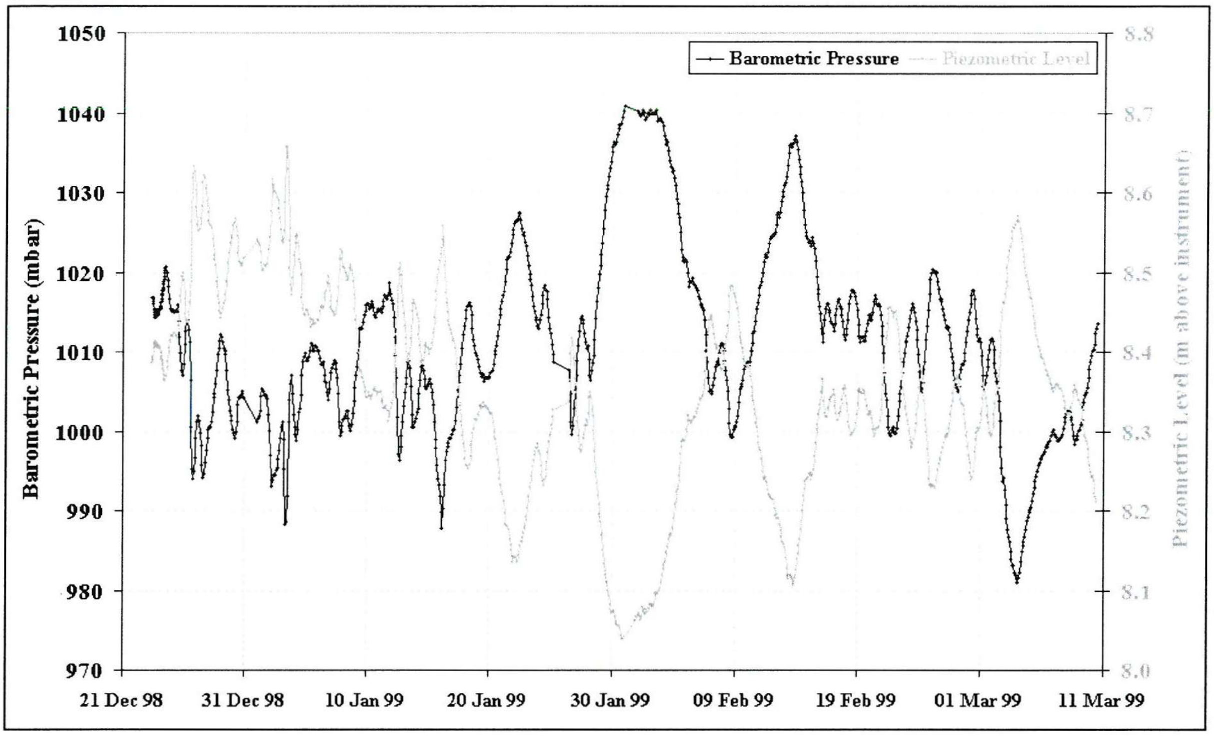

Figure 6.40 Inverse relationship between barometric pressure and piezometric level of H05.

The inverse relationship is obvious from the chart. Using these data and the same method shown in Table 6.9, the barometric efficiency was calculated as 91.6%. With half hourly readings there were many more data points (900+) and the result was regarded as significantly more accurate. It is interesting to note that this figure of 91.6% is almost 20% higher than the previously calculated figure of 73.8% for this same piezometer. The lower figure may have been produced by inaccuracies resulting from (a) the infrequent monitoring schedule, and (b) the ongoing pumping. If these inaccuracies have also affected the other piezometers, the barometric efficiency values listed in Table 6.10 and Table 6.11 may be underestimated.

What are the implications of this clear relationship between barometric pressure and piezometric level? Firstly, as already suggested, the waste can be regarded as confined to some extent by the layers of waste above with the degree of confinement increasing with

* The pressure transducer was of the 'vented' type so that the air pressure in the transducer body was in equilibrium with barometric pressure and the readings referred directly to actual changes in piezometric level.

depth. Secondly, landfill operators may overlook barometric pressure when analysing leachate level data. In a worst case scenario, the barometric pressure may have risen (or fallen) by up to 60mbar since the last leachate level reading so that levels may appear to have fallen (or risen) by 0.6m where the efficiency of the piezometer is 100%. This may prompt unnecessary action, i.e. engaging pumps, or may result in no action when action is required. As the barometric efficiency of the piezometers generally increased with depth, readings taken from near the base of the site may be more sensitive. The accuracy of pore pressure readings near the base of the site is particularly significant at landfills like Rainham where tight licensing conditions are in force and the leachate head on the base should not exceed 2m.

6.10 Waste Compression

It was anticipated that the waste would compress as a result of the depressurisation of pore fluids and, as described in section 5.1.2, four magnet extensometers were installed to indicate the distribution of settlement with depth in addition to the total amount of settlement at the surface. The waste was also expected to compress through ongoing decomposition although the relative importance of each factor was unknown.

In the months following the start of pumping, it became obvious that large settlements were occurring as the exposed length of each piezometer tube gradually increased. This is illustrated in Figure 6.41 which shows piezometer HW1C seven months after installation with its cement plug and metal cover (which had originally been at ground level) stuck to the tubing some 0.5m above ground.

Figure 6.41 Settlement left the piezometer covers well above ground level (HW1C shown).

6.10.1 Estimation of Settlements

Before pumping commenced it was not known how the pore pressure distribution would be affected and consequently it was difficult to estimate to what degree the waste would compress. Once the changes in pore pressure were established, an estimate of the settlements related to depressurisation were made, and these were compared with actual settlements. This analysis is presented below.

To calculate the expected settlement due to depressurisation it was assumed that soil (waste) movements were predominantly vertical [Terzaghi, 1943], so that the relationship between the vertical stress and stain increments would be governed by the one-dimensional stiffness modulus, E'_0. Variations in stiffness with depth could then be taken into account by dividing the waste into a number of layers, each of which was characterised by a uniform E'_0 value.

The settlement of each layer, ρ, was estimated from

$$\rho = (\Delta\sigma'_v \times T) / E'_{0(av)} \tag{6.9}$$

where T is the thickness of the layer.

$E'_{0(av)}$ was derived from Beaven (2000), who suggested that

"there is an approximate linear increase in stiffness of all wastes with increasing effective stress, the stiffest waste is $E'_0 = 3.5(\sigma'_v)$ and the most compressible is $E'_0 = 7(\sigma'_v)$."

$E'_0 = 5.25(\sigma'_v)$ was therefore taken as an average. As the effective stress itself changes during depressurisation, the average E'_0 value of the initial and final conditions was used.

Initial and final pore pressures were derived from piezometer readings (taking the unit weight of leachate as $10kN/m^3$) and total stress, σ_v, was calculated from equation 3.12, which was derived during the site investigation and describes total stress as a function of depth, z

$$\sigma_v = 0.1791z^2 + 4.2447z$$

Using these equations, the expected settlements were calculated for locations directly above and at a distance of 25m from HW1 (Table 6.12 and Table 6.13).

Table 6.12 Settlement using maximum recorded reductions in pore pressure centred on HW1.

Stratum	**Unsaturated**	**Upper**	**Middle**	**Lower**
Boundaries (mOD)	31 – 16.8	16.8 – 7.5	7.5 – 4.5	4.5 – 1.5
Thickness of layer, T (m)	14.2	9.3	3	3
Elevation of mid-point (mOD)	23.9	12.15	6	3
Depth of mid-point, z, (m)	7.1	18.85	25	28
Total stress, σ_v (kPa)	39.2	143.7	218.1	259.3
Initial pore pressure, u_{ini} (kPa)	0	46.5	104.2	109.9
Final pore pressure, u_{fin} (kPa)	0	34.5	50	51.3
Initial effective stress, $\sigma'_{v(ini)}$	39.2	97.2	113.9	149.4
Final effective stress, $\sigma'_{v(fin)}$	39.2	109.2	168.1	208
$\Delta\sigma'_v$ (kPa)	0	12	54.2	58.6
$E'_{0(ini)}$	206	510	598	784
$E'_{0(fin)}$	206	573	883	1092
$E'_{0(av)}$	206	542	741	938
Settlement, ρ (m)	0	0.206	0.219	0.187

The total maximum estimated settlement centred over HW1 was calculated as;

$$(0 + 0.206 + 0.219 + 0.187) = \mathbf{0.61m}$$

Table 6.13 Settlement using reductions in pore pressure 25m from HW1.

Stratum	**Unsaturated**	**Upper**	**Middle**	**Lower**
Boundaries (mOD)	31 – 16.34	16.34 – 7.5	7.5 – 4.5	4.5 – 1.5
Thickness of layer, T (m)	14.66	8.84	3	3
Elevation of mid-point (mOD)	23.67	11.92	6	3
Depth of mid-point, z, (m)	7.33	19.08	25	28
Total stress, σ_v (kPa)	40.7	146.2	218.1	259.3
Initial pore pressure, u_{ini} (kPa)	0	44.2	99	109.8
Final pore pressure, u_{fin} (kPa)	0	40.1	68.1	67.9
Initial effective stress, $\sigma'_{v(ini)}$	40.7	102	119.1	149.5
Final effective stress, $\sigma'_{v(fin)}$	40.7	106.2	150	191.4
$\Delta\sigma'_v$ (kPa)	0	4.2	30.9	41.9
$E'_{0\,(ini)}$	214	536	625	785
$E'_{0\,(fin)}$	214	558	788	1005
$E'_{0\,(av)}$	214	547	707	895
Settlement, ρ (m)	0	0.068	0.131	0.140

The estimated settlement at a distance of 25m from HW1 was calculated as;

$$(0 + 0.068 + 0.131 + 0.140) = \mathbf{0.34m}$$

The estimated settlement due to depressurisation was therefore 0.61m above the well where pore pressure reductions were greatest, and 0.34m at a distance of 25m from the well. Reductions in piezometric level ceased around 500 days after pumping first began, so related settlements may also have ceased after this time.

In the same 500 days, settlements of the landfill surface above HW1 were measured at two locations, the first directly above the well and the second 12m away. Settlements were 1.45m and 1.79m respectively, considerably more than the estimated values. The additional settlement may therefore have been a result of waste decomposition.

Some data were available for the settlement of the landfill surface in an area away from the influence of HW1*. The area used for comparison was also on Phase 2 of the landfill, of a similar initial waste depth and had a similar depositional history. The settlements recorded varied between 0.9m and 1.8m over the same 500 day period. This wide variation between the two control locations may have been due to the heterogeneous composition of the waste (and therefore differences in its rate of decomposition). Nevertheless, the comparison suggested that decomposition alone could account for the settlements recorded around HW1.

While the above analysis suggested that depressurisation due to pumping may have caused significant settlements, the relative importance (in comparison to settlements thought to be caused by decomposition) was difficult to define. Only a more thorough investigation with a greater number of monitoring points in both the active and control areas would be able to differentiate and quantify the settlements caused by depressurisation alone.

6.10.2 Distribution of Settlement with Depth

The magnet extensometers enabled the distribution of settlement with depth to be examined. Figure 6.42 to Figure 6.45 show the total settlement recorded by each magnet during the 500 days after pumping began.

* Taken from Cleanaway 'airspace' surveys.

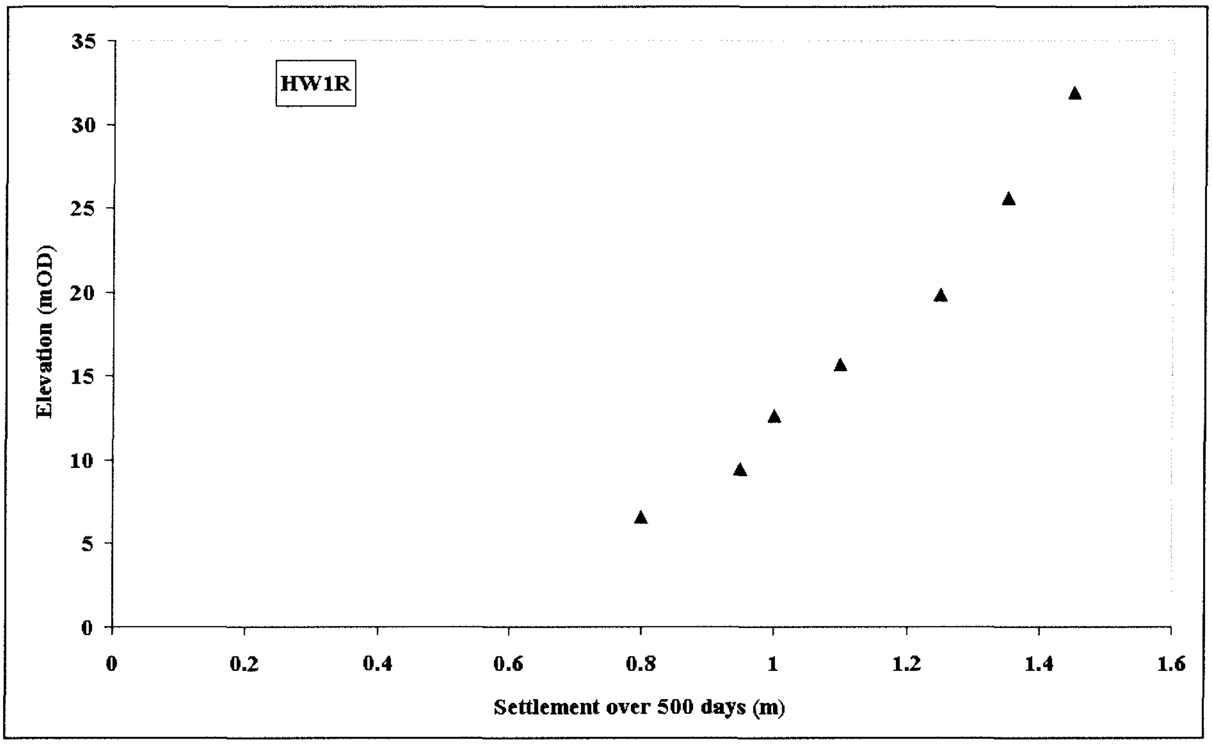

Figure 6.42 Settlement of Magnets at Extensometer HW1R

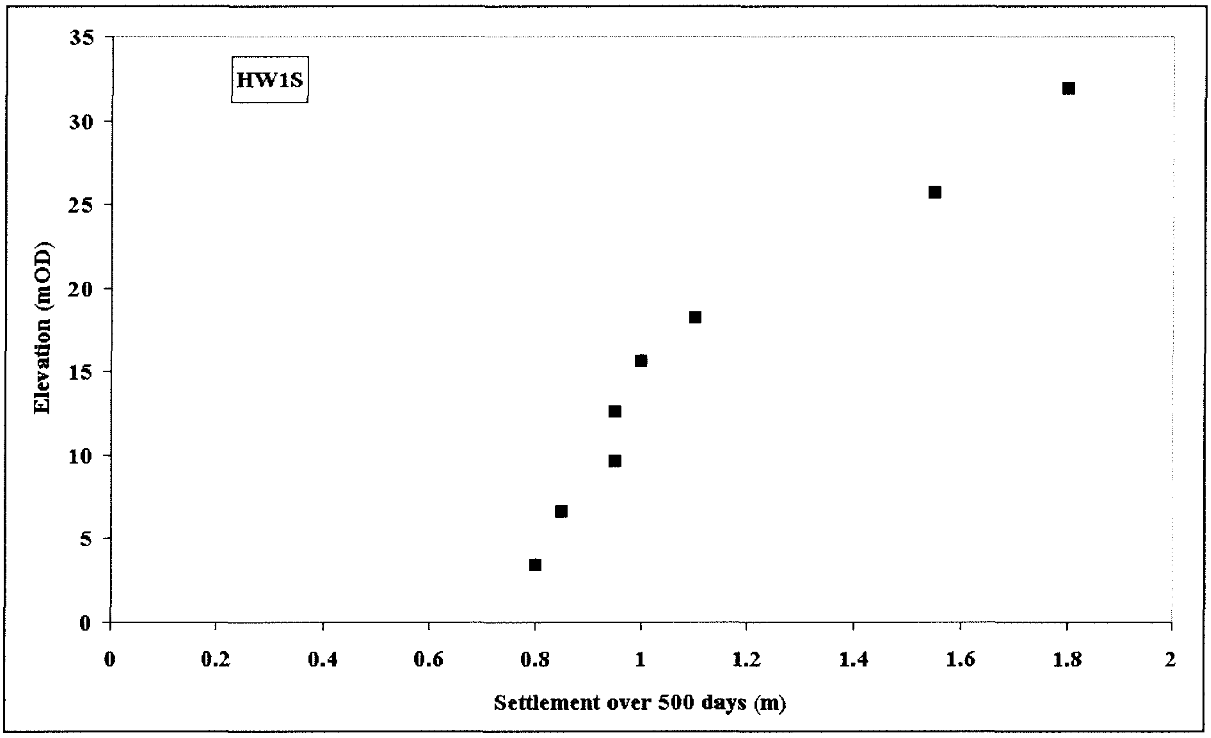

Figure 6.43 Settlement of Magnets at Extensometer HW1S

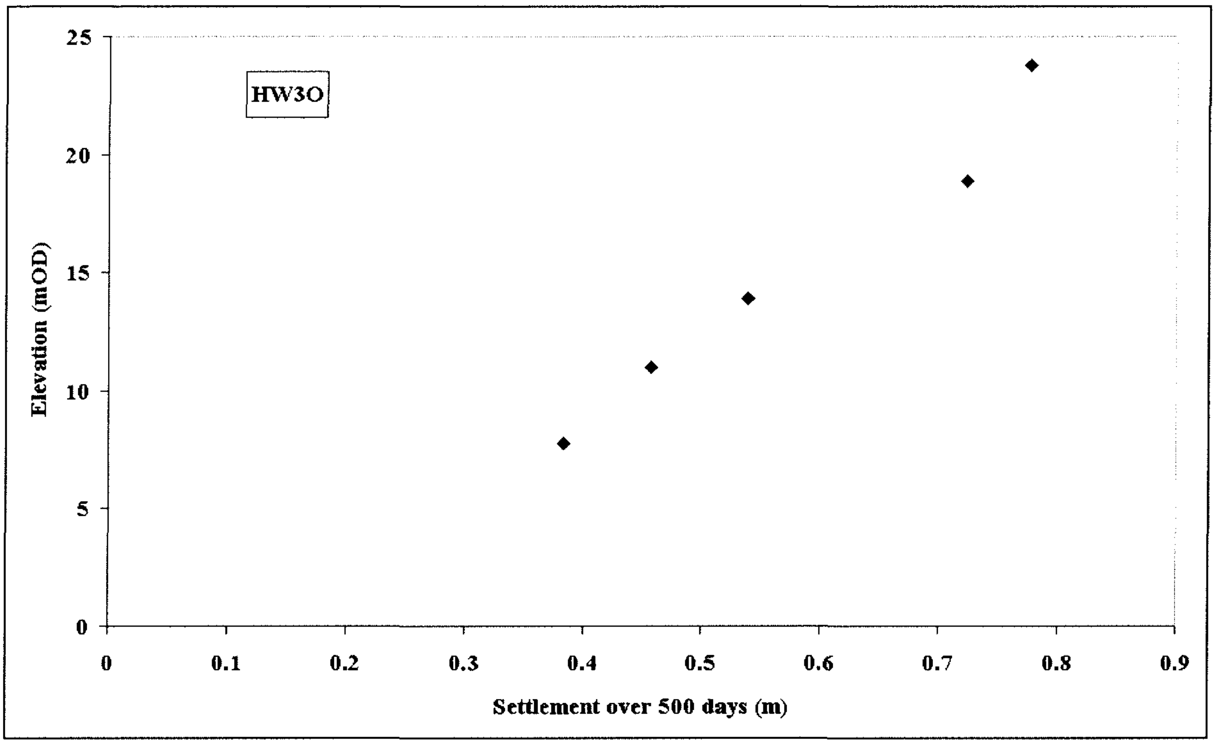

Figure 6.44 Settlement of Magnets at Extensometer HW3O

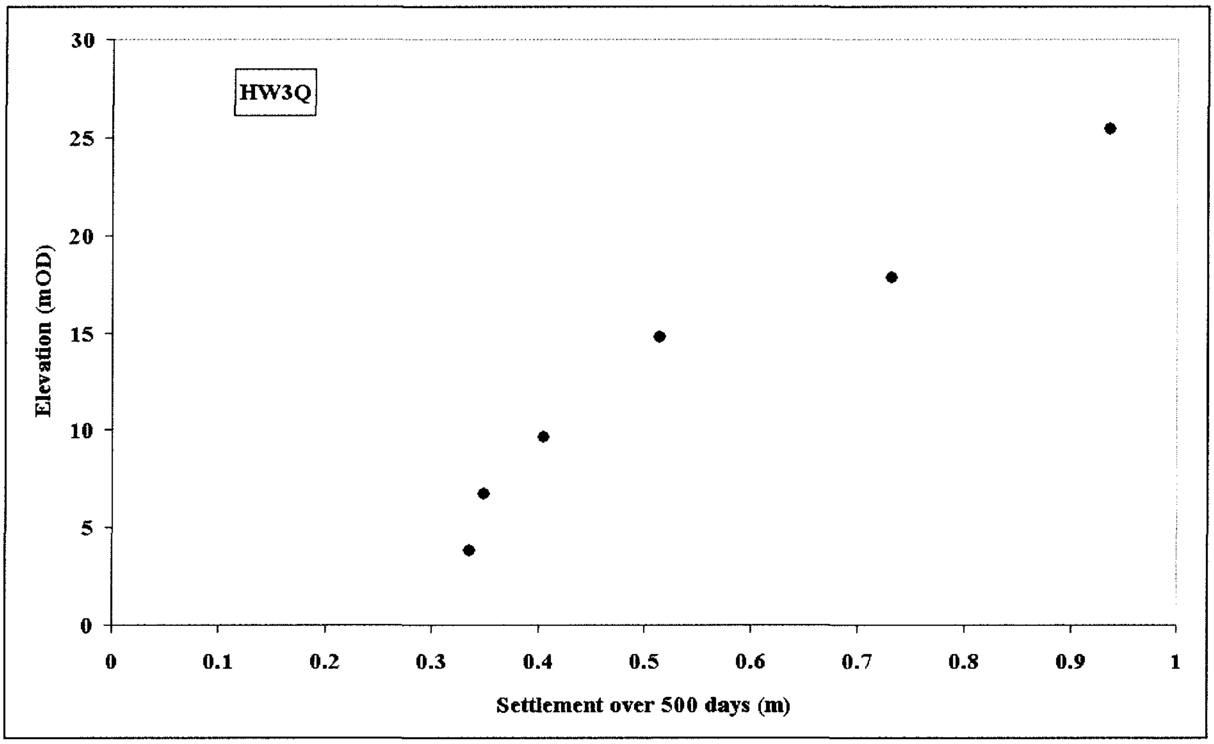

Figure 6.45 Settlement of Magnets at Extensometer HW3Q

In all of the extensometers, settlements were recorded within the uppermost 5m to 10m where the waste was unsaturated and settlement could not have been related to depressurisation. Indeed, data from HW1S and HW3Q indicate that settlements were notably greater in the uppermost layers than in the saturated layers below, which may be related to more rapid decomposition in the more recently deposited waste.

As pore pressure reductions increased with depth and were greatest in the Lower Horizon, settlements were expected to follow the same pattern. However, this was not reflected in the above Figures. Rather, the data followed a broadly linear trend (particularly below 15mOD) with no increase in gradient as the Lower Horizon was approached. Unfortunately, only HW1S and HW3Q had magnets within the Lower Horizon as the depth of HW1R and HW3O was shortened to avoid drilling through the horizontal well screen. HW3Q did record proportionally larger settlements in the lowest magnet (at the same level of the well) which may have been related to greater depressurisation around the well, although HW1S did not.

At the start of the 500 day monitoring period the landfill surface at HW1 was at 31.5mOD and, from site investigation data, the waste/silt interface was at 0.5mOD giving a waste depth of 31m. The total settlement (indicated by HW1R and HW1S) was between 1.5m and 1.8m but the above Figures show that only half of this total settlement occurred in the upper 25m or so of waste. There are two possible explanations to account for the remaining and substantial settlements that must have occurred below the level of the magnets.

(1) Settlements in the Lower Horizon (beyond the reach of the magnets) may have been much larger than those recorded in the Middle, Upper and Unsaturated Horizons, possibly as a result of depressurisation caused by pumping.

(2) Significant consolidation may have occurred below 0.5mOD, and may be related to two factors.

(a). Depressurisation in the Lower Horizons may have increased the pore pressure gradient between the silt and the waste, accelerating the flow of pore fluid out of the silt (and underlying clay) and increasing the rate at which these underlying layers consolidated (such an increase in gradient was previously suggested in section 6.8.1).

(b). Ongoing consolidation of the silt and clay was likely to have occurred in the 20 months between the site investigation and the start of pumping. As evidence for this, Figure 3.6 shows the gradual reduction in pore pressures of the basal layers between successive periods of tipping which are likely to have been accompanied by consolidation. Furthermore, the landfill surface during the site investigation was at 24mOD and the subsequent addition of a 7m thick layer of waste before the start of pumping may have accelerated consolidation of the silt and clay layers. This may have resulted in the silt/waste interface being lower than the 0.5mOD determined during the site investigation so that the waste depth was actually greater than 31m. Some of the remaining settlement may have occurred in this additional depth of waste.

Potentially indicative data from the Lower Horizon are absent from both HW1R and HW3O as they were terminated short so as to avoid drilling through the horizontal well screen. With hindsight it would have been more useful to install these extensometers to the full depth even if they were offset from the horizontal well by a few metres. Moreover, all the extensometers were terminated short of the silt/waste interface (so as not to compromise the integrity of the containment layers), yet to provide a full understanding of the distribution of settlement with depth the extensometers would need to monitor the silt and clay layers and would ideally be terminated in the underlying chalk bedrock.

6.10.3 Waste Compression - Conclusions

The above analyses have demonstrated that settlements during the pumping period were substantial, up to 1.8m over an initial waste depth of approximately 31m, and it was indicated that the following three factors may have contributed to this settlement;

(1) consolidation of the waste due to depressurisation as a result of pumping from the horizontal wells,

(2) ongoing consolidation of the basal silts and clays which previously showed evidence of elevated pore pressures, and may have been accelerated by depressurisation of the overlying waste, and

(3) decomposition, especially in the most recently deposited waste.

However, the analyses were inconclusive as to the relative significance of each of these factors. Only a more thorough investigation with deeper magnets and remote extensometers (for more accurate comparisons) would provide a fuller understanding of the settlements associated with depressurisation caused by pumping from the horizontal wells.

CHAPTER 7

Conclusions

7.1 Project Overview

The origins of this project lay in the idea that horizontal wells might be used in landfills to control leachate and gas as an alternative to existing methods. The project has focused on the retro-fitting of horizontal wells at older landfill sites where initial site preparation for the control and/or removal of leachate and gas had been less rigorous than is now practiced at modern sites.

The research had to first consider whether the relatively new technology of directional drilling could be adapted for the installation of horizontal wells in heterogeneous waste materials of unknown and random composition. Following an initial desk study of both the directional drilling industry and site records of Rainham landfill, a site investigation was carried out. The site investigation concluded that while drilling was likely to be difficult, it was nevertheless feasible. The site investigation was followed by a study of similar installations in the American Midwest and a series of drilling trials, with each trial building on the experience of the previous ones. In Spring 2000, three prototype horizontal wells were successfully installed at Rainham and a single well at Metallic Tile landfill.

Once the Rainham wells had been installed, they were operated for approximately 18 months. The data collected from the extensive monitoring system enabled a thorough assessment of both the performance of the wells and the effect of their operation on the surrounding waste mass.

A number of important and at times surprising conclusions were made during both the well installation phase and the pumping tests. These conclusions have been detailed in the corresponding chapters of this thesis and are now summarised below.

7.2 Desk Study & Site Investigation

The desk study helped to clarify the method by which waste had been deposited at Rainham. This method, as is common at UK landfills, was to deposit the waste in a sequence of layers, each with a thin covering of low permeability material (daily cover). The waste was also compacted vertically using machinery weighing up to 54 tons. Layering and compaction in only one dimension was thought to be partly responsible for the anisotropic pore pressure response described in Chapter 6.

Before the site investigation boreholes were drilled it became apparent that there were no definitive standards to provide a consistent description or classification of the waste samples collected. As a result, a protocol was derived for use in the Rainham site investigation. This protocol was later adapted before publication and can now be used to provide a basis for similar investigations.

Due to the problems experienced with the site investigation drilling rig (principally inadequate rotational speed) some of the more quantitative data, such as moisture content and density measurements, were unreliable. However, the qualitative descriptions of the waste samples were invaluable in providing an insight into the composition and characteristics of the waste as much as twenty-five years after deposition. The waste samples were found to be very heterogeneous, with some containing sizeable pieces of tough material such as concrete while others consisted of soft slurry-like soil. With regard to directional drilling, the hard items were a concern as they could impede progression of the pilot borehole and the slurry was a concern as it would prevent effective steering of the drill bit. Indeed, obstructions delaying progress were encountered during the directional drilling trials and soft ground did cause steering difficulties on a number of occasions. However, such problems were infrequent and each was successfully overcome.

The site investigation also provided information on the exact position of the site base. This was an important consideration for maximising the depth of the wells without compromising the containment layers of silt and clay, but one that had been overlooked in the Midwest trials.

Despite the insight provided by the desk study and site investigation, little information was obtained to assist with design specifications for drilling of the horizontal wells - for example, the required capacity of the drill rig and diameter of the borehole. It was therefore decided to conduct a small scale drilling trial prior to attempting a full scale installation.

7.3 Field Trials

7.3.1 First Rainham Field Trial

The pilot borehole was completed with little incident. A number of hard materials were encountered which deflected the drill bit but none that prevented its progress. Of greater concern were materials with high tensile strength in strands (e.g. wire, fabrics, steel rope) as these became wound around the drilling tools preventing cutting of the waste and making their progress through the waste more difficult. It soon became clear that cutting or milling of the waste was unlikely and that 'drilling' should focus on pushing a hole through the waste rather than removing the material to form one. At this stage it was realised (and later reaffirmed at Metallic Tile) that aggressive drilling tools with protruding teeth should be avoided.

It became apparent during the drilling of the pilot borehole that no drilling fluids were likely to return. Indeed, little if any fluids returned during subsequent installations and it was concluded that bentonite based fluids should be avoided so as not to reduce the hydraulic conductivity of the waste in the vicinity of the well.

The pilot borehole demonstrated that while walkover guidance systems were suitable for shallow applications in waste, they suffered from interference at depths greater than 8m or so. A wireline system was therefore necessary for the full scale trial whose maximum depth would be nearly 30m.

Essentially, the pilot borehole was completed with relative ease and the reaming stages that followed proceeded quickly. The major difficulty was encountered during installation of the well screen. Analyses were performed to calculate the resistance to movement caused by friction in both (a) a borehole that remains open (where the well screen only contacts the borehole along its base), and (b) a borehole with full closure (where contact between the well screen and the borehole walls occurs all around the screen). The analyses suggested that the borehole had undergone at least a partial closure which prevented installation because the friction exceeded the pullback capacity of the rig. Three main recommendations resulted from this.

1) a drilling rig with a larger pullback capacity than 120kN would be required for the full scale trial,
2) in order to reduce friction with the partially collapsed borehole, the diameter of the well screen should be minimised, and
3) an alternative drilling method that prevented borehole collapse would be desirable.

7.3.2 Midwest Trials

Investigation of the horizontal well installations at two landfills in the American Midwest reinforced a number of the conclusions drawn from the first Rainham field trial and provided some additional recommendations. The major points were as follows;

(a) drilling fluid returns (and therefore cuttings) were negligible,
(b) a wireline guidance system could be successfully operated at depths of around 30m,
(c) a large capacity drilling rig (possibly a maxi-rig with more than 1000kN capacity) would be required for the full scale trial at Rainham,
(d) borehole closure led to a number of installation failures,
(e) pilot drilling may have skimmed along, or possibly entered, the clay layer beneath the waste at one of the sites, stressing the importance of accurately determining the position of the site base,

(f) the well screen open area (and its distribution) should be maximised to increase the likelihood of higher permeability pathways in the waste matching up with perforations in the well screen, and

(g) use of a carrier casing may reduce the installation stresses on the screen; thereby allowing the design of the screen to be optimised for fluid extraction.

7.3.3 Second Rainham Field Trial

Based on the experience of these trials the major concern for directional drilling through waste was the partial closure of the borehole in materials that could not support an open borehole nor retain low viscosity fluids that would support it. This led to the decision that the full scale trial at Rainham would proceed using the alternative method of overwashing. Essentially this method involved the drilling of an overwash pipe into the waste following pilot drilling. This pipe would then remain in the waste, retaining the borehole walls, while installation of the screen continued within. In addition to preventing borehole closure, overwashing gave the option of drilling shorter one-ended or 'blind' wells which was an advantage at Rainham due to the topography. Furthermore, if an impassable obstruction had been encountered, the borehole need not have been abandoned as installation of well screen would have been possible up to that point. This is an advantage over back-reaming which requires a two-ended borehole before well screen installation can begin.

However, during operation of the wells it became clear that continuous wells have two important advantages over blind wells. Firstly, blockage of the well (through clogging or damage from waste settlement) may well occur sooner or later and if this blockage is near the single entrance point of a blind well, the performance of the well may be severely impaired. Secondly, evidence from pore pressure reductions suggested that the wells were more effective closer to the outflow point. Pumping from both ends of a continuous well may therefore have a more pronounced effect on pore pressure reductions in the waste. Ideally the overwashing technique should be used to install continuous wells, only reverting to a blind-ended well if an impassible obstruction is encountered.

Pilot drilling during the second Rainham trial encountered only minor problems with one notable incident when the drill bit could not be steered out of the soft silt base. Some interference and loss of accuracy was experienced with the wireline guidance system due to the heterogeneity and metallic content of the waste, but when the strength of the induced magnetic field was increased by both shortening and double looping the wire coil, the guidance system worked adequately. As expected, no drilling fluid (and therefore no cuttings) returned from the boreholes and water was used successfully as a drilling fluid substitute.

Despite some success in the Midwest, the back-reaming technique was considered unreliable. The second Rainham trial demonstrated that overwashing could be successfully used although further trials would be required to confirm that the experiences at Rainham were not unique. The success was essentially due to the limitation of borehole closure as a result of the support provided by the overwash casing. Although the borehole may have partly closed around the overwash casing itself, this was limited owing to the small difference in diameter between the casing and the cutting shoe. As more overwash casing was installed, the increasingly slow progress indicated that friction with the waste was increasing. Had the boreholes been longer or of a larger diameter then the rig would have required greater thrust and torque. Severe wear of the cutting edge on hard materials in the waste was also thought to have contributed to the reduced rate of progress.

The selection of a suitable well screen was a major concern during the design stage. The diameter had to be small enough to run inside the 150mm overwash casing yet large enough to accommodate a submersible pump, so a 100mm diameter screen was chosen. It is not clear whether a larger diameter well would have performed more successfully in terms of leachate and gas yield. However, the flow rate of only 0.04 to 0.1 m^3/day per m^2 of screen, suggests that the waste was the limiting factor and that a larger diameter well would have given no significant advantage, although it may have enabled simultaneous use of the jetting and suction hoses during well development.

It was not possible to install a filter pack around the well screen as is usual with vertical wells. Specialist screens with built in filter packs were considered but were either not robust enough or were prohibitively expensive. As a result the well screen would be in direct contact with the waste, thus making the selection of a suitable slot size even more critical. Due to a combination of the broad distribution of particle sizes and the rather inconclusive recommendations in the literature, it was not clear what the optimal slot size might be. It was therefore decided that each well would be completed with a different slot width (1mm, 2mm and 5mm) in an attempt to make comparisons. Johnson Vee-wire screen was selected for a number of reasons, but principally because the open area was large and evenly distributed (unlike the crude well screen used in the Midwest which had only a small number of widely-spaced holes).

7.3.4 Metallic Tile Field Trial

As well as providing a reassessment of the installation techniques used during the first Rainham trial, the Metallic trial was an experiment to establish whether installations using a low powered rig could be achieved by keeping the well screen diameter small. The first Rainham trial had shown that the 55mm diameter drilling rods passed through the borehole without difficulty, suggesting that if the well screen had been similarly small, then it too would have passed through. This was confirmed by an analysis of the force that would be required to overcome the friction on the drill rods assuming borehole closure occurred. Like the first Rainham trial, a small drilling rig (200kN pullback capacity) was used at Metallic along with 55mm diameter pilot rods. The well screen diameter was 62mm (instead of 150mm) and was successfully drawn through the borehole without difficulty. However, there are limits to the applications of such small diameter wells as submersible pumps and most development tools will not fit inside.

Unlike the Rainham trials, hard materials were encountered during the pilot drilling at Metallic. However, on the third attempt a pilot borehole was completed and although these hard obstructions caused a delay to installation it was thought unlikely that they would prevent the completion of future installations. Perhaps of greater significance were the two vehicle tyres encountered during back-reaming which became caught on the protruding teeth of the reaming tool. This incident reaffirmed that aggressive drilling tools should be avoided as, unlike natural formations, waste contains materials with considerable tensile strength (e.g. wire, fabrics, steel-rope). It is anyway unlikely that cutting and milling of the waste could be achieved with any type of reamer, so drilling tools should be designed to push through the waste rather than cut through it.

Metallic Tile landfill is classified as a hazardous waste site and one aim of the trial was to establish whether wells could be installed by directional drilling without the production of spoil. With the exception of 1.5m^3 of returning drilling fluids, no cuttings were produced, thus avoiding the problems of spoil redisposal.

7.4 Operation of the Horizontal Wells

7.4.1 Leachate Flow

Once the monitoring system had been installed at Rainham, pumping began from the three horizontal wells. After 17 months of pumping at maximum yield, the flow rate from HW1 had reduced from an initial 11m^3/day to 4m^3/day.

The average flow rate from HW2 was 4.84m^3/day although the flows were highly variable, particularly after the well was damaged during development. The flow rate from HW3, after initially high flows in the first month of pumping, declined from 4m^3/day to 1.2m^3/day after 500 days.

Without further trials using alternative well screens it was difficult to evaluate Johnson Vee-wire in comparison to other possible alternatives. Nevertheless the large open area and uniform distribution of Vee-wire proved successful in lowering pore pressures. The only cause for concern with Vee-wire was its low compressive strength, a factor which may have been responsible for the damage sustained to HW2.

Attempts to evaluate the relative performance of the three slot sizes (1mm, 2mm or 5mm) proved inconclusive because of the following differences between each of the three wells.

1) Leachate heads above the horizontal section at the start of pumping were significantly different, ranging from 8.5m (HW1) to 1.5m (HW3).
2) The design of the well screen was such that as the slot width increased, the percentage open area also increased. It would have been more useful to compare different slot widths with a uniform percentage open area.
3) HW1 was possibly blocked at 210m and HW2 was damaged at 36m and possibly blocked at 90m. These blockages may have had a significant affect on flow rates.

4) The gas flow rate, which ranged from 360m^3/day (HW1) to zero (HW2), may also have influenced the leachate flow rate.

In addition to the hydraulic conductivity of the surrounding waste, leachate flow rates were thought to be a function of the following factors; (a) change in leachate head, (b) percentage open area, (c) slot width, (d) gas flow rate, and (e) possible clogging of the well screen.

7.4.2 Gas Flow

Gas flow rates from HW1 and HW3 were high, averaging 360m^3/day and 207m^3/day respectively although it was unclear why gas did not flow from HW2.

Consideration of the solubility of landfill gas concluded that that only a small fraction of the gas volumes recorded could have been dissolved within the extracted leachate before it was drained from the waste. Further analysis suggested that ongoing decomposition of the waste within 66m of the well would be sufficient to produce the gas flow rates recorded. Ongoing decomposition could also explain why gas flow rates did not deteriorate over the duration of the pumping.

HW1 and HW3 produced gas and leachate in surges while HW2 did not produce gas and the flow of leachate was smooth. This suggested that the surging was a product of the gas flow or that the two fluids were interacting to create the surges. The mechanisms of gas flow were thought to involve two discreet stages. The first stage is the gradual accumulation of gas within the waste, which then migrates towards the well driven by the pressure gradient. The second stage involves the two-phase flow of leachate and gas along the well. The gas flow rate was 50 to 85 times greater than the leachate flow rate and although it was not possible to calculate the actual velocity of each fluid phase as it passed along the well (as the distribution of flow areas was unknown), it was clear that the velocity of the gas was much greater than the velocity of the leachate. This is likely to have caused waves to form at the phase boundary which became large enough to fill the pipe so that fluids were released intermittently, a mechanism described as 'slug flow'. Undulations in the profile of the well may also have contributed to the surging through a mechanism described as ‘terrain induced slugging’.

The gas flow rate was found to be inversely correlated with fluctuations in atmospheric pressure. This is consistent with observations by other researchers who found the rate of gas migration from landfill sites to be governed by the difference in gas pressure between the interior of the landfill and external atmospheric pressure. An inverse correlation between leachate flow rate and atmospheric pressure was also found, but only in the two wells which produced gas, suggesting that the correlation was an indirect result of the relationship between gas flow and atmospheric pressure. Why leachate flow should increase with gas flow is not clear but this an interesting observation that requires further study.

The gas venting from HW1 and HW3 was consistently 60% methane and 38% carbon dioxide (plus a host of trace compounds), and was therefore of a high quality in terms of energy generation potential. With a vertical well the screened section is often near the landfill surface which can result in an ingress of air when methane is extracted under suction. Such dilution was avoided in the horizontal wells as the first 50m of each well was completed with plain casing, thus forming an effective seal that prevented the ingress of air into the well screen. Using gas composition and flow rate data, the energy potential of the gas from HW1 and HW3 was calculated. Taking into account efficiency losses in the conversion to electricity, the gas flow rate of 567m^3/day could be expected to generate approximately 52kW.

7.4.3 Pore Pressure Changes

The pore pressure reductions were markedly different in each of the Lower, Middle and Upper waste horizons. The largest reduction was in the Lower Horizon in which the horizontal wells were situated. Indeed, at a distance of 12m to either side of HW1, pore pressures fell a total of 8m (from 13mOD to 5mOD, with the well at 3.6mOD). The lateral extent of the well's influence on pore pressures in the Lower Horizon was approximately 50 to 70m either side of the well. However, pore pressure reductions in the Upper Horizon were as little as 0.5m only 5.5m directly above the well. The anisotropic permeability resulting from analysis of these data was consistent with the findings of Hudson *et al* (1999) who carried out tests on a large scale compression cell and reported that the horizontal hydraulic conductivity of waste may be 2 to 5 times greater than the vertical depending on the applied vertical effective stress. New research (2002) by Hudson and Beaven suggests that this ratio may be even higher, up to 10 times greater.

The markedly different response of each waste horizon highlighted the absolute need for piezometers that monitor discrete layers of waste. At present, landfill operators rely principally on leachate level measurements assessed using observation wells that are screened over their entire length.

One of the driving factors behind this research was the need to lower the pressure head in the waste to reduce the likelihood of leachate breaking through the underlying containment layers of silt and clay. The horizontal wells have targeted pore pressures in the Lower Horizon and caused a significant lowering of the pressure acting on the base. Although the pore pressures in the Upper Horizons remain high, further extraction of leachate from these horizons would have little if any beneficial effect in terms of preventing outward leachate migration.

Analysis of the fluctuations in pore pressure and barometric pressure revealed that the piezometers had an average barometric efficiency of 69.4%, suggesting that the waste could be regarded as confined to some extent by the layers of waste above. This has considerable implications for landfill operators who may overlook barometric changes when analysing leachate level readings. In a worst case scenario, when barometric pressure has changed by 60mbar since the last reading and the piezometer has an efficiency of 100%, the leachate level may appear to have risen (or fallen) by 0.6m. This may prompt unnecessary action such as the engagement of pumps, or result in no action when action is required.

After 500 days of pumping the pore pressures in the waste surrounding HW1 and HW3 had reached equilibrium yet the leachate flow rates continued to fall. This suggested that the hydraulic conductivity was declining, possibly as a result of clogging of the well screen. The horizontal wells in Cardiff Bay also experienced a gradual decline in flow rate which was accompanied by a gradual rise in the surrounding pore water pressures, suggesting that these wells were working less effectively, also as a possible result of clogging.

There was evidence to suggest that the Rainham wells may have caused a significant redistribution of pore pressures along their length before pumping commenced. Piezometers were installed after the horizontal wells to enable accurate positioning. However, the prior installation of around five piezometers across the planned route of the well would have provided invaluable information on the pore pressure distribution prior to well installation.

7.5 Recommendations for Future Research

While this research has revealed a great deal about the installation, operation and performance of horizontal wells installed in waste, it also represents the first attempt to analyse scientifically the application of the technology in a landfill. There is therefore much work still to be done to gain a complete understanding of the use of horizontal wells for leachate and gas control. The following section provides a number of recommendations for further investigation.

With the exception of the wells installed in the Midwest, the wells installed for this research project are thought to be the first of their kind and it is possible that some of the observations made were unique to these wells. The installation and operation of additional wells, particularly at different landfill sites, is required to support the observations reported herein. Building on the findings of this research, additional wells could incorporate improvements to both the method of installation and well design.

In October 1999 two further horizontal wells were being installed at Livingston County landfill in the Midwest. Enquires in February 2002 revealed that due to contractual problems, commissioning of the wells had been severely delayed. It is now (December 2002) probable that the wells are in use and are providing valuable data on well performance. With the international expansion of the directional drilling industry over the last decade, it is a distinct possibility that other horizontal wells have been installed or attempted at other landfill sites. A review of the wells in the Midwest together with a search for other examples is therefore recommended.

The wells at Rainham were installed deep within the saturated zone yet still produced surprising volumes of gas. Although this was explained in terms of ongoing gas generation in the lower horizons, the presence of all this gas raises questions as to whether the waste can truly be regarded as saturated. Evidence was presented for (a) pore pressure confinement, (b) distinct variations in piezometric level between horizons, and (c) the presence of large quantities of gas in the saturated zone. In light of this, the concept of a singular phreatic surface or 'leachate level' marking a line of zero pore pressure and a simple demarcation between unsaturated and saturated layers is of little meaning or use. In this respect, an investigation of the way in which landfill operators monitor pore pressures is required to ensure such data are being interpreted and used correctly by both operators and regulators.

While the interactions between gas and leachate clearly play an important role in the study of fluid movement in landfills, the horizontal wells did not provide a controlled environment in which these interactions could be assessed with confidence. Research which investigates and analyses the relationship between gas and leachate in laboratory scale experiments is required, indeed, such research is ongoing at Southampton University and it is hoped that the findings of this work will be applied to subsequent field investigations.

The single horizontal well installed at Metallic Tile landfill was intended both as a reassessment of the installation technique used in the first Rainham trial and as a study of the gas collection performance of horizontal wells at shallow, unsaturated depths. Due in part to the intensive operation and assessment of the Rainham wells, the performance of the Metallic Tile well was never thoroughly assessed and the suitability of horizontal wells for such applications remains unresolved. This is an area requiring investigation and the Metallic Tile well may represent a good starting point.

Gas was passively vented from the Rainham wells for the majority of the time they were in use, although toward the end of the project, HW1 was connected to the existing site gas extraction system and gas flow rates increased significantly. However, with (a) only a limited capacity to obtain flow rate measurements and (b) the gas extraction contractor making unannounced adjustments to the system, a controlled assessment of the effect of suction was not possible. Evidence has been presented suggesting there may have been a positive correlation between gas and leachate flow rates and it remains a possibility that by increasing gas flow using suction, the rate of leachate flow may also increase. Further investigation requires the independent control of gas suction from each of the wells without connection to the main site extraction system over which no control can be exercised. Gas quality monitoring during such an investigation should provide valuable information as to whether the quality remains high during prolonged suction as it did during passive venting.

The effect of the Rainham wells on the pore pressures and settlement of the surrounding waste may be modelled using finite element analysis. By modifying element properties such as waste density and the change of permeability with effective stress, it may be possible to match the model to field observations. This may enable quantification of important parameters such as the k_h to k_v ratio. Once this model has been constructed, tested and revised it may be possible to use it as a tool for predicting the likely effect of horizontal and even vertical well installations for future projects.

7.6 Concluding Remarks

The research had a broad objective; to study the installation and performance of directionally drilled horizontal wells for leachate and gas control. It was demonstrated that a modified directional drilling technique could be used successfully to install horizontal wells in heterogeneous waste materials. Building on the findings of this research operators wishing to perform similar installations can now approach the task with a great deal more confidence. Furthermore, operators considering the use of horizontal wells may use this research as an indication of how such wells are likely to perform. The research has also raised a number of questions concerning the interpretation of pore pressure data and the current understanding of fluid movement within landfills. This provides an interesting starting point for future work.

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APPENDIX A

Parameters to be Logged During Site Investigation

1) Location

General information identifying where the waste sample originated, e.g. site name, borehole number, surface elevation, coordinates and date of drilling.

2) Depth

Depth of the borehole before and after the extraction of each waste core. Subsequent conversion to *metres ordnance datum* (mOD) may allow layers such as temporary caps and perched leachate to be identified between boreholes.

3) Temperature

A temperature probe should be inserted into the base of the core immediately upon extraction. Temperature may be linked to the stage of decomposition and gas generation rates. When possible, further temperature readings should be taken from different areas of the same waste core to provide a more reliable average.

4) Moisture Content

This should be assessed using two methods, firstly by observation and secondly by oven drying in the laboratory. To provide a structure for the qualitative manual observations the following five point scale should be used:

i. DRY
ii. MOIST (suggestive of moisture)
iii. DAMP
iv. WET (within field capacity)
v. SATURATED (beyond field capacity)

Samples of the waste are to be taken to a laboratory for quantitative analysis by oven drying. However, the waste is likely to contain individual particles too large for oven drying and only the matrix material should be used for the subsample.

5) Density

Each core of waste should be extruded from the barrel auger into an appropriate container (such as a $1m^3$ building materials sack). The container should then be weighed on platform scales with an area of an appropriate size. For 200mm diameter cores up to 3m in length an area of at least $1m^2$ will be required. The mass, combined with depth and volume data (calculated from the core diameter), should then be used to calculate the bulk density of the waste. Combining the wet density value with the moisture content will enable calculation of the dry density.

6) Age

From legible newspapers found in many waste samples it should be possible to determine an age profile of the waste. Other sources such as sell-by-dates on food packaging can also be used for this purpose.

7) Water Strikes and Water (Leachate) Level

The depths at which free leachate is encountered should be recorded. If the leachate is perched the thickness of the perched layer should also be recorded. Following installation of the permanent casing, the depth to leachate should be measured.

8) Waste Description

The recovered waste can be regarded as having two principal components

i. the *Main Contents*, and
ii. the *Matrix*.

The description of the main contents should include a list of the material types found (e.g. paper, plastic, fabric, wire, wood, concrete) together with a note if any particular material is found in large quantities. The matrix has a generally smaller particle size and fills the spaces around the main contents. The matrix materials are likely to be more difficult to identify. A significant proportion of the matrix may be comprised of the semi-decomposed remains of the main contents. Other materials may include soil, silt and clay (possibly from daily cover). The matrix should be described according to its properties rather than its composition, e.g. soft/firm/stiff, fibrous/granular, spongy/plastic/crumbly. A definition of these terms is given in BS 5930 (1981). The volume of the matrix as a percentage of the total should be estimated.

9) Degree of Decomposition

As with moisture content, the degree of decomposition is to be assessed manually using a five point scale:

i. NOT DECOMPOSED (showing no evidence of decay)
ii. PARTLY DECOMPOSED (clear signs of decay)
iii. SEMI-DECOMPOSED (most readily degradable materials no longer present)
iv. WELL DECOMPOSED (only most resistant degradable materials are present)
v. FULLY DECOMPOSED (only non-degradable materials remain identifiable)

10) Photographs

A photographic log of the waste cores should be made. An identity card stating the borehole and sample number should be included in each photograph to ensure they do not become confused. The card will also provide scale for the photograph.

APPENDIX B

Landfill Wastes Borehole Logging Sheet

Site Code:________ BH Number:_______ Core Number:________ Date:_________ Time:__________

Depth to base BEFORE drilling of core (mBGL):_________ mOD:_________
Depth to base AFTER extraction of core (mBGL):_________ mOD:_________
Calculated depth of core (m):__________ ID of barrel:_________
Calculated volume of core (m):__________

Gas Composition,
Methane (%):_______
CO_2 (%):_______
Oxygen (%):_______
H_2S (%):_______
Pressure (mbars):_______

Temperature inside barrel (°C):_________
Subsequent temperature readings (°C):_________ :_________

Total Mass on Scales (kg):_________
Mass of Container. (kg):_________
Calculated Mass of Waste Core (kg):_________

Moisture Content, Sample Taken (Y/N):________ Sample reference number:____________
Observation ________ (Dry - 1, Moist - 2, Damp - 3, Wet - 4, Saturated - 5).

Water Strike (Y/N):_________ Depth Struck (mBGL):_________
Perched Leachate (Y/?/N):________ Thickness of Perched Layer (m):_________

Approximate Date of Waste Emplacement (month & year):_____/_____
Source of date:__________________________

Degree of Decomposition:________
(1 Not Decomposed, 2 Partly Decomposed, 3 Semi-Decomposed, 4 Well Decomposed, 5 Fully Decomposed)

Photograph taken (Y/N):_____ Photo reference number:_______________

Main Contents (✓), (Note if in unusual quantities)

Plastics		Paper	
Wood		Other Vegetation	
Glass		Rubber	
Fabrics/Carpets		Brick/Hardcore	
Metal		Other	

If other, specify:

Nature of Matrix (✓),

1) Quantity; <25% [ ], 25-50% [ ], 50-75% [ ], >75% [ ]

2) Soil [ ], Soil forming material [ ], Clay [ ], Silt [ ], Sand [ ], Gravel [ ], Other (specify):

3) Soft [ ], Firm [ ],

4) Malleable/Shapeable [ ], Crumbly [ ], Fibrous [ ], Sticky [ ]

5) Black [ ], Dark Brown [ ], Brown [ ], Light Brown [ ], Dark Grey [ ], Grey [ ], Light Grey [ ], Cream [ ], Other:

Any other notable finds:

APPENDIX C

The reference for the following paper is:

Board, M., Painter, A., Cox, S., Cole, D.J. and Powrie, W. (2001) The Description and Logging of Samples Arising from Boreholes Drilled in Landfilled Wastes. *Proc. Sardinia 2001, Eighth Int. Waste Management and Landfill Symposium,* S. Margherita di Pula, Cagliari, Italy, Vol.4, pp447-456.

THE DESCRIPTION AND LOGGING OF SAMPLES ARISING FROM BOREHOLES DRILLED IN LANDFILLED WASTES

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SUMMARY: The description of a rock or soil sample written in the field on the basis of arisings recovered during a geotechnical site investigation is the first step in understanding the physical characteristics of a geomaterial. Similarly, the drilling of a borehole is often the only practical method for sampling the complete thickness of a landfilled waste formation. Observations made during drilling such as wetness, degree of degradation, and age and composition of waste materials can indicate waste stratigraphy, be used to augment the laboratory testing of samples and provide vital information for the design of engineering works in and on landfills. However there is currently no definitive standard or protocol that provides a framework for the description and classification of degraded waste samples arising from landfills. This paper presents a review of previous work, proposes an adaptable framework for the detailed description of waste samples, provides an aide memoir to assist the recording of waste descriptions and observations made during drilling, and gives examples of the benefits of accurate, consistent waste descriptions.

1. INTRODUCTION

The description of a rock or soil sample written in the field during a site investigation is the first step in understanding the physical characteristics of a geomaterial. The procedure for describing a sample may be set out in a standard such as BS 5930 *Code of practice for site investigations* (BSI, 1999) or some other established protocol. Although substantial research has been carried out on the characterisation of individual waste streams disposed of to landfill, there is no definitive UK standard or protocol that provides either

a) a framework for the accurate and consistent description of degraded, co-mixed waste samples arising during the site investigation of landfills, or

b) a materials classification for degraded, co-mixed waste materials.

It is apparent from existing literature (e.g. Watts & Charles, 1990) and discussions with UK waste management companies that the lack of a protocol to describe waste samples has led to the use of terms such as "municipal solid waste " to indicate the general composition and physical properties of a waste sample; or even an entire waste formation within a landfill. Simple descriptions of waste sample components are frequently given in publications without additional data concerning their physical properties, making it difficult if not impossible to compare or relate experimental results to other sites (Grisolia *et al*, 1995).

The construction of systems to manage leachate and gas production on operational and closed landfills, and research carried out to improve the design and installation of such systems, has highlighted the need to determine the physical properties of degraded waste materials encountered during engineering works. The drilling of boreholes is often the only practical method for sampling the complete thickness of a waste formation. Drilling boreholes in landfills is expensive, potentially hazardous, difficult due to obstructions and generally only a small volume of heterogeneous material can be recovered from a core for further laboratory testing (Grisolia *et al*, 1995; Read 1998).

It is therefore important that as much data as possible are recorded during the intrusive phase of a landfill site investigation or during the installation of a gas or leachate well. Observations of materials arising during drilling such as wetness, degree of degradation, and the age and composition of waste materials encountered can indicate waste stratigraphy, augment laboratory test data from samples and provide vital information for enhancing the design of engineering works in and on landfills. Common UK practice is for the drilling contractor to produce a simple log sheet recording minimal information such as the depth of the borehole, simple descriptions of materials encountered, depths of leachate strikes and if applicable brief well or instrumentation installation details.

Clearly there is a cost associated with the detailed logging of waste during a site investigation or engineering works, including the employment of suitably qualified personnel and possibly the increased time taken to drill each borehole. However the data gained by the accurate description and logging of a waste formation can be of considerable financial benefit. For example, drilling logs in combination with other investigation techniques such as geophysical resistivity tomography have been used to identify the location, shape and composition of unsaturated zones within a waste formation. Recirculation of leachate into the dry zones identified with the aid of the borehole logs proved to be of considerable financial benefit to a waste management company in reducing leachate treatment costs, promoting more rapid stabilisation of the landfill and indicating possible gas reservoirs.

This paper presents a framework for the detailed description of waste samples recovered during drilling and for the recording of other important data generated before, during and on completion of drilling. An aide memoir to assist the accurate recording of waste descriptions and observations made during drilling is proposed. The framework and aide memoir have been developed during landfill site investigations carried out by both research and industrial practitioners and could if desired be incorporated into a waste management company's quality assurance system.

2. EXISTING STANDARDS, PROTOCOLS AND CLASSIFICATIONS

The terms characterisation, description, classification, and composition are commonly used in waste management literature to describe the process of identifying and categorising waste materials into groups. These terms are used in an apparently interchangeable manner and may also have different connotations according to individual disciplines within the waste management community. For the purposes of this paper the terms description and classification are used as defined in BS 5930:1999.

A **description** gives detailed information on the colour and nature (plasticity and particle characteristics) of a soil, as well as on the state (strength condition) and structure (bedding, lamination) in which it occurs in a sample, borehole or exposure. Few if any soils have identical descriptions (BSI, 1999).

For engineering interpretation purposes, a **classification** is the grouping of a soil on the basis of the geological origin of the stratum, some engineering property or properties of the strata, or any of a large number of combinations of geological or engineering parameters. Such classifications are usually adopted to provide a framework for description and assessment of the ground conditions at a site. The most common type of classification places a soil in a group based on the grading and plasticity of disturbed samples. A classification can therefore provide information as to how a disturbed soil may behave (BSI, 1999).

The need for the accurate description and classification of waste materials has arisen as a result of the increasing amount of engineering work occurring on operational and closed landfills. To design engineering works that are safe and economical it is vital to understand the physical properties of the ground that will be encountered. As stated previously, an important first step is to describe the materials as accurately as possible, followed by classification according to defined physical or chemical properties. Considerably more research has been carried out on fresh waste classification than on degraded waste description.

3. EXAMPLES OF WASTE DESCRIPTIONS AND CLASSIFICATION.

A number of papers indicate the current lack of a suitable description protocol and the requirement for a standard classification method. Boland *et al*, (1998) reported the heterogeneity of waste samples arising from five boreholes drilled in a UK landfill that had received predominantly municipal solid waste (MSW). Of 102 samples collected from the boreholes, no two samples were considered the same. However no detailed descriptions were given and the method of classification was not clearly defined, although the samples were reported to have been sorted into 24 materials categories. There are several reported examples of waste samples from boreholes in landfills being bagged and then analysed under laboratory conditions (Gabar & Valero 1995, Townsend *et al*, 1996 and Zornberg *et al*, 1999). Gabar and Valero (1995) provided a limited description of the recovered samples with observations on perched leachate tables and the degree of waste degradation made during drilling. Zornberg *et al* (1999) reported borehole logs recording perched leachate tables and classified waste samples as soil, household waste, green waste and inert waste.

4. EXISTING DESCRIPTION STANDARDS AND PROTOCOLS FOR WASTE MATERIALS

Procedures for describing a waste sample are set out in standards such as BS 5930 *Code of practice for site investigations* (BSI, 1999), and other established protocols such as the *Geotechnics of Landfill Design and Remedial Works Technical Recommendations – GLR* (GLR, 1993).

BS 5930 (BSI, 1999) does briefly cover the investigation of 'waste tips' and the description of made ground samples. This standard recommends the use of a soils description protocol for samples of made ground with an emphasis on recording the types of waste materials present. Although this approach may be adequate for made ground with a high proportion of natural material, the use of this standard to describe a landfill waste sample may result in a list of constituent materials, providing little indication of the likely properties of the material. However the standard does recognise the importance of detailed descriptions due to the heterogeneous nature of waste materials, and the need for recording additional characteristics of waste samples such as the degree of degradation, age indicators and odours.

The GLR recommends that wastes should be described in such a manner that their mechanical behaviour can be defined, where wastes should be divided into two types, "soil-like wastes" and "other waste". **Soil-like waste** is defined as granular waste, for which conventional soil mechanics theory is applicable. It is stated that soil-like wastes can be compared, in respect of composition and geotechnical behaviour, with soil groups as defined in soil classifications such as BS 1377 (BSI, 1990). **Other waste** is defined as non-sorted municipal waste, etc (GLR, 1993).

The GLR recommendations for the description and classification of waste materials are not particularly helpful. No guidance is given as to the defining properties of a soil-like waste or other waste. In practice it is unlikely that a sample of waste from a borehole in a landfill will consist solely of either a soil-like waste or non-sorted municipal waste; the majority of samples are a mixture of waste and soil materials. The GLR recommendation that a soil-like waste containing an uncertain proportion of MSW or other waste will have geotechnical properties comparable with a standard soils classification seems optimistic in that in some respects the applicability of soil mechanics principles to household waste is currently unclear (Powrie *et al*, 1999). In any case only appropriate in situ or laboratory mechanical testing can determine the mechanical behaviour of a waste material (Landva & Clark 1990), so that at best the description of a material can only give an indication of its nature.

5. EXISTING CLASSIFICATION SYSTEMS FOR WASTE MATERIALS.

Although this paper is concerned with the description of degraded waste materials, any description method must be considered in conjunction with an associated classification system. Current classification systems for waste can be divided into two categories:

a) classification of wastes at the site of production or following mixing at collection points or waste transfer stations and as received at a landfill;

b) classification of a waste according to its geotechnical properties.

The classification of waste arising at the site of production or before disposal is considered crucial for the planning, design and operation of waste management systems (Savage & Diaz, 1997). Work carried out to classify the sources of waste arising and entering UK landfills has shown that waste is often unrecognisably mixed at transfer stations before entering landfills (DoE, 1993), where further mixing will occur during deposition. Mechanical compaction, settlement, biological degradation and chemical reactions lead to further mixing and disintegration of materials within a waste formation. It therefore appears inappropriate to describe or classify a waste sample from a borehole or trial pit according to the origin of its components. However it may be appropriate to describe samples arising from mono-fill landfills according to the origin of the waste material e.g. Pulverised Fuel Ash. Geotechnical classifications for waste have been proposed by Landva & Clark (1990) and Grisolia, *et al*, (1995). Both of them divide waste materials into categories according to the potential strength or degree of degradation of a material. Landva & Clark, 1990 suggest two major categories, Organic and Inorganic materials, each sub-divided in to two further groups.

However there is no discussion as to how this classification system could be applied in practice or how it could be combined with a description protocol.

Grisolia, *et al*, (1995) suggest three classes of materials. Inert stable elements (A), highly deformable elements (B) and readily biodegradable elements (C). The classification system was used as a tool for analysing data on the composition of fresh waste entering a landfill using ternary diagrams. The paper also reports the potential of this method to correlate waste sample composition with its likely geotechnical properties. Although the system was developed to classify fresh waste samples it could be adapted for the classification of degraded wastes. It is considered that with further work this method could be used to indicate the mechanical behaviour of a waste material from its description. This classification has therefore been incorporated into the suggested description method.

6. FRAMEWORK DESIGNED FOR THE DESCRIPTION OF DEGRADED CO-MIXED WASTE SAMPLES

Having recognised the importance of recording data during the drilling of landfill formations a framework to aid the accurate recording of data was drawn up. The framework is designed as a template that can be adapted according to the task being undertaken e.g. landfill site investigation or drilling of a leachate or gas well. It is suggested that the data are recorded daily in a notebook by the nominated engineer overseeing the work. An example of the description template is provided in Figure 1. Relevant borehole data can be divided into four main categories.

Category 1. Information on the purpose and location of the borehole.

These data set out the location and purpose of the borehole. Details include the drilling method to be used and the health and safety precautions to be taken. The data should be collated before drilling commences.

Category 2. Depth, description and composition of waste sample.

These data detail the depth from which a sample is taken, a qualitative description of the sample including indicators of its strength, degree of decomposition and compaction, moisture content, colour and the presence of landfill gas. It is suggested that the composition of a sample is described according to the materials present, and the particle sizes and proportion of each. This allows the material to be given a simple classification based on the Grisolia, *et al* (1995) system. Additional data such as waste density and age, smells and any unusual observations should also be included. Samples may be retained for further physical and chemical testing e.g. particle size distribution test or methane potential test. The identity of the sample and the tests to be undertaken should be recorded here.

Category 3. Depth of leachate strikes and leachate properties.

This category includes the depths at which leachate is encountered in the landfill. Regular dipping during drilling together with the description of waste samples will aid the identification of perched leachate within a landfill. Details of leachate colour, smell and temperature may also help indicate the degree of waste degradation.

Category 4. Information on the completed borehole

This category is used to record the details of the completed installation including its depth (m) and the level (mAOD) of the finished wellhead, and the methods used to develop the well. These data are particularly important as the initial design recorded in Category 1 may have been modified as a result of the conditions encountered during drilling. The results of any testing (e.g. pumping tests and gas composition) conducted on the installation on completion should also be recorded and are vital for quantifying any future deterioration in well performance. These data may only become available some time after drilling has been completed.

7. EXAMPLE APPLICATIONS OF LOGGING BOREHOLES IN LANDFILLED WASTES

Gas and leachate wells in landfills are often located and installed according to a standard plan and design drawn up prior to installation, with no previous site investigation. This practice takes no account of the material properties encountered during drilling. The logging of waste during drilling allows the design of a well to be modified according to the waste materials encountered, thus improving the performance of the installation. Well location can be assisted using geophysical resistivity tomography to map the resistivity values of waste within a landfill site. Resistivity profiles along pre-selected traverse lines are carried out to produce 2 dimensional cross sections of the waste formation (Figure 2), enabling zones of dry, partly saturated and fully saturated waste to be identified following calibration between resistivity values and samples recovered from boreholes. The cost of a commercial resistivity survey for a 600m cross section is equivalent to the cost of a single well installation, so significant savings may be gained from the use of resistivity surveys to site wells in the optimum location. This combined technique has also proved extremely useful in highlighting unsaturated waste for leachate recirculation and indicating true leachate levels within landfills.

8. CONCLUSION

There is an increasing requirement for engineering works on operational and closed landfills to manage leachate and gas production, and for developments on stabilised landfills. Geotechnical engineers have established standards for the description and classification of rocks and soils but there are no such standards for landfilled materials. The requirement for a degraded, co-mixed waste material classification for use by industry, researchers and environmental enforcement agencies is apparent. The framework presented in this paper for the accurate description of a sample of waste is considered to have potential financial benefits for the waste management industry and provide a first step towards the development of a degraded waste material classification. Further work is required to relate the description and classification of degraded waste materials to their mechanical properties.

Aide memoire for the description of waste samples recovered during drilling works on landfill sites. Side A.

Box 1. Start of each borehole.

Site name and area
Grid reference
Borehole number
Ground level (MAOD)
Health& Safety precautions
Purpose of borehole
Depth to be drilled
Drilling method
Name of drilling contractor

Box 2. Start of each day

Date
Weather conditions
Atmospheric pressure
Leachate level start of day
Time drilling commenced
Other comments

Box 3. Sample recovery

Depth of sample From.....To	Temperature of waste	Sample No
	Photograph No	Samplling method
Time of recovery	Sample type waste, gas, leachate ,	Tests to be carried out

Box 4. Description of sample

Strength	Degree of decomposition	Observed water content	Odours / Gases	Colour
			Visible Steam / vapour	
Slurry	Not decomposed	Dry	Description and strength of smell	
Soft	Moderately decomposed	Damp	Odorless	
Firm	Well decomposed	Wet	Very Strong Odour	
Stiff	Fully decomposed	Saturated		

Figure 1. Example of a description template for drilling of a leachate extraction well in a landfill formation.

Aide memoire for the description of waste samples recovered during drilling works on landfill sites. Side B.

Box 5. Sample composition

Material Class	Generic materials	Specific Materials	material %	particle sizes mm
A Hard inert	Soil			
	Metal	food Cans		
		industrial		
	Glass			
	Concrete			
B Highly deformable	Paper	newspaper		
		cardboard		
	Plastic	plastic bags		
		bottles		
		packaging		
	Fabric			
	Wood			
C Soft degradable	Putresibles	food		
		nappies		
	Vegetation			

Box 6. Additional observations

Age of waste

Density of waste

Degree of waste compaction

Features of the waste formation

Gas volumes measured or observed

Box 7. Leachate strike

Colour of Leachate

Temperature of leachate

Depth leachate encountered

Rate of leachate rise or fall (if any)

Box 8. Finish of day

Leachate level

Time drilling stopped

Other comments

Box 9 Following completion of borehole

Time taken to complete installation

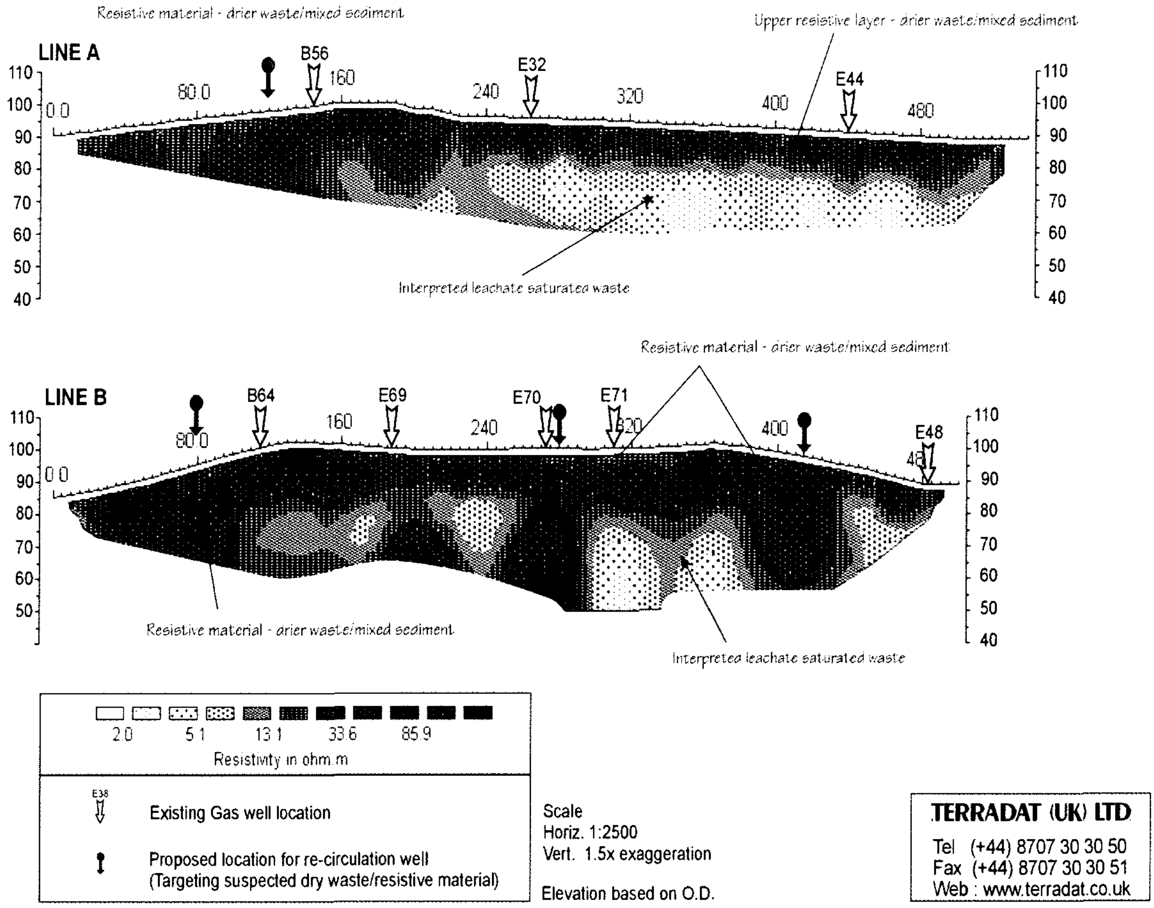
Design details of well installation

Height of well head (mAOD)

Development method used

Tests carried out & results

Figure 1. continued.



Resistivity Tomography Survey to locate areas of saturated/unsaturated waste at a landfill site in the UK

Figure 2. Resistivity tomography survey used in conjunction with borehole sampling to locate areas of saturated and unsaturated waste at a landfill site in the UK.

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APPENDIX D

Piezometer Installations

Ref No.	Well/Horizon Monitored	Cross Section Number	Distance to Well (m)	Direction from Well	Response Zone (m OD)	Tube Top (m OD)	Grid Ref.	
							E	N
H03	HW1/Lower	-	5.3	E	2.9	19	552412	179372
H04	HW1/Lower	-	7.4	E	2.8	22.5	552403	179390
H05	HW1/Lower	2	0.8	E	1.6	25.6	552383	179413
H06	HW1/Lower	2	-3.4	W	3	28.9	552367	179431
H08	HW1/Mid/Upp	1	-14	W	8	29.3	552326	179478
HW1B	Middle	1	-4.2	W	6	31.4	552332	179480
HW1C	Middle	1	-12.6	W	5.7	30.8	552324	179483
HW1D	Middle	1	-25.3	W	5.7	30.9	552314	179476
HW1E	Upper	1	12	E	10	31.6	552348	179492
HW1F	Lower	1	12	E	3.5	31.7	552348	179492
HW1G	Upper	1	-0.4	W	9.4	31.4	552337	179486
HW1H	Middle	1	-0.4	W	6.3	31.5	552337	179486
HW1I	Silt	1	-18.9	W	1.7	32	552336	179452
HW1J	Upper	1	-4.4	W	9.4	31.3	552334	179484
HW1K	Lower	1	-4.4	W	3.5	31.5	552334	179484
HW1L	Upper	1	-12.1	W	9.2	30.1	552327	179480
HW1M	Lower	1	-12.1	W	3.4	30.4	552327	179480
HW1N	Upper	1	-25	W	9	31.1	552316	179473
HW1O	Lower	1	6.9	E	4.3	33	552357	179467
HW1P	Lower	1	40.7	E	3.5	32.4	552387	179482
HW1Q	Lower	1	-25	W	3.2	31.3	552316	179473
HW1T	Upper	2	11.9	E	9.5	25.7	552389	179423
HW1U	Lower	2	11.9	E	3.4	25.7	552389	179423
HW1V	Upper	2	1.1	E	9	25.7	552380	179417
HW1W	Middle	2	1.1	E	5.9	25.7	552380	179417
HW1X	Upper	2	-12.2	W	9	26.8	552369	179411
HW1Y	Lower	2	-12.2	W	3.6	26.9	552369	179411
HW1Z_1	Upper	2	-24.3	W	8.9	27.2	552358	179404
HW1Z_2	Lower	2	-24.3	W	3.4	27.3	552358	179404
240	HW1 / All	-	-60.5	W	Full	22.2	552355	179340
243	HW1 / All	-	25.6	E	Full	24.8	552411	179413
358	HW1 / All	2	-99.1	W	Full	30.5	552295	179364
368	HW1 / All	1	-84.4	W	Full	33.1	552284	179411
HW2B	Upper	-	20.5	E	9.4	26.4	552015	179202
HW2C	Lower	-	20.5	E	3.8	26.8	552015	179202
HW2D	Upper	-	20.5	E	9.3	26.9	552011	179207
HW2E	Lower	-	20.5	E	3.9	27	552011	179207
HW2F	Upper	-	20.3	E	9.4	27.1	552006	179216
HW2G	Lower	-	20.3	E	4	27.2	552006	179216
235	HW2 / All	-	-0.2	W	Full	21.8	552046	179105
308	HW2 / All	-	-0.2	W	Full	19	552062	179075
312	HW2 / All	-	14.7	E	Full	23.8	552050	179122
322	HW2 / All	-	22.8	E	Full	27.6	552038	179167
332	HW2 / All	-	31.7	E	Full	29.7	552024	179199
341	HW2 / All	-	-3.7	W	Full	26.9	551977	179218

Ref No.	Well/Horizon Monitored	Cross Section Number	Distance to Well (m)	Direction from Well	Response Zone (m OD)	Tube Top (m OD)	Grid Ref.	
							E	N
HW3B	Middle	3	4.3	NE	6.6	24	551940	179287
HW3C	Middle	3	12	NE	6.8	24.8	551945	179293
HW3D	Middle	3	24.8	NE	6.4	26.4	551951	179304
HW3E	Middle	3	-12	SW	7	22.1	551928	179275
HW3F	Lower	3	-12	SW	4.3	22.5	551928	179275
HW3G	Upper	3	0.3	NE	9.6	23.2	551934	179285
HW3H	Middle	3	0.3	NE	6.6	23.4	551934	179285
HW3I	Upper	3	4	NE	9.4	23.6	551937	179288
HW3J	Lower	3	4	NE	4.1	23.9	551937	179288
HW3K	Upper	3	12.2	NE	9.1	25	551941	179296
HW3L	Lower	3	12.2	NE	4.7	25	551941	179296
HW3M	Upper	3	25.3	NE	9.5	26.7	551948	179306
HW3N	Lower	3	25.3	NE	4.1	26.8	551948	179306
HW3R	Upper	-	12.2	NE	9.7	21.5	551888	179331
HW3S	Lower	-	12.2	NE	4.3	21.5	551888	179331
351	HW3 / All	-	1.7	NE	Full	27.1	551961	179271

Magnet Extensometer Installations

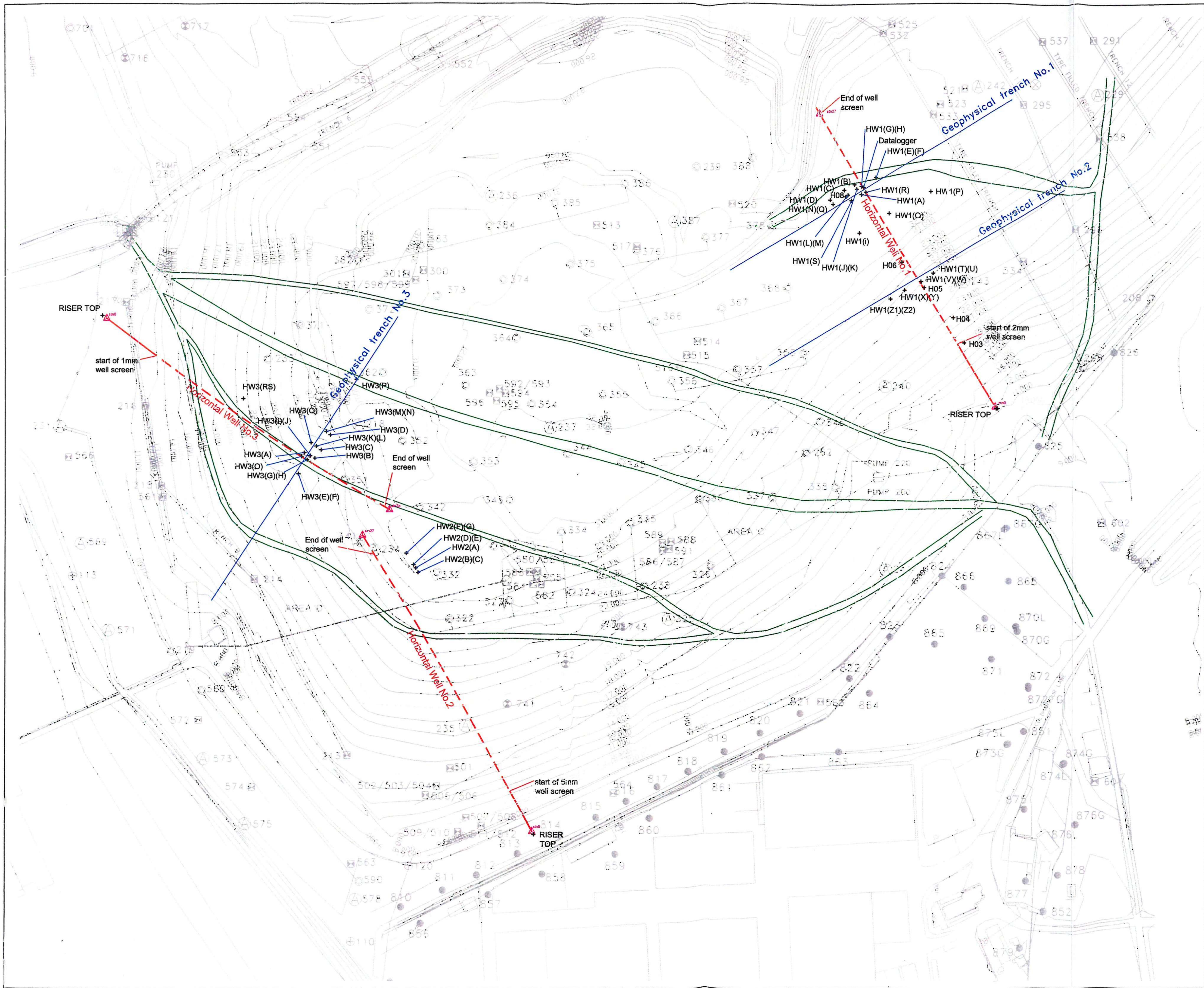
Ref No.	Well Monitored	Distance to Well (m)	Direction from Well	Tube Top (m OD)	Ground Level (m OD)	Elevation of Magnets (m OD)							Grid ref.	
						1st	2nd	3rd	4th	5th	6th	7th	E	N
HW1R	HW1	0.5	E	31.5	31.3	25	19.6	15.5	12.5	9.3	6.4	-	552341	179482
HW1S	HW1	-11.6	W	31.6	31.1	25	17.7	15.2	12.3	9.3	6.3	3.1	552330	179476
HW3O	HW3	0.2	NE	23.8	23.5	18.6	13.7	10.8	7.7	-	-	-	551930	179288
HW3Q	HW3	12.5	NE	25.4	25.1	17.6	14.6	9.5	6.6	3.7	-	-	551937	179298

APPENDIX E

RAINHAM LANDFILL

PHASE 2

SITE PLAN



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KEY

- As Built - Horizontal Well Plain Casing
- As built - Horizontal Well Screen/Slotted
- Geophysical Investigation Lines
- Haul Road @ 9m AOD (1979-1988)
- Monitoring Points

Rev.	Details	Int.	Date

CLEANAWAY

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LEIGH-ON-SEA, ESSEX, SS9 4EL
TEL. BRENTWOOD (01277) 234567
FAX No. (01277) 723508

Location
RAINHAM, ESSEX

Project
LEACHATE DRAINAGE

Title
AS BUILT HORIZONTAL WELLS & MONITORING POINTS -PHASE 2

	Initials	Date	Scale:
Drawn By	BDS	20JUL00	1:1250
Checked By	SC	01AUG00	
Approved By	DJC	01AUG00	ORIGINAL SHEET SIZE : A1
Project No. JOB 817			Drg.No. RAI/LEA/497

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