TRL Limited

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A STUDY OF WATER MOVEMENT IN ROAD PAVEMENTS

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Prepared for: Transport Research Foundation

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Executive summary

There is very little knowledge of the volumes of water that move into and out of road pavement layers. The mechanical properties of unbound materials such as strength and stiffness vary dramatically with changes in moisture content, so knowledge of water movements is important. It is also important to know the fluxes of water so that an accurate estimate can be made of the leaching of contaminants into groundwater or surface water. This is particularly important where recycled or secondary aggregates, which may contain high concentrations of metals and other contaminants, are used in unbound form in the road. Water movement in roads was identified as an important issue requiring further investigation by three collaborative European research projects in the late 1990s, ALT-MAT, POLMIT and COURAGE.

In response to this problem, the Transport Research Foundation (TRF) commissioned TRL to undertake a study into water movement in roads. The study involved a critical review of the literature on water infiltration into and movement beneath roads, followed up by field and laboratory studies to estimate the water movements in an existing section of trial road at TRL with a concrete pavement, granular sub-base and clay subgrade. There was a paucity of information in the literature on infiltration through pavements and particularly through cracks in pavements. Many of the studies that reported field observations of infiltration were based on pavement designs or climatic conditions that were significantly different from those used in the UK. A wide range of values for infiltration were reported.

An existing 24 m long test section of concrete road at TRL provided an opportunity to obtain data on actual water movement in a road under UK conditions to compare with the various theoretical models and to trial a number of techniques for estimating water content and movement. The road was instrumented to record, rainfall, run-off, flow from the sub-base and pore pressures in the sub-base and subgrade. The permeability and other characteristics of the materials were measured and the chemistry of sub-base drainage, precipitation and surface runoff analysed on several occasions. Investigations were made with ground penetrating radar and tracer experiments using a saline solution, and cracks were induced in the concrete to see if they increased the amount of infiltration.

Measurements started in August 2001and continued until the end of December 2002. Results indicate that the bulk of the instrumentation functioned correctly for most of the period, although there were repeated problems with blockages of the raingauge and the piezometer de-airing tubes and valves were damaged by frost during the winter. The piezometer data indicate that the pore water pressures varied with the rainfall in a broadly predictable manner. Laboratory experiments showed that the permeability of the concrete and the clay subgrade was very low, whereas the granular sub-base had high permeability.

It was found that the measured outflow from the sub-base was only a very small proportion of the incident rainfall. This was initially ascribed to the very low permeability of the concrete pavement, but subsequent investigations showed that the sub-base drain was not functioning properly due to incorrect positioning, so the low recorded sub-base flows may have been due to leakage from the system. Estimates of the actual amounts of sub-base flow were made indirectly from the available data. The sub-base flow was expressed as a percentage of surface runoff, which was considered to be the most accurate parameter. The recorded sub-base flows were in the range 0.02 – 0.03% of surface runoff. By looking at individual large storm events, it was found that this rose up to a maximum of 0.37%. An optimisation model was set up to estimate possible leakage from the recorded data; from this, sub-base flows were estimated to be in the range 0.5 - 0.75% of the surface runoff. A maximum value of 2.75% was obtained from consideration of changes in piezometer levels in the sub-base from summer to winter. It is felt that the values from the leakage model of 0.5 - 0.75% of surface runoff are most likely to be correct. The model was particularly useful as it matched predicted times and amounts of flow to the recorded values and allowed estimation of unrecorded leakage, and hence the total sub-base drainage flows.

Chemical analyses of the different waters and leaching tests revealed information about water movement through the trial road. The sub-base drainage water had higher chemical concentrations than the runoff, and both were significantly higher than the rainfall. The biggest influence on the

chemistry of the sub-base drainage and runoff was contact with the concrete. The higher concentrations in the sub-base drainage reflect the longer period of contact of this water with the concrete than the runoff.

A number of other techniques were investigated to obtain further information on water movement in the road. A number of cracks were induced in the pavement in September 2002, after completion of a full year's monitoring, to simulate an old, badly maintained pavement and to see to what extent this increased the amount of water penetrating into the sub-base. However, the ratio of infiltration to runoff did not show any measurable increase after cracking. An unsuccessful experiment using saline solution as a tracer was carried out in January 2003 to find if water infiltrating the cracks could be shown to be exiting the sub-base. Ground penetrating radar surveys showed that this non-intrusive technique could detect changes in moisture content of the sub-base between surveys in summer and winter.

The study highlights potential problems in obtaining accurate data from field experiments to compare with models of water movement in roads. The very low permeability of most concrete and asphalt road pavements means that the amounts of water infiltrating to the sub-base will be very small unless the pavement is badly cracked. Errors in measuring any of the variables thus have a major effect on the reliability of the overall water balance. This perhaps explains why there are so few recorded studies of water movements in road pavements. The present study was conducted by retrofitting instrumentation to an existing facility that was designed for another purpose. Problems arose due to the presence of trees close to the road and the steep slope. It is recommended that future trials use custom-built facilities that are carefully sited in the open to avoid the problems encountered in this study. They should be continued for a number of years to enable a full picture of the hydraulic regime to be obtained under a range of weather conditions.

The study has underlined the importance of understanding the movement of water in roads. Most theoretical models and field studies use roads with much higher permeability than those commonly encountered in UK practice, yet observation from real sites shows that movement of water through sub-base and subgrade layers does occur, often with significant adverse effects on the performance of the road. Better field studies are required to yield reliable data that can be used to test theoretical models. The present study suggests that leakage through uncracked pavements of low permeability will be less than 1% of the surface runoff. However, water can enter unbound sub-base or subgrade layers from leaking drains, leading to softening and settlement, cracking of the concrete or asphalt pavement layers and further infiltration; a vicious circle that can significantly reduce the life of the road, requiring early and expensive maintenance to put right. Further research is required to address this issue.

A study of water movement in road pavements

Abstract

A study was carried out to improve knowledge of movement of water into and out of road pavement layers under UK conditions. The theoretical background and results of recent international studies were reviewed critically. Field and laboratory studies were carried out to obtain data to test the theoretical models and to trial various investigative techniques. A section of existing trial concrete pavement was instrumented and rainfall, runoff, sub-base drainage and pore pressures in the sub-base and subgrade measured over a full year. The chemical composition of the rainfall, runoff and sub-base flows was measured and compared with laboratory leaching tests to assess the factors controlling the chemical composition of the flows. The permeability of the pavement layers was estimated from laboratory-based experiments . Cracks were induced in the concrete after one year, to see if the sub-base flows increased as a result. Ground probing radar was used to investigate changes in sub-base moisture content, and a tracer study using saline solution performed to find if water from the surface could be detected at the sub-base outflow. Theoretical modelling methods, identified by the literature review, were compared with the field data. The models predict greater flows than those measured. Difficulties were encountered in obtaining reliable data for some parameters, but several of the investigative techniques showed promise. Attempts to obtain an indication of the water fluxes from a combination of models and the field data are described.

1 Introduction

There is very little knowledge of the volumes of water that move into and out of road pavement layers. The mechanical properties of unbound materials such as strength and stiffness vary dramatically with changes in moisture content. It is also important to know the fluxes of water so that an accurate estimate can be made of the leaching of contaminants into groundwater or surface water. This is particularly important where recycled or secondary aggregates, which may contain high concentrations of metals and other contaminants, are used in unbound form in the road. Water movement in roads was identified as an important issue requiring further investigation by three collaborative European research projects in the late 1990s, ALT-MAT, POLMIT and COURAGE.

In response to this problem, the Transport Research Foundation (TRF) commissioned TRL to undertake a study into water movement in roads. The study involved a critical review of the literature on water infiltration into and movement beneath roads, followed up by field and laboratory studies to estimate the water movements in an existing section of concrete trial road at TRL.

This report describes the construction of the facilities for the experiment and instrumentation and the results of a one-year period of observation. A comprehensive literature review, which identified suitable theoretical methods, is included. The most appropriate of these have been used to estimate the water flows, and compared with the acquired data. The results have been compared with the observed flows to evaluate the accuracy and usefulness of the methods.

The chemistry of the drainage has been compared with the results of specialised laboratory methods that simulate the movement of water in granular sub-base material. The project has shown some deficiencies of existing methodologies for predicting the movement of water and chemicals in roads.

This project has been funded in full by the TRF. The work was carried out by TRL Limited over the period July 2000 to March 2003.

2 Review of existing work

2.1 Introduction

The movement of water into, through and subsequently out of road pavements has received little attention by either the geotechnical or highway engineering communities. Conversely, the presence of water in soils, rocks, aggregates and the like has received much attention, as has the measurement of pore water pressures and suctions in natural soils and the various layers of unbound materials making up the lower layers of road pavements. For much of the time, the latter measures of water - or more commonly moisture - content and pore water pressure provide good descriptions of the water regime in a road and its foundations. Both moisture content and pore water pressures directly affect soil strength and stiffness; moisture content affects the compaction properties of materials, and pore water pressures affect the pressures exerted by soils on adjacent materials and structures.

On the other hand, water movement in roads has not been considered a major determinant of pavement performance. Some consideration has been given to potential problems, such as piping and filtration, but these are generally covered by relatively simple and empirical expressions, such as the piping and permeability ratios: see for example Clause 513 of the Specification for Highway Works (MCHW1). However, in recent years, the use of industrial by-products and recycled materials has become commonplace in road construction (see Baldwin *et al*, 1997). These may contain contaminants, such as hydrocarbons or heavy metals, which could be leached from the unbound material by groundwater movement and released into the environment. In addition, there is anecdotal evidence that the mechanical properties and performance of materials used in pavement foundations and drainage may be affected by water movement, not just by the presence of water itself.

Problems associated with these potential effects might become more tractable, if the movement of water in roads were better understood. A simplified model of water flow is shown in Figure 2.1 and a useful summary of the mechanisms of water ingress to and egress from pavements has been provided by Dawson (1985).

The various components of water flow of concern to this project, as defined on Figure 2.1 are

P = precipitation on road surface

E = evaporation from road surface

R = surface runoff

Q = quantity of runoff at end of road

I = infiltration through road pavement

∂S = change in water content of sub-base

V = water draining horizontally from the sub-base

D = infiltration into subgrade

Figure 2.1 shows the main components of water flow in the trial road at TRL used in this project (see chapter 3). This has a concrete pavement, poured in a single layer, a granular sub-base of crushed rock complying with Type 1 of the Specification for Highway Works and a subgrade of well compacted London Clay. The low permeability of the pavement and the fact that the dominant water movement in this layer will be vertical mean that horizontal movement of water within this layer will be insignificant. Thus, for the purposes of this experiment, it can be assumed that the pavement is effectively impermeable in the horizontal direction.

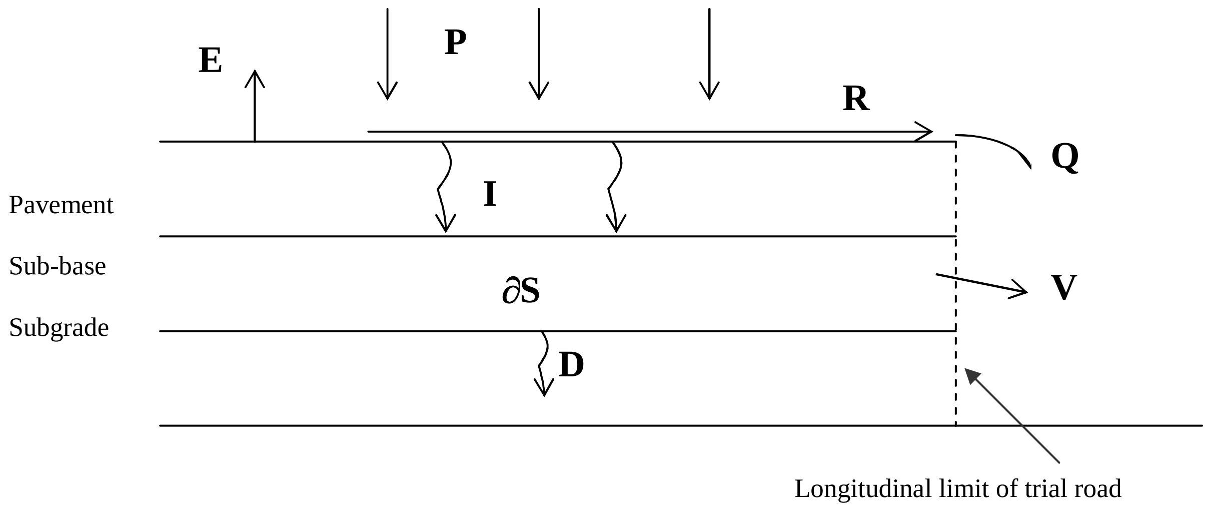

Figure 2.1 Schematic view of water flow along road at trial site

Furthermore as the subgrade is clay the permeability will be much lower than the granular sub-base and it might be assumed, at least initially, that D = 0. If this is the case then

$$V = I - \partial S$$

Over the course of a year, the change in ∂S can be expected to be very small: if it is zero then V = I, and

since $P - E = R + I$

and $R = Q$

then $P - E = Q + V$

Hence by measurement of P, Q and V and calculation of E, the value of I can be estimated.

From the foregoing, the three fundamental questions that can be asked are,

- What is the value of I?
- What is the value of the ratio I/P?
- What factors influence I?

The results can then be compared with the information from other studies presented in this chapter.

Ingress

• Through the pavement surface	through construction joints through cracks due to thermal or traffic loads through cracks due to pavement failure penetration through intact bound layers
• From the subgrade	by artesian head in the subgrade by pumping action at formation level by capillary action in the sub-base
• From the road margins	by reverse falls at formation level by lateral/median drain surcharging by capillary action in the sub-base through an unsealed shoulder collecting pavement and ground run-off

Egress

• Through the pavement surface	through cracks under pumping action through the intact surfacing
• Into the subgrade	by soakaway action by subgrade suction
• To the road margins	into lateral/median drains under gravitational flow in the sub-base into positive drains through cross drains acting as collectors

Unfortunately Dawson (1985) does not quantify any of the above mechanisms. Dawson and Hill (1998), in reprising the earlier work of Dawson (1985), added the following routes for water ingress:

- leaking pipes and gullies
- direct rainfall onto the pavement during construction

Some work has been conducted on modelling the flow of water through unbound materials, including unsaturated soils, but rarely has the accuracy of such predictions been tested through comparison with real measurements. Where they have been performed, the measurements have often been at laboratory or model scale and may therefore be influenced by scale effects. As an aside, the testing of soil permeability is one of the most problematical measurements in geotechnics. Testing suffers from poor accuracy and poor reliability and is particularly susceptible to problems of scale. Indeed, the correlation between *in situ* permeability in the field and laboratory permeability is frequently very poor. The problem is further complicated by the fact that the permeability of an unsaturated soil is strongly dependent on the mean value of the suction and the suction gradient at that point. As the moisture content increases, the permeability increases rapidly.

The arguments presented above suggest that improving our understanding of water fluxes in roads will in turn improve our ability to predict the leaching of contaminants from recycled materials used in road construction and may also help to reduce the incidence of failures in pavement foundations. Recent collaborative EU projects, such as ALT-MAT, POLMIT and COURAGE have also identified water movement as an important issue requiring further investigation, although none of them has attempted to quantify it. However, COURAGE (Anon, 1999) concluded that moisture content was a major determinant of pavement performance and was very dependent on:

- precipitation levels
- the integrity of the sealed surface
- the level of the pavement (raised or in cutting)
- the ability of the pavement to drain (its permeability and the adequacy of the drainage system).

As an initial step, the following sections seek to put the problem into perspective by reviewing the published literature in this subject area.

2.2 Water movement and recycled materials

As mentioned earlier, Baldwin *et al* (1997) reviewed the use of industrial by-products in road construction and their effects on water quality. *Inter alia*, they discussed the following mechanisms for water movements in roads.

2.2.1 Seepage of water into the subgrade from adjacent ground

Seepage may occur where the road is in a cutting and may result in groundwater entering the sub-base or capping layers, either of which may contain recycled materials. This water entry may be either direct or through the subgrade. However, the road design is likely to include some provisions to limit seepage in the cutting to improve stability, either by the use of cut-off drains or slope drains. Such provisions should also be designed to limit seepage into the pavement foundation and subgrade.

2.2.2 Rise and fall of the water table

Seasonal changes in the position of the water table may occur, which in turn may allow leaching from the pavement layers. In most situations, however, the water table is likely to be at sufficient depth to limit the applicability of this mechanism. It is generally considered that the water table should be maintained at least 1.3 m below the formation level to ensure that the subgrade remains stable. Highway Construction Details (MCHW3) require the edge of pavement drains to extend to at least 600 mm below formation level i.e. below the bottom of the capping layer.

2.2.3 Transfer of water to or from the verges

This may be a problem in roads constructed without specific drainage provisions. In the UK, there is likely to be a net movement of water into the pavement in winter and a net movement into the verge in summer. However, such effects are likely to be small and as secondary materials are likely to be used in new construction, provisions to limit these flows can easily be incorporated into the design.

2.2.4 Transfer of moisture to or from lower soil layers

Some transfer of moisture up through the subgrade may occur and in turn this may affect any suctions in the sub-base. However, this effect is unlikely to lead to the movement of any significant volume of water, although it may have a substantial effect on the performance of the pavement foundation.

2.2.5 Transfer of water vapour through the soil

This may occur where there are substantial diurnal and seasonal changes in temperature, but is unlikely to lead to significant water movement in the UK. It may however be a significant mechanism in less temperate climates.

2.2.6 Exposure of sub-base to rainfall

During construction, water falling on the sub-base (and the capping layer if present) should be directed towards the edge of the road construction, but some may percolate into the sub-base or capping. In turn, this may drain horizontally to the edge of pavement drains or percolate into the subgrade.

2.2.7 Percolation of water through the surface of the road

During construction the need to protect the lower layers of the pavement from water ingress should reduce the risk of substantial quantities of water reaching the foundation layers. However, in the case of a completed but deteriorated pavement, prolonged percolation of water through cracks and joints may occur. Successive freeze-thaw cycles may cause crazing of bituminous pavements, leading to their increased porosity. Inadequate sealing of joints and cracks in concrete pavements will also allow infiltration of rainfall. Some of this water will then percolate through the pavement layers and capping to the subgrade, but most is likely to be removed from the pavement by the drainage system.

2.2.8 Example calculations

Baldwin *et al* (1997) presented simple calculations to illustrate the flow of water in different situations. In each example, the time in years for a specific volume of water to flow out of the example geometry was calculated. The examples were:

- exposure of an embankment without topsoil cover
- exposure of an embankment with topsoil cover in place
- exposure of an embankment with barrier soil cover in place
- a deteriorated cracked road built on an embankment

Several assumptions were made in these calculations, which were given as illustrations only.

- It was assumed that 20 per cent of the mean intensity of a one year, two hour rainfall would percolate into a finished embankment not covered by topsoil. This value is derived from the Design Manual for Roads and Bridges (DMRB) Volume 7, but is at variance with the value quoted in the Notes for Guidance on the Specification for Highway Works (NFG; MCHW2), which suggests that the mean intensity of such a storm is used to dimension the drainage system (see Section 2.3.1).
- It was assumed that 10 per cent of the mean intensity would percolate into a finished earthwork covered in topsoil.
- It was assumed that the barrier soil had a permeability of 10^{-9} m/s i.e. 0.03 m/year. In turn this leads to a time delay of 16 years for water to percolate through a 0.5 m thick layer.
- Based on operational experience and advice from the then Department of Transport, it was assumed that 1 per cent of the rainfall falling on a cutting slope would arrive at the subgrade. In turn, it was assumed that 25 per cent of the moisture reaching the subgrade would be available to wet up the sub-base through capillary action.
- Ridgeway (1976) concluded that for design purposes, the infiltration rate through a pavement could be assumed to be 2.8×10^{-3} l/s per metre run of crack (see Section 2.3.2). By using OECD (1991), the UK maintenance intervention level of 10 per cent area cracked was considered by Baldwin *et al* (1997) to be equivalent to a crack length of 0.2 m/m^2. In turn, this translates to an infiltration rate of 2 l/m^2 per hour[†] or a uniform percolation rate of 2 mm per hour.

[†] Note that Dawson and Hill (1998) wrongly quote the infiltration rate as 20 l/m^2 per hour.

Baldwin *et al* (1997) then went on to consider the exit of water to the subgrade, drainage of the pavement foundations and dilution effects. Cedergren (1974) used Darcy's Law to calculate the outflow from a layer of permeable material on an impermeable base, assuming that the phreatic surface remains in the permeable layer and that the outflow and infiltration rates are equal:

$$\frac{q}{k} = \left(\frac{h}{l}\right)^2 \tag{1}$$

where q = outflow rate

k = coefficient of permeability

h = thickness of permeable layer

l = half-width of layer

Next, the authors made different assumptions about the permeability of the subgrade. These were:

- impermeable i.e. 100 per cent of water is carried to the permanent drainage
- 80% impermeable i.e. 80 per cent of water is carried to the permanent drainage
- 60% impermeable i.e. 60 per cent of water is carried to the permanent drainage

Because subgrade materials have to be well compacted to provide adequate bearing capacity, it was considered unlikely that more permeable subgrades would be encountered.

Finally, the dilution effects likely to occur when any leachates percolate from the pavement or the subgrade were discussed. These fall into three categories:

- Dilution effects which occur when leachate from the subgrade mixes with groundwater. This will depend on the groundwater storage capacity and is likely to be a non-renewable effect as the timescale for groundwater movement will be long compared to the timescale of the leaching process.
- Dilution effects, which occur when drainage from the pavement layers is mixed with surface run-off water. This will depend on rainfall patterns and intensities.
- Dilution effects due to recharge, which depend on the pattern of water ingress, again primarily from rainfall.

All these effects will be very sensitive to the water and drainage regime at each particular site, but it was tentatively suggested by Baldwin *et al* (1997) that dilution rates are likely to be between 10 and 100 times. For leachates entering the drainage system, Luker and Montague (1994) cover the control of pollution from highway drainage discharges.

2.3 Previous studies of water movement

2.3.1 Work by TRL and its predecessors

Much of the early work concentrated on understanding the movement of water held in the interstices between soil particles by surface tension, rather than the movement of groundwater which is controlled by gravity. Croney and Gwatkin (1947) describe measurements of soil suction undertaken as part of the work to understand how water was held in the interstices and how it migrated from areas having different pressures or suctions. The relations between soil suction and moisture content were also investigated in depth. Croney and Lewis (1947) also discussed the ways in which water may enter and leave road subgrades:

- through a pervious or cracked road surface
- by seepage from surrounding high ground
- as a result of suction differences between the subgrade, the verge and the soil at depth
- as a result of water vapour movements driven by temperature gradients

Unfortunately, the discussion of percolation through the pavement was rather limited. Some discussion of the pumping of fines causing premature pavement failure was provided, together with recommendations to use permeable sub-base material and a suitable geometry to promote drainage to the side of the road. The use of stabilised subgrade to reduce its susceptibility to moisture ingress was also suggested. The authors concluded that further research would be needed before the problem of subgrade regression - the steady change in moisture content leading to gradual deterioration in subgrade strength - could be fully understood. It would be interesting to know whether either of these major protagonists of UK pavement engineering thought we would still be looking for the answer fifty years later. It is equally interesting to note that one of their areas of further work was to look at the use of waterproofing layers to restrict percolation through the pavement layers. With the advent of geomembranes, this avenue is now available at relatively low cost.

By the early 1950s, the work on soil moisture had advanced considerably, but had narrowed so that it covered only the relations between soil suction and soil moisture content (Croney, 1952). Methods were developed to predict the distribution of moisture under road and airfield pavements, but the surface was assumed to be impervious in these models. Black *et al* (1958) reported similar work in more detail, but again the surfaces were impermeable. Comparative data from adjacent areas with grass cover were presented, and whilst these showed differences in moisture distribution, it would be difficult to relate this to percolation of rainwater, since the effects of evapotranspiration would also need to be taken into account. In the context of the present work, it is disappointing that the opportunity to obtain data from beneath a concrete pavement containing a leaking joint and compare it to an adjacent impermeable section was passed over.

Russam and Coleman (1961) extended the earlier work of Black *et al* (1958) to climatic conditions other than those in the UK. They gave some consideration to the balance between precipitation, evapotranspiration, drainage and field capacity for moisture retention: various quantities were used to express moisture regimes, including soil moisture deficit and moisture index. Further details are given in Russam (1962), but again the assumption has been made that the soil is covered by a relatively impervious pavement. A detailed consideration of the thermodynamic description of moisture in soil is given by Aitchison *et al* (1966), together with details of the laws determining fluid flow. These include Darcy's Law for saturated two-phase flow and its modification to cover polyphase flow in unsaturated soils. Some introductory comments are made on the application of finite difference methods to compute water movement in unsaturated soils, and the effect that improvements in computing power will have on such methods. However, the engineering considerations are once again limited to soils overlain by relatively impervious pavements.

Croney (1977) gave an overview of water movement in Chapter 6 of *The design and performance of road pavements*. This divided water into:

- Ground water - below the water table the ground is saturated and Darcy's Law applies.
- Held water - water held in either the liquid or vapour phase in the soil interstices.
- Gravitational water - water flowing under the action of gravity from the surface towards the water table: as the soil is unsaturated, a modified form of Darcy's Law is needed.

Comparative data for negative pore water pressures between the soil under a concrete slab, that adjacent to the slab but covered by vegetation and a further adjacent area of bare soil showed that the suctions under the pavement and the bare soil remained small throughout the year, whereas those under the vegetated area increased. In all areas, the soil suction increased approximately linearly with depth and lay between the values expected for extremes of the water table. The data suggest that grassed areas adjacent to carriageways, together with positive drainage to remove surface water, are likely to be beneficial to pavement longevity.

Farrar (1994) reviewed work by TRL on edge of pavement and slope drains. Measurements under the concrete pavement of the M40 motorway at Denham showed that pore water pressures in the London Clay embankment were similar to those under the central reserve. Pore water pressures were close to zero at pavement formation level, with negative pressures in the upper part of the embankment. These suggest that infiltration must be occurring through the pavement. This agreed with earlier work by Farrar (1968) which showed that in a heavy clay soil with a permeability of about 10^{-10} m/s, the water table could only be lowered by trench drains at the edge of the concrete pavement slab if the slab were covered with polythene. It was estimated that as little as 1 mm of water infiltrating through the pavement concrete was sufficient to recharge the water table under the slab.

Farrar (1994) used a simple model developed by Maugeri and Motta (1987) to show that for a permeability of the London Clay of 10^{-8} m/s, a uniform infiltration through a 10 m wide pavement of 150 mm per year was sufficient to keep the water table at formation level, even if 2 m deep edge drains were incorporated. Farrar suggests that this value for infiltration rate is similar to the value quoted in NFG (MCHW2). NFG 514 states that in the absence of better information, the infiltration rate through the pavement might be assumed to be not less than the mean intensity of a one year, two hour rainfall. Values for rainfall events in the UK are given in TRRL Road Note 35 (1976): the mean intensity for a one year, two hour storm in the area around TRL is 7.8 mm per hour.

The model developed by Maugeri and Motta (1987) is a simple one-dimensional analysis for seepage and pore water pressures in the soil between two vertical trench drains. The soil is assumed to be homogeneous and isotropic and underlain by an impermeable layer: the geometry of the problem is shown in Figure 2.1. The flow process is considered as a sequence of steady state conditions during the time in which the flow evolves. The flow is assumed to be essentially horizontal and the ratio of L/D must be large for the Dupuit assumptions to be valid: for a discussion of these, see Chapter 4 of Scott (1969). The quantity of seepage at any vertical section is given by:

$$q(x) = k\left(\frac{h^2 - D^2}{2L}\right) + N\left(x - \frac{L}{2}\right) \qquad (2)$$

where q = rate of seepage at x

k = coefficient of permeability of the soil

h = total head

N = uniform rate of infiltration

The other variables are defined in Figure 2.2.

Thus, the seepage at the drain is given by substituting x = L in Equation 2:

$$q(L) = k\left(\frac{h^2 - D^2}{2L}\right) + \frac{NL}{2} \qquad (3)$$

The water balance equation for infiltration (see Figure 2.2) is given by:

$$h(t) = p + \frac{2pe^{-2p\alpha t}}{\dfrac{h_0 + p}{h_0 - p} - e^{-2p\alpha t}} \qquad (4)$$

where h(t) = total head at time t at x = 0

h_0 = total head at x = 0 and t = 0

$$p = \left(D^2 + \frac{NL^2}{k}\right)^{0.5} \qquad (5)$$

$$\alpha = \frac{2k}{\pi n_e L^2} \qquad (6)$$

where n_e = porosity of the soil†.

By substituting Equation 4 into Equation 3, the flow rate q(t) into the trench drain can be determined. Also, in the long term, p becomes equal to h(t): this is the theoretical value derived from steady state models and the Dupuit assumptions. Further details and example calculations are given in Maugeri and Motta (1987): the model appears to be quite straightforward to use and provides results which are generally in good agreement with the charts derived by Hutchinson (1977) using finite element methods.

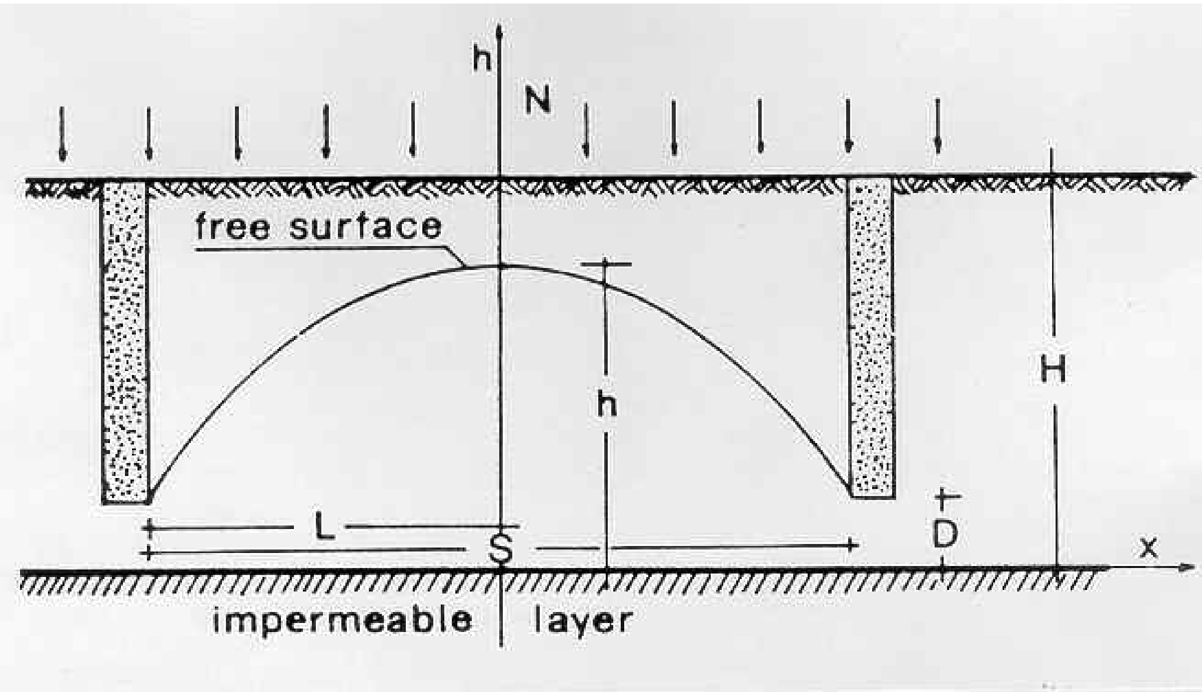

Figure 2.2 Cross section of typical trench drains (after Maugeri and Motta, 1987)

† The original paper defines n_e as the effective volume of voids, but it appears from the text that it is more likely to be the porosity i.e. the ratio of the volume of voids to the total volume.

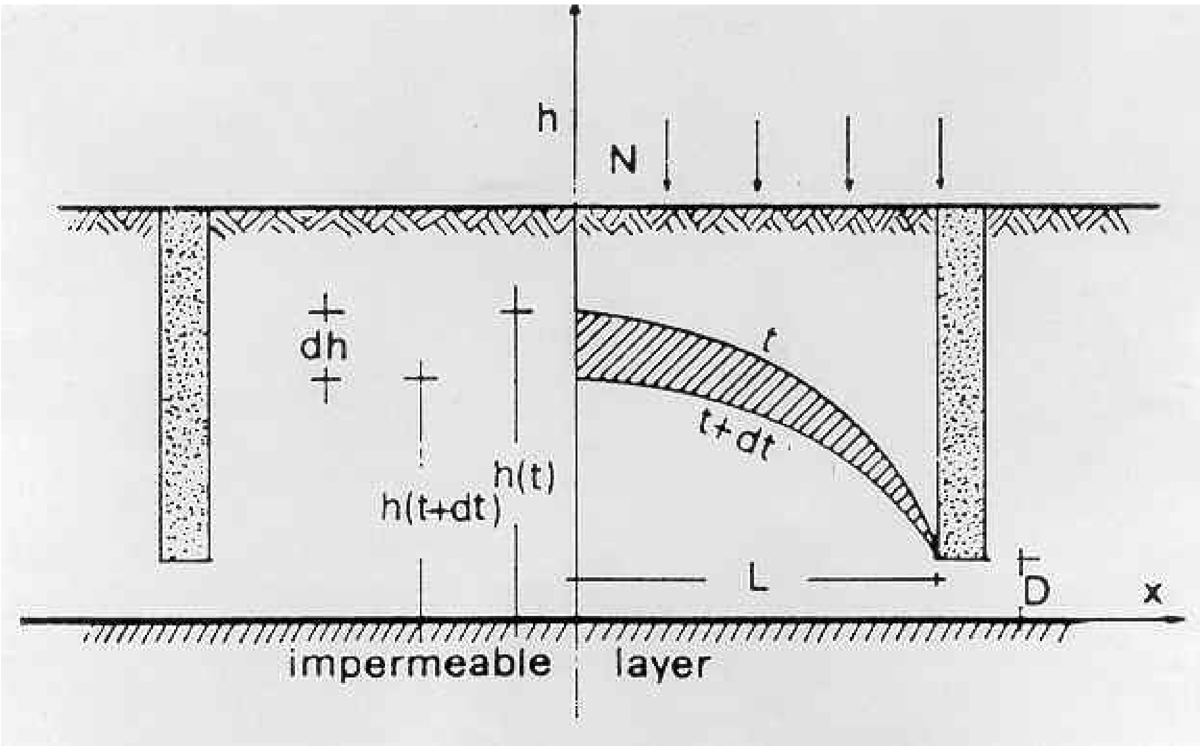

Figure 2.3 Representation for the water balance equation (after Maugeri and Motta, 1987)

As part of the EU funded project ALT-MAT, which was led by TRL, Raimbault (1999) reviewed the hydrological characteristics of road materials and the hydrological state of roads. This covered, *inter alia*, the relations between water content, matrix or suction potential and hydraulic conductivity in unsaturated materials. In these conditions, vertical flow is governed by the Richards (1931) equation: this is covered in the review of Choo and Yanful (2000) in Section 2.3.2.

Raimbault cites work by van Ganse (1981) in Belgium, in which he related the infiltration rates through a permeable surfacing, to the relative duration and relative intensity of the rainfall event, and the drying period of the pavement. Van Ganse (1981) showed that for one site in Belgium, the relation between the ratio of the infiltration (I) to the total precipitation (P) and the permeability of the surfacing (k) was as detailed in Table 2.1.

Table 2.1 Relation between permeability and infiltration ratio (after van Ganse, 1981)

k (m/s)	10^{-9}	10^{-8}	10^{-7}	10^{-5}
I/P (%)	0	5	50	~100

Van Ganse (1981) also reported that infiltration rates through a 3 mm edge crack were similar to those found in the USA and concluded that in a mild climate such a crack would absorb all the rainfall. However, the biggest problem with this model is that it requires significant analysis of the profile of rainfall intensity to determine the proportion of the duration and precipitation of the overall rainfall event which do not exceed certain thresholds. This will vary markedly from site to site and with time of year, making the analysis cumbersome.

Raimbault (1999) also reported studies by LCPC at Nantes, where levels of infiltration between 0.45×10^{-7} and 1.7×10^{-7} m/s were recorded through a thin overlay. A typical value of 10^{-7} m/s was also quoted for bituminous materials compacted to 93 per cent (presumably of the refusal density). Finally, Raimbault (1999) reviewed the Proceedings of the International Symposium on Sub-drainage in

Roadway Pavements and Subgrades, held by PIARC in Granada, Spain in 1998. These proceedings are considered along with the review of them by Raimbault in Section 2.3.4.1.

2.3.2 Studies in North America

Dempsey and Elzeftawy (1976) reviewed earlier work performed in the USA and elsewhere and presented thermodynamic equations describing the movement of moisture under the action of moisture and thermal gradients. Brief details were given of the numerical methods used - mainly finite difference - to solve the water movement and heat flow equations. Considerations of water percolation through the pavement were limited to a statement that the amount of rainfall that infiltrates the pavement is a function of the rainfall intensity and duration, the surface run-off, and the pavement surface permeability. A similar level of detail was provided in Dempsey (1979).

Dempsey *et al* (1982) undertook a state of the art review on pavement drainage for FHWA. This suggested that infiltration through the pavement could be predicted from the following equation:

$$q_i = I_c\left(\frac{N_c}{W} + \frac{W_c}{WC_s}\right) + K_p \qquad (7)$$

where q_i = design infiltration rate

I_c = crack infiltration rate

N_c = number of contributing longitudinal cracks

W = width of granular base or sub-base subjected to infiltration

W_c = length of contributing transverse cracks

C_s = spacing of transverse cracks or joints

K_p = infiltration rate through uncracked pavement surface

Dempsey *et al* (1982) also presented data from several sources where attempts had been made to measure infiltration into the pavement structure through longitudinal and transverse joints, cracks and permeable surface materials. Laboratory tests confirmed that open cracks or joints in concrete pavements had the potential to admit large volumes of water. Table 2.2 presents a summary of the data from Cedergren *et al* (1973) which were measured for a precipitation rate of 51 mm/hour. The slabs used did not have any obstruction at the bottom of the cracks. Open cracks and joints in the field would be expected to have lower infiltration rates after the available void space had been filled with water: the rate of percolation would then depend on the permeability of the materials under the slab. The percolation rates may also be affected by the thickness of the slab: unfortunately this was not given.

Dempsey *et al* (1982) also reported earlier work by Barksdale and Hicks (1975). They measured the rate of infiltration at two sites on an interstate highway in Georgia, which had plain jointed concrete traffic lanes and asphalt concrete shoulders. At one site only one per cent of the precipitation entered the pavement, whereas at the other site 64 per cent entered. From the data presented, it is difficult to see why this difference arose. However, Dempsey *et al* report that prior to testing, the pavement had experienced heavier than average precipitation. It may be that the site where infiltration was lower was already close to saturation before the measurements were made.

Table 2.2 Infiltration into cracks in concrete pavements

Crack width (mm)	Pavement slope (%)	Run-off entering crack (%)
0.89	1.25	70
0.89	2.50	76
0.89	3.75	79
1.27	2.50	89
1.27	3.75	87
3.18	2.50	97
3.18	3.75	95

Dempsey *et al* (1982) also reported on earlier work by McCullough *et al* (1975). They conducted laboratory tests on cracks in reinforced concrete pavements in an attempt to determine the maximum acceptable crack width at which corrosion would not occur. Salt water was ponded on top of the cracked slab and the permeability measured by head loss. Typical data showed that the permeability rose from 5×10^{-4} l/s per metre run of crack for a width of 0.25 mm, to 2×10^{-3} for a crack width of 0.5 mm and 2.5×10^{-2} for a crack width of 0.75 mm. This represents a fifty-fold increase in permeability for a three-fold increase in crack width.

These values are similar to those determined by Ridgeway (1976) and also cited by Dempsey *et al* (1982). Ridgeway found infiltration rates for bituminous concrete ranged from 5×10^{-5} to 2.5×10^{-2} l/s per metre run of crack: for concrete pavements he quoted values up to 2×10^{-3} l/s per m run of crack and 1.3×10^{-3} l/s per metre run for sealed transverse cracks. Ridgeway (1976) also presented a form of Equation 7 and suggested that for design an infiltration value of 2.8×10^{-3} l/s per metre run of crack might be used. This makes no allowance for the properties of an individual crack or joint but may be sufficiently accurate given the vagaries of the problem.

Dempsey *et al* (1982) also cite measurements of the permeability of asphalt concrete surfaces by Cedergren *et al* (1973), although it appears that they in turn are quoting from several other sources. The data show that for new pavements, the permeability varied between about 1.5×10^{-4} and 5×10^{-4} m/s: for old pavements (excluding laboratory tests) the permeability ranged from 5×10^{-6} to 2.5×10^{-5} m/s. These results are rather surprising, since it might generally be expected that older pavements would be more cracked and hence more permeable than new pavements. However if the material were porous asphalt it may be due to clogging of the pores with time.

However, Cedergren (1989), citing data from Barber and Sawyer (1952), states that well compacted laboratory samples of bituminous concrete had permeabilities within the range 2.9×10^{-6} to 5.6×10^{-7} m/s, whereas a traffic compacted sample had a permeability of 7×10^{-10} m/s. In contrast, Cedergren quotes a permeability of 3×10^{-4} m/s for a moderately compacted sample of a dense-graded bituminous pavement slab. Indeed, Cedergren (1989) avers that:

> “Most of the world’s pavements are so leaky that far more water soaks in than can drain away into the subsoil.”

In a typical percolation test on a cracked airfield taxiway, he measured what he considered to be a typical permeability of 7×10^{-9} m/s, which is equivalent to an infiltration of 0.6 mm per day, assuming a hydraulic gradient of unity. A value of unity is considered by most workers to be appropriate for infiltration into a cracked pavement.

Crovetti and Dempsey (1991) expand on methods of estimating infiltration rates based on the infiltration ratio and Equation 7. However, the section is not very well presented and the reader is left with some misgivings over the units to be used for the variables in Equation 7. However, the report does suggest that, for the conditions likely to be found in Illinois, infiltration rates using the infiltration ratio are likely to be about 93 litres per day per metre run of pavement, whereas values of about half this would be calculated using Equation 7. Finally, the report does present many useful data on the permeability of different sub-base materials.

Fortunately, a much better presentation of the above work is given in Section 5 of FHWA (1992). This is a notebook for participants and describes the use of both infiltration ratios and Equation 7 in some detail. The method of calculating the infiltration ratio is explained in such a way that it could be translated into UK highway engineering practice without too much difficulty. The units in Equation 7 are also clarified and worked examples are provided. The infiltration rates are then used to calculate the discharge rate that is likely to occur from a permeable base course and hence the capacity required in the edge of pavement drainage system.

FHWA (1992) also deals with the time taken for infiltration to drain from the permeable base. The model is a straightforward analysis based on porosity, permeability and the geometry of the pavement layers. Graphs are provided allowing hand calculations to be performed, and example calculations are provided. However, given that sensitivity analyses are likely to be needed for this type of design, a computer program, DAMP, has been developed (Carpenter, 1990) and is available from FHWA.

The computer model, like the FHWA method, is concerned with estimating the maximum flow likely to occur from the unbound layers under storm conditions in order that the sub-base drains can be correctly sized. It is not designed to predict the total amount of water moving through the unbound layers over an extended period such as a year. In order to estimate the extent of leaching of contaminants from the unbound layers, a model is required which can estimate the overall water movement rather than the maximum flow rate.

Ok-Kee *et al* (1994) and Buettner *et al* (1996) presented results of electrical resistance tomography (ERT) showing the distribution of moisture under pavements. In a subsidiary experiment they undertook ERT at one section before filling a borehole in the section with water. They then repeated the ERT at regular intervals for several hours. Comparison of the images produced allowed the progress of the water permeating through the lower layers of the pavement to be traced. However, the time taken to perform each ERT scan meant that the response obtained was a heavily damped time average. Given the significant increase in computing power available since this experiment was conducted in 1993, it should now be possible to trace moisture movements resulting from infiltration through the pavement, rather than down a borehole, with reasonably short time increments.

Choo and Yanful (2000) studied the vertical flow of water in unsaturated cover soils using both analytical and numerical methods. The analytical studies were based on published solutions of the Richards (1931) equation:

$$\frac{\partial \theta(h_p)}{\partial t} = -\frac{\partial}{\partial z}\left(k_z(h_p) \frac{\partial h_t}{\partial z} \right) \tag{8}$$

where θ = volumetric water content
k_z = unsaturated hydraulic conductivity as a function of pressure head h_p
h_t = total head h_t
t = time

The numerical modelling was performed using SEEP/W (1994) from Geo-slope International Ltd. The form of the function $k_z(h_p)$ was obtained from the soil moisture characteristic (suction curve) using the method of van Genuchten (1980). Two relatively simple cases were evaluated:

- downward vertical steady state flow in a multilayer soil
- transient vertical flow in a homogeneous layer

Transient flow through heterogeneous or multilayer soils is not very amenable to analytical methods, but by inference is amenable to solution using SEEP/W. Reasonable agreement was obtained between the analytical and numerical methods, suggesting that SEEP/W may be an appropriate tool with which to study pavement infiltration. Some problems were encountered in analysing the flow of water through laboratory models of multilayer cover soils, particularly at times greater than three days. These were attributed to problems with the unsaturated hydraulic conductivity - pressure function, which did not accurately model “locked-in” non-equilibrium water pressures in the laboratory models. Attention was also drawn to the fact that water fluxes will be significantly over-predicted if evaporation is not considered. As most of these problems were encountered in the fine sand used as one of the cover soils, the effects may be somewhat lessened when considering pavement drainage.

2.3.3 Studies in Australia

The Country Roads Board of Victoria provided guidance on infiltration rates in Technical Bulletin No 32 (1982). These are in the form of infiltration factors, which for design are multiplied by the mean intensity of a two year, one hour rainfall: in the UK this is equivalent to an intensity of about 10 mm per hour. Thus the factors given in Table 2.3 represent the proportion of rainfall assumed to permeate through the pavement.

Table 2.3 Infiltration factors from Technical Bulletin No 32 (1982)

Surface type	Infiltration factor
Sprayed seal	0.2 - 0.25
Asphalt	0.2 - 0.4
Cement concrete	0.3 - 0.4
Unsealed shoulders	0.4 - 0.6

ARRB Special Report 35 (Gerke, 1987) suggests infiltration factors of 0.33 for asphalt concrete and 0.66 for cement concrete roads but that the use of the one year, one hour mean rainfall intensity is likely to over-predict infiltration. Thus the two year, one hour value suggested by Victoria will lead to even greater over-prediction.

2.3.4 Other studies

OECD (1973) reported the discussions of an expert group set up to study water in roads. This mainly covered the prediction of moisture contents in subgrades, but made a few points on water infiltration. It was suggested that an “adequately waterproof” pavement was one, which had a coefficient of permeability an order of magnitude lower than that of the soil. This was considered not to be a problem for sandy soils, but silty soils with permeabilities in the range from 10^{-7} to 10^{-8} m/s would need very good quality pavements to fulfil the waterproof criterion and it would be almost impossible to achieve for pavements founded on clay.

2.3.4.1 The Granada symposium

The Granada symposium on Sub-drainage in Roadway Pavements and Subgrades, was held by PIARC in Granada, Spain in 1998.

Hornych *et al* (1998) used the computer program CESAR developed by LCPC to model water flows within pavements and subgrades. The paper provides some discussion on measuring unsaturated permeabilities and the values of this and moisture retention to use in numerical methods. Similarly, Gamir and Perez (1998) used the program FADES, developed by the Polytechnic University of Catalonia. Neither of these programs appear to have significant advantages over SEEP/W, nor do they appear to be available as commercial packages.

Lebeau *et al* (1998) reported that SEEP/W provided outputs which were in agreement with other models, but the prediction of soil water retention and unsaturated hydraulic conductivity was not straightforward. The authors used the extended soil water retention model of Fredlund and Xing (1994):

$$\theta = \left[1 - \frac{\ln\left(1 + \frac{u_w}{u_{w,r}}\right)}{\ln\left(1 + \frac{10^6}{u_{w,r}}\right)}\right] \times \frac{\theta_s}{\left[\ln\left(e + \left(\frac{u_w}{\alpha}\right)^{\upsilon}\right)\right]^{\omega}} \tag{9}$$

where θ = volumetric water content

θ_s = saturated volume water content

u_w = pore water pressure

$u_{w,r}$ = pore water pressure corresponding to the residual water content, θ_r

α, υ, ω = three different soil parameters

The unsaturated permeability was predicted from the work of Fredlund *et al* (1994):

$$k_r(u_w) = \frac{k(u_w)}{k_s} = \frac{\displaystyle\int_{y=\ln(u_w)}^{\ln(10^6)} \frac{\theta(e^y) - \theta(u_w)}{e^y}\theta'(e^y)\,dy}{\displaystyle\int_{y=\ln(u_{w,aev})}^{\ln(10^6)} \frac{\theta(e^y) - \theta_s}{e^y}\theta'(e^y)\,dy} \tag{10}$$

where k_r = relative hydraulic conductivity as a function of u_w

k_s = saturated hydraulic conductivity

$u_{w,aev}$ = pore water pressure near the air entry value

$\theta(e^y)$ and $\theta'(e^y)$ are Equation 9 and its derivative evaluated at e^y

y = dummy variable of integration representing the logarithm of pore water pressure

Some guidance on the above equations is given in the paper, along with typical data for a uniform sand and a well graded gravel.

Quibel (1998) presented a simpler, empirical model in which the pavement system is broken down into an array of cells, each of which is ascribed an unsaturated permeability (see Figure 2.4). The model showed that the infiltration rate fell from 30 per cent of the incident precipitation to 5 per cent as the permeability of the surfacing decreased from 10^{-7} to 10^{-8} m/s. Alonso (1998) presented a much more comprehensive model, CODE_BRIGHT, in which the thermal and mechanical behaviour of the pavement is considered, as well as its hydrological state. Although it is not entirely clear from the text, it appears that the unsaturated properties used in CODE_BRIGHT were derived from the work of Alonso, Gens and Hight (1987). The model assumed 65 and 15 per cent of the incident rainfall intensity permeated unpaved and paved areas respectively. Detailed results were presented, but the model seems overly complicated for such an ill-defined problem and the computer program appears to have been developed by Alonso and his co-workers for their own use, rather than as a commercial package.

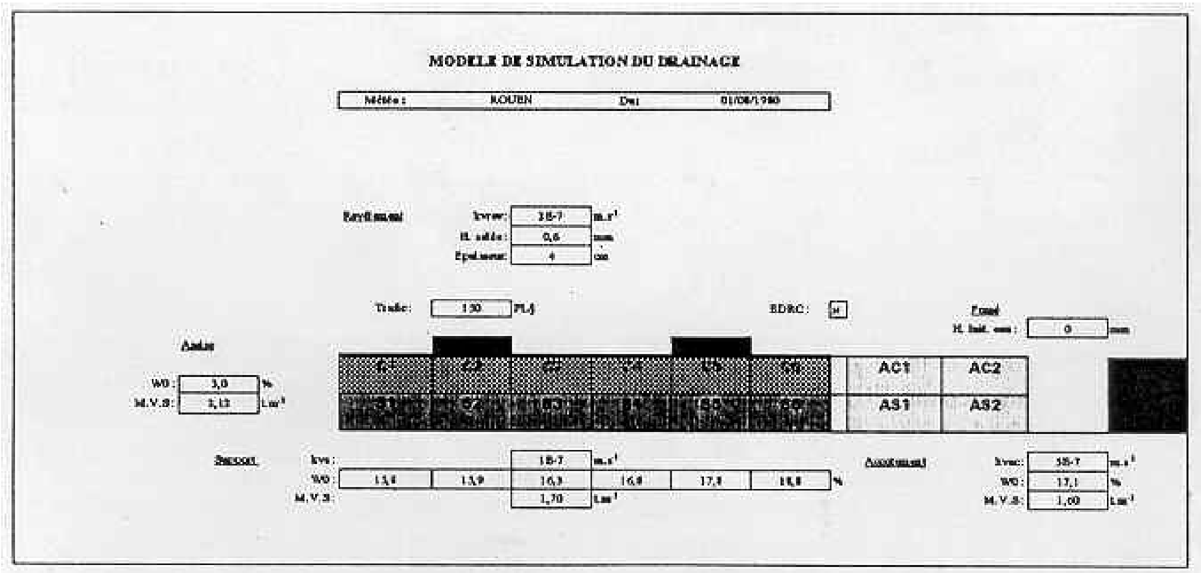

Figure 2.4 Cellular model for simulating drainage (after Quibel, 1998)

Akai, Ohtsu and Ohnishi (1998) presented an analytical model similar to that proposed by Maugeri and Motta (1987) which assumes the shape of the water table is parabolic. This model also extends the scope of the earlier work as it covers the case of an inclined road base. The analytical model was also shown to predict accurately the results of simple laboratory model tests, giving increased confidence that the model of Maugeri and Motta is robust.

Robertson and Birgisson (1998) report the use of time domain reflectometry (TDR) to measure *in situ* water content of soils and other materials. TDR utilises the markedly higher dielectric constant of water compared to other soil constituents: thus there is a well defined relation between effective dielectric constant and water content. The authors propose to use this to assess the effectiveness of different edge drains, whilst monitoring the quantity of water entering the pavement and leaving the drain. Details of the study are rather sparse, but the experimental set up appears to be similar to that proposed in the present study.

Waters (1998) reports details of the relation between permeability and normalised air voids for a range of asphalts typical of those used in Australia. Overall, a very high correlation was obtained using the following equation:

$$\log k = 5.469 \log NV - 4.085 \tag{11}$$

where k = permeability in µm/s
NV = normalised air voids in per cent

$$NV = \frac{VD_{50}}{4.75} \tag{12}$$

where V = air voids in per cent
D_{50} = the sieve size through which 50 per cent of the material passes

Thus the normalisation reflects the fact that the median particle size for most Australian asphalts is 4.75 mm. Using Equations 11 and 12, the permeability is found to vary between 10^{-8} and 10^{-2} m/s as the air voids vary from 2.5 to 30 per cent.

Finally, Grogan (1998) gives details of the infiltration performance of a pavement consisting of 115 mm of sub-base, 240 mm of unbound base and 90 mm of hot mix asphalt concrete, with an average fall of 1.5 per cent. Based on the paved area, the outflow from the drainage system and five unique rain events, which could be separated from the continuous data, the results in Table 2.4 were obtained.

Table 2.4 Infiltration ratios for different storm events

Storm event		Tipping bucket output	
Duration (hours)	**Rainfall (mm)**	**Volume (l)**	**Volume as % of**
1	0.5	209	23
5	2.0	338	11
7	2.5	640	14
7	10.7	994	5
38	51.3	2127	2.5

These data clearly show that the infiltration ratio decreases as the storm duration increases, indicating that the lower layers of the pavement become less permeable as they become more saturated.

2.4 Discussion

At the start of this review, it was assumed that there would be relatively few data available on water infiltration into pavement foundations. However, rather more than anticipated have been found, although there are no strong themes connecting what appear to be a number of disparate studies. In the following sections, an attempt is made to summarise some of the main points to emerge from these previous studies.

2.4.1 Experimental results

The experimental results presented in the literature provide data on infiltration which are expressed in several different ways, as well as showing rather large scatter in the values obtained. Three main ways of presenting the data are used:

- a uniform infiltration rate, flux or permeability, which gives an absolute measure of the volume or rate of water entering a unit area of the pavement
- an infiltration ratio, which expresses infiltration as a fraction (or percentage) of incident rainfall
- a measure of infiltration volume or rate related to a particular feature, such as a crack or joint

All these measures are related, but the form of the relations requires a knowledge of the main parameters describing rainfall events at the site and a measure of the number of cracks or joints per unit area. In addition, there is strong evidence to suggest that the rate of infiltration will vary with time: as the pavement foundation becomes more saturated, the flow of water through the pavement will slow down. In simple terms, there is nowhere for the water to go and so run-off will take over from percolation. There is also strong evidence to suggest that relatively small changes, in the properties of a pavement surface or in the geometry of the cracks within it, will result in major changes to the rate of infiltration.

The data on infiltration rates suggest that typical permeabilities for uncracked pavement surfaces are likely to lie in the range from 10^{-4} to 10^{-10} m/s, with a typical value of 10^{-7} m/s mentioned by several authors. This corresponds to an infiltration rate of 8.6 mm per day, but of course the actual volume will depend on the duration of the rainfall event. For infiltration ratios, the data suggest that values are likely to range from about 0.2 to 0.4 for bituminous roads, 0.3 to 0.6 for concrete roads, up to 0.9 if they are cracked, and from about 0.4 upwards for unsealed shoulders. For cracks, the infiltration rates lie in the range from 2.5×10^{-2} to 5×10^{-5} l/s per m run of crack, with Ridgeway (1976) suggesting a design value of 2.8×10^{-3} l/s per m run of crack.

In order to link together the above measures of infiltration, the following points may be useful.

The relation between permeability and infiltration ratio given by van Ganse (1981) and reproduced in Table 2.1 provides a good starting point and is in reasonable agreement with data from other sources.

Similarly, the data of McCullough *et al* (1975), which showed a fifty-fold increase in permeability for a three-fold increase in crack width, provide a reasonable link between these variables.

Finally, the data of Grogan (1998) reproduced in Table 2.4 provide a starting point in defining a relation between storm duration and intensity and infiltration ratio.

2.4.2 Models for water movement

The work of Baldwin *et al* (1997) provides the best starting point from which to consider infiltration and its likely consequences for leaching pollutants from secondary materials used in road construction. Unfortunately, it has a number of shortcomings, largely caused by the paucity of data on which it is based. The main problems are:

- The calculation of the volumes of water likely to percolate into earthworks with or without topsoil or a barrier soil needs to be refined.
- The estimation of the volume of water entering a cutting slope and subsequently reaching the subgrade needs to be improved.
- The infiltration rate through the pavement is based on the design value given by Ridgeway (1976), but the method of linking the crack length he used to the percentage area cracked, which is used in the UK as the basis for maintenance intervention, is tenuous.

Other models for infiltration, which appear to be particularly relevant to the present study, are as follows.

- The model DAMP and the associated description appears to be a useful method of predicting pavement infiltration. The work extends that of Ridgeway (1976) and may go some way to overcoming the problem of linking crack geometry to cracked area.
- The model of Maugeri and Motta (1987) for predicting the water regime between vertical trench drains appears to be particularly useful, especially as it has been calibrated against the work of Hutchinson (1977). Using this model and the extension to a sloping road base proposed by Akai *et al* (1998), it should be possible to translate some of the case histories of long-term pore water pressure data, collected in embankment and cutting slopes and under pavements by TRL, into infiltration rates. Some details of this are given by Farrar (1994) for the pavement at the M40 site at Denham but it should be relatively straightforward to extend it to some of the cutting and embankment slopes cited in the same paper.
- SEEP/W appears to be the best documented of the numerical methods used to analyse the pavement infiltration problem. It uses the models of unsaturated soil behaviour developed by

Fredlund and his co-workers, rather than the better known alternatives of van Genuchten (1980) or Alonso *et al* (1987). That said, the results obtained from the model by Lebeau *et al* (1998) and Choo and Yanful (2000) appear to be reasonable and have been calibrated to some extent against analytical methods of known validity.

- Finally, the cellular permeability model developed by Quibel (1998) appears to offer a relatively simple simulation of pavement infiltration and the subsequent drainage of the pavement foundation.

2.4.3 Relation to UK conditions and future work

The studies discussed in the previous sections come from a range of countries around the world, and many relate to pavement designs and climatic conditions that are significantly different from those in the UK. The results therefore have to be viewed with caution. Many of the studies relate to thin surfacing layers and unbound bases, in contrast to the thick layers of asphalt, concrete and cement bound material used in the UK. The range of permeability given for road surfacing material, 10^{-4} to 10^{-10} m/s with a typical value of 10^{-7} m/s, is much greater than would be expected in normal UK practice. With the exception of porous asphalt, most UK pavement materials are designed to be impermeable and to shed water into the surface drainage system. Fresh pavements would be expected to be at the lower end of the range given, around 10^{-10} m/s, and hence to allow virtually no infiltration according to Table 2.1. However, with age and traffic loading cracks would start to appear, and potentially infiltration could increase until by the time the intervention level of 10% area cracked is reached it could be significant (Baldwin *et al*., 1997).

The theoretical models may be put into some form of perspective by an example observed on a major motorway in South East England in the 1990s. The motorway was on a high embankment and the pavement consisted of a reinforced concrete slab above a cement bound sub-base. This was underlain by a silty sand layer about 2 m thick, with the rest of the embankment consisting of heavily overconsolidated clay fill. The motorway had been open for less than 15 years, but the pavement was badly cracked, especially at the expansion joints between the concrete slabs. Water had penetrated into the silty sand layer, where it had been retained because of the fine grain size of the particles. As a result the layer had a consistency like jelly and was not offering adequate support to the pavement, which shook visibly when heavy lorries passed by. Because the water could not drain a vicious circle had developed, whereby the loss of support led to more cracking, leading in turn to further infiltration. The pavement had clearly been designed to be impermeable, or a moisture-susceptible material would not have been placed beneath it, but infiltration had occurred and was compromising the performance of the road.

On the basis of this review and the discussion of the experimental results and models available, the following suggestions for future work are put forward.

The model of Baldwin *et al* (1997) should form the basis for future analysis, with the existing shortcomings being investigated using the model DAMP to attempt to improve the prediction of infiltration through the pavement. In addition, the models proposed by Maugeri and Motta (1987) and by Akai *et al* (1998) should be used to try to improve the prediction of infiltration through adjacent slopes and verges.

The model proposed by Quibel (1998) should be evaluated against the results for pavement infiltration obtained from experimental work and other modelling studies, to determine if it provides a simple way of predicting infiltration behaviour.

SEEP/W appears to be the best documented of the numerical models, which are commercially available. It should be used to try to develop improved design values for use in the model of Baldwin *et al* (1997) by conducting a limited suite of parametric studies covering the range of infiltration conditions likely to be encountered in the UK.

Data should be sought from instrumented sections of 'live' or trial roads to confirm assumptions about infiltration through fresh and cracked pavements in the UK and assess the implications in terms of performance and potential leaching.

3 Field study at TRL

3.1 Background

The previous section illustrates that several recent projects have highlighted the fact that there is little detailed understanding of the flow and volumes of water which move into and out of road pavement layers. The water regime in road structures can have a great effect on both the mechanical properties of construction materials, and also on the amount of potential contaminants, which may leach from the road materials into the surrounding ground.

The leaching of contaminants is particularly important where alternative materials, which may contain relatively high concentrations of potential contaminants, are used as unbound road sub-base or capping. Current methods of estimating the environmental impact of such materials in road constructions are conservative, because they are based on laboratory tests where the leachant liquid to material solid ratio is high. Conditions in road structures are generally unsaturated, and are therefore more likely to be at lower liquid to solid ratios than laboratory tests. The amount of water percolating into the sub-base from the surface of a road is likely to be small, and amounts of leachate reaching surface or ground waters are likely also to be small. However, the actual amounts have not been quantified, and conservative estimations have been made with the result that alternative materials are seen as likely to leach out contaminants in much higher quantities than is likely to be the case in the field. Alternative materials may thus be rejected unnecessarily for use in road construction.

Water movements in road structures also affect the development of performance criteria for the mechanical properties of construction materials. The mechanical performance of road materials depends on the moisture conditions of the road, and these vary in response to seasonal, rainfall and groundwater conditions. The need for further research in this area was identified by the collaborative EC funded projects ALT-MAT, COURAGE and POLMIT, and is highlighted by the case study given in Section 2.4.3.

To advance understanding of the subject and its implications under UK conditions, it was decided to carry out a field study to monitor the actual movements of water through a road pavement. In order to monitor the water fluxes, a suitable section of road that could be monitored and controlled was required. A detailed specification for a trial road to monitor water movements is given in the final report from the ALT-MAT project (Reid *et al*., 2001). Within the scope of the project, there was not sufficient time or resources to build an instrumented trial road that would meet these requirements. Consideration was therefore given to adapting an existing trial road.

At the TRL Crowthorne site a trial road had been constructed in 1996 as part of an earlier research project. The road was situated within a firebreak in the pine forest at the north-eastern corner of the Small Roads System, National Grid Reference SU 8523 6568. The aim of the original project was to investigate the potential of various geophysical and non-destructive test methods for assessing the structure and condition of a road pavement, in particular the detection of voids. That project had been completed, and it was felt that with modifications the trial road could be used to investigate water fluxes in roads. It also provided an opportunity to trial various techniques for understanding water movement in roads, such as ground probing radar and tracer studies.

A brief summary of the trial road structure is given below.

3.2 Existing trial road

The section of road is 24 m long, 4 m wide and has a significant slope along its length, running downhill to the north. (Plate 1) The surface level drops by 1.31 metres over the length of the road, giving a gradient of 5.5% (1 in 18). It has a concrete pavement approximately 240 mm thick and a 200-250 mm thick unbound Type 1 sub-base of crushed limestone aggregate. Beneath this is a 200-300 mm thick layer of London Clay, which in turn overlies natural ground – at this site a sandy clay deposit (Bracklesham Beds). The site is named as ‘Clay Hill’ on the 1:25,000 OS map of the area. There are six artificial voids at various points in the subgrade section of road, below the road centre line; these were constructed using sealed air-filled plastic containers. A cross-section of the road is shown on Figure 3.1.

Half of the length of the concrete pavement of the road contained steel reinforcement (which allowed investigation into the effect of reinforcement on geophysical and non destructive testing in the previous experiment), and three voids were present under each half of the trial road. The road is shown in plan on Figure 3.2.

The presence of the simulated voids in the road subgrade should not affect the overall water flux in the road. It was ensured that the plastic containers used to simulate voids were sealed and air-tight when placed, and care was taken to ensure that they were not damaged during placement in the road construction. If subsequent damage has occurred, and there is the potential for ingress of water into the containers, their small volume compared to the overall volume of the road structure should mean that the nature of the water flux is not greatly altered within the road.

The location of the existing trial road (on site at TRL Crowthorne) ensured that it was readily accessible, and had the potential to be used for experimental work and monitoring for a long period. However, because it was not constructed for the purposes of monitoring water movements there were limitations to the information that could be obtained. The extent of these limitations became more apparent as the project progressed. Despite these, it was felt that useful information could be obtained that would contribute to the knowledge of water movement in roads in the UK and provide data to compare with the existing work reported in Section 2.

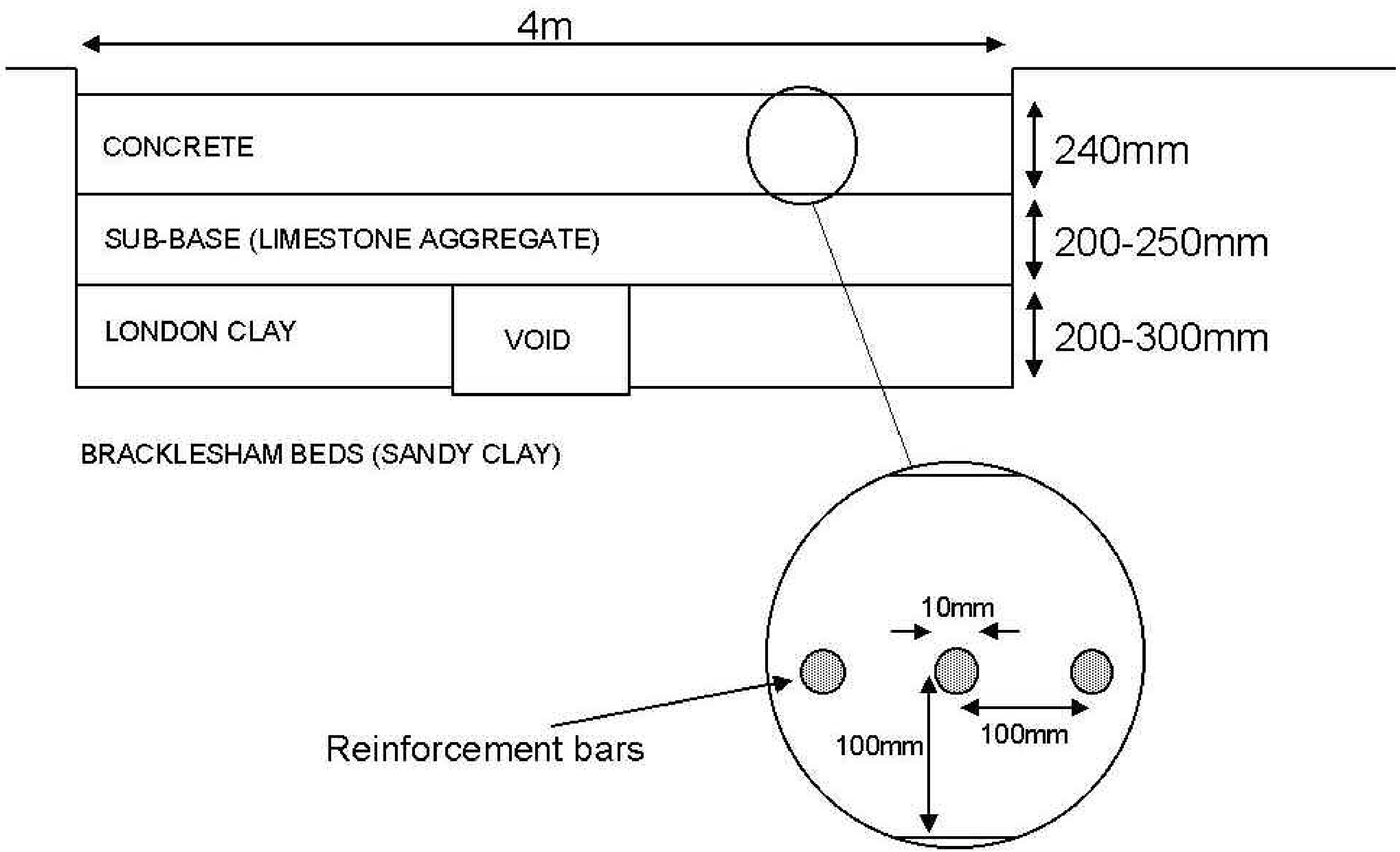

Figure 3.1 Simplified cross section showing design of voids trial road

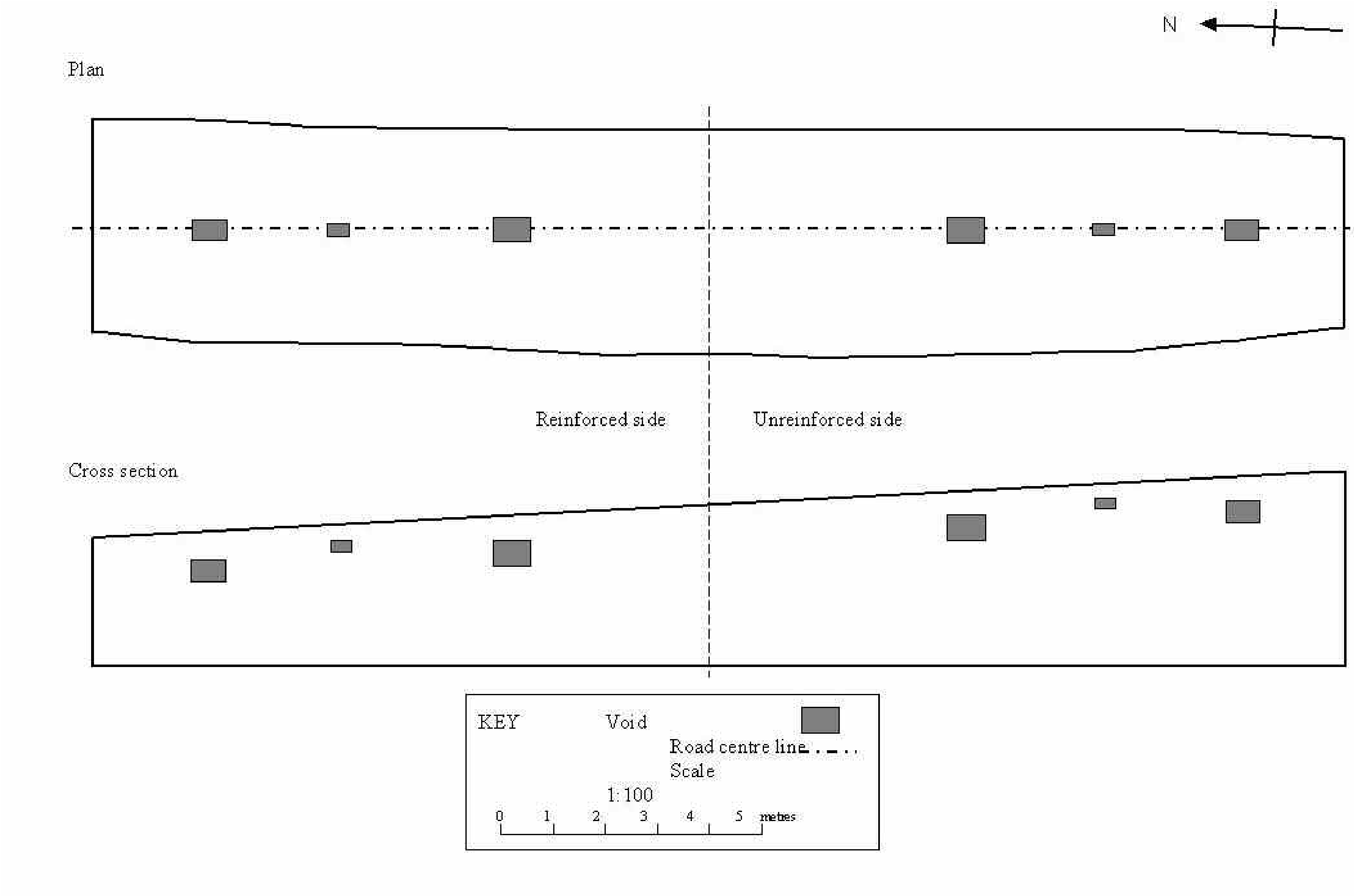

Figure 3.2 Plan and cross-section of trial road indicating void locations

4 Modifications to trial road and instrumentation

4.1 Modifications required

The trial road at TRL was potentially suitable for the experiment, but a number of modifications were required in order to ensure that all the movements of water were controlled, and a programme of instrumentation had to be developed in order to measure and record the fluxes of water.

In order to prevent flows of water entering the sub-base from the sides, A method of sealing the perimeter was required. A method of preventing run-on to the surface of the road and directing the run-off to the measurement equipment was needed to provide parameter “Q”. The London Clay below the sub-base was expected to provide an adequate seal against entry of water from below. A method of isolation and measurement of the drainage from the sub-base was required to provided parameter “V”.

A measurement of incident rainfall was required to give parameter “P”. An estimation of evapotranspiration, “E”, was required and was obtained, for August to November 2001, from the Meteorological Office. These data were calculated for their Beaufort Park site in Bracknell, which is less than 1km from the trial road.

It was considered likely that D and δS might vary over the length of the slab, so it was decided to install a series of monitoring points near the centre line. It was also decided that it would be prudent to measure both moisture content and porewater pressure, in order to have the maximum information to calculate the movement of water within the sub-base. The layout of the instrumentation is shown on Figure 4.1. A total of six positions for piezometers were used, with three instruments in the subgrade and three in the sub-base. (Plates 2 and 3) Six locations for moisture content determination were chosen, adjacent to the piezometer locations.

Vibrating wire piezometers were selected as the most suitable for use in both the subgrade, where pore pressures were expected to be positive at all times, and in the sub-base, where small suctions might develop in prolonged dry spells in summer. The piezometers were connected to a data logger so that a virtually continuous record of pore pressures was obtained. This gave information on the hydraulic gradient along the road within each layer and vertically between the layers, and how this varied with time.

The repeated determination of moisture content at the same point in a soil below a concrete pavement poses problems in terms of instrumentation. The method chosen was to use a neutron moisture probe, with access tubes installed through the concrete pavement into the sub-base and subgrade. The instrument was calibrated by taking a set of readings immediately after installation of the access tubes which were compared with the moisture content of samples of the material excavated during installation. This method allowed readings to be taken manually at regular intervals, but did not allow continuous monitoring of the moisture content. The monitoring programme therefore had to be designed with this in mind.

An alternative system, which would have allowed continuous logging of moisture content, would have been to install time domain reflectometers (TDRs). These are small probes, which could be installed at close intervals in the soil, and hence give a more accurate picture of the variation in water content vertically through the sub-base. However, there would have been problems installing these probes in the existing trial road, and they were much more expensive than the neutron moisture probe. The TDR alternative was therefore not used in this experiment, but in a situation where a trial road was being constructed from scratch and the budget permitted, they would be the preferred method of monitoring moisture content.

In order to install the piezometers and access tubes for the moisture probe, cores were drilled through the concrete pavement. The cores were used to determine the permeability and bulk density of the concrete. The sub-base and subgrade were excavated by hand, and samples taken. The moisture content of all samples was determined. In addition the grading of the sub-base was determined.

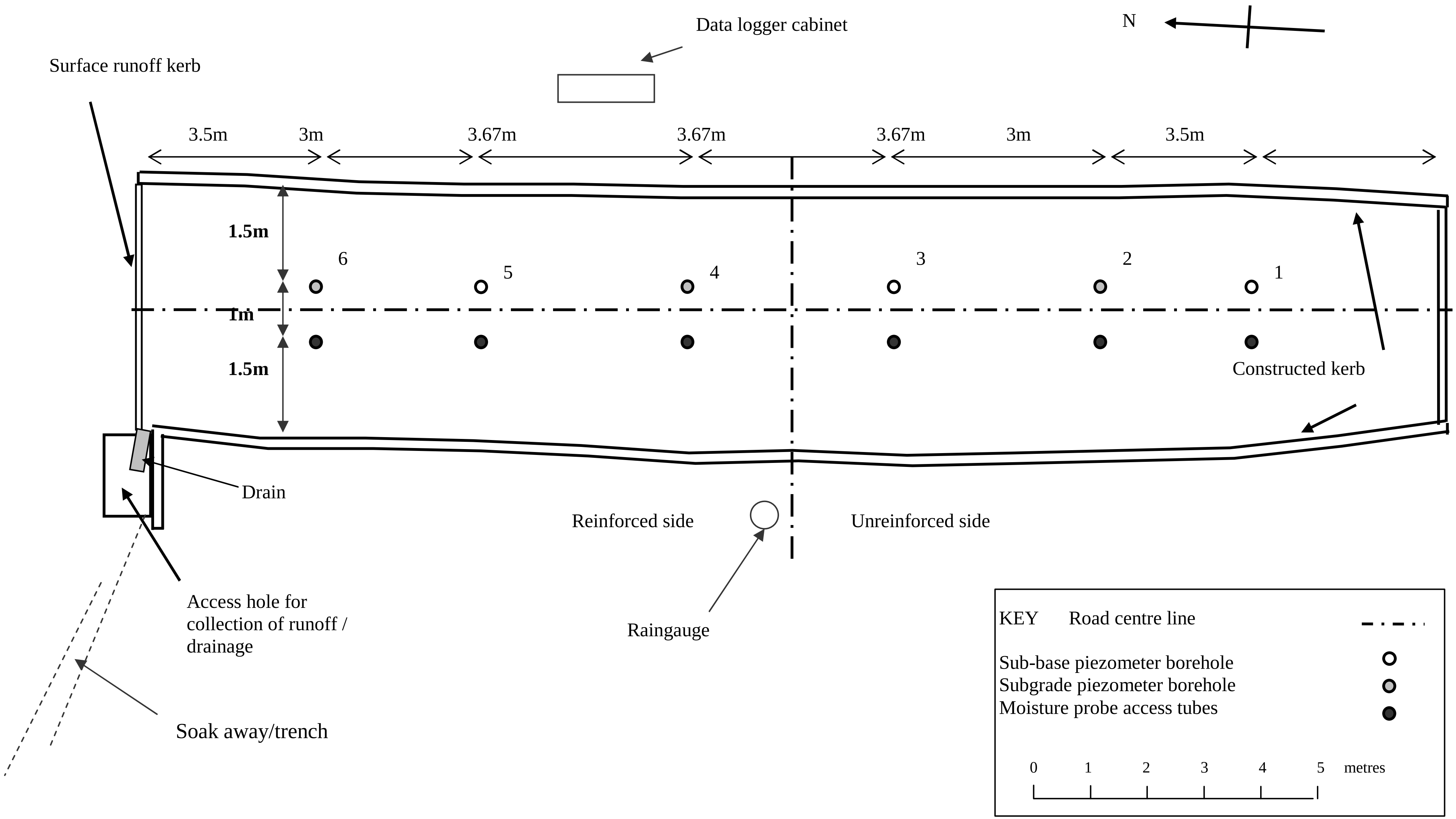

Figure 4.1 Plan of modified trial road – showing locations for piezometers

This was used to give an indication of the permeability of the material. The liquid and plastic limits of the London Clay subgrade were determined. This allowed characterisation of the material.

Further details of the modification to the trial road and the installation of the instrumentation are given in Section 4.2.

4.2 Carrying out the modifications

There were a number of issues that needed to be addressed, prior to taking any measurements of the variables outlined above.

4.2.1 Clearance and inspection of road

Prior to this experiment the road had been covered with a layer of hardcore. This had been placed down the length of the firebreak in which the trial road is situated; it was placed to facilitate the movement of vehicles involved with the haulage of felled trees from the surrounding woods. One of the first tasks of the project was to clear the hardcore from the road surface. This was carried out in August 2000.

It was important that the trial road was in an adequate structural condition for the experiment; and reasonably representative of a road on the public highway. The condition of the trial road was assessed after the clearance of the hardcore. There appeared to be no cracking or other defects in the road surface, the pavement appeared intact and undamaged along the full length of the trial road, and the structure was judged to be in a satisfactorily good condition to permit its usage for this project.

4.2.2 Modification of the trial road

Modifications were required to control water movements and enable them to be monitored accurately. This work was carried out between December 2000 and July 2001. This coincided with a very wet period, and had been preceded by an extremely wet autumn in late 2000. This caused problems with the construction works as ground conditions became very soft. This was exacerbated by the location of the site in a firebreak in a pine forest. The site was surrounded by trees, which kept it very damp even when it was not raining, especially in winter. Vehicular access to the site was by means of the Small Roads System and had to be organised in advance to avoid conflict with other users. As a result the works took considerably longer than anticipated. It had been hoped to start recording water movements on 1 April 2001, but in the event all the instrumentation was not fully installed and working until the beginning of August 2001.

Trenches were dug on both sides and across the top of the road and filled with compacted London Clay, to prevent the ingress of water from the adjacent ground. The London Clay was surplus material from TRL’s pavement test facility (PTF). This was mixed with water and rotavated to a soft consistency that would ensure a low permeability when compacted. The clay was then covered with about 150 mm of granular material, to act as a capillary break layer and prevent the clay drying out and cracking in the summer. Laboratory-based experiments to determine the index properties were carried out on the London Clay and the natural ground, which had been excavated from the trenches. To contain the surface run-off, kerbstones were installed around the perimeter of the road, with a collection point in the north-western corner. This arrangement is shown schematically in Figure 4.1

4.2.2.1 Road structure edges

Levels taken on the surface of the trial road showed the north-western corner to be the lowest point. Any water flow through the sub-base was expected to follow the gradient of the running surface, i.e. south to north with a tendency to flow towards the western side of the road.

During the clearing and inspection of the road in August 2000, a trench was excavated along the western edge of the road to examine the water flow in this area. From the trench it could be seen that water flow laterally out of the sub-base was minimal. It was subsequently decided that installation of a drain along this edge was unnecessary, and simply sealing the edges of the structure would be sufficient to proceed with the experiment.

Water from the sub-base was collected by a drain installed at the northern end of the road (see Figure 4.1). A trench excavated at this end of the road, during the inspection of the trial road, confirmed that the sub-base was saturated at this point.

4.2.2.2 Runoff / drainage system

The system was installed at the lowest point of the road to collect the runoff water from the road pavement and the water draining from the sub-base. The arrangement is shown in lateral cross-section in Figure 4.2. To measure the volume of water collected, two tipping bucket gauges were installed, and an access chamber in the north-western corner was excavated to accommodate them. The work was carried out between January and June 2001.

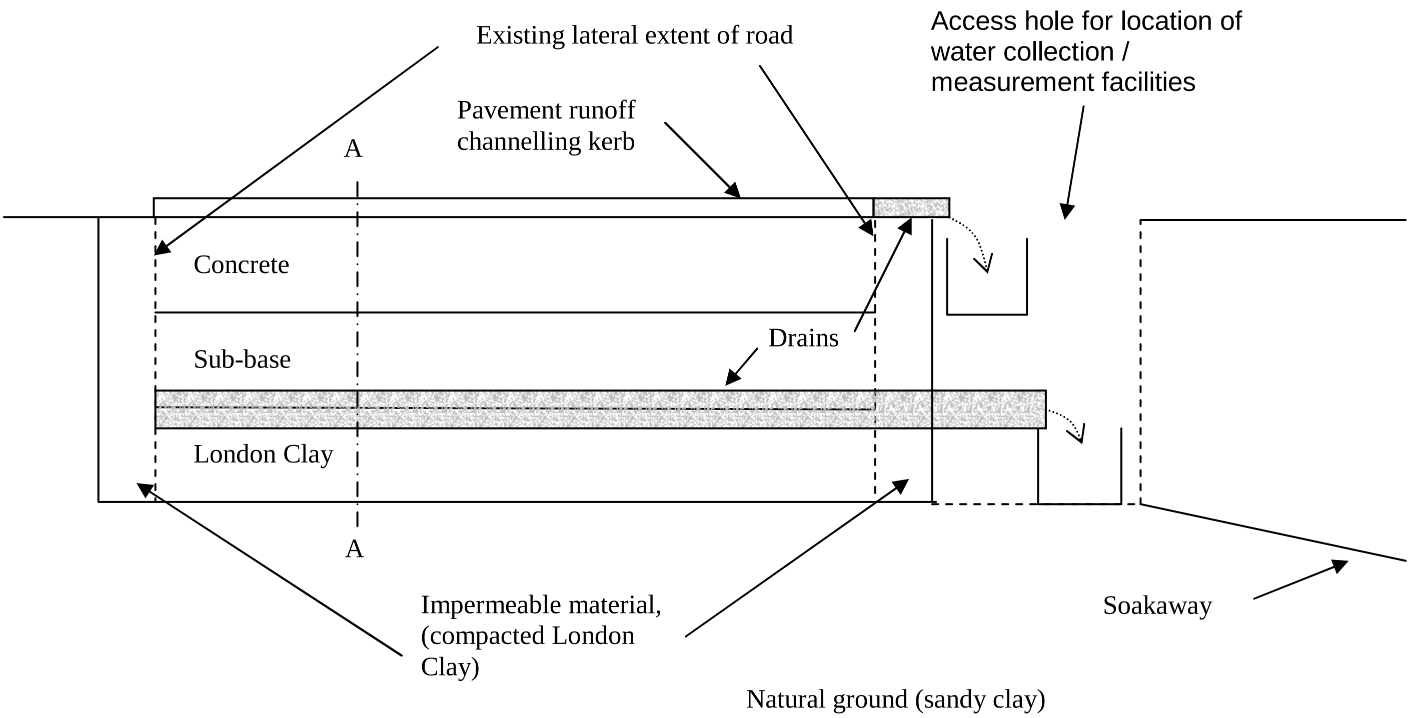

Figure 4.2 Simplified lateral cross section at northern end of road, showing drain arrangement for runoff and sub-base drainage measurement/collection (not to scale)

4.2.2.3 Installation of instrumentation

Cores were drilled through the concrete pavement in March 2001 to allow installation of the piezometers and in April 2001 for access tubes for the moisture probe. The holes were 150 mm in diameter. The cores were bagged and taken to the laboratory for testing. The holes were excavated through the sub-base and London Clay subgrade by hand. The excavated materials were double bagged and sealed to prevent loss of moisture, and were taken to the laboratory for determination of moisture content and index properties. These results are presented in Section 6.

Once all the instruments were installed and all the construction work was complete, the site was surveyed and levelled relative to a temporary benchmark, assigned a level of 10.00m, adjacent to the edge of the small road system.

4.3 Instrumentation

The instrumentation comprised six vibrating wire piezometers; three tipping bucket gauges and a set of six access tubes for the neutron moisture content probe. The piezometers and two of the tipping bucket gauges were installed during March 2001 and the third tipping bucket gauge, to measure the runoff from the surface of the road, during June 2001.

The data from the three tipping bucket gauges and six piezometers was collected using a Campbell CR10X datalogger, values being recorded every hour.

4.3.1 Piezometers

The piezometers were of the de-airable vibrating wire type. They were fitted with high-air-entry-pressure ceramic filter elements to minimise the entry of unwanted air into the instruments, and 3-bar range vibrating wire pressure transducers. A schematic drawing of a piezometer installation is given in Figure 4.3.

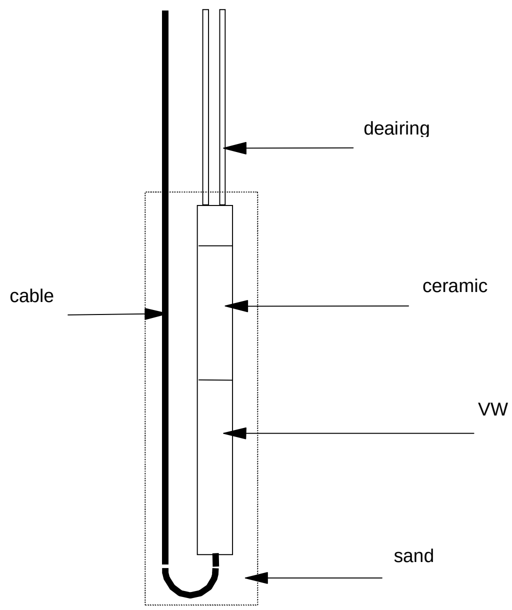

Figure 4.3 Piezometer installation

Three instruments were installed in the granular sub-base and three in the clay just below the sub-base. The instrument locations are shown in plan in Figure 4.1. Details of the installation depth of each instrument are shown on Figure 4.4. The pressure transducer is at the base of the ceramic filter. Any air which passes the filter collects at the top from where it can be removed by circulating de-aired water through the two tubes connected there. These tubes were led from the borehole and terminated at valves in the data-logger cabinet.

Before installation the piezometers were de-aired and their outputs logged for several weeks to check for faults. One was found to behave erratically and was replaced. The calibrations supplied for the transducers by the manufacturer were checked by lowering each transducer to the base of a water filled tube three metres deep. None showed any significant variation from the supplied calibration.

Following installation, five of the piezometers were found to be working satisfactorily and showed a small response to rainfall events. The piezometer in the clay subgrade at the north end of the road (hole 6) showed very high readings immediately after installation, and the readings remained high and did not vary over time. The frequency of automatic reading was increased to every five minutes for a week in an attempt to get the piezometer to respond, but without success. The loss of one out of six piezometers was not considered sufficiently serious to warrant replacement. No data was obtained from this piezometer throughout the project.

4.3.2 Tipping bucket gauges

The three gauges installed were all of the tipping bucket type. This type of gauge provides a digital output, simplifying operation with a data-logger. The gauge measuring rainfall and the gauge measuring the sub-base outflow were similar standard raingauges. The resolution of each gauge was 2ml per tip of the bucket. For the gauge measuring the rainfall this corresponded to 0.1mm of rainfall. Such gauges are known to exhibit degradation in accuracy of around 4% at rainfalls of 25mm/hr rising to 8% at 133mm/hr (Parkin *et al* 1982).

The gauge measuring rainfall was mounted on a pole about 1.5m above the ground to monitor the rain falling on the experimental section. The gauge was located beside the pavement at roughly the mid-point of the eastern side, as far as possible from any trees that might have caused erroneous values to be measured. Its elevation above ground level prevented water splashing up into the gauge causing errors, and reduced the likelihood of debris entering the gauge causing blockages in the mechanism. However, the gauge was still close to the trees (Plate 1), which are mature pines and considerably higher than the gauge. There was a risk of the gauge being affected by pine needles or other debris.

The second gauge was mounted below the outflow pipe leading from the sub-base and designed to be isolated from all other sources of water. The purpose of this gauge was to measure the amount of water flowing from the sub-base.

The third gauge was purpose-built with a capacity of 1.254 litres per tip. This was mounted below the outfall pipe from the surface of the slab to measure the surface runoff. This gauge was larger than the other two as the peak flows would clearly be greater than for either the rainfall or sub-base discharge. The surface area of the pavement was 99.3 m^2, so 1mm of rain would potentially produce 99.3 litres of run-off.

All three gauges were connected to the datalogger. This recorded hourly the number of tips counted in the previous hour.

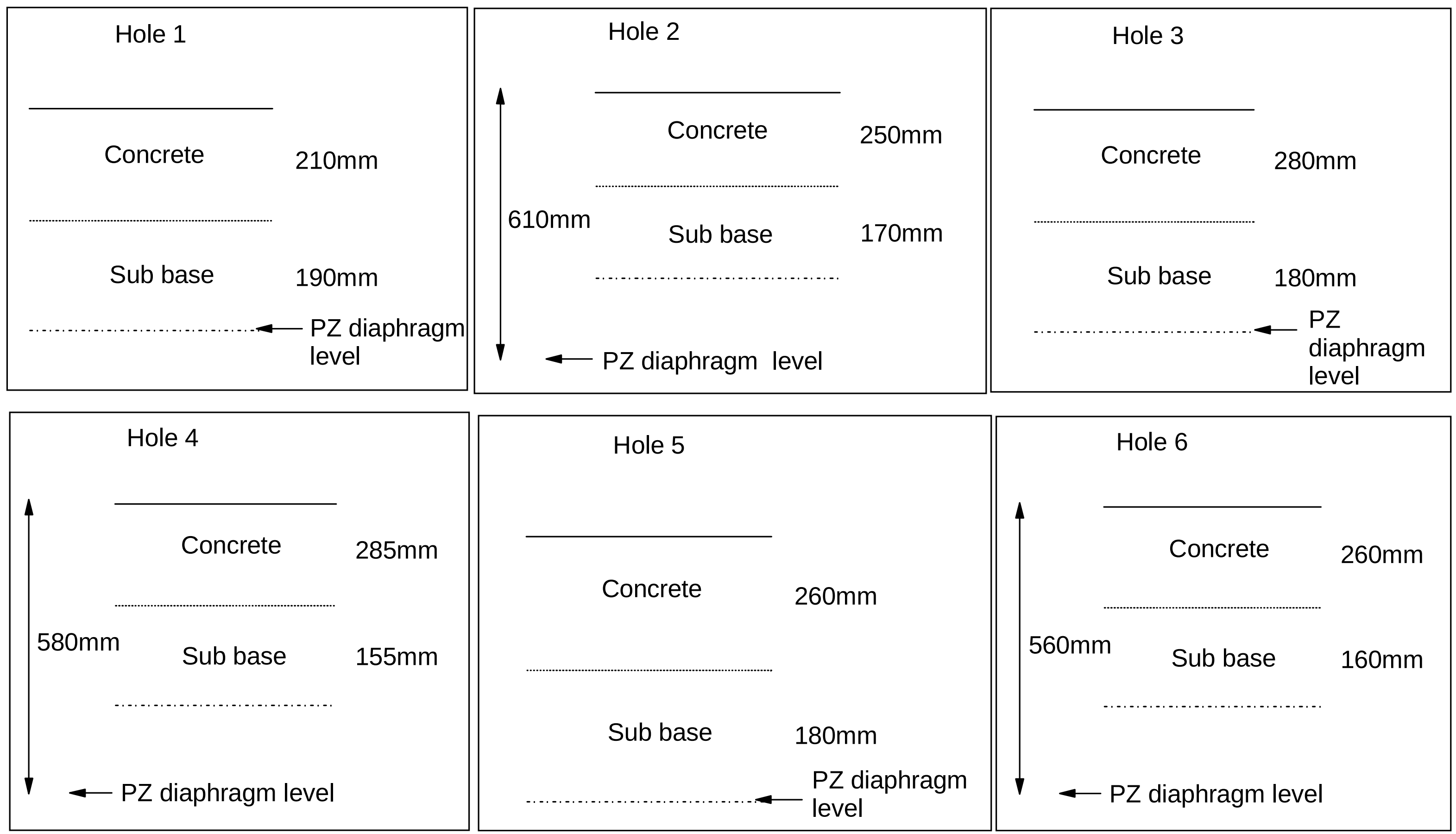

Figure 4.4 Piezometer installation details

4.3.3 Access tubes for neutron moisture probe

The locations of the access tubes for the neutron moisture probe are shown on Figure 4.1. The tubes were made of thin aluminium to allow readings of moisture content to be made in the sub-base. The bases of the tubes had to be 150 mm below the lowest reading point, so it was not possible to take readings in the subgrade clay, in case the tubes punctured the base of the clay layer.

In use, the neutron moisture content probe is lowered to the required depth down an access tube of internal diameter 39.4mm, external diameter 41.4mm. To avoid serious errors in the measured moisture content of the soil, it is essential that the outside of the tube is in intimate contact with the surrounding soil and that all moisture be excluded from the inside. With a wall thickness of one millimetre, it was not possible to thread the tubes and screw sealing plugs into their bases. Neither could blanking pieces be welded to the bases as this might have caused sufficient distortion of the tubes to prevent intimate contact of the outside of the tube with the soil. The manufacturer of the probe explicitly excludes both these methods and suggests that a parallel-sided rubber stopper be sandwiched between two washers and expanded against the side of the tube by tightening a nut and bolt passing through the stopper and washers. Despite considerable effort no suitable stopper could be obtained.

Various options were investigated to plug the base of the access tubes. Several methods were investigated, but were found to leak when subjected to a head of 2.5m of water overnight. The final system consisted of two cylindrical aluminium plugs as shown in Figure 4.5. Assembly consisted of gluing in the first plug and, once the adhesive was cured, applying a crack-sealing compound. A layer of adhesive was then applied and, once set, further crack sealer used. The second plug was then installed and sealing compound applied. A final layer of adhesive was then used to completely cover the second plug. Aluminium plugs were employed to minimise any thermal effects that might cause stresses to be developed in the adhesive/sealant in use. The tops of the tubes were sealed with removable covers to prevent water ingress.

A tube sealed as described was subjected to head of 2.5m of water for a period of several days and no leakage occurred. Once assembled the tubes were individually tested for leaks.

The tubes were installed through holes cut in the concrete pavement, and during excavation through the granular sub base samples were taken for moisture content determination. Holes were then made into the underlying clay to a depth that allowed moisture content readings to be obtained at the bottom of the sub base. The granular material was re-compacted carefully by hand around each tube to prevent damage. The concrete surface was then reinstated.

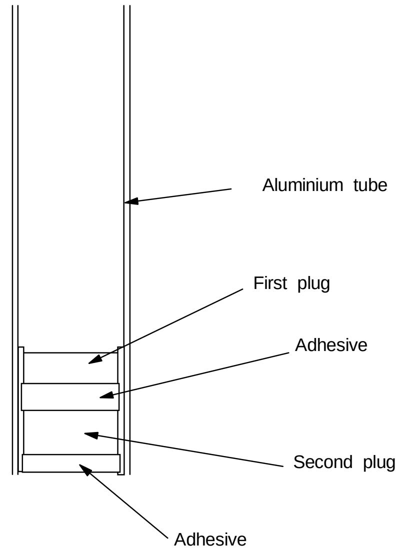

Figure 4.5 Access tube for neutron moisture probe

5 Results of water movement monitoring

5.1 Survey of road profile and area

A levelling and taping survey was carried out to determine the plan area and slopes of the pavement as accurately as possible. The results are given in Figure 5.1. The average fall over the length of 24.5m is 1.29m giving a slope of 1 in 18.95 or 3.0 degrees. The area of the pavement is 99.3 m^2.

5.2 Commissioning of gauges

The rain gauge and the sub-base outfall gauges were commissioned on the 14 March 2001. There was a serious delay in receiving the runoff gauge from the manufacturer, with the result that this was not commissioned until 27 July 2001. During the initial period of monitoring large volumes were recorded by the sub-base outflow gauge. This caused concern that there might be considerable leakage into the collector drain. However, this reduced in the first months as shown in Table 5.1 . In an attempt to reduce possible leakage from the surface above the collector drains, material was removed from this area and a layer of polythene sheeting placed and buried beneath the excavated material on 3 August 2001.

Table 5.1 Reducing sub-base outflow in early months

Start date	Finish date	Rainfall (mm)	Equivalent runoff (l)	Sub-base outflow (l)
14/03/01	31/03/01	73.6	7308	115.7
01/04/01	02/04/01	22.3	2214	42.6
25/04/01	29/04/01			
30/04/01	30/05/01	53.5	5313	1.59

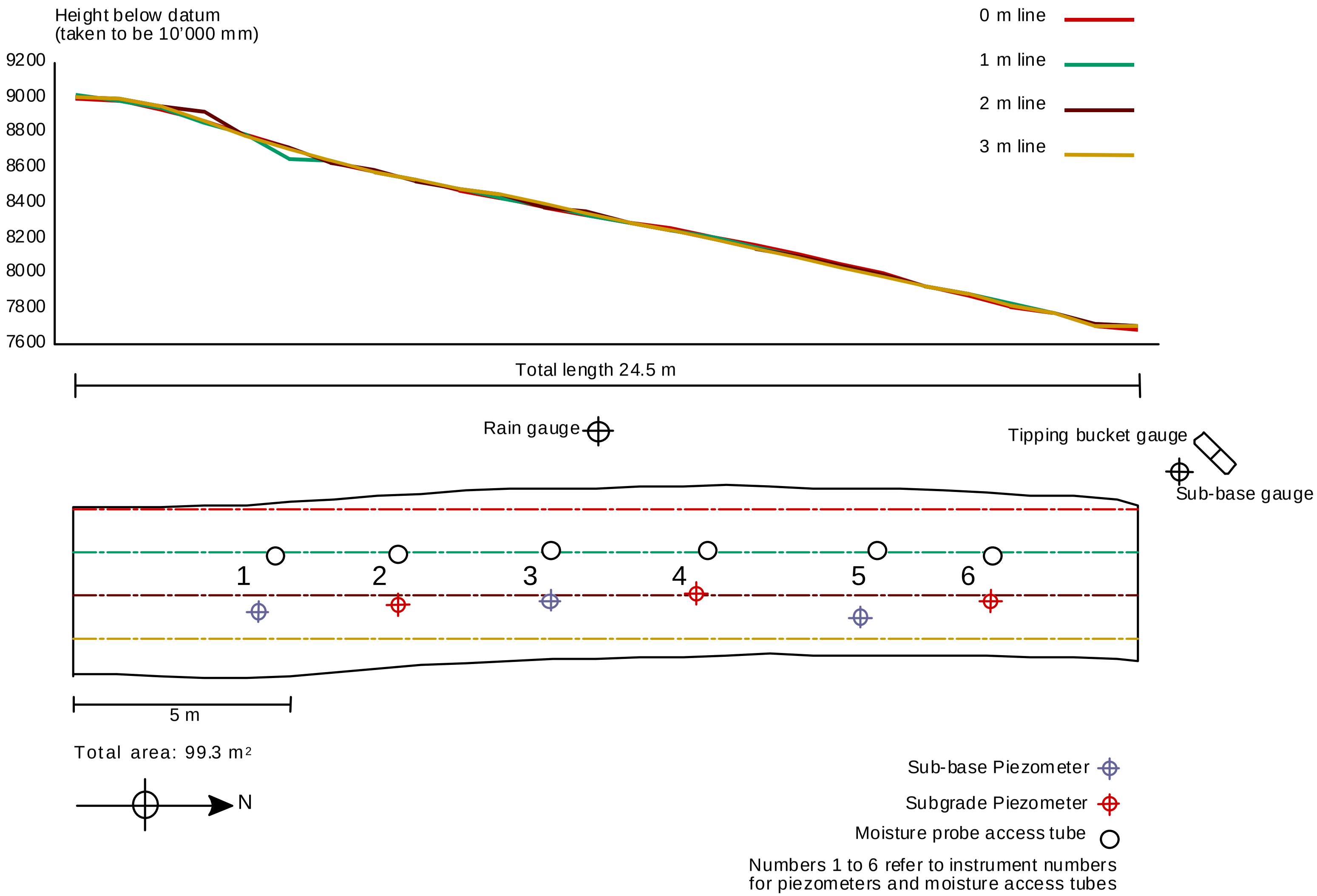

Figure 5.1 Plan and section of trial road

The monthly sub-base flows were very variable after this so it was difficult to assess the effect of the membrane (see Table 5.2). Subsequent to the completion of the project, the sub-base drain was excavated in August 2003 as part of another project. The construction was found to be faulty, and it was evident that the drain had not been functioning properly due to incorrect positioning. It was suspected that leakage may have been occurring behind the headwall and possibly elsewhere. The readings of sub-base flow will thus be less than the actual values. Attempts to assess the actual sub-base drainage flows are described in Section 10.

5.3 Water balance

A recording interval of one hour was chosen to give a reasonable compromise between temporal resolution of events and data storage. This data was transferred from the logger to a laptop computer once a month during most of the trial period. The rain gauges were checked for blockage or improper operation at the same time. The rain gauge collecting direct precipitation was particularly prone to blockage by plant debris from the surrounding trees. This led to the collected water backing up in the gauge funnel, with very slow release to the tipping bucket. Any ponded water was released and measured during checks. However this volume will have been under-measured as the accuracy of the gauge decreases at high flow rates.

Evaporation data was purchased from the Meteorological Office for the months August to November 2001 for the estimation of water lost by this route. This information is derived from measurement of a range of parameters and is available in the MORECS system for a variety of surface types. The figures acquired for this project were those for an urban area, which is assumed to be 100% paved, so this was expected to be good match for the experimental site. The figures are daily totals of “actual evapotranspiration” (AE). AE takes account of the available water, which is low for paved areas. The data provided were calculated for Beaufort Park, Bracknell, which is close to the trial road. Acquisition of this data was discontinued after this initial period because the quantities were relatively small and did not improve the water balance calculation. The outflows measured were in excess of the inflows without the contribution from AE.

All the data was entered into a spreadsheet for analysis. Table 5.2 shows the monthly summary of the water balance calculation for the period August 3 to December 31 2001 and Table 5.3 shows the monthly summary for the period January to December 2002. This is the period for which all the instruments were functioning.

For completeness, the data for the period before the run-off gauge was functioning in 2001 are included above the double line in Table 5.2. The symbols in the top row are those defined in Figure 2.1. The raingauge data are logged in units of mm of rainfall. The logger converts the number of tips recorded to rainfall in mm by multiplying by the bucket constant of 0.1mm. The rainfall in mm has been multiplied by the catchment area of the pavement ($99.3m^2$) to yield the “Equiv litres” (P) figure in the table. The volume per tip of the sub-base gauge is 2ml so the number of tips recorded has been multiplied by $2x10^{-3}$ to give the collected volume in litres (V). The surface run-off gauge has a volume-per-tip of 1.254 litres. The recorded number of tips has been multiplied by this to give the run-off volume (R). The next columns contain the MORECS data in mm and the equivalent evaporated volume when multiplied by the pavement area (E). Inflows were to have been totalled as the sum of the raingauge equivalent volume (R) and the sub-base water content change (dS). In view of the uncertainties involved it was felt that calculations of dS would not be sufficiently reliable. Therefore the change in water content of the sub-base has been left at zero.

Outflows have been totalled as the sum of the sub-base flow, the run-off and the evaporation. As indicated in Section 5.2, the recorded sub-base flows underestimate the actual values. Whilst it is customary to calculate I/P in this instance R was found to be a more reliable measurement and hence I/R has been used.

Table 5.2 Water balance for 2001

Symbol			P		V	R	E		dS	P + dS	V + R + E	(V+dS)/R	
Note	Start timedate	Finish Timedate	Rain gauge (mm of rain)	Equiv litres	Sub base (Litres)	Surface run off (Litres)	Morecs AE mm	Eq Litres	Increase in water content (Litres)	Inflows (Litres)	Outflows (Litres)	Difference (Litres)	I/R Ratio (%)
Runoff gauge commissioned - present	27/07/01 10:00	30/11/01 23:00	324.6	32,233	9.46	36,335.90	15.7	1,559	0	32,233	37,904	- 5,672	0.026
14 to 31 Mar	14/03/01 11:15	31/03/01 23:00	73.6	7,308	115.66					7,308	115.7		
April 1-2, 25-31	01/04/01 00:00	29/04/01 23:00	22.3	2,214	42.64					2,214	42.6		
May	30/04/01 00:00	30/05/01 23:00	53.5	5,313	1.59					5,313	1.6		
June	31/05/01 00:00	29/06/01 23:00	29.6	2,939	0.05					2,939	0.0		
July-Aug 3	30/06/01 00:00	03/08/01 10:00	53.2	5,283	5.85					5,283	5.9		
August-3 on	03/08/01 11:00	31/08/01 23:00	62.2	6,176	0.11	5,049	6.7	665	0	6,176	5,714	462	0.002
September	01/09/01 00:00	30/09/01 23:00	74.0	7,348	0.03	6,689	9.0	894	0	7,348	7,583	- 234	0.000
October	01/10/01 00:00	31/10/01 23:00	160.2	15,908	3.49	19,982	11.1	1102	0	15,908	21,088	- 5,180	0.017
November	01/11/01 00:00	30/11/01 23:00	25.4	2,522	0.01	2,494	7.1	705	0	2,522	3,199	- 677	0.000
December	01/12/01 00:00	31/12/01 23:00	23.6	2,343	3.77	3,102	0.0	0	0	2,343	3,106	- 763	0.122
TOTALS (Aug 3 - Dec 31)			345.4	34,298	7.41	37,317	33.9	3366	0	34,298	40,690	- 6,392	0.020

Table 5.3 Water balance for 2002

Symbol			P		V	R	E		dS	P+dS	V+R+E	(V+dS)R
			Rain gauge				Morecs AE		Increase in water content (litres)			
Month	Start timedate	Finish timedate	(mm of rain)	Equiv litres	Sub-base (litres)	Surface run off (litres)	(mm)	Eq litres		Inflows (litres)	Outflows (litres)	I/RRatio (%)
January	01/01/2002 00:00	31/01/2002 23:00	*77.9*	7735	1.25	8954	0.00	0	0	7735	8955	0.014
February	01/02/2002 00:00	28/02/2002 23:00	*89.7*	8907	0.012	9972	0.00	0	0	8917	9972	0
March	01/03/2002 00:00	31/03/2002 23:00	*51.9*	5154	0.002	6680	0.00	0	0	5154	6680	0
April	01/04/2002 00:00	30/04/2002 23:00	*42.8*	4250	0.640	4677	0.00	0	0	4250	4878	0.014
May	01/05/2002 00:00	30/05/2002 23:00	*82.1*	8153	0.002	8485	0.00	0	0	8153	8485	0
June	01/06/2002 00:00	31/06/2002 23:00	78.3	7725	3.304	8069	0.00	0	0	7725	8073	0.041
July	01/07/2002 00:00	31/07/2002 23:00	102.4	10168	0.294	12385	0.00	0	0	10168	12385	0.002
August	01/08/2002 00:00	31/08/2002 23:00	86.6	8599	1.224	12596	0.00	0	0	8599	12598	0.010
September	01/09/2002 00:00	30/09/2002 23:00	21.5	2135	7.716	2233	0.00	0	0	2155	2241	0.345
October	01/10/2002 00:00	31/10/2002 23:00	73.3	7279	1.646	5248	0.00	0	0	7279	5250	0.031
November	01/11/2002 00:00	31/11/2002 23:00	192.4	19105	1.610	26965	0.00	0	0	19105	26966	.0060
December	01/12/2002 00:00	31/12/2002 23:00	*141.3*	14031	5.848	20699	0.00	0	0	14031	20704	0.043
TOTALS			1040.2	103292	23.54	126963	0.00	0	0	103271	127187	0.027

Note that rainfall figures in italics are substitutions from the Wooden Hill raingauge during periods when the TRL raingauge was clearly not reading correctly, often due to being blocked by debris. Comparisons when the gauge was functioning correctly demonstrate that the two gauges were in close agreement.

The only months in which the inflows exceeded the outflows were August 2001 and October 2002. This must cast some doubt on the accuracy of the measurements. Possible sources of this are under-reading of the precipitation raingauge, and runoff from the surroundings penetrating or overtopping the kerbs surrounding the pavement. At the beginning of November 2001 the following work was carried out in an attempt to eliminate the latter possible causes.

- Excess soil behind the kerbs at the sides of the road was removed to form a drainage channel.
- A drainage channel was dug around the south-east corner to prevent ponding of run-off against the southern (uphill) kerb and possible overtopping.
- A leak in the same curb was sealed with clay.

However the excess of outflows over inflows has not abated since then as can be seen in Table 5.4.

Table 5.4 Measured outflows as a percentage of measured inflows

2001		2002			
Month	**%**	**Month**	**%**	**Month**	**%**
-	-	Jan	116	July	122
August	93	Feb	112	August	146
September	103	Mar	130	Sept	105
October	133	April	110	October	72
November	127	May	104	November	141
December	133	June	104	December	122

Some extra confidence in the readings of the raingauge was gained by comparing them with some freely available, privately measured, monthly rainfall data for a nearby location at Wooden Hill, Bracknell. This is provided on a website by a Meteorologist/Weather Forecaster employed by the UK Met.Office. Table 5.5 shows that, given that the variability between sites even this close together is likely be quite high, there is no evidence that, in months in 2001 when the TRL gauge was operating correctly, it was consistently under-reading. There was a greater incidence of problems with blockage of the raingauge in 2002. In these cases the data from Wooden Hill was substituted for the rain gauge data, as the evidence showed that they were broadly similar.

Table 5.5 Comparison of TRL readings with Wooden Hill

2001		
Month	**Wooden Hill (mm)**	**TRL (mm)**
August	73.2	62.2
September	66.3	74.0
October	129.0	160.2
November	40.3	25.4
December	22.3	23.6
2001 TOTAL	331.1	345.4

A further figure of note is that over the five-month period August to November 2001, rainfall equivalent to 894 litres of run-off from the area of the road was measured by the rain-gauge during hourly intervals when no actual run-off was measured. It must be assumed that all of this rainfall evaporated. It is interesting to compare this with the evaporation figures from MORECS for the months where this data has been obtained. It seems reasonable to assume that the most of the volume in the right-hand column of Table 5.6 would have evaporated, but that there would be little more water available for evaporation. The reason for the MORECS figures being so much higher than the right-hand column of Table 5.6 is not known, although an unknown further amount of water would be expected to evaporate from the surface during and after rainfall events.

Table 5.6 Evaporation compared with rainfall while run-off = zero

Month	MORECS AE (litres)	Calculated runoff equivalent of measured rainfall, while runoff = zero (litres)
August	665	298
September	894	100
October	1,102	278
November	705	218
TOTAL	3,366	894

The measured sub-base outflow has been very small. As shown by Table 5.2 the total for August to December 2001 was 7.4 litres, which represents 0.020% of the 37317 litres of run-off. The total for January to December 2002 was 23.5 litres, which represents 0.027% of the 126,962 litres of run-off. As indicated in Section 5.2, the recorded flows underestimate the actual sub-base flows by an unknown amount.

5.4 Piezometer data

Numbering from the top of the slope piezometers 1, 3 and 5 were installed in the granular sub-base and 2 and 4 were installed in the clay below the sub-base. Piezometer 6 was also installed in the clay subgrade but failed soon after installation. The depths of the piezometers are given in Table 5.7.

Table 5.7 Piezometer installation depths

Piezometer	Depth below road surface (m)	Depth below clay surface (m)	Reduced level at pavement surface (m)	Level relative to P5 (m)
1	0.40	-	8.757	+0.938
2	0.61	0.19	8.569	+0.390
3	0.46	-	8.393	+0.484
4	0.58	0.14	8.228	+0.059
5	0.44	-	8.029	0.000
6	Failed	-	7.889	-

Figure A1 shows the recorded porewater pressure in kPa for the sub-base piezometers and Figure A2 those in the clay subgrade for the whole measurement period.

The shapes of the curves for the three sub-base piezometers are very similar, with P3 and P5 almost coincident. Although P1 is at the top of the slope it consistently showed a pressure about 2kPa higher than the other two (representing a head difference of 0.2m). This was not expected given the slope of the trial road and the free-draining nature of the sub-base (see Section 6) and may represent an offset error in P1. Pressures on all three piezometers were highest in the winter months, declining in the summer periods by about 2kPa in 2001 and 3kPa in 2002. There was an overall reducing trend to the pressures, which were about 1.5kPa lower by December 2002 than they had been at the same time in 2001. The traces show rapid fluctuations of up to about 3kPa throughout the monitoring period. There are also some outlying points, which are thought to be spurious. The readings of P3 and P5 were mainly negative (suction).

These results show that:

- the hydraulic gradient may be steeper in the upper part of the slope between P1 and P3,
- between P3 and P5 the hydraulic gradient is parallel to the pavement slope,
- the hydraulic gradient does not vary seasonally and
- the phreatic surface varied in height seasonally by about 0.2m
- except for P1 most of the readings were negative, representing unsaturated conditions

The two working piezometers in the clay showed a much greater disparity, although, again, the traces are similar in overall shape. P4 read as high as 29kPa in March 2001, followed by a steady decline to about 22kPa in early September. After this the trend was fairly level until March 2002, when a steady decline set in until the end of October, after which there was a slight rise. P2, which is higher up the slope, initially read about 4kPa, declining to about –3kPa by September 2001 and then rising a little. After this the trend is very similar to that of P4. The readings of P2 are thought unlikely to be correct, because they represent a phreatic surface which is up to 2.9m above the piezometer and 2.3m above the pavement surface. Table 5.7 shows that this piezometer is closer to the top surface of the clay layer than P2. It is therefore unlikely that the high readings are due to artesian pressure beneath the clay. The shape of the trend of P2 being very similar to that of P4 suggests that the piezometer is responding correctly to changes in pressure. It is therefore likely that there is a large offset error on this piezometer. In this case the trends in pressure will be correct even though the absolute values indicated may not be accurate.

5.5 Moisture content

The neutron probe moisture content readings from the six holes are plotted against time in Figures A3 to A8, and against depth in Figures A9 to A13. Readings were taken at the same six distances from the bottom of each hole. These positions were as given in Table 5.8 and shown graphically in Figure 2.1.

Table 5.8 Neutron probe measurement positions

Depth from bottom (m)	Depth from surface (m)					
	Hole 1	Hole 2	Hole 3	Hole 4	Hole 5	Hole 6
0.000	-0.434	-0.427	-0.452	-0.472	-0.455	-0.448
0.075	-0.359	-0.352	-0.377	-0.397	-0.380	-0.373
0.150	-0.284	-0.277	-0.302	-0.322	-0.305	-0.298
0.225	-0.209	-0.202	-0.227	-0.247	-0.230	-0.223
0.300	-0.134	-0.127	-0.152	-0.172	-0.155	-0.148
0.375	-0.059	-0.052	-0.077	-0.097	-0.080	-0.073

These depths correspond to close to the top, middle and bottom of each of the concrete and sub-base layers, although in holes 3 to 6 the lowest reading point is well into the clay subgrade. Readings were taken on 5 July, 9 August, 5 September, 5 October, 9 November and 23 November in 2001 and on 16 May, 24 June, 9 July, 07 August, 6 September, 31 October and 26 November in 2002.

The neutron probe does not measure at a discrete depth, but over a radius of neutron penetration and reflection, which may be as large as 150mm. Readings closer to the surface than this are likely to be erroneously low. Each reading therefore represents the average moisture content over a layer of about 300mm thickness, the diameter of this "sphere of influence". The neutron probe gives readings of volumetric moisture content, which will be greater than the moisture content by mass obtained by weighing samples of the material before and after drying. The relationship between volumetric moisture content (θ) and moisture content by mass (w) for a soil is given by:

$$\theta = w / (w + 1/G_s) \qquad \text{where } G_s = \text{specific gravity of solid particles}$$

If Gs = 2.7, which is the normal value for soils, then

$$\theta = w / (w + 0.37)$$

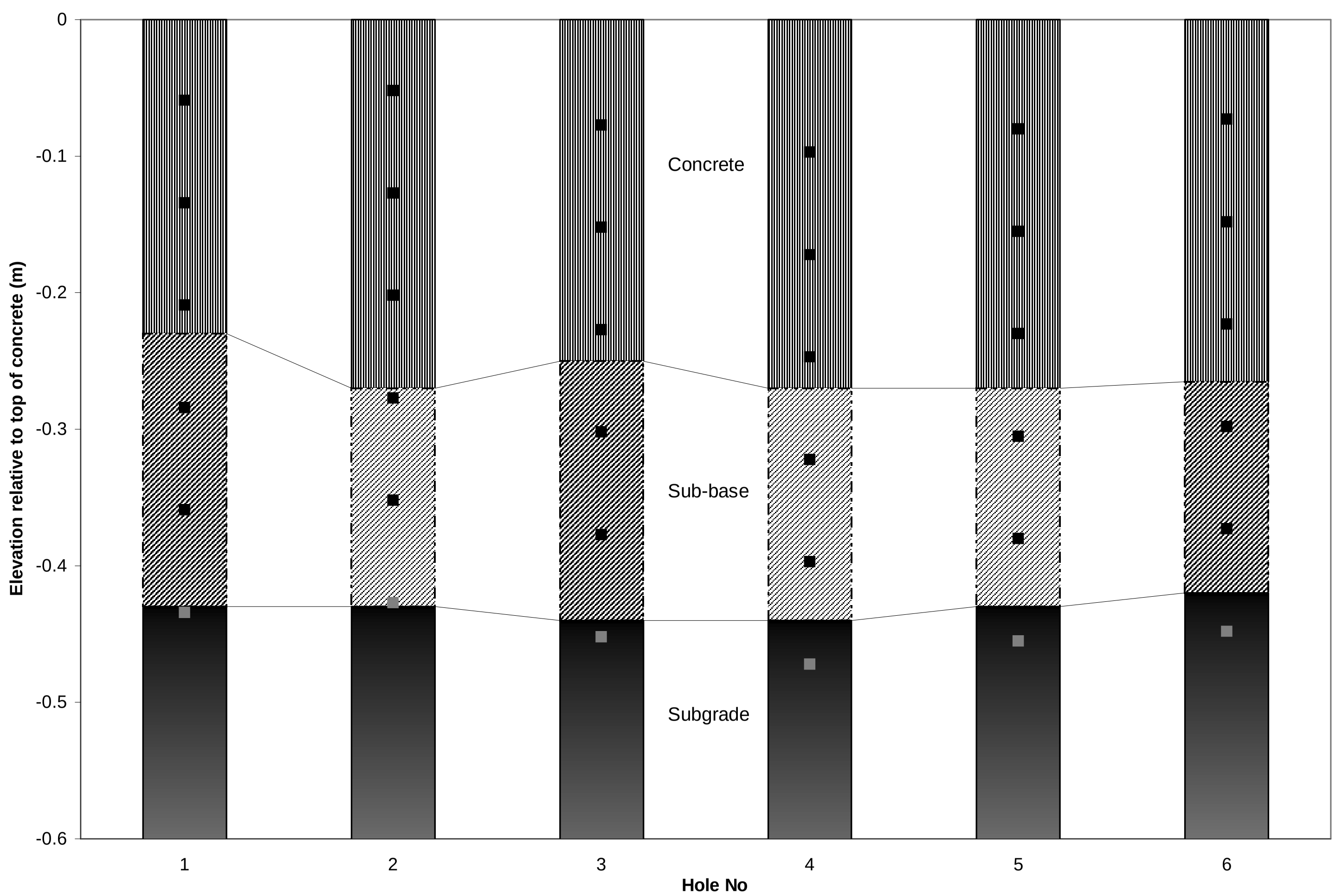

Figure 5.2 Neutron probe measurement points

The curves of volumetric moisture content versus depth (see Appendix A) show a similar "S" shape in each hole, which has not varied much between readings. The indicated moisture content increases rapidly from about 10% close to the surface of the concrete to about 20% at a depth of about 0.2m. This is followed by a slight fall to the bottom of the slab at about 0.26m. The indicated volumetric moisture content reaches a minimum of around 15% (approximately 6.5% by mass) in the sub-base and then rises again towards the top of the clay subgrade. There may have been larger short-term variations in moisture content, as shown by the piezometers, but to detect this would have required an impractical frequency of readings.

6 Laboratory experiments

Laboratory-based experiments were carried out to determine the permeability of the concrete, sub-base and subgrade, and the composition of the rainfall, the run-off and the sub-base drainage.

6.1 Permeability of the concrete

Two 150mm diameter cores were taken and prepared for testing in a pair of triaxial cells. Each specimen was fitted with a porous plate and 150mm diameter top and bottom caps with water connections. A double sealing membrane was fitted over this assembly, which was then inserted in the cell. The specimens were subjected to a cell pressure of 395kPa to seal the membrane against the core. A variable inlet pressure was applied to the top of each specimen, while the water exiting the bottom cap was collected and measured. Only one of the tests proceeded to completion, because of problems with the sealing of the membrane on the second specimen. This test was continued until the flow rate became constant, and was stopped after 455 hours.

The test that was completed gave a permeability measurement of 3.62×10^{-11} m/s (Figure 6.1). This represents a very impervious pavement. If it was possible to keep the $99.3m^2$ pavement continuously saturated, and assuming a continuous hydraulic gradient of unity, this would correspond to a total of 0.31 litres of water passing through per day. However in practice the surface of the pavement will often be dry, and it is likely that much of the concrete will not be saturated. This will considerably reduce the likelihood of water penetrating to the base of the slab. As discussed in Section 2.3.1 van Ganse (1981) reported zero infiltration through a pavement when the average permeability was 10^{-9} m/s or less.

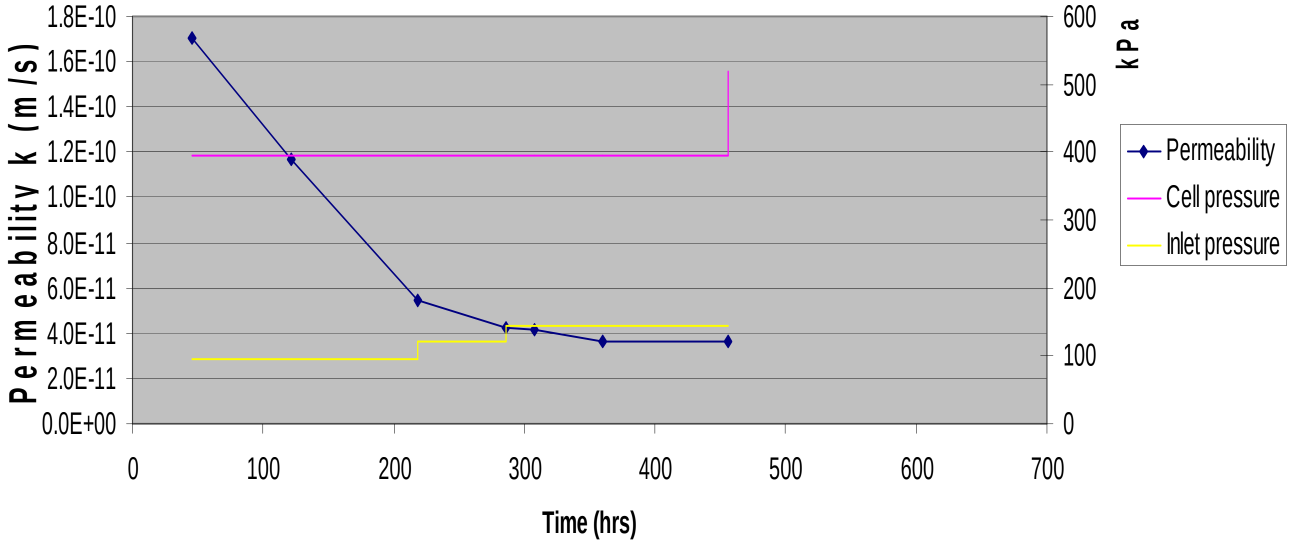

Figure 6.1 Results of pavement concrete permeability investigation

6.2 Sub-base and subgrade properties

Table 6.1 gives the results of the moisture content by mass and index tests that were carried out on the sub-base and the clay subgrade. The samples were taken when the holes for the instrumentation were excavated, but the instrumentation was not installed until later, so there is not a direct correlation between the two sets of readings. The moisture content by mass of the sub-base was in the range 4.1 to 6.1%. These values are close to those estimated from the volumetric moisture content readings from the neutron probe (Section 6.1) and are what would be expected for an unsaturated granular material complying with the grading requirements for Type 1 sub-base. The moisture content of the underlying clay is much higher, ranging from 37.5% to 41.8%, suggesting some ponding of water on top of this stratum.

Also shown on Table 6.1 are the D10 and D60 particle sizes, and the Uniformity Coefficient D60/ D10. The grading curves shown in Figure 6.2 indicate the sub-base material broadly complies with Type 1 grading requirements. The sample size was restricted because it was derived from the material excavated from the 150mm diameter holes drilled through the slab for the piezometers, hence it is less than required to obtain a truly representative grading. In particular, it is likely that the coarse particles have been underestimated because of the sample size and method of collection, and the in-situ material may correspond more closely to the Type 1 grading requirements.

Table 6.2 gives approximate permeabilities of the sub-base calculated using five different empirical methods. The first of these is Hazen’s formula, which was developed for sand filters with a particle size in the 0.1mm to 3mm range. A coefficient C appropriate to coarse but well-graded sand was used.

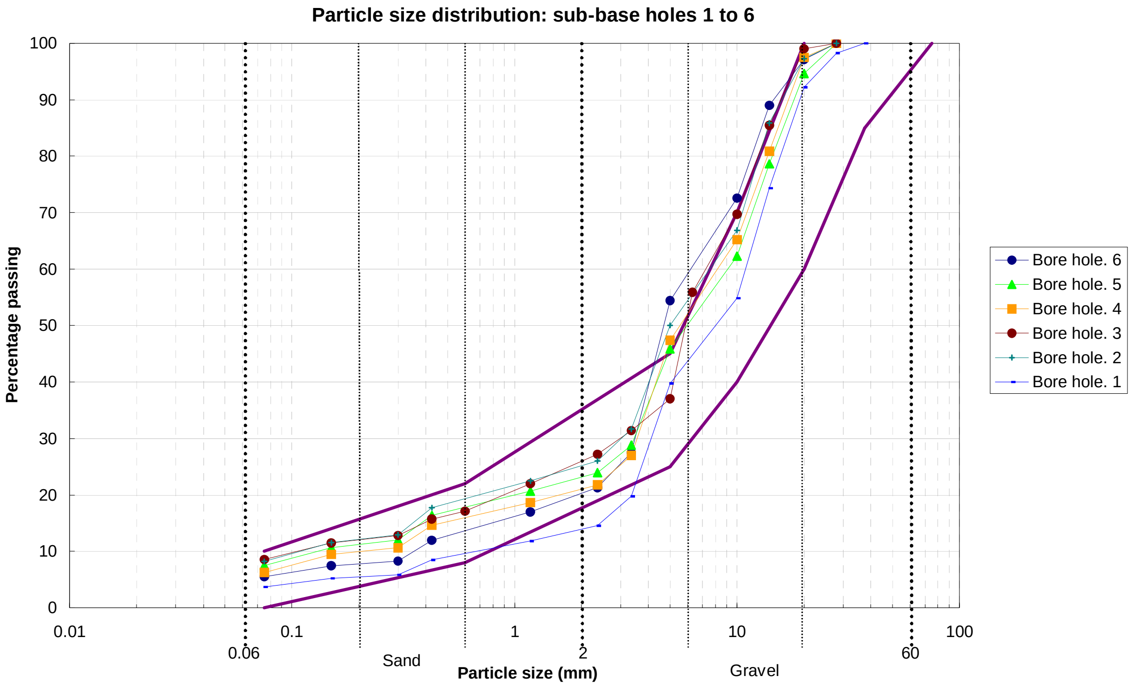

Figure 6.2 Sub-base grading curves – limits shown are for Type 1 sub-base

The other permeability estimates are based on the application of empirical relations between permeability and various functions of the shape of the particle distribution curve. These were established by Zohrabi and Temporal (2001) for a range of granular materials. There is a wide scatter apparent in the values determined by these approximate methods. Being for a similar material, the permeability calculated from Uniformity Coefficient for crushed limestone should be the most reliable indication of the permeability of the trial road sub-base.

Table 6.1 Results of index tests on soil samples

Hole No.	Material	Moisture content by mass (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	D_{10} (mm)	D_{60} (mm)	UC (D_{60}/D_{10})
1	Sub-base	6.1				0.70	10.5	15
	Clay	39.4						
2	Sub-base	5.0				0.11	7.5	68
	Clay	37.5	67	26	41			
3	Sub-base	5.9				0.11	7.2	65
	Clay	38.2						
4	Sub-base	4.1				0.20	8	40
	Clay	37.9	65	23	42			
5	Sub-base	5.2				0.12	9	75
	Clay	39.7						
6	Sub-base	4.5				0.35	6	17
	Clay	41.8	74	26	48			
Trench	Clay	44.3	69	27	42			
Natural ground	Clay	37.6	48	21	27			

Table 6.2. Estimates of permeability of sub-base from grading of samples

Sample no.	Permeability from Hazen's formula (m/s)	Permeability from UC (crushed limestone)[1] (m/s)	Permeability from UC (all soils)[1] (m/s)	Permeability from C_z [1] (m/s)	Permeability from modified shape factor [1] (m/s)
1	4.9×10^{-03}	2.0×10^{-03}	6.2×10^{-04}	3.8×10^{-05}	8.1×10^{-04}
2	1.2×10^{-04}	5.0×10^{-04}	5.8×10^{-05}	1.2×10^{-03}	6.6×10^{-04}
3	1.2×10^{-04}	5.2×10^{-04}	6.2×10^{-05}	1.3×10^{-03}	7.2×10^{-04}
4	4.0×10^{-04}	8.2×10^{-04}	1.3×10^{-04}	9.5×10^{-04}	8.3×10^{-04}
5	1.4×10^{-04}	4.5×10^{-04}	5.0×10^{-04}	1.2×10^{-03}	5.9×10^{-04}
6	1.2×10^{-03}	1.8×10^{-03}	5.0×10^{-04}	7.4×10^{-04}	1.5×10^{-03}

[1] after Zohrabi and Temporal (2001)

UC Uniformity coefficient

Cz Coefficient of curvature

Hazen's formula: $k = C(D_{10}^2)$; where k is the coefficient of permeability in m/s; C is a coefficient (0.01 in this case) and D_{10} is the sieve size in mm through which 10% passes.

The following empirical relations were established by Zohrabi and Temporal (2001).

- Permeability and Uniformity Coefficient, for crushed limestone and for all soils investigated.
- Permeability and curvature (C_z).
- C_z is an alternative expression for describing the shape of the particle size $C_z = \left(\frac{D_{30}{}^2}{D_{10} \times D_{60}}\right)$ distribution curve given by
- Permeability and Modified Shape Factor for all soils investigated $\left(\frac{D_{60}}{D_{10} + D_{30}}\right)$

Although there is a wide scatter on the empirically determined permeabilities of the sub-base, it is clear that it is unlikely to be below 5 x 10^{-4}m/s. This represents a free-draining material compared with the seven orders of magnitude lower permeability of the concrete.

The London Clay subgrade and trench fill material was found to be clay of high plasticity (CH) using the system on Figure 18 of BS 5930 (British Standards Institution, 1999) (LL 69, PL 25, PI 44, LI 32). It can be expected to have a permeability of less than 10^{-10}m/s and so can be treated as impermeable. A sample of the natural strata at the site, the Bracklesham Beds, was found to be a clay of intermediate plasticity (CI). The clays were generally in a firm or soft to firm condition.

6.3 Chemical analyses of rainfall, run-off and leachate

Samples of rainfall, pavement run-off and sub-base outflow were collected on three occasions and sent for chemical analysis. The first samples were collected on 10 October 2001 and the second samples were collected on 10 December 2001, apart from the sub-base sample, which was collected on 5 December 2001. The third samples were collected during November 2002 prior to the commencement of the tracer study.

Material for the laboratory leaching tests was obtained during the excavation of the neutron probe and piezometer holes.

6.3.1 Field samples

The rainfall and sub-base outflow samples were collected in sample bottles over a few days from the drainage tubes of the tipping bucket gauges. The runoff sample was collected from the residual water in the flow-meter bucket.

6.3.2 Laboratory tests

Two types of leaching test were performed on the limestone sub-base material. One was a NordTest Column Method test (1995) and the other was a CEN two-stage batch leaching test (1997), now BS EN 12457-3. These tests were recommended for assessing the environmental properties of materials used in road construction in the final report of the ALT-MAT project (Reid *et al*, 2001). The column test is a characterisation test, designed to reproduce the conditions the material will be exposed to in the field, i.e. slow percolation of water. The batch test is a compliance test, designed to give a rapid indication of whether leaching is likely to be a problem for the material. In the column test a known weight of material is lightly tamped into a cylindrical cell which is then sealed. Water is forced through the column of material from top to bottom. A fixed flow rate of 12ml/hr was achieved using a small peristaltic pump. The volume passing is measured and samples of the leachate are collected when the liquid to solid (L/S) ratio of the efflux is 0.1, 0.2, 0.5, 1.0 and 2.0 (l/kg).

In the CEN test a weighed sample of material is first sealed in a container with sufficient water to give an L/S ratio of 2 and tumbled for 6 hours. The sample is filtered and the leacheate is taken for chemical analysis. Water is added until the total L/S ratio is 10. The sample is tumbled for a further 18 hours, filtered, and a further sample of the water taken for analysis.

A sample of leachate from the permeability test on the concrete was also sent for chemical analysis, and the results are shown on Table 6.3.

6.3.3 Results

Table 6.3 gives the results of all the chemical analyses of the water samples. All of the samples had pH values in the neutral range (pH6-pH8). To put the measured concentrations in context, none of the concentrations of the substances measured exceeded the threshold permitted in UK drinking water. This is not a surprising result, given that there were no deleterious materials present.

The conductivity measurements show that the sub-base outflow and the surface runoff had higher conductivities than the rainfall. This shows that these flows have taken ionic compounds into solution during their passage over the concrete pavement and through the sub-base. Compared with the incident rainfall the outflows contained more sulfate, calcium, sodium and potassium and alkalinity. As would be expected the sub-base outflow contained the highest concentrations. There was no clear general increase or reduction in concentration with time. Values are similar to those expected for surface water in the UK.

In the CEN tests the two samples of the limestone sub-base material (from Hole 1 and Hole 3) gave similar results. The concentrations of all the detectable substances were lower than in the field sub-base outflow, even at L/S = 2l/kg (6 hour). At L/S = 10 l/kg (18 hr) levels were lower still. The decrease in concentration between the two stages of the test suggests that most of the species in solution were washed off the surface of the particles in the initial stage. Only aluminium and silica tend to remain at constant concentration in both stages, although the concentrations are very low.

In the NordTest column method the concentrations were lower still, even at an L/S ratio of 0.1 l/kg. There was no agreement between the two test methods at the common L/S ratio of 2. As with the CEN test, the concentrations of most species decrease with time, suggesting they are being washed off the surface of the particles. For calcium and alkalinity, concentrations remain constant or decrease only slightly, suggesting that at the slow flow rate of this test the concentrations are maintained by slow dissolution of the limestone. The pH also remains constant, suggesting that the solution is buffered by the limestone.

The low concentrations of dissolved solids in the leaching tests are consistent with the nature of the sub-base material, which is crushed limestone. The higher concentrations in the sub-base drainage and surface runoff may arise from various sources:

- Contact with the concrete
- Contact with silt and organic matter on the road surface
- Contact with the underlying clay
- A longer period of time of contact between the percolating water and the sub-base material, allowing more material to come into solution

While all these factors may have contributed, the most important is likely to be contact with the concrete. This can be seen from the analysis of the leachate from the permeability test on concrete (Table 6.3). The leachate has higher values of alkalinity, chloride, sulfate, calcium, sodium and potassium than the rainfall and the leaching tests on the sub-base, and the values are similar to those recorded in the samples of surface runoff and sub-base drainage. The higher concentrations in the sub-base drainage are consistent with longer periods of time in contact with the concrete than the surface runoff.

The chemical analyses thus indicate that the flows of water in the system are consistent with the model set out in Section 2.1.

Table 6.3 Results of chemical analysis of leachates

TRL tests				Independent laboratory tests															
Sample	Date	pH	Conductivity mS/m	pH	Conductivity µS/m	Alkalinity as $CaCO_3$ mg/l	Chloride mg/l	Sulfate as SO_4 mg/l	Ca mg/l	Na mg/l	K mg/l	Pb mg/l	Mn mg/l	Fe mg/l	Al mg/l	Nitrate mg/l	TOC mg/l	Si mg/l	Ammoniacal nitrogen as N mg/l
Field samples																			
Raingauge	10.10.01	7.3	31	7.3	<100	8	2	1.88	3.67	1.47	.71	.04	N/A	0.02	N/A	N/A	5	0.8	0.3
Raingauge	10.12.01			7.0	165	3	3	6.17	3.48	1.84	1.12	N/A	N/A	N/A	N/A	N/A	2.8	1	0.1
Raingauge	11.02			6.5	<100	<2	8	3.04	1.99	3.94	0.42	<0.03	0.01	0.01	<0.01	<0.5	3.1	<0.1	<0.1
Runoff gauge	15.10.01	6.6	239	7.9	249	92	3	18.6	31.9	5.75	12.6	0.03	N/A	0.01	N/A	N/A	14	2.7	0.2
Runoff gauge	10.12.01			7.8	344	130	3	33.9	52.2	4.51	7.9	N/A	N/A	N/A	N/A	N/A	9.5	2.2	N/A
Runoff gauge	11.02			7.8	211	81	4	12.5	29.7	3.33	3.3	<0.03	<0.01	0.01	<0.01	<0.5	4.8	0.8	0.3
Sub-base raingauge	10.10.01	6.9	379	8.2	363	96	8	70.4	48.3	10.3	9.31	N/A	N/A	N/A	N/A	0.07	6.5	1.7	0.2
Sub-base raingauge	5.12.01			7.3	230	79	5	26.6	32.1	4.77	6.31	N/A	N/A	0.15	0.08	N/A	8.3	1.6	0.2
Sub-base raingauge	11.02			8.0	591	231	7	53.7	81.9	17.2	10.9	<0.03	0.01	0.01	<0.01	1.7	8.9	0.9	0.4
Cen 2 Stage Leaching Test																			
H1 6hr	11.10.01	7.5	169	I S	I S	43	14	13.8	20	7.94	4.87	0.03	N/A	N/A	0.1	0.6	8.2	1.2	0.4
H1 18hr	12.10.01	7.5	86	8.4	N/A	37	N/A	2.19	14.9	0.84	0.95	N/A	N/A	N/A	0.46	N/A	1.9	1.2	0.2
H3 6hr	11.10.01	7.5	172	I S	I S	51	I.S	13.4	25.3	8.64	4.6	N/A	N/A	N/A	0.16	0.5	5.1	1.8	0.4
H3 18hr	12.10.01	7.5	80	8.4	N/A	32	N/A	2.42	13.2	0.9	0.42	N/A	N/A	N/A	0.19	N/A	1.6	1.4	0.2
NordTest Column Method																			
L/S 0.1	18.10.01			7.8	<100	12	N/A	5.97	9.5	3.24	3.65	N/A	N/A	N/A	0.12		2.9	0.8	N/A
L/S 0.2	19.10.01			7.7	<100	10	N/A	4.42	8.93	1.74	2.69	N/A	N/A	N/A	0.14		3.2	0.8	N/A
L/S 0.5	22.10.01			7.8	<100	26	N/A	3.67	9.23	1.2	2.44	N/A	N/A	N/A	0.14		3	0.9	N/A
L/S 1.0	27.10.01			7.7	<100	23	3	1.7	7.82	0.56	1.72	N/A	N/A	N/A	0.19		1.5	0.8	N/A
L/S 2.0	06.11.01			7.8	<100	20	6	0.49	6.78	0.27	1.03	N/A	N/A	N/A	0.14		1	0.4	N/A
Permeability Tests																			
Perm				7.3	394	100	38	37.3	47.9	18.9	4.01	N/A	.0.56	N/A	0.01	0.8	25	1.8	1.7

NOTE

N/A: not available, IS: insufficient sample

7 Ground probing radar study

In an attempt to gain more information about the variations of moisture content with plan position under the slab and time, both resistivity imaging and ground probing radar were considered. A review of available resistivity techniques and equipment and discussions with two universities were carried out. Following this, it was judged unlikely that resistivity probing from the surface would yield any information about the moisture content of the sub-base, through the concrete slab. Ground probing radar (GPR) was then selected as the most promising method, based on earlier work performed at TRL. It was expected that GPR, using the most appropriate wavelength, would be able to penetrate the slab and the sub-base to sufficient depth. It was recognised that, without detailed calibration for the precise materials present, it would not be possible to measure the absolute moisture content of the materials, but only any change with time. This change would alter the dielectric constants of the materials and be seen by the radar as an apparent change of depth to discernible interfaces between the structural layers. No other property changes with time, which could change the dielectric constant, were expected.

Two ground probing radar surveys were performed. One was on 6 August 2002 at the end of a period of dry weather, and the other on 25 November 2002 at the end of a wetter period. This was also after the cracking of the pavement described in the next section. There was a measurable difference between the two surveys, which indicated a 15% overall increase in the moisture content of the sub-base. This correlates with a decrease of approximately 2kPa in the suctions recorded in the sub-base piezometers over this period (see Figure A1). A fuller description of the theory of GPR and its application to this project is provided in Appendix B.

8 Cracking of pavement

Because the flow from the sub-base was negligible during the first five months it was decided to increase the effective permeability of the pavement by inducing cracks in it once a year of monitoring in the un-cracked condition was complete. The study of existing work reported in Section 2 indicated that for pavements of low permeability, cracks were likely to be the main method of water infiltration. Information on the nature of cracks in typical trafficked pavements was sought to help with the decision on the spacing and width of the cracks to induce. Two main methods of cracking the trial pavement were considered. These were:

- a falling-weight guillotine as used to “crack and seat” old concrete pavements and
- the drilling of holes, followed by expansion or “bursting”.

The guillotine method was found impractical as the pavement was obstructed by instrumentation tubes and pipes, and a small-enough machine could not be located. A hydraulic bursting machine was chosen and the crack inducing was carried out on 11th September 2002. This method required the drilling of 50mm diameter holes into the pavement using a rotary percussive drill and then inserting the burster, which used the “plug and wedge” principle to generate a large horizontal force within each hole. It was found that operating the burster in a single central hole was sufficient to burst a transverse crack the full width of the pavement. In all 5 cracks were formed at approximately 3.6m spacing. Three of these were in the un-reinforced part of the pavement and two in the reinforced (Figure 8.1). The crack widths were surveyed immediately after cracking and found to average 0.76mm in the un-reinforced section and 0.22mm in the reinforced section. The cracks in the reinforced

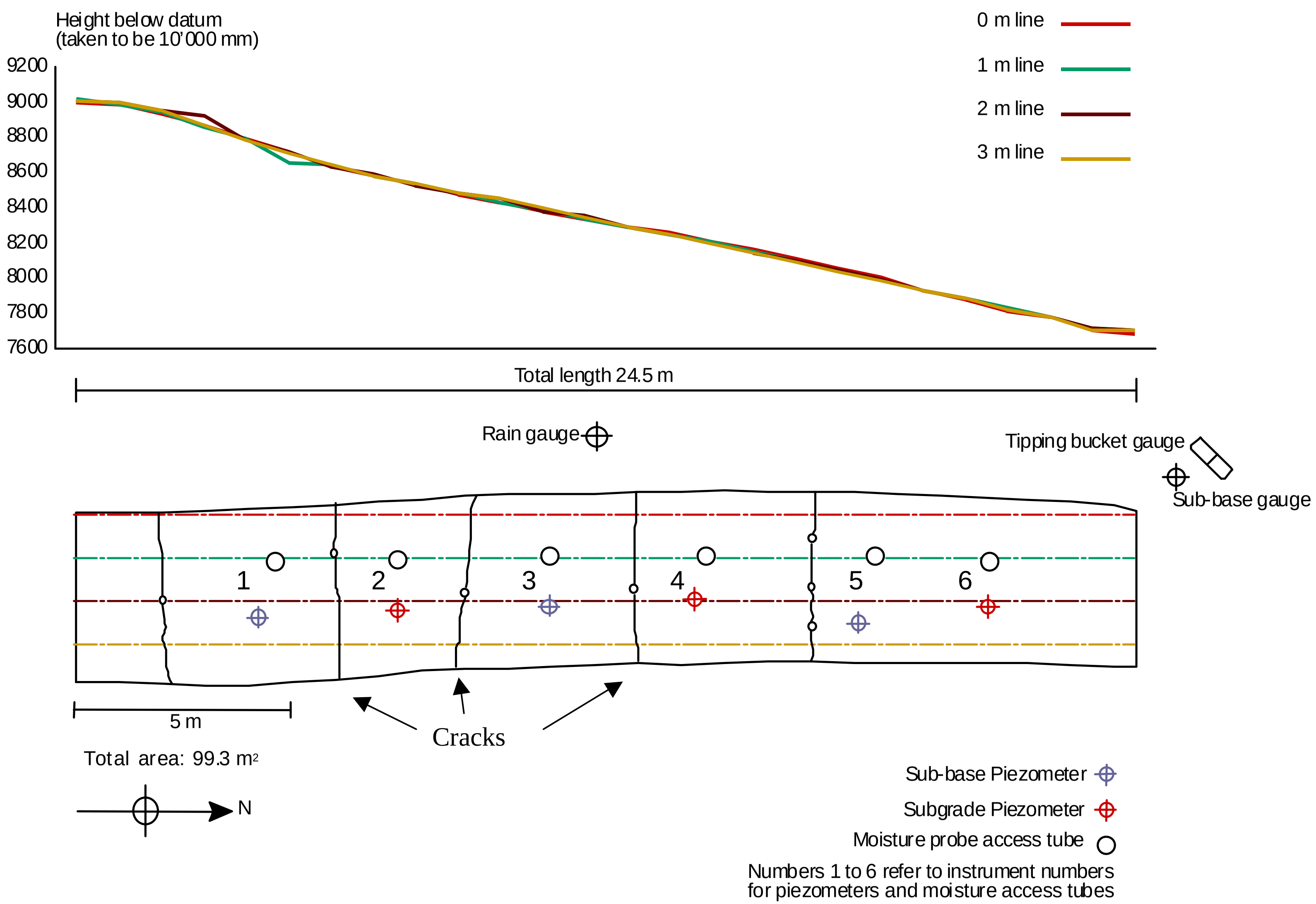

Figure 8.1 Plan of pavement showing crack positions

section remained tightly closed due to the action of the reinforcement. Finally the bursting holes were filled with cement mortar to seal them against water ingress.

All the instruments were then monitored until the end of December 2002 to determine if there was any change in the sub-base outflow. The data in Table 5.3 show that there was no significant increase in the sub-base outflow after the cracking was induced, despite a period of particularly heavy rainfall in November and December 2002. This appeared to demonstrate that the hydraulic conductivity of cracks of this size was very low. However, given the subsequent discovery that the sub-base drain was not functioning correctly, this conclusion cannot be made with certainty. This is discussed in more detail in Section 10.

9 Tracer study

To determine whether any surface water could be detected at the sub-base outflow a tracer study was carried out. A Campbell CS547A temperature and electrical conductivity probe was acquired, which would interface with the existing logger. The range of the conductivity probe was from $0.005mScm^{-1}$ to $7.0mScm^{-1}$. For comparison, this range runs from about five times the conductivity of distilled water to one fifth the conductivity of seawater. The probe and the supplied software were tested prior to installation. The logger was set to acquire data every minute during the recording period.

The probe, which was of a flow-through design, was attached to the outlet from the sub-base tipping-bucket gauge in such a way that all the flow would pass through it. To ensure that the bore of the transducer was always full of water it was arranged so that the outlet was slightly higher than the inlet.

Sixty litres of saturated salt solution was spread on the surface of the pavement on 30 January 2003. The outflow from the sub-base was then monitored for 24 days. It was anticipated that if salt solution reached the outflow there would be an increase in conductivity of successive readings, possibly followed by a fall. During the recording period, 3480 litres of surface runoff, equivalent to 35mm rainfall, were recorded. Because of programming or possibly hardware, problems the logger failed to record the raingauge data during this period. However it is believed that the conductivity data was correctly recorded. Unfortunately, during the whole of this period, only 2ml (one bucket tip) of sub-base outflow was recorded, at 19:06 on 2 February. This prevented any analysis of any variation in the conductivity of the outflow.

10 Comparison of model predictions for cracked pavements with observations

The information obtained from the literature review suggested that very little infiltration would be expected through an uncracked pavement with a permeability as low as that measured for the trial road (3.6×10^{-11} m/s), see Table 2.1. This is consistent with the very low observed sub-base flows, though the sub-base flows were underestimated because of problems with the drain. This aspect is discussed in Section 11.

The literature review recommended that the work presented by Baldwin *et al* (1997) would provide the best starting point from which to consider infiltration. This work depends heavily on the infiltration rate through pavement cracks estimated by Ridgeway (1976). For concrete pavements the suggestion was that a figure of 2.8×10^{-3} l/s per metre run of crack should be used. This also approximately agreed with McCullough *et al* (1975) who provided data showing an infiltration rate of 2×10^{-3} l/s/metre run of 0.25mm wide crack (see Section 2.3.2).

Details of the lengths and measured widths of the cracks induced in the TRL trial pavement are given in Table 10.1. The main rainfall events that occurred in the post-crack period are tabulated in Table 10.2. From these the three giving rise to the highest total rainfall were chosen for analysis. It was not possible to estimate infiltration from the difference between incident rainfall and run-off, because of two factors that were present even before the pavement was cracked. First, the run-off measured was commonly greater than that predicted by the measured rainfall. Second, often no run-off was recorded in hours when the rainfall was light. It is expected that a large part of this rainfall will have wetted the surface without significant surface flow taking place and then evaporated.

Table 10.1 Crack length and area

Crack number	Length (m)	Mean width (mm)	Crack area (mm^2)
1	4.1	0.90	3600
2	4.0	0.33	1300
3	4.1	1.05	4200
4	3.9	0.33	1300
5	3.9	0.10	400
Total	20.0	0.54	10800

Table 10.2 Rainfall events

Event no.	Start date	Rainfall (mm)	Sub-base out flow (ml)	Measured run-off (l)	Run-off predicted from rainfall (l)	Duration (hrs)
1	12/10/2002	3.1	0	24	308	11
2	13/10/2002	19.9	0	617	1976	32
3	15/10/2002	21.9	1236	2474	2175	24
4	21/10/2002	8.9	408	1238	884	30
5	25/10/2002	9.7	0	631	963	12
6	01/11/2002	52.0	786	7831	5164	70
7	08/11/2002	19.0	0	2001	1887	32
8	11/11/2002	63.0	574	8378	6256	69
9	21/11/2002	55.0	246	8505	5461	190
10	30/11/2002	10.8	2	1130	1072	42
11	04/12/2002	5.0	0	682	496	13
Totals		268.3	3252	33511	26642	525

Table 10.3 shows the comparison between the crack infiltration calculated using 2.8 x 10^{-3} l/s per metre run of crack and the sub-base outflow that was measured. It is seen that in all cases the predicted infiltration capacity is higher than the total rainfall volume falling on the pavement. This strongly suggests that the infiltration figure proposed by Ridgeway is grossly in error.

Table 10.3 Predicted infiltration compared with sub-base flow

Event number	6	8	9
Duration (hours)	70	69	190
Rainfall (mm)	52	63	55
Rainfall volume (l)	5164	6256	5461
Predicted infiltration (l)	9172.8	1.39E+07	3.83E+07
Measured sub-base outflow (l)	0.786	0.574	0.246

11 Discussion

The small volume of water flowing from the sub-base in the first phase of the experiment (pre-cracking) suggested that the overall condition of the pavement slab was very good and that little, if any, water entered the slab through the sealing trenches or the clay layer beneath the sub-base. It was subsequently discovered that there was some leakage from the sub-base drain. However, given that the outflows expected from the models were several orders of magnitude higher than those measured, it is not believed that the difference could be accounted for by leakage.

Cracking of the slab during September 2002 produced no significant increase in the ratio of flow from the sub-base to run-off. The following three month period did produce an increased flow from the sub-base, however the volume of water running off the slab also increased approximately in proportion (see Table 5.3).

The fall from top to bottom of the slab, of about 1.2m over its length of 24.5m, caused most of the water falling on the slab to run off the surface more quickly than it would on a less-steep gradient. This ensured that there was never any standing water on the slab, which ensured that the hydraulic head available to drive water into the cracks was at a minimum. It is notable that in the work described in Section 2.3.2, from which the infiltration rate of 2.8 x 10^{-3}l/s/metre run of crack was derived, water was ponded on the surface and the rate of head-loss measured. The conditions beneath the slab are not described. It is likely that this would result in this estimate of crack infiltration being excessively high.

The amount of precipitation recorded as falling on the slab was significantly less than would be expected from the measured surface run off. There are several possible explanations for this. The site was surrounded by trees, which could have affected the rainfall distribution across the site. The raingauge was blocked on several occasions by debris and spiders entering the gauge and interfering with the tipping of the bucket. Although the rain gauge was fitted with heating elements no mains power was available leading to periods during the winter when the tipping bucket mechanism was frozen up. During periods of heavy rainfall some ponding may have occurred at the top end of the site leading to overtopping of the kerbs, though this was never observed, even soon after heavy rain. The sub-base outflow gauge and the large surface runoff gauge have performed well.

On three occasions during 2002, inexplicably large volumes of water were measured flowing from the sub-base, in each case within a single hour. These were not included in the analysis as their nature was anomalous and they did not have the expected characteristics of water permeating through the concrete slab. Other, presumed “normal” events showed a gradual build up over a few hours, followed by decay, whereas these “abnormal” events each contained an anomalously large flow in a single hour. Some large flows had been observed early in 2001, but since the commencement of full observations in August the phenomena had not been repeated.

The ground probing radar (GPR) study confirmed earlier work which showed that the technique has considerable promise as a method of detecting water under un-reinforced slabs. Depending on its spacing, any steel reinforcement is likely to prevent a useful radar return from the sub-base. In this trial the reinforcement spacing was different in the longitudinal and transverse directions. This allowed the GPR technique to be successful only in the longitudinal direction. Significant changes in the dielectric constant were measured between two surveys separated by five months. However conversion to moisture content change was problematic, as it was not possible to calibrate the GPR equipment against accurately known moisture contents of the materials present. In order to provide more accurate measurements with this technique it would have to be carried out on similar material in the same state of compaction at different moisture contents, so that the variation of dielectric content with moisture content could be established.

The water tracing trial using common salt solution and frequent conductivity measurement proved inconclusive as very little water flowed from the sub-base and it was therefore not possible to detect any variation in the conductivity of the outflow. This should however be a viable technique with higher flows.

12 Indirect estimates of infiltration

The problems with measurement of rainfall and sub-base flows meant that direct measurements of the infiltration through the pavement were not reliable. Attempts were therefore made to estimate infiltration indirectly from analysis of the data that were considered reliable. Four methods were used:

- Analysis of large unexplained sub-base flows (see Section 11)
- Analysis of individual storm events where data was thought to be reliable
- Estimation of sub-base flows from changes in piezometer levels
- Development of a model to estimate leakage from the sub-base drain

12.1 Large unexplained sub-base flows

As mentioned in Section 11, on several occasions large flows from the sub-base drain were observed. It was considered that these might represent the release of water that had been trapped in the sub-base drain since the last event. The sub-base flows could then be compared to the surface runoff and/or rainfall over the same period to obtain a measure of the infiltration, assuming that this was all recorded in the sub-base outflow events. Surface runoff was used as it was considered the most reliable variable, with results expressed as the ratio of infiltration (I) to surface runoff (S).

The largest single sub-base flow event in 2002 was on 9th September, when 7636 ml was discharged in a period of 3 hours. The previous flow from the sub-base was on 21st August. Comparing the sub-base flow to the surface runoff over this period gave an I/S ratio of 0.29%, i.e. the amount of infiltration was 0.29% of the surface runoff. This compares with values of 0.02% to 0.027% from the overall recorded sub-base flows in 2001 and 2002 (Section 5.3).

12.2 Individual storm events

The sub-base flows of 9th September were associated with a major storm event. If it is assumed that the sub-base flows only represent infiltration due to this event, a comparison with surface runoff gives an I/S ratio of 0.37%. As with the previous estimate, this is an order of magnitude higher than that obtained for the annual figures.

12.3 Changes in piezometer levels

The sub-base piezometers show a change of about 2 kPa between summer and winter over the period of the observations from March 2001 to January 2003 (Figure A1). For most of this period, the piezometers were in suction, i.e. indicating negative porewater pressures. However, during 2001 the piezometer at the top of the slope, P1, varied between 0 and +2.0 kPa, roughly corresponding to a head of 0.2 m of water, between summer and winter. If it is assumed that these readings are accurate – and there may be some doubts about this, see Section 5.4 – then this change reflects a flux of water through the sub-base equivalent to a head of 0.2 m. This can be translated into a volume of water by considering the area of the slab and thickness, density and void content of the sub-base, giving a flux of approximately 993 l over the six month period between summer and winter 2001. Assuming this is due entirely to infiltration through the pavement, this gives an I/S ratio of 2.75%.

This model is probably the least reliable, as it involves a number of assumptions and relies on readings from one piezometer for one year. However, it may give a possible upper limit to estimates of infiltration through the pavement.

12.4 Leakage model

A model was fitted to the recorded runoff and sub-base outflow data to estimate the potential unrecorded leakage from the sub-base drain, and hence the total flows from the sub-base. The model assumed that the drainage from the sub-base was into a leaky sump from which outflow occurred only during high flow events, when the sump overflowed. The leakage from the sump was modelled as proportional to the square of the stored volume. The volume is directly proportional to the head driving flow from the sump, and a square law was chosen for the model to reflect that flow will increase in a non-linear fashion with head, being proportionally greater the bigger the difference in head between the sump and the surrounding ground. The measured outflow was modelled as a proportion of the total outflow. Excel’s nonlinear optimization “add-in” was used to find solutions which most closely matched the measured total outflows over the period of the trial. The model went through a series of iterative calculations until the predicted times and amounts of sub-base outflow matched the observed values.

The model gives estimates of I/S ratio from 0.5% to 0.75%, slightly higher than those from individual storm events and large sub-base flows. It probably is the most reliable estimate of infiltration, and is likely to be considerably more accurate than the overall values based only on the recorded sub-base flows. A summary of the results obtained by different methods is given in Table 12.1.

Table 12.1 Estimates of infiltration/surface runoff (I/S) from analysis of the data

Estimation method	I/S (%)
Direct measurements of sub-base flow and surface runoff	0.02 – 0.-027
Large unexplained sub-base flow events	0.29
Individual storm events	0.37
Leakage model	0.5 – 0.75
Seasonal change in sub-base piezometer readings	2.75

13 Conclusions

The study has underlined the importance of understanding the movement of water through roads, and has also highlighted the difficulties involved in measuring the component parts of the water cycle accurately. It has yielded useful information on the amount of infiltration that might be expected in road pavements under UK conditions and identified the possibilities of several techniques for better understanding of the mechanisms and quantities involved. It has identified several aspects that need to be addressed if future trials are to be successful.

There is very little information available in the literature on the infiltration of water through road surfaces. The data that are reported show wide variations and often relate to pavement designs and climatic conditions that are significantly different from those in the UK. This study has added to the knowledge on infiltration into roads and the potential for leaching from the sub-base. It has showed that a cracked concrete pavement with a gradient of 5.5% is unlikely to give rise to the amounts of water infiltrating through cracks in the pavement predicted by the few models which exist. Leaching of any contaminants from the sub-base caused by this source is therefore unlikely. However it should be remembered that there could be other sources of water movement through the sub-base, such as faulty carriageway edge drainage or carrier pipes.

The study utilised a short section of concrete road that had been constructed for an earlier experiment on a different topic. Resources did not permit construction of a bespoke experimental road. This resulted in problems with the measurement of some variables, but allowed useful measurements to be made and techniques such as ground probing radar, tracer studies, chemical analyses of drainage waters and inducing cracks to be trialled. The experiment provided a full year's monitoring of the un-cracked concrete road, which established a baseline from which to damage the pavement in an attempt to increase the infiltration through the pavement.

It was demonstrated in both the field and the laboratory that the permeability of the concrete pavement of the trial road was so low that the amount of infiltration to the sub-base from this source was not significant. During this first (uncracked) phase of the experiment a very small amount of flow was measured at the sub-base outflow, mainly during very intense rainfall. In order to allow the measurement of a significant quantity of water, the pavement was "damaged" by the formation of six full-width transverse cracks in an attempt to increase the infiltration to a measurable proportion of the incident rainfall.

There was no significant change in the ratio of outflow from the sub-base to the runoff from the slab in the three months after cracking. This suggests that the infiltration rates into cracks given by Ridgeway (1976) and Dempsey *et al* (1982) are much too high for a cracked concrete road on a gradient of 5.5%, even if the higher outflows of the leakage model are taken into account.

Chemical analyses and leaching tests have revealed the factors that control the composition of the different types of water. Sub-base outflow and surface runoff water showed higher concentrations of a range of soluble substances than the incident rainfall. This is mainly due to reaction with the concrete pavement. Concentrations were higher in the sub-base outflow than the surface runoff, reflecting the longer time period that the sub-base water was in contact with the concrete. Reaction with the limestone sub-base material seems to have contributed little to the chemistry of the sub-base outflow water.

Problems had been experienced with recording rainfall data due to the proximity of trees to the site. After completion of the trials it was discovered that the sub-base drain was not functioning correctly due to incorrect positioning. However, by using the data that was reliable and employing several indirect approaches, including looking at individual storm events and developing a model to estimate leakage from the sub-base drain, it was possible to obtain an estimate of the infiltration to surface runoff (I/S) ratio for the pavement. The leakage model proved particularly effective, as it enabled predicted flows to be matched to the observed flows by an iterative optimisation technique. From the leakage model, the I/S ratio is likely to lie in the range 0.5 to 0.75%, whereas the values recorded from the actual sub-base flows are of the order of 0.02 to 0.03%.

A particular factor that should be avoided in future trials is siting the trial road near trees or other vegetation that could shed debris into rain gauges and drains. Another factor is siting the trial road on an excessively steep slope, as this increases the risk of inflows from the adjacent area and outflows from the bottom end of the slab at times of intense rainfall. The drainage outfall for the sub-base flows requires particularly close attention during design and construction. Difficulties are likely to be encountered in retrofitting instrumentation to an existing road, and new sections should be built where possible.

The project has given useful insight into the problem, provided some data on infiltration into pavements under UK conditions and demonstrated the potential of several techniques for investigating water movement in roads. For further work in this area, a fully instrumented bespoke trial road should be constructed following the recommendations given in the final report for the ALT-MAT project (Reid *et al*., 2001). This would be much more expensive than the work carried out under this project, and would need to be continued for a number of years to cover the full range of conditions that might occur.

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15 Plates

Plate 1 Trial road showing kerbing, clay trench and raingauge on right

Plate 2 Excavating sub-base from core hole for piezometer installation

Plate 3 Core hole through concrete pavement showing sub-base at bottom

Appendix A. Additional figures

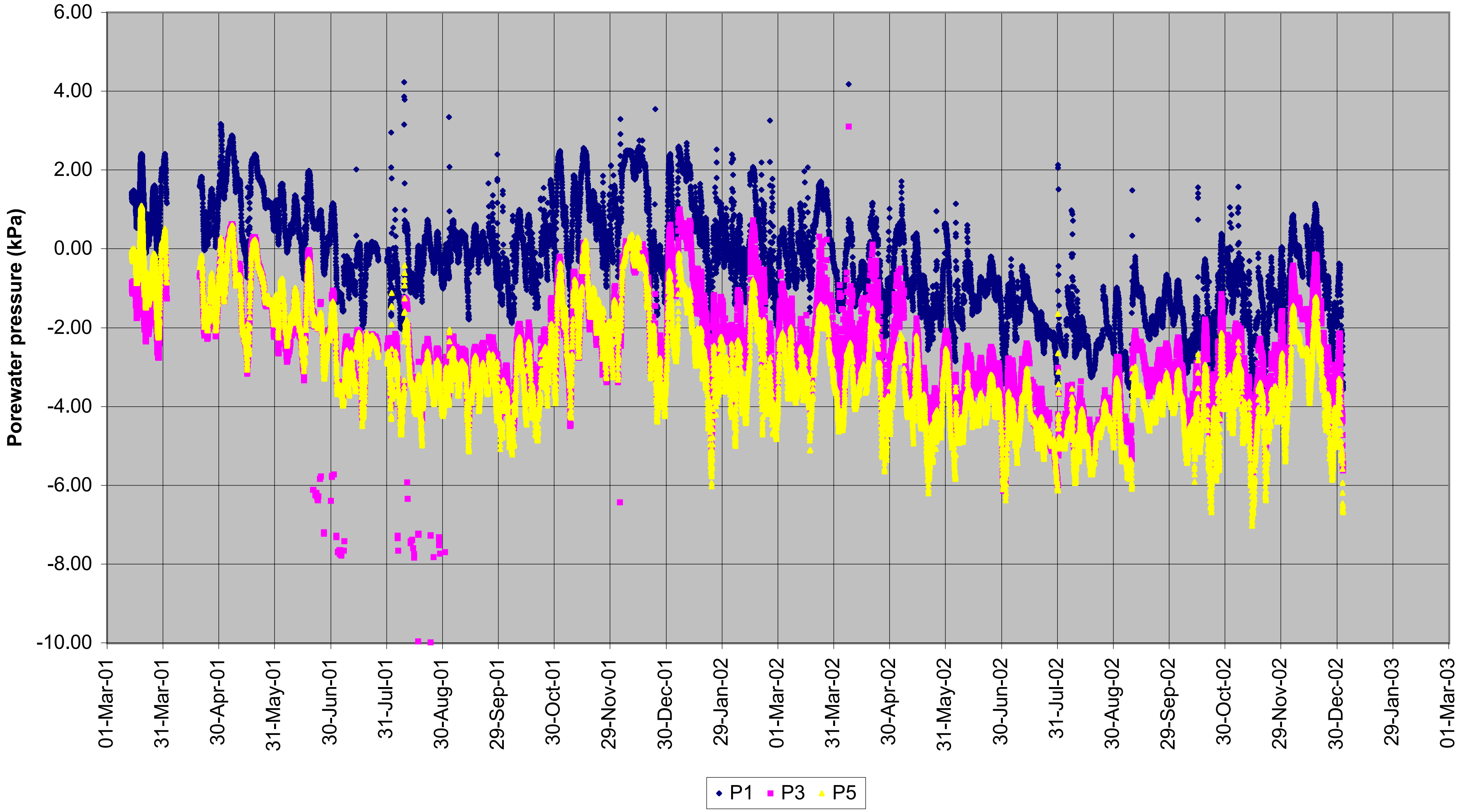

Figure A1 Porewater pressures in the sub-base

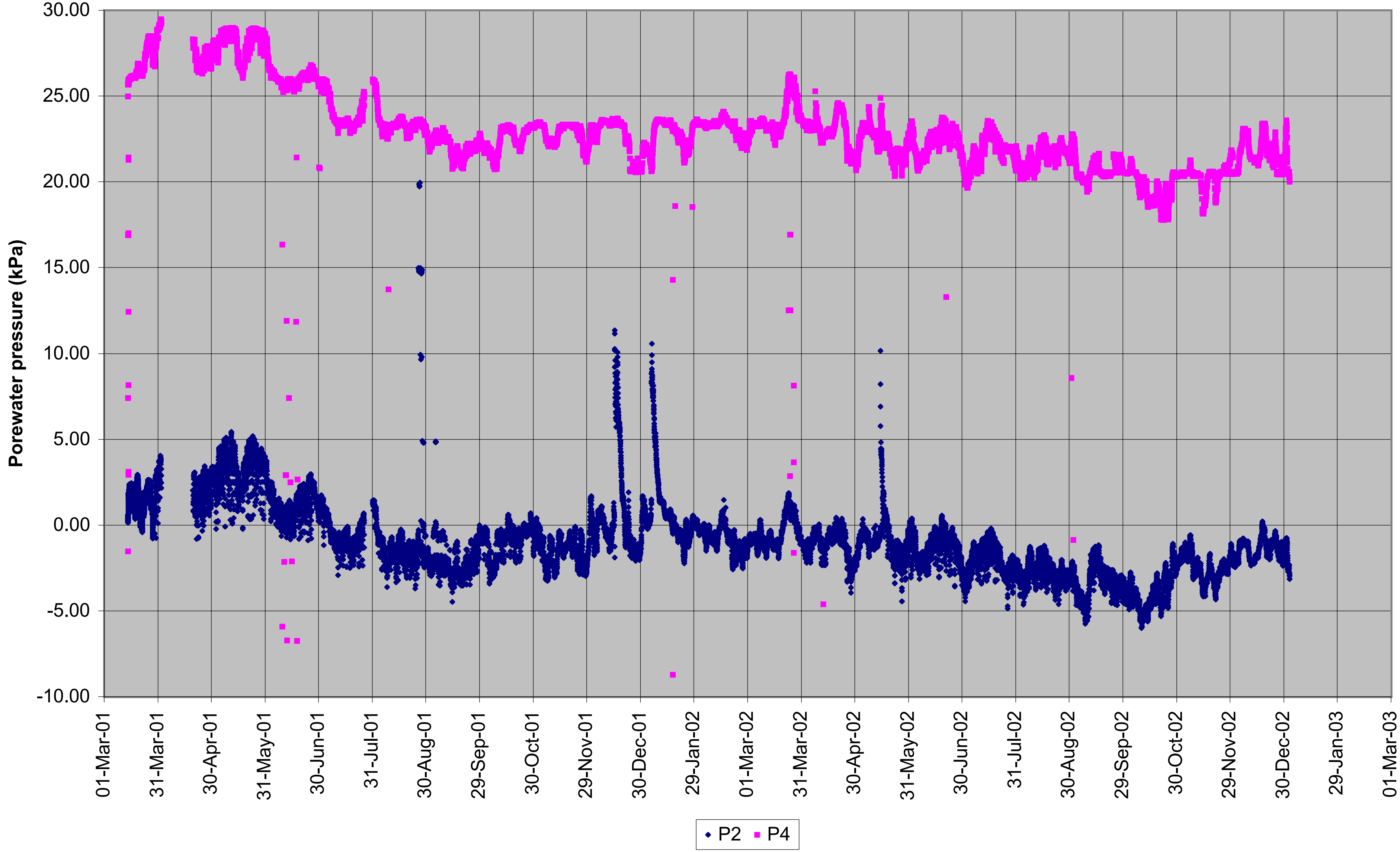

Figure A2 Porewater pressures in the clay subgrade

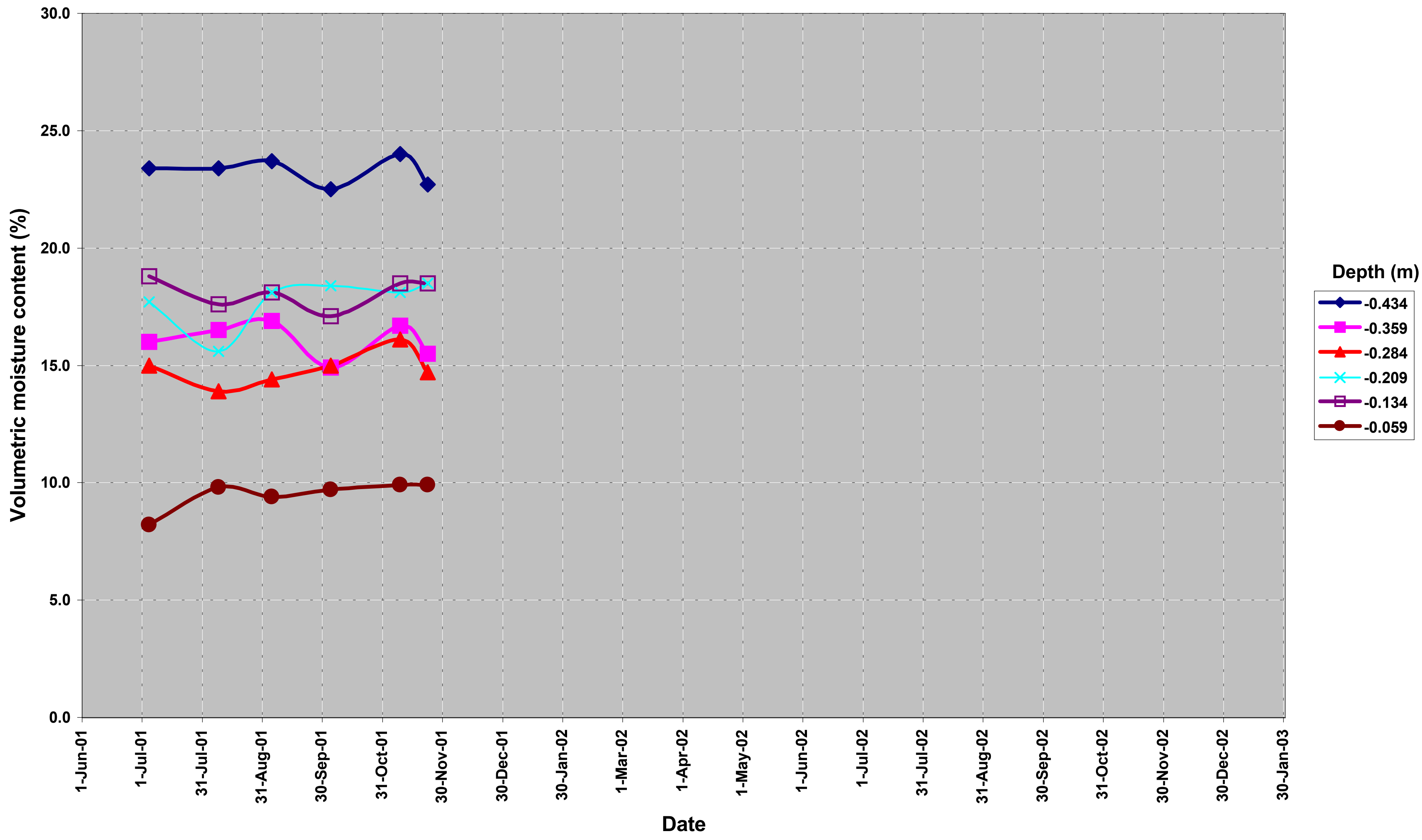

Figure A3 Moisture content readings in Hole 1

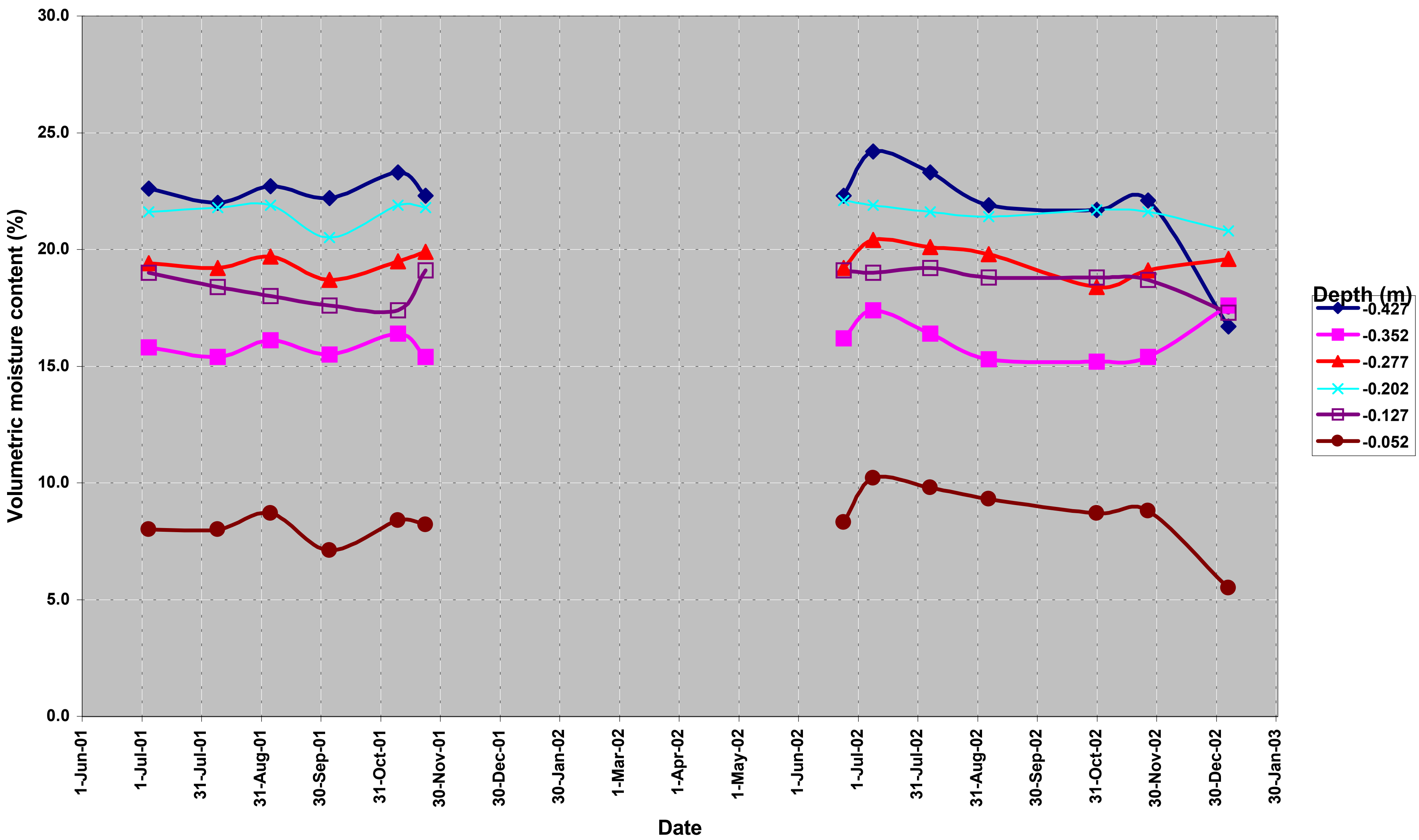

Figure A4 Moisture content readings in Hole 2

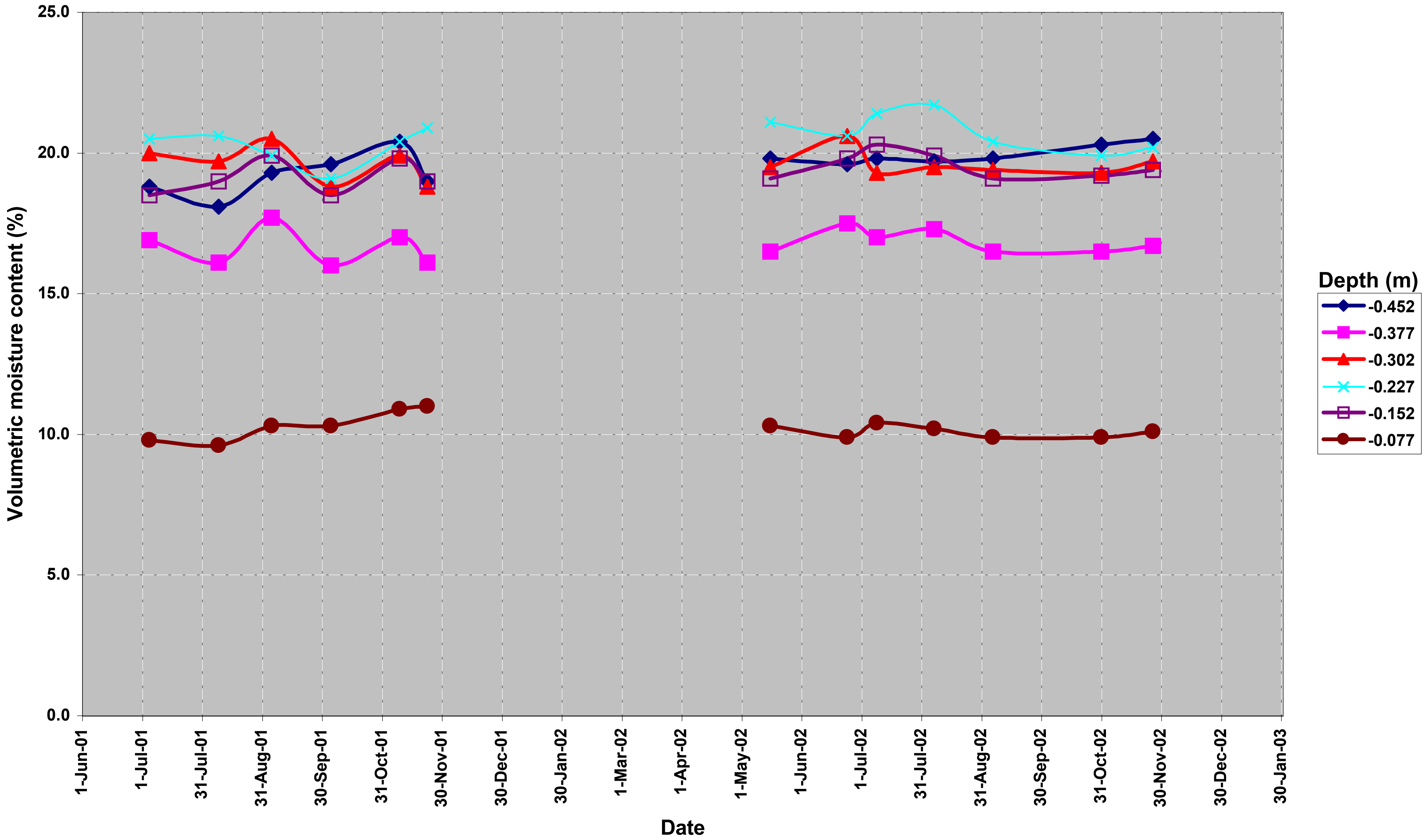

Figure A5 Moisture content readings in Hole 3

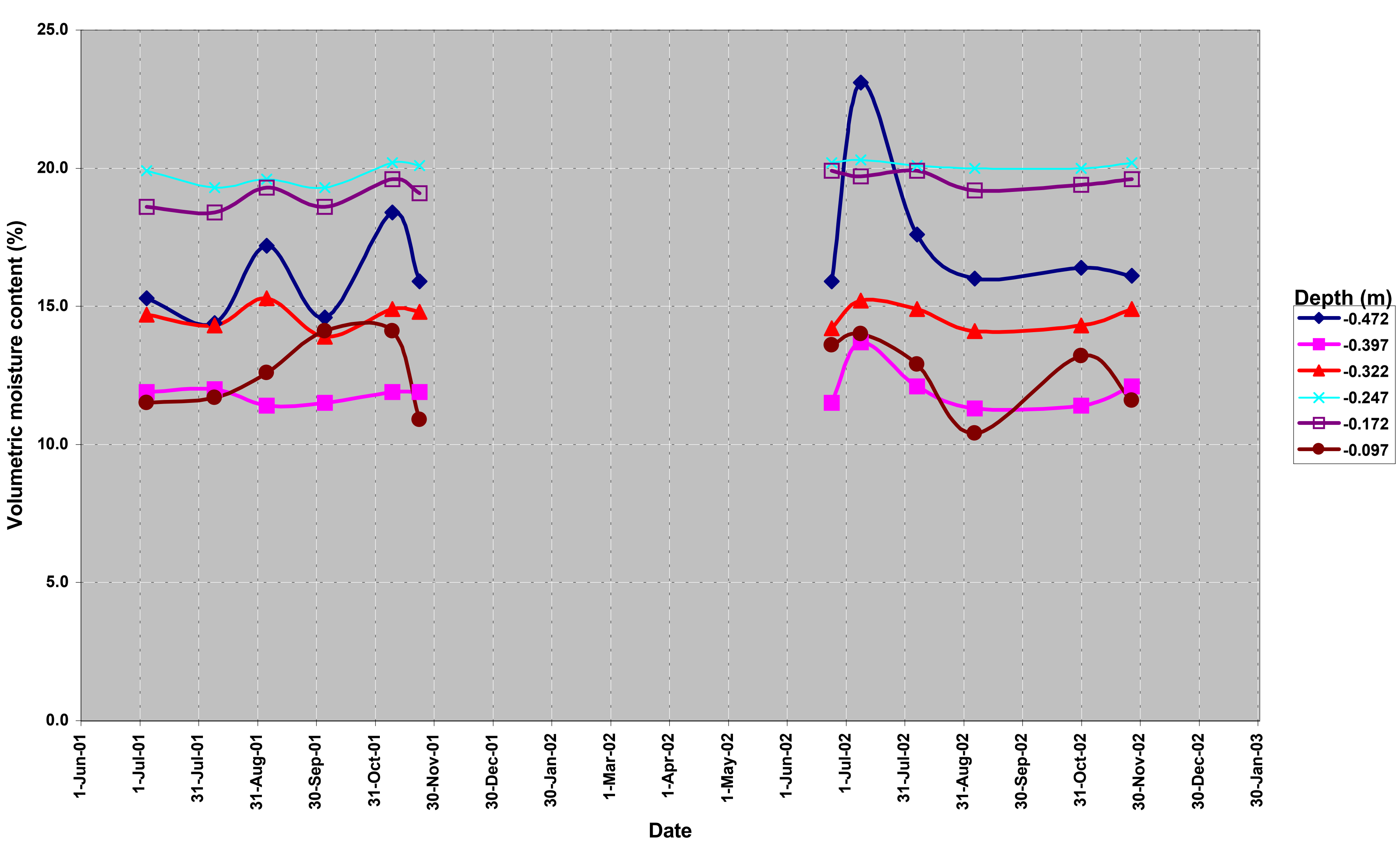

Figure A6 Moisture content readings in Hole 4

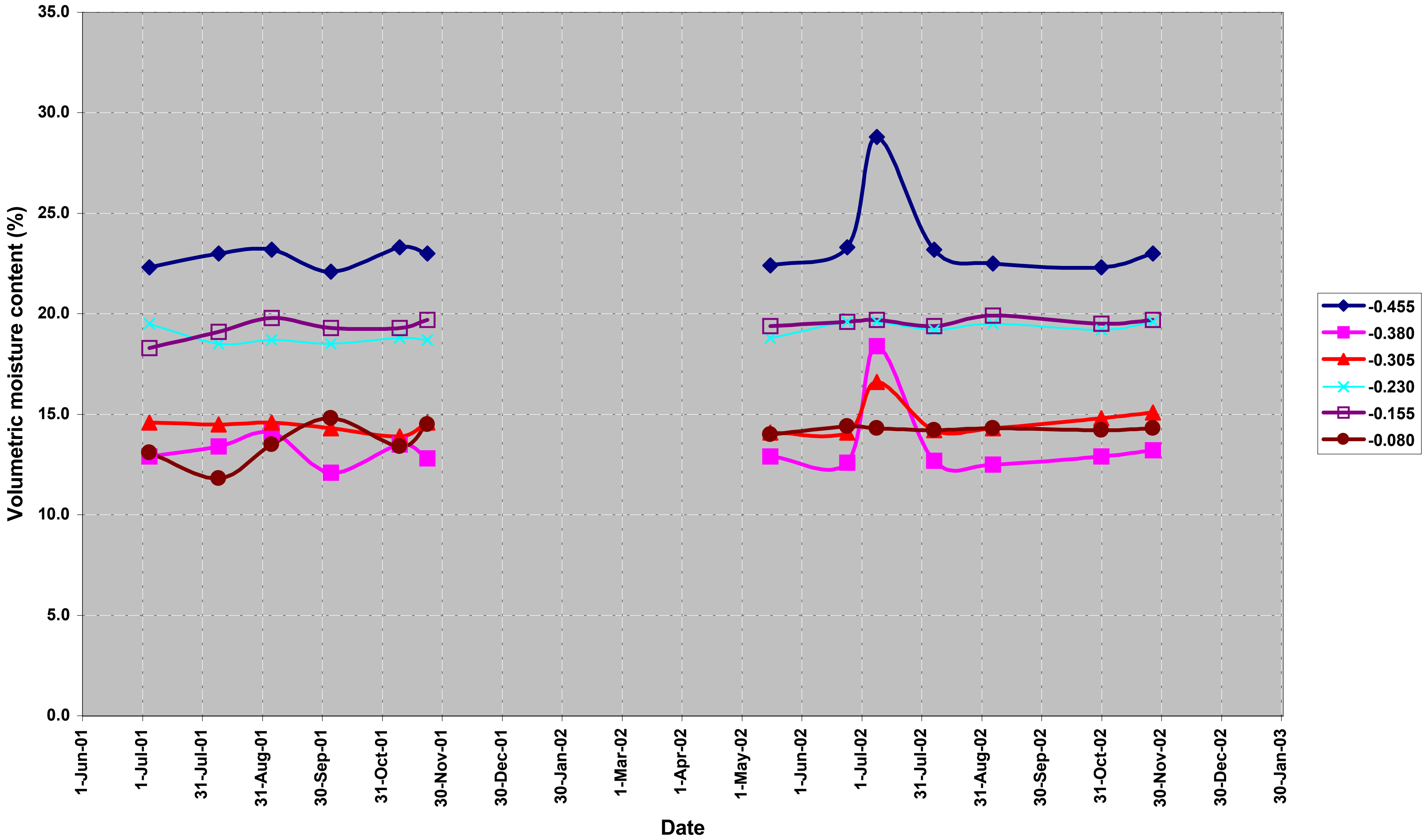

Figure A7 Moisture content readings in Hole 5

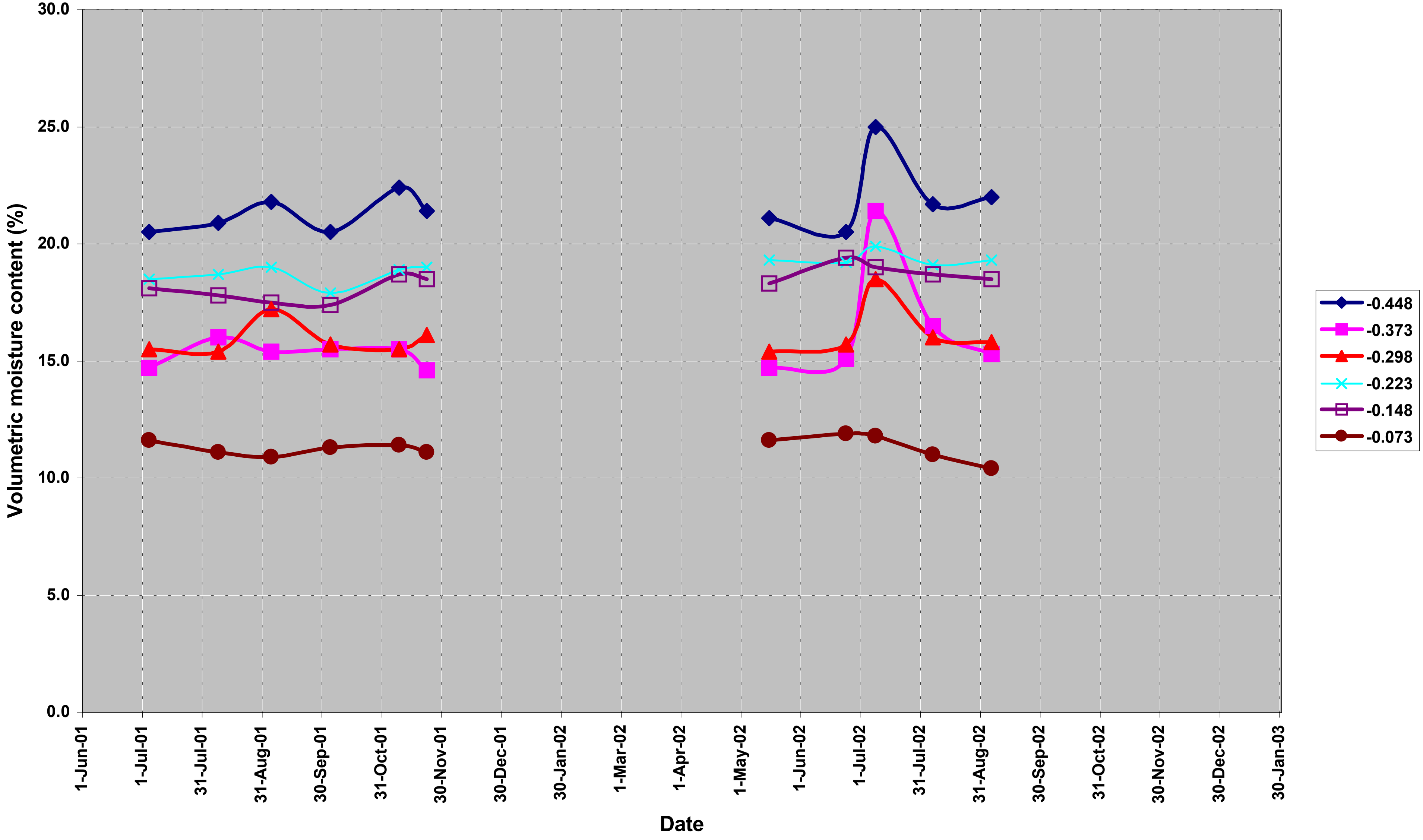

Figure A8 Moisture content readings in Hole 6

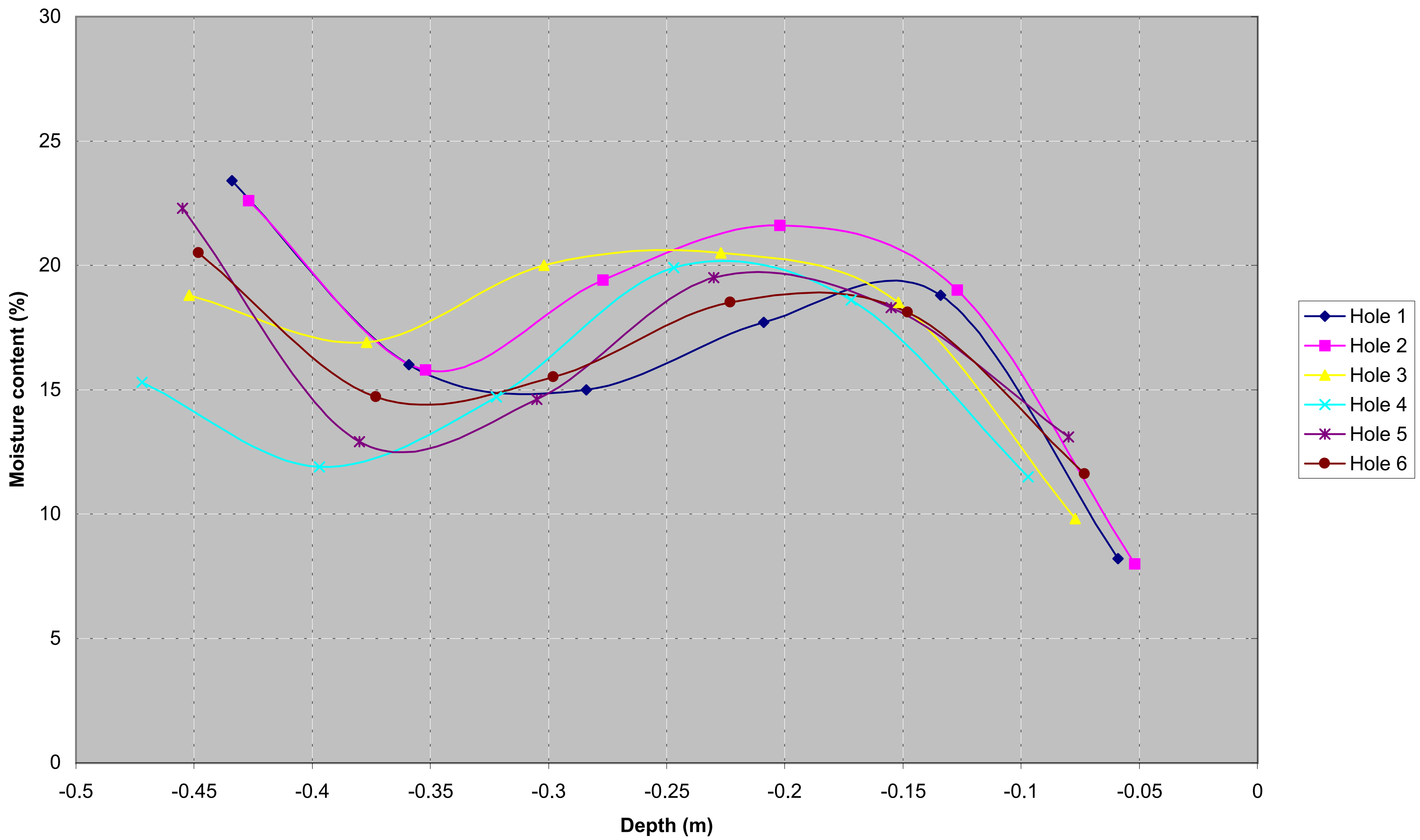

Figure A9 Vertical moisture distribution on 5 July 2001

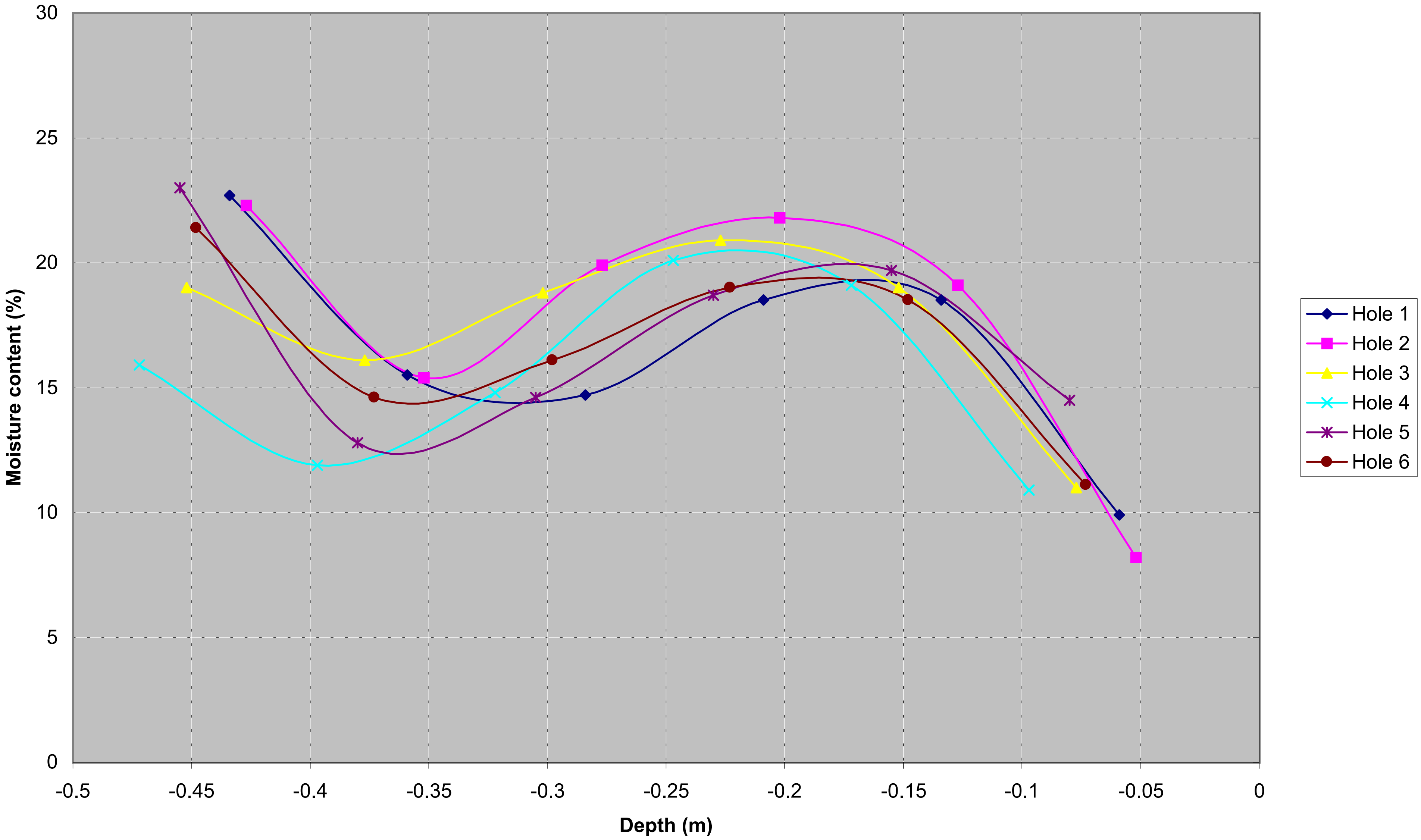

Figure A10 Vertical moisture distribution on 23 Nov 2001

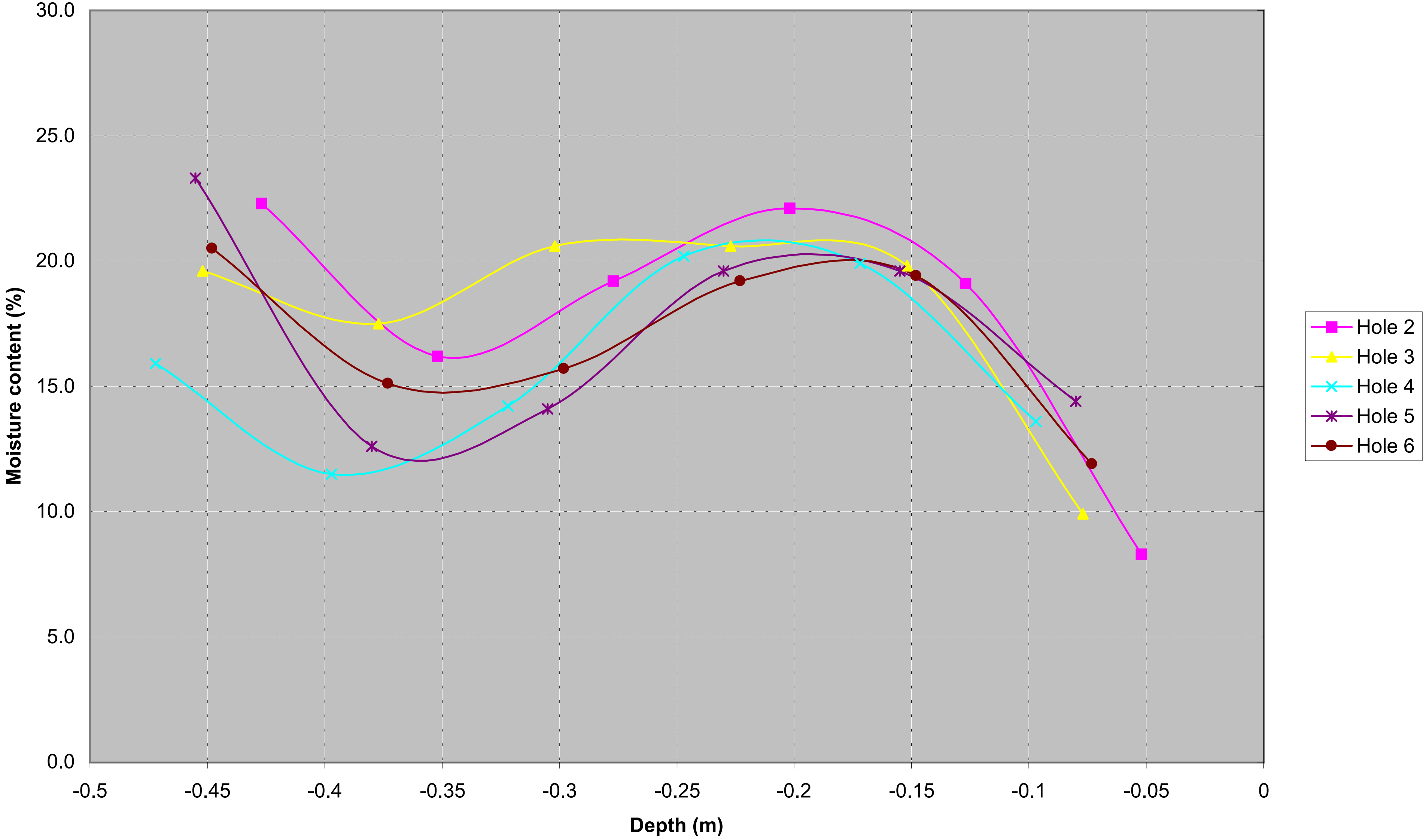

Figure A11 Vertical moisture distribution on 24 June 2002

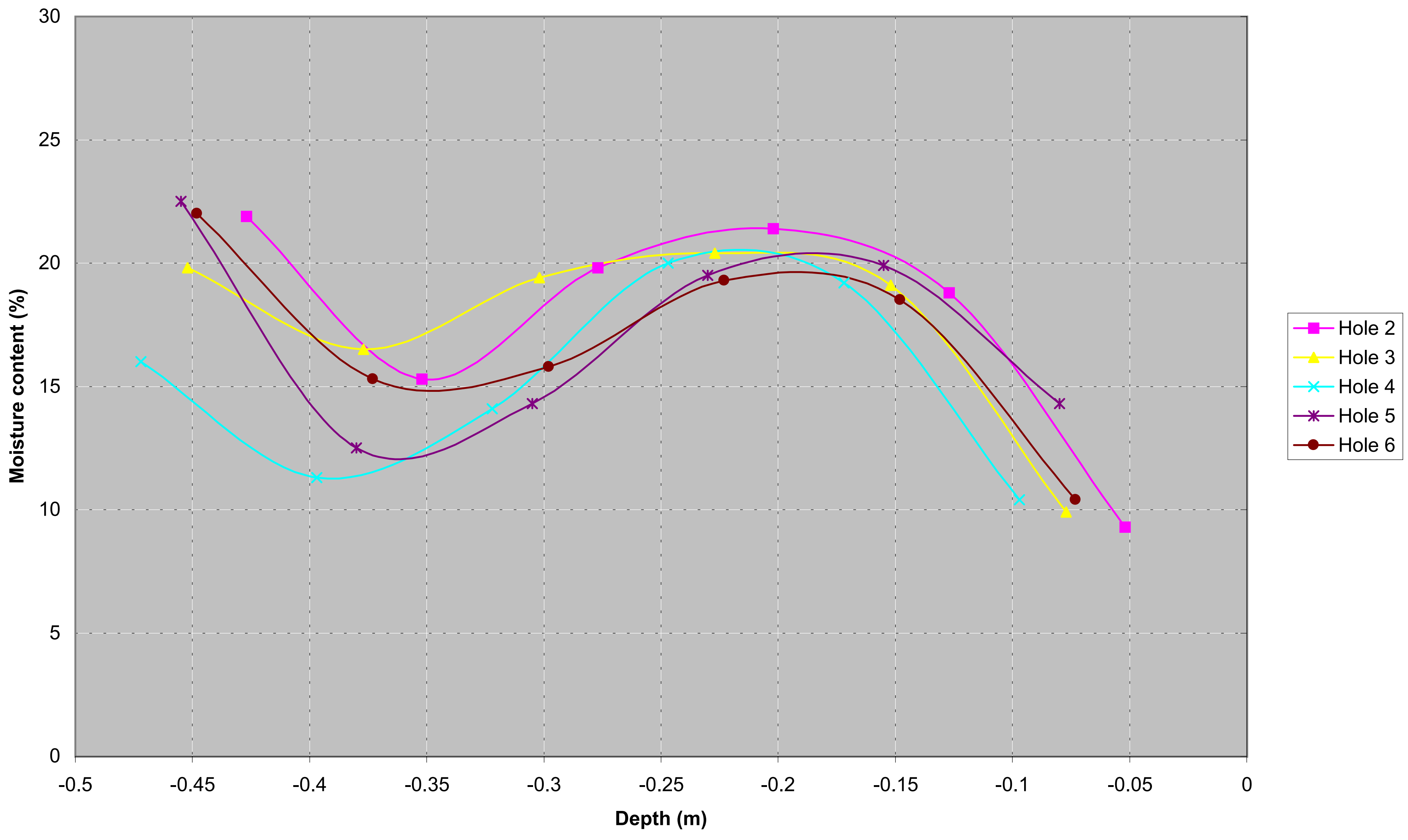

Figure A12 Vertical moisture distribution on 6 Sept 2002

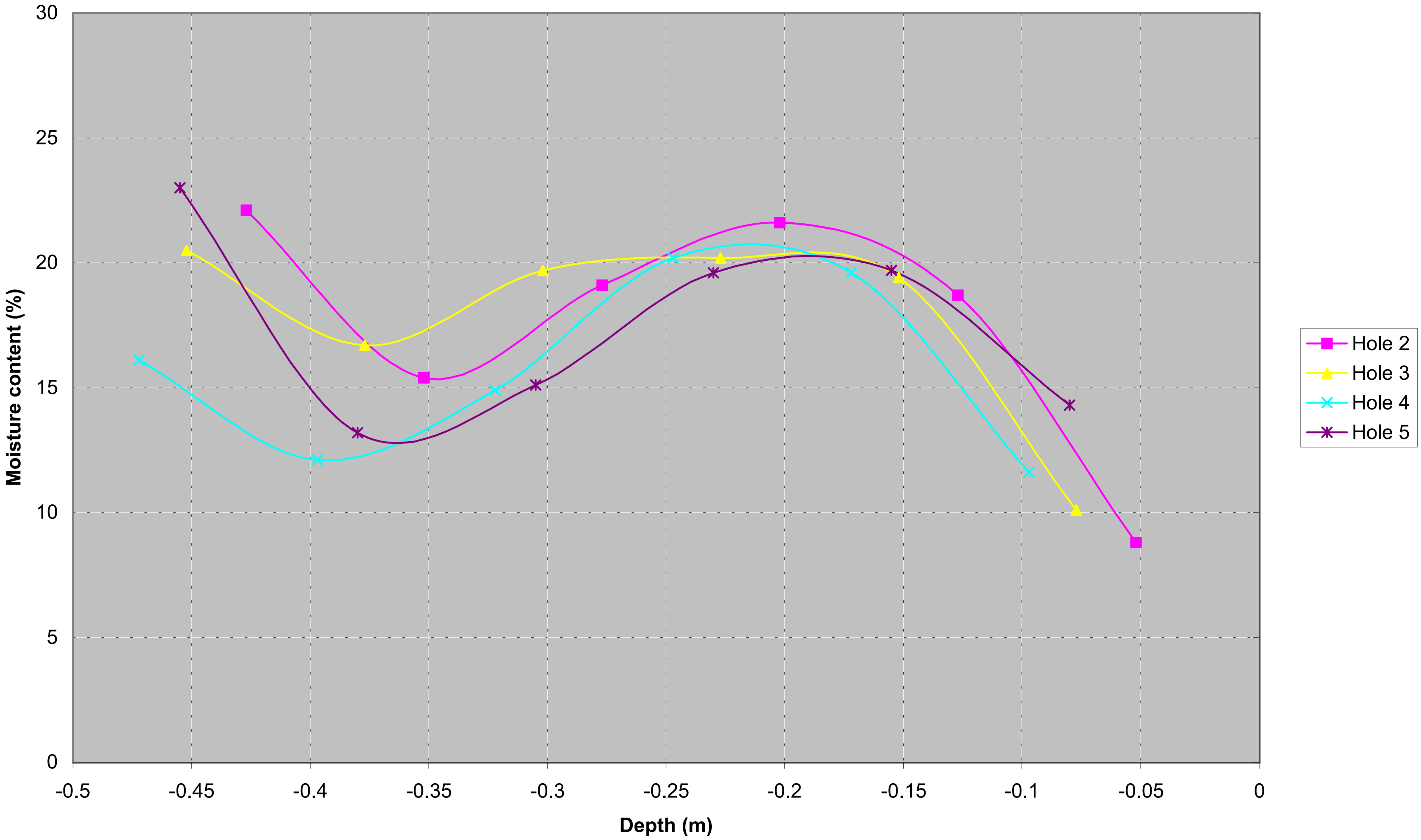

Figure A13 Vertical moisture distribution on 26 Nov 2002

Appendix B. Ground probing radar surveys

Water Movement- Radar Report

Ground penetrating radar (or GPR for short) is the general term applied to techniques that employ radio waves, typically in the 10 to 2000 MHz frequency range. Historically radar was primarily focussed on mapping structures in the ground; more recently GPR has been used in non-destructive testing of non-metallic structures.

Early work focussed on permafrost soil applications (Annan and Davis, 1976). The application areas broadened, to include mapping of soil and rock stratigraphy, (Davis and Annan,1989) which can be useful in mining applications (Scaife and Annan, 1991). Other early applications have included the profiling of contaminated or waste water (Bensoon *et al*, 1984 and Ulriksen, 1982). More recently, GPR has been shown to be an extremely versatile technique (Hobbs *et al*, 1993) and in some applications is the only technology, which can be used successfully for the location of hidden features. For instance GPR has been used to investigate roads; (Gordon *et al*, 1998 and Hugneschmidt *et al*, 1998) and to find archaeological remains (Goodman and Nishimura, 1992). GPR has also been used in other techniques as shown in Binda *et al* (2000) where radar was used to assess earthquake damaged towers in Italy. Flint *et al* (1999) and Colla *et al* (1997) have shown that GPR can be used on masonry arch bridges, to determine the internal state. Forde *et al* (1999) have shown that GPR can not only be used to assess geological condition or assessing structures but also through water to investigate potential scour holes in a non saline river bed. The most recent application of GPR has been to determine the in-situ condition of railway ballast (Clark *et al* 2001 and Clark, 2001).

Figure B1 shows the basic principles of radar. The fundamentals of GPR are described in Daniels (1989). GPR systems work by emitting short pulses of electromagnetic energy, typically of frequencies in the range of 1 to 2,000 MHz, into the transmission medium by means of an antenna. The pulse length can range from 1ns to 10ns depending on the type of material and the resolution required (see Table B2).

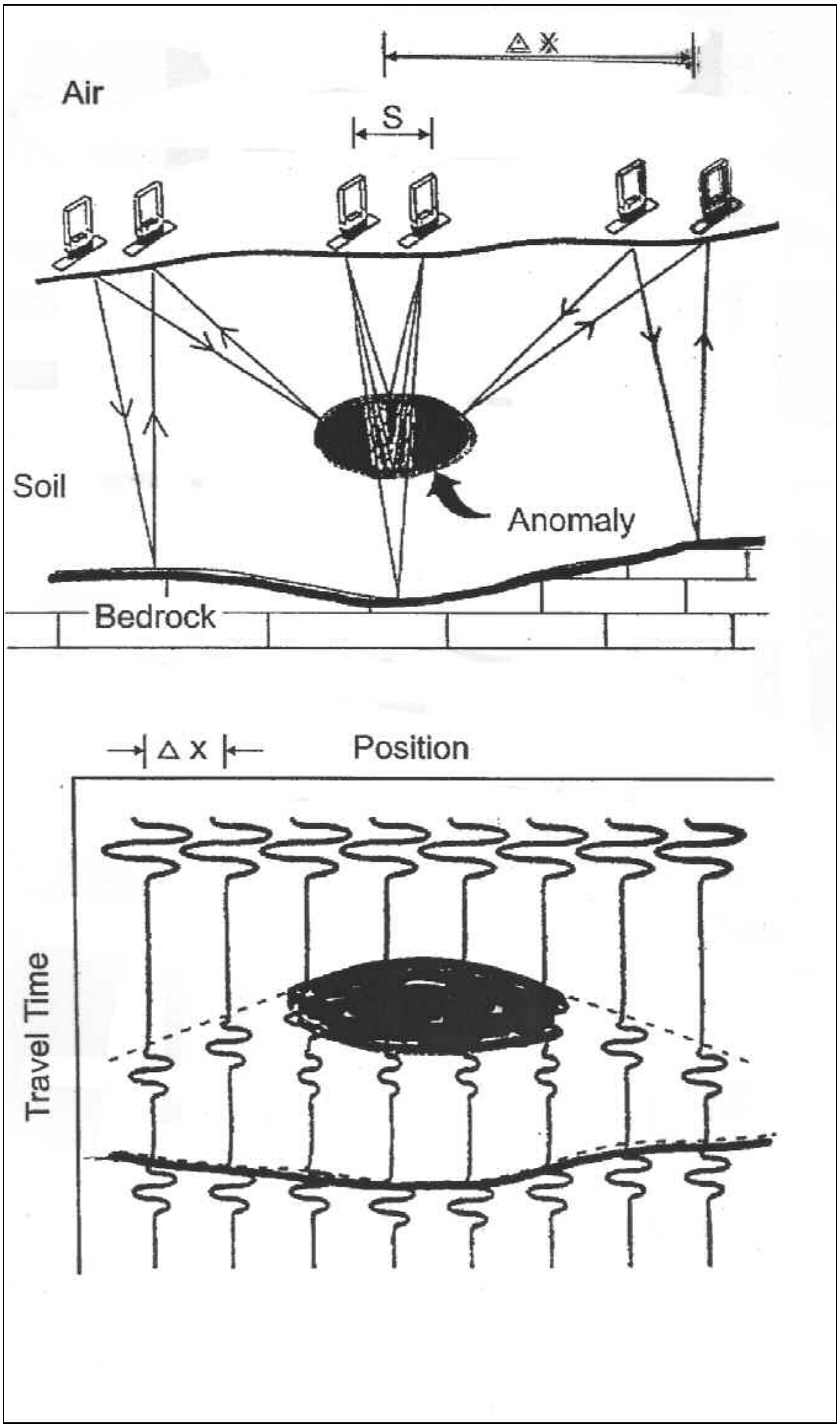

Figure B1 Basic principle of radar testing

The signals transmitted into the medium of interest are partially reflected on encountering a change in the electrical properties of that medium. The reflected signal is recorded at a receiver while the transmitted part continues through the new material. This process is repeated when further electrically different media are met by the transmitted signal. The series of reflections recorded at the receiver allow an image of the interior structure to be built up.

In most practical applications, geological and building materials are classified as low loss materials at radar frequencies and therefore the general equation for wave velocity is often simplified to:

$$v = \frac{c}{\sqrt{\varepsilon_{\mathrm{r}}}} \tag{1}$$

c = Speed of light (m/s)

v = Velocity of the electromagnetic wave (m/s)

ε_r = Dielectric constant which is independent of frequency and conductivity.

The depth is determined from the time it takes the reflected wave to be detected at the receiver. Knowing the velocity of the wave through the relevant media the depth is calculated using the equation 2 below:

$$d = v\left(\frac{t}{2}\right) \tag{2}$$

where: $d =$ Depth of reflector (m)

$v =$ Velocity of electromagnetic wave (m/s)

$t =$ Two-way travel time (s)

The two-way travel time is defined as the time it takes for the signal to travel from the transmitter to the reflector and back to the receiver, covering twice the distance (d) under investigation

The dielectric constant is a dimensionless measure of the capacity of a material to store a charge when an electric field is passed through it. In partially saturated or fully saturated materials, the dielectric constant is primarily determined by the water content, salinity and porosity. This effect of water on the dielectric constant is described in Wensink (1993). Table B1 shows typical values for some common materials.

Table B1 Typical values of dielectric constant at 100MHz, and conductivity of common materials (Colla, 1997; Davis and Annan, 1989; Sharma, 1997 & Daniels, 1996)

Material	Dielectric constant (ε_r)
Air	1
Fresh Water	81
Ice (Fresh Water)	4
Seawater	81
Sand: dry	2 – 6
Sand: wet*	20 – 30
Silt: dry	2 – 6
Silt: wet*	10 – 30
Clay: dry	4 – 6
Clay: wet*	15 – 40
Limestone: dry	4
Limestone: wet	8
Sandstone: dry	2 – 3
Sandstone: wet	6
Concrete: dry	6
Concrete: wet*	20
Brick	4

* Saturated with fresh water

Surface penetrating radar systems can use a variety of antennae tuned to various frequencies suited to different applications. Propagation losses, antenna size, material type and the size of the object to be detected dictate the choice of frequency of operation and therefore the resulting penetration and resolution of the measurements. For example, an antenna with a centre frequency of 500 MHz can typically penetrate to depths of 2 metres in clays with a corresponding resolution of 5 cm (about a half of a wavelength [$\lambda/2$]). Table B2 shows the propagation and resolution through concrete of different dielectric constants. The table shows how the velocity, wavelength, resolution (minimum distance between two interfaces to be identified) and Z_{min} (the distance to the first possible identifiable interface) will vary with dielectric constant.

Table B2 GPR propagation and penetration through materials with different dielectric properties

	Frequency (MHz)		Frequency (MHz)		Frequency (MHz)		Frequency (MHz)	
	900	450	900	450	900	450	900	450
Dielectric Constant	Velocity (cm/ns)		Wavelength (cm)		Resolution (λ /2) (cm)		Z_{min} (λ /3) (cm)	
4	15.0	15.0	16.7	33.3	8.3	16.7	5.6	11.1
5	13.4	13.4	14.9	29.8	7.5	14.9	5.0	9.9
6	12.2	12.2	13.6	27.2	6.8	13.6	4.5	9.1
7	11.3	11.3	12.6	25.2	6.3	12.6	4.2	8.4
8	10.6	10.6	11.8	23.6	5.9	11.8	3.9	7.9
9	10.0	10.0	11.1	22.2	5.6	11.1	3.7	7.4
10	9.5	9.5	10.5	21.1	5.3	10.5	3.5	7.0
15	7.7	7.7	8.6	17.2	4.3	8.6	2.9	5.7
20	6.7	6.7	7.5	14.9	3.7	7.5	2.5	5.0

An antenna of higher frequency will operate with a higher resolution providing increased clarity, but the depth of penetration is proportionately reduced. Conversely, a lower frequency antenna will provide greater penetration but less clarity. Materials with a high conductivity such as wet clays and soils containing a large amount of dissolved salts are the most difficult to penetrate, whereas materials such as granite and sandstone are relatively easy to penetrate. In regions where soils of high conductivity such as clay exist, penetration may be reduced to the point where radar may no longer be the preferred method of testing. An important consideration when choosing equipment for an application is to determine the exact trade off between resolution, antenna size and the penetration required. As a general guide, it is better to trade resolution for penetration. Good resolution is not useful if the target cannot be detected.

Targets which possess a dielectric constant similar to that of their surroundings may be difficult, perhaps impossible, to identify using radar. Conversely, because electromagnetic waves are unable to penetrate metallic objects, large reflections can usually be detected.

Experiments:

Two radar surveys were conducted over the trial concrete pavement, one on 6/8/02 during a relatively dry period and one after a wet period on 25/11/02. For each survey two parallel scans were recorded from the top to the bottom of the slope. One was near the edge of the pavement and the other close to the centreline. Two different radar frequencies were used, to see if it was easier to identify the layer changes with a lower frequency or a higher frequency. The radar frequencies used were 900 MHz and 450 MHz. The 900 MHz would give better definition of defects, however the 450 MHz would penetrate further.

A plan view of the survey site and the location of the survey lines are shown in Figure B2. Figure B3 shows a radar plot of the concrete pavement running from the top of the slope to the bottom in a straight line. It shows a cross-section in the longitudinal direction of the test area. The radar plot was taken with a 900 MHz antenna at the beginning of the test period. A number of features can be seen in the image, the reinforcement [A],

the interface between the air and the concrete (the surface reflection) [B],

the interface between the concrete and the sub-base [C],

the interface between the sub-base and the clay layer below [D]

The horizontal scale represents the 48 equally-spaced survey points along the length of the pavement. The vertical scale represents the two-way time, in nanoseconds, taken by the radar signal to travel from the antenna to the feature and back to the antenna. The colour scale represents the magnitude and polarity of the reflected signal, i.e. a green colour shows a small magnitude while a large magnitude reflected signal is either blue or red depending on the associated polarity. With the 900 MHz antenna the three interfaces in Figure B3 i.e. air/concrete, concrete/sub-base and sub-base/clay, can clearly be seen.

Figure B4 shows a radar plot of the same line as in Figure B3, but scanned with a 450MHz antenna. This antenna could penetrate deeper but gave less resolution. The individual bars within the reinforcement [A] in Figure B3 can be clearly seen but in Figure B4 it is harder to distinguish between the individual reinforcing bars. The 450 MHz antenna has greater penetration making it easier to identify the location of the sub-base/clay interface [D]. It is possible to identify this layer below the reinforcement due to the spacing of the reinforcement (100 mm) in the longitudinal direction. The reinforcement is spaced more closely in the transverse direction. In a transverse scan across the concrete slab with the 900 MHz antenna, it is not possible to identify any features below the reinforcement (Figure B5). This confirms the theoretical data shown in Table B2.

Figure B3 and Figure B4 show that both the 900 MHz and 450 MHz antennae can be used to identify the layers (concrete and sub-base). A number of radar scans were carried out over the same points during the survey. This showed that the results shown in **Figure B3** and Figure B4 were repeatable within 10% of the average dielectric constant over 4 radar surveys. The scans were taken at different times of the day with different gain and software settings as well as different time windows.

The radar survey was then repeated on the 25/11/02, after a period of relatively high rainfall. This was also after the concrete slab had been cracked. The 900 MHz and 450 MHz scans are shown in Figure B6 and Figure B7. In these figures the three interfaces identified in the first survey can be identified. The amplitude (size and intensity of the colour) has marginally increased.

The radar data were analysed by knowing the specific locations of a number of radar scans along the survey line. The radar travel time difference between the first reflection (surface reflection) and the reflection from the concrete/sub-base interface was measured, along with the time difference between the reflections from the concrete/sub-base interface and the sub-base/clay interface. The depths of the sub-base and concrete layers were known as they were measured when the neutron probe holes were drilled. Using this information the dielectric constants of the concrete and sub-base were calculated from equations 1 and 2. From Figure B3 and Figure B4 the average dielectric constant, along the length of the survey lines, of the concrete was found to be 8 and that of the sub-base to be 12.

For the second survey, the average dielectric constants for the two layers were calculated from Figure B6 and Figure B7 and found to be 12 and 20. The dielectric constant had therefore increased for both the concrete layer and the sub-base layer. Cross-sections of the concrete slab, the actual depth of the concrete slab and the sub-base are plotted in Figure B8 and Figure B9 . The depth of the concrete and sub-base from the radar surveys is also plotted assuming the dielectric of 8 for the concrete and 12 for the sub-base found in the first survey. It can be seen from the second survey that the apparent depth has increased. This cannot be true, so must be interpreted as an increase in the dielectric constant. The data show that:

The dielectric constant increased more in the sub-base than in the concrete.

The dielectric constant along the survey line on the west (lower) side of the concrete slab increased more than that along the central survey line.

There is a marked increase in the dielectric constant towards the north end of the survey line (bottom of the slope).

Along the side of the concrete slab the new dielectric constants are 14 (concrete) and 24 (sub-base) while the along the middle of the concrete slab the dielectric constants are 10 (concrete) and 17 (sub-base) which represent a marked increase.

In this case, where the physical properties of the sub-base and concrete cannot change over time, the change in the dielectric constant must relate to an increase in moisture content.

The results were interpreted by reference to those of Pynn and Todd (1997), who carried out some radar experiments on granular sub-base overlaid with concrete, to explore the variation of radar velocity with moisture content. Over a limited moisture content range their results showed an approximately linear relationship between radar velocity and moisture content.

However, the prediction of the moisture content from the dielectric constant is very difficult. The variation of dielectric constant with moisture content differs from material to material (Ansoult *et al* 1985, Davis *et al* 1966, de Loor 1983, Tsui *et al* 1997, Bungey *et al* 2002, Olhoeft 2003, Reppert *et al* 2000, Robert 1998, Maser 1994 and Ulaby 1974). A large amount of research has been conducted to study the effect moisture content has on the dielectric properties of concrete and pavements (Bungey *et al* 2002, Tsui *et al* 1997, Olhoeft 2003 and Maser 1994). This literature suggests that the relationship between dielectric constant and the moisture content is not linear. The majority of authors state that the dielectric constant increases exponentially with an increase of moisture (apart from uncontaminated sand and distilled water, in which Davis *et al* (1966) found a linear relationship). The increase in moisture content will affect the dielectric constants of materials in different ways depending on the void ratio, density, absorption of water ions, the chemical contents of the material and the water and the ratio of absorbed water particles to those water particles held in the air voids.

Pynn and Todd (1997) showed that a 10% increase in the volumetric moisture content, in a similar limestone aggregate sub-base, resulted in a 43% increase in dielectric constant, (from a dielectric constant of 7 to one of 10). Assuming Pynn and Todd’s result to be applicable to the current experiment, Table B3 shows the increase in moisture constant as the average dielectric content increases over the different survey lines. The accuracy of this result is expected to be poor due to the assumptions made by Pynn and Todd (1997) and the application of their results to the current experiment.

Table B3 The increase of moisture content for the sub-base

Material	Location	Initial dielectric constant	Final dielectric constant	Increase in dielectric constant (%)	Increase in moisture content (%)
Sub-base	Side	12	24	100	23
	Middle	12	17	41	10

The data were also used to calculate the change in moisture content at half-metre intervals along the survey lines and this is plotted at Figure B10 . This shows that at the south end of the survey line there was a decrease in the moisture content of the sub-base. The moisture content change increased along the length of the survey line. There is also a similarity in shape between the moisture content change profiles along the side of the slab and at the middle.

Conclusions

Ground probing radar is a suitable technique for determining the location of layers below a concrete road surface.

900 MHz and 450 MHz antennae are both capable of identifying the layers in a concrete pavement..

Ground probing radar is capable of detecting a change in the moisture condition of sub-base below a concrete surface.

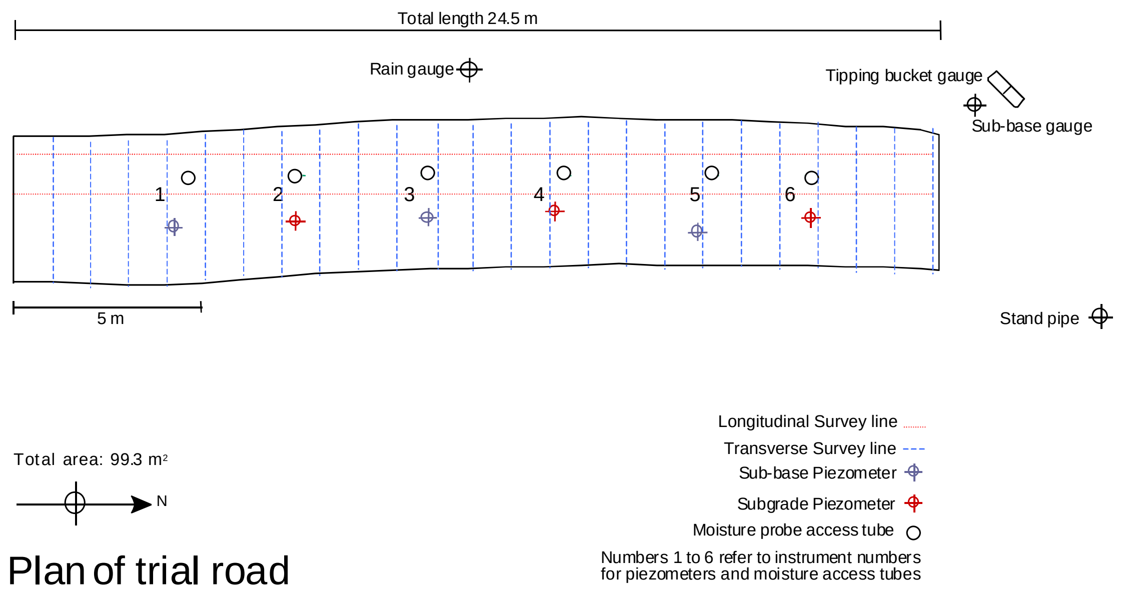

Figure B2 Plan view of the survey area including the survey lines

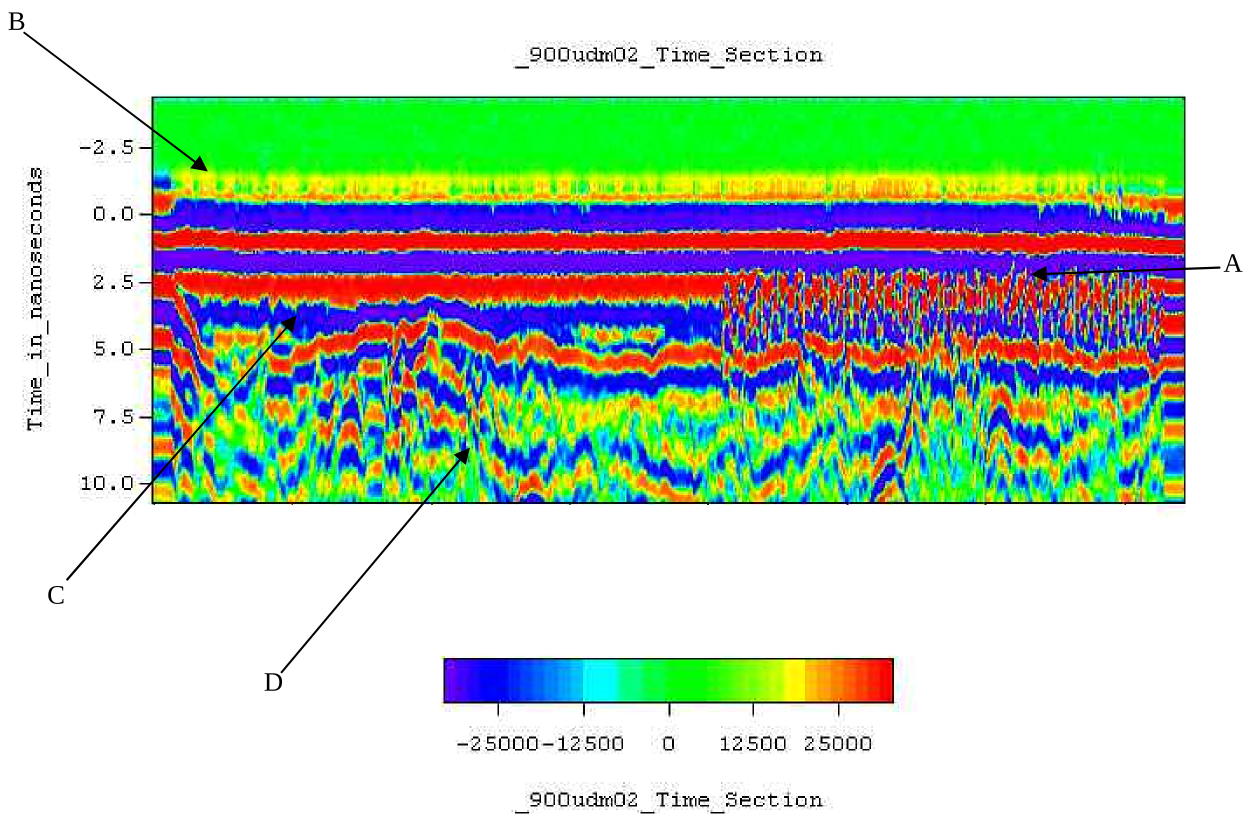

Figure B3 Radar scan of the length of the concrete section using a 900 MHz antenna.

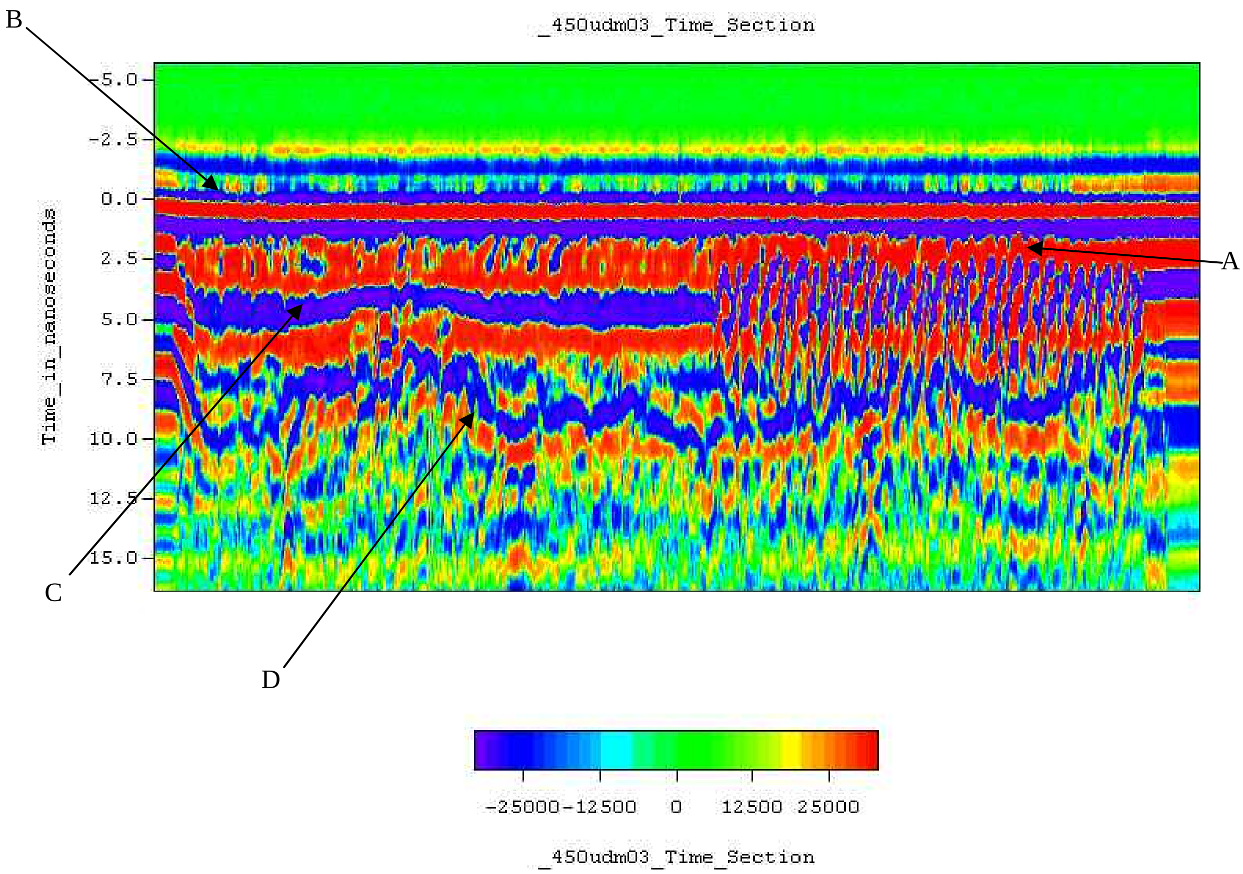

Figure B4 Radar scan of the length of the concrete section using a 450 MHz antenna.

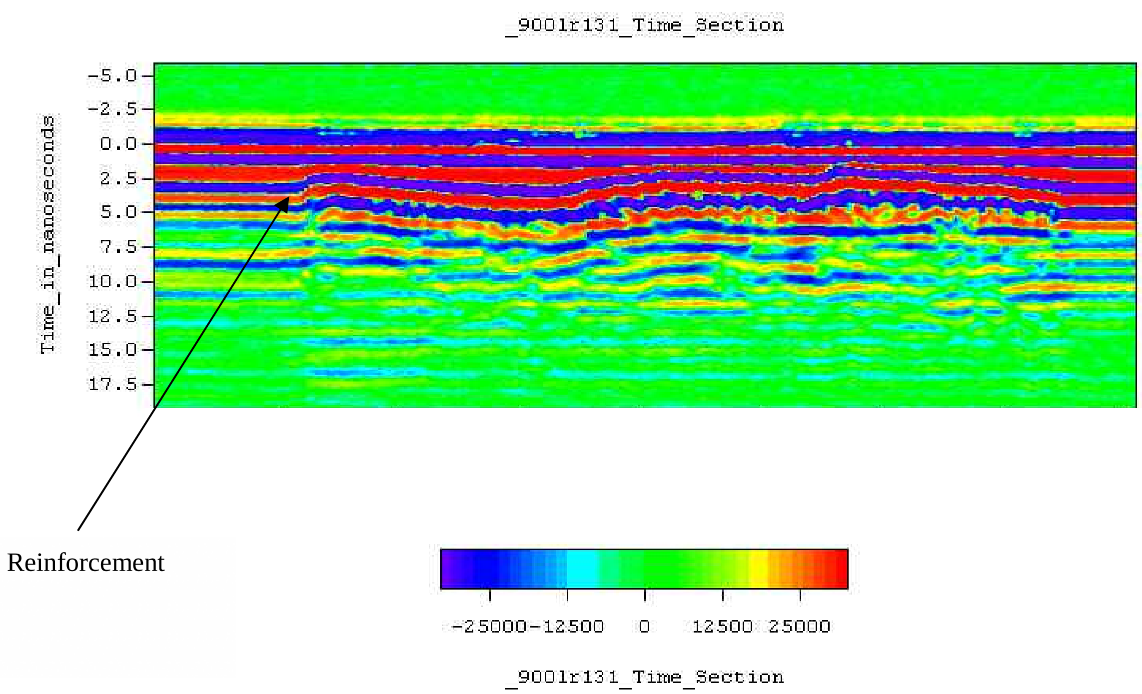

Figure B5 Transverse radar scan using the 900 MHz antenna over the reinforcement area.

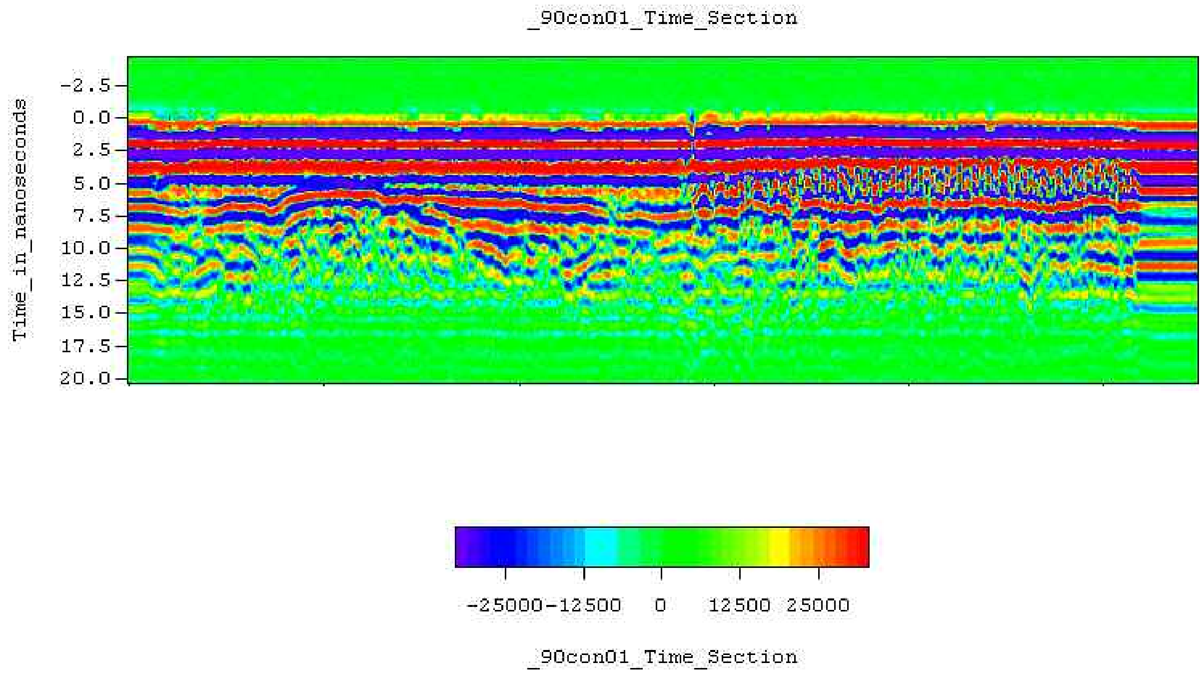

Figure B6 A 900MHz radar scan after water ingress

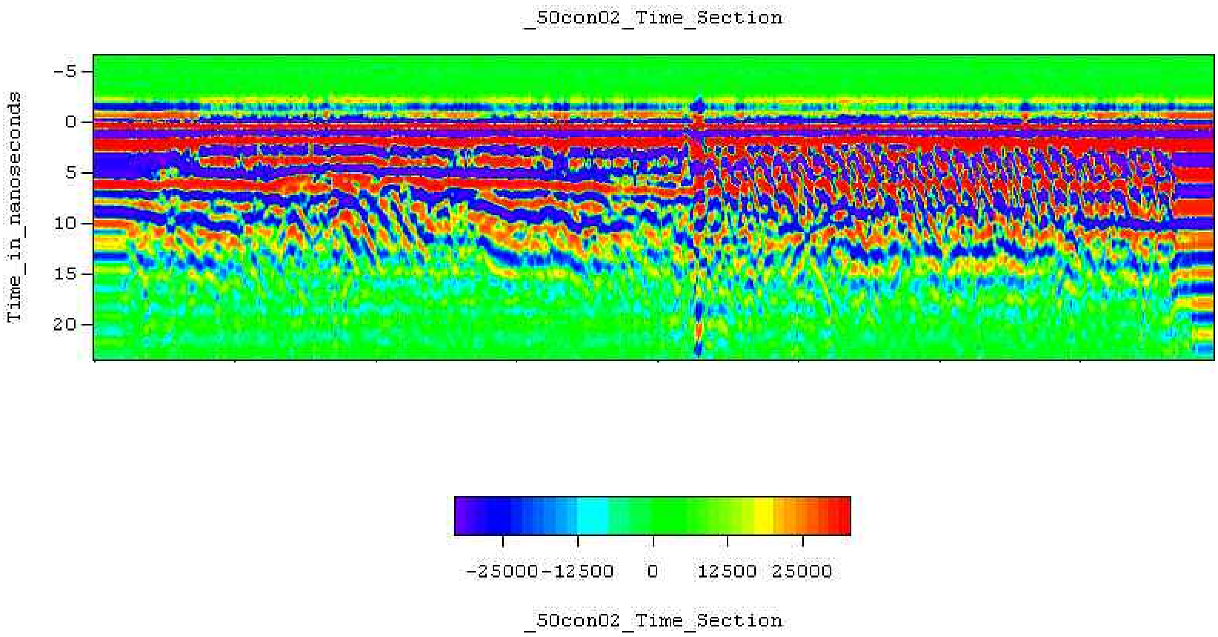

Figure B7 A 450MHz radar scan after water ingress

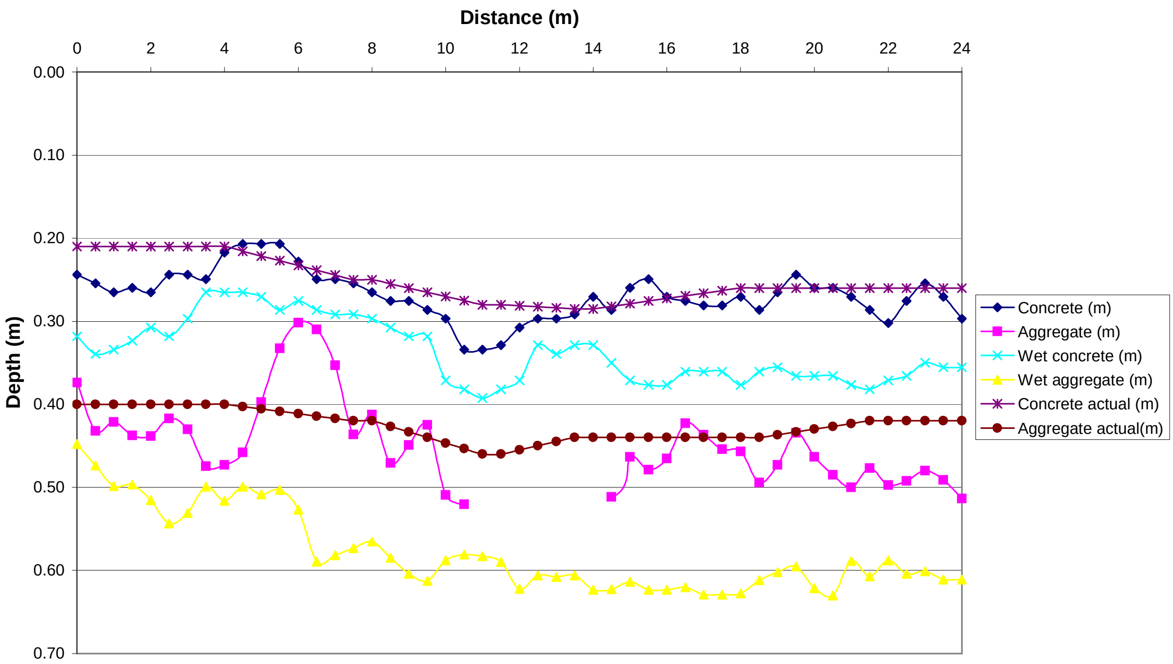

Figure B8 The longitudinal cross-section of the side of the concrete slab wet and dry.

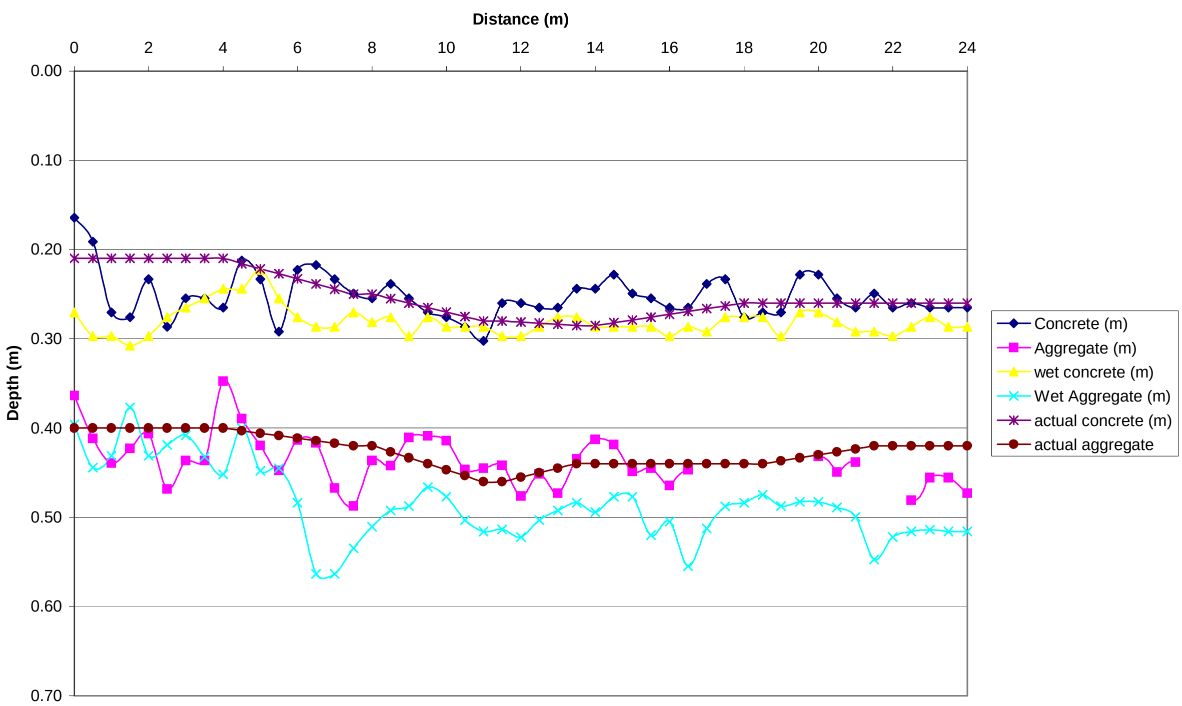

Figure B9 The longitudinal cross-section of the centre of the concrete slab wet and dry.

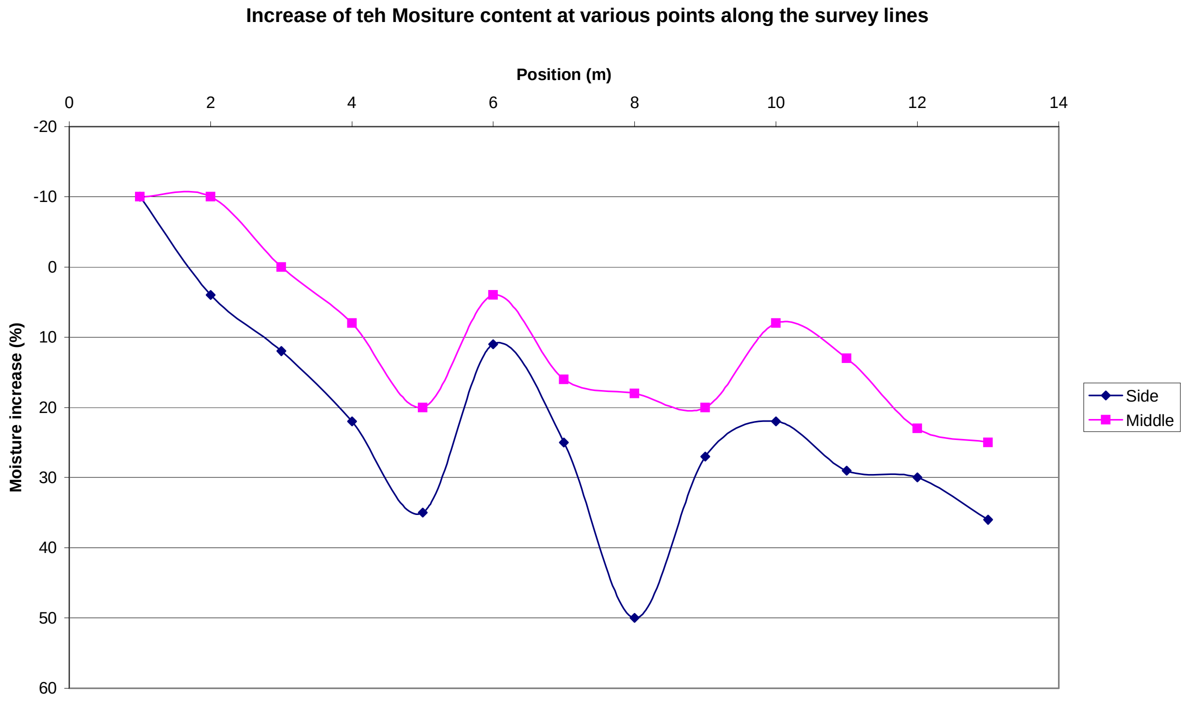

Figure B10 Indicated percentage increase in moisture content along the length of the survey lines.

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