The Hydraulic Design and Performance of Soakaways

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HR Wallingford

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Abstract

Soakaways are infiltration drainage devices which dispose of urban stormwater by recharge into the ground. Soakaways store water during a storm event and then allow it to infiltrate into the soil over a period of time. For a soakaway to be effective, it must be of sufficient size to both store and allow infiltration of stormwater runoff. The size required depends on the hydraulic properties of the soil and the chosen design rainfall events.

A soakaway designer needs to be able to assess the ability of the soil to infiltrate and disperse stormwater. The designer then needs to choose appropriate dimensions for the soakaway to be effective and to predict the likely return period of any overflows. The study addresses hydraulic problems associated with the application of field measurements to determine the size and corresponding performance of soakaways responding to design rainstorm events.

Existing guidelines on soakaway design, field testing and site conditions were examined and the hydraulic analyses used are reviewed and evaluated.

The study includes a review of the hydraulic principles governing the groundwater infiltration process which controls the hydraulic behaviour of soakaways.

A numerical model of coupled saturated and unsaturated groundwater flow was constructed to simulate soakaway tests and soakaways performing under working conditions. The results of the numerical simulations were compared with simplified analytical models of soakaway hydraulics. An analytical model based on two infiltration coefficients was found to provide quite an accurate description of the numerical results.

The project provides an increased understanding of soakaway hydraulics and the analytical model developed can be used to evaluate field tests and soakaways under realistic working conditions.

The research is a prerequisite to the formulation of authoritative guidelines on the hydraulic design and performance of soakaways.

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1. Introduction

1.1 General introduction

Rainfall runoff from impervious areas can be disposed of by use of infiltration drainage systems such as soakaways which recharge the water into the sub-soil. The main advantage of such a policy is to reduce the burden on the sewerage system. This minimises the quantity of water requiring piped transport and decreases the risks of overflows in stormwater sewers and combined sewer systems. By controlling stormwater close to source, infiltration drainage attenuates flooding in open channel systems and reduces the hydrological impact of urbanization. Soakaways may be used where existing piped sewers are fully laden or connections are impractical. Enhanced recharge of groundwater is another reason why infiltration drainage may be considered desirable in some circumstances.

Many thousands of soakaways are constructed each year in England and Wales. Applications include the drainage of small patios and single roofs, car parks and pavements, roads and motorways, large factory roofs and paved urban areas. The use of soakaways is likely to increase in future due to their potential for reducing the burden on piped sewerage systems and attenuating stormwater discharges in the urban environment.

A recent survey, conducted on behalf of the Deparment of the Environment and the Building Research Establishment, (Ove Arup, 1989) revealed a need among users for improved guidelines relating to site testing and choice of soakaway size. In order to provide a better understanding of this subject, a research project on the hydraulic design and performance of soakaways was undertaken.

The research was funded by a consortium comprising the Department of the Environment, six of the major Water Companies and three of the regional National Rivers Authorities.

1.2 Soakaways

Where an infiltration drainage system serves a number of properties it is termed as centralised and decentralised where each property is drained individually. Soakaways in the United Kingdom tend to be based on the decentralised approach and are therefore relatively small.

Overseas, the centralised system appears to have received much more attention. Overseas practice has therefore concentrated on open systems such as grass-lined ditches and ponds which overflow into infiltration trenches or percolation basins. In contrast, soakaways in the United Kingdom tend to be closed and buried systems.

Infiltration drainage systems which are not considered to be soakaways for the purposes of this study include open percolation basins, plane infiltration systems such as permeable pavements, long infiltration trenches and deep recharge boreholes.

Soakaways may be shallow dry wells, rubble filled pits, dry-jointed brick-lined cavities or perforated concrete ring cylinders. A typical soakaway for a decentralised infiltration drainage system would probably be between one and three metres in diameter (or square) in plan and one to three metres in depth.

1.3 The soakaway problem

When a soakaway is to be constructed, it is necessary to choose appropriate dimensions for it to be effective. It must be able to cope with chosen design rainfall events without overflowing. It must provide sufficient storage capacity to accept the runoff from short intense rainfall events and to provide sufficient infiltration capability to disperse the water from long steady rainfall events. The performance of a soakaway will depend on the size and shape of the excavation and the hydraulic properties of the soil in which it is founded. These properties are generally site specific and cannot be determined by simple inspection of the soil so it is necessary to conduct a field test. Once the relevant soil properties are determined, an appropriate design can be selected which will provide the required performance of the soakaway.

A site test for soakaways is shown in Figure 1 where a relatively small excavation is made and the rate at which water disperses is measured, Figure 1(a). This is often termed a soakage test. The results of the soakage test are applied to a design rainstorm event, Figure 1(b), to predict the size of excavation required to ensure that the soakaway does not overflow, Figure 1(c).

The scaling factors involved in analyzing soakage tests are highly complex. Until now, little research has been applied to soakaway hydraulics and there is a requirement to provide guidance for those seeking advice on soakage tests, soakaway design size and the corresponding soakaway performance.

The soakaway designer needs to know how to conduct soakage tests and how to analyze the results in order to determine the size of soakaway required. He needs to estimate the likely maximum water level that will occur in the soakaway during different rainfall events and the frequency with which the soakaway could overflow.

1.4 Objectives of the study

The objectives of the study were to evaluate the hydraulics of soakage tests and to assess how such field measurements can be related to the required dimensions and the corresponding performance of soakaways.

A further objective was to provide a better understanding of soakage tests and soakaways and to elucidate the physical principles that govern their behaviour. This improved understanding will benefit the assembly of informed and authoritative guidelines on the design of soakaways.

The objectives were achieved through a mainly theoretical study in which the soakaway problem was modelled by numerically solving the equations that govern the flow processes. This was conducted for a large number of situations in order to provide an understanding of the relationships between soil properties, soakage tests, soakaway dimensions and the resulting soakaway performance.

1.5 Description of the study

Existing procedures for finding the design size of soakaways were reviewed. The hydraulic analyses used were examined in order to determine the technical basis and relative merits of the different methods. Brief descriptions of the available design procedures and the hydraulic analyses involved are provided in Section 2 of this report.

Relevant published research in the field of infiltration theory was reviewed. This has included work in the disciplines of agronomy, soil physics, soil mechanics, hydraulics and hydrology. The flow processes which determine the hydraulic behaviour of soakaways and the soil properties which influence these flow processes were identified. These processes are described in Section 3 of the report.

The soakaway problem was modelled by numerically solving the governing equations for the flow processes. Computational simulations of the infiltration process were conducted for constant and falling head soakage tests, for soakaways receiving stormwater in the form of hydrographs and for different types of soil. Section 4 describes the modelling approach and also presents the results of the computational simulations of soakage tests and soakaways.

A simplified analytical approach was developed for the analysis of soakage tests and the modelling of soakaways. Section 5 describes the development of this approach and presents comparisons with the numerical model simulations.

In order to obtain field data, for comparison with theoretical behaviour, a series of soakage tests were carried out. These tests and the analysis of the results are described in Section 6.

In Section 7, the existing procedures and analyses for choosing the dimensions of soakaways are reconsidered. An example problem was modelled and compared with the results obtained by using each of the different design procedures. The numerical and analytical mathematical models developed within the study were also applied to the example problem and thew results compared with the present guidelines.

The implications of the results of the study on future guidelines are discussed in Section 8. A possible methodology for field testing and the hydraulic design of soakaways is outlined.

The conclusions and recommendations of the study are presented in Section 8 of the report.

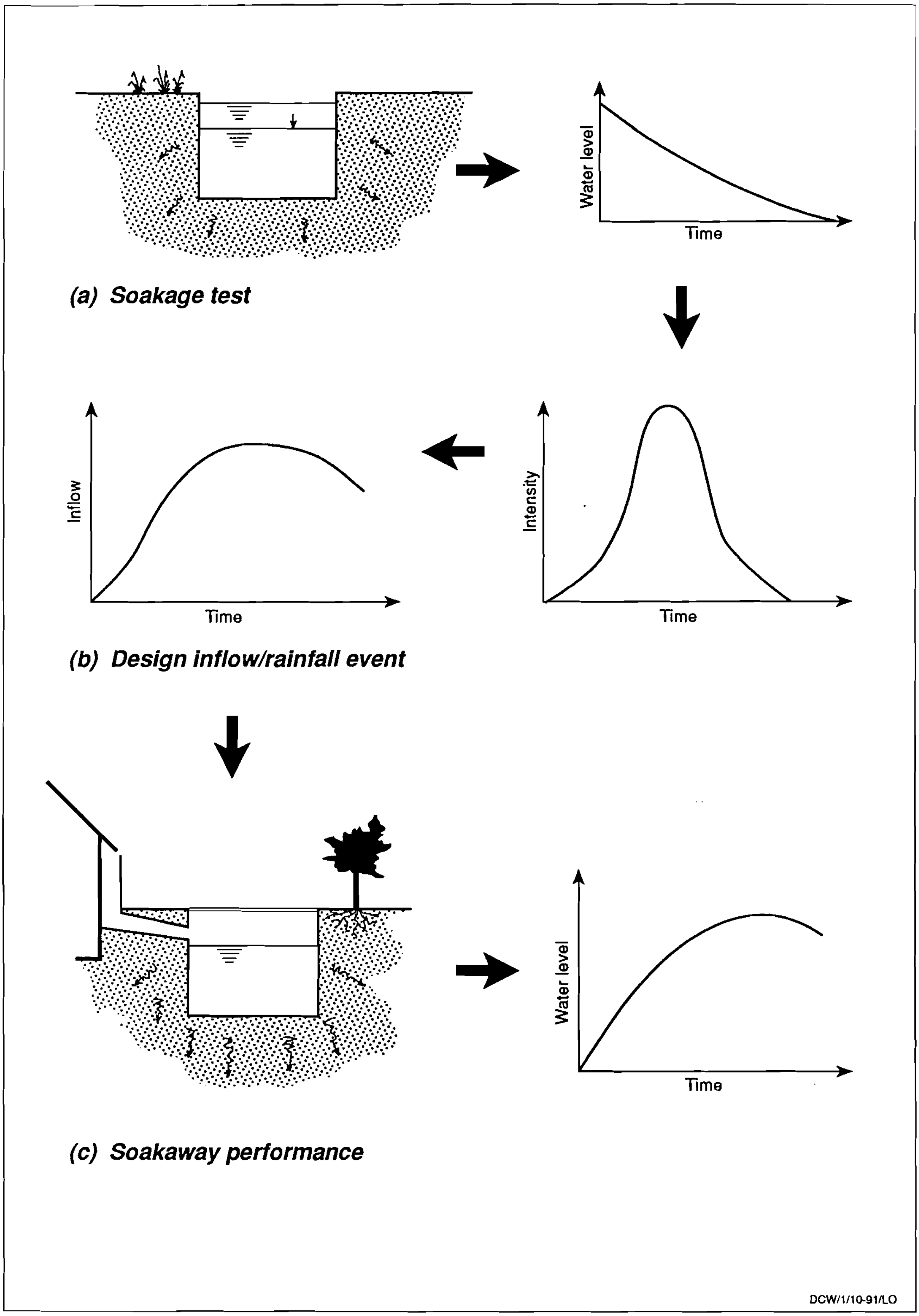


Fig 1 Diagrammatic representation of the soakaway problem

2. *Existing design procedures*

2.1 Introduction

At present there is no universally accepted procedure for designing soakaways. The most commonly quoted reference to soakage tests in the UK is BRE Digest 151 (see Section 2.2.3 below). Some Local Authorities apply guidelines of their own to choosing the design size of soakaways (Ove Arup, 1989) whilst many Authorities have no standard procedures.

There are essentially three types of approach to the hydraulic design of soakaways, taken in the guidelines available. In the first approach a standard size is used irrespective of the soil properties. The other approaches involve conducting a soakage test, measuring the infiltration rate and applying the result to prototype conditions.

Type I: The first approach is to simply specify the volume of soakaway required to store all the rainwater from a specific storm event. The porosity of any fill material in the soakaway must be taken into account. This type of method takes no account of the soil's ability for infiltration. A soakaway designed by this method may store the rainwater from one event but, if the water does not disperse by infiltration, it will be unable to cope with successive events.

Type II: The second approach is to conduct a soakage test and to analyze it by calculating the mean flow rate per unit wetted surface area achieved for the test. This infiltration rate is then assumed to be constant and to apply to the prototype soakaway. The required size of soakaway is then found by calculating the surface area required to infiltrate the runoff derived from a given rainfall event.

Type III: The third approach is to relate the soakage test to saturated groundwater flow theory. In hydraulic terms, the infiltration rate is related to the hydraulic conductivity of the soil, the hydraulic head acting on the soil surface and a known hydraulic gradient by Darcy's Law. In order to do this, assumptions have to be made about the extent and geometry of the saturated part of the soil system and the effective hydraulic gradient which drives the flow.

Procedures for designing soakaways described in some published guidelines are briefly outlined below.

2.2 Common UK design methods

2.2.1 Code of Practice for Building Drainage, BS 8301

The British Standard Code of Practice for Building Drainage (British Standards Institution, 1985) briefly mentions soakaways. The advice given is based on a Type I approach. It states that soakaway dimensions should be based on a storage capacity equal to 12 mm of rainfall over the impervious drained area.

If infiltration capacity is felt to be of concern, the reader is referred to the BRE Digest 151 to conduct a soakage test.

2.2.2 Property Services Agency Technical Instruction on soakaways, PSA CE 125

This document (Department of the Environment, 1977, 1984) contains two methods for conducting and analyzing soakage tests. For soakaways serving impervious areas of less than 400 m^2, the method given in BRE 151 is reproduced. For areas greater than 400 m^2, a Type II method is presented.

The latter procedure involves the excavation of a rectangular test pit. The pit is then filled with water and allowed to drain. Measurements of the distance that the water level falls are taken at regular intervals. The percolation rate is calculated as the volume outflow divided by the average wetted surface area. A version of the calculation is also given that assumes the base of the pit to be impermeable. It is then suggested that, to provide a factor of safety, a value of one third of the percolation rate obtained from the soakage test should be adopted for the design calculations.

The soakaway design procedure is based on a storm event of 15 mm/hour intensity and 2 hours duration. It is assumed that 11/12 of the total runoff (27.5 mm/unit area drained) needs to be accommodated by storage and 1/12 of the runoff (2.5 mm/unit area drained) is dispersed by infiltration during the storm event (1.25 mm/hour). The soakaway is then designed to have sufficient storage volume to satisfy the first criterion and also sufficient surface area to satisfy the second criterion based on the adopted percolation rate.

2.2.3 Building Research Establishment Digest on soakaways, BRE 151

A procedure for determining the size of soakaway from a soakage test is given in this document (Building Research Establishment, 1973). It is not clear whether the test is based on a type II or type III approach.

The test involves an auger hole 150 mm diameter drilled to the depth to which the soakaway would be founded. The hole is filled with water to a depth of 300 mm and the time taken for the hole to empty is recorded.

A graph is given which is used to convert the time taken for the test hole to empty and the area to be drained to a suggested soakaway size. It is assumed that the soakaway depth will be equal to the soakaway diameter.

The soakaway design is based on an inflow of 15 mm/hour over the area drained; stated as corresponding to a rainfall event of two hours duration and a one in ten year return period.

The analysis of the test is not explained. The design graph provided can be obtained by assuming the following theory, however.

The mean percolation rate is given by the volume flow rate divided by the average wetted surface area during the test. The mean hydraulic head is taken as half the depth which also equals the radius of the test pit. The percolation rate is related to the hydraulic head by a constant of proportionality, $[T^{-1}]$. The required soakaway size is found by calculating the wetted surface area required to infiltrate a flow rate equivalent to 15 mm/hour precipitation over the area drained, given the percolation rate obtained from the test and a hydraulic head equal to the depth and diameter of the prospective soakaway.

One of the main problems occurring with the use of this guideline is the fact that the procedure cannot be applied to different sized test holes, different shaped soakaways or different rainfall events because no details of the calculation procedure are given.

2.3 Other methods

2.3.1 Pratt (proposed replacement of BRE 151)

A replacement guideline to BRE 151 (Pratt, 1990, in draft) has been formulated and is likely to be issued in 1991. The test analysis is a Type II method.

A rectangular soakage pit is excavated 0.3 to 1 metre wide, 1 to 3 metres long and to the depth anticipated for the soakaway. The pit is filled with water and allowed to drain to almost empty, three times. The soil percolation rate is calculated on the basis of the time taken for the water level in the pit to fall from 75% full to 25% full and using the wetted area at 50% full.

The calculation of soakaway size is based on a ten year return period storm event but the method allows a range of storm durations to be considered. First, a depth and length of soakaway are assumed. For a ten year return period storm event with a chosen duration, the outflow from the soakaway is calculated per unit width assuming the percolation rate from the test and the wetted area of the soakaway at half full. The volume of soakaway required to store the difference between inflow and outflow at the end of the storm event is also calculated per unit width of soakaway. Combining the equations for percolation and storage, results in an equation which is solved to find the width of the soakaway.

The calculation is repeated for a range of storm durations to calculate the largest soakaway width required to cope with a ten year return period storm event. In order to provide a factor of safety, the base of the soakage pit is included in the calculation of percolation rate but the base of the soakaway is excluded when applying the percolation rate to the soakaway. A further criterion given is the fact that the soakaway should half-empty within 24 hours, assuming the constant percolation rate and a half-full wetted area.

When applying the method to square or cylindrical soakaways, the depth is chosen and the procedure is used to formulate a quadratic equation which is solved to find the length of side or diameter of the soakaway.

2.3.2 *King (Surveyor magazine)*

A procedure for designing soakaways appeared in an issue of Surveyor magazine (King, 1974) and is sometimes quoted as a guideline. The procedure is for the analysis of a test similar to BRE 151 and is a Type III method.

Two sizes of test pit can be used; an auger hole 150 mm diameter or a rectangular pit of 300 mm length and 250 mm width. The hole is filled with water to a depth of 300 mm and the time taken for it to empty is measured.

The soakaway is designed to provide a storage volume to accept 12.5 mm of rainfall over the impermeable area. This is taken from a storm of 75 mm/hour intensity and 10 minutes duration. The soakaway is also designed to provide a continuous infiltration capacity of 6 mm/hour over the drained area. This is to cope with 0.89 metres of annual rainfall falling on 160 days of the year and in 1 hour of the day.

The calculation procedure is not clear but it appears to be based on some form of free-surface theory whereby some, possibly dome-shaped, region of the soil is considered to be saturated. There appear to be some misconceptions in the analyses which involve a dimensionless 'coefficient of permeability'. For the calculation of soakaway size, a constant inflow to the soakaway and a constant and average outflow are assumed in order to calculate the required volume and surface area. The method is presented in the form of charts, graphs and formulae.

2.3.3 *Piezometer analysis*

The British Standard Code of Practice for site investigations (British Standards Institution, 1981) contains a method of analyzing falling head piezometer tests to obtain values of hydraulic conductivity. This procedure is sometimes applied to the analysis of soakage tests.

The method is based on the Hvorslev analysis (Hvorslev, 1951) using shape factors to represent the geometry of the system. In the original reference it is explicitly stated that the flow takes place beneath a water table of infinite extent. The analysis, therefore, cannot be applied to soakage tests above the water table.

2.3.4 *American Society of Civil Engineers*

The practice suggested by the Urban Resources Research Council of the American Society of Civil Engineers is presented in a recent book on stormwater detention (Stahre and Urbonas, 1990). This book contains a chapter on the design of infiltration facilities.

First, the site is classified for its suitability for infiltration drainage. This is done by allocating points for parameters such as soil type, vegetation and after-use. The points system used is intended for open infiltration systems and is not applicable to soakaways.

A minimum infiltration surface area of one half of the drained area is specified. Again, this criteria does not seem reasonable for soakaways.

It is then suggested that a percolation pit or trench should be designed according to guidelines issued by the Swedish Water and Sewer Works Association.

2.3.5 Swedish guidelines

The Swedish guidelines (Swedish Water and Sewer Works Association, 1983) provides a method for designing soakaways. The procedure is a Type III method but no soakage test is suggested. Instead it is assumed that the designer already has an independent measure of the hydraulic conductivity of the soil.

The inflow to the soakaway is calculated from the rain envelope method (Sjoberg and Martensson, 1982) which allows some water to infiltrate directly without runoff to the soakaway. The rain envelope method is a way of calculating the storage volume as the inflow minus a steady outflow due to the infiltration component (see Section 7.4.3 below). The size of soakaway is found by calculations of storage volume and constant infiltration rate by applying the rain envelope method to the soakaway itself (Jonasson, 1984). The calculations are based on Darcy's Law using the known hydraulic conductivity, a unit hydraulic gradient and flow through the sides of the soakaway only.

The validity of some of these assumptions are discussed in Section 2.4 below.

2.3.6 Danish guidelines

Present guidelines in Denmark are based on a Type I approach. The design rainfall is based on a high intensity event of two year return period. This is taken as 50 mm/hour for 10 minutes duration or 8.4 mm over the area drained. Assuming a porosity of 25% for the soakaway fill material, the required volume is 33.6 mm over the area drained. This is equivalent to 1 m^3 of soakaway for every 30 m^2 of drained area, which is how the advice is stated.

Plans are currently being made in Denmark to supersede the existing guidelines with a method which is similar in essence to the Swedish guidelines described in Section 2.6.5 above, but applied to vertical flow from percolation basins only.

2.4 Discussion

There are various guidelines available but none are universally adopted. The soakaway designer is faced with a number of options, each of which will result in a different size of soakaway being constructed.

Table 1 provides a summary of the guidelines available.

The majority of guidelines contain two stages: the determination of a measure of the infiltration capability of the soil and the application of this measure to the prototype soakaway under design storm conditions.

The first stage includes the soakage test. The infiltration capability is usually obtained by calculating the volume outflow rate when the water level is allowed to fall. Some guidelines require the pit to drain to empty. There are many discrepancies over the meaning, use and dimensions of the term 'permeability' within the guidelines.

In the second stage, the result of the soakage test is applied to the design storm. This is often taken as a 1 in 10 year, 2 hour duration, rainstorm event. The Pratt and the Swedish guidelines use a range of 1 in 10 year events to find the rainstorm with the most critical duration. In order to do this though, both methods assume a constant outflow rate from the soakaway throughout the event irrespective of the water level in the soakaway.

Table 1 Summary of existing guidelines on soakaway design

Guideline	hydraulic approach	design criteria
BS 8301	Type I	12 mm storage
PSA 125	Type II	27.5 mm storage and 1.25 mm/hour infiltration
BRE 151	Type II/III	15 mm/hour infiltration
Pratt	Type II	1 in 10 year intensity critical duration
King	Type II	12.5 mm storage 6 mm/hour infiltration
Swedish	Type III	rain envelope method, 10 year frequency
Danish	Type I	1 m^3 storage per 30 m^2 area

Type I methods provide for short intense storms only and do not allow for long duration rainfall or sequential events, because they take no account of infiltration.

Type II methods assume the infiltration to be uniformly distributed throughout the surface area of the soakage pit. They are likely to predict different size soakaways depending on the size and shape of the soakage test pit unless the test is carried out at prototype size.

Type III methods contain many assumptions which largely invalidate the theory being used. They often assume the flow to be always perpendicular to the infiltration surface, neglecting the three-dimensional nature of the soakaway problem. In applying Darcy's Law, all unsaturated flow effects are ignored and various geometries are assumed for the saturated region. To calculate the hydraulic conductivity and apply Darcy's Law, a unit hydraulic gradient is usually assumed.

The reason behind the assumption of a unit hydraulic gradient is the fact that, for vertical steady-state gravity-controlled infiltration, a phenomenon known as the 'piston effect' can occur. It has been shown (Phillip, 1969) that under the above conditions the pressure can be considered to be zero everywhere and so the hydraulic head becomes equal to the elevation head (see equation 11, Section 3.3 below). The steady infiltration rate then becomes equal to the hydraulic conductivity of the soil which is equivalent to assuming a unit hydraulic gradient.

If the piston effect does occur in a soakage test, the unit hydraulic gradient can only be applied in the vertical direction. The Swedish guidelines, for example, ignore vertical flow and apply the unit hydraulic gradient to the horizontal flow only, totally invalidating the assumption. This is the reason why the proposed Danish guidelines are intended to be applied only to vertical flow from large percolation basins. In this context, the assumption has been shown to be acceptable (Hovgaard and Mikkelsen, 1990).

If the Type III approach is taken with a simplified geometry for the flow domain and a unit hydraulic gradient, the analysis becomes equivalent to a Type II approach (see Section 5.2).

With either Type II or Type III approaches, further assumptions are made about the design inflow rates and the relative proportions accounted for by storage and infiltration. The different analyses used in the existing guidelines are considered further in Section 7 and their differences are highlighted by use of an example.

In order to advance our understanding of soakaway hydraulics and to provide a realistic analysis of soakage tests, it is necessary to consider the flow processes taking place.

3. *Flow processes*

3.1 Introduction

Soakaways are founded in the unsaturated soil zone above the water table. If this were not so, they could be classified as recharge wells or recharge galleries and methods from standard (saturated) groundwater hydraulics theory, such as the piezometer analysis described in Section 2.3.3, could be used for the analysis of the soakaway problem.

Figure 2 shows diagrammatic representations of four different conceptual models of soakaway hydraulics. Each part of the figure shows a cross-section through a soakage pit or soakaway and the extent and directions of the groundwater flow taking place in the surrounding soil.

The concept shown in Figure 2(a) assumes a uniform infiltration rate and corresponds to a Type II approach to the soakaway problem. The flow system generated around the infiltration surface is not taken into account in any way.

The representation shown in Figure 2(b) is equivalent to the reasoning behind the analysis given in the Swedish guidelines. Here the groundwater flow is taken to be saturated, always perpendicular to the soil surface and to extend as far in length as the head acting on the soil surface (unit hydraulic gradient).

Figure 2(c) shows a conceptual model assuming the free-surface approach. Here the soil is still separated into a region of saturated groundwater flow and a totally dry region where no flow can occur. Various assumptions need to be made to delineate the free surface. Groundwater flow within the saturated zone can then be analyzed.

If groundwater flow in the unsaturated zone is taken into account, the concept shown in Figure 2(d) is then applicable. It is envisaged here that a 'bulb' of saturation would become established around the soakaway. As groundwater flows away from the soakaway, the area through which it passes increases due to the three-dimensional nature of the flow. The specific discharge decreases and unsaturated flow effects become dominant. This is considered to be the most accurate conceptual model of soakaway hydraulics. Both the saturated and the *unsaturated* components of groundwater flow were therefore considered in this study in order to realistically describe the hydraulic behaviour of soakaways.

3.2 Saturated groundwater hydraulics

The theory of saturated groundwater flow was derived from an empirical observation known as Darcy's Law. This states that the rate of fluid flow in a porous medium, or soil, is directly proportional to the hydraulic head difference across the soil and the distance over which the head difference applies (Darcy, 1856).

The ratio of the head difference to the distance over which it acts is known as the hydraulic gradient and the coefficient of proportionality is known as the hydraulic conductivity of the soil with respect to the fluid. It has been extensively shown that, for laminar flow of Newtonian fluids in uniform saturated soils, the hydraulic conductivity is a constant for each fluid.

More universally, the intrinsic permeability of a soil is a constant for any fluid and is related to the hydraulic conductivity by the fluid properties (e.g. Bear, 1979).

$$\rho g K = \mu k \tag{1}$$

where	K	= intrinsic permeability	$[L^2]$
	k	= hydraulic conductivity	$[LT^{-1}]$
	ρ	= fluid density	$[ML^3]$
	μ	= fluid viscosity	$[ML^{-1}T^{-1}]$
	g	= acceleration due to gravity	$[LT^{-2}]$

The analysis of soakaway hydraulics was considered to involve water with constant properties. The theory below is therefore stated in terms of hydraulic conductivity rather than intrinsic permeability.

Darcy's Law acting in one dimension may be written as

$$Q = -Aki \tag{2}$$

where	Q	= volume flow rate	$[L^3T^{-1}]$
	A	= area perpendicular to flow	$[L^2]$
	i	= hydraulic gradient	$[-]$

The negative sign indicates that flow takes place down the hydraulic gradient, from a high head to a low head. Introducing the specific discharge as the flow rate per unit area and writing the hydraulic gradient in differential terms, equation (2) can be written as

$$q = -k\frac{d\phi}{dl} \tag{3}$$

where	q	= specific discharge	$[LT^{-1}]$
	ϕ	= hydraulic head	$[L]$
	l	= distance	$[L]$

Assuming the hydraulic conductivity to be isotropic, equation (3) can be applied to three dimensional coordinate systems by writing it as

$$q = -k\nabla\phi \qquad (4)$$

where ∇ is the differential operator

$$\nabla = \frac{\partial}{\partial x} + \frac{\partial}{\partial y} + \frac{\partial}{\partial z}$$

By considering the conservation of mass for fluid flow in a representative elemental volume of porous media, the change in specific discharge across an element is equal to the change in hydraulic head potential times the available storage within the element. The continuity equation can therefore be written as

$$-\nabla q = S\frac{\partial\phi}{\partial t} \qquad (5)$$

where S = storage coefficient [-]

t = time [T]

Combining equations (4) and (5) produces the equation of motion for saturated groundwater flow

$$k\nabla^2\phi = S\frac{\partial\phi}{\partial t} \qquad (6)$$

Equation (6) can be solved for saturated groundwater flow problems if the initial and boundary conditions are defined and the hydraulic conductivity and storage coefficient are known. In phreatic aquifers (water table conditions) the storage coefficient is usually taken to be the specific yield of the soil, S_y, and is defined as the volumetric quantity of water that will drain from the saturated soil under the influence of gravity.

3.3 Unsaturated groundwater hydraulics

Unsaturated groundwater flow differs from saturated groundwater flow in four main respects:

1. The pressure head in the fluid (or soil water tension) is due to the capillarity of the soil (or soil suction) and is negative with respect to atmospheric pressure.

2. The storage coefficient depends on the degree of saturation which is not constant but is a function of the pressure head, due to the presence of air in the soil pores.

3. The hydraulic conductivity depends on the geometry of the water-filled part of the system and so is not constant but is a function of the degree of saturation and thus the pressure head.

4. Because the soil is not saturated, there is a potential for gravity induced vertical flow into underlying unsaturated soil.

Some of the above concepts were introduced by Buckingham (1907). Richards (1931) hypothesised that Darcy's Law could be modified to apply to the movement of moisture in unsaturated soils and he is generally credited with formulating the first general equation of motion for unsaturated groundwater flow. The hypothesis was experimentally verified (Childs and Collis-George, 1950) and is generally accepted in various forms, such as equation (12) below.

Taking account of point 3 above, Darcy's Law in the form of equation (4) can be written as

$$q = -k(\theta)\nabla\phi \qquad (7)$$

where θ = volumetric moisture content [-]

Taking account of point 2 above and allowing the moisture content to vary, the continuity equation, equation (5), for unsaturated flow may be formulated as

$$-\nabla q = \frac{\partial\phi}{\partial t} \qquad (8)$$

Combining equations (7) and (8) leads to an equation of motion for unsaturated groundwater flow.

$$\nabla k(\theta)\nabla\phi = \frac{\partial\phi}{\partial t} \qquad (9)$$

The fluid pressure can be related to the (negative) pressure head according to

$$p = \rho g \psi \qquad (10)$$

where p = fluid pressure $[ML^{-1}T^{-2}]$

ψ = pressure head [L]

For unsaturated groundwater flow, the hydraulic head is the sum of the pressure head and the elevation head.

$$\phi = \psi + z \qquad (11)$$

where z = elevation head [L]

Furthermore, the hydraulic conductivity and the moisture content functions can be written in terms of the pressure head as $k(\psi)$ and $\theta(\psi)$ respectively. Substituting these features into equation (9), provides the equation of motion for unsaturated groundwater flow stated in terms of the pressure head, ψ, as the independent variable.

$$\nabla k(\psi)\nabla(\psi + z) = \frac{\partial\theta}{\partial\psi}\frac{\partial\psi}{\partial t} \qquad (12)$$

Equation (12) is often referred to as the modified Richards equation and takes account of each of the four features listed above that distinguish unsaturated from saturated groundwater flow.

3.4 Unsaturated soil properties

In order to solve equation (12), it is necessary to know the functional relationships between the hydraulic conductivity and the pressure head, $k(\psi)$ and between the moisture content and the pressure head, $\theta(\psi)$.

For convenience, the hydraulic conductivity - pressure head function may be written as

$$k(\psi) = k_s k_r(\psi) \qquad (13)$$

where k_s = saturated hydraulic conductivity

k_r = relative hydraulic conductivity $(0<k_r<1)$

and the moisture content - pressure head function can likewise be written as

$$\theta(\psi) = n\theta_r(\psi) \qquad (14)$$

where n = porosity

θ_r = relative saturation $(S_r < \theta_r < 1)$

and S_r = specific retention

The specific retention is the volumetric quantity of water that will not drain from the soil under the influence of gravity and so is related to the specific yield as

$$n = S_y + S_r \qquad (15)$$

A large number of possible relationships have been proposed for $k(\psi)$ and $\theta(\psi)$. Many of these have been chosen because they are mathematically expedient rather than physically correct. Much of the analytical work that has been applied to solving equation (12) has assumed an exponential relation for $k(_\iota)$ and a linear relation for $\theta(\psi)$. These functions lead to a quasi- linearization of the Richards equation (Pullan, 1990). Another commonly used approach is to relate the $k(\psi)$ and $\theta(\psi)$ functions to the air-entry pressure of the soil (e.g. Brooks and Corey, 1964). This line of reasoning allowed the development of permeameters based on measuring the air-entry pressure in the soil rather than the flow rates (Bouwer, 1966).

Mualem and Dagan (1976) compiled a catalogue of the unsaturated flow relationships between relative hydraulic conductivity, relative saturation and pressure head for one hundred soils which had been studied in soil physics literature. Mualem (1976) then proposed a new model for describing the relative hydraulic conductivity function which could be applied to all the soils considered in the catalogue. From Mualem's work, Van Genuchten (1980) proposed an equation to describe both the functions $k(\psi)$ and $\theta(\psi)$ using the same coefficients. This is a useful relationship because only two curve fitting coefficients are introduced to describe both relations and it is applicable to a wide range of natural soils.

For the purposes of this study, therefore, the van Genuchten functional relationships between relative hydraulic conductivity, relative moisture content and pressure head were adopted.

These relations are

$$k_r(\psi) = \left[1 - (\alpha\psi)^{N-1}\right]\left[1 + (\alpha\psi)^{N}\right]^{\frac{1-N}{2N}} \tag{16}$$

and

$$n\theta_r(\psi) = S_r + S_y\left[1 + (\alpha\psi)^{N}\right]^{\frac{1}{N}+1} \tag{17}$$

Where α [L^{-1}] and N [-] are curve fitting coefficients.

A further complication in unsaturated groundwater hydraulics is the fact that both $k(\psi)$ and $\theta(\psi)$ are hysteretic functions. The relationships with pressure depend on whether the soil is wetting or drying at the time (Childs, 1969). The soakaway problem basically concerns wetting, or imbibition, of the soil and so hysteresis of soil properties is ignored within this study.

3.5 Infiltration Borehole Permeameters

Infiltration borehole permeameters are shallow boreholes located in soil above the water table and used for estimating the saturated hydraulic conductivity of the soil. A constant head of water is maintained in the borehole and the flow rate required to maintain a steady water level is measured. Because of the similarity with the soakaway problem, it is worth considering developments in the analysis of borehole permeameters, here.

The earliest analyses proposed for infiltration borehole permeameters were based on the free-surface approach, Figure 2(c). Simplified solutions were given based on applying Darcy's Law to the saturated flow region that is assumed (e.g. Glover, 1953). A more detailed analytical model of the problem was proposed by Reynolds et al (1983). They developed a general pressure flow solution, of which the Glover solution is a special case.

The researchers later extended their theory to take account of the effects of unsaturated flow, assuming quasilinear unsaturated soil properties (Reynolds et al, 1985; Elrick and Reynolds, 1986).

Approximate analytical solutions were also developed by Phillip (1985) for the conceptual model shown in Figure 2(d), again using the quasilinear approach to unsaturated soil properties.

Stephens and Neuman (1982a, 1982b, 1982c) modelled infiltration borehole permeameters by numerically solving the modified Richards equation. They demonstrated that large errors can occur when free-surface theory is applied and unsaturated groundwater flow effects are ignored. They also predicted the establishment of a distinct and limited zone of saturation around the borehole, which is in keeping with the concept shown in Figure 1(d).

Stephens et al (1987) applied the numerical model to a number of soils with hydraulic properties taken from Mualem's catalogue (Mualem and Dagan, 1976). They also applied both the Brooks-Corey (1964) and the Van Genuchten (1980) models of unsaturated soil properties to each soil. They conducted parameter sensitivity analyses on the borehole dimensions to generate a large number of data sets describing the results of numerically modelling the infiltration borehole permeameter problem. The results of their study were presented in the form of simplified equations derived from multiple non-linear regression analyses on the data sets. The formula that they derived using the Van Genuchten unsaturated soil properties model is

$$\log\left(\frac{Q}{r\,H k_s}\right) = 0.653\ \log\left(\frac{H}{r}\right) - 0.257\ \log(\alpha) - 0.633\ \log(H)$$

$$+\,0.021\ \left(\frac{H}{r}\right)^{0.5} - 0.313\ N^{-0.5} + 1.456\ r + 0.453 \qquad (18)$$

where H = depth of water in the borehole [L]

r = radius of borehole [L]

and α is in units of cm^{-1}

The hydraulic parameters not included in equation (18), S_y and S_r, were not considered to significantly effect the steady flow rate measured or the saturated hydraulic conductivity calculated.

An infiltration borehole permeameter test was instrumented and the field measurements proved to be in accordance with the theory assumed. The field measurements also demonstrated the concept shown in Figure 2(d) to be appropriate.

The reason why equation (18) cannot be applied directly to the soakaway problem is that it was formulated within limits which apply to borehole type geometries and cannot sensibly be extrapolated to the dimensions of soakage test pits and soakaways.

3.6 Discussion

The hydraulic properties which influence saturated groundwater flow are the saturated hydraulic conductivity, k_s, and the specific yield, S_y, of the soil. The equation of motion which describes saturated groundwater flow is equation (6).

The hydraulic properties which influence unsaturated groundwater flow are the specific retention, S_r, and the Van Genuchten soil coefficients, N and α. Equation (12) describes the motion of saturated and unsaturated groundwater flow.

The hydraulic analysis of infiltration borehole permeameters is similar to that for the soakaway problem. Developments in this field have led from analytical solutions of Darcy's Law, assuming a geometry bounded by a free surface, to regression analyses of the results of numerically solving the modified Richards equation. The most current research reported on infiltration borehole permeameters indicates that the concept of combined saturated and unsaturated groundwater flow, as shown in Figure 2(d), provides the best representation of the hydraulic problem. Numerical model simulations of infiltration borehole permeameter tests have been supported by field measurements (Stephens et al, 1987).

The hydraulic concept shown in Figure 2(d) seems to be the most accurate description of soakaways. It is therefore appropriate to consider a Type IV approach to soakaway hydraulics based on saturated and unsaturated groundwater flow in accordance with equation (12). This approach was investigated by using numerical modelling techniques.

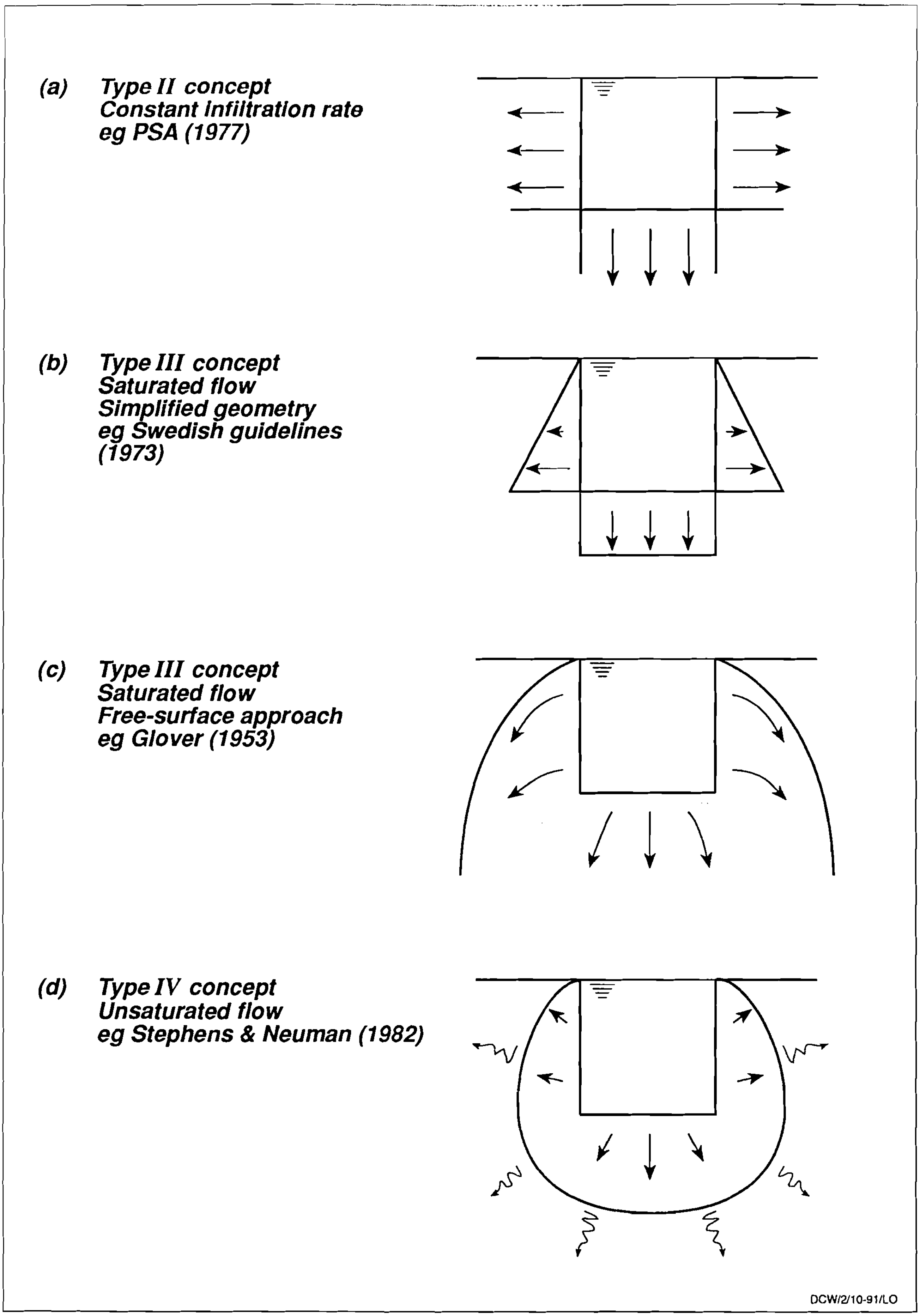


Fig 2 Diagrammatic representation of some conceptual models of soakaway hydraulics

4. *Numerical modelling*

4.1 Introduction

Equation (12) for saturated and unsaturated groundwater flow may be solved analytically for some simple situations using simplified relationships for the unsaturated soil properties; such as the quasilinear approach (eg. Phillip, 1968). In order to solve equation (12) for a range of realistic conditions which are relevant to the soakaway problem, it is possible to obtain numerical solutions. The techniques for doing this have been available for some time. It is only in recent times, however, that the computational ability for obtaining practical numerical solutions has been readily available.

Equation (12) is in terms of the pressure head, ψ. The soil properties, $k(\psi)$ and $\theta(\psi)$, are also in terms of pressure. The equation is therefore highly non-linear and is computationally demanding and very intensive to solve.

It was established that a computer code existed which was capable of solving equation (12) under suitable conditions to model the soakaway problem. The code, named SUTRA, was obtained and set up to simulate saturated and unsaturated groundwater flow from a cylindrical soakaway.

4.2 Finite element code SUTRA

The finite element code SUTRA (Voss, 1984) was developed jointly by the United States Geological Survey and the United States Air Force. It is based on a hybridization of finite element (in space) and integrated finite difference (in time) methods. The method allows the simulation of flow in irregular regions through the use of quadrilateral elements with four corner nodes. Parameters may vary in value throughout the modelled region. Specified boundary and point conditions may be constant or varied with time. Simulations are in two space dimensions but a three-dimensional quality is provided by allowing the thicknesses of the two-dimensional regions to vary in a third direction.

The code is primarily designed for two-dimensional simulation of saturated flow and either solute or energy transport in variable density systems. SUTRA contains appropriate numerical algorithms for dealing with the non-linearities involved in these problems, and is therefore capable of simulating unsaturated flow, but it was not specifically designed for this application (Voss, 1984). Fine spatial and temporal discretization are therefore required in the model to avoid problems with non-convergence of the solution. In fact, numerical convergence problems hampered the modelling work throughout the course of the study. One advantage of using SUTRA for the soakaway modelling is that the code is modular and is therefore relatively simple to modify.

Because it is designed to cope with density dependent flow, SUTRA works in terms of pressure and intrinsic permeability. The data and results listed in the tables and figures in this report were therefore converted to the relevant units according to equations (1) and (10). For these purposes it was assumed that the acceleration due to gravity was equal to 10 m/s^2, the density of water was 1000 kg/m^3 and the viscosity of water was 0.001 Ns/m^2. Consequently, 1 metre of pressure head was considered to correspond to 10,000 Pascals (N/m^2) of pressure and a hydraulic conductivity of 10^{-3} m/s corresponded with an intrinsic permeability of 10^{-10} m^2.

4.3 Soils modelled

Three different soils were modelled, comparable with soil types which might typically be considered for soakaway drainage. The soils used were chosen from Mualem's catalogue (Mualem and Dagan, 1976) and corresponded with some of the soils modelled in the analysis of infiltration borehole permeameters (Stephens et al, 1987).

The soil properties used corresponded with coarse sand, fine sand and gravelly sand. The hydraulic properties of these soils are listed in Table 2 and the hydraulic conductivity and the relative saturation - pressure head relationships are shown in Figure 3.

Table 2. Hydraulic properties of the soils modelled

soil description	saturated properties		unsaturated properties		
	k_s (m/s)	S_y	S_r	N	α (m^{-1})
gravelly sand	2.8×10^{-6}	0.247	0.079	2.84	1.5
fine sand	4.4×10^{-6}	0.248	0.05	4.36	1.6
coarse sand	1.4×10^{-4}	0.265	0.04	2.64	6.0

4.4 The model

4.4.1 Discretization

The domain of the model was taken as a cylindrical volume of soil, 10 rn diameter and 10 m in depth. A soakage pit or soakaway was taken to be located at the centre and at the ground surface.

Taking advantage of the radial symmetry of the problem, it was only necessary to discretize a two-dimensional vertical slice of 5 m radius and 10 m depth. The three-dimensional nature of the problem was restored by assigning a thickness to each nodal point equal to the circumference of a circle at that radius. In this way, the model grid produced a series of stacked hollow cylindrical elements.

The vertical slice was discretized as shown in Figure 4. The horizontal axis, representing the radius from the centre of the soakaway, was divided into nineteen segments and the vertical axis, representing the depth below ground level, was also divided into nineteen segments. This created a grid consisting of 361 elements and 400 nodes.

4.4.2 *Boundary conditions*

All the model boundaries were set so that no flow could occur across them. At the upper boundary at the ground surface, AB in Figure 4, no flow can cross the boundary because no soil exists above the ground. The boundary along the centreline, AD on Figure 4, is a line of symmetry and so must be a streamline across which no flow can occur.

The lower and outer boundaries, DC and BC respectively on Figure 4, were also assigned no-flow conditions. In this case these are not realistic conditions but it was assumed that the boundaries were set far enough away from the source of flow that they would not influence the model results. In order to ensure that this assumption remained valid during the simulations, it was necessary to observe the pressures on boundaries DC and BC to ensure that no response occurred there. If a response at the boundary did occur, the condition was considered to be violated and the results of the simulation were discarded. In this way, the distances to the two boundaries could be considered to be great enough to have no influence on the problem modelled.

The soakaway was located around the origin, A on Figure 4. Many different sizes and shapes of soakaway were simulated in the modelling exercises by incorporating different numbers of elements in the soakaway excavation. The soakaway region was essentially removed from the domain of the model by assigning a very low hydraulic conductivity, $10^{-9}k_s$, to the elements encompassed by the soakaway. Specified pressures were assigned to the nodes which coincided with the boundary of the soakaway excavation. The values used were obtained from a hydrostatic pressure distribution beneath the water level in the soakaway.

4.4.3 *Initial conditions*

For each soil that was modelled, it was necessary to establish appropriate initial conditions from which to start the simulations.

Unsaturated groundwater, like saturated groundwater, is at equilibrium under hydrostatic conditions. This occurs when the potential for flow, ψ, is zero. From equation (11), $\theta = 0$ when $\psi = -z$. At the position of a water table, the interface between saturated and unsaturated flow, the pressure head is zero. Below the water table the soil is saturated, the pressure is positive and increases linearly with depth as a hydrostatic distribution. Above the water table the soil is unsaturated and the pressure is negative but continues to decrease linearly with height above the water table in accordance with a hydrostatic distribution.

An appropriate pressure distribution for the initial conditions could therefore be based on hydrostatic pressure in relation to a water table at a specified level. For each soil, the moisture content distribution will be related to the distance above the water table by the relations shown in Figure 3, with the pressure head being equivalent to the height above a water table.

Following this principle, in a situation where the water table is at a great depth, there would be a very high negative soil water pressure close to the ground surface. From Figure 3, at high negative pressures, there are only negligible quantities of mobile water present in the soil and, in particular, the hydraulic conductivity is close to zero. Extremely high hydraulic gradients develop for flow to occur under these circumstances and numerical problems were encountered when modelling very dry soils. It therefore proved to be impractical to use a hydrostatic pressure head distribution for initial conditions greater than a certain level above a water table; the level depending on soil type and the properties shown in Figure 3. This provided a lower limit on the values used for the initial pressures and corresponding moisture contents.

If the initial moisture content is too high, it will influence the results of the simulation. Stephens et al (1987) found that the initial conditions had little effect on the results of the simulations if the initial moisture content was below a certain level. The criteria that they used was an initial pressure corresponding to a relative hydraulic conductivity, k_r, of 0.03 or less. They applied this initial condition uniformly across the domain of their model.

In order to determine upper limits for the initial moisture contents, a series of constant head test simulations (see Section 4.5.1 below) of a one metre diameter and one metre deep soakage pit were conducted for each soil using different initial conditions. By comparing the results, upper limits for the initial moisture contents were determined by trial and error. Beneath these limits, the initial conditions did not influence the modelling results.

The limits found for the initial conditions are listed in Table 3 in terms of pressure head. Values using the criteria given by Stephens et al are also given.

Table 3. Pressure head limits on initial conditions (m)

soil	Stephens criteria $k_r = 0.03$	trial and error upper limit	lower limit
gravelly sand	-0.90	-5.0	-2.5
fine sand	-0.80	-4.0	-2.0
coarse sand	-0.25	-0.5	-0.3

Because of the need to observe the boundary pressures, it proved impractical to use uniformly distributed initial conditions. These created a degree of 'background' flow as the system settled toward a hydrostatic equilibrium. This also tended to obscure the flows taking place solely due to the presence of the soakaway. It therefore proved necessary to use a combination of uniform and hydrostatic initial conditions depending on the soil being modelled and its properties as shown in Figure 3.

The initial conditions described above were used when applying the model to constant head soakage tests. Once the constant head simulations were run to steady conditions, it was possible to simulate falling head tests using the situation at the end of the constant head tests as the initial conditions. The simulations of time-varying inflows to soakaways were conducted using the results of the falling head tests, after the soakaway has emptied, to provide the initial conditions.

4.5 Simulations

4.5.1 Constant head tests

In the constant head test, the soakage pit is filled with water and kept full. The rate at which water needs to be added to maintain the water level is measured and the value is used for calculations of infiltration rate.

The model was set up to simulate constant head soakage tests in order to conduct the sensitivity analyses on the initial conditions described above. Further simulations were conducted, for each soil, varying the dimensions of the soakage pit. The constant head simulations were achieved by specifying fixed pressure heads to each of the model nodes at the borders of the soakage pit. The values assigned corresponded with hydrostatic positive pressure beneath the water level in the pit. In all the simulations, it was assumed that the pit was filled with water to ground level.

In agreement with other work (Stephens and Neuman, 1982c), it was found that initially high flow rates occurred which reduced with time, toward a steady level. In fact, a steady-state solution does not occur because the wetting front emerging from the soakage pit continues to move outward, though at an ever-decreasing rate (Phillip, 1966) and the flow rate will also decrease continuously (Fitzsimmons, 1972). In an engineering sense, however, steady conditions can be considered to become established after some time from the start of the constant head test. A judgement was therefore required as to when the flow rate could be considered to be steady, for the purposes of the modelling results.

Figure 5 shows the flow rates required to maintain a constant head in a one metre diameter and one metre deep soakage test pit in each of the soils modelled and Table 4 lists the times by which conditions were considered to be steady for modelling purposes. Under field conditions and within experimental error, flow rates could be considered to be stable much earlier than the times listed in Table 4. The time ranges listed reflect the different size of soakage pit modelled; larger pits take longer to establish steady conditions.

Figures 6 and 7 show some typical results. These simulations relate to a one metre diameter and one metre deep soakage test pit. Figure 6 shows the evolution of pressure head through a soil profile beneath the centreline of the pit during the course of a soakage test. Figure 7 shows the distribution of pressure head throughout the upper part of the soil profile at the end of the test.

Table 4. Times taken to establish steady conditions

soil	range of times taken
gravelly sand	1 - 2.5 days
fine sand	6 - 24 hours
coarse sand	15 - 60 minutes

4.5.2 Falling head tests

In practice, it is more common to measure the rate at which the water level in the soakage pit falls as the water is allowed to drain away. This is an easier measurement to make than the flow rate in a constant head test.

In order to simulate falling head tests, the specified pressure heads along the border of the soakage test pit were required to respond to the fall in water level within the soakage pit. At the beginning of the simulation the pit is full. After each timestep in the model simulation, the volume outflow from the soakage pit was computed and the new water level in the soakage pit was calculated, taking account of the porosity of any fill material that may have been placed in the pit for sidewall stability. For the next timestep, the nodes were reassigned specified pressures in accordance with hydrostatic pressure beneath the new water level.

When the soakaway became empty the simulation was terminated.

4.5.3 Time-varying inflows

Simulations were conducted with time-varying inflows in order to model prototype soakaways under realistic working conditions. The inflow rates were based on synthetic unit hydrographs (Shaw, 1983). These assume a linear increase in flow rate up to a peak and a linear decrease from the peak to the end of the hydrograph. In this way, the hydrographs are described in terms of the peak flow, Q_p, the time at which the peak occurs, T_p, and the time at which the flow ceases, T_b. In simple cases, it can be assumed that the rainfall landing on an impervious surface immediately reaches the soakaway, in which case the unit hydrograph is symmetrical so that $T_b = 2T_p$. It was felt that the synthetic unit hydrograph provided a significantly better representation of the inflow rates than block rainfall hyetographs, for the purposes of soakaway modelling.

The specified pressure heads on the soakaway border were calculated in a similar manner to those for the falling head tests. This time, however, the calculation at each timestep was based on the difference between the inflow to the soakaway, in the form of the hydrograph, and outflow due to infiltration. Again, the porosity of any fill material in the soakaway is included in the mass balance calculation.

In Figure 8, a typical response of a soakaway to an approximate synthetic unit hydrograph is shown in terms of both the inflow and outflow rates. For this example, Q_p is 0.25 l/s, T_p is 1 hour and T_b is 2 hours which corresponds approximately to a hyetograph of 15 mm/hour for 2 hours duration over a 30 m_2 drained area. Figure 9 shows the corresponding water level that occurred in the soakaway as a result of the inflow event shown in Figure 8. For the coarse sand soil, during much of the inflow event, the soil is drying faster than it is wetting so no ponding of water occurs.

4.6 Results

4.6.1 Constant head tests

Constant head test simulations were conducted for soakage test pits ranging in size from 0.15 to 2.5 metres in diameter and from 0.15 to 2.0 metres in depth. The steady flow rates achieved for the tests simulated are shown in Figures 10 and 11 for each of the soils modelled.

The nature of these results is discussed further in Section 5.2 below.

4.6.2 Falling head tests

Figure 12 shows model results representing the fall in water level during a falling head test in a 1.0 metre diameter soakage pit with various depths and Figure 13 shows the corresponding information for a 0.63 metre deep soakage pit of various diameters.

4.6.3 Time-varying inflows

Figure 14 presents the results of modelling a one metre diameter and one metre deep soakaway in each of the soils modelled, under a range of inflow hydrographs. The figure shows the maximum water level that occurred in the soakaway during each simulated storm event. For the example in Figure 14, it is assumed that the synthetic unit hydrograph is symmetrical.

An approximate correlation can be made between the parameters describing the synthetic unit hydrograph and block rainfall events, for comparison with the existing guidelines (see also Section 7.5.1 and Figure 27, below). It can be assumed, approximately, that the duration of the rainfall is the same in each type of event. The block rainfall intensity will then be equal to half of the peak flow rate per unit area drained. For example, for a block rainfall of 15 mm per hour intensity and 2 hours duration over an area of 33 m^2 the peak flow rate, Q_p, will be 30 mm per hour per unit area or 1 m^3 per hour which is 0.28 litres per second. For the example given in Figure 13(b), the maximum water level in the soakaway would therefore be 0.9 m in response to a 15 mm intensity and 2 hour duration block rainfall event.

4.7 Discussion

The numerical simulations of groundwater infiltration from soakage pits and soakaways were performed to provide parameter sensitivity analyses and an insight into hydraulic behaviour of soakaways. The modelling procedure was designed to minimise the number of factors involved (e.g. the initial and boundary conditions) so that a clear picture of the problem might be achieved.

The numerical modelling process is complex and laborious and so it would not be expected to be used as part of a standard design procedure. Although a numerical model has been used to assess the performance of an infiltration drainage scheme in the past (Herath and Musiake, 1987), the author is not aware of any schemes that have been specifically designed with the aid of numerical solutions of the Richards equation.

In order to apply the results of the numerical modelling more universally to soakaway design, simplified analytical solutions are needed.

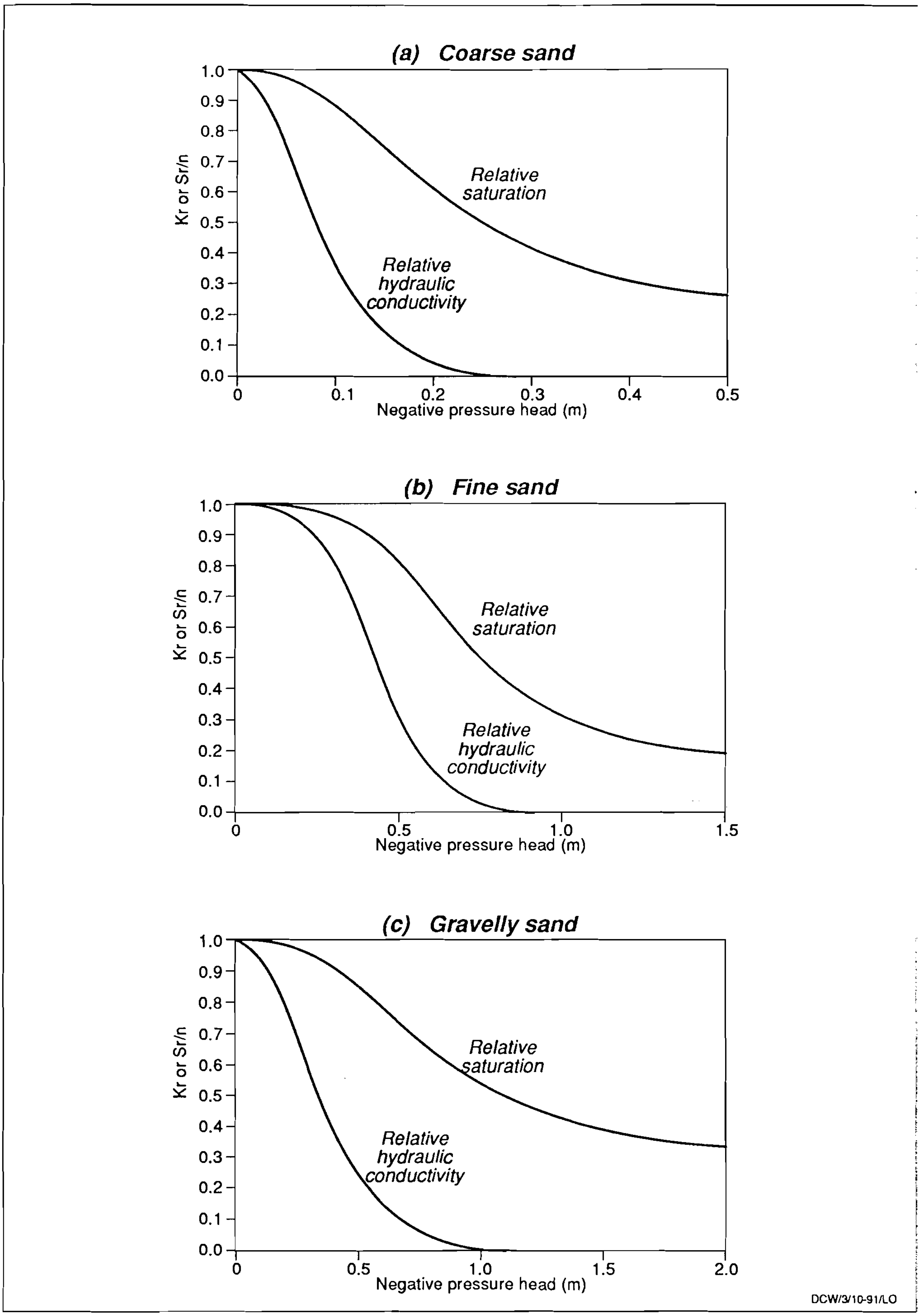


Fig 3 Unsaturated soil properties – pressure head relationships

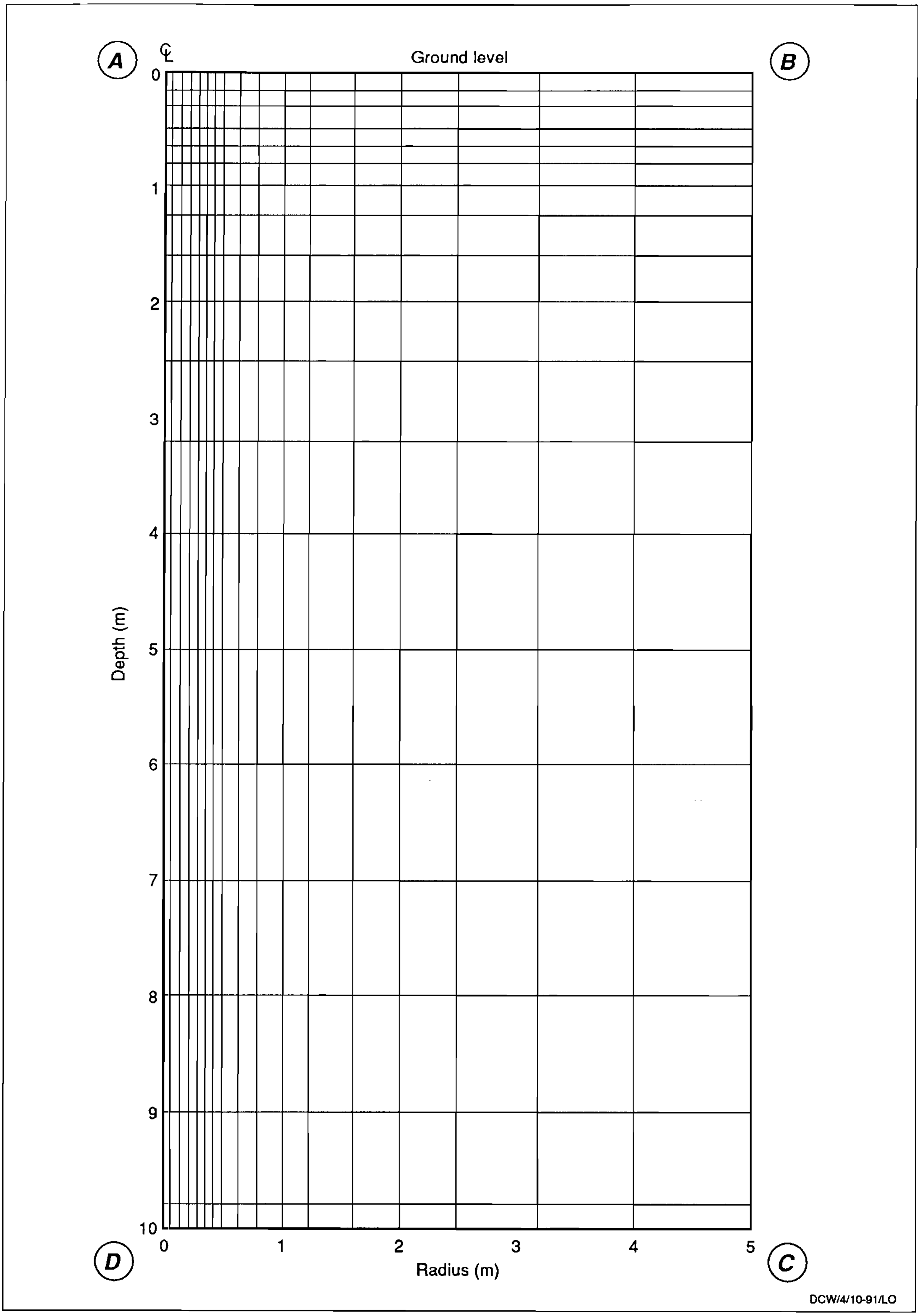


Fig 4 Model domain discretization

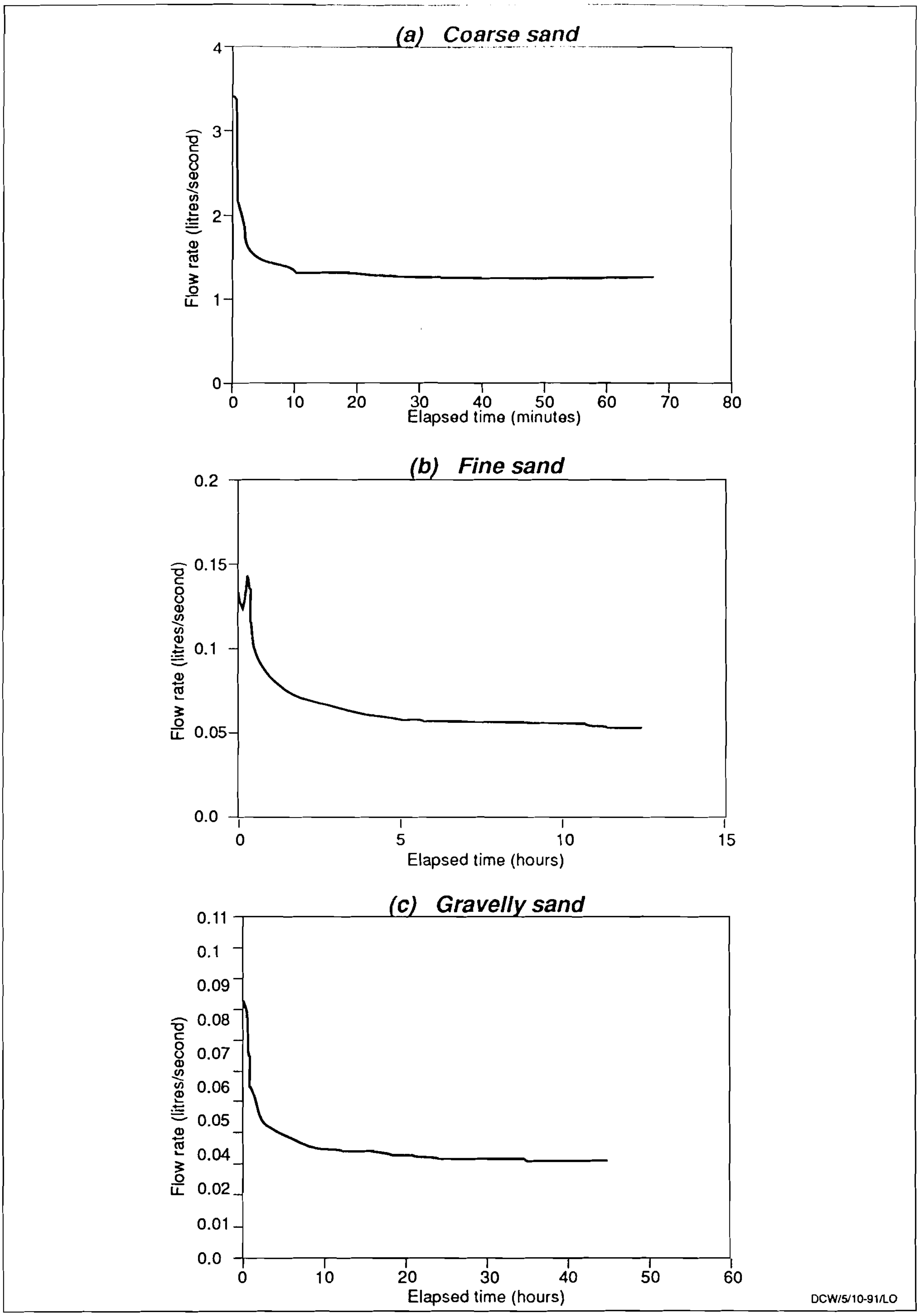


Fig 5 Flow rates required for a constant head test

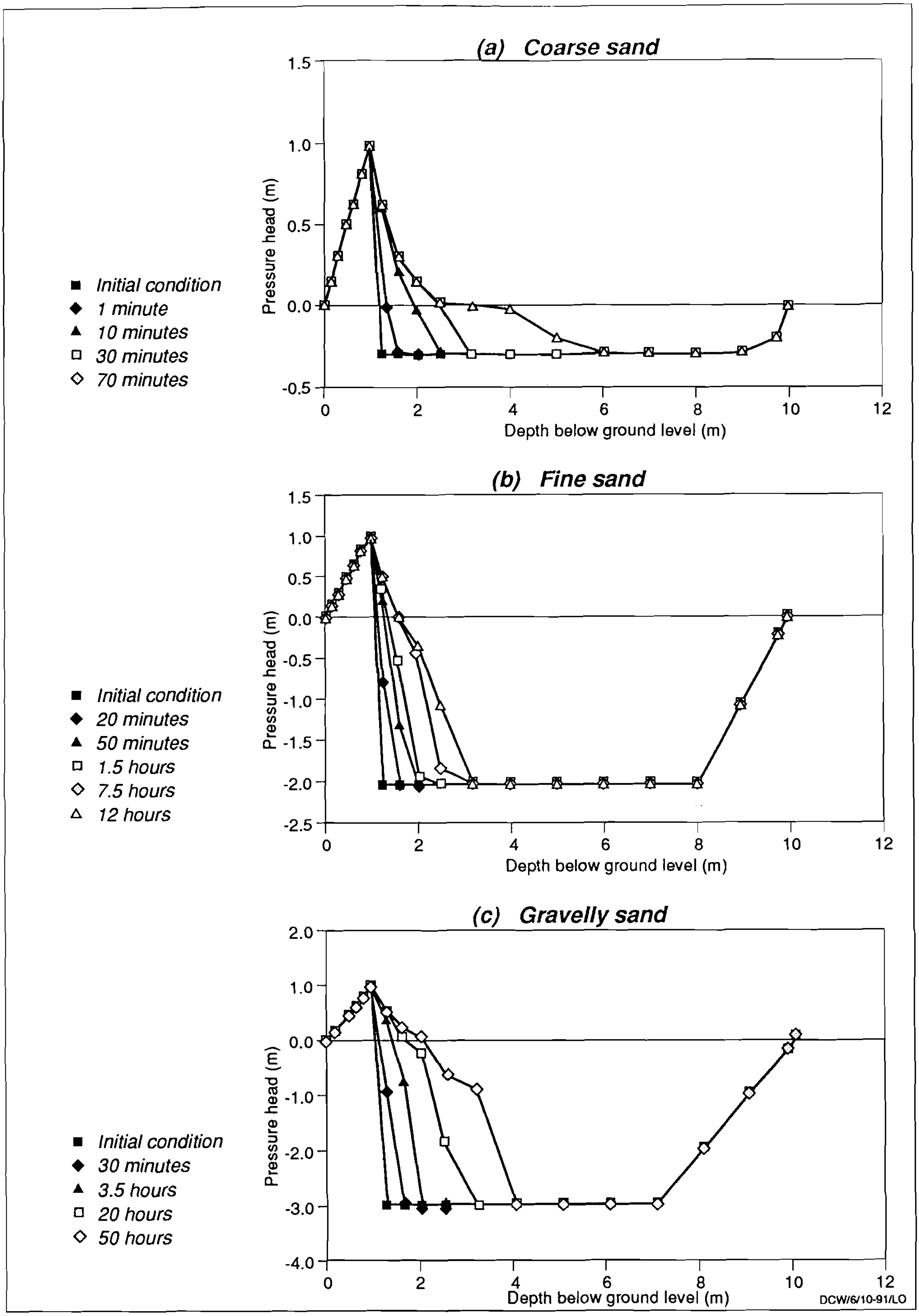


Fig 6 Evolution of pressure beneath a soakage test pit during a constant head test

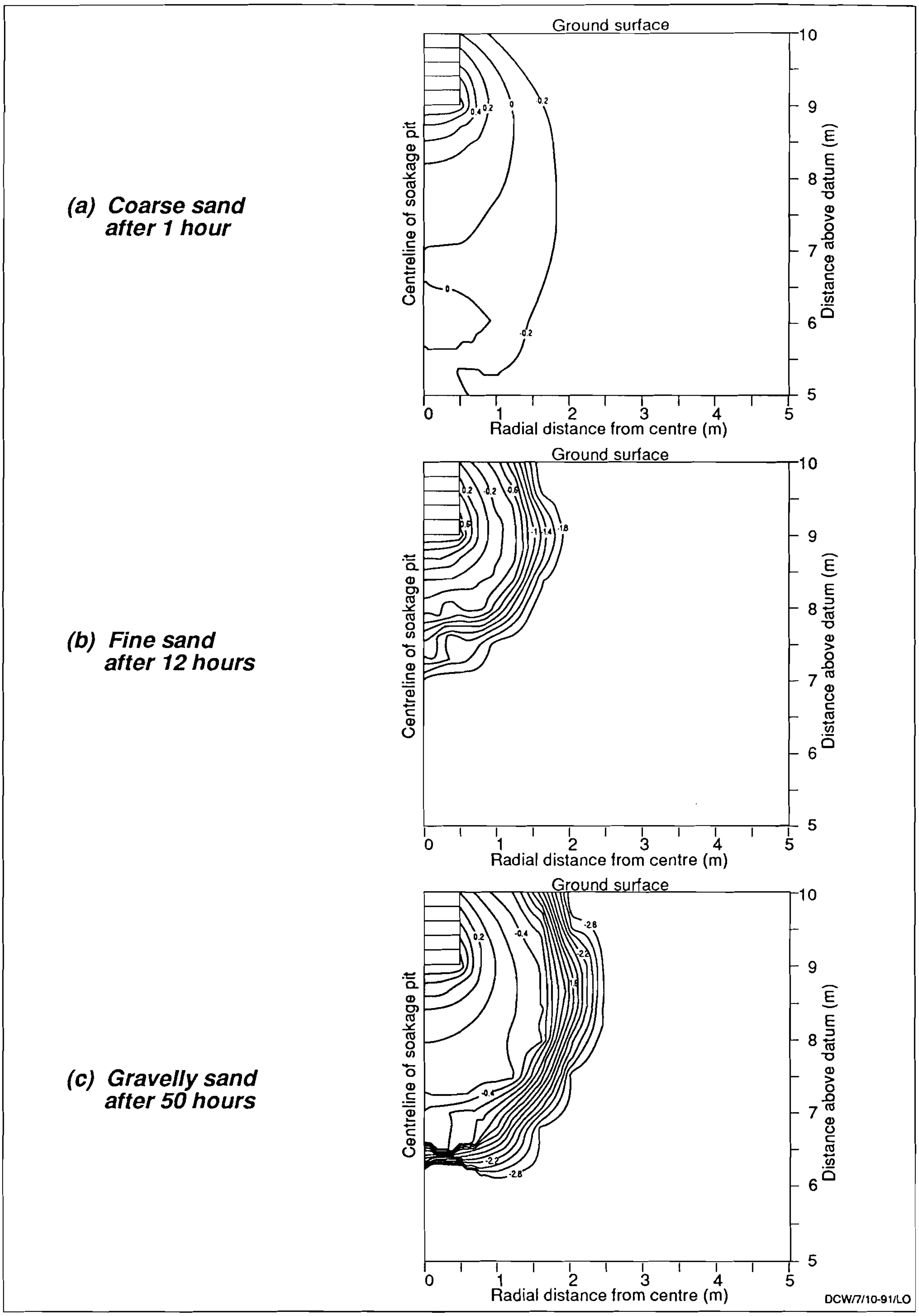


Fig 7 Distribution of pressure head (m) around a soakage test pit after a constant head test

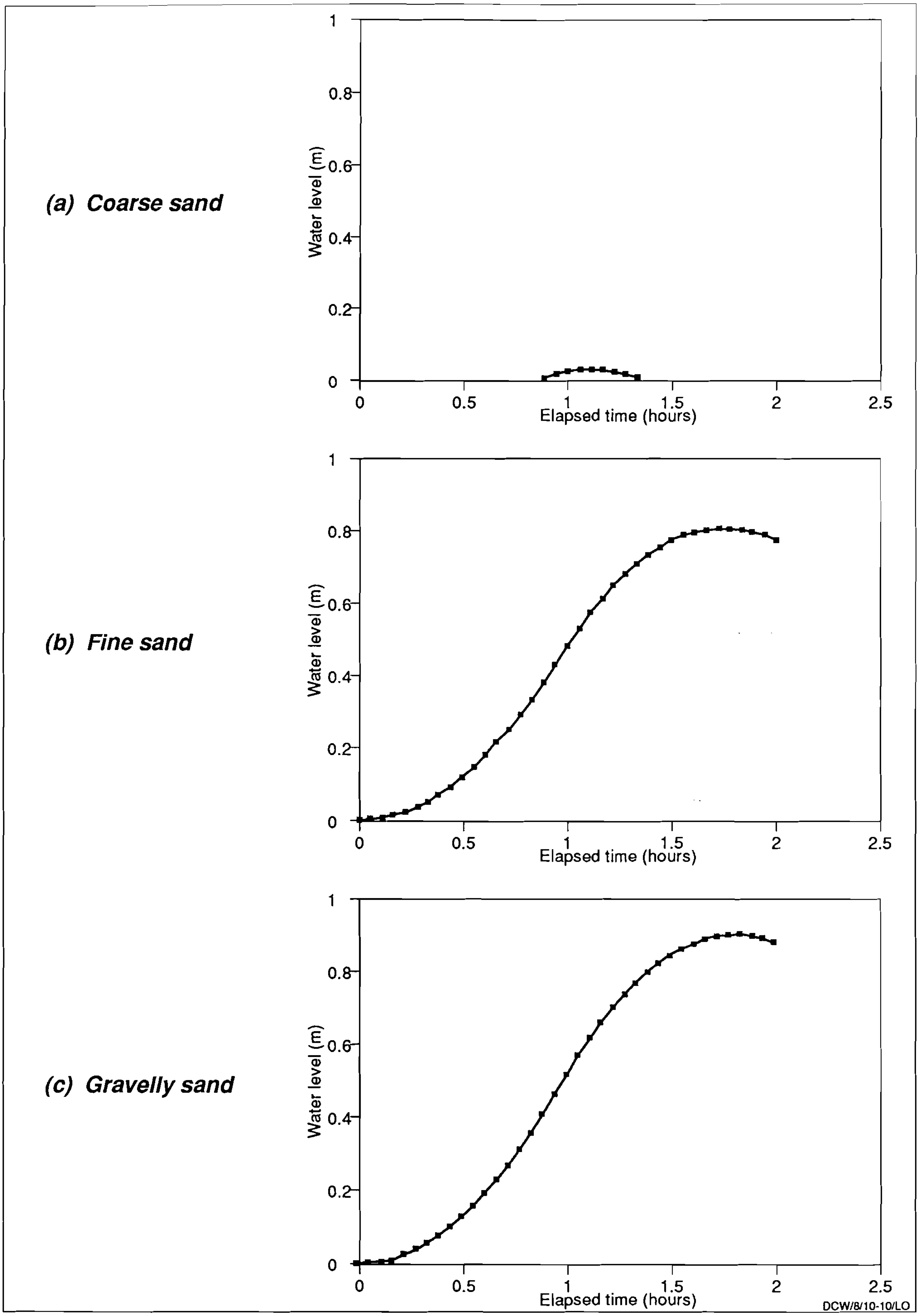


Fig 8 Water level response in a soakaway – numerical model results

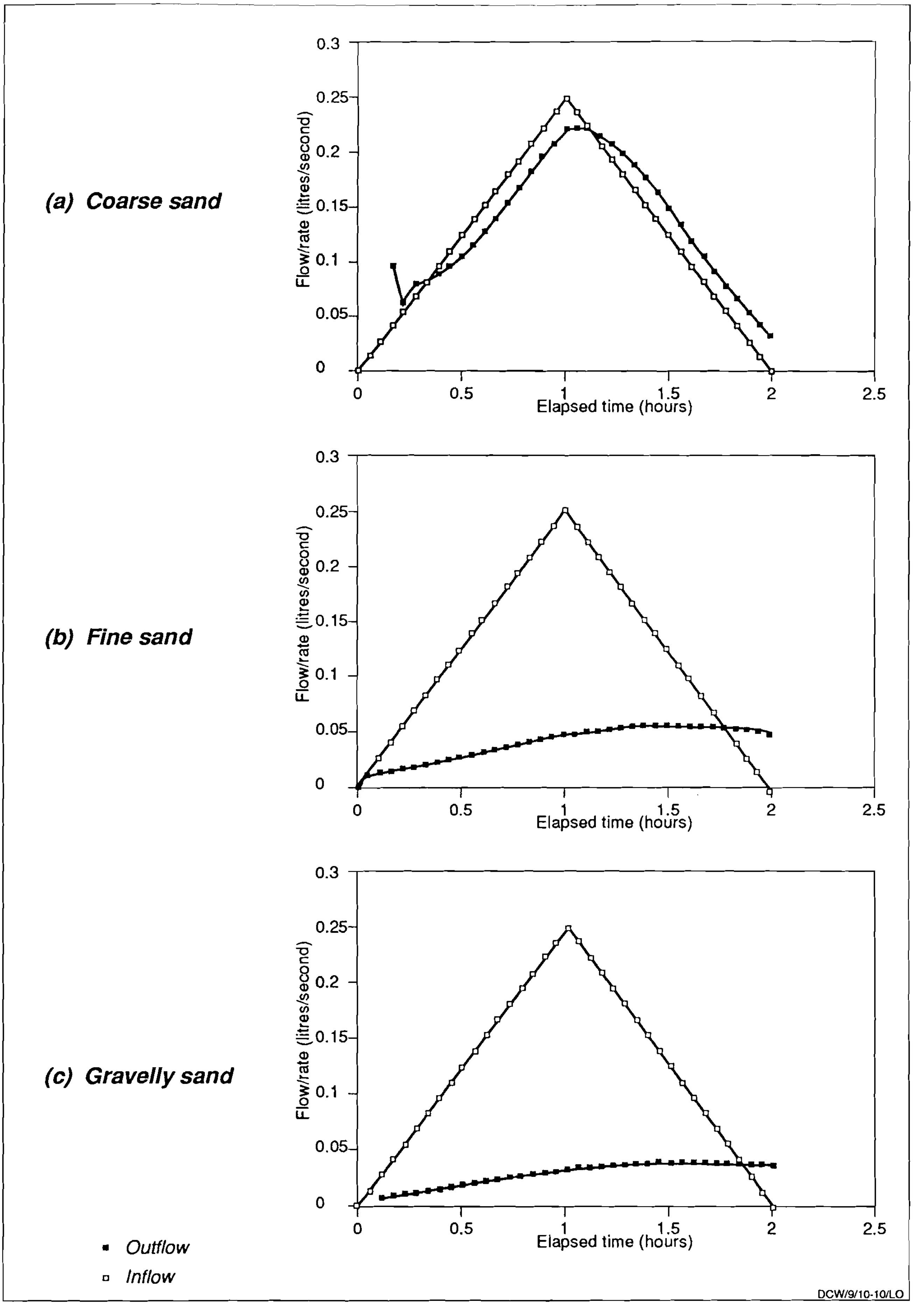


Fig 9 Outflow response of a soakaway – numerical model results

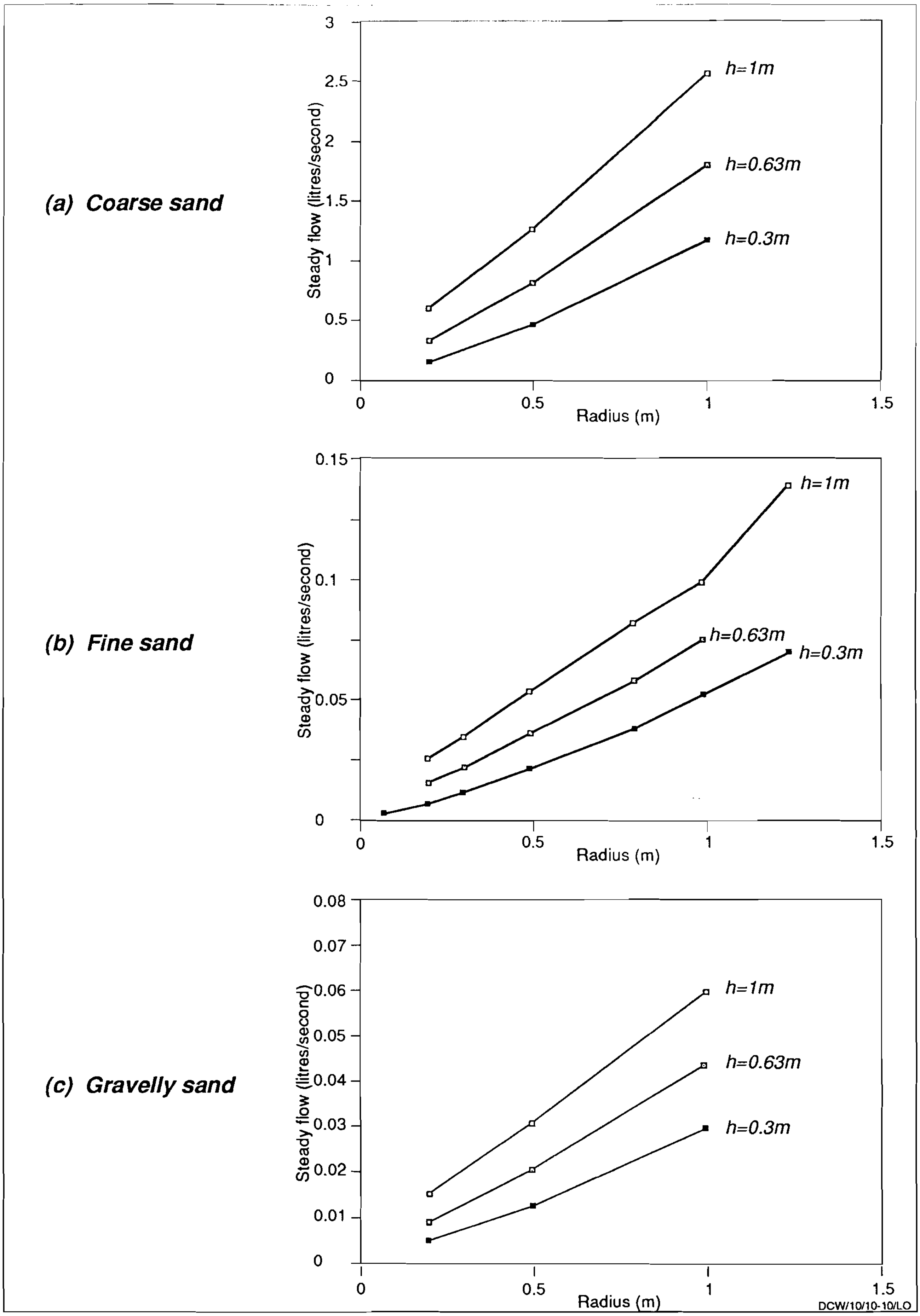


Fig 10 Steady flow rates for fixed depth constant head tests – numerical model results

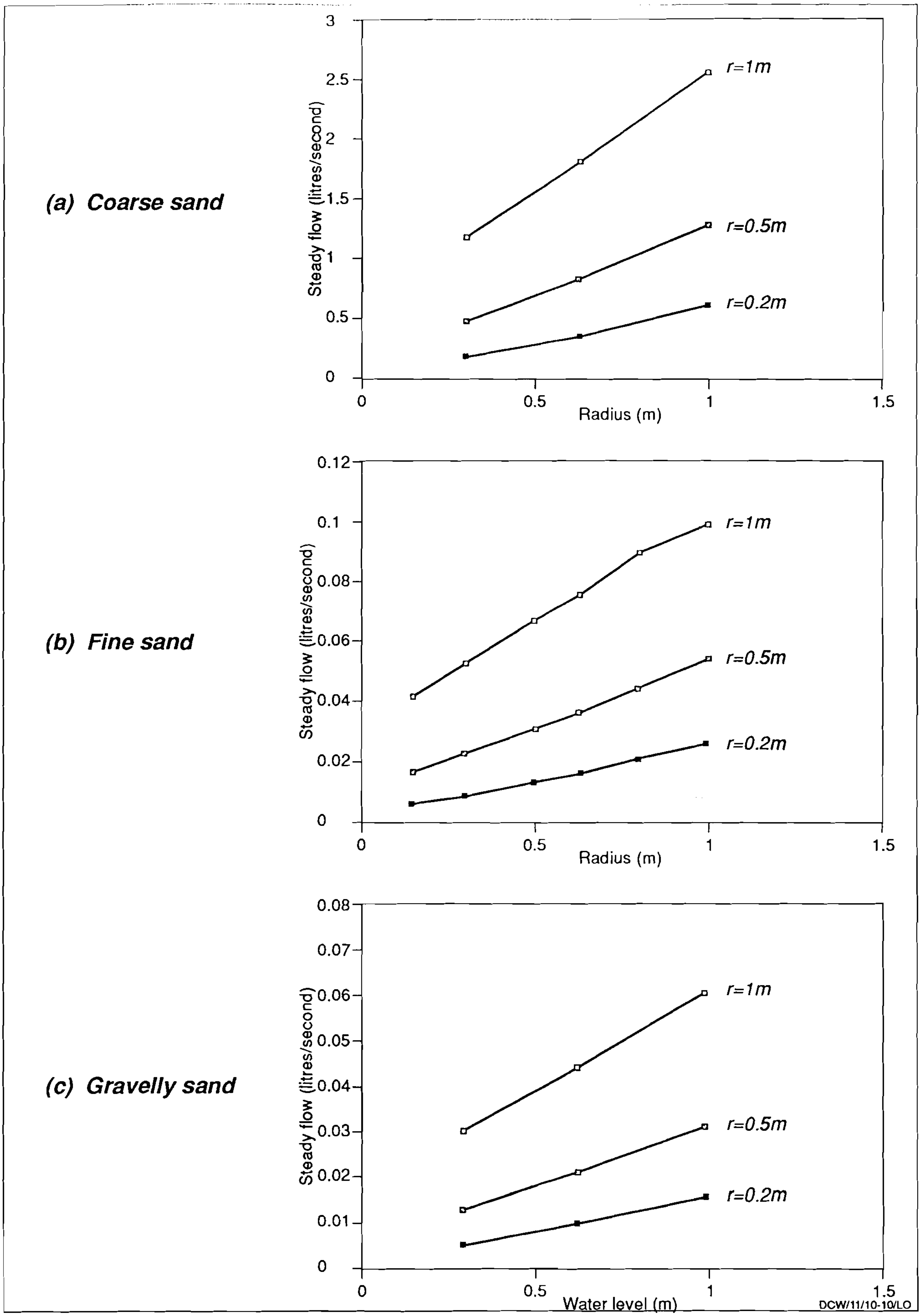


Fig 11 Steady flow rates for fixed radii constant head tests – numerical model results

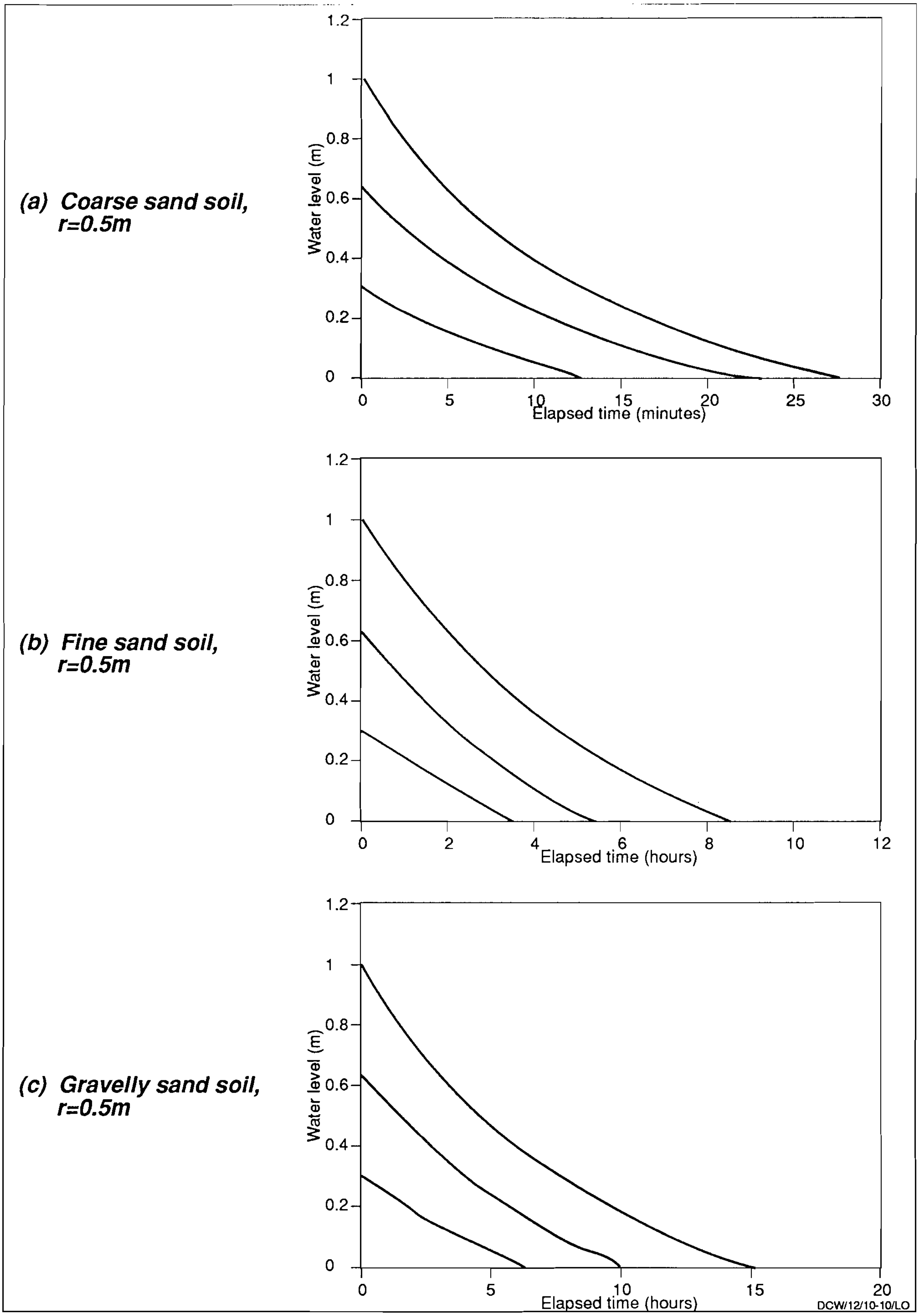


Fig 12 Falling head tests in a 1.0m diameter test pit – numerical model results

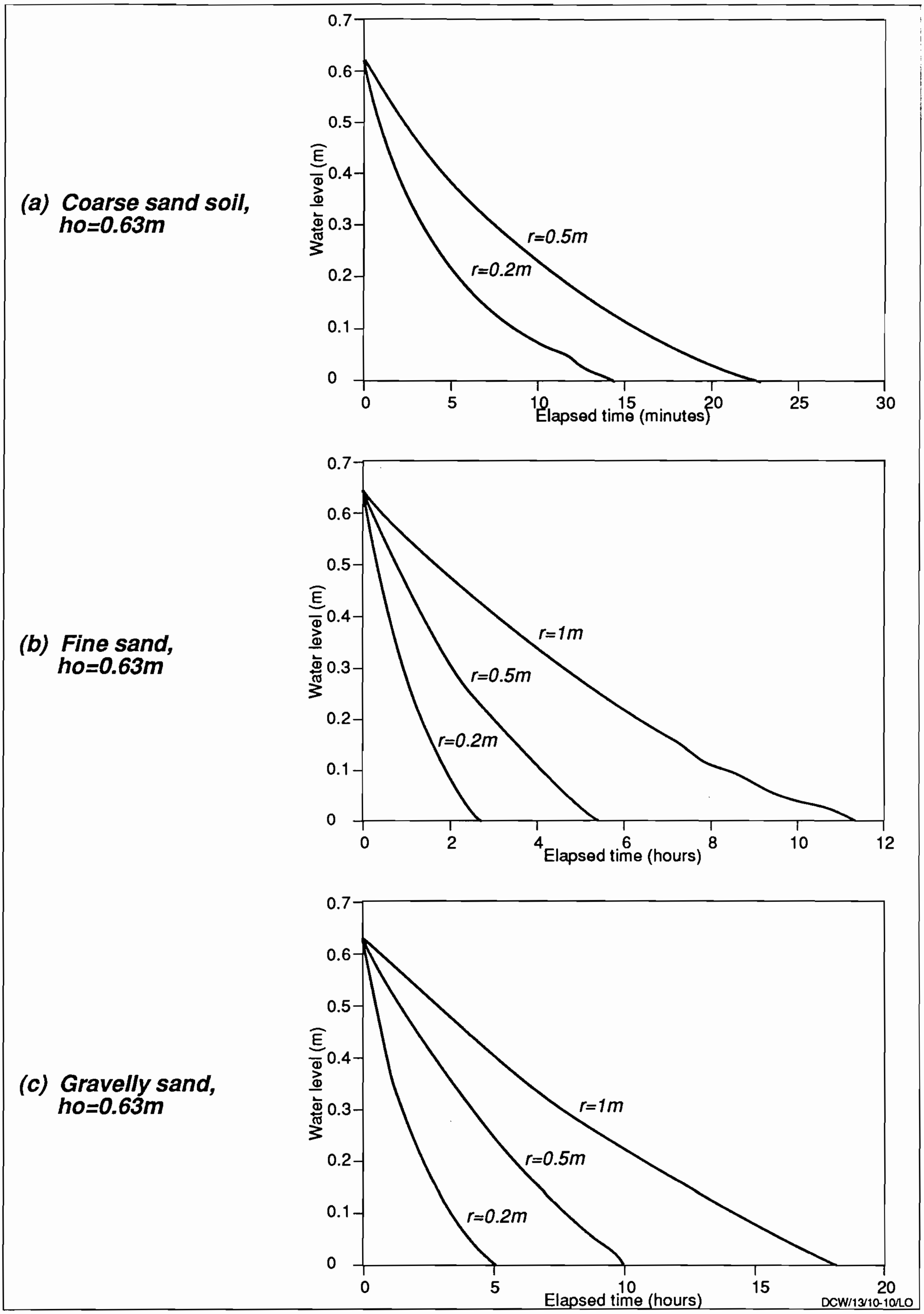


Fig 13 Falling head tests in a 0.63m deep soakage pit – numerical model results

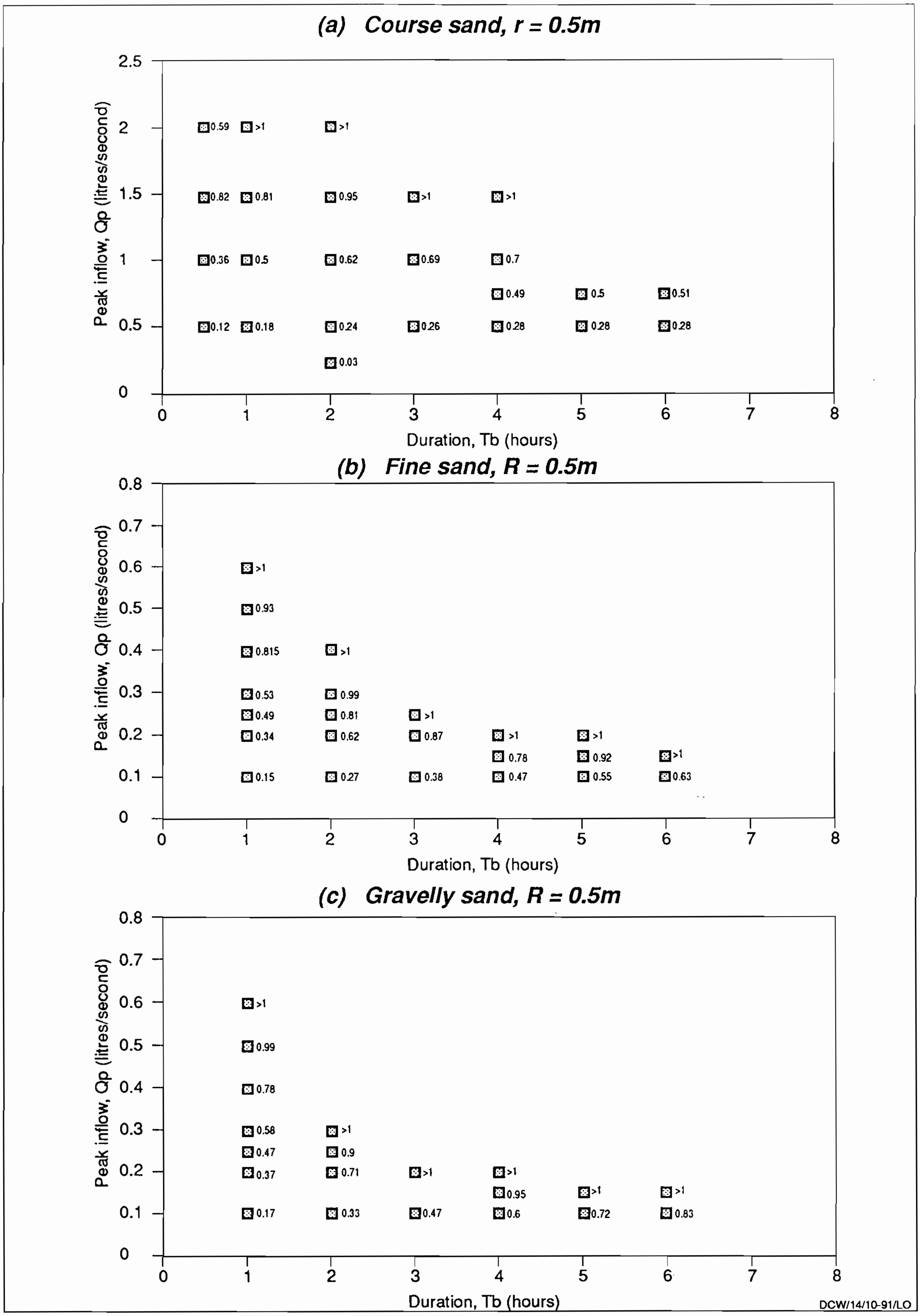


Fig 14 Soakaway performance in response to time-varying inflows – Numerical model results. Peak water level for various storms

5. *Analytical modelling*

5.1 Introduction

As an alternative to using numerical modelling techniques directly, the results of sensitivity analyses, such as those described above, can be used to test and justify any simplified analytical solutions to the soakaway problem. Any simplified solution should predict similar results to the numerical model, if the theory is to be considered valid.

A number of simplified analytical solutions were considered and tested against the steady flow rates achieved during constant head tests as predicted by the numerical simulations.

One simple analytical model was further developed to calculate water levels in soakage test pits during falling head tests and in soakaways during time-varying inflows.

5.2 Method

5.2.1 Constant head tests

According to the theory invoked in the analysis of the test in BRE 151, for both the test and the prototype soakaway, the ratio of the flow to the product of the wetted surface area and the water depth is assumed to be a constant. This hypothesis can be written as

$$Q = CAh$$

or, for cylindrical soakaways,

$$Q = C(\pi r^2 h + 2\pi r h^2) \qquad (19)$$

where	C	= constant	$[T^{-1}]$
	A	= wetted surface area	$[L^2]$
	r	= radius of soakage pit	$[L]$
	h	= depth of water in soakage pit	$[L]$

Figure 15 shows the results of the numerical simulations of the constant head tests, as shown on Figures 10 and 11, compared with those predicted using equation (19). The value of the constant, C, used in Figure 15 is listed in Table 5. This value was obtained from a least-squares-best-fit comparison between the numerical and analytical model results. As can be seen from Figure 15, the hypothesis is not very accurate.

According to the Type II approaches such as used by PSA 125 and Pratt (1990), the infiltration rate per unit wetted area is a constant for both soakage test and soakaway. This hypothesis can be written as

$$Q = q_1 A$$

or

$$Q = q_1(\pi r^2 + 2\pi rh) \qquad (20)$$

where q_1 = infiltration rate $[LT^{-1}]$

If a Type III approach is used with the assumption of a unit hydraulic gradient then, from equation (2), the outflow is also described by equation (20) but with the infiltration rate equal to the hydraulic conductivity of the soil.

Figure 16 presents a comparison between the results of the numerical simulations and equation (20) using the values of the infiltration coefficient obtained by least-squares-best-fit comparison and listed in Table 5. The best-fit value is similar to, but consistently higher that, the saturated hydraulic conductivity. From Figure 16, it can be seen that this model is a considerable improvement on the previous hypothesis.

This single infiltration coefficient model can be improved by separating the vertical and horizontal infiltration components to produce a two-coefficient model. Here it is assumed that the infiltration rate through the base and the infiltration rate through the sides of the pit or soakaway are both constant (but not necessarily equal) under steady conditions. For cylindrical soakaways, this hypothesis may be written as

$$Q = q_b(\pi r^2) + q_s(2\pi rh) \qquad (21)$$

where q_b = infiltration rate through the base $[LT^{-1}]$
q_s = infiltration rate through the sides $[LT^{-1}]$

Figure 17 presents a comparison between the results of the numerical simulations and equation (21), using the best-fit values of the two infiltration coefficients as listed in Table 5. Although all of the soils modelled were isotropic, a greater infiltration rate appears to have been achieved through the sides than the base in each case. It is clear from Figure 17 that the two-coefficient model provides a reasonable description of the steady flow rates achieved from constant head soakage tests, as compared with the results of the numerical simulations over the range of dimensions modelled.

Table 5. Results of analyses of constant head soakage tests

soil	C (s^{-1})	q_1 (m/s)	q_b (m/s)	q_s (m/s)	k_s (m/s)
gravelly sand	0.00756	6.5×10^{-6}	3.75×10^{-6}	8.7×10^{-6}	2.8×10^{-6}
fine sand	0.0187	1.1×10^{-5}	7.7×10^{-6}	1.45×10^{-5}	4.4×10^{-6}
coarse sand	0.317	2.7×10^{-4}	1.4×10^{-4}	3.5×10^{-4}	1.4×10^{-4}
field experiment			1.7×10^{-5}	1.2×10^{-5}	1.0×10^{-5}

5.2.2 Falling head tests

The two-coefficient model for constant head tests, equation (21), can be extended to time-varying conditions such as falling head tests. If the water level is not maintained steady but is allowed to vary, the change in water level may be equated with the difference between the inflow and outflow of the soakaway or test pit.

$$n_s \pi r^2 \frac{dh}{dt} = Q_i(t) - Q \tag{22}$$

where $Q_i(t)$ = inflow function $[L^3T^{-1}]$

n_s = porosity of soakaway fill material [-]

n_s = 1.0 if no fill material is present

Combining equations (21) and (22)

$$n_s \pi r^2 \frac{dh}{dt} = Q_i(t) - q_b \pi r^2 - q_s 2\pi r h$$

or

$$\frac{dh}{dt} + \frac{2q_s h}{n_s r} = \frac{Q_i(t)}{\pi r^2} - \frac{q_b}{n_s} \tag{23}$$

which is of the form

$$\frac{dh}{dt} + ah = f(t)$$

where

$$a = \frac{2q_s}{n_s r}$$

and

$$f(t) = \frac{q_b}{n_s} + \frac{Q_i(t)}{\pi r^2}$$

which is of the form

$$f(t) = b + ct$$

The form of f(t) is linear, so the general solution of equation (23) is

$$h = \frac{b}{a} - \frac{c}{a^2} + \frac{c}{a}t + d\left[\exp(-at)\right] \quad (24)$$

where d is an arbitrary constant.

The value of d is determined from the initial conditions.

when $t = 0$, $h = h_0$, where h_0 is the depth of water in the pit at the start of the test.

Substituting the initial conditions into equation (24),

$$d = \left(h_0 - \frac{b}{a} + \frac{c}{a^2}\right)$$

so

$$h = \frac{b}{a} - \frac{c}{a^2} + \frac{c}{a}t + \left(h_0 - \frac{b}{a} + \frac{c}{a^2}\right)\exp(-at) \quad (25)$$

Equation (25) is the general equation describing the water level in a soakage pit or soakaway according to the two-coefficient model.

For falling head tests, the inflow, Q_i, is zero.

Hence

$$b = -\frac{q_b}{n_s}$$

and

$$c = 0$$

Substituting values of a, b and c in equation (25),

$$h = -\frac{r\,q_b}{2q_s} + \left(h_0 + \frac{r\,q_b}{2q_s}\right)\exp\left(-\frac{2q_s}{n_s r}t\right) \qquad (26)$$

Equation (26) describes the water level at any time, t, during a falling head soakage test, according to the two-coefficient model developed earlier for constant head tests.

Equation (26) can be used to describe the fall in head once the two coefficients are known. Equation (26) cannot easily be solved, however, for the inverse problem of determining the coefficients from known water levels. To do this it is necessary to substitute two different water levels and the corresponding times into equation (26) and to solve the two equations simultaneously for q_b and q_s. The resulting equations are highly non-linear in terms of the two coefficients and so an iterative procedure is required to solve the simultaneous equations computationally.

In order to provide a simpler procedure for the inverse problem, a method was developed based on the gradients of early and late water level falls. The evaluation of the constants relied on the difference between the rates of fall when the test hole is nearly full and when the test hole is almost empty. These proved to be insensitive parameters and so the procedure is not reported here.

5.2.3 Time-varying inflows

Equation (25) describes the water level in a soakaway with any functional inflow, $Q_i(t)$. The falling head condition described by equation (26) is just a special case of equation (25) where $Q_i(t)=0$.

The water level resulting from an inflow due to a synthetic unit hydrograph can be modelled by using equation (25) with the appropriate function for $Q_i(t)$. Because the synthetic unit hydrograph is a discontinuous function, the time to peak and the time after the peak must be treated separately.

First considering the rising inflow ($0 < t < T_p$),

$$Q_i = \frac{Q_p}{T_p}t$$

$$b = -\frac{q_b}{n_s}$$

$$c = \frac{Q_p}{\pi r^2 T_p}$$

Substituting these values into equation (25)

$$h = -\frac{r\,q_b}{2q_s} - \frac{Q_p n_s}{4\pi q_s^2 T_p} + \frac{Q_p t}{2\pi r\, q_s T_p}$$

$$+ \left(h_0 + \frac{r\,q_b}{2q_s} + \frac{Q_p n_s}{4\pi q_s^2 T_p} \right) \exp\left(-\frac{2q_s}{n_s r} t \right) \qquad (27)$$

Equation (27) can be used to model the water level in a soakaway at any time up to the peak of a synthetic unit hydrograph, according to the two-coefficient hypothesis.

The maximum water level in the soakaway, h_m, cannot occur before the peak inflow, however. In order to predict the maximum water level due to the inflow it is therefore necessary to model the conditions after the peak inflow has occurred. To do this it is first necessary to calculate the water level, h_p, at the time of the peak inflow, T_p, from equation (27).

Now considering the falling inflow ($T_p < t < T_b$),

$$Q_i = \frac{T_b Q_p}{T_b - T_p} - \frac{Q_p t}{T_b - T_p}$$

$$b = -\frac{q_b}{n_s} + \frac{Q_p}{n_s \pi r^2}$$

$$c = -\frac{Q_p}{n_s \pi r^2 (T_b - T_p)}$$

Substituting these values into equation (25) starting at $t = T_p$ and $h = h_p$

$$h = -\frac{r\,q_b}{2q_s} + \frac{Q_p}{2\pi r\, q_s} + \frac{n_s Q_p}{4\pi q_s^2 (T_b - T_p)} - \frac{Q_p (t - T_p)}{2\pi r\, q_s (T_b - T_p)}$$

$$+ \left(h_p + \frac{r q_b}{2q_s} - \frac{Q_p}{2\pi r\, q_s} - \frac{n_s Q_p}{4\pi q_s^2 (T_b - T_p)} \right) \exp\left(-\frac{2q_s}{n_s r} (t - T_p) \right) \qquad (28)$$

Equation (28) can be used to model the water level in a soakaway at any time after the peak of the synthetic unit hydrograph inflow, according to the two-coefficient hypothesis.

The maximum water level, h_m, at time, t_m, can be determined from the conditions

$$h = h_m, \quad t = t_m, \quad \frac{dh}{dt} = 0$$

Differentiating equation (25) with respect to time, for $t > T_p$

$$\frac{dh}{dt} = \frac{c}{a} - a\left(h_p - \frac{b}{a} + \frac{c}{a^2}\right)\exp(-a(t - T_p))$$

so the time at which the maximum water level occurs,

$$t_m = T_p + \frac{1}{a}\ln\left[1 - \frac{ab}{c} + \frac{a^2 h_p}{c}\right]$$

or

$$t_m = T_p + \frac{n_s r}{2q_s}\ln\left[1 + \left(\frac{2q_s q_b \pi r}{n_s Q_p} + \frac{2q_s}{n_s r} - \frac{4\pi q_s^2 h_p}{n_s Q_p}\right)(T_b - T_p)\right] \quad (29)$$

In order to predict the maximum water level occurring in a soakaway in response to a synthetic unit hydrograph according to the two-coefficient hypothesis, the following procedure can be used, based on equation (25).

Use equation (27) to calculate h_p, the water level that occurs at the time of the peak inflow, T_p. Use equation (29) to calculate t_m, the time at which the maximum water level in the soakaway occurs. Use equation (28) to calculate h_m, the water level at time, t_m.

5.3 Results

Figures 18 and 19 present analytical predictions of the water level falls according to equation (26), for comparison with the results of the numerical simulations of falling head tests shown in Figures 12 and 13. The results shown in Figures 18 and 19 were calculated using the best-fit values of the infiltration coefficients derived from the constant head test results, as listed in Table 5, and not from the numerical falling head results, with which they may be compared.

Figure 20 shows the response of a soakaway to a time-varying inflow as predicted by the two-coefficient analytical model using equations (26) and (27) to determine the water level. Figure 21 shows the inflow and corresponding outflow calculated by using equation (21) and the water levels shown in Figure 20. These results can be compared with the numerical predictions shown in Figures 8 and 9.

Figure 22 shows the predicted soakaway performance in terms of the peak inflow that is able to completely fill up each of four different sized soakaways for different storm durations. Also, Figure 23 shows the maximum water level that occurs in response to four different rainstorm events in different sizes of soakaway. The results presented in Figures 22 and 23 were achieved by solving equations (26), (27) and (28) for synthetic unit hydrograph inflows.

5.4 Discussion

The results of the numerical simulations of constant head tests were compared with those obtained by assuming a number of simple hypotheses. An analytical model based on two infiltration coefficients, equation (21), provided the best correlation with the numerical results.

The formula proposed by Stephens et al (1987), equation (18), was also compared with the numerical model results. For borehole radii greater than about 0.2 metres, this model provided totally unreasonable predictions. For small radii holes, reasonable flow rates were calculated but the comparison with numerical results was certainly not as accurate as that obtained by the two infiltration coefficient model developed here.

The two-coefficient model was extended to describe water levels in a soakaway during a falling head test and also in response to a time-varying inflow. A procedure was also developed to allow calculation of the maximum water level in a soakaway occurring in response to a synthetic unit hydrograph inflow. This simplified analytical model agreed well with numerical results and provides a method of analyzing and evaluating soakaway hydraulics.

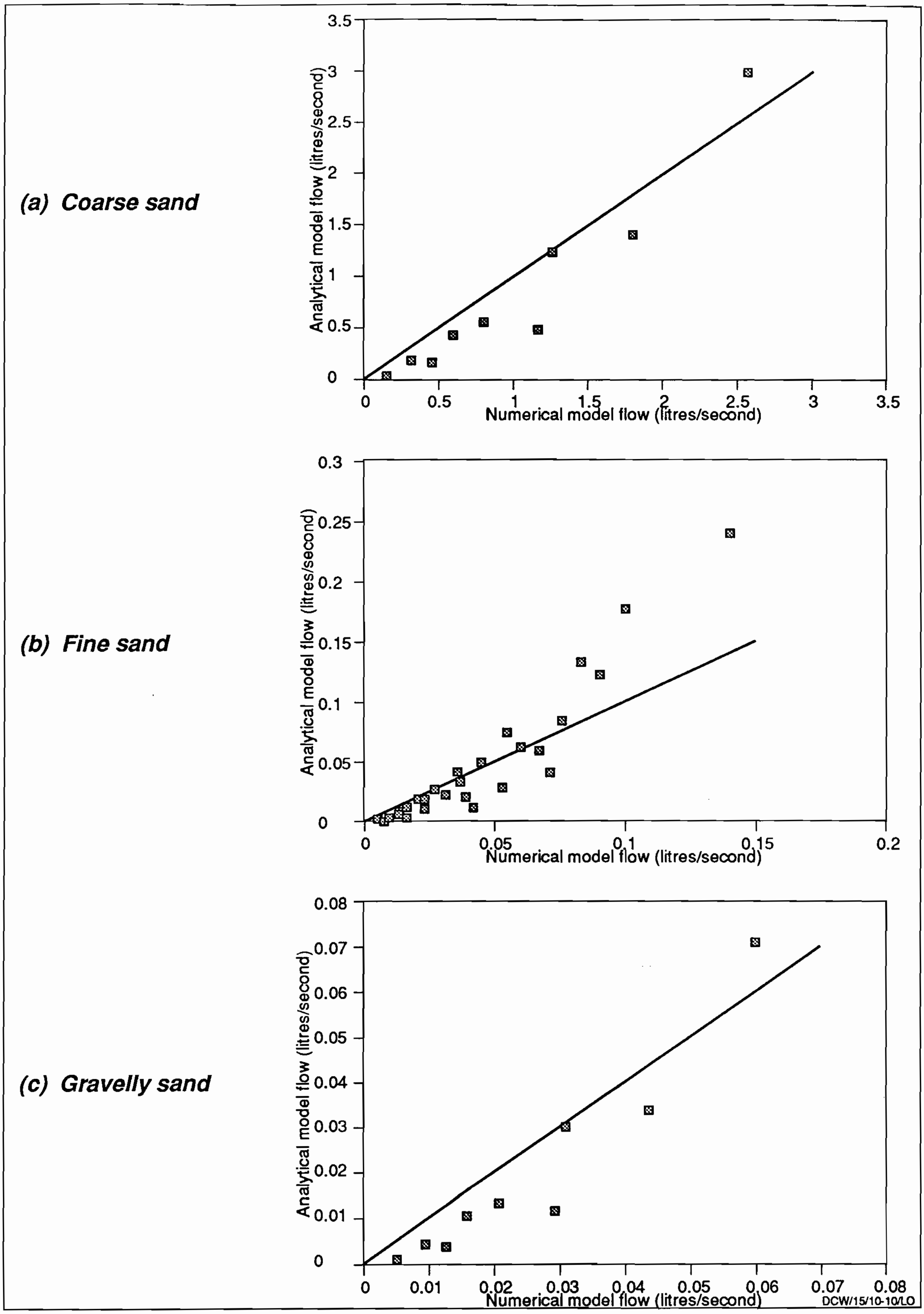


Fig 15 Comparison between BRE151 type analytical model and numerical model results for constant head tests

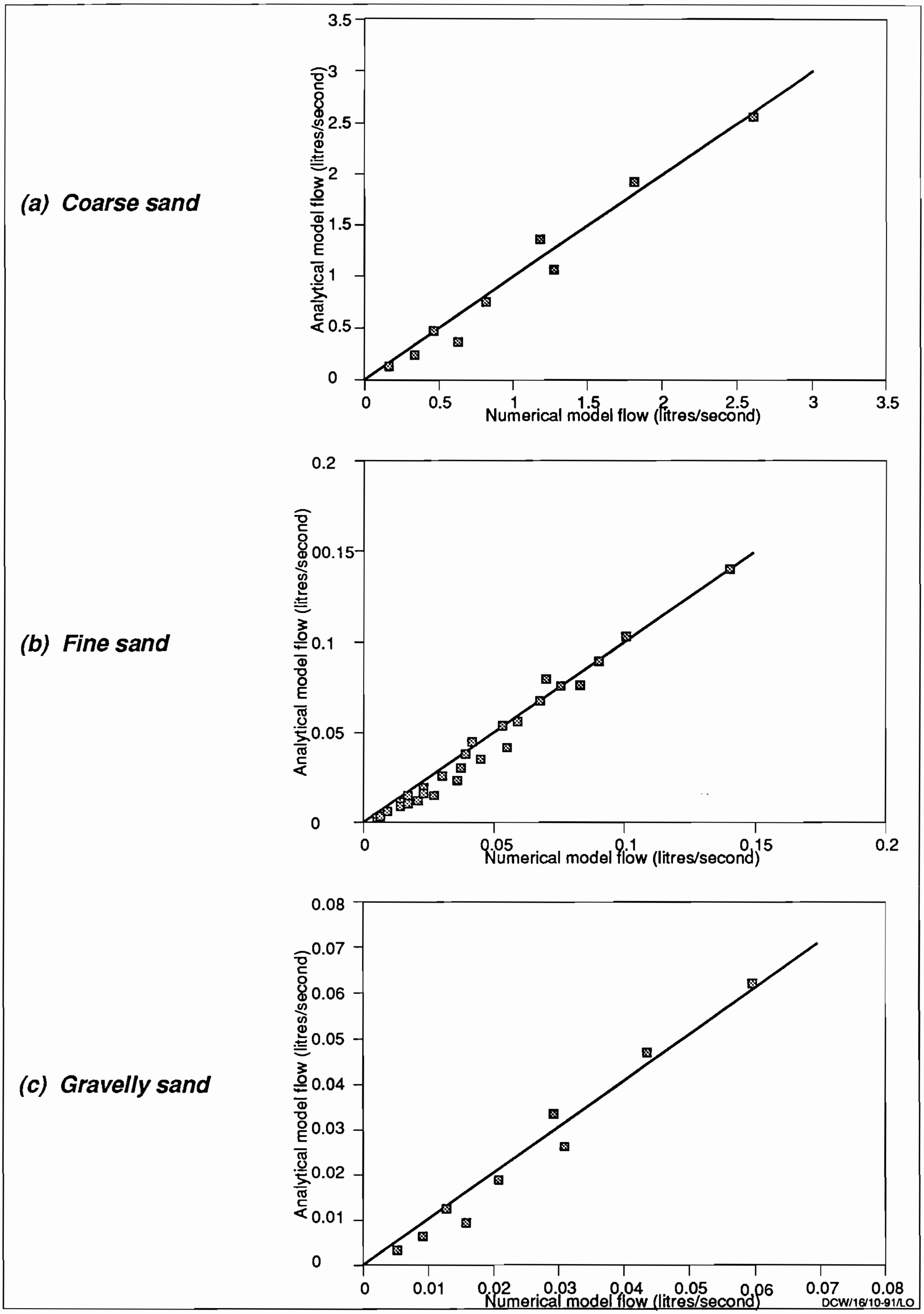


Fig 16 Comparison between one coefficient analytical model and numerical model results for constant head tests

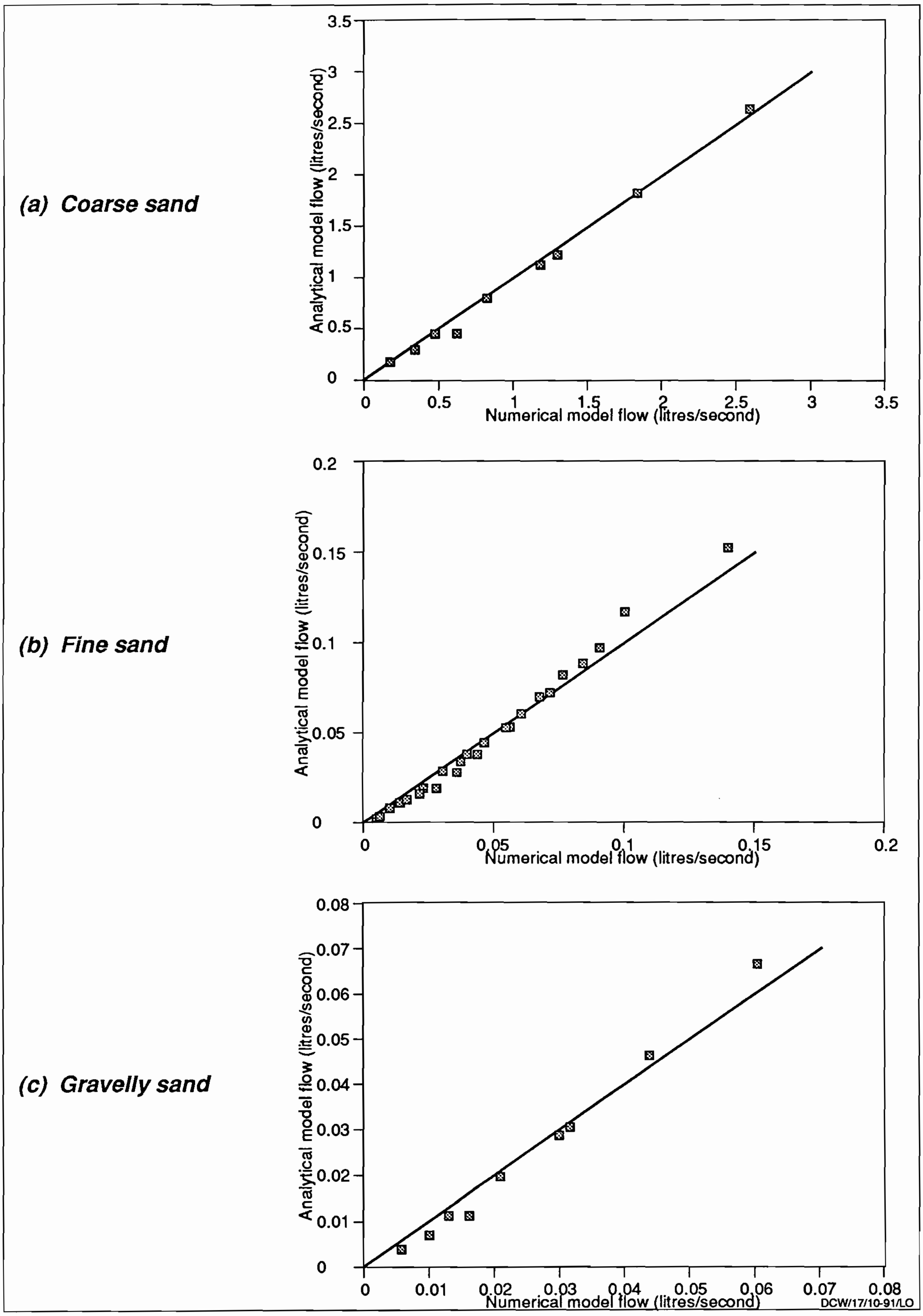


Fig 17 Comparison between two coefficient analytical model and numerical model results for constant head tests

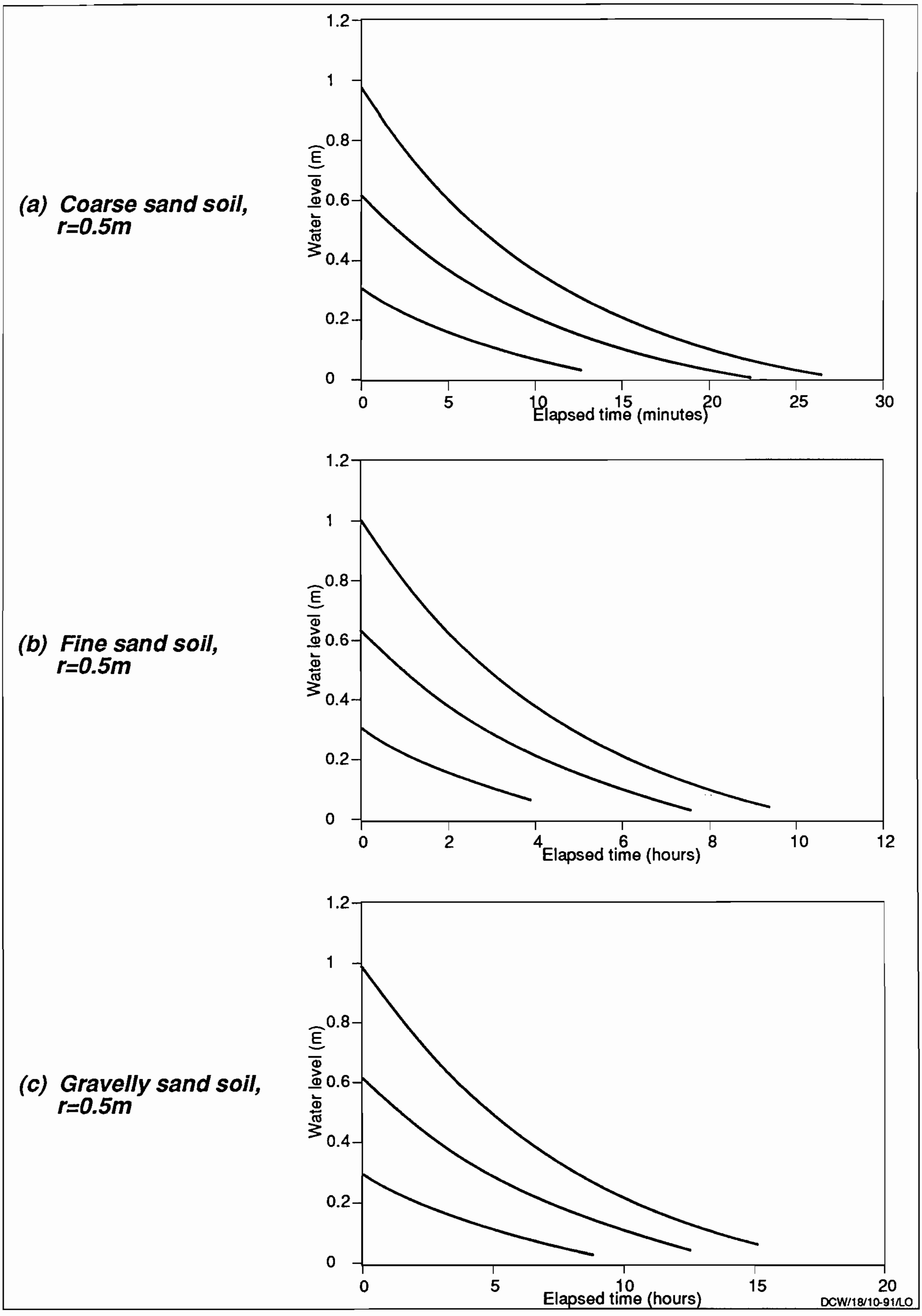


Fig 18 Falling head tests in a 1m diameter soakage test pit – analytical model results

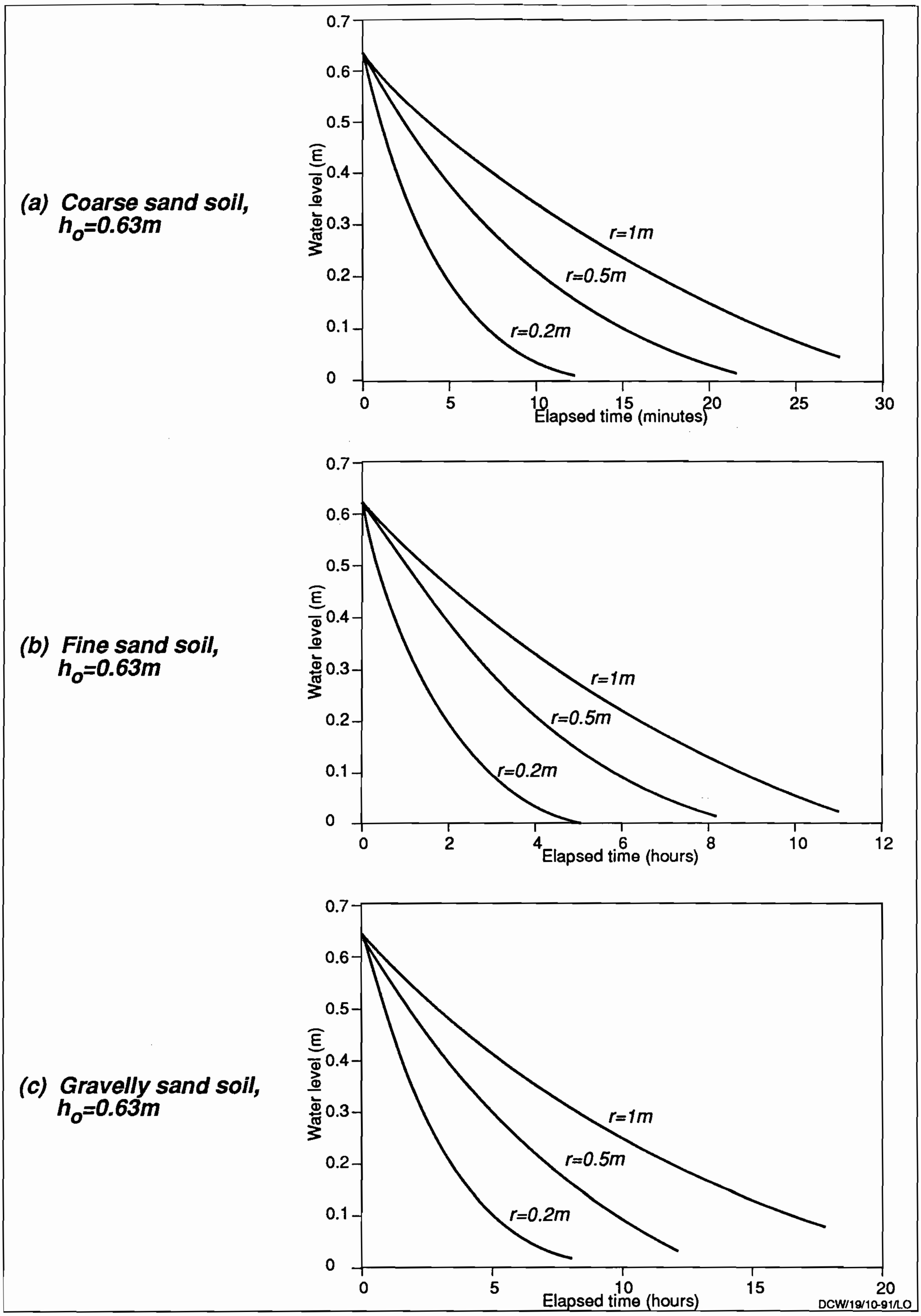


Fig 19 Falling head tests in a 0.63m deep soakage test pit – analytical model results

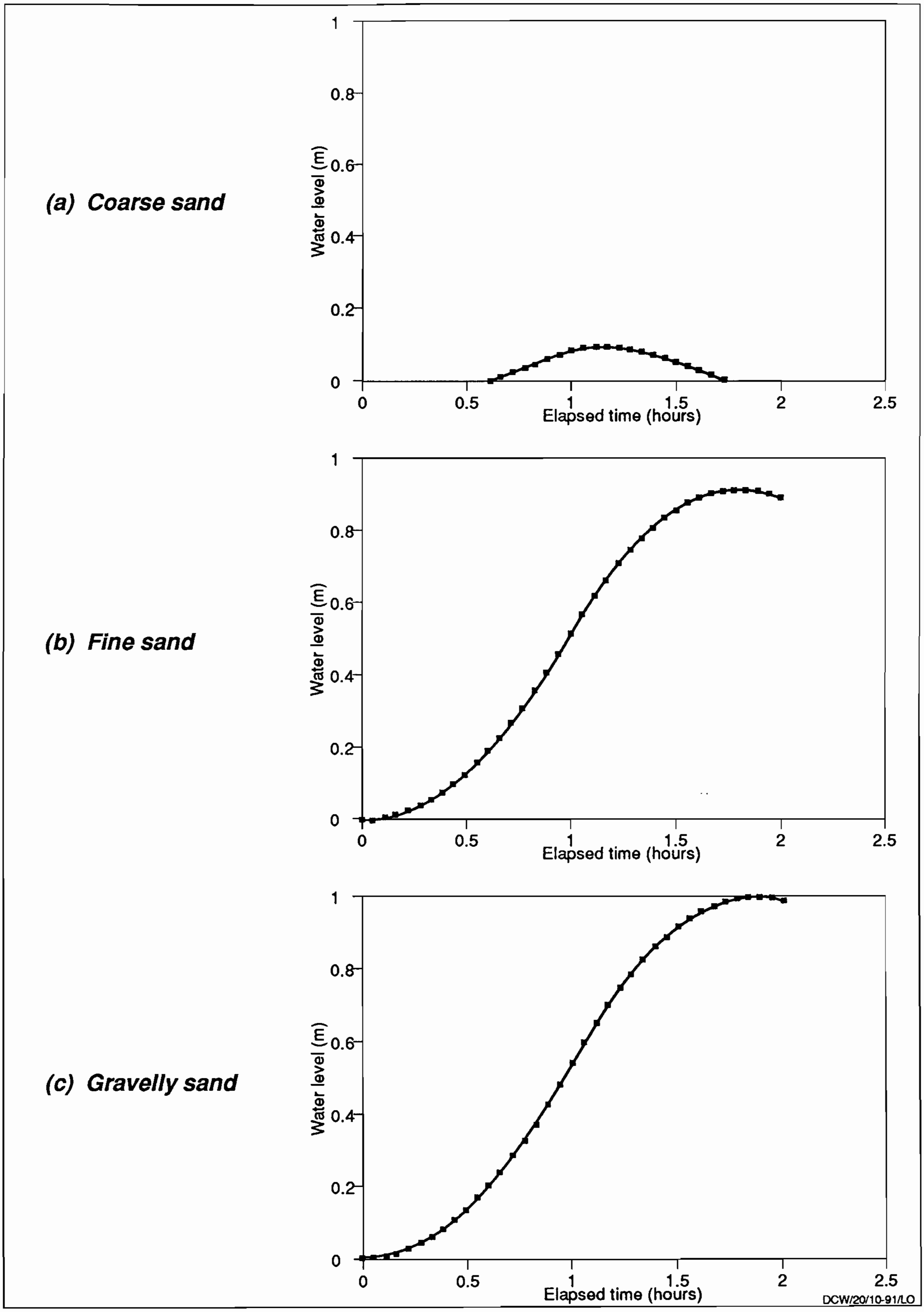


Fig 20 Water level response in a soakaway – analytical model results

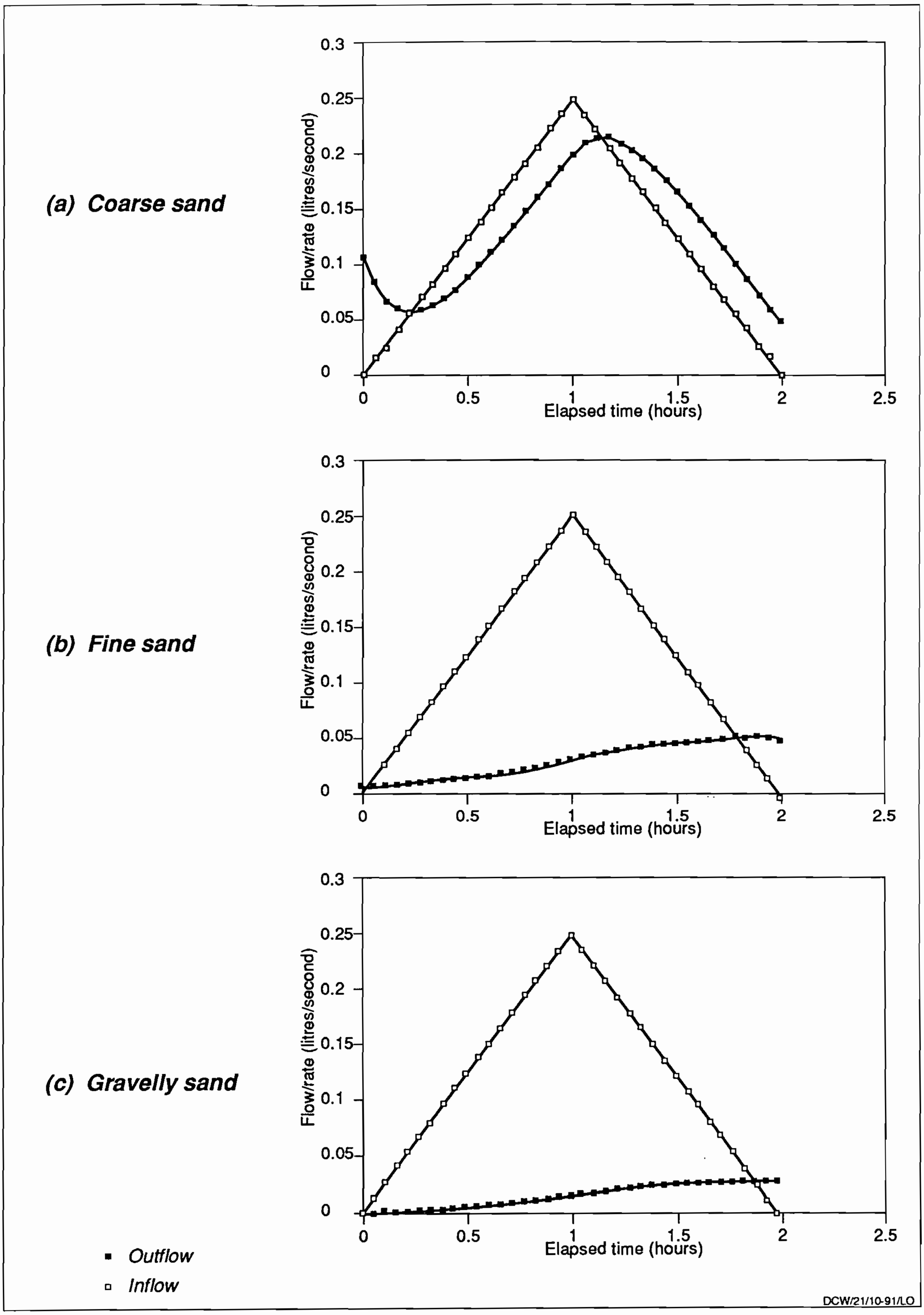


Fig 21 Outflow response in a soakaway – analytical model results

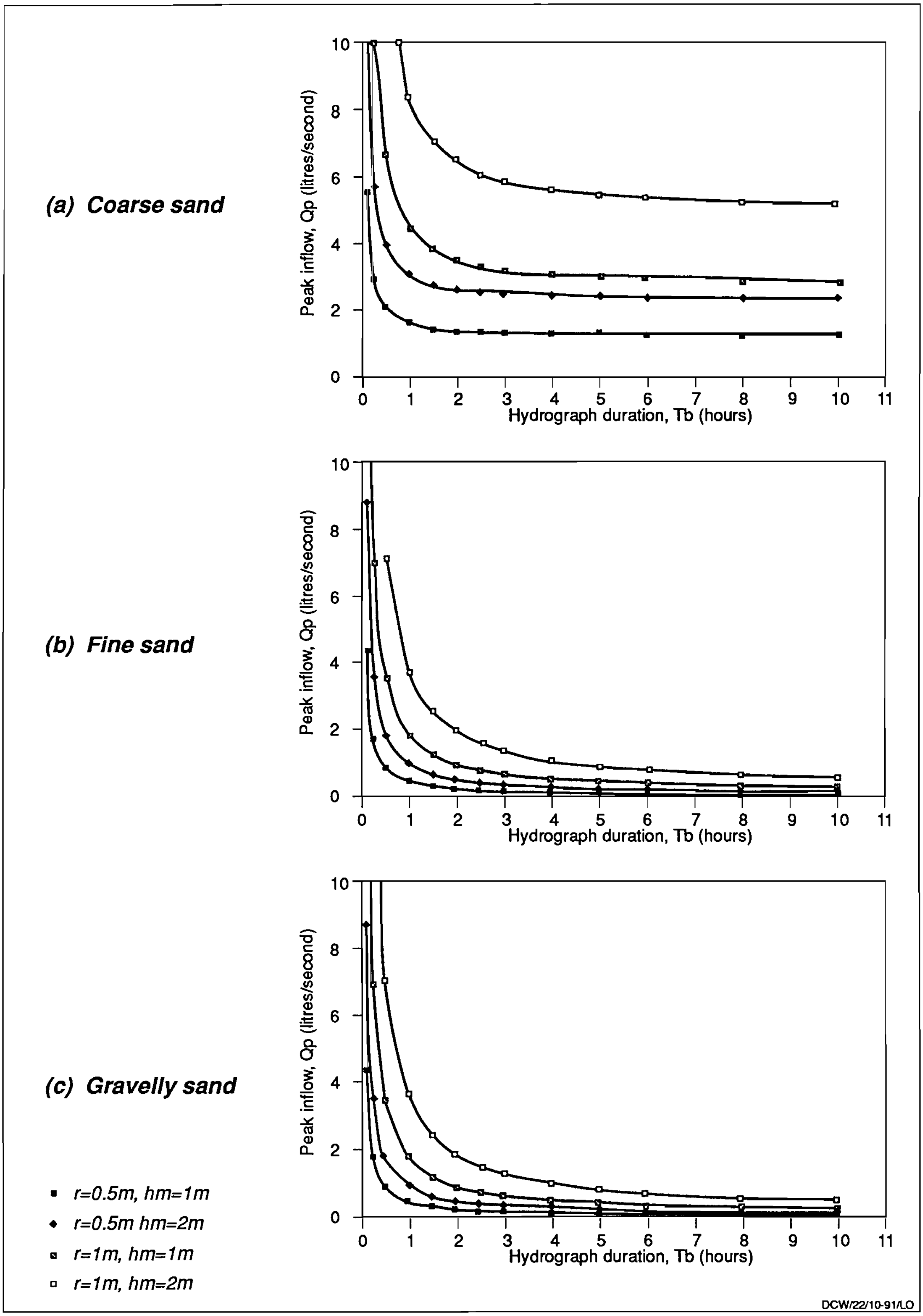


Fig 22 Soakaway performance, fixed soakaway dimensions – analytical model results

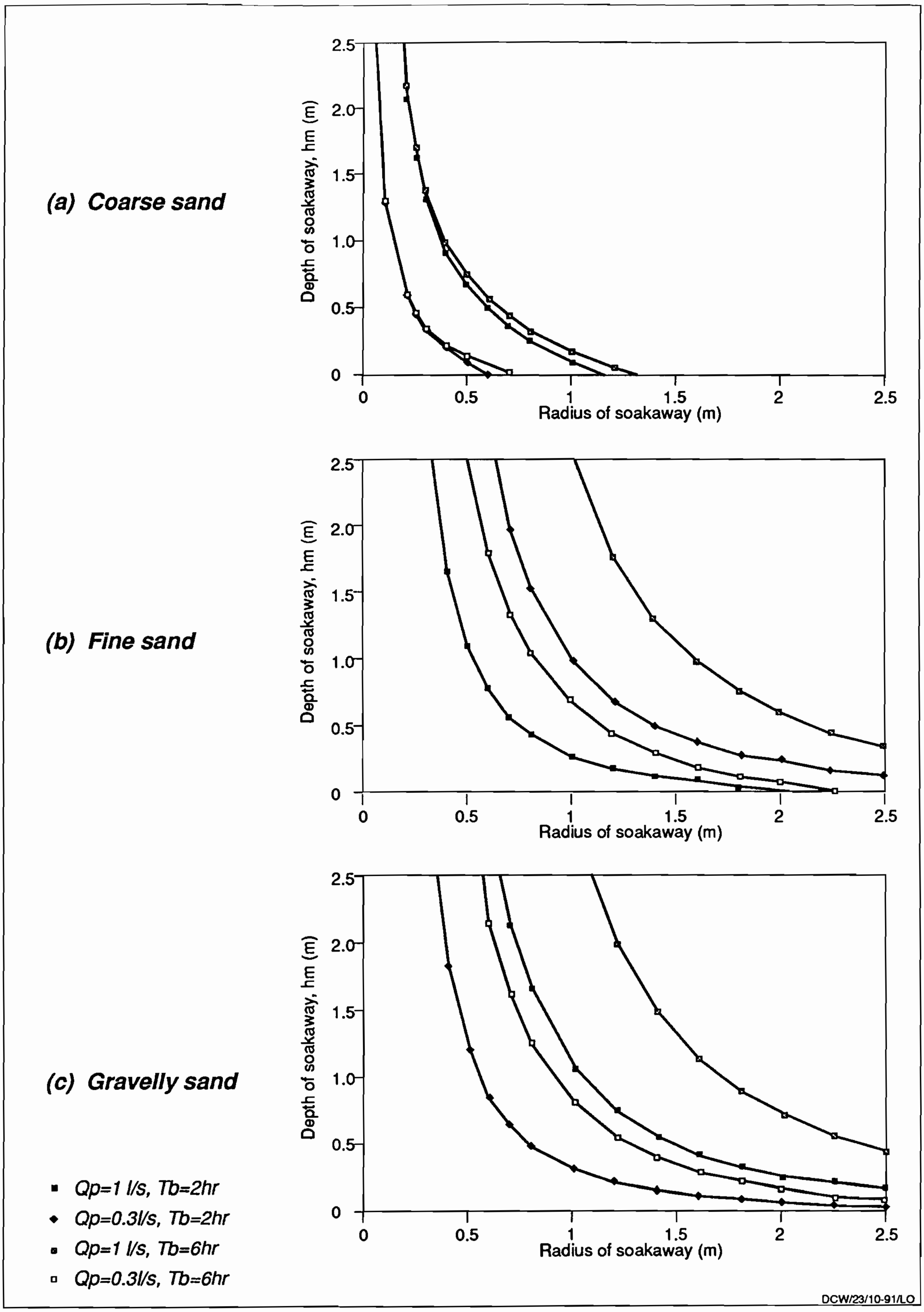


Fig 23 Soakaway performance, fixed inflow hydrographs – analytical model results

6. Field experiment

6.1 Introduction

A field experiment consisting of five soakage tests in different sized test pits was conducted. In order to minimise problems caused by vertical inhomogeneity of the soil, the depth of the test pit was kept constant and the diameter was enlarged between successive tests.

The soil in which the experiment was conducted is best described as a coarse-sandy silty clay loam with gravel. The hydraulic conductivity of the soil was measured independently using a Guelph permeameter (Reynolds and Elrick, 1985). This test involves measuring the flow rate in a small auger hole with a steady water level maintained by a Mariotte bottle. Tests are carried out using two different water levels in the auger hole. Simultaneous equations are then used to solve the formulae proposed by Reynolds et al (1985) for infiltration borehole permeameters, to obtain the saturated hydraulic conductivity of the soil.

For each size of soakage test pit, constant head tests were conducted first. Once steady conditions appeared to have been achieved, the water level was allowed to fall providing a falling head test with measurements of water level and time.

6.2 Method

The topsoil in the area chosen for the experiment was removed to expose the subsoil. The first soakage test was conducted in an auger hole of 150 mm diameter and 300 mm depth, in accordance with the BRE 151 test.

The test hole was filled with water and a steady level was maintained for 2 to 3 hours. The average rate at which water needed to be added to maintain a constant water level was recorded. The water in the hole was then allowed to drain away while measurements of the fall in water level were taken. The soil was then allowed to dry out overnight. The next day, the diameter of the test pit was enlarged and the soakage test was repeated. Five soakage tests were conducted in this way in 0.3 m deep test pits between 0.3 and 1.2 m in diameter. Each test was conducted within one day.

6.3 Results

The results of the constant head test field experiments are listed in Table 6 and are also shown in Figure 24. The flow rate required to maintain a steady water level in the 0.15 m diameter test hole was found to be greater than that for the 0.3 m diameter test hole. This observation is at odds with theoretical behaviour and is probably due to experimental error in the measurement of the flow rate. The falling head measurements demonstrate that a lower volume flow rate was, in fact, achieved for the smaller test pit.

In order to analyze the field experiment, the two-coefficient model developed in Section 5 was applied to the results listed in Table 6. In determining the best-fit value of the coefficients, the experimental error mentioned above was included in the data set to ensure that the analyses remained realistic in field-experimental terms. The calculated coefficients for the soil in which the experiment was conducted are included in Table 5 which also lists the saturated hydraulic conductivity as determined by the Guelph permeameter. Again, the infiltration coefficients were found to be similar in magnitude, but slightly larger than, the saturated hydraulic conductivity of the soil.

Table 6. Results of the field experiment

radius of soakage pit (m)	constant head flow rate (l/s)
0.075	0.0048
0.15	0.0045
0.25	0.011
0.4	0.015
0.6	0.033

The falling head test results are presented in Figures 25 (a-e). Also included on these figures are the predicted water level falls as determined by the two-coefficient analytical model, equation (26), using the coefficients derived from the constant head tests listed in Table 5.

6.4 Discussion

The comparison between the analytical model and the field data is very encouraging in terms of model verification. The analytical model was calibrated against the constant head measurements and the result was compared with falling head measurements, made independently. The predicted water levels exhibited similar trends and rates of fall to those measured in the field.

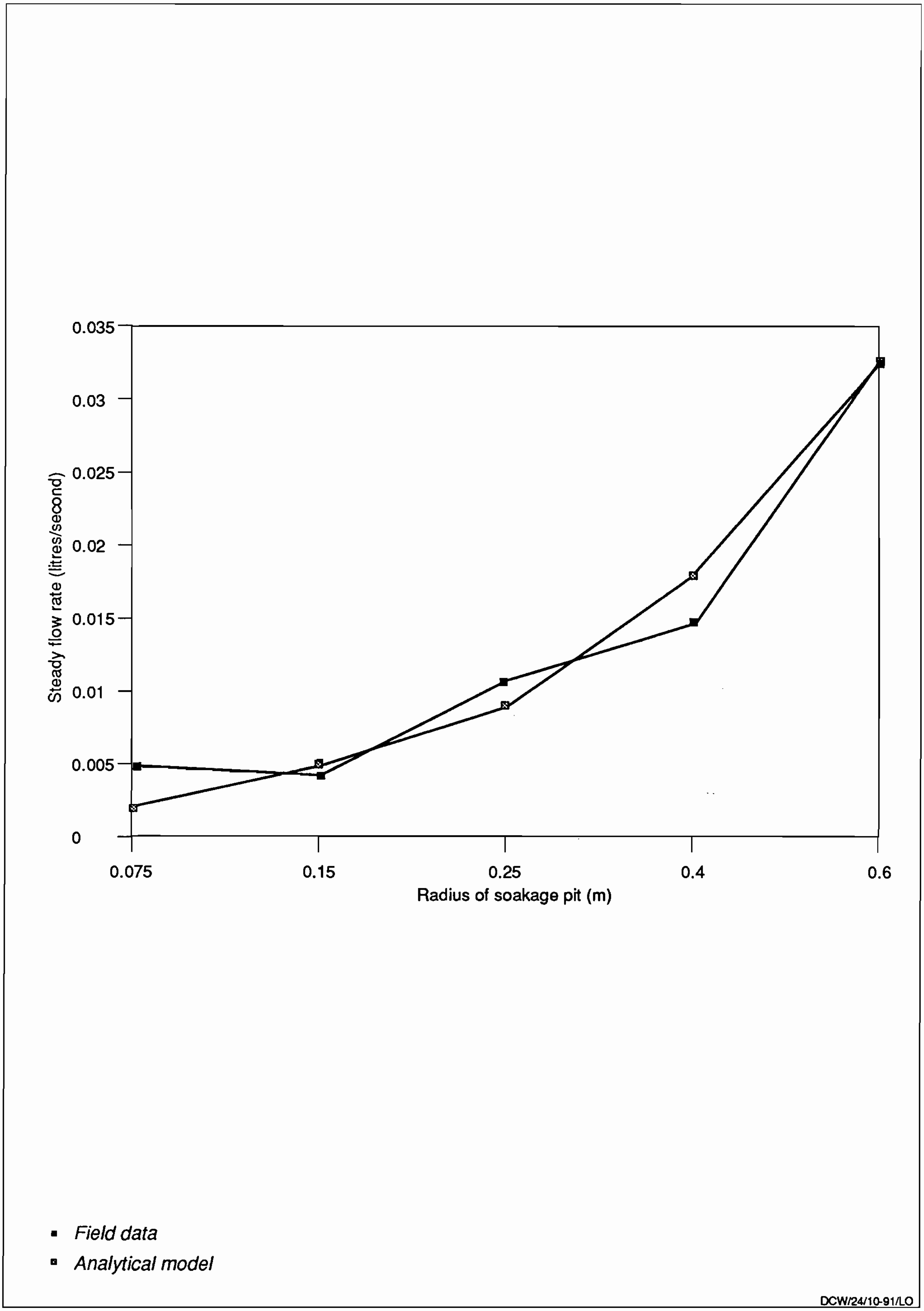


Fig 24 Results of constant head tests – field experiment

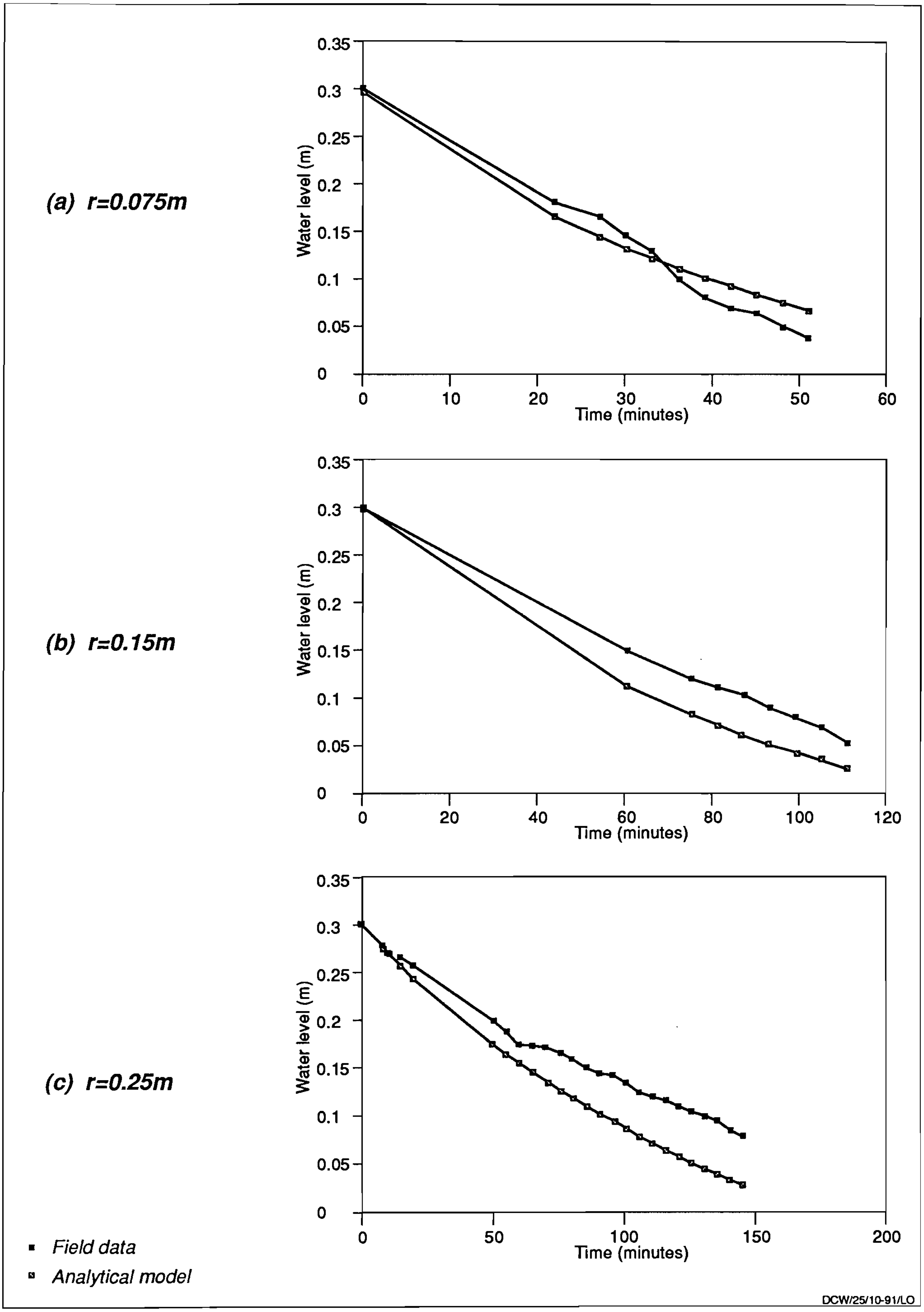
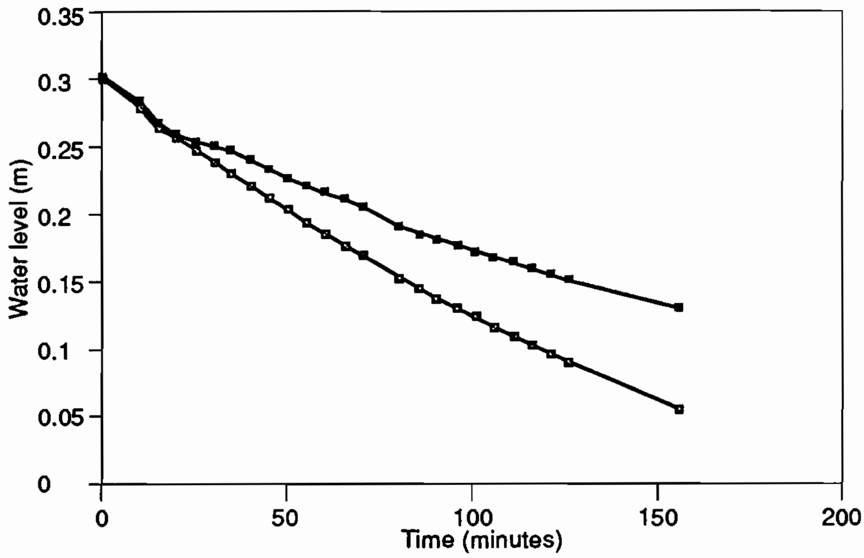
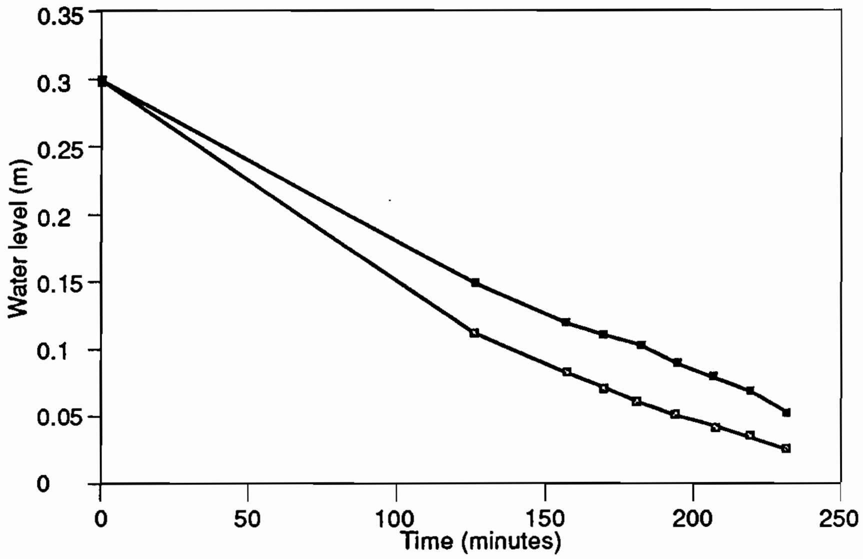


Fig 25 Results of falling head tests – field experiment

(d) r=0.4m



(e) r=0.6m



▪ *Field data*

▫ *Analytical model*

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Fig 25 (continued) Results of falling head tests – field experiment

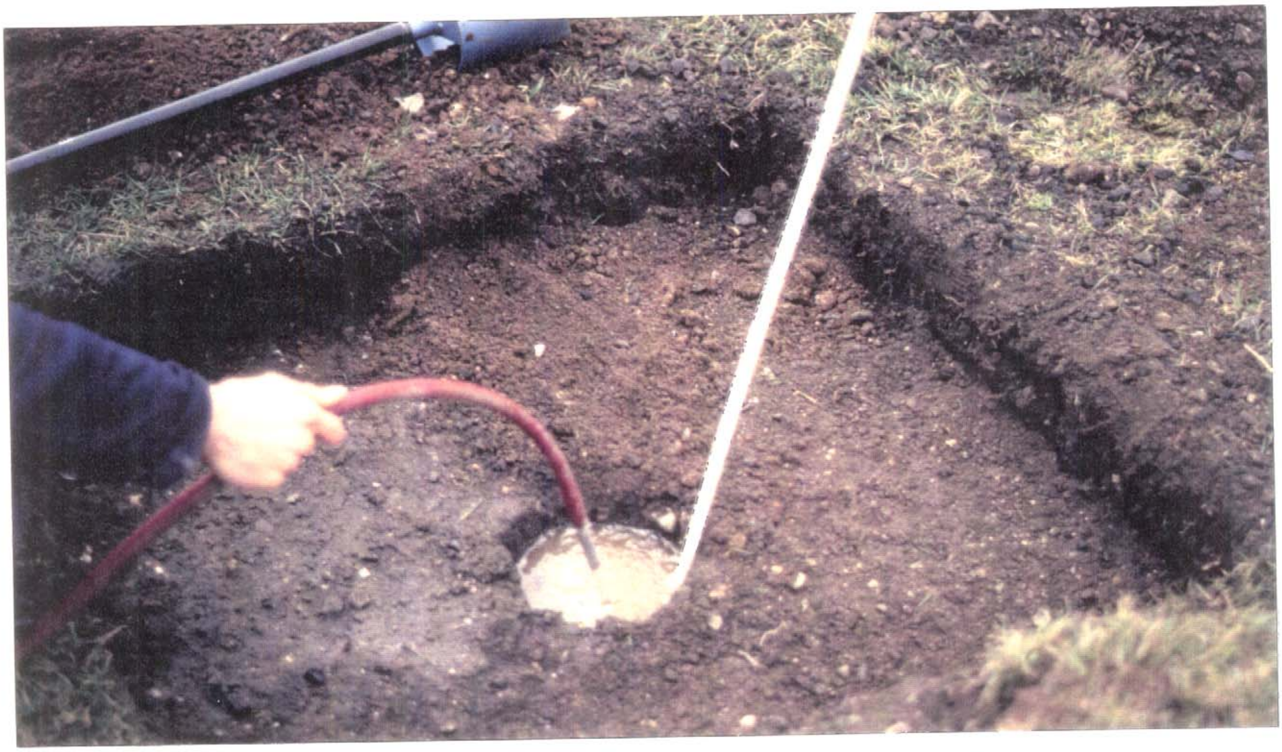

Plate 1 Soakage test pit 0.15m diameter, field experiment

Plate 2 Soakage test pit 0.5m diameter, field experiment

Plate 3 Soakage test pit 0.8m diameter, field experiment

Plate 4 Soakage test pit 1.2m diameter, field experiment

7. Evaluation of design procedures

7.1 Introduction

In the light of the results of the mathematical modelling work, it is appropriate to re-evaluate the design methods. This is to allow an assessment to be made of the implications of this study on soakaway design procedures.

To allow a comparison between the guidelines available, an example problem was taken and a soakaway was designed by each of the different available methods. For the example problem, it was assumed that an impervious area of 20 metres by 20 metres was to be drained to a soakaway. It was further assumed that the subsoil consisted of the fine sand soil with the hydraulic properties listed in Table 2. To simplify comparisons between the different designs, it was assumed that the resulting soakaway would be cylindrical with the diameter equal to the depth and would not contain any fill material.

The numerical and analytical models were also used to predict the theoretical performance that may be expected from a soakaway in the example problem.

7.2 Type I methods

7.2.1 BS 8301

As with all Type I methods, the size of soakaway recommended is irrespective of the soil properties.

The recommendation to construct a soakaway of sufficient volume to store water equivalent to 12 mm over the 400 m^2 drained area will result in a soakaway capacity of 4.8 m^3 assuming that no fill material is used. Given the criterion for the example, that the depth is equal to the diameter, the designed soakaway would need to be 1.85 metres in diameter and depth.

7.2.2 Danish guidelines

These guidelines recommend a soakaway volume of 1 m^3 per 30 m^2 drained area. The required volume for the example would therefore be 13.3 m^3 in which a porosity of fill material of 25% is already taken into account. For the example problem, the soakaway depth and diameter dimensions would be 2.6 metres.

Because it is implicit in the design procedure that the soakaway contains a fill material with a porosity of 25%, this design may be considered to contain a factor of safety of 4.0, for the example problem.

7.2.3 ASCE recommendations

These recommendations state that infiltration drainage should not be considered in soils for which the hydraulic conductivity is less than

$2x10^{-5}$ m/s. The fine sand soil being considered in the example has a saturated hydraulic conductivity of $4.4x10^{-6}$ m/s. The recommendation would therefore be not to construct a soakaway.

7.3 Type II methods

7.3.1 PSA 125

This guideline suggests a soakage test in a rectangular pit. To allow a comparison here, it is assumed that the test is conducted in a cylindrical test pit.

Figure 26 shows the fall in water level for a falling head soakage test conducted in a 0.15 metre diameter and 0.3 metre deep test pit in the fine sand soil, according to the numerical model. This is equivalent to the BRE 151 soakage test. For the purposes of the example, it is assumed that the pit was allowed to drain from full to half full. From Figure 24, the time taken to do this was 650 seconds and the volume outflow was 0.00265 m^3.

The design infiltration rate is the outflow per unit time per mean unit area and is therefore $3.3x10^{-5}$ m/s. The infiltration rate used for design purposes is $1.1x10^{-5}$ m/s including a factor of safety of 3.

The soakaway is designed to provide a storage volume equivalent to 27.5 mm of runoff from the drained area, or 11 m^3. The surface area must provide an infiltration capability of at least 1.25 mm/hour from the drained area, or 0.5 m^3/hour. According to the infiltration rate calculated from the soakage test, this would require a surface area of 12.6 m^2.

Maintaining the depth equals diameter constraint on the soakaway design, the dimensions that satisfy both the storage and infiltration criteria can be found from

storage = $2\pi r^3$ = 11 m^3

r = 1.2 metres

infiltration surface area = $5\pi r^2$ = 12.6 m^2

r = 0.9 metres

A soakaway of 2.4 metres diameter and depth would therefore be required to satisfy the storage criterion and would also provide sufficient infiltration capability.

7.3.2 Pratt (1990)

For this method, it was assumed that a soakage test pit of 1.0 metre diameter and 1.0 metre depth was used. The numerical simulation of a falling head test in this size pit is shown in Figure 12(b). The design method uses an infiltration rate based on the time taken for the pit to drain from 75% to 25% full. From Figure 12(b), this time was 13,500 seconds.

The volume dissipated in this time was 0.39 m^3 and the mean wetted surface area was 2.36 m^2. The infiltration rate, q_1, was therefore 1.23×10^{-5} m/s.

According to the guideline, the volume of rainfall precipitated during the design storm event can be equated with the sum of the volume dissipated during the event, through half of the sidewall area, and the volume stored after the event.

inflow = outflow + storage

or

$$A_d R = q_1 D \pi r H + \pi r^2 H \qquad (30)$$

where	A_d	= impervious area drained	[L^2]
	R	= rainfall per unit area	[L]
	D	= rainfall duration	[T]

Equation (30) is solved for the radius of the soakaway, r, with various values of block rainfall intensity and duration, based on a 1 in 10 year return period event.

Values for R and D for the location of Wallingford, Oxfordshire, were found according to the Flood Studies Report (NERC, 1977). These values are shown on Figure 27(a). Figure 27(b) shows the required radius of soakaway, assuming that the diameter equals the depth, as calculated from equation (30).

The critical storm duration is about 6 hours with an intensity of 6.4 mm/hour and the corresponding minimum size of soakaway is 1.27 metres in diameter and depth.

A further criterion given in the guideline is that the soakaway should half-empty within 24 hours. This can be tested by equating half the storage volume divided by half the sidewall area with the infiltration rate. For the case whereby the diameter equals the depth, this leads to the condition:
$r^2 < 86400\ q_1$.

The maximum radius of soakaway that can satisfy this half-emptying criterion is 1.03 metres. It will therefore be necessary to drain the area to more than one soakaway, each of which must be less than 2.06 metres diameter.

The procedure for calculating the size of soakaway for 1 in 10 year storm events was repeated using drained areas of 200 m^2 and 100 m^2 and the results are also shown in Figure 27(b).

If the area is drained to two soakaways, dimensions of about 2.0 metres in depth and diameter could be used but this would be close to the limit of the half-emptying criterion. The area could be drained to four soakaways each of 1.6 metres diameter and depth.

7.4 Type III methods

7.4.1 BRE 151

The BRE 151 test is based on the time taken for a 0.15 m diameter and 0.3 m deep soakage test pit to empty completely. From Figure 26, the time taken to do this was 1500 seconds or 25 minutes.

This time is too short, the infiltration rate is too high, to be able to estimate the soakaway size required by reading off the graph. This demonstrates one of the shortcomings of the guideline.

It could be assumed that the minimum size mentioned, 1.0 m diameter and depth, should be used. Knowing the function plotted in the BRE graph, however, it is possible to calculate the design size. The function used is

$$A_d = \frac{\pi d^3}{2 I T} \qquad (31)$$

where d = depth = diameter [L]
= 2r

I = rainfall intensity [LT^{-1}]
= 15 mm/hour

T = time taken for the test pit to empty [T]

Solving equation (31) for the example problem, the required soakaway diameter and depth is therefore 1.2 metres.

7.4.2 King

This method is based on the same soakage test as BRE 151 and so the time to empty, T, used in the design calculation is 1500 seconds.

The equation given for the analysis of the test is

$$T = \frac{1}{C_p}\left(\frac{6r}{4g}\right)^{0.5} \tan^{-1}\left[\left(\frac{4h_o}{3r}\right)^{0.5}\right] \qquad (32)$$

where C_p = a so-called coefficient of permeability [-]

This leads to a value for C_p of $4.7x10^{-3}$.

The equation given for the soakaway design is

$$L = \frac{A_d R_d}{C_p Q_n} \tag{33}$$

where L = length of soakaway [L]

R_d = design rainfall intensity and is in units of m/hour

and

$$Q_n = \left(\frac{4}{3}H + W\right)(2gH)^{0.5} \tag{34}$$

where H = height of soakaway [L]

W = width of soakaway [L]

The guideline is intended primarily for trench type soakaways. Substituting H = W = L into equation (34) produces a cubic soakaway design for ease of comparison with cylindrical soakaway sizes.

For soils with a C_p value of less than 10^{-6}, it is recommended that a rainfall design intensity of 0.006 m/hour is used and for soils with a C_p value greater than $5x10^{-5}$, a design rainfall intensity of 0.075 m/hour should be used. Substituting this latter value into equation (33), the required soakaway size is a cube with sides of 4.75 metres.

7.4.3 Swedish guidelines

With this method, no advice on soakage tests is given. It is assumed that the designer knows the hydraulic conductivity of the soil, in this case $4.4x10^{-6}$ m/s.

The soakaway is designed using the rain envelope method. For a range of 1 in 10 year probability rainfall events, the rainfall volume is plotted against duration. This is shown in Figure 28(a) using the rainfall data for Wallingford, presented in Figure 27(a) and for a 400 m^2 drained area. Constant outflow rates are also plotted on Figure 28(a). The storage required in the soakaway is taken to be the maximum difference between the inflow and outflow, S_d on Figure 28. Figure 28(b) shows the storage volume plotted against different values of outflow rate as determined from Figure 28(a).

To calculate the required soakaway size, a value of the dimensions (radius in this case) is chosen. The outflow rate is calculated from a version of equation (20) but with the infiltration constant equal to the hydraulic conductivity divided by a factor of safety of 2.5 and flow only through the sidewall area, equation (35) below.

$$Q = \frac{k}{2.5} 2\pi r H \tag{35}$$

When the outflow rate has been estimated from equation (35), the corresponding storage volume required is found from Figure 28(b). This value is compared with the actual volume available and, if required, the soakaway size is increased and the calculation repeated.

Figure 28(c) presents the results of a series of calculations to determine the radius of soakaway required to fulfil the storage requirements, assuming the diameter is equal to the depth. The required soakaway should be 2.5 metres in diameter and depth.

7.5 Mathematical models

7.5.1 Inflow hydrographs

Figure 14 shows the response of a 1.0 metre diameter and 1.0 metre deep soakaway to a time-varying inflow, according to the numerical model. Likewise, Figures 22 and 23 indicate the performance of soakaways operating under time-varying inflows according to the analytical model. Both sets of results refer to inflows based on synthetic unit hydrographs.

The numerical and analytical models of soakaways, developed within this study, were applied to the example problem. In order to make comparisons with the existing guidelines, the mathematical models were used to simulate conditions with inflow hydrographs based on block rainfall events, in addition to synthetic unit hydrographs.

The following types of inflow hydrograph were modelled:

(a) Block rainfall hydrograph

In accordance with block rainfall hyetographs, the block rainfall hydrograph assumes a constant inflow, Q_i, equal to the product of the mean rainfall intensity, I, and the area drained, A_d, maintained for the duration of the storm event, D.

(b) Approximated synthetic unit hydrograph

This is the symmetrical hydrograph as used in the earlier modelling exercises, Figures 8, 9, 14, 20, 21, 22 and 23. It is assumed that the rainfall immediately reaches the soakaway so the hydrograph is symmetrical and the hydrograph duration, T_b, equals the storm duration, D. Assuming that the volume of rainwater is conserved, the peak inflow, Q_p, equals twice the equivalent constant inflow rate, Q_i.

(c) Estimated synthetic unit hydrograph

In this case the runoff delay is taken into account so that the hydrograph is attenuated before reaching the soakaway and is therefore asymmetrical.

The Flood Studies Report (NERC,1977) provides a method of converting rainfall events to synthetic unit hydrographs for idealised catchment areas. The 10 mm intensity and 1 hour duration synthetic unit hydrograph is given by the equation (Chadwick and Morfett, 1986)

$$T_p = 46.6\ MSL^{0.14}s^{-0.38}(1+URB)^{-1.99}RSMD^{-0.4} \quad (36a)$$

$$Q_p = 2.52T_p \quad (36b)$$

(36c)

where MSL = mainstream length in km.

s = surface slope [-]

URB = fraction of urbanised catchment[-]

RSMD = 1 in 5 year, 24 hour duration rainfall in mm.

For the example problem, the mainstream length is the diagonal of the 20 m by 20 m area and is 28.3 m. For the sake of the example, it was assumed that the surface slope is 0.1 %. The drained area is fully impervious so the fraction of urbanised catchment area is 1. The 1 in 5 year, 24 hour duration, rainfall for Wallingford is 44 mm. Substituting these values into equation (36)

$$T_p = 5640 \text{ s}$$

$$T_b = 14200 \text{ s}$$

$$Q_p = 5.6\times10^{-4} \text{ m}^3/\text{s}$$

The values of T_p, T_b and Q_p for the 10 mm/hour intensity and 1 hour duration event, listed above, can be scaled linearly to correspond with other intensity and duration rainfall events.

Figure 29(a) shows a block rainfall event of 15 mm/hour intensity and 2 hours duration. For the example, the area drained is 400 m^2 and so the volume inflow rate is 6 m^3/hour for 2 hours, 12 m^3 in total. Assuming that this volume and duration are fixed, the peak inflow for the synthetic unit hydrograph, Q_p, must be 12 m^3/hour as shown on Figure 29(b). Figure 29(c) shows the corresponding synthetic unit hydrograph for the 15 mm/hour and 2 hour duration block rainfall event, shown in Figure 29(a) in which the runoff characteristics have been approximately taken into account using equation (36).

7.5.2 Numerical model

The time dependent boundary condition in the model code was modified to represent a block rainfall hydrograph inflow to the soakaway, as described above, in addition to the synthetic unit hydrograph inflows modelled previously. The model was set up to simulated a soakaway of 2.0 metres diameter and 2.0 metres in depth.

Figure 30 shows the maximum water level that occurred in the soakaway in response to a range of inflow hydrographs based on block rainfall events. The rainfall event probabilities included on Figures 30 and 31 relate to the location of Wallingford and are derived from the Flood Studies Report (NERC, 1977).

Rainfall events that produce a maximum water level in the soakaway, h_m, greater than 2 metres will result in an overflow. The soakaway will accommodate all those events for which h_m<2.0 metres, shown on Figure 30.

7.5.3 Analytical model

The analytical model can easily be applied to the example problem.

Equations (26), (27) and (28) were developed for synthetic unit hydrograph inflows, from equation (25). A version of these equations was developed for constant inflow rates corresponding with block rainfall hydrograph inflows, to produce the results shown in Figure 31(a).

Under these conditions,

$$Q_i(t) = Q_i = I\,A_d$$

$$Q_p = 2.2\,\frac{Ad}{T_p}$$

$$c = 0$$

Substituting values of a, b and c into equation (25)

The maximum water level must occur at the end of the storm event.

Therefore at

$$t = D,\ h = h_m$$

The maximum water level occurring in the soakaway, due to a block rainfall hydrograph (constant) inflow can therefore be found by substituting the storm duration, D, for the time, t, in equation (37).

Figure 31 presents the corresponding results to Figure 30 but obtained by using the two-coefficient analytical model. This was achieved by using equation (37) for the block rainfall hydrographs and equations (27), (28) and (29) for the synthetic unit hydrographs. In these cases the performance can be calculated directly for a given soakaway size. In Figure 31, the results of four soakaway designs; diameter and depth equal to 1.0, 1.5, 2.0 and 3.0 metres, are presented. Figure 31 shows the inflow conditions above which each of the soakaway designs would overflow.

7.6 Discussion

The different guidelines tested produced a wide range of design sizes for the example problem. A summary of the results is provided in

Table 7. The volume of the excavation required is also listed in Table 7 as this provides a better indication of the effort and cost involved.

Even ignoring the design according to King, which may probably be regarded as unreasonable, the range of the volumes requiring excavation varied by almost an order of magnitude. This occurred even though the guidelines were applied to the same theoretical soil, which excludes any experimental error from affecting the comparisons. Virtually any soakaway size within the range 1 to 3 metres diameter and depth could be designed for the example problem.

Table 7. Summary of soakaway sizes recommended for the example problem

guideline	diameter and depth (m)	volume excavated (m^3)	factor of safety
BS 8301	1.85	5.0	1
Danish	2.6	13.8	4 on storage
PSA 125	2.4	11.0	3 on infiltration
Pratt	2 x 2.0	12.6	various assumptions
BRE 151	1.2	1.4	1
King	4.75	107	1
Swedish	2.5	12.3	2.5 on infiltration

The size of soakaways designed according to BS 8301 or the present Danish guidelines do not depend on the soil properties.

The methods used by Pratt (1990) and PSA 125 both involve constant head soakage tests. Although a falling head is used in the test, the measurements are used to find the mean constant head flow rate which is then applied by using equation (20) for constant head tests.

The procedures used by BRE 151 and by King (1974) both involve falling head tests though the analyses involved in these methods are somewhat tenuous. The smallest and largest soakaway sizes were recommended by these two methods, respectively, for the example problem. In practice, The times measured in the field are likely to be longer than those predicted mathematically, tending toward the design of larger soakaways. This is because it is difficult to measure the moment at which the test pit becomes completely empty.

The Pratt and the Swedish methods are, in fact, similar in many respects. Although a number of different assumptions are made, they produced very similar results for the example problem.

The guidelines were only tested and compared for the one example problem and one soil type. To fully evaluate the methods, a range of problems would have to be considered.

The numerical model results, Figure 30(c), show the maximum water levels expected to occur in a 2 metre diameter and 2 metre deep soakaway in the example problem, taking account of the infiltration process according to equation (12), the hydraulic properties for the fine sand soil listed in Table 2 and a realistic inflow hydrograph. According to the numerical predictions, the soakaway would be likely to overflow due to 1 in 10 year storm events between about 15 and 600 minutes duration, close to overflowing during 1 in 5 year events around 100 to 200 minutes duration but will not overflow during any more frequent events. It will be able to cope with short duration rainfall events through its storage capacity and with long duration events through its infiltration capability.

The analytical model results are closely comparable with the results of the numerical model simulations. Because the analytical model is more flexible than the numerical model, it is possible to obtain more detailled predictions.

According to the results presented in Figure 31(c), a soakaway of 1 metre diameter and 1 metre depth would often overflow and would only cope with 1 year return period events of less than about 2 minutes and more than about 2000 minutes in duration, without overflowing. A soakaway of 1.5 metres in diameter and depth would overflow during 1 year return period events between 10 minutes and 300 minutes duration and 1 in 10 year return period events between 3 and 1500 minutes duration. The 2 metre soakaway would accomodate all 1 year return period events but would be liable to overflow during 1 in 5 year events between 30 and 180 minutes duration and 1 in 10 year events between 15 and 500 minutes duration. The 3 metre soakaway would not be expected to overflow during any events other than those with return periods in the order of hundreds of years.

These designs, however, would not provide any factor of safety. A factor of safety is required to ensure against problems due to experimental error, soil variability, changes in soil properties and silting up of the soakaway. The most appropriate form and consequences of different factors of safety, and also the effects of conservative assumptions, require further study and analysis.

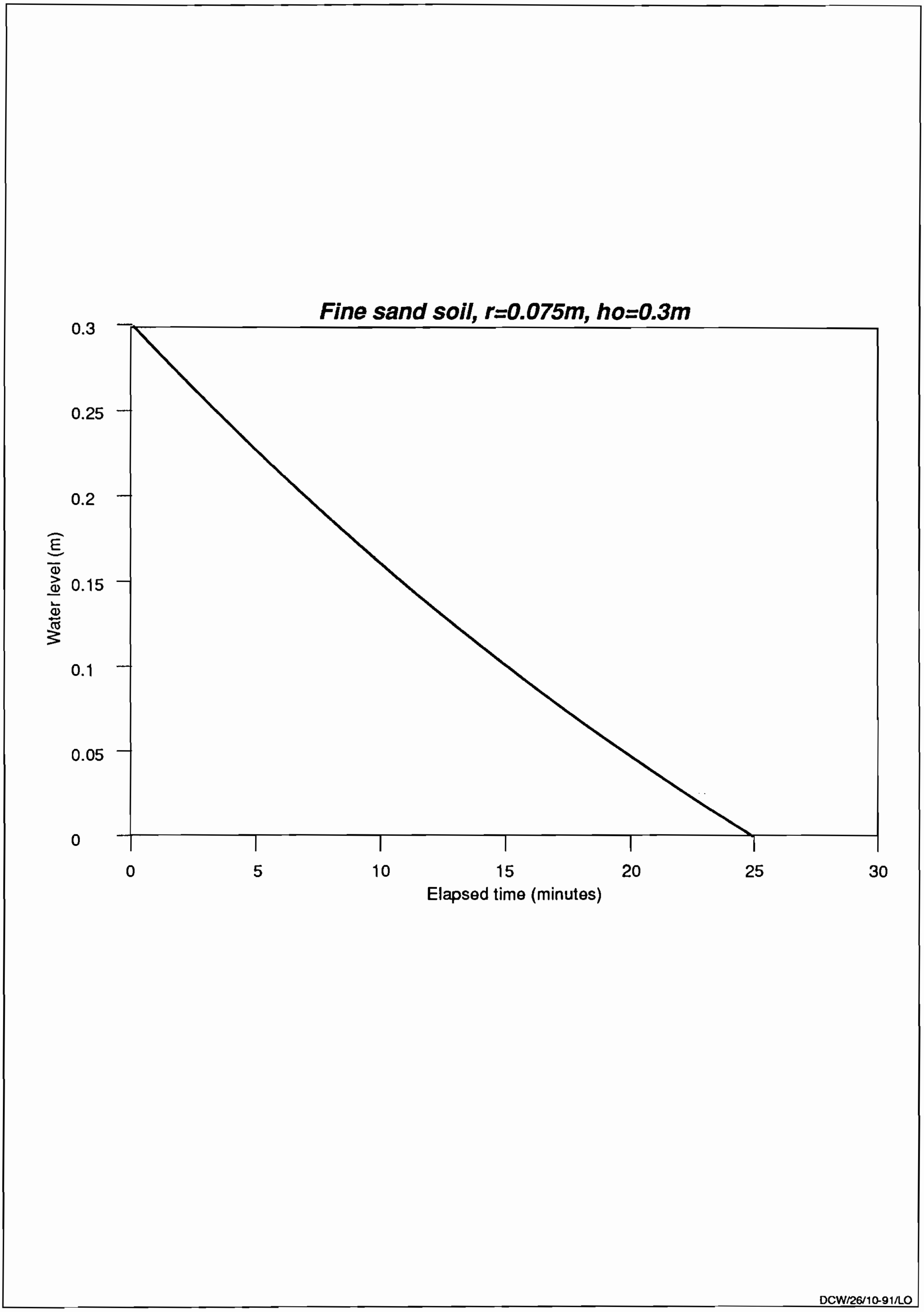

Fig 26 Simulation of a falling head test in a 0.15m diameter soakage pit – numerical model results

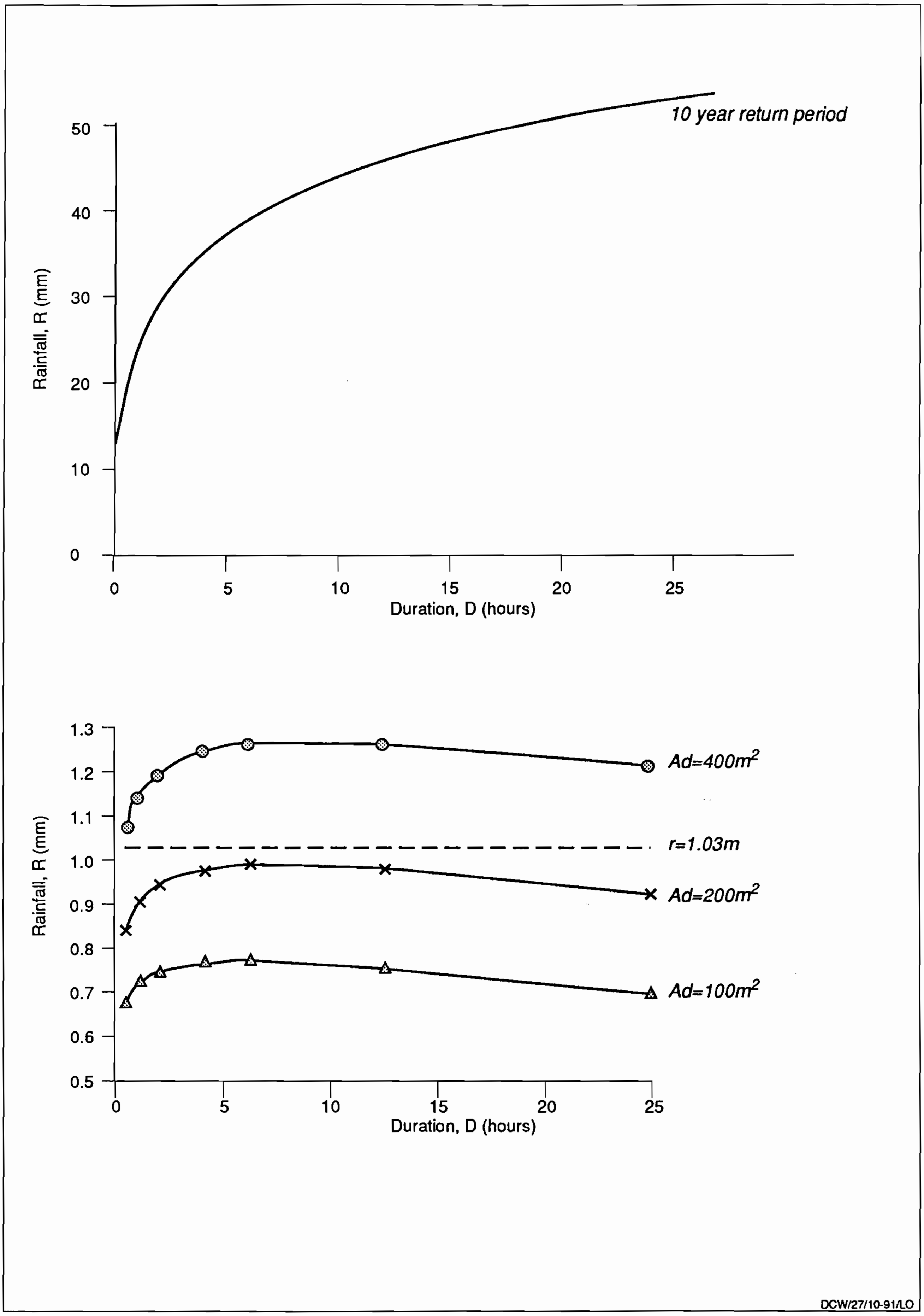

Fig 27 Rainfall events and the Pratt method of soakaway design for the example problem

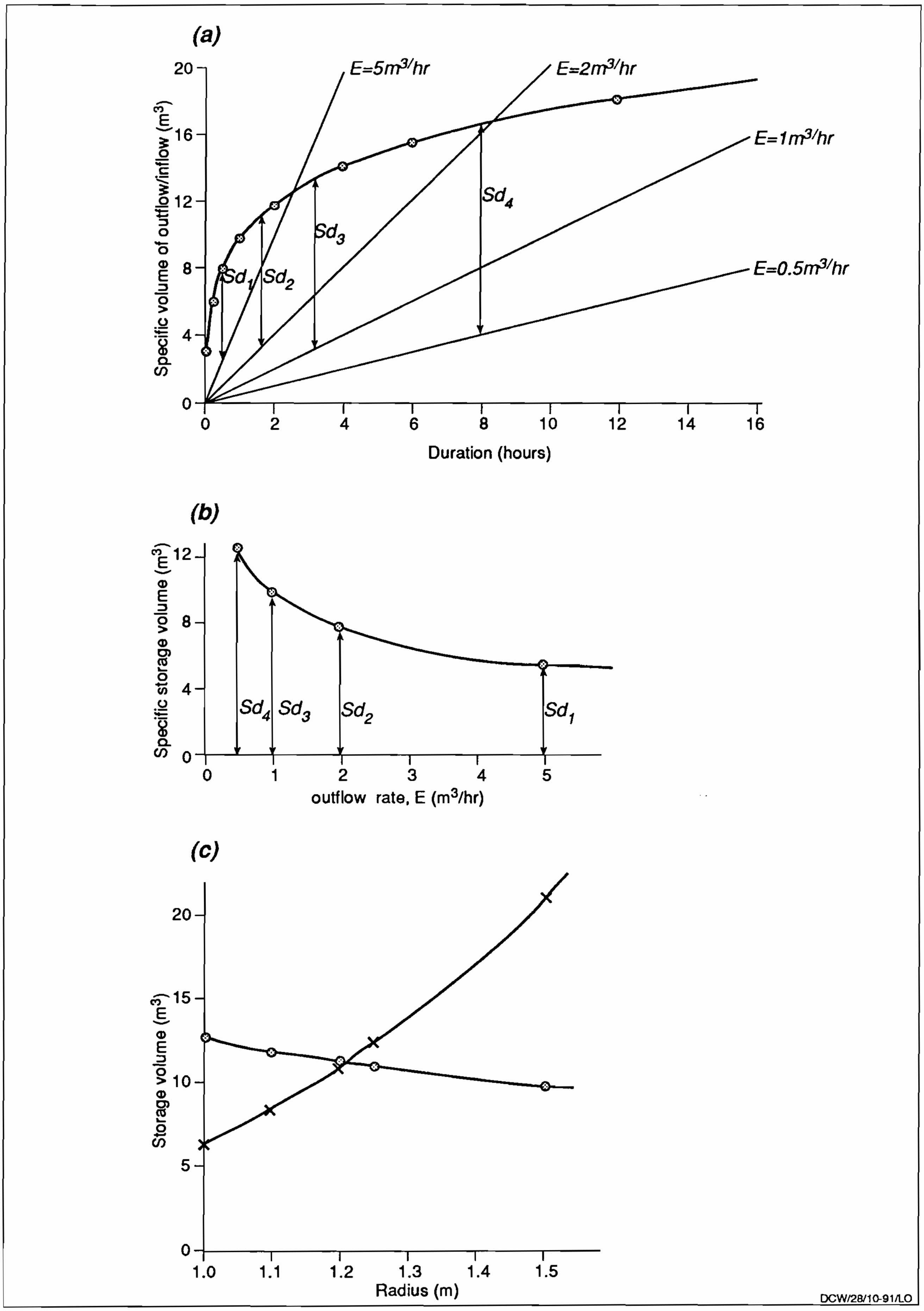

Fig 28 The rain envelope method of soakaway design for the example problem

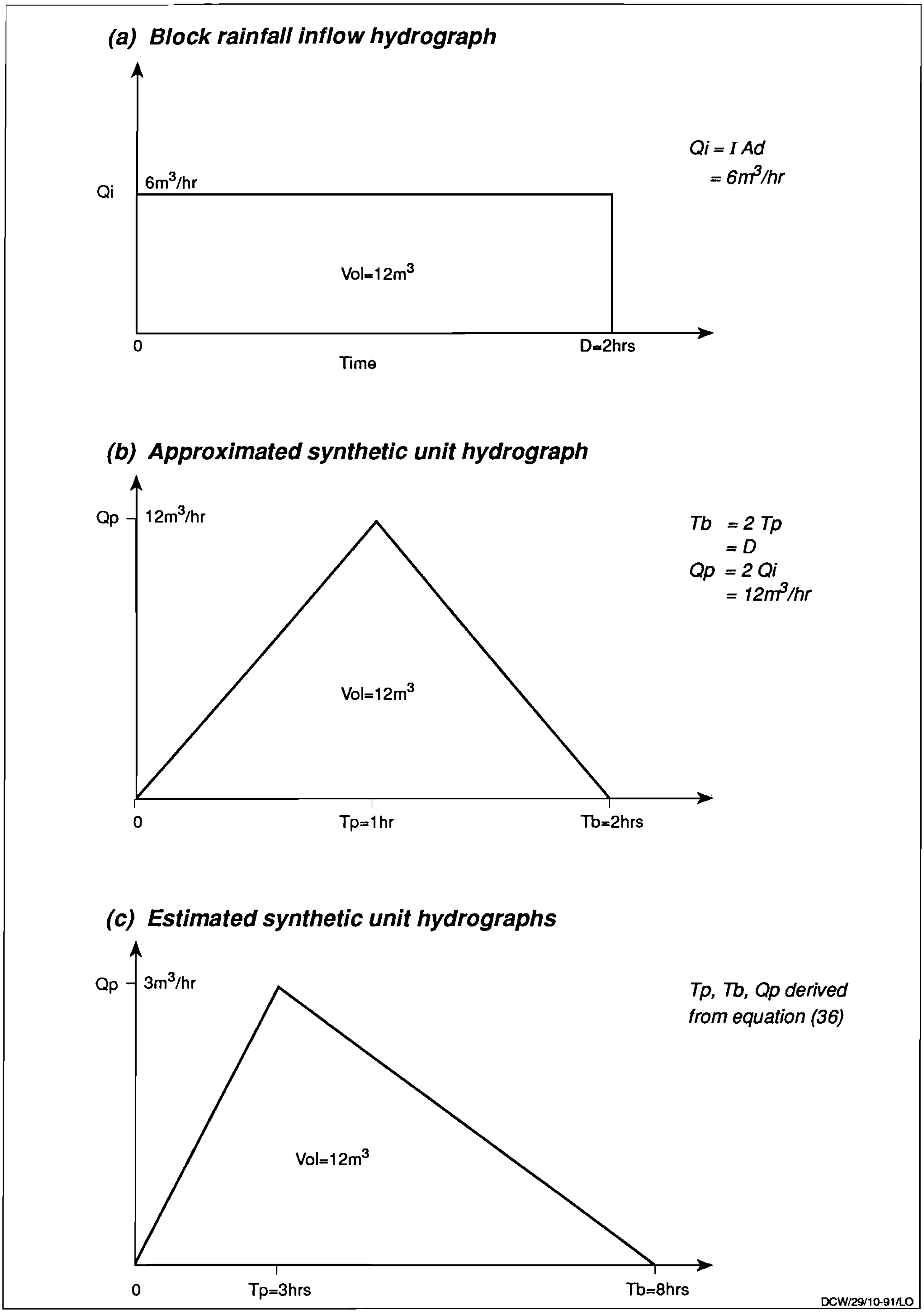

Fig 29 Approximate relation between block rainfall inflows and synthetic unit hydrograph inflows

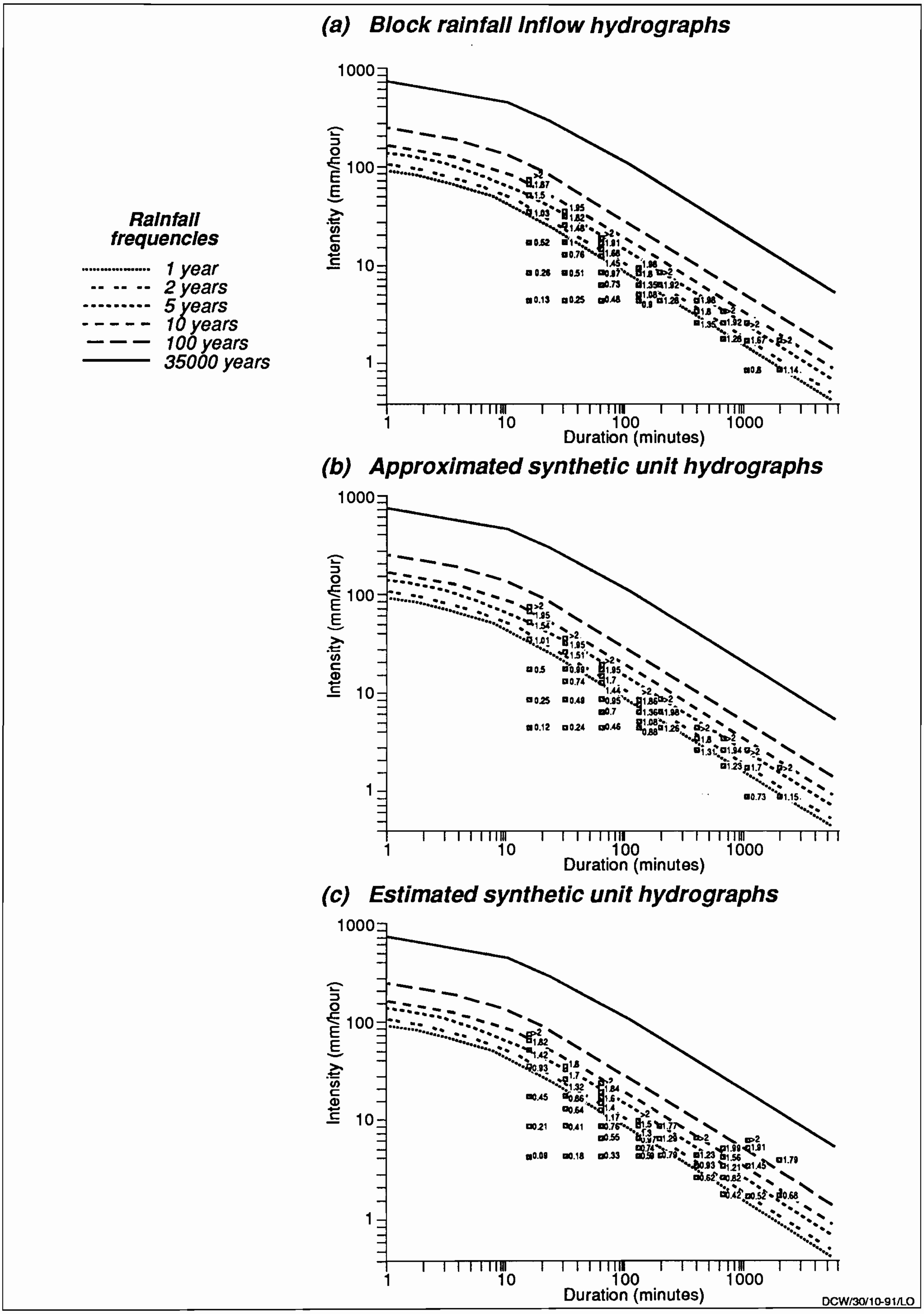

Fig 30 Soakaway performance for the example problem – Numerical model results. Peak water level for various storms

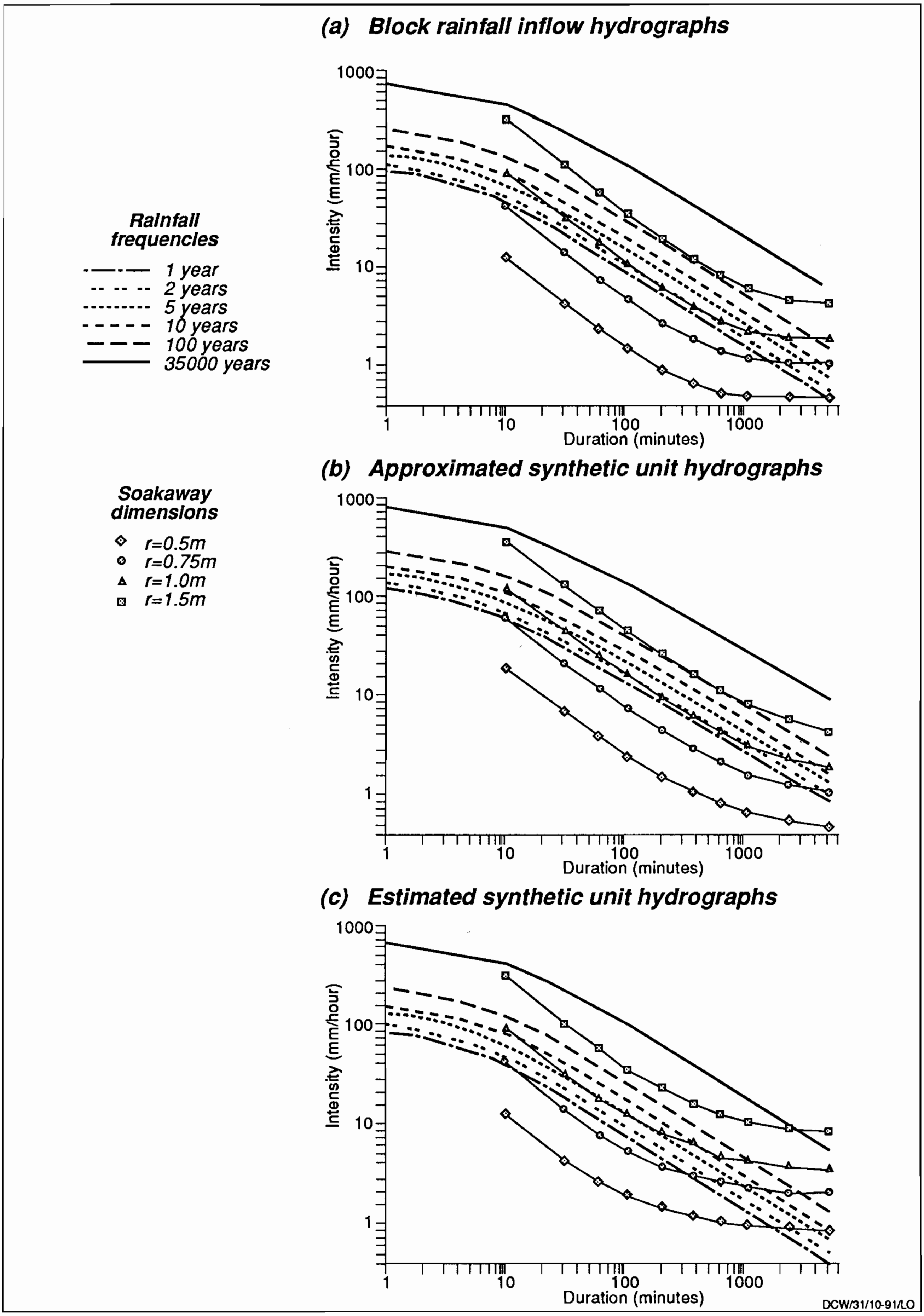

Fig 31 Soakaway performance for the example problem – Analytical model results

8. Implications for future guidelines

Guidelines could be based upon the two-coefficient analytical model developed in Section 5. Although this method takes no explicit account of the hydraulic conditions around the soakage pit or soakaway, it is shown to provide a close agreement with numerical simulations of unsaturated groundwater flow. This simple model is a type II method of soakage test analysis and the one coefficient version of equation (21) corresponds with theory used in the PSA (1974) guideline and the proposed replacement of BRE 151 (Pratt, 1990).

The soakage test results can be applied to any inflow based on synthetic unit hydrographs. In order to apply the analytical model as a guideline, however, simplified procedures would need to be formulated and tested, ensuring that such guidelines can be easily used.

A possible methodology for designing soakaways could be based on the following outline procedure. First a soakage pit is excavated and a soakage test is conducted. This will involve filling the pit with water and keeping it full for a period of time. The constant head flow rate is found either from noting the average rate at which water needs to be added or by periodically measuring the time taken for the water level to fall by a small amount. Once steady conditions are achieved, the soakage pit can be allowed to drain and the falling head water levels may be monitored. Subsequently, the soakage pit excavation is increased in size and the test is repeated.

To analyze the tests, equation (21) is simultaneously solved for the two sets of results, using the appropriate flow rates and soakage pit dimensions, to obtain values of the two infiltration coefficients. To provide a check on the values obtained, equation (26) could be used to predict the falling head water levels for comparison with the field data.

This procedure worked well in the evaluation of the field experiment described in Section 6.

Because two coefficients need to be determined, it is necessary to conduct at least two soakage tests. With the one coefficient version, equation (20), it is only necessary to conduct one soakage test. The equations that describe falling head tests and time-varying inflows, however, are the same as those given for the two coefficient model, but with $q_b = q_s = q_1$, and so detailed soakaway analysis is not significantly simplified. An advantage of the two coefficient model is that it can be applied accurately to soils with anisotropic properties, which are commonly encountered in the field.

In order to apply the results of the soakage tests, equations (27), (28) and (29) can be used to predict the performance of different sizes of soakaway. As part of a usable design guideline, this part of the procedure would need to be simplified somewhat or automated into a graphical technique.

The work reported here studied cylindrical soakaways only. Future guidelines should allow for maximum flexibility and provide recommendations regarding a wide range of infiltration techniques. These should include trenches and permeable pavements and also the types of system used in the centralised approach to infiltration drainage such as percolation basins.

9. Conclusions and recommendations

A study of the hydraulic design and performance of soakaways was undertaken.

In order to design a soakaway, it is necessary to conduct an on-site soakage test. The results of the test are applied to prototype soakaways under design rainstorm conditions in order to select an appropriate design. The soakaway problem lies in the application and scaling of field test results to prototype conditions and design rainfall events. The procedures for doing this can vary considerably with many different assumptions and virtually any size of soakaway could be recommended using the different design guidelines currently available.

The hydraulic behaviour of soakage tests and soakaways is determined by the infiltration process and involves both saturated and unsaturated groundwater flow. The governing flow equation is the modified Richards equation, equation (12), which was solved numerically to simulate the hydraulic behaviour of soakage tests and soakaways. This provided a valuable insight into the relations involved in the soakaway problem.

The results of the numerical simulations were used to test simplified analytical models of the problem. A simple model which provides a reasonable agreement with the numerical results is the hypothesis that the flow from a soakaway can be described in terms of two infiltration coefficients, the infiltration per unit area through the base and the infiltration per unit area through the sides of the excavation, equation (21). This analytical model can be used to describe the water level in a soakage pit or soakaway during a falling head test and the water level in a soakaway responding to a time-varying inflow. A procedure was formulated to calculate the maximum water level in a soakaway in response to an inflow function described by a synthetic unit hydrograph, according to the two-coefficient model.

This work has enabled the hydraulic design and performance of soakaways to be better understood and evaluated. It is now possible to formulate a design methodology, such as that outlined in Section 8 above, and design procedures for which the consequences of the assumptions made are known. Future design procedures may draw upon some aspects of existing design procedures which are considered applicable and disregard aspects which are considered invalid.

Further hydraulic analyses are recommended. These are needed to study such factors as the effects of seasonal changes in ambient or antecedent conditions and the vertical and lateral extent of different soil horizons. The numerical model should also be applied to a wider range of soil types to test the range of validity of the analytical model.

Future guidelines should also be made applicable to types of infiltration drainage systems other than soakaways.This would encourage the use of more centralised types of infiltration drainage systems in the United

Kingdom. Further hydraulic analyses would make it possible to compare and assess the relative merits of different types of infiltration drainage systems. The results of this work should enable Authorities to determine local policies regarding the use of infiltration drainage and optimise the use of this option for controlling urban stormwater.

Other technical, but non-hydraulic, issues which must also be addressed in future guidelines are the water quality implications and requirements for maintenance. Non-technical issues which should be considered are the legal implications and division of responsibilities for centralised systems and the comparative costs involved in using different methods.

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