Hydraulics Research
Wallingford

Groundwater flow beneath flood embankments

Modelling procedures

D C Watkins BA

Report SR 169
March 1988

Registered Office: Hydraulics Research Limited,
Wallingford, Oxfordshire OX10 8BA.
Telephone: 0491 35381. Telex: 848552

This report describes work funded by the Ministry of Agriculture, Fisheries and Food under contract CSA 557-138, River Flood Protection. It is being carried out in the River Engineering Department of Hydraulics Research. Dr W R White was the Company's nominated project officer and the section leader was Dr R Bettess. The Ministry of Agriculture nominated officer was Mr R Buckingham.

The report is published with the permission of the Ministry of Agriculture, Fisheries and Food, but any opinions expressed are not necessarily those of the Department.

ABSTRACT

Embankment schemes are often built to contain river flooding on alluvial flood plains. These flood plains invariably contain permeable fluvial deposits and an interaction between the river flood and the groundwater system may result in high groundwater pressures evolving inside the area protected by an embankment. This may result in either seepage of water to the ground surface causing flooding or instability of the ground due to high porewater pressures.

The purpose of this study has been to investigate this phenomenon and to examine ways in which the reaction of the groundwater system to an imposed river flood, contained behind an embankment, may be predicted.

This report describes the problem in terms of conceptual, mathematical and numerical models. A simple numerical model has been used to conduct sensitivity analyses on parameters that govern the groundwater flow. A simple formula is presented which may be used to obtain a first estimate of the severity of the problem from the aquifer properties. The importance of various aspects of the groundwater system under consideration are discussed.

CONTENTS

		Page
1	INTRODUCTION	1
	1.1 The problem	1
	1.2 Tackling the problem	1
	1.3 Work carried out at HR, 1987-88	3
2	CONCEPTUAL MODELS	3
	2.1 The unconfined aquifer	5
	2.2 The confined aquifer	5
	2.3 The semi-confined aquifer	6
	2.4 Distribution of surface ponded water	7
3	MATHEMATICAL MODELS	9
	3.1 Non-steady flow in the aquifer	9
	3.2 Leaky aquifer approach	10
	3.3 Uplift pressures and seepage gradients	11
4	NUMERICAL MODELS	11
	4.1 The finite difference method	12
	4.2 The explicit finite difference method	12
	4.3 Scope of models	13
5	THE FLOODPLAIN MODEL	14
	5.1 Discretisation	14
	5.2 Data input	15
	5.3 Boundary conditions	15
	5.4 Timesteps	16
	5.5 Calculation procedures	16
	5.6 Results output	17
6	SENSITIVITY ANALYSES	17
	6.1 The standard case	18
	6.2 Deviations from the standard case	18
	6.2.1 Storage coefficient	19
	6.2.2 Hydraulic conductivity	19
	6.3 Susceptibility to failure	20
	6.4 Further deviations from the standard case	22
	6.4.1 Varying transmissivity	22
	6.4.2 Unconfined/confined models	22
	6.4.3 Semi confined models	23
	6.5 Discussion of sensitivity analyses	24

CONTENTS (CONT'D)

		Page
7	CONCLUSIONS	26
8	RECOMMENDATIONS	27
9	REFERENCES	28

TABLE

1. Model results presented

FIGURES

1. Groundwater flow beneath a flood embankment
2. Numerical modelling options

3-23 FLOODPLAIN model results. See Table 1 for details

1 INTRODUCTION

1.1 The problem

Flood alleviation schemes often incorporate earth embankments to protect prime agricultural or developed areas of the flood plain. The embankments themselves, if suitably constructed, are relatively impermeable but the ground beneath them may be permeable. River flood plains are commonly built up of permeable fluvial sands and gravels overlain by less permeable alluvial silts and clays.

As a result, significant groundwater flow can take place through the soil once a high head of water is maintained in the channel between the artificial banks of a flood alleviation scheme. Such a head of water will increase groundwater pressures which may be transferred through the permeable strata, forcing groundwater to the surface and flooding the land inside the embankment. This may be regarded as a partial failure of the embankment scheme even though overtopping has not occurred.

The hydraulics of such a system need to be considered during the design of flood embankment schemes in order to identify areas at risk and to assess the true degree of flood protection provided.

1.2 Tackling the problem

In order to provide a view of the problem and to quantify anticipated effects, mathematical modelling techniques may be used. By using mathematical methods to model a known system under flood conditions, a design engineer may assess the potential for groundwater to short-circuit an embankment scheme. A prediction of the behaviour of the groundwater system to chosen flood events can enable an identification to be made of particular areas at risk and provide a realistic determination of the degree of protection provided by a proposed scheme.

The first step in this procedure is to collect and analyse data on the physical and hydraulic characteristics of the hydrogeological system under investigation. This is achieved by carrying out a field investigation which includes the drilling of boreholes to identify sub-soil horizons, pumping tests to determine the hydraulic properties of the sub-soils and piezometric monitoring to study the nature of the groundwater regime.

The next step is to form a conceptual model of the system by interpretation of the field data. Here, the engineer must gain an understanding of the physical processes governing the groundwater flow in the system under investigation in order to appreciate which simplifying assumptions are reasonable. By applying simplifying assumptions, such as purely horizontal flow in the aquifer, it is possible to resolve a complex 3-dimensional problem to a simpler 2 or 1-dimensional problem.

Once this has been done, the model can be formulated in mathematical terms. The partial differential equations governing groundwater behaviour may be written in terms of the parameters measured during the site investigation and set for given boundary conditions. The problem we are now faced with is the solution of the partial differential equations under transient flow with a time-variant boundary condition. In other words, we wish to model the groundwater reaction over a time period during which the river level acts in a specified manner.

The most suitable method of solving this mathematical problem is by using an approximate numerical method, carrying out lengthy and repetitive computations on a digital computer. This method can allow a review of the predicted groundwater situation at specified time periods during a particular flood event simulation.

1.3 Work carried out by HR, 1987-88

The purpose of this study has been to follow through the procedure of setting up numerical models of typical, but hypothetical, flood embankment situations. This has served to identify potential obstacles to the application of the technique, to the particular problem of groundwater flow beneath flood embankments. It has highlighted areas in which a choice of conceptual model may lead to very different results and has provided information on the sensitivity of a model to the input data.

Section 2 of this report describes three conceptual models, the unconfined, confined and semi-confined aquifer models. The equations governing the groundwater flow are presented in Section 3. Numerical methods are introduced in Section 4 but in the briefest of detail. The interested reader should refer to the references cited for further information. Section 5 describes the 1-dimensional explicit finite difference model constructed at HR. The sensitivity analyses carried out with the HR FLOODPLAIN model are described and discussed in Section 6. The conclusions and recommendations of the study are contained in Sections 7 and 8 of the report respectively.

2 CONCEPTUAL MODELS

The models which are envisaged here consist of an aquifer of known thickness, which may vary across the site, and known hydraulic conductivity and storage capacity which is intercepted by a river channel. The aquifer may be capped by an overburden of considerably lower permeability than the aquifer. Figure 1 shows a cross-section sketch of such a floodplain system where a development is protected from overland flooding by an embankment.

The groundwater flows are initially in a steady-state condition, usually with a net flow of water to the river which acts to drain the floodplain (Figure 1a). With the onset of a flood event, the water level in the river rises and creates a hydraulic head difference with the groundwater in the adjacent soil. This reverses the direction of groundwater flow and the river acts to recharge the aquifer (Figure 1b). In hydrological terms, the river changes from an effluent to an influent nature. The piezometric response and resulting flow of groundwater within the aquifer due to this head difference depends upon the transmissivity of the aquifer, which is the product of the hydraulic conductivity of the aquifer and the saturated thickness and also upon the degree of storage of water that can occur within the aquifer. The head of water in the river rises above ground level and is prevented from innundating the ground above the aquifer by a flood embankment, so a high head difference is maintained. When the river level recedes, the excess groundwater heads dissipate and the groundwater drains back toward the river (Figure 1c).

There are three basic models that the engineer may wish to consider: unconfined, confined and semi-confined aquifers, depending upon the degree of influence of the alluvial overburden. The unconfined aquifer is one above which the overburden is either not present or has negligible effect and so does not restrict the flow from the aquifer. In the confined case, the overburden is considered to be totally impermeable and so high groundwater pressures within the aquifer do not result in a flow of water through the overburden to the surface, though for this situation, other dangers exist (Section 2.2). The unconfined and confined models are the two extremes when considering the effect of the overburden. The most likely situation to occur naturally, however, is that of a low permeability (but not impermeable) overburden which semi-confines the aquifer but may also allow seepage to the ground surface.

2.1 The unconfined aquifer

In the case of the unconfined (phreatic) aquifer, the water table (phreatic level) is a reflection of the hydraulic head in the aquifer. If a volume of water is added to a finite unconfined system, the water table rises in accordance with the amount of fillable pore space or storage coefficient of the aquifer. This storage coefficient is termed the specific yield and is defined as the difference between the porosity of the soil and the specific retention; the specific retention being the background moisture content of the unsaturated soil due to water that does not drain out under the influence of gravity alone. In an unconsolidated granular soil, the specific yield may be in the region of 0.2 - 0.4.

As there is no restricting layer in an unconfined aquifer, if a hydraulic head above ground level is predicted, exfiltration of water to the surface is implied resulting in surface flooding. This is considered further in Section 2.4.

2.2 The confined aquifer

A confined aquifer is fully saturated and the hydraulic head is reflected by the piezometric level which is above the top of the aquifer. If the piezometric head dropped below the top of the aquifer, an unconfined condition would exist. With this definition, there can be no storage due to the specific yield of the soil. Instead, the elastic storage (or storativity) of the aquifer constitutes the storage coefficient. This elastic storage is due to a slight reorientation of the soil grains that takes place in response to changes in hydraulic head imposed on the soil and also to the slight compressibility of water. The value of this storage coefficient is usually small, frequently in the region 0.001 - 0.01. Due to this low storage capacity in comparison with unconfined aquifers, high piezometric heads may evolve in a confined aquifer without a corresponding large flow of water being required.

As no exfiltration of water through the overburden can occur, there is no danger of flooding by seepage when the piezometric level exceeds ground level. There is, however, a danger of uplift pressures exceeding the weight of the overburden which may result in floatation of the soil providing a flow path to the surface. Under high piezometric heads, this mode of failure may be highly destructive. A further consideration here is that high uplift pressures may occur beneath the foundations of buildings which penetrate through the confining layer and an investigation of this is an appropriate application of the type of model considered in this report.

2.3 Semi-confined aquifers

The theory of leaky aquifers describes the flow of water within two aquifers separated by a semi-permeable layer, the features of which are incorporated in a leakage term. By treating the space above the ground surface as an upper aquifer with a storage capacity of 100%, it is reasonable to apply the leaky aquifer approach to our model and calculate the head of water that ponds on the ground surface.

A difficulty arises here when the hydraulic head in the aquifer is above the base of the overburden but below the ground surface. The 'upper aquifer' then has no effect on the flow and so the leaky aquifer theory cannot be applied. This means that the aquifer must be treated as fully confined when the piezometric head is between the levels of the top and base of the overburden. To account for the head in the overburden, it is necessary, therefore, to make a simplifying assumption such as: the head in the overburden is equal to the head in the aquifer until the latter exceeds ground level. The error introduced by this approximation is unknown. It is essentially assuming the overburden to be just fully saturated with a zero pore pressure at all points outside the influence of the piezometric level of the aquifer. This may well be justified as the effect of any infiltrating rainwater upon the moisture potential of the overburden is unknown.

When modelling the semi-confined case, it is possible to take account of both groundwater seepage to the surface, as with unconfined aquifers and also uplift pressures, as with confined aquifers.

2.4 Distribution of surface ponded water

Once water ponds on the ground surface, there is uncertainty as to whether it remains in its ponded location or whether it is distributed by flowing over the land surface to an equilibrium level.

If a small depth of water ponds, it is unlikely to be redistributed due to the effects of vegetation, footpaths, hedgerows etc which are not accounted for on the topographic scale of the model. Furthermore, groundwater would be likely to pond at the lowest lying areas first, obviating the need for redistribution, though this is not necessarily the case. The problem arises when substantial ponding occurs which, if not redistributed, leads to considerable gradients appearing on the free water surface. The best line of approach is probably to assume that no surface flow takes place unless the results indicate significant redistribution. If this happens, the model may be rerun using surface water redistribution.

When considering the semi-confined aquifer condition, these two cases can be modelled very easily using the leaky aquifer approach because an upper aquifer with 100% storage capacity is envisaged above ground level to handle the surface ponded water. If the ponded water is not to be redistributed, the upper aquifer is assigned a zero transmissivity value and if the water is to be redistributed over the land surface, an extremely large value of transmissivity is assigned to the upper aquifer. In order to do this, however, the explicit FDM cannot be used as this would lead to excessively small timesteps (Section 4.2) and so an implicit scheme is required.

In the unconfined aquifer case, once the head is predicted above ground level, changes in hydraulic head are calculated using a storage coefficient of unity. This takes account of the fact that a 100% storage capacity is available above ground level.

3 MATHEMATICAL MODELS

3.1 Non-steady flow in the aquifer

In the absence of sources and sinks, the partial differential equation describing non-steady groundwater flow in an aquifer is given by

$$S \frac{\partial h}{\partial t} = T \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}\right) \qquad (1)$$

where S = storage coefficient
T = transmissivity
h = hydraulic head
t = time
x,y,z = length dimensions

If we make the assumption that only horizontal flow occurs in the aquifer and that the flow does not vary along the length of the embankment (1-dimensional flow) , the partial differential equation simplifies to

$$S \frac{\partial h}{\partial t} = T \frac{\partial^2 h}{\partial x^2} \qquad (1a)$$

This equation may be used to model flow in the aquifer. In the unconfined case, the storage coefficient is the specific yield and the transmissivity is the product of the hydraulic conductivity and the hydraulic head (saturated thickness). In the confined case, the storage coefficient is the elastic storage and the transmissivity is given by the product of the hydraulic conductivity and the aquifer thickness (saturated thickness).

3.2 Leaky aquifer approach

A low permeablility layer separating two aquifers will have a resistance to flow, c, whereby

$$c = d/K' \tag{2}$$

where d = thickness of layer
K' = vertical hydraulic conductivity of layer

Flow through such a layer between two aquifers is described by a leakage term,

$$\ell = \frac{h_2 - h_1}{c} \tag{3}$$

where h_1 = hydraulic head in aquifer 1
h_2 = hydraulic head in aquifer 2

Assuming only horizontal flow in the aquifer and only vertical flow in the semi-confining layer, the flow in each aquifer is governed by the coupled equations

$$S_1 \frac{\partial h_1}{\partial t} = T_1 \frac{\partial^2 h_1}{\partial x^2} + \ell \qquad \text{in aquifer 1}$$

and (4)

$$S_2 \frac{\partial h_2}{\partial t} = T_2 \frac{\partial^2 h_2}{\partial x^2} - \ell \qquad \text{in aquifer 2}$$

where $S_{1,2}$ = storage coefficients of aquifers 1 and 2
$T_{1,2}$ = transmisivities of aquifers 1 and 2

3.3 Uplift pressures and seepage gradients

An upward hydrostatic pressure on the overburden occurs when the hydraulic head in the aquifer exceeds the hydraulic head above the overburden. Uplift and mechanical failure of the overburden can occur when the upward hydrostatic pressure exceeds the downward soil loading pressure.

Net uplift pressure = upward hydrostatic pressure - downward soil pressure

$$U = \gamma_w (h_1 - h_2) - \gamma_s d \qquad (5)$$

where U = uplift pressure
γ_w = unit weight of water
γ_s = saturated unit weight of soil

The critical hydraulic gradient across the overburden, i_c, at which failure occurs is therefore

$$i_c = \frac{h_1 - h_2}{d} = \frac{\gamma_s}{\gamma_w} \qquad (6)$$

4 NUMERICAL MODELS

The two main numerical methods used for solving the groundwater flow equations are the finite difference method (FDM) and the finite element method (FEM). The FDM is the most widely used but the FEM is equally applicable to the problem we wish to model here. The FEM has the advantage that non-regular grid spacings may be used, allowing specific areas of interest to be examined in greater detail than the general domain of the model, but is much more complex than the FDM.

4.1 The finite difference method

The basic concept of the FDM is to replace the derivatives at a point by ratios of the changes in appropriate variables over a small but finite interval. "The approximation is made at a finite number of points and reduces a continuous boundary problem to a set of algebraic equations", Ref 1. Various techniques may be applied to the FDM depending on the type of procedure used to solve the equations. The model described in Section 5 and used for the sensitivity analyses in Section 6 is based on the explicit finite difference method.

4.2 The explicit finite difference method

Consider a series of points in a line, distance Δx apart, at positions

$$i-1,\ i,\ i+1,\ \ldots,\ i+n-1,\ i+n$$

By applying a finite difference approximation to equation 2, the new hydraulic head that occurs after time interval Δt, at time $j + 1$, may be given by

$$h_{i,j+1} = h_{i,j} + \frac{T\Delta t}{S\Delta x^2}\left(h_{i+1,j} + h_{i-1,j} - 2h_{i,j}\right) \qquad (7)$$

This is an explicit formula which is obtained from fact that a forward finite difference approximation has been made for the time derivatives. If $i - 1$ $i + n$ are boundary points with heads given at time $j + 1$, the above formula may be used to compute the new heads at time $j + 1$ for all the points between $i - 1$ and $i + n$.

One drawback of the explicit FDM is that there is maximum size of timestep for which the approximation is ~~valid~~ stable. If too large a value of Δt is used, the solution becomes unstable. The magnitude of the maximum timestep is given by the formula

$$\Delta t \leqslant \frac{S}{2T\Delta x^2} \qquad (8)$$

and so in order to avoid excessively small timesteps it is necessary to choose sufficiently small values for the length increments.

4.3 Scope of models

The finite difference formula, equation (7), relates to 1-dimensional flow in the aquifer but may be expanded to incorporate 2 and 3-dimensional flow by discretising the aquifer into elements in the x and z, x and y or x, y and z dimensions.

Figure 2a shows a cross-section through which flow is modelled in the x direction only. This type of model makes the assumptions that

(a) all flow in the aquifer is horizontal

(b) all flow in the aquifer is perpendicular to the line of the embankment

Flow is modelled between the grid points at equal spaces along the cross-section.

Figure 2b shows a cross-section discretised to incorporate vertical flow. Approximation (a) is no longer required but approximation (b) still applies. Flow is modelled between the nodes of the superimposed mesh.

A plan view of an embankment scheme is shown in Figure 2c, discretised for flow in the horizontal plane. For this model, approximation (a) is applied but not approximation (b).

It is equally possible to model an aquifer in all three dimensions by discretising the aquifer into cubes. Neither of approximations (a) and (b) then need to be applied. The resulting model, however, will be very complex and the question arises as to whether sufficient field data can be supplied in order to justify such a detailed model.

5 THE FLOODPLAIN MODEL

A 1-dimensional explicit finite difference model was constructed at HR on a small desk-top computer. The model was based on a cross-section through the flood plain such as those shown on Figures 1 and 2a. The assumption that only horizontal flow perpendicular to the line of the embankment occurs in the aquifer is applied to provide fully 1-dimensional flow.

The model may be used to simulate the response of groundwater pressures (hydraulic heads) in an aquifer due to a flood event contained by an embankment. The aquifer may be confined, unconfined or semi-confined and the principles described in Sections 2, 3 and 4 were followed.

The level of the hydraulic head within the aquifer or above the ground surface is calculated. No attempt is made to redistribute surface ponded water.

5.1 Discretisation

The length of cross-section we wish to model, L, is divided into N segments of length, $\Delta x = L/N$. There are then N + 1 points separated by the segments. For convenience we may say that the end point is at i - 1 on the line of the embankment and the other end point at i + n is the limit of the model at distance L from the embankment.

5.2 Data input

At each point, or node, the model requires information on the hydraulic conductivity of aquifer and overburden, appropriate strorage coefficient and levels of the base of the aquifer (Z1), top of aquifer/base of overburden (Z2), top of overburden/ground level (Z3) and initial hydraulic head in the aquifer (h). The levels are all input as height relative to a common datum.

5.3 Boundary conditions

The hydraulic head at node i-1 is given by the river level according to the design flood hydrograph. The hydrograph should be that expected for the embanked scheme taking account of the channel restrictions.

The value of the other boundary head at node i + n is approximated as being equal to the hydraulic head at node i + n - 1 and therefore constitutes a no-flow boundary. If the effect of the flood reaches as far along the cross-section as i + n - 1, slight errors will occur due to this approximation. In effect, we are assuming an axis of symmetry around the point $i + n - \frac{1}{2}$. This may well be justified as the effect of groundwater entering the model domain from higher ground is not otherwise taken into account.
This boundary condition may be modified to suit a particular conceptual model. For instance, an area of the flood plain protected by embankments on both sides may be modelled by applying boundary heads given by flood hydrographs at both ends of the cross-sections. Another method is to set the length L sufficiently large so that no effect of the river flood occurs there. The model may be run once and the hydraulic

head at a point, say L/20, may be computed. The model may then be rerun with a new boundary set at the previous value of L/20 which experiences the hydrograph recorded there during the first run.

5.4 Timesteps

The minimum timestep, Δt, is calculated from the parameters Δx, S and T. The length increment, Δx, is constant throughout the model and assuming that the storage coefficient and hydraulic conductivity are constant throughout the aquifer, Δt depends upon B, the thickness of the aquifer. The node with the greatest value of B = (Z2-Z1) is therefore used to calculate the minimum timestep according to the formula (8) given in Section 4.2.

5.5 Calculation procedure

Considering the case of the semi-confined aquifer and assumming a zero transmissivity and a storage coefficient of unity for the upper aquifer, the coupled equations (4) may be written

$$S \frac{\partial h_1}{\partial t} = T \frac{\partial^2 h_1{}^2}{\partial x^2} + \ell \qquad \text{lower aquifer (1)}$$

$$\frac{\partial h_2}{\partial t} = - \ell \qquad \text{upper aquifer (2)} \qquad (9)$$

where ℓ is given by equation (4)

In finite difference terms this may be written

$$h_{1_{i,j+1}} = h_{1_{i,j}} + \frac{T\Delta t}{S\Delta x^2} (h_{1_{i+1,j}} - 2h_{1_{i,j}}) + \frac{\Delta t}{S} \left(\frac{h_{2_{i,j}} - h_{1_{i,j}}}{c}\right)$$

$$h_{2_{i,j+1}} = h_{2_{i,j}} + \Delta t \frac{(h_{2_{i,j}} - h_{1_{i,j}})}{c} \qquad (10)$$

In the cases of the fully confined or fully unconfined aquifers, the upper aquifer and the leakage term are both ignored and so equation (10) reduces to equation (7). This equation is then solved using the appropriate values of the storage coefficients.

5.6 Results output

The hydraulic heads occuring at each node may be listed at selected time intervals after the start of the flood simulation. For the case of the semi-confined aquifer, two sets of results are output referring to the hydraulic heads above and below the overburden.

The model may be run for some time after the flood has passed because the effect of the flood in the aquifer may be prolonged compared to the effect in the river.

The results may be displayed graphically to provide a view of the groundwater response, as has been done with the results of the sensitivity analyses.

6 SENSITIVITY ANALYSES

The 1-dimensional FLOODPLAIN model was run for a set standard configuration and hydraulic conditions and also for deviations from that standard in order to study the sensitivity of the model to the parameters derived from the field investigation, for the fully confined and fully unconfined cases.

A dimensionless parameter has then been used to describe the susceptibility of the aquifer to failure of an embankment scheme due to flow beneath the embankment. Finally, some model runs have been carried out to simulate more realistic situations using leaky aquifer theory to model water ponding above ground level.

Table 1 lists the model runs carried out with the FLOODPLAIN model, the results of which are presented in Figures 3 to 23.

6.1 The standard case

The standard case consists of a uniform horizontal aquifer which may be either fully confined or fully unconfined. The base of the aquifer (Z1) is at datum level. The top of the confined aquifer (Z2) is at 5m above datum. The initial river level (F1) and initial hydraulic heads are also at 5m above datum. The river level rises according to a flood hydrograph which is a simple harmonic cycle with a peak (F2) at 10m above datum and a period (t) of 100 hours. The ground level is at least 10m above datum. The floodplain is modelled over a distance (L) of 500m from the embankment using 20 length increments. The hydraulic conductivity of the aquifer (K) is 0.001m/s the specific yield (Sy) is 0.3 and the elastic storage (Se) is 0.001.

The results from this standard case are shown for an unconfined aquifer in Figure 3 and for a confined aquifer in Figure 8.

6.2 Deviations from the standard case

In order to demonstrate the sensitivity of the model to the parameters derived from field tests, model runs have been carried out using the standard case but with different values of storage coefficient and hydraulic conductivity.

6.2.1 Storage coefficient

Figures 3, 4 and 5 show the resulting phreatic head profiles for an unconfined aquifer with specific yield of 0.3, 0.2 and 0.1 respectively. The effect of the river flood can be seen to increase with decreasing storage though the effect is small over these ranges. Figures 6, 7 and 8 show the resulting piezometric head profiles for a confined aquifer with elastic storage of 0.1, 0.01 and 0.001 respectively. It can be seen that these order-of-magnitude reductions in storage cause a significant increase in the effect of the river flood on groundwater pressures.

Figures 5 and 6 present the unconfined and confined cases with the same storage coefficients. These two cases are not quite identical because flow in the confined aquifer is restricted to a thickness of 5m and flow in the unconfined aquifer may occur through the entire saturated thickness, up to 10m. Comparison of Figures 5 and 6, however, shows that for this case, where the river fluctuation is equal to the thickness of the confined aquifer, the difference between the unconfined and confined cases with equal storage coefficient is very small indeed. Close inspection of Figures 5 and 6 reveals that the effect of the river flood is slightly greater in the unconfined case due to the temporarily increased transmissivity.

6.2.2 Hydraulic conductivity

The standard model assumes a constant aquifer thickness and a change in hydraulic conductivity produces a proportional change in transmissivity.

Figure 9 presents the results for an unconfined aquifer of hydraulic conductivity 0.01m/s and Figure 10 the results for a confined aquifer of hydraulic conductivity 0.0001m/s. Figure 11 shows a confined aquifer of hydraulic conductivity 0.01 and storage coefficient 0.1.

Note that the results of Figure 10, K = 0.0001 S = 0.001 are identical to those of Figure 11, K = 0.01 S = 0.1 and of Figure 7, K = 0.001, S = 0.1.

6.3 Susceptibility to failure

The susceptibility of the embankment scheme to failure due to groundwater flow in the aquifer is increased with the time that flooding is sustained and with increasing transmissivity of the aquifer but is decreased with increasing storage coefficient and with increasing distance from the embankment.

From Equation (7), it can be seen that proportional changes in the hydraulic head in the aquifer,

$$\frac{\Delta h}{h} \propto \frac{T\Delta t}{S\Delta x^2} \qquad (10)$$

The susceptibility of a scheme to failure due to groundwater flow through the aquifer may be expressed in terms of the rise in hydraulic head at the point of concern. Substituting the period of the flood event, t for Δt, the distance to the model limit, L for Δx and the susceptibility for failure of the scheme, say E, for $\Delta h/h$

$$E = \frac{Tt}{SL^2} \qquad (11)$$

where E is a dimensionless parameter which describes the suspectibility of a floodplain to raised groundwater levels due to a flood event contained behind an embankment. Because E is non-dimensional, it can be used to characterise the problem.

Values of E are included in Table 1 which explains the similarity in the results presented in Figures 7, 10 and 11.

Figures 12, 13, 14 and 15 are type curves which show the hydraulic head profiles for situations where the value of $E = 0.01$, 0.1, 1 and 10 respectively. For ease of comparison these are plotted on the same scale as Figures 3 to 11. Inspection of these figures show that for $E = 0.01$ the effect of the river flood at distance L from the embankment is negligible. If $E = 0.1$, the effect is very small. If $E = 1$, then the effect is considerable and the location at L experiences more than half the river flood peak, though at a later time. In the situation where $E = 10$, the aquifer at the location L experiences about 99% of the river flood peak.

In order to assess the severity of a given situation, we may make the following generalisation:

Susceptibility	Severity
$E \leqslant 0.1$	low
$0.1 \leqslant E \leqslant 1$	moderate
$1 \leqslant E$	high

and use the type curves from Figures 12 to 15 as a guide to the magnitude of groundwater rise to be anticipated. In fact, the true severity of a given situation depends upon the ground level at the location of concern in relation to the river levels F1 and F2. It is important to note that in this model, L represents an assumed point of symmetry and not just a distance from the embankment. The positioning of this point in the model is obviously an important factor affecting the value of E.

6.4 Further deviations from the standard case

The standard case refers to an aquifer of uniform transmissivity which is either fully confined or fully unconfined. Real situations are likely to involve aquifers with undulating boundaries which are confined for some of the time, for part of the aquifer. Also, the standard case has not considered the effect of water ponding on the ground surface. These cases are considered below.

6.4.1 Varying transmissivity

The standard case assumes a constant transmissivity due to a constant thickness of aquifer. An increase or reduction in depth of flow will lead to a corresponding change in transmissivity. Changes in aquifer thickness, however, are likely to be fairly small (not orders-of-magnitude) and so the corresponding changes in the hydraulic head profile are also likely to be small. Figures 16 and 17 show the effect of a gradually increasing and gradually decreasing transmissivity respectively, on the confined case with a mean value of E = 0.432 in both cases.

6.4.2 Unconfined/confined models

Figures 18 and 19 present the results of model runs with a confined aquifer with the top of the aquifer (Z2) at 5.01 and 5.1m above datum (0.01 and 0.1m above the initial water level) respectively. The case where Z2 equals the initial water level is given by Figure 10. This means that the aquifer is unconfined until the hydraulic head reaches Z2 and then acts as a confined aquifer with a correspondingly reduced storage coefficient. The possibility of failure of the scheme is reduced dramatically if the aquifer remains in the unconfined condition for just a small amount of time.

6.4.3 Semi-confined models

The semi-confined model is treated as unconfined for hydraulic head in the aquifer, $h_1 \leqslant Z2$, confined for $Z2 \leqslant h_1 \leqslant Z3$ and leaky for $h_1 \geqslant Z3$.

Figures 20-24 consider the effect of the low permeability overburden on a hypothetical system where the base of the aquifer (Z1) varies between 0 and 2m above datum, the top of the aquifer (Z2) between 4m and 6m above datum and the ground level (Z3) between 7m and 9m above datum.

Figure 20 shows the resulting profile for a fully unconfined aquifer which is the case when the hydraulic conductivity of the overburden $K' = K$. Figure 21 presents the profile for a fully confined aquifer which is the case when $K' = 0$. Note how the phreatic portions of the aquifer restrict the development of the piezometric head.

Figure 22 shows the piezometric head and the resulting surface water profile for the case where the over burden has a vertical hydraulic conductivity of 10^{-7}m/s ($K' = K \times 10^{-4}$). Comparing Figures 21 and 22 demonstrates that for this configuration, virtually all of the piezometric head above ground is expressed as surface ponding. Figure 23 shows a similar situation but with the vertical hydraulic conductivity of the overburden reduced to 10^{-9}m/s ($K' = K \times 10^{-6}$). In this case the resistance to flow offered by the overburden is sufficient to restrict the surface ponding to a negligible amount.

6.5 Discussion of sensitivity analyses

The susceptibility of a floodplain to raised groundwater levels, E, as defined in Section 6.3, demonstrates the effect of the design parameters, t and L, and of the measured parameters, T and S, on the resulting groundwater behaviour of a model similar in nature to the standard case of the FLOODPLAIN model and is given by equation (11). The implications of the relative importance of aquifer properties are generally applicable to variations of this particular model and, therefore, to alluvial floodplains in general.

An error in the value of E of one order of magnitude is sufficient to drastically alter the results predicted by the model as demonstrated in Figures 12 to 15. An error of two orders of magnitude is sufficient to make the difference between a predicted low severity and high severity problem such as between the results shown in Figures 6 and 8. As the value of E is equally sensitive to S, T, t and L^2, each of these parameters should be known to the same order of accuracy.

The parameter t represents the length of time over which flooding occurs and is fixed by the design conditions. All the model runs presented herein relate to a flood hydrograph based on a simple harmonic cycle. Should a real flood hydrograph reach its peak quickly and/or maintain the peak for some time the susceptibility to flooding would be greater than for the cases presented in Figures 3-23.

The parameter L represents a boundary condition. As E is inversely proportional to L^2, the correct positioning of this boundary is important but with a good conceptual model and carefully chosen boundary conditions, it is unlikely that L^2 would be in error by an order of magnitude. The alternative methods of specifying the boundary conditions discussed in Section 5.3 may be more applicable to many situations.

The storage coefficient, S, is unlikely to vary much for an unconfined aquifer (Figs 3,4,5) but the range of values that may apply to a confined aquifer can easily vary by orders of magnitude (Figs 6,7,8).

The transmissivity, T, depends directly on the value used for the hydraulic conductivity of the aquifer as the aquifer thickness is unlikely to be much in error. Hydraulic conductivities and elastic storage coefficients are derived from carrying out pumping tests. Both parameters can be highly variable and are usually only quoted to an accuracy of one order of magnitude.

The area in which the greatest error is likely to occur is in assessing the degree of confinement provided by the overburden. E is liable to be in error by several orders of magnitude if a confined aquifer is treated as unconfined or vice versa (Figs 3,8 and Figs 18,19). The implication of this is that it is worthwhile expanding particular effort to assess the degree of confinement provided by the overburden.

If groundwater flow through the overburden is anticipated, the vertical hydraulic conductivity of the overburden may be taken into account using the leaky aquifer theory. The hydraulic conductivity of the overburden material within the normally unsaturated soil zone is a particularly difficult quantity to measure (Ref 6) and again, order of magnitude errors are possible which may lead to large discrepancies (Figs 22,23).

7 CONCLUSIONS

The reaction of a floodplain groundwater system, with known hydraulic properties, to an imposed river flood contained behind an embankment, may be predicted using numerical modelling techniques and by following the procedures outlines in this report.

The calculation is clearly defined for fully unconfined or fully confined aquifers but not for the situation where a semi-confining layer overlies the aquifer, which is common in alluvial floodplains.

A first estimate of the severity of the problem may be made for a generalised model of a flood plain using equation (11) to obtain a value of the dimensionless parameter, E. This provides an indication of the severity of the problem for given conditions. A range of possible conditions may then be examined to provide an indication of the likelihood of a problem existing and whether more detailed modelling is, indeed, required.

The main area in which error is likely to occur is in defining the degree of confinement provided by the overburden and the degree of storage available within it.

The model results are also sensitive to errors in the hydraulic conductivity and elastic storage coefficient of the aquifer, as derived from the field investigation. It is important, therefore, to gain a first estimate of these values from the field investigation, but to be prepared to adjust these by calibrating the model against a measured reaction of the groundwater system to fluctuations in flow conditions. This should provide more representative values of the parameters T and S and help verify and improve the conceptual model as much as possible.

8 RECOMMENDATIONS

In order to extend the work carried out in 1987-88 under this contract, the following directions of further study are recommended.

1. Use the basic FLOODPLAIN model to examine the applicability of the type curves based on the dimensionless parameter, E, to situations with different boundary conditions to those used herein and also to a range of true flood hydrographs.

2. Carry out more detailed modelling of hypothetical situations. This would enable a comparison of more complex models with the simple type of model used here and test its assumptions and limitations.

3. Refine the semi-confined aquifer theory. The standard groundwater equations need to be related to an aquifer which is semi-confined by a layer of lower permeability and non-negligible storage capacity. At present, the overburden can only be modelled using leaky aquifer theory once it is fully saturated thus ignoring the storage.

4. Obtain field data from a site at which a flood embankment scheme already exists and monitor groundwater reactions to real flood events. This would provide information on the choice of conceptual model and test the relative importance of the various aspects of the hypothetical schemes considered in this report.

5. Use the field data obtained to revise/refine theory and concepts as necessary.

9 REFERENCES

1. Remson, I, et al. "Numerical Methods in Subsurface Hydrology". Wiley, 1971.

2. Bear, J. "Dynamics of Fluids in Porous Media". Elsevier, 1972.

3. Wang, H, Anderson, M. "Introduction to Groundwater Modelling". Freeman, 1982.

4. Verruit, A. "Groundwater Flow". Macmillan, 1982.

5. Kinzelbach, W. "Groundwater Modelling". Developments in Water Science 25, Elsevier, 1986.

6. Watkins, D. "Evaluation of an Air-entry Permeameter". Report SR 102, Hydraulics Research, 1987.

TABLE.

TABLE 1 Model results presented

Fig No	T	Sy	Se	E	Remarks
3	0.005	0.3		0.024	Standard case
4	0.005	0.2		0.036	
5	0.005	0.1		0.072	
6	0.005		0.1	0.072	
7	0.005		0.01	0.72	
8	0.005		0.001	7.2	Standard case
9	0.05	0.3		0.24	
10	0.0005		0.001	0.72	
11	0.005		0.1	0.72	
12				0.01	
13				0.1	
14				1	
15				10	
16	~0.00003		0.001	0.43	T decreasing
17	~0.00003		0.001	0.43	T increasing
18	0.000501	0.3	0.001	0.0024-0.72	Z2 = 5.01
19		0.3	0.001	0.0024-0.72	Z2 = 5.05
20		0.3	0.001		Unconfined
21		0.3	0.001		Confined
22		0.3	0.001		Semi-confined
23		0.3	0.001		Semi-confined

FIGURES.

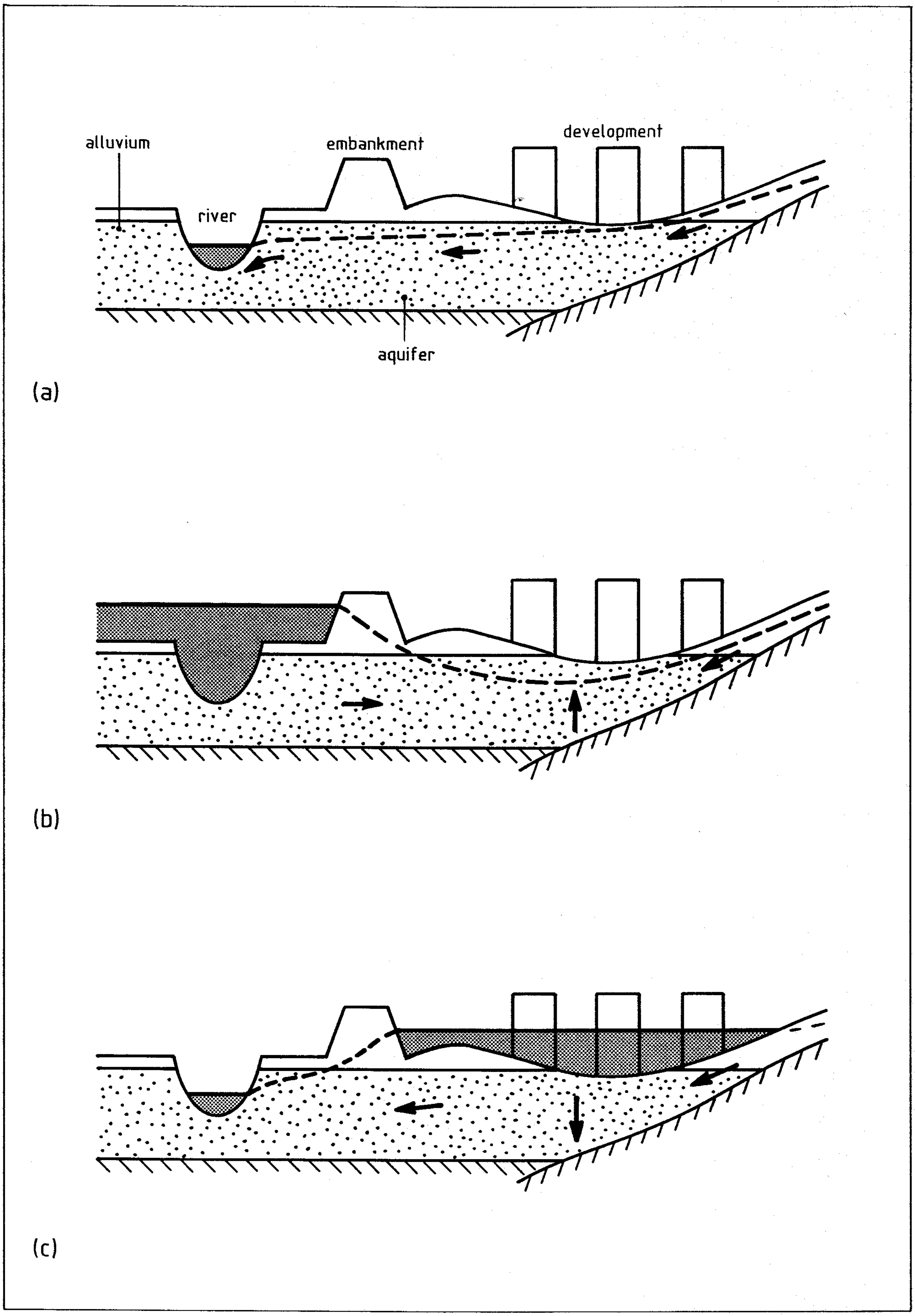

Fig 1 Groundwater flow beneath a flood embankment

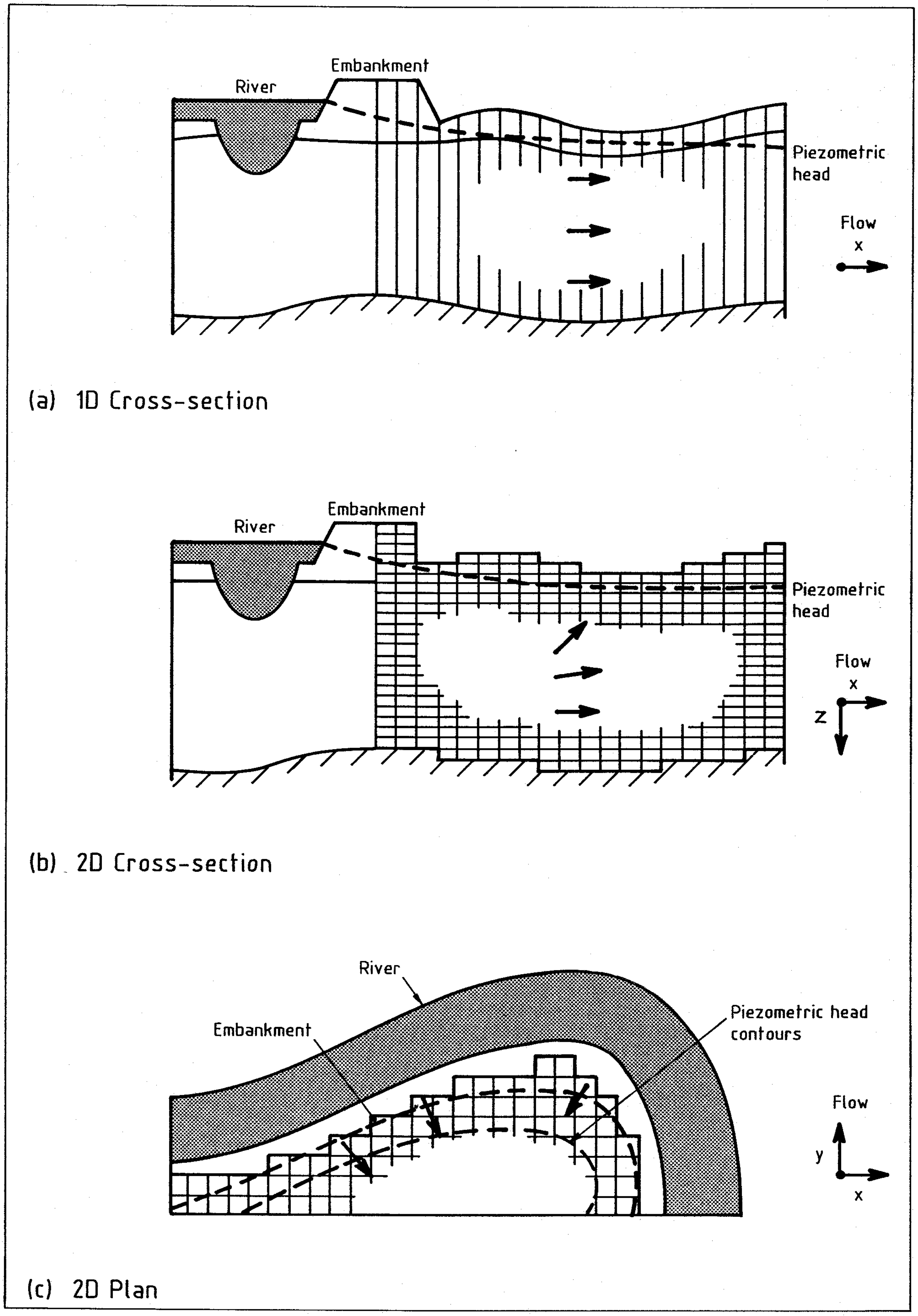

Fig 2 Numerical modelling options

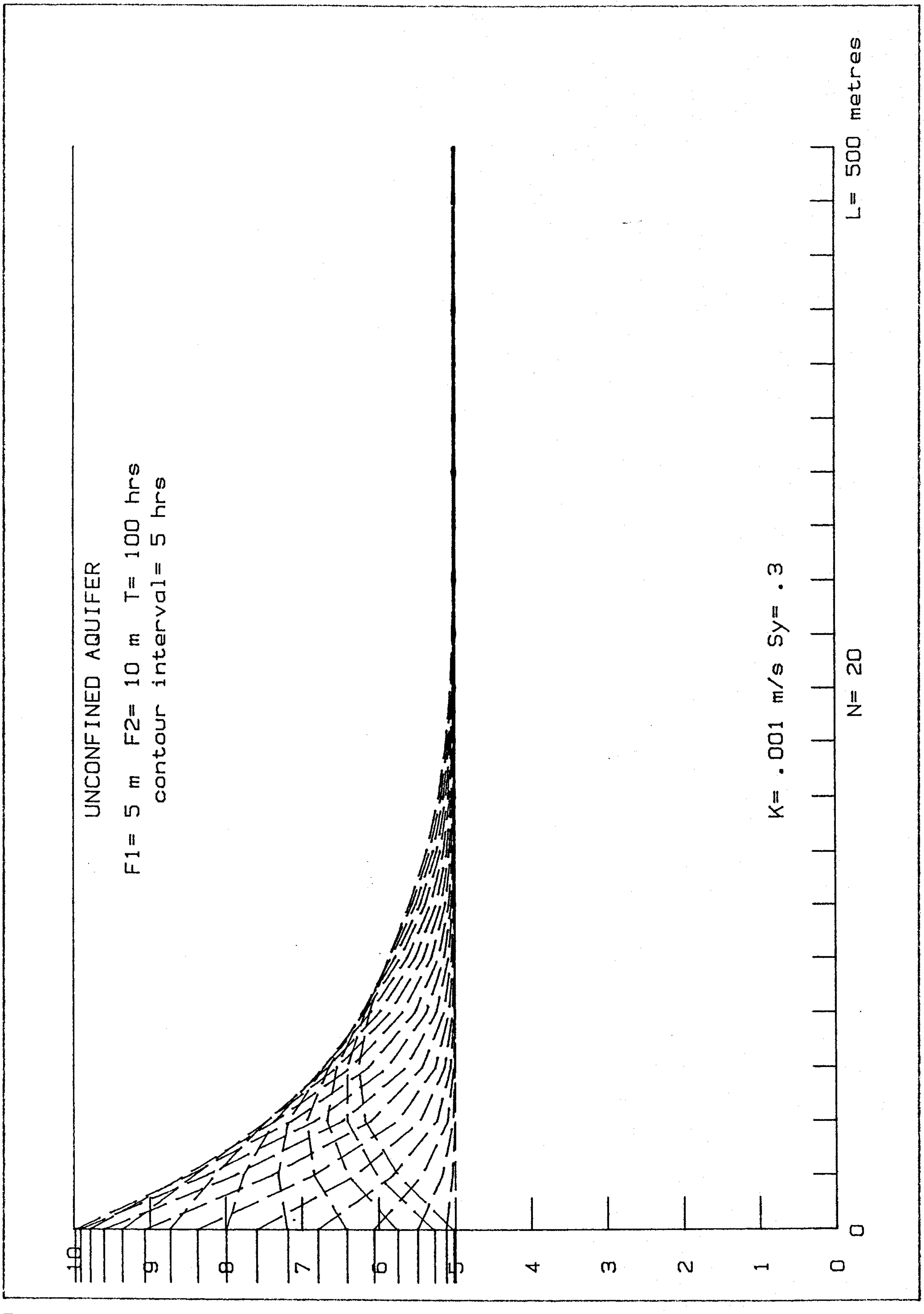

Fig 3 FLOODBANK model results
see text for details

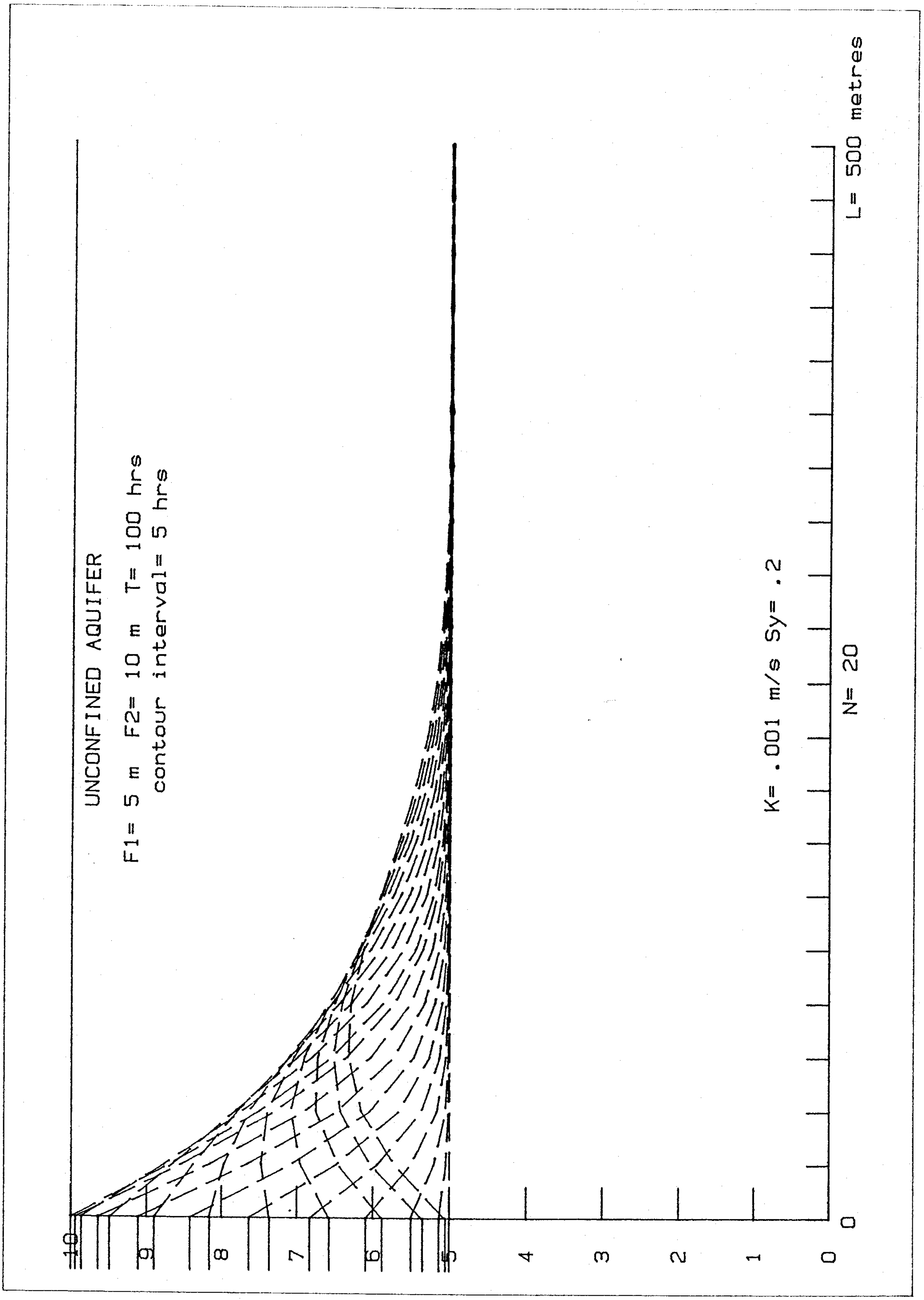

Fig 4 FLOODBANK model results
see text for details

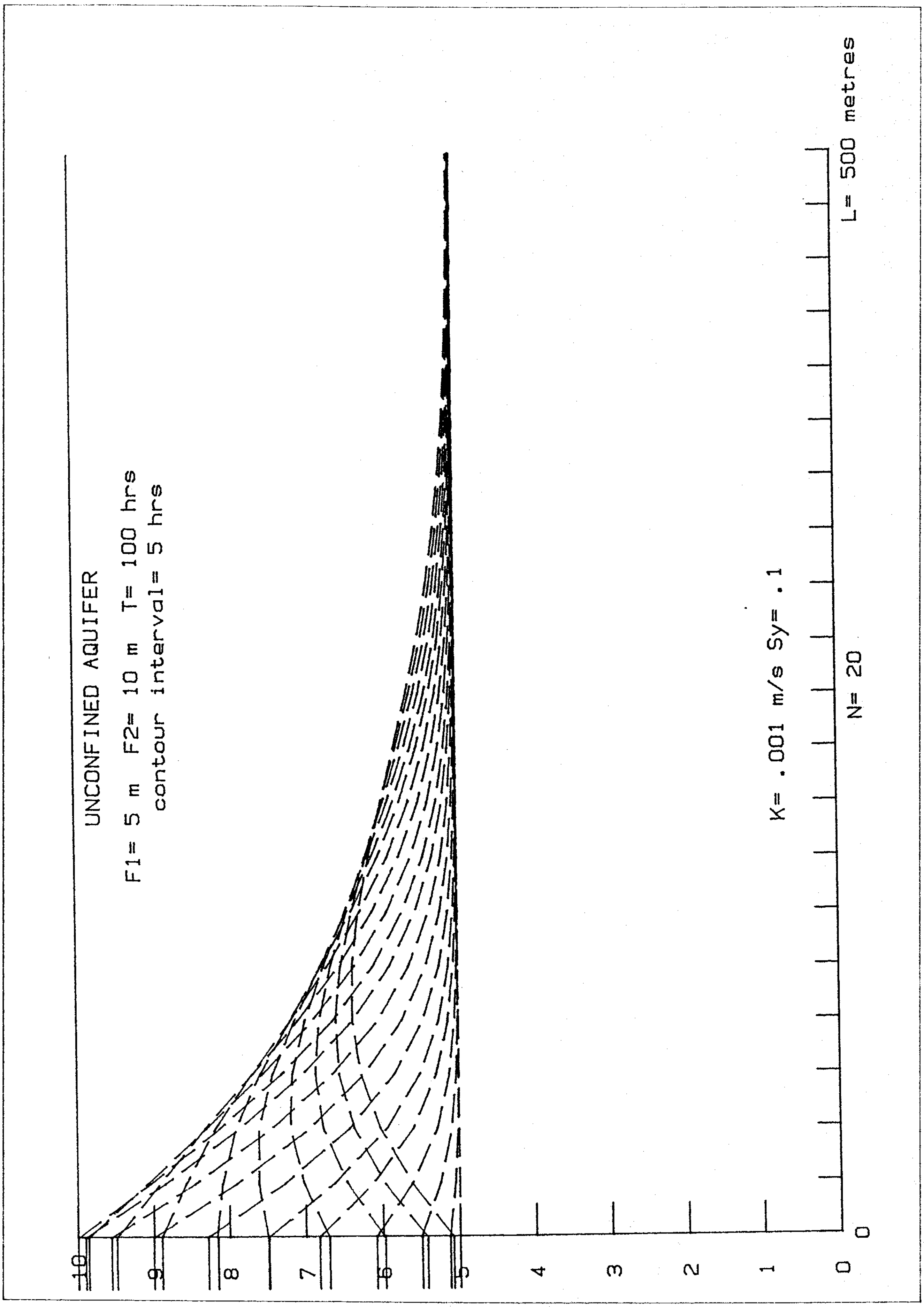

Fig 5 FLOODBANK model results
see text for details

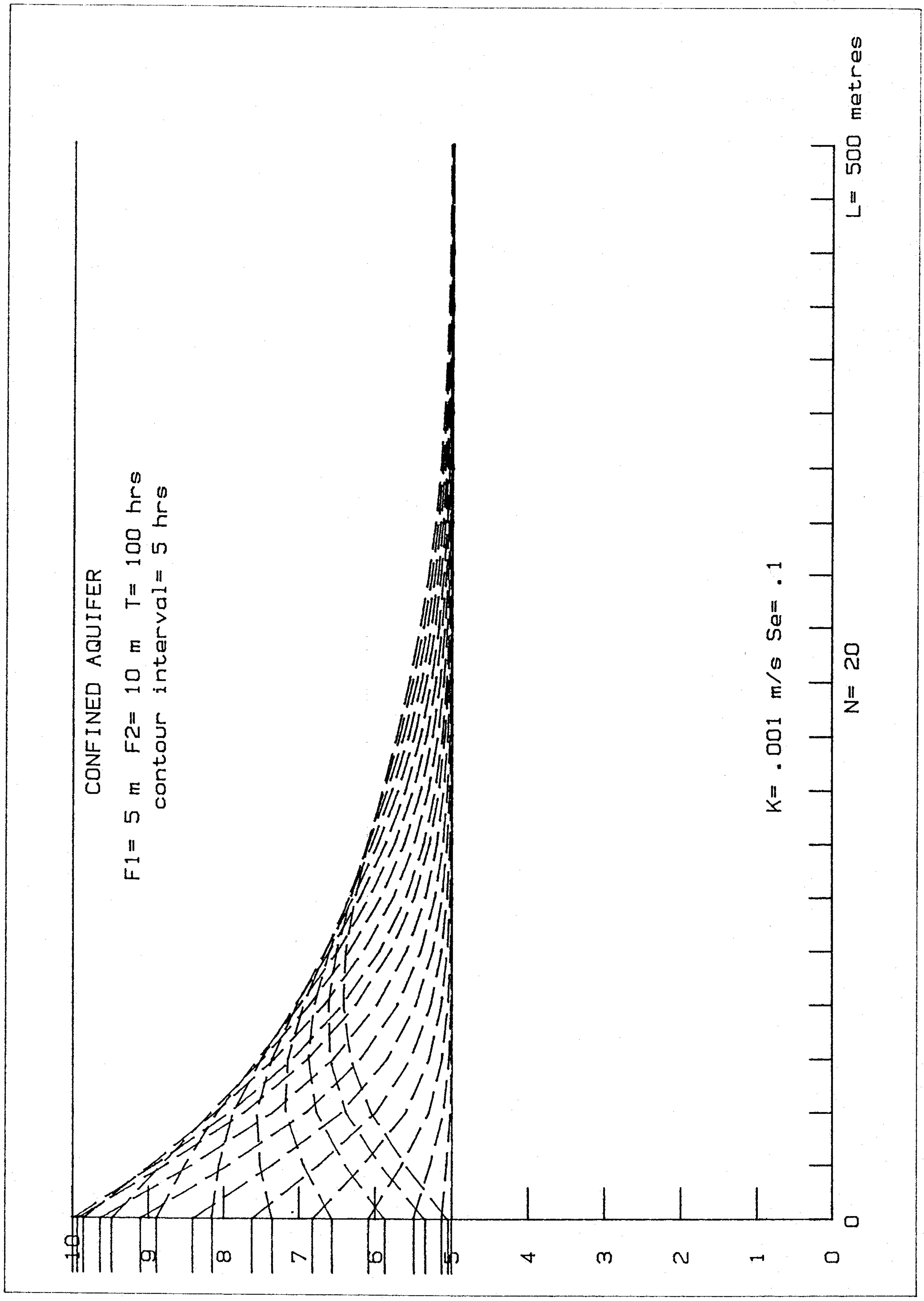

Fig 6 FLOODBANK model results
see text for details

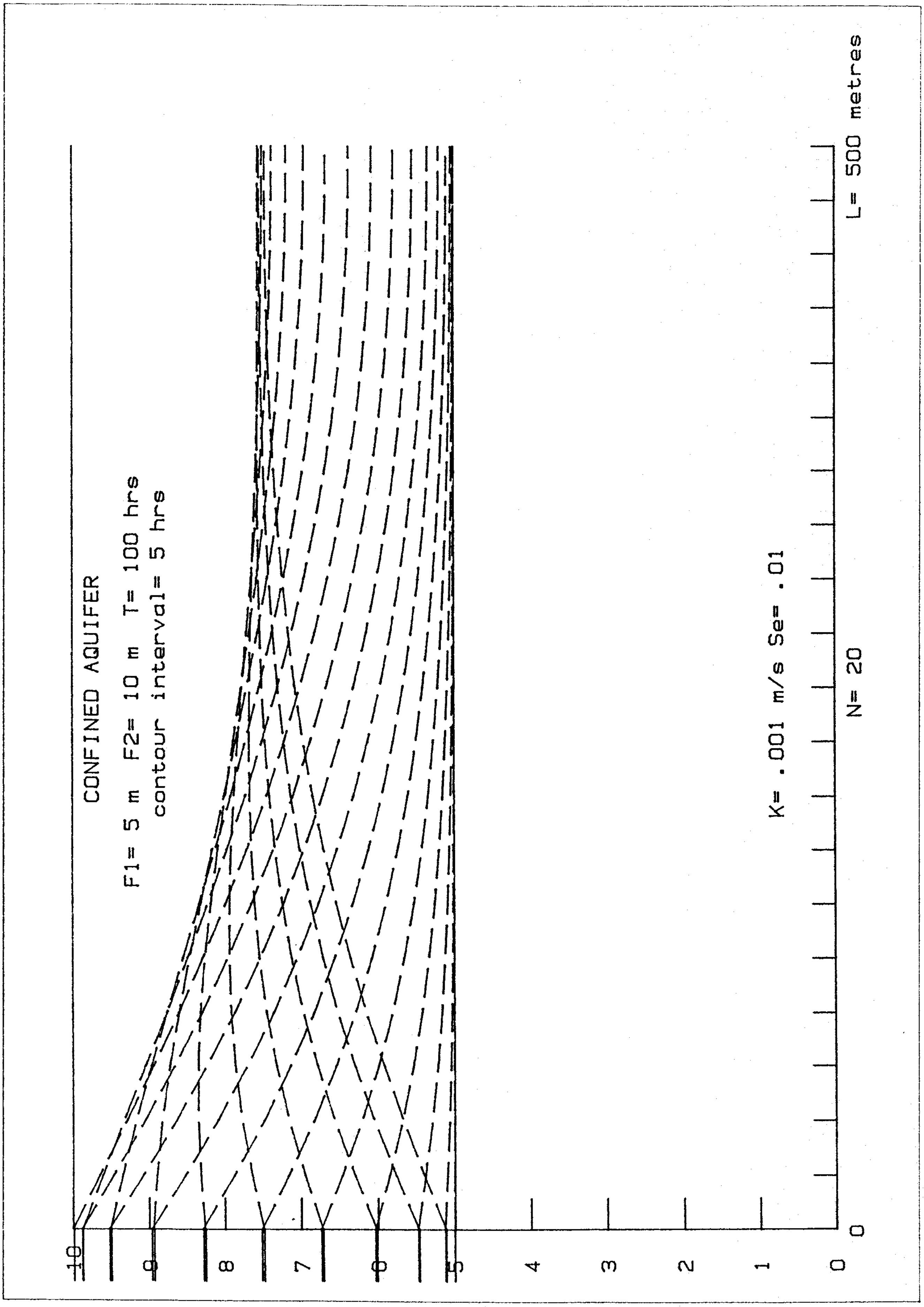

Fig 7 FLOODBANK model results
see text for details

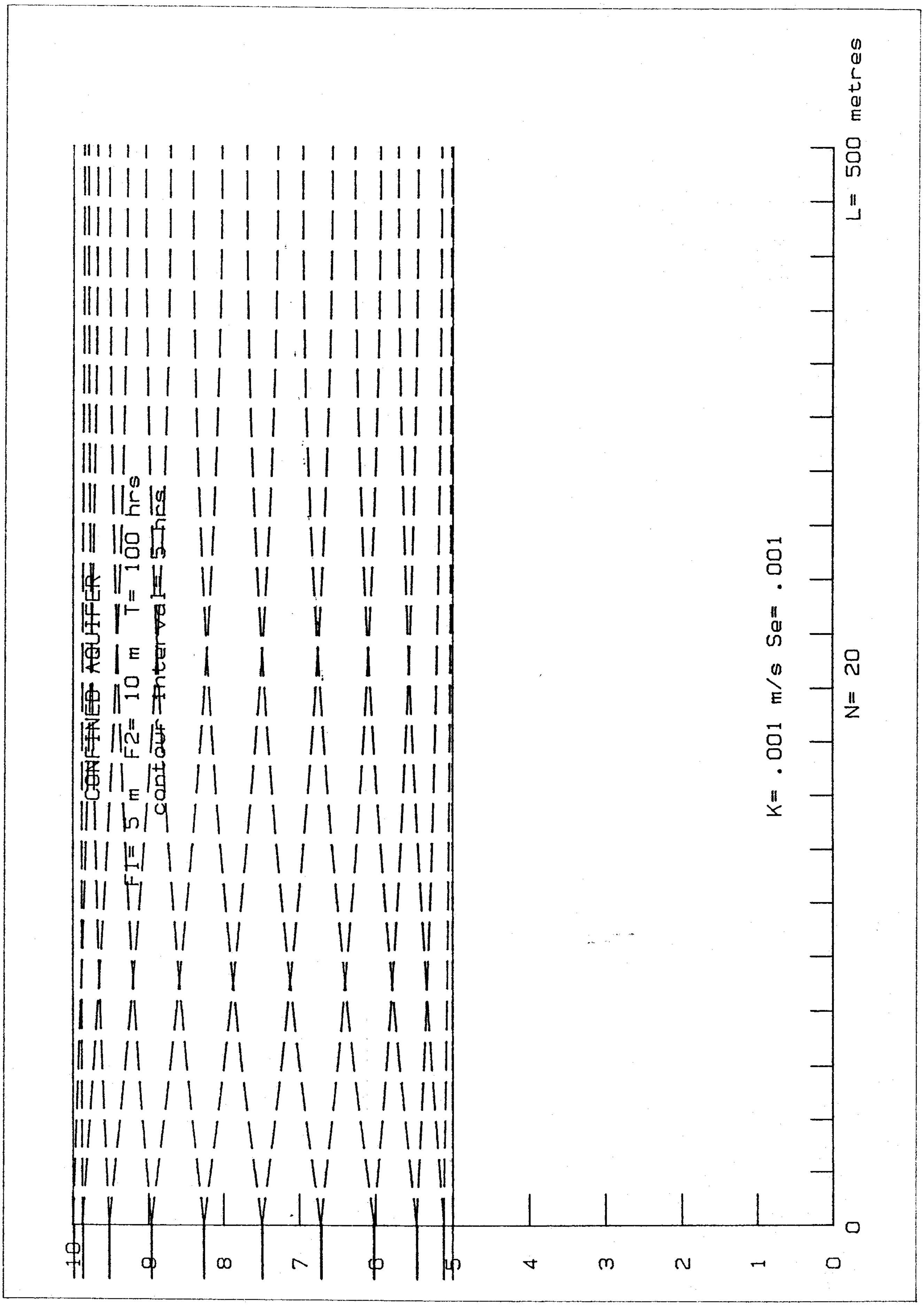

Fig 8 FLOODBANK model results
see text for details

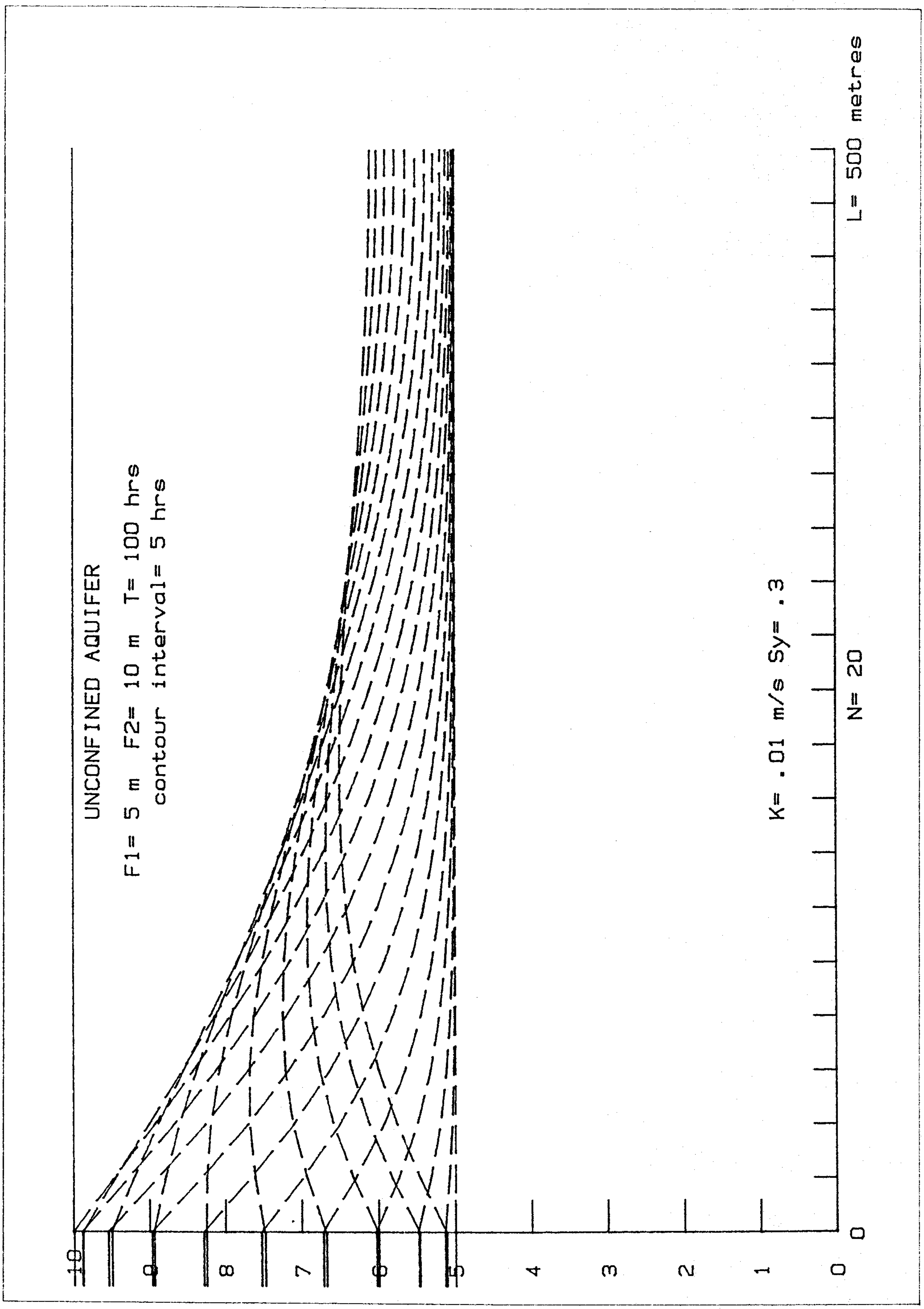

Fig 9 FLOODBANK model results
see text for details

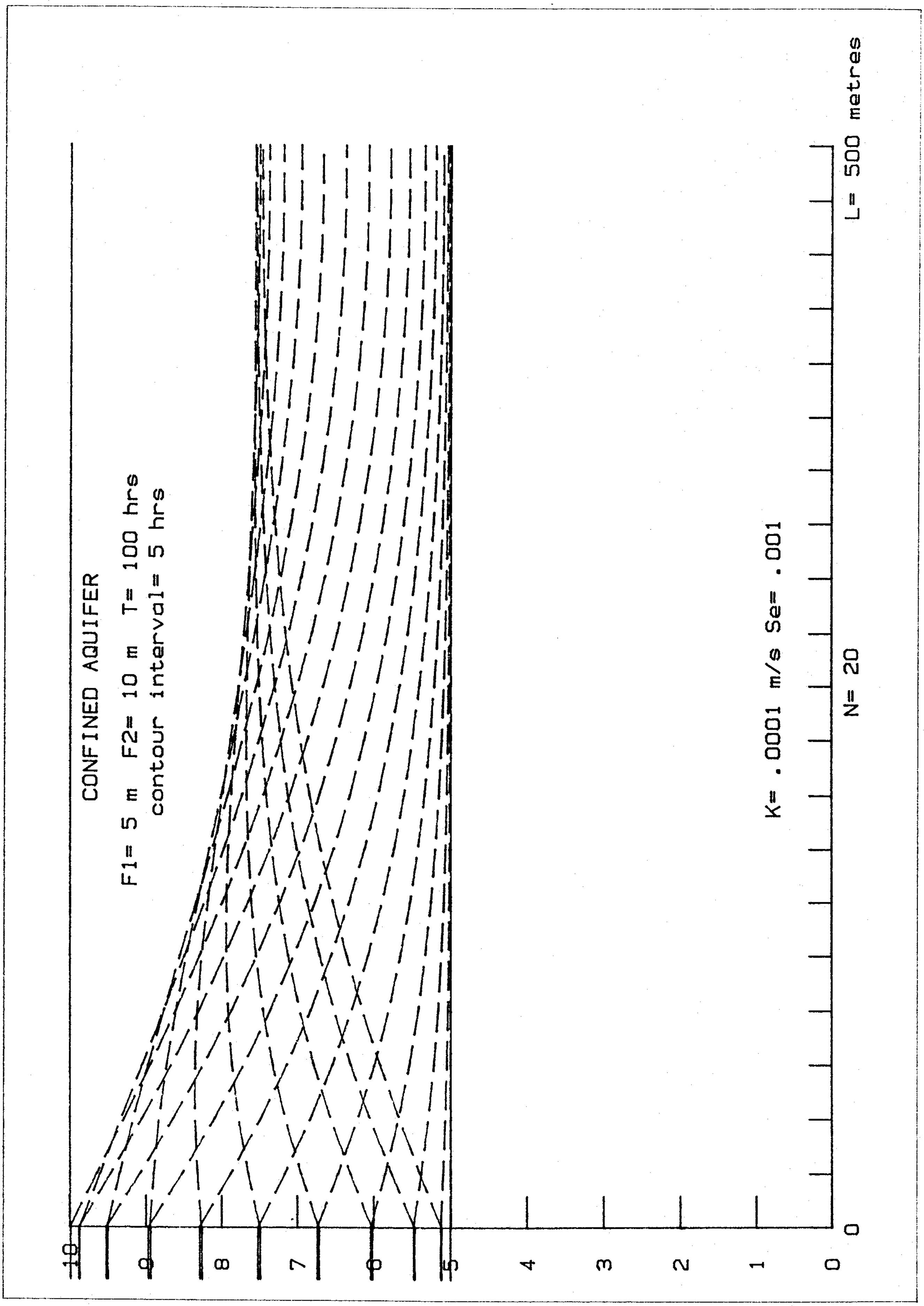

Fig 10 FLOODBANK model results
see text for details

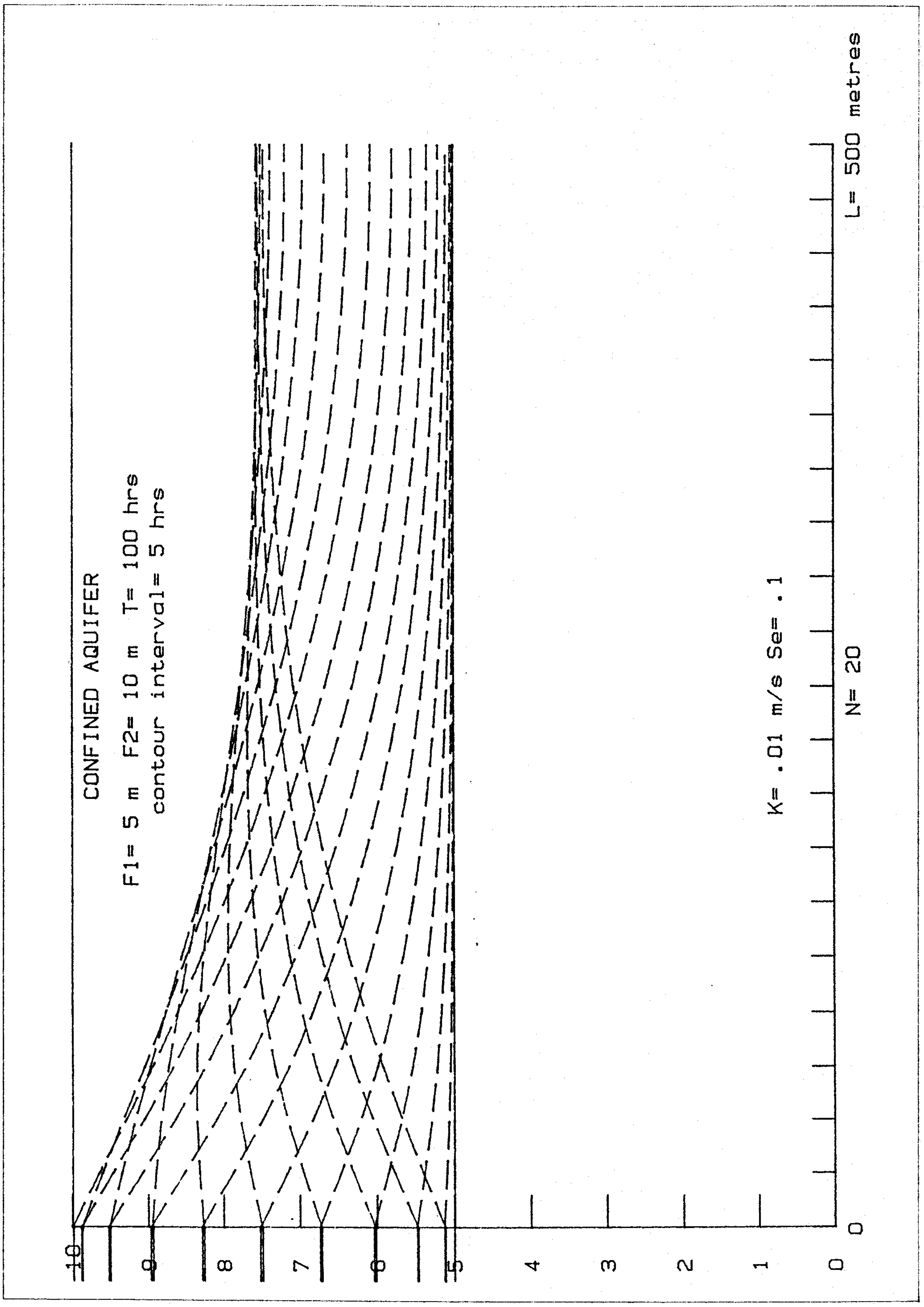

Fig 11 FLOODBANK model results
see text for details

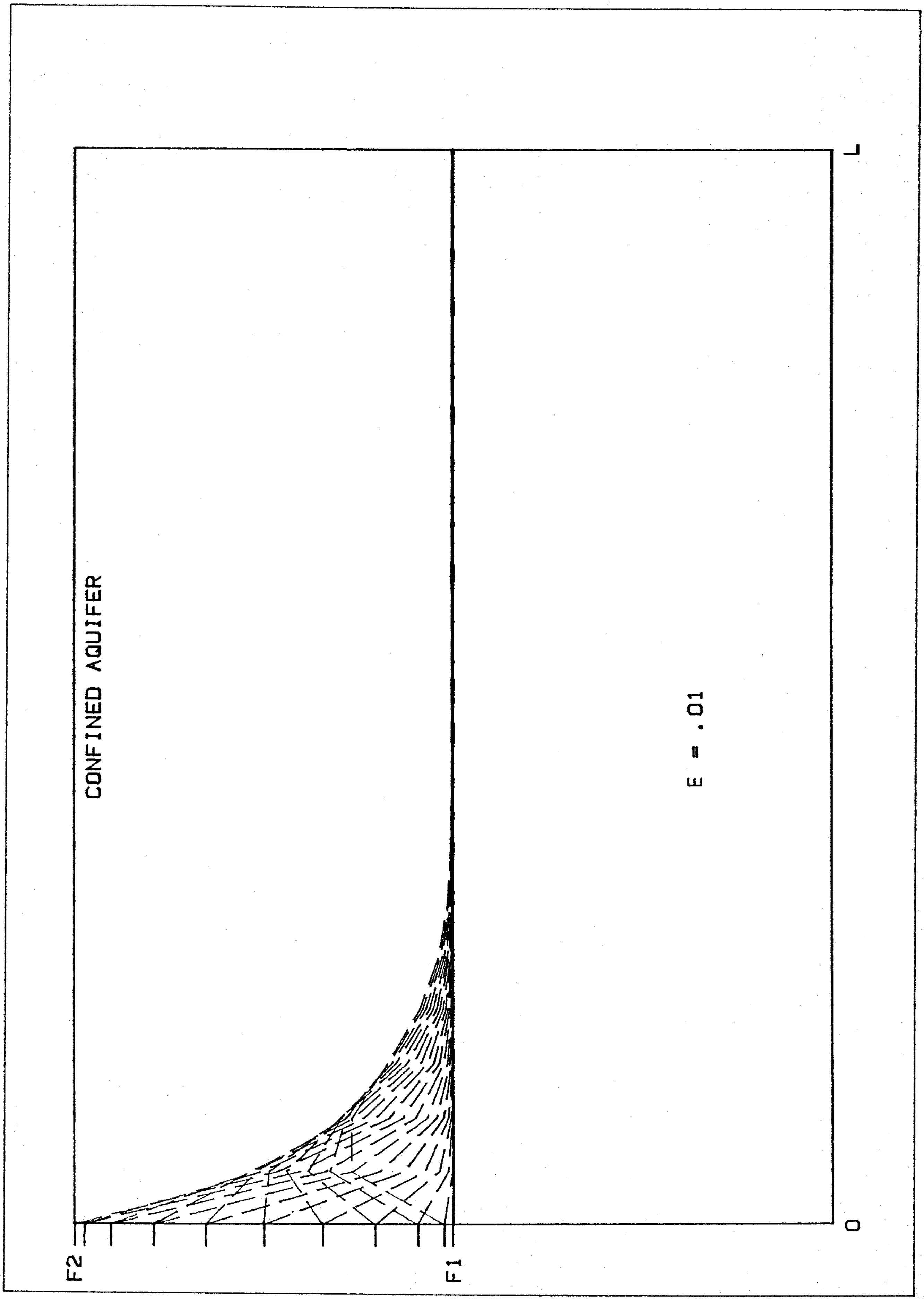

Fig 12 FLOODBANK model results
see text for details

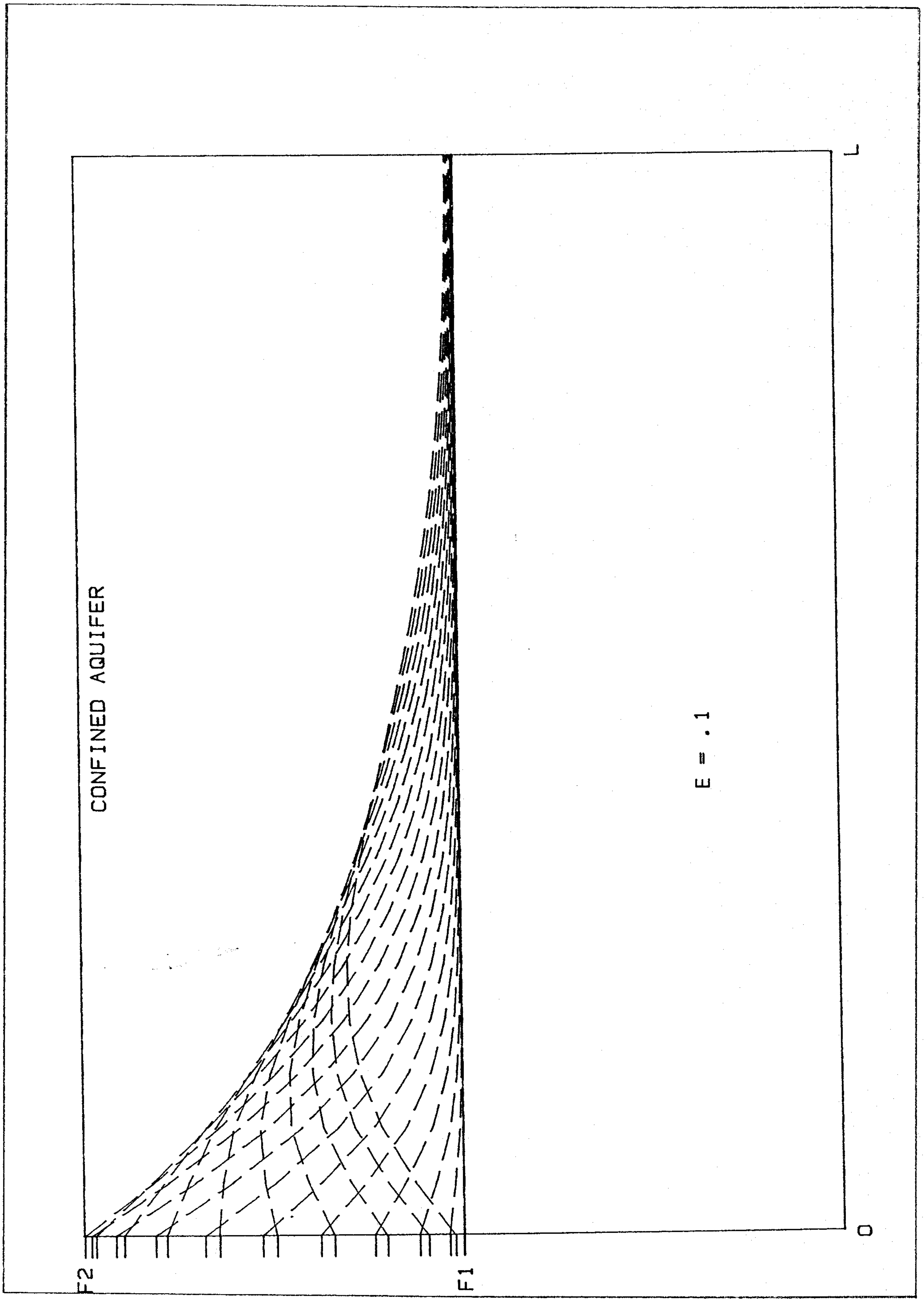

Fig 13 FLOODBANK model results
see text for details

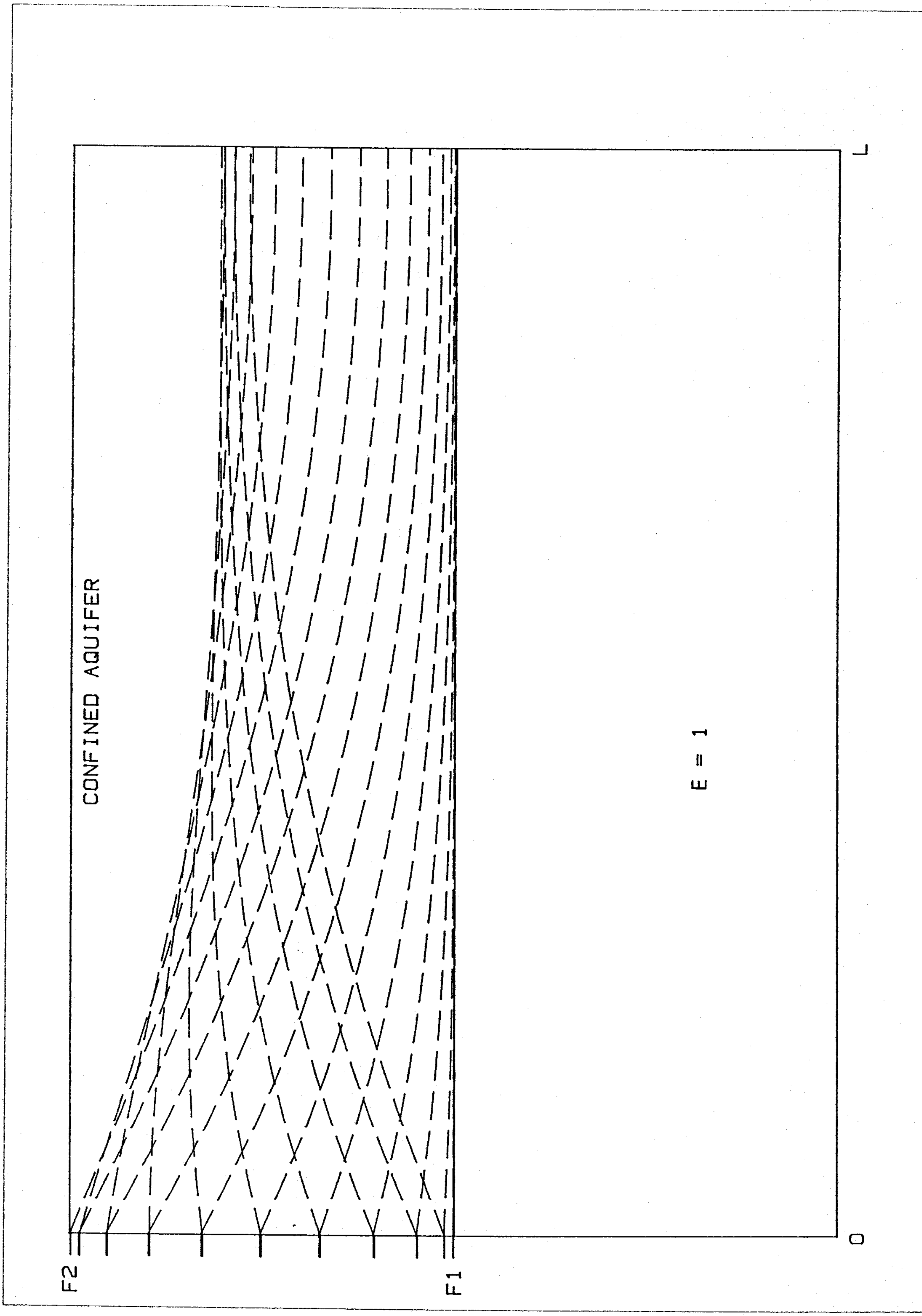

Fig 14 FLOODBANK model results
see text for details

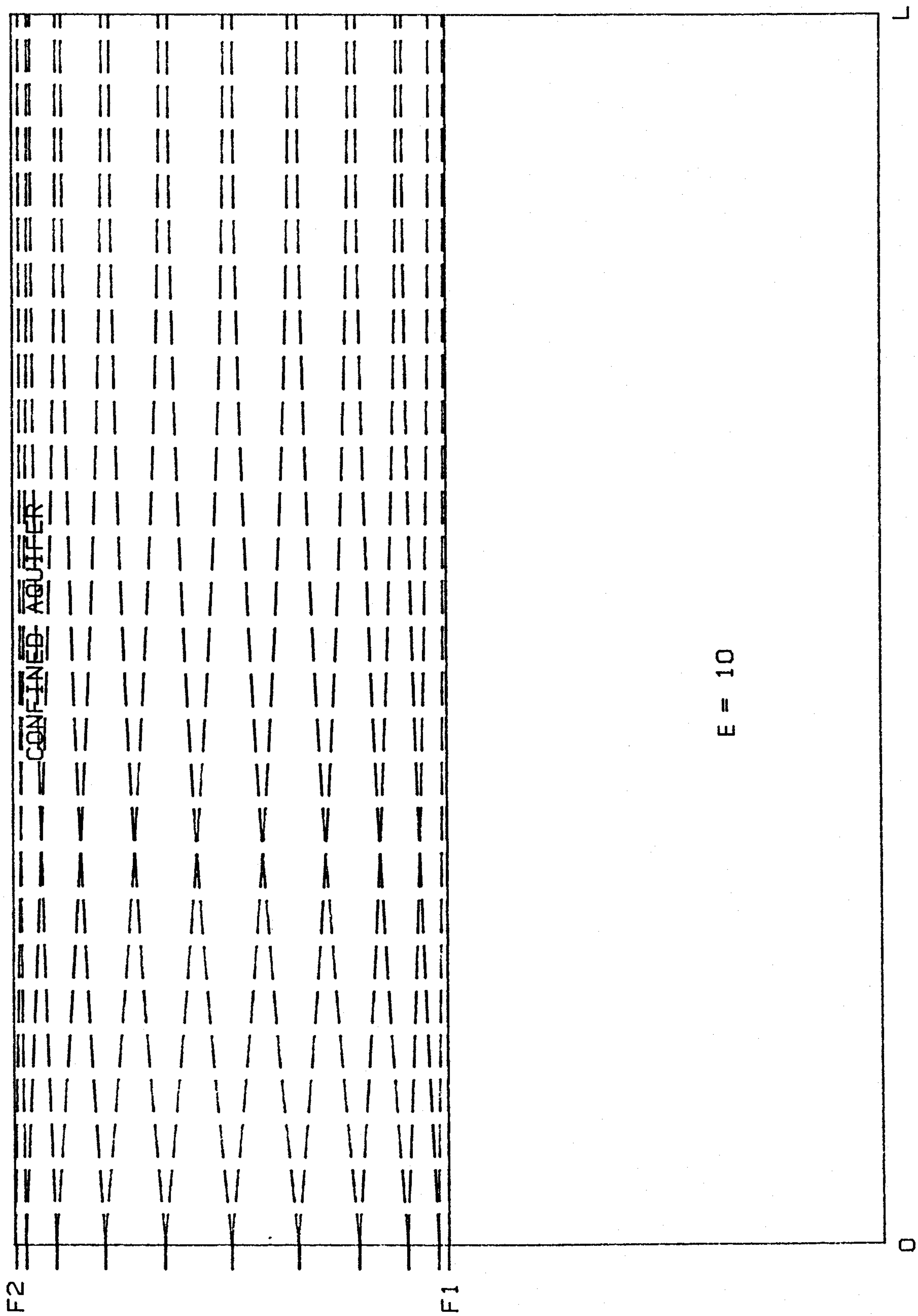

Fig 15 FLOODBANK model results
see text for details

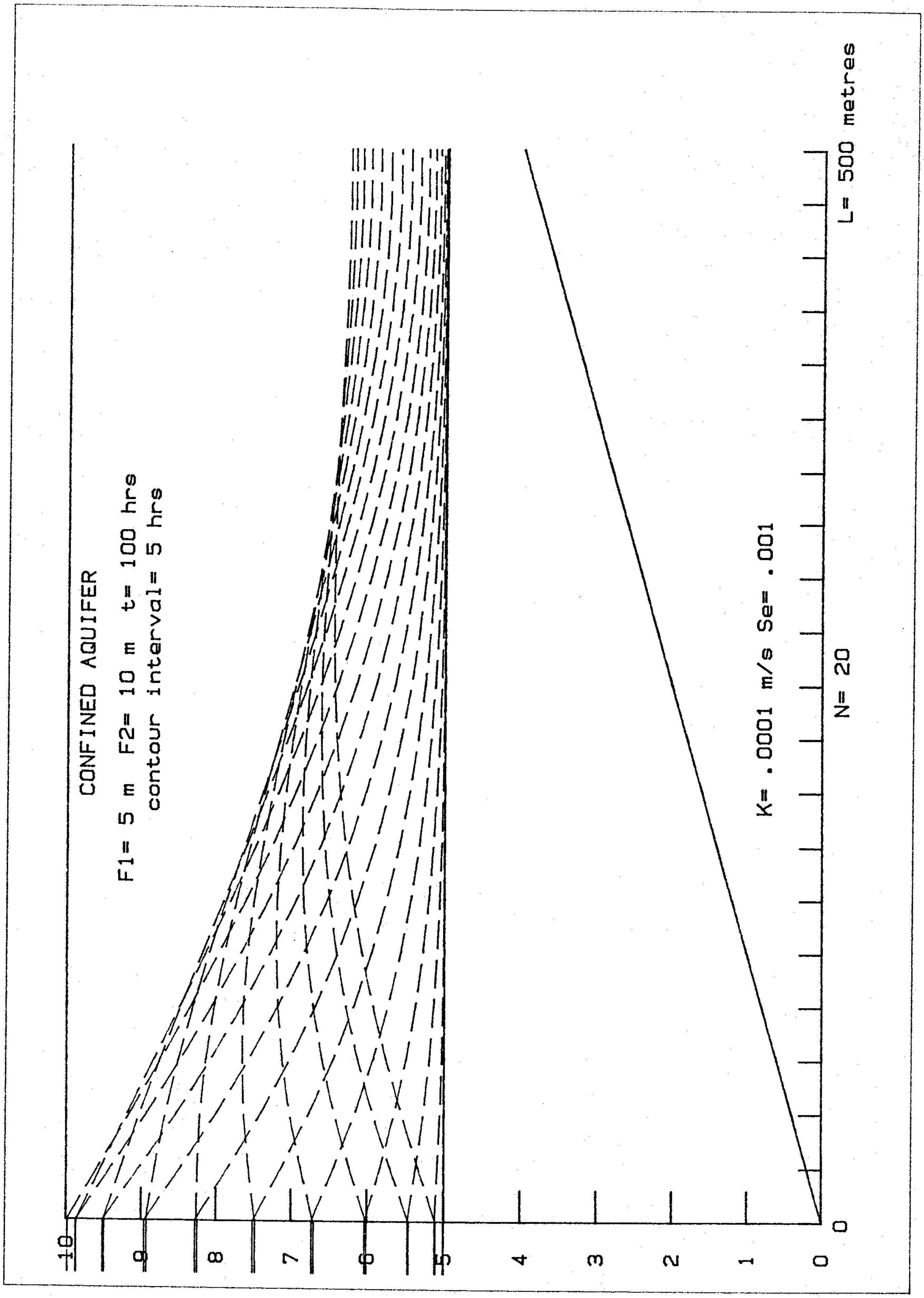

Fig 16 FLOODBANK model results
see text for details

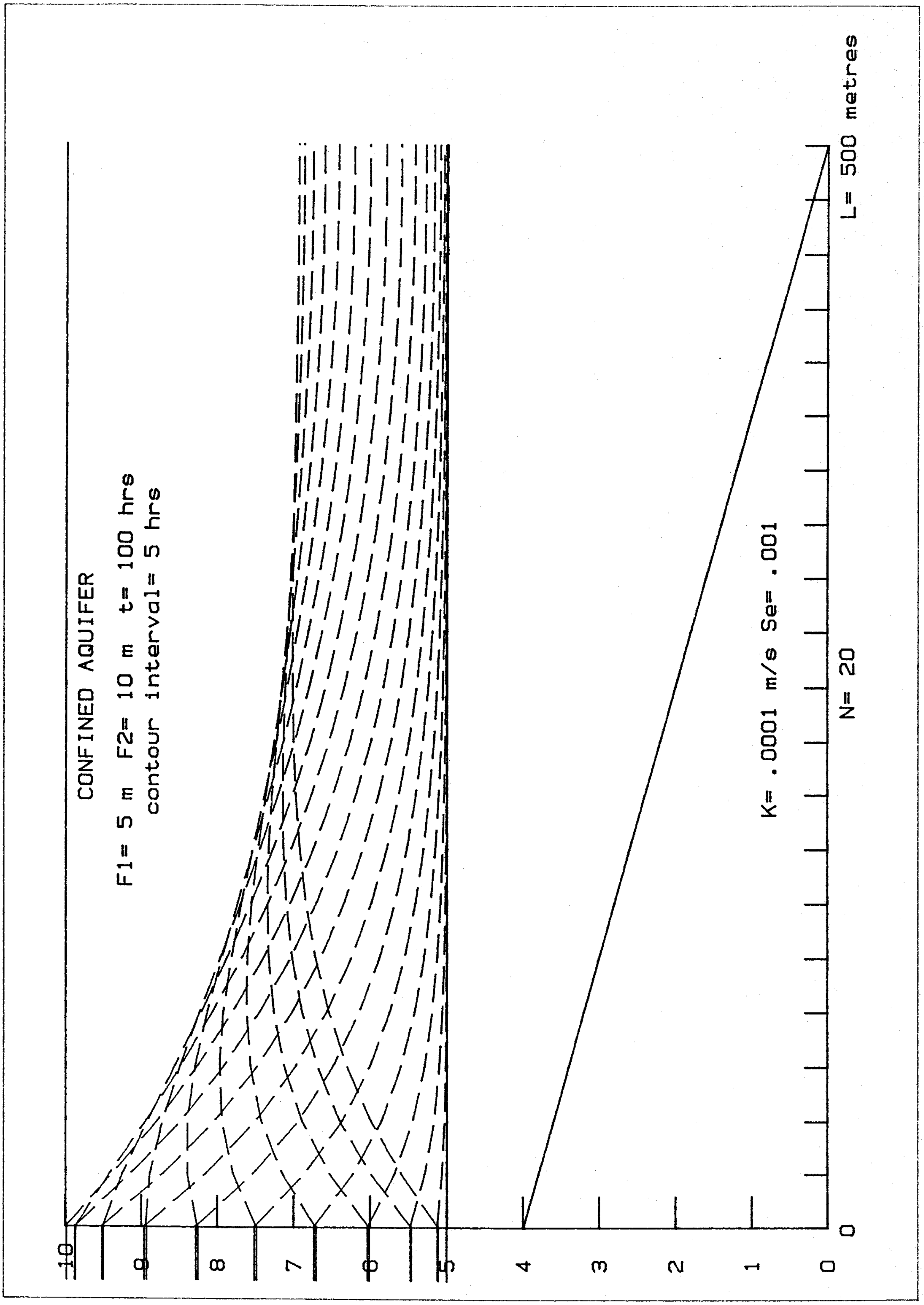

Fig 17 FLOODBANK model results
see text for details

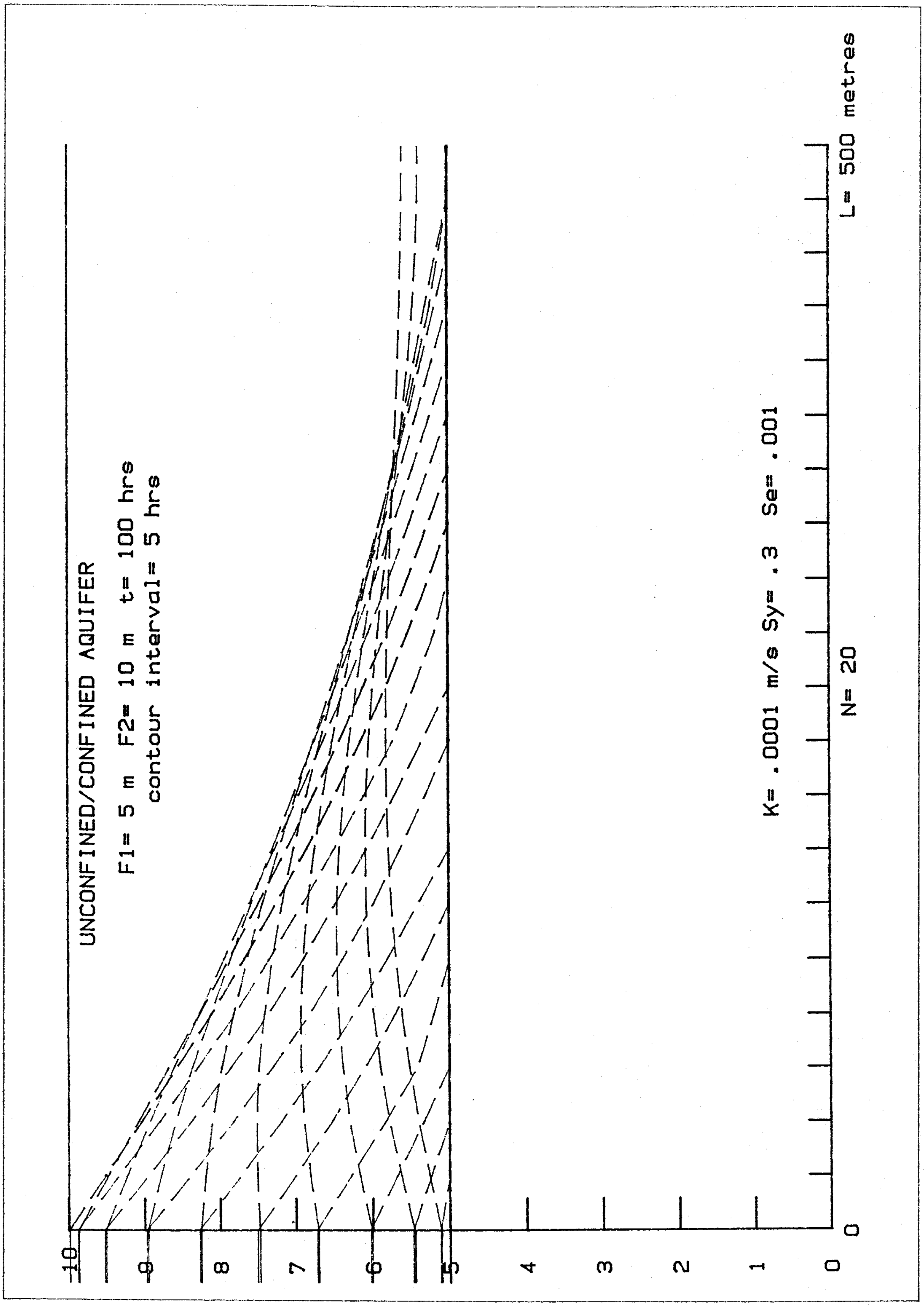

Fig 18 FLOODBANK model results
see text for details

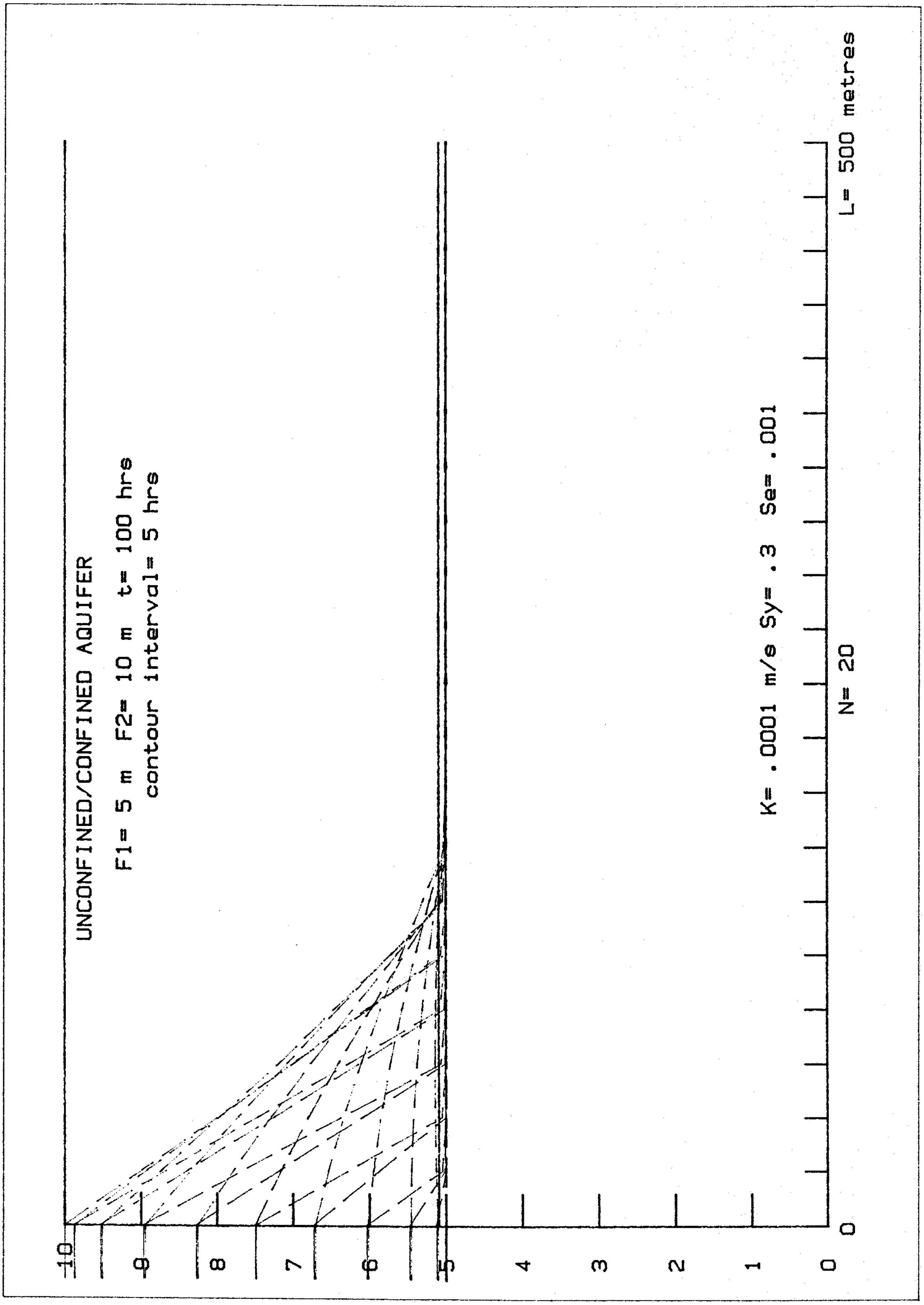

Fig 19 FLOODBANK model results
see text for details

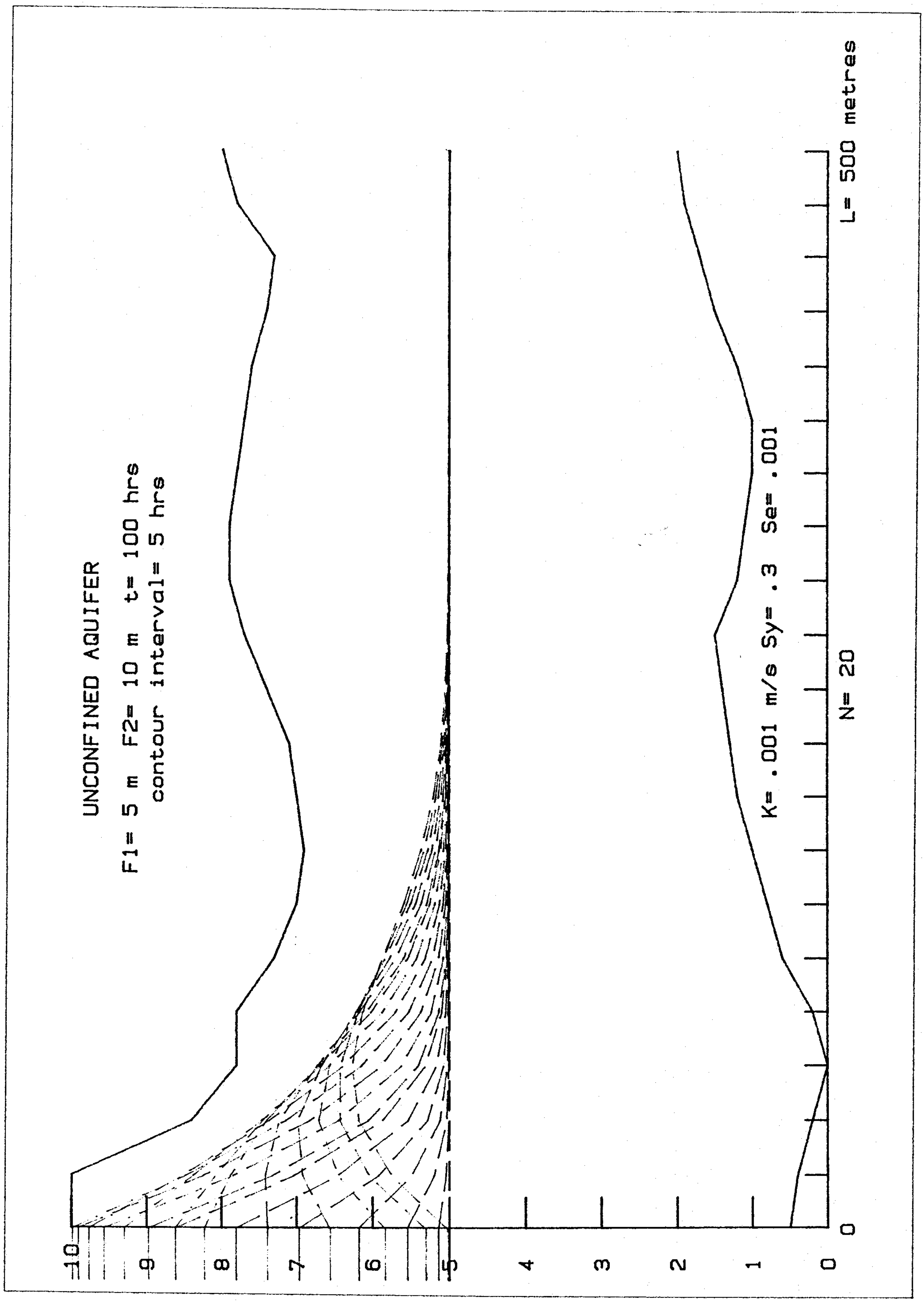

Fig 20 FLOODBANK model results
see text for details

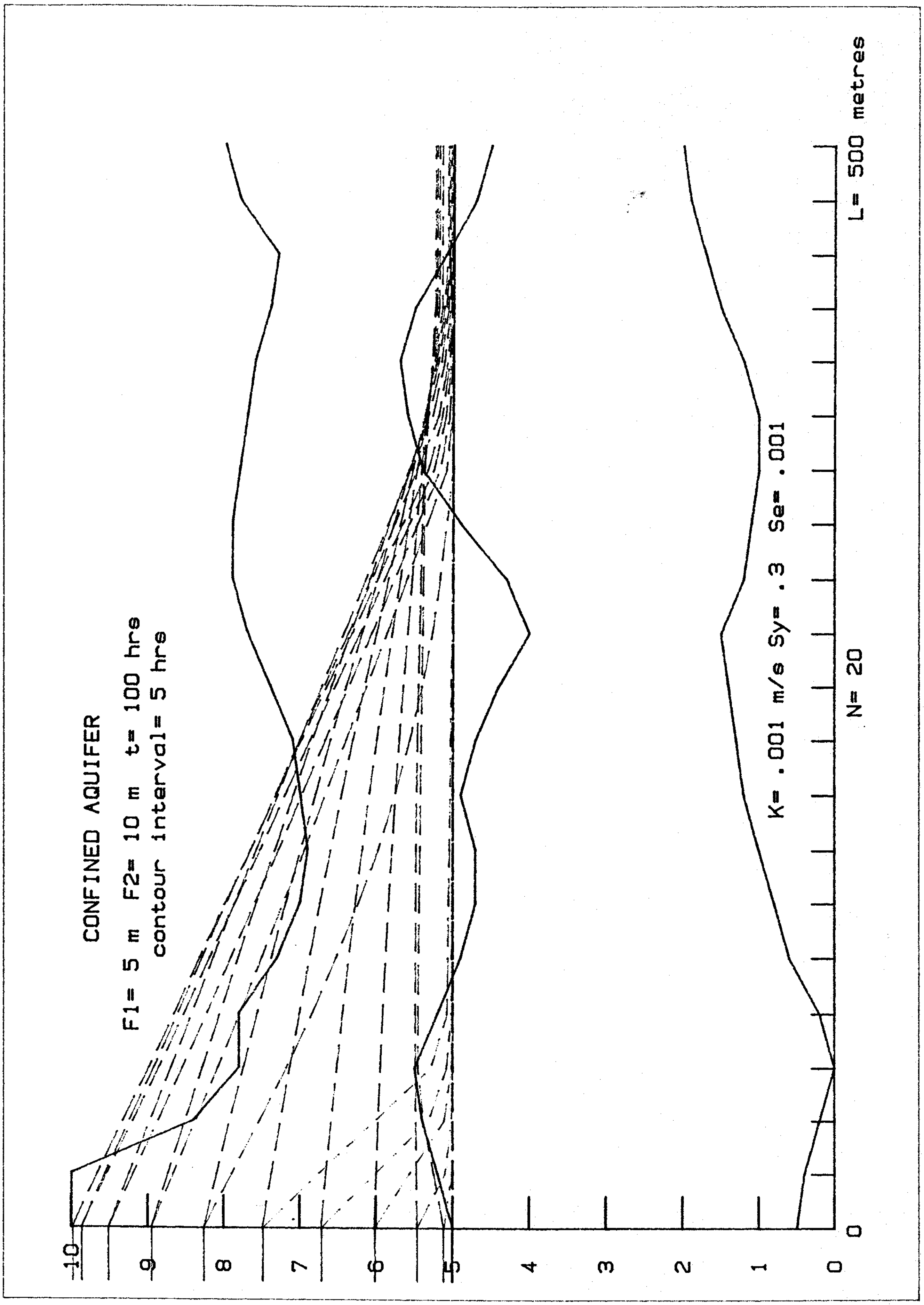

Fig 21 FLOODBANK model results
see text for details

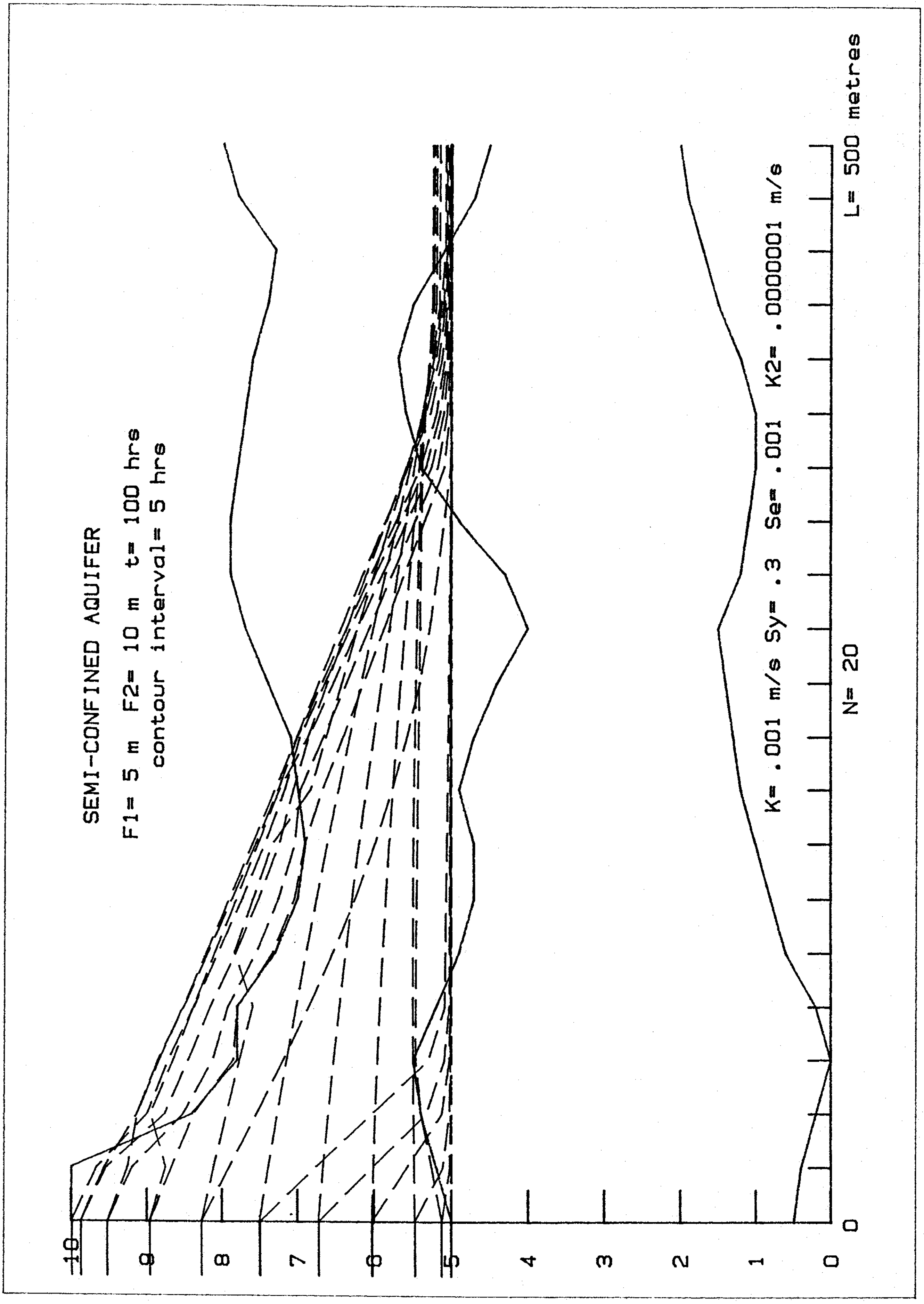

Fig 22 FLOODBANK model results
see text for details

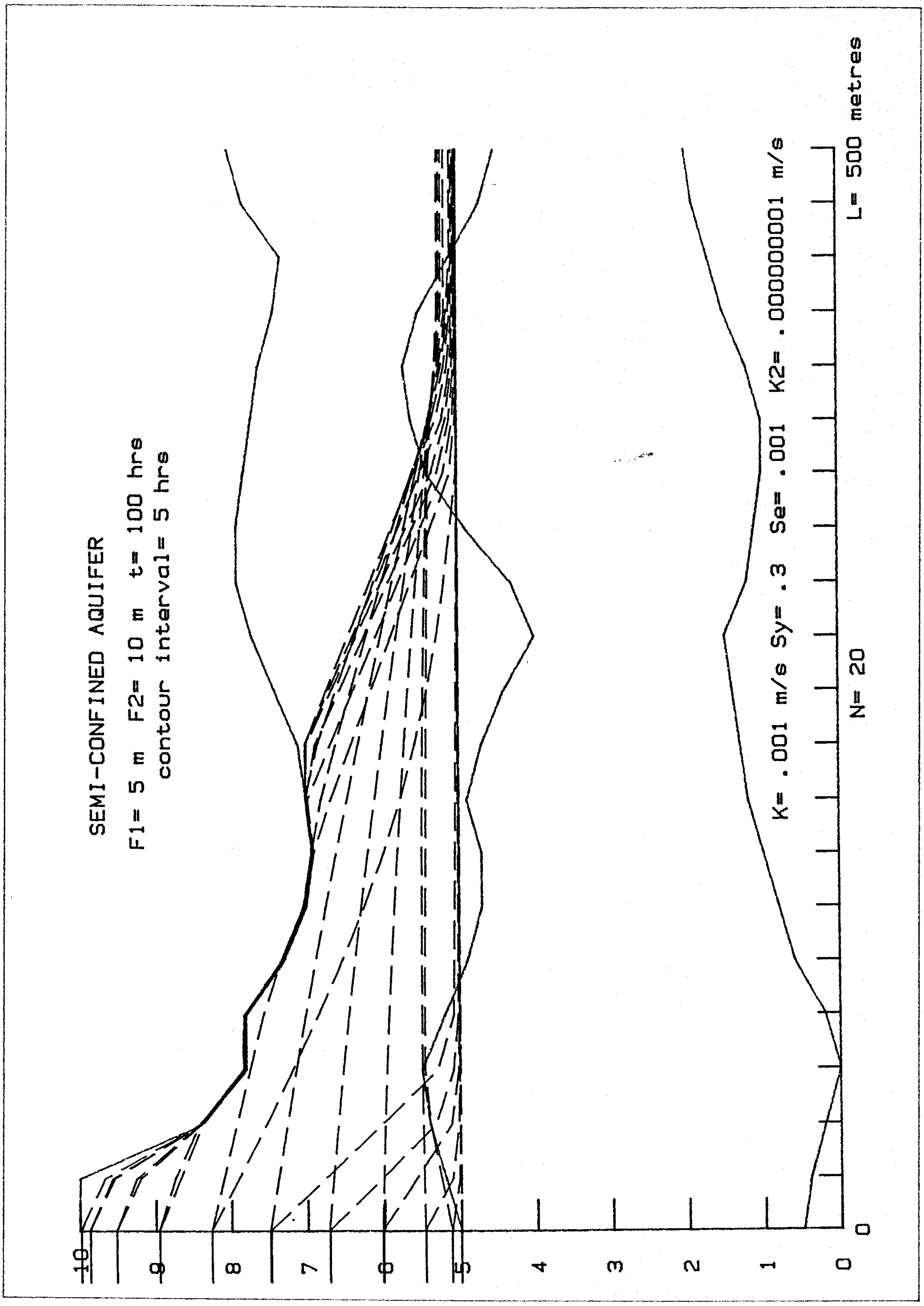

Fig 23 FLOODBANK model results
see text for details