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INSTITUTE of HYDROLOGY

MUQDISHO WATER SUPPLY

EXPANSION

STAGE IIA

EXPLORATION AND MODELLING

STUDIES

VOLUME II

This report is prepared for

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SECTION 3

MODELLING STUDIES

6. REGIONAL RECHARGE MODEL

6.1 INTRODUCTION TO MODELLING WORK

In order to estimate the available groundwater resource the hydrological characteristics of the Muqdisho area have been brought together within the framework of mathematical models. These are used as a tool to analyse the aquifer under natural conditions and to make a quantitative assessment of the impact of groundwater abstraction.

It is not possible to obtain accurate field estimates of aquifer recharge. However the river Shebeelle has been shown to be the only source of recharge and areas of preferential recharge along this river have been identified. In order to quantify the recharge at these locations a mathematical model which indirectly calculates recharge from transmissivity and hydraulic head is utilized. The hydraulic heads used in the modelling studies are those which relate to the main water table; the influence of perched water levels has been ignored. The recharge models are used to analyse aquifer behaviour prior to any large scale abstractions. A steady state formulation is used since under these conditions it is reasonable to assume that groundwater levels are not changing rapidly, changes in storage are very small and flows through the aquifer are constant. In this chapter the model is used to examine recharge in a regional context.

This is done since the major recharge sources are beyond the area within which wellfield location is viable. In Chapter 7 recharge estimates are obtained using the same grid as is subsequently used for the water management model. This grid is finer than that used in the regional recharge model since within the water management model steeper hydraulic gradients are encountered and hence a finer grid is required to represent these adequately. The recharge estimates incorporated in the management model should be those associated with this finer grid since the recharges are partly dependent on the transmissivity distribution which is given at the grid nodes.

In Chapter 8 the results of the regional and study area recharge models are used to develop a water management model. This is a time varying model which is used to examine the changes in storage and the the development of cones of depression associated with various abstraction regimes. In order to avoid contamination of the groundwater resource water table constraints have to be developed and using these certain abstraction regimes can be rejected as being unacceptable. Previous experience has shown that the introduction of large quantities of water into a non or partly sewered urban area such as Muqdisho will result in substantial local recharge. As a consequence a groundwater mound will develop which in turn will restrict saline intrusion. These effects are investigated by using the water management model. The model is also used to examine the consequences of a reduction in recharge from the river Shebeelle.

6.2 RECHARGE MODEL: THEORETICAL BASIS

The steady state flow of groundwater within an unconfined, inhomogeneous, isotropic aquifer, subject to suitable boundary conditions is described by:

$$\underline{\nabla} \cdot (T\underline{\nabla\phi}) = \underline{r} \qquad (1)$$

where: $\underline{\nabla} = (\frac{\partial}{\partial x}, \frac{\partial}{\partial y})$ is the differential operator

$T(x,y)$ is the spatial distribution of transmissivity (m^2/day)
$\underline{\phi}(x,y)$ is the hydraulic head (m)
$r(x,y)$ is the recharge or abstraction (m^3/day)
$r<0$ representing recharge

In order to obtain equation (1) certain assumptions have been made about the flow:

(a) that vertical flow is negligible compared to horizontal flow. In the area being modelled this is reasonable since the aquifer thickness is approximately 170 m compared to a horizontal extent of 80 to 130 km.

(b) that the transmissivity can be considered independent of the saturated thickness. Again this is a reasonable assumption since the maximum water level fluctuations are expected to be less than 10% of the saturated thickness.

The boundary conditions which have been applied are of two types. Along the coast we have assumed that hydraulic heads are constrained to be zero (i.e. at mean sea level). The remaining boundaries have been treated mathematically as no flow boundaries but with recharge allowed at the nodes adjacent to these boundaries. The 'recharges' at these boundary nodes consist of two components - a component of boundary flux and a component of recharge or abstraction; as a result of the two dimensional formulation there is no mathematical difference within the model between these components.

The mathematical basis for the method which has been used for the estimation of the recharge within the modelled area has been fully described by Smith and Wikramaratna (1981).* Below is a summary of the most important points concerning the method.

Equation (1) together with appropriate boundary conditions may be solved numerically using the finite difference method. This leads to a set of difference equations of the form

$$A\underline{\phi} = \underline{r} \qquad (2)$$

where A is a n*n matrix (n is the number of nodes or grid points) of coefficients involving the transmissivity and the boundary conditions, $\underline{\phi}$ is an n-vector of the hydraulic head at the nodes and $\underline{r}$ is a vector representing the recharge or abstraction. The vector $\underline{r}$ is assumed to consist of two parts; $\underline{b}$ is that part of the recharge or abstraction which is known while $\underline{q}$ represents the parts to be estimated. The inferred recharge method has been used to estimate $\underline{q}$. This technique depends on the unknown component of recharge being restricted to certain known areas and being constrained to be equal to zero

*Smith, P.J. and R.S. Wikramaratna (1981): A method for estimating recharge and boundary flux from groundwater level observations. Hydrological Sciences Bulletin 26(2), 113-136.

elsewhere. The nodes at which components of q are to be estimated are known as inferred recharge nodes. In order to minimise the model's sensitivity to errors in the observed data the number of inferred recharge nodes (m) needs to be significantly less than the number of grid points. The inferred recharge method consists of finding the vector q which is constrained to have (n-m) of its components equal to zero which minimises the sum of squared errors between the observed hydraulic heads at the nodes (h_i) and the predicted heads at the nodes (ϕ_i); thus

$$S(q) = \sum_{i=1}^{n} (h_i - \phi_i)^2 \qquad (3)$$

The inferred recharge $\hat{\underline{q}}$ is the value of $\underline{q}$ which minimises S(q).

During the preliminary modelling work the inferred recharge model described above was utilised. During this work a problem was encountered as the water level elevations are only available at data points and in general do not correspond with the nodes of the model. In order to obtain the water level elevations at the nodes, subjective interpolation is carried out on the data point values. There is no way in which the error involved in this interpolation can be quantified. In an attempt to overcome this problem, the inferred recharge model was modified so that the recharges were found which minimised the sum of squared errors between the observed heads at the data points and the model predicted heads at the data points. The model predicted heads at the data points were calculated by bi-linear interpolation from the predicted heads at the four adjacent grid points. The modified inferred recharge model requires the number of inferred recharge nodes to be significantly less than the number of data points in order to minimise its sensitivity to observational errors.

Whether water level fitting is carried out at the nodes or the data points, the equation that has to be solved to find the inferred recharges is of the same form

$$G\hat{q} = d \qquad (4)$$

where G and d are known matrices which depend upon the matrix of finite difference coefficients, any fixed abstractions, the observed heads at the data points or the model nodes, and where relevant the interpolation matrix from the predicted heads at the nodes to the predicted heads at the data points. The solution of equation (4) is found by using the numerical techniques of successive over relaxation and Gaussian elimination (Smith 1978)*.

The root mean square error is used as a quantitative measure of the quality of the fit between the observed water levels and those predicted by the model. The root mean square error is the sum of squared errors corrected for the number of estimated recharge parameters. For the inferred recharge method this can be expressed as

$$rms = (S(\hat{q})/(n-m)) \tag{5}$$

When water table fitting is carried out at the data points, rather than the model nodes, the correction factor in equation (5) is the number of data points, less the number of inferred recharge nodes rather than the number of grid points, less the number of inferred recharge nodes. It must be stressed that the root mean square error should not be the only measure of goodness of fit. It is very important that there should be a qualitative agreement between the water table configuration predicted by the model and that observed.

6.3 REGIONAL AREA FRAMEWORK

The boundaries of the regional model have been chosen to incorporate all of the major recharge locations that we have identified earlier. Consequently the recharge model is initially applied to an area much greater in extent than the study area. The relationship between the two modelled areas is illustrated in Plate 1.

*Smith, G.D. (1978): Numerical Solution of Partial Differential Equations: Finite Difference Methods. Oxford University Press. 304 pp.

The southern boundary of the regional area coincides with the coast and is taken as a zero head boundary. The location of the other three boundaries is somewhat arbitrary. All have been chosen to be sufficiently far removed from the study area so that their location will have minimal effect on the boundary fluxes into that area. These boundaries have been treated mathematically as no flow boundaries with recharge allowed at the nodes adjacent to them. The grid confined by these boundaries consists of (13 * 8) 10 km squares aligned parallel to the coast. Thus there is a total of 104 grid points in a modelled area of 10,400 square km. This would appear to be a very small number of grid points to cover such a vast area; however when the sparseness of the data within the area is taken into account it is the finest grid that can be justified.

6.4 HYDROGEOLOGICAL INPUT

6.4.1 Introduction

The validity of all subsequent modelling work and results depends to a significant degree upon the quality and reliability of the hydrogeological input data. For this reason considerable effort has been made to ensure that such data are consistent with our understanding of the hydrogeological regime. Where possible all values entered have been verified against field data.

Three types of information are required to build up the model framework. These comprise water table elevations at data points, transmissivity values for each node of the model grid and identification of nodes where recharge is taking place.

Assigning water table elevation is straightforward and simply involves transferring field water level observations directly to the relevant model data point. Once assigned the values are fixed and require no further modification. Transmissivity distribution and the identification of recharge nodes, on the other hand, present more difficulty. Unlike water levels, transmissivity needs to be assigned node by node and since direct measurement is only possible at a few sites extrapolation over large areas is necessary. As a result some error within the initial distribution is inevitable. For this reason values assigned to nodes are modified where subsequent model runs show this to be necessary. Newly-assigned values, however, are always set within the limits imposed by our understanding of the hydrogeology. In this way field observation and computer modelling operate hand in hand to provide a consistent and realistic pattern of transmissivity distribution (Figure 6.1).

The model is also used to modify the initial selection of recharge nodes where it indicates changes are necessary. But once again any alteration is required to be consistent with the known hydrogeology. In the following sections we outline the procedure used for transferring data from field to model.

6.4.2 Water Levels

Water level data have been obtained at 99 sites throughout the area. Readings were taken at 52 sites during the current investigation. At the remaining 47 sites which are restricted mainly to the alluvial plain, we have had to rely on published records, due to boreholes being either disused or in production. In every case the location and elevation of the site has been carefully checked and the data assessed for reliability before being entered into the model. For example water levels clearly referring to perched aquifers have not been included. In addition to borehole data points further water level information is required along all boundary nodes and these have been estimated from the water table map.

The eventual groundwater levels assigned and the resulting water table configuration, are shown in Figure 6.2.

6.4.3 Transmissivity

In terms of assigning model transmissivity the sand, alluvium and limestone formations each present their own problems and each has been solved in a different manner.

Regional recharge model : Distribution of transmissivity (m^2/day) INITIAL DISTRIBUTION

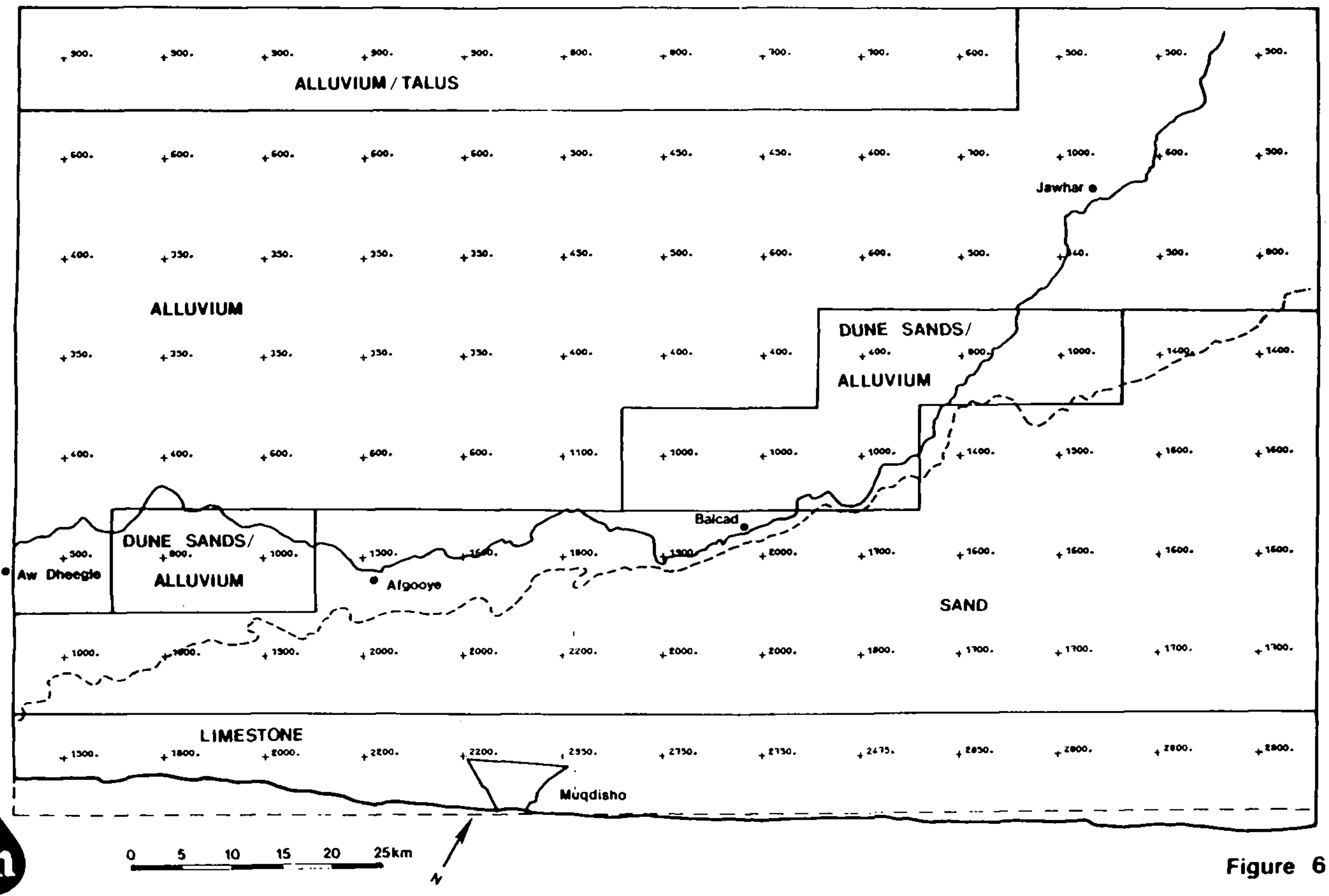


Figure 6·1

Regional recharge model : Given water table elevations (m)

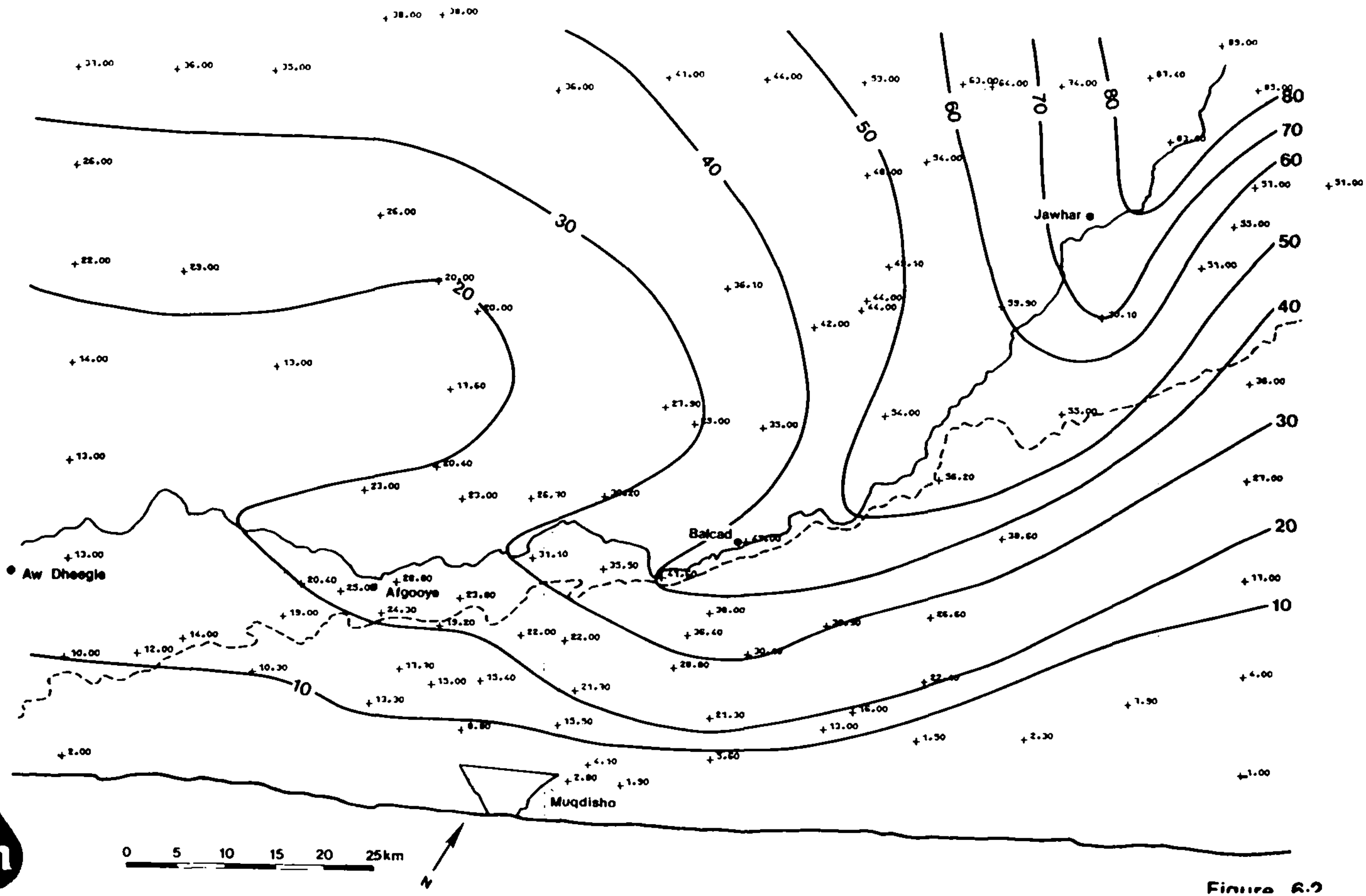


Figure 6.2

For the sand aquifer the results of 7 pumping tests have been used to derive an overall permeability of 15 m/d and 17 m/d for buff and red sands respectively. These values, in conjunction with total saturated thickness, obtained from Figures 5.3 and 4.2, have been used to evaluate nodal transmissivity throughout the sand aquifer.

Within the alluvial formation, complex lithology has demanded a slightly different approach. Here the results of two pumping tests have been used to derive a regional permeability of 16 m/d for the sand fraction, with clays and silts taken to be non-water bearing. Based on this, transmissivity has been calculated at 13 sites with known geology. The resulting network has been used to assign nodal transmissivity by extrapolation from the nearest data point.

At points where sand and alluvium are interbedded, beneath and along the southeast flanks of the Shebeelle, transmissivity has been adjusted to take account of the relative percentage of each formation present.

However, of all formations the coastal limestones proved most difficult to evaluate. The single pump test result shows low transmissivity, but contradicts the evidence of water table gradients adjacent to the coast, which point to high values. Initially the weight of evidence appeared to be in favour of high transmissivity and as a result our first estimates were made along these lines. At coastal nodes transmissivity was correlated inversely with gradient; the flatter the gradient the higher the transmissivity. Values were taken from the sand aquifer nodes immediately inland, with these being increased in proportion to the coastward decrease in gradient.

Subsequent modelling, however, showed the estimates to be in error and transmissivities more consistent with the field data were introduced.

6.5 MODEL DEVELOPMENT

6.5.1 Introduction

The regional recharge model is designed to quantify recharge taking place at designated nodes. It does this by distributing recharge in such a way that the match between the simulated and true water table is as close as possible. The true water table in this instance is that measured during the current study (Fig 5.3). A measure of the water table fit during any model run is given by the 'root mean square (RMS) error', defined in section 6.2. Essentially, the lower the error the better the fit.

In this section we describe the various changes made to the input data of the initial model and explain the reason for each modification. These modifications are traced to a point where the true water table is simulated with minimum error.

The first translation of field data into the model framework resulted in the transmissivity and water level distributions shown in Figures 6.1 and 6.2, with recharge being assigned to all river nodes. This represented the best initial interpretation of field data. When run, the model at once highlighted three regions of poor water level fit. These were:-

1. The region of the groundwater trough. Here simulated water levels were up to 10 m higher than true values.

2. Along the coast to the northeast of Muqdisho where simulated levels were up to 7 m too high.

3. At the site of E08 where the modelled water level was 13 m lower than the real value.

Elsewhere the fit was reasonably good resulting an an overall RMS error of 5.56.

At the outset the large errors occurring within the groundwater trough and along the coast were seen in terms of incorrectly assigned transmissivity. To obtain a better fit in these regions simply appeared to require an upward adjustment of local transmissivity in order to simulate lower water table elevations. With this in mind each problem region was tackled independently in a subsequent series of runs, beginning with the groundwater trough.

6.5.2 Modifications to transmissivity in coastal and trough regions

Along the length of the trough transmissivity was progressively raised to a maximum of 1200 m^2/day over three successive runs. This increase failed to improve the water table fit. The RMS error was 5.58. To reduce errors significantly would have required increasing transmissivity to levels clearly at variance with the known lithology.

Along the coast to the north east of Muqdisho a similar attempt was made to lower simulated water levels by increasing transmissivity. Already set at a high level in an endeavour to anticipate the requirements of low gradients in the region, transmissivity was raised still further by a factor of between 2 and 2.5. In this way values were increased to a maximum of 6800 m^2/day, as shown in Figure 6.3. Once again, however, no solution was obtained because although water level fit at the coast was improved, those inland now developed large errors in consequence, giving rise to an overall RMS error of 6.00: a situation clearly not acceptable.

These results showed that errors in the trough and coast regions could not be lessened by manipulation of the input data within the constraints set by the known hydrogeology. It followed that we needed to look beyond the model framework to obtain a satisfactory explanation for the poor fit in these areas.

In due course the reason for the errors was recognised: water levels in the coastal and trough regions are not in a steady state condition with respect to the present recharge regime. At the coast levels are controlled by a rising coastline, while in the trough they are the product of recharge from a much earlier period (see chapter 5). Such processes cannot be simulated by the models within

Regional recharge model : Distribution of transmissivity (m^2/day) HIGH COASTAL VALUES

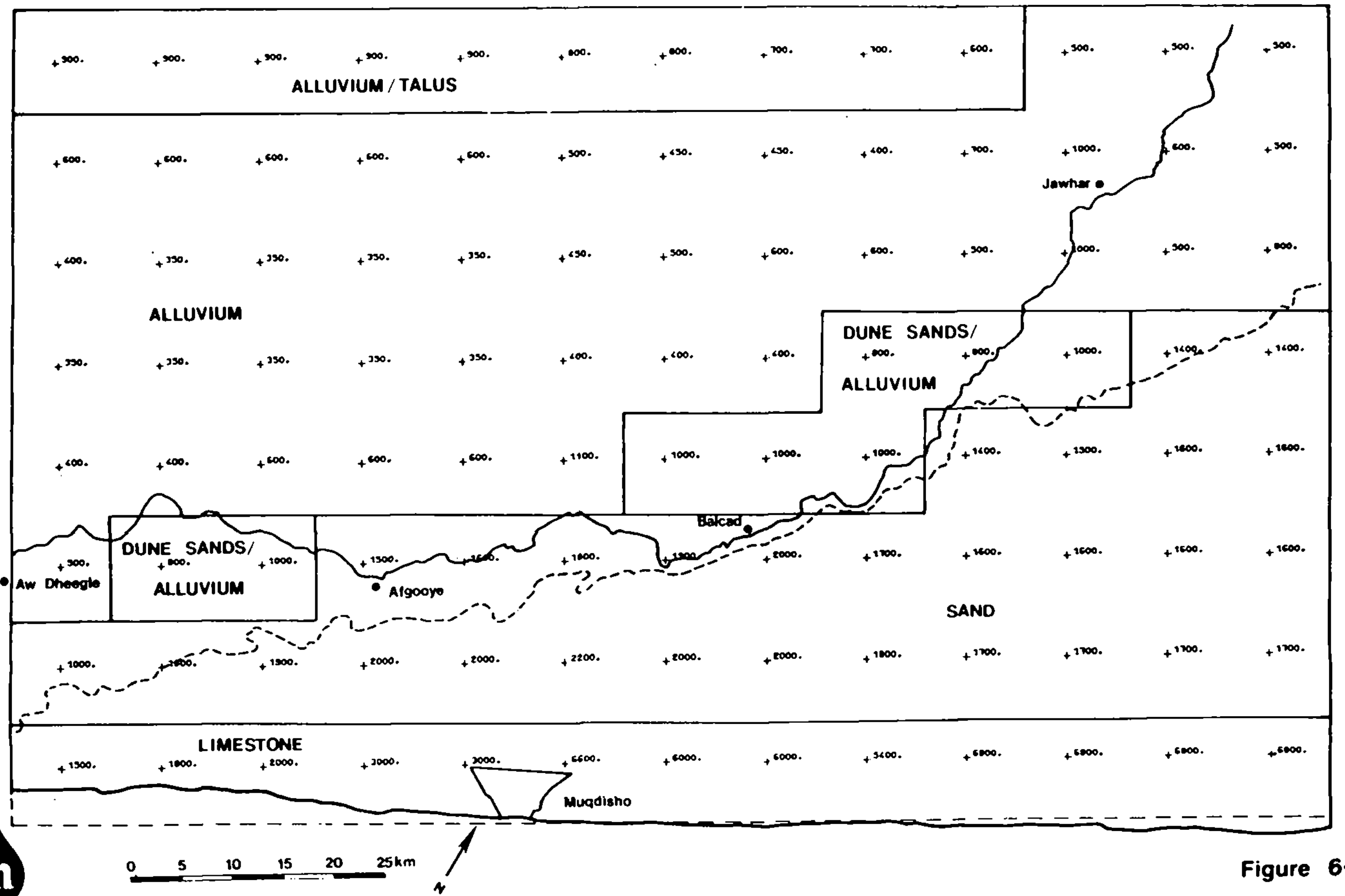


Figure 6·3

which water levels are taken to be a function solely of transmissivity, water table gradients and present day recharge. Attempts to simulate water levels caused by other factors invariably fail.

Since the model was shown to be inapplicable in the trough and coastal regions transmissivity was eventually assigned purely on the basis of field data. Within the trough values used in the initial run were retained, but in the coastal region lower figures were allocated to reflect the result of the single pumping test undertaken in the coastal limestone (Section 5.4.3). This contrasts with the initial distribution which erroneously was based on the evidence of water table gradients. The final distribution selected is shown in Figure 6.4.

Subsequent model runs incorporating these transmissivities repeated the poor fit in the trough and at the coast, but in the light of our increased hydrogeological understanding the errors now became admissible. Elsewhere the water table fit was good and an overall RMS error of 5.25 resulted.

6.5.3 Location of recharge nodes

Up to this point recharge had been assigned to all nodes beneath the river. But by now the evidence of conductivity, stable isotope and chemistry data pointed to recharge being restricted to selected reaches of the river (see Section 5.5). Subsequent runs were thus undertaken with nodes removed from those lengths of river shown to contribute no recharge. These included 2 nodes to the west of Afgooye, one between Balcad and Afgooye and one to the south of Jawhar. (Figure 6.5). At the same time one extra recharge node was located beneath the Jawhar sugar estate to simulate irrigation infiltration known to be taking place at this site. When run with this recharge node configuration a slightly reduced RMS error of 5.021 resulted.

6.5.4 The problem of E08

Apart from the trough and coastal region to the northeast of Muqdisho the water table fit, at this stage, was remarkably good. One

Regional recharge model : Distribution of transmissivity (m^2/day) FINAL DISTRIBUTION

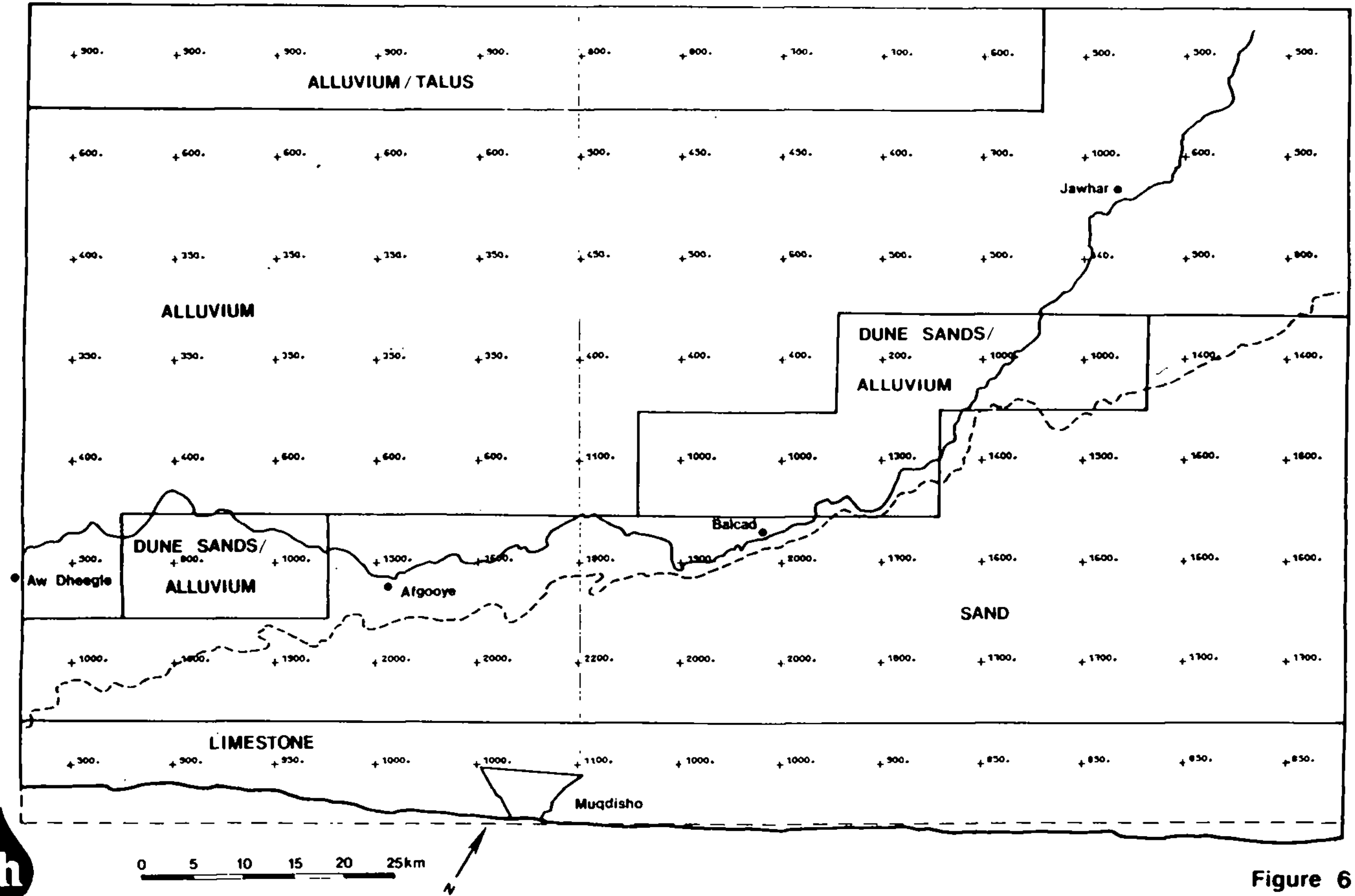


Figure 6·4

data point, however, retained a persistent error of greater than 10 m; this was the site of E08. In an attempt to remove this error several runs were undertaken which incorporated modifications to, and in the vicinity of, the node concerned. Transmissivity was progressively reduced to the lowest acceptable limits based upon field data and a number of recharge nodes introduced over 5 successive runs, but to no avail. The error at E08 failed to respond to any reasonable modification. Indeed the best of the runs only reduced the difference between simulated and true water tables to 10 m, giving an overall RMS error of 4.98.

Eventually it was accepted that the problem related to the position of the site with respect to the 'step' in the water table profile (see Section 5.2). At this point the model attempts to smooth out the step, which is a non-steady state feature, leading to an underestimate of levels at E08. Thus the error is associated with non-steady state conditions in much the same way as those in the trough and along the coast. For this reason the error became admissible.

6.5.5 Run with 'non-steady state' data points removed

To demonstrate the close fit of simulated and given water tables beyond the regions not in steady state, a final run was undertaken with seven data points in the trough and coastal region removed. Under these circumstances the overall RMS error reduced to 3.81, a remarkably low figure in view of the sparseness of data.

We have, however, taken the earlier run with a RMS error of 4.98, which includes all data points, as the final version of the regional recharge model. This incorporates the transmissivity and recharge node distributions shown in Figs 6.4 and 6.5. The best water table fit is reproduced in Figs 6.6 while the errors between this and the true water table are given in Figure 6.7.

6.6 RESULTS OF THE REGIONAL RECHARGE MODEL

The recharges derived from our final model are shown in Figure 6.5. Here individual recharge nodes are shown to be either

Regional recharge model : Recharge nodes and inferred abstractions (million m^3/year)

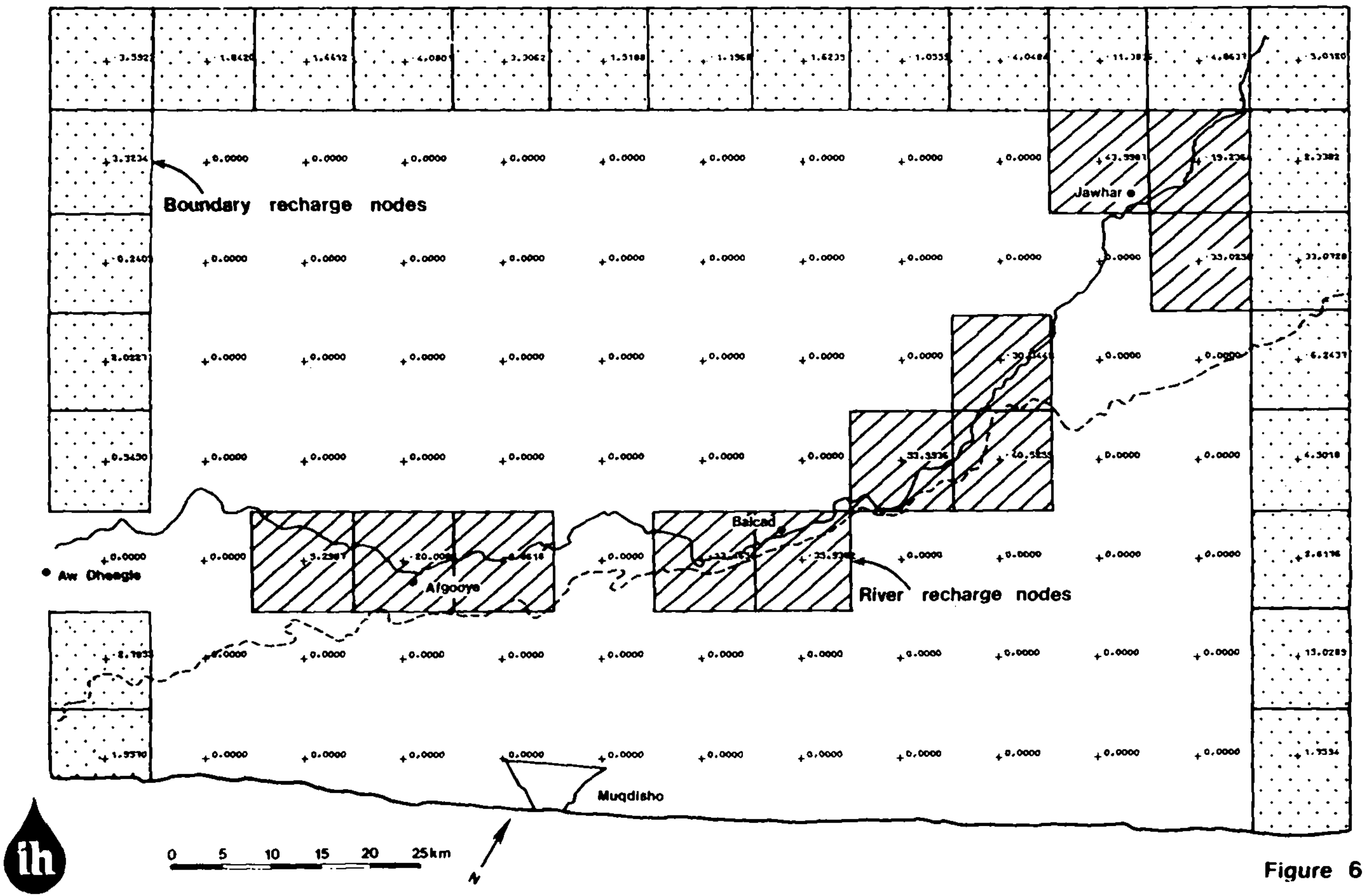


Figure 6·5

Regional recharge model : Best fit water table elevation (m)

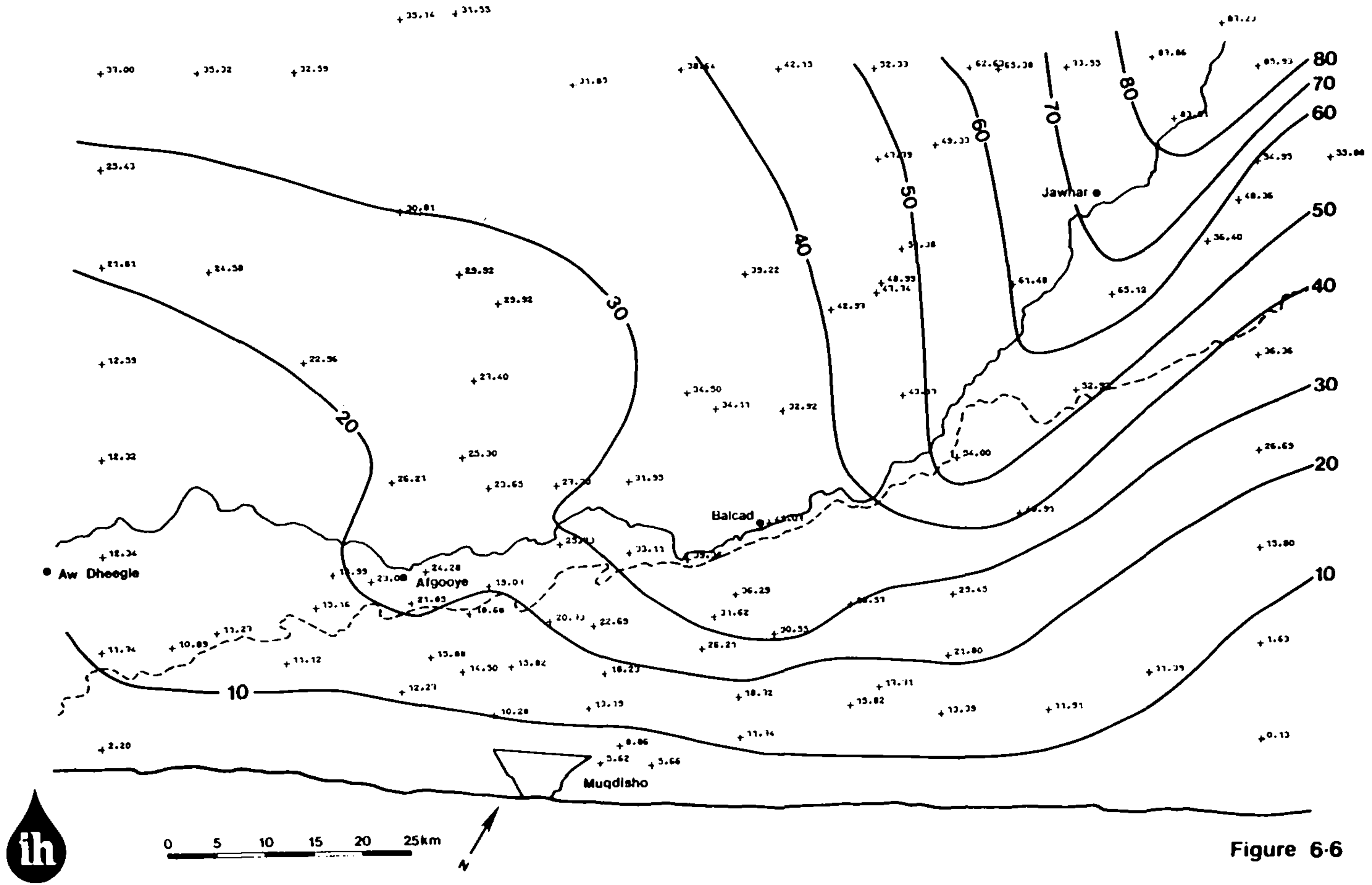


Figure 6·6

Regional recharge model : Errors in water table fits (m)

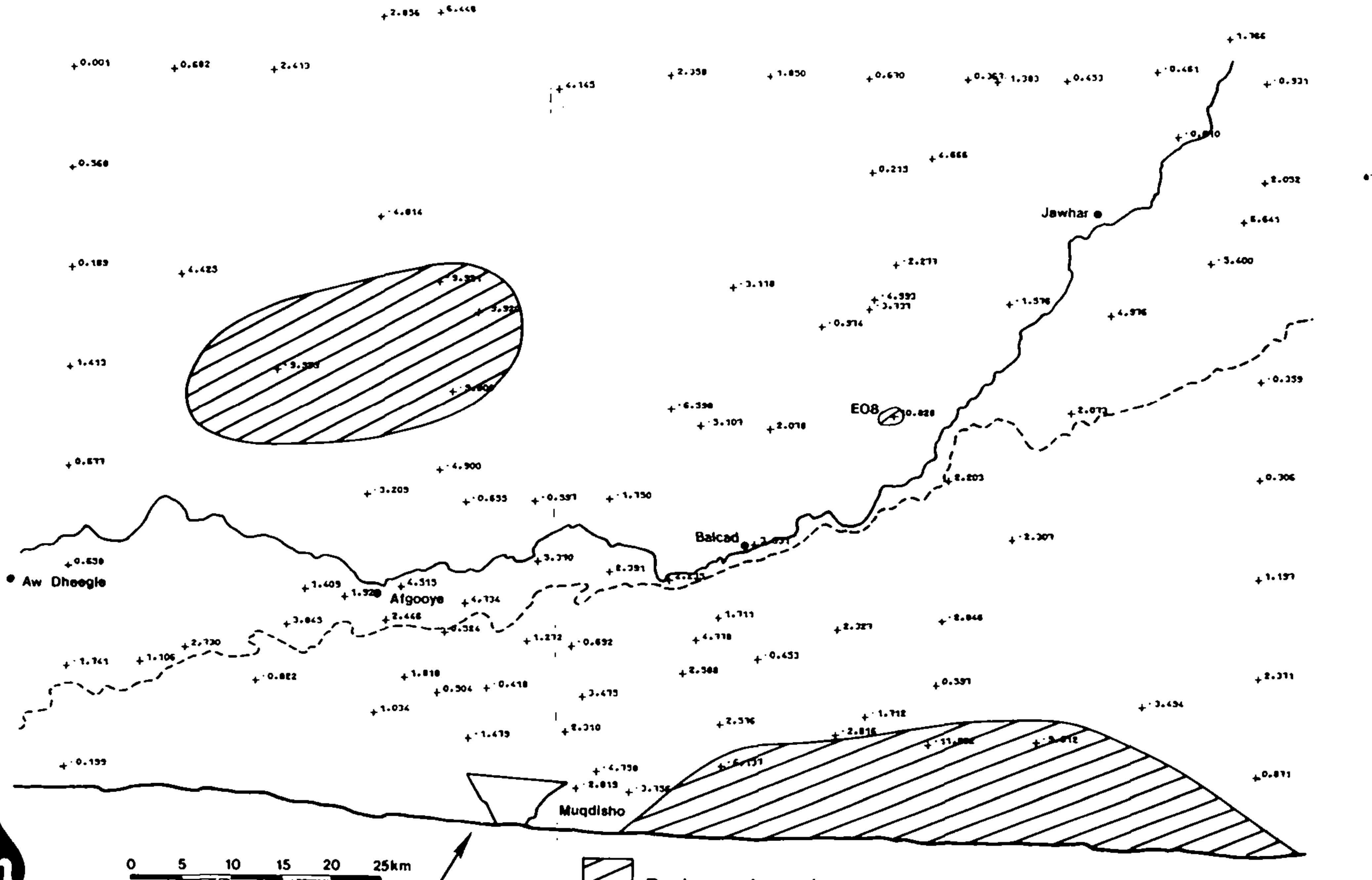


Figure 6·7

sustaining abstraction or receiving recharge (-ve sign). This is not intended to imply that real abstractions and recharges are taking place in adjacent nodes, but is simply a manifestation of the way in which the model calculates and distributes recharge. In fact net recharge is the algebraic sum of all values assigned to recharge nodes. Direct interpretation of this type of presentation is, however, difficult so to help clarify the picture net recharges have been listed by regions in Table 6.1.

TABLE 6.1 Net recharge to aquifer

Region	Recharge to Aquifer (million m^3/year)
20 km Upstream) of Jawhar 15 km Downstream)	23.6
15 km Upstream) of Balcad 25 km Downstream)	40.1
20 km Upstream) of Afgooye 35 km Downstream)	5.9
TOTAL	69.6

Recharge is blocked into 3 regions, for convenience. The breakdown shows that most recharge takes place along the stretch of river centred about Balcad, with somewhat lower levels of recharge taking place in the Afgooye region. Recharge taking place along the Jawhar stretch of river is concentrated toward the Balcad end with relatively little being introduced in the Jawhar region itself. Overall the total net recharge to the area is shown to be 69.6 million m^3/year.

At this point a comparison can be made with independent estimates of recharge obtained from surface water studies. (Section 5.7). Comparison on a region by region basis is not feasible because those used by the surface water study are difficult to duplicate using the coarse model grid. Nevertheless total net recharge can be compared.

TABLE 6.2 Net recharge from river system

	Upper Limit	Best Estimate (million m^3/year)	Low Limit
Recharge from irrigation	117.2	64.4	35.2
Recharge from river	44.5	17.8	7.1
TOTAL	161.7	82.2	42.3

Results of the surface water study given in Table 6.2, are expressed in terms of upper and lower limits and are broken down into recharge from irrigation and river bed infiltration. Total recharge is set between limits of 42 to 162 million m^3/year with a best estimate of 82 million m^3/year. This compares with the figure of 69.6 million m^3/year derived from the model, which is unable to separate river bed and irrigation components.

STUDY AREA RECHARGE MODEL

7.1 INTRODUCTION

In this chapter recharge estimates are obtained using the same grid as used in the water management model. This is done in order to ensure that the recharge estimates are compatible with the transmissivity distribution used in the management model.

The theoretical basis for the study area application of the recharge model is identical to that described for the regional area. As with the latter runs of the regional recharge model water table fitting is carried out at the data points.

7.2 MODEL FRAMEWORK

The study area is the region within which the water management modelling is to be carried out. Since the wellfields to be analysed during that work must produce water above a certain quality the location of the study area boundaries are determined by water quality. Within this report conductivity has been used as the measure of water quality. A map showing the variations in groundwater conductivity in the vicinity of Muqdisho is shown in Figure 5.8. Using this information the study area boundaries have been located.

The extent of the study area is shown in Plate 1. The southern boundary corresponds to the coast and, except for the eastern third, the northern boundary corresponds to the river Shebeelle. The river was chosen as the northern boundary since the groundwater to the north of the river is of too poor a quality to be considered as a feasible water resource. The groundwater to the west of Afgooye is of poor quality hence the western boundary of the study area was located just to the south-west of Afgooye. To the east of Balcad field work has indicated that the groundwater is of good enough quality for it to be a potential water source. Hence the eastern boundary of the study area is located approximately 35 km north-east of Balcad. As with the regional area the study area boundaries, with the exception of the coastal boundary, are chosen so that their location will have minimal influence on the results obtained.

The southern boundary of the study area is treated as a zero head boundary whereas all the other boundaries are represented as mathematical no flow boundaries. Recharges are allowed at the nodes adjacent to these boundaries to take account of boundary fluxes and/or recharge and abstraction.

The study area is divided into a grid of (17 * 6) 5 km squares aligned parallel to the coast. At the centre of each of these squares is a grid point. Thus the study area grid consists of 102 nodes covering an area of 2550 square km.

7.3 HYDROGEOLOGICAL INPUT

The hydrogeological data initially entered into this model was based upon that incorporated within the final regional recharge model, but broken down in a more detailed manner for inclusion into a finer grid. Water levels, transmissivity and recharge node distributions were all transferred in this way. The water level transfer posed few problems since these were based upon the same data points present within both grids. However, the re-allocation of transmissivity and recharge nodes within the finer grid inevitably led to the introduction of some initial errors and inconsistencies.

In the following section we outline the manner in which these were resolved and trace the development of the model to its final form.

7.4 MODEL DEVELOPMENT

Transmissivities initially allocated to the study area model are those shown in Figure 7.1, while true water table elevations are given in Figure 7.2. The finer grid used within this model has allowed the inclusion of a limestone/sand transition zone; this was missing from the regional recharge model because of the coarse grid. At the same time recharge nodes were positioned along the northern boundary to simulate river infiltration and groundwater flow from the north of the boundary. They were also assigned along both eastern and western boundaries in order to meet the mathematical requirements of the model as explained in Section 7.2.

Study area model : Distribution of transmissivity (m^2/day) (INITIAL DISTRIBUTION)

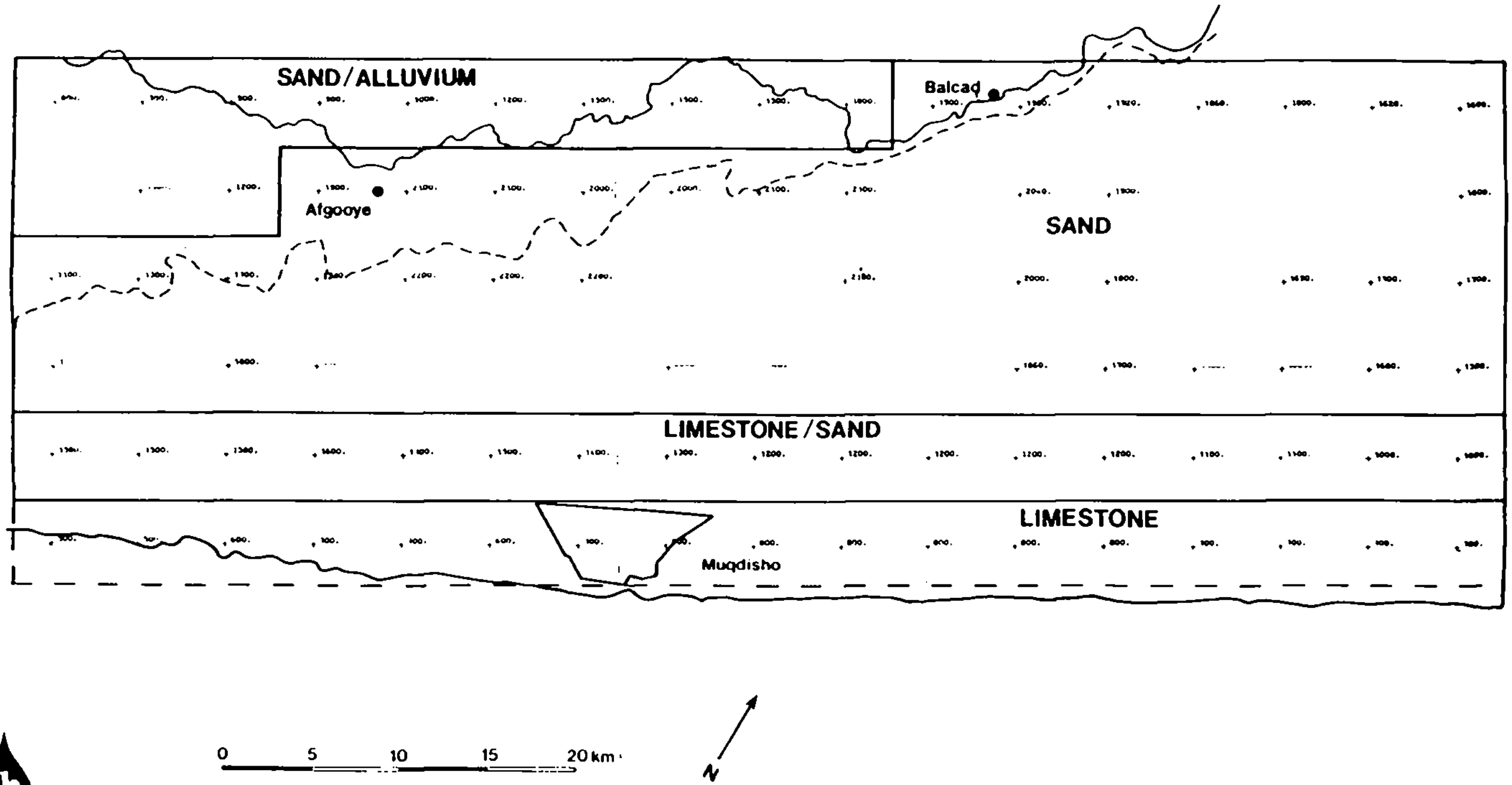


Figure 7·1

Study area model : Given water table elevations [m]

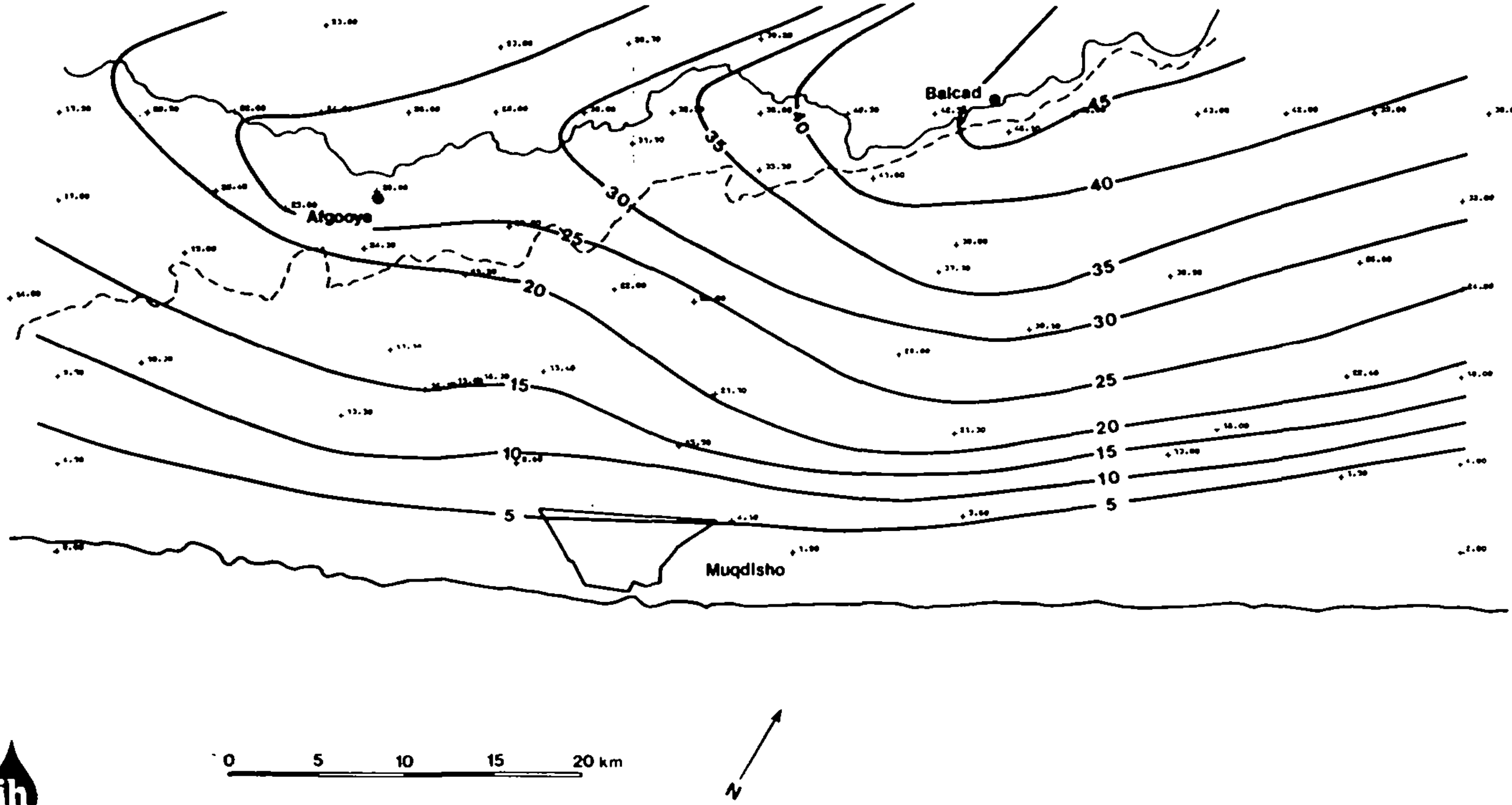


Figure 7·2

With such input data the model, when run, gave a RMS error of 3.88. Most of this error was caused by a poor water table fit along the coast to the north east of Muqdisho and at two points adjacent to the river. In an endeavour to eliminate these errors a number of recharge nodes and transmissivity modifications were incorporated in a subsequent series of four model runs.

To combat the poor water table fit close to the river, three extra recharge nodes, one row in from the northern boundary, were introduced to represent points where meander loops in the river extend southward (Figure 7.3). At the same time a number of minor transmissivity modifications were made along the sand-alluvium contact in order to reflect field data more closely. This strategy succeeded in lowering the RMS error to 3.14, virtually eliminating errors adjacent to the river but retaining those along the coast to the north east of Muqdisho.

Through the regional recharge model, however, we had already accounted for this coastal error in terms of non-steady state conditions. Hence, given the good fit of the remainder of the region, further work was strictly speaking not necessary. Nevertheless, since a finer grid was now being used, we felt it worthwhile investigating the effects of attempting to estimate these errors by increasing the value of coastal transmissivities. By raising values to a maximum of 2700 m^2/day (Figure 7.4) we obtained a greatly improved fit with the RMS error reduced to 2.36. At the same time the calculated net recharge into the area was increased by 50% to over 67 million m^3/year.

Purely in modelling terms this was the best obtainable result. But from a hydrogeological point of view it was unacceptable, for two reasons; firstly the values given to coastal limestones were not consistent with their known field characteristics, and secondly the resultant net recharge for the study area was unacceptably high. As a result, despite having the best water table fit, this version of the model was not accepted. Instead the model finally adopted was that having low, but realistic, coastal transmissivities, (Figure 7.5). This generated the water table shown in Figure 7.6 and gave a RMS

Study area model : Recharge nodes and net abstractions [million m^3/year]

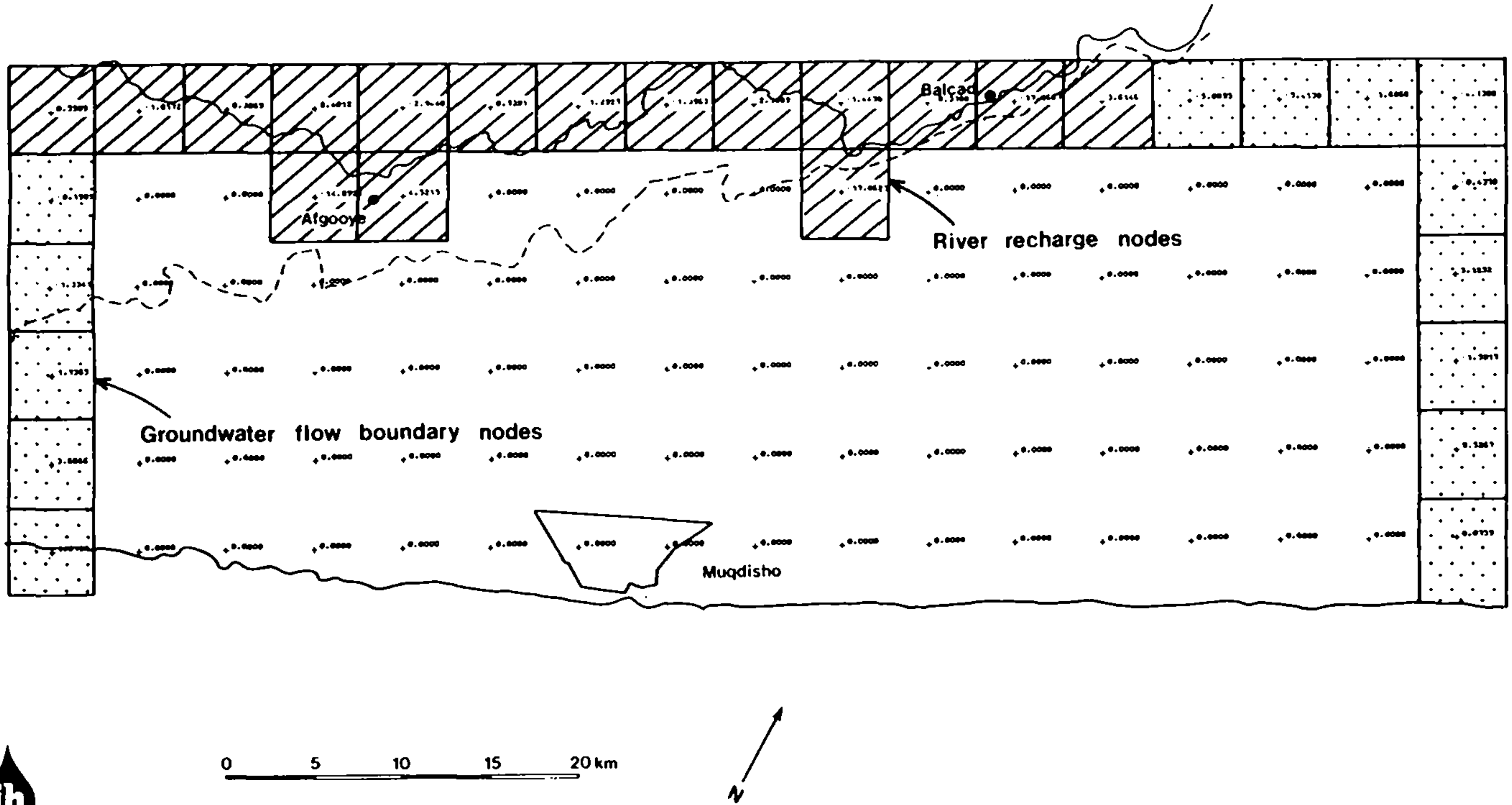


Figure 7·3

Study area model : Distribution of transmissivity (m^2/day) (HIGH COASTAL VALUES)

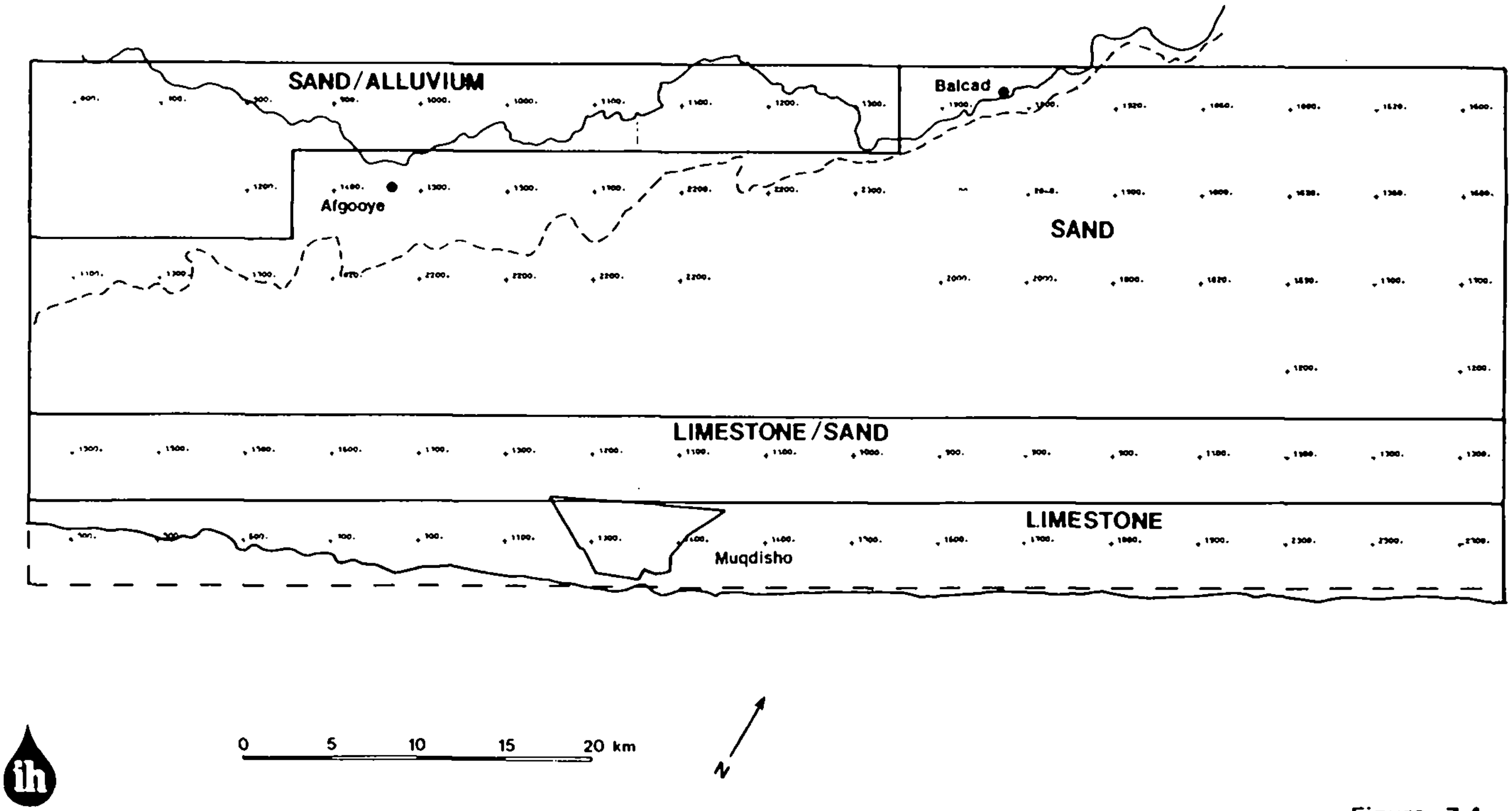


Figure 7·4

Study area model : Distribution of transmissivity (m^2/day) (FINAL DISTRIBUTION)

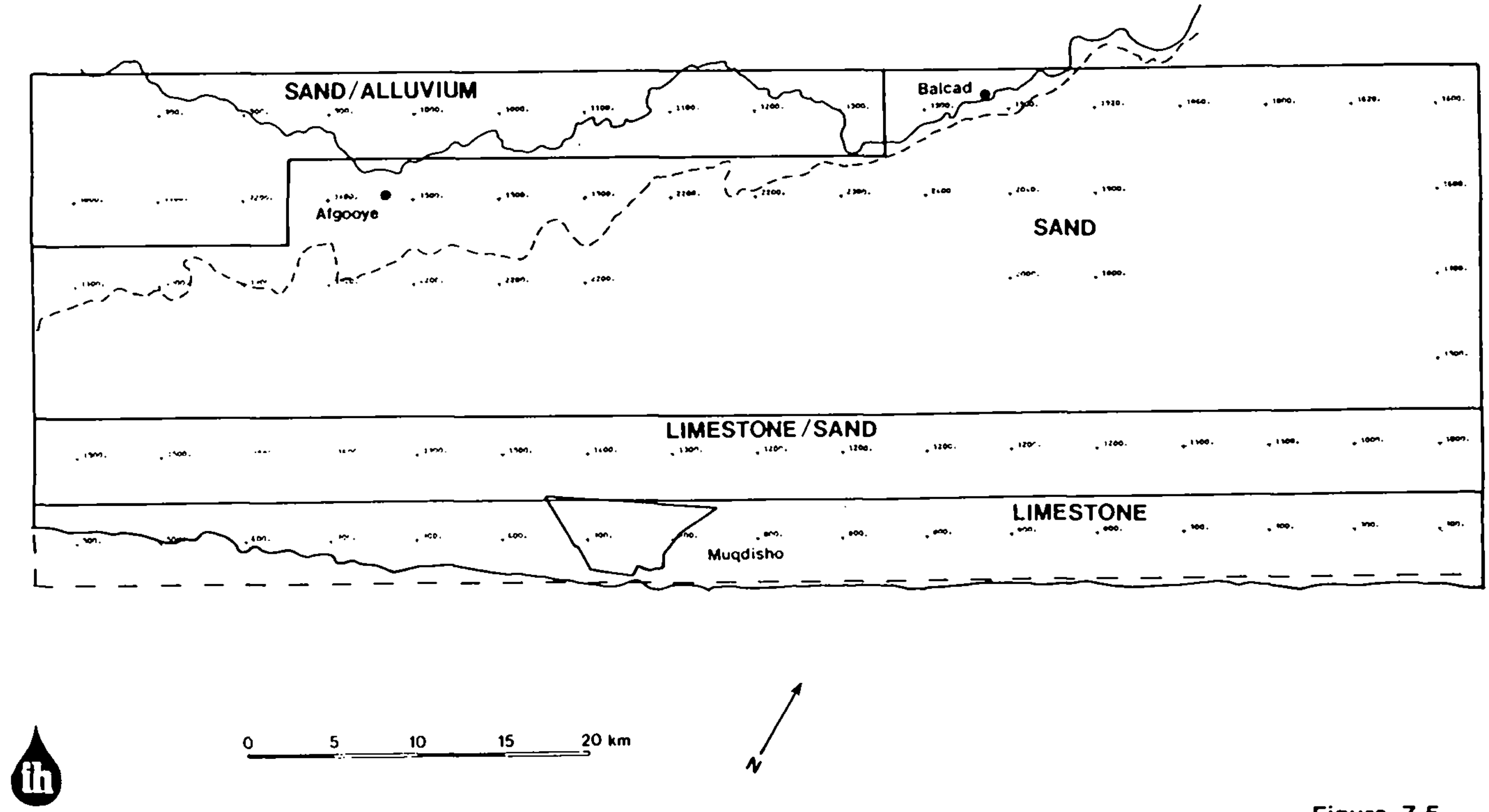


Figure 7·5

Study area model : Best fit water table elevation [m]

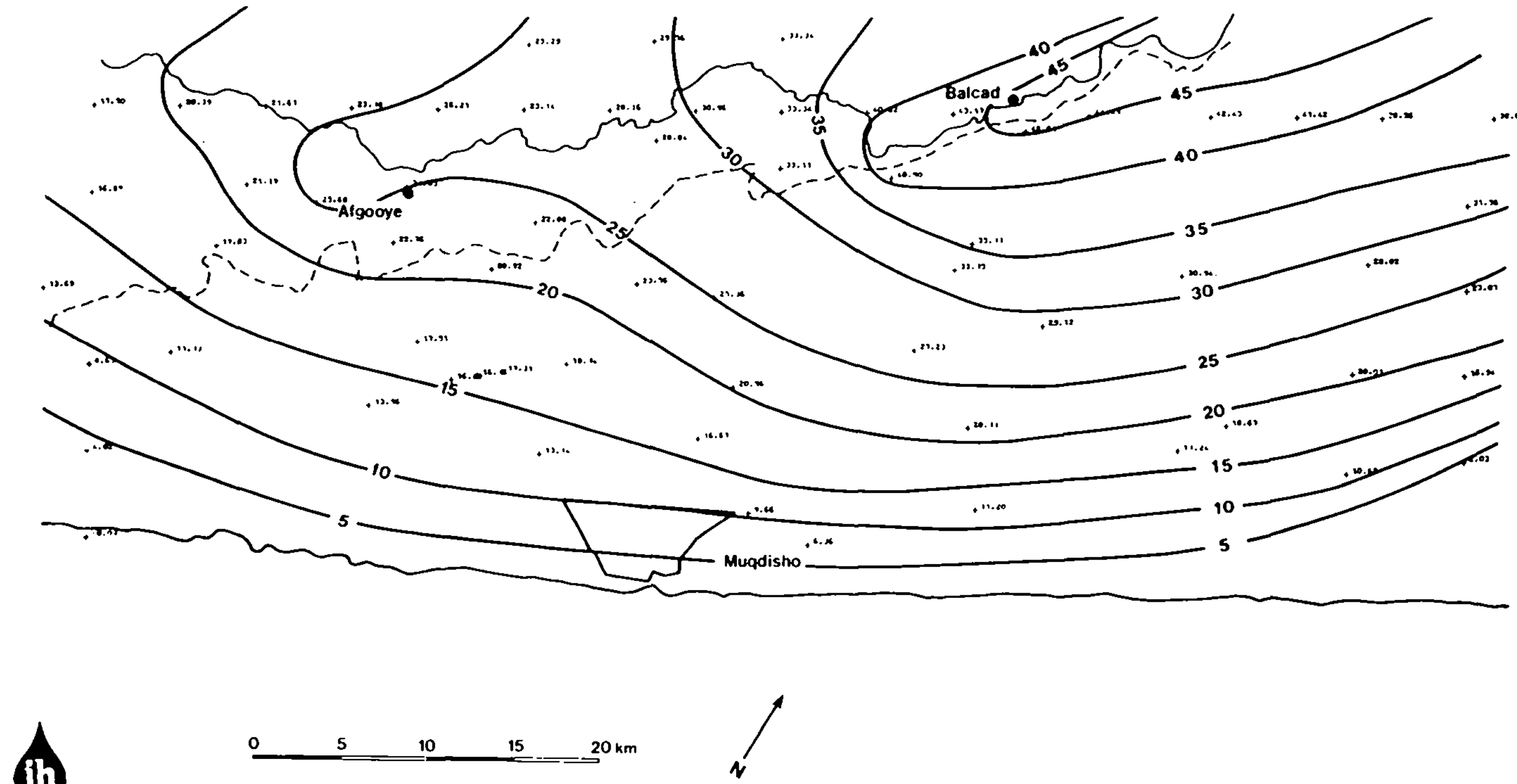


Figure 7·6

error of 3.14. The distribution of errors between this and the given water table are presented in Figure 7.7.

7.5 MODEL RESULTS

This section compares the recharges generated by the coarse grid of the regional model with those of the finer study area grid, and verifies their consistency.

The recharges to the study area calculated from the regional and study area recharge models are listed in Table 7.1. The regional model figures refer only to recharge entering the section of the regional model covered by the study area grid. These figures are calculated from the recharges shown in Figure 6.5 and the internodal flows for that run of the regional recharge model. The study area recharge model figures come directly from Figure 7.3. Because of the ways the two sets of figures have been calculated they are directly comparable.

The table shows both net recharge totals to be very similar; 46.7 million m^3/year and 43.9 million m^3/year for the regional and study area models respectively. The models show that the maximum recharge to the aquifer takes place at Afgooye and Balcad with much less recharge in the area between them.

These results confirm a close agreement between the models. The differences that occur are a result of the finer grid used in the study area recharge model. For eventual input into the management model the recharge distribution given by the study area model was chosen, since this was calculated using the same size and area of grid.

Study area model : Errors in water table fits [m]

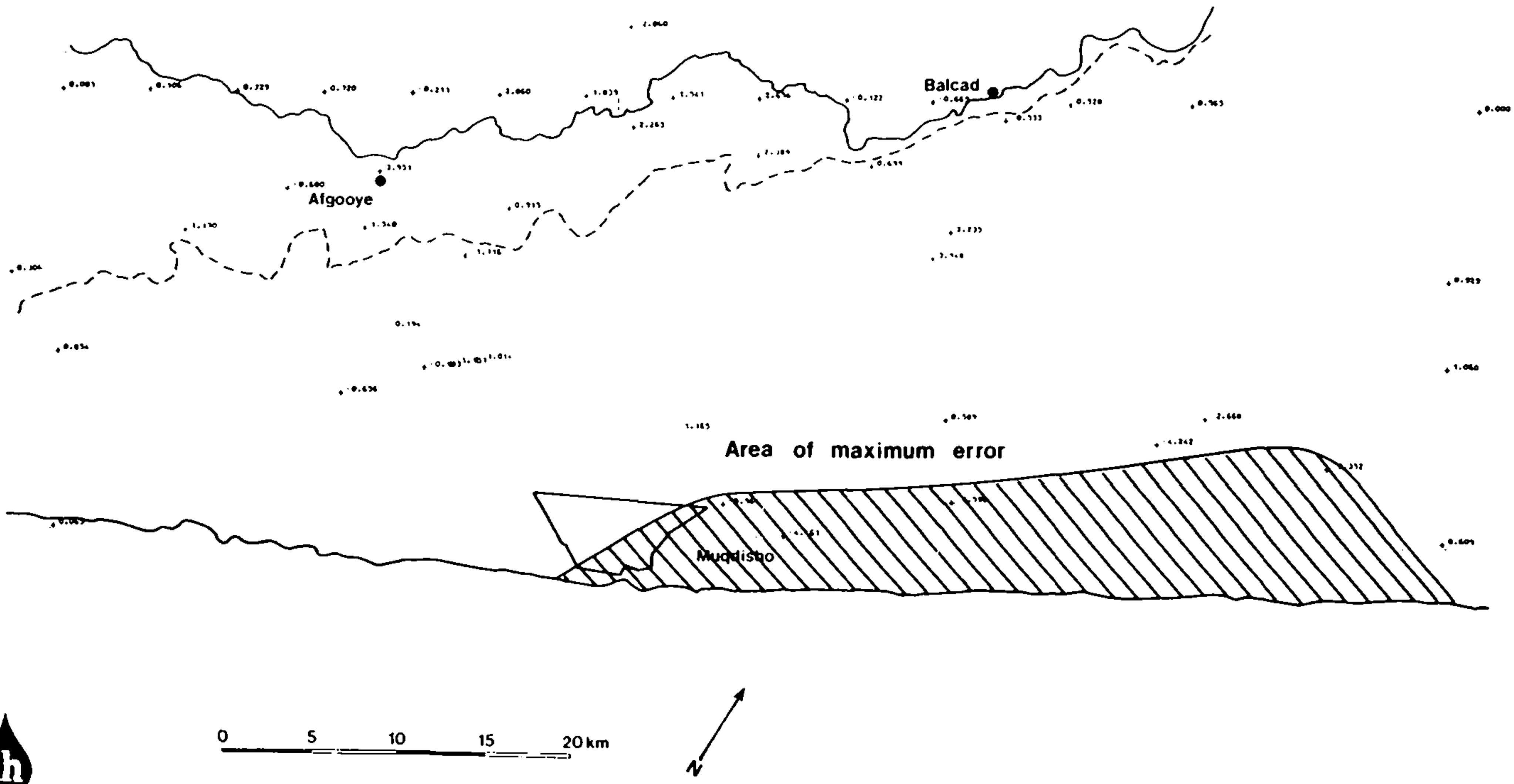


Figure 7·7

TABLE 7.1 COMPARISON OF RECHARGES (million m^3/year)

	REGIONAL MODEL	STUDY AREA MODEL
Region	Recharge	Recharge
West of Afgooye	1.6	0.63
Afgooye	12.3	10.9
Afgooye-Balcad	(-) 0.76	(-) 0.39
Balcad	32.3	36.1
East of Balcad	6.0	9.3
Eastern boundary	(-) 1.73	(-) 6.56
Western boundary	(-) 3.1	(-) 6.1
TOTAL LOSS DUE TO G/W FLOW	20.2	34.4
TOTAL RECHARGE	67.0	78.3
NET RECHARGE	46.7	43.9

8. WATER MANAGEMENT MODEL

8.1 INTRODUCTION

In this chapter the results of the recharge modelling are brought together in a water management model which is then used to investigate the consequences of various abstraction regimes. The model development and a description of the numerical experiments is given here and the results are discussed in chapter 9.

8.2 THEORETICAL BASIS

The transient, horizontal flow of groundwater within an inhomogeneous, isotropic aquifer is described by the following equation,

$$\frac{\partial}{\partial x}\left(T\frac{\partial \phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(T\frac{\partial \phi}{\partial y}\right) = S\frac{\partial \phi}{\partial t} - N \qquad (1)$$

where: (x,y) is the horizontal coordinate system (m)
t is time (days)
$T(x,y,t)$ is the transmissivity (m^2/day)
$\phi(x,y,t)$ is the hydraulic head (m)
$S(x,y,t)$ is the storage coefficient (nondimensional)
$N(x,y,t)$ is the excess of inflow over outflow per unit area - positive for recharge and negative for abstraction (m/day)

Assumptions similar to those made during the recharge modelling have been used to obtain this equation. These assumptions are that vertical flow is negligible relative to horizontal flow and that transmissivity can be considered independent of saturated thickness. As with the recharge models these are physically reasonable assumptions.

In order to investigate the effect that different abstraction regimes have on the water table configuration equation (1) has to be solved for the hydraulic head, within a study area, subject to certain boundary conditions. As with the recharge model the boundary conditions are of two types; the coast is considered to be a zero head boundary whilst the other boundaries are mathematical no flow boundaries. At these, boundary fluxes are represented as recharges at nodes adjacent to the boundary.

Within the study area the continuous solution of equation (1) has been approximated by a discrete solution using the finite difference scheme. In order to do this the block-centred finite difference grid shown in Figure 8.1 is used. By considering the water balance of the i,j element we obtain the implicit equation,

$$\frac{1}{\Delta x_{i-}} \left[T_{i-,j} \left(\frac{\phi^K_{i-1,j} - \phi^K_{i,j}}{\Delta x_{i-}}\right) - T_{i+,j} \left(\frac{\phi^K_{i,j} - \phi^K_{i+1,j}}{\Delta x_{i+}}\right)\right]$$

$$+ \frac{1}{\Delta y_j} \left[T_{i,j-} \left(\frac{\phi^K_{i,j-1} - \phi^K_{i,j}}{\Delta y_{j-}}\right) - T_{i,j+} \left(\frac{\phi^K_{i,j} - \phi^K_{i,j+1}}{\Delta y_{j+}}\right)\right] \qquad (2)$$

$$= \frac{S_{i,j}}{\Delta t} \left(\phi^K_{i,j} - \phi^{K-1}_{i,j}\right) \qquad N^{K-}_{i,j}$$

where: the superscripts are positions in time, the subscripts are positions in space

$T_{i\pm,j}$ and $T_{i,j\pm}$ are respectively the internodal transmissivities in the x and y directions (m^2/day). See Figure 8.1.

Δ_{xi} and $\Delta x_{i\pm}$ are the x grid spacings (m)

Δy_j and $\Delta y_{j\pm}$ are the y grid spacings (m)

Δt is the time step (days)

$$N^{K-}_{i,j} = (N^K_{i,j} + N^{K-1}_{i,j}) \quad (m^3/\text{day})$$

All other terms in equation (2) have been defined.

In obtaining this equation it has been assumed that the distributions of transmissivity and storage coefficient are independent of time. The internodal transmissivities have been

Index scheme for finite difference grid and coefficients of finite difference equation

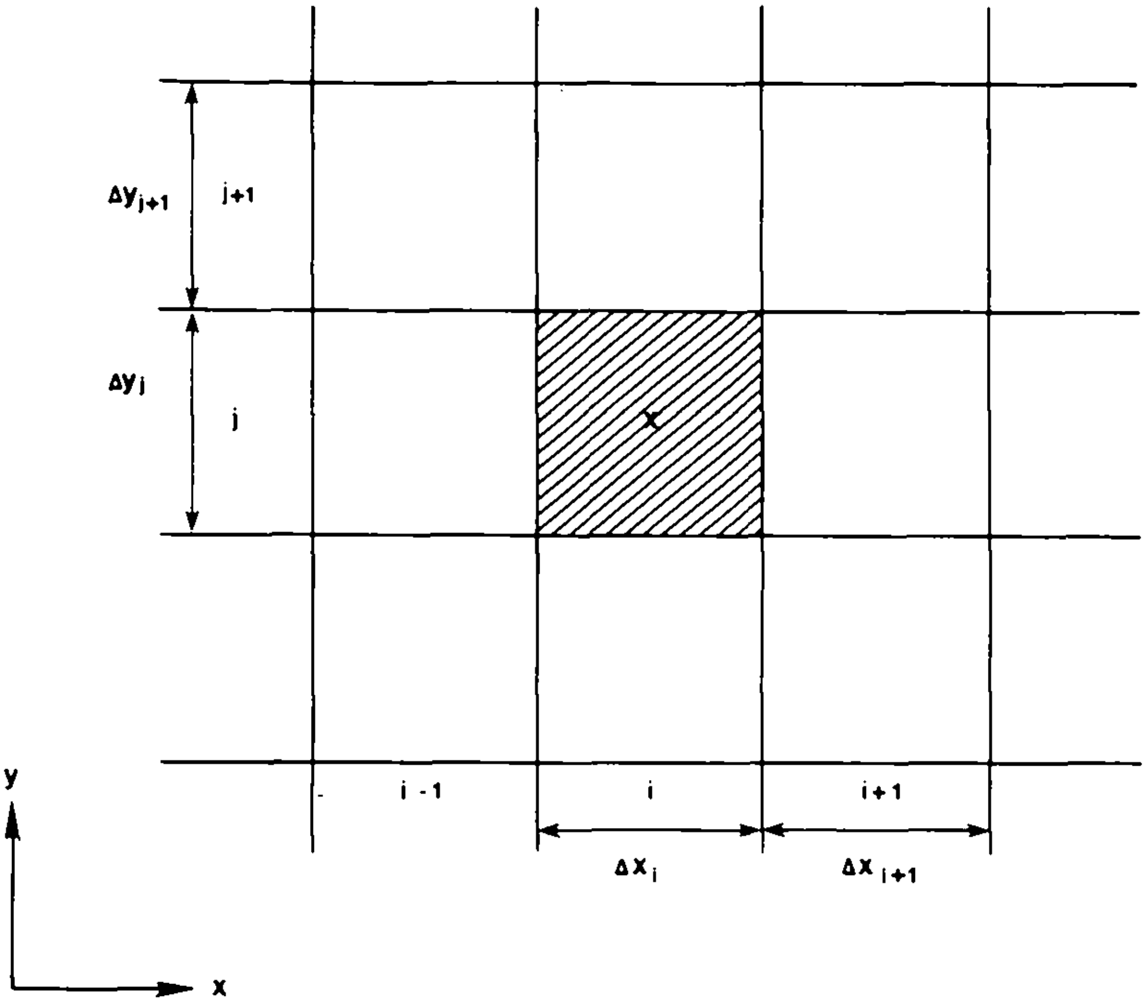


Figure 8.1

calculated as being the harmonic mean of the transmissivities at the adjacent nodes. The harmonic mean has been used since this is less influenced by extreme values. An implicit scheme has been preferred to an explicit one since its stability is independent of the length of time step. However the implicit scheme must be applied with caution since the time step length may influence the accuracy of the solution. The influence of time step length on the amount of drawdown is investigated in Appendix E.

The excess of inflow over outflow consists of two components; the natural recharges and the fixed abstractions,

$$\text{Thus, } N_{i,j}^{K-} = [(R_{i,j}^{K} + R_{i,j}^{K-1}) + (B_{i,j}^{K} + B_{i,j}^{K-1})]/2 \qquad (3)$$

where $R_{i,j}$ are the natural recharges

$B_{i,j}$ are the fixed abstractions

Within this study it has been assumed that the natural recharges do not vary with time, thus equation (3) can be rewritten as,

$$N_{i,j}^{K-K} = R_{i,j} + (B_{i,j}^{K} + B_{i,j}^{K-1})/2 \qquad (4)$$

By rearranging and incorporating equation (4) into equation (2) the basic implicit scheme can be rewritten as,

$$\frac{T_{i-,j}}{\Delta x_i \Delta x_{i-}} \phi_{i-1,j}^{K} + \frac{T_{i+,j}}{\Delta x_i \Delta x_{i+}} \phi_{i+1,j}^{K} + \frac{T_{i,j+}}{\Delta y_j \Delta y_{j+}} \phi_{i,j+1}^{K} + \frac{T_{i,j-}}{\Delta y_i \Delta y_{j-}} \phi_{i,j-1}^{K}$$

$$- [\frac{T_{i-,j}}{\Delta x_i \Delta x_{i-}} + \frac{T_{i+,j}}{\Delta x_i \Delta x_{i+}} + \frac{T_{i,j-}}{\Delta y_j \Delta y_{j-}} + \frac{T_{i,j+}}{\Delta y_j \Delta y_{j+}} + \frac{S_{i,j}}{\Delta t}] \phi_{i,j}^{K} \qquad (5)$$

$$= - \frac{S_{i,j}}{\Delta t} \phi_{i,j}^{K-1} - R_{i,j} - (B_{i,j}^{K} + B_{i,j}^{K-1})/2$$

All parameters within equation (5), with the exception of the ϕ^K's are known, thus this equation can be used to estimate these. The numerical solution of this equation was found by using successive over relaxation.

8.3 MODEL FRAMEWORK

The region within which the water management modelling is performed is the study area. The location of the study area boundaries together with the boundary conditions are discussed in chapter 7 which also contains a description of the model grid.

8.4 HYDROGEOLOGICAL INPUT

The model calibration described in the preceding chapters has furnished values for the major hydrogeological parameters required as input for the management model. The transmissivity distribution, and the distribution and magnitude of the natural recharges used in the management modelling, are identical to those used in the final run of the study area recharge model. The only other hydrogeological input the management model requires is the distribution of storage coefficient.

The values of storage coefficient have been discussed in the section of chapter 5 on aquifer properties. Most of the aquifer covered by the study area consists of sand; thus a storage coefficient of 4 per cent has been used. In the extreme coastal areas where the aquifer is mainly composed of limestone no field estimates of storage coefficient are available; thus it has been assumed that the aquifer in this area has the same storage coefficient as the sand aquifer. In the north-western part of the modelled area the aquifer consists of a mixture of sand and clay, within this area a storage coefficient of 2 per cent has been assumed.

The pre-abstraction water levels used by the model are those which are numerically compatible with the given recharges and transmissivity distribution. These water levels will differ from the true water table elevations because in certain parts of the study area the water table is not solely a function of present day recharge and transmissivity. The numerically compatible water levels must be used, otherwise the drawdown patterns obtained will reflect the lack of compatibility between the initial water level elevations and the rest of the input data as well as the response to given abstraction regimes.

8.5 WATER QUALITY CONSTRAINTS

The estimation of the maximum amount of water that can be recovered from an aquifer is not only a problem of quantity; the groundwater quality must also be taken into account. Thus the rates of groundwater abstraction which can be sustained without causing a reduction in water quality need to be estimated. In the vicinity of Muqdisho there are two potential sources of groundwater contamination; firstly due to saline water at the coast and secondly, due to poor quality water from north of the river Shebeelle. The absolute values used for the constraints are described in a later section of this chapter. Within this section the philosophy behind and the problems associated with the selection of these values is discussed.

8.5.1 Saline intrusion constraints

Because of the differing densities of fresh and sea water a saline wedge will exist at the coast. Despite the fact that hydraulic gradients are toward the sea this wedge may extend several kilometres inland. If the freshwater above this wedge is pumped excessively the groundwater will become contaminated; thus all wellfields must be located inland of the saline wedge.

In recent years much theoretical modelling work has been carried out to investigate the factors which influence both the position and thickness of saline interfaces. Because of the complex nature of saline interfaces a very large quantity of sophisticated field data is required to calibrate these models. In all practical problems, such as the one currently being studied, which are not directed solely at analysing saline interfaces the field data required by these models is not available. Hence a simple mathematical model has been used to locate the saline interface. The ratio between the depth to the saline water (h_s) and the fresh water (h_f), both measured relative to sea level, can be approximated using the Ghyben-Herzberg relation,

$$h_s/h_f = \rho_f/(\rho_s-\rho_f) = 40$$

where ρ_f, the density of fresh water is 1.0 and ρ_s, the density of salt water is 1.025.

This model assumes that the saline interface is sharp. In reality the interface is a transition zone but in the Muqdisho aquifer the sharp interface approximation is acceptable since the width of the transition zone will be small relative to the saturated thickness.

The limited field data available from the Muqdisho aquifer suggests that here the Ghyben-Herzberg proportionality constant is much greater than 40 (Figure 5.9). Within this study a value of 80 has been used which is conservative realtive to the field data. A problem with the Ghyben-Herzberg relation is the dependence of the results on the choice of proportionality constant. For example if there is a fresh water head of 2m five kilometres from the coast and the aquifer base is 140 m below sea level proportionality constants of 80, 120 and 40 would respectively locate the toe of the saline wedge 4.38 km, 2.9 km and 8.75 km inland.

For the Muqdisho area it has only been possible to obtain one field value for the proportionality constant. This value is 112; as a result the Ghyben-Herzberg model with a value of 80 used in this work may overestimate the inland penetration of the saline interface.

Because the Ghyben-Herzberg relation is applied in areas close to the coast, the fresh water heads involved are small. This introduces a further problem, which is that relatively small changes in the fresh water head can result in dramatic differences in the predicted penetration of the saline interface. This can be illustrated by referring back to the example cited previously. If the fresh water head 5 km inland were 2.5 m instead of 2 m this would result in the toe of the interface being located 3.5 km inland rather than 4.38 km. This difficulty is of great relevance to the management model since the predicted drawdowns will be greater than those which will actually occur because of the conservative values used for the hydrogeological parameters.

The factors which control the water table configuration on the recently emerged coastal plain are poorly understood, hence the Ghyben-Herzberg relation is applied inland of this area. At the points where the Ghyben-Herzberg relationship is applied the numerically compatible heads used by the model are in general higher than the true heads. Because of this problem the fresh water heads at these points were obtained by subtracting the predicted drawdowns from the observed water level elevations. Clearly this resulted in lower fresh water heads than if the model values had been used and, because of the problems associated with the Ghyben-Herzberg relation, will tend to cause an overprediction of saline intrusion.

Because of the problems associated with the Ghyben-Herzberg formulation the saline intrusion results must be treated with discretion. It must be stressed that because of the conservative approach used the model will overestimate the penetration of saline water. Despite the problems involved the Ghyben-Herzberg relation was the only way to analyse saline intrusion in this study because of the lack of field data.

8.5.2 Northern Boundary Constraints

Large quantities of clay in the aquifer to the north of the river Shebeelle result in the groundwater in that region being of poor quality. This is due to the presence of clay minerals and also because low flow rates result in long residence times. Overpumping within the study area may result in a reversal of water table gradient under the river. This of course would result in the flow of non potable water into the wellfields and the associated contamination of the water resource. In order to avoid this constraints have been applied to the drawdowns allowed under the river. In estimating the maximum drawdowns which can be allowed under the river, account must be taken of the amount of drawdown which will occur in the area to the north of the river Shebeelle. It has not been possible to quantify this; however because of the lower aquifer transmissivity north of the river it is probable that drawdowns in that region will be greater than in the study area. Since we have been unable to quantify the drawdowns to the north of the river it has been assumed that no drawdown occurs in that region. The northern boundary constraints are based on this assumption - clearly once again the constraints applied are conservative.

8.6 MODEL DEVELOPMENT

8.6.1 Introduction

In this section the numerical experiments, which were carried out with the water management model, are described and the manner in which the results of these experiments are analysed is discussed. The actual results together with an interpretation of their meaning are presented in the chapter 9.

All experiments were executed until 2020, which is beyond the period to which this study was originally confined. This was done in order to ascertain the long term consequences of proposed abstraction regimes. It was decided to terminate the analysis in 2020 since by that year it is likely that a new source of water will have been exploited. The water management modelling assumes that recharge from the river Shebeelle does not change with time. Because of irrigation demands and off-stream storage schemes it is almost certain that this assumption will have been violated by 2020 - this is a further justification for terminating the analysis in that year. The effect of a reduction in river Shebeelle recharge is discussed in chapter 9.4.3.

8.6.2 Verification of 1980 report conclusions

Prior to utilizing the water management model to investigate proposed new wellfields it was used to confirm the final conclusions of the 1980 study. The new distributions of transmissivity and recharge were incorporated, and the reinterpreted storage coefficient values used. The wellfield locations and pumping rates used were those described in Chapter 4 of the 1980 report. Using this input data the management model was executed from 1973 to 2005. The drawdown patterns obtained were compared with those shown in Figures 4.9, 4.10 and 4.11 of the aforementioned report. This comparison revealed close agreement between the two sets of drawdown patterns. Hence, the improved understanding of the Somali coastal aquifer obtained during this study does not alter the conclusions reported in 1980.

8.6.3 Wellfield locations and pumping rates

All wellfields up to and including the Stage IIb, wellfield, are located in the positions proposed in the 1980 report. Prior to 1984 the pumping rates from these wellfields are identical to those in the previous study except that the Stage IIa wellfield is not brought into production. From 1984 onwards the wellfields pump at the rates shown in Table 9.1. These rates are based on the predicted water demand for Muqdisho.

The new wellfield proposed in this study is located in a region of good quality groundwater to the north-east of the Stage IIb Balcad road wellfield. The exact locations of this and other existing and proposed wellfields are shown in Figure 8.2. The new wellfield should not be located closer to the Stage IIb wellfield since the interference effects of the adjacent wellfields will result in excessive drawdowns. This in turn may result in the contamination of the water resource from the plume of poorer quality ground water between Balcad and Afgooye. A further reason for not locating the wellfield adjacent to Stage IIb is that it will intercept a proportion of the groundwater flow to the wellfield and thus may result in a reduction in yield. With the new wellfield in its proposed location it will be accessing groundwater flow not tapped by the Stage IIb wellfield. Location of the Stage III wellfield further east is impractical since this will result in contamination of the water resource from poorer quality water beyond the eastern boundary of the study area. Location further to the north cannot be considered because of the problem of contamination from the poorer quality water to the north of the river Shebeelle and location further to the south will result in excessive inland penetration of the saline interface. It must be stressed that the proposed location of the new wellfield is the optimum location when all hydrogeological constraints are taken into consideration. However it may not be the optimum economic location for the Stage III wellfield. Alternative locations were

Water management model : Wellfield locations

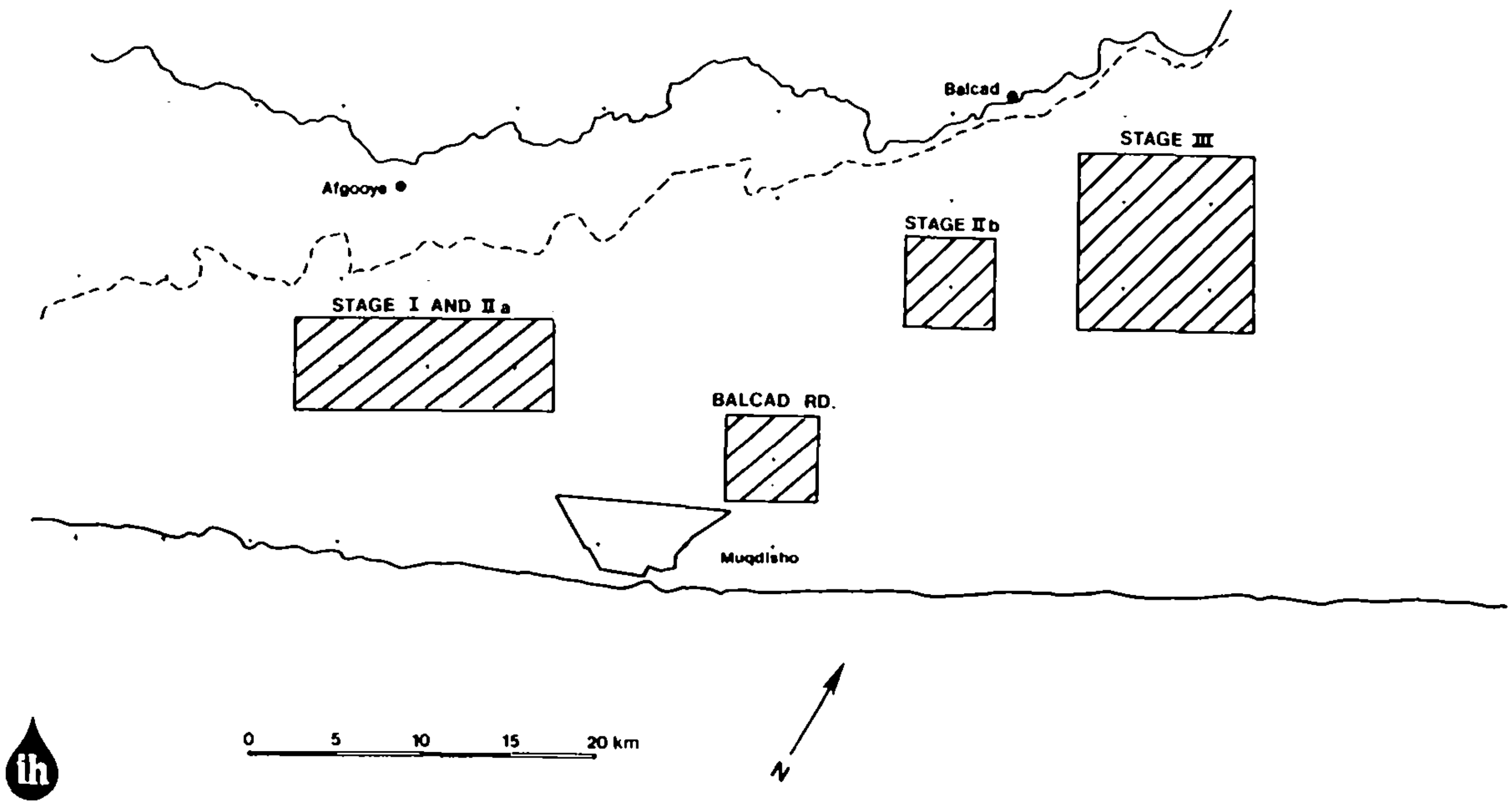


Figure 8·2

considered. These are discussed in Appendix F. The pumping rates from the new wellfield are presented in Table 9.1. These are required to meet the predicted water demand of Muqdisho to and beyond the year 2000. Because of the large quantity of water required from the new wellfield it is spread over four of the management model nodes.

In subsequent sections of this chapter and the next chapter, numerical experiments carried out with the management model are referred to by the pumping rates they use and the final year to which they are run. For example an experiment may be described as using the 1998 pumping rates and run until 2020. This means that up to and including 1998 the abstraction rates presented in Table 9.1 were used, and from 1999 until 2020 the pumping rates are kept at their 1998 values.

8.6.4 Absolute values for water level constraints

The water level constraints used within the water management model are illustrated in Figure 8.3.

The northern boundary constraints are the maximum drawdowns that can be allowed in order to prevent the transport of non-potable groundwater from north of the river Shebeelle. Drawdowns have been restricted to between 8 and 13 m depending upon the quality of water on the far bank (Figure 5.8), being most severe where quality is poorest.

The saline intrusion constraints are illustrated as a line, which is the maximum distance inland to which the toe of the saline interface can be allowed to penetrate. Across most of the study area this distance is 10 km. In the extreme east of the modelled area, however, the saline interface is already approximately 13 km inland so the limit of saline water penetration in this area is set at 15 km. The reason for applying these limits to the saline intrusion is to avoid pollution of major wellfields. Clearly if the toe of the saline interface penetrates 10 km inland a large number of coastal wells will become contaminated. This is an unavoidable consequence of abstracting the proposed quantities of water from the groundwater system.

Water management model : Water table constraints

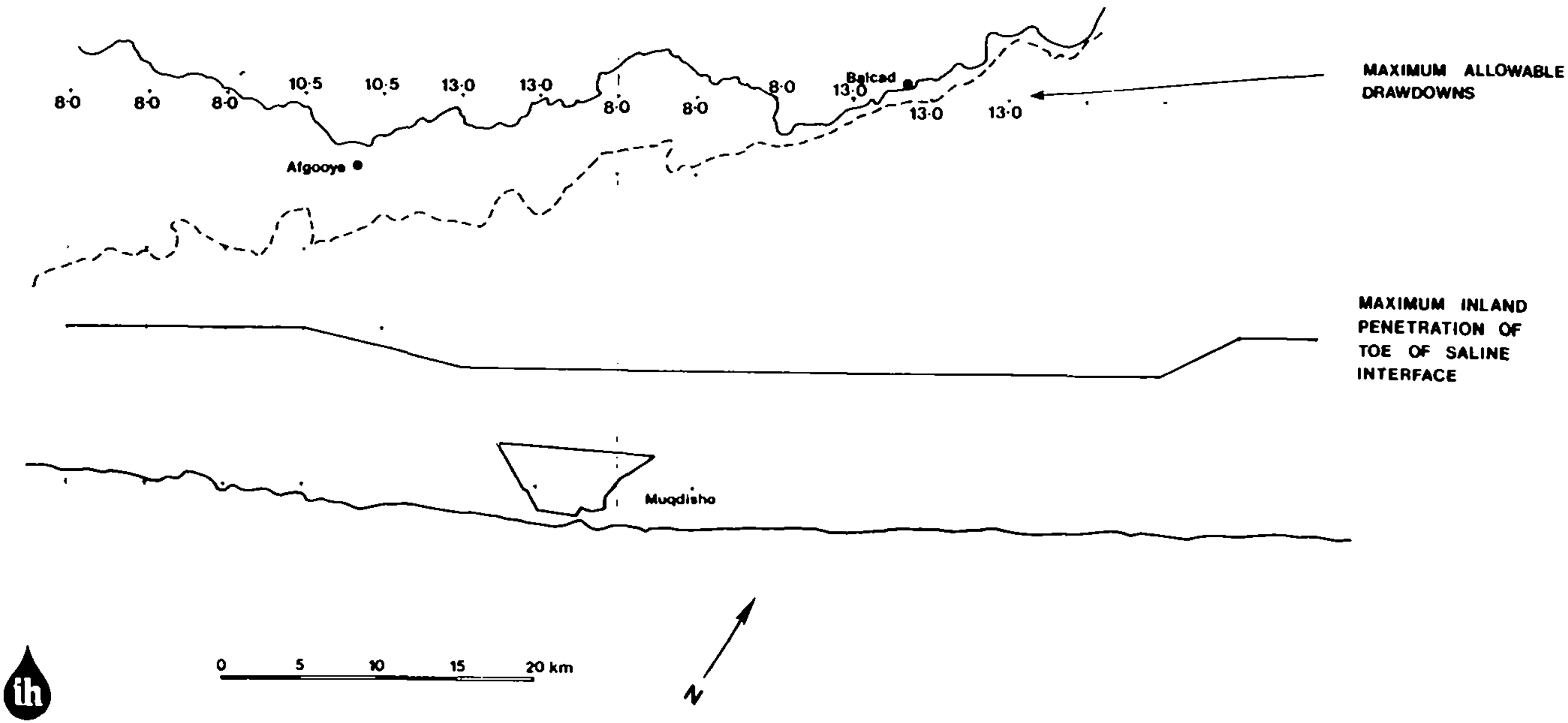


Figure 8·3

8.6.5 The numerical experiments

All water management model experiments were executed until 2020 in order to ascertain the long term effects of various abstraction regimes. It was unknown at the beginning of this study whether or not all the water required by Muqdisho until the year 2000 could be obtained from groundwater. Thus experiments were carried out to investigate the consequences of abstracting various proportions of the predicted demand. This was done by executing model runs which satisfied requirements up to a certain year and then kept the abstraction rates at that year's values until 2020. Numerical experiments were carried out which satisfied the requirements for each year from 1989 until 2000 and then maintained the appropriate abstraction rates until 2020. The results of this work together with examples of the drawdown patterns obtained are shown in chapter 9.

From experience we know that when large quantities of water are introduced into a non or poorly sewered urban area such as Muqdisho a proportion of the water is recharged to the aquifer within that area. Work we have carried out in Tehran (Iran) and Doha (Qatar) indicates that this proportion is approximately 35%. In order to investigate this phenomenon all the aforementioned numerical experiments were repeated with 35% of the total abstraction being allowed to recharge at the management model nodes which correspond to Muqdisho. The results of these experiments together with typical drawdown patterns are shown in the next chapter.

8.6.6 Experiment acceptance criteria

Each numerical experiment produces a large amount of data in the form of water table elevations, predicted drawdowns and inland penetrations of the toe of the saline interface. Clearly criteria have to be devised which will allow a decision to be made about whether the abstraction rates incorporated within an experiment are acceptable. Since the only constraints on the abstraction rates are the northern boundary and saline intrusion constraints it is evident that the acceptance criteria have to be based on these.

In order to obtain the acceptance criteria the model predicted drawdowns for 1980 were compared with those observed in the field at both the Balcad road and Stage I wellfields. This illustrated the model approach to be over predicting drawdowns by 25 to over 50 per cent because of the conservative values used for all hydrogeological parameters. It was not practical to recalibrate the water management model since accurate field estimates of drawdown were only available at two locations. If drawdowns which are 80 per cent of those predicted by the model do not violate any of the water table constraints the experiment is accepted. If any constraints are violated by drawdowns which are 60 per cent of the model predicted drawdowns the experiment is rejected. Any experiments falling within these limits are provisionally accepted. The abstraction rates which fall into these categories are respectively those which present no problems of groundwater contamination; those which are almost certain to result in a deterioration of groundwater quality in the modelled area; and those which are exploiting the aquifer's potential to the full and thus may, unless carefully managed, result in groundwater contamination. The acceptance criteria are applied rigidly. Discretion must be used in interpreting the conclusions of this analysis since it is dependent on the choice of water level constraints and the acceptance criteria. These are the best estimate based on the sparse data available.

The results of the acceptance criteria analysis are presented in the form of a decision matrix in chapter 9.

SECTION 4

GROUND AND SURFACE WATER RESOURCES AND CONCLUSIONS

9. WATER RESOURCES: GROUNDWATER

9.1 INTRODUCTION

In this chapter we assess the extent to which groundwater resources can meet the projected water demands of Muqdisho up to the year 2000. These demands are presented in Table 9.1. Up to, and including 1989, all requirements are to be met by existing or planned well fields, but from this time onward a shortfall will progressively build up culminating in an annual deficit of 32.7 million m^3 by the year 2000. We have been required to assess whether this shortfall can be met by the installation of a further wellfield or series of wellfields, within a reasonable distance of the city.

In order to do this a time-varying computer model designed to simulate the effects of various abstraction regimes upon the water table, has been developed (see Chapter 8).

Part of the future water demand is to be met by existing and planned wellfields on the Balcad and Afgooye roads. Abstraction from these are simulated using the pumping schedules shown in Table 9.1. New sources, intended to make up the shortfall, are not introduced until 1990. When these are incorporated, abstraction is built-up from year to year as shown in Table 9.1. Successive model runs for a series of wellfield locations were executed. The optimum hydrogeological location is described in Section 8.6.3 and shown in Figure 8.2. Alternative wellfield locations are described in Appendix F.

Although the problem is concerned essentially with meeting demand up to the year 2000 we have extended our model studies to investigate the impact of abstraction up to 2020. This is to take account of the fact that we are dealing with a dynamic situation. The decline of water levels will not stop in 2000 and it is important to know the scale of decline to be expected after this time. In this way an abstraction scheme can be chosen that will minimise damage to the aquifer until at least to the year 2020.

9.2 THE PRESENTATION OF THE RESULTS

The results of our resource investigations have been presented in the form of a series of decision matrices, rather than a unique solution. This is done

TABLE 9.1

MUQDISHO WATER SOURCE ABSTRACTIONS

Year	All Sources		Afgoi Road Stage I and IIa		Balad Road		Garas Bintow Stage IIb		Future Sources Stage III	
	Total	Maximum	Total	Maximum	Total	Maximum	Total	Maximum	Total	Maximum
1984 †	15926	54540	10512	41760	5414	12780				
1985	17706	60640	12089	41760	5627	18880				-
1986	19240	65890	12089	41760	7151	24130	-	-	-	-
1987	20907	71600	10231	35190	3559	10920	7117	25490	-	-
1988	22718	77800	11117	38240	3867	11870	7734	27690	-	-
1989	24688	84550	12081	41560	4202	12900	8405	30090	-	-
1990	26824	91860	12089	41760	4205	12960	8410	30240	2120	6900
1991	28995	99300	12089	41760	4205	12960	8410	30240	4291	14340
1992	31342	107340	12089	41760	4205	12960	8410	30240	6638	22380
1993	33879	116020	12089	41760	4205	12960	8410	30240	9175	31060
1994	36621	125410	12089	41760	4205	12960	8410	30240	11917	40450
1995	39585	135570	12089	41760	4205	12960	8410	30240	14881	50610
1996	42643	146040	12089	41760	4205	12960	8410	30240	17939	61080
1997	45937	157320	12089	41760	4205	12960	8410	30240	21223	72360
1998	49486	169470	12089	41760	4205	12960	8410	30240	24782	84510
1999	53308	182560	12089	41760	4205	12960	8410	30240	28604	97600
2000 *	57426	196660	12089	41760	4205	12960	8410	30240	32722	111700

Total = total volume abstracted in the year (in thousands of cubic metres.)

Maximum = maximum volume abstracted in any day in the year (in cubic metres.)

* Beyond the year 2000 abstraction rates are maintained at the 2000 level.

† Prior to 1984 the abstraction schedule is that given in our 1980 report.

because the problems posed by the terms of reference cannot be satisfied with a single answer.

The solution eventually chosen will need to be made in the context of an overall water policy. Essentially the amount of water that can be abstracted depends upon the degree of aquifer contamination that can be tolerated. Contamination will result from the ingress of saline water from the coast, and poor quality water from the northern and western banks of the river, in response to falling water levels.

Where the limits are to be drawn is a decision that involves factors other than those of a hydrogeological nature. For this reason we have, in this chapter, put forward the various options open in order to help provide the basis for the formulation of a sound abstraction strategy.

9.3 THE DECISION MATRICES: INTERPRETATION

9.3.1 The Constraints

The results of our management model runs are given in Tables 9.2 to 9.7. These represent a series of 6 decision matrices which are a straightforward way of showing whether a particular abstraction rate is accepted or rejected.

Acceptance or rejection is expressed in terms of whether or not certain constraints imposed upon the model are exceeded. The constraints involved are drawdown constraints, which have been introduced along two boundaries. These are:-

1. The Northern Boundary, where we have set limits to the drawdown allowable along the groundwater mound beneath the river. This has been done to prevent movement of poor quality water from the northern and western side of the river. Drawdowns have been restricted to between 8 and 13 m depending upon the quality of water on the far bank, being most severe where quality is poorest (see Section 8.5.2 and 8.6.4). Where these values are exceeded the model run is rejected.
2. The coast, where drawdowns have been restricted to those which allow a movement of the toe of the saline interface no more than 10 km inland, except in the extreme north east, where a 15 km ingress is permitted (see Section 8.5.1 and 8.6.4). Again where these constraints are violated, the run is rejected.

Table 9.2

Pumping Only - No Muqdisho recharge (summary)

Abstraction Rates	To 2010	To 2020
1989 (24.688 M m^3/annum)		
1990 (26.824 M m^3/annum)		
1991 (28.995 M m^3/annum)		
1992 (31.342 M m^3/annum)		
1993 (33.879 M m^3/annum)		
1994 (36.621 M m^3/annum)		
1995 (39.585 M m^3/annum)		
1996 (42.643 M m^3/annum)		
1997 (45.937 M m^3/annum)		
1998 (49.486 M m^3/annum)		
1999 (53.308 M m^3/annum)		
2000 (57.426 M m^3/annum)		

Accept (80%) Provisionally Accept (60%) Reject

Table 9.3

Pumping Only - No Muqdisho recharge (Northern Boundary)

	To 2010	To 2020
1989 (24.688 M m^3/annum)		
1990 (26.824 M m^3/annum)		
1991 (28.995 M m^3/annum)		
1992 (31.342 M m^3/annum)		
1993 (33.879 M m^3/annum)		
1994 (36.621 M m^3/annum)		
1995 (39.585 M m^3/annum)		
1996 (42.643 M m^3/annum)		
1997 (45.937 M m^3/annum)		
1998 (49.486 M m^3/annum)		
1999 (53.308 M m^3/annum)		
2000 (57.426 M m^3/annum)		

Accept (80%) Provisionally Accept (60%) Reject

Table 9.4

Pumping Only - No Muqdisho recharge (Saline Intrusion)

	To 2010	To 2020
1989 (24.688 M m^3/annum)		
1990 (26.824 M m^3/annum)		
1991 (28.995 M m^3/annum)		
1992 (31.342 M m^3/annum)		
1993 (33.879 M m^3/annum)		
1994 (36.621 M m^3/annum)		
1995 (39.585 M m^3/annum)		
1996 (42.643 M m^3/annum)		
1997 (45.937 M m^3/annum)		
1998 (49.486 M m^3/annum)		
1999 (53.308 M m^3/annum)		
2000 (57.426 M m^3/annum)		

Accept (80%) Provisionally Accept (60%) Reject

Table 9.5

Pumping with 35% Recharge at Muqdisho (Summary)

	To 2010	To 2020
1989 (24.688 M m^3/annum)	Accept	Accept
1990 (26.824 M m^3/annum)	Accept	Accept
1991 (28.995 M m^3/annum)	Accept	Accept
1992 (31.342 M m^3/annum)	Accept	Accept
1993 (33.879 M m^3/annum)	Accept	Accept
1994 (36.621 M m^3/annum)	Accept	Provisionally Accept
1995 (39.585 M m^3/annum)	Accept	Provisionally Accept
1996 (42.643 M m^3/annum)	Accept	Provisionally Accept
1997 (45.937 M m^3/annum)	Provisionally Accept	Provisionally Accept
1998 (49.486 M m^3/annum)	Provisionally Accept	Reject
1999 (53.308 M m^3/annum)	Provisionally Accept	Reject
2000 (57.426 M m^3/annum)	Provisionally Accept	Reject

Accept (80%)   Provisionally Accept (60%)   Reject

Table 9.6

Pumping with 35% Recharge at Muqdisho (Northern Boundary)

	To 2010	To 2020
1989 (24.688 M m^3/annum)		
1990 (26.824 M m^3/annum)		
1991 (28.995 M m^3/annum)		
1992 (31.342 M m^3/annum)		
1993 (33.879 M m^3/annum)		
1994 (36.621 M m^3/annum)		
1995 (39.585 M m^3/annum)		
1996 (42.643 M m^3/annum)		
1997 (45.937 M m^3/annum)		
1998 (49.486 M m^3/annum)		
1999 (53.308 M m^3/annum)		
2000 (57.426 M m^3/annum)		

Accept (80%)   Provisionally Accept (60%)   Reject

Table 9.7

Pumping with 35% Recharge at Muqdisho (Saline Intrusion)

	To 2010	To 2020
1989 (24.688 M m^3/annum)		
1990 (26.824 M m^3/annum)		
1991 (28.995 M m^3/annum)		
1992 (31.342 M m^3/annum)		
1993 (33.879 M m^3/annum)		
1994 (36.621 M m^3/annum)		
1995 (39.585 M m^3/annum)		
1996 (42.643 M m^3/annum)		
1997 (45.937 M m^3/annum)		
1998 (49.486 M m^3/annum)		
1999 (53.308 M m^3/annum)		
2000 (57.426 M m^3/annum)		

Accept (80%) Provisionally Accept (60%) Reject

The matrices are broken down and presented in such a way that it is possible to identify which constraint is being violated.

9.3.2 Acceptance - rejection criteria

Comparison with a small number of observed values at the Balcad road and Stage I wellfields has revealed a tendency for the model to overestimate drawdowns (see Section 8.6.6). Bearing this in mind, we felt that to use the constraint violations caused by model predicted drawdown, as a basis for rejection, would be unduly pessimistic. Thus, for reasons explained in Section 8.6.6, acceptance criteria have been based on whether or not certain proportions of the model predicted drawdowns cause the water table constraints to be violated. These acceptance criteria are:-

1. If no constraints are violated by drawdowns which are 80 per cent of the model predicted values, the model run is unconditionally accepted. No aquifer contamination will result.
2. If water table constraints are violated by drawdown at 80 per cent of model predicted values but not at the 60 per cent level, then the run is provisionally accepted. But here we are exploiting the aquifer to its limit, and unless a careful management policy is adopted localised contamination may occur.
3. If constraints are violated by drawdowns at the 60 per cent level of predicted model values, the run is rejected. Under these circumstances there is potential for widespread aquifer contamination.

9.3.3 Inclusion - exclusion of Muqdisho recharge

Two sets of three matrices are presented. One set is based on the assumption that water imported into Muqdisho does not recharge the aquifer, but is all transported, after use, beyond the study area. A second, more realistic set, assumes on the other hand that up to 35 per cent of imported water will be recharged to the aquifer via septic tanks, municipal irrigation, leaking water pipes and so on, in the manner described in Section 8.6.5. The figure of 35 per cent is derived from our experience in other non or poorly sewered urban areas, specifically Doha (Qatar) and Tehran (Iran).

9.3.4 Some examples

Interpretation of the decision matrices presented in Tables 9.2 - 9.7 can best be explained by reference to some specific examples. Let us first consider a case where abstraction is being made at the 1996 rate, with recharge taking place beneath Muqdisho. Results under these circumstances are summarised in Table 9.5. By referring to abstraction at the 1996 rate, we mean that the abstraction schedule shown in Table 9.1 is adopted up to the point where the 1996 levels are obtained. After this time pumping rates are maintained at the 1996 level, shown on Table 9.5 to be 42.64 million m^3/year. The table shows that abstraction at this level is acceptable at the 80 per cent level up to the year 2010, but violations of northern boundary constraints reduce this to a 60 per cent level acceptance by 2020.

Thus, in this case we could provisionally accept the abstraction rate up to and beyond the year 2020.

If on the other hand we consider an abstraction rate at the 2000 level, which amounts to 57.4 million m^3/year a different picture emerges. Here there is a provisional acceptance to the year 2010, but beyond this time the run is rejected. Reference to Tables 9.6 and 9.7 shows that once again it is the northern boundary constraints that are violated. With recharge taking place at Muqdisho, no coastal constraints are violated at all even at the 80 per cent acceptance level.

9.4 THE DECISION MATRICES: RESULTS

9.4.1 Introduction

In this section we present and discuss, in two stages, the results summarised in Tables 9.2 to 9.7. Firstly the 1998 pumping rate is taken as a specific example to demonstrate the way in which resulting cones of depression develop from year to year and how constraints are eventually violated. Secondly, we move on to discuss the implications of the overall results and provide some broad guidelines relating to the selection of a sound pumping strategy.

9.4.2 Development of a typical cone of depression

The drawdown pattern resulting from abstractions at the 1998 rate, for the years 1990, 1998, 2010 and 2020, are illustrated in Figures 9.1 to 9.10. The drawdowns shown are these using 60 per cent of model predicted values. Two sets of conditions are simulated; the first set (Figures 9.1 to 9.4) represents drawdown with no recharge at Muqdisho, while the second (Figures 9.5 to 9.8) assumes recharge to be taking place as discussed in section 9.3.3.

Without recharge, Figures 9.1 to 9.4 show that by 2010 no constraints have been exceeded even though water levels over much of the eastern part of the area have been lowered by more than 5 m and locally over 10 m. Thus Table 9.2 shows the abstraction rate to be provisionally acceptable up to this time. But by 2020 continued lowering of regional water levels causes the northern boundary constraints to be exceeded at one point. This results in rejection as shown in Figure 9.4 and Table 9.2. At the same time coastal constraints, while not exceeded, are closely approached, especially over the western part of the area, and the saline interface has moved a considerable distance inland (Figure 9.4). By 2020 therefore, the aquifer, under these conditions, has been pushed beyond its limit.

The identical situation, but with recharge taking place at Muqdisho, is recorded in Figures 9.5 to 9.8. Here recharge leads to the build up of a mound beneath the city and has the effect of significantly reducing regional drawdowns. In particular the introduction of the recharge component helps reduce coastal drawdowns and prevents significant inland movement of the saline interface. Up to 2010 no constraints are exceeded but by 2020, however, even the extra recharge is unable to prevent violation of northern boundary constraints (Figure 9.8). Once again levels are exceeded at only one point but is sufficient to merit rejection of the model run. Nonetheless, with the extra recharge component, it is clear that the pressure on the aquifer is significantly eased, particularly in coastal regions.

If a more rigorous 80 per cent acceptance level is adopted, then drawdown patterns for the year 2020, without and with recharge, are those shown in Figure 9.9 and 9.10 respectively. Without recharge, northern boundary constraints are exceeded at six points, while the toe of the saline interface is drawn dangerously close to the Stage I and IIA wellfield. The situation is clearly

Drawdown response at the 1998 rate of abstraction at the 60% level of acceptance with no extra recharge

[figs 9·1 to 9·4]

1990

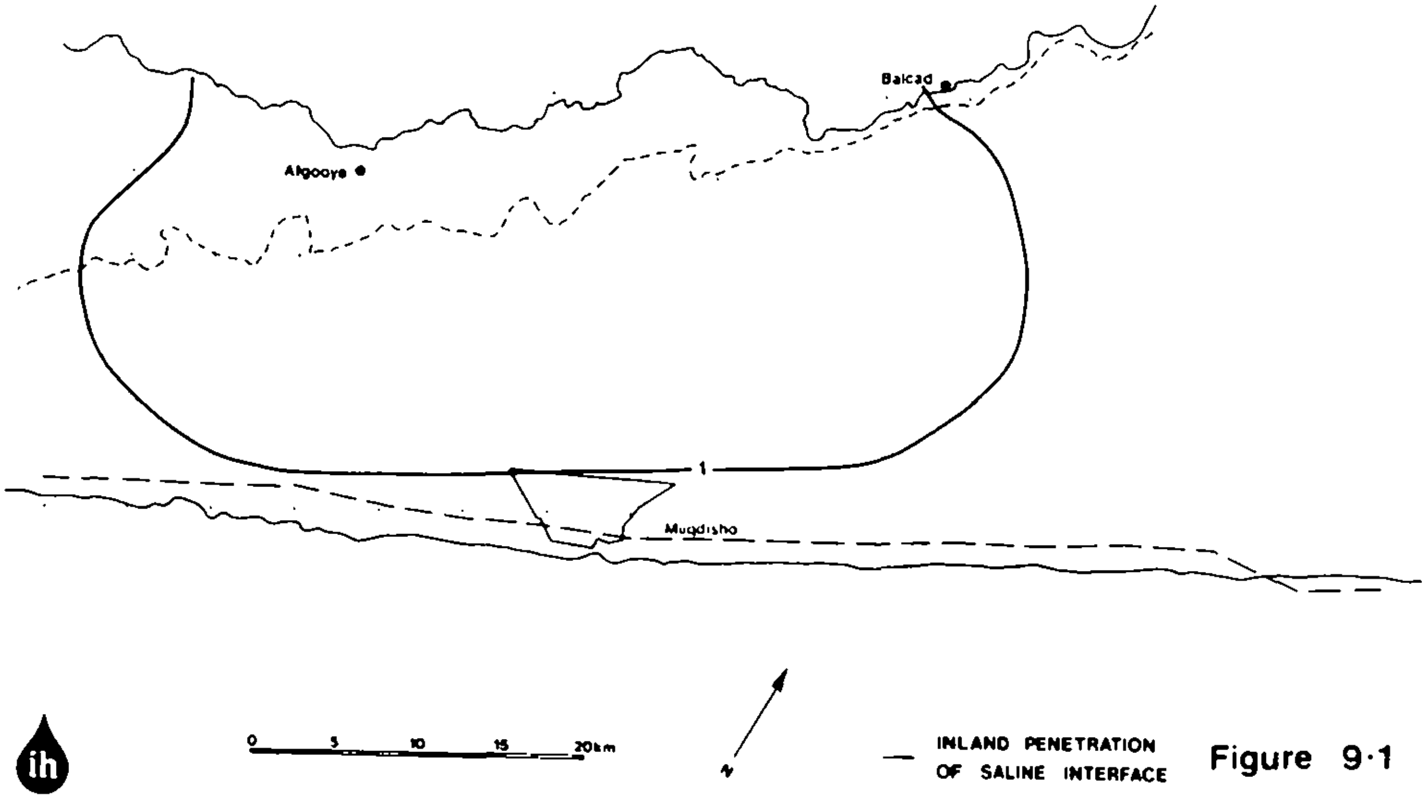


Figure 9·1

1998

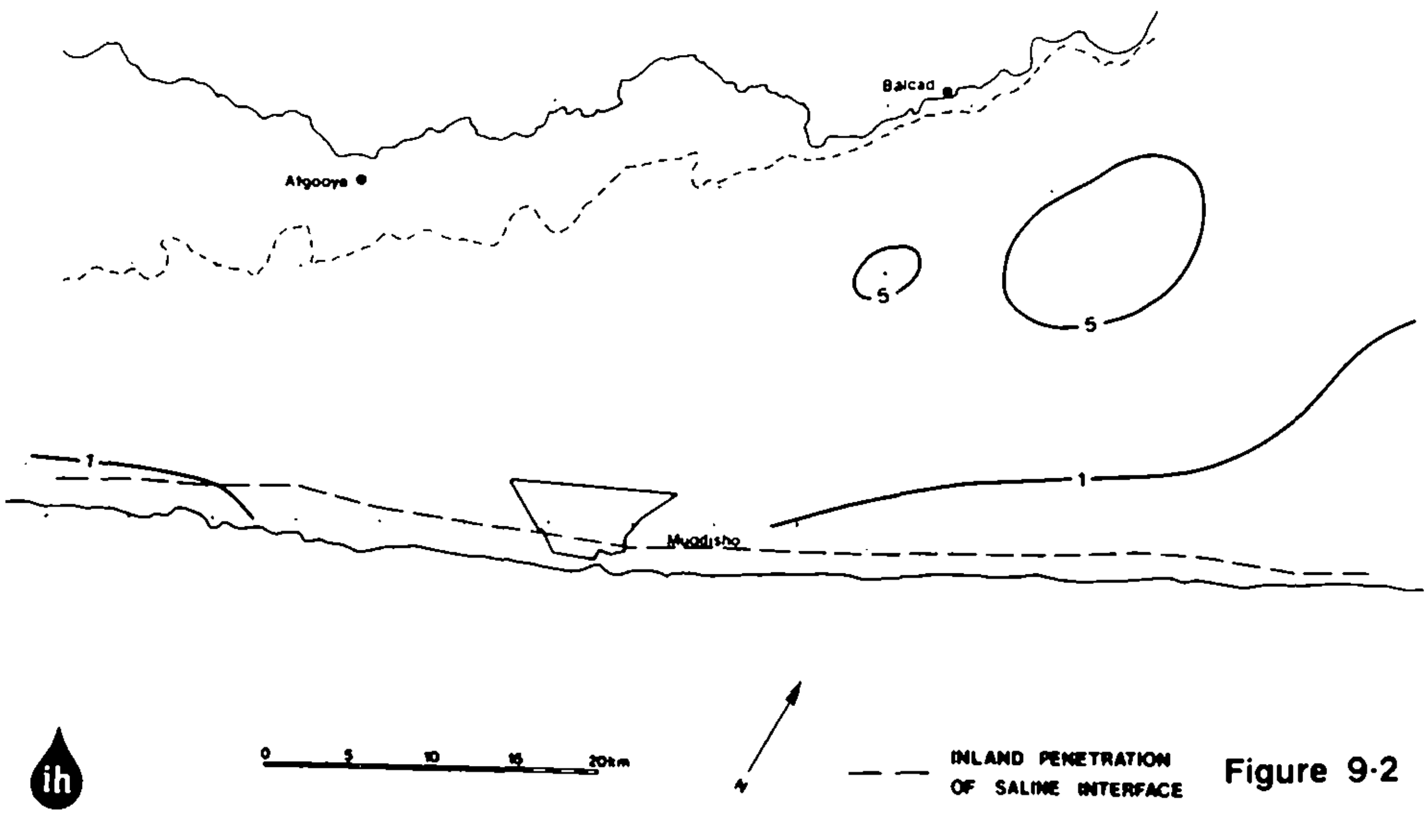
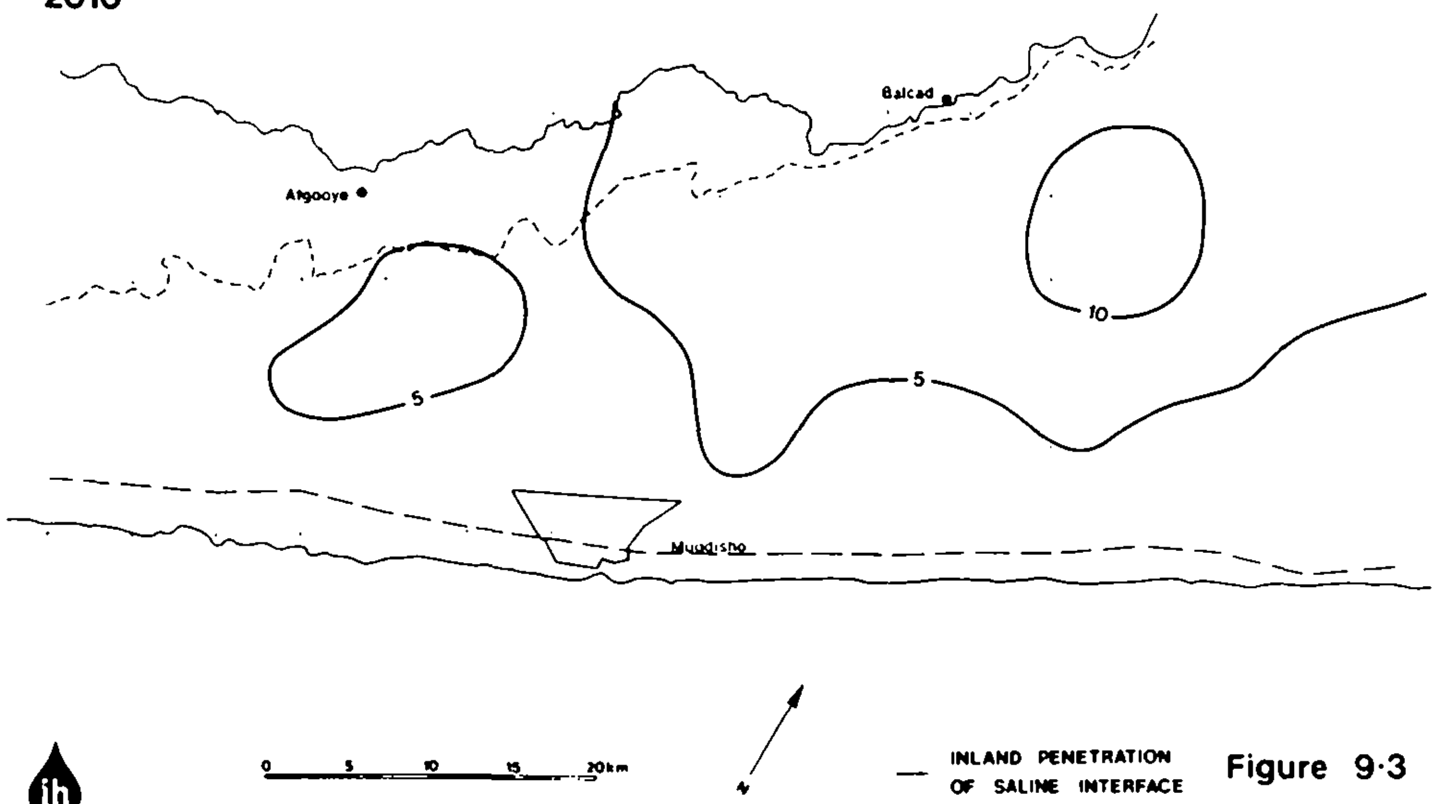


Figure 9·2

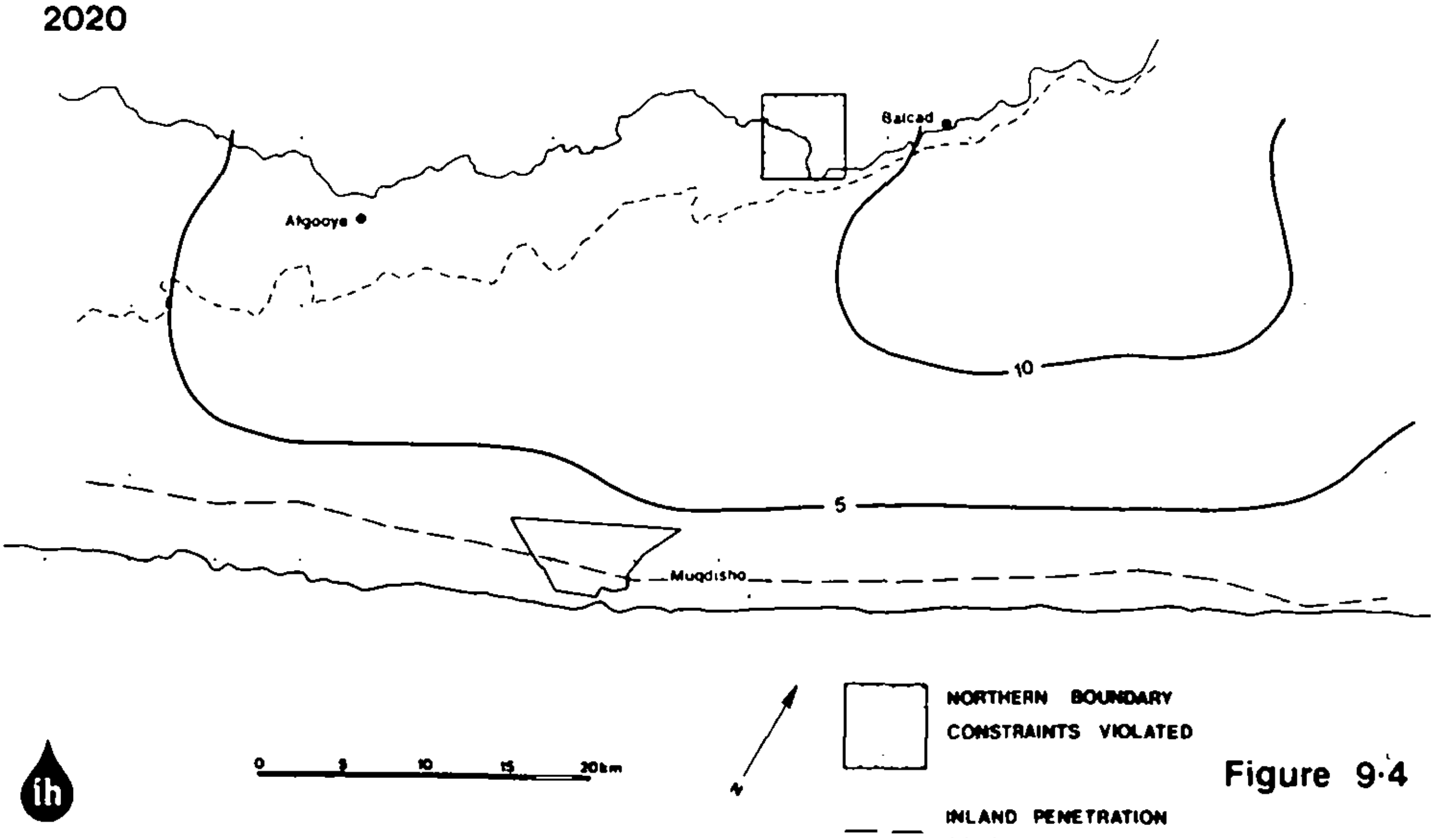
2010



INLAND PENETRATION OF SALINE INTERFACE

Figure 9·3

2020



NORTHERN BOUNDARY CONSTRAINTS VIOLATED

INLAND PENETRATION OF SALINE INTERFACE

Figure 9·4

Drawdown response at the 1998 rate of abstraction at the 60% level of acceptance with extra recharge

(figs 9·5 to 9·8)

1990

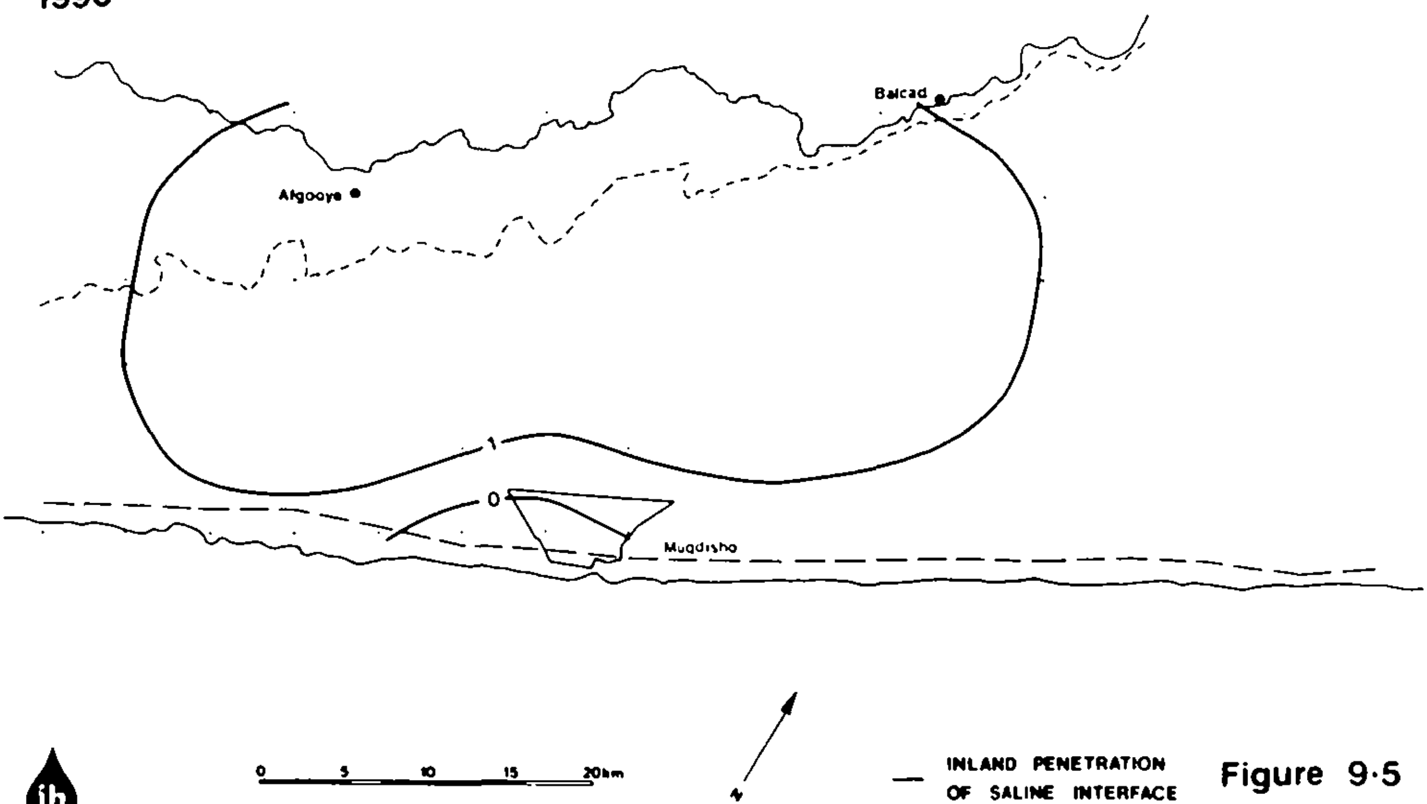


Figure 9·5

1998

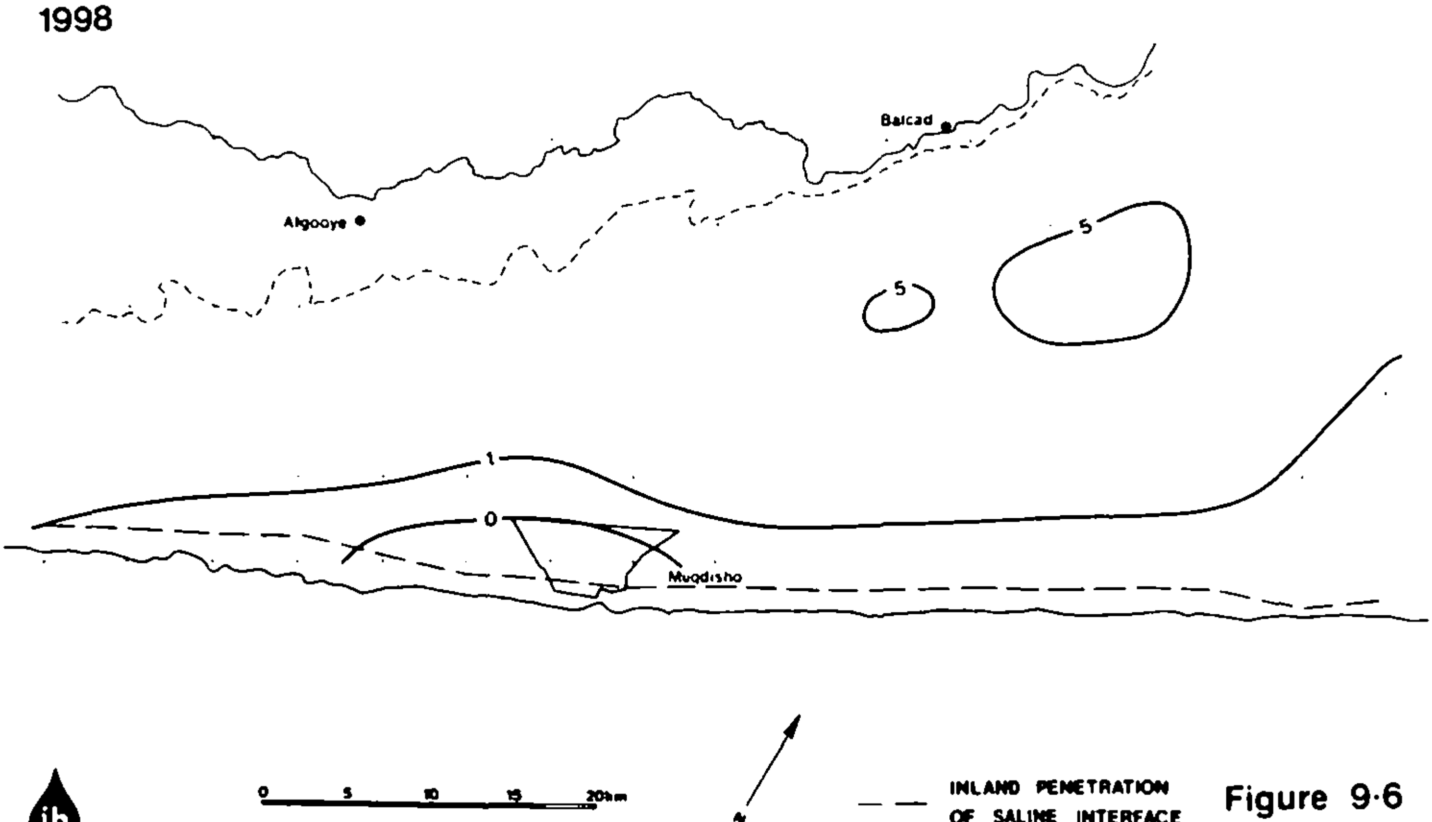


Figure 9·6

2010

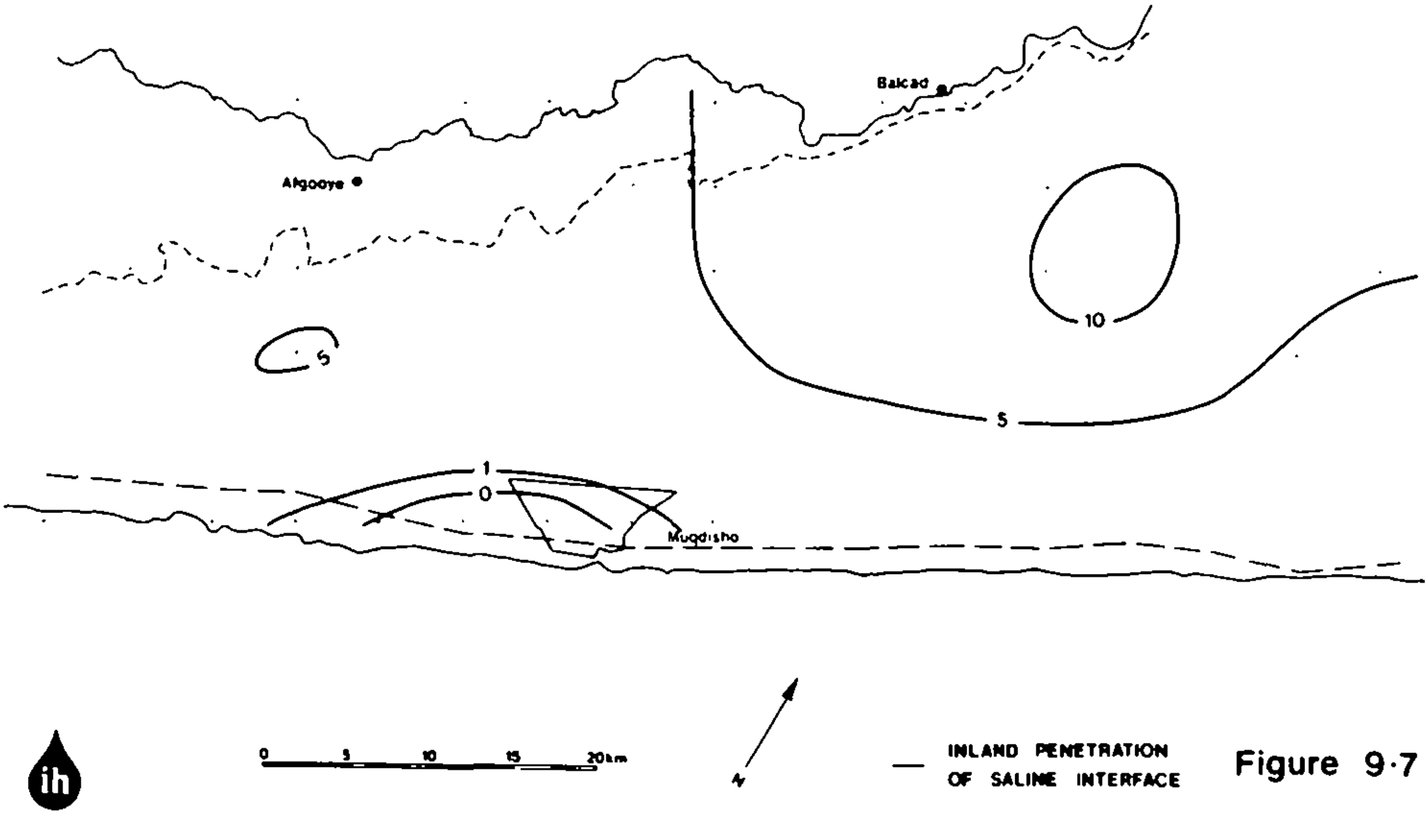


Figure 9·7

2020

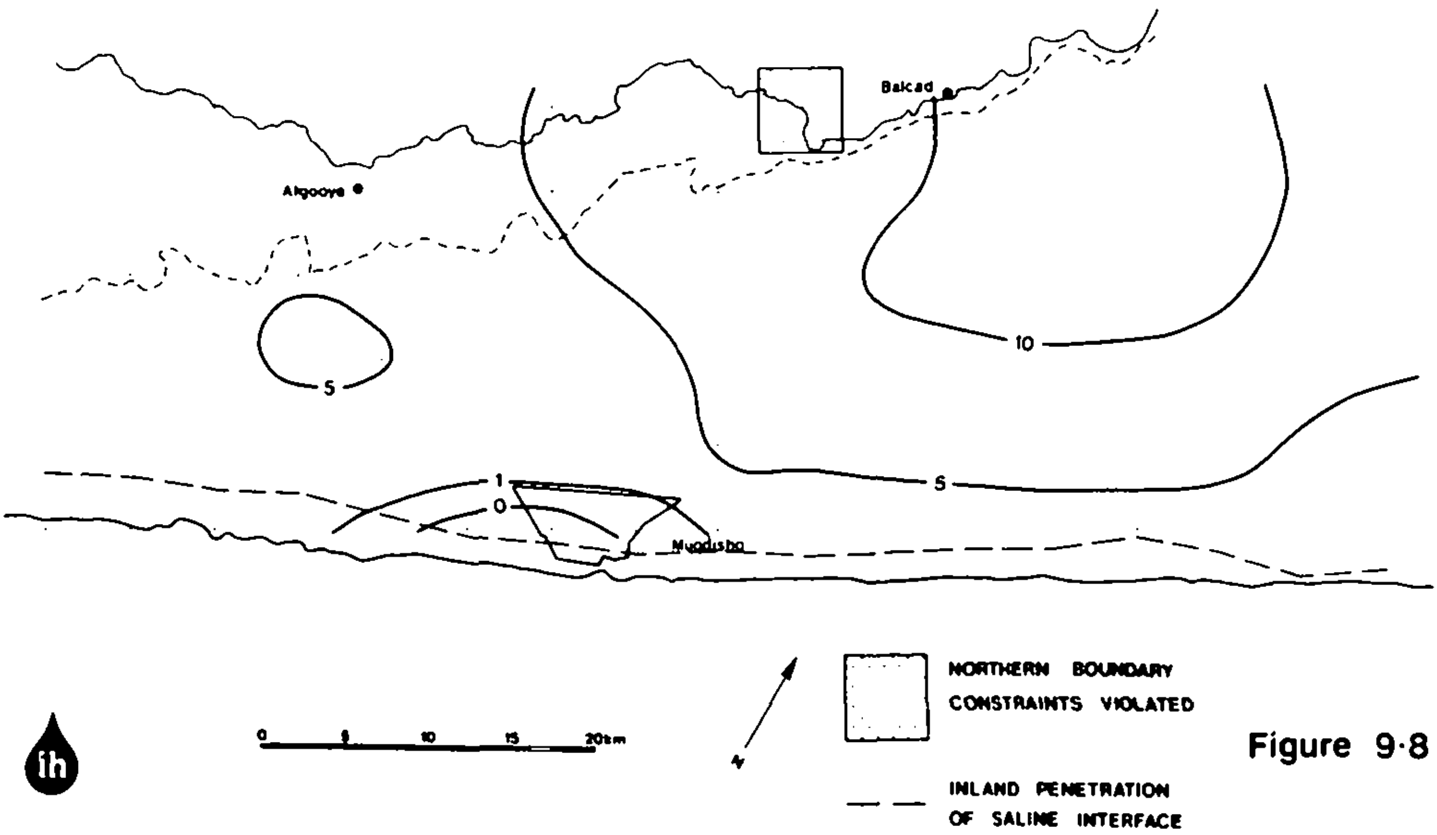


Figure 9·8

Drawdown response at the 1998 rate of abstraction at the 80% level of acceptance for the year 2020 [figs 9·9 to 9·10]

with no extra recharge

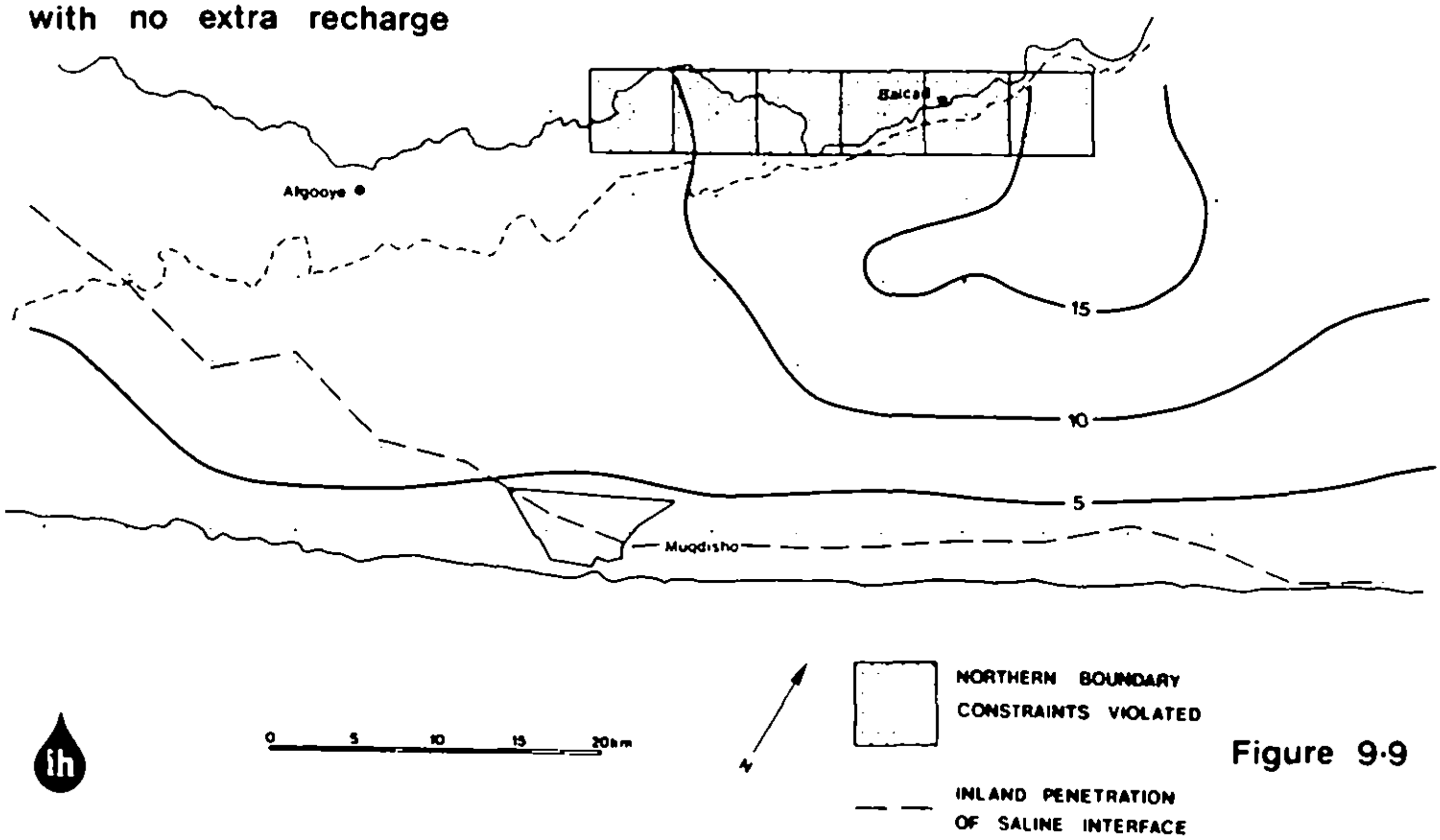


Figure 9·9

with extra recharge

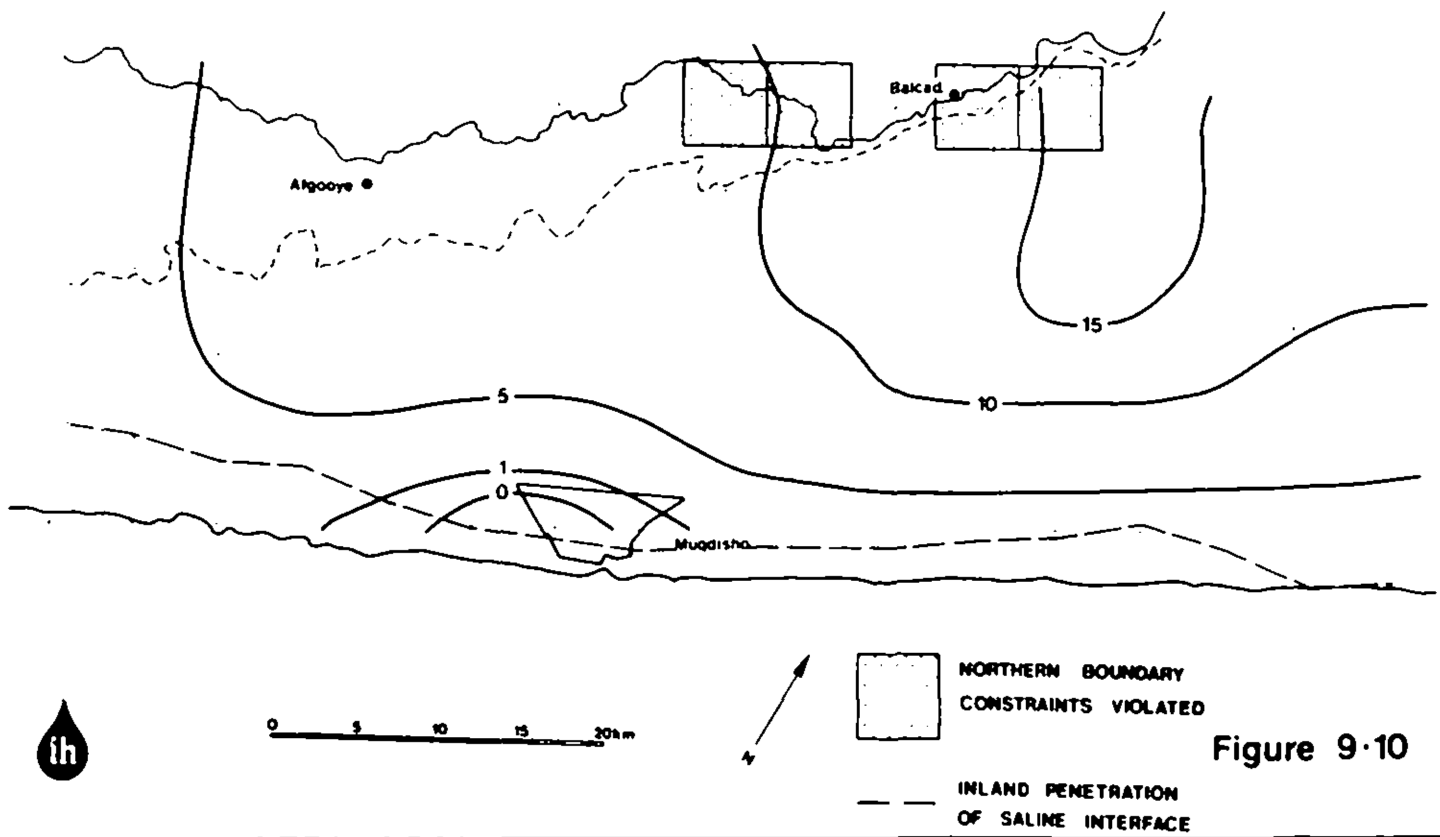


Figure 9·10

not acceptable, and is indicated as such in Table 9.2. Where recharge does take place the picture is improved, as shown in Figure 9.10. Here the northern boundary constraints are exceeded marginally at 4 points, while the recharge mound developed beneath the city succeeds in preventing excessive inland movement of the saline interface. Once again, however, because of northern boundary constraint violations, the situation is not acceptable.

9.4.3 Implications of the overall results

Overall the matrices demonstrate that for most abstraction regimes, with or without recharge at Muqdisho, the limiting factor is the violation of constraints along the northern boundary. Saline intrusion along the coast does not appear to pose a problem since even in the worst case with pumping at the 2000 rate (57.42 million m^3/year) and no recharge at Muqdisho, drawdowns at the 60 per cent acceptance level do not exceed the given coastal constraints (Table 9.4). Thus at the 60 per cent level, no abstraction scheme need be rejected purely in terms of saline intrusion violations.

By far the most severe limits on abstraction are provided by drawdown constraints applied along the northern boundary, beneath the river. Unlike drawdown along the coast those beneath the river are not significantly reduced by the introduction of extra recharge in the vicinity of Muqdisho. Tables 9.3 and 9.6 show that only slightly higher abstraction rates can be tolerated with extra recharge taking place. This applies to both the 60 per cent rejection and the 80 per cent acceptance levels.

On balance we feel that, even with future sewering of the city, recharge will inevitably take place beneath Muqdisho, and consequently recommend that Table 9.5 be used to select a suitable abstraction rate.

We further recommend that if there are any plans to exploit the aquifer significantly beyond the year 2020, then only solutions at the 80 per cent level should be considered. This in effect means restricting abstraction to the 1993 rates, equivalent to 33.87 million m^3/year. If on the other hand short term pressures are sufficiently severe for conservation of the aquifer beyond the year 2020 not to be a major consideration, then any rate up to a maximum of 45.9 million m^3/year, the 1997 rate, can be adopted. Selection of a rate intermediate to these two extremes may provide an attractive compromise.

Finally we feel that two points in particular need to be emphasised. Firstly, the results of all our work are based on the assumption that recharge from the Shebeelle will be maintained at its present level. If however this source of recharge were to be significantly reduced in the future, the effect upon regional groundwater levels would be very serious.

To emphasise this point a model run was undertaken in which river recharge was reduced by half, with no pumping taking place from the aquifer. In this way we measured the natural decay rate of the water table recharge mound beneath the river under conditions of 50 per cent less recharge. The results showed that, even without abstraction regional water levels are lowered sufficiently to exceed constraints beneath the river by the year 2000. Clearly with abstraction this stage would be reached at an even earlier time. Thus the importance of maintaining present recharge rates cannot be overemphasised. However, comfort can be drawn from the fact that the current practice of increasing areas of irrigation and the installation of offstream storage schemes is likely to lead to an overall increase, rather than a decrease, in recharge.

A second and final point to stress, is the necessity to have in place, before the commencement of any abstraction scheme, a network of observation wells to monitor water levels and quality at sensitive locations. In particular the position of the saline interface between the coast and stage IIA wellfield will need to be carefully recorded, as well as water levels adjacent and beneath the river in the vicinity of the Stage IIB wellfield. At these places water quality and water level monitoring needs to be undertaken at least once every six months in order to give adequate warning of contamination.

This applies particularly if an abstraction scheme is selected which falls into the 'provisionally accepted' category. However, under any circumstances we would wish to stress the importance of monitoring water levels and quality no matter which scheme is eventually adopted.

10. HYDROLOGY

This chapter describes the surface water studies carried out to estimate the available resource of the Shebeelle and the effect of future abstractions on this resource.

10.1 THE AVAILABLE DATA

10.1.1 Climate of the study area

The climate of Southern Somalia, including the study area, is tropical and semi-arid with a bi-modal rainfall pattern influenced by monsoon winds.

(i) Rainfall

The rainfall occurs during two seasons, the Gu from March to May and the Der from September to November. Occasionally the Gu season is extended into June or July, in coastal regions, by the Haggai rains which are produced by the onset of moist onshore winds. The Gu and Der rains are caused by the passage of the Inter Tropical Convergence Zone (ITCZ) where the surface winds of the northern and southern hemispheres meet. The ITCZ is thus a zone of low pressure and considerable atmospheric instability. This instability causes rain to fall in isolated storm cells which result in an extremely irregular distribution, both areally and from year to year.

Long term mean monthly rainfall records are listed in Table 10.1 for Jawhar, Balcad and Afgooye (Hunting 1977) together with the effective rainfall, described in section 10.2.

(ii) Wind

The annual pattern of wind direction is also controlled by the movement of the ITCZ. From May to September, when the ITCZ is 15° south, the winds blow from the southwest; and from December to February, when the ITCZ is 15° north, the winds blow predominantly from the northeast. During the transitional periods, the wind drops and becomes erratic in direction.

Table 10.1 Rainfall and effective rainfall (ER) (mm)

Long term mean rainfall	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	Total
Jawhar (1921-60)	5.5	1.3	21.7	94.4	87.1	24.5	25.6	16.1	11.7	104.1	83.5	21.1	496.6
Balcad (1922-58)	8.7	2.3	9.6	93.6	68.8	36.2	21.7	11.9	16.2	75.1	93.9	30.4	468.4
Afgooye (1922-75)	2.2	2.6	5.1	82.1	85.0	57.1	54.2	23.5	7.0	55.3	91.4	28.8	494.3

Source: Arab Organisation for Agricultural Development (1977)

Effective rainfall													
Jawhar	5.0	1.2	19.5	70.8	65.3	22.1	22.4	14.5	10.5	68.7	62.6	19.0	381.6
Balcad	7.8	2.1	8.6	70.2	57.3	31.7	19.6	10.7	14.6	56.3	70.4	26.6	375.9
Afgooye	2.0	2.3	4.6	61.6	63.8	47.6	45.1	21.1	6.3	48.4	68.6	25.0	396.5

(iii) Temperature

The mean daily temperature is very constant throughout the year, the hottest months March and April being only a few degrees warmer than the coolest months July and August. At Afgooye, the mean daily temperature for the period 1953 to 1976 ranges from 25.2°C in July to 28.8°C in March with an annual mean of 27.0°C. However, the diurnal temperature fluctuations are much greater and can range from 20° to 35°C.

(iv) Humidity

The pattern of mean daily relative humidity follows that of temperature with high humidity coinciding with low temperatures. At Afgooye for the period 1953 to 1976 the relative humidity ranges from 66.3 per cent in March to 76.9 per cent in July with an annual mean of 71.8 per cent.

10.1.2 The Shebeelle river flows

The Shebeelle river rises on a plateau on the eastern side of the Ethiopian highlands, and has a catchment area of about 200,000 km^2. Only about a third of the catchment is in the low lying areas of southern Somalia. The main source of runoff is from Ethiopia; thus the seasonal distribution of runoff is controlled by the incidence of rainfall in the highlands, rather than in the coastal region around Muqdisho.

There are 5 gauging stations of interest along the Shebeelle river, Beled Weyne, Mahadday Weyne, Balcad, Afgooye and Aw Dheegle which are shown in Figure 10.1; these are controlled by the Ministry of Agriculture in Muqdisho. Each site has gauge boards which are read twice daily, and occasional current metering exercises have been carried out. The mean daily water level measurements and calculated mean daily flows are kept at the Irrigation department of the Ministry of Agriculture, Muqdisho.

Beled Weyne stage records are available for earlier periods than any of the other stations. Lockwood (1968) carried out hydrometric analyses and established correlations between the stages of Beled Weyne and the other sites. These relationships were used to construct synthetic series of flows for infilling

Location map of gauging stations on the River Shebeelle

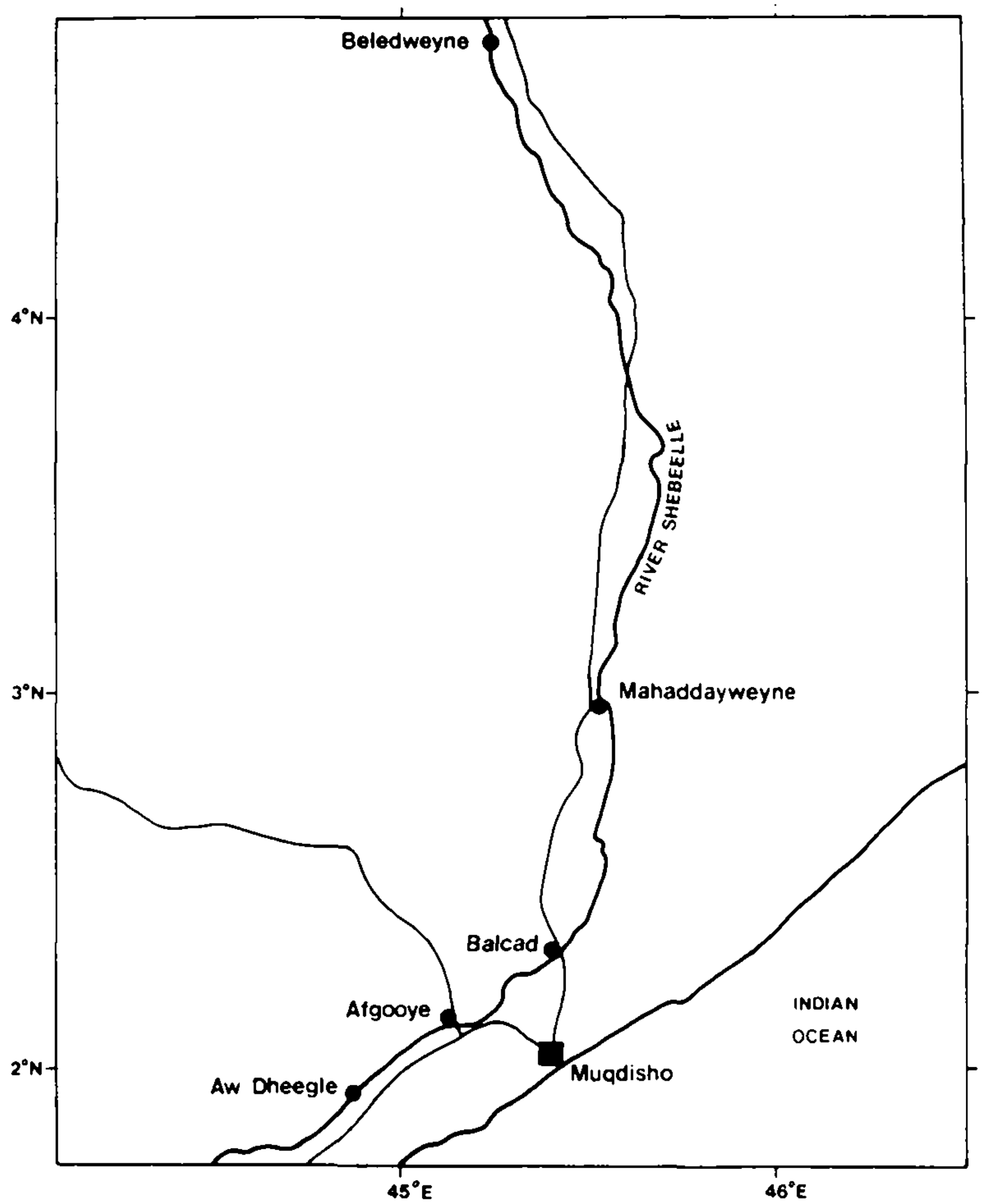


Figure 10·1

data at each site, between 1951 and 1962. The original records have been mislaid and it is not possible to trace which data are real and which synthetic.

Lockwood also carried out a programme of gauging at each site during 1963 to 1965 to enable the construction of stage discharge curves from flows. These rating tables were used to convert the water levels of 1953 to 1962 to flow records.

Unfortunately, very few complete years of record are available from 1973 to the present. One of the major problems is caused by the nature of the flood flows; that is, a sudden rapid rising limb occurring after a prolonged monotonous low flow. This rising limb is often missed completely by the gauge readers.

The basic water level data contain some erroneous values. The main reason for this is that the metre level is not clearly labelled on the staff gauges and it is obvious that readings are often associated with the wrong metre. Historically flows have been calculated from uncorrected water levels and the errors are not readily spotted from the flow data. Some corrections to flows have been made in recent reports (Hunting & MMP (1969) and MMP (1976)) but we believe that there are still many errors. Because of these problems we decided to use the basic daily water levels,and to calculate the flows from these and their corresponding rating tables, rather than accept the computed flow values.

The daily water level data for each station were screened using a computer program to identify large changes of recorded mean daily stage. These occurrences were examined individually and, with cross reference to the other stations, were corrected. Infilling was carried out using linear interpolation for short periods of less than 5 days during steady flow conditions.

10.1.3 Stage-discharge relationships

Rating tables were constructed by Lockwood from data collected from 1963 to 1965. Their study concluded that the river bed at each site was fairly stable and that these rating tables could be used for the 1953 to 1962 records. We have adopted these rating tables and used them for the translation of early stage data . Some recent gaugings were carried out under the auspices of the Ministry of Agriculture during 1980 and 1981. We also carried out a gauging programme as part of the field work in Somalia, but gaugings were restricted to the flood period and one low flow measurement in 1980 at Balcad. Therefore the ratings of 1980 and 1981 by the irrigation department have formed the basis of the stage discharge relationships produced for recent data. The stage discharge information available for each site are shown in Figures 10.2 to 10.5.

Rating tables were produced from the 1980 and 1981 data, for each of the sites, using a linear regression on the logarithmic transformation of the stage and discharge information. There are so few stage discharge measurements recorded at Balcad that the recent rating table was not used. For other stations, these recent results were employed for conversion of Mahadday Weyne, Afgooye and Aw Dheegle water levels to flows. Table 10.2 sets out the periods of data and origin of the rating curves used for each station. The mean daily flows were thus calculated using the mean daily stage measurements and the appropriate stage discharge curves.

10.1.4 Record infilling

The flow records obtained, by the method described above, were discontinuous. However the periods of missing data were not the same for each site so the deficiencies in flow data could be filled, to some extent, by reference to the other stations. This was achieved by computing a correlation between pairs of records. The resulting relationships which were used are shown in Table 10.3. This table also indicates the order of stations used for infilling. Generally the closest stations were used first as these would provide the most reliable relationship, unless there were problems with predicted negative flows. Once the flows had been treated in this way the series covered identical periods and had identical missing periods. There is, therefore, no information on which to base the flows during the missing periods.

Stage discharge curve for the River Shebeelle at Mahaddayweyne

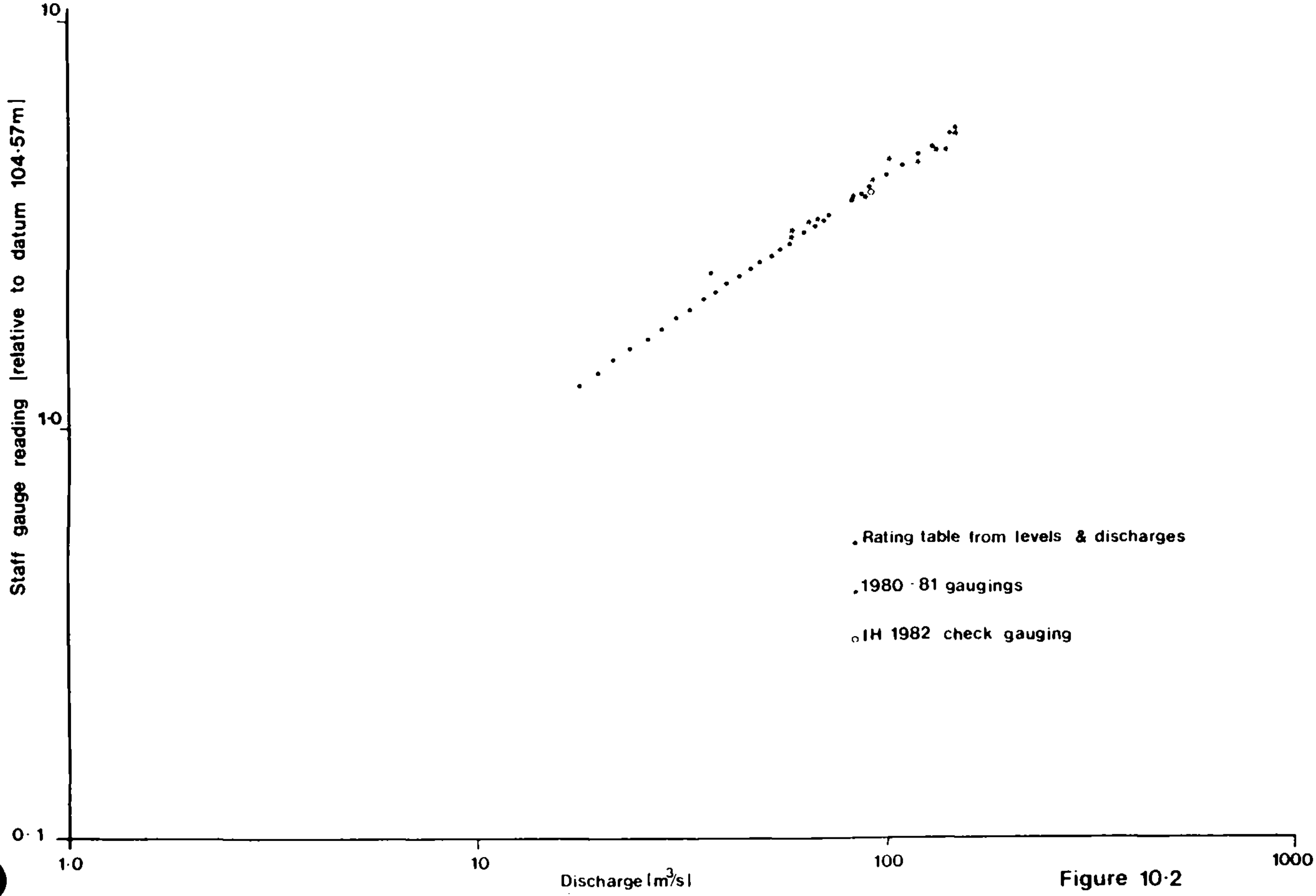


Figure 10·2

Stage discharge curve for the River Shebeelle at Balcad Bridge

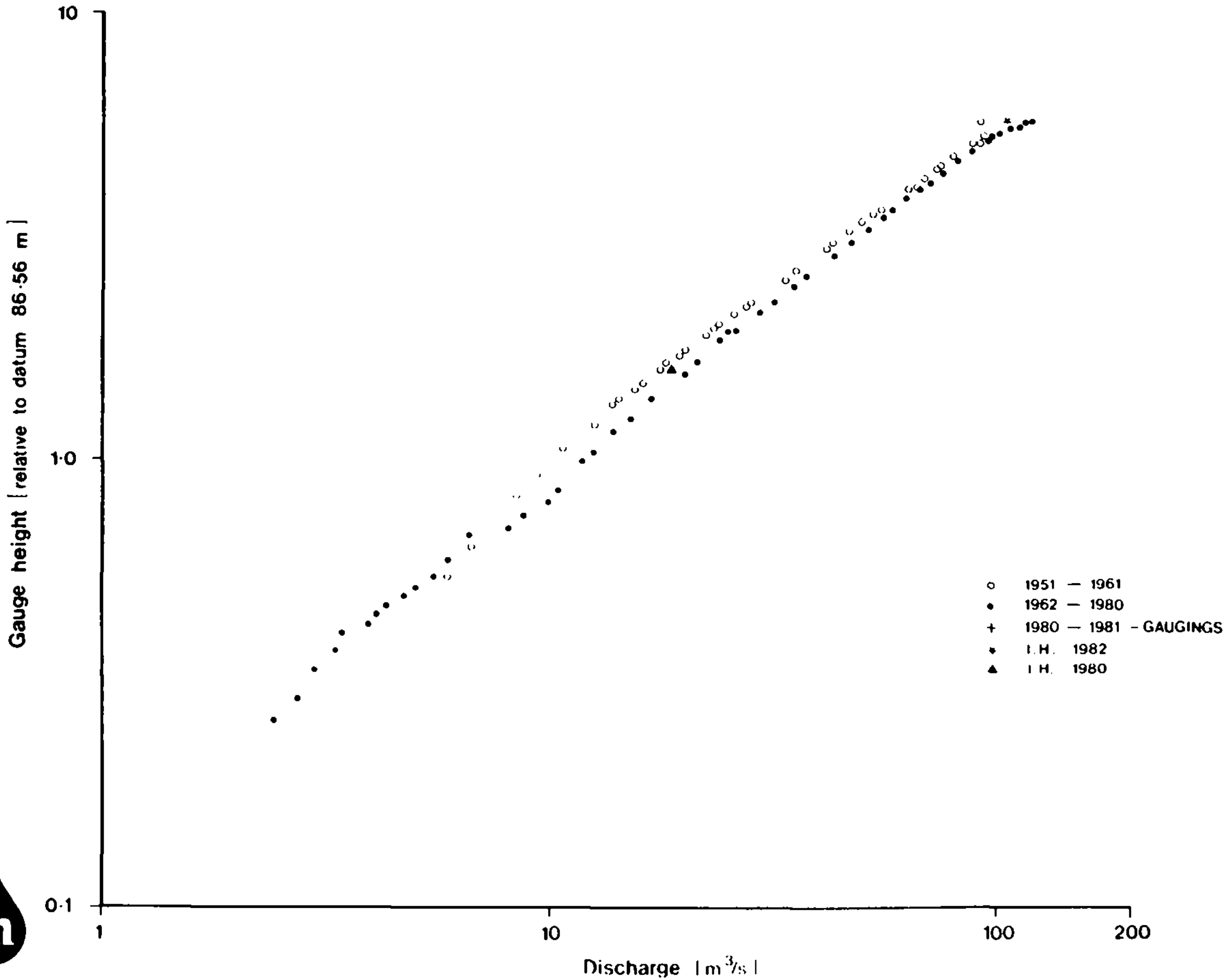


Figure 10·3

Stage discharge curve for the River Shebeelle at Afgooye Bridge

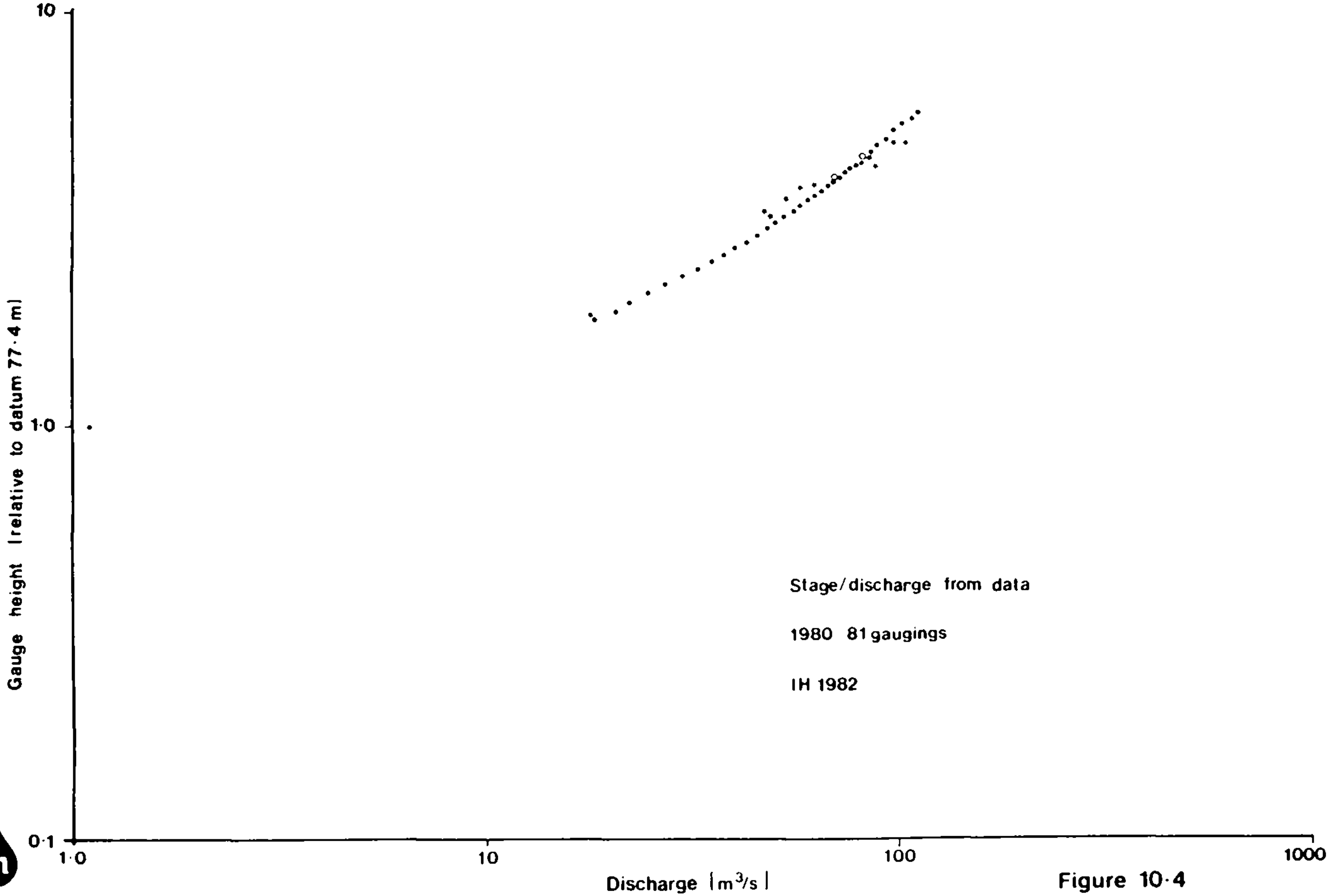


Figure 10·4

Stage discharge curve for the River Shebeelle at Aw Dheegle

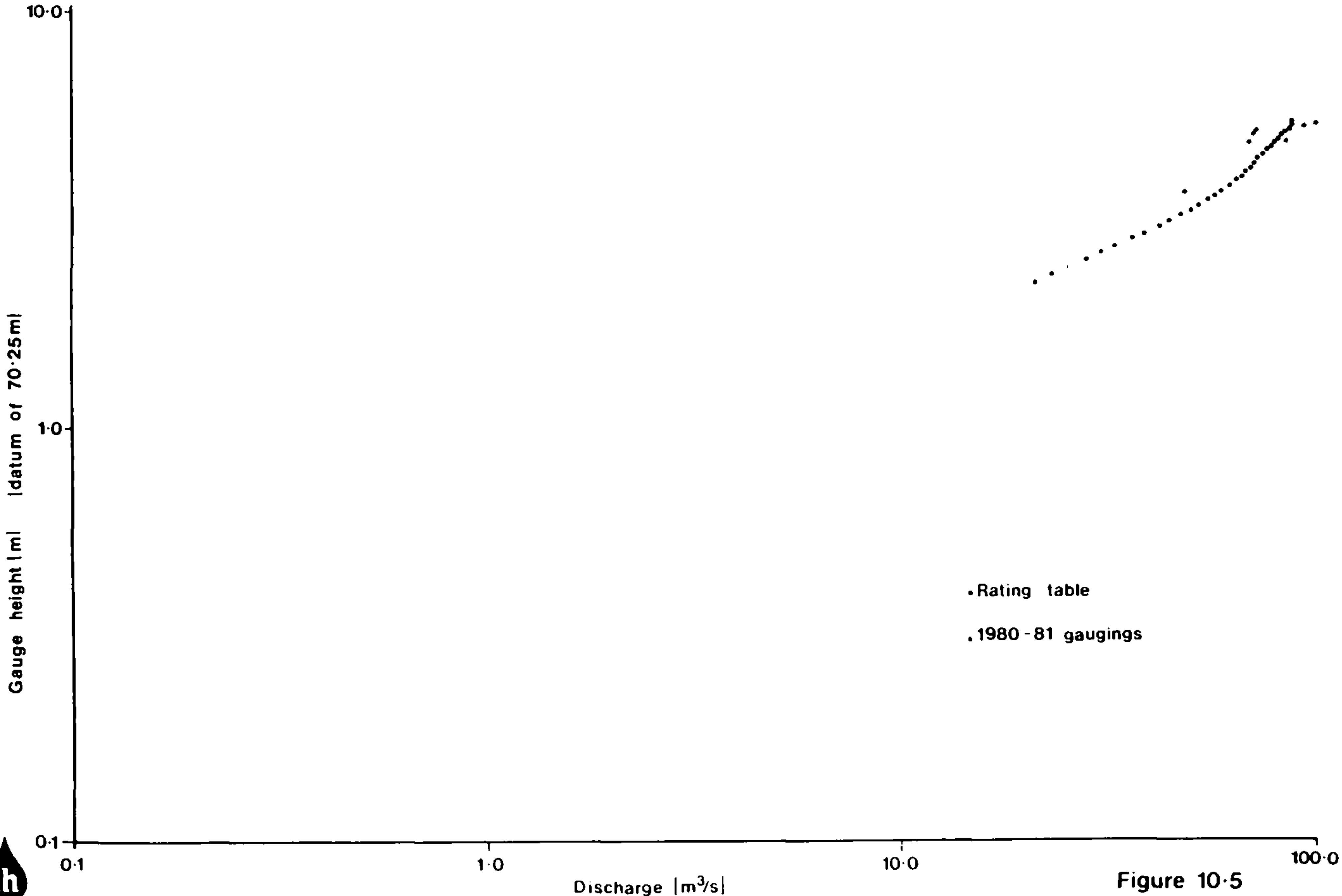


Figure 10·5

Table 10.2

Origin of rating tables for each station

STATION	PERIOD	ORIGIN
BELED WEYNE	1953 to 1967	MINISTRY OF AGRICULTURE MUQDISHO
	1968 to 1982	MINISTRY OF AGRICULTURE MUQDISHO
MAHADDAY WEYNE	1953 to 1967	MINISTRY OF AGRICULTURE MUQDISHO
	1968 to 1982	I.H. LINEAR REGRESSION
BALCAD	1953 to 1982	MINISTRY OF AGRICULTURE MUQDISHO
AFGOOYE	1953 to 1973	MINISTRY OF AGRICULTURE MUQDISHO
	1974 to 1982	I.H. LINEAR REGRESSION
AW DHEEGLE	1953 to 1975	MINISTRY OF AGRICULTURE MUQDISHO
	1975 to 1982	I.H. LINEAR REGRESSION

TABLE 10.3

Inter station relationships and progression of infilling

1. MAHADDAY WEYNE (MW)

(i) From Beled Weyne (BW) $MW = 0.89 \times BW$

(ii) From Balcad (B) $MW = 1.099 \times B$

If $MW > 172$, $MW = 172$ in each case.

BALCAD (B)

(i) From Mahadday Weyne (MW) $B = 0.91 \times MW$

(ii) From Beled Weyne (BW) $B = 0.81 \times BW$

(iii) From Afgooye (A) $B = 1.048 \times A$

In all cases if $B > 115$, make $B = 115$.

AFGOOYE (A)

(i) From Beled Weyne (BW) $A = 0.77 \times BW$

(ii) From Balcad (B) $A = (0.955 \times B) + 1.4$

In all cases if $A > 115$, make $A = 115$.

AW DHEEGLE (AW)

(i) From Afgooye (A) $AW = 1.302 \times A^{0\ 8688}$

(ii) From Balcad (B) $AW = 0.855 \times B^{0\ 964}$

(iii) From Beled Weyne (BW) $AW = 0.63 \times BW$

In all cases if $AW > 80$, make $AW = 80$.

A summary of the original series and the infilled series for Mahadday Weyne is shown in Table 10.4 where the incomplete years are 1953, 60, 74-76 and 1979 and missing values are indicated by 999.99. We therefore had 26 calendar years of complete data for each site.

10.1.5 Open water evaporation Eo

Climatological data collection is not widespread in Somalia. Some long term mean open water evaporation (Eo) values have been published but their validity is difficult to assess. Only two sites have suitable records for estimation of Penman daily and monthly Eo. These are the Jawhar Estate and Afgooye Research Station. We have used the data from the Afgooye Research Farm which seem to be the most reliable.

From these data the average climatological values for 1980 to 1981 provide the most comprehensive inputs to the Penman equation for calculation of Eo and the resulting values are shown in Table 10.5.

10.1.6 Crop evaporation

Crop evaporation was estimated by calculating Reference Crop Evapotranspiration (ETo) and applying Crop Coefficients (kc) for different stages of plant growth. In the FAO Irrigation and Drainage Paper on Crop Water Requirements (Doorenbos and Pruitt 1977), the authors propose ETo as a concept for evaporation from a well watered, freely transpiring vegetated surface. As the definitions of the Doorenbos and Pruitt ETo and Penman Et are very nearly identical (Gunston and Batchelor 1983), Reference Crop Evapotranspiration has been calculated for the present study using the Penman formula as follows:-

$$ETo\ (mm.day^{-1}) = Et = (IT - BT) + AT$$

$$IT\ (\text{Incoming Radiation Term}) = \frac{\Delta}{\Delta+\gamma}\left[0.75\ Ra(0.25 + 0.5\ n/N)\right]$$

$$BT\ (\text{Back Radiation Term}) = \frac{\Delta}{\Delta+\gamma}\left[\sigma\ Ta^{4}(0.1 + 0.9\ n/N)\ (0.56 - 0.08\sqrt{e_d})\right]$$

$$AT\ (\text{Aerodynamic Term}) = \frac{\gamma}{\Delta+\gamma}\left[0.26\ (1 + U_2/160)(e_a - e_d)\right]$$

where

= slope of the saturation vapour pressure (pressure/temperature curve (mb $^{o}K^{-1}$)

γ = psychrometric constant

Ra = solar radiation on a horizontal surface at the top of the atmosphere (cal. cm^{-2} day^{-1})

n = actual daily sunshine (hours)

N = theoretically possible daily sunshine (hours)

σ = Stefan - Boltzmann constant

Ta = mean air temperature (^{o}K)

e_a = saturated vapour pressure (mb)

e_d = actual vapour pressure (mb)

U_2 = run of wind at 2 metres height (km day^{-1})

ETo values were derived using the above method and climatological data for 1980 and 1981 from Afgooye Research Farm. These are shown in Table 10.5.

Table 10.4 Summary of Mahadday Weyne flows before infilling process

Units

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTALS
1951	999.99	999.99	999.99	999.99	335.40	250.11	62.05	142.32	198.03	106.19	310.10	237.31	999.99
1952	24.20	9.68	10.71	10.37	197.59	60.83	19.34	999.99	275.31	203.07	101.23	10.94	999.99
1953	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1954	23.50	21.53	23.84	999.99	154.98	62.69	8.79	167.95	324.97	336.41	174.42	74.28	999.99
1955	13.04	16.11	14.47	16.24	999.99	30.27	14.45	50.14	232.95	999.99	999.99	14.59	999.99
1956	11.25	7.36	19.54	999.99	999.99	67.21	57.96	241.32	321.67	309.66	304.23	74.22	999.99
1957	30.31	11.64	37.09	999.99	330.93	256.99	117.54	270.76	301.28	121.83	107.53	131.04	999.99
1958	19.28	34.06	106.11	37.18	162.12	24.23	30.77	263.64	325.14	336.41	999.99	47.88	999.99
1959	18.59	12.60	10.95	10.37	999.99	59.34	32.85	179.26	289.31	302.98	999.99	56.15	999.99
1960	159.30	999.99	23.03	48.41	153.62	90.60	44.11	91.71	198.26	999.99	999.99	999.99	999.99
1961	24.19	26.00	11.20	17.15	115.55	36.40	97.30	295.54	325.18	334.92	325.24	999.99	999.99
1962	999.99	5.99	7.99	23.82	129.31	41.78	15.55	61.79	164.91	210.08	310.87	192.51	999.99
1963	20.64	5.74	5.98	97.93	332.19	237.02	131.09	252.52	326.35	240.18	115.06	208.84	1973.54
1964	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	128.03	142.84	999.99	999.99
1965	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	87.16	210.80	268.74	113.33	999.99
1966	999.99	999.99	999.99	74.48	209.76	101.52	94.44	149.97	263.04	248.21	187.39	39.87	999.99
1967	999.99	999.99	999.99	999.99	261.97	185.21	72.01	247.37	340.31	341.28	292.33	311.26	999.99
1968	95.45	42.16	175.57	191.30	360.84	999.99	214.70	271.77	322.32	300.17	172.45	176.32	999.99
1969	70.11	53.33	258.90	273.44	295.51	301.36	135.27	288.74	337.03	255.86	124.25	39.37	2433.17
1970	20.23	46.67	111.38	259.49	353.58	103.25	45.94	251.95	354.72	360.94	247.56	61.43	2217.14
1971	34.69	18.65	8.83	84.04	241.62	135.96	212.61	999.99	999.99	999.99	999.99	999.99	999.99
1972	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1973	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1974	999.99	999.99	999.99	999.99	999.99	181.71	190.91	260.51	302.02	227.22	69.83	25.33	999.99
1975	9.21	999.99	999.99	999.99	151.77	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1976	999.99	999.99	999.99	999.99	327.37	999.99	227.29	291.77	343.46	999.99	200.78	999.99	999.99
1977	999.99	999.99	16.21	999.99	426.90	217.10	166.85	289.60	336.02	412.03	434.41	392.11	999.99
1978	999.99	46.04	999.99	173.06	999.99	999.99	999.99	41.11	112.76	278.60	231.73	999.99	999.99
1979	132.77	176.11	96.86	271.05	165.89	266.28	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1980	999.99	999.99	999.99	999.99	999.99	999.99	52.99	999.99	999.99	999.99	54.07	15.81	999.99
1981	999.99	999.99	999.99	420.50	430.63	207.46	85.97	266.40	394.27	420.16	181.17	64.04	999.99
1982	37.94	20.80	999.99	145.93	330.58	273.53	999.99	999.99	999.99	999.99	999.99	999.99	999.99
MEAN	36.993	35.247	125.420	210.287	327.093	213.877	104.100	264.403	339.367	285.660	162.290	103.213	2207.950
STD DEV	28.68	25.77	127.04	97.55	29.37	101.06	50.41	21.08	14.33	65.66	73.99	92.14	229.95
COEFF OF VARIATION	0.78	0.73	1.01	0.46	0.09	0.47	0.48	0.08	0.04	0.23	0.46	0.89	0.10

TABLE 10.4 continued.

Summary of Mahadday Weyne flows after infilling process

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTALS
1951	999.99	999.99	999.99	294.33	335.40	250.11	62.05	142.32	198.03	106.19	310.10	237.31	999.99
1952	24.20	9.68	10.71	10.37	197.59	60.83	19.34	58.06	275.31	203.07	101.23	10.94	981.33
1953	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99	999.99
1954	23.50	21.53	23.84	128.92	154.98	62.69	8.79	167.95	324.97	336.41	174.42	74.28	1502.28
1955	13.04	16.11	14.47	16.24	95.10	30.27	14.45	50.14	232.95	326.18	104.21	14.59	927.75
1956	11.25	7.36	19.54	37.55	297.53	67.21	57.96	241.32	321.67	309.66	304.23	74.22	1749.50
1957	30.31	11.64	37.09	129.68	330.93	256.99	117.54	270.76	301.28	121.83	107.53	131.04	1846.62
1958	19.28	34.06	106.11	37.18	162.12	24.23	30.77	263.64	325.14	336.41	186.61	47.88	1573.43
1959	18.59	12.60	10.95	10.37	179.59	59.34	32.85	179.26	289.31	302.98	284.67	56.15	1436.66
1960	159.30	56.28	23.03	48.41	153.62	90.60	44.11	91.71	198.26	999.99	999.99	999.99	999.99
1961	24.19	26.00	11.20	17.15	115.55	36.40	97.30	295.54	325.18	334.92	325.24	391.07	1999.74
1962	41.61	5.99	7.99	23.82	129.31	41.78	15.55	61.79	164.91	210.08	310.87	192.51	1206.21
1963	20.64	5.74	5.98	97.93	332.19	237.02	131.09	252.52	326.35	240.18	115.06	206.84	1973.54
1964	93.94	32.42	16.01	52.87	78.99	35.30	53.74	271.32	354.46	128.03	142.84	68.98	1328.90
1965	118.99	30.18	14.01	23.86	109.75	24.48	9.39	73.99	87.16	210.80	268.74	113.33	1084.68
1966	3.15	13.29	68.27	74.48	209.76	101.52	94.44	149.97	263.04	248.21	187.39	39.87	1453.39
1967	4.87	4.26	3.97	114.14	261.97	185.21	72.01	247.37	340.31	341.28	292.33	311.26	2178.98
1968	95.45	42.16	175.57	191.30	360.84	313.42	214.70	271.77	322.32	300.17	172.45	176.32	2636.47
1969	70.11	53.33	258.90	273.44	295.51	301.36	135.27	288.74	337.03	255.86	124.25	39.37	2433.17
1970	20.23	46.67	111.38	259.48	353.57	103.25	45.94	251.95	354.72	360.94	247.57	61.43	2217.13
1971	34.69	18.65	8.83	84.04	241.62	135.96	212.61	249.47	342.53	248.63	172.92	75.93	1825.88
1972	24.56	49.56	35.77	119.21	417.46	150.96	197.30	280.85	333.17	258.13	165.14	41.60	2073.71
1973	17.94	9.93	7.43	15.59	135.03	56.24	64.97	215.23	293.67	239.07	59.83	11.18	1126.11
1974	999.99	999.99	999.99	160.95	149.96	181.71	190.91	260.51	302.02	227.22	69.83	25.33	999.99
1975	9.21	999.99	999.99	999.99	151.77	999.99	157.35	302.91	439.38	295.65	78.09	19.23	999.99
1976	999.99	999.99	999.99	999.99	327.37	322.28	227.29	291.77	343.46	250.32	200.78	107.18	999.99
1977	37.65	21.45	16.21	206.29	426.90	217.10	166.85	289.60	336.02	412.03	434.41	392.11	2956.62
1978	64.44	46.04	199.53	173.06	244.06	75.39	84.47	41.11	112.76	278.60	231.73	99.52	1650.71
1979	132.77	176.11	96.86	271.05	165.89	266.28	161.25	277.33	165.39	193.09	131.33	29.42	2066.77
1980	18.39	10.71	15.55	31.56	254.58	999.99	52.99	999.99	999.99	999.99	54.07	15.81	999.99
1981	8.48	9.63	168.51	420.50	430.63	207.46	85.97	266.40	394.27	420.16	181.17	64.04	2657.22
1982	37.94	20.80	28.76	145.93	330.58	273.53	999.99	999.99	999.99	999.99	999.99	999.99	999.99
MEAN	39.745	29.350	59.547	116.188	238.620	127.112	88.523	209.003	288.497	275.697	201.090	113.578	1786.950
STD DEV	36.57	34.86	73.37	107.97	110.66	96.02	65.60	88.02	79.61	77.42	90.79	111.65	555.37
COEFF OF VARIATION	0.92	1.19	1.23	0.93	0.46	0.76	0.74	0.42	0.28	0.28	0.45	0.98	0.31

Table 10.5 Evaporation Estimates for Afgooye Research Farm

Month	E_o (mm)	ET_o (mm)
JAN	243	200
FEB	229	189
MAR	249	205
APR	192	155
MAY	169	135
JUN	167	135
JUL	169	135
AUG	177	141
SEP	195	156
OCT	202	161
NOV	176	140
DEC	202	162
TOTAL	2430	1914

ETo calculated as Penman potential transpiration

10.2 IRRIGATION ABSTRACTIONS FROM THE SHEBEELLE

10.2.1 Introduction

To assist future planning of the water supply for Muqdisho, a knowledge of irrigation patterns along the Shebeelle river was needed to quantify the effects of irrigation water demands on river flows, and to estimate likely groundwater recharge due to deep percolation beneath irrigated areas.

This Section is concerned with the first of these requirements and covers definition of irrigated areas, estimation of crop evaporation and calculation of irrigation demands from the river. Three reaches of the river are covered:-

(a) Mahadday Weyne to Balcad Bridge

(b) Balcad Bridge to Afgooye Bridge

(c) Afgooye Bridge to Aw Dheegle

The two principal cropping seasons along the Shebeelle inland of Muqdisho, are closely linked to the bimodal rainfall distribution and the two periods of high flow in the river. Because the river flows are generated by rainfall over the main catchment area in Ethiopia, the river flows and local rainfall are not directly related.

Within the study area, broadly Mahadday Weyne to Aw Dheegle, irrigation is normally carried out in one of the following three ways:-

(a) Controlled flood irrigation - water is routed down established channel systems during the Gu and Der flood periods only. There is little precise control on water distribution at the field level, and the total area irrigated in any season is dependent on the height and duration of flood flows in the river.

(b) Defined irrigated estates or schemes - water is abstracted through controlled headworks upstream of a river barrage, or by pumping, and distributed through networks of canals of decreasing size until field boundaries are reached.

(c) Farm level pumping - water is pumped directly to one or more small farms close to the river.

Controlled flood irrigation is most common above Balcad Bridge, and pumped irrigation downstream of the Balcad Scheme Barrage. The two major irrigation schemes currently operating are the Jawhar Estate (c 6400 ha) and the Balcad Scheme (c 2000 ha).

10.2.2 Defining irrigated areas

The areas of various crops being irrigated in each of the three river reaches were determined by two methods. The first involved a field survey of cropping patterns in October 1982 to define the crop patterns for calibration of Landsat imagery and the second involved a study of the total length of mapped irrigation canals.

(i) The field survey

The field survey involved measuring all significant cultivated plots. The type of cultivation within each plot was noted as being either perennial or unspecified seasonal crops. The perennial crops were broken down into either bananas, citrus or papaya but as no seasonal crops had been planted at such an early date in the wet season, no distinction between various types of seasonal crops was possible.

The survey was carried out for the irrigated areas on both banks of the Shebeelle between Balcad and Afgooye and on the left bank only from Afgooye to Mordille. The arrival of the Der rains prevented further access onto the alluvial tracks. However, as the aim of the survey was to establish some observed "ground truth" data to enable calibration of the available Landsat imagery, the data acquired was adequate.

The survey data were plotted onto the available 1:100,000 scale topographic maps, distinguishing between the various crop types. The total area of each crop was then planimetered for the Balcad to Afgooye reach, and the surveyed area of the Afgooye to Aw Dheegle reach. An illustration of the type of data collected is shown for the area around Afgooye in Figure 10.6.

The results of the field survey of irrigated areas made in October 1982 are given in Table 10.6. The area of seasonal cropping may be an underestimate as the survey was made prior to the planting of seasonal crops. It was possible to identify fields commonly under seasonal crops however. This observed cropping pattern for the early part of the Der season was used to calibrate the best Landsat image available for the area which was February 1973, just prior to the Gu rains. Unfortunately frequent and extensive cloud cover over the study area meant that this particular image was the only one available to us with an unobscured view of almost the whole study area.

(ii) Use of the Landsat imagery

A description of how the Landsat imagery was obtained and utilised to map the geology and aquifer has been given earlier in this report (Section 1). However, a brief description of the methods used to estimate irrigated areas follows.

The Landsat multi spectral scanner (MSS) simultaneously records the reflectance from the ground within four wave bands designated 4, 5, 6 and 7. The last two are both effective within the near infrared range and produce similar images. The data are recorded and stored digitally on magnetic tapes. Analysis of these data is usually carried out by displaying the image of the data, on a colour monitor, either as a single waveband or a composite of two or more wavebands. The composite image can be coloured by assigning different coloured guns in the monitor to each waveband. A true colour composite cannot be produced as none of the sensors record reflectance in the blue wavelength; therefore a standard false colour composite is normally adopted consisting of blue for band 4, green for band 5 and red for either 6 or 7.

The near infrared bands are particularly useful for the detection of water and vegetation because water almost totally absorbs near

Results of the Crop Survey in the Afgooye area

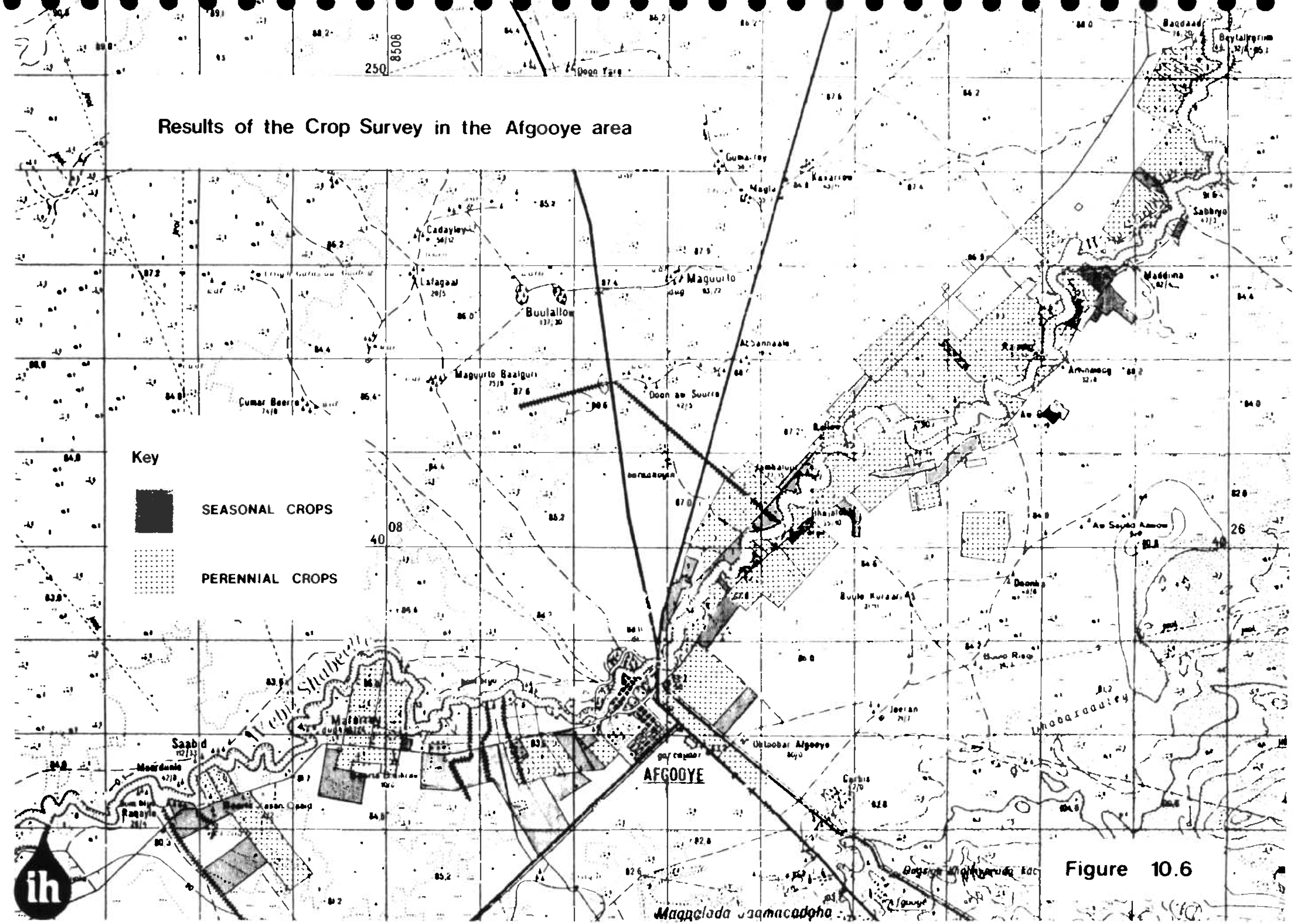


Figure 10.6

TABLE 10.6 Comparison of survey results

SOURCE	MAHADDAY WEYNE TO BALCAD	BALCAD TO AFGOOYE		AFGOOYE TO AW DHEEGLE
1. IH Field Survey	- -	860 S 3250 P	4100	*3400 S 960 P
2. Landsat Feb, 1973	8260 S 600 P 2600 sugar 1460	3980 S 1792 P	5600	7060 S 696 P 800 O
3. IH Canal survey	1700 T	4400 T		17500 T
4. Adjusted canal survey	8600 T	4400 T	4400	9700 T

all values in hectares

S = Seasonal P = Perennial T = Total

*Survey of only part of the reach, therefore an underestimate.

infrared radiation whilst healthy vegetation is a strong near infrared reflector. Therefore, assigning a red colour to band 7 will produce a deep blue for water and a bright magenta-red for healthy, green vegetation. The false colour composite is shown in Plate 1 where the magenta can be clearly seen in the Jawhar Sugar Scheme and along the line of the river. The river itself can be inferred, along most of its length, from the healthy bank vegetation. It must be emphasized that the resolution of the image is based on "pixels" which are 79 m by 56 m and therefore any feature smaller than this will not be apparent. (This amounts to an area of approximately 0.443 ha).

From our field survey of October 1983 we were able to identify various areas of cropping and their particular colour tone on the image. Plate 1 illustrates this as the sugar scheme exhibits a different shade of red to the flood irrigation along the river banks. We could, therefore, classify areas according to their colour tone by choosing cultivated areas which had not changed their land use in the last ten years. It was hoped to divide the crops into each of the different types we had noticed on the survey, but colour tone varies not only with crop type but also with crop conditions, spacing etc and therefore three basic divisions were chosen. These three divisions were for sugar, perennial crops (including citrus, banana and papaya) and seasonal crops.

After modest adjustments of the calibration, an image of the Balcad to Afgooye reach was obtained which was the best possible reproduction of our observed field data shown in Figure 10.6.

A count of the total number of pixels, and hence an estimate of the total area of each crop type, was made for the image from February 1973 for each section along the river.

(iii) Irrigation canal survey

The other method of estimating present irrigated areas utilised was a study of the total lengths of irrigation canals shown on the 1:100,000 scale maps. Some of these are shown in Figure 10.6. These maps were produced in 1976 and from our observations in October 1982 give a good indication of the location of primary distribution canals. Whilst some of the mapped canals, particularly in the Afgooye to Aw Dheegle reach, seem to have fallen into disrepair and do not appear to be used, it was felt that an examination of these total canal lengths would provide a crude estimate of irrigated areas. From our field survey we estimated that the average area irrigated by each linear kilometre of an average canal to be of the order of 100 hectares. This indicative average area was multiplied by the total length of mapped canals in each reach to provide a second estimate of the total irrigated area.

(iv) Results of the survey

The results of each survey are shown in Table 10.6. The Balcad to Afgooye field survey was used to calibrate the Landsat image which, in turn, was used to estimate cropped areas in the other two reaches. The Landsat classification will tend to overestimate irrigated areas as it is difficult to distinguish between crops and the healthy bankside vegetation.

The validity of the February 1973 estimates of irrigated area for each reach may, to some extent, be questioned. Some changes in irrigation practices may have taken place between 1973 and the period of our field calibration in 1982. This could partly account for the differences in Table 10.6 between the division of total irrigated area into seasonal and perennial. Our chosen division of the total area in each reach was based on the field survey and discussion with the local farmers.

The canal survey proved to be too inaccurate and was improved by a detailed study of each reach. The Mahadday Weyne to Balcad reach was very complicated. The reticulation system was so dense in some areas that it was unrealistic to assume 100 ha per 1 km canal length. Therefore an adjusted canal length was estimated. The Balcad to Afgooye reach was fairly simple and the original survey produced reasonable results. The Afgooye to Aw Dheegle reach contained several long canals which, from our field survey, did not lead to irrigated land. Therefore a maximum length of 3 km was imposed on canals in this reach and an adjusted irrigated area calculated as shown in Table 10.6. Considering the nature of the surveys carried out these results are reasonably consistent and form the basis of irrigated area estimation in each reach. The results of the analyses for each of the three reaches are given below.

(v) Mahadday Weyne to Balcad Bridge

Controlled flood irrigation can cover a significant area along this reach, but the extent from season to season is difficult to estimate. The 1:100 000 maps show irrigation channels in some detail, and the total length of channels shown was measured as 170 km. Additional information came from the Landsat imagery of February 1973 and low level flying over the area in October 1982. Combining data from all three sources, the best estimate for controlled flood irrigation along the reach is 9000 ha. This is an upper limit, and when flood volumes and river stages are below average the areas irrigated by flooding in a Gu or Der season will be lower.

Due to salinity problems it is not possible for the entire area of the Jawhar Estate in command to be irrigated. From discussions with senior managers in October 1982, the current area cultivated for sugar cane is now around 6400 ha. Plans to improve drainage and reclaim saline soils should increase the potential area for cane growing, and 7000 ha is a reasonable medium term prediction for the effective area of the estate.

In the Inter-Riverine Agricultural Study (Hunting 1977), the only areas quantified as being under irrigation between Mahadday Weyne and Balcad Bridge were the Jawhar Estate and various "small schemes". Controlled flood irrigation was presumably excluded as it was not a demand on the river during low flow periods. In the absence of any further details of the small schemes, their areas - both 'present' and 'proposed' - have been taken as those presented by Hunting (1977) (see Table 10.7).

Two planned projects of this reach are the Romsoma and Iraqsoma irrigated livestock feedlot schemes. The irrigated fodder area at

Table 10.7 Irrigated areas and cropping patterns (ha)

		Present		Possible by c 1990	
		Gu	Der	Gu	Der
(a)	<u>Mahaddey Weyne & Balcad Bridge</u>				
(i)	Controlled flooding	9,000	9,000	9,000	9,000
(ii)	Jowhar Estate sugar cane	6,400	6,400	7,000	7,000
(iii)	"Small schemes"				
	cotton	-	100	-	425
	groundnuts	100	-	425	-
	maize	210	50	625	-
	paddy rice	-	50	-	415
	pulses		-	200	-
	sesame		120	-	410
(iv)	Feedlots				
	Romsoma			5,000	5,000
	Iraqsoma			5,000	5,000
(b)	<u>Balcad Bridge to Afgooye Bridge</u>				
(i)	Balcad Scheme				
	cotton	-	500	-	5,600
	maize	2,000	-	8,000	-
	sesame	-	1,500	-	2,400
(ii)	Farm pumping (seasonal crops)				
	cotton	-	387	-	387
	maize	1,400	323	1,400	323
	pulses	600	161	600	161
	sesame	-	1,129	-	1,129
(iii)	Farm pumping (perennial crops)				
	bananas	1,500	1,500	1,500	1,500
	citrus	900	900	900	900
	papaya	600	600	600	600
(iv)	Feedlot				
	Gisoma			4,000	4,000

TABLE 10.7 (continued)

		Present		Possible by c 1990	
		Gu	Der	Gu	Der
(c)	Afgooye Bridge to Awdheegle				
(i)	Farm pumping (seasonal crops)				
	cotton	-	968	-	968
	maize	3,500	808	3,500	808
	pulses	1,500	402	1,500	402
	sesame	-	2,822	-	2,822
(ii)	Farm pumping (perennial crops)				
	bananas	375	375	375	375
	citrus	225	225	225	225
	papaya	150	150	150	150

Romsoma is planned to be 5,000 ha (MMP 1981), but details for Iraqsoma are less precise. An estimate of 5,000 ha of irrigated fodder for the scheme has been made. Both schemes are planned to lie between Jawhar and Balcad Bridge.

(vi) Balcad Bridge to Afgooye Bridge

The Balcad Scheme was originally proposed as a controlled flood irrigation project (Hunting 1969b). However, it was subsequently developed as a conventional surface irrigation scheme, taking water from behind a barrage constructed a short distance downstream of the road bridge at Balcad. The current cropping programme is:

(a) Gu - 2000 ha maize

(b) Der - 1500 ha sesame
500 ha cotton (Information from Project Manager, October 1982).

Various reports, including the Somali Five Year Development Plan, 1982-86 (MMP 1981) forecast expansion of the scheme to various levels up to a maximum of 10,000 ha. 8,000 ha has been selected as the best estimate for likely future development following discussions with the Project Manager in October 1982.

The remaining land currently irrigated between Balcad and Afgooye comprises smaller farms fed by pumping from the river. The methods discussed in (ii), (iii) and (iv) were used to produce best estimates of irrigated areas which were:-

(a) Seasonal crops - 2000 ha

(b) Perennial crops - 3000 ha
(Banana, citrus, papaya)

A third irrigated livestock feedlot project, the Gisoma Scheme is planned to be developed "near Afgooye", but it is not clear whether it would be upstream or downstream of Afgooye Bridge. The proposed irrigated area for fodder is 4,000 ha (MMP 1981) and a site upstream of Afgooye has been assumed for river abstraction calculations.

(vii) Afgooye Bridge to Aw Dheegle

Small farms predominate on this reach, irrigated channels being mainly pump-fed, and cropping is both seasonal and perennial. The extent of the irrigated land near Afgooye was estimated using the methods described earlier. Best estimates for irrigated areas were:

(a) 5,000 ha seasonal crops

(b) 750 ha perennial crops

Just downstream of Afgooye on the south bank is the site of the Afgooye-Mordile Scheme (Hunting 1969 c, FAO 1975). Although this pump-fed scheme has a nominal existing area of 1,500 ha, the field survey revealed that the current cropped area appeared to be considerably less, and that cropping patterns were closely similar to surrounding lands. The land nominally within the Afgooye-Mordile Scheme was therefore included in the totals listed in the last paragraph.

10.2.3 Crop evaporation

Crop evaporation was estimated as in Section 10.1.6 by calculating Reference Crop Evapotranspiration (ETo) and applying Crop Coefficients (kc). ET_O was calculated using the form of the Penman Potential Transpiration (Et) equation detailed in Section 10.1.6.

10.2.4 Cropping patterns and crop coefficients

(i) Mahadday Weyne to Balcad Bridge

For controlled flood areas it was decided not to compute crop evaporation, as it appears that they only receive one significant irrigation (during flood periods). There is no indication of further irrigation being applied through a season to satisfy crop water needs. As an upper limit for river abstraction calculations, a value of 300 mm water depth/season is proposed. This includes all allowances for losses between the river and the crop root zone.

Table 10.8 Crop coefficients (kc)

Crop	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Banana	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.7	0.7	0.8	0.8
Citrus	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55
Cotton	0.44							0.16	0.74	1.05	1.09	0.98
Fodder	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Groundnuts				0.42	0.81	1.01	0.83	0.09				
Maize	0.29			0.13	0.55	1.02	0.97	0.26	0.12	0.46	1.04	1.06
Papaya	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Pulses	0.18			0.14	0.65	0.80	0.26			0.15	0.79	0.90
Rice (paddy)								0.46	1.07	1.30	1.22	0.48
Sesame									0 15	0.73	0.99	0.45
Sugar cane, planted* GU	1.1	1.1	0.9	0.7	0.5	0.8	0.9	1.0	1.1	1.1	1.1	1.1
DER	1.1	1.1	1.1	1.1	1.1	1.1	0.9	0.7	0.5	0.8	0.9	1.0

On the Jawhar Estate both plant and ratoon sugar crops last around 12 months, and there is planting and cutting in both the Gu and Der seasons. For crop evaporation it was assumed that the estate was equally divided between Gu and Der crops, and crop coefficient values used were derived from Doorenbos and Pruitt (1977).

For the "small schemes" referred to in the Inter Riverine Agricultural Study (Hunting 1977) the values for areas, cropping patterns and crop coefficients used here (see Tables 10.7 and 10.8) are from that Study, although ETo values used are from Afgooye 1980/81.

For the Romsoma and Iraqsoma irrigated livestock feedlot schemes it was assumed that fodder cropping for grazing or cutting would be continuous through the year. The constant crop coefficient of 0.8 proposed in Hunting (1977) was used.

(ii) Balcad Bridge to Afgooye Bridge

Cropping patterns for the Balcad Scheme were obtained during a discussion with the Project Manager in October 1982. Crop coefficients for maize in Gu and sesame and cotton in Der were taken from Hunting (1977).

For areas of farm level pumping, seasonal cropping patterns have been scaled from those given in Hunting (1977) for the Balcad to Aw Dheegle reach, although total cropped areas have been defined separately as discussed in Section 10.2.2. Crop coefficients are from Hunting (1977).

For perennial crops a pattern of 50% bananas, 30% citrus and 20% papaya has been assumed. Crop coefficients for bananas have been derived from Doorenbos and Pruitt, assuming a four year growth cycle and for citrus the kc value of 0.55 all year, proposed by Hunting (1977), has been used. For papaya an all year value of 0.7 was assumed.

For the proposed Gisoma irrigated livestock feedlot the same assumptions have been made as for the Romsoma and Iraqsoma scheme.

(iii) Afgooye Bridge to Aw Dheegle

For seasonal crops fed by farm level pumping, cropping patterns have been selected from the Hunting (1977) values for the Balcad to Aw Dheegle reach, and crop coefficients have also been taken from that Study. For reasons discussed in Section 10.2.2, the Afgooye-Mordile Scheme has not been considered as a separate entity, and cropping patterns across it have been assumed to be similar to the rest of the irrigated area.

For perennial crops the same percentage distributions and crop coefficients have been used as for the Balcad to Afgooye reach.

10.2.5 Effective rainfall

Long term mean monthly rainfall totals are listed in Table 10.1 for Jawhar, Balcad and Afgooye. Effective rainfall, also listed, was derived using the USBR method as shown in Table 10.9.

Table 10.9 Effective rainfall

Monthly Rainfall (mm)		Percentage Effective
0.0	24.9	90.0
25.0	49.9	87.5
50.0	74.9	83.3
75.0 -	99.9	75.0
100.0	124.9	66.0
125.0 -	149.9	56.7
> 150.0		(Constant value of 85 mm)

(Source: Hunting 1977, p 109).

From Mahadday Weyne to Balcad, Jawhar rainfall was used; from Balcad to Afgooye, Balcad rainfall; and from Afgooye to Aw Dheegle, Afgooye rainfall.

10.2.6 River abstractions for irrigation

Excluding controlled flood irrigation, which was discussed in Sectin 10.2.4, it was assumed that all irrigation was managed to satisfy crop water requirements at the field level represented by:-

$$FCWR = ETo.kc - ER$$

where FCWR = monthly field crop water requirement (mm)
ETo = monthly reference crop evapotranspiration (mm)
kc = crop coefficient, for a given crop and month
ER = monthly effective rainfall (mm)

For overall water application efficiency from the point of river abstraction to the crop root zone, the conclusions of Hunting (1977) were accepted, and an overall efficiency value of 45% used for all irrigation (except controlled flooding).

River abstraction for irrigation was obtained by multiplying the irrigated area for each crop, AC and converting to flow volumes in cubic metres per month:-

$$RAI = \frac{10 \times (FCWR) \times AC}{0.45}$$

where RAI = abstraction at river to satisfy irrigation demand ($m^3.mth^{-1}$)

RAI values were calculated for each crop on each component area of each reach, and a total monthly irrigation demand was then built up for the three reaches between river gauging stations at Mahaddey Weyne, Balcad Bridge, Afgooye Bridge and Awdheegle (Malable), as shown in Tables 10.10 a, b and c.

10.2.7 Discussion

The data in Tables 10.10a, b and c provide a basis for calculating irrigation abstractions from the Shebeelle between Mahadday Weyne and Aw Dheegle. As discussed earlier, a number of assumptions have had to be made, e.g. a set value of overall

Table 10.10a River abstraction for irrigation, RAI($m^3.10^6$) - Mahadday Weyne to Balcad Bridge

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Controlled flooding; 9000 ha		27, Gu season							27, Der season			
Jawhar Estate, Sugar cane;												
(i) Gu planting, "Present", 3200 ha	15.3	14.7	11.7	2.7	1.6	6.1	7.1	9.0	11 5	7.7	6.5	11.3
(ii) Der planting, "Present", 3200 ha	15.3	14.7	14.7	7.1	5.9	9.0	7 1	6.0	4.8	4.3	4.5	10.2
(iii) Gu planting, "Proposed", 3500 ha	16.7	16.1	12.8	2.9	1 7	6.7	7.7	9.8	12.5	8.4	7.1	12.4
(iv) Der planting, "Proposed", 3500 ha	16.7	16.1	16.0	7.8	6.5	9.8	7.7	6.6	5.3	4.7	4.9	11.1
"Small schemes"; (i) "Present"	0.2				0.1	0.8	0.7	0.2	0.4	0.5	0.6	0.7
(ii) "Proposed"	0.8	-	-	-	0.7	3.1	2.4	0.8	2.6	2.7	2.5	2.4
ROMSOMA; 5000 ha fodder	16.9	16.6	17.3	6.0	5.6	8.5	9.8	11.3	12.2	8.1	4.6	11.4
IRAQSOMA; 5000 ha fodder	16.9	16.6	17.3	6.0	5.6	8.5	9.8	11.3	12.2	8.1	4.6	11.4

Table 10.10b River abstraction for irrigation, RAI($m^3.10^6$) - Balcad Bridge to Afgooye Bridge

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Balcad Scheme; (i) "Present", 2000 ha	0.9				0.8	4.7	4.9	1.3	1.4	3.3	3.2	3.0
(ii) "Proposed", 8000 ha	10.0				3.0	18.8	19.8	6.1	13.0	17.3	13.8	18.9
Farm Pumping; (i) Seasonal Crops	1.3				0.7	3.6	2.7	0.5	1.5	3.0	3.2	3.8
(ii) Perennial Crops	9.9	9.4	10.0	3.7	2.5	3.6	3.8	5.7	6.4	3.8	2.0	6.0
GISOMA; 4000 ha fodder	14.0	13.2	14.2	5.6	3.9	5.4	5.6	8.1	10.5	7.2	3.9	9.3

Table 10.10c River abstraction for irrigation, RAI($m^3.10^6$) - Afgooye Bridge to Aw Dheegle

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Farm Pumping; (i) Seasonal Crops	3.2				1.6	9.0	6.7	1.2	3.7	7.4	8.0	9.6
(ii) Perennial Crops	2.5	2.3	2.5	0.9	0.6	0.9	1.0	1.4	1.6	1.0	0.5	1.5

irrigation efficiency of 45%, but it does not appear that experimental measurements of irrigation water use have been widely carried out along the Shebeelle. In addition, there seem to be few data available from field surveys of cropping patterns and areas of the sort carried out for this study around Afgooye.

The "Proposed" future areas of irrigated land are the best present forecast estimates. To test alternative demand patterns, the monthly abstraction figures for a given crop in Gu or Der seasons can easily be scaled by area from the values given.

When rainfall or river flow are low in a season, abstraction figures based on full satisfaction of crop water needs during growth, and an overall efficiency of 45%, will not apply over the whole cropped area. Links between land tenure, water rights and actual abstractions by farmers and estates during drought periods could not be investigated in detail during the present short study.

10.3 THE SHEBEELLE RIVER AS A SOURCE OF WATER

This section describes the analysis carried out to identify the availability of water in the Shebeelle for a water supply for Muqdisho. 57 million cubic metres per year will be required for Muqdisho by the year 2000, of which 25 million cubic metres can be met using the present groundwater schemes. Therefore a further 32 million cubic metres per year (or approximately 1 cubic metre per second) is required.

10.3.1 The Shebeelle river flows

Section 10.1.2 describes the available river flow data and the analyses carried out to produce daily, five-daily and monthly flow series. The flows have not been naturalised as the historic irrigation abstractions have not been recorded and would be difficult to estimate with enough accuracy to justify a study of their variation with time. However, many of the existing schemes have been in operation since the beginning of the data collection period and the historic values should represent a stable system up to the beginning of the operation of the Jawhar offstream storage reservoir.

10.3.2 Availability of water in the Shebeelle

A useful technique for indicating the availability of water for a run of river scheme is to plot the data as a flow duration curve. This curve describes the relationship between a given discharge and the percentage of time that discharge is not exceeded (hence non-exceedence probability).

Daily flow duration curves have been constructed for each of the gauging stations and are shown in Figure 10.7. These curves highlight the characteristics of flow in the Shebeelle. For example the peak flows reduce as one proceeds downstream from approximately 170 m^3/sec at Mahadday Weyne to only 80 m^3/sec at Aw Dheegle. This is the result of considerable overbank flooding upstream of Balcad which reduces the peak flow at Balcad. The river in the reach between Balcad and Afgooye is well contained

Flow duration curves for the Shebeelle gauging stations

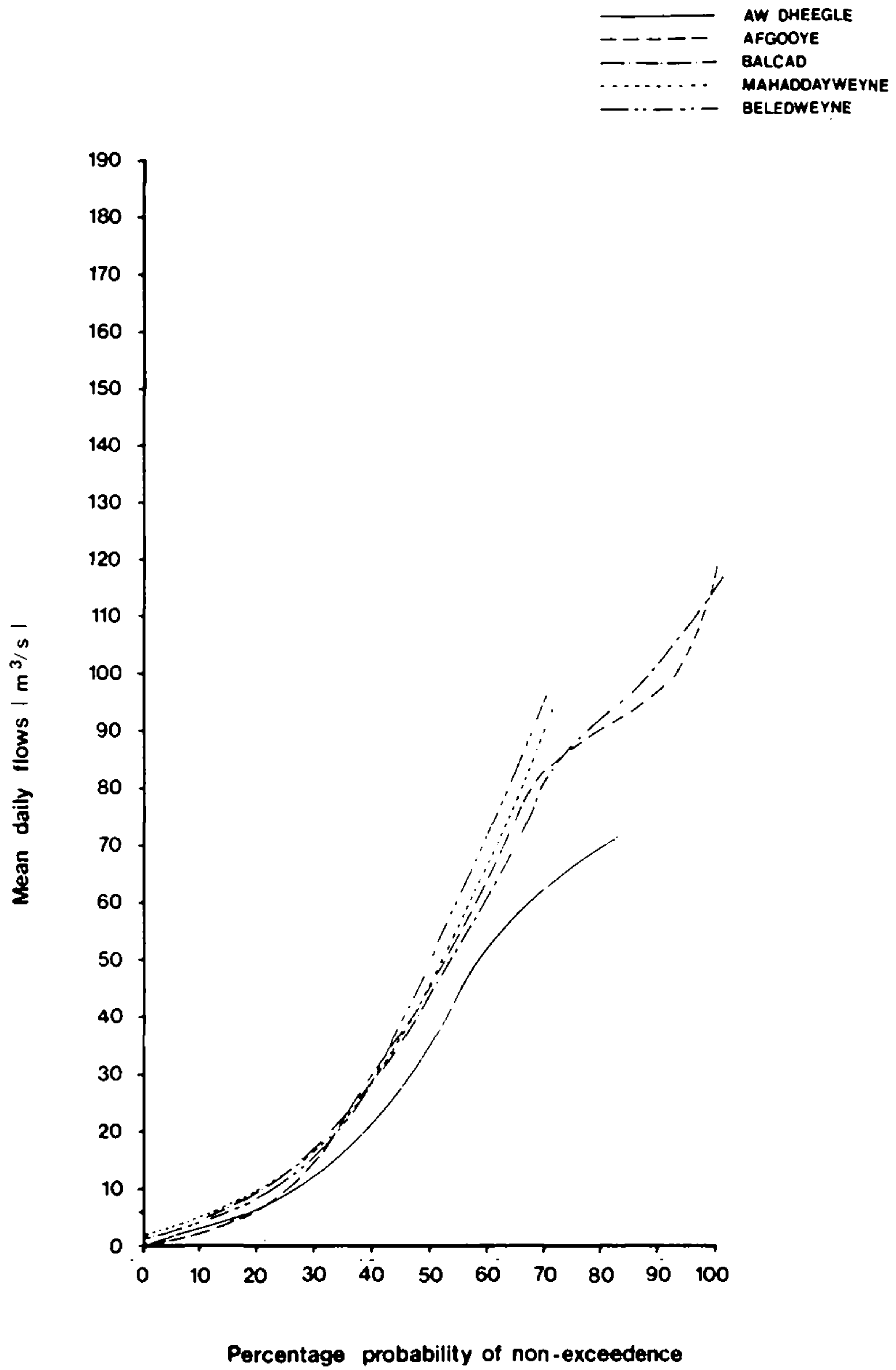


Figure 10.7

within fairly steep banks, about 5 m high, and hence the peak flows at these two sites are very similar. There must be some additional overbank flooding between Afgooye and Aw Dheegle as the peak flows are reduced from 115 m^3/sec to 80 m^3/sec.

Figure 10.7 provides the information for estimation of availability of water. If an abstraction of 1 m^3/sec is required the flow in the river must be greater than 1 m^3/sec. This will ensure the provision of continuous submergence of the pump inlet and will also allow for irregular flow and the irregular shaped river bed. For the purpose of this study we consider that a river flow of 5 m^3/s is the minimum for an abstraction of 1 m^3/s. The percentage of time that this condition is not satisfied varies between 10 and 18 from Mahadday Weyne to Aw Dheegle. The most likely sites for an abstraction works would be at or near Balcad or Afgooye for ease of access and other utilities. The likely availability of 5 m^3/sec at these two sites is 87% and 82% respectively (100 minus non-exceedence probability).

10.3.3 Seasonality in river flows

The Shebeelle flow records show that the monthly distribution of runoff corresponds closely with the distinct two-peaked pattern of rainfall observed in the headwaters. From December to March there is a dry period associated with the northeast monsoon along the coast, when the river levels fall to a minimum. Then follows the Gu season, which is relatively wet, when the river flows remain high before falling again in the summer. A further period of high flows, during the Der season, generally begins during August and finishes in November or December. This seasonality will ensure that the likely availability of water will vary throughout the year. To investigate this, mean daily flow duration curves were constructed for each calendar month and the results for Balcad are shown in Figure 10.8.

The progression of the seasonal flow can be clearly seen and the likelihood of a flow greater than 5 m^3/sec during each month is plotted, for both Balcad and Afgooye, in Figure 10.9. The general shapes of the probability curves are similar but Afgooye exhibits more extreme troughs. Once the required reliability of

Seasonal variation of flow duration curves at Balcad

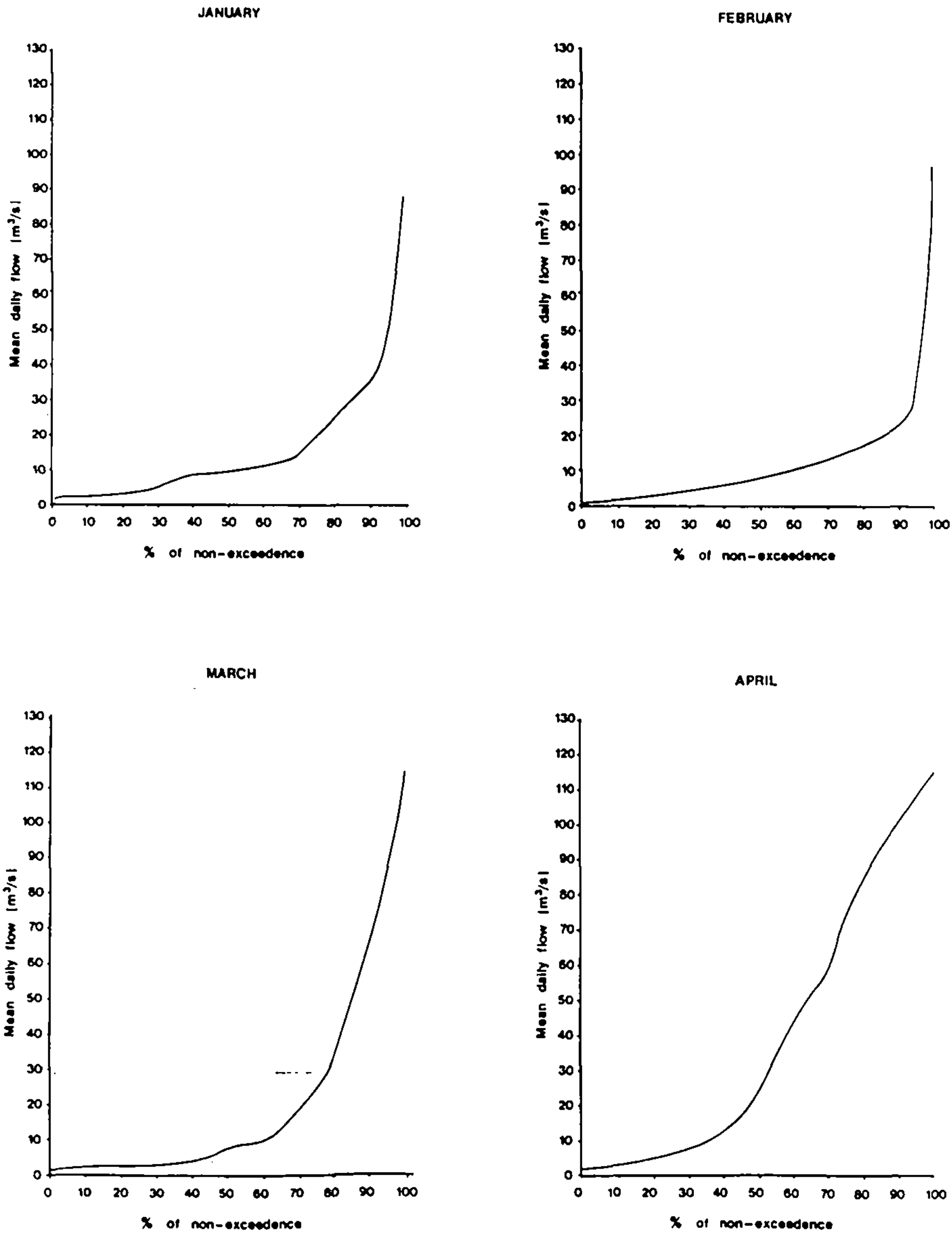
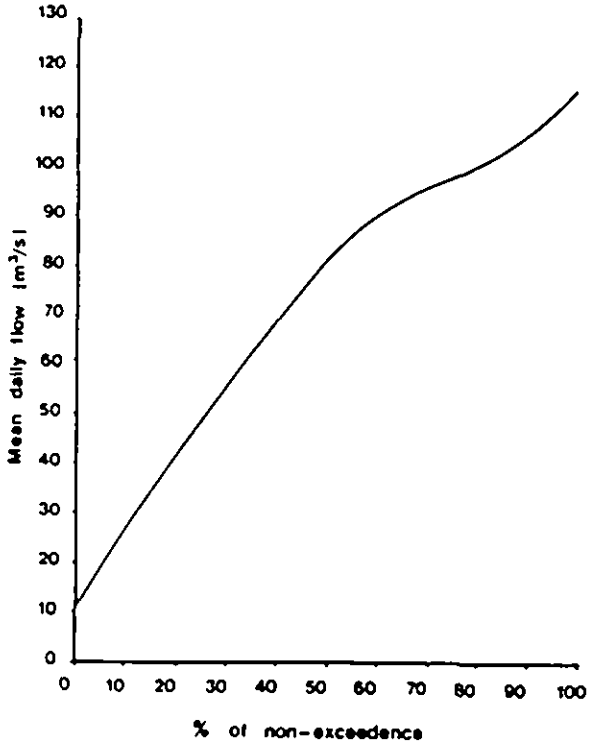


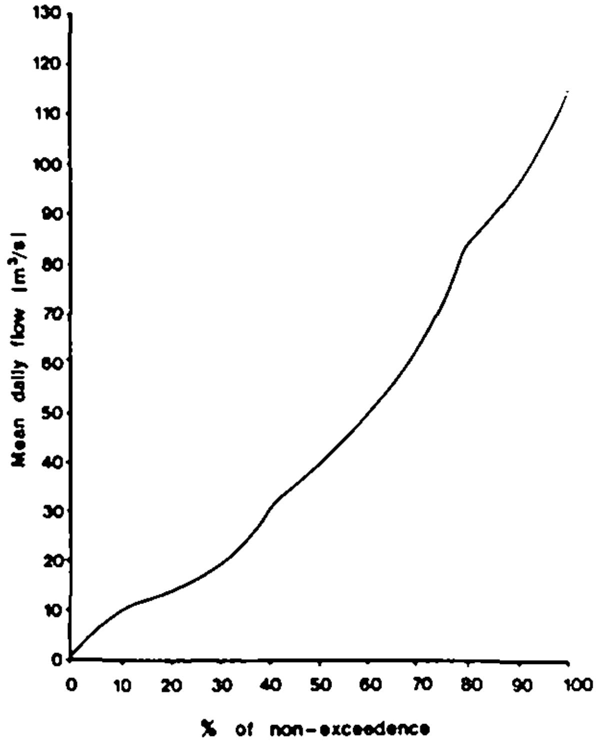
Figure 10·8

Seasonal variation of flow duration curves at Balcad

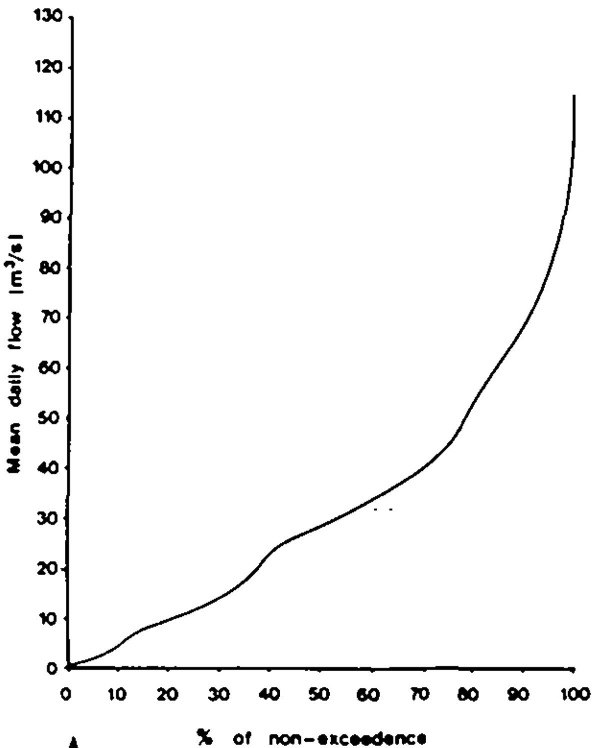
MAY



JUNE



JULY



AUGUST

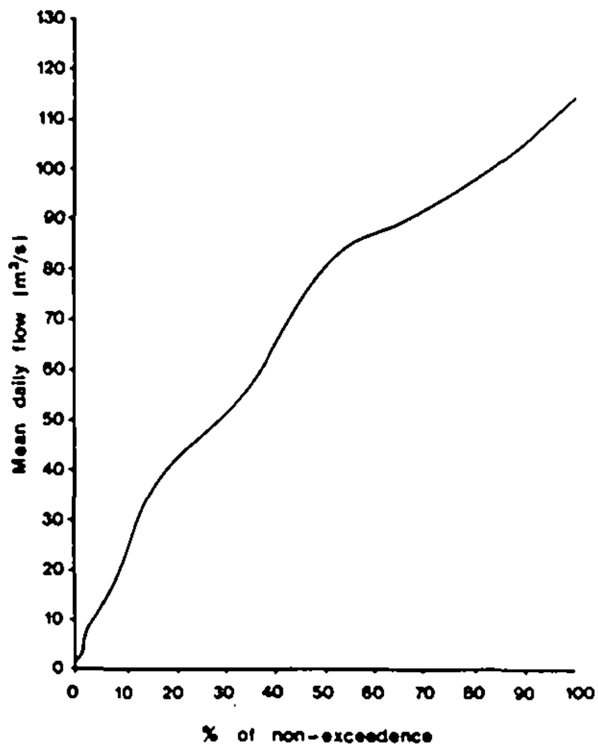


Figure 10·8 cont

Seasonal variation of flow duration curves at Balcad

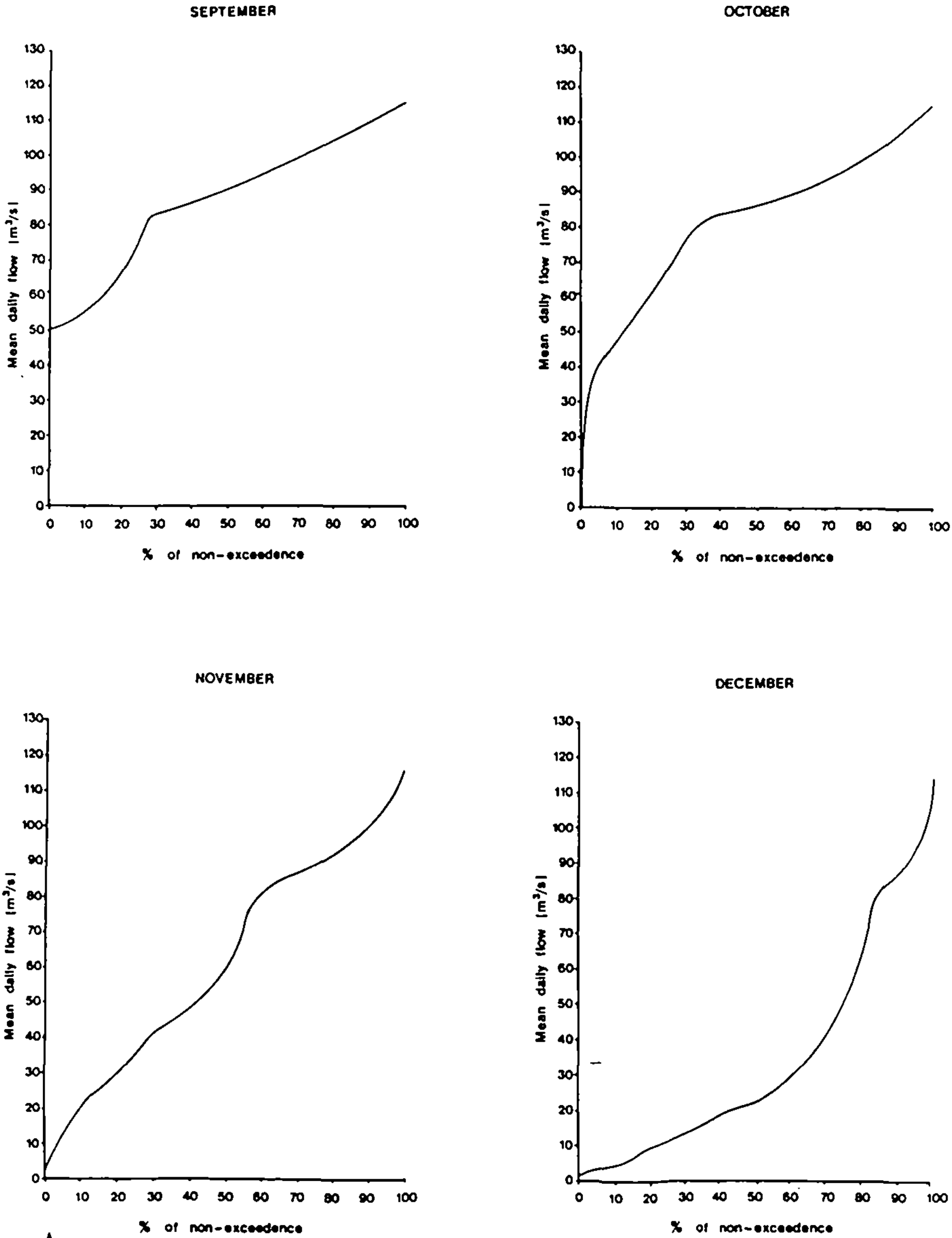


Figure 10·8 cont

The probability of exceedence of $5\,m^3/s$ flow

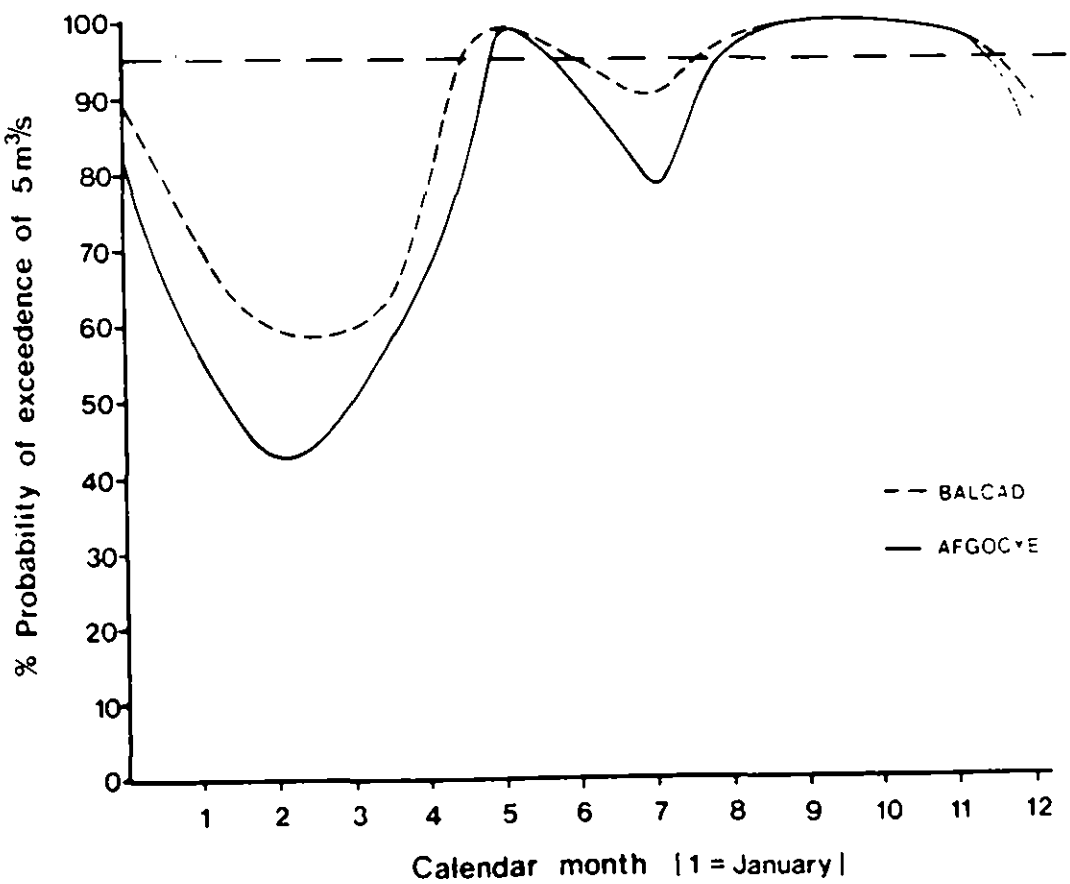


Figure 10·9

the scheme has been fixed the possibility of the river supplying the water can be readily determined. From Figure 10.9 it seems that historically, the river would not be able to supply a demand of 1 m^3/s throughout the whole year. However, it could supply the demand, 95% of the time, during the months of April to May and July to November. This suggests that the river may be more valuable as part of a conjunctive scheme with groundwater rather than an isolated safe yield source.

10.3.4 The future availability of water

All the considerations so far have concerned the historic sequence of flows and hence have implied the historic level of irrigation demands. Also the flow duration curves described so far have not included years during which the Jawhar offstream storage reservoir (JOSR) was in operation.

(i) The model structure

To assess the future availability of water a model has been constructed, shown in Figure 10.10, which divides the river into sections between gauging stations. The model predicts the outflow from each section dependent on the Mahadday Weyne 5-day inflows and intermediate flooding and abstraction.

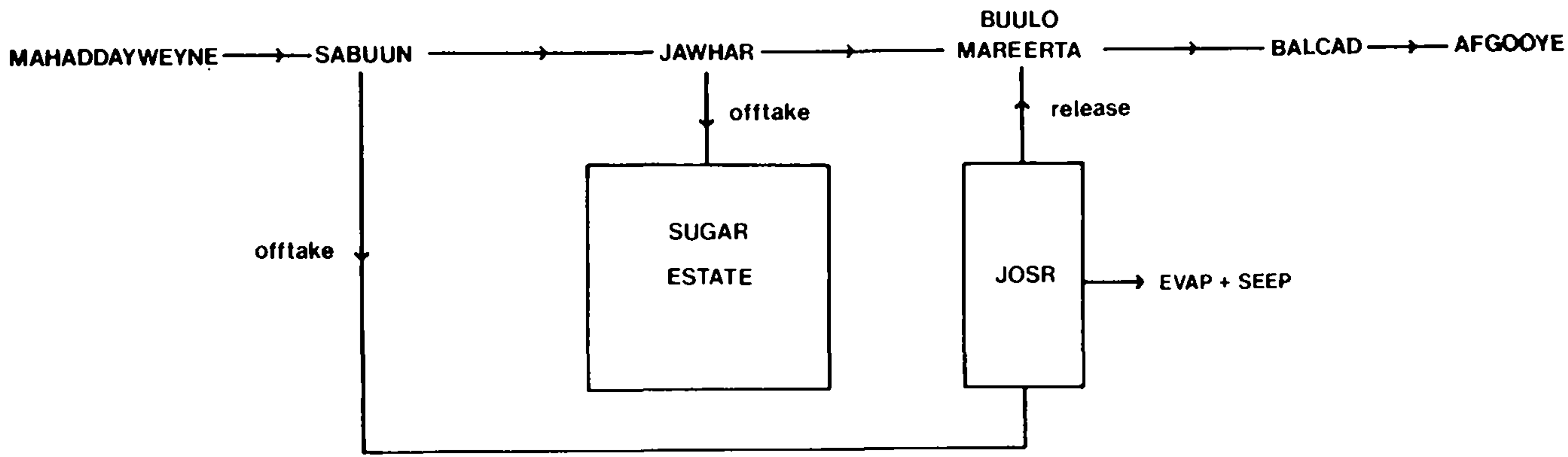
(ii) The inflow sequence for the model

A continuous sequence of flows at Mahadday Weyne was required for input to the model. The historic flows were examined for evidence of serial correlation in the annual statistics. The autocorrelation function for lags of 1 to 5 years are shown below for hydrological data (April to March) for the 13 years of continuous data available..

1	2	3	4	5
0.173	- 0.126	- 0.135	- 0.234	- 0.191

With a null hypothesis the confidence limits are ($1.96/\sqrt{n}$) (where n is the number of years of data) for the lag 1 autocorrelation

Schematic representation of River Shebeelle model



Future inclusions : Flood irrigation equally spaced along reach

Figure 10·10

coefficient i.e. 0.544. Any coefficient lower than this could equally well be produced from random data and does not indicate serial correlation. Thus this set of data does not provide any evidence to suggest that the serial correlation is different from zero at the 95% confidence limit. We have therefore assumed that the flows do not exhibit significant annual serial correlation. The years containing missing data can therefore be discarded and the remaining data used as a continuous series of flows. These flows were collated into a 25 year sequence of 5-day flows.

(iii) Testing the model

The model was tested by using the inflow sequence described above, our best estimate of the historic irrigation demands described in 10.2 and the evaporation estimates from Table 10.11. Flooding was allowed above a certain threshold flow and irrigation was linearly reduced during very low flow periods. An annual summary of the results is shown in Table 10.12. The input to and output from the JOSR is always set to zero as this is for comparison with historic records. These results are very encouraging as the mean annual flow at Afgooye is very close to the historic mean of 1500 million m^3.

A further check was carried out by plotting the 5-day flow duration curves for Balcad and Afgooye and comparing them with the curves produced from the model output. These comparisons are shown in Figures 10.11 and 10.12. The flow duration curves from the model output fits the historic flow duration curves very well. The main difference is the failure of the model to produce a kink in the curve at the peak flows.

This departure from the usual S-shaped curve is probably due to the occasional change of course of river above Balcad during extreme floods. The last occasion when this occurred was the Gu flood of April/May 1981 documented by Gemmell (1982) where the river departed from its usual course above Mahadday Weyne and travelled parallel to the coast. The Landsat image of southern Somalia (Plate 1) clearly delineates several paths of previous flood flows across the plain, by the distinctive winding red traces of healthy vegetation. When this change of course takes

TABLE 10.11 Rainfall and evaporation for the Shebeelle model

	RAINFALL (mm)	EVAPORATION (mm)	NET EVAPORATION (mm)
JAN	7.1	243	236
FEB	1.8	229	227
MAR	15.6	249	233
APR	94.0	192	98
MAY	78.0	169	91
JUN	30.4	167	137
JUL	23.6	169	145
AUG	14.0	177	163
SEP	14.0	195	181
OCT	89.6	202	112
NOV	88.7	176	87
DEC	25.8	262	236
TOTAL	482.6	2430	1946

Five day flow duration curves for Balcad

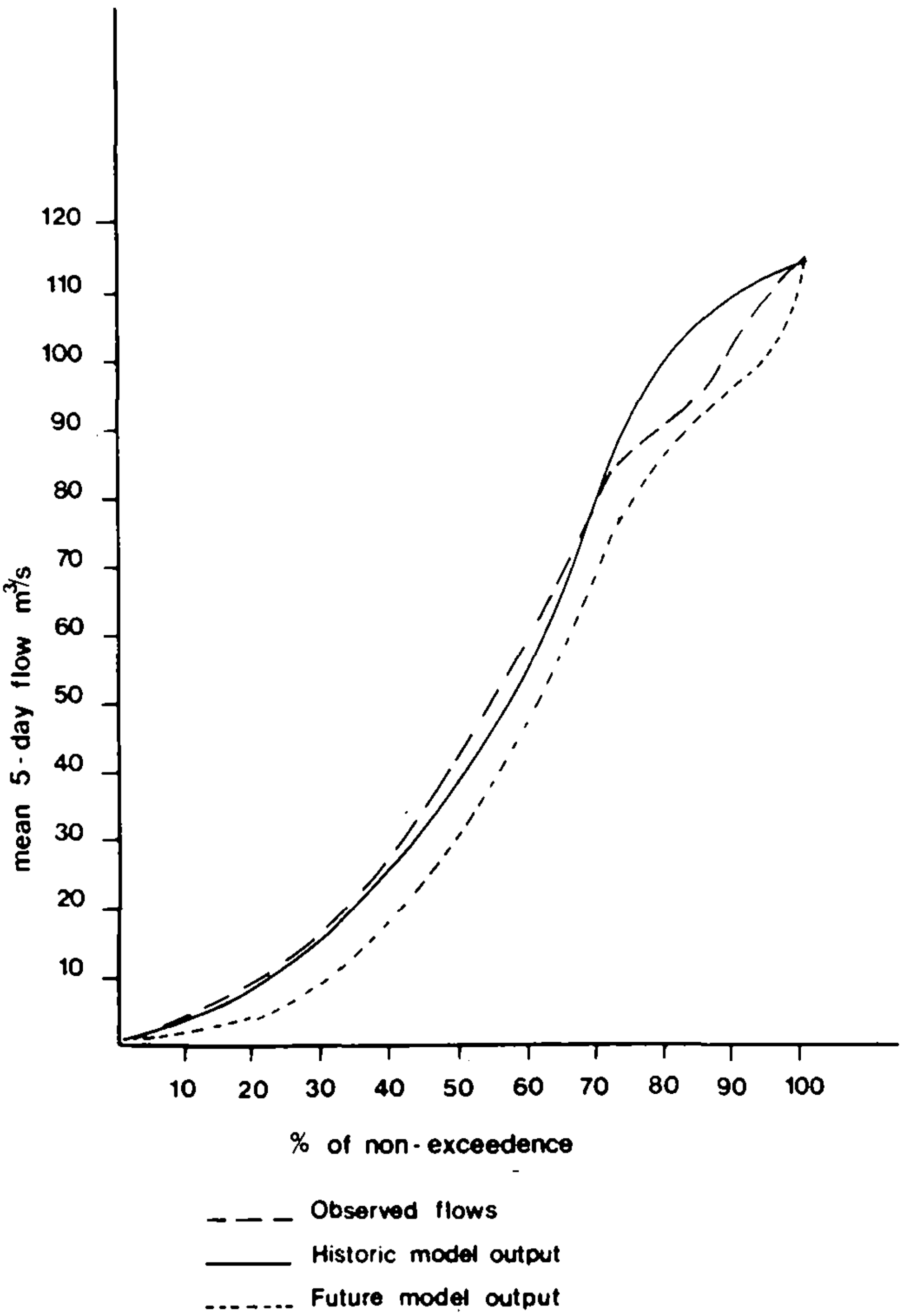


Figure 10.11

Five day flow duration curves for Afgooye

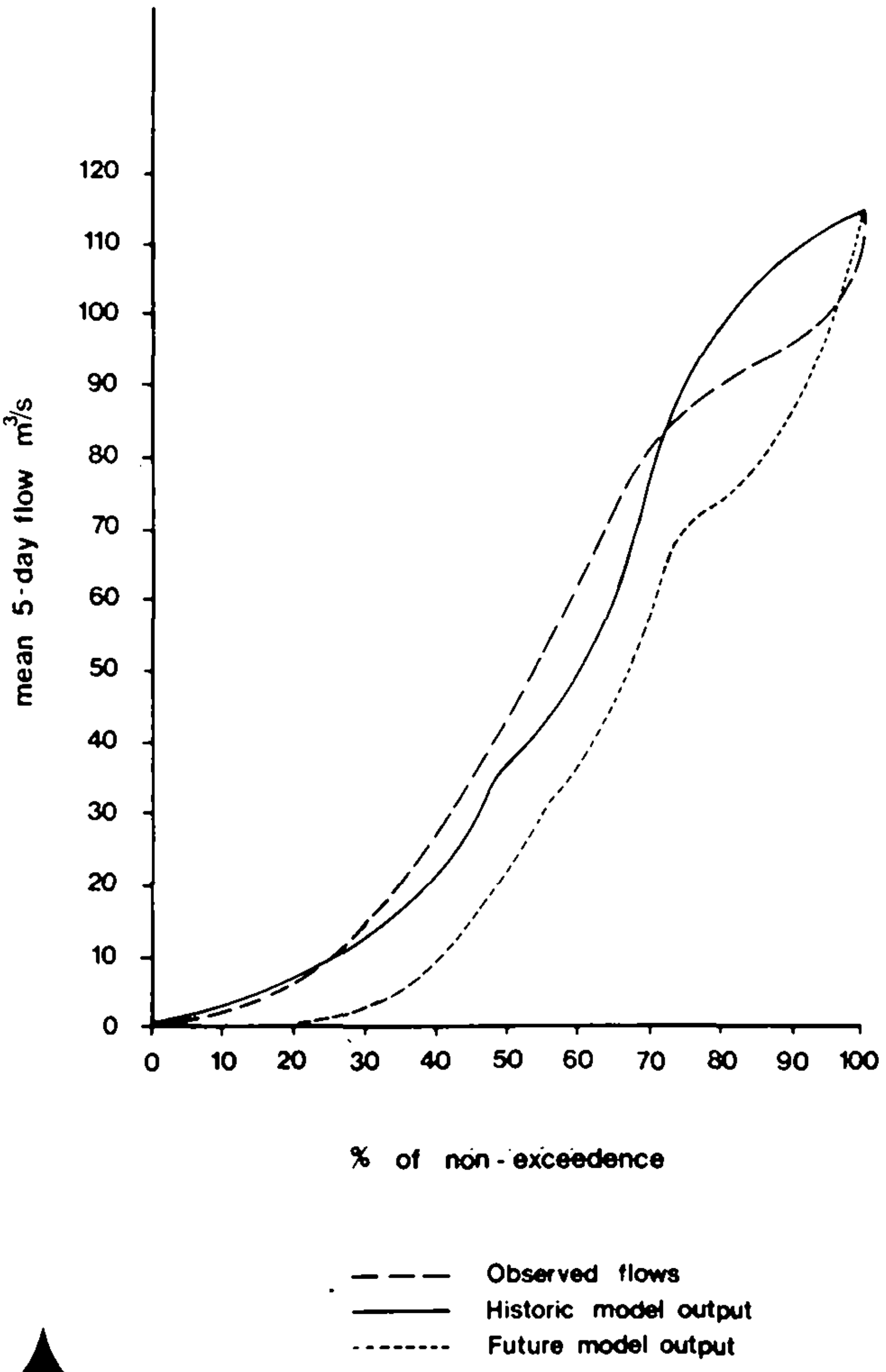


Figure 10.12

Table 10.12 Annual Summary and Mean Annual Values of the Present Behaviour of the Shebeelle River

MAHADDAY WEYNE TO BALCAD REACH							BALCAD TO AFGOOYE REACH			
INFLOW	JOSR INFLOWS	JOSR RELEASES	JAWHAR SUGAR SCHEME	FLOOD IRRIG.	SMALL SCHEMES	SPILL	BALCAD SCHEME	BALCAD FLOWS	AFGOOYE FLOWS	SCHEMES D/S BALCAD
1980.44	0.	0.	90.16	45.00	2.65	12.78	23.45	1806.40	1743.11	63.29
1005.55	0.	0.	40.73	18.00	1.21	0.	23.45	922.15	875.03	47.12
1476.99	0.	0.	56.25	40.50	1.66	8.94	23.45	1346.19	1294.56	51.63
922.26	0.	0.	35.73	20.25	1.16	0.	23.45	841.67	802.82	38.85
1790.43	0.	0.	74.67	47.25	2.15	6.56	23.45	1636.35	1575.38	60.97
1927.01	0.	0.	100.75	27.00	2.50	11.89	23.45	1761.42	1687.58	73.84
1456.12	0.	0.	59.10	40.50	1.65	2.78	23.45	1328.64	1278.84	49.80
1455.91	0.	0.	61.82	36.00	1.87	0.49	23.45	1332.28	1276.14	56.14
1993.93	0.	0.	81.90	54.00	2.60	67.86	23.45	1764.12	1706.04	58.09
1182.98	0.	0.	68.30	20.25	2.31	0.28	23.45	1068.39	1020.14	48.25
2083.59	0.	0.	101.99	31.50	3.15	14.56	23.45	1908.94	1834.77	74.17
1349.78	0.	0.	63.21	20.25	1.24	55.09	23.45	1186.53	1119.48	67.06
1006.24	0.	0.	36.39	9.00	1.36	0.50	23.45	935.54	882.63	52.91
1381.83	0.	0.	63.99	18.00	1.85	3.46	23.45	1271.08	1216.94	54.14
2479.06	0.	0.	123.32	54.00	3.19	39.48	23.45	2235.62	2153.12	82.50
2705.60	0.	0.	135.73	49.50	3.76	87.26	23.45	2405.90	2320.02	85.88
2229.08	0.	0.	95.09	33.75	2.74	56.59	23.45	2017.46	1945.67	71.79
2091.03	0.	0.	72.29	47.25	1.94	107.83	23.45	1838.26	1774.39	63.87
1873.61	0.	0.	93.49	29.25	2.90	21.53	23.45	1702.98	1633.35	69.63
1999.10	0.	0.	80.81	33.75	2.59	112.02	23.45	1746.48	1687.75	58.73
1166.11	0.	0.	53.79	22.50	1.48	0.	23.45	1064.89	1013.51	51.38
3191.32	0.	0.	132.40	54.00	3.63	418.71	23.45	2559.12	2475.85	83.27
1746.46	0.	0.	84.99	20.25	1.68	0.	23.45	1616.08	1534.05	82.03
1847.68	0.	0.	106.26	36.00	2.76	35.02	23.45	1644.19	1585.25	58.94
2558.12	0.	0.	80.52	42.75	2.24	387.52	23.45	2021.63	1950.77	70.86
1796.01	0.	0.	79.75	34.02	2.25	58.05	23.45	1598.49	1535.49	63.01

All values in million cubic metres

place between Mahadday Weyne and Balcad the predicted flow at Balcad will be much greater than actual flow. This would be a very difficult phenomenon to model and as we are primarily interested in the low flows this effect has been ignored.

These two tests have indicated that the model adequately represents the historic river system. The next task is to apply it to future conditions.

(iv) The future river scheme

The historic model was modified by introducing the operation of the JOSR and by increasing the irrigation demands as shown in Table 10.13. The annual summary of the results are shown in Table 10.14 and the 5-day flow duration curves are superimposed on the historic results in Figures 10.11 and 10.12.

The most important result is that the availability of 5 m^3/s is very much reduced from 87 per cent to 75.6 per cent and from 82 per cent to 66.5 per cent at Balcad and Afgooye respectively. The seasonal flow duration curves are similarly affected as shown in Figure 10.13.

10.3.5 Conclusions and discussion

We can conclude from the modelling results that the Shebeelle would not provide a reliable source of water for Muqdisho for the whole year. However it could constitute a very valuable input as part of a conjunctive use scheme, providing more than 1 m^3/s at certain times of the year.

An interesting point from Table 10.14 is the apparent high losses from the JOSR. If one accepts the seepage estimates from Chapter 5, then evaporation and seepage losses account for a high proportion of the available water. This is illustrated by the following crude calculations:

TABLE 10.13 Comparison of historic and future demands

	MAHADDAY WEYNE TO BALCAD					BALCAD TO AGOOGYE
MONTH	JAWHAR SUGAR	FLOOD	SMALL SCHEMES	ROMSOMA	BALCAD	SMALL SCHEMES
JAN	30.6		0.2	0.0	0.89	11.2
FEB	29.4		0.0	0.0	0.00	9.4
MAR	26.4	27	0.0	0.0	0.00	10.0
APR	9.8		0.0	0.0	0.00	3.7
MAY	7.5		0.1	0.0	0.75	3.2
JUN	15.1		0.8	0.0	4.71	7.2
JUL	14.2		0.7	0.0	4.94	6.5
AUG	15.0		0.2	0.0	1.28	6.2
SEP	16.3		0.4	0.0	1.41	7.9
OCT	12.0	27	0.5	0.0	3.29	6.8
NOV	11.0		0.6	0.0	3.18	5.2
DEC	21.5		0.7	0.0	3.01	9.8
TOTAL	208.8	54	4.2	0.0	23.46	87.1
JAN	33.4		0.8	16.91	9.97	25.2
FEB	32.2		0.0	16.57	0.00	22.6
MAR	28.8	27	0.0	17.27	0.00	24.2
APR	10.7		0.0	5.98	0.00	9.3
MAY	8.2		0.7	5.63	3.00	7.1
JUN	16.5		3.1	8.48	18.84	12.6
JUL	15.4		2.4	9.82	19.76	12.1
AUG	16.4		0.8	11.34	6.06	14.3
SEP	17.8		2.6	12.24	13.00	18.4
OCT	13.1	27	2.7	8.06	17.26	14.0
NOV	12.0		2.5	4.62	13.82	9.1
DEC	23.5		2.4	11.44	18.92	19.1
TOTAL	228.0	54	18.0	128.36	120.63	188.0

All values in million cubic metres.

Table 10.14 Annual Summary and Mean Annual Values of the Future Behaviour of the Shebeelle River.

MAHADDAY WEYNE TO BALCAD REACH							BALCAD TO AFGOOYE REACH			
MAHADDAY FLOW	JOSR INFLOW	JOSR RELEASE	JAWHAR FLOOD ABSTR.	FLOOD IRRIG.	SMALL SCHEMES	SPILL	BALCAD SCHEME	BALCAD FLOW	AFGOOYE FLOW	SCHEMES D/S BALCAD
1980.44	224.14	10.97	157.43	45.00	79.49	0.	119.09	1366.25	1252.88	113.36
1005.55	58.75	0.	136.15	18.00	50.33	0.	91.26	651.05	586.03	65.02
1476.99	189.63	31.07	140.88	40.50	60.88	0.	98.24	977.93	890.81	87.12
922.26	73.79	5.90	115.13	20.25	43.62	0.	82.77	592.60	534.82	57.78
1790.43	204.62	20.28	166.65	47.25	73.98	0.	115.41	1202.81	1105.11	97.70
1927.01	158.96	0.	190.71	27.00	91.68	0.	118.13	1340.54	1212.93	127.61
1456.12	172.73	12.28	144.97	40.50	60.26	0.	98.57	951.38	875.41	75.97
1455.91	139.88	11.00	158.56	36.00	64.33	0.	111.83	956.31	875.78	80.53
1993.93	337.81	13.19	150.96	54.00	77.59	0.	113.11	1273.66	1168.02	105.64
1182.98	58.80	16.76	129.47	20.25	59.57	0.	96.07	835.58	750.98	84.61
2083.59	181.38	0.	191.25	31.50	98.38	0.	120.60	1460.49	1323.81	136.67
1349.78	130.03	1.83	187.49	20.25	70.58	0.	117.28	825.99	724.31	101.68
1006.24	39.41	0.	149.45	9.00	51.51	0.	88.47	668.40	588.00	80.40
1381.83	58.33	1.89	136.42	18.00	66.35	0.	109.03	995.60	903.18	92.42
2479.06	317.12	8.07	223.69	54.00	109.44	0.	120.60	1662.27	1512.72	149.55
2705.60	304.07	4.98	225.21	49.50	119.23	0.	120.60	1891.97	1734.31	157.66
2229.08	250.13	19.74	191.21	33.75	94.23	0.	116.37	1563.13	1435.05	128.07
2091.03	354.67	21.01	169.17	47.25	75.80	0.	119.15	1346.00	1246.21	99.79
1873.61	100.61	10.77	182.46	29.25	89.82	0.	119.07	1363.16	1247.87	115.29
1999.10	228.74	13.76	146.38	33.75	78.35	0.	116.04	1409.60	1308.08	101.52
1166.11	40.12	0.	150.76	22.50	60.27	0.	95.86	796.60	724.38	72.22
3191.32	671.04	19.20	224.90	54.00	115.23	0.	120.46	2024.90	1874.96	149.94
1746.46	44.55	6.22	228.12	20.25	90.81	0.	119.58	1249.37	1112.33	137.04
1847.68	123.08	11.62	150.18	36.00	85.47	0.	107.38	1357.20	1248.36	108.84
2558.12	593.03	18.32	183.02	42.75	83.87	0.	119.63	1554.14	1439.61	114.53
1796.01	202.22	10.35	169.22	34.02	78.04	0.	110.18	1212.68	1107.04	105.64

All values in million cubic metres

The predicted probability of exceedence of $5\,m^3/s$ flow

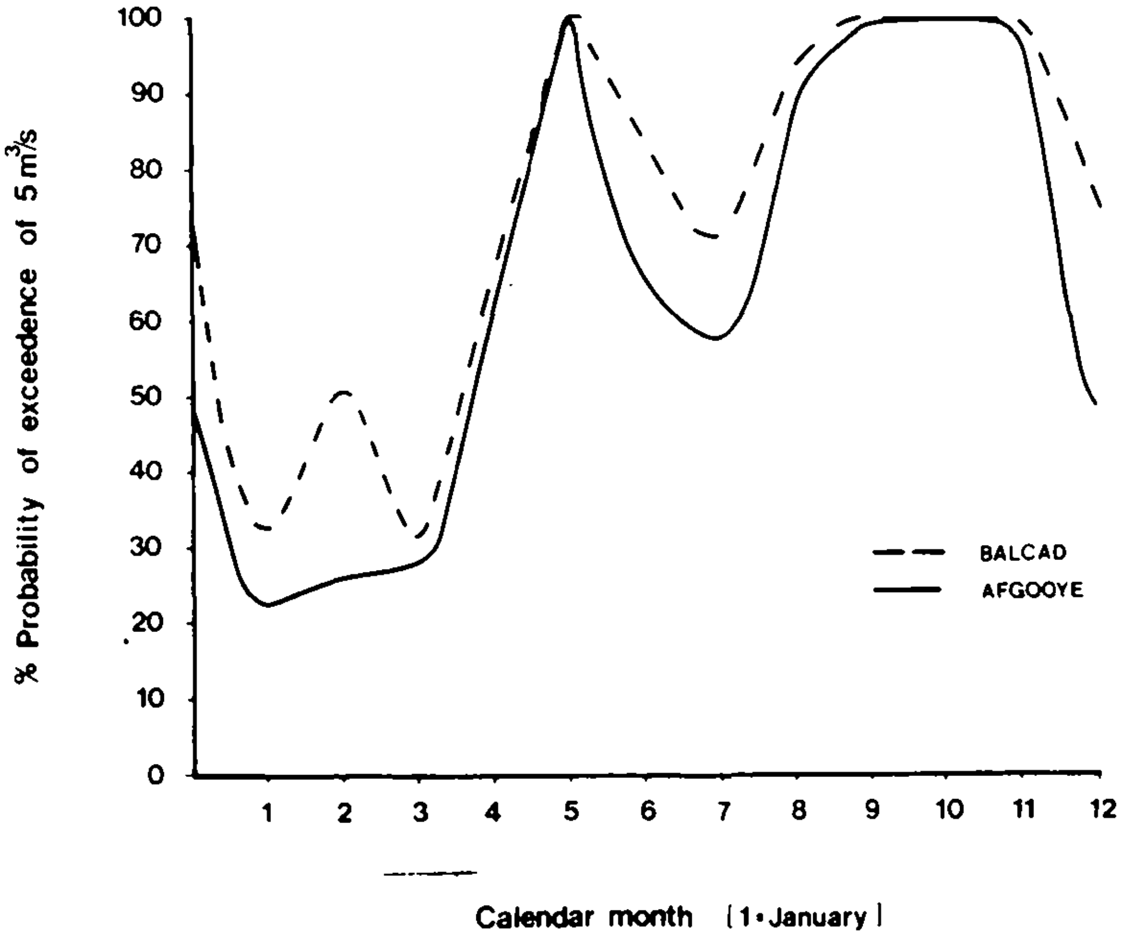


Figure 10.13

Assume that the reservoir is approximately half full and therefore the surface area is about 69 km^2. From Table 10.11, describing the net evaporation over the reservoir, the annual evaporation would be about 1946 mm or 134 x $10^6 m^3$. Seepage, at a rate of 0.4 x $10^6 m^3$/day (Chapter 5), would total approximately 146 x $10^6 m^3$ per year and thus the total losses could be as high as 280 x $10^6 m^3$. These calculations are very imprecise but do show that very large losses are possible. The seepage losses could reduce as the reservoir base becomes silt covered but the present high estimate seems reasonable considering the rise in perched water table described in Chapter 5. These preliminary findings are based on approximate seepage estimates.

Tables 10.12 and 10.14 show that the irrigation requirements from Table 10.13 are not always fully met. It was necessary to impose a reduction in the abstraction during low flow periods to ensure a fit of the model output with the historic data.

The historic case predicts a "SPILL" term in most years which sums the excess flows in the reach Mahadday Weyne to Balcad by considering the maximum possible flow at Balcad. This excess can be thought of as an additional flood irrigation component thus allowing the flood irrigation to be close to that estimated.

In contrast, the output from the future model (Table 10.14) predicts no "SPILL" term; thus there is little evidence to suggest the flood irrigation could be augmented. This shows the effect of increasing the large pumped scheme abstractions on the flood irrigation areas and is an important consideration for planning the expansion of irrigated areas.

10.4 CONCLUSIONS AND RECOMMENDATIONS

10.4.1 The available data

Section 10.1 describes the data that were available for the hydrological analysis and the data processing procedure used to obtain daily, five daily and monthly flow sequences for each of the gauging stations.

Towards the end of this study a report by Gemmell (1982) was made available which incorporates a vast amount of work and knowledge concerning the Shebeelle and Juba rivers. At this stage we had already recomputed the flows and carried out the majority of the analyses. However, we have compared the annual flow series produced by both Gemmell and ourselves for Afgooye and the two are virtually identical. We have therefore not attempted to repeat the analyses presented in this report with the newly available data. In the light of this new report we would recommend the future use of the river flow information produced by Gemmell as it represents the culmination of three years work in Somalia.

Evaporation data are very sparse in Somalia but climatological information is of prime importance when discussing irrigation projects. We recommend that more effort should be made to collect these data on a national basis.

10.4.2 Irrigation abstractions from the Shebeelle

In Section 10.2 the river has been divided into the three main reaches: Mahadday Weyne to Balcad, Balcad to Afgooye and Afgooye to Aw Dheegle. The different methods of irrigation have been described and irrigated areas defined in each reach.

Finally the abstractions necessary for irrigation requirements have been quantified for the present and for the year 2000 including the proposed schemes which seem most likely to be constructed. The results of these analyses are summarized in Tables 10.10 a, b and c. This information provides the irrigation inputs for the river model discussed in the next Section.

The irrigation requirements are calculated using cropped area and crop water requirements so that the effect of alterations in the future crop distributions can easily be estimated.

10.4.3 The Shebeelle River as a source of water

The analyses in section 10.3 have investigated the availability of water in the Shebeelle at the present and in the future. The results show that the Shebeelle is not capable of sustaining a constant water supply adequate for future Muqdisho demands. However there are considerable seasonal resources available at both Balcad and Afgooye and these could be useful as part of a future conjunctive scheme.

The study of the future water availability has not mentioned the effect of the recent barrage developed at Balcad. The abstractions at Balcad have been incorporated but the ponding of water at Balcad will tend to reduce flows at Afgooye. No operational details of the Balcad barrage are available but the effect will be to reduce available water resources at Afgooye and make abstraction at Balcad seem more favourable. Because of this barrage any future gaugings and flow measurements at Balcad will be meaningless.

The study carried out to assess the future availability of the Shebeelle water included a simple model of the operation of the JOSR based on estimates of evaporation and seepage. This indicated very high losses from the reservoir due to seepage and evaporation.

11. CONCLUSIONS

11.1 INTRODUCTION

In this chapter we bring together the main conclusions reached during our investigations of ground and surface water resources in the vicinity of Muqdisho.

The conclusions for each type of water resource are dealt with separately. They relate specifically to the capability of each resource of meeting future water demand in Muqdisho. The conclusions do not take account of engineering, economic, or management aspects of source development.

11.2 GROUNDWATER RESOURCES

1. There is considerable potential for groundwater development in the area to the north east of Muqdisho. However the extent to which this potential can be realised will be limited due to poor water quality.

2. There are two sources of potential pollution. The first is the intrusion of salt water into the aquifer from the coast. The second is the movement of high conductivity groundwater from the north of the Shebeelle River.

3. Recharge to the aquifer at Muqdisho is certain to occur as a result of improved water supplies. The magnitude and extent of this recharge will be affected however by the introduction of a sewerage system. Within the time scale of this study it is unlikely that a complete sewerage system will be constructed in Muqdisho. It is therefore recommended that groundwater source development be based on the decision matrices assuming a 35 per cent recharge at Muqdisho, as summarised in Table 9.5.

4. The intrusion of salt water into the aquifer from the coast has been shown by the model studies to present no constraint on development within the time scale and rates of abstraction considered. Even without recharge to the aquifer at Muqdisho all abstractions considered up to 2020 lie within the accept or provisionally accept categories of the decision matrices.

5. The possible movement of high conductivity water into the proposed area of abstraction is the major constraint to realising the potential of the aquifer. Movement will be greater at higher abstraction rates become less marked at lower rates of abstraction.

   The current data suggests that the aquifer can sustain abstractions up to the level required to satisfy the year 2000 demands. However if abstractions are increased to and maintained at that level the poor quality water is likely to be drawn into the wellfield sometime between 2010 and 2020, if a lower level of abstraction is adopted, say to meet the 1997 level of demand then the poor quality water is unlikely to be drawn into the wellfield until beyond the year 2020. This is illustrated by Table 9.5.

6. It is extremely difficult to quantify limits to abstraction on the basis of water quality criteria. We have therefore been forced to adopt a conservative approach in our analysis and recommendations. In view of this it is recommended that future groundwater development be based on our findings as indicated by Table 9.5 but that detailed monitoring of groundwater movement be carried out so that our predictions can be checked against field data. This check will indicate if abstractions are too high or if the aquifer will sustain further development.

7. All our predictions assume that the recharge from the Shebeelle will be maintained at the magnitude indicated by the analysis of the 1983 regional ground water levels. Any reduction in recharge could have serious consequences.

irrigation is likely to increase rather than to decrease the rate of recharge.

8. Finally we stress the necessity for regular monitoring of groundwater levels and quality, particularly in sensitive areas. Three particularly sensitive areas are between Afgooye Road wellfield and the coast, between the Stage 2B wellfield and the Shebeelle River and lastly inland of Muqdisho, where the quality of water flowing from the recharge mound created beneath the city will need to be monitored.

11.3 SURFACE WATER RESOURCES

1. The predicted flows for the Shebeelle are shown on Figure 10.13, and our conclusions largely relate to these predicted flows.

2. To satisfy year 2000 demands in Muqdisho a flow of 1 cubic metre per second must be abstracted from Shebeelle River. Our studies have suggested that in order to abstract this flow from the Shebeelle a minimum flow of 5 cubic metres per second must be available in the river.

3. On the basis of the above it is possible to abstract the required 1 cubic metre per second for only five months of the year with a five per cent risk of failure. The months during which water can be abstracted are May, August, September, October and November.

4. If the minimum flow that will permit abstractions of 1 cubic metre per second is reduced from 5 cubic metres per second to 2 cubic metres per second the month of June may be added to the above list.

5. There is no reliable flow in the Shebeelle River in the months of January, February and March. The full requirements for demand in Muqdisho would therefore have to be satisfied by other sources during a minimum of these three months in every year.

6. In the transition months of April, June, July and December it would be possible to abstract some water for supply purposes in some years. The threshold discharge of 5 cubic metres per second is exceeded on average 65 per cent of the time in April, 83 per cent in June, 71 per cent in July and 75 per cent in December.

7. The Shebeelle River does not therefore form a reliable source of water to satisfy demand in Muqdisho.

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