Operational considerations in the development of a spate irrigation system

E. P. Sephton and B. J. Allum,
Sir M. MacDonald & Partners

1. Background to the project

1.1 Location and brief description of project area

The Wadi Mawr project is located in the Tihama coastal plain of the Yemen Arab Republic, some 100 km to the north of the main port of Hodeidah. The wadi has been used as a source of irrigation water for centuries, using a network of individual offtakes from the wadi, each controlled by some rudimentary form of diversion works. The total area commanded by the existing system is of the order of 15 000 ha.

The wadi catchment area is primarily a rugged mountainous region composed of weathered and fractured rock with relatively steep slopes; only about 17 percent of the area is terraced or cultivated. The surface runoff comes from high intensity, short duration thermal storms and gives rise to highly variable spate flows in the wadi. A large amount of sediment accompanies the surface runoff, and is deposited in the wadi and its floodplain, in the irrigation canals and on the fields.

The project area is in a gently sloping plain of deltaic materials laid down by the wadis which flow from the mountains. The soils, derived from the sands and silts deposited by Wadi Mawr, have generally good texture and water-holding characteristics and thus favourable sustained irrigation. The depth to water table ranges from about 5 m in the west to 30 m in the east. The relief is generally featureless except for a few isolated rocky hills in the east and some sand dunes in the west. Natural vegetative cover is sparse or non-existent.

The wadi bed in the upstream reaches consists mainly of layers of sand and gravel, with some surface armour of shingle and cobbles. The wadi floods are contained within a floodway 200 m to 300 m wide, bounded by low cliffs of weakly cemented sands and gravels, with occasional outcrops of weathered rocks. The main channel is generally poorly defined and unstable with low flows following a typical riffle and pool form. On reaching the coastal plain there is a change to a sand bed with a floodplain. The banks are less resistant to erosion and the floodway widens considerably to over 1 km in places, partly covered with reeds and scrub growth. The wadi regime is affected by the activities of the local cultivators who construct temporary sand banks and groynes to deflect flows from the wadi into their canals. Average wadi bed slopes range from about 4.4 m/km in the upstream reaches to 2.9 m/km in the lower reaches.

1.2 Traditional irrigation practice

The overriding principle for traditional water distribution is the right under Islamic Law of the upstream user to abstract water before passing flows on to the next downstream user. Administration of the distribution is undertaken by Water Masters. One is assigned to each canal to determine allocations to individual areas, organise maintenance and repairs to canal systems and to arbitrate in disputes. Wadi Mawr is a perennial river but baseflow is very low for most of the year. During the months of December, January and February daily mean discharge is normally only 2-3 m^3/s. During the winter months, therefore, only a limited number of upstream canals are capable of abstracting the wadi baseflow.

Early studies identified those canals which traditionally received low flow, and the distribution of flows within the perennial area was investigated. No set pattern of distribution appears to predominate, but lower canals are generally dependent on the availability of residual flow downstream of the upper canals. However, since the larger spates frequently destroy the temporary diversion works of the upper canals, the lower canals benefit from enhanced flows on the falling spate flows.

For those canals which receive winter flows only a proportion of their total potential irrigable area is actually given over to perennial cropping. For the uppermost canal this is generally of the order of 40 percent. For other lower canals this reduces to about 20 percent. Cropping intensities are thus generally low.

The actual area under irrigation in a particular season varies with the location of the canal offtakes, the frequency of the floods in the wadi, the performance of the temporary diversion works and those of the upstream diversion works and the standard of maintenance in the canals. This is particularly so for those canals offtaking in the less reliable zones in the wadi.

Generally those canals in the middle wadi area feed a proportion of their total irrigable area with varying supplies throughout the early and late rainy periods. This normally allows the production of two crops each year, although such double cropping is highly variable. Further downstream single cropping predominates, although some areas may be planted for a second crop in anticipation of some further yield. The canals furthest downstream may receive only a single watering; some none at all in dry years. The efficiency of agriculture in these areas is thus low.

The principle crop grown is sorghum. Some cotton, millet and sesame are also grown.

1.3 Wadi hydrology

The wadi catchment area is about 7 900 km² and is located in the mountainous terrain to the east, rising to 3 000 m above the coastal plain. Annual rainfall over the catchment is highly variable, but generally lies within the range 900 mm in the south-west to 200 mm in the north-east, and results predominantly from high intensity thermal storms.

Of principal interest in the design of the irrigation system were records of wadi flows. The Shat el Erge gauging station, located some 7 km upstream of the proposed diversion site was established in 1975 during the feasibility study and together with the results of field hydrology studies in 1982 provided the basic data required for hydrological analysis. Estimated return periods for floods in Wadi Mawr were derived from the data available and a value of 3 100 m^3/s was assigned to the 100 year design flood for the principal diversion structure. Provision was made in the design for catastrophic floods in excess of the estimated design flood. The total mean annual runoff was estimated at about 216 million m^3, of which approximately 88 percent occurs as a result of discharges less than 40 m^3/s, the adopted headworks capacity. The mean monthly discharges are given in Figure 1. Monthly discharge-duration analyses carried out using the available records indicated the predominance of low flows. Table 1 illustrates this.

The hydrograph at the gauging station reflects the direct surface runoff and interflow from the catchment areas. Within a short time period after a storm the stage rises very quickly and then recedes rapidly. The recession then drops

Figure 1 Mean monthly wadi flows

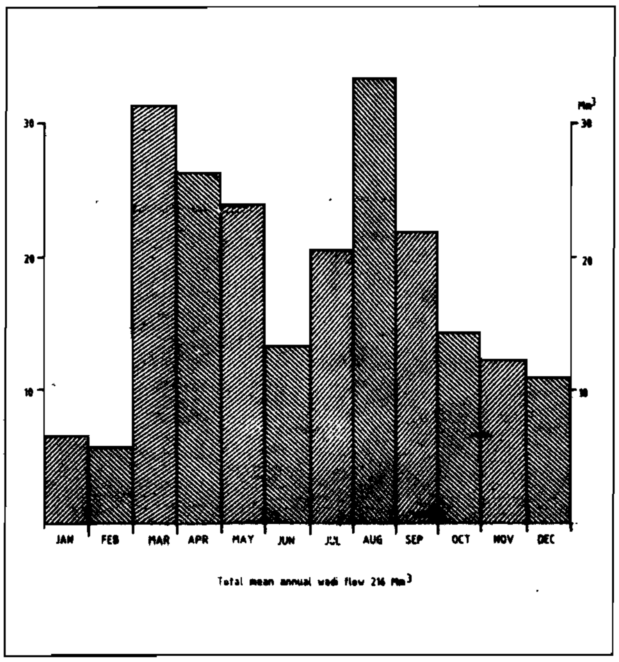


Table 1 Proportion of time a given discharge is equalled or exceeded

Discharge (m^3/s)	Proportion of time
5	0.44
10	0.14
20	0.06
40	0.03

at a lower rate as the slower moving interflow reaches the wadi. The discharge continually declines until another storm event occurs over the catchment, which, during the rainy seasons is nearly every day. Typical wadi flood hydrographs are shown in figure 2.

1.4 History of the scheme and outline of the irrigation works

In 1973, following a survey of groundwater resources in Wadi Mawr, a full feasibility study of the potential for surface water and groundwater development was carried out. This indicated that major engineering works on the wadi to divert and distribute flow, limited groundwater development and the provision of infrastructural works were feasible from all technical and economical considerations. Detailed field survey works and design work started in 1982, with construction work on the initial contracts commencing in 1983.

The major portion of the adopted project is the construction of a cross wadi diversion weir, headworks and irrigation canal system. The design of the weir and headworks was primarily dominated by two factors:

- the sudden rise in spate flows; and
- the large concentration of sediment carried.

To prevent sediment entering the canal system, twin settling basins have been provided. While one basin is in operation ('settling mode'), the sediment in the other basin can be flushed back into the wadi ('flushing mode').

The principal components shown on figure 3 are thus:

(i) a scour sluice—to scour sediment from the intake forebay and to maintain a clear leading channel in the wadi adjacent to the intake;

(ii) twin canal head regulators—to control the flow into the intake feeder canals;

(iii) twin vortex tubes—to remove the very coarse sediment moving along the bed of the feeder canals and so prevent it from entering the settling basins where it would be difficult to flush out;

(iv) two settling basins—to capture sediment during normal system operation;

(v) two flushing sluices—to eject sediment deposited in the settling basins and pass it back to the wadi; and

(vi) a by-pass sluice—to pass low base flows from one settling basin in which water level is controlled to reduce seepage losses.

During normal operation canal supplies pass over a

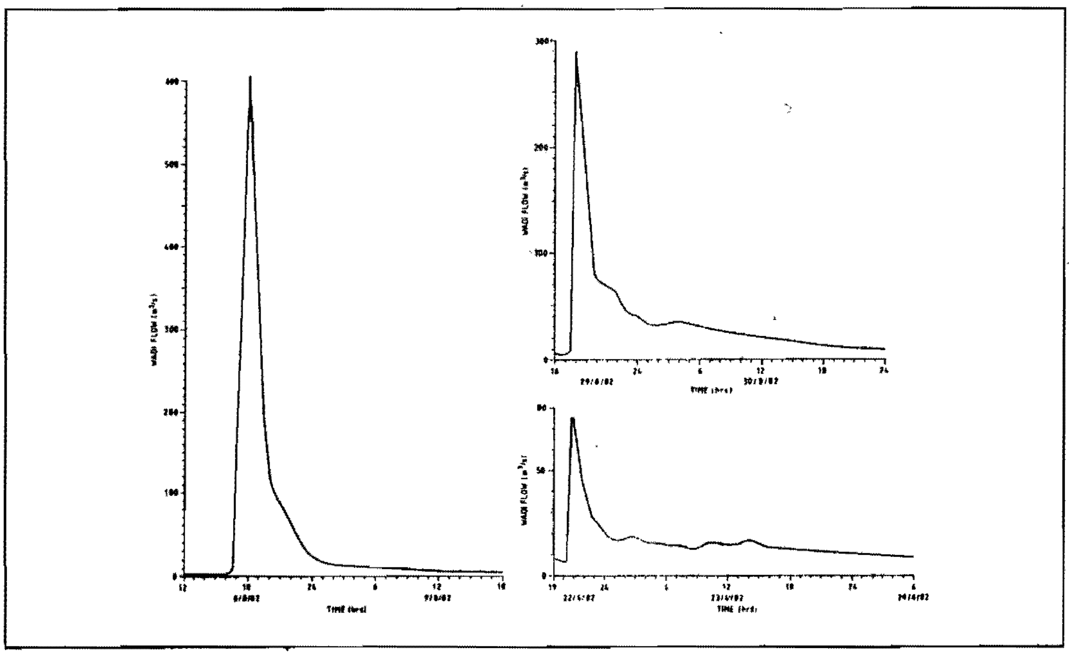


Figure 2 Typical Wadi Mawr hydrographs

labyrinth weir into a common pool, the peak discharge from which is regulated to within a maximum variation of 10 percent by Neyrtec baffles. Any excess water escapes back to the wadi via an overflow weir.

Diverted flows are delivered to the 39 existing primary canals served by the scheme by offtakes from a combined headreach canal and two supply canals; one on each side of the wadi. The north supply canal serves 17 primary canals over its 19 km length and commands 5 100 ha. The south supply canal, after crossing the wadi in a siphon underpass, serves 21 primary canals over its 25 km length and commands 10 800 ha.

2. Scheme design concepts

2.1 Introduction

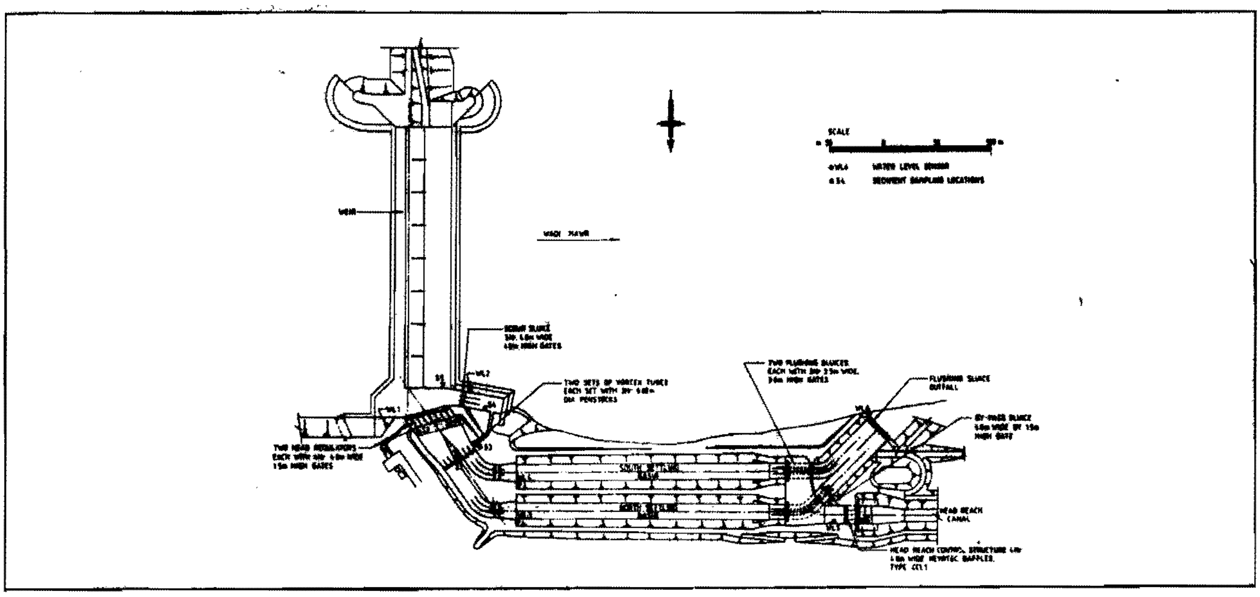
In the early stages of design, consi ion was given to providing several diversion structure . the wadi, each serving groups of canals. A number of options were investigated in detail and costed. As a result, a single structure serving the whole irrigable area was found to be economically preferable, as well as offering technical advantages. A single offtake on one bank would simplify the exclusion of sediment and trash, increase the effectiveness of the scour sluices and would avoid the difficulty of maintaining flow on both banks to serve separate offtakes. The diversion weir is located so that all existing primary canals served by the upper reaches of the wadi would derive benefit from the scheme, although site selection was based mainly on wadi morphology and local topography.

The main aims of the new irrigation system are:

(i) to maximise the volume of water which can be economically abstracted from the wadi each year; and
(ii) to distribute the water abstracted from the wadi as efficiently and equitably as possible taking account of existing local practices.

It was found that increasing the canal headworks above 40 m³/s had progressively less effect on the diversion

Figure 3 General layout of headworks



efficiency; at 50 m³/s, a 25 percent increase in abstraction capacity, the diversion efficiency increases only 2 percent from 88 percent to 90 percent. An analysis of costs showed a similar pattern. Thus the canal headworks capacity was set at 40 m³/s, slightly in excess of that required to provide an average duty of 2 l/s per hectare over the estimated net irrigable area of 18 800 ha.

2.2 Model studies

Because of the anticipated operational problems, a special mobile bed hydraulic model was commissioned at Hydraulics Research Ltd., in the UK. The model, built to 1:50 scale, had the facility for spate flow simulation and sediment input. Figure 4 gives a plan of the model and shows the approach conditions.

The size of the mobile bed model sediment was based on theoretical calculations and the experience gained from previous wadi models on Wadi Zabid. A natural sand with D_{35} of 0.13 mm was chosen, being based on the D_{35} sediment fraction of the coarser samples of the bed material observed in the wadi low flow channels.

Based on the hydrological analysis of data from the Shat el Erge gauging station, a standard unit hydrograph shape was derived for the simulation of spate flows. The sediment supply rate was determined from the results of field data and computations based on the Ackers White sediment transport equations.

Following acceptable proving tests, main tests were carried out to assess:

- the influence of upstream flow conditions and the stability of the low flow channel;
- sediment exclusion performance; and
- flow patterns and the stability of flow downstream.

The main points of interest of operational significance resulting from the model studies are described briefly below:

(i) the approach flows to the canal headworks were stable and satisfactory; and

(ii) the sediment exclusion tests indicated that the sediment concentration in the canal would increase with increasing wadi discharge. However, limitations on the operation of the scour sluices, namely their closure during canal operation, will prevent the ingress of large amounts of sediment.

A point of interest was the variation of sediment intake on the flood recession, where the concentration of sediment entering the canal decreased until it reached a minimum and then started to increase again.

Model studies were thus effective in providing solutions by design and by the adoption of certain operating procedures to satisfy the somewhat conflicting objectives of low flow channel stabilisation and minimum sediment entry to the canal system.

2.3 Sediment removal

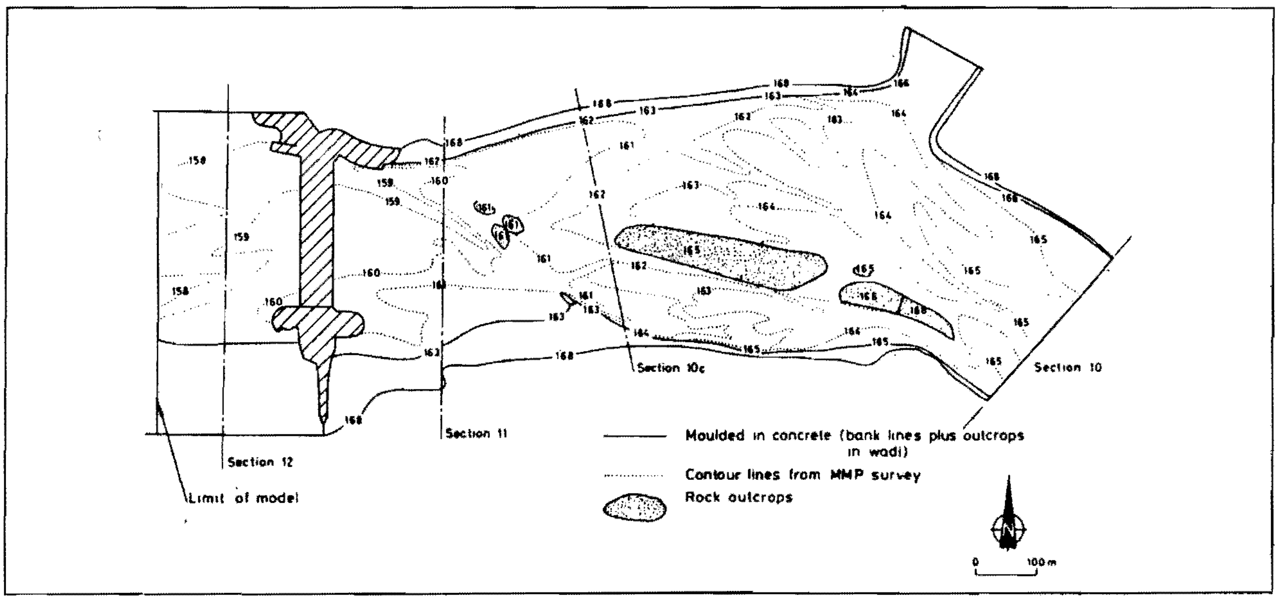
Early designs for sediment removal works featured vortex tube sand extractors, combined with settling tanks and vacuum operated pumps for the ejection of sediment. However, largely as a result of very high sediment concentrations measured at Wadi Zabid, the methods of sediment removal were revised to incorporate twin settling basins coupled with vortex tube extractors.

The characteristics of the spate floods, namely the unpredictable frequency and magnitude of events, led to some variation to the normal settling basin design:

- the increase in through velocities and bedshear in the basins during the settling mode, to reduce the amount of sediment that will be deposited in the settling basin. This has the advantage of extending the time required for the basins to fill up and consequently also reduces the time to flush out the sediments;
- the incorporation of limitations on operation, to restrict sediment entry at high concentrations; and
- the provision for mechanical clearance in case of an emergency.

The performance of the settling basins, both in the settling and flushing modes, relies on theoretical sediment

Figure 4 Plan of model



calculations. The estimated sediment concentrations in the wadi are based on sediment sampling carried out by Hydraulics Research Ltd. on Wadi Zabid and adjusted to take account of the differences between Wadi Zabid and Wadi Mawr catchments with respect to rainfall concentrations and catchment characteristics.

2.3.1 *Settling mode*. The design discharge for the settling mode is 40 m³/s. The theoretical maximum spate flood at which the sediment concentrations carried exceeds the carrying capacity of the canals, is about 300 m³/s for a rising flood. The limiting wadi discharge for the settling mode on a falling flood has been set arbitrarily at 400 m³/s. The reason for this is that sediment concentrations on a falling flood are known to be correspondingly less than those during rising floods. A typical spate flood with a 300 m³/s peak flow would half fill one basin in 12 hours. At a constant 40 m³/s an empty basin would fill to the same extent in about 31 hours.

2.3.2 *Flushing mode* To maximize the rate of scouring of the basins and to increase the spate capacity at which the basins can be flushed, a peak flushing capacity of 80 m³/s has been adopted. However, the potential for flushing depends on several additional factors, namely the Manning coefficient, the slope of the bed and the sediment concentration entering the basin. Each of these factors has been investigated in some detail and expected values determined for an analysis of typical flushing rates.

The Manning coefficient is expected to be low under scouring conditions, particularly since the sediment load is high. An n value of 0.015 has thus been assumed. The bed slope is expected to lie within the range 4 to 8 m/km, although sufficient head is available for initial slopes of 12 m/km to develop. The D50 size of the deposited sediment is assumed to be coarser than that of the incoming sediment which has a D_{50} size in the range 0.10 to 0.125 mm. A value of 0.15 mm has thus been assumed for median floods.

On the basis of the above assumptions it was found that the typical sediment load of 10 920 m³ deposited during a 300 m³/s flood will be removed by flushing during a similar event (in approximately 4 hours) thus confirming the design criteria.

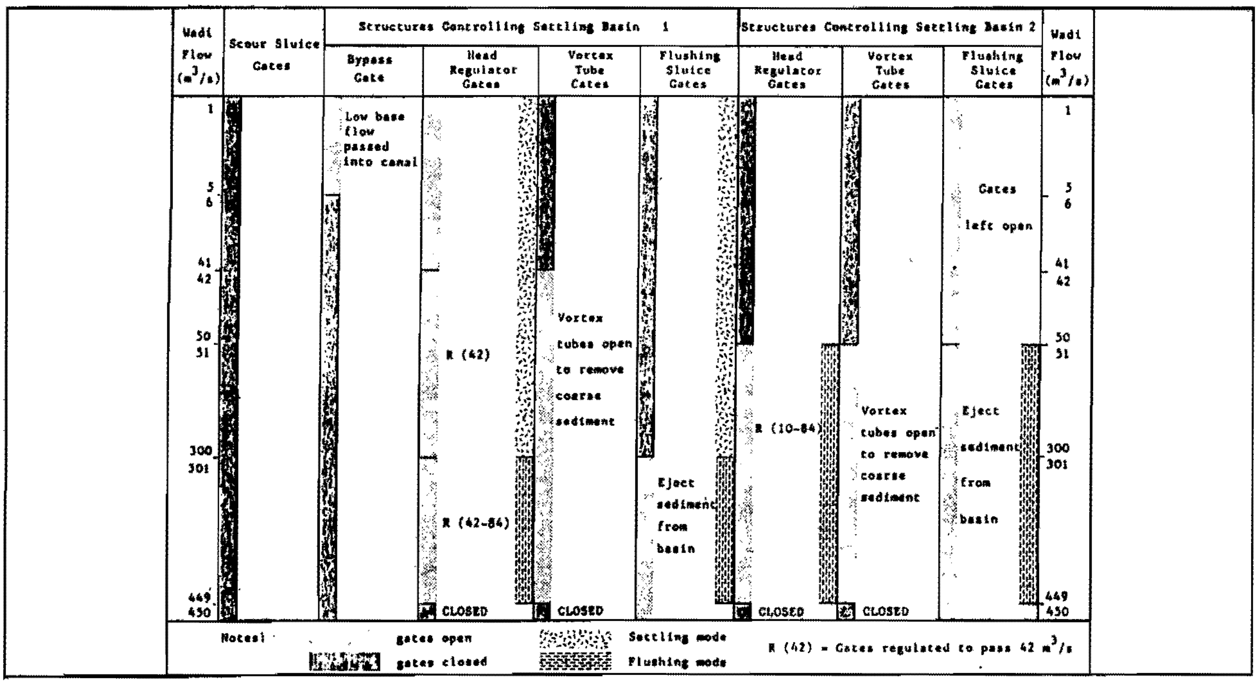
2.4 *Water distribution*

Two supply canals serve the irrigated area, one on each bank of the wadi. Each supply canal has offtakes to divert flow into the existing primary canal system which was traditionally served directly from the wadi. Experience has shown that provision of a secondary distribution system is best left as a follow-on operation once the primary system has 'settled-in'.

The need to ensure equitable distribution of such unpredictable flows within a relatively high sediment load dictates the design features of the supply canal system. Each canal serves an upper area which traditionally received perennial supplies and a lower area supplied only by spate flows. To ensure flexibility in the distribution of low flows to the perennial areas each head regulator was provided with low level gates of capacity equal to the estimated requirements of the area served. All flows in excess of the combined requirements of the north and south perennial areas over fixed weirs whose crest lengths are proportional to the respective areas served. Thus the division structure allows proportional splitting of low flows to the north and south canals, rotation of low flows between canals and the automatic division of spate flows.

The average natural ground slope parallel to the wadi is approximately 3 m/km, much greater than the maximum permissible canal slope. The longitudinal slopes of the supply canals have therefore been set at the maximum permissible slope. Head loss is accomplished at each cross

Figure 5 Guidelines for operation of headworks during a rising flood



regulator, located to provide control for the major canal offtakes. Residual head is lost by the inclusion of straight drop fall structures.

Each offtake has been fixed to discharge 2 l/s per hectare of area commanded. Typical sizes range from 0.5 m^3/s to 2.5 m^3/s. In all other respects the design is similar, regardless of location along the canal. Each incorporates a tilting gate, which is set so that when the gate is fully open the design discharge can be passed with the supply canal water level at cross regulator crest level, facilitating the distribution of low flows.

Water levels vary considerably in the supply canals, therefore some control is necessary. Twin orifices, downstream of the gate, effectively throttle the flows which would otherwise result. Thus the offtake allows design discharge to be passed over the full range of supply canal flows. In addition excess flows are able to pass over the top of the raised offtake gate, thus providing an automatiic escape facility. Immediately ahead of each offtake a scour sluice has been provided to assist clearance of the intake channel and each component of the offtake has been designed to facilitate the passage of trash.

3. Principles of headworks operation

3.1 Introduction

There are three main modes of operation of the headworks, namely:

- settling;
- flushing; and
- scouring.

For normal operation, low flows will be passed via the north head regulator, along the north settling basin and into the canal system. Very low flows are routed through a bypass gate which allows the settling basin water level to be retained at a lower level thus reducing seepage losses. Flushing is recommended at every opportunity, within the limitations described below, on rising and falling flood stages when the diverted discharge exceeds the canal requirements. Vortex tube operation accompanies both settling and flushing operations at discharges in excess of 40 m^3/s. Scouring is only recommended on a falling flood stage outside the operational limits of the settling basins. A schematic representation of the operation of the headworks during a rising flood is shown in figure 5.

3.2 Settling mode

The head regulators are used to control the flow of water to the basins in the settling mode. As described in Section 2 at wadi flows greater than 300 m^3/s on a rising flood, or 400 m^3/s on a falling flood, water should not be released into the canal system as excessive quantities of sediment are expected to pass through the settling basins. These limiting values should, however, be reviewed in the light of operational experience.

Only one settling basin is operated in the settling mode at any one time. The appropriate head regulator gates are adjusted to pass the design flow of 40 m^3/s plus 2 m^3/s for the operation of the vortex tubes, or the full wadi flow if less than this. During the settling mode the vortex tubes are operated only for flows in excess of 40 m^3/s, since at low flows it is unlikely that very coarse sediment will enter the headworks and conservation of irrigation water is important.

For flows up to about 5 m^3/s the bypass gate would be adjusted to regulate the water level in the north settling basin to minimize seepage losses.

3.3 Flushing mode

Flushing is recommended at every suitable opportunity since it is expected that favourable conditions for this operation will be limited.

The basin not being used for settling is considered to be in the flushing mode and its sluice gate is left open. The normal limits of operation have been set at a wadi flow of between 50 m^3/s and 450 m^3/s. The upper limit corresponds to the estimated wadi stage at which drowning of the outflow will occur but in practice the upper limit will be determined by experience. In the interests of water conser-

Table 2 Gate operating times

Gate location and number of gates	Distance opened or closed (m)	Time taken to operate all gates at each location (minutes)	
		By electric power	By hand (2 men)
Scour sluices 3 gates	4.8	8	96
Head regulator 4 gates	1.5	3	15
Vortex tubes 3 penstocks	0.6	4	12
Flushing sluices 3 gates	3.0	6	30
Bypass sluice 1 gate	1.5	3	8

Notes: (1) All gates can be operated simultaneously by electric power.
(2) Times given by hand are for 2 men operating the gates.

vation, flows up to the canal requirements should not be used for flushing purposes, particularly since the potential for scouring at low flows is limited and flushing times would be long. Therefore layout of the flushing sluices and the size of the tunnel barrels are such that earthmoving plant can drive into the basins.

To maximise the rate of scouring of sediment from one basin, whilst the other is in the settling mode, a wadi discharge of 126 m^3/s is required. Lower wadi flows decrease the scouring flows available. However, flushing operations should be carried out at all wadi stages within the operational limits of the basins to maximise the efficiency of the sediment collection and removal facilities and reduce accretion of sediment in the wadi downstream of the flushing sluice outfalls.

3.4 Scouring mode

The purpose of the scouring mode is to eject sediment from the intake forebay and to maintain a clear lead in channel in the wadi adjacent to the intake.

The gates would usually remain closed during a rising flood to prevent the coarser sediment entering the intake forebay. The normal limits of operation of the scour sluice are between 700 m^3/s and 450 m^3/s on a falling flood. The 700 m^3/s upper limit was chosen because it corresponds to an upstream flood level at which the scour sluice gates can be fully opened without causing the hydraulic jump to sweep out from the downstream cistern. The 450 m^3/s lower limit is the flow at which the settling basin flushing would commence on a falling flood. The scour sluice gates have not been designed to open at flows in excess of 700 m^3/s. However, flood flows can pass over the closed gates and will discharge floating debris that may otherwise collect in the intake forebay.

Constant observation and water level monitoring both upstream and downstream of the scour sluices is essential during operation since at all times the stability of the hydraulic jump is essential. For this reason remote control of the scour sluice gates has not been provided. Limiting gauge levels for scour operation with the gates fully open have been calculated to assist operation.

3.5 Electric operation

An important feature of the control equipment design is the incorporation of electric drive. This is an essential requirement for most major operations, particularly scouring, because of the time taken to operate the gates manually compared with the very rapid rise and fall of wadi spates. Table 2 below gives comparative manually and electrically operated times for the major control equipment.

Consideration of typical wadi hydrographs indicates that between 40 to 60 minutes available for the main scouring operation (for wadi flows falling from 700 m^3/s to 450 m^3/s. It is also quite possible for the wadi flow to increase from practically nothing to more than 450 m^3/s in 90 minutes. During this time one set of head regulator gates should be opened, regulated and then closed, the other set regulated and then closed, one set of flushing sluice gates opened and two sets of vortex tube penstocks opened and closed. Water level sensors have been incorporated at critical locations to provide data for remote control. As a back-up all of the gates can be operated manually in the event of a power failure.

3.6 Sediment sampling

The preliminary assessment of the operation of the headworks is based on the estimated sediment concentration in Wadi Mawr and the estimated sediment concentration that can be transported by the project canals. This is described in Section 2. However, because of the inexact knowledge of the sediiment behaviour, the predicted conditions may vary considerably from what may occur in practice. It is important therefore that, particularly in the early stages of operation, as much information as possible on sediment and headworks performance should be collected and analysed in order to confirm the operation procedures.

Sediment samples should be taken in the turbulent areas downstream of the major control structures using specially adapted bottles. The frequency and timing of sampling will be determined from practical experience but in the early stages of operation, samples should be taken during floods at as frequent intervals as practical. Rapid sampling will be required at certain locations during critical flood stages.

It is recommended that analysis is carried out by Imhoff cone and the concentration of particles greater than 0.063 mm in size plotted against time to give an instant indication of at what stage it will be necessary to change the mode of operation. Samples taken routinely can be analysed by normal laboratory methods, to determine the sediment particle size distribution and the results used for comparison with those of other floods to assess uniformity or trends in wadi sediment behaviour.

4. Development of canal system operating strategy

4.1 Introduction

The major consideration in the development of the operating strategy for the new system was the established system of water distribution; the aim being to guarantee more assured supplies to the project area thus enabling greater control over its distribution. However, since no storage has been introduced in the system, the supplies will be no more predictable than in the past. Thus, to achieve a major aim of the project, the equitable distribution of flows, a comprehensive water allocation plan was developed to define

Table 3 Duration of flows exceeding 10 m^3/s in average year

Winter period Month	Duration (days)	Early rains Month	Duration (days)	Late rains Month	Duration (days)
Nov	1.7	Mar	6.4	Jul	7.0
Dec	0.4	Apr	4.8	Aug	12.2
Jan	0.7	May	5.8	Sep	7.8
Feb	0.8	Jun	1.7	Oct	1.8

operational parameters for the new system.

The water allocation plan aims to provide an operating system which incorporates relatively simplified procedures while maintaining an efficient use of available flows. However because of the inherent flexibility of the system, variations can be easily incorporated. A comprehensive flow measurement system has been provided to allow continuous monitoring of the distribution of flows, for a comparison of actual performance against that predicted.

4.2 Water allocation principles

On the basis of the existing irrigation calendar and on monthly flow patterns, the year has been split into three seasons as shown below:

Winter period	November to February
Early rains period	March to June
Late rains period	July to October

Low baseflows predominate during the winter period. In accordance with existing practice, and to minimize canal seepage losses, this is allocated only to the upstream section of the project area, to those canals which traditionally receive winter baseflows (the 'perennial' canals). Water will only be passed further down the supply canals when diverted flow is surplus to the requirements of the upper canals.

During the early and late rains, when baseflows are enhanced and spate flows are frequent, the diverted flows will be allocated both to the upper canals and to those canals which traditionally receive spate flows only.

The continuous irrigation requirement at the primary canal offtake is 0.8 l/s per hectare. This figure was based on generalised cropping patterns and efficiencies and represents a target annual depth of 2.5 m for areas of all-year-round cropping. Proportional seasonal target depths of irrigation are applicable for areas of double and single cropping. These generalised field irrigation requirements have been adopted for the formulation of the water allocation plan. Further refinements of these estimates can be made to suit actual cropping patterns and intensities as operational experience is gained.

Hydrological analysis of the available data indicated that the maximum diverted flow of 40 m^3/s is likely to occur approximately 20 to 30 times in a year. Typical spates of this magnitude last for less than 6 hours but on rare occasions they continue for 24 to 36 hours. Under these conditions every offtake would be capable of discharging design flows. However flows significantly less than the maximum diversion capacity predominate.

Table 3 below summarizes the average duration each month of flows which exceed 10 m^3/s, the total requirement of the perennial canals.

It can be seen that during the winter period there is very little water available for distribution to lower canals, but in the early and late rainy periods residual flow is available some of the time. Over the year as a whole this flow is exceeded 14 percent of the time. A system of allocation between the individual offtakes is therefore necessary to distribute available flows, such that each offtake has a reasonable chance of obtaining its target allocation, regardless of location along the supply canal.

From an analysis of the hydrological data and an estimation of likely canal seepage and other losses, preliminary volumetric allocations were determined for each offtake to match the estimated average annual diverted flow. These were based on the irrigation requirements of the generalised cropping patterns assumed for each area, and were used for initial computer simulation studies.

4.3 Computer simulation studies

To test the effectiveness and reliability of the allocation principles previously described, computer simulation studies were carried out on historic flow data. A continuous flow record is best to make best use of a computer program to investigate water allocation. However, the record from Shat el Erge gauging station has much missing data. Mean daily flows were filled in by a stochastic modelling technique using the three parameter log normal distribution fitted on a monthly basis and showed excellent correspondence. Additionally, in order to develop a consistent data set only daily flow data, which is available for all periods of record, were distributed into a representative daily hydrograph. One season was chosen to calibrate a daily hydrograph model, and a relationship between maximum daily flow and mean daily flow determined for the normal range of discharge encountered. For input into the computer program a variable time step was introduced according to the mean daily flow value. The time step duration adopted varied from 12 hours with mean daily flows of less than 5 m^3/s to 1 hour with mean daily flows in excess of 30 m^3/s.

The program was written specifically for the scheme and distributed diverted wadi discharges between the various offtakes taking due account of seepage losses. A specific area of investigation was the operation of the division structure, which controls the flow into the two feeder canals. At the commencement of each irrigation cycle all offtake gates are assumed to be open to receive design discharge. At each time interval the instantaneous discharge passing into the system is distributed to open offtakes, uppermost offtakes receiving supplies before lower offtakes. Once the target allocation for the cycle has been received, the offtake gate is assumed to be closed. At the end of each cycle the actual volume of water received by each offtake is calculated and compared with the target allocation. Seasonal volumes are output for a determination of reliability.

To assess the individual effects of changes to the variables an initial series of runs was carried out to develop a basis for the comparison of options. One area of special study was the ability of the system, under various conditions, to effectively escape residual flows without threatening the integrity of the canal embankments. Over the ten years of analysis, average areas receiving target depths were computed on the basis of the recommended operating strategy. It was found that on average 81 percent of the estimated annual diverted flow of 185 million m^3 is utilised by the scheme canals, the remaining 19 percent being lost through seepage (16 percent) and escape over the tail regulators (some of which is utilised by the tail canals).

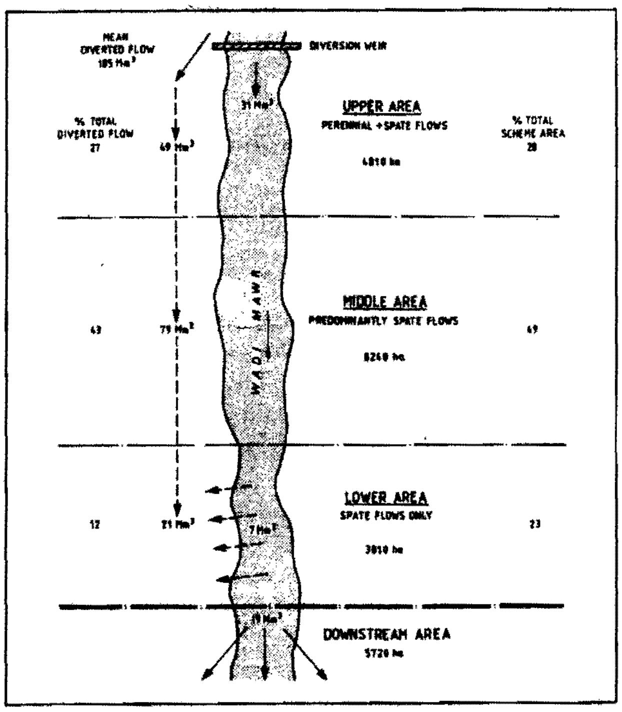
This results in potential cropping intensities of between 130 percent for upper canal commands to around 80 percent for lower canal commands. A schematic representation of the distribution of flows is shown in Figure 6.

4.4 Principles of operation

As a result of the comprehensive simulation studies, an operating strategy was developed which incorporated relatively simplified procedures whilst maintaining an efficient use of available flows. The simulation results demonstrate that all scheme offtakes, when operated within a defined range of parameters, will receive, in an average year, a proportionate share of available water in relation to their size and position within the system. In addition, it has been demonstrated that in drier than average years all scheme offtakes would receive a fair allocation of available water.

The operating strategy comprises the opening and closing of the offtake gates on a calendar basis with volumetric control being exercised over closure. Thus, in a wetter than average season, early gate closure is effected as soon as the target volume has been received to maintain an equitable distribution of flows. In drier than average seasons the flow to lower offtakes is preserved by upper offtake gate closure on the assigned date, before the target volume has been received. Each offtake has been assigned a window of opening which, for uniformity, remains the same for each irrigation cycle within a particular season. The length of the irrigation cycle for each primary area was optimised during computer studies and resulted in the adoption of a cycle frequency of twice monthly for upper perennial areas and monthly for lower spate areas. This gives frequent allocations to all offtakes thus providing flexibility to varying agricultural requirements. At the commencement

Figure 6 Schematic representation of distribution of available flow



of each cycle all offtake gates are set fully open to receive supplies. Each offtake is then closed when either:

(i) its target allocation has been received; or
(ii) the indicated closure date is arrived at.

This results in a progressive closure of gates from the upstream end of the system. Excess flows are automatically spilled over the top of the upstream closed gates which have been set at an escape position. This is particularly important at the end of each irrigation cycle when the majority of offtake gates are closed. Thus maximum use is made of available flows and tail losses are kept to a minimum.

The distribution of low flows during the winter period to the canals on either side of the wadi is governed by a priority system which changes with each irrigation cycle. Thus low flows are either diverted to the north or south bank of the wadi depending on priority. Flows in excess of the requirements of the perennial areas of the priority canal are diverted to the other canal.

4.5 Performance monitoring

The computer studies have shown that the above operating system ensures sufficient regular supplies to each scheme offtake in an average year to suit the overall agricultural requirements assumed. However, the importance of regular data collection for analysis cannot be over-emphasised, not only for the efficient day-to-day operation of the scheme, but also for an assessment of overall performance, in comparison with that predicted. It is anticipated that refinement of the operating strategy will be carried out during an initial period of review as operators become familiar with the operation of the water control equipment and the response of the system to variations in the operating parameters.

Comprehensive discharge measuring facilities have been incorporated into the scheme to allow system monitoring. These comprise gauge boards for use in conjunction with water control structures, autographic water level recorders at selected locations to provide continuous supply canal discharge data, and flow meters at every canal offtake.