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ENGINEERING MANUAL FOR SPATE IRRIGATION

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Thanks must also go to the unnamed people and consultants who, through their efforts to apply well-intentioned engineering improvements to traditional spate irrigation schemes, have unintentionally lengthened my list of actions to avoid.

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I must also mention the role of the late Robert F Camacho in stimulating my interest in irrigation and showing that the starting place for design is the application of common sense. Only when that route is exhausted does one get into the detailed calculations. It was very interesting to visit some of Bob Camacho's early spate structures in southern Yemen in order to see the context for development of his spate engineering approach.

Finally, some words of appreciation to the long-suffering spate irrigation farmers who, over many generations, have developed some impressive spate irrigation schemes and actually understand the process of spate irrigation better than the engineers. I have learned to listen to them and value their knowledge and suggestions.

Report submitted by
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ABBREVIATIONS AND ACRONYMS

DEM	Digital Elevation Model
EDF	European Development Fund
ELWDP	Eastern Lowlands Wadi Development Project
ETo	Reference crop evapotranspiration
EU	European Union
FAO	Food and Agriculture Organization of the United Nations
GIS	Geographical Information System
IFAD	International Fund for Agricultural Development
MAF	Mean Annual Flood
MCM	Million cubic metres
MoA	Ministry of Agriculture
O&M	Operation and Maintenance
SRTM	Shuttle Radar Topography Mission
TDA	Tihama Development Authority (in Yemen)
TRMM	Tropical Rainfall Measuring Mission

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EXECUTIVE SUMMARY

This manual is intended to both provide an introduction to the engineering of spate irrigation schemes and provide a practical guide for selection and designing of interventions.

Engineering of spate irrigation schemes, particularly attempts to improve existing schemes, is as much about process, with emphasis on understanding the existing situation, as it is about doing design calculations. Among the key features of spate irrigation that have to be addressed during any design work are (i) diversion of sufficient water during flood conditions; (ii) management of heavy sediment loads; (iii) accommodating the progressive rise of command levels; and (iv) simplicity of operation so that flood waters can reach the fields, usually at night.

This manual starts by providing background about spate irrigation and common problems, then discusses the key considerations of hydrology, agronomy, sediment and beneficiary involvement before moving to the detailed design of irrigation infrastructure. The performance of existing engineered interventions in spate irrigation is discussed at length because it is only through understanding of where, when and why things go wrong then similar problems be avoided in future.

While the content of this manual mainly relates to the improvement of existing spate irrigation schemes, the possibility of creating new spate irrigation schemes is also discussed. This is a particularly challenging area because of the even greater uncertainty about the external influences that may impact on the development.

One engineering challenge for spate irrigation is to provide improvements that are compatible with the modest economic benefits that usually prevail in spate irrigation systems. In some situations economics are over-ridden by poverty alleviation or other objectives which would increase the acceptable level of expenditure.

However, this is still likely to leave a designer with the challenge of creating affordable robust solutions that perform and survive under the flood conditions that prevail in spate irrigation schemes, although value engineering may make it appropriate to reduce the factors of safety inherent in the normal design process. Alternatively, it may be acceptable to provide structures that have a significant risk of failure. A low-cost structure that needs repair after 10 years may provide better value for money than a structure with twice the cost that survives for 30 years.

Cost-effective engineering is more likely to be achieved through development of proposals in close coordination with the beneficiaries (the farmers), particularly if they are required to make a contribution towards the costs. Engineers must not underestimate or ignore the knowledge and experience of farmers in existing spate systems who should be encouraged to propose what improvements should be made to the existing infrastructure. In addition to such proposals building on the farmers' experience, implementation of the proposals, if technically sound (which is the engineers' task) and financially viable, will leave the farmers with a stronger sense of ownership and responsibility.

1. INTRODUCTION

1.1. Objective of this manual

The objective of this manual is to both provide an introduction to the engineering of spate irrigation schemes and provide a practical guide for designing of interventions and, in particular, to try to avoid the problems already found, or created, by others.

Engineering of spate irrigation schemes is as much an art and not a science. Conventional irrigation and hydraulics books and design manuals fail to give specific guidance about the unusual requirements of spate irrigation and application of the standard design methodologies to spate irrigation can frequently result in unsatisfactory designs.

Much has been written about spate irrigation during the past 10 years, most notably the FAO Irrigation and Drainage Paper. 65: Guidelines on Spate Irrigation (2010)[1] and its predecessor document, the DFID-funded Improving Community Spate Irrigation (2005)[2]. However, while both of these documents discuss design issues, neither contain sufficient detail to provide guidance for making judicious decisions about spate irrigation design. This manual attempts to fill that gap. A further valuable reference is the proceedings of the 1987 conference on spate irrigation[3].

One of the challenges for the engineers is to reduce the expectations of the farmers who start off with the assumption that engineers can solve their problems whereas, in reality, the realistic objective can only be to reduce their problems. To this end, the dialogue needs to start by asking the farmers to list their problems in order of priority instead of making an engineering diagnosis about what needs to be done.

Whilst farmers would like the engineers to provide the infrastructure to (i) enable all the flow to be diverted to the fields; (ii) allow only the fertile silt to be carried by the water with the rest of the sediment diverted elsewhere; (iii) reduce the severity of the flood peaks; and (iv) provide a system which requires minimal operation and maintenance, in reality these objectives are difficult to achieve at an affordable cost. There is, however, one intervention which could go a long way to fulfilling the above objectives: It is called a spate breaker dam. This provides temporary storage of flood flows for a few hours only thereby attenuating the peak flows and causing deposition of the coarse sediment. Unfortunately, the process of sediment deposition will progressively reduce the capacity for temporary storage and the structure may become ineffective within a few years. Equally unfortunately, although the concept has been around for many years, there is no documentation relating to the implementation and performance of these structures.

1.2. Scope of this Manual

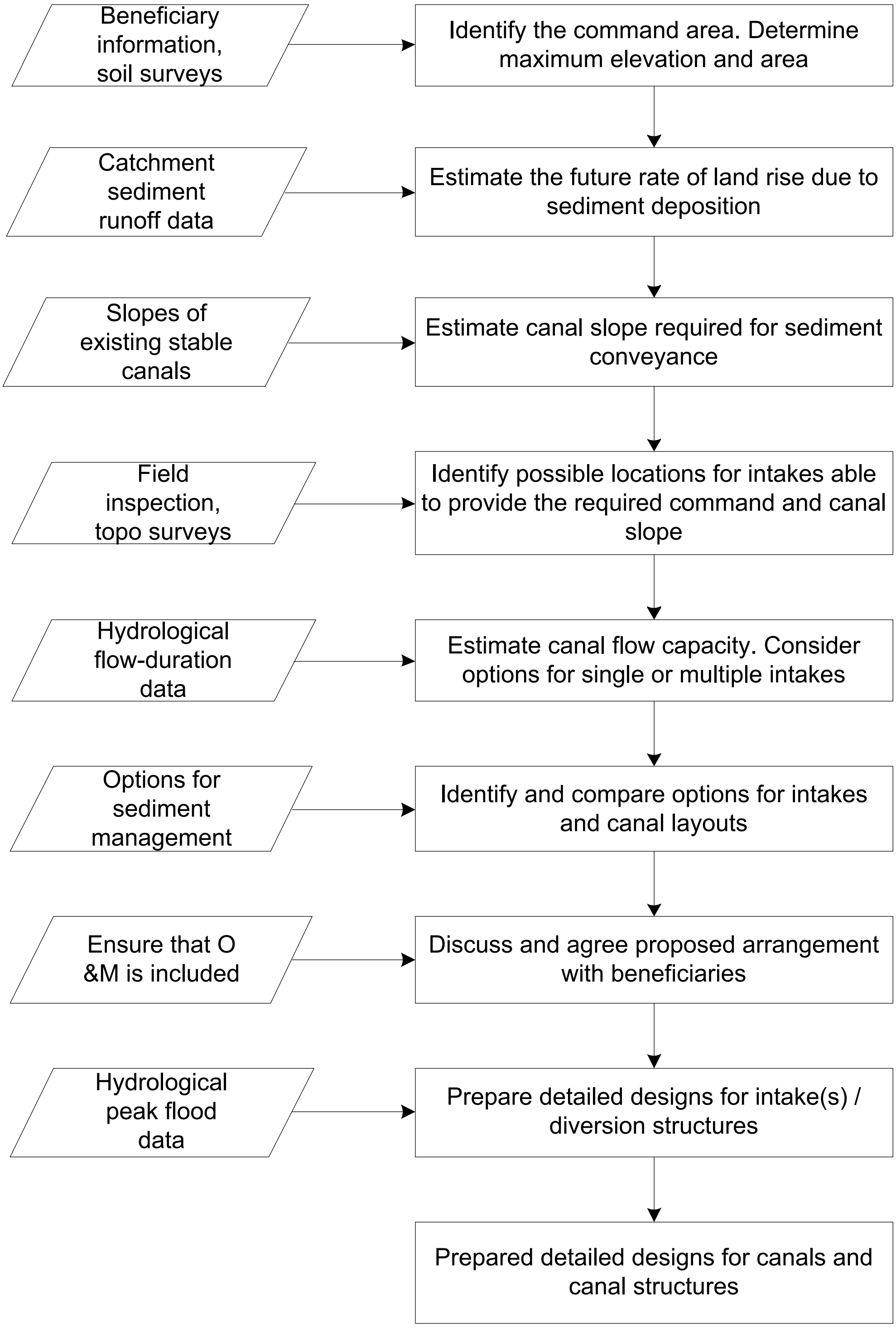
The first part of this manual (sections 1 to 4) discusses the background to spate irrigation and describes, with examples, the many problems that can be encountered and provides advice on the collection or estimation of background information. The second part of the manual (sections 5 to 12) provides guidance for design procedures.

This manual focuses on the specific requirements of spate irrigation. Some engineers may consider this manual deficient because it is relatively devoid of complex formulae. This is deliberate: One can easily be blinded by theory and overlook the realities (in computer terminology: "Garbage in, garbage out") or to quote the economist John Maynard Keynes "It is better to be approximately right than precisely wrong". Hence the emphasis on process to maximise the understanding of the situation on the ground and appreciate the consequences, for better or worse, that any engineered intervention may cause.

However, once the concepts, potential problems and priorities have been identified then the design process can use the appropriate engineering formulae and criteria to develop the designs of whatever has been proposed for implementation. There is still a need to input data into formulae and this manual either provides suggested values or recommends procedures to determine the input data. The principal steps in the design process are shown in Figure 1.

[1] *Guidelines on spate irrigation. FAO irrigation and drainage paper 65. F van Steenbergen et. al, 2010*
[2] *Improving community spate irrigation. HR Wallingford report No. OD154. F van Steenbergen et. al, 2005*
[3] *Spate Irrigation. UNDP / FAO Proceedings of the Subregional Expert Consultation on Wadi Development for Agriculture in the Natural Yemen, Aden, 1987.*

Figure 1-1: Flow chart for principal design steps



It can be seen that understanding the situation, data collection and analysis form a substantial part of the process and that the full engineering design of the proposed works is only a final step.

2. LESSONS LEARNED IN SPATE IRRIGATION

2.1. Background to Spate Irrigation

In simple terms, spate irrigation is the diversion of flood flows in ephemeral rivers for irrigation. In reality, most irrigation schemes include some element of lower flows which may only be the recession of each flood event but can also be a base flow that may run continuously during the flood season. As such, there is no clear boundary between spate irrigation and a seasonal irrigation system where water is available continuously for several months of the year. Some irrigation schemes are supplied by snow melt water but are also subject to floods when rain accelerates the snow melt process. If the farmers want to capture the total flow, which is often the case, then designers for those schemes should also take spate irrigation engineering into account.

For the purpose of classification, an appropriate definition of spate irrigation could be to include those schemes where flood flows (complete with the associated high sediment loads) **need to be utilised** for irrigation and the capacity of intakes and canals is based on flows much larger than any base flow.

The boundary between spate irrigation and water harvesting is also poorly defined. They are both forms of capturing flood water in order to create sufficient depth of water application to support a crop. Perhaps a suitable boundary between the two would be where the flow being captured or diverted is more (spate) or less (water harvesting) than $1m^3/s$.

Spate irrigation is practised in many arid or semi-arid countries where floods arising from heavy rainfall represent much of the annual water resource. It is economically very important in countries such as Yemen, Pakistan, Eritrea and Ethiopia where agriculture is an important component of the economy. However, care is needed when transferring ideas that work in one country or region to another. For example, topography, rainfall, flood patterns and soils are different between eastern and western Eritrea.

Spate irrigation has been practised for hundreds, if not thousands, of years[4]. Of greater relevance to the engineer is that some of the existing spate irrigation schemes have their roots going back 100 years or more. The current Wadi Zabid system in Yemen can be traced back several hundred years.

2.2. Modernisation and Improvement

In the 1970's to 1990's there was a spate of well-intentioned modernisation of existing traditional spate irrigation schemes, particularly in Yemen. A key component of the modernisation was the provision of engineered diversion structures, some with elaborate sediment exclusion facilities. However, many of the investments have failed to create the expected benefits and some have created new problems. These are discussed further in Section 2.3 below.

2.3. Lessons Learned

2.3.1. Potential Pitfalls in Spate Irrigation Development

Examination of well-intentioned investments in spate irrigation during the past 50 years has revealed a number of reasons for poor performance that had not been anticipated. These include:

- Failure to achieve the expected increase in irrigated area due to over-optimistic assumptions about water resource availability or incorrect characterisation of the flood flows
- Increased inequity of water distribution resulting from the construction of stronger diversion structures which give the benefiting farmers a greater share of the available flow to the detriment of others
- Failure to appreciate the problems associated with high sediment loads resulting in blocked canals and, in the longer term, inability to command raised fields
- Use of in appropriate design parameters and formulae (such as those for clear water irrigation)
- Unrealistic assumptions about operation and maintenance in particular the ability of government departments to fund, manage and maintain the irrigation systems
- Changes in irrigation practice, such as the development of groundwater

2.3.2. The Learning Process

Unfortunately, the feedback loop for the performance of the modernised spate systems followed long after the modernisation process and has only really come to prominence when the nominally 30 year life cycle of the modernisation investments approaches its end and a further round of investments can be justified.

[4] *See page 3 of FAO Irrigation and Drainage Paper 65*

As such, the Irrigation Improvement Project in Yemen, which commenced in 2002, was very informative in understanding the performance of the previous round of investments. That project prescribed a participatory approach to the design of improvements and the scope of the works prioritised by the farmers was somewhat different to the list of works (primarily deferred maintenance) prepared during project preparation (see Annex A for a comparison of the planned and actual work).

The growth of the internet has also facilitated the sharing of knowledge about the performance of spate irrigation schemes. It is human nature that failures do not achieve the same coverage as successes but, so far, there has been a "no blame" culture attached to spate engineering problems because it is an inexact science. Nonetheless, with improved knowledge dissemination there is little excuse for repeating old mistakes.

2.3.3. Lesson 1: Sediment and Trash

Sediment is both a key resource (fertile silt from the catchment) and a key problem (boulders, gravel and sand). Traditional diversion works usually breached in the larger floods which also enabled the worst of the sediment to be flushed downstream instead of entering the canal system.

Box 2-1 : Wadi Laba Sediment Management

The problem of sediment was appreciated during the design of improved diversion works for Wadi Laba in Eritrea and skimming weir (curved channel sediment excluder) was provided at the intake. However, it is only effective if sufficient flow is allowed to pass through the sluiceway to keep the area in front of the weir flushed.

A gravel trap was also provided on the head reach of the main canal to catch coarse material entering the intake. The gravel trap was successful at this task. However, project budgets could not afford a concrete lined flushable basin so reliance had to be made on equipment to clean the gravel trap. In reality this proved to be unsatisfactory because the basin could be substantially filled by a single large flood and equipment could not enter the basin to remove the material until it had dried out.

The normal design criteria for design of canals indicate velocities set to avoid erosion with clean water. However, spate water has a sediment load, which may be substantial, and the velocity needs to be sufficient to keep that sediment moving. Regime theory formulae can be used to predict velocities to give an overall balance between sedimentation and scour. One challenge for designers is to make an assumption about the effectiveness of any sediment exclusion arrangements such that the chosen canal can convey the sediment remaining in the flow.

Traditional spate canals are usually relatively wide and shallow. Deposition of sediment in the bed has a limited impact on the flow capacity and can be easily compensated for by raising the banks. Engineered canals are usually much narrower and deposition of the same volume of sediment results in a substantial reduction in flow capacity. Wide, shallow canals are also hydraulically inefficient which means that they can naturally dissipate sufficient energy to main suitable velocities on relatively steep slopes without any drop structures.

Floating trash can be a major problem during floods. An unfortunate fact of physics is that any intake, particularly if located on the outside of a bend and designed to attract the cleaner surface flow, will also attract the trash. There are two basic options for trash management: (i) keep it away from the intake using a suitable trash screen large enough to pass sufficient water even when substantially blocked and (ii) make openings large enough to let trash pass.

Box 2-2: Small Gates Block Easily

This irrigation intake show the hazard of using small gates: They easily block with trash. The sluiceway gate openings in this photo are completely blocked so the sediment cannot be flushed through and enters the canal (on the left of the photo - with gates also partly blocked by trash). The raised headwall on the sluiceway is to protect the gate operating mechanisms (on the downstream side of the wall) but also reduces the flood flow capacity.

Box 2-3: Canal Slopes at Wadi Zabid

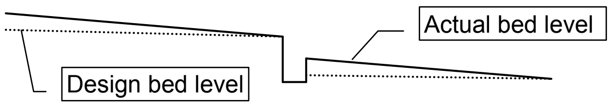
Bed Slopes of Old Canals, Wadi Zabid

Canal	Maximum Capacity (m'/s)	Average bed (m/km)
Mansury	40	3.8
Rayyan	60	3.7
Bagr	40	3.7
Gerhazi	50	3.9
Mawi	60	4.8

The Wadi Zabid canals were remodelled with a bed width/ water depth ratio of 6:1 to 8:1 and bed slopes ranging from 0.0003 to 0.0001. .... Heavy deposition took place in these canals after the first year of operation. Regular excavation of the head reaches is required to keep the intakes functioning and sediment has to be removed from the first 2 to 3 kms of the canals almost annually. The canals in their middle and lower reaches are modifying their cross-sections and attaining some measure of equilibrium with bed slopes of 1 to 2 metres/km. All the indications are that much steeper bed slopes are necessary.

(From Camacho, page 69 of Proceedings of 1987 conference on spate irrigation)

This photograph shows one of the Wadi Zabid drop structures. The bed level downstream of the structure has risen in order to increase the bed slope and sediment transport.



2.3.4. Lesson 2: Flow Capacity of Intakes

The cost of engineered intakes and canals increases with flow capacity. There is, therefore, a desire to reduce the capacity and related costs. A theoretical justification for this is to use a flow-duration curve which shows the proportion of the flow volume below a flow value. This approach has been found to have two drawbacks: (i) the hydrological analysis on which the floods are based may not be accurate and (ii) changes in the catchment may reduce the amount of low flows (eg diverted for small-scale irrigation) or increased the amount of flood-runoff (eg catchment degradation). The farmers tend to look upon any flow passing the intake and going downstream as a lost resource (this is less of an issue where there are multiple intakes). To enlarge the capacity of an existing intake is expensive.

Box 2-4: Additional Intakes at Wadi Zabid

When the irrigation system in Wadi Zabid was modernised in the late 1970's the large number of traditional canals taking flow from the wadi were grouped so that the combined canals could be supplied from five diversion weirs. The modernised intakes were of lower capacity than the sum of the traditional intakes that they replaced and the layout inherently gave the upstream users along a canal ability to take more than their share of water. One of the farmers' requests during the Irrigation Improvement Project was for reactivation of some of the traditional intakes to improve the access to water for downstream farmers within some irrigation blocks.

2.3.5. Lesson 3: Rising Field Levels

Sediment entering spate irrigation schemes has to go somewhere. Coarser sediment is usually deposited in canals and finer sediment reaches the fields. Often this fine sediment is someone else's fertile soil and helps to produce a good crop. However, the annual deposition of sediment raises the required land level to be commanded by the irrigation system. This problem tends to affect the upstream fields most because they usually receive a disproportionate share of the water and sediment load. However, as those fields rise, the farmers usually block the canal to try to get enough water onto their land. This encourages sediment deposition in the canal, can reduce the flow in the intake and reduces the amount of water reaching other farmers.

Failure to allow for rising field levels in the design of engineering works will result in the progressive deterioration in system operation. However, making provision in the design for rising levels adds to the costs. In traditional systems the farmer's solution to rising levels is to move the intake further upstream in order to maintain adequate command.

Box 2-5: Rising Canal Level in Wadi Zabid

These photos from Wadi Zabid illustrate the problem of rising command levels. When the canal culvert was constructed in about 1980, the bottom of the culvert was at canal bed level. By 2002 (left photo) the canal bed was within 0.5m of the top of the culvert when was creating a major obstruction to flow. The headwalls were therefore removed. By 2008 (right photo) the canal bed had risen further.

2.3.6. Lesson 4: Existing Water Rights

Existing traditional irrigation schemes usually have an established system of water rights established by the community to provide reasonably equitable access to water within the constraints of variability of supply and the need to give irrigated land enough water to support a crop. Downstream farmers may agree to the upstream farmers having first right to the water in the knowledge that the larger floods will overwhelm the upstream diversion arrangements and pass downstream. Providing new structures will often disturb this natural balance of water distribution and enable the upstream farmers to take a greater share of the water.

Therefore, if any improved structures are planned that will change the farmers' traditional ability to divert water, appropriate changes to the established water rights must be discussed and agreed between the benefiting farmers and those downstream.

In a few irrigation schemes, such as Wadi Tuban in Yemen, the water allocation schedule is revised annually by the local administration after taking account of irrigation in the previous year in order to give all canals a fairer share of the water.

Box 2-6: Wadi Zabid Water Rights

Wadi Zabid in Yemen has a date-based allocation of water between the upstream (29 March–2 August), middle (3 August–13 September) and downstream (14 September–18 October) users. This allocation took account of the larger floods breaching the upstream diversion arrangements and passing downstream. When the Wadi Zabid irrigation system was being modernised in the 1970s it was realised that the provision of permanent diversion structures would enhance the upstream users' ability to divert water and modification of the time periods to reduce the upstream users' time period was discussed. However, the diversion structures were built before a new water allocation was signed and the upstream users then refused to agree to any change. They have benefited substantially from the engineering works while the downstream users receive less water than before.

2.3.7. Lesson 5: Avoid Increasing Beneficiary Dependency

Modernisation of traditional irrigation schemes has usually created greater beneficiary dependency on government. Part of this is institutional, with government having ownership of the new structures, and part is a consequence of the modernised system requiring more sophisticated operation and maintenance, for which the farmers lack capacity. Often, the modernised system requires extra maintenance due to factors, such as sedimentation, overlooked at the design stage. These aspects, combined with the lack of an effective system for raising funds for the O&M costs, result in poor operation.

A better strategy is to consider O&M at the start of the design stage and, for existing traditional schemes, ensure that ownership remains with, or is transferred to, the beneficiaries. Simplicity of operation and maintenance must also be a primary design objective.

Box 2-7: The Tihama Development Authority

The Tihama Development Authority (TDA) was created to manage the modernisation of the traditional irrigation schemes in the Yemen Tihama (the area near the Red Sea) and undertake the operation and maintenance of the improved systems. However, no system was created to raise revenue from the beneficiaries and funding from central government covered little more than the salaries of staff. Farmer's became increasingly dissatisfied about the level of service provided by government. One of the objectives of the Irrigation Improvement Project was to reduce the role of TDA and give greater responsibility for operation and maintenance back to the beneficiaries. Considerable effort was put into the establishment and training of Water User Associations to improve the beneficiaries' ability to undertake this work. A similar situation exists in many irrigation schemes worldwide.

2.3.8. Lesson 6: The Benefit of Participatory Design

The farmers have hands-on experience of the problems with operation and maintenance of their systems. Engineers should make maximum use of their knowledge when formulating proposals for improvements. Often, the farmers' priorities for problems to be addressed differ from the engineers' interpretation of the situation. Participatory design is the process of jointly identifying and agreeing the proposed interventions. One important outcome of this process is that the farmers retain (or receive) ownership of the improved system and, because they have been involved in planning the improvements, have a greater moral obligation to achieving successful operation.

Box 2-8: Participatory Design in Yemen

The Irrigation Improvement Project in Yemen mandated a participatory approach to design of the improvement works. This required the farmers to prioritise their problems, design engineers to understand the farmers' problems, discuss and agree possible solutions, identify and agree how the farmers could contribute a nominal 10% to the cost and get written agreement by the farmers representatives prior to the implementation of any works. Pending the formal establishment of the Water User Associations, each major group of beneficiaries was asked to appoint a Farmers Design Committee as representatives to work with the engineers. This arrangement worked satisfactorily. In addition, the beneficiary contribution was often provided through minor works contracts awarded to the WUAs at rates below contractor prices. These contracts provided a valuable opportunity for capacity building of the WUAs to manage and undertake minor construction works and build their skills for future operation and maintenance.

2.3.9. Lesson 7: Vulnerability to Hydrological Assumptions

Spate irrigation involves managing floods which are one of the less certain aspects of hydrology. Erroneous assumptions about flood magnitude affect the performance of structures designed to manage the floods while erroneous assumptions about flood frequency and duration affect the sizing of the hydraulic structures and the probability of irrigating the command area.

Hydrological assumptions may also fail to recognise longer term trends in flood water availability due to either climate change or land use changes. Often there was a period of diligent collection of hydrological data at some time in the past and this information is still used many years later because of the lack of sufficient recent data. The availability of satellite imagery covering over 30 years can help in identification of changes in land use in the catchment.

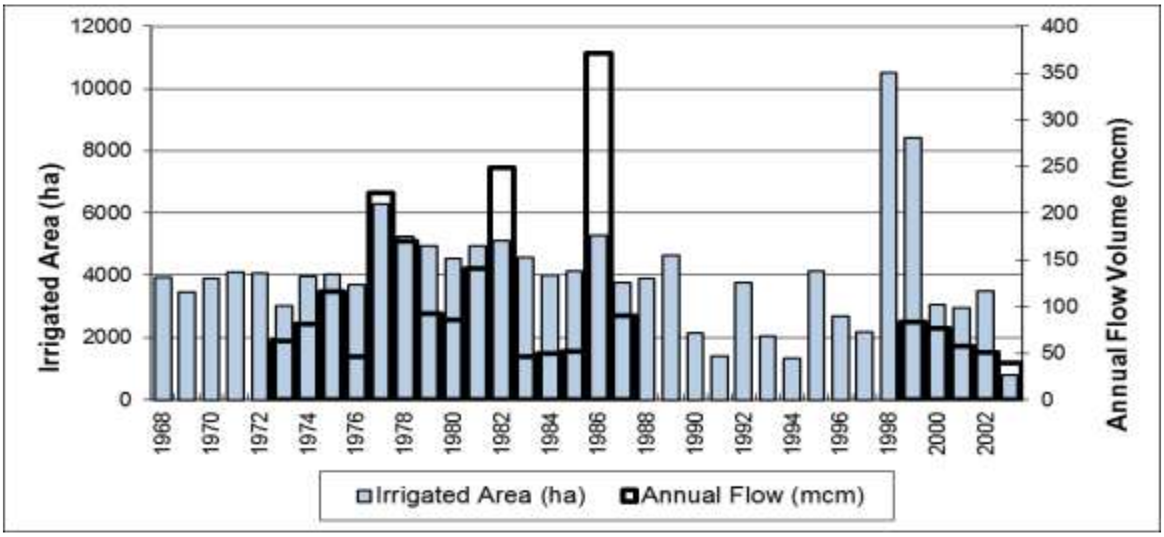
Box 2-9: Wadi Mai Ule Hydrology

Wadi Mai Ule in Eritrea is a relatively small catchment next to Wadi Laba in the Eritrean eastern lowlands. A hydrological analysis was undertaken to try to correlate flood events for Wadi Laba with rainfall in the area of the catchment. This suggested that 3 day rainfall was a reasonable indicator of flood events for Wadi Laba. While it was appreciated that Wadi Mai Ule's catchment was smaller and would have different rainfall characteristics, it was assumed that spate recession characteristics would be similar. The reality, discovered after the diversion works were constructed, was that the flood characteristics were somewhat different from Wadi Laba, with the flow at Mai Ule tending to be in short duration events with high flood peaks. Consequently, the intake, designed for 20m^3/s and assumed to be able to divert 80% of the total flow, could only divert a much lower proportion. The farmers divert water from downstream of the weir (itself undersized) to their fields in order to capture sufficient flow.

Had there been local sub-daily rainfall data which demonstrated that much of the rain was provided by local storms then the methodology for estimating floods generated off the local steep could have been different.

Box 2-10 : Example of Changing Flows and Areas

Flow measurement in Wadi Tuban was carried out between 1973 and 1987 and then resumed in 1999. There is a poor correlation between cropped area and annual flow volume, perhaps attributable to increased used of groundwater but the data confirm the farmers' perception that the volume of floods has reduced.



However, there is the perception of reduced floods which are unable to support the extent of irrigation that occurred in the 1970's and '80's. Those floods that do occur tend to be of shorter duration. The reduction in flow is attributed to climate change but one possible cause is the substantially increased cultivation of qat in the upland catchment which is intercepting more rainfall at source and depleting groundwater in the upland valleys. Both these factors contribute towards the reduction in the amount of flood runoff. The floods that occur are mainly from the catchment nearer the irrigated area.

Why hasn't the reported irrigated area dropped in the same way? Because the farmers have made increasing use of ground water. However, this is not sustainable because water levels are dropping and saline intrusion is developing.

2.3.10. Lesson 8: The Benefit of Multiple Intakes

For the engineers, the most attractive diversion structure is a large weir at the head of the system. It is usually, on paper, the least cost satisfactory solution. While this arrangement can usually provide a robust and reliable means of diverting water there are a number of disadvantages including:

(i) Water used for sluicing and flushing is lost to the system
(ii) The upper part of the canal system must be large to provide the required capacity
(iii) The upstream farmers have the opportunity to take control over the water supply

Traditional systems usually contain multiple intakes so that the flood flows that pass the upstream intakes can be captured by the intakes further downstream and in normal years the overall diversion efficiency can be 100%.Having multiple intakes also results in self-contained smaller systems which are easier to manage. While the intakes further down the wadi / river usually get less access to floods, the coarse sediment loads have also reduced so the maintenance burden is less.

Box 2-11: Reactivation of Old Intakes in Yemen

Both Wadi Zabid and Wadi Tuban in Yemen contain multiple weirs and canal intakes. However, among the requests from farmers for works during the Irrigation Improvement Project was for improvement of traditional intakes to increase the opportunity to capture flood water when there were floods. The modernised intakes had, in the farmers' view, insufficient capacity to supply all their command areas. This represents the conflict between the engineers who want to keep the capacities and costs down (and also reduce the ingress of sediment during large floods) with the farmers who see the flood peaks as a resource. Multiple intakes represent one way to harvest the water without receiving all the sediment, some of which will be deposited in the wadi before reaching the downstream intakes.

2.3.11. Lesson 9: Abrasion and Impact Damage

Sand in high velocity water is abrasive. Gravel and cobbles can cause substantial impact damage. Structures, particularly in the upstream part of a system where the velocities and bed loads are highest, are very vulnerable to damage from the larger sediment in the flow. Reinforced concrete is particularly at risk because, once the covering layer of concrete has been eroded, the structural reinforcement can be damaged. Protective measures include durable stone (basalt is good) cladding and steel angle protection on exposed corners of concrete such as chute blocks.

Figure 2-1: Eroded Chute Blocks and Base Slab

2.3.12. Lesson 10: Breaching Bunds

One strategy for reducing the size and cost of weirs is to design the weir for floods up to a 1 in 10 year probability and provide a "breaching bund" or earthen fuse plug as an extension of the weir which will overtop and wash away when flows exceed the design magnitude. Should the hydrological analysis under-estimate the flood magnitudes and frequency then the breaching bund will breach more frequently resulting in reduced diversion of flows and unplanned operation and maintenance problems resulting in crop losses and extra cost. These structures are best provided where the hydrology is better understood.

Box 2-12: The Wadi Laba Breaching Bund

The diversion weir for Wadi Laba in Eritrea was provided with a breaching bund designed to breach in a 1 in 5 year or larger flood event. However, the embankment was washed away twice in each of the first 3 years of operation causing considerable reduction in the volume of water diverted to the canal system. (It has now been replaced by a weir). But what might be the causes of breaching being much more frequent than designed? The most likely cause is the hydrological assessment failing to quantify the flood peaks arising from intense local rainfall on the nearby catchment. Although such floods are hydrologically insignificant in terms of their volume, the flood peaks may be sufficient to overtop the breaching bund. Another possible factor is that water approaching the breaching bund will decelerate and regain some of the velocity head (up to $v^2/2g$) so the water level adjacent the breaching bund will be higher than the adjacent water flowing over the weir.

2.3.13. Lesson 11: Sophisticated Engineering can Work

In case the foregoing comments are construed as meaning that sophisticated engineering can't work in the context of spate irrigation, there are examples of successful interventions such as flushing sediment basins. However, such interventions are expensive.

Box 2-13: Wadi Mawr - Sophisticated Engineering

Wadi Mawr scheme in Yemen uses a single diversion weir with a head regulator and sluiceway. The head regulator feeds two separate flushable concrete lined sediment basins one of which can be flushed while the other is supplying the main canal. All the gates are electrically operated.

The diversion structure is sufficiently far from the mountains that the main sediment load is sand and silt, which is flushable. However, the operational problem is that the farmers do not like to see water being "wasted" for flushing.

2.3.14. Lesson 12: Avoid Over-complex Operation

Spates most frequently occur at night - a natural consequence of afternoon or evening rainstorms. Farmers may, or may not, be aware of the spate depending on proximity of the rainfall and whether there is any warning / communication system. Diversion structures are sometimes manned during the flood season. The size and duration of a flood is generally unknown until it arrives (although it may be possible to get information from someone upstream) and the key operational objective is to get the water onto the land with the minimum of effort.

Box 2-14: Gated Canal Division Structure

Here is a canal division structure with 8 gates: Try to operate this in the night! Could two fixed weirs have been suitable?

Gates will cause problems if they are not operated to suit the flow conditions. They also take time to operate. Therefore, fewer gates enable easier overall operation. Fewer gates is achieved by (i) providing bigger gates at those locations where they are essential and (ii) not providing gates where they are not essential. Earth bunds, for example, are an effective alternative to gates in many situations and have the advantage that they can be set to over-top and wash away during larger floods.

3. BENEFICIARY CONSULTATION AND INVOLVEMENT

3.1. The Benefits of Beneficiary Involvement

Often the participatory design process is undertaken concurrently with the establishment of a beneficiary organisation such as a Water Users' Association which will manage the operation and maintenance of the improved system. Experience has demonstrated that close involvement of the beneficiaries in the design process can help achieve several key objectives including:

- Treating the beneficiaries as partners in the development process
- Building on the beneficiaries' experience in working with spate irrigation
- Ensuring that the proposed works are compatible with the existing water rights or, if they are not compatible, then revisions to the water rights are agreed between the beneficiary groups before construction commences
- Utilising, as far as practicable, the beneficiaries' proposals for development so that they have psychological ownership of the proposed development
- Ensuring that the beneficiaries understand what development is proposed together with the implications for operation and maintenance (O&M)
- Reducing the risk that the development will increase the risk of dependency on government
- Ensuring that the beneficiaries appreciate that engineers cannot completely control the floods but the planned works will reduce the burden on the farmers.

Development of the plans for improvements to spate irrigation systems should take account of the following overall objectives:

- Distribute water in line with accepted rules and rights while at the same time providing flexibility
- Find an optimum balance between different uses (agriculture, drinking water, etc)
- Where appropriate, be an activity supporting transfer of responsibility for system management, operation and maintenance to the beneficiaries
- Manage sedimentation
- Manage possible changes to the river system
- Be easy and affordable to operate and maintain
- At a cost compatible with the benefits

Conversely, a failure to involve the beneficiaries in the design process is often the main cause of project failure. To achieve the above objectives the beneficiary consultation needs to be far more than the occasional meeting to brief them on what the design team is doing but requires a process.

The time and effort to undertake this process thoroughly should not be underestimated but the benefit is an increased probability of a successful project. One related aspect to be agreed before the start of any participatory design process is whether any beneficiary contribution to the costs is required. Experience indicates that where the beneficiaries have to contribute towards the cost of development then they have an increased sense of ownership. The requirement also increases the work that can be undertaken with the available funding. The beneficiary contribution is typically 5% or 10% of the construction cost. Often the beneficiaries cannot afford to contribute in cash so they contribute labour or undertake simple works contracts at below contractor prices. This latter mechanism also allows capacity building of a beneficiary organisation in preparation for future maintenance work.

3.2. The Participatory Design Process

The main steps of the participatory design process (shown as a flowchart on Figure 3-1) are:

- Identification of the beneficiaries including traditional institutions such as hydraulic units or villages
- Introduction to the project and explanation of its objectives
- Identification of problems
- Ranking of problems
- Examination of the more highly ranked problems both in the field and by desk study (how many problems can be addressed will depend on both the costs of solutions and the
- Identification of possible solutions to the highly ranked problems and agreement on the preferred solutions (possibly more than one if the final decision will depend on cost)

- Outline design and cost estimates for the preferred solutions
- Identification and resolution of any potential impacts of the proposed works on the existing water rights;
- Agreement on scope and responsibilities for future operation and maintenance
- Discussion of the outline designs and either agreement to proceed with detailed design or, if too expensive, proceed with a more affordable solution
- Preparation of final designs and cost estimates
- Final agreement by the beneficiaries to the proposed works prior to implementation

Figure 3-1: Flowchart for Participatory Design

			Location
<--- Focus on problem identification --->	1	**Beneficiary groups list of irrigation problems** - Focus on irrigation problems and their causes - Indicate how many farmers are affected	Each village in the project area
	2	**Problem ranking by meeting of groups** - Identify the irrigation problems, their cause and how many farmers/ area are affected - Rank the irrigation problems - Group representativess must sign the ranked problem list - Make sure farmers understand that funds are limited - Beneficiaries appoint representatives as Farmer Design Committee (FDC) to work with project design staff	Suitable meeting place in or close to the project area
	3	**Walk-thru survey** Joint field inspection of the most serious problem areas by the FDC and the design staff to discuss and understand the problems	In the project area
<--- Focus on solutions to problems --->	4	**Evaluation of possible solutions** - Design staff assess alternative solutions to the irrigation problems, the O&M implications, economic feasibility and potential benefits - Estimate (roughly) the costs for each alternative solution	Design office
	5	**Design meeting** - The design staff explain and discuss the alternative technical solutions and corresponding costs with the FDC - The preferred list of proposed interventions is agreed	Suitable meeting place in or close to the project area
	6	**Agreement on beneficiary contributions (Optional)** - Discuss options for farmer contribution towards rehab costs - Agree on how farmers will contribute their share	
	7	**Detailed design** - Design staff prepare more detailed design drawings/sketches - Quantity and cost calculation	Design office
	8	**Design presentation** - Explain the final design + costs (use photos & sketches) - Farmer representatives sign for approval (minor revisions possible)	
	9	**Bidding documents** Finalize bidding docs for rehab works	

The staff inputs required for participatory design are substantially more than the “top-down” approach to planning and design that has often been used in the past. Meetings and the study of alternatives are time-consuming, while proposals that are subsequently dropped represent wasted effort. The participatory design process probably requires a doubling of design engineer time. While farmers do not understand engineering, they are familiar with the history of their system and what happens during operation. It is essential that design engineers understand the farmers’ experiences, views and ideas, relate then to conventional engineering theory and identify where spate irrigation does not conform to conventional theory.

Design engineers need to embark on the participatory design process without a firm opinion about what needs to be built because the farmers’ and engineers’ perceptions of problem priorities may be different (see Box 3-1 and Annex A) but, during the design process they guide the farmers about what is feasible and affordable to construct and their operation and maintenance implications. Provided that the farmers can understand what is proposed and how it will work (site visits to examples, if available, or photographs will help them understand) then they will be able to judge if the proposals are likely to be successful.

Box 3-1: Example of Beneficiaries’ Priorities

Participatory design in Yemen resulted in significant differences between the type of work prioritised through the participatory design process and that identified during preparation by engineers familiar with the project (see Annex A for a detailed comparison). The farmers gave much more emphasis on rehabilitation / improvement of existing control structures and provision of additional structures while the engineers anticipated substantial expenditure on canal earthworks. The exact reasons for this change of emphasis are not fully understood but could be; (a) the farmers consider the ability to control flows under spate conditions is more important than the amount of flow; (b) the farmers consider that external assistance is best directed on structures while they can always do earthworks themselves (either with available equipment or paying the direct operating costs for government equipment).

3.3. Understanding the Current Situation

3.3.1. Layout

Spate irrigation schemes comprise five main components:

- River regulation components – river bed stabilization, diversion points and embankments
- Some form of works to divert spate flows from the wadi. In exceptional circumstances where a wadi naturally splits into several braided channels forming an alluvial fan, the flow may be automatically diverted.
- Land to be irrigated. These are usually basins that have been constructed by the formation of perimeter bunds.
- Channels to convey the water from the diversion point(s) to the land to be irrigated. In some situations the land may be adjacent to the wadi, or water may be passed from field to field without separate channels.
- Drains (usually in the form of enabling safe passage of excess water out of the end of the system)

Understanding the existing layout (or, for a new irrigation scheme, the layout of the existing features) is a priority activity because it provides the context for the beneficiaries’ requests for improvements. If the overall system is not fully understood then the full nature of the problems or possible effects of solutions may not be fully appreciated. The best approach for understanding the existing layout is to combine fieldwork, where the positions of significant features are recorded by GPS[5], with recent satellite imagery. Spate irrigation schemes are dynamic and can change relatively quickly. Consequently satellite imagery more than about 5 years old may not accurately show the current situation, but is better than nothing. Any maps should also be used with caution because they may not represent the current situation but will help to explain the history of the scheme development.

[5] *A modern simple hand-held GPS should record positions within about 5m accuracy provided that it has a good view of the sky. If the GPS has a USB connector then it is often convenient to leave the GPS running all day recording a track of where it has been. The track can then be uploaded into GIS software or to Google Earth. Cameras with built-in GPS are also relatively affordable (<US$400). These store the position of a photograph within the image file. Some of these cameras include an electronic compass and will also record the direction of the photograph.*

3.3.2. Water Rights

Any existing spate irrigation system is likely to have established operational rules which prescribe who is entitled to water and when. Such rules will have evolved to take account of the interests of upstream and downstream users and reflect the limitations of the existing diversion arrangements. For example, traditional diversion structures will be breached by the larger floods and automatically provide the downstream users with an opportunity to obtain water. Engineered interventions are likely to affect this balance.

Box 3-2: Impact of Engineering on Traditional Water Rights

Example from Wadi Zabid
When modernisation of this scheme was being designed it was appreciated that permanent structures would affect the natural water distribution which was reflected in agreed water rights based on periods within each year when different groups of farmers were entitled to take water. A modification of the existing rules was discussed and agreed in principle with the different groups. However, construction then proceeded without getting the agreement signed. Thereafter the upstream group, realising that they would benefit from the new infrastructure, refused to accept any change to the traditional water allocation.

3.3.3. Irrigation and Agronomic Practice

Planning of any engineering interventions needs to be based on an understanding of when and how the crops are irrigated and grown. If a new scheme is proposed then it will be necessary to draw upon the experience elsewhere in the region supported by theoretical analysis (see Section 4).

The field-to-field water distribution practised in most spate irrigation schemes also means that the upstream farmers in an irrigation block may not be entitled to close off the water into their fields so that any floods can pass through to those farmers who are still waiting for sufficient water.

The farmers, particularly if growing grain crops such as sorghum, usually plant their crops at the same time to minimise the problem with bird pests. Anyone who plants late will have difficulty in protecting their crop from the bird population which has expanded after feeding on the other farmers' crops.

Participatory design increases the probability that investments will meet the needs of the final users. These users will have an increased sense of ownership of the final outcome because they have been involved in its identification and design and have to contribute towards the cost. This makes it more difficult for the users to complain that whatever is constructed is unsatisfactory and they should be more willing to take an increased responsibility for operation and maintenance (which was the ultimate objective of the project).

Those responsible for formulation of projects need to make adequate provision for the time and cost of the participatory process, both in terms of programme duration and the resources needed. Typically, the time and resources needed for the engineering work are doubled relative to a "top-down" approach. The main additional inputs are for the various meetings (including preparation), outline design of options and abortive work, where farmers change their requirements. Other staff are also needed to organise and negotiate with the farmers.

3.4. Addressing O&M at the Design Stage

3.4.1. Planning for Easy Operation and Maintenance

Specialisation of engineers tends to result in design engineers having limited "hands-on" experience in operation and maintenance and are, therefore, vulnerable to exercising poor judgement on O&M aspects. The common approach of designers to operation and maintenance is to first complete the designs and then, almost as an afterthought, write something about how the works should be operated and maintained. Problems arising from failure to undertake the prescribed O&M cannot then, in the designers view, be blamed on the designs.

In reality, for all design work, and particularly for spate irrigation, ease, feasibility, affordability and sustainability of operation and maintenance should be the first consideration. Only after those aspects have been thoroughly examined, in consultation with those more experienced in operational problems (such as the field staff and the farmers), should design work be undertaken. Normally simplicity and robustness in operation and maintenance should take precedence over achieving maximum efficiency in water diversion and distribution. The aspects to be considered include:

Gate number and operation: Gates can make water control easier but, unless motorised, take time to operate. Gates can also make it easier for farmers to take water out of turn while water passing down a canal to a closed gate may result in the canal banks being overtopped and breached. Electric motors can make operation much faster but add to the cost and create an additional maintenance burden. One option is to use a small portable generator which can be carried to the gates when it is needed. One related question is what happens if the generator doesn't work? Motorised gates usually have very high gear ratios and using a winding handle if there is an equipment problem can result in extremely slow operation.

Gate type and size: Small gates may be easier to fabricate and to operate individually, but cumulatively create a bigger operational problem because there are many more gates to operate. Small gates with small openings are also much more vulnerable to blockage by trash (see Section 2.3.3). Radial gates are more appropriate at headworks or major flow division structures so that the openings can be larger without the friction loads associated with sliding gates. Counterbalancing will reduce the effort required to operate large gates.

Sediment Management: Intakes should be designed to minimise the entry of coarse sediment into the canal system. While a sensible operational guideline is to restrict the flow into the canals during high floods when sediment loads are greatest, operation of gates in response to rapidly changing water levels may be difficult. Some spate irrigation schemes include sediment basins designed to intercept any bed load that does enter the canals. However, these can have disadvantages such as:

- Basins trap finer sediment when the flow is low
- Flushing sediment basins are expensive to construct and the farmers may consider any water used for flushing to be a waste of irrigation water unless there is another intake further downstream
- Non-flushing basins that have to be excavated are vulnerable to being filled by a large flood early in the flood season with subsequent floods providing no opportunity to clear the basin. Sufficient funds and equipment also need to be available to clear the basin.

Water that enters the canal system should be kept moving with sufficient velocity the ensure that suspended sediment is passed through to the fields.

Trash: Floods can carry substantial amounts of trash which can quickly block smaller gate openings or normal trash screens. Trash screens are best avoided because they quickly collect trash and block up. It is best to provide larger gate openings which will pass most trash and also provide a winch or gantry which can be used to pull out any trash once the flood has reduced. Sluiceways are best constructed without breast walls and with large gates that can lift clear of the flood water to enable trash to pass downstream.

Management of excess flows: If excess flows enter the canal system then there should not be significant damage. Measures to achieve this objective include:

- Provision of a side spillway on the head reach of a canal just upstream of a control structure to reject much of the excess flow
- Provision of proportional weirs on canals instead of gated control structures to distribute any excess flow through the system
- Provision of drop structures to control erosion of any drainage channels emerging from the field system.

Automatic operation: Floods will often arrive at night time when few people may be available to undertake any operation. Those few people cannot see what they are doing. A design which enables floods to pass to the fields with the minimum of intervention will be appreciated by the operators. Potential measures include:

- A sluiceway which will automatically open once a flood has reached a threshold, thereby allowing smaller floods to be diverted completely (this can be accomplished using an earth or gravel bund)
- Proportional dividers for any main canal branches

- The use of earth bunds in canals instead of cross regulators. These can breach automatically if there is any excess of water and can be easily breached if the farmers want to release water downstream. Once breached, they can only be reconstructed after the flood, which gives the downstream fields more likelihood of receiving water.

3.4.2. Establishing Responsibilities for O&M

Responsibilities for O&M should be discussed early in the design stage because this may affect design decisions. The farmers may be hoping for government to carry a greater share of the O&M burden but experience in many countries shows that funding constraints may limit the ability to fulfil any additional obligations in a timely manner.

It is, therefore, preferable to leave the overall responsibility for O&M with the farmers (or, if the system is currently government managed, to pass the responsibility back to them) and enhance their capacity to manage the tasks. Unless the farmers are growing cash crops, their contributions are more easily made as labour (using livestock where appropriate). Therefore designs compatible with labour-based maintenance are more appropriate than those that need machinery such as bulldozers (which may not be locally available even if the farmers have the money). Where the farmers are growing cash crops then equipment rental may be feasible provide such equipment is locally available.

An example of the design decision to be made is how to manage the sediment which enters the canal system. The traditional, labour-based approach is to let the sediment spread widely while the equipment-based approach is to collect the sediment in a sediment basin. However, cleaning a sediment basin using labour is difficult, particularly if gravel and cobbles have been collected while large excavators working from the sides would be needed to excavate a basin while it is wet. Basins may therefore be a source of problems unless the available funding permits flushable basins.

3.4.3. Flood Warning System

One of the operational challenges of spate irrigation schemes is to know when the floods are going to come. This is one area where modern technology can be used to advantage. National weather forecasts should provide a good indicator of whether rain is expected.

Increasing availability of mobile phones and service coverage may make it possible to identify someone living near a watercourse upstream of a spate irrigation scheme who can then phone someone in the irrigation scheme if they see or hear a flood or there is a heavy rainstorm in the area.

Alternatively, but more expensive, is to provide automatic water level monitoring equipment upstream of a scheme. The equipment will automatically dial one or more key personnel with a pre-recorded message (or send text message) should a flood be detected. Additional messages can be sent if there are significant changes to the flow. Such systems can work through the satellite phone system if the monitoring site is out of range of the terrestrial coverage. Such monitoring stations, if calibrated, can also provide flow gauging data. The challenge may be to identify a monitoring location that is sufficiently far upstream to give, for example, 30 minutes warning but also intercepts all the tributaries that bring significant floods. This arrangement was investigated for Yemen in 2003 but it was considered that the cost could not be justified.

4. SPATE AGRONOMY AND WATER REQUIREMENTS

4.1. Soils in Spate Irrigation Schemes

4.1.1. Soils in Existing Spate Schemes

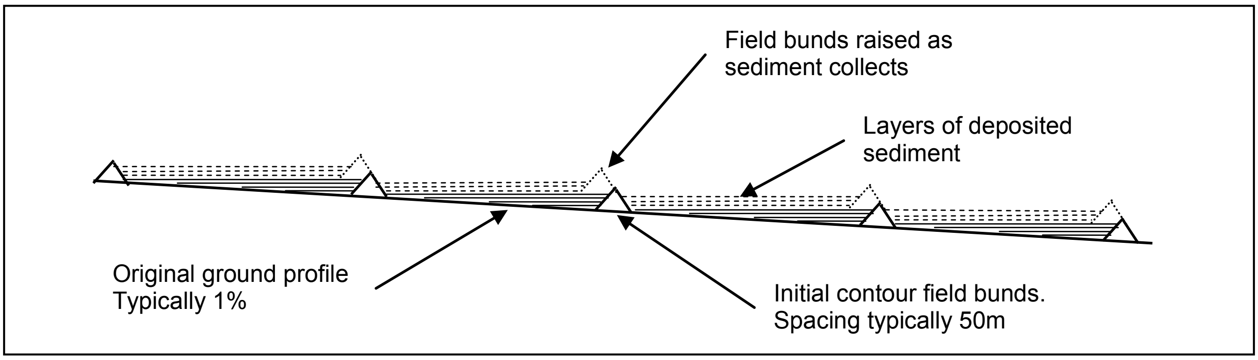
Fields in existing spate irrigation schemes are usually formed of deposited sediment, not the original ground. However, the type and thickness of sediment will vary. Fields in the upstream parts of system tend to receive more, but coarser, sediment and fields in the downstream parts of the systems tend to receive less water and, therefore, less sediment. Some blocks of fields may only have been developed relatively, either in response to greater potential for irrigation or to utilise a new diversion point further upstream.

As part of the planning process for any irrigation improvement, it is recommended that a soil survey be undertaken with a minimum point density of 1 per 100ha with sufficient samples taken to characterise the soils over a 2m depth. Knowledge of the soil moisture holding capacity can be used to identify whether the traditional irrigation arrangements developed by the farmers can be further refined.

4.1.2. Soils in New Spate Schemes

New irrigation schemes, or extension of existing irrigated areas, should be designed to accommodate the sediment that will be deposited during irrigation. Soil surveys should be undertaken to determine the crop production potential of the existing soils although this will be gradually improved by the deposited sediment. Suitable construction of field bunds will not only harvest the sediment but also use it to create level terraces, thus avoiding expensive land levelling. A typical arrangement is shown in Figure 4-1.

Figure 4-1: Schematic Arrangement for Building Terraces from Deposited Sediment



The field bunds are typically 0.6 m to 1m high and can be formed using oxen pulling scoops if the existing soil is moderately soft. The same arrangement would be used for maintaining and raising the bunds. Where the natural ground is relatively hard or gravelly it will be necessary to use a bulldozer to create the initial bunds. During the first few years (depending on the size and number of floods), most of the sediment will tend to collect in the areas immediately upstream of the bunds. Unless the underlying soil also has excellent water holding capacity, the best crop production will initially be in progressively widening strips of the deposited soil upstream of the bunds.

4.1.3. Salinity and Sodicity

Salinity and sodicity are potential hazards where the flood water contains significant salts and/or sodium content (usually derived from the parent rocks) and irrigation applications are not sufficiently generous to flush these minerals below the root zone. Both types of minerals can adversely affect crop growth although some crops are more tolerant than others. This potential problem has been evaluated in detail[6] for the Sheeb area in Eritrea which came to several significant conclusions including:

- The large and very large floods had a higher, and potentially hazardous, saline and sodic content
- Analysis of soils samples from the field did not reflect these levels of salinity and sodicity perhaps because, until the recent construction of a diversion weir, the larger floods would break the traditional diversion arrangements and pass down the wadi

[6] *A tradition in transition. PhD Thesis Abraham Mehari Haile, 2007. Chapter 8.*

- At least 10% excess water should be applied in order to help flush chemicals through the root zone
- The downstream fields were most at risk of problems because they usually only received water during the larger floods and were less likely to receive sufficient water for flushing
- System water management should therefore provide some water from the medium floods to the downstream areas in order to help flush the chemicals
- Maize is more vulnerable than sorghum to yield loss due to salinity problems
- The effect of moderate salinity or sodicity on crops has similar appearance to water stress and result in reduced yields.

Sodic soils can also cause engineering problems because the soil structure becomes very weak when wet because the soil particles tend to disperse. This makes embankments very vulnerable to failure.

Vulnerability to salinity and sodicity will depend on catchment and water quality which will vary with individual spate systems. Periodic monitoring of the soil in the spate irrigated fields is recommended in order to detect any trends. Any visible build-up of white material on the surface should be followed up by laboratory testing.

4.2. Spate Agronomy

4.2.1. The Relevance of Agronomy to Design

Why write about agronomy in the design manual? Because growing crops is the output of any irrigation scheme and the design needs to be compatible with the crops that might be grown. Otherwise, a well-designed, but inappropriate, irrigation system may not provide the expect benefits. Designers therefore need to understand the on-farm end of irrigation before designing the rest of the system.

4.2.2. Suitable Crops

Usually, the principal focus of spate agricultural is subsistence agriculture which is to produce enough food to support the household. Also, farming in spate irrigation systems usually relies predominantly on livestock for draught power which requires that providing sufficient forage for the animals is also one of the objectives of crop production.

Drought-tolerant varieties of crops such as sorghum and millet are preferred because they will usually provide some yield even if the irrigation is limited. Shorter duration varieties of maize may also be grown and under favourable conditions can both yield better than sorghum and are less vulnerable to losses due to birds. Medium or long duration varieties of maize may, under favourable soil moisture conditions, give the best yields but are more vulnerable to drought resulting in substantial yield reductions.

Box 4-1: Cash Crops

Marketing of cash crops - Sheeb
In 1996 one of the farmers tried growing water melon under spate conditions and was successful. He made a good profit selling the produce at the local market. In 1997 many more farmers invested in this cash crop. However, supply outstripped local demand and, with Sheeb being 45km from the main highway, prices plummeted.

However, cash crops including vegetables can also be grown on soils with good water-holding capacity. However, the profitability of cash crops depends on access to markets. Limited production of cash crops can be sold in the local market but many spate schemes are relatively remote from the main cities so transport cost will be a consideration where production of cash crops exceeds local demand. For crops which have to be transported fresh then speed is also relevant. Cash crops that can be dried and stored may be more attractive.

4.2.3. Cropping Patterns

Irrigation is interlinked with the cropping pattern which requires understanding which crops can be grown when. With spate irrigation the cropping has to fit the irrigation and local rainfall (if any). The starting place for identifying the cropping pattern is to study the current farming practice including discussing with farmers the consequences of doing things differently. Most likely they have already tried the alternatives and discovered the disadvantages. If planning a new spate irrigation system then the starting place for planning cropping is any other nearby spate irrigation schemes. Considerations that feed into the planning of the cropping pattern include:

- When do the floods occur?
- When does any rainfall occur?
- Is there potential for supplementary irrigation from groundwater?

- What is the soil water holding capacity?
- How much water is available (floods + rainfall)?
- Climate
- Crop growth period
- Crop rooting depth

An example of how these factors define the cropping pattern in the Eritrean eastern lowlands is given in Box 4-2. This shows how the farming has evolved to suit the local conditions. Other spate irrigation systems may benefit from having small areas set aside for crop trials to test alternative crops and varieties and discover whether there would be benefits from modifying the general cropping pattern.

Box 4-2: An Example Cropping Pattern

The evolution of the cropping pattern for Sheeb (Eritrea)
The main flood season is July and August but rainfall is most likely in January - February (but quantities usually small relative to the irrigation). The soils are generally deep (>2m) with potential water holding capacity of 1.0m within a 2m depth. Irrigation will exceed this provided there are at least two floods per farm The climate is very hot from May to September (mean maximum temperatures >40°C) but cooler in the other months. Fields are lightly ploughed after irrigation and before planting to leave a loose top layer of soil which breaks the capillary and reduces evaporation losses.

The basic cropping pattern has evolved to comprise:

- Planting in September when the probability of floods is reduced (flooding will damage or kill the growing plants) and harvest of the first crop in about December. The main crop is sorghum or short-duration maize but some vegetables may be sown in about October as the weather cools down. All farmers tend to plant their main crop over a short period so that crops mature together and minimise the problem of progressive pest build-up
- A second crop can then be grown using the remaining soil moisture supplemented by any rainfall or small floods. This is either sorghum (ratoon) or short-duration maize.
- Sometimes a second sorghum ratoon is grown to provide fodder
- Finally the land is ploughed in preparation for the next irrigation season. Water will infiltrate faster into a ploughed field.

Most of the families in the area migrate to the highlands with their livestock in April and do not return to the project area until November when the weather is cooler and harvesting is about to commence. During the intervening period farmers with oxen temporarily visit the project area to undertake irrigation work, ploughing and planting.

4.2.4. Agro-Chemical Usage

Normally, agro-chemicals can be used to boost yields of irrigated crops. Chemicals may also be required for pest control, although integrated pest management is preferable. The use of artificial fertilizer, however, needs to be considered carefully because of the uncertain water availability. For example, application of fertilizer without the corresponding irrigation can damage the crop while a large flood could flush out the fertilizer which is wasted. The timing of fertilizer application is also important because of the need to apply the fertiliser at appropriate crop growth states in order to obtain the maximum benefit.

In many spate systems the sediment carried by the floods contains enough nutrients to support good yields of spate irrigated crops. Otherwise, natural means of improving soil fertility may be more effective. The use of green manure by ploughing in a green (preferably leguminous) crop to increase the soil organic content can be included in a crop rotation. Other leguminous crops can also be included in the cropping pattern (possibly as a second crop using residual soil moisture) to both provide produce and improve soil fertility by adding nitrogen to the soil.

4.2.5. Agro-Forestry

Growing tree crops is an integral part of spate irrigation in some countries but not in others. Trees are not competing with other crops for water because, once their roots are established, they are able to use the water that has passed below the root zone of field crops. The potential benefits of agro-forestry include:

- Production of wood for building purposes and fuel
- The trees may produce edible fruit
- Provision of shade
- Rows of trees can form wind breaks both reducing potential for dust storms and reducing crop evapotranspiration as shown on Figure 4-2. Lines of trees as wind breaks will not only reduce the

wind velocity but also tend to trap air containing transpired water, thus raising the humidity of the air surrounding the crops. Lower wind and higher humidity both reduce evapotranspiration.

However, one potential drawback of trees is that they can provide convenient habitat for grain-eating birds that are a pest for farmers. However, careful selection of trees varieties less suitable for nesting can reduce this problem.

4.3. Crop Water Requirements

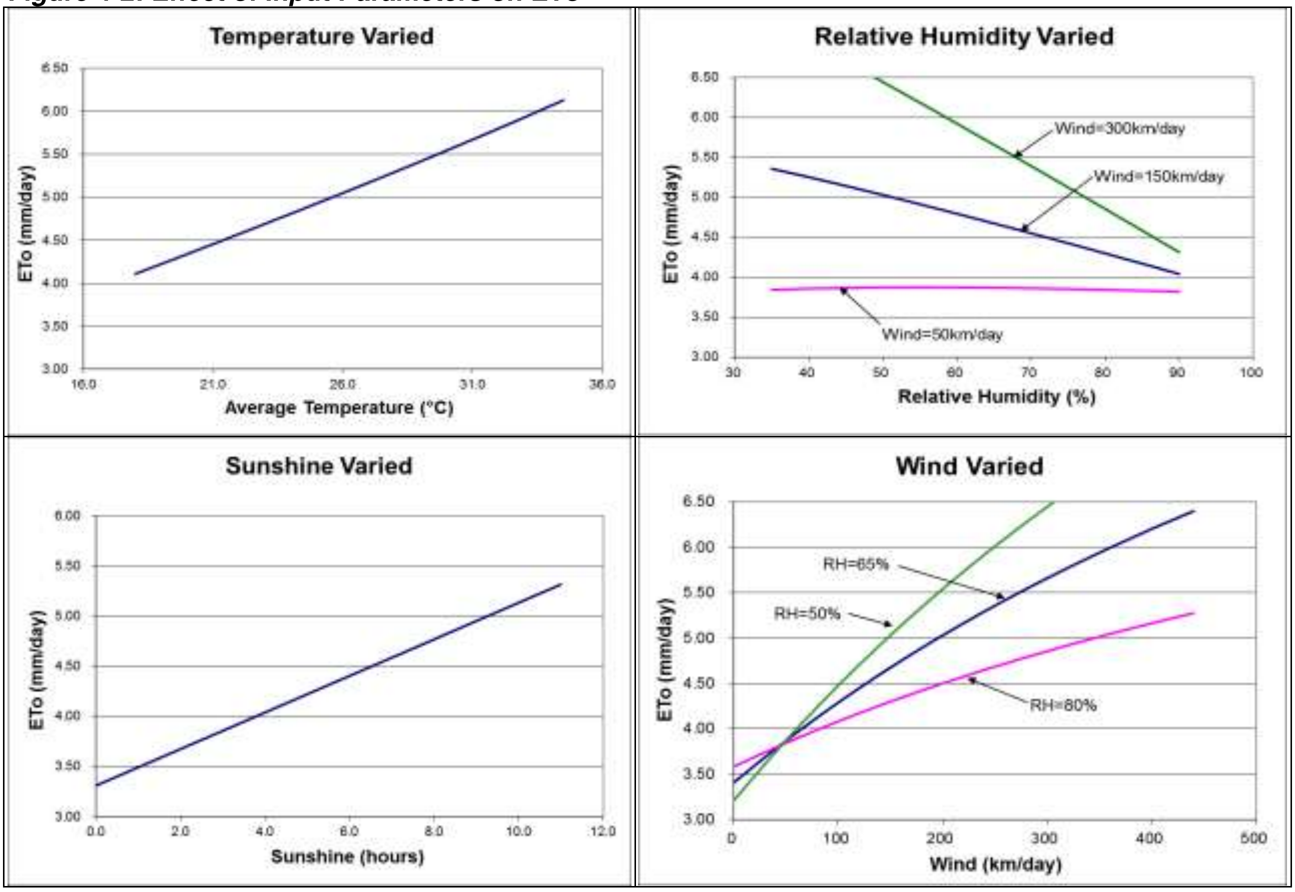
4.3.1. Estimation of Reference Evapotranspiration

Calculation of crop water requirements is an integral part of the design of any new irrigation scheme in order to quantify the balance between available water and crop water usage. The procedure as described by FAO[7] is standard for any irrigation scheme and it is recommended that the FAO Cropwat 8 program is used for the calculations. The challenge is to select appropriate input data for the calculations. This is discussed below.

4.3.2. Climate Data

Most spate irrigation schemes do not have the benefit of a meteorological station collecting a long-term series of climate data. Even if a meteorological station exists, it should be verified for being representative of field conditions. It should be within the cropped area but often the equipment is in an open area convenient for the office. As such, the climate station does not benefit from the micro-climate arising from the cooling effect of the crops as they absorb the sunshine during their growth period, the reduction in wind due to the resistance provided by the crops and the increased humidity from the water transpired by the crops. The sensitivity of the reference evapotranspiration, ETo, to the key input parameters is shown in Figure 4-2.

Figure 4-2: Effect of Input Parameters on ETo



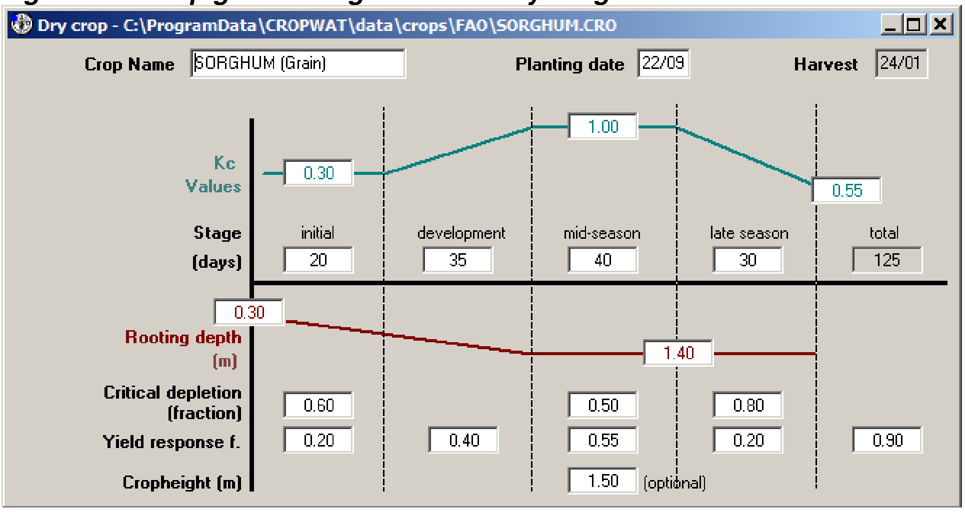
[7] *Crop evapotranspiration - Guidelines for computing crop water requirements, FAO Irrigation and Drainage paper 56: 1998*

If there are no meteorological stations nearby to the irrigation scheme then the climate data will have to be estimated from the nearest regional stations. Some average data are available in the FAO Climwat database. However, all climate data should be given some basic quality checks. Procedures for doing the checks are described in the Annex 5 to the FAO paper 56. Annex 6 of this paper explains the relationship between minimum temperature, dew point and humidity. This check should be undertaken. Tmin may be higher than Tdew, particularly when no crops are growing, but it cannot be lower than Tdew. Also, an average daily relative humidity of near to 100% is impossible unless the daily temperature variation is very small. The humidity cannot exceed 100% at the daily minimum temperature (when moisture condenses out of the air as dew) and the humidity reduces as the temperature rises so the average humidity will drop below 100%. Quite often, daily climate data contains an unrealistically high minimum humidity value. This may be the result of twice daily manual data collection not including the time of lowest humidity.

4.3.3. Crop and Soil Data

Crops use different amounts of water depending on the growth stage. The FAO Cropwat program contains pre-defined data for typical crop growth parameters but these may need to be adjusted to suit the durations of the proposed crops. Figure 4-3 shows a typical pre-defined set of crop data held within the Cropwat database.

Figure 4-3: Crop growth stages for 125 day Sorghum



The erratic nature of water supply in spate irrigation schemes means that the soil needs to hold as much water as possible in the root zone. Soil samples should therefore be tested for water holding capacity and the soil properties entered into the Cropwat program.

4.3.4. Crop Yield and Crop Stress

There are some periods, such as during flowering or grain filling, when crop stress due to water shortage can substantially reduce the yield which water shortage at other growth stages has much less effect. Crop yield under conditions of intermittent irrigation can be estimated by the Cropwat program by varying the timing of irrigation. The FAO Aquacrop program enables a more precise simulation of the crop production cycle.

Selection of crop varieties should focus on the inclusion of shorter duration crop varieties. Although these have lower maximum yield potential, they also reduce the risk of crop failure if there is a deficit in water availability.

4.3.5. Irrigation Development Alternatives

Planning of cropping should consider different options for timing of planting. For example, in the Eritrean eastern lowlands the crops are planted after the main spate irrigation season and can also benefit from winter rainfall but in the western lowlands the crops are planted at the start of the rainy season, which is also the flood season. This increases the challenge of managing floods without drowning the crops.

4.4. Groundwater Development

Availability of groundwater in sufficient quantities for irrigation substantially increases the options for irrigation development by making water availability independent of floods. Water stored as groundwater does not require expensive reservoirs that lose capacity due to sediment deposition and the water is not lost by evaporation. However, it may gradually migrate downstream towards the sea.

Groundwater is used extensively in Yemen to enable year-round cropping in spate irrigation areas including production of high value perennial crops and fruit trees. Farmers try to maximise the irrigation of their fields during floods in order to increase the recharge to groundwater. However, over-exploitation of groundwater is taking place in many locations resulting in declining groundwater levels, higher pumping costs and saline intrusion along the coastal fringes.

Recharge to groundwater as a by-product of spate irrigation (the conveyance and percolation losses in Table 5-2) can be around 50% of the diverted flow. Where conditions permit the re-use of this percolated water then the economics of investment in spate irrigation infrastructure are much improved for two main reasons:

(i) The overall use efficiency of diverted water is approximately doubled meaning twice as much crop per unit of diverted water; and
(ii) The availability of the groundwater to provide supplementary irrigation during any gaps in the supply of spate water improves the likelihood of achieving consistently good yields.

There is, however, the additional cost of pumping the groundwater, but the ability to supply water to match the crop demands can substantially improve the yields.

At present, groundwater usage in the spate irrigation systems in Eritrea is currently negligible. In the eastern lowlands, exploration for water sources for village water supplies in 1997 is reported to have encountered saline water in many areas which suggests that the potential for groundwater use for irrigation may be limited (the chosen source of water supply for Sheeb was at the mouth of the wadi). However, further investigation would be merited in order to confirm the groundwater situation since, if suitable water is available in reasonable quantities, then it changes the whole potential for agricultural development because of greater security of water supply. Similarly, if groundwater is available in the western lowlands then the dependency on unreliable rainfall is reduced. However, if groundwater is discovered and developed then it must be carefully managed to avoid over-exploitation as is the current situation in Yemen.

5. SPATE HYDROLOGY

5.1. Background Information

A detailed discussion of methods for analysis of floods and sediment yields is presented in Section 3 of the FAO Guidelines on spate irrigation[8]. Where possible, the analyses should be undertaken by a hydrologist who has previous experience of hydrology for arid and semi-arid conditions. The discussion below is aimed at providing a background for design engineers who should have an understanding of what is involved in order that they can undertake preliminary calculations, ask the appropriate questions, check that suitable procedures are used (including reconciling field information with theoretical calculations), appreciate the limitations in the output they are provided and can prudently apply the information for the design work.

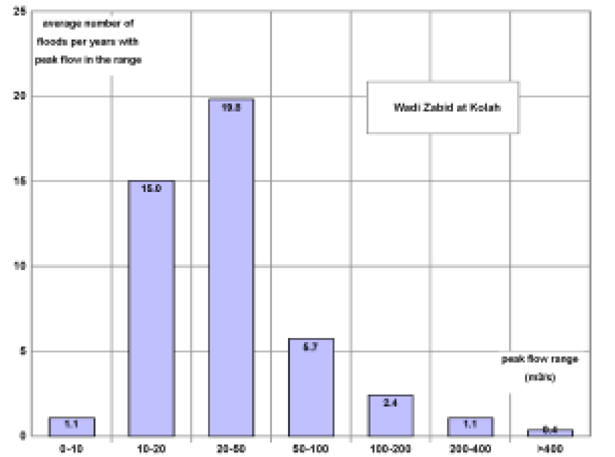
5.2. Understanding the Floods

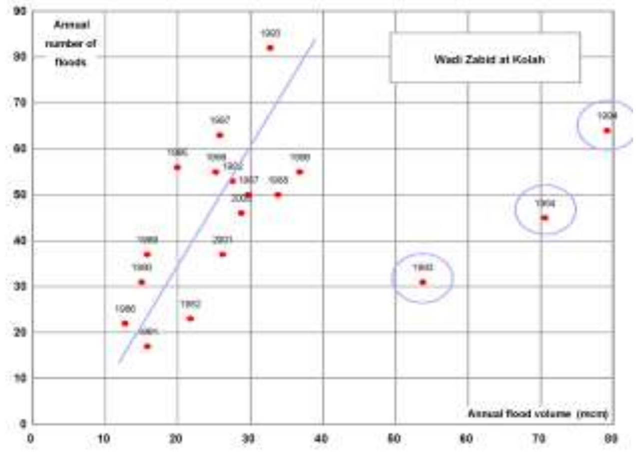
5.2.1. Field Hydrology

Rule number 1 in spate hydrology is do not trust the theoretical output without making some field checks. Among the checks that can be made are:

(i) Find out the number of floods in each of the past few years from the local population. Try to classify them into large, medium and small and whether the years were wet, dry or average.
(ii) Also find out the flood durations and whether length is dependent on the peak flow.
(iii) Ask about the biggest floods in both history and living memory and, if possible, the years when they occurred
(iv) At a location where the wadi / river is relatively confined, ask the local population to indicate the maximum water levels for both the highest floods in recent years and the highest in memory. The flow can then be estimated by surveying the channel cross section and the water surface slope at that location (but note that, unless the channel has a rock bed, the cross section area is probably enlarged during the flood peak). The channel Manning's roughness values are likely to be in the range 0.03 (relatively smooth with gravel bed) to 0.07 (relatively rough bed of cobbles and boulders). This roughness range can affect the calculated flow substantially, so it is desirable to calculate flows using more than one location or method (for example to take some velocity measurements during floods).

Figure 5-1: Flood Variability for Wadi Zabid



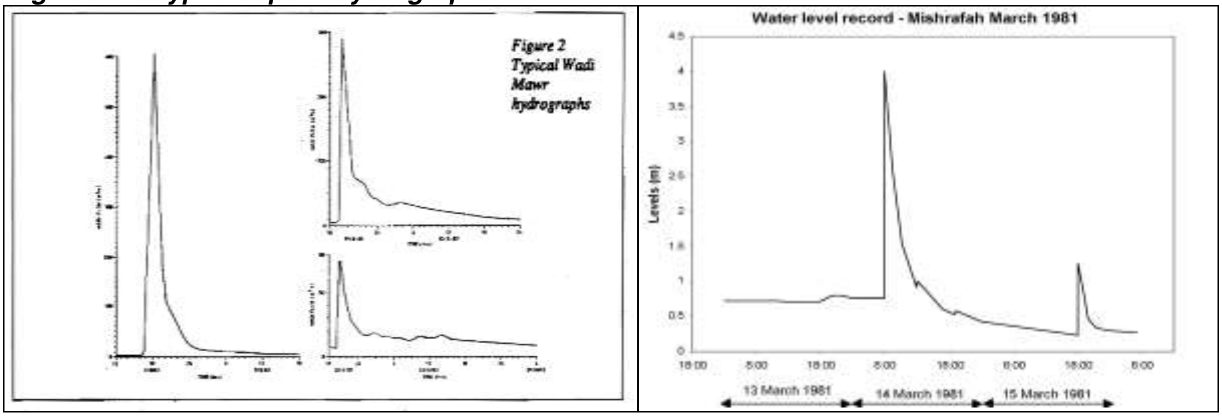


An average of 5.7 floods per year between 50 and 100m³/s.	Less than 20 to more than 80 floods per year

Designers and hydrologists are better able to comprehend the problems they are trying to address if they have seen the existing wadi / river and spate irrigation system during floods. Unfortunately, the erratic nature of flood events makes this difficult. The next best alternative is to have photographs taken during the floods (although this can be difficult since many floods peak occur at night). Photographs taken during the recession usually show the high water marks.

[8] *Guidelines on spate irrigation. FAO Irrigation and Drainage Paper 65. FAO 2010*

Figure 5-2: Typical Spate Hydrographcs



5.2.2. Gauging Data

There may be flow gauging data for a nearby site but the quality needs to be verified before use. Manual readings are particularly vulnerable to misrepresentation because the reader can enter a reading even if they have never visited the gauge. Automatic gauging is also vulnerable to problems. For example, the minimum reported rise time for a flood is the minimum increment on the recorder. This is a particular risk with electronic equipment with data loggers. If the recording interval is 20 minutes but the flood rises in 2 minutes it will show as 20 minutes. Also, the recorder could miss the peak flow of a very short duration flood. Ideally, the data loggers should be programmed to record data for both fixed time intervals and incremental changes in water level (such as 0.5m).

A further uncertainty about gauged flow data is the rating curve to convert the water level to flow. If the gauge is at a fixed cross section with rock beds and banks then the cross section will not vary so the only unknown is the channel roughness at that cross section. Where there is a mobile bed then the channel cross section may vary both within and between floods. Gauging data is therefore best used as a broad indicator of the flood characteristics (frequency, duration and approximate magnitude) but unless the gauge has been frequently calibrated, the calculated flows may not be very accurate particularly at the higher stages where physical flow measurement is difficult, if not impossible.

The TRMM (Tropical Rainfall Measuring Mission) precipitation dataset[9] is another possible source of hydrological data. This uses various sensors to collect 3 hourly rainfall-related data which is then post-processed with data from other meteorological satellites to produce estimated daily rainfall for 0.25-degree by 0.25-degree (approximate 25km x 25km) spatial resolution. Spatial coverage extends from 50 degrees south to 50 degrees north latitude. The satellite was launched in 1997. The resolution is sufficient to provide an improved understanding of rainfall patterns on a sub-catchment basis than is provided by most rain gauge networks.

5.2.3. Flood Peak Flow Estimation

Estimates of the peak flood flows are required for design of diversion structures. Substantial errors in flood peak estimation can have major repercussions on the design. Underestimation of floods can result in operational problems and damage such as from overtopping or scour while overestimation of flood severity will result in unnecessary cost.

Several methods can be used for flood estimation:

a) analysis of long-term records of measured flood discharges;
b) analysis of synthetic long-term runoff data derived from stochastic modelling;
c) the rational methods based on a “design” rainfall intensity, a time of concentration derived from catchment parameters and a runoff coefficient that depends on catchment conditions;
d) regional flood frequency relationships;
e) slope-area calculations to estimate the size of the largest historical flood that has occurred, for which local informants can provide a reasonably reliable estimate of the flood water level;
f) Velocity-area measurements of actual floods in which the surface velocity can be measured using floats at a measured cross section.

[9] *http://disc.sci.gsfc.nasa.gov/precipitation/*

In practice, the first method is virtually never feasible as long-term flow data only exist for a small number of wadis worldwide. Short datasets can be misleading. For example, is a large flood in the data an outlier or representative? The second method would only be considered for large projects that have the resources to commission specialized hydrological modelling. Rational methods are used in some areas but require information on catchment characteristics for the selection of appropriate runoff coefficients and rainfall intensity, which are data not available in the regions where many spate irrigation systems are located. Nonetheless, they can provide an overall check on results obtained by other methods. Possible methods are listed in Table 5-1. Rainfall-runoff methods may be appropriate for small catchments (<50km²) if regional rainfall intensity data are available.

Table 5-1: Methods for Estimating Annual Flood Peak Discharge

Method	Formula	Remarks
Binnie (1988)	$MAF = 3.27 \cdot A^{1.163} \cdot MSL^{-0.935}$	Regional flood formula developed for wadis in Southern Yemen but probably OK in the Red Sea region
Bullock (1993)	$MAF = 0.114 \cdot A^{0.52} \cdot MAP^{0.537}$	Developed using data from 43 semi-arid catchments in Botswana, Zimbabwe, South Africa and Namibia
Nouh (1988)	$MAF = 0.322 \cdot A^{0.56} \cdot ELEV^{0.44}$	Developed from regressions on data from 26 gauging stations
Farquharson et al. (1992)	$MAF = 0.172 \cdot A^{0.57} \cdot MAP^{0.42}$	Developed from 3,637 station years of data collected from arid zones worldwide.

(From FAO 2010)

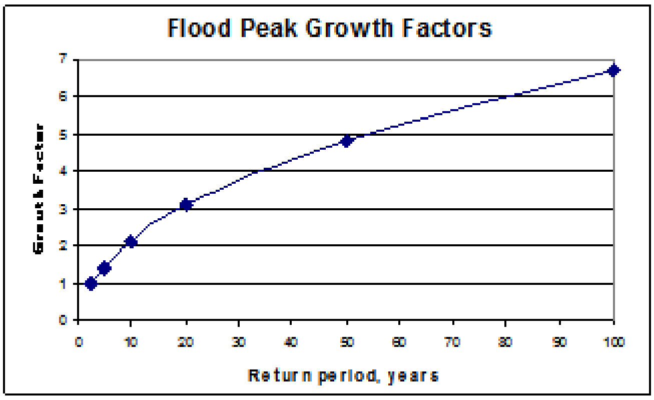
Note: MAF = Mean annual flood peak discharge (m³/s)
A = Catchment area (km²)
ELEV = Mean catchment elevation (m)
MSL = Main stream length (km)
MAP = Mean annual precipitation (mm)

The hydrological analysis needs to take account of a number of rainfall scenarios. For example, the flood with the highest volume of water may be generated by moderate rainfall over the whole catchment but the highest peak flow may be generated by an intense small storm on hills just upstream of the diversion site.

Maximum rainfall intensity data need to be reduced as the rainfall area increases. Indicative area reduction factors are shown on Figure 5-4. It can be seen that the intensity drops to below 80% of the peak for catchments greater than a few km² with the intensity of shorter storms dropping faster than for longer storms.

One factor to be considered in the flood estimation process is the potential for natural flood attenuation upstream of the structure site. A diversion site that is at the mouth of a valley will have little potential for upstream attenuation because flood peaks will be retained in the confined channel. However, a diversion site that is further downstream may benefit from flood peaks being attenuated by in-channel storage or temporary overbank flooding once there is space for the floods to spread. This effect is particularly pronounced for floods with short duration. See Box 5-1 for example data for short-duration rainfall intensity.

Figure 5-3 : Indicative Flood Peak Growth Factors



Return period (years)	Growth factor
Mean annual	1
5	1.4
10	2.1
20	3.1
50	4.8
100	6.7

Note: Graph from hydrological analysis for the Yemen Irrigation Improvement Project

Box 5-1: Indicative Sub-daily Rainfall Intensities

For small and medium catchments where the main source of rainfall is short storms and the time of concentration of flow is less than one day, sub-daily rainfall becomes important. Ideally, sub-daily rainfall intensity data should be derived from recording rain gauge data in or near to the catchment being studies. If this information is not available then regional data can be used. The data in this box are for the south-west Arabian peninsula and illustrates the potential intensity of short storms. For example, the 1 hour rainfall can be about two-thirds of the daily total.

The sub-daily rainfall can be expressed as a proportion of the daily rainfall (Table 1) and combined with the probability data for maximum daily rainfall (Table 2) to give sub-daily rainfall amounts for different levels of probability (Table 3).

TABLE 1: SW SAUDI ARABIA. Ratio of the storm rainfall of duration 'D_t' to the 1 day depth. '$D_{1\ day}$' (Source: Wheater. H., Larentis. P And G. S. Hamilton. (1989). "Design Rainfall Characteristics for SW. Saudi Arabia.". Proc Inst Civ Eng. (London) Series 2. Vol 87. December.)

Duration (D_t)	10 min	30 min	1 hour	2 hours	6 hours	1 day
D_t / $D_{1\ day}$	0.33	0.56	0.68	0.79	0.92	1.00

TABLE 2: Estimated Probability Distribution of Annual Maximum Daily Rainfall (mm)

Recurrence Interval (years).						
2	5	10	20	50	100	200
74	90	100	110	125	133	143

Note: Data are for Taiz in Yemen

TABLE 3: Sub-daily Rainfall Amounts (mm)

Recurrence Interval (years)	**Duration**					
	10 min	**30 min**	**1 hour**	**2 hours**	**6 hours**	**1 day**
2	25	41	50	59	68	74
5	30	50	61	71	83	90
10	33	56	68	79	92	100
20	36	62	75	87	100	110
50	41	70	85	99	115	125
100	44	75	90	105	122	133

Note: Data are for Taiz in Yemen

It should be appreciated that these rainfall amounts apply to point rainfall which will not occur at the same intensity over a significant area, one reason being that storms usually move. As the area increases then the average rainfall intensity for that area will decrease.

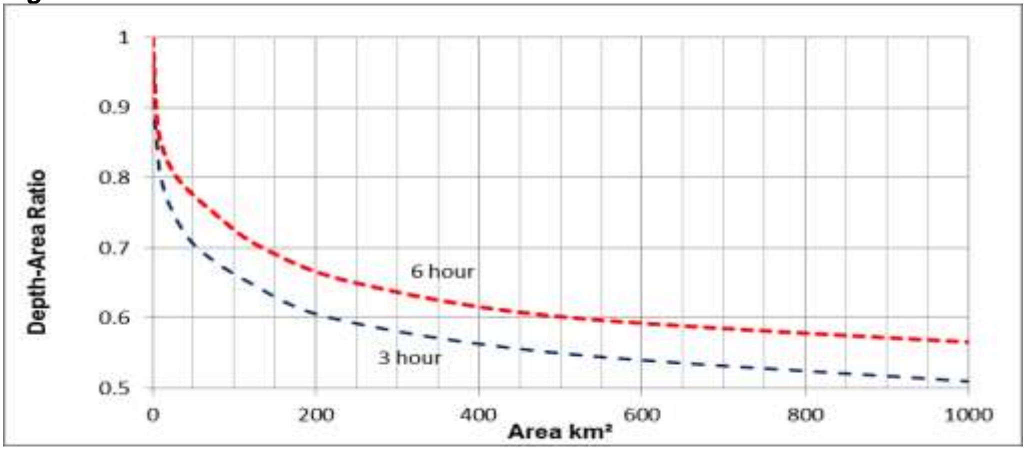
5.2.4. Slope-Area Method

The basis for the slope-area method is being able to observe the maximum water level for a flood event using flood marks, usually debris. The peak discharge of the flood is estimated using the high water marks to determine the slope. Three or four cross sections are usually surveyed so that two or more independent estimates of discharge, based on pairs of cross sections, can be made and averaged. Additional field work required for slope-area estimates consists of selecting the stream reach, estimating "n" values and surveying the channel profile and high water profile at selected cross sections. The work is guided by the following:

(i) The selected reach is as uniform in channel alignment, slope, size and shape of cross section, and factors affecting the roughness coefficient "n" as is practicable to obtain. The selected reach should not contain sudden breaks in channel bottom slope, such as shallow drops or rock ledges.
(ii) Elevations of selected high water marks are determined on both ends of each cross section.
(iii) The three or more cross sections are located to represent as closely as possible the hydraulic characteristics of the reach.

Distances between sections must be long enough to reduce the errors in estimating stage or elevation but should not include major changes in channel properties.

Figure 5-4: Indicative Area Reduction Factors



From: United States National Weather Service. 1984

5.2.5. Estimation of Mean Annual Runoff and Potential Irrigated Area

The proportion of the mean annual runoff (MAR) that can be diverted to the fields is an important parameter in determining the potential command area, although in spate schemes the areas that are irrigated can vary widely from year to year. MAR is conventionally expressed as a runoff depth from the catchment, in mm, but can easily be converted to a volume by multiplying it by the catchment area. The proportion of the runoff volume that can be diverted for irrigation depends on the diversion arrangements and the patterns of spate flows that are experienced. This is difficult to determine accurately without extensive long-term site-specific flow data but can be estimated from the observed flood characteristics.

The proportion of rainfall on a catchment emerging as runoff is usually between 5% and 10%, the higher end of this range being for smaller and relatively steep catchments. As an example, 5% runoff from a catchment of 100km² with a mean rainfall of 500mm is 2,500,000 m³. This is sufficient for a gross irrigation application of 1000mm over 150ha at 60% diversion efficiency. (Diversion efficiency is the proportion of the total flow that is diverted for irrigation.) The variability in rainfall can result in the area potentially irrigated varying from, typically, less than 50% to more than 200% of the mean value. Part of the runoff can be base flow which may be a substantial volume of water due to its long duration. This volume should be deducted from the quantity of water assumed for spate irrigation as it may only reach a few upstream farmers or become recharge to groundwater. In arid areas the mean (ie average) rainfall may be skewed by the occasional very wet year. In this case the median value (ie the middle value in a ranked range) may be a better indicator of the typical rainfall.

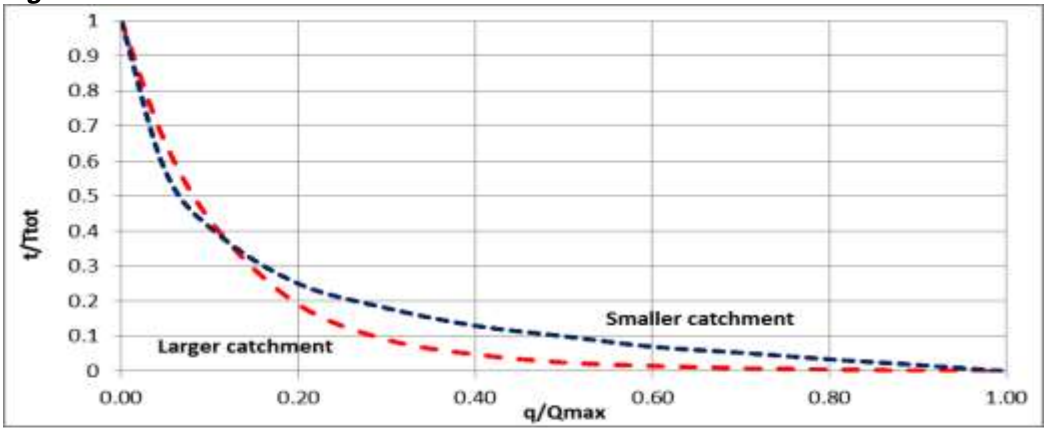
5.2.6. Estimation of Flood Frequency, Volume and Duration

The number of floods and the volume of water they contain are variable from month to month and year to year. As the catchment area increases then smoothing of fluctuations in rainfall distribution will occur. Analysis of about 20 years of records for Wadi Zabid in Yemen showed the number of floods per year ranged between 18 and 82 while the estimated total annual flow in the wadi (both base flow and flood flow) ranged between 48 MCM and 240 MCM.

Within the annual variability of water availability lies the variability of individual floods. A single flood could contain much of the annual volume of flood water, but if the flow is large and exceeds the capacity of the diversion works then water will go to waste. Similarly, if the first flood of a season is large and washes out diversion works then the subsequent floods cannot be fully utilised until the diversions are repaired. Planning of spate irrigation development needs to take account of how many floods, how much water they contain and how long they last. Two floods may have the same volume of water but one may have a low peak and long duration while the other has a high peak and short duration. The former flood is easier to manage and has better potential for irrigation. From the farmers' perspective a "good" flood is one that has a moderate peak but a long recession.

The flood flow data can be consolidated into a flow-duration curve which relates the flow to the proportion of the time for which it occurs. Figure 5-5 below shows example flow-duration curves for two typical catchments. The flow is less than 50% of the mean annual peak flow for 90% of the time.

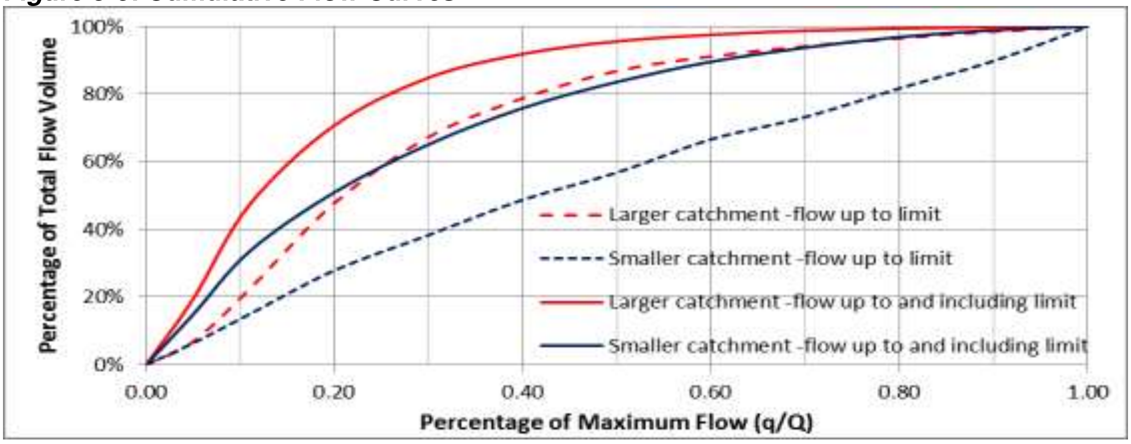
Figure 5-5: Indicative Flow-Duration Curves



It must be appreciated, however, that the shape of the flow-duration curve will vary depending on rainfall and catchment characteristics. For example, a small, steep catchment is more likely to have short duration floods with relatively high peaks because there may be little potential for attenuation. In such circumstances a larger proportion of the total flood volume occurs at higher flows. This should be verified by catchment-specific analysis.

Basic checks should be undertaken on the output of theoretically derived from flow-duration curves by using the number, size and duration of flood events reported by the farmers to estimate the volume of water that could be diverted. Should this be substantially different from the theoretical output then the two results need to be reconciled.

Figure 5-6: Cumulative Flow Curves



It is possible to integrate the time and flow data to provide the proportion of the total flow volume that is below a specific flow value. Figure 5-6 shows the cumulative flow curves based on the data in *Figure 5-5*. This example graph covers two scenarios:

(a) the flow above the limiting value is ignored (this is equivalent to closing an intake when the wadi flow reaches a threshold value); and
(b) the flow above the limit is quantified at the limiting value (this is equivalent to leaving the intake open but the maximum capacity is fixed) and the volume above this value is excluded.

The graph shows that for a smaller catchment, with a lower volume of flow in the flood recession, it is necessary to size the intake capacity for a larger proportion of the maximum flow in order to divert the same proportion of the total flood volume. Thus, two similar-looking flow-duration curves actually have very different distributions of the total flow volume due to the different shape of the recession part of the graph.

5.2.7. The Probability of Irrigation

Relying on spates for irrigation is inherently uncertain and risky while distributing the water, often at night, is challenging. For those with no other source of water, this challenge has to be accepted. The area developed for irrigation tends to expand until the probability of receiving water is low, perhaps one time in five years. This approach can be justified if the investment in canal systems and field development is minimal, as is the case in traditional systems, but a higher probability of irrigation is required where significant engineered infrastructure is proposed.

One of the outputs of a hydrological analysis needs to be an estimate of the variability of floods both in terms of the distribution within a year (what proportion of the annual volume per month) and the distribution between years (how much the annual flood volume varies from the average). Unless there are long-term gauging data, this will have to be estimated from the variability of rainfall within the catchment or region.

Fairness of water distribution may be an issue. In some schemes farmers may have land in different areas with different probability of receiving water, which spreads their risks. This is practical in smaller schemes where distances are no more than a few kilometres. In larger schemes other forms of water allocation need to be adopted. Two such systems are (a) a fixed calendar giving time periods for the entitlement of intakes to receive water (as practised in Wadi Zabid in Yemen) and (b) central allocation of floods based on flood size and which areas had already been irrigated (as used to be practised in Wadi Tuban in Yemen). Existing spate irrigation systems will have usually developed rules for water allocation, at least between the major irrigation blocks.

The command areas of existing spate irrigation systems have often grown beyond the capacity of the source of water. One reason for this is that developments usually start in the downstream areas where the floods have attenuated and are easier to manage. Over time, competition for water results in land being progressive developed further upstream which can ultimately leave the original downstream land without water except in the occasional very wet year or when there are exceptionally large floods. Mapping and quantification of irrigated areas should, therefore, include collection of field data about the overall development history and the water availability and crop production in the previous 10 years or more for different sub-areas. Consideration should also be given to trends in water availability due to either climatic cycles or changes in water use upstream. The farmers can usually explain if long term changes in water availability or flood characteristics are taking place even if they do not know the cause.

5.2.8. Irrigation Efficiency

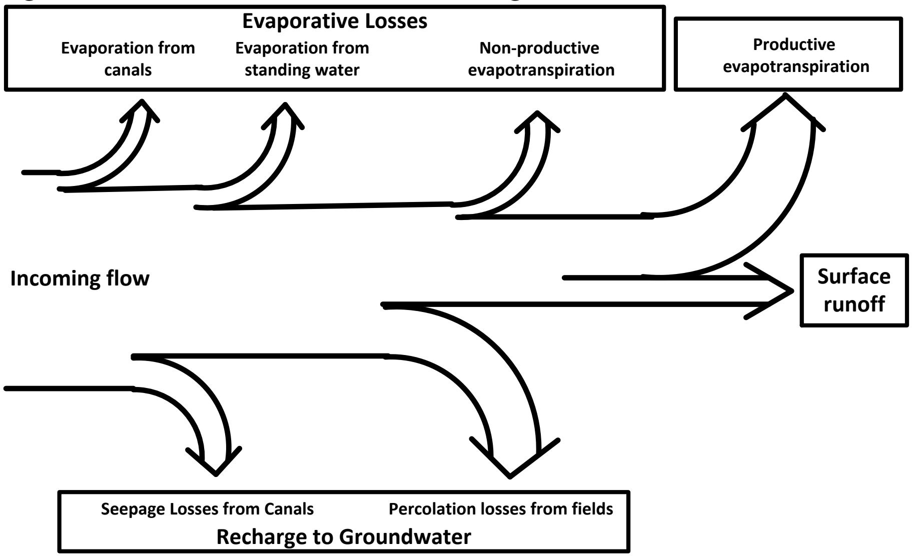
Only part of the water diverted into an irrigation scheme will ultimately be used by the crops. This proportion gives the overall irrigation efficiency. In Figure 5-7, the water used by the crop is productive evapotranspiration. The remaining water may be lost to evaporation, percolation or surface runoff. Whether percolation is actually a “loss” depends on whether the groundwater is re-used by pumping, in which case the percolation is beneficial - it is water going to temporary storage for subsequent use - but it is considered as a loss in normal efficiency calculations.

In normal surface irrigation systems the overall irrigation efficiency is typically about 40%. That is, if the crop requires 0.4m depth of water for full production, then 1m depth has to be diverted to the system. Specific factors relating to the efficiency of spate irrigation are:

- Conveyance losses through seepage can be substantial, particularly where canals are on sandy or gravelly material. However, canals in mature systems with substantial sediment deposition may contain substantial fine sediment in their bed material which reduces the percolation losses
- Evaporation losses tend to be limited because water is only flowing for limited periods
- Application of water is very erratic, both in terms of timing and the difficulty of control over how much water goes where
- Surface runoff only occurs during very large floods. Under normal conditions the field-to-field irrigation captures all the inflow and runoff from one field becomes irrigation water for another

Where the fields are on deep soils with good water holding capacity then percolation losses can be relatively small (10% to 30%) unless the applied water is excessive. However, if the soils are thin or sandy then the percolation loss can be much greater.

Figure 5-7: Schematic Distribution of Irrigation Inflow



An example of the estimation of the overall irrigation efficiency to take account of site-specific conditions is given in Table 5-2.

Table 5-2: Estimation of Irrigation Efficiency

Condition	Conveyance loss (canals)	Evaporation loss	Percolation loss from fields	Overall efficiency
Sandy or gravelly foundation + thin soils	30%	15%	50%	25%
Most of system on silty loam with thick soils	20%	20%	30%	40%

Note: Overall efficiency = (100-distribution loss) * (100-(evaporation loss+ percolation loss))

Conveyance losses down a wadi can also be very significant. Measurements during low flow conditions have indicated losses of between 1% and 5% of the upstream flow per kilometre. This proportion will be reduced during floods because the rate of percolation into the channel bed will be the constraint.

5.2.9. External Future Changes to the Hydrological Regime

Climatic conditions are inherently variable and possible future changes should be considered when planning water-related developments. One of the symptoms of global climate change in many countries is a more frequent occurrence of severe rainfall events which will result in more severe floods and a lower proportion of the overall flood volume contained in more easily managed moderate floods. Increased variability between very wet and very dry years may also occur.

Another external factor is changes to the conditions in the catchment. Forest cover is generally considered to reduce the rate of runoff. It will also reduce the sediment yield because of protection of soil from the direct impact of rain drops. Land cover degradation due to unrestrained agriculture will result in increased, and faster, runoff while conservation measures such as terracing will reduce both the water and sediment runoff. Development of small-scale irrigation in the catchments will cumulatively reduce the total runoff volume.

While it is difficult to predict the future but the implications of potential changes in the catchment land use should be considered in the evaluation of water resources for an irrigation scheme.

6. WADI MORPHOLOGY AND MANAGEMENT

6.1. Overall Wadi Morphology

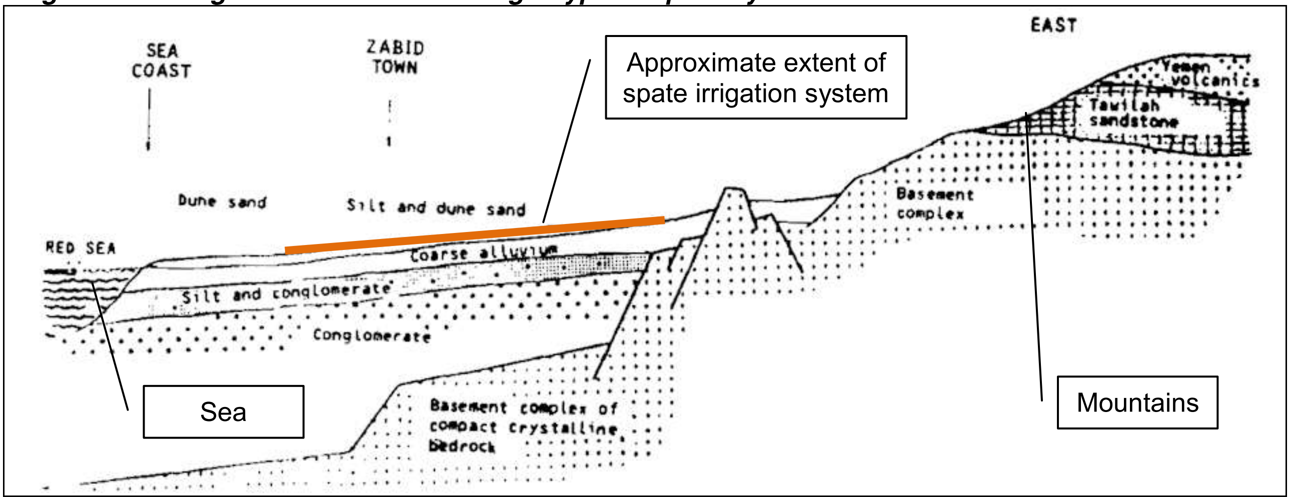
6.1.1. Longitudinal stability

Wadis usually, but not always, originate in mountains and tend to be steep and erosive in their upstream section where sediment transport is controlled only by the rate of erosion and supply of material. Where the wadi emerges from the mountains the channel slope tends to flatten and material is deposited. This is a morphological process that occurs over geological time. The spate irrigation system is usually superimposed on alluvium that has been deposited by the natural erosion - deposition process over many thousands or millions of years. An example is shown in Figure 6-1.

However, not all spate systems are founded on natural alluvial fans. Some, particularly smaller systems, are founded on thin soils within otherwise mountainous areas where there is a convenient location to divert flood flows onto flatter land.

The amount of sediment that can be transported depends on sediment size and density and the flow velocity. If the carrying capacity of the water is less than the incoming sediment load then some material will be deposited. Conversely, if the sediment load is less than the carrying capacity, then erosion will occur unless the channel is passing through non-erodible material. Varying flows result in variable sediment transport capacity and significant changes can occur during floods as material is first eroded and then deposited. A channel that has not changed substantially over many years is said to be “in regime”. Various formulae, known as regime formulae, exist that attempt to relate sediment transport to the channel and sediment characteristics.

Figure 6-1: Longitudinal Section Through Typical Spate System



Longitudinal section through Wadi Zabid in the Yemen Tihama (after Tesco-Viziterv-Vituki 1971)

Degradation and aggradation are the processes of long-term erosion and deposition of bed material in a river that affect its longitudinal profile. They normally occur as a series of progressive steps, predominantly during floods, but exclude the more localised effects of scour during a particular flood event.

Degradation usually appears as a general lowering of bed levels along a reach of river, and is caused by the reach seeking to adjust its longitudinal gradient to match the requirements of the flows and sediment loads that it carries. If the sediment load entering the reach is lower than the actual transport capacity within the reach, degradation starts at the upstream end and works its way downstream, so as to reduce the overall longitudinal gradient. However, if the channel downstream of the reach in question has a greater sediment transport capacity, degradation starts at the downstream end of the reach and works its way upstream, leading to an overall increase in the longitudinal gradient. In the case of aggradation, the above causes and effects are reversed. Clearly, channel degradation is the more critical condition when considering scour at structures.

In many rivers there is an approximate equilibrium or "regime" (with no continuing degradation or aggradation in the area of interest). However, the stable regime conditions to which the river has become adjusted may be disturbed by changes resulting from natural processes and/or human interference. These changes may include:

- Catchment changes
- increased runoff and/or sediment supply from deforestation, increased urbanisation or land drainage
- River channel changes
- natural morphological changes, such as meander progression and cut-otfs
- sand and gravel mining from the channel bed
- The influence of other structures
- removal of a downstream "control", such as a weir or bridge, that previously inhibited degradation at the site of interest
- creation or removal of an upstream structure that affects the sediment supply

6.1.2. Channel Vertical Stability

A meandering channel can give the wadi the correct overall balance between slope, velocity and sediment transport ability. Alluvial channels have some basic characteristics:

- Meanders tend to migrate downstream with time as shown on Figure 6-2.
- Interventions in one location can cause effects in another location by changing the natural erosion - deposition process. This process is illustrated in
- Figure 6-3.
- Sand and gravel extraction from the wadi bed can upset the morphological balance and cause erosion downstream because bed load material will refill any holes caused by the extraction.

Figure 6-2: Typical Channel Meander Movement

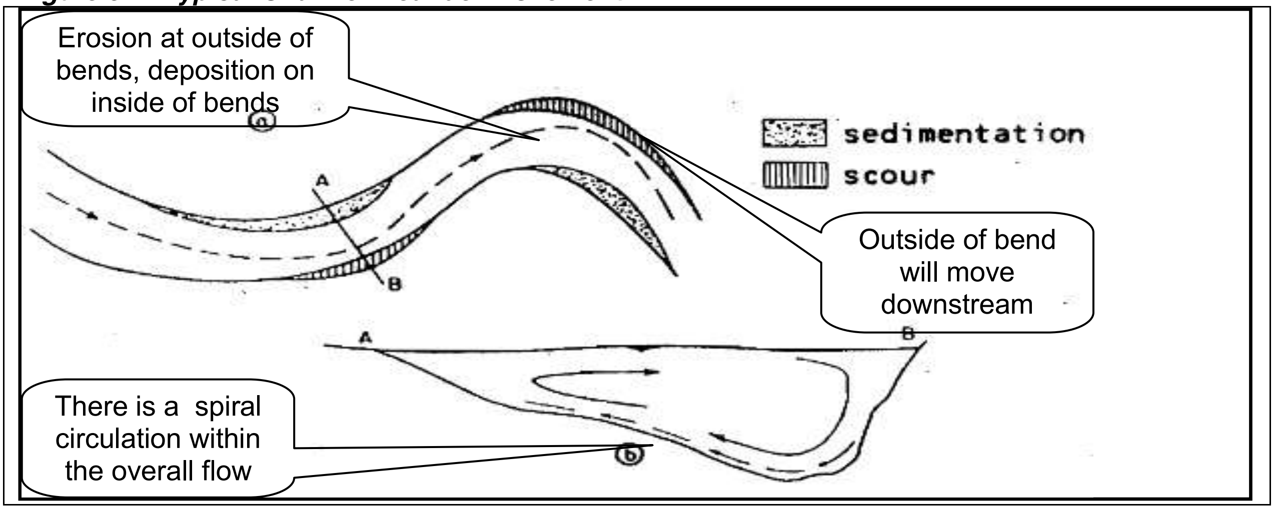


Figure 6-3: Changes to Wadi Bed Profile Caused by a Structure

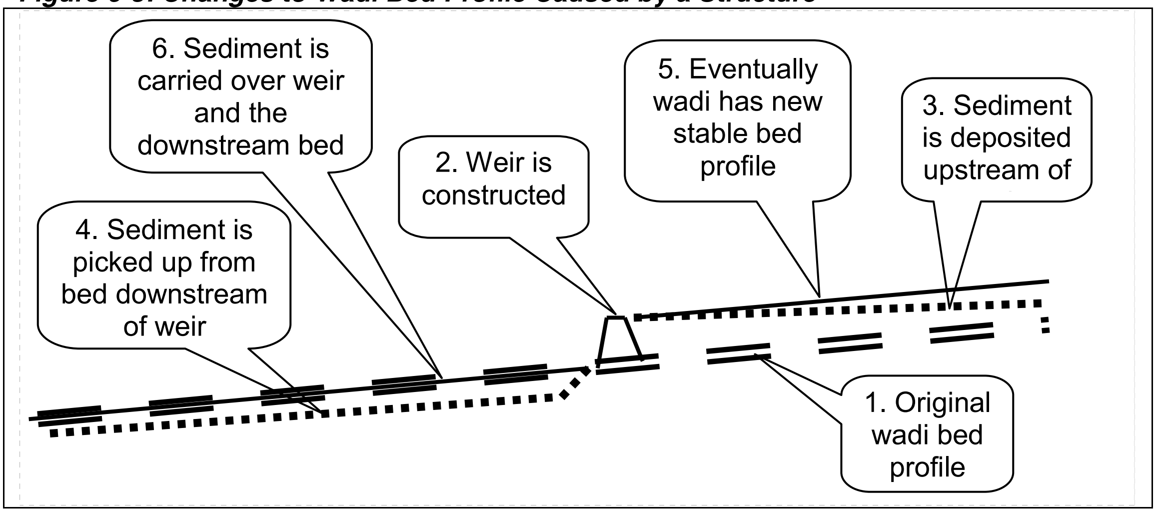


Figure 6-4: Effect of River Morphological Changes

	
Deresa weir, Eritrea: The bed level downstream of the weir has dropped by more than 1m and scour holes are developing, probably in response to sediment deposition upstream of the weir	Faleg ladh weir, Yemen: The bed level downstream of the weir has dropped by up to 2m, probably caused by extensive gravel extraction in the section of river downstream of the weir

6.2. Channel Horizontal Stability

Channel migration may occur naturally or as a result of human activity, and may be associated with any of the causes that give rise to degradation and aggradation. Migration of the entire river channel as part of the process of meander progression, or movement of the deep-water channel within the same overall channel banks, can affect the scour exposure of a bridge or other structure whose foundations may have been fixed in relation to an earlier channel position. In some cases, migration may occur rapidly in response to a particular flood event, but in other cases it may be gradual. In a braided channel, the channel positions are continuously changing.

Taking account of the potential for channel migration is an important part of the design or assessment of fluvial structures. As a general rule, if there is potential for channel migration, the foundations should be designed or assessed on the basis of any credible shifts of the deep-water channel or channels. Alternatively, training works may be carried out to limit the possible movement of the deep-water channel.

Whether a channel is straight or meandering depends on both whether there are hard features (natural or artificial) to control the channel alignment and whether the natural ground slope is steeper than the morphologically stable river slope. If the ground is too steep then the river channel may meander in order to achieve a stable slope. If the channel is meandering then it is normal for the meanders to move as shown in Figure 6-2. Erosion will take place at the outside of bends, unless there is erosion resistant material. This erosion occurs because the momentum of the water carries it to the outside of the bends until the water encounters something sufficiently solid to deflect the flow path. This deflective force is a potential source of erosion. There is also a spiral circulation within the overall flow as some of the water at the outside of the bend moves towards the inside of the bend. The amount of scour depends on:

- bend curvature
- width-to-depth ratio
- vulnerability of bank material to erosion
- bed material grading and strata.

Some meandering of wadi channels with time is normal. However, once people start to develop land or irrigation infrastructure along the wadi then they want to stabilise the channel and protect their investment. Farmers often try to develop land within the wadi channel because there is easy access to water and some, who own the land at the side of the wadi, try to expand their land holding. Any works that intrude on the wadi channel will tend to upset the natural balance and deflect water away from the intrusion. This, in turn will tend to cause erosion of the opposite bank further downstream, which will create another demand for protection.

The nature of wadi hydrology means that severe floods can be expected at unpredictable intervals. Designers should ensure that any works in or close to the wadi channel respect the need to pass the major floods. Requests to protect land inside the overall wadi channel must be resisted. Often this can be achieved by reference to the extent of inundation and damage by the last major flood.

Any proposals for bank protection works must consider the possible interaction between the works and the wadi morphology. That includes both how the wadi behaviour will impact on the works as well as how the works will impact on the wadi behaviour. There is the risk, particularly with narrower wadis, that works to protect one bank may deflect the flow to the other bank and create a new problem.

Existing maps, satellite imagery and aerial photography should be obtained and consulted in order to provide an overview of the wadi and how it has changed with time. Local people should be consulted in order to find out whether channel movement has been progressive or have been sudden changes, possibly associated with major flood events.

For high flood discharges the roughness of the channel can be assumed as n = 0.035. This assumption should give a reasonable estimate of the flow depths and velocities. The channel bed slope should be averaged over about 1km. The equation can be easily transformed to find any one unknown.

6.3. Works in Wadis

Works to stabilise the wadis fall into two main categories: Training works which stabilise the horizontal alignment and sometimes the vertical profile; and protection works that protect or stabilise one bank at a specific location. The components of these two types of works are similar but the configurations and objectives can be different.

6.3.1. River Training

In cases where a bridge or hydraulic structure is located on a river or channel that is unstable, river training works should be considered. The purpose of river training works is to constrain the river locally to reduce instability and thus pass flows through the structure under good hydraulic conditions. There are three main types of river training:

- longitudinal structures, such as guide bunds, which are parallel to the flow and define the river banks and prevent lateral movement
- transverse structures, approximately perpendicular to the flow, to deflect flow away from a bank, reducing flow velocities at that bank of the river, thereby reducing lateral movement and encouraging build-up of sediment
- bed control structures, mainly taking the form of sills or weirs, which fIx bed levels, so reducing degradation of the river bed upstream of the sill. Longitudinal river training protects the river banks from erosion. They are often useful for velocity control at expansions to avoid separation of flow and eddy formation downstream of abutments.

Construction materials for training works may include include riprap, gabion mattresses, concrete blocks (interlocking or articulated) and sheet piling. In addition, various bio-engineering solutions using soil reinforcement and vegetation cover are coming in to more widespread use, generally in locations of low flow velocities (less than about 2 m/s).

6.3.2. Bank Protection

The main function of bank protection is to prevent erosion of one bank of the wadi. The normal cause of erosion is the flow being directed towards the bank due to upstream flow conditions or the bank being on the outside of a bend. The protection may be active, which works by changing the flow pattern, or passive, which strengthens the bank to make it resistant against erosion.

Active protection uses components such as spur dikes which project into the flow and change the overall flow pattern.

Passive protection is often called revetment. It is constructed along the face of the bank and may be formed of materials such as gabion mattresses, concrete blocks, stone rip-rap or stone pitching.

6.3.3. Spurs

Spur dikes (or groynes, as they are alternatively termed) are structures constructed projecting from a bank to protect the bank from erosion. These are widely used for the purpose of river training and serve one or more of the following functions:

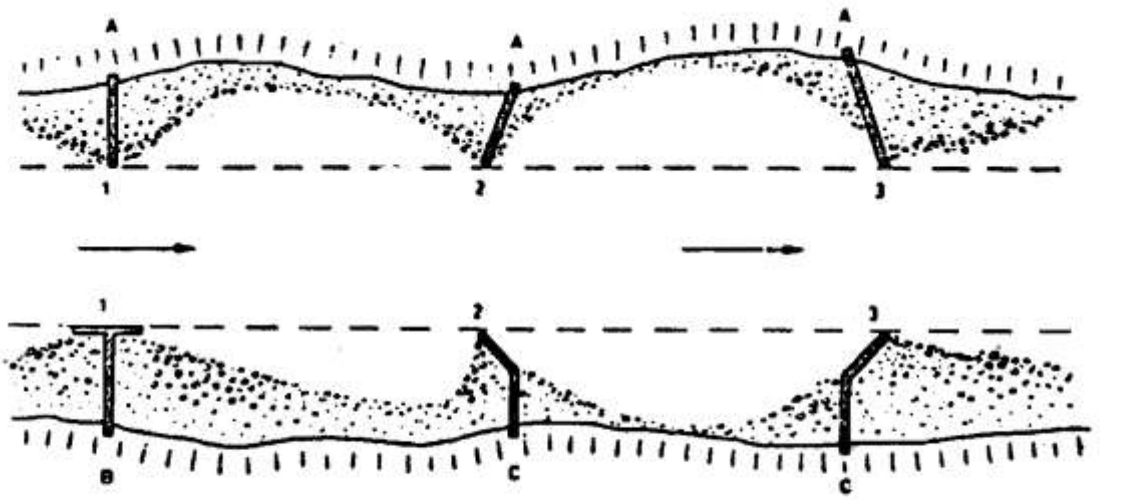
- Training the river along a desired course by attracting, deflecting (or repelling) and holding the flow in a channel. An attracting spur creates deep scour near the bank; a deflecting spur shifts deep scour away from the bank, and a holding spur maintains deep scour at the head of the spur.
- Creating a zone of slack flow with the object of silting up the area in the vicinity of the spur.
- Protecting the river bank by keeping the flow away from it.

These structures may either be impermeable (eg formed with dumped rock or of embankment type with a soil core protected by rock armour) or permeable (eg constructed using timber, steel or concrete piles) so as to allow some flow parallel to the bank, but at a low enough velocity to prevent erosion and / or encourage sediment deposition. Care needs to be exercised in the use of spurs to ensure that they do not simply transfer erosion from one location to another, or initiate unforeseen changes in the general channel morphology.

By acting on the flow around them, spurs dikes tend to increase local velocities and turbulence levels in their vicinity. The structure of the dike itself may be liable to erosion; flow moving parallel to the bank is intercepted and accelerates along the upstream face of the dike towards the nose. The high velocities and strong curvature of flow near the nose of a spur can cause significant scouring of the adjacent channel bed. Unless the foundations of the structure are deep enough or are well protected, the end section of dike may be undermined by local scour and could lead to a progressive failure of the whole structure.

The design of bank protection should consider whether the protection should be passive (ie not attempting to change the flow direction) or active (ie affecting the flow intended). Bank protection by simple revetment is passive but spurs can be active because they alter the flow direction. Examples of different types of spurs are shown on Figure 6-5. The pattern of deflection will depend on the angle of the spur relative to the channel flow direction. A spur inclined downstream is most effective at deflecting flow away from the bank to be protected and consequently is most likely to cause unwanted erosion of the opposite bank. A spur inclined upstream is less likely to deflect the flow and is more likely to promote sediment deposition in front of the bank being protected. Local turbulence around the ends of spurs will result in a high risk of scour damage during major floods unless sufficient protection is provided. Stepped ends to spurs will reduce this problem.

Figure 6-5: Different types of spurs



(A) ***Straight type*** **- (1)** ***At right angles to stream*** **(2)** ***Slanted upstream*** **(3)** ***Slanted downstream.***
(B) ***Hammer type.***
(C) ***Bayonet type*** **(2)** ***slanted upstream*** **(3)** ***slanted downstream.***

Commonly used materials for revetment include gabions, masonry walls and large concrete blocks. Gabions are the most common material for spurs. All erosion protection works need adequate provision against undermining by scour. Engineered erosion protection works to protect agricultural land are unlikely to pass any cost-benefit analysis, particularly if there is land elsewhere which does not currently receive irrigation water. There are, however, alternatives for erosion protection which have lower costs and durability, and may give better cost-benefit performance. Such works include:

- Bank protection and spurs formed of cut timber, placed with the trunk ends facing outwards. This option requires availability of sufficient raw material without risk of threatening the environment.
- Bank protection using vegetation. Suitable grass which has a dense root network may be appropriate for small channels and Tamarisk trees have been suggested for wadi banks.

Nose of gabion spur collapsed due to scour

Spur with stepped nose and extensive apron

6.3.4. Spacing of Spurs

The spacing between spurs depends on the channel width, length of the spur from the bank and its projected length. General recommendations are:

- In a straight reach the spur spacing should be about five (5) times the projected spur length.
- Spurs may be spaced further apart, with respect to their projected lengths, in a wide river than in a narrow river, having similar discharge.
- The location of spurs affects their spacing. The recommended spacing for convex bends is 2 to 2.5 times the projected spur length; and for concave bends, equal to the projected spur length.

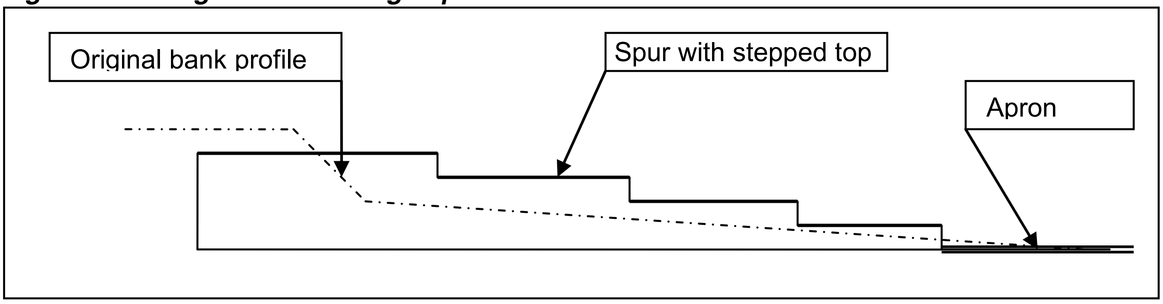
6.3.5. Length of Spurs

No general rules can be formulated for fixing the length of spurs. It depends entirely on the corresponding conditions and requirement of the specific site. The length should not be shorter than that required to keep the scour hole formed at the nose away from the bank. Too short a length may cause bank erosion upstream and downstream of the spur due to eddies formed at the nose. A long spur may encroach into the main river channel and would not withstand flood attack from discharge concentration at the nose and a high head across the spur. Normally spurs are shorter than one fifth (1/5) the channel width.

6.3.6. Profile of Spurs

Spurs are often constructed with a flat top set above flood level and then a vertical end. While the end of the spur nearest the bank should be above flood level and keyed into the bank sufficiently to avoid the risk of flood water passing around the back of the spur, there is no need for most of the spur to be above flood level. In fact, if some water flows can pass over the spur then the turbulence around the nose of the spur will be reduced.

Figure 6-6: Long section through spur



6.3.7. Revetment

Revetment can be classified as a passive protection because it directly protects the surface of the wadi bank but does not interfere with the flow (unless part of an encroachment on the wadi channel). Revetment should be used where there could be unwanted side effects if the flow pattern is disturbed, such as upstream of an intake. Revetment can be constructed of various material included stone pitching, cemented stone pitching, rip-rap, gabion mattresses or concrete blocks. Slopes are normally 1 unit vertical to 1.5 or 2 units horizontal.

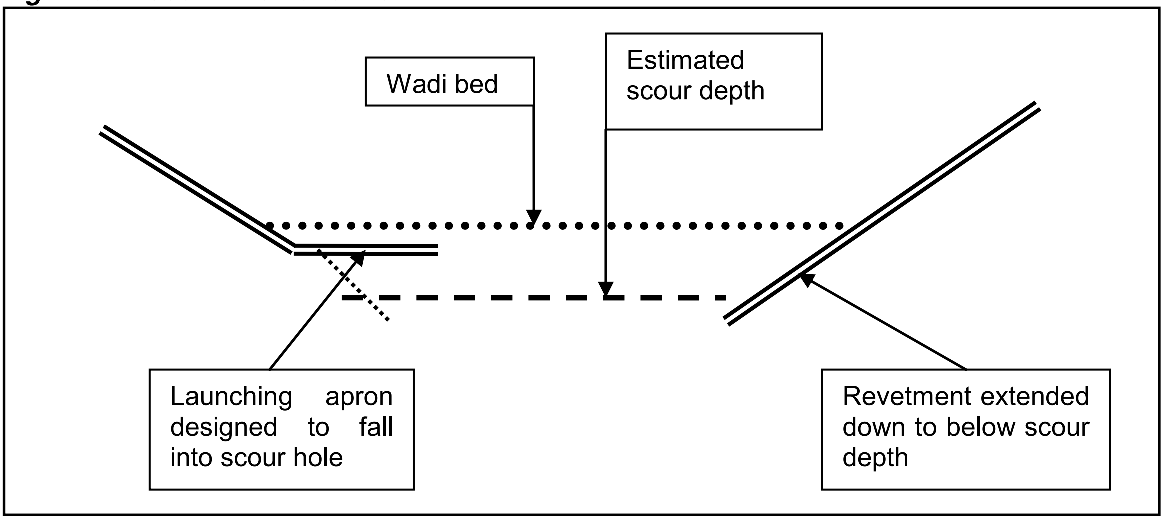
The revetment should either be extended to below the estimated scour depth by excavating the wadi bed or provided with an apron designed to fall into a scour and limit the extent of scour as shown on Figure 6-7. A horizontal apron can be either a gabion mattress of stone boulders. Extension of revetment into the wadi bed is the recommended option unless there are water problems or the depth is beyond a reasonable excavation depth. If an apron is provided there is less certainty that it will perform correctly. A mattress may not bend as expected, or may break, while boulders may migrate.

The construction of longitudinal training works may result in the depth of scour adjacent to the works being greater than would be the case if the banks were in a natural state. The presence of a protected bank can have two separate effects on local flow conditions.

First, the bank may alter the magnitude and direction of the flow velocity adjacent to it. Changes in the slope or surface roughness of a bank may also alter the local flow velocity. Thus, constructing a length of vertical or steeply sloping guide wall with a smooth finish would cause the bed at the toe of the wall to be subject to higher velocities and scour than would have been the case with the original natural bank. Conversely, if a section of natural bank were replaced by a revetment of the same slope but with an armour layer of greater roughness, local flow velocities near the toe of the bank might be reduced.

The second effect that a revetted bank may cause is a change in the level of turbulence within the flow near the bank. For straight sections of bank, the turbulence level is affected by the surface roughness of the armour layer; if the revetted bank is smoother (and straighter) than the natural bank, the level of turbulence and the amount of scour may be reduced.

Figure 6-7: Scour Protection for Revetment



It is apparent from this description of the processes involved that a factor such as the surface finish of a revetment can have opposing effects in terms of local scour. On the one hand, a high roughness tends to reduce velocities close to the bank (which is beneficial), but on the other it tends to increase turbulence levels (detrimental). Few systematic studies on changes in scour depth caused by the presence of revetments have been carried out, so it is not possible to quantify the effects of the different factors.

6.3.8. Selection Principles

Usually, protection of a given length of bank using spurs is less expensive than revetment. However, revetment is less likely to have unexpected effects on flow patterns. For example, revetment should be used upstream of intakes where spurs might deflect the flow away from the intake.

6.4. Choice of Materials

6.4.1. General Requirements

Scour protection measures are designed to protect the channel bed and banks from the erosive forces causing scour. They fall into two main categories: flexible and rigid systems. Flexible systems can cope with some movement without losing their armouring capability and so can adjust to settlement or movement of the underlying and adjacent surface or bed. Such systems are susceptible to failure from movement of the armour material, either because it is undersized or because of loss of material at its edges.

Rigid systems cannot adjust to changes in the underlying surface and are often impermeable. While nominally more resistant to erosion, they are susceptible to failure by undermining and uplift (seepage pressure).

Table 6-1: Typical Materials for Revetment

Revetment	Description	Advantages	Disadvantages
1 Gabion Mattress	300mm thick wire basket containing 150 - 200mm dia (5 - 10kg) stones on filter fabric	1 Flexible, can settle with bank 2 Can form lauching apron 3 Farmers can supply stone	1 Wires can break 2 Limited longevity
2 Rock Rip Rap	500mm thick layer containing 250 - 300mm dia (20 - 60kg) stones on filter fabric	1 Flexible, can settle with bank 2 Can form lauching apron 3 Requires machines for placement	1 Can be dislodged under high velocity 2 Too heavy to be supplied by farmers
3 Cemented Pitching	300mm thick layer containing 250mm dia (20kg) stones bedded in mortar and mortared joints	1 Good abrasion resistance	1 Not flexible, can crack and settle 2 Requires good compaction of 3 Difficult to be supplied by farmers 4 Needs toe to prevent undermining
4 Plain Concrete	300mm thick layer on compacted sub grade	1 High longevity 2 Good compressive strength	1 Not flexible, can crack and settle 2 Requires good compaction of 3 Difficult to be supplied by farmers 4 Needs toe to prevent undermining

Factors influencing materials choice include:

- construction cost
- underwater or dry construction
- availability of materials
- construction and maintenance constraints (for example access)
- channel stability laterally and vertically
- environmental considerations
- Potential for accidental or deliberate damage
- future maintenance costs and access.
-

The cost of the system is dependent on various factors, including availability of materials, such as rock, the length of haulage routes to the site, and the type of access available for construction. In general, the systems incorporating concrete are more expensive, unless there are long haulage distances for rock.

In general, the flexible systems can accommodate larger changes in channel stability than rigid systems, and are preferred where there is significant channel instability. The rigid systems are generally more resistant to surface erosion, so can provide good protection against high velocity and high turbulence.

All construction works are vulnerable to human interference. Materials may be removed for use elsewhere, thus endangering structural integrity. This risk is reduced if the size and weight of materials is too large for manual handling. It has been observed that gabion bank protection adjacent to villages is vulnerable to damage because rubbish is often thrown onto the channel bank and then periodically burnt. The fires can damage the wires and accelerate corrosion.

Another form of protection used successfully in southern Yemen is linked concrete precast slabs as shown in Figure 6-8. Each slab contains two diagonal reinforcing bars with hooks at each end. The ends of these bars are then jointed by welded steel loops which ensure integrity while providing moderate flexibility.

Figure 6-8: Linked Concrete Slab Protection

	
Overall view of linked concrete slabs	Joint detail of linked concrete slabs

6.4.2. Riprap

Riprap is the term used to describe loose quarry stone with a wide grading, laid as scour protection. It is one of the most versatile and commonly used types of revetment, as it can generally be readily sourced, easily placed and can be specified to suit particular flow conditions. It is flexible and can accommodate small ground movements and some loss of stones without failure. Suitably sized riprap is appropriate as protection up to very high velocities and turbulence. It can be used to protect banks with slopes up to IV: 1.5H, without requiring additional restraint. Because of the flexibility in the shape of the area that can be covered, it is useful for protecting small awkwardly shaped areas and transitions between hydraulic structures and natural channels.

Six main failure mechanisms for riprap can be identified from experience and research:

- hydraulic failure due to the size (in fact the weight) of individual stone being inadequate for the flow conditions, characterised by the scattering of riprap stone around the protected area and loss in thickness of the riprap
- winnowing failure caused by erosion of the underlying bed material through the voids of the riprap, due to failure or omission of filter layers, from inadequate riprap thickness due to under-design or poor placing or as a result of poor grading of the rock, characterised by the stones being submerged within the bed of the channel
- edge failure due to the erosion of a scour hole in the natural bed adjacent to the protection, with stones at the outer edge of the riprap falling into the hole and leading to progressive failure, characterised by scour around the protection and loss of riprap around the edge of the protection
- bed movement undermining, where significant natural scour takes place, if riprap is placed on or at the original bed level - this type of failure can appear similar to winnowing failure, although more extensive movement of stone usually occurs laterally. Sloping riprap can suffer from two further failure mechanisms:
- translational slide of the riprap down the slope, which normally occurs if the angle of the slope is too steep or if the toe of the riprap has not been keyed in adequately - where riprap is laid on geotextile there may be less friction between the rock and underlying soil, thus increasing the risk of sliding
- rotational slip failure of the soil mass beneath the riprap owing to an unstable slope. Apart from slide and slip failures, collapse tends to occur gradually, allowing time for repairs to be carried out, provided that the failure process is observed early enough.

6.4.3. Riprap Sizing

Many formulae have been proposed for sizing riprap and, like scour estimation, designers have been faced with several possible solutions which may give greatly differing results. Nevertheless, designers have to make decisions based on the best available guidance. Research on stability has shown that the main parameters affecting the stability of riprap are:

- flow velocity
- flow conditions (degree of turbulence)
- stone properties (density, shape)
- the location of the riprap (bed or banks).

When sizing riprap for a scour protection system, the worst case conditions in terms of water depth and flow velocity should be established. During the design flood, the main incised channel tends to increase its cross-sectional area as a result of natural and contraction scour, leading to a reduction in flow velocity for a given value of discharge. For design purposes, the riprap should be sized on the assumption that the discharge in the design flood may initially occur while the channel still has its "normal" or long-term cross-sectional area; this is likely to be more severe than the condition that will apply later in the flood, when scouring of the channel may have temporarily increased its cross-sectional area towards the regime value corresponding to the design discharge.

USBR[10] recommends the following formula for determining the size of stone that will not be dislodged under turbulent flow conditions:

$$D_{50} = (V_{av} / 4.915)^2 \quad \text{(turbulent flow conditions) [metric units]}$$

Where:

V_{av} = average velocity of flow for maximum discharge [m/s]

D_{50} = average stone size [m]

The specific gravity of the stones was assumed to be 2.65 (ie density of 2,650kg/m³). If less dense stone is used, then the stone size should be increased correspondingly.

For low-turbulent flow conditions, such as exist along the shank of a flood protection bund, the required stone size will be less than that given above. A reduction in the D50 stone size of 40% is acceptable.

The grading of the stone pitching should be as follows:

- Maximum stone size = 1.5D50
- Minimum stone size = 0.5D50

Not more than 40% of the stone should be smaller in size than D50

6.4.4. Filter Design

Stone protection placed on embankments should be laid on a filter layer to prevent piping. When one filter layer is sufficient it is called a “graded filter”. When more than one filter layer is used, the coarser filter is placed on top of a finer filter (ie the permeability increases outwards), and the filter is called an "inverted filter".

The gradation of a graded filter should conform to the following guidelines established originally by Terzarghi:

d_{15} filter / d_{85} soil < 5;

d_{15} filter / d_{15} soil > 5; and

d_{50} filter / d_{50} soil < 25

Where d_{85} is the sieve size which will pass 85% of the material, and similarly for other percentages (d_{15} and d_{50}).

10/ USBR “Hydraulic Design of Stilling Basins and Energy Dissipators” United States Bureau of Reclamation. 1983.

The above criteria relate respectively to:

- stability (ie preventing the movement of soil particles into the filter);
- permeability; and,
- uniformity.

If this cannot be achieved with a single filter layer, then two layers shall be used, where the upper layer of the filter is designed using the above criteria, where the soil parameters are replaced by the parameters relating to the filter below.

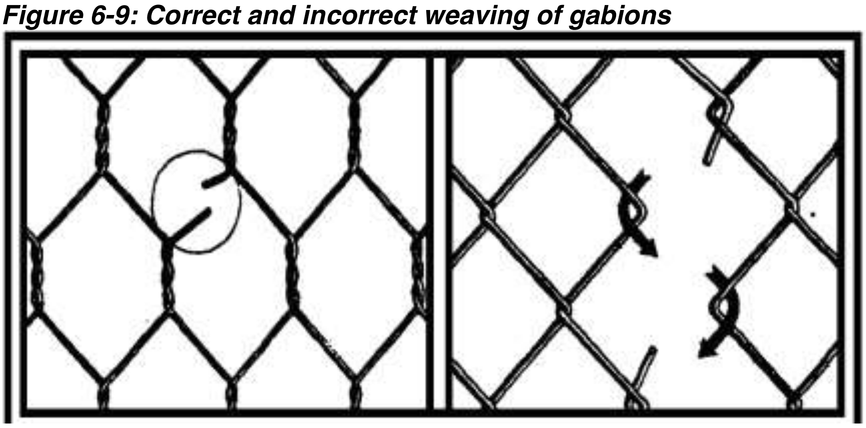
Geotextiles are increasingly used instead of filters. Care must be taken to ensure that the geotextile has an appropriate strength for the conditions and does not lose strength if exposed to sunlight for long periods.

6.4.5. Gabion Mattresses and Boxes

Gabions are wire mesh containers filled with stone. The flexibility of the mesh allows the containers to deform to the bed profile, while preventing the stone contained within from either shifting to expose the bed or from being removed from the revetment. Enclosing the stone within the mesh allows smaller sized stone to be used. Research comparing gabions with riprap also shows that a thinner revetment layer can be used typically to two thirds of those required for riprap. In gravel and cobble bed rivers, where abrasion may be a problem, the larger diameters of wire available (typically 3.0mm) can be used. However, in highly abrasive conditions a concrete coating is necessary to protect from damage.

Local manufacturers in many countries can produce gabions, and the mesh can even be hand woven. However the quality of locally produced mesh can be variable and hence a careful specification is needed to ensure that the appropriate quality is supplied. The gabions need to be woven correctly as shown on Figure 6-9 to avoid unravelling should a wire break. A wide range of sizes and shapes of gabion is available to suit different applications. Gabion boxes are normally produced with dimensions between 0.5m and 2.0m.

Figure 6-9: Correct and incorrect weaving of gabions



Gabion mattresses are more flexible than gabion boxes because they are thinner and because smaller stones and thinner wire are used in their construction. Due to their flexibility, mattresses are probably the most commonly used form of gabion for erosion protection. Mattresses are available in a range of sizes but typically have a thickness of 0.2m or 0.3m. Where a protection thickness of 0.5 m or more is necessary, gabion boxes may be preferable to mattresses because of their greater strength. However, if additional flexibility is also needed, for example in a falling apron, the required thickness can be obtained by placing two mattresses on top of each other but not tied together. Gabions are usually filled in situ by hand, although machine filling, particularly of gabion sacks, is possible. Careful construction is the key to a robust and successful gabion protection system.

There are several failure mechanisms for gabions that should be borne in mind; they can normally be avoided by good construction practice:

- failure of mesh leading to loss of stone from compartments within the protection system - causes include corrosion and abrasion, vandalism and theft, and damage during construction

- edge failure, due to the erosion of a scour hole in the natural bed adjacent to the protection - although mattresses can accommodate significant movement, excessive movement can lead to mattresses breaking or being undermined
- excessive movement of stone within compartments. High flows will usually cause some slight displacement of stones towards the downstream end of each compartment. However, the amount of movement can become excessive if the velocities exceed those designed for, if the stone is poorly packed, or if the partitions forming the compartments are not spaced closely enough. The underlying material may then become exposed to current attack and, in extreme cases, the mesh may fail due to the additional stresses imposed on it.

Gabions are normally placed on a filter of either geotextile or granular material to prevent loss of the underlying material through the gabion voids.

6.4.6. Concrete Blocks

Concrete block revetment comprises a single layer of precast concrete blocks laid on a geotextile or granular filter. The blocks may be cellular, with up to about 20 per cent of their plan area open, although solid blocks are also manufactured. They may take the form of individual blocks that interlock with adjacent blocks (interlocking blocks), or they can also be linked into a mat using cables running through the blocks (cable-tied blocks). Large blocks can be joined by welded links formed of bent steel bars (see Figure 6-8)..

6.4.7. Non-Structural Protection

Alternative methods of protection which may be considered including:

- Tamarix or other local bushes along the toe of the protection bund or along a channel bank between spurs
- stone spurs with tamarix or other local bushes planted in them

Encouragement of vegetation growth will then provide some natural protection in the event of long-term decay of the engineered infrastructure.

6.4.8. Bank Protection using Alternative Materials

For small channels good quality hollow concrete blocks placed with the openings facing outwards and the voids filled with sand or gravel may be effective. Similar, specially manufactured hollow revetment units are manufactured in Europe. Such materials offer the advantage of enabling vegetation growth while providing stability. A further form of revetment is to use old vehicle tyres, which often form a disposal problem. Bales of old tyres have been used in Europe for bank stabilisation. A similar result can be achieved by placing tyres horizontally in staggered layers and the voids well filled with gravel and soil.

6.4.9. Design of Works

The information in this section guides the planning of the overall layout of the works. Detailed design of works must take account of other parameters such as providing sufficient conveyance capacity for floods (see Section 5.2.3) and scour (see Section 8.10.4).

7. SEDIMENT MANAGEMENT

7.1. Introduction

Sediment is a fundamental feature of most spate irrigation schemes. It is both a benefit (deposited silt creating fertile land with good water-holding properties) and a problem (clogged canals and structures, rising fields and command problems). Development of a strategy for managing, or living with, the sediment is the first requirement in planning work on a spate irrigation scheme.

7.2. Sediment

7.2.1. Sources of Sediment

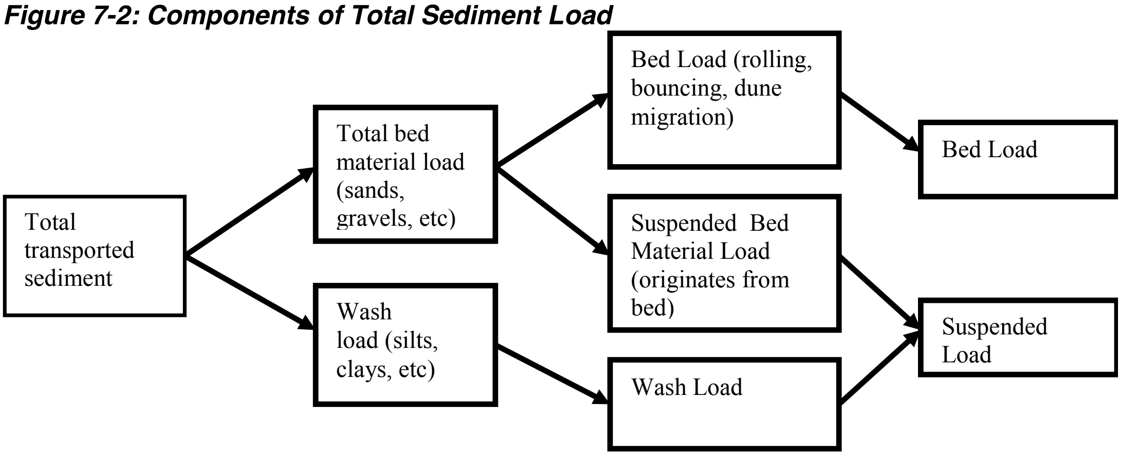
The sediment carried by floods is material washed off the catchments or scoured from upland channels during heavy rainfall. One source is fine sediment directly washed from the soil by the direct impact of rain or the sheet flow of water over the surface. Bare surfaces with little vegetation cover are the most vulnerable to this erosion. Coarser sediment is washed off wherever the flow becomes more concentrated. The amount of sediment is therefore very sensitive to catchment condition including factors such as soil type, vegetation cover and slope. The channel conditions upstream of a point of interest are also important as shown in Figure 7-1.

Figure 7-1: Sediment Upstream of Weirs

Engulet: Western Eritrea. Moderate slopes upstream of weir - fine sediment	Wadi Mai Ule: Eastern Eritrea: Steep slopes upstream of weir - coarse sediment

Once the fine sediment (called “washload”) is in suspension then it remains in suspension because it takes some hours of standing in still water to settle out. Coarser material may be in suspension or is moved along the bed (“bed load”) and can quickly stop moving if the flow velocity reduces. The various components of total sediment load are shown on Figure 7-2.

Figure 7-2: Components of Total Sediment Load



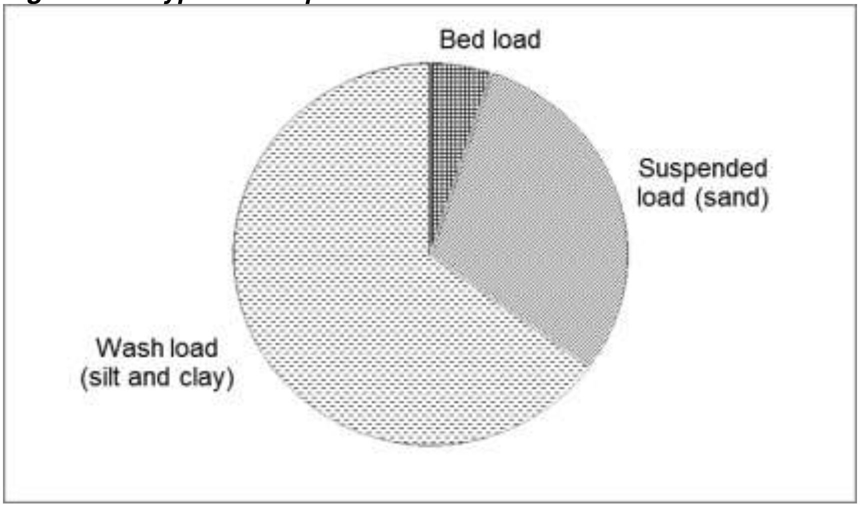
7.2.2. Measurement of Sediment Loads

The limited information that is available on sediment measurements for floods suggests that:

- Total load sediment concentrations rising to and exceeding 100 000 ppm, or 10% by weight can occur in floods in some wadis. Sediment concentrations up to 5% by weight in floods are common. Sediment runoff will be influenced by catchment conditions (slope, soil type and land use) as well as rainfall intensity.

- Sediment transport is dominated by the finer sediment fractions. The proportion of silt and clay (material finer than 63 microns) in the sediment load varies widely during and between floods and between catchments but typically ranges between 50 and 90% of the total annual sediment load. The load of sand and coarser material is influenced by the channel conditions upstream of the diversion point. As the slope and velocity reduces then this material will be deposited. The fine sediment concentrations are 'supply controlled' and do not correlate well with wadi discharge. Figure 7-3 shows the typical composition of different types of sediment load.

Figure 7-3: Typical Composition of Total Sediment Load



The bed load and suspended sediment grading curves (excluding the wash load) measured for two wadis in Yemen and Eritrea are compared on Figure 7-4. The higher proportion of coarser bed material for Wadi Laba is probably the result of the steeper (>3%) channel slope upstream of the sediment measuring station.

Figure 7-4: Example Bed and Suspended Bed Load Grading Curves

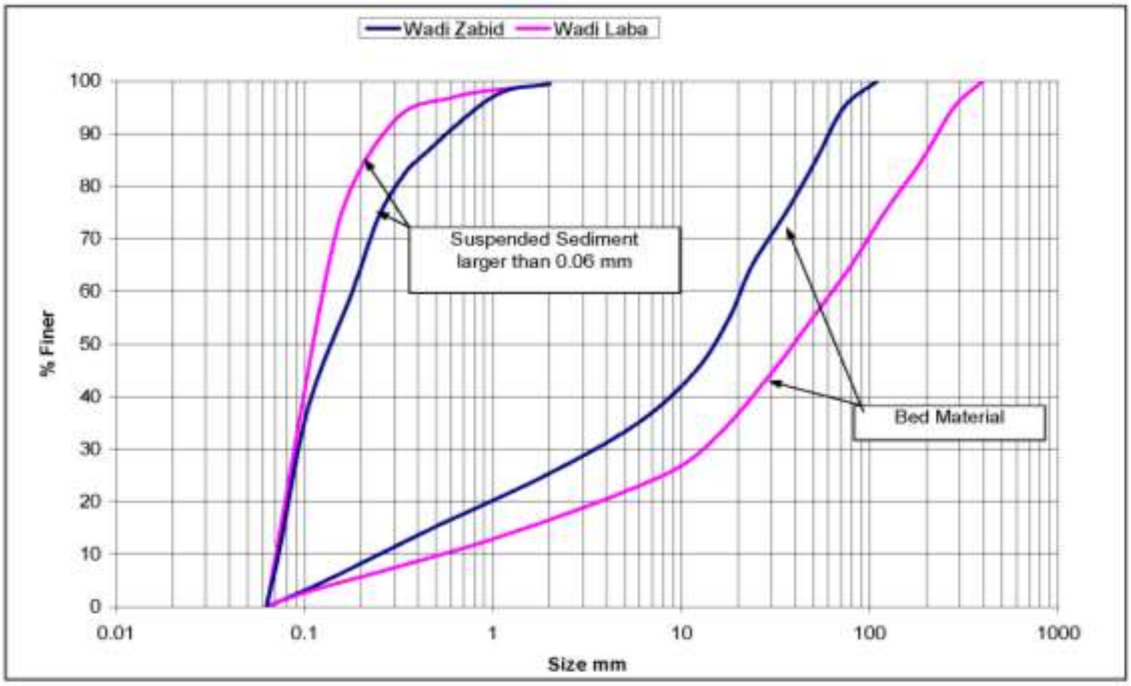
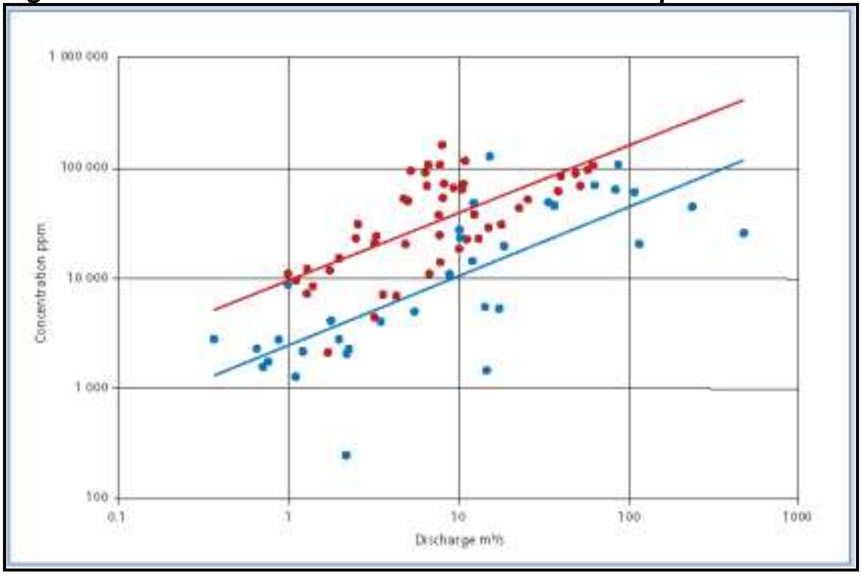


Figure 7-5 shows fine sediment concentrations measured in for two spate rivers in Baluchistan and Eritrea. It can be seen that the sediment loads increase substantially as the flow increases. Fine sediment is likely to be most concentrated early in the flood season when the source catchment has less vegetation cover.

Figure 7-5: Fine Sediment Measurements For Two Spate Rivers



The sand load transported in suspension in wadi flows, most of which will be diverted to canals even at well-designed intakes, is also relatively fine (generally between 0.1 and 1 mm) when compared with the parent bed material. Estimates of the sand load can be derived from empirical equations (see Section 7.2) but should be supported, wherever possible, by measurements of the sand load during floods.

Coarse sediments transported near the wadi bed by rolling and sliding represent only 5% or so of the total annual sediment load. Sediments of this size range from coarse sand, through gravel, to cobbles and in some cases boulders. These settle and block intakes and canals. Estimates of bed load sizes and concentrations are needed to design sediment control facilities where these are included in larger intake structures. The estimates are usually derived from empirical equations and much of the coarse bed load may only move during the larger floods. The extent of bed movement can be monitored using methods such as vertical chains buried in the wadi bed. Excavation after a flood can reveal the extent of bed movement as shown by movement downstream of the upper part of the chain. During a high flood the bed may scour significantly with deposition as the flood recedes.

In the medium term, sediment tends to be deposited in the wadi channel during the small and medium floods and will then be eroded and carried downstream during the large floods. This has implications for the design of any structures in the wadi channels because the bed level may change significantly.

7.2.3. Sediment deposition in canals

Sediment deposition in canals is, as a minimum, a nuisance and can be a major problem resulting in substantial reduction in flow capacity. Any very coarse sediment (gravel or larger) will tend to be deposited in the very first section of canal while the deposition of sand can be spread along the canal depending on slopes and velocities. Ideally, intakes should be closed during the peak of large floods in order to keep the coarse sediment out of the system, but the desire to divert water means that this often does not happen. However, the consequence can be a choked head reach on the canal that becomes a major constraint to the flow.

Figure 7-6: Sediment in main canal head reaches

	
Engulet main canal: The slope is relatively flat and the canal is choked with fine sediment	Wadi Laba gravel trap: The channel is full of gravel but the extra width provides additional flow capacity

7.2.4. Sediment Deposition in Fields

The finer sediment entering the irrigation system will normally pass through to the fields unless canal slopes are very flat or control structures in the canal system cause substantial reductions in velocity. The extent of sediment deposition depends on both sediment load and the depth of irrigation applications but can average about 2 cm to 5 cm per year, or 20 cm to 50 cm in 10 years. The sediment deposition has two main consequences, one good and one bad:

(a) **The Good Consequence:** The deposited sediment, usually predominantly soil with nutrients, can be fertile material with good water holding capacity. It is often better for crop production than the pre-existing natural soils. This is a key feature of many of the spate irrigation systems around the Red Sea, where the natural soils are very sandy but are now covered by several metres of good silty soil with excellent water-holding properties and the annual resupply of fertile sediment reduces the need to use any artificial fertilizers.

The land build-up is an inherent characteristic of spate irrigation schemes and can be used for overall scheme development. The first step is to use the deposited sediment to form terraces that are easier to irrigate than the natural sloping land. This process can take some years but the cost is low.

Figure 7-7: Examples of Spate Irrigated Fields

	
A cascade of terraced fields in Wadi Zabid	Contour map in Sheeb scheme showing up to 6m of land build-up in 100 years around old village site

(b) **The bad consequence:** The progressive rise in the field levels necessitates a progressive rise in the associated infrastructure such as field bunds, canal earthworks and structures, between-field drops and drainage structures and eventually any diversion structures. This problem should not be under-estimated during the design of permanent infrastructure.

The adverse effects of field rise can be mitigated by designing structures, where they are needed, to accommodate future rises in level. For example, pipes become buried but open channel structures can be easily raised. Alternatively, low cost structures which have a short working life can be used.

Figure 7-8: Examples of Irrigation Structures Affected by Sediment Deposition

	
Wadi Zabid: Canal piped offtake unable to command fields and becoming buried	Wadi Zabid: Traditional drop structure showing at least three increases in height

7.3. Estimation of Annual Sediment Loads

7.3.1. Introduction

Estimates of annual sediment loads transported by wadis are needed to enable the quantities of sediment diverted to irrigation systems to be determined. Overall catchment sediment yield can be estimated using regional values for tonnes of sediment per unit area of catchment. Background discussion of sediment has been presented in section 7.2. This section addresses the problem of estimation of sediment quantities which needs to take account of specific local conditions. The annual quantity of sediment carried by the wadi at the proposed diversion point needs to be estimated based on the catchment sediment runoff. The proportion of sediment is normally split between material coarser and finer than 63 microns (the silt and clay). The latter is in suspension and will not settle out until the water is ponded. Several different methods can be used to estimate the annual sediment runoff and these are discussed below.

7.3.2. Using parameters based on regional data

Estimates were made by Euroconsult[11] of sediment runoff for some Eritrean catchments based on sediment sampling and evaluation of reservoir sedimentation. The predictive equation proposed for estimating sediment yields was:

$$Y = EHI * A^{-n1} * Runoff^{\,n2} * K$$

Where: Y = Sediment yield Tonnes/ km²
A = Catchment Area, km²
EHI = Erosion hazard index, tabulated below
Runoff = Annual runoff mm (typically 5% to 10% of rainfall - see section 5.2.5)
n1 = a constant
n2 = a constant
K = a constant

Erosion hazard class	Expected erosion rate range t/ha/year	Erosion hazard index (EHI)
Low	1 to 15	1
Moderate	15 to 50	7
Severe	50 to 100	18
Very severe	> 100	35

The coefficients n1, n2 and K were derived using an iterative optimisation procedure with the long term sediment yields estimated for the river gauging sites. This produced the following values:

n1 = 0.25 n2 = 0.97 K = 11.8

[11] *Euroconsult 1998, Sector Study on National Water Resources and Irrigation Potential. Annex 5 - Sediment Studies*

The selection of the erosion hazard class is based on desk study and field observations to take account of slopes, soil type and vegetation cover (including the extent of any soil conservation measures). Ideally, the catchment should be evaluated at the end of the dry season when vegetation will be at a minimum and the soil most vulnerable to erosion from heavy rainfall.

Using this method the sediment runoff for Wadi Laba in Eritrea can be estimated as:

- Catchment area = 638 km^2
- EHI = 18 = severe on account of steep slopes and limited soil conservation measures
- Annual runoff = 86 mm (from total estimated runoff / catchment area)

$Y = 18 * 638^{-0.25} * 86^{0.97} * 11.8 = 18 * 0.20 * 75.24 * 11.8 = 3{,}196$ tonnes/km^2.

This is in the same range as the estimate of 3,757 tonnes/km^2 calculated using field measurements of sediment load during floods.

7.3.3. Using Evaluation of Catchment Condition

For smaller catchments it is possible to estimate the sediment runoff based on a field assessment of the catchment condition using the procedure described in Annex B.

7.3.4. Estimation of Sediment Entering the Canal System

The proportion of the total sediment load that enters the canal system will depend on two main factors:

- The effectiveness of any sediment exclusion at the intake
- The extent to which the high floods (containing a larger proportion of the coarser material) are restricted from entering the canals.

The calculation presented in Table 7-1 provides an example of estimation of coarse sediment (ie larger than 0.063mm) entering the canal system. The calculation is based on the large catchment flow-duration curve presented in Section 5.2.6 and assumes that mean annual flood peak (Qmax) is 200 m^3/s and the total flow duration (Ttot) is 100 hours but the limiting flow to the canal is 50m^3/s. The relationship between coarse sediment load (X) and flow (Q) is assumed to be $X = 80 * Q^{1.2}$.

Table 7-1: Calculation of Coarse Sediment Entering Canal System

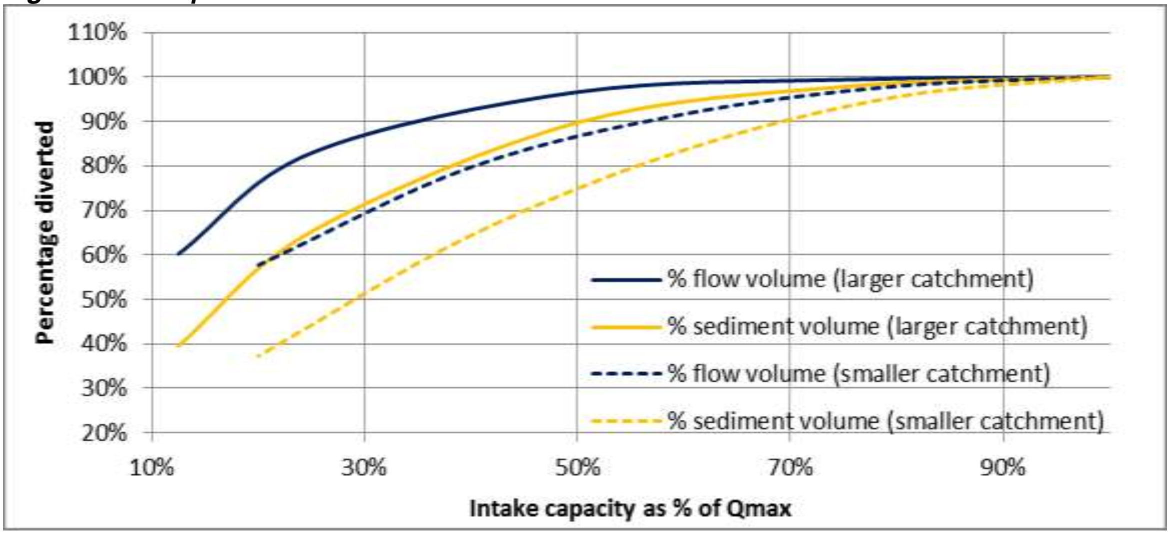
Q wadi (m³/s)	Q canal	T (hours)	Average Q canal	Incremental time	Incremental Flow Volume	Sediment Concentration	Sediment Load
200	(m³/s)	100	(m³/s)	(seconds)	(m³)	(ppm)	(tonnes)
0	0	100			0		
10	10	65	5	126,000	630,000	1,268	799
22	22	40	16	90,000	1,440,000	3,266	4,703
40	40	19	31	75,600	2,343,600	6,692	15,684
60	50	9	45	36,000	1,620,000	10,886	17,635
80	50	4.8	50	15,120	756,000	15,374	11,623
100	50	2.5	50	8,280	414,000	20,095	8,319
120	50	1.5	50	3,600	180,000	25,010	4,502
140	50	0.9	50	2,160	108,000	30,091	3,250
160	50	0.5	50	1,440	72,000	35,321	2,543
180	50	0.2	50	1,080	54,000	40,683	2,197
200	50	0	50	720	36,000	46,166	1,662
					7,653,600		72,917
					82.9%		65.2%

The calculation assumes that the intake has a maximum capacity of 50 m^3/s (25% of Qmax) and will take all the wadi flow up to this threshold. It also assumes that the sediment concentration in the canal flow is the same as that in the wadi. Under this operating condition the intake can divert 83% of the total flow but only receives 65% of the coarse sediment. The volume of water diverted (7.6 MCM) would be sufficient to irrigate

760 ha with a gross irrigation application of 1m. In reality, the flow through the intake would increase during the period of highest flow, resulting in an increase in both the flow volume and sediment ingress, unless the gate is operated to restrict the flow.

Figure 7-9 shows the proportions of flow and sediment diverted to the canal for different intake capacities based on the flow-duration curves for the larger and smaller catchments as presented in *Figure 5-5*. As noted above, the intake for a smaller catchment will need to be a larger proportion of Qmax to divert the same proportion of the flow volume. However, the generally lower flows mean that the sediment loads tend to be less.

Figure 7-9: Proportions of Diverted Flow and Sediment



This calculation does not include the fine sediment. This is likely to be in the range of 100% to 300% of the quantity of coarse sediment depending on the catchment condition. The quantity of fine sediment is controlled by the potential for erosion of the catchment and not the ability of the wadi to transport the sediment.

The amount of coarse sediment entering the canal system can be further reduced by providing a sediment exclusion facility at the intake which may reduce the intake of coarse sediment by, typically, 25% to 50%. This is discussed further in Section 7.4.2. Locating the intake on the outside of a shallow bend will result in a below-average proportion of the bed load approaching the intake. However, the flow remaining in the wadi needs to be sufficient to carry the remaining sediment away from the diversion structure. Otherwise a shoal of excluded sediment may develop downstream of the intake and adversely affect the flow and the channel morphology.

7.4. Sediment Management Options

7.4.1. Selection of Options

If evidence points towards a significant sediment problem (and it is highly unlikely there will not be) then deciding how it will be managed will influence the type of intervention that can be made. The coarse sediment load increases with increasing flow while the silt and clay washload can only be controlled by catchment management. There are various options for managing sediment and these are summarised in Table 7-2. In all cases the finer sediment (fine to medium sand as well as silt and clay) should pass through to the fields provided that the canal structures do not cause ponding of water.

Table 7-2 : Sediment Management Options

Intake/scheme type	***Sediment management strategy***
Type A. Basic intake without a weir	• Locate intake at the outside of a channel bend • Limit flows entering canal with flow throttling structure • If provided, close gates during periods of very high wadi flows • Provide canals with sufficient slope to keep the medium to fine sand component of the coarse sediment moving, minimise ponding and use flow division • Make provision for rising command levels

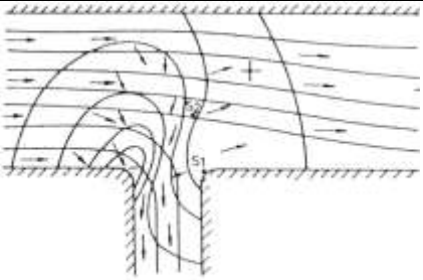
Intake/scheme type	Sediment management strategy
	• Consider arrangements for and sustainability of canal cleaning or period bank raising to accommodate sediment deposition
Type B. Basic (probably small) intake with a low weir	As for type A plus: • Provide skimming weir in front of intake • Provide a simple sediment sluice, possibly gated or provided with earth embankment to breach during larger floods • Align canal intake to minimise flow diversion angle • Consider if mechanically excavated gravel trap is appropriate
Type C. Higher cost intakes	As for type A plus: • Incorporate a sediment sluice, consider curved channel sediment excluder if bed sediments are coarse. • Align canal intake to minimise diversion angle • Consider if mechanically excavated gravel trap is appropriate, or whether flushed settling basin might be feasible • Where high investments costs might be justified by reduced de-silting costs, consider hybrid vortex extractor /settling basin system located in the canal head reach

Selection of the appropriate intake type will be determined by a consideration of site conditions, potential benefits and cost. Multiple simple intakes may provide a better overall solution than a single high cost structure.

Box 7-1 : Avoid Right-angled Intakes

FAO's 2002 irrigation design manual, (Volume 2 module 7 figure 39), specifically recommends right angled intakes for silt laden rivers. However, physical and numerical models and field experience all demonstrate that **frontal intakes divert the minimum of bed load** to canals and right angled intakes increase the amount of sediment entering the canal.

This is because the lower momentum of flow near the bed of a channel makes it more easily divertible if the flow direction is changed. In spate intakes the angle of diversion is only relevant during high flows, when water is passing through the sluiceway or over the weir. In addition, sudden changes in flow direction cause vortices which can lift bed load into the flow.



The arrows show the direction of the flow near the bed.

7.4.2. Sediment Excluders

Sediment excluders can be of several different types including skimming weirs, tunnels and undersluices. All rely on a flow of water to sluice the bed load, while the remaining flow enters the canal. Tunnel type structures are not advisable where the bed load contains gravel or larger material because there is a significant risk of blockage. Skimming weirs are a proven effective solution for spate conditions with a relatively narrow angle of divergence between the canal flow and the sluice flow. One objective is to minimise any causes for vortices which will cause the bottom flow to rise. The skimming weir may be a simple straight wall that directs the bed load towards the sluiceway or part of a more sophisticated curved channel sediment excluder.

The curved channel excluder relies on artificially creating a bend where the helicoidal flow will move to the bed load to the inside of the bend, away from the intake) and the cleaner surface flow will move towards the intake. The basic layout of the curved channel excluder is shown in Figure 7-11. Model testing has demonstrated that the excluder performance will be more effective if the curved channel has slight convergence to help accelerate the sluicing flow towards the sluiceway. The curved channel is created by provision of a guide pier. However, the extent to which a sophisticated structure can be justified depends on site conditions. A large and robust guide pier is a significant extra cost. Performance is more likely to be effective in locations where the predominant wadi bed material is sand and the approach flow is not turbulent.

In many cases the "loss" of water for sluicing is not acceptable to the farmers. A compromise can be to only undertake sluicing when the wadi flows exceed a pre-determined threshold and the bed load becomes more substantial. However, this creates the operational problem of when and how quickly to operate the sluiceway. One possible solution is a float-operated gate which is triggered by the upstream water level. However, float mechanisms are vulnerable to debris and, ideally, the gates are opened clear of the water during large floods. An alternative is a counter-balanced gate mounted on a vertical spindle as shown on Figure 7-12. This operates like a door which opens when the flood is high.

Figure 7-10: Examples of Skimming Weirs

Figure 7-11: Typical Layout of Curved Channel Excluder

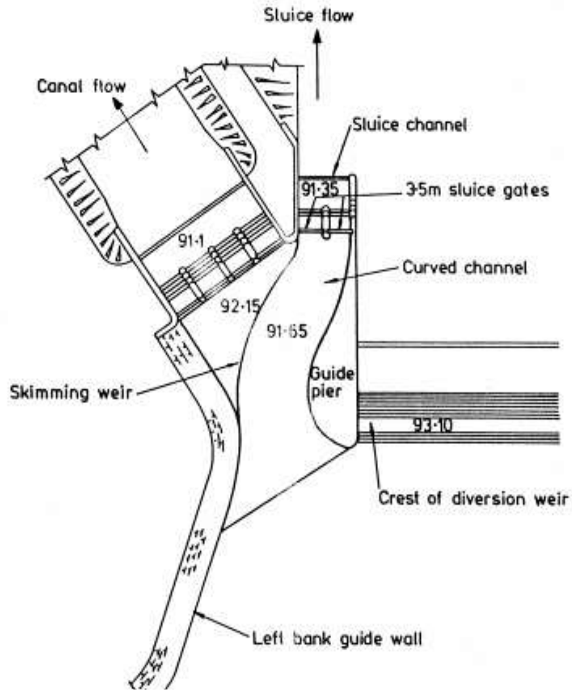
A typical curved channel excluder has the axis of the canal intake at 30° to the axis of the river.

A guide pier is provided to create an artificial bend and a skimming weir (typically 0.6m to 1m high) is placed in front of the canal head regulator to form the outer edge of the curved sluice channel. This arrangement is more effective where the predominant bed load is sand.

The weir is built high enough to command the canal and provide the driving head through the sluiceway (note that the worst design case may be during large floods when the tailwater level is high).

Placing the weir at the upstream end of the guide pier will enable some flushing of sediment from the upstream site of the weir.

Curved channel excluders are less effective has the approach velocities and sediment size increase because the helicoidal flow will have less chance to develop.



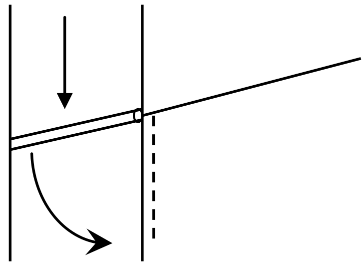
At small intakes it may be sufficient to use an earth or gravel bund in the sluiceway which will breach during large floods. However, it will be a challenge for farmers to close the sluiceway during the flood recession so water will pass downstream. This may be considered acceptable if there is another intake which will use the flow.

The use of tunnel sediment excluders is not recommended for spate irrigation because they need a continuous flow to avoid blockage by finer sediment and may block with coarse sediment during high floods. Open channels are preferable for ease of maintenance.

Figure 7-12: Hinged Sluiceway Gate

Example of a hinged gate: This solution is feasible for sluice channels up to about 1.5m wide. The gate is mounted on a vertical spindle (recessed into the wall on the right side of the photo) and as the upstream water level rises it will gradually push the gate open until the gate is contained in a recess in the wall.

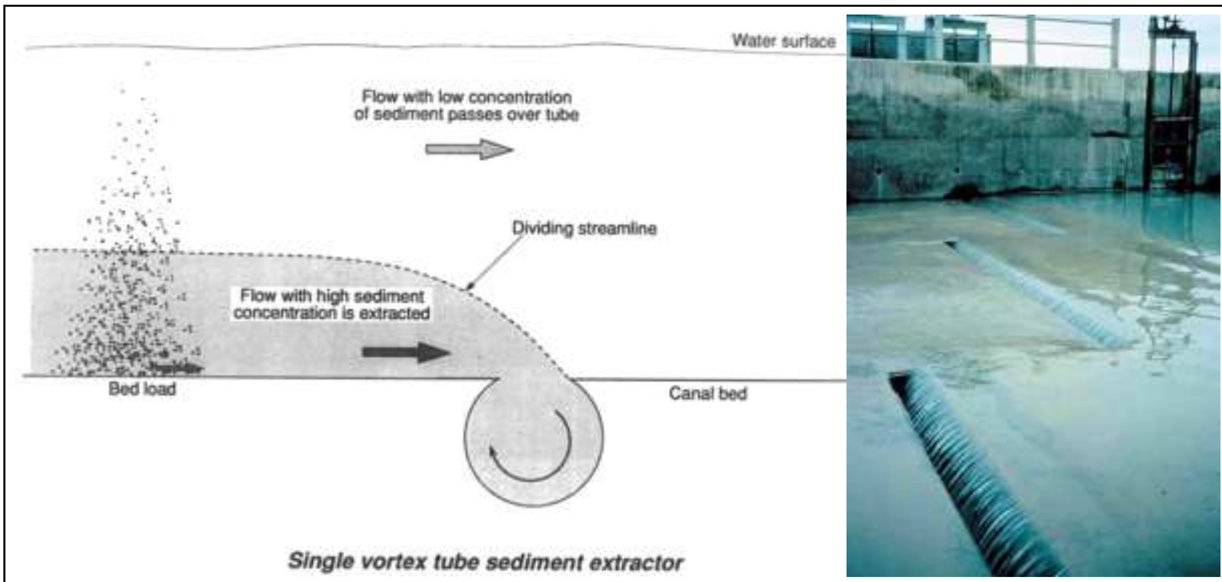
The gate is normally held closed by a counterbalance (with weights to be adjusted to achieve satisfactory operation) but once open will be held in place by a ratchet mechanism to stop the gate swinging in the flow. A long lever arm is provided to enable the operator to close the gate when the water level has reduced.



7.4.3. Sediment Extractors

Bed sediment entering the canal system can be removed by a vortex tube sediment extractor. This is a tube buried in the bed of the canal with an open slot at the top. The bed flow in the canal drops into the tube which then drains sideways. On wider canals several tubes may be required, each serving part of the width. The solution is proven to be effective at intercepting sand bed load but uses 5% to 10% of the flow in the canal.

Figure 7-13: A Vortex Tube Sediment Extractor



Where the water cannot be spared and there is sufficient head drop in the canal, the vortex tube can discharge into a sediment basin where the sand is deposited and the water returns to the canal. This basin can then be cleaned by intermittent flushing.

7.4.4. Sediment Trapping

The sizing of sediment basins has to be a compromise because if they are too small then they fill up before there is an opportunity for cleaning, and if they are too large they will trap sediment that would otherwise pass through the system.

While mechanically cleaned sediment basins work successfully on perennial irrigation schemes, experience indicates that such sediment basins are unlikely to be successful in spate systems due to the erratic and uncertain nature of the canal flows (see Box 7-2). It is usually assumed that any basins are cleaned either by bulldozers or by loaders + trucks. However, it is not practical for this equipment to work in wet conditions so any flow has to be stopped and standing water drained off. This may not easy be easy to carry out within the context of unpredictable floods. Furthermore, if the trapped material contains a significant proportion of fine material, then it may have to dry out before equipment can enter the basin.

Box 7-2: Gravel Trap at Wadi Laba

> The new diversion works for Wadi Laba in Eritrea included a gravel trap in the head reach of the main canal. The design assumed that the gravel trap would be cleaned as needed.
>
> In early July 2003, two relatively large floods filled the gravel trap. However, although a bulldozer was available, conditions never became dry enough during the flood season for the bulldozer to operate. The large accumulation of gravel both caused flow to spill back into the wadi and caused blockage of a culvert passing below the wadi.
>
> The lesson is that excavated gravel traps and sediment basins are only workable if suitable equipment is provided that can clean them during flow conditions. Flushable basins do not have this problem but may not be able to handle very coarse sediment.

7.4.5. Sediment Flushing

Flushed sediment basins have proved to be effective but are very expensive to construct (see Box 2-13 for an example). Two parallel basins and associated gates are normally provided so that one basin can be flushed while the other is passing flow to the canal. The basins are normally concrete lined to facilitate the flushing. Flushing requires a substantial amount of flow at intervals whenever a basin is substantially full. If flushing is not permitted by the farmers due to possible loss of irrigation water then either mechanical cleaning has to be carried out (with the risk of damage to the concrete lining) or sediment will pass down the canal system, which is not designed to transport the high sediment load.

7.4.6. Sediment Deposition and Land Accretion

Farmers generally welcome the fine sediment as it is believed to improve fertility and will add to the thickness of soil with good water retention properties. The rate of field level rise varies between and within schemes and can range between about 5mm and 50mm per year. The deposition is usually greatest adjacent to the head reaches of canals where farmers get water most frequently. The silt and clay are in suspension and cannot be removed from the water until it is ponded. During high floods the canal water may also contain substantial quantities of sand which will settle more easily. The deposition of sediment causes a progressive rise in field levels that will eventually reduce the difference in water level between the source and the fields such that the flow rates are reduced.

The best guide to the rate of rise in existing schemes is to estimate the depth of sediment on existing fields and the period for which they have been irrigated. Village sites which are usually on the original land can provide a convenient datum (see Figure 7-7 for an example). Improvements to irrigation diversion efficiency may increase the rate of rise if more irrigation water is applied. For new irrigation schemes the best reference is existing irrigation schemes with similar catchment conditions.

Farmers who realise that their land is becoming unirrigable because of sediment deposition have limited options for action. Such fields are most likely to occur on the head reach where the water contains a greater proportion of sand.

The usual approach is to block the canal until it is ponded to a sufficiently high water level to irrigate the land. This can both reduce the flow through the intake and cause sediment deposition in the canal, to the detriment of all farmers served by the canal. Where the intake is a traditional structure then it can be relocated upstream at relatively low cost, but this is not a viable option for a modernised structure.

Another approach is to not pond water on the fields, but allow the water to pass over as a sheet flow. Some water will percolate although much of the sediment will be transported by the remaining flow. This practice is most attractive with field-to-field irrigation where high fields will impede flow passing downstream. Ideally, the canal users, as a group, will be proactive in requiring that irrigation of the highest land is undertaken in a way that minimises sediment deposition and a worsening of what is already a problem.

A third approach is to look for a different source of water for the high fields, such as from another, higher canal. This will require negotiation between farmers and groups of canal users. No farmer will like to be relegated from being at the head of one irrigation canal to a lower position on another canal even though the change benefits everyone else.

A final approach to high fields is to remove some of the soil. This will usually incur a cost but may sometimes be combined with raising canal banks for which a source of material is required.

7.4.7. Progressive Rise of the Canal System

In traditional spate schemes, the canal system progressively rises in harmony with the rising field levels. The typical regime cross section for a canal conveying a high sediment load is a wide, but shallow, channel, which provides substantial capacity for holding deposited sediment. Some of the deposited material can be used for raising the banks, which will need to be carried out as annual maintenance in order to retain capacity and freeboard. The banks are easy to construct or raise because the canal is shallow and in some cases the canal is effectively a route through a field. The longitudinal profile for a canal in regime will also enable sediment to be distributed along the length of the canal.

Figure 7-14: Examples of Traditional Canals

	
This log deposited in a field is the only indicator that it is also a canal	Traditional canal used for growing crops after the flood season

The traditional solution to the command of rising field levels is to move the canal intake further upstream. A permanent diversion structure does not offer this flexibility although a modest raising may be possible without compromising structural safety. The design of permanent diversion or canal structures should allow for an appropriate rise in field levels (typically 30 to 50 years accumulation is in the range 0.3m to 2.5m depending on the amount of irrigation water supplied and the concentration of fine sediment in the water).

Further discussion about the design of canals is presented in section 9.

8. INTAKES AND DIVERSIONS

8.1. Diversion Strategy

8.1.1. Diversion Objectives

The basic objective of spate irrigation is to capture the spate flow and spread it onto farmland before it goes to waste. The meaning of “waste” in this context means either evaporating or flowing to the sea. Percolation to groundwater is only waste if the groundwater is saline. Otherwise groundwater is an effective way to store the spate water for subsequent use in pumped irrigation. Unlike storage in open reservoirs, groundwater is not subject to evaporation losses although there may be movement towards remote aquifers or the sea. Increased availability and reduced cost of pumps has resulted in groundwater becoming the predominant source of water in some spate irrigation areas. This can change the economics of investment in spate irrigation infrastructure.

However, spate irrigation farmers may not be the exclusive users of the flood water and consideration should be given to possible downstream uses, either human or environmental, which may be adversely affected by increased upstream diversions.

The proportion of the spate flows that are diverted and used beneficially will depend on local conditions and resources. Diversion of a small proportion of the flood flow at a particular location can be achieved with relatively small effort, but diversion becomes more difficult as the proportion of flow to be diverted increases.

Application of spate irrigation water may be a single large application (300mm or more) or several smaller irrigations, in accordance with local practice. Highly suitable soils are capable of holding up to about 600mm of water within the root zone of deeply rooting plants such as sorghum or cotton. Crops may be planted immediately after flooding or may be delayed until after the end of the flood season, depending on local conditions.

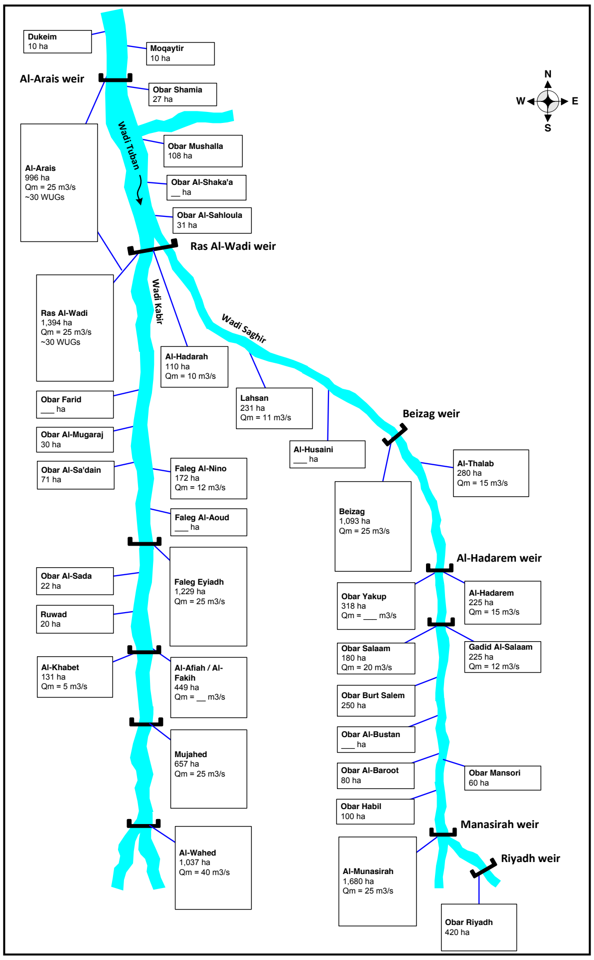
Traditionally, diversion of spate flows is carried out at a number of locations along the wadi. An example is shown on Figure 8-1. Smaller floods would not reach the downstream intakes, which rely on the larger floods which flow past (and often damage) the upstream diversions. Multiple intakes provide an efficient solution where the cost of each diversion is low, with each supplying a relatively small canal system carrying manageable flows but having a large combined diversion capacity.

However, where substantial improvements to diversion arrangements are planned, then the cost of making these improvements to many intakes becomes very high. As a result, where traditional spate schemes have been modernised, the normal practice is to provide a limited number of major diversion structures serving new canals that connect to the existing canals. This can create problems with water management where there are well-established water rights and operational rules for the traditional system.

Cost considerations resulting in the diversion capacity of the improved diversions usually being less than the combined capacity of the traditional intakes. There are many examples of farmers re-activating their traditional intakes in order to capture a larger proportion of the flood flows.

Box 2-11 describes experience at 2 modernised spate irrigation schemes where the farmers wanted reactivation of some traditional intakes in order to improve their diversion capability.

Figure 8-1: Schematic layout of Wadi Tuban



8.1.2. Diversion Capacity

The flow capacity of each intake from a wadi is based on the command area and the time for which water will be available. Guidance on the estimation of the time that water will be flowing is given in Section 5.2.6. The design flow capacity can be estimated based on the volume of water to be applied and the time the water is available using the formula:

$$Q_i = \frac{2.778\ A_i\ W}{t}$$

where:

Q_i	is the design discharge (m^3/s)
A_i	is the irrigable area (ha)
W	is the gross (ie allowing for irrigation efficiency) depth of irrigation in m
t	is the time of application (hours)

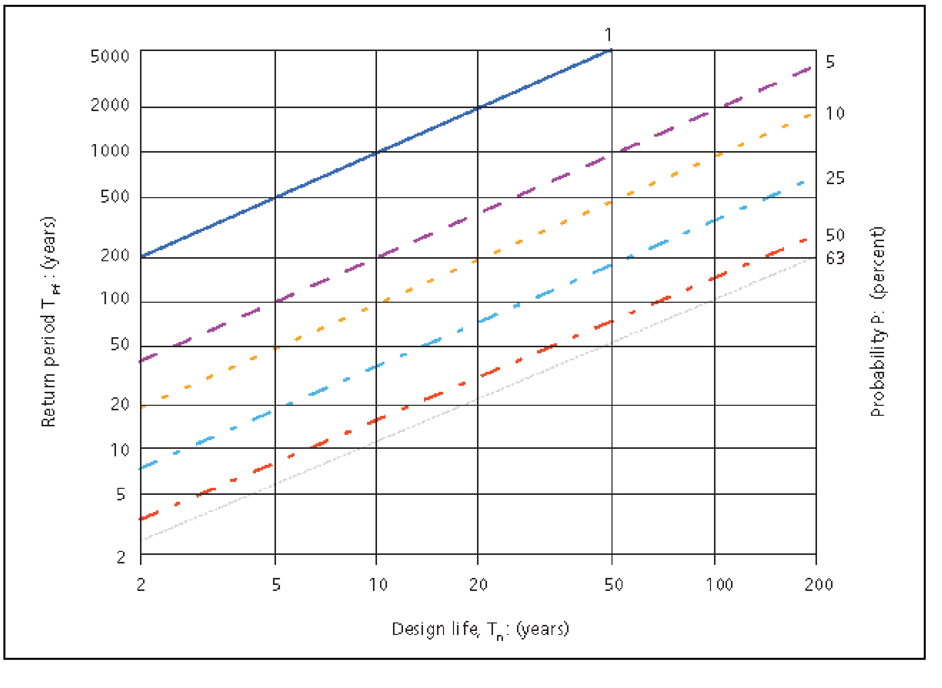
The time of application is assumed to be the time period in hours for which the flood flows in the wadi at the intake will exceed the canal capacity during the irrigation season. This time period will probably become shorter the further an intake is down a wadi as the upstream intakes take part of the flow. Percolation into the wadi bed may be another source of flow reduction.

In situations where there is a prolonged flood recession then the above formula over-estimates the capacity because the volume of water that can be diverted during the recession can be substantial. This is most likely to apply to an upstream intake which has the best opportunity to divert the recession flow. Where multiple intakes exist along a wadi then a simple calculation should be undertaken using the inflow and the flow diverted at each intake.

8.2. Flood Resilience

Any engineered structures built in the wadis need to withstand severe floods with minimal damage. A common criterion is to design a structure to withstand a flood of a specified probability such as 1 in 100 years. However, there is a probability that one or more flood events with a specified return period, or greater, will occur over the design life off the structure as shown on Figure 8-2.

Figure 8-2: Probability That Floods with Specified Return Period will Occur



Designers should therefore consider what is likely to happen under exceptional floods. For example, will a structure be damaged if it is submerged? Will canals be destroyed due to excess flow through the intakes? Designers also need to be aware of the behaviour of high velocity water. If fast-flowing water slows down then the kinetic energy of water has to be converted. An example of flow under turbulent conditions is shown in Box 8-1. Typical flood growth factors are given on Figure 5-3.

For example, if the design life of a structure is 50 years, then Figure 8-2 shows that there is a 10% probability of a 1 in 500 year flood being encountered within the structure life. Consideration has therefore to be given to the performance of the structure under an exceptional flood event while balancing the cost of repair and the losses) with the cost of providing a more robust structure.

Box 8-1: Impact of High Velocity Water

What happens when high velocity flow is obstructed? A big splash!

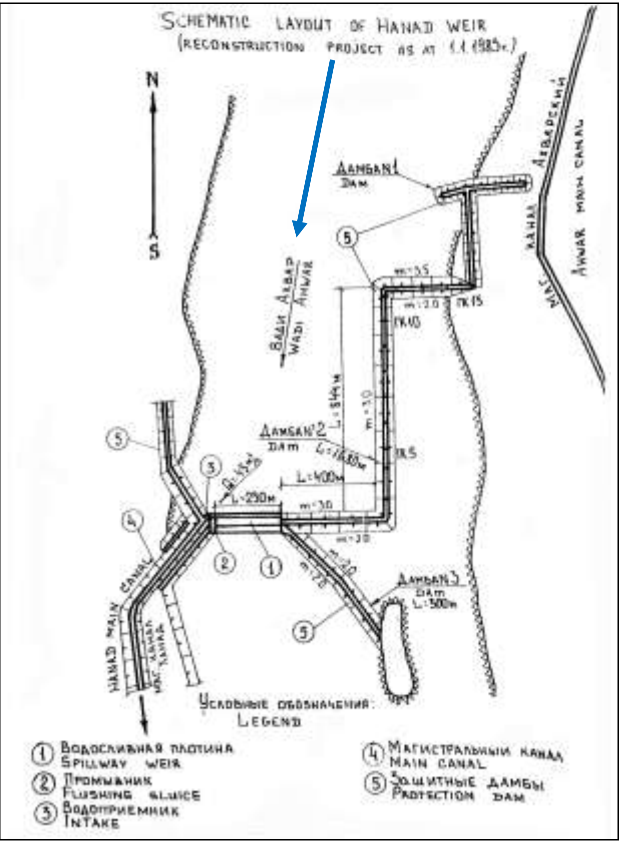
Energy dissipators destroy the velocity energy through turbulence but gradual deceleration of flow can be converted into potential energy, thus raising the water level.

Box 8-2 : Al Hanad weir, Yemen

Al Hanad weir - not one but two big floods!
Al Hanad weir on Wadi Ahwar in southern Yemen was originally built in about 1973. The original design of the Al Hanad weir used a weir crest length of 250m which involved a substantial constriction in the natural wadi channel which was about 650m wide prior to the construction of the weir. The weir functioned satisfactorily until the major flood of 1982 which bypassed the weir on the left side.

The weir was repaired but the left bank head regulator was closed off and an embankment constructed to connect the left weir abutment to the new left side of the wadi at a location about 1.5km upstream. The closing bunds were aligned either parallel or perpendicular to the flow. In 1989 a flood reported to be 4800m³/s again breached the left side embankments. The layout of the embankments must have increased their vulnerability since they did not smoothly guide the flood flow towards the weir. The weir itself was also damaged with about 50m of weir near to the right head regulator destroyed and part of the head regulator structure damaged.

The layout of the embankments did not guide the water smoothly towards the weir. This may have resulted in higher than design water levels against some sections of the embankments where the water decelerated. The flow velocity under an extreme flood would be about 3m/s giving a possible velocity head of up to 0.5m.



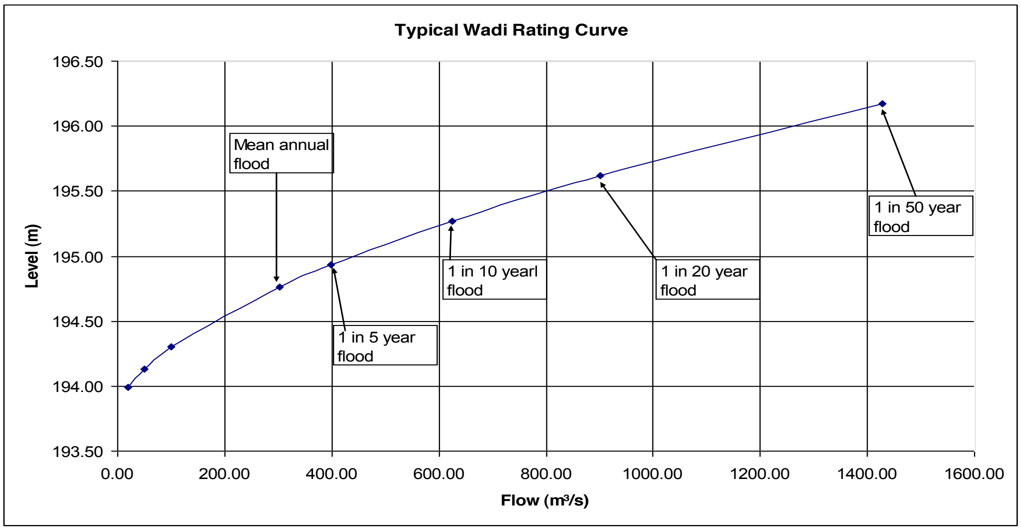
Earthworks are particularly vulnerable to large floods: Once overtopped they are likely to be completely washed away. Overtopping can be caused by either wave action and/or the water level increase associated with deceleration. Earthworks are usually easier and cheaper to replace than structures but the flow through breached earthworks can sometimes cause more extensive damage. An example of a layout where the layout of the earthworks may have contributed to failure is given in Box 8-2. Embankments perpendicular to the flow may have caused increased local water levels.

8.3. Flow Rating Curves

8.3.1. Introduction

It will be necessary to prepare rating curves for both upstream and downstream of a structure (or proposed structure location) in order to see the relationship between flows and water level. If the wadi channel is very uniform then it is possible to make a reasonable estimate of the rating curve using Manning's formula with two channel cross sections (to provide an average cross section) and bed slope which is assumed to be the same as the water surface slope) and the Manning's roughness coefficient based on the bed material. However, where flow conditions are less uniform then the water surface and bed slopes will not be parallel and a more detailed calculation is needed. The method usually used is the slope-area method (see Section 5.2.4).

Figure 8-3: Example Rating Curve



One set of observations will provide only one point on the rating curve. Ideally, measurements are made for several floods of different magnitudes to get additional points on the curve.

A supplement to the slope-area method is the velocity-area method. This requires people to be present during the flood to measure the velocity at various points across the channel by timing floats passing between two marked section lines. The additional velocity information avoids the need to estimate the manning's roughness coefficient.

The upstream rating curve will be affected by the structure itself and any associated works such as confinement of the channel by embankments that guide the flow towards the structure. Therefore, if the rating curve is calculated without a structure then it will need to be recalculated to include the structure. The rating curves will be used to determine the water levels at the site for use in hydraulic and structural design. In some circumstances it may be necessary to prepare more than one set of rating curves to take account of changes such as in channel morphology.

8.4. Conceptual Design

8.4.1. Basic Requirements

For the purposes of this discussion the term "diversion works" is used to cover all the works associated with diverting water from a wadi into a canal. This may be a simple free intake, a complex structure including a weir, or any combination of structures in between.

The basic requirements for normal spate diversion works are:

(i) To be compatible with accepted water rights but incorporate the flexibility to cope with future changes

(ii) To divert the maximum flow of water that the distribution system can handle without surcharging the distribution system such that damage is caused
(iii) The be able to command the land to be irrigated
(iv) To divert the water for the longest possible period while water is available
(v) To minimise the sediment load entering the canal system
(vi) To be adaptable for future changes, particularly rising land levels and the movement of the approach channel
(vii) To have construction and operating costs compatible with the financial and economic benefits of spate irrigation
(viii) Where the structure will be operated by the beneficiaries, operation and maintenance requirements should be compatible with their resources

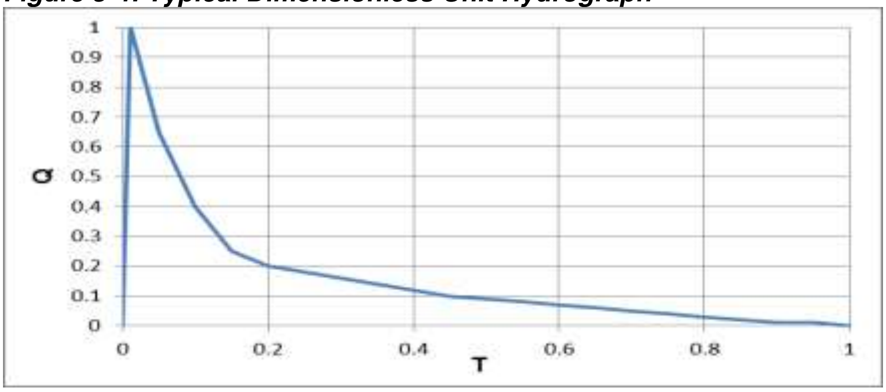
8.4.2. Capacity of Intake

A head regulator for a spate irrigation system should have a large flow capacity compatible with the downstream canal capacity and the required irrigation duty. A common deficiency of many engineered structures, at least from the farmers' viewpoint, is that the capacity is too small.

Head regulator capacity can be determined theoretically using the flow-duration curve which shows the proportion of the total flow volume occurring below a specific flow value (see *Figure 5-5*). However, unless the flow-duration curve is based on several years of gauging data it may be misleading. The alternative is to estimate the volume of water in the floods based on the farmers' memory about the flood characteristics, size, duration of peak and overall duration, and the number of floods of each size group in recent years. A method that can be used is described below.

First, a typical unit hydrograph is prepared for the catchment as shown in Figure 8-4. In reality, each flood will have a different shaped hydrograph but the typical unit hydrograph should represent the average situation. The unit hydrograph is defined by a table of Q against T.

Figure 8-4: Typical Dimensionless Unit Hydrograph



The floods are then categorised into several groups based on maximum flood flow and duration. The categories will depend on the local catchment. The farmers may indicate a typical maximum flood water level for floods of different groups. The engineers then have to undertake a survey and calculate the corresponding flow using the slope - area method. The flood peak and duration data can then be combined with the unit hydrograph data to calculate the total flood volume by summing the flow volume at each time increment to give typical flood volumes for each size of flood as shown on Table 8-1.

Table 8-1 : Example Flood Classification

	Flood size			
	Very large	Large	Medium	Small
Peak (m^3/s)	200	100	50	30
Total duration (hours)	40	20	10	5
Volume (m^3)	4,456,800	1,114,200	278,550	83,565

The average flood volumes can then be combined with the records of flood occurrence to give an estimate of annual flood volume. An example computation, based on the farmers' memories of the floods in the previous 5 years, is presented in Table 8-2.

The total flood volume should be in the same range as estimated from the catchment runoff. If not, further study is required to reconcile the differences. The advantage of calculating flood volumes using the flood characteristics is that it can be easily taken one step further to calculate the volume of water for a specific intake size (the same calculation can be undertaken using a flow-duration curve provided it reflects the catchment characteristics). In the above example, restricting the intake capacity to 40m³/s reduces the maximum diversion potential by over 20%. If a gross water application of 1,000 mm (400mm net application at 40% efficiency - see Table 5-2) is assumed then the area that can be fully irrigated ranges between 252 ha and 962 ha for the years in the data sample.

Table 8-2: Example of Annual Flood Volume Estimation

	Number and size of floods reported by farmers					
Size	**Year 1**	**Year 2**	**Year 3**	**Year 4**	**Year 5**	**Average**
Very large	0	1	0	0	2	0.6
Large	2	1	1	2	3	1.8
Medium	4	2	5	3	3	3.4
Small	3	5	3	4	2	3.4
Volume	3,593,295	6,545,925	2,757,645	3,398,310	13,258,980	5,910,831
% of average	61%	111%	47%	57%	224%	
Volume <40m³/s	3,179,295	4,819,725	2,523,645	2,993,310	9,626,580	4,628,511
Ha irrigated @ 1m	318	482	252	299	962	462

An appropriate development area for this example may be 500ha but the full area would only be irrigated in one or two years out of 5 and in a dry year only 50% would be fully irrigated. Under such a situation the prudent risk sharing strategy is for farmers to have their landholding spread between plots in the upper, middle and lower parts of the system to reflect the likelihood of getting sufficient irrigation and a crop. A design intake capacity of 40m³/s for 500ha represents an irrigation duty of 80 litres/second/ha (compared with, typically, 1 l/s/ha for perennial irrigation).

Reduction in the intake capacity will also reduce the area that can be fully irrigated by that intake. However, this can be offset by providing a second intake further downstream.

8.4.3. Ensuring Sufficient Command

The earliest intakes for spate intakes tended to be some distance from the mountains, so that the flood peaks would attenuate, the velocities reduce and often the flow would have naturally split, which makes it easier to manage. The diversion works under such conditions would be easier to construct and maintain with the limited resources available to the farmers.

However, intakes have moved upstream as competition for water has increased. In most locations the over-riding water right is "upstream first", so whoever is furthest upstream has the first opportunity to take water. At the same time, they have the risk of the most damaging flows. With traditional diversion structures, floods following a larger damaging flood would pass downstream until the upstream diversion was rebuilt, thus increasing the equity of distribution. However, where upstream diversion structures have been constructed as fixed weirs, the canals served by that weir have greater certainty of receiving water, to the disadvantage of downstream users.

At first sight, traditional diversion and water distribution structures often seem crude, but they enable water to be diverted from uncontrolled ephemeral rivers using only local materials and indigenous skills. A relatively high water diversion efficiency can be achieved overall when multiple traditional intakes are used along a wadi,.[12] The principal disadvantage of traditional diversion methods is the excessive inputs

[12] *In some cases this was a result of comparing the diversion efficiency of well-designed permanent gated diversion structure with the much lower efficiency obtained with traditional intakes in floods. A permanent gated intake and the combined diversion efficiency of the many independent traditional intakes that form most systems should be compared over the whole range of flows, including the easily diverted low flows that make up a significant part of the annual flow volumes in many spate schemes. In other cases over-optimistic assumptions of increases in cropped areas following modernisation may have been influenced by the need to justify large donor driven projects with conventional cost/benefit criteria. However, there is some evidence that increased abstraction of low flows upstream*

of labour and materials needed to rebuild the intakes and the other water control structures that are frequently, sometimes by design, damaged or scoured out by flood flows.

Diversion structures need to be located where they have sufficient command of the land to be irrigated (including provision for rise in the land due to sediment deposition), plus sufficient slope in the intervening canals to allow for satisfactory sediment conveyance plus the head needed to operate any sediment removal facilities. Traditional diversion structures are relocated further upstream as needed to maintain command. However, this is not feasible for an engineered structure.

8.4.4. Single or Multiple Intakes

A key conceptual design decision to be made is whether to have one or more diversion structures. An engineered solution that uses diversion weirs will need to minimise the number of diversion structures because of overall cost. However, one or more combined structures introduce problems as have been encountered in the modernised spate irrigation systems. These include:

- Creating potential conflicts between groups of farmers over the sharing of water from an intake (although sharing of water between different intakes is also a potential source of conflict but may have been resolved by the existing water rights)
- The use of one or few or intakes inherently gives greater power to the upstream people along the canals to take water whenever it is flowing. This more than offsets the potential for upstream users to divert more than their entitlement of flood water from the wadi
- Canals with sufficient capacity to serve the whole irrigated area from the upstream end will be relatively expensive to construct complete with appropriate flow management structures and significant land will be required. The wadi already exists as a conveyance system
- Upstream structures will receive the greatest amount of incoming sediment in the wadi flow. Therefore, a single upstream intake will have to manage the maximum sediment concentration in diverted flow for the whole of the system
- Multiple intakes along a wadi can achieve a higher overall diversion efficiency than a single structure
- Weirs with intakes on each bank are not recommended but are less vulnerable than having an intake on one side with an inverted siphon to cross the wadi. A siphon requires very effective sediment exclusion to avoid the risk of blockage.
- If the upstream structures are close to the mountains then the peak floods that they have to be designed for may be more severe than the structures further downstream. Provision of breaching bunds to reduce weir costs is hazardous unless the hydrology is well understood.

The above considerations indicate several advantages of using multiple simpler diversion structures whether for existing or new canals instead of a single structure at the upstream end of the system. Each of these can then become a self-contained management unit (except for the overall wadi water allocation). Downstream intakes may receive less water, but they also have lower expenditure on sediment management.

8.4.5. Site Selection

Where feasible, intakes should be located on the outside of a moderate bend where the channel bank is made of firm material. The deepest part of the channel will also be at the outside of the bend which facilitates diversion of the lower flows and the bend will tend to cause cleaner surface water to move towards the intake and the bed sediment to move away from the intake due to the helicoidal flow pattern as shown in Figure 6-2. Although the development of helicoidal flow will be less in wide, shallow wadis than in normal rivers there will still be the tendency for the low flow channel to be at the outside of bends.

While a rocky foundation may be considered attractive, this may result in expensive rock excavation which more than offsets the cost of building a structure suitable for a non-rock foundation. Sharp bends are more vulnerable to erosion and the intakes sites will be less stable or require protection but are preferable to a location on the inside of a bend. Should it be decided to have two intakes opposite each other then they should be located on a straight section of channel to reduce the likelihood of water consistently going to one side. Some weirs have been constructed with an intake on each bank. While this shares the investment cost, the flow will not naturally divide itself between both intakes and temporary embankments are often required to split the flow.

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from new intakes in modernised systems has reduced the flow volumes available for diversion and the hence the areas that could be irrigated from the new facilities.

8.4.6. Sediment Exclusion

Some provision for sediment exclusion should be provided at all but the most simple of structures. General options for sediment exclusion have been discussed in section 7.4.2 and summarised in Table 7-2. The selection of suitable facilities for specific diversion structures is discussed later in this chapter.

8.4.7. Operation and Maintenance

Consideration of the operation of the structure should be made early in the design process so that realistic assumptions can be made in the design. This is discussed in more detail in section 3.4 but some points to note here are:

- Fewer, larger gates are likely to be easier to operate
- Clear gate operating rules are required based on easy indicators
- Ease and simplicity of maintenance are important
- The design and performance of the intake will affect the ease of operation and maintenance of the related canal system
- The design of the canal system must take account of the operational characteristics of the intake

8.5. Traditional intakes

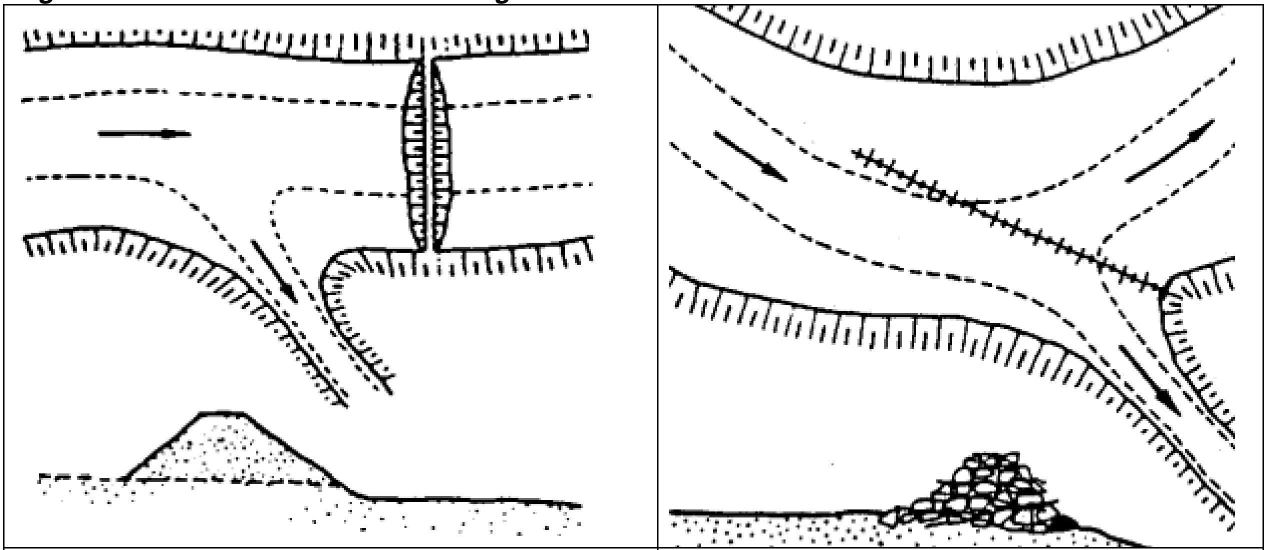
8.5.1. Types of Traditional Intakes

Traditional diversion works can have two basic forms as shown on Figure 8-5: (i) total blockage of the wadi by an earth or gravel embankment which diverts all the flow until it is overtopped and breached; and (ii) a spur of earth, gravel or brushwood (or combination) that intercepts part of the flow and diverts it to a canal. This approach does not divert all the flood flow but repair and reconstruction is less costly. They are described here so that design staff can appreciate the features of the traditional structures that the farmers may wish to be improved.

Diversion bunds are constructed from wadi bed material and completely block the channel to deflect all the flow to a canal as shown in Figure 8-5 (a). They are most usually constructed in the downstream sections of the wadi systems where they are able to divert the total flow of smaller floods and the likelihood of larger floods is small. Reconstructing a breached diversion bund is a substantial task using oxen and scraper boards, but can be achieved quite quickly if a bulldozer is available.

The deflector method of diversion is shown schematically in Figure 8-5 (b). Deflector intakes are usually located at the upstream and middle sections of wadis, usually at the outside of a bend, where the deep water channel created by flood flows forms a low flow channel close to the wadi bank. A low spur, constructed of gravel or brushwood, is constructed close to the river bed to intercept and head up low and medium wadi flows. The alignment and length of the spur depend on the conditions in the river. If the low flow channel is well established at the outside of a wadi bend then the spur will be aligned almost directly upstream. The position and length of the spur can be adjusted to follow changes in the alignment of the low flow channel, or to increase or reduce the proportion of the small and medium flood flows that are captured. In larger floods the deflector is usually damaged or completely washed out. However, this also reduces the flow entering the canal, reducing the risk of damage, and also reduces the entry of large bed sediments into the canal.

Figure 8-5: Traditional diversion arrangements



(a) Diversion bund	(b) Deflecting spur

Figure 8-6: Examples of traditional diversion works

Yemen: Gravel embankment across wadi channel	Eritrea: Repair of brushwood spur in wadi

A free intake may consist of either (a) of a channel excavated through the wadi bank and emerges onto lower ground or (b) a spur that separates part of the wadi flow from the main channel and gradually raises it to the bank top level. The intake is usually constructed at the outside of a bend, where the wadi makes a smooth curve and the low flow channel is close to the bank. The width of the canal head regulates the amount of water entering to the system. This structure is simple to construct, and is often used to irrigate small areas from small wadis. However, it is vulnerable to substantial excess flows during large floods.

An advantage of low cost traditional diversions is that they can be easily reconstructed at different locations to follow the main flow channel and provide additional command. Engineered structures lose this flexibility and it is not attractive (or usually feasible) to relocate them either during their design life or beyond, should the structure remain intact but increasingly ineffective.

8.6. Engineered intakes

8.6.1. Types of Intakes

Whereas a traditional intake is normally an open channel, an engineered intake includes some form of a structure, normally called a “head regulator” that will restrict the maximum flows entering the canal. The type of structure to be selected will be determined by various factors including: probability of big floods; design life and benefiting area (as an indicator of budget).

Head regulators may be combined with other components such as sluiceways and weirs to create a full diversion structure. The chosen configuration will be determined by site conditions and the available budget.

An engineered improvement of a traditional intake offers the advantage of no change to the existing canal system and probably no change to the water rights. The basic objective of the engineering intervention is to reduce the costs of rebuilding and maintenance, which may have to be carried out after every flood. There are many options for improvement, which depend on site conditions and the available resources.

However, any work to strengthen the diversion arrangements will tend to increase the proportion of floods diverted and it may be necessary to provide an intake control on the canal to limit the maximum flows. it may be best to retain the traditional intake where frequent intake relocation is necessary on account of an unstable wadi channel or rapidly rising field levels. Improvement could be confined to provision of access to machinery for rebuilding of temporary works.

Engineered intakes are more difficult to modify than traditional intakes to accommodate rising field and canals levels. It is advisable to design the intake structure for a higher canal level than currently needed and provide a drop downstream of the intake. This drop would progressively become buried as the farm land and the canal rises.

8.6.2. Open Intakes

The most simple design of improved intake is an open channel with a fixed invert, two vertical side walls with associated scour protection. This can be easily and cheaply constructed of gabions and the rectangular opening will reduce the maximum flows relative to an open trapezoidal channel.

Figure 8-7: Examples of simple intakes

	
Open intake in Yemen with rectangular flow constrictor	Improved traditional intake in Yemen with bed bar across wadi

8.6.3. Ungated Orifices

The next level of flow control is provided by an ungated orifice such as shown in Figure 8-8. An orifice provides free flow at lower water levels but once the water surface reaches the top of the opening the rate of increase in flow is restricted, although the flow will still increase as the upstream water level rises. If it is required to close off the intake then an embankment is constructed in front of the structure. Model testing was undertaken to explore the possibility of having two successive orifices as a way to further limit the increase in flows but the configuration resulted in unstable flow conditions as the orifices changed flow modes. Excess flows are best managed by providing a rejection spillway on the canal (see section 9.1.8).

8.6.4. Gated orifices

A gate creates a variable orifice so that the flow can be further controlled or closed off completely either due to water rights or during peak flood flows. Normally, the gate is built into a wall for a head regulator on a canal intake so that the gate cannot be overtopped and the flow is limited by the wall should the gate be raised more than intended. Provision of a gate further increases the cost compared to an ungated orifice but provides greater operational flexibility.

Figure 8-8: Ungated orifice intake in Yemen

This intake comprises a reinforced concrete breast wall and 3 openings (the left opening is not visible). The farmers build an embankment in front of the intake when it is not the turn of the canal to take water while, when it is the canal's turn, the orifice will limit the maximum flow during floods. A bed bar set at intake invert level was also provided across the wadi to help manage the flow.

Gated intakes provide the capability to regulate the flow into the canal and, if necessary (for example in an emergency, or for water allocation reasons), stopped completely. The gates should be largest possible size suitable for the means of operation since intermediate piers in the headworks increase the hydraulic losses and catch trash during floods. Ideally, the gate openings should be sufficient to pass substantial trash since any blockage represents a loss of water for the farmers. Vertical gates are less expensive to manufacture than radial gates but require greater operating effort. Vertical gates greater than about 2m width become are inappropriate for manual operation whereas manually-operated radial gates can span up to about 5m. Usually a breastwall is provided above the gate, with the bottom of the breastwall being set at the upstream water level required to give the design flow. This limits the maximum flow when the gate is open and minimises the size and weight of the gate.

Figure 8-9: Simple gated orifice intake in Yemen

This structure was originally designed as a single ungated orifice. The central pier and gates (already available) were added later. The reduced size of the openings increased vulnerability to blockage by trash.

Provision of gates avoids having to build an embankment to close the flow and inflows can also be regulated.

The head regulators provided on major diversion structures are also gated orifices, but are combined with a sluiceway and diversion weir.

Operation guidelines often recommend that intake gates are closed during the flood peak in order to exclude the water with the highest sediment load from the canal system. However, farmers are usually reluctant to accept any closure, which represents lost water. Without power assistance, it is usually not feasible to close and open the gates within a short period. This is a major deficiency of most spate intakes that do not have electric gate operation. However, provision of motorised gates and a generator creates another maintenance burden.

8.6.5. **Trash Management**

All floods tend to pick up and transport trash. Major floods can often uproot trees along the wadi. The design of diversion works needs to make allowance for the trash. One design challenge is that the cleaner water on the outside of a bend, preferred for its lower sediment load, tends to have the highest trash load. There are three basic options for managing trash:

(i) Encourage the trash to pass down the wadi through careful design
(ii) Detain or divert the trash upstream of the intake (eg with a floating boom) where it will not significantly obstruct the flow to the intake
(iii) Allow the trash to pass through the intake

Letting the trash pass through the intake or sluiceway is usually the most attractive. However, there is an upper limit to the size that can be passed and once something becomes trapped, then it will obstruct the passage of smaller trash so that a blockage follows. Provision of any trash screen on the intake in front of the gates will substantially accelerate the blockage process. Examples of trash problems and solutions are shown on Figure 8-10.

Figure 8-10: Examples of Trash Problems and Solutions

	
Mai Ule, Eritrea. Head regulator after a large flood: A tree truck has caused a built-up of debris blocking one opening, but the other two are open.	Wadi Zabid, Yemen: No breast walls are provided on the sluiceway in order to help trash pass downstream
	
Waqir weir, Yemen. Fine trash screen vulnerable to blockage (many bars have now been removed)	Barquqa weir, Yemen: Large trash deflector upstream of intake (weir to right of photo)

A trash blockage during a large flood needs to be removed as quickly as possible once the flood recedes. The design of the intake structure should facilitate trash removal. The material is best pulled out in an upstream direction. An overhead gantry would be an expensive investment, but provision of a secure mounting point for a small winch upstream of the intake would require limited investment. Alternatively, if some tractor access is possible, provision of one or more simple pulleys upstream of the intake could be used in conjunction with a tractor-mounted rope.

8.7. Weirs and Bed Bars

8.7.1. Purpose of Weir

A weir will create a rise in the water level which would provide two benefits: (i) Increased ability to command an irrigation system and (ii) provides a differential head for flushing of sediment, either at the structure and/or from the canal head reach. A weir will also cause a rise in the bed level upstream, which will also raise the upstream water levels and increase the risk of out-of-bank flooding, but may also improve the ability of traditional intakes to take water. Weirs are inherently expensive structures because of the need to provide energy dissipation.

Bed bars are basically a protected wall built in the wadi bed and is arranged to guide water towards an intake and prevent the development of low flow channels at other points across the wadi. A bed bar should not significantly raise water levels under flood conditions so there is no need to provide for energy dissipation although scour has to be considered. Bed bars are a small fraction of the cost of weirs[13] and are most likely to be appropriate in the middle to lower reaches of a wadi system where flow conditions are less aggressive.

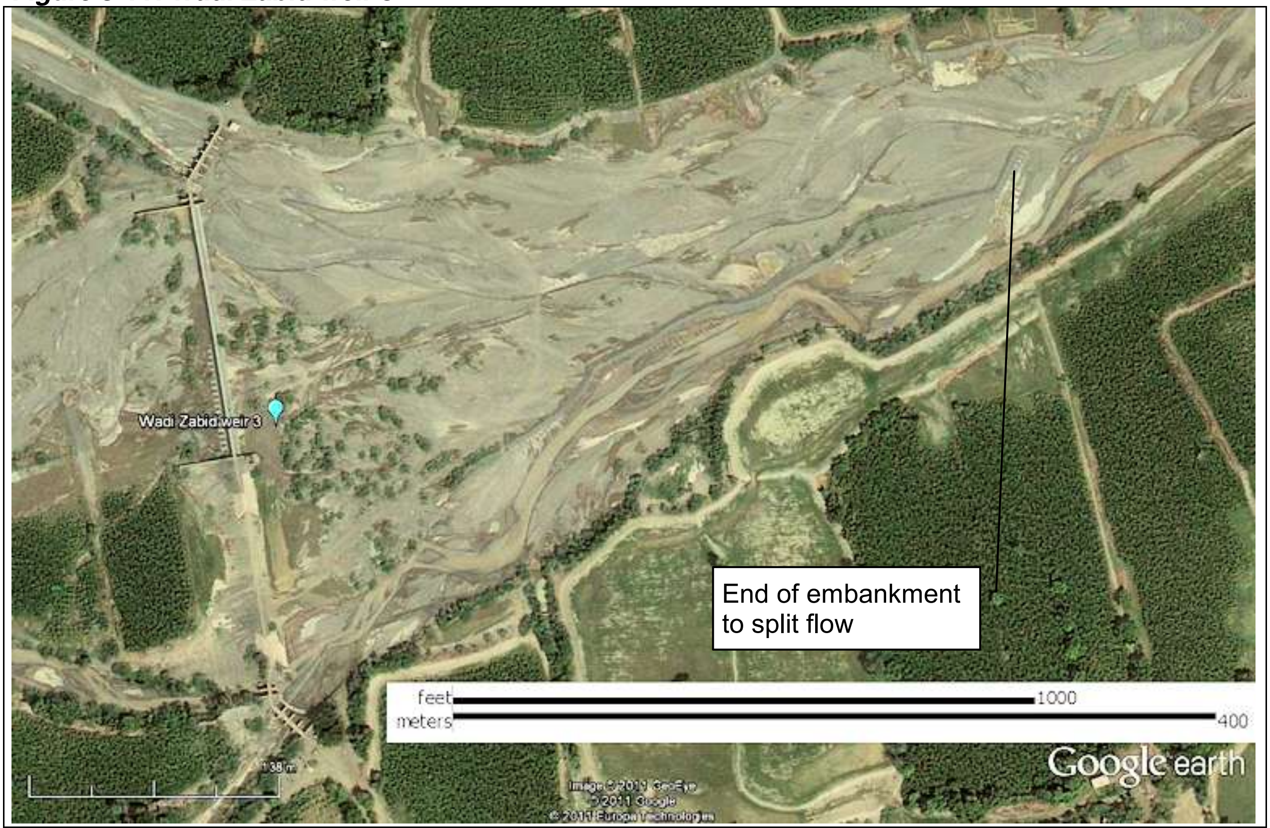
8.7.2. General Design Considerations

Diversion weirs with intakes at each end are not recommended because of the difficulty with splitting the wadi flow to both sides. However, this may be the only way to justify the cost of the weir. Regular use of any sluiceways will help maintain channels towards the intakes and if a bulldozer is available then it can be used to create temporary embankments to help divide the flow. An example of this work is shown on Figure 8-11.

The use of an inverted siphon to enable land on both sides of a wadi to be irrigated from an intake on one side has a mixed record. The approach has been successful in engineering terms at Wadi Rima and Wadi Mawr in Yemen because upstream of the siphon are large, flushable, sediment basins. However, a culvert built at Wadi Laba in Eritrea under the wadi was unsuccessful because it was vulnerable to blockage once a mechanically cleaned gravel trap became full and could not be cleaned during the flood season. Gravel then overflowed into and blocked the culvert. Siphons or culverts are expensive structures because they need to be built of good quality reinforced concrete.

It is recommended to slope the weir crest at 1% to 2% downwards towards the head regulator in order to encourage the low flow channel to stay close to the intake. If there are two intakes then the weir will be highest in the middle of the channel with slopes down to each side.

Figure 8-11: Wadi Zabid weir 3



In order to divide the flow between the two intakes an embankment over 400m long has been constructed upstream of the weir. This has to be rebuilt using a bulldozer after large floods or whenever the main flow channel moves.

8.7.3. Weir Construction Options

Weirs comprise 3 main components: (i) the weir crest; (ii) a basin for dissipating energy; and a glacis slope between the crest and the basin. Sometimes a vertical drop is used, in which case no glacis slope is provided and, exceptionally, if the weir is founded on rock then no basin is required. Weirs with vertical

[13] *Silva & Makin A low cost approach for wadi flow diversion. Proceedings of Spate Irrigation Conference, 1987 pages 102 - 106.*

drops are vulnerable to failure from overturning. Ogee weir crests have the highest discharge coefficient but are not recommended unless a high standard of construction quality is anticipated.

Most weirs are constructed of reinforced concrete, mass concrete, stone masonry or gabions, or combinations of these materials. However, rockfill weirs are an alternative as described in Box 8-2 and may be less expensive than rigid weirs where suitable rock is available within reasonable distance, the wadi slopes are flatter (<0.5%) and the foundations are permeable. See Box 8-3 for an example design. Gabion weirs are another option but need to be protected by a thin layer of reinforced concrete if there is substantial bed load of large or abrasive material.

8.7.4. Breaching Bunds

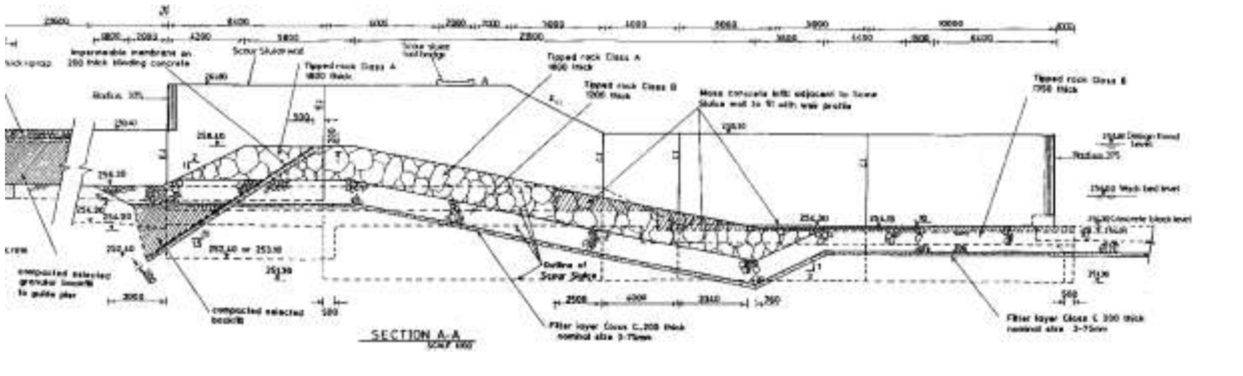
The cost of a weir depends on the length of the crest which is controlled by the design flow. The 1 in 10 year flood is typically about double the mean annual flood peak while the 1 in 100 year flood is usually about 6 to 7 times larger than the mean annual flood peak. Figure 8-2 shows there is quite a high probability that a flood larger than this will occur during the structure design life. The weir cost can be substantially reduced by providing an embankment (called a breaching bund or fuse plug) as an extension of the weir to close off part of the wadi channel. Larger floods (greater than about 10 year return period) will overtop, erode and breach the embankment in order to keep the maximum flow over the structure within its design capacity and avoid damage.

There are mixed results of the performance of breaching bunds. Those provided for some of the Wadi Zabid weirs have breached once in over 30 years during an exceptionally large flood. However, the breaching bund for Wadi Laba in Eritrea breached once or twice per year when designed to breach every 5 years. Similar problems have been encountered in Pakistan. Such discrepancy between theory and reality suggests that the design was unsatisfactory. It is, for example, highly vulnerable to any errors in the flood estimation. The biggest problems with the breaching bunds have occurred where the structures are close to mountains where they are vulnerable to local flash floods. A brief overtopping is, by design, enough to trigger a washout of the bund. A further possible factor is that, as relatively fast water approaches an embankment the water slows down and there is recovery of the velocity head causing a locally higher water level. For example, if the approach velocity is 3m/s then $v^2/2g$ is 0.46m. Full recovery of kinetic energy is unlikely but this illustrates the potential magnitude of the velocity head.

Box 8-3: Rockfill Weirs

Rockfill weirs

Weirs made of large rocks (typically 1 to 2 tonnes in weight) are an alternative to masonry or concrete weirs. They are particularly suitable at sites with permeable and relatively weak foundations where the wadi slope has reduced. Design of a rockfill weir involves selection of a glacis slope which is stable for the selected rock size (bigger rocks allow steeper slopes) under all flow conditions. A relative steep channel slope lengthens the glacis. The weir crest contains an impermeable membrane to prevent direct seepage through the crest. The rocks are placed on graded filters that can accommodate seepage through the foundation. These weirs are quick to construct once the materials have been stockpiled.



Breaching bunds are, therefore, more appropriate further down the wadi system where short floods with high peaks will have attenuated and the flatter slopes mean that the velocity heads are much less. The objective is to avoid unnecessary overtopping and failure while quickly washing out in the event of a large flood.

Suggested design criteria for breaching bunds are:

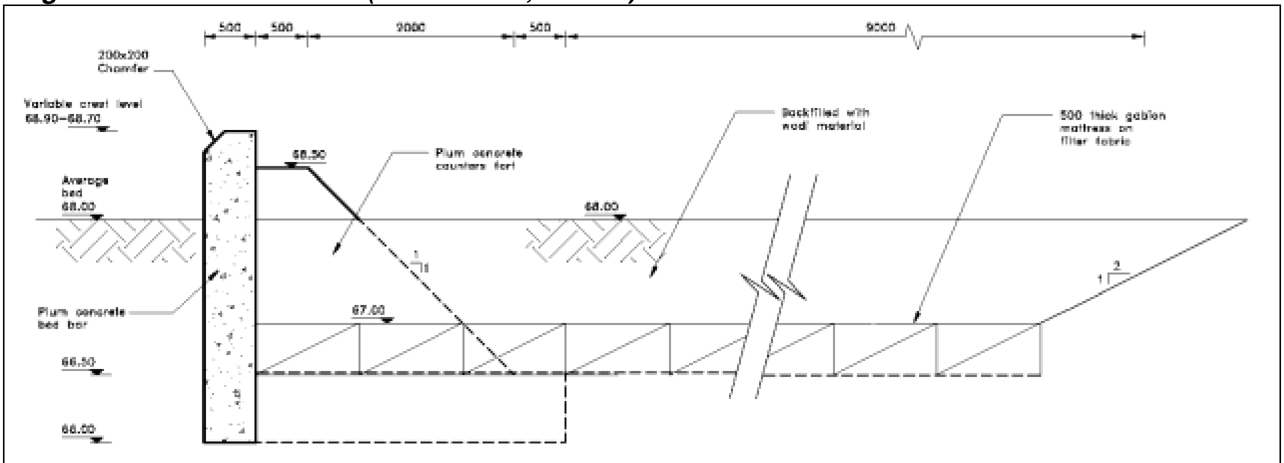
- Constructed of available wadi bed material
- Pushed up by bulldozer and not compacted
- 1V:2H upstream slope and 1V:3H downstream slope
- Top width 2m
- Top level 1m below the top of the adjacent abutments

Traffic passing over the breaching bund where the wadi also serves as a rural road will create a low point which can result in premature failure. It is therefore advisable to incorporate a permanent ramp for vehicles at one side of the structure.

8.7.5. Bed Bars

A bed bar is essentially a wall in the wadi bed that doesn't attempt to significantly raise water levels but cuts off any low flow channels. It therefore provides greater predictability of water levels at intakes. The bed bar may be inclined in plan across the wadi bed as shown on Figure 8-13 with the top level sloping towards the intake in order to guide the water towards the intake.

Figure 8-12: Bed Bar Detail (Wadi Tuban, Yemen)



Bed bars may be vulnerable to failure due to downstream scour during large floods. Two measures are suggested to mitigate this risk:

- Provision of buttresses to the wall on the downstream site at intervals along the wall to reduce the risk of overturning
- Provision of a gabion mattress on the downstream side of the wall to limit the scour depth and protect the bed bar and buttress foundations

The gabion mattress is best buried to avoid excessive abrasion damage during normal floods and it should be anchored to the wall to prevent possible migration downstream.

8.8. Diversion Works Combinations

The main options for diversion works and intakes are summarised in Table 8-3. Selection of an appropriate option is determined by the site conditions and available budget (which may be based on the area served).

Table 8-3: Diversion Works Options

	Description	Head regulator	Weir / bed bar	Sediment excluder	Upstream guide bund	Breaching bund
	Traditional intakes					
1a	Free intake	None	Optional bed bar	None	Optional	None
1b	Free intake	None	Earth embankment	None	None	None
	Improved traditional intakes					
2a	Intake with fixed orifice	Fixed orifice	Optional bed bar	None	Optional	None
2b	Intake with gated orifice	Gated orifice	Optional bed bar	Optional sluice channel	Yes	None
	Modern diversion structures					
3a	Intake with weir	Gated orifice	Weir	Gated sediment excluder	No	Optional
3b	Weir with intake at each side	Gated orifice	Weir	Gated sediment excluder	No	Optional (central?)

Figure 8-13: Schematic Layout of Improved Traditional Intake

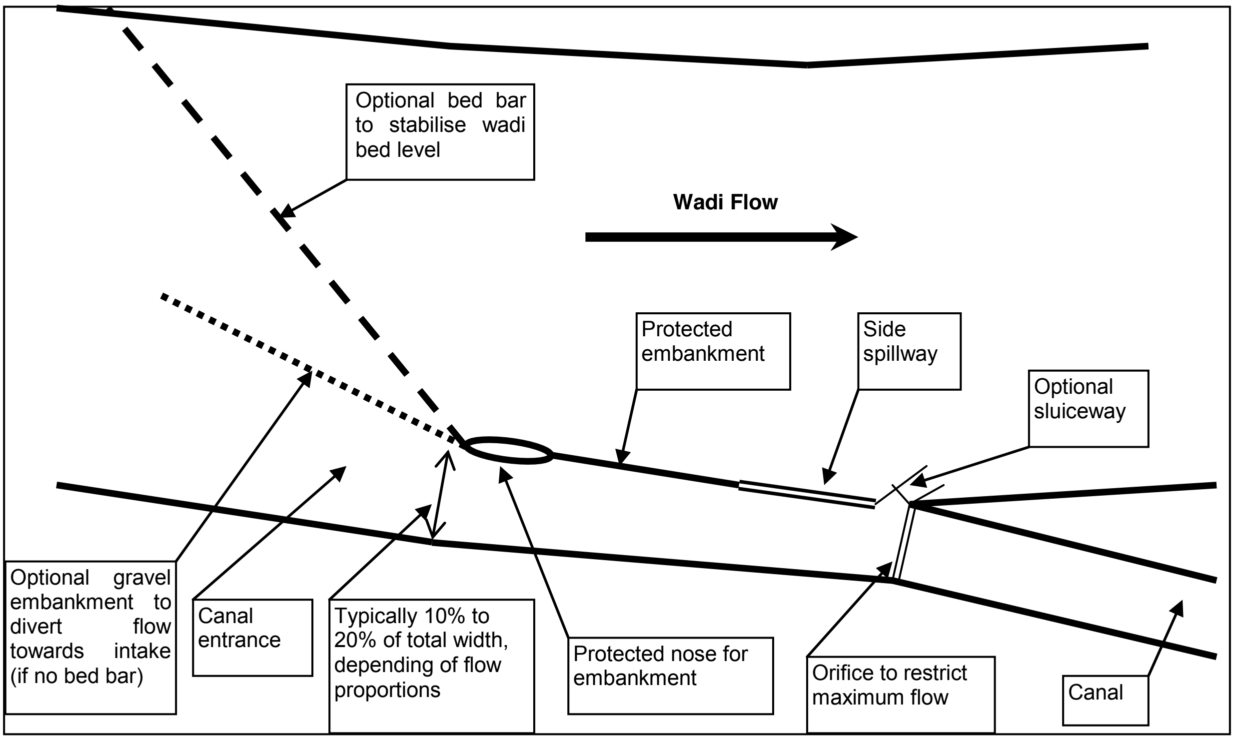
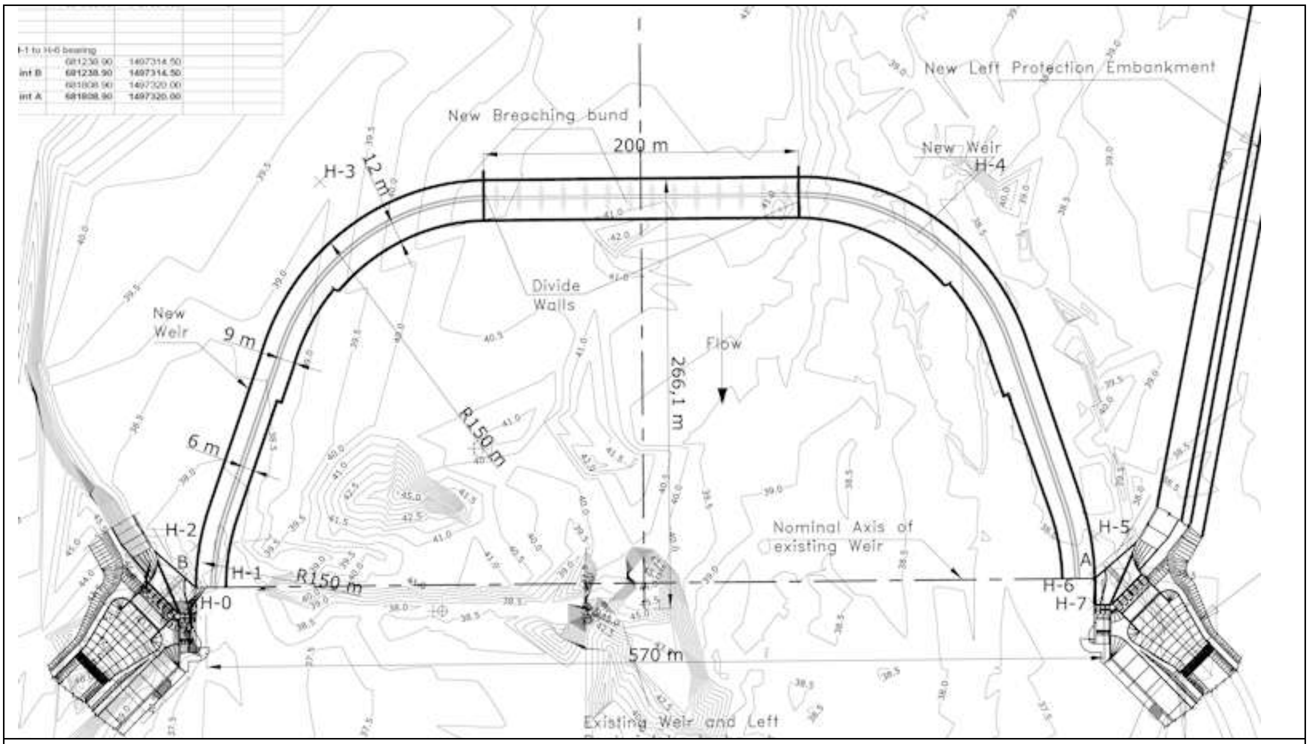


Figure 8-13 shows the indicative design for an improved traditional intake. If a sluiceway is provided then an embankment extending upstream of the intake creates the operating head for the sluiceway. This embankment can be omitted if there is no sluiceway.

Figure 8-14 shows the proposed layout for a diversion structure that needs to supply intakes at both sides of the wadi. Curved weirs are used to guide water towards the intakes. This layout takes into account the successful performance of existing structures shown on Figure 8-15.

Figure 8-14: Proposed Layout for Al Hanad Weir, Yemen



This proposed design has an intake on each side. The central breaching bund provides capacity for very large floods and the curved weir at each side both guides water towards the canal intakes and creates the head for the sluiceways.

Figure 8-15: Examples of Existing Unusual Diversion Structures

The quadrant shaped weir at al Manasirah in Yemen guides flow towards the intake and sluiceway	Al Afiyah - Al Fakih diversion structure in Yemen uses two oblique weirs and a central breaching bund

Standard intake designs often use a divide wall between the weir and the intake / sluiceway. Box 8-4 suggests that it may be preferable to avoid divide walls under conditions with heavy sediment loads.

Box 8-4: Divide Walls

Divide walls are often provided to separate weirs from the intakes in order to (i) create smoother flow conditions approaching the canal head regulator and (ii) enable more effective flushing of the area in front of the head regulator. However, the divide wall separates the area upstream of the weir from the flushing which makes it more difficult to ensure that the main flow channel passes towards the intake. Modifications to shorten the divide wall at some structures in Wadi

Zabid have been successful in enabling flushing upstream of the weir as shown in the photographs below.	
	
Wadi Zabid weir prior to cutting back of divide wall. Sediment upstream of weir has accumulated to weir crest level	The divide wall has been cut back and enables flush of sediment upstream of the weir.

8.9. Bridges

If a weir is constructed then consideration should be given to providing a bridge along the crest. A full road bridge will add substantially to the cost but a footbridge about 1.25m wide will be sufficient for people, motorcycles and animals while being too narrow for anything heavier as shown on Figure 8-16.

Figure 8-16: Footbridges

	
Footbridge at Wadi Zabid weir 3	Bridges should be wide enough for the traffic

8.10. Detailed Design Considerations

8.10.1. Water and Structure Elevation Calculations

Rating curves must be prepared for (i) the proposed structure design (which controls the upstream rating curve) and (ii) the water level in the channel downstream of the structure. The following considerations apply to the selection of design levels:

- The minimum weir crest level has to be sufficient to command the canal. The worst case is usually when the full flow is being diverted so weir crest level = water level at the upstream of the intake under low flow conditions = canal level (including any allowance for field level rise) + head loss through the canal head regulator
- The weir crest level must also be sufficient to ensure operation of the sluiceway. The worst case is usually under flood conditions if the tail water level is high.
- The possibility of a lowering of the channel bed downstream (and water level) of the structure (see Figure 6-3) must be considered. This may not be a permanent change and therefore the design has to consider two cases: (a) the likely long-term downstream bed level and (b) a temporary lowering of the downstream bed level after the structure is built. Sluiceway and stilling basin design has to consider the two options.
- Where the wadi is steep and flow velocities greater than 1m/s then the possible conversion of kinetic to potential energy should be allowed for in design freeboards.

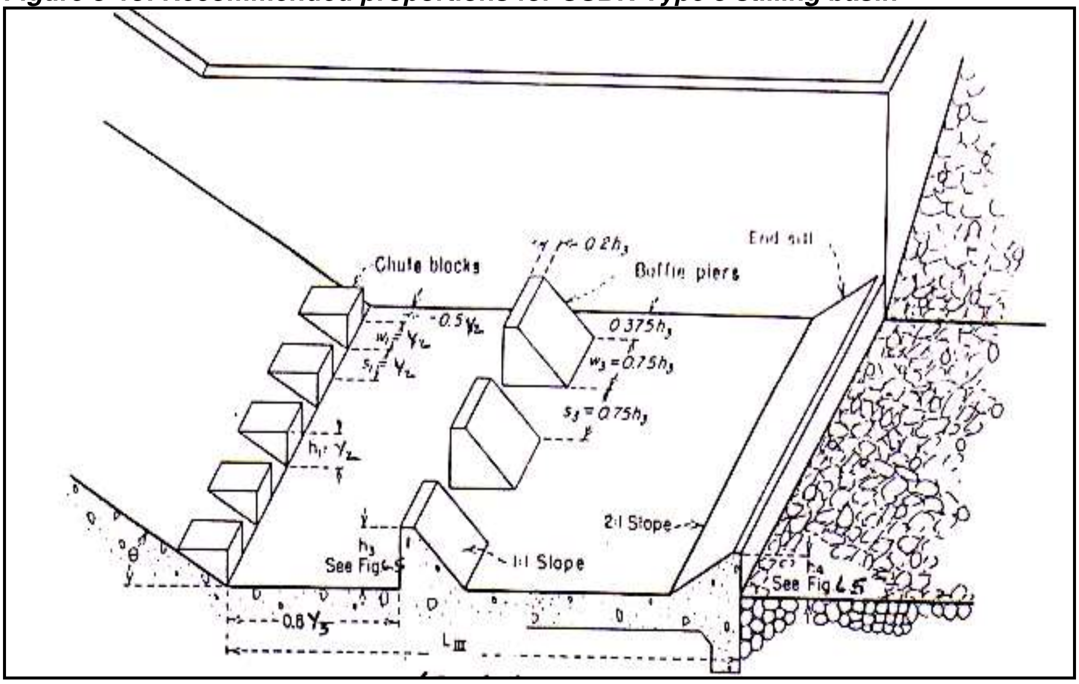
8.10.2. Weir and Stilling Basin

Stilling basins usually follow one of the USBR standard designs[14]. It is usually cost-effective to use a Basin Type 3 which includes chute blocks, baffle piers and a solid end sill to reduce the basin length. Note, however, that the chute block and baffle blocks will need protection is the wadi bed load includes gravel or larger material. The normal method for protection is to provide steel angle protection on the exposed edges as shown on Figure 8-17. The steel angle will need to be securely anchored to the concrete reinforcement.

Also, where concrete is used and conditions are highly abrasive, the weir and the invert of the sluiceway will require protection. For these conditions, the recommended protection is to provide a cladding of carefully dressed hard masonry stone. The joints between the masonry blocks must be small to minimise the risk of scour of the mortar between the blocks which could then result in loosening of the blocks and progressive failure of the protection.

Figure 8-17: Protection to edges of chute and baffle blocks

Figure 8-18: Recommended proportions for USBR Type 3 stilling basin



8.10.3. **Uplift**

The need to design the stilling basin against uplift (floating) depends in site-specific circumstances. Where either shallow groundwater conditions or an impermeable layer immediately below the structure exist then flotation is a risk. However, where the groundwater is deep then it is unlikely that significant uplift forces can develop. The normal engineering measure to prevent flotation is to provide sufficient mass to more than balance any uplift forces assuming that any stilling basin is dry. However, this substantially adds to the cost of any structure. The alternative is to provide under-drainage such as

[14] *Hydraulic Design of Stilling Basins and Energy Dissipators, USBR, May 1983*

shown in Box 8-5: An example of Effective Under-drainage so that any water underneath the structure can emerge. This both relieves the pressure underneath and allows water into the basin to provide ballast.

8.10.4. Scour

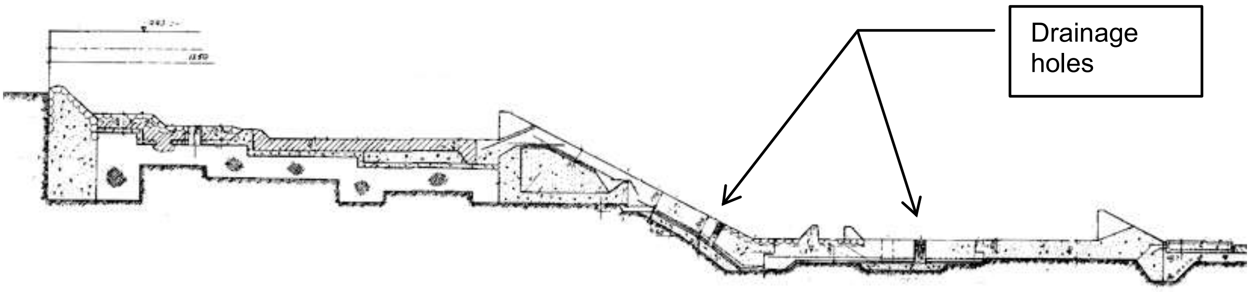
Local scour is associated with particular local features that obstruct and deviate the flow, such as bridge piers, abutments and dykes, and occurs in their immediate locality. The structures increase the local flow velocities and turbulence levels and, depending on their shape, can give rise to vortices that exert increased erosive forces on the adjacent bed. As a result, the rates of sediment movement and erosion are locally enhanced around the structures, leading to local lowering of the bed relative to the general level of the channel.

In the case of structures in the path of the flow, part of the flow is deflected downwards to the bed and rolls up to create what is often described as a "horseshoe vortex" (Figure 8-19) around the front face of the structure; the vortex intensifies the local flow velocities and acts to erode sediment from the scour hole and transport it downstream. Normally, the deepest scour tends to occur at the upstream face of the structure, as a result of the action of the horseshoe vortex.

Box 8-5: An example of Effective Under-drainage

Underdrainage and weep holes

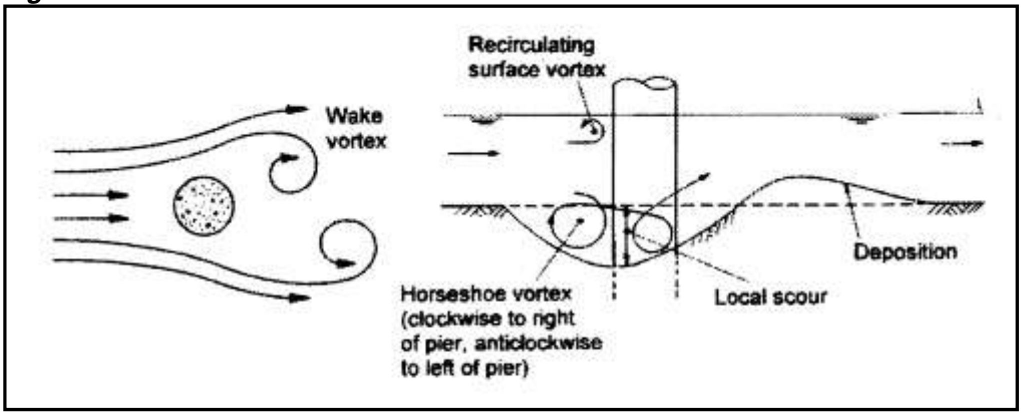
Al Arais weir in Yemen is of very lightweight construction considering its height and design flood (>2,000m³/s). The weir also has an unusual intermediate stilling basin. Under low flow conditions the residual flow seeps into the wadi bed upstream of the weir and emerges through a series of drainage holes near the bottom of the weir glacis slope. If this drainage did not work then the weir would be damaged by the uplift pressures, but the drainage has worked for over 30 years and provides a cost-effective alternative to providing sufficient mass to ensure that the structure does not float.



Material eroded from this hole is usually deposited towards the downstream end of the structure, to a level above that of the surrounding bed. The wake vortices are transported downstream by the flow and can create twin longitudinal scour holes; this type of scour may need to be considered if there is another structure farther downstream that is located within the wake created by the first structure.

As the scour develops, the increase in local flow depth decreases the strength of the erosive action at the bed; as a result, the rate of scour decreases and eventually reaches an equilibrium. For livebed scour, equilibrium occurs when the rate at which sediment is eroded from the hole matches the rate at which it enters due to bed load transport over the upstream section of channel bed.

Figure 8-19 : Local scour at structure



8.10.5. Effect of Debris

The accumulation of debris against bridges and other hydraulic structures can significantly affect the hydraulic behaviour, the amount of scour and risks of failure. This can lead to significant rises in upstream water levels, flooding and overtopping. Accumulation of debris against a bridge structure can increase the amount of scour due to:

- the increased effective width of the structure (which is a significant factor in the amount of scour)
- the increased velocities resulting from the flow constriction and the rise in upstream head.

Debris can also be a contributory factor in structure failure due to:

- increased drag and hydrodynamic forces
- impact forces resulting from the debris colliding with the structure.

8.10.6. Estimation of Scour

During a major flood, higher-than-average flow velocities may cause a short-term lowering of bed levels within an incised channel if the bed material is erodible. There may also be a tendency for the flow to attack the banks and thereby widen the channel. When designing structures to withstand possible scour, however, it is recommended to assume that any erosive action is primarily concentrated towards the bed.

The amount of short-term scour that occurs within a channel during a single flood is difficult to predict with certainty because information on rates of natural scour is very limited. A key factor to be remembered is that a general lowering of bed level within a particular channel reach will only occur if the rate at which sediment is transported downstream from the reach exceeds the rate at which sediment arrives from upstream. An overall increase in the transport rates produced by a higher flow velocity does not itself cause scour, unless there is an imbalance between the amounts of sediment in transport at the upstream and downstream ends of the channel reach. However, any existing imbalances tend to be accentuated during floods, leading to more rapid short term changes.

The usual approach to assessing short-term natural scour is to rely on an extension of regime theory. The basic assumption made is that, during a major flood, the main incised channel tends to increase in size towards the regime geometry corresponding to the peak flow rate. It is unlikely that a full adjustment to the higher regime condition will be achieved during an individual flood, not least because the appropriate changes in channel width and longitudinal gradient take a considerable time. Nevertheless, because it is not possible to be certain how far any short-term changes will progress, it is customary to assume that the full regime condition corresponding to the design flood would be reached.

The Lacey empirical equation may be used to compute the depth of scour. The design scour depth below bed level (D) is given by:

Design scour depth (D) = XR – Y [metric units]

Where:
X = scour factor dependent on type of reach (see Table 8-4 below)
Y = design depth of flow [m]
$R = 1.35\ (q^2/f)^{1/3}$
q = the maximum discharge per unit width [m^2/s]

f = Lacey's silt factor

Table 8-4 : Table of Scour Factors

Type of Reach	Mean Value of Scour Factor "X"
Straight	1.25
Moderate bend (most transitions)	1.50
Severe bend (also Shank protection at spurs)	1.75
Right angled bend (and pier noses and spur heads)	2.00
Nose of Guide Banks	2.25

Where the bed material size is known, the Lacey silt factor (f) may be calculated from the formula:

$$f = 1.76 \sqrt{D_{50}}$$

D_{50} = the sieve size through which 50% of the material passes by weight [mm]. Alternatively, the silt factor is given in **Table 8-5** below for various materials.

Table 8-5 : Lacey's Silt Factor

Soil Type	Lacey's Silt Factor "f"
Large boulders and shingle	20.0
Boulders and shingle	15.0
Boulders and gravel	12.5
Medium boulders, shingle and sand	10.0
Gravel	4.75
Coarse sand	1.5
Medium sand	1.25
Standard silt	1.0
Medium silt	0.85
Fine silt	0.6
Clay	5.0

8.11. Gates

8.11.1. Selection of Gate Type

Gates are an additional construction cost and form an operation and maintenance burden. They often form a source of conflict between water users. The general recommendation is to not provide gates at the intakes unless there is a strong demand for their installation. If intake closure is required under the water rights then a gravel embankment can be used.

If gates are provided then these recommendations should be followed:

(i) gate openings should be a large as possible to reduce the risk of blockage by trash
(ii) gates should be designed to allow operation in a short period of time; and
hydraulic design and energy dissipation should consider the possibility of a high upstream water level and a part open gate which can result in a jet of water flowing under the gate which is more severe than either the gate open of gate closed condition.

8.11.2. Gate Operation

Gates are normally either hand operated or motorised. The latter offers the capacity for much faster operation but needs a reliable supply of power. Mains power, even where available, is vulnerable to disruption since floods are often associated with storms. Having one generator on site is no guarantee that it will work when needed, so should there be a backup generator, which adds to the cost?

It is possible to provide both hand and motorised winding for a gate. However, this should be designed as a motorised add-on to a hand-operated mechanism with the facility for the motor section and associated gearing to be easily disconnected in the event of power failure. The alternative of providing a handle for the motor shaft does not work because hand-winding through all the extra gearing can take hours.

An intermediate technology which has been tried but is not widely used is a portable electric winder which can be connected to each gate as needed. This saves on the cost of fixed motors at each gate but the use of portable electrical equipment, particularly at night and in rain, is a safety hazard.

8.11.3. Vertical Gates

Vertical gates can be simple slide gates or roller gates. With slide gates, the leaf gate slides on bearing plates fixed on to the gate frame. Generally, the two surfaces formed by the bearing plate and its mating surface on the gate form the water tight seal. With roller gates, on the other hand, the gate is provided with rollers on each side which run on rails fixed onto the piers in the gate guide. Water-tightness is achieved with suitable rubber seals.

The selection of the appropriate gate type will depend on gate size and possible upstream head since these factors control the horizontal force on the gate which, in turn, controls the friction load. Gates at diversion structures should be relatively large to reduce risk of blockage by trash and therefore the roller arrangement should be used. Whatever type of vertical gate is used, considerable care has to be given to ensure that neither the gates nor the guides are plane and distorted. Otherwise the loading will not be uniformly distributed and the gates become more difficult to operate.

Slide gates are usually operated by steel spindles which can apply both upwards and downwards forces while roller gates are often operated using steel wire ropes. The gates will then roll down under gravity. Gates that are wider than they are high should have a lifting point at each end of the gate.

8.11.4. Radial Gates

Radial gates are preferred for larger openings because the friction load from water pressure is lower. While the gates themselves are more complex to design and manufacture than vertical gates, they are less demanding for highly accurate installation than large vertical gates. They also require lower lifting effort than vertical gates since part of the load is carried by the pivots. Radial gates are normally operated by steel wire ropes. These raise the gate which will go down under its own weight. The gates are normally designed so that the top of the gate, when closed, is vertical.

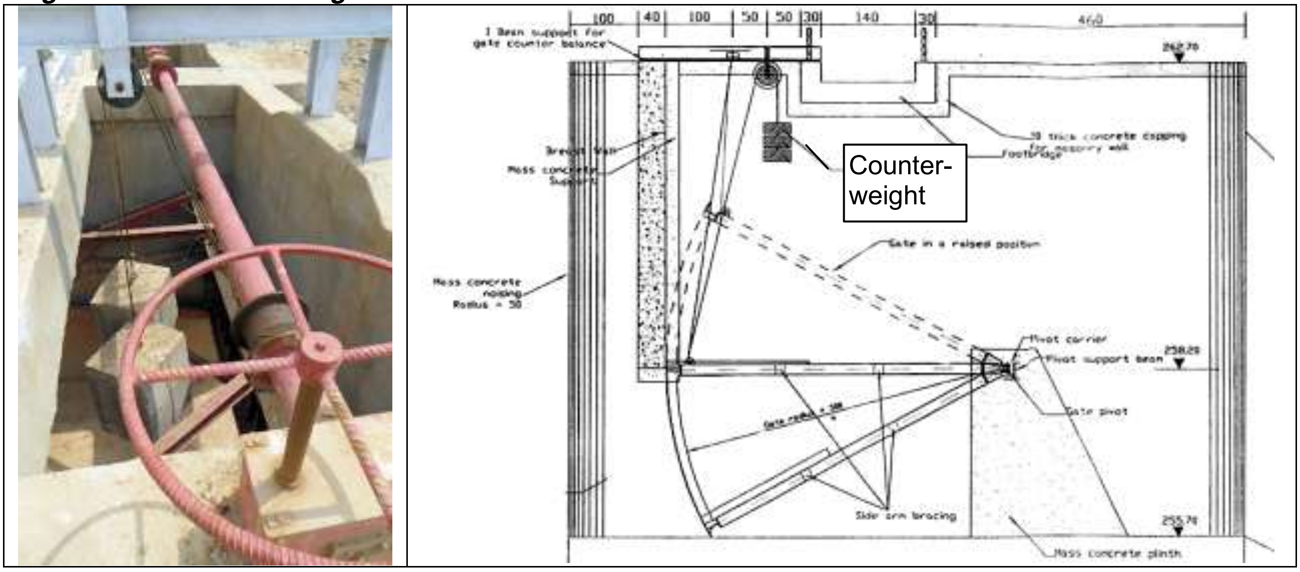
8.11.5. Automatic Gates

Automatic gates can either be electrically operated or hydro-mechanical. The former option is too sophisticated for consideration in spate irrigation. However, hydro-mechanical gates may merit consideration. They normally about a horizontal axis (pivot) and are counterweighted to be controlled by either the upstream or downstream water level. The design water level is controlled by adjustment of balance weights during commissioning. The upper part of the gate needs to be protected by a breastwall. This arrangement may be suitable for sluiceways if there is no need to allow for trash to be flushed downstream.

8.11.6. Counter-weights

One way to reduce the lifting effort of gates and thereby enable faster operation is to provide counter-weights to offset part of the weight of the gate. This mechanism can be used for both vertical and radial gates. The counter-weights are attached to the gate using steel wire ropes passing over a pulley above the gate. The simple arrangement shown in Figure 8-20 can be used where there is a breast wall much higher than the gate such that when the counter-weight is still clear of the water when the gate is raised. An alternative arrangement can be used to reduce the movement of the counter-weight. Care is required to leave the gate with sufficient weight to close against flowing water.

Figure 8-20: Counter-weights for Radial Gate



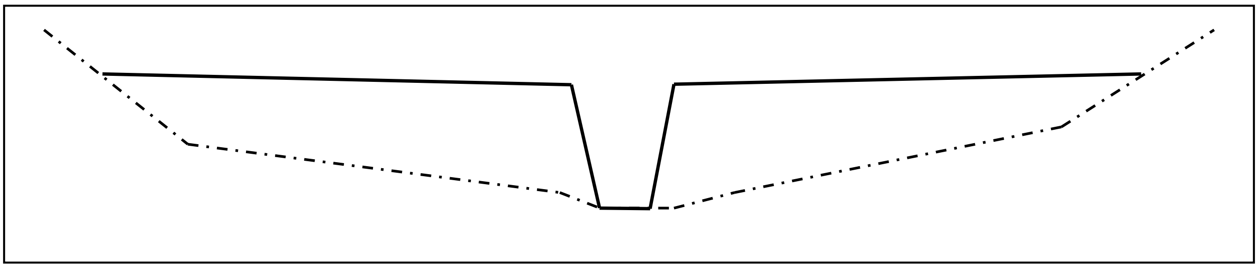
A counter-balanced gate on a sluiceway without a breast wall will require that the lifting gear and pulley are placed high enough that the counter weight remains clear of the water under high flood conditions.

8.12. Spate Breakers

Spate breakers are dams designed to temporarily store the peak of a flood in order to increase the availability and manageability of water for irrigation. Water will not be stored for long enough to cause significant deposition of silt and clay. However, gravel, cobbles and boulders will tend to be deposited although some may be flushed through. Spate breakers have been considered for various countries but the rapid loss of storage capacity due to sediment deposition means that implementation is rare.

The dam will contain either a large orifice or an open slot and will be designed to be overtopped by an exceptionally large flood because provision of sufficient capacity to store such floods would substantially add to the cost. The outlet would either need to be large enough to pass trees or be protected by a sufficiently substantial screen that debris would be held upstream and not substantially affect the outflow. Blockage of the outlet during a large flood would result in accelerated sediment deposition and loss of capacity. An indicative dam design is shown on Figure 8-21. Ideally it would be constructed at a natural constriction in the valley with a rock foundation and where substantial upstream storage can be mobilised. The slot in the dam would be sized to restrict the normal flood flows but must be large enough to pass floating debris such as trees.

Figure 8-21: Possible Section Through Spate Breaker Dam



Construction would probably be of mass concrete or stone masonry to ensure integrity of overtopped. While the dam would attenuate most floods and make irrigation easier, downstream structures would still need to be designed for the full flood flows to provide for conditions such as floods on successive days and loss of capacity due to sediment deposition.

9. CANALS

9.1.1. Introduction

The first problem of getting water out of the wadis has been discussed in Chapter 8. The next challenge is to get the water down the main canal system without excessive management effort, sedimentation or scour. Experience with modernised systems has revealed several consistent problems of which the most significant is the rapid loss of capacity due to sediment deposition. This has been caused by three features of the designs:

(i) Slopes that are too flat and velocities too low to transport the incoming sediment
(ii) Cross sections that are relatively narrow and have limited capacity to hold sediment
(iii) Cross regulator or check structures that further reduce the velocities while water is ponded to command offtakes or fields (this problem can also occur on traditional canals which use early embankments but often the sediment is flushed downstream when the embankment is breached)

The problem is then made worse by rising field levels, particularly in the upstream part of the system where farmers tend to get more water (and sediment). These farmers pond up the water in the main canal for longer while they struggle to get water onto their land. Not only does this increase the sediment deposition but also, unless the designers allowed a head loss between the intake and the head reach of the canal, reduces the flow through the intake into the canal.

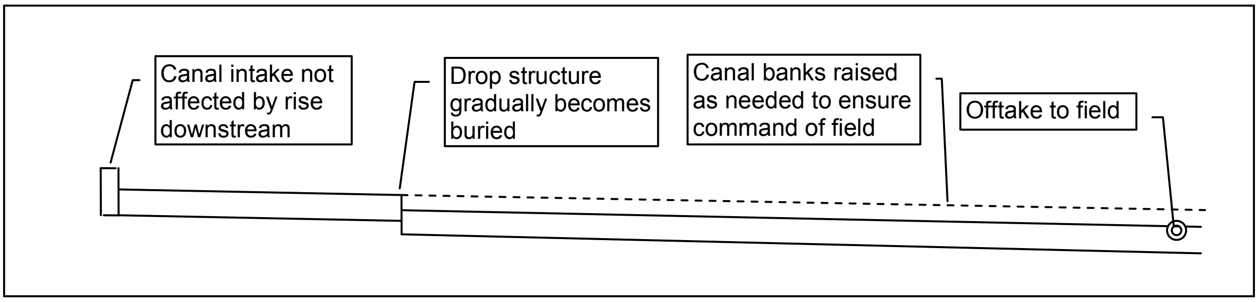
A further problem with engineered canals is the operation of control structures, particularly under conditions of rapidly changing flow and often at night.

The objective of this chapter is to provide guidance on how these problems can be minimised in future designs.

9.1.2. Accommodating Field Level Rise

Design of improved canal systems and structures needs to take account of the progressive rise in the field levels. The first measure is to incorporate a drop between the canal intake and the section of canal commanding the most upstream fields as shown on Figure 9-1. This will delay the time when rising field levels will affect the flow through the intake. The amount of time until the intermediate drop is buried will depend on the height of drop and the rate of field level rise. For example, if the fields rise at 3cm per year and the drop is 1m then the drop structure will become ineffective after 33 years.

Figure 9-1: Providing Canal Drop Downstream of Intake



Selection of a suitable location for the drop structure will depend on site conditions. It may be best to place it near to the intake in order that the first reach of canal is below ground level and reduce the temptation of farmers to start irrigation from that reach. Provision of allowance for field level rise will probably mean that the intake location is moved upstream from the initially considered location. Farmers may need to be reminded that this move upstream is to allow for future change, not to enable an upstream extension in the irrigated area.

While the entire canal system could be initially constructed to command raised fields, this will add substantially to the cost. It is best to leave raising of the banks to be undertaken as needed because the height of the bank will continue to be relative to the raised bed and field level.

Canal structures may be needed for various functions. The main ones are check structures / check/drop structures, offtakes, division structures (which may be a pair of check structures side by side) and crossings. Design of canal structures to accommodate future rises in levels is more difficult. The options are:

(i) Provide a low cost structure that can accommodate up to 0.5m rise and is then replaced
(ii) Provide a modular structure that can be dismantled and rebuilt to a higher level
(iii) Provide a structure that is designed to accommodate future raising
(iv) Provide a structure that can accommodate 1m or greater change in canal level without structure modification

The most appropriate option will be influenced by the type of structure. Weirs are more suited to progressive raising than gated structures where the gates may have to be removed and repositioned. Modular structures may appear to be cost-effective but are inappropriate unless the equipment is available for lifting the various components. Low cost structures made of gabions may also be worth considering, but care has to given to avoiding potential seepage paths through or adjacent to the structures. Solutions where the modifications can be undertaken by the farmers as necessary are to be preferred. Figure 7-8 shows a traditional brick masonry drop structure that has been raised several times by the farmers.

9.1.3. General Requirements

Water distribution structures should facilitate operation using the traditional methods, unless there is a specific requirement from the users for a change in water distribution practice. Designers must give consideration to the performance of distribution structures over a wide range of flow conditions. Traditional irrigation often follows the “upstream first” principle. This is logical because, traditionally, water from the canal is usually diverted by earth embankments. Once these have breached, they cannot be rebuilt until after the flood is over and the agreed operational rules may require that the upstream embankments cannot be rebuilt during a flood season until after the downstream fields have received water. Often, the water users want to continue with the traditional water management practises and the associated water rights and downstream users are unhappy about “improvements” that give upstream farmers a greater ability to take more water with the implicit reduction in water for the downstream users.

The design flows for spate irrigation canals are usually much higher than for canals in normal irrigation systems. For upstream canals where the period of water availability is longer, the capacity of the canal can be determined from the area to be irrigated and the period of water availability as described in 8.1.2. For example, if it is planned to supply an area of 100ha with a 0.60m gross application of water within 10 hours then the required flow is:

$$(0.60 \times 100 \times 10{,}000) / (10 \times 3600) = 16.7\text{m}^3/\text{s}$$

This calculation assumes that the required flow can be diverted for the whole period.

9.1.4. Regime Design

Spate canals encounter a wide range of flows and sediment loads that affect scour and sediment deposition. Design of spate irrigation canals and canal structure should take account of regime conditions. “Regime” means the long-term stable condition and represents the combined effect of the various flow and operating conditions. In spate irrigation an overall rise may be superimposed upon the regime condition to meet the needs of irrigating rising field levels. Canals will tend to progressively fill with sediment during smaller floods and flush out during larger ones. As a broad rule, high sediment loads result in wide, shallow and relatively steep canals.

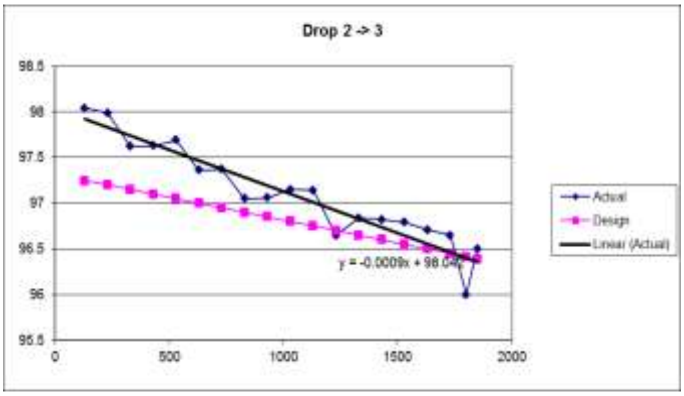
In conventional irrigation, the peak design discharge is used to determine the canal bed slopes and cross-sections. Following this approach for spate canals will result in serious siltation problems at lower flows. This is because spate canals flow at their full design discharge for very short periods of time. Most of the time the canal flow is much lower than the peak discharge and a steeper canal bed slope than that set by the maximum flow is required to avoid sediment deposition. As a rule of thumb, about 70% of the peak discharge could be used to determine the slope and width of spate canals. The capacity to convey the maximum discharge (probably 120% of the design flow) is then provided by increasing the depth and freeboard. There may be some erosion of the canal bed and banks when the flow in the canal is high but, as very high flows are maintained for short periods and will be carrying very high sediment loads, there is limited risk of serious scour problems occurring.

Examination and analysis of the existing canals will enable any modifications to be designed with greater confidence that they will perform satisfactorily. The SHARC software package includes a module containing different regime formula which facilitates the evaluation of prevailing conditions within any spate irrigation system provided some basic information such as channel slopes, cross sections and bed sediment grading are available. If the existing conditions can be modelled using one of the regime formulae, then the same equation can be used to guide the design using appropriately modified parameters. The existing natural slopes of canals should not be changed without an assessment of the possible consequences. The slopes of an engineered canal should not be used for guidance unless the canal has not been maintained for many years and has reached a natural condition. Box 9-1 shows an example calculation.

Box 9-1: Example of Canal Slope Evaluation

Fuad main canal in the Wadi Ahwar project in Yemen was constructed in 1990. By 2008 it was badly silted so cleaning was undertaken. The slope of the canal prior to maintenance (0.09%) was used as guidance for regime slopes on other proposed canals. The slope may still be an under-estimate if the canal is not fully in regime.

The slope of these canals is much lower than encountered in some other spate irrigation schemes. The natural ground slopes are also lower (0.3%) than many schemes, suggesting a lower sediment load and/or finer sediment.



The design of an engineered canal often aims for the most efficient conveyance (ie minimum ratio of area / wetted perimeter) in order to have the minimum cross section and cost. However, where ground slopes are steeper than the hydraulic slope then a less hydraulically efficient design will reduce the number of drop structures because the energy is used in friction losses on the large wetted perimeter. A wide canal also has a greater capacity to accommodate with a small reduction in flow capacity. Providing canals with lower b/d ratios would require drop structures to be employed and would also likely result in significant deposition of sediments in the canal system. An example canal design is presented in Box 9-2.

Box 9-2: Regime Canal Design for Wadi Laba (Eritrea)

Sediment loads in the wadis and in the canals are high, with values of up to 100,000 ppm being obtained from wadi flow sampling. More commonly the suspended sediment loads are in the range of 5,000 to 50,000 ppm except at the tail end of recessions when the low flows become relatively clear.

The canals generally exhibit a large width to depth ratio (b/d), and are very shallow. Ratios of between 50 and 150 are common with flow depths rarely exceeding 0.5 m. Existing canal slopes up to 2% have been observed.

Of the various regime methods tested, that of Chang seems to most closely relate to conditions in the project area. This method has been incorporated into the suite of channel design methods software prepared by Hydraulics Research Wallingford, Design of Regime Channels (part of SHARC).

Chang Regime Method - Canal Geometry for Various Discharges
(Sediment Concentration assumed at 10,000 ppm)

Discharge Q100 (m^3/s)	Discharge Q70 (m^3/s)	Bed Width (m)	Depth (m)	Invert Slopes - %	
				D50 = 5mm	D50 = 1mm
35	25	26	0.63	1.14	0.51
25	18	22	0.56	1.16	0.52
15	11	17	0.48	1.19	0.54
10	7	14	0.42	1.23	0.55
5	4	10	0.34	1.29	0.58

For new irrigation schemes where there are no existing canals to provide a basis for design, slopes will have to be based on measurements for the wadi in the vicinity of the diversion site to determine an appropriate regime formula that gives a reasonable fit to the wadi conditions. The following parameters are required:

- Channel slope
- Channel bed width

- Bed material grading analysis (this may require measurement of a large sample from a pit)
- Estimated flow depth, roughness and velocity

Then the same formula can be applied for the canal design with two main changes:

- The assumed dominant flow is 70% of the design canal discharge
- The grading curve is adjusted to exclude the material which could be expected to be excluded by the intake (provided it is equipped with an effective sediment excluder).

It will be better to design a canal that is too steep than too flat: Should scour start to occur then it is relatively easy to provide an intermediate drop structure to reduce the overall slopes but if the canal is too flat then it will keep filling with sediment and will be a permanent operation and maintenance problem.

Freeboard should be calculated using the full design discharge, Q. Freeboard computations are often based on the design discharge, such as the Lacey formula :

Freeboard : $F_b = 0.2 + 0.15\ Q^{1/3}$

However the USBR formula, based on depth of flow seems more appropriate in these very wide shallow channels. This is recommended for application. The formula is :

Freeboard : $F_b = \sqrt{c\ d}$

with c taken as 0.5 and d being the depth of flow in metres.

The design bank top levels should be the higher of:

(i) full design freeboard above water level at 100% of design flow
(ii) 50% of design freeboard above the water level at 120% of design flow
(iii) 50% of design freeboard above the backwater created by a structure at 120% of design flow

A minimum freeboard value of about 0.3 m would be practicable for smaller canals with $Q \leq 2m^3/s$.

9.1.5. Flow Management Structures

As already stressed in section 9.1.2, canal structures probably have to be designed to accommodate rising field, and eventually, canal levels. Given the potentially limited life working life, the level of investment in canal structures needs to be considered carefully. Operation and maintenance considerations, as explained in section 3.4, indicate that structures should be easy to operate and maintain. Weirs are preferable because their operation is predictable and automatic. Gates, if required, should be relatively large in size and few in number. Box 2-14 shows a complex canal structure with numerous gates.

Canal structures may be required to control flows, particularly where the flow is divided between two branches. Division structures are important and may be one of the most justifiable investments. In many existing systems it has been agreed that the flow at a particular location is divided into fixed shares but often there is no structure to ensure the shares. Any imbalance in the shares can cause disputes and may not represent the best use of the water.

If possible, fixed proportional weirs should be used to provide automatic division of flow over the whole operating range. However, when the flow is low the operating rules provide for only one branch to take the flow. In this case a gate can be provided within each weir to be opened, when appropriate), under low flow conditions. The top of the gate should be at weir crest level and can act as a weir when closed. Division structures are best designed as two similar structures side by side (these may be check structures or offtakes according to conditions) so that they have similar hydraulic properties. The design needs to take both present and future tailwater conditions into account since a high tailwater into one branch will change the balance of flow distribution.

Hydraulic structures for spate irrigation schemes should be designed to operate satisfactorily for a range of flows such as 50%, 100% and 120% of the design flow. The water level at 120% of design flow must lie within the design freeboard. Where structures include gates, the design must take account of the possibility of the possibility of high inflows and closed gates. It is therefore usually necessary to make provision for water to flow over gates and / or weirs at the site of the gates.

Figure 9-2: Combined Gate and Weir Cross Regulator

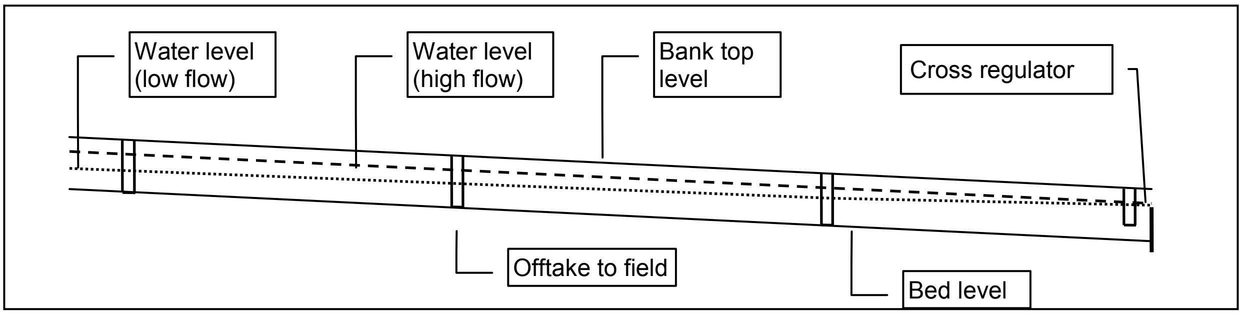
Combination gate / weir cross regulator. If the gates are closed then the water can pass downstream	The same structure during a flood. Note the turbulent flow which will affect any careful calculation of flow distribution

Figure 9-3: Potential Problem Canal Structures

Where does all the flow go when the gates are closed?	Farmers using available material to try to raise water level to command the fields

The normal upstream-first heirachy for spate irrigation means that the flow capacity of offtakes may need to be sufficient to take the entire incoming canal flow if there is only one offtake commanded by a cross regulator. Alternatively there may be an offtake on each side of the canal or several offtakes along a reach served by a cross regulator. However, the latter gives variable flow into the offtakes as the backwater effect of the cross regulator changes depending on the canal flow as shown on Figure 9-4.

Figure 9-4: Backwater Effect of Cross Regulator



Engineered large capacity offtakes are expensive. Open channel offtakes are less expensive than culverts and less vulnerable to loss of capacity when the field levels rise. Whether offtakes need gates or other means of closing will depend on the canal water level when any cross regulator structure on the canal is open. However, whereas water level at an offtake adjacent to a cross regulator will be controlled by the cross regulator, there will be less impact on any offtakes further upstream in the same reach.

Therefore, operationally, the optimum canal configuration is to have only one offtake per reach adjacent to a cross regulator and to rely on field-to-field water distribution to the land between these offtakes. Effectively, a strip of land through the fields may function as a distribution canal which will be planted after the irrigation season is finished.

9.1.6. Crossings and Inverted Syphons

Inverted siphons may be required to enable canals to cross wadis or low-lying land. As noted in section 8.4.5, it is not desirable to have intakes on opposite banks of the wadi at the same location. One intake serving two canals plus an inverted siphon under the wadi is a possible design alternative. However, design of inverted siphons must take account of the high sediment loads that may occur in spate systems. They are very vulnerable to blockage during either low flow or abnormal sediment load conditions even if designed for high velocities. Such blockage might cut irrigation supplies for the balance of the season. Wadi Mawr and Wadi Rima in Yemen both have inverted siphons and both have flushable sediment basins upstream to minimise the sediment being transported downstream. Structure costs will enforce a constraint on flow capacity that may not be acceptable to the farmers. At both of the above-mentioned schemes the farmers have reactivated traditional intakes to supplement their irrigation supplies.

Figure 9-5: Examples of Cross-drainage Structures

Wadi Mawr inverted siphon inlet with trashrack	Wadi Laba culvert outlet blocked after large flood

Ideally, an inverted siphon would be designed as a culvert with a gravity outfall so that it is self-draining but, even so, the structure will be difficult to clear if blocked. However, even though a culvert or inverted siphon can be designed to have a self-cleaning velocity for all but the largest material, the canal downstream of the structure is unlikely to be not be designed for such high, and potentially erosive, velocities. Excess sediment may there first fill the canal and then back up to block the structure as happened at Wadi Laba (see Figure 9-5). It is, therefore, best to avoid any siphons or culverts unless they are associated with very robust sediment management facilities.

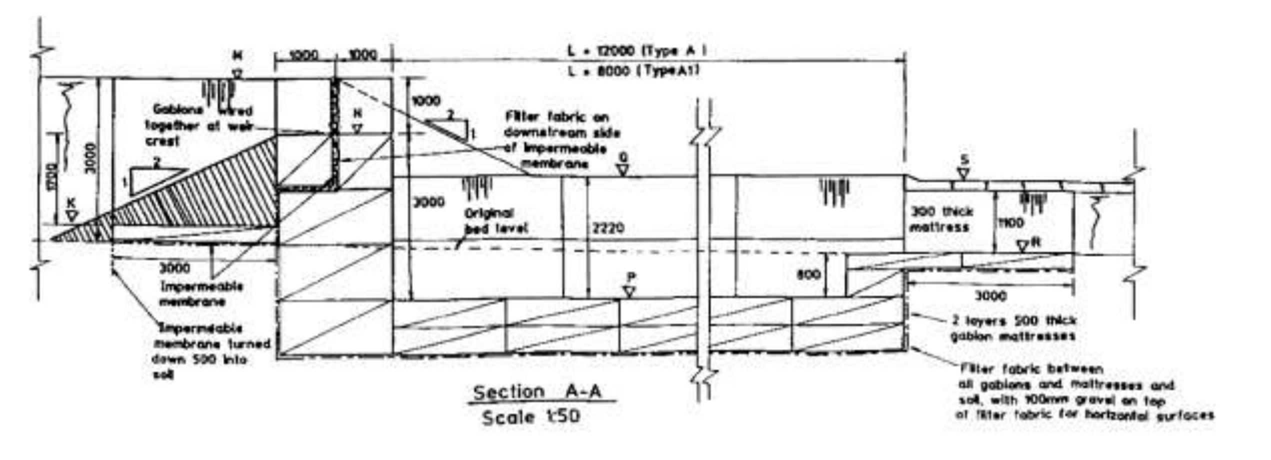
Culverts along canals such as for road crossings should be given generous capacity for two reasons: (i) to ensure that excess flows do not back up and overtop the upstream banks and (ii) to reduce the possibility of blockage by trash. A further possible cause of damage is seepage under a culvert washing out its foundations should it become blocked.

9.1.7. Drop Structures

Use of regime design for canals carrying significant sediment loads will indicate steeper design slopes and therefore fewer drop structures than indicated by clear water design methods. However, it is likely that some drop structures may be needed, if only as tail escapes where any excess flow can drop off the end of the irrigated area onto the lower natural ground.

Drop structures need to be designed with a view to them being raised in the future as both the land and canal levels rise due to sediment deposition. The traditional masonry drop structures used in Wadi Zabid (see Figure 7-8 for an example) present one possible solution. Each raising of the structure adds another step and the lower part of the structure becomes buried. Relatively low cost gabion vertical drop structures are another alternative that are suitable for raising or complete replacement. However, where the soil is silty, care is required to prevent any seepage paths that can quickly become failures.

Figure 9-6: Example Design of Gabion Canal Check / Drop Structure

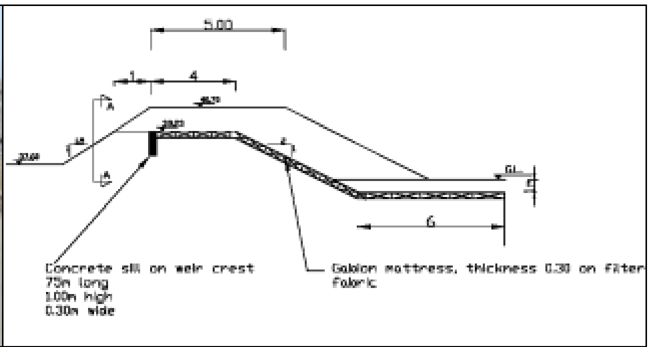


9.1.8. Rejection Spillways

Under high flood conditions excess flow will still pass through a fixed orifice or gate to enter the canal system unless a gate is operated to restrict the flow. It is therefore normal practice to provide a rejection spillway in the first reach of canal which allows some of the excess flow to spill back to the wadi.

The effectiveness of the spillway is in proportion to its length. The spillway will be more effective if a further flow control structure is provided on the canal just downstream of the spillway because water level changes at the structure will be much more sensitive to excess flow than in an open trapezoidal channel. Unit discharges over the rejection weirs are not large so gabion structures are appropriate and cost-effective (see Figure 9-7 for examples). A fixed orifice control or undershot gate on the canal would be the most effective at causing rejection, with the top of the orifice being set at spillway crest level. This type of structure would also not impede normal flow or sediment transport along the canal.

Figure 9-7: Gabion Rejection Weirs

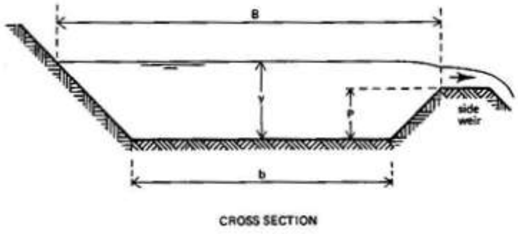
	
Gabion box rejection weir at Wadi Laba just upstream of a canal control structure	Design for rejection weir using gabion mattresses and concrete sill

Calculation of flow over a side spillway has to take account of the drop in water level along the canal resulting from the flow going over the rejection weir. An example of the flow calculation for a side spillweir is given in Box 9-3.

Box 9-3: Example Flow Calculation for Side Spillweir

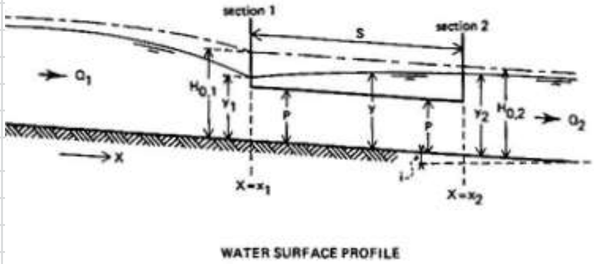
Design of escape (side weir) in first reach of Hanad Left main Canal

The theory on flow over side weirs is only applicable if the area of water surface drawdown perpendicular to the centre line of the canal is small in comparison with the water surface width of this canal.
In other words, If $Y-P_1 < 0.1$ B.



In Hanad Left canal the water has a water surface of about 20 m and P of the projected side weir is 1.63 m (10 cm obove FSL)
Y is assumed maximal 2.50 m, so that the height over crest is approximately 0.80 m, which is only 0.04 B.

Also, the expected flows are sub-critical, so that we may assume that the energy line is parallel with the bottom profile of the canal.



Therefore, we may write:

$$H_{0,1} = y_1 + Q_1^2/2gA_1^2 = y_2 + Q_2^2/2gA_2^2 = H_{0,2}$$

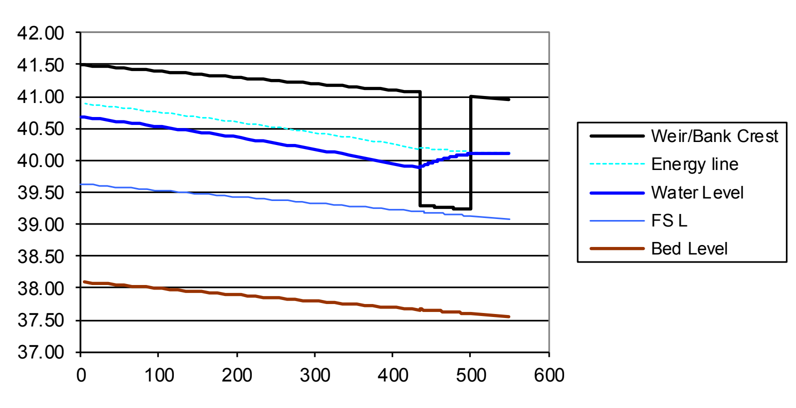
The shape of the water surface and length of the weir have been calculated in a step-wise calculation, taking steps of 1 m.
(steps of 0.5m, 2m or 5 m in the calculation give approximately the same result)

From: Discharge Measurement Structures

The result is calculation is:

Q_1	105	m^3/s
Q_2	38	m^3/s
Qs (over weir)	67	m^3/s
Length of weir:	65	m
Y_2 - P	0.87	m
Y_1 - P	0.61	m

Also, if all the gates are closed, the full 105 m3/s will flow over the weir.
In that case the freeboard near the control structure will still have a freeboard of 0.6 m



longitudinal section through right bank of Hanad Left canal, km 0.000 to km 0.550

levels for km 0.000 - 0.550

Chainage	0.000	0.435	0.500	0.550
Bank Level	41.50	41.07	41.00	40.95
Weir crest level		39.30	39.23	
Standard Bank level (0.90 m above FSL, not shown)	40.53	40.10	40.03	39.98
Emergency water level (Q intake is 105 m^3/s)	40.70	39.90	40.10	40.09
Full Supply Level, FSL, (0.35 m3/s)	39.63	39.20	39.13	39.08
Bed Level	38.10	37.67	37.60	37.55

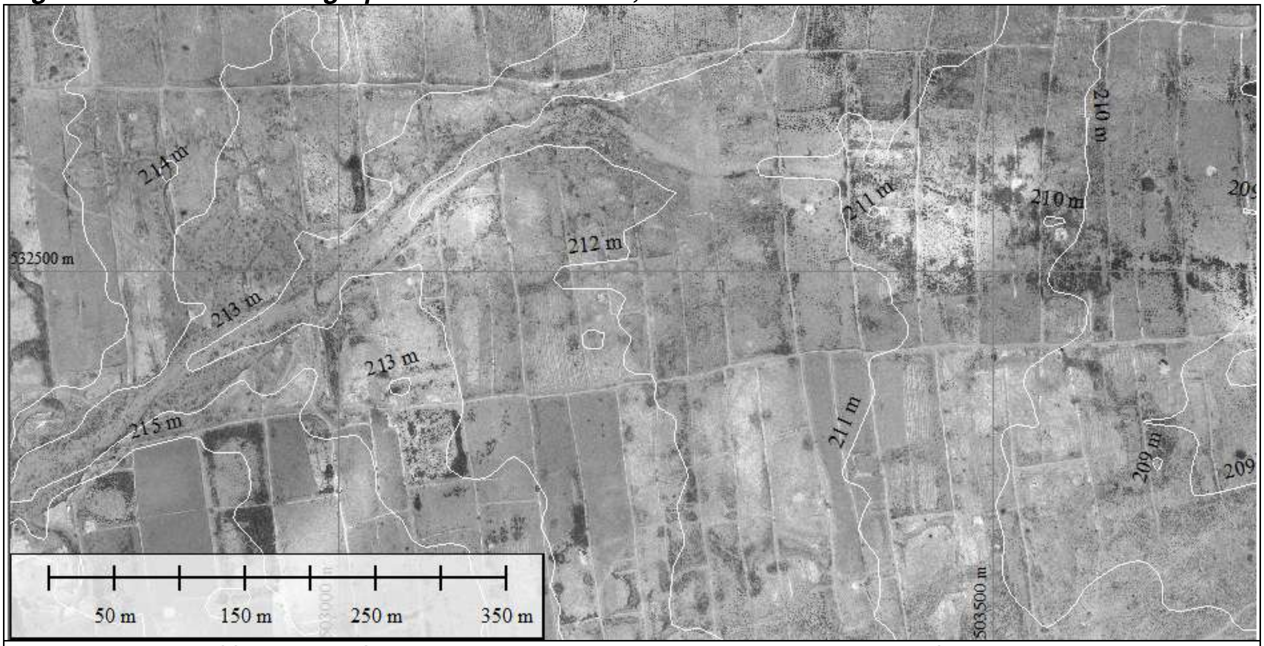
The escape weir (spillway) is 65 m long and is located 50 - 115 m before the first control structure in Hanad Left Canal, on the right bank so that the water spilling over the crest will flow back to Wadi Ahwar.
The escape weir starts at km 0.435 and ends at km 0.500.
Level of the weir is 0.10 m above FSL.
The banks of this section of canal should be raised by approximately 0.85 m compared to the original design without escape.
The control structure at km 0.550 shall get a modified design with the aim to limit the flow to the downstream section in case of an emergency.

10. ON-FARM WORKS AND WATER MANAGEMENT

10.1.1. Introduction

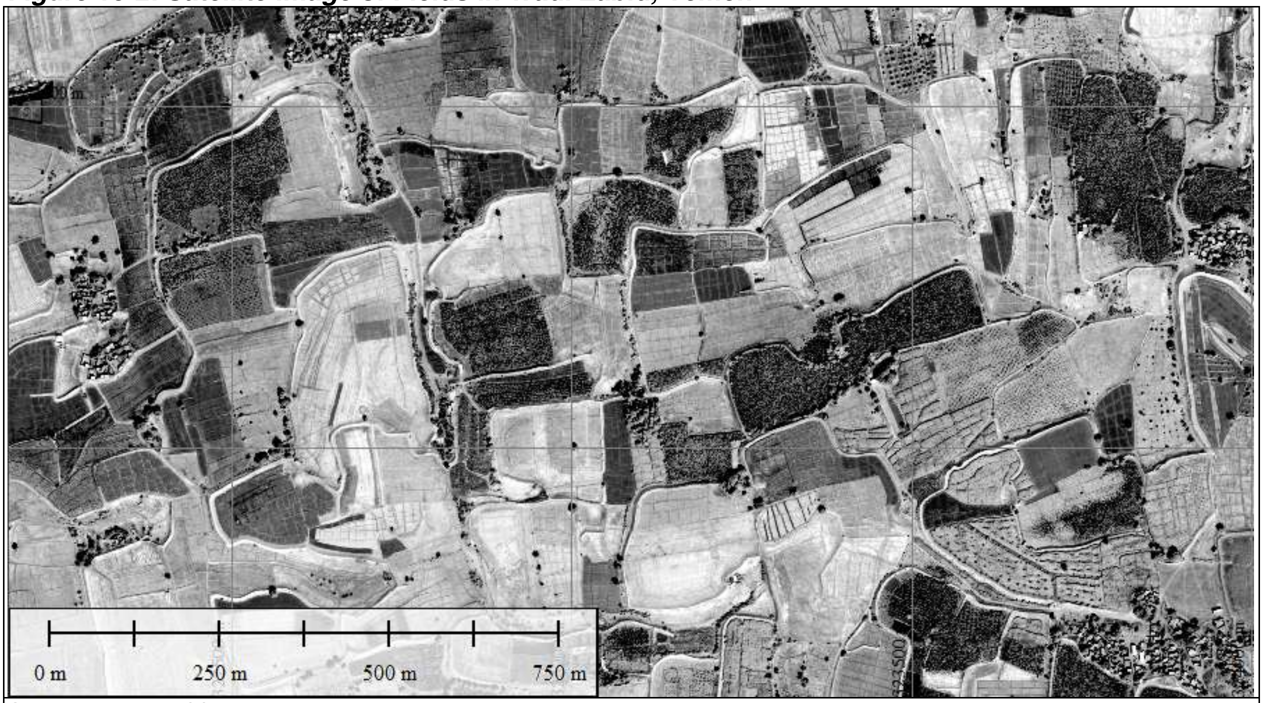
A few modernised spate irrigation schemes have included construction of a conventional network of canals with individual offtakes to each field. However, such an arrangement is only feasible when water is available for long periods because offtake capacity (which is relatively low to limit overall costs) limits the rate of irrigation. Also, this arrangement needs good land grading to provide uniform application of water otherwise the water can all infiltrate near the entry point and not reach the far parts of the field. If the irrigation water has a high sediment load then the infrastructure will progressive become unusable as the fields rise due to sediment deposition. A comparison between having field-to-field distribution and having individual offtakes to each field is given in Table 10-1.

Figure 10-1: Aerial Photograph of Fields in Sheeb, Eritrea



Aerial photograph of fields in the Sheeb area showing the end a canal merging into the fields

Figure 10-2: Satellite Image of Fields in Wadi Zabid, Yemen



Satellite image of fields in Wadi Zabid, Yemen, showing very irregular layout.

In most spate irrigation schemes field-to-field conveyance of water is practised. This involves low investment in infrastructure and can accommodate field level rise. The normal approach to the application of spate water is to apply a large flow of water to each field, which is surrounded by a bund at least 0.6m high, in turn. This enables the whole field to be flooded. Once the water depth has reached a pre-determined amount (typically knee-deep) the water is released to the next field.

Table 10-1: Comparison of Irrigation Options

Aspect	Field-to-field irrigation	Individual offtakes
Land take	No land is required for the distribution canals because water passes through the fields	Land required for secondary and tertiary canals is estimated to be about 10 percent of total area, though at the end of season canal beds are sometimes cultivated
Construction cost	No significant infrastructure development cost	Requires expensive investment in gated flow control and division structures and field offtakes with a high flow capacity
Maintenance cost	Regular repair required to in-field scour on lands result from the breaching of downstream bund	Potential cost in removal of sediment from canals
Ease of maintenance	Farmers responsible for repair of field bunds in accordance with local rules	Farmers need to contribute to system operation and maintenance
Enforcement of water rights	Water distribution usually well regulated by local rules, although timing of breaching to release water from one field to the next can be a source of conflict	Gated control structures make it possible to divert water at any time and in contravention of established water rights. Difficult to measure when fields have received the correct share of water
Ease of operation	Relatively simple because the focus is on the field currently being irrigated.	Requires continuous monitoring and adjustment of gates along a canal
Flexibility	Is reasonably tolerant of wide ranges in inflow	Flows into individual fields will vary according to flow in canal and distance upstream of a control structure
Sediment management	Facilitates uninterrupted flow of water from canals through to the fields minimising sediment deposition in the canals. Continuous movement of water helps sediment reach the downstream fields	Sediment deposition in canal occurs when water is ponded in canals to command a series of offtakes
Uniformity of irrigation	Rapid inundation of whole field promotes relatively uniform irrigation	Unless inflow is high, land near the offtake and low-lying areas receive most water
Water distribution and management	The breaching of the field bunds helps to remove large quantities of sediment from the command area and reduce the risk of rising command areas getting out of command	Less scope to remove sediments from the command areas naturally – as signified by very high field bunds. In flat areas this can be a significant problem
Land levelling	Help to level land in irrigation fields because more sediment is deposited in low lying areas	When plots are large, the lack of levelling will create uneven irrigation
Vulnerability to damage	Damage of upstream field bunds may jeopardize flows to lower areas. Large floods may cause additional damage	Group water supply is not vulnerable to breaking of individual field bunds. Gated structures reduce risk of scour and improve water application regulation
Crop damage	Possible damage to growing crops when passing water downstream. Replanting may be required	No risk of crop damage due to water passing through the fields
Out-of-season irrigation	Smaller floods do not reach tail-end plots. Smaller floods later in season are not diverted because upstream plots are cultivated	Individual offtakes allow for more flexibility and the possibility of irrigating downstream fields even later in the season without damage to upstream crops
Longevity	Will automatically adapt to rising levels due to sediment deposition	Depending on rate of sediment deposition, system may cease to function within 20 years
Other factors	Large drops between fields will still require drop structures to minimise scour damage	

10.1.2. Field-to-Field Irrigation

Under this system, the most upstream field receives the water first and it is allowed to pond to a pre-determined depth (typically knee deep). When that depth is reached the field bund is breached and the ponded water is released to the next field. Meanwhile, any incoming flow passes through the first field to the next one. This process is progressively repeated until water reached the most downstream field in the block (provided the flood is long enough).

The main advantage of this system is that there is no investment in, or land lost to, a separate canal system. Disadvantages include any crop in the upstream fields being damaged if there is a flood and the downstream land is still entitled to water, and the lack of separate channels means that more water will percolate en-route and less reaches the downstream area (an advantage for the intermediate fields).

Figure 10-3: Field-to-Field Irrigation in Eritrea

Field-to-field irrigation can work very successfully when the level difference between fields is up to about 0.5m. In Sheeb, Eritrea, (see Figure 10-1) the typical ground slope is about 1% and the fields are about 50m wide, so the drop between fields is about 0.5m. With this difference in level the velocity of water as it passes from one field to the next normally does not result in much erosion although large floods can be damaging as shown in Figure 10-4.

Often the flow path through fields is arranged to be indirect, that is the inlets and outlets are not aligned, in order to lengthen the overall flow path and reduce the overall water slope. Farmers often avoid a larger drop between fields by passing water laterally along fields in what is effectively the same terrace to where the drop is smaller.

Figure 10-4: Erosion Between Fields

	
Erosion after breaching of field bund	Major erosion gully at large drop between fields

10.1.3. Full Distribution Network with Individual Offtakes

Provision of a full canal network with offtakes to each field allows more precise control of water delivery. This approach may be appropriate at the upstream end of a spate irrigation system where there is a prolonged base flow between floods. It is unlikely to be cost-effective in areas where most of the irrigation water arrives in short floods and/or has his sediment content. High design flows will make the system expensive to construct while high sediment loads will result in rising fields that canals and offtakes can no longer command.

10.1.4. On-farm Structures

If a full irrigation network can be economically justified then the full range of irrigation structures will be required as for perennial irrigation systems, except that higher design flows will necessitate larger structures.

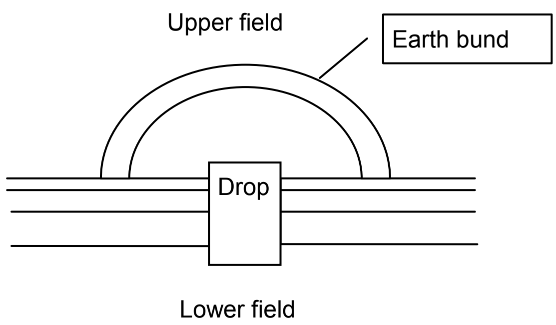
For the more likely situation of field-to-field irrigation then the key structures that may be required are between-field drop structures to minimise erosion as water is passed between fields. Traditionally, these structures were found necessary the farmers would construct them of the locally used building materials such as stone masonry, brick or, more recently, concrete blocks as shown on Figure 10-5.

Figure 10-5: Farmer-built Drop Structures

	
This drop is several metres high and is constructed of concrete blocks	This drop is about 1m high and is constructed of stone masonry

Where mechanised handling of materials is available then stone rip-rap may be cost effective. Gabion mattresses or pre-cast units could also be used to form a chute where a large number of structures are planned.

Figure 10-6: Earth Bund Upstream of Drop Structure

	
Normal practice is to provide a semi-circular earth bund upstream of the drop	Example of embankment upstream of a drop structure (Wadi Zabid)

It is unusual to provide a gate for these structures. A gate would not only add substantially to the cost but also could be used to over-ride the normal rule that once the embankment is breached then water is allowed to flow downstream. It is normal practice to close the structure by building a semi-circular earth bund upstream of the structure that connects to the banks each side of the structure as shown on Figure 10-6. This avoids the risk of seepage along the interface between embankment and structure which could cause a premature washout of a soil plug. Breaching of an earth bund can be at least as fast as opening a gate.

10.1.5. Land Levelling and Terracing

If contour bunds are built then spate irrigation will naturally form terraces as shown on Figure 4-1. The process is self-levelling because the most water will accumulate and will deposit the most sediment in the lowest places. Progressively, depending on the volume of water and sediment content, the sediment deposition will level the field. Therefore specific land levelling is not necessary provided the spate water has a moderate to high sediment load and water is quickly applied to the fields to achieve overall inundation. The number of years required to create fields that will grow a reasonable crop depends on the amount of water and sediment received but a minimum of 5 to 10 years is likely.

Figure 10-7: Creation of New Fields Using Bunds

	
Stone bund waiting for flood to bring sediment	Earth bunds forming new fields

Spate irrigated fields naturally form terraces because each is level but within overall sloping terrain. What is less easy to understand is why a wide range of level differences between fields progressively develops. The main factor is probably a relative difference in the amount of irrigation water and hence sediment that is received. When the height difference between two fields becomes excessive, the farmers tend to route the flow via other fields. This will result in the lower field receiving flow even less frequently and the height difference between the fields will further increase.

11. COSTS, RISK AND VALUE ENGINEERING

11.1. Costs

11.1.1. Quality Requirements

The specified quality requirements should be based on what is easily achievable provided this is suitable for the purpose. Higher than basic quality specifications should only be requested when essential for the performance and function of the works. Examples of where materials and workmanship specifications can often be relaxed compared with normal engineering practice are:

- Materials to be what is locally available if there are existing examples of satisfactory work using these materials
- Concrete strengths should be appropriate (but subject to good practice such as minimum water content and curing)
- Compaction of earthworks (unless subsequent consolidation would cause problems, such as under structures)

11.1.2. Minimising Costs

The lowest construction costs will be achieved by the maximum use of local resources, both for materials and manpower (including both project beneficiaries and local contractors), although there may be additional costs for construction supervision and contract management. Making maximum use of local resources also provides capacity building at the community level which can enable the community to undertake further construction, repairs and maintenance without external support. Costs can be further reduced by requiring beneficiaries to contribute to the cost, such as through providing labour. Small works can be contracted to the community themselves to implement, either directly or through village level contractors and artisans, thus enhancing the capacity of the community to implement more works in the future.

The use of large contractors simplifies procurement and contract management but is unlikely to provide capacity building at the community level. The construction costs are likely to be 20% to 30% higher. However, a large contractor may be justified either to bring in resources to meet a demanding programme or to bring in specialist skills or equipment not locally available.

11.1.3. Construction Materials and Methods

The traditional materials for spate irrigation comprise those materials that could be found in the vicinity. Large embankments have been built with animal powered scrapers, but this type of equipment cannot easily handle coarse gravel and cobbles. Traditional diversions in the upper wadis therefore tend to built of brushwood (acacia bushes interlock very well) weighed down with hand-placed stones. Lower down the system it is feasible to build embankments in wadis and canals using animal power or tractor-mounted scrapers. Where bulldozers are available, they can be used to build embankments in the upper wadis.

For permanent structures, the preferred construction materials and forms of construction should reflect the locally available materials and skills in using them. This is particularly important for smaller works which could be implemented by water users. In addition, designs that can be implemented by the users using local resources are designs that can be replicated by the users in the future without external intervention.

Walls of masonry, mass concrete (possibly using selected sand/gravel wadi bed material) or concrete blockwork (if local block production capacity exists) may be preferable than reinforced concrete because a lower level of skill is required, and are normally less expensive. Gabions are often used, where there is suitable stone, but have the disadvantage that the wire materials are not often locally available and have to be imported. Gabions are vulnerable to damage under certain conditions. The wires are vulnerable to abrasion by the coarse bed material and may be snagged by large trash as shown on Figure 11-1.

Where the spate irrigation system is large, the most cost-effective materials may change with distance from the mountains. Masonry may be cheapest at the upstream end, near the mountains, and mass concrete cheapest at the downstream end, where only gravel is available. However, the overall design of structures would be unchanged.

Figure 11-1: Example of Damage to Gabion Wall

One side wall of the stilling basin of Engulet weir is constructed of gabions. These are progressively being broken as debris in the turbulent water gets snagged in the wires and breaks them. The damage is at the bottom of the wall which can be expected to eventually collapse.

11.1.4. Cost Estimation

Cost estimation is an important part of the design process. Unless the designer appreciates the cost of what is being proposed and can easily compare the costs of alternatives, there is the risk that an unaffordable solution may be proposed. The cost estimation can be based on either of two methods:

- Evaluation of recent bids for similar works in the area. In the absence of irrigation works contracts, roads contracts often provide a good guide. They are similar in nature to irrigation, comprising spread-out earthworks and isolated structures. The average of several bids should be used, both to average out contractors' different methods of pricing and distribution costs within a bid and because the lowest bid for a contract may not be a good indicator of the correct pricing to provide the required quality of works. Often, the lowest bidder wins a contract either because they were desperate for work or they made a mistake in pricing. They will then put additional effort into claims in order to avoid making a loss. Such costs need to be reflected in the cost estimates for the work.
- Derivation of unit rates from the basic costs of people, materials and equipment combined with assumptions on production and overheads. This method makes it easy to calculate updated unit costs should input costs change and also provides a systematic method for estimating the costs of works for which unit costs are not available. The cost build-ups also make it easy to identify the major input costs which may reveal where costs can be further reduced by changing some work items.

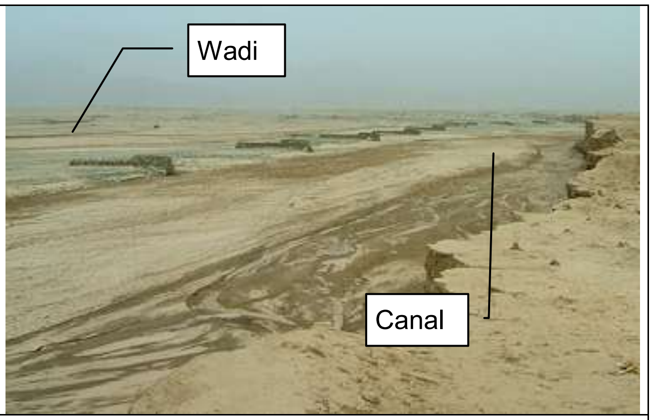
Ideally, cost estimation will combine both methods. Recent construction costs can provide the data to check and improve the quality of unit rate build-ups.

11.2. Risk of Failure

Normal engineering design practice aims to produce safe solutions that may only fail in extreme circumstances. Factors of safety are applied to both strengths (assumed to be lower than expected) and loadings (assumed to be higher than expected) in order to make the likelihood of structural failure remote. The nominal design life is often assumed to be the project life assumed in any economical or financial analysis (typically 30 years) although the actual life of well-built structures may be substantially longer. However, the relatively low economical and financial benefits of spate irrigation make it difficult to justify the use of normal design practice. Furthermore, under spate irrigation conditions, structures may need to be modified or abandoned within 30 years on account of rising land levels.

The other threat to the longevity of spate irrigation infrastructure is a severe flood. The incidence of these is unpredictable and whatever the event that is provided for, there is the risk that something bigger will arrive within the design life. Figure 8-2 shows the probability of floods of a specified probability occuring within a specific period.

Figure 11-2: Examples of Flood Damage

	
Guide pier at Mai Ule weir after a major flood. This component was damaged (but easily repairable) while the other parts of the structure were intact.	Gabion spur protection was ineffective because a large flood entered the canal at the upstream end and washed out the land behind the spurs

An additional uncertainty is the accuracy of any estimates of the design flood magnitudes, particularly where these are derived theoretically without any good quality gauging or flood peak measurements. Allowing for such uncertainties by allowing generous margins for error in the design can substantially increase the costs. A designer should be able to determine which modes of failure can be progressive, in which a local failure can turn into a catastrophic failure, and which local failures will remain as local damage to be repaired after a flood. For example, overtopping of a headwall may not damage the wall itself but may wash out the material behind it. This can be catastrophic if the wall then collapses. Existing schemes will often contain examples of works which have either survived or failed. The use of successful existing structures as the basis for new designs can provide the data for appropriate design parameters.

11.3. Seeking Best Value

Spate irrigation is usually associated with subsistence agriculture using an erratic and uncertain supply of water. As such the returns on the investment are relatively low and any cost-benefit analysis will indicate that only modest levels of investment are justifiable. The engineering challenge for spate irrigation is to provide improvements that are compatible with the relatively poor and variable economic benefits that usually prevail in spate irrigation systems. In some situations economics are over-ridden by poverty alleviation or other objectives which would increase the acceptable level of expenditure.

However, this is still likely to leave a designer with the challenge of creating robust solutions that perform and survive under the flood conditions that prevail in spate irrigation schemes, although value engineering may make it appropriate to reduce the factors of safety inherent in the normal design process. Alternatively, it may be preferable to provide structures that have a significant risk of failure. A low-cost structure that needs repair after 10 years may be better value than a structure with twice the cost that survives for 30 years.

Cost effective engineering is more likely to be achieved through development of proposals in close coordination with the beneficiaries (that is the farmers), particularly if they are required to make a contribution towards the costs. The engineer must not underestimate or ignore the knowledge and experience of farmers in existing spate systems who should be encouraged to propose what improvements should be made to the existing infrastructure. In addition to such proposals building on the farmers' experience, implementation of the proposals, if technically sound and financially viable, will leave the farmers with a stronger sense of ownership and responsibility.

While both the farmers and the engineers may prefer interventions involving permanent structures, one alternative that should not be overlooked is the procurement of a bulldozer to help manage the floods by constructing simple embankments. This is the modern version of the traditional solution and is easily adapted to rising command levels. The capital investment is relatively low and the running costs are also not very high, given that the equipment is used for only a few months per year. The two drawbacks are (i) more water will be lost downstream than with engineered structures; and (ii) ownership and responsibility for the equipment operation and maintenance. Private bulldozers are available for rental at some of the modernised irrigation schemes in Yemen because simple flow training works are required to supplement the engineered structures. The rental costs are affordable to groups of farmers and the owner has the responsibility to keep the equipment working otherwise he will get no income.

ANNEX A: COMPARISON OF ENVISAGED AND ACTUAL IMPROVEMENT WORKS CARRIED OUT IN WADI ZABID AND WADI TUBAN[15]

Rehabilitation and Improvement Works as included in the PAD and actually undertaken.		
Category of work	**Quantity As per the RFP**	**Actual quantity designed (approx.)**
a) A. WADI TUBAN		
(a) Repair or replacement of radial gates at diversion weir	48	80
(b) Repair or replacement of gates in canal control structures	1,024	1,828
(c) Improvement or replacement of diversion and canal structures	• 13 division/canal structures • 5,600 m³ of concrete • 4,700 m³ of stone masonry • 11,000 m³ of gabion work	• 184 as follows: 9 existing diversion structures, 16 improved traditional diversion structures and 159 new/improved canal structures.
(d) Desilting and reshaping of main, secondary and tertiary canals including installation of sediment basis at the head of main canals	• 170,000 m³ embankments • 580,000 m³ excavation	• 5,300m. Farmers did not request this type of work.
(e) Rehabilitation of roads, mainly in the form of shaping and gravelling	47 km	44 km of which 5 km was designed to a higher standard in order to allow for future black topping
(f) Wadi protection works for villages.	• 11 villages • 2. 8 km length	11 locations in total: 3 locations in priority works and 8 in participatory works.
b) B. WADI ZABID		
(a)Improvement of existing diversion structures, including improving silt exclusion arrangements at the intakes, raising weir crests, repairs and alterations to intake gates, new head regulators and sluice	• 2 Diversion weirs: 3 and 4. • 2 new head regulators • 1 new sluice	• 2 Diversion weirs 1 and 3. • 3 new free off takes with headworks and bed bar (Jarahazi, Bagr & Mansury) • 2 Division structures (Weir 5 + wadi Nassery) -
(b) Linking of traditional ogma systems in Wadi Nassery through new canals to diversion 4.	1	• At Wadi Nassery studied but not implemented as farmers objected that they would lose land required for the new canal. • 1 feeder canal & ancillary structures at Mansury
(c) Desilting and reshaping of the canal systems including construction of new canals and sediment basins at the head of the primary canals	550,000 m³	Desilting and reshaping itself not done, due to farmer objections about loss of land resulting from increased channel width, but canal capacities increased at 9 locations
(d) Improvement of canal structures including cross regulators, turnouts and drop structures	• 10 cross regulators • 26 turnouts • 32 drop structures In total 68	• 12 Cross regulators • 67 turnouts • 70 drop structures • 89 Miscellaneous In total 238
(e) Rehabilitation of roads, mainly in the form of shaping and graveling	33 km	Total of 26.7 km of road designed of which 25.8 was required to be to a standard to allow for blacktopping and a further 7.1 km of road was surveyed and then cancelled.
(f) Wadi protection works	• For new splitter weir upstream of weir 4 • For 1 village between diversion 3 and 4	• 1 bed bar and divide wall u/s of weir 4 • Village/land protection undertaken at 66 locations (major works at Matea, Al Marra and Mahal Al Skeikh villages + major work which was later deleted, however was designed)
(g) Repair canal gates	None envisaged	Gates for 3 WUAs repaired

[15] Designs were required for rehabilitation works at Tuban and Zabid. The Description of the Services in the RFP referred to two tables T1 and Z1 and text, which summarized the indicative works to be designed at each scheme. The indicative work required to be designed was summarized in Annex 2 of the PAD.

ANNEX B: FIELD ESTIMATION OF SMALL CATCHMENT SEDIMENT YIELD

Yield = 2650 $A^{-0.13}$

Where:
Yield = The annual sediment yield, t/km^2
A = The catchment area km2

The procedure is:

a) Locate the proposed diversion point on a 1:50000 map. (A GPS is useful) Mark and measure the upstream catchment area from the 1:50000 maps. Calculate the average catchment slope from the elevation difference between the catchment boundary, and the river bed at the intake location, divided by the distance, measured along the main stem river, from the catchment boundary to the diversion.

b) Carry out transect walks across the catchment, and with the assistance of local informants score the three catchment factors shown on the check list on the next page.

c) Estimate the uncorrected catchment sediment yield from the equation above.

d) Calculate a correction factor (F) to account for catchment condition as:

$F = Slope^{0.3} * SASE^{1.2} * STD^{0.7} * VC^{0.5} /500$

Where:
Slope = River slope (as metres of elevation / metres of length) from the catchment boundary to the diversion
SASE = Signs of active soil erosion (Score from catchment characterisation)
STD = Soil type and drainage (Score from catchment characterisation)
VC = Vegetation condition (Score from catchment characterisation)

d) Multiply the predicted sediment yield by the correction factor F to obtain an indicative sediment yield to be used in the subsequent calculations.

Characterisation is carried out by completing a checklist, shown above, during a rapid field appraisal. The only essential field tools required are:

- A 1: 50 000 topographic map covering the catchment area.
- A compass.

A Hand held GPS (global positioning satellite) is also useful

Assessments are based on information collected partly from interviewing people resident in the catchment, and observations made while walking a number of randomly chosen transects across the catchment. The direction and siting of transects are selected after careful study of a 1:50 000 topographic map. They may follow footpaths and tracks where they cross the catchment (running down from the upper slopes down to the watercourses and up the other side). Where there are no suitable footpaths, transects are walked following a bearing. It is particularly important to walk along random sections of the main watercourses to examine the condition of the riverbanks and riverbeds.

Ideally a local informant who knows the location and direction of the footpaths will accompany the person(s) making the assessment. They can be important sources of information on the past land use and land conditions within the catchment.

It is possible to assess catchment characteristics at a rate of about 1.5 km^2/hour, and to finish a typical small dam catchment within a day's work. If there are roads and motorable tracks across the catchment, these may be used to speed up the work rate, but to avoid missing key characteristics some transects must be walked. Assessments are best carried out at the end of the main dry season, when the vegetation cover is at its lowest, with the soils bare or almost bare. It is under these conditions that soils are most prone to erosion during intensive storms in the early part of the rainy season, storms that usually generate a large part of the sediment runoff from catchments.

Catchment Characterisation form

Catchment name:
Date:
Observers:

Factor	Extreme		High		Normal		Low	
Soil Type & Drainage	No effective soil cover; either rock or thin shallow soils	40	Poorly drained compacted soils; much ponding on soil surface after heavy rains	30	Moderately well drained medium-textured soils; some ponding on soil surface after heavy rain	20	Well drained coarse-textured soils; little ponding on soil surface after heavy rain	10
Vegetation Condition over Whole Catchment	Little effective plant cover, ground bare or very sparse cover over 80% of catchment	40	Fair cover: >50% of catchment is cultivated with annual crops;	15	Good cover: 20-50% of catchment is cultivated with annual crops;	10	Excellent cover: <20% of catchment is cultivated with annual crops;	5
			<30% of catchment is under good grass cover or protected forest cover	15	30-60% of catchment is under good grassland or protected forest cover	10	>60% of catchment is under well-maintained grassland and/or protected forest cover	5
Signs of Active Soil Erosion	Many actively eroding gullies draining directly into watercourses; active undercutting of riverbanks along main watercourses	40	Some actively eroding gullies draining directly into watercourses; moderate undercutting of riverbanks along main watercourses	20	Few actively eroding gullies (dongas) draining directly watercourses; little undercutting of riverbanks along main watercourses	10	No actively eroding gullies draining directly into watercourses; no undercutting of riverbanks along main watercourses	5