DESIGN MANUAL

VOLUME 2

GUIDELINES FOR WADI DIVERSION AND PROTECTION WORKS

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The European Union's Food Security Programme for Yemen
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VOLUME 2: GUIDELINES FOR WADI DIVERSION & PROTECTION WORKS

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1. Wadi Hydrology

1.1 Flood flows and durations

Spates are relatively short duration floods which are caused by heavy range on the catchment and are characterised by a very rapid increase in the flow, a short peak and then a less rapid decline. The total duration of a spate can range between a few hours and a few days. Sometimes a spate can have more than one peak if there is rainfall over more than one sub-catchment and sometimes there can be spates on successive days. A "good" spate, for irrigation, is one where the peak is not very big and the duration (and therefore total volume of water) is several days.

Figure 0-1 : Example spate hydrographs
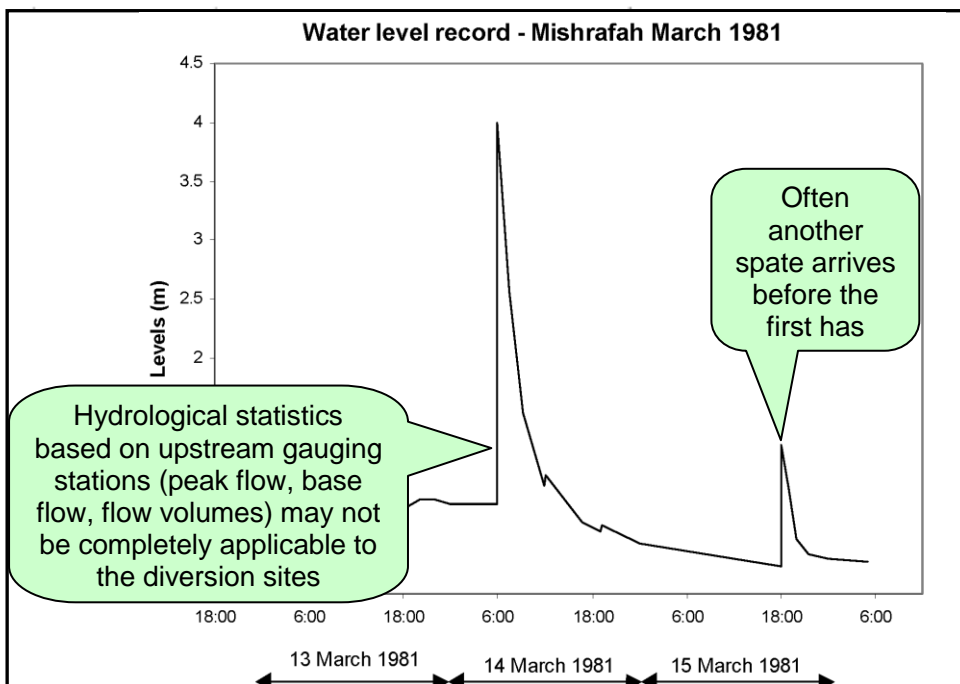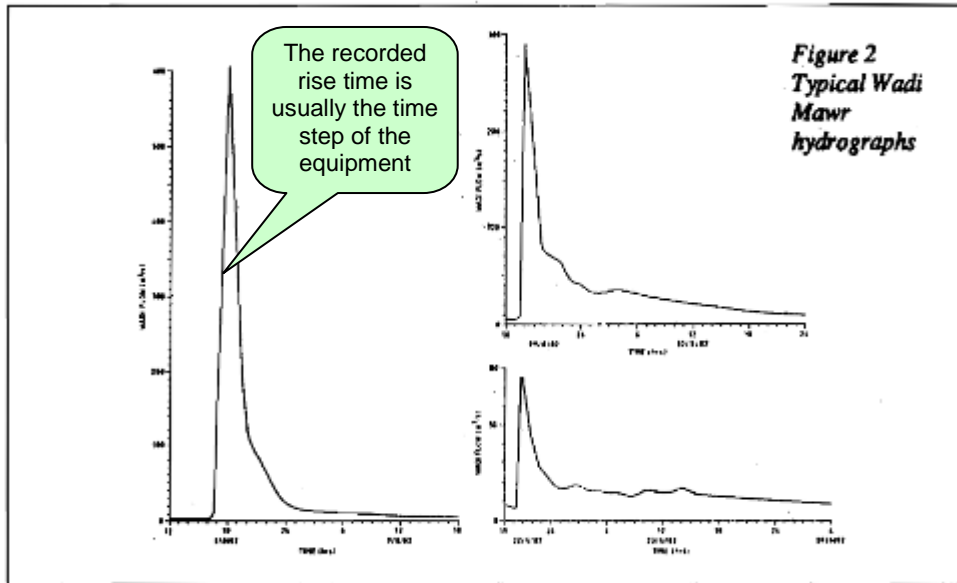
The peak flow of the spates can vary considerably. Figure 0-2 shows a classification of the spates for Wadi Zabid based on the peak flow. The higher peaks, which may cause damage, may be less than one per year on average.

Figure 0-2 : Wadi Zabid - distribution of spate peak flows
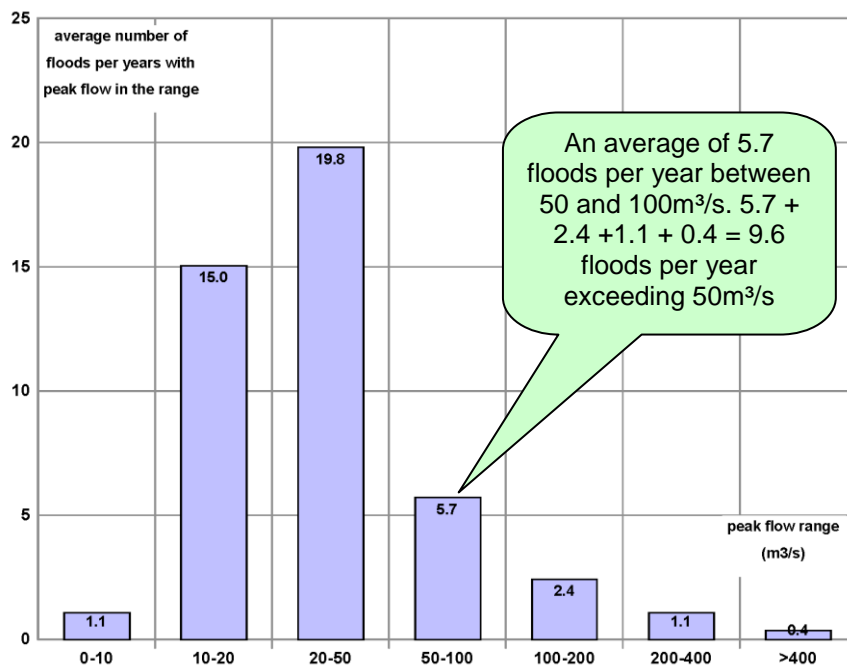
The number of spates per year can vary widely between the wadis and years. Wadi Zabid, for example, received less than 20 spates in 1991 and more than 80 in 1993 as shown on Figure 0-3.

Figure 0-3 : Wadi Zabid - annual number of spates and total flow volume
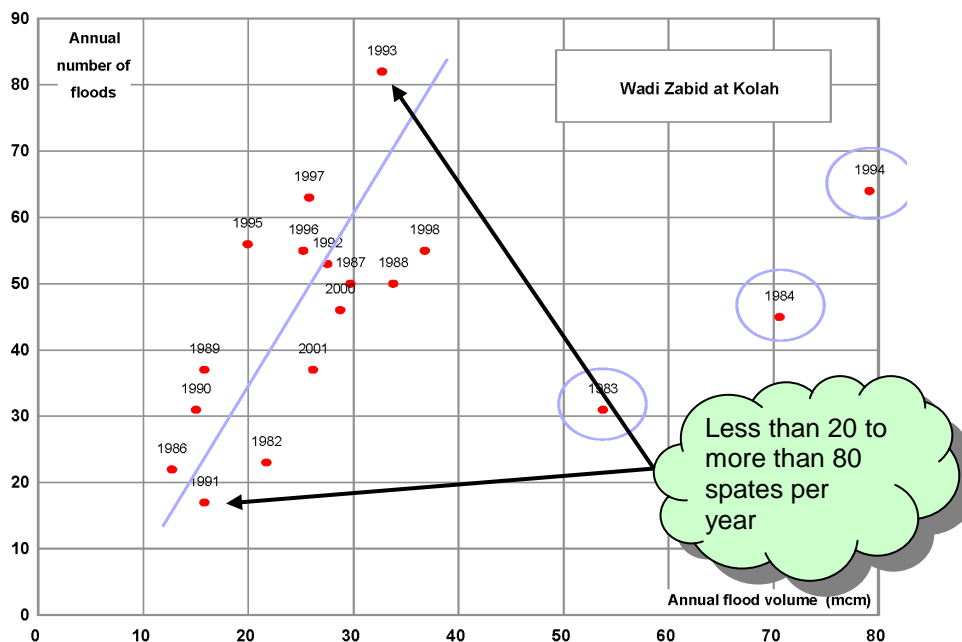
Design of the intake capacity of the modernised spate irrigation schemes took into account the proportion of the flow volume which would exceed the intake capacity. An example is shown on Figure 0-4. This shows that 80% of the flood volume is at flows less than 100m³/s and 90% of volume is below 200m³/s. However, the farmers want to use the remaining water which exceeds the intake capacity.

Figure 0-4 : Wadi Zabid - proportion of flood volume below limiting discharge
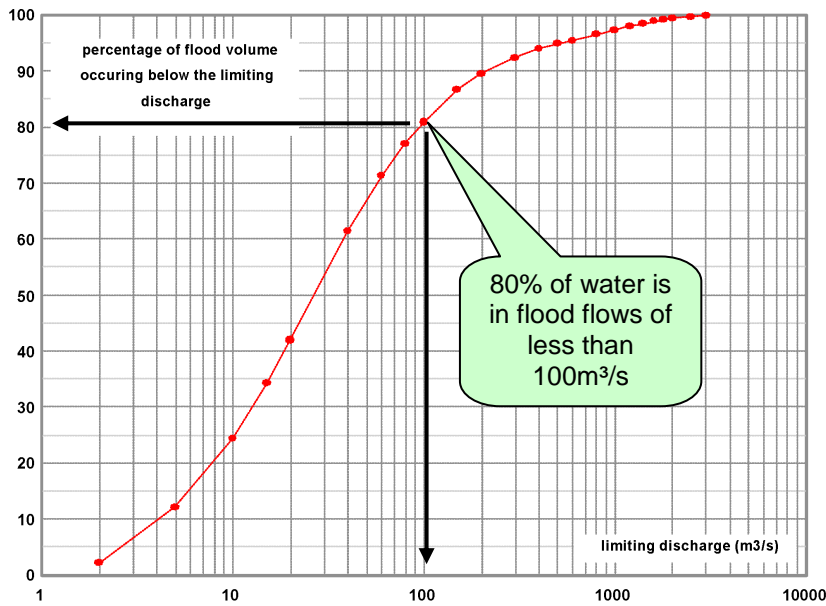
The wadis may also include a component of base flow. Use of this was also considered in the design of the modernised spate schemes. However, the flow gauging stations are often located some distance upstream of the diversion structures and part or all of the base flow may percolate into the wadi bed before it reaches the intake. In addition, increased ground and surface water usage in the catchment may have reduced the base flow.

Figure 0-5 : Wai Zabid - annual base flow and flood flow volumes
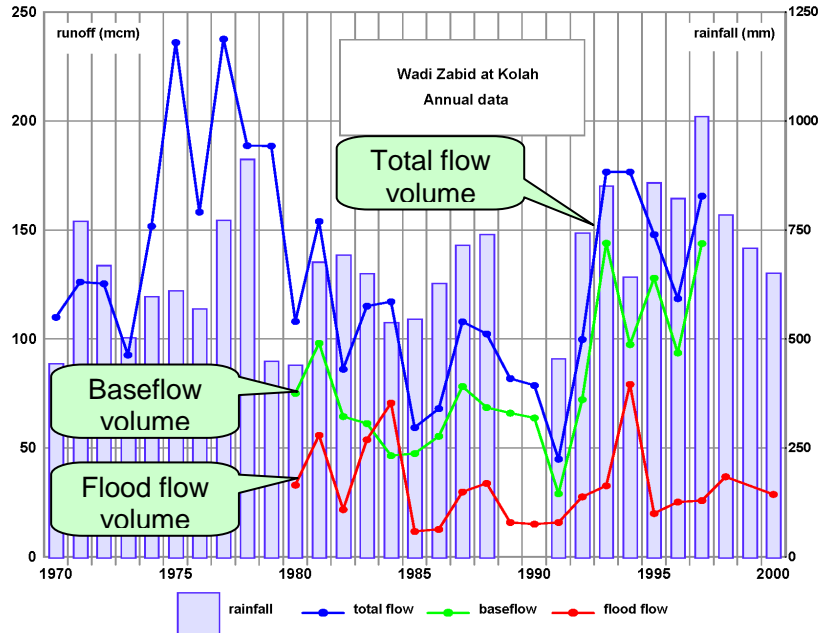
1.2 Sediment

High sediment loads is another key feature of the wadi systems. During peak flood flows the sediment load, excluding the washload (the fine material which will only settle very slowly) may exceed 10% by weight, as shown on Figure 0-6. The measurements will also not include the coarse bed material which migrates along the bottom of the wadi during the more severe floods but cannot be measured using sediment sampling equipment.

Figure 0-6 : Wadi Zabid - sediment load (from Report OD 73)

Fig 9   Wadi Zabid - sediment concentration as a function of discharge, HR data (washload excluded)
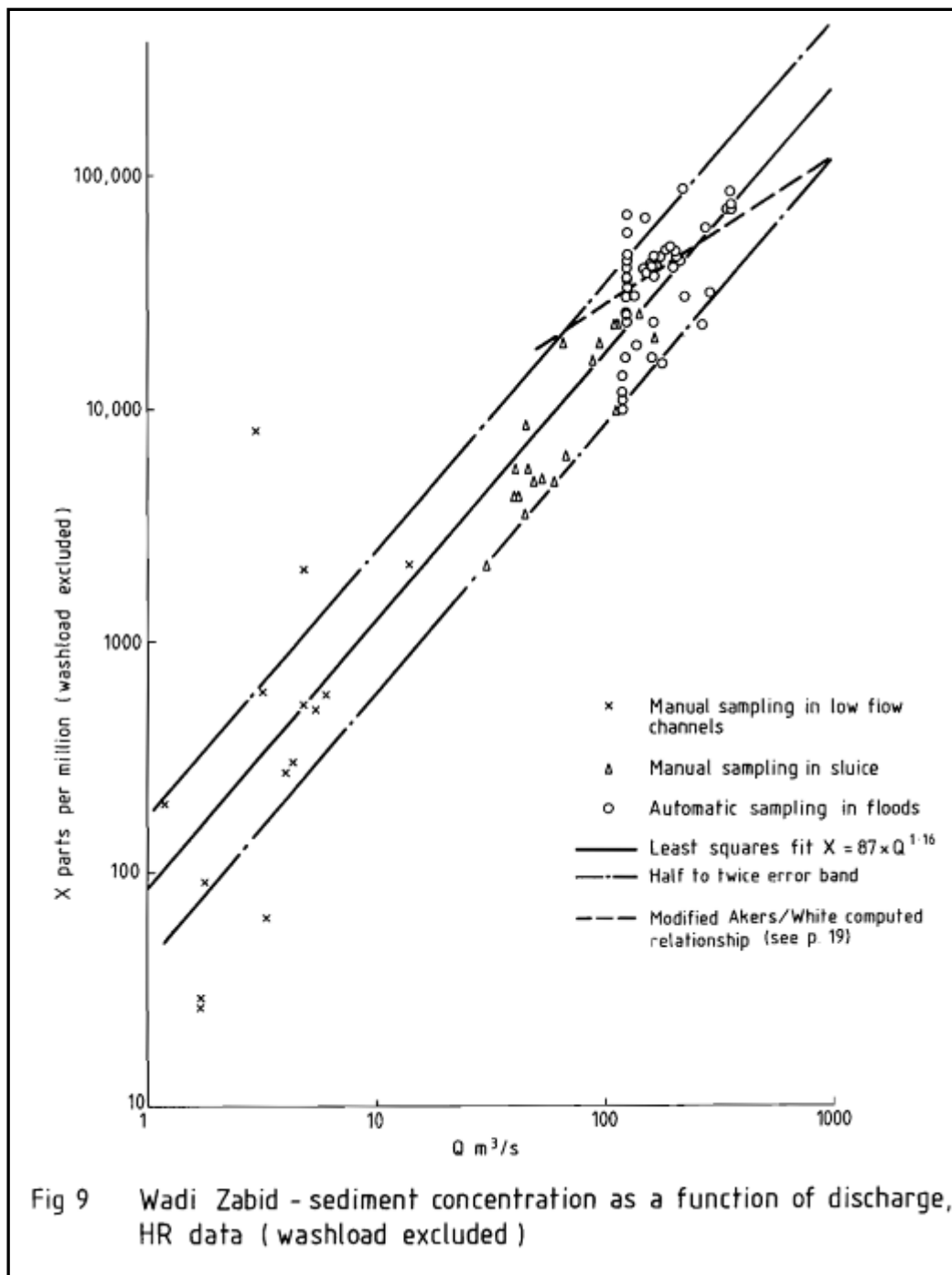
The sediment load, particularly during the larger floods, can cause major operation and maintenance problems for spate irrigation systems.

1.3   Wadi Morphology

1.3.1   Degradation, aggradation and regime conditions

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

Interventions within a river can disturb the morphological balance. **Error! Reference source not found.** shows the progressive effect on channel morphology of constructing a weir. The weir will prevent the free migration of sediment downstream until the bed profile upstream of the structure has adjusted.

The extraction of sand and gravel from wadi beds will also disturb the overall morphological balance: Sediment being carried into the area where extraction has occurred will get deposited and the flow will tend to erode the bed further downstream in order to regain the stable sediment load.

Figure 0-7 : Effect of weir construction on channel morphology
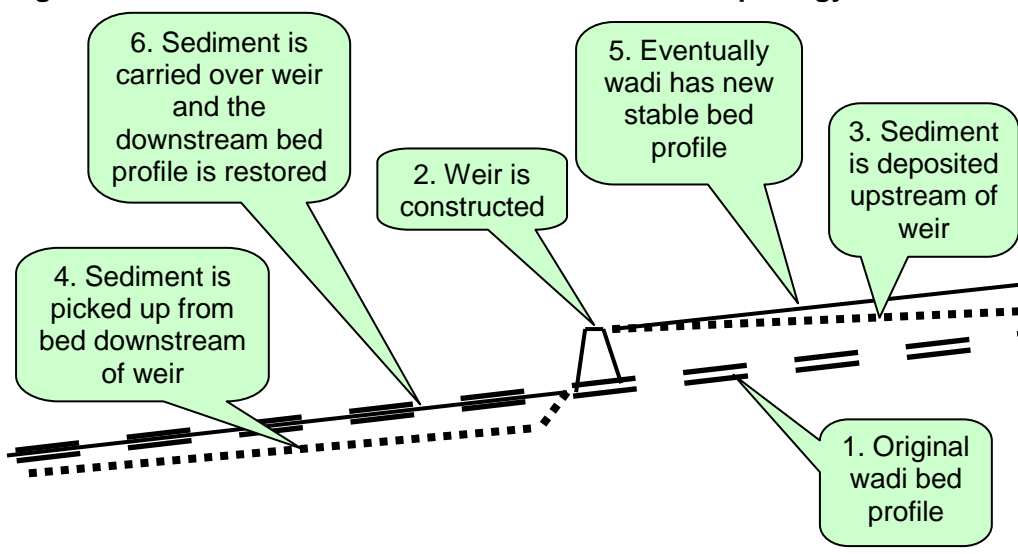
1.3.2   Channel migration

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 0-8. 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 which will deflect the flow path. 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
- bank erodibility
- bed material grading and strata.

Figure 0-8 : Typical meandering channel
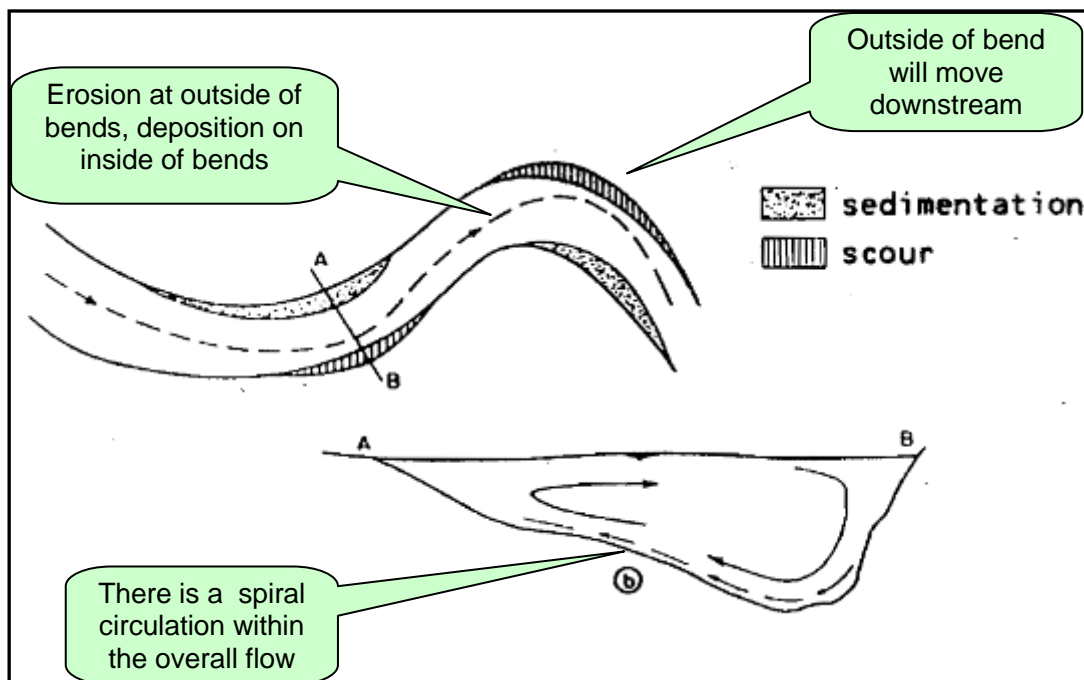
1.4 Scour

1.4.1 Local scour at structures

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 0-9) 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.

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 0-9 : Local scour at structure
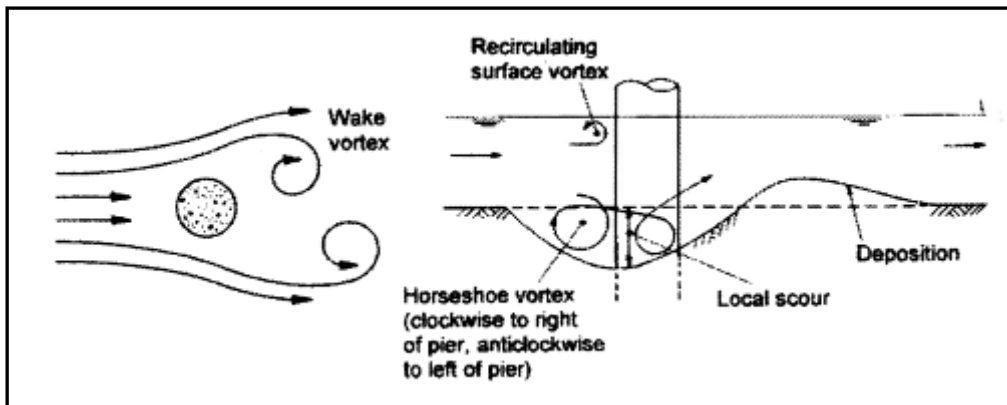
1.4.2 Transient scour

Observations of scour holes made after flood events can be misleading. Maximum scour depths are likely to occur near the peak of a flood, and during the recession stages the holes may be partially refilled. This phenomenon, which has been observed from the strata exposed in trial pits in scour holes, is principally a feature of scour in live-bed conditions, where bed material is transported from upstream. However, it can also occur over time following clear-water scour, due to the deposition of finer material or slumping of the scour slopes.

1.4.3 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.

1.4.4 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 0-1 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 0-1 : 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 well known, the Lacey silt factor (f) may be calculated from the formula:

$$f = 1.76 \, \theta\overline{D_{50}}$$

Where:

D50 = the sieve size through which 50% of the material passes by weight [mm]. Alternatively, the silt factor is given in **Table 0-2** below for various materials.

Table 0-2 : 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

2　Diversion Structures

2.1　General considerations

Traditional diversion and water distribution structures often seem crude at first sight, but they enable water to be diverted from uncontrolled ephemeral rivers using only local materials and indigenous skills. When multiple traditional intakes are used along a wadi, a relatively high water diversion efficiency can be achieved overall. The principal disadvantage of traditional diversion methods is the excessive inputs of labour needed to rebuild the intakes and the other water control structures that are frequently, sometimes by design, damaged or scoured out by flood flows. Relatively sophisticated (when compared with traditional structures) and costly diversion structures, linked to new canal systems, have been introduced in several countries to modernise and improve the performance of existing traditional systems. These well-intentioned engineering interventions were designed to reduce or eliminate the need for the frequent reconstruction of intakes and in some cases to increase the volumes of water that could be diverted. However, these have often failed to produce the benefits that were expected.

The disappointing performance of many of these systems has been variously attributed to:

- Failures to achieve an expected increase in irrigated area due to over-optimistic assumptions about water resource availability and the water diversion efficiencies that can be achieved with rapidly varying spate flows and manually operated control gates.[1]
- An increased inequity of water distribution, resulting from the construction of permanent diversion structures at the head of spate systems, which gave the upstream farmers control over a large proportion of the available flows, to the detriment of downstream irrigators.
- Serious command problems due to high rates of sediment deposition on the fields and canals.
- Unrealistic assumptions concerning levels of operation and particularly the maintenance, mostly canal de-silting, required to keep conventionally designed irrigation networks running in spate systems.

This is due in part to a development approach that focussed on relatively conventional engineering solutions. Experience has shown that if they are to be successful interventions in spate schemes need to be based on a sound understanding of the social and institutional strengths and the simple technologies that are used, that have enabled many traditional systems to function successfully over centuries. To be successful the minimum requirements for engineering inventions designed to improve farmer managed spate systems are that they should:

- Be easy for farmers to operate and not require the large inputs of labour or other resources to maintain.
- Prevent large and uncontrolled flood flows from damaging canals and field systems.
- Distribute water in line with accepted rules and rights, while providing flexibility to accommodate future changes in water distribution and cropping. They should also ensure an appropriate balance between the needs of different water uses and users, (agriculture, drinking water, downstream users etc).

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[1] 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 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.

- Continue to function with high rates of sedimentation on the fields and canal beds.
- Cope with frequent and sometimes large changes to the levels and alignments of unstable river channels resulting from large floods.

Improvements requested by farmers are usually aimed at reducing their maintenance burden, through the provision of more robust and more permanent diversion and water control structures. The economic benefits of improvements may be modest and only low cost improvements, that almost inevitably have a short life and significant ongoing maintenance requirements, are viable. This is not always a disadvantage in spate irrigation, as these characteristics provide the flexibility to adjust to a rapidly changing physical environment. Where improvements can be justified from a wider range of benefits, such as poverty alleviation, improved resilience to cope with droughts, or enhanced re-charge of shallow groundwater aquifers, it may be possible to justify an increased level of expenditure.

Improvements to a traditional intake may therefore include several components:

- Works to direct the wadi flow towards the intake
- Works to ensure sufficient water level at the canal intake
- Works to control the flow entering the canal
- Works to flush excess sediment away from the canal intake
- A spillway to reject excess flows that enter the canal

Different parts of the work may be designed to different standards. Some may be damaged or destroyed by major flood events while other parts are designed to survive.

2.2 Hydraulic design

2.2.1 Design Parameters

The Hydraulic design of improvements to traditional intakes needs to take into account a number of parameters including:

- Wadi flow and water levels over a range of flows
- Canal flow
- Canal command level
- Stability of wadi and low flow channel
- Sediment load in wadi
- Rate of land rise due to sediment deposition
- Design life of structure

The general arrangement of a typical improved canal intake is shown below :

Figure 2-1 : Typical Layour of Improved Canal Intake

Optional bed bar to stabilise wadi bed level

Wadi Flow

Protected embankment

Side spillway

Optional sluiceway

Gravel embankment to divert flow towards intake

Canal entrance

Typically 10% to 20% of total width, depending of flow proportions

Protected nose for embankment

Orifice to restrict maximum flow

Canal
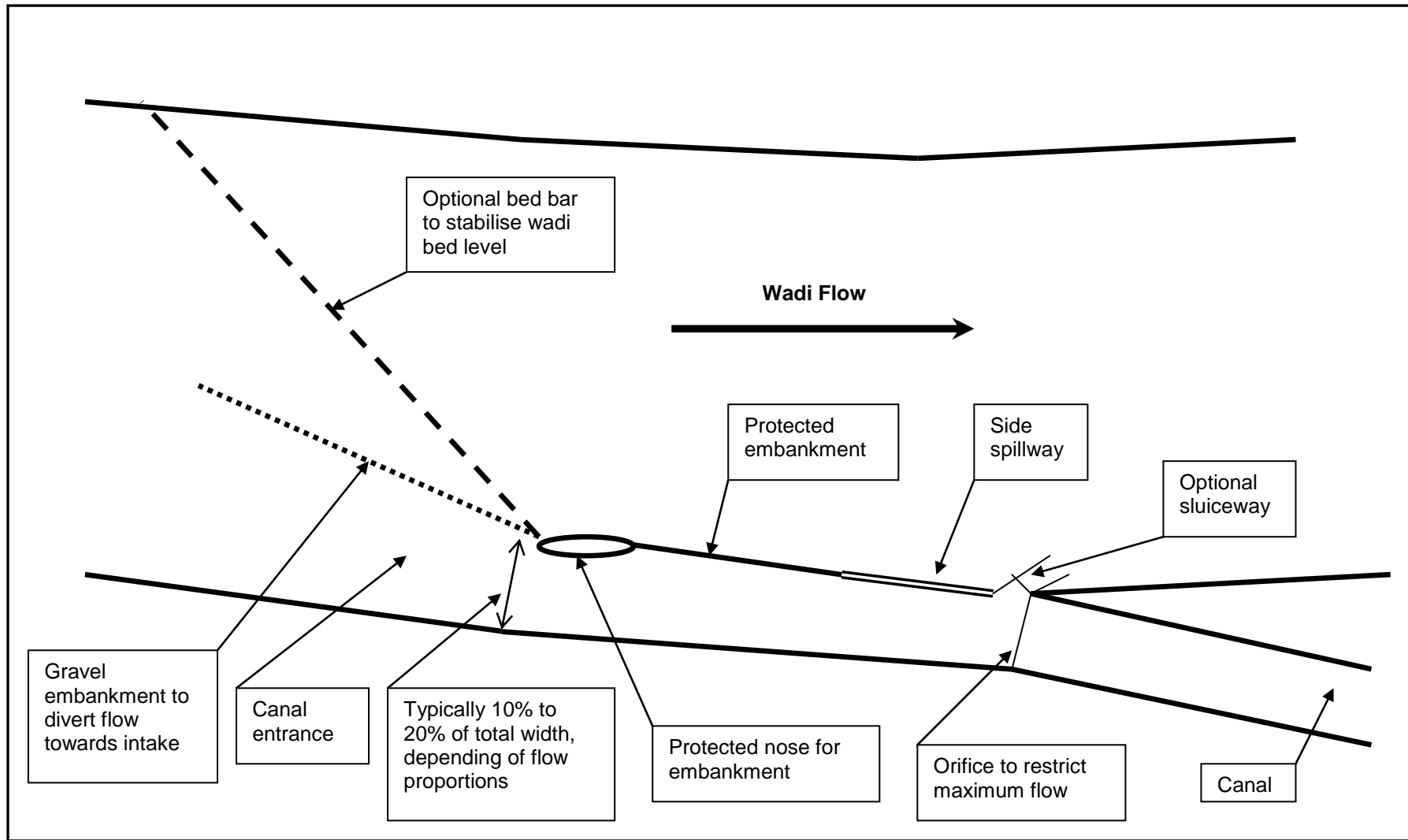
2.2.2   Wadi flood flows and water levels

Wadi flows and water levels need to be determined for a range of conditions. Available hydrological analyses should be consulted. However, the floods determined by existing hydrological analysis may be at a different location on the wadi, usually upstream of the irrigated area, and the peak flows will reduce as water spreads out or is diverted.

Therefore, if either flood estimates are either not available or are some distance from the location being studied, it will be necessary to prepare new flood estimates. One way to derive estimated flood peak values is to use the slope-area method. This method is detailed in books on hydrology but, in summary, uses the flow cross section area and slope as input to Manning's equation. It is necessary to have one or more maximum flood level marks with some indication of the frequency of the flood. If the year is known, then it may be possible to correlate with other flood data for the wadi to get the probability.

It will be necessary to relate the peak floods with the different return periods. In the absence of other information it is suggested that the following growth factors can be used.

Figure 2-2 : Indicative Flood Peak Growth Factors

Return period (years)	Growth factor
mean	1
5	1.4
10	2.1
20	3.1
50	4.8
100	6.7
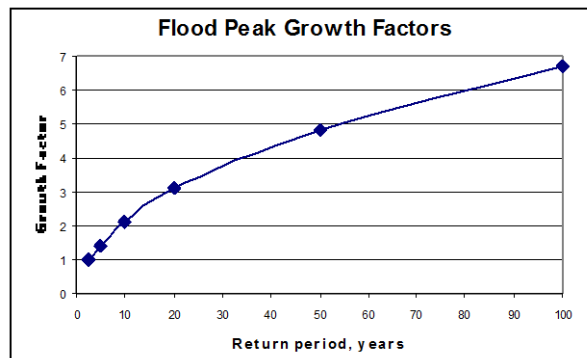
It is also necessary to prepare a rating curve (graph of water level and flow) for the wadi at the intake location so that the wadi flood levels for a range of flood flows can be estimated.

2.2.3   Canal design flow and water level

The design flow for the canal can be determined using the formula:

$$Q_i = \frac{2.77\ A_i\ W}{\varepsilon\ t}$$

where:

Q_i	is the theoretical discharge (m^3/s)
A_i	is the irrigable area (ha)
W	is the depth of application (assume 0.60m)
ε	is the application efficiency (assume 40%)
t	is the time of application (hrs)

The time of application is the time period for which the flood flows in the wadi will exceed the canal capacity during the irrigation season. The time period will probably become shorter the further an intake is down a wadi.

Determine the depth of flow in the canal for the maximum design discharge using Manning's equation:

$$Q = (1/n) \, AR^{2/3} \, s^{1/2}$$

where:
Q   is the max. design discharge (m^3/s)
n   is the rugosity (assume n = 0.03)
A   is the cross-sectional area of flow
R   is the hydraulic radius (A/P)
P   is the wetted perimeter
s   is the canal slope

The canal cross-section parameters (bed width and side slope) is estimated from the field visit, or from survey data if available.

Checks should be made to confirm that the canal water level will be sufficient to irrigate the highest fields (which are usually at the start of the canal). Otherwise, water may be ponded up to irrigate this land and the potential flow in the canal will be reduced.

2.2.4 Orifice at canal intake

The improvements to the diversion increase the risk of excess flow being diverted into the canal during floods. It is therefore recommended that an orifice is provided at the canal intake to reduce the maximum flow which may enter the canal under high flood conditions. The water level for normal flow in the canal during small floods should be just below the top of the orifice to minimise the head loss.

Determine the size of the orifice and the head difference across the offtake structure for the design discharge Q applying:

$$Q = C_d \, C_v \, bw \, [2g \, (y_1 - y) \,]^{1/2}$$

where:
Q   is the design discharge (m^3/s)
C_d is the coefficient of discharge is as given in Table 1
C_v is the coefficient of approach velocity taken as 1.00
b   is the width of the opening
w   is the height of the opening
y_1 is the upstream depth
y   is the vena contracta determined from y = δw
δ   is the contraction coefficient taken from Table 1

Table 2-1 : Orifice Coefficients

y_1 /w	Δ	C_d
1.6	0.642	0.599
1.8	0.634	0.597
2.0	0.630	0.596
2.2	0.628	0.596
2.4	0.626	0.596
2.8	0.625	0.598
3.0	0.625	0.599
3.5	0.625	0.602
4.0	0.624	0.604
4.5	0.624	0.605
5.0	0.624	0.607

Under flood conditions the upstream water level will rise and the orifice will be drowned. The discharge through the orifice can be estimated using:

$$Q = C_d\, C_v\, bw\, [2g\, (y_1 - y_2)\,]^{1/2}$$

where:
Q   is the design discharge (m^3/s)
C_d is the coefficient of discharge taken as 0.60
C_v is the coefficient of approach velocity taken as 1.00
b   is the width of the opening
w   is the height of the opening
y_1 is the upstream depth
y_2 is the downstream depth

Small orifices are more vulnerable to blockage by trash. Therefore, multiple orifices should only be used if a single orifice is more than about 5m wide unless gated orifices are used, in which case the maximum width may be 3m to 4m. For guidance, the approximate discharges for typical orifice openings are given in Table 2.

Table 2-2 : Typical Orifice Sizes

Orifice size width x height (m)	Approx. Discharge (m^3/s)
1.5 x 1.5	7.5
2 x 1.5	10
2.5 x 1.5	12.5
3 x 1.5	15
4 x 1.5	20

Gates may be required to close canal intakes if water rights require that the intake has to be closed at certain times. However, the alternative to gates is a gravel embankment immediately upstream of the orifice.

2.2.5   Side spillweir

Determine the height and length of the side spillweir upstream of the orifice structure. The spill weir should start to reject flow as soon as the water depth upstream of the orifice reaches the depth at the design discharge y1.

The equation for the flow over the side spillweir is taken as:

$$Q_w = K\, C\, B\, h^{3/2}$$

where:
Q_w is the discharge over the side spillweir
K   is the coefficient for oblique flow over the weir
C   is taken as 1.7 for a broad crested weir
B   is the crest length of the spillweir
h   is the design head over the crest where $h = (y_o - y_1)$
y_0 is the upstream head at the head of the diversion embankment

The crest level b is determined from level d + y_1 where d is the wadi bed level at the entrance to the canal.

To determine the depth y_0 Manning's equation can be applied to the wadi flow taking the full width of the wadi W, similar to step 2). This depth is calculated for the design flood in the wadi. For a flood less than this, a backwater curve would occur. The distribution of flow between the wadi channel and that passing into the canal entrance and over the spillweir for different return periods is assessed by trial and improvement, but in general with the offtake closed the flow distribution is

about 5% over the spillweir and 95% in the wadi. This ratio changes if the offtake gates are opened or if a sluice is provided as shown in Figure 1.

The modular ratio at the spillweir needs to be checked such that:

$$(h_d / h) < 0.7$$

where h_d is the downstream head over the weir.

If $(h_d / h) > 0.7$ then the coefficient of discharge C would need to be reduced, however it is expected that this situation should not occur.

If the flow into the canal under flood conditions is likely to endanger the canal integrity then a further spillway should be provided in the canal headreach to reject excess water back to the wadi. A gabion mattress protected embankment may be suffcient.

2.2.6   Embankment nose

Determine the height of the embankment nose. This should be set above the maximum anticipated design flood level. A freeboard of 1m above the depth y0 is suggested, (a = c + y0 + 1). The expected height of the embankment is between 3 to 4 m above wadi bed level. A check to see that level a is close to the wadi bank top level at this point needs to be made.

Determine the length of the embankment nose. This is done by calculating the maximum scour depth likely to occur at the nose as follows:

The scour depth is calculated using Lacey's formula:

$D_s = XR - Y$
where
D_s = the scour depth below bed level in m
X = the severity factor depends on the type of reach (see Table 2-3)
Y = the depth of flow in the channel in m
$R = 1.35 (q^2 /f)^{1/3}$
q = the unit discharge m^3/s/m
f = Lacey's silt factor (see Table **0-2**), where $f = 1.76 d_{50}^{1/2}$

For designing the extents of the scour protection works launching aprons are used as follows:

The minimum length of the launching aprons are 2Ds at the head of the nose, but this can be reduced to 1.7Ds if the scour depth is excessive, where 1.7Ds is equivalent to an angle of repose of 30°.
The launching aprons should be buried below wadi bed level by at least 1m. This is to prevent them from being damaged by bulldozer activities.

Table 2-3 : Severity factors

Location	Severity factor X
Upstream of structure	1.5
Downstream of structure	2.0
Nose of spur	2.25
Transition from nose to straight	1.5
Straight reach of guide bank	1.25

The width of the entrance to the canal intake should not be greater than 1/5th of the width of the wadi. (see Figure 2-1).

2.2.7   Diversion of flow to intake

Work may be required to ensure that the smaller floods in the wadi flow towards the intake. One technique is to ensure that the intake is located on the outside of a bend. However, this also increases the risk of erosion damage and may require bank protection upstream and downstream of the intake itself.

Construction of a weir across the wadi is a major investment and may also cause water rights disputes. This option is therefore not considered further. An alternative may be a bed bar with its top at the average wadi bed level. This will stop any channel within the wadi from cutting below the intake level.

The final option is to construct a gravel embankment across part, or all, of the wadi to deflect the flows to the intake. This may appear to be no significant improvement on the current practice. However, if the intake structure includes a spillway, medium floods in excess of canal capacity will pass over the spillway and not breach the gravel embankment.

In addition, if a sluiceway is provided immediately upstream of the orifice on canal intake, this can be used to (i) increase the capacity to pass flood flows and (ii) help to maintain a flow channel towards the canal entrance. The sluiceway should be 4m to 5m wide so that it can be closed by a gravel embankment pushed up by a bulldozer. The top of the embankment should be about 60cm above the spillway crest.

2.3   Construction materials

The selection of construction materials should take account of (i) wadi conditions and (ii) available materials.

Sites lower down the wadi systems may have locally available sand and gravel and therefore mass concrete may be more attractive than gabions, for which the stones would need to be transported some distance. However, sites where gabion stone is available may have aggressive wadi conditions where concrete or masonry structures would be more durable.

Categories of exposure conditions are set out in Table 2-4 and options for construction materials for high exposure conditions are given in Table 2-5.

Table 2-4 : Exposure categories

Exposure Category	Description
High	• Location of high flow velocities in excess of 3.5m/s (11.5ft/s) with continuous high concentrations of sand / fine gravel causing <u>severe abrasive damage</u> • Locations of high flow velocities in excess of 3.5m/s (11.5ft/s), or areas of high turbulence carrying large cobbles / large boulders causing <u>severe impact damage</u> • Potential for severe uncontrolled surcharges including high discharges and high flow velocities
Medium	• Location of high flow velocities in excess of 3.5m/s (11.5ft/s) with intermittent high concentrations of sand / fine gravel causing <u>significant abrasive damage</u> • Locations of high turbulence occasionally carrying large cobbles / large to medium boulders causing <u>some impact damage</u> • Potential for uncontrolled surcharges inducing high discharges and high flow velocities
Low	• Location of medium flow velocities of 1.5-3.5m/s (5-11.5ft/s) carrying predominantly wash-load material and some coarse material with <u>limited potential for abrasive damage</u> • Locations normally free from transported material larger than sand size.

Table 2-5 : Construction materials options for high exposure conditions

Type of Construction	Ashlar faced Mass Concrete	Massive Rockfill	Concrete wearing course on Mass Concrete	Composite concrete and gabion (protected)	Unprotected Gabion
Serviceability	Good 30 yrs +	Good 30 yrs +	Fair: requires replacement of wearing course	Very poor, 5 years	Unacceptable, less than 5 years
Failure Mode	Extensive failure unlikely, localised surface damage	Extensive failure unlikely, localised surface damage	Extensive failure unlikely, surface damage	Extensive failure likely	Extensive failure very likely
Annual Repair Requirement	3% Wearing Course	2% wearing course	5% Wearing Course	20 - 25% of gabion surface	Not classified
Conclusion	Suitable – preferred	Suitable[1] - alternative	Less suitable	Unsuitable	Totally unsuitable

Notes

1   Subject to availability and price. The required rock size is likely to be about 1 m^3 (2.5 tonnes), and would require a special quarrying operation.

2.4   Sediment Management

Wadis carry considerable sediment load, particularly during high floods as is illustrated by **Error! Reference source not found.** which shows the results of field measurements for Wadi Zabid. The sediment load comprises a range of sizes from cobbles and gravel which move along the bed during the larger floods to silts and clays in suspension. When the flow velocity is reduced ten the sediment is deposited.

The farmers appreciate the fine sediment (called the "washload") which is normally carried through to the fields because it is usually fertile. However, this sediment progressively raised the land being irrigated as shown in Table 2-6. Design of the canal command levels needs to take account of the likely rise in field levels during the design life of the proposed improvements to the canal intake.

Table 2-6 : Typical rates of sediment deposition

Scheme	Annual rise rate, mm/year
Wadi Laba Eritrea (Measured 1998/99)	Upstream fields 8–32
	Middle fields 6–18
	Downstream fields 5–9
Wadi Laba Eritrea (Long term estimate)	30
Eastern Sudan	139
Baluchistan mountain systems	> 50
Wadi Zabid	Upstream fields 20–50

Coarse sediment may be deposited in the canal and have to be removed from time to time. The entry of coarse sediment into the canal can be reduced by providing a sluiceway at the intake which is used during the larger floods to eject sediment from the intake back into the wadi.

2.5   Trash Management

Large floods can carry considerable amounts of trash which can quickly block canal intakes. Intakes sited on the outside of bends (the best location to capture the flow) are particularly vulnerable because trash moves with the surface flow to the outside of a bend.

Figure 2-3: Barquqa weir trash screen at intake

Trash screens are usually provided but may make the risk of blockage worse because they will tend to catch smaller trash which would otherwise pass through the intake and into the canal. Trash already captured will effectively reduce the opening size and stop smaller trash from passing through. This can be seen on Figure 2-4.

The solution shown in Figure 2-3, which is to provide a very large screen upstream of the intake, is likely to not incur sufficient blockage to stop flows, but it is also expensive. The inclined angle of the screen also means that some trash is likely to be deflected and washed downstream past the intake.

Figure 2-4 : Intake partly blocked by trash

Access to areas where trash may collect should be included in the design and where trash is a regular problem then it may be appropriate to provide trash clearance equipment, such as a winch with a hook.

2.6 Gates

Gates provide an additional construction cost and form an operation and maintenance burden. The general recommendation is to not provide gates at the intakes unless there is a compelling reason for their installation. They often form a source of conflict between water users. 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 not be smaller than the minimum recommended sizes for orifices; (ii) gates should be designed to allow operation in a short period of time; and (iii) hydraulic design and energy dissipation should consider the possibility of a high upstream water level and a part open gate.

3 Channel Stabilisation Works

3.1 Channel properties

A general introduction to wadi morphology was given in Section 1.3 and will not be repeated here. 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.

In order to assess the potential effects of scour the velocity and depth of flow for a particular discharge needs to be established. This can be done by a number of equations but the simplest is the Manning's equation:

$$Q = (1/n) \, A \, R^{2/3} \, s^{1/2}$$

where:
Q	is the discharge (m^3/s)
n	is the roughness of the channel bed
A	is the cross-sectional area of flow
R	is the hydraulic radius A/P
P	is the wetted perimeter
s	is the slope of the energy line, which approximates to the slope of the channel bed

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.

3.2 Types of Works

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.

3.2.1 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 flx 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).

3.2.2   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.

3.3   Selection of Works

3.3.1   Spurs or groynes

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.

Figure 3-1 : Collapsed nose of spur
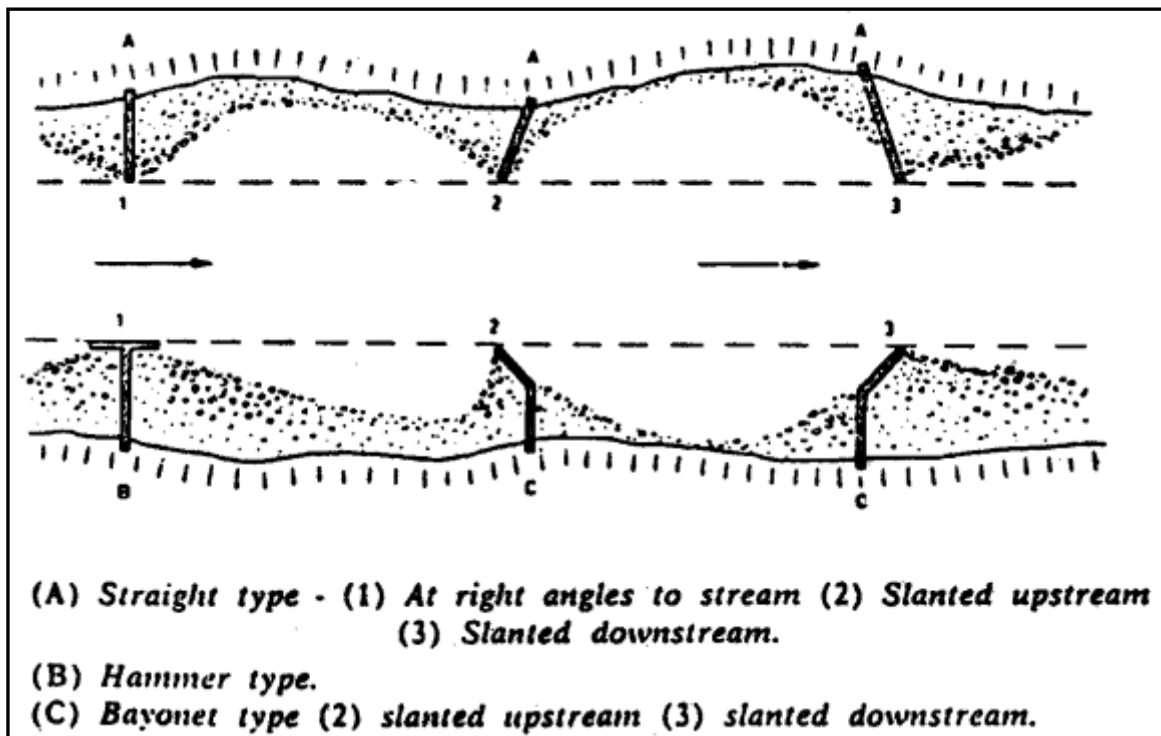

The requirements of a spur are:

- Optimum alignment and angle consistent with the objective.
- Availability of a high river bank to anchor (or tie) the spur back, by extending it into the bank a sufficient distance to avoid it being outflanked.
- Sufficient freeboard provision (in case of non-submerged spurs).
- Adequate protection to nose/head against anticipated scour.
- Shank protection with stone pitching and stone apron for the length which is vulnerable to flow attack.

Depending upon the purpose, spurs can be used singly or in series. Spurs may be aligned either perpendicular to the bank line or at an angle pointing upstream or downstream. They can also be used in combination with other training measures. Their use in series is introduced if the river reach to be protected is long, or if a single spur is not efficient/strong enough to deflect the current and also not quite effective for sediment deposition upstream and downstream of itself. The structure located the farthest upstream in a series of spurs is much more susceptible to flow attack both on the riverward and landward ends. Thus it should be given special treatment to ensure its structural stability.

The position, length and shape of spurs depend on site conditions, and requires significant judgement on behalf of the designer. No single type of spur is suitable for all locations.

A spur angled upstream repels the river flow away from it and is called a repelling spur. These are preferred where major channel changes are required. A spur originally angled upstream may eventually end up nearly perpendicular to the streamlines after development of upstream side silt pocket and scour hole at the head. Repelling spurs need a strong head to resist the direct attack of swirling current. A silt pocket is formed on the upstream side of the spur, but only when the spurs are sufficiently long. Repelling spurs are usually constructed in a group to throw the current away from the bank. Single spurs are neither strong enough to deflect the current nor as effective in causing silt deposition upstream and downstream.

Figure 3-2 : Examples of spur types

(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.*

A spur angled downstream attracts the river flow towards it and is called an attracting spur. The angle of deflection downstream ranges between 30 to 60 degrees. The attracting spur bears the full fury of the frontal attack of the river on its upstream face, where it has to be armoured adequately. Heavy protection is not necessary on the downstream slope. It merges into the general stream alignment more easily. The scour hole develops off the riverward end of the structure.

When the upstream angled spur is of short length and changes only the direction of flow without repelling it, it is called a deflecting spur. It gives local protection only.

The angle which the spur makes with the current may affect the results. A spur built normal to the stream usually is the shortest possible and thus most economic. An upstream angle is better to protect the riverward end of the spur against scour. A downstream angle might be better for protecting a concave bank, especially if spacing and the lengths of the spurs are such to provide a continuous protection by deflecting the main currents away from the entire length of bank.

Figure 3-3: Generous scour protection at head of spur

Stepped end reduces impact on high floods but still protects the bank

Spur keyed into bank to avoid outflanking behind spur

Apron to protect against scour

3.3.2 Spacing of Multiple Spurs

The spacing between spurs depends on the length of the spur from the bank, 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.

3.3.3 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 river width.

3.3.4 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 3-4: Long section through spur
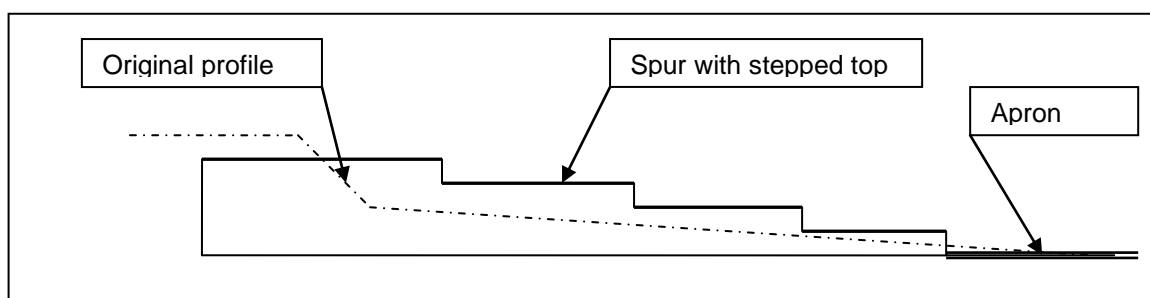
3.3.5   Revetment

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. Revetment should be used where there could be unwanted side effects if the flow pattern is disturbed. For example, 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 3-5. 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.

**Table 3-1 : 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

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 revetted 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 3-5 : Scour protection for revetment
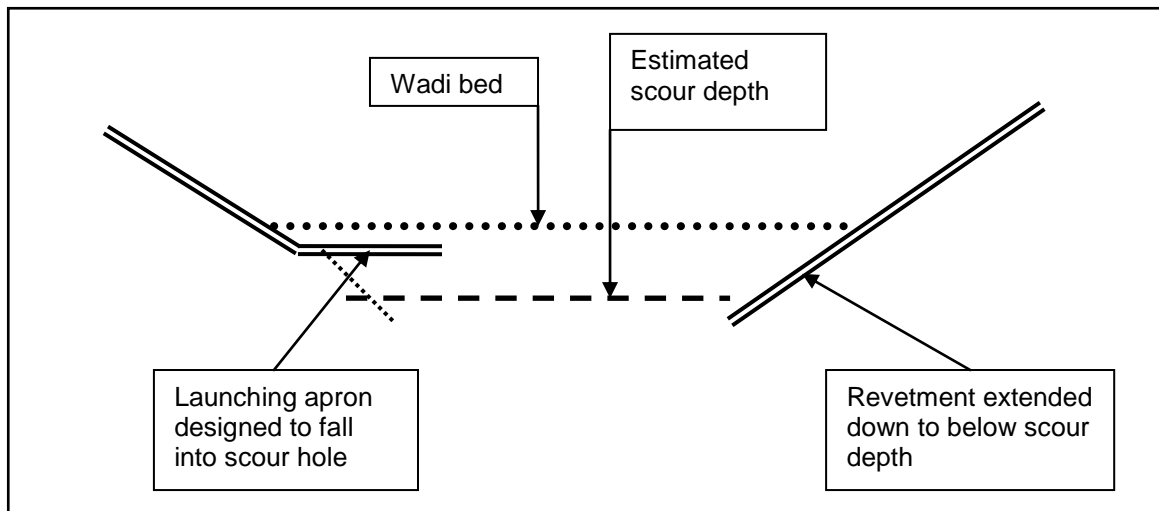
It is apparent from this description of the processes involved that a factor such as the surface finish of a revetment can have opposing effects in terms of local scour. On the one hand, a high roughness tends to reduce velocities close to the bank (which is beneficial), but on the other it tends to 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.

3.3.6 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.

3.4 Choice of materials

3.4.1 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).

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 haul routes 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.

3.4.2   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.

Riprap can be placed by machine and does not require hand placing or compaction. It can be placed underwater, although good quality control is needed to ensure an even coverage, and it is normal to increase the thickness placed to compensate for the greater difficulty of accurate placing. It can be placed in flowing water, although care is needed to avoid segregation and loss of the smaller stone fraction. The layout of riprap protection is often designed so that there is a surplus of material at the edges, so that if there is scour adjacent to the protection, stone will fall into the scour hole but will continue to provide protection. This is known as a falling (or launching) apron. Riprap is normally placed on a filter of either geotextile or granular material to prevent loss of the underlying material through the riprap voids.

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, failure tends to occur gradually, allowing time for repairs to be carried out, provided that the failure process is observed early enough.

Where stone is scarce, randomly placed precast concrete blocks have been used instead. In coastal situations, specially shaped proprietary concrete blocks (eg tetrapods) have been used instead of rock armour. The special shapes are designed to interlock better than standard rock shapes, in order to improve resistance to wave attack. They have also been used, although less extensively, in tidal and fluvial situations. Riprap can be grouted with cement or bitumen for a less flexible, less permeable but stronger revetment.

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.

3.4.3   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. Thicknesses between one and two thirds of those required for

riprap have been found from research. The mesh in gabions is usually fabricated from steel wire. The wire is either woven or welded. Woven wire tends to be more flexible and may therefore be more appropriate where significant deformation may occur. Welded mesh is more rigid and is more suitable where it forms an earth-retaining function, rather than just an erosion protection function. Welded mesh containers are easier to handle and fill. The wire can be galvanised, or galvanised and then coated with PVC, for additional corrosion protection. Gabions are less suited if there is significant wave attack because this tends to move the rock within the mattress, leading to stressing and abrading of the mesh. Their resistance to abrasion and wave attack can be improved by partially or fully grouting with mastic.

In gravel and cobble bed rivers, where abrasion may be a problem, the larger diameters of wire available (typically 3.0mm) can be used. In abrasive conditions a second panel of mesh is sometimes laced onto the exposed face of the gabion mesh, at right-angles to the lower layer, to reduce the risk of holes developing in the facing mesh. However this is usually only considered where repairs are being carried out to an existing structure that has suffered abrasion since, if these conditions exist, then an alternative type of scour protection is probably more appropriate.

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 3-6 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. These units are relatively large and inflexible and are used where flow velocities are high, or where they perform a river-training or earth-retaining function.

Figure 3-6 : Correct and incorrect weaving of gabions
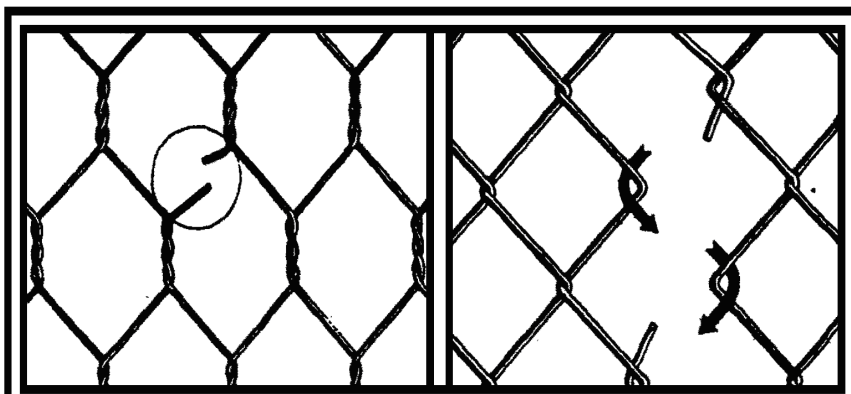
Gabion mattresses are more flexible than gabion boxes because they are thinner and because smaller stones and thinner wire are used in their construction. Due to their flexibility, mattresses are probably the most commonly used form of gabion for erosion protection. Mattresses are available in a range of sizes but typically have a thickness between about 0.15m and 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. Farther away from the structure or area of greatest risk of erosion, the protection can be reduced to a single layer of mattress. Gabions can also be filled in situ on banks. Gabions are normally filled 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.

Other concerns are the problems of vandalism and theft. These problems tend to be relatively rare, but should be considered in known problem areas. This can take the form of cutting out mesh for resale. In developed countries vandalism and theft of stone (for domestic construction work, for example) may be a problem. Riprap can suffer from the same problems of theft and vandalism. These problems can be reduced by random spraying of asphalt over the surface of the gabions or riprap, but this leaves them visually unattractive.

Under high flow velocities or high turbulence, using two mattresses, each of less than the required thickness laid on top of each other, provides better protection than a single mattress of the required thickness. Joints between mattresses should be staggered. Mattresses should be laid in such a way that the maximum dimension of a compartment in the direction of flow does not exceed 1.0m. Internal diaphragms are normally provided to reduce the size of individual compartments and limit any movement of the stone. On a sloping bank, the diaphragms should normally be set parallel to the line of the top of the bank, while on a flat bed they should be at right angles to the direction of the flow.

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

3.4.4   Stone pitching

Stone pitching is hand-placed stone, a traditional form of erosion protection that is still in widespread use worldwide. It usually consists of a single layer of single-sized stone, closely packed and laid on a gravel bed in dry conditions. It is labour-intensive, but needs little or no equipment, so it is particularly suited to isolated locations. small areas and where labour is cheap and plentiful. Stone can provide a more attractive finish than riprap or concrete block revetments. It can be placed with no mortar in the joints ("dry pitching"). or with the joints can be grouted with a cement mortar or bitumen.

Cemented stone pitching increases the stability of the protection and reduces the risk of loss of individual stones that could lead to progressive collapse. It also allows smaller stone sizes to be used. Either grouted or dry, stone pitching is relatively inflexible and can only accommodate small movement or settlement. Because of its inflexibility, a sound, well prepared subgrade is important to minimise settlement. In addition, regular inspection is recommended, as loss of individual stones can rapidly develop into widespread collapse. Placing the pitching on a gravel bed is important because it facilitates placing and provides a filter, reducing the risk of loss of subgrade. Typically, pitching stones would
be laid to a depth of 0.2 - 0.4m, on a gravel backing (20-50mm size), 0.1 - 0.2m thick. Slopes should not exceed 1V: 1.5H and 1:2 is preferred.

Like other inflexible systems stone pitching mainly fails by undermining. As such, a secure toe is needed on slopes. It is not well suited to applications where large lateral or vertical movement of the channel could occur. For example, where laid on slopes, if the toe of the slope is inadequately

keyed in, then scour at the toe can lead to extensive loss of stone. In similar conditions, slabs of grouted stone can collapse under their own weight in a cantilever action and slip down the bank.

Cemented pitching requires careful quality control to ensure that the cement mortar is applied for the full thickness of the pitching and is not a thin decorative layer placed over the joints which provides no structural strength.

3.4.5 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.

3.4.6 Stone size, grading and thickness

USBR[2] 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:
<div style="margin-left:2em">

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

D_{50} =   average stone size [m]
</div>

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 = $1.5D_{50}$
- Minimum stone size = $0.5D_{50}$
- Not more than 40% of the stone should be smaller in size than D_{50}

The thickness of the pitching should be 1.5 times the stone D_{50} size. It is usual to place the pitching on a filter layer, to prevent fines being washed out from the embankment during the flood recession. The stone pitching is laid on a filter layer, usually 0.15m thick although geotextile may be used.

3.4.7 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:

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[2]/   USBR ⱯHydraulic Design of Stilling Basins and Energy Dissipators⌐ United States Bureau of Reclamation. 1983.

$$d_{15} \text{ filter } / d_{85} \text{ soil} \quad < \ 5;$$
$$d_{15} \text{ filter } / d_{15} \text{ soil} \quad > \ 5; \text{ and}$$
$$d_{50} \text{ filter } / d_{50} \text{ soil} \quad < 25$$

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

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 singe 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.

3.5 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 evnt of long-term decay of the engineered infrastructure.