KONSO DEVELOPMENT ASSOCIATION
AND
FARM-AFRICA

SPATE IRRIGATION IN KONSO

REVIEW OF METHODS FOR IMPROVEMENT OF SPATE IRRIGATION

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R F Camacho
Spate Irrigation Specialist

FARM-Africa
9-10 Southampton Place
London WC1A 2EA
England

SPATE IRRIGATION IN KONSO - ETHIOPIA

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REFERENCES

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SPATE IRRIGATION IN KONSO - ETHIOPIA

REVIEW OF METHODS FOR IMPROVEMENT OF SPATE IRRIGATION

1. INTRODUCTION

1.1 Background

The Konso people of Southern Ethiopia have a long history of the effective utilisation of the rainfall that falls on their land by trapping, diverting and impounding runoff through elaborate systems of terracing and channelling.

The traditional homeland of the Konso is in a highland massif rising above dry savannah. It is in these highlands that their systems of traditional soil and water conservation are most developed. In recent years though, due to population pressure, the Konso have started cultivation in the lowlands, and in doing so have adapted some of their highland technologies to the lowland environment.

FARM-Africa has recently implemented a new project in Konso which aims to build local capacity to withstand the recurrent droughts afflicting this region. The Konso Capacity Building Project works with local communities and local organisations representing those communities to strengthen their drought coping mechanisms. Part of this programme is designed to address the issue of crop production in the lowlands by improving the effectiveness and reliability of the systems of spate irrigation being built in these alluvial areas.

From August 1999 Konso has been in the grip of a severe drought and FARM-Africa has facilitated an Employment Generation Programme utilising relief food from the European Union, Christian Aid and DFID to undertake a number of emergency employment activities paid on a food for work basis. These have included work on the spate irrigation systems, both in rehabilitation of existing systems and in construction at new sites.

This emergency phase will continue until the Konso are able to produce food again following the expected *beig* rains of March to May 2000. The work on spate irrigation sites is built into the original project design and will continue as an ongoing activity of the project beyond the emergency phase.

At this stage therefore FARM-Africa needs to be sure that the technical standards being used in the design and construction of spate irrigation are appropriate and technically suitable for their design use; i.e. that they will effectively convey water to, and distribute water within the field areas to allow crop production, withstanding the erosive impact of high volume/velocity spate flows and minimising the risk of massive erosion to structures and field areas. Using Konso indigenous technical knowledge as a starting point recognises the skills and experience available locally and this knowledge can be built upon in any improvements that may be suggested to existing systems.

1.2 Terms of Reference

A Spate Irrigation Specialist's was given a brief assignment to visit the spate irrigation schemes in Konso with the following terms of reference:

1. Review local knowledge and experience of spate irrigation and water spreading;
2. Suggest minor improvements in the light of Konso indigenous knowledge;
3. Run a short training workshop to introduce these minor improvements; and
4. Make an assessment within the *woreda* of the overall potential for spate irrigation.

1.3 Consultant's Visit Programme

The timing of the Consultant's visit was intended to be during the *beig* rains of 2000, which should have allow him to assess spate flows as they occur and the ability of existing systems to utilise those flows and the design modifications necessary to utilise them more effectively.

A Diary of the Consultant's visit, extending over a period of 17 days in Ethiopia is given at Appendix A.

On site, the Consultant worked under the direction of Michael Asefa, Acting Project Coordinator and Resource Planner and worked closely with Vincent Gainey of FARM-Africa and Mengistu Gebre, Water Engineer.

2 REVIEW OF SPATE IRRIGATION IN KONSO

The Konso people have started to farm the lowland areas of the Woreda in recent times, as they had previously avoided this zone, largely because of the incidence of malaria.

They have started to utilise the seasonal flows in the rivers, to irrigate the fertile alluvial plains in the valley bottoms.

The traditional techniques used include building stone and brushwood spurs into the stream beds to divert a proportion of the stream flow, and then channelling it along hand dug canals to the fields. The positioning, height and length of these spurs is a matter of sound judgement and considerable experience at each particular site. It requires a knowledge of the flood flows, their magnitude, frequency and duration, soils, crop water requirements, area to be irrigated, etc..

The spurs are partially or completely washed away by unmanageable flood flows which pass on downstream to spurs lower down the river where irrigation supplies are again diverted to other areas. The spurs are often washed away before the total command area has been irrigated and sometimes water cannot be diverted to the fields again until the spur has been rebuilt. How soon this can be achieved will depend on the time interval to subsequent river flows, and the availability of the labour force. Thus, the spur cannot always be rebuilt before the ensuing flood.

Traditional systems are relatively cheap to build but require considerable repairs to remain operative. If small to medium flows arrive, the spur can be effective, but medium to large floods can result in the expenditure of much effort with very little benefit as all the spurs may be swept away. Thus, the probability of irrigation of the total command area of each spur is variable and risk-prone. However, the basic principle is sound enough - that of diverting water at low stage and allowing large floods to pass unchecked.

The hand dug primary canals are deep and narrow and have steep bed slopes. Thus, the high river flows often produce scouring velocities in the canals, which are quite long, become more incised, thereby causing some fields to go out of irrigable command.

Where there is inadequate control at the canal intake a substantial and unmanageable proportion of the river flow can be diverted into the canal, widening and deepening it and in time the canal can be so damaged that it forms part of the river channel. This has occurred on the upper Taho River. Canal slopes may need to be modified, but care must be taken to insure that the reduction in bed slope does not cause sedimentation in the canals.

The primary spate canals taking off from the rivers generally have a large capacity in relation to the area irrigated because of the relatively short duration of the spate flows. The primary canal sub-divides to smaller canals as it reaches the irrigable area, and farmers exercise traditional forms of control so that the higher lands generally receive water first. Normally, irrigation supplies within the command area are distributed proportionally or by blocking the canals and distributaries with temporary earth bunds and cutting them when demands are met.

Within the fields, the water is distributed and controlled by an internal system of basins and small tertiary canals. These look very similar to what you might expect to see on a perennial irrigation system, with bunds about 300-400 mm high forming the basins. This indicates that the soils do not have the water retention capacity to hold a single spate application to mature the crops grown and that two to four applications may be required. Thus it is vitally important to have the ability to replace the diversion spurs quickly after they have been washed away.

Water rights are determined by the communities and seem to provide fair and equitable distribution and usage of available water supplies to all the participating farmers.

The construction and replacement of the spurs and distribution of flows through the canals are communal activities organized by the farmers. Costs are shared on the basis of benefits received, which depends on the area of land owned by each farmer, its elevation, and its proximity to the irrigation intake.

3 BASIC CRITERIA FOR SPATE IRRIGATION HEADWORKS

The basic criteria for satisfactory performance of a spate irrigation headworks should be to:

(1) divert all flows up to the design capacity of the primary canal;

(2) permit continued diversion whilst surplus wadi flows bypass the canal intake;

(3) incorporate a sediment excluder to deal with bed load;

(4) remain undamaged by large floods, or capable of repair quickly and economically; and

(5) continue to function with aggrading and degrading river bed levels.

A major headworks incorporating a permanent weir, canal head regulator, sediment excluder sluiceway and a breachable bund or "fuse plug", is usually required to meet all these criteria, but these would be much too costly for the small areas commanded by the proposed improvement schemes and lower cost solutions are required.

The design of the low cost headworks must be uncomplicated, so that they can be constructed by the farmers themselves, with some additional materials, labour and machinery and with a minimum of technical supervision.

These improved spate diversion works are intended to ensure that the canal intake and headreach are rendered semi-permanent and that the traditional diversion bund or spur would be damaged less frequently.

Some river training groynes will be required upstream to provide some protection to the intake against out-flanking and groynes will also be required downstream to protect the canal bank from scour by river flows, as the canal bank rises from the river bed.

Provision of a settling basin and a rejection spillway may be required in the headreach of the primary canal to help in managing the sediment entering the canal system and to reduce the damage which can be caused by excessive flows entering the ungated system at high river flows.

Each diversion site will have different characteristics. Plans must clearly be prepared in collaboration with the farmers as the improved spate diversion works would be operated in a manner to which the local farmers are well accustomed.

The design flood discharge of the spillweirs (or intakes) will be chosen for a return period of 2 to 5 years. The length of the concrete/masonry or rock-fill diversion weirs (or intakes) will be determined from the broad crested weir formula which may be taken as:

$$Q = 1.70\ LH^{1.5}$$

where
- Q = design flood discharge in m^3/s
- L = length of weir crest in meters
- H = total energy head in meters above the weir crest, with a maximum of 1.0 m, to restrict energy dissipation requirements.

The canal full supply discharge is determined by the formula:

$$Q = \frac{2.77 \times A \times W}{E \times N}$$

where
- Q = design full supply discharge (m^3/s)
- A = net commanded area (ha)
- W = average irrigation application depth (m).
- E = overall irrigation efficiency of the whole system.
- N = the average number of hours per crop season, that the river will flow in excess of Q.

As there is very little hydrological data for the two rivers, the design of each improved intake will have to be developed in collaboration with the farmers, who have knowledge of flood flows and their characteristics, as well as their diversion requirements.

The technical problems with diversion structures and canal systems on spate irrigation schemes can be summarised by the following deficiencies in planning and design of the spate improvement schemes:

- underestimation of the maximum flood in the design concept;
- underestimation of the sediment transported by the floods and inadequate provision for sediment exclusion and ejection at canal headworks;
- insufficient provision of total diversion capacity from the river or wadi and neglect of investment in downstream areas which have a reduced probability of receiving spate irrigation;
- underestimation of canal bed slopes and sediment transport capacity;
- inadequate assessment of the size and capacity of the irrigation inlets to the fields due mainly to a misunderstanding of the probable duration of the spate flows and the reduction of the flood wave in the irrigation channel as the wave swiftly passes downstream in the river or wadi.

The consultant's general overview of the technical problems with hydraulic structures and canal systems on spate schemes in Yemen are set out in his paper on "Traditional Spate Irrigation and Wadi Development Schemes" presented in the FAO/UNDP working paper on "Spate Irrigation", 1987, which is attached at Appendix B for easy reference.

The consultant's "Concepts and Design Philosophy for Spate Irrigation Improvement Schemes" are set out in worked examples using data from schemes in Yemen and Eritrea and is attached at Appendix C. The consultant reviewed and explained these concepts and design features to Mengistu Gebre in some detail during his visit.

The consultant's specific comments on the technical problems with diversion structures and canal systems on the traditional spate schemes in Konso are given below.

4. MINOR IMPROVEMENTS IN SPATE IRRIGATION AND BANK PROTECTION

4.1 Yandafero - Yanda River

A general description of the soils in the Yanda River area is given in a report on "Development Prospects in the Southern Rift Valley" by Land Resources Division (LRD), MOD, (Ref. 3), which classifies the area as one of the lateral fans of the upper Segan Valley.

"Lateral Fans

> Much of the area of the Upper Segan Valley comprises gently sloping fans. These border the hill masses and have been formed from material deposited by intermittently flowing streams originating in the hills. Both the Adei and the Gato (Yanda) tributaries flow into the Sagan Valley across extensive fans; here in consequence the flood plain is restricted. The materials on the fans are derived from basement rocks and become coarser in texture away from the river. Soils vary from clay loams on the more level areas at the foot of the fans, to sandy loams, often with gravelly layers, on the more undulating land towards the hills. The slopes would preclude irrigation on all but a few of the more level sites bordering the flood plain. Nevertheless, there is scope for diverting and spreading floodwater across certain of the larger fans. Indeed, this is already done by several farmers of the Conso tribe irrigating sorghum near the point where the Gato (Yando) River enters the valley."

A portion of a map presented in the LRD Report at Ref:3, shows the ecoclimatic zones in the Konso area and is attached at Figure 1. The Yandafero area is categorized as semiarid or arid lands having some potential for agriculture e.g. cotton, beans or early-maturing cereals.

The LRD Report at Ref:3 also presented an average rainfall isohyetal map covering the Konso Special Woreda. A copy of a portion of the map is attached at Figure 2. It would be interesting to find out whether rainfall records over more recent years have significantly altered these average isohyets.

The existing Yandafero irrigation scheme consists of a series of some 27 small intakes and canals from the Yanda River. These are shown on the sketch plan at Figure 3, prepared by Foreman, Worku Karaffo from Jarsso, which shows the general layout and other information. Most of these intakes were severely damaged by recent floods.

Vince Gainey's Report (Ref:1) alludes to an estimated potential irrigable area of some 4,000 ha farmed by about 4,000 families. The irrigable area is probably over estimated, because the whole lateral fan of the Yanda down to the Segan is about 4,000 ha and it is highly unlikely that this whole area is suitable for irrigation and the water is available to do so. The topographic map of the Yanda lateral fan is shown at Figure 4. Each square of the grid is 1 km^2 = 100 ha and on this basis the total area of the lateral fan was estimated at about 4,000 ha. The irrigable area is probably of the order of 1,000-1,500 ha.

The number of families benefiting from the scheme needs to be agreed. Vincent's Report mentions - nearly 4,000 families; Michael spoke of about 1,000 and if the figures shown on Worku Karaffo's sketch are numbers of farmers, then when added, the number exceeds 8,000 farmers.

The recent large flood in addition to sweeping away most of the intakes, has also caused very severe erosion thereby deepening of the bed of the river by about 3-4 metres and as a result several spate diversions intakes can no longer command their former areas. This erosion of the bed extends from the flood plain of the Segan for some 6 km up the Yanda.

Around 1995, the local NGO Mekane Yesus (MY) conducted a survey and feasibility study for a dam upstream on the Yanda. The preferred site is between Gandayla mountain on the right bank and Komboroti on the left bank. Two other sites about 1 km and 1.6 km upstream of the preferred site were also surveyed. The preferred site would have a large reservoir storage volume, with side spillway on a low saddle on the right flank. The consultant was hoping to see this feasibility report to get some information on the catchment area, hydrology, water resources, etc. of the Yanda at that site, but without success.

The KDA are anxious to proceed with the spate irrigation improvement scheme and the MY proposal would provide control for the spate flows, but little further action seems to have been taken to implement this proposal.

The most pressing needs for assistance on this scheme are:

(1) redress the severe problem of erosion of the river bed to bring previously irrigated areas back into command;
(2) improvements to the spate irrigation intakes;
(3) possible improvements to the primary diversion channels;
(4) providing improved river bank protection work.

(1) River Bed Erosion

The causes of this erosion of the bed are not clear; see photograph on the cover of this report. It might be partly due to larger floods with more intense peaks, caused by roadworks and erosion in the catchment, or it may be due the bypassing of an obstruction in downstream channel which has caused the bed slope to adjust to the new flow regime. It may be of interest to note that similar bed and bank erosion is taking place in some Yemen wadis, notably Wadi Mawr, mainly due to degradation in the catchment.

What are the possible short term and long term options for redressing this natural phenomenon?

The first short term proposal put forward by the farmers was to build a check dam in the river some distance downstream. The farmers had built a small one but it was washed away.

The topographical map of the Yandafero area at Fig. 4 shows the 880 and 900 contours and the distance between them is about 3 km. Thus, the ground slope in the vicinity of a check dam site would be around 1 to 150. As the depth of the cut in the bed is about 3-4 m and the widened and deepened channel downstream is about 6 m deep, the check dam would have to be sited about 500 to 900 m downstream of the start of the cut, to check further erosion at that point. But even if the check could be immediately put in place there would be some further cutting back until sufficient ponding and siltation occurred to create the necessary backwater effect.

The height of dam would have to be about 3 m above ground level at the selected site and thus some 9 m high at the deepest river section. A conceptual outline of a check dam of this height is at Figure 5. There would be flanking embankments 3 m high at the river section, which would end at ground level 1 m higher than at the river section. Gabion spillways would be installed at both ends of these diversion embankments.

Site surveys and investigation are required to determine the length of the embankments and their alignments and the upstream and downstream side slopes of the dam and flanking embankments. The slopes shown on Fig.4 have been used as it is assumed that the fill material would be borrowed from the upstream area of land adjacent to the site and its suitability has not been checked.

Hydrological data will be required to determine suitable lengths for the gabion spillways.

It is unlikely that the necessary funds can be raised and earth moving equipment mobilised (scrapers, bulldozers, wheel loaders, etc.) in the next few weeks or months to implement the work and it would be difficult to raise money for this purpose when it is realised that the storage volume of the check dam will be filled with silt in a few years and a new channel or channels will be established around the embankments.

This is not a sustainable proposal and is not recommended.

The farmers have partially accepted this natural phenomenon. They have already commenced work on the second option and the only one which is sustainable and which they can implement immediately. They have started to construct new and longer intake channels from the eroded and lower bed of the river to command new areas. These new canal banks will need more and better bank protection as the intake canals rise out of the river bed.

The third option is long term and concerns the construction of a storage dam which was the subject of a feasibility study undertaken under the auspices of Mekane Yesus. The site would be located between Gandayla mountain on the right bank and Komboroti on the left. The dam would largely control flood flows and release water supplies in manageable quantities to farmers downstream. Their diversion works would thus be much less susceptible to damage.

Such a dam with its spillway and outlet works might be difficult to justify in economic terms as the area which could benefit incrementally, might only be of the order of 1,500 ha. However, the social and humanitarian aspects may carry more weight in the context of the prevalent food shortages in Konso.

The fourth option is also long term and concerns the construction of two spate diversion headworks providing all the facilities shown in Figure 6. One to command right bank areas and the other, the left bank areas.

Each headworks would comprise: a diversion weir, a breachable bund, a canal head regulator, a sediment excluder sluice, a sediment settling basin, sediment ejector and rejection spillway. A description of the purpose and design of these works is given in Appendices B and C.

Primary canals from the canal head regulators would distribute water to the existing canals systems and to extended areas downstream. Separate intakes from the river would no longer be necessary.

A full feasibility study of this outline proposal would be required as such spate diversion headworks might also be difficult to justify in economic terms as the area which could benefit incrementally, might only be of the order of 1,500 ha. Much emphasis would have to be given to the social and humanitarian aspects of the situation in Konso.

FARM could seek to generate interest in such a study from international agencies such as UNDP/FAO, IFAD and ADB.

(2) Improvements to the Spate Irrigation Intakes

Unfortunately, there is not much scope for improving the brushwood diversions from the river because there are virtually no boulders in this reach, it has a sand and gravel bed and there is no animal or mechanical power available, only manpower.

Boulders would have to be imported to the sites for gabion or rock-fill structures and masonry and concrete structures would require a greater degree of technical assistance and supervision. All these options would be very expensive in relation to the relatively small areas commanded by each intake and none would be easily sustainable by the farmers, without some outside assistance.

Usually, farmers have animal power for ploughing. This power could be put to alternative use with scrape boards for rapid replacement of diversion bunds, in sand and gravel river beds, after a flood had washed out the bund. See photographs at Figure 7, showing dam rebuilding in Yemen some 50 years ago, after a flood had washed it out. Unfortunately, Yandafero farmers do not use ploughs. The farmers on the Taho use oxen and ploughs, but the bed material in the Taho is mainly boulders, cobbles and gravel, which is unsuitable for scrape boards.

(3) Possible Improvements to the Primary Diversion Channels

Until there is better control of flows entering the primary canals it is difficult to suggest improvements in canal design. It is highly probable that, from practical experience, the farmers have adopted the shape (deep and narrow), and the length of the incised cut channel, to partially control the flows entering the distribution system.

One means of providing better control of the flow entering the primary canal is to provide a rejection spillway in the head reach of the diversion channel. There is clear evidence that, in some cases, the farmers provide this facility or that the channel naturally breaks back the the river at high intake flows.

Notes on primary canal design and design tables for Simons and Albertson Canals is presented in Appendix C.

(4) River Bank Protection

Farmers regard their land as a priceless asset and bank protection work is therefore given a high priority by them. However, the armouring of river banks to resist erosion and subsequent loss of agricultural land is costly and difficult to justify in economic terms as water, not land, is the major limiting factor in spate irrigated areas.

Such river bank protection and training works are more readily justified:

- where villages, bridges, roads, canals and diversion structures need a greater degree of security.

In practice, however, account has to be taken of environmental factors and the progressively deleterious effect of doing nothing to check erosion and reclaim land. Moreover, specific action will be necessary in some cases, to safeguard the proposed improvements to the distribution system where these may be threatened by river bank erosion.

Typical erosion control works using boulders and gabions are shown at Figure 8. Both the gabions and the boulders would have to be imported for such works in the lower Yanda. The works would be expensive and could not be sustained by the farmers without assistance.

Appendix D is an extract from a FAO Preparation Report on a Second Land and Water Conservation Project in Yemen, prepared by Salah Rouchiche, FAO Forestry Consultant. The project proposal was to experiment with the use of vegetation to assist in wadi bank protection. Pages 27 & 28 of Appendix D describe the proposals and rational. The plant **Tamarix aphylla** was to be used. It grew naturally in the wadis. It is suggested that **Lowland Grawa** be tried as an experiment in Yandafero, instead of Tamarix.

The use of vegetation may be justified because it can withstand normal floods and when damage occurs in large floods, it sprouts again and regenerates. Observations in the field indicate that vegetation reduces the flow velocity, causing sediment to be deposited in front and behind the vegetative barrier. Coarse sediments and silt transported during high and medium flows mix with vegetative debris at low flows combine and build up to form a natural protective structure.

The difficulty is in trying to establish the vegetation where it is needed to assist in protecting the river bank as natural vegetation in wadis usually occurs where the flow velocity is low and seeds are deposited and covered with enough sediment to cause germination. Unfortunately, these sites are not where the bank protection is needed.

Vegetation can only be established where it is required by planting cuttings deep and giving them protection against scour and wash out. Illustrations of how this might be achieved is shown in the Figures attached to Appendix D. The vegetation would be planted in good wet soil at the bottom of the various sections of ditch and backfilled with graded material ranging from sand immediately above the soil, through gravel and shingle to large boulders on top.

It is suggested that FARM discuss this subject and its possibilities with the Yandafero farmers to see if they are receptive to carrying out trials on selected and agreed sites. If the farmers show interest, plans would be prepared in close consultation with them on the basis that

FARM would assist with the provision of boulders, supervision and any technical assistance that may be necessary. The farmers would provide the labour. FARM would monitor the outcome of the trials.

4.2 Taho River

There are two diversion sites on the upper Taho River that once commanded two quite large areas. First, at Hagayawa left bank intake site, serving the Merssa area of about 1,200 ha and farmed by about 600 families and the second, an upper intake site serving the right bank Kantada area of some 800 ha and farmed by about 400 families. Both intakes were completely destroyed by large floods and require reconstruction, which will be difficult and costly.

The upper intake for Kantada will require considerable earthworks or a quite long (70 m) and high (4-5 m) masonry retaining wall to Separate the diversion canal from the river flow or both may be required. The reconstruction of the intake will be difficult for the farmers to carry out without assistance, but the cost may be more than FARM can readily find. A survey of the site is needed initially. FARM could talk to the farmers to find out how they might be able to help. It is reported that the Regional Development Office at Awasa have plans for a Dam or Spate Breaker upstream on the Taho.

At Hagayawa intake site, the river has taken a course down the canal and by-passed a loop of the old river bed. This will be costly to rectify. The command area served is about 1,200 ha and may be large enough to justify more permanent headworks in concrete and masonry on the lines indicated in Figure 6. Certainly the remaining canal section in the head reach seems large enough to justify it.

Alternatively, a gabion headworks might be more appropriate in the circumstances as boulders are available on site and in the vicinity. Figure 9 shows some typical improvements that could be adopted for this traditional spate canal intake and Figure 10 is a sketch showing improvements to traditional spate diversion works using gabions. Both these Figures show right bank offtakes, but at Hagayawa the canal is on the left bank, so these Figures would have to be studied when turned through 180°.

The reconstruction, whether in masonry or gabions, needs careful study and detailed survey of the site is essential.

Lower down the Taho, near the road, there are two small spate diversion schemes at Beayde and Fuchucha. Several small offtakes serve these potential irrigable areas which total about 225 ha.

Soils are fertile and easily workable silty clays with some vertisol development close to the river bank and some stone and gravel deposits at a depth of about 1.5 metres in the alluvial material of the river banks. Main crops grown are maize and sorghum.

It is estimated that 100-150 families are farming in these two areas.

The irrigation intakes are temporary diversion structures built by the farmers from stone and brushwood. All of these structures were badly damaged by recent floods. The farmers report that, when the intakes are intact, the peak flows of the river, which is sufficient to reach the intake, only occur for two to three days at a time when the river is in flood.

The main technical requirement at these sites is improvements to the spate irrigation intakes and primary diversion channels.

The areas commanded by each intake range from about 10 to 25 ha. These small areas would have to be served by a single primary link canal taking off from an improved headworks upstream. This would be the only way of justifying the inputs and the effort that will be involved.

The general arrangements and layout of an improved spate diversion headworks, designed on the basis of using rock-fill, boulder riprap and filter fabric (such as Terram), is shown at Figure 11. It is possible that such a structure could be built by the farmers, under the food for work programme, with supervision provided by FARM.

The following notes give some guidance for the design of a rock-fill (tipped stone) headworks.

(a) The intensity of the design flow over the spillweir should not exceed $3m^3/s$ per metre. This intensity of flow can be partially controlled by setting the top level of the breaching bund at about 1.2 metres above the crest level of the spillweir. This will allow for about 0.3 m of further flood rise while the bund is being over-topped and the breach is developing.

(b) The average weight per stone of the top layer of stones should be about 0.07 tonne which is equivalent to a sieve size (d_s) of about 0.35 m. The thickness of the top layer should be about 1.1 d_s.

(c) The underlayer thickness under the tipped stones of the top layer should not be less than the top-stone seive size (d_s), and the median stone size of this underlayer should not be less than 1/6 of the top-stone size.

(d) Scour protection at the end of the weir and downstream of the abutments, based on a falling apron principle, is necessary.

(e) In the top layer of stones, no stone should protrude by more than half its diameter above the design surface profile. No hollow should be left below the design surface profile if it can be filled without infringing the above condition.

(f) In areas where access is required the surface should be evened up with smaller material after the conditions in (e) have been satisfied.

A detailed site survey and data on hydrology and river flows are essential information necessary for the preparation of a plan for a specific site.

5. ASSESSMENT OF SPATE IRRIGATION POTENTIAL

5.1 Yanda River

The Yandafero area has a potential irrigable area of possibly 1,500 ha., which might be developed with the construction of two spate diversion headworks providing all the facilities shown in Figure 6. One to command right bank areas and the other, the left bank areas.

Primary canals from the canal head regulators would distribute water to the existing canals systems and to extended areas downstream. Separate intakes from the river would no longer be necessary.

A full feasibility study of this outline proposal would be required as such spate diversion headworks might be difficult to justify in economic terms as the area which could benefit incrementally could be relatively small. Much emphasis would have to be given to the social and humanitarian aspects of the situation in Konso.

FARM could seek to generate interest in such a study from international agencies such as UNDP/FAO, IFAD and ADB.

5.2 Taho River

The reconstruction of the two improved spate diversion headworks to serve the Kantada and Merssa areas, totalling some 2,000 ha, deserves serious consideration. Site surveys an a feasibility study would be necessary as the structures would be quite expensive.

5.3 Segan River

5.3.1 Lower Segan

There is a good potential site for (spate) irrigation at Segan Gete (Tebela and Kochela) on the lower Segan River about 13 km from its confluence with the Weito River, see Figure 12. There is about 1,500-2,000 ha of land which could be commanded from the Segan, which is virtually a perennial river. A full feasibility study would be a prerequisite to any irrigation development. At present, a small portion of this area is used for growing maize and sorghum and the remainder used mainly for livestock and cattle rearing.

At Burgitcha (Kochela) a group of farmers divert water from the Segan, into a small canal about 1.5 m wide a 0.8 m deep, to irrigate their fields. They use stakes, logs and brushwood to divert flow from the Segan. They irrigate their crops two or three times depending on the type of soil.

This area is classified in Ref. 3 as "Semiarid or arid lands where, despite relatively low or erratic rainfall, a combination of altitude and latitude or local site conditions confer some potential for agriculture (e.g. cotton beans or early maturing cereals). These areas are characterised by dryland acacias, with some broad-leaved trees or shrubs." This is a fitting description of the Kochela area.

The farmers in the area have taken the first steps in developing the land for irrigated agriculture and should be encouraged and given every assistance.

At Segan Gete (Tebela), a gated intake and the head reach of the canal is being constructed of masonry. The canal capacity will be about 1.75 m^3/s and is intended to irrigate an area of about 200 ha in the first instance. After this initial development it is intended to expand the scheme to take in the Burgitcha (Kochela)? area.

There will be no weir or other structure to divert the river flow. Thus low river flows will bypass the intake as only high river flows can be taken in. One of the disadvantages of this design is that the higher river flows will transport a much higher concentration of sediment, much of which will be deposited in the canal system.

The higher river flows will be more difficult to control at the intake gates, but provision is to be made for a rejection spillway at a suitable site on the head reach of the canal.

5.3.2 Middle Segan

The consultant was not able to visit the Middle Segan area due to an impassable road, but the following extract is taken from Vincent Gainey's report of August 1998 (Ref:1).

"The Segan River is a semi-permanent water course on the eastern boundary of Konso with Borana. There is a promising diversion site adjacent to a bridge across the Segan River into Borana.

Soils here have quite a high clay content but are considered quite workable with hand tools or oxen.

This site has already been developed by the local farmers who are irrigating about 120 ha from a number of sources including the Segan and from ephemeral streams flowing out of the hills to the west.

There are about 150 farming families growing maize, sorghum, potato and soybeans in this scheme area.

Again, the most important technical requirement at this site is improvements to the spate irrigation intake and primary diversion channel."

The Mid-Segan area definitely has potential for spate irrigation development but the extent of the possible command area has not been determined.

6. DISCUSSIONS AND TRAINING WORKSHOP

6.1 Professional Discussions and Training

The consultant had three sessions, totalling about 9 hours, of discussion and professional training with Mengistu Gebre on the technical aspects of spate irrigation engineering. The concepts and design features given in Appendix C were reviewed and explained in greater detail.

6.2 Workshop Feedback

A Workshop Session, which lasted about 3 hours, was also held with discussion group of 24, comprising Development Assistants, Foremen, EGS Supervisors & Farmers. Problems which they encountered were discussed and possible solutions reviewed with the group. Potential development sites were also considered.

6.3 Possible Visit to Yemen

A visit to Yemen of a selected group from Konso might be usefull and informative. The group could go to Hodeidah and visit Wadi Mawr, Wadi Rima, as well as some of the small wadis on the Tihama. They will then see both the traditional spate irrigation practiced on the small wadis at present, as well as the major spate irrigation improvement schemes of Wadis Mawr and Rima. At least three days on site in Yemen plus travelling time would be required for such a visit.

7. SUMMARY OF OBSERVATIONS

7.1 Observations on Minor Improvement Works

It is suggested that FARM discuss the possibilities of utilising vegetation to assist with river bank protection work, with the Yandafero farmers to see if they are receptive to carrying out trials on selected and agreed sites. If the farmers show interest, plans could be prepared in close consultation with them on the basis that FARM would assist with the provision of boulders, supervision and any technical assistance that may be necessary. The farmers would provide the labour. FARM would monitor the outcome of the trials.

On the lower Taho river an improved spate diversion headworks might be tried. The general arrangements and layout of such a headworks, designed on the basis of using rock-fill, boulder riprap and filter fabric (such as Terram), is shown at Figure 11. It may be possible that such a structure could be built by the farmers, under the food for work programme, with supervision provided by FARM.

A detailed site survey and data on river flows are essential information necessary for the preparation of a plan for a specific site.

7.2 Spate Irrigation Development Potential in Konso Woreda

The proposal for a spate irrigation improvement scheme in the Yandafero area concerns the construction of two spate diversion headworks providing all the facilities shown in Figure 6. One to command right bank areas and the other, the left bank areas.

A full feasibility study of this outline proposal would be required as such spate diversion headworks might be difficult to justify in economic terms as the area which could benefit incrementally, might only be of the order of 1,500 ha. Much emphasis would have to be given to the social and humanitarian aspects of the situation in Konso.

FARM could seek to generate interest in such a study from international agencies such as UNDP/FAO, IFAD and ADB.

The reconstruction of the two improved spate diversion headworks to serve the Kantada and Merssa areas, totalling some 2,000 ha, deserves serious consideration. Site surveys an a feasibility study would be necessary as the structures would be quite expensive.

There is a good potential site for (spate) irrigation at Segan Gete (Tebela and Kochela) on the lower Segan River about 13 km from its confluence with the Weito River, see Figure 12. There is about 1,500-2,000 ha of land which could be commanded from the Segan, which is virtually a perennial river. A full feasibility study would be a prerequisite to any irrigation development.

The Mid-Segan area definitely has potential for spate irrigation development but the extent of the possible command area has not been determined.

Ecoclimatic Zones

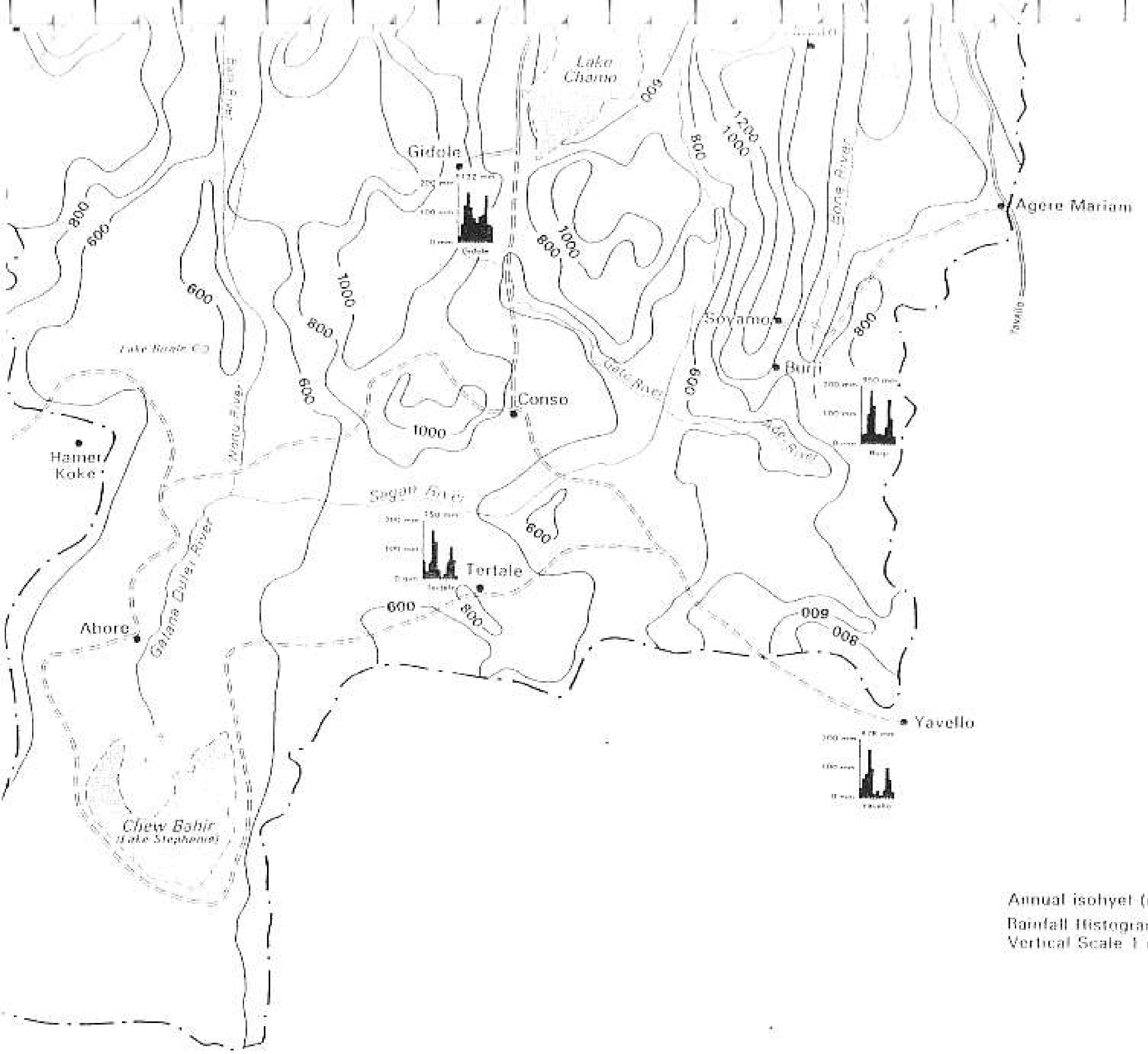
1 Humid to dry subhumid lands, now mostly under coffee or other intensive agricultural use, but formerly forest or montane grassland.

2 Dry subhumid or semiarid lands characterised by evergreen shrubs, *Combretum* or allied vegetation, usually of good potential for agriculture (soil and topography permitting) though with a restricted choice of crops and farming systems.

3 Semiarid or arid lands where, despite relatively low or erratic rainfall, a combination of altitude and latitude or local site conditions confer some potential for agriculture (e.g. cotton, beans or early-maturing cereals), characterised by dryland acacias, with some broad leaved trees or shrubs.

4 Arid lands, mostly dry thorn bushlands unsuitable for rainfed agricult... though ... erati... tial fe... ssive ... k pro... r w...

IG. Ec lim c Z es i Kon Spe l W eda



Annual isohyet (mm) — 1000 —
Rainfall Histograms
Vertical Scale 1 mm=20 mm Rainfall

Specialist information by T.J. Kingham

Stations for which histograms of monthly average rainfall are shown

Rainfall station	Record used
Adamitulu	1956-69
Alaba Colito	1966-73
Arba Minch	1960; 1963-6; 1969-72
Asella	1966-73
Awassa	1961-73
Bacco	1954-61, 1964-6
Bilate Agricultural Estate	1971-3
Burji	1956-72
Gidole	1954-71
Hosaina	1953-68; 1972-3
Kofele	1955-72
Soddu	1954-66; 1971-2
Tertale	1970-3
Yavello	1957-72
Yirga Alem	1955-71
Yirga Chefe	1966-8, 1970-3

Rainfall records are summarised in Appendix 7

FIG. 2 Average Rainfall Isohyetal Map in Konso

FIG. 3 General Layout of Intakes & Canals in Yandafero Area

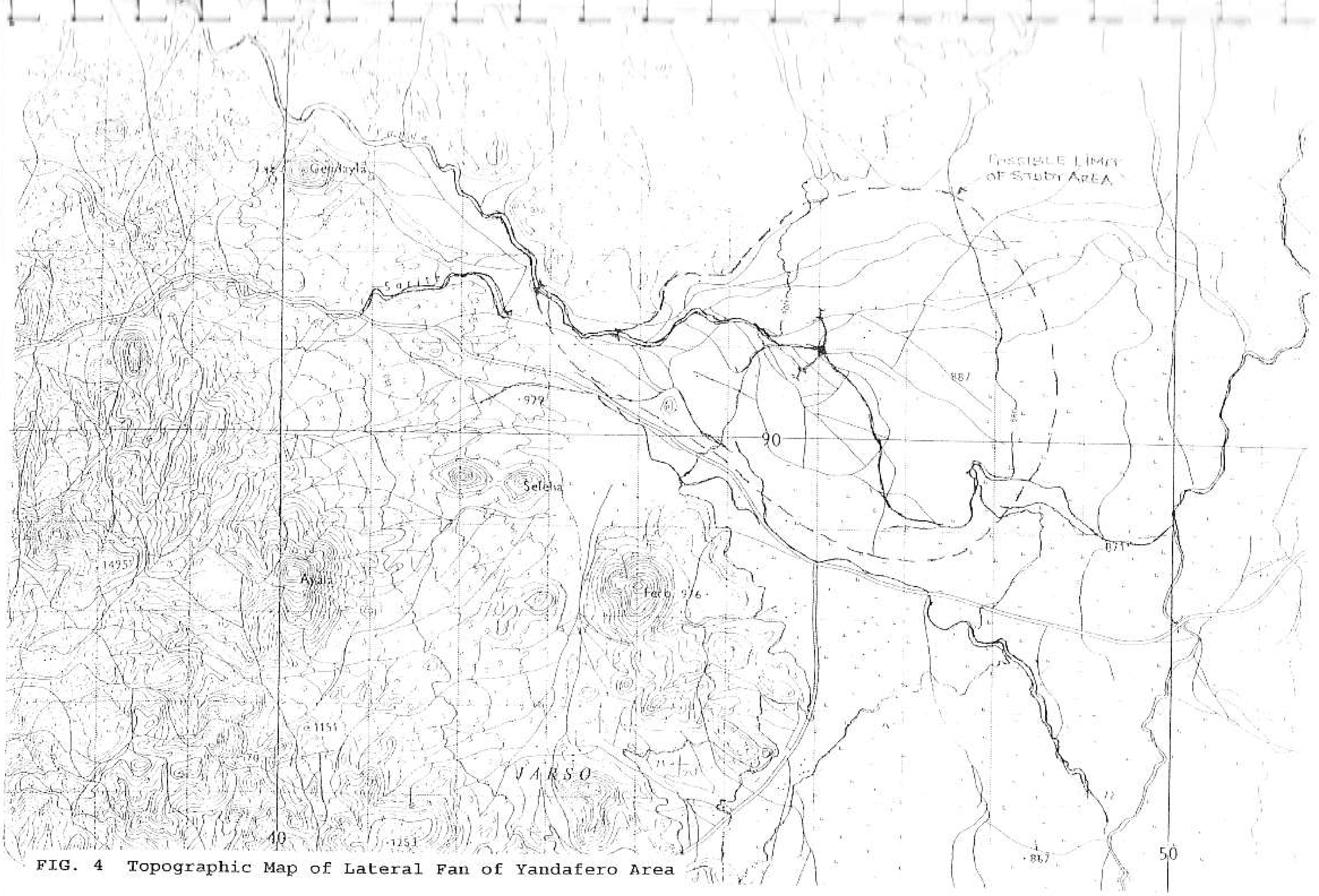
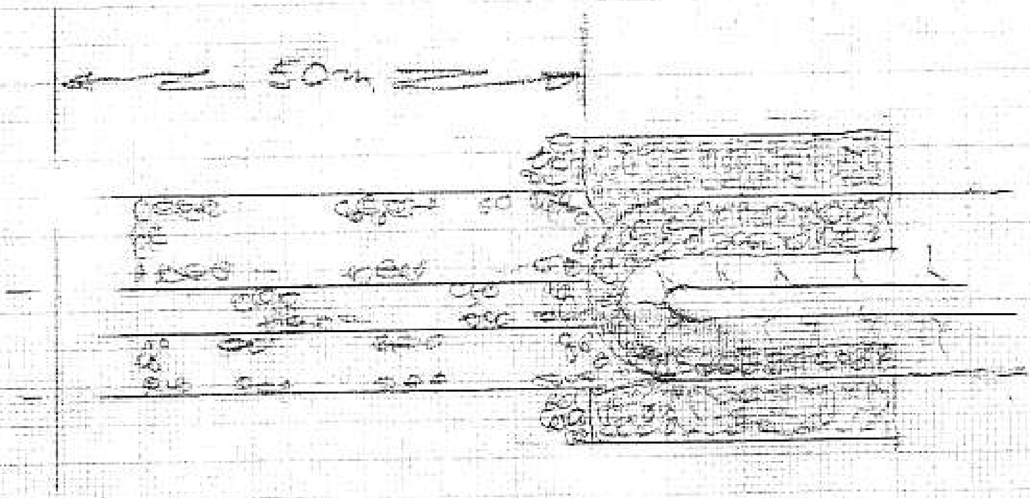
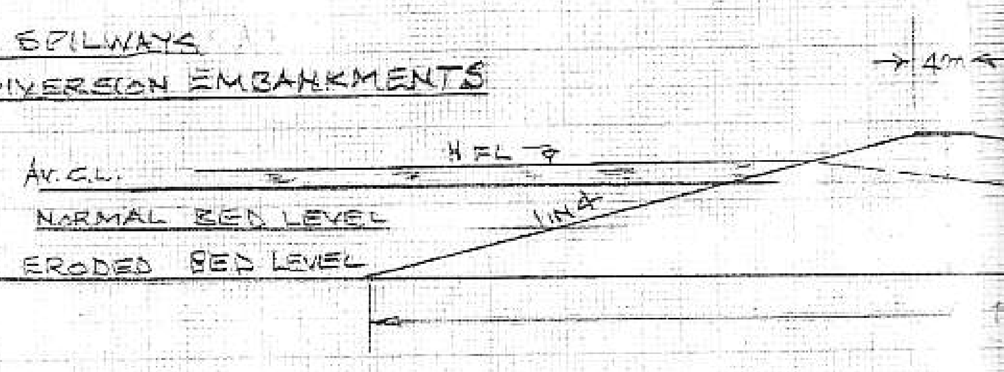


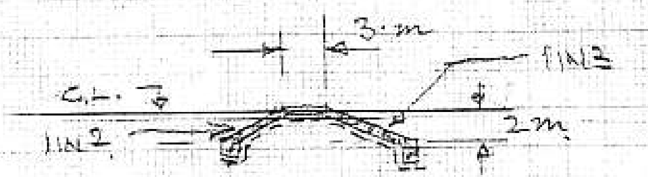
FIG. 4 Topographic Map of Lateral Fan of Yandafero Area



PLAN OF SPILWAYS
AT ENDS OF DIVERSION EMBANKMENTS
SCALE 1:500

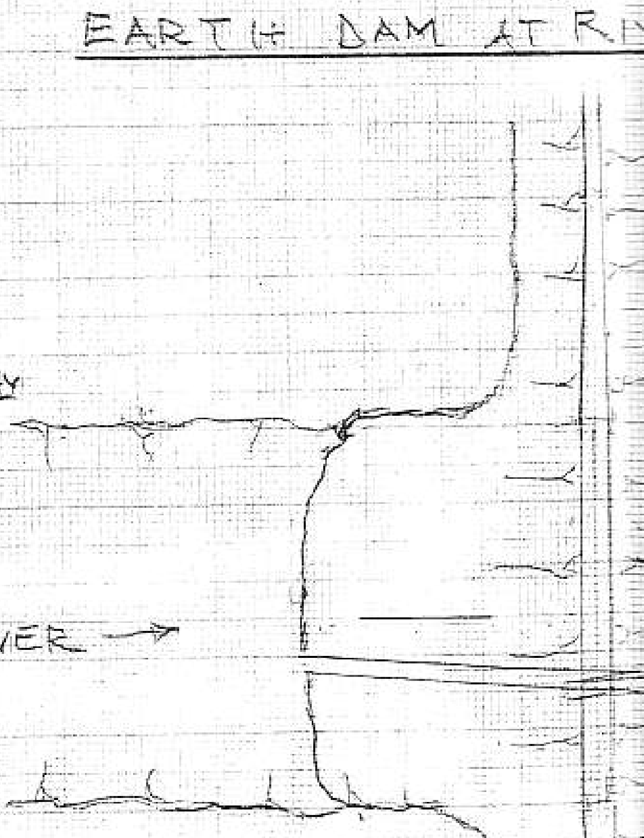


EARTH DAM AT RI



SECTION OF GABION SPILWAY

SCALE 1:500



PLAN

RPC
21/5/00

CONCEPTUAL PLAN
FOR CHECK DAM
ON YANDA RIVER

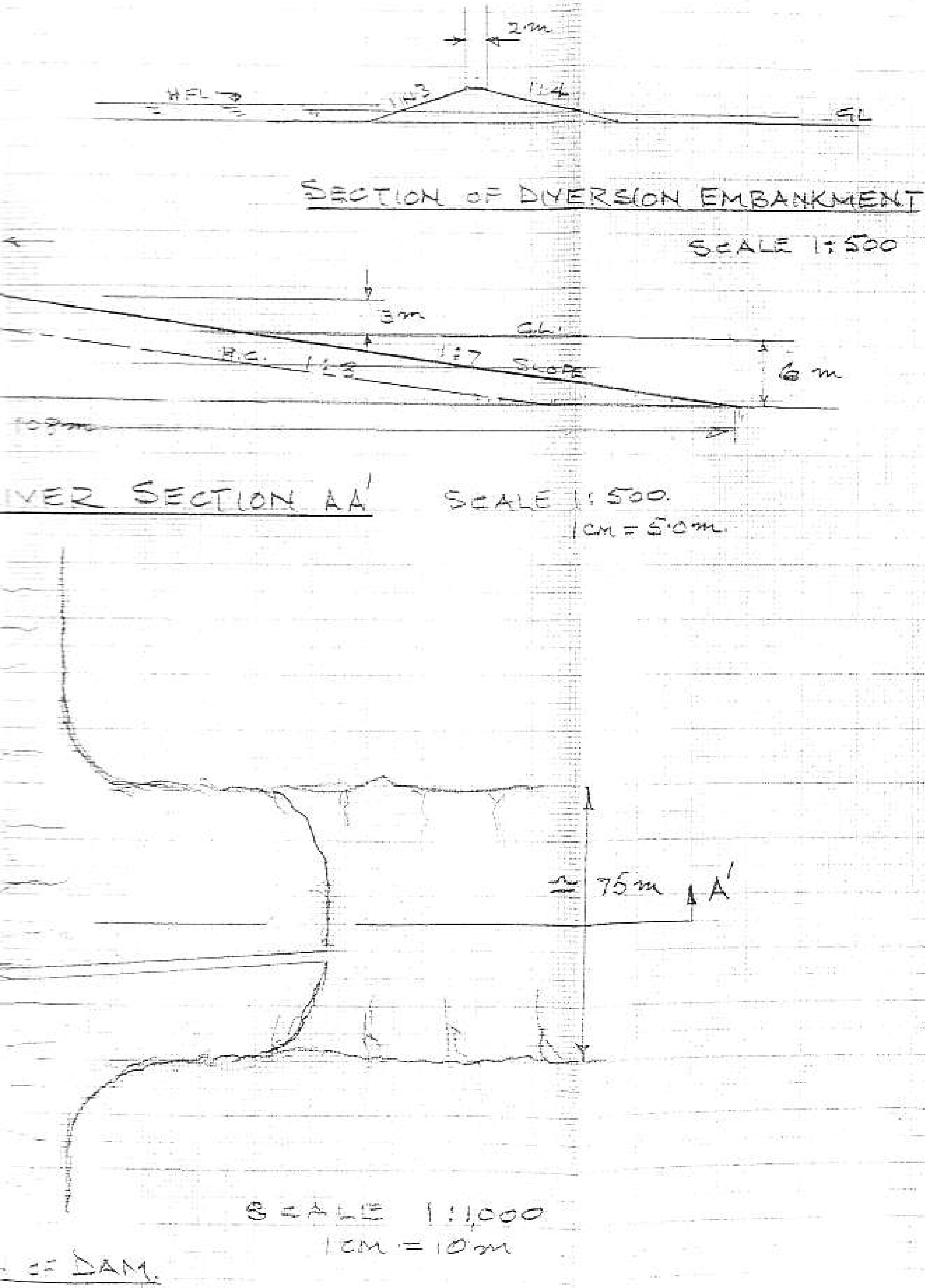


FIG. 5 Conceptual Plan for Check Dam on Yanda River

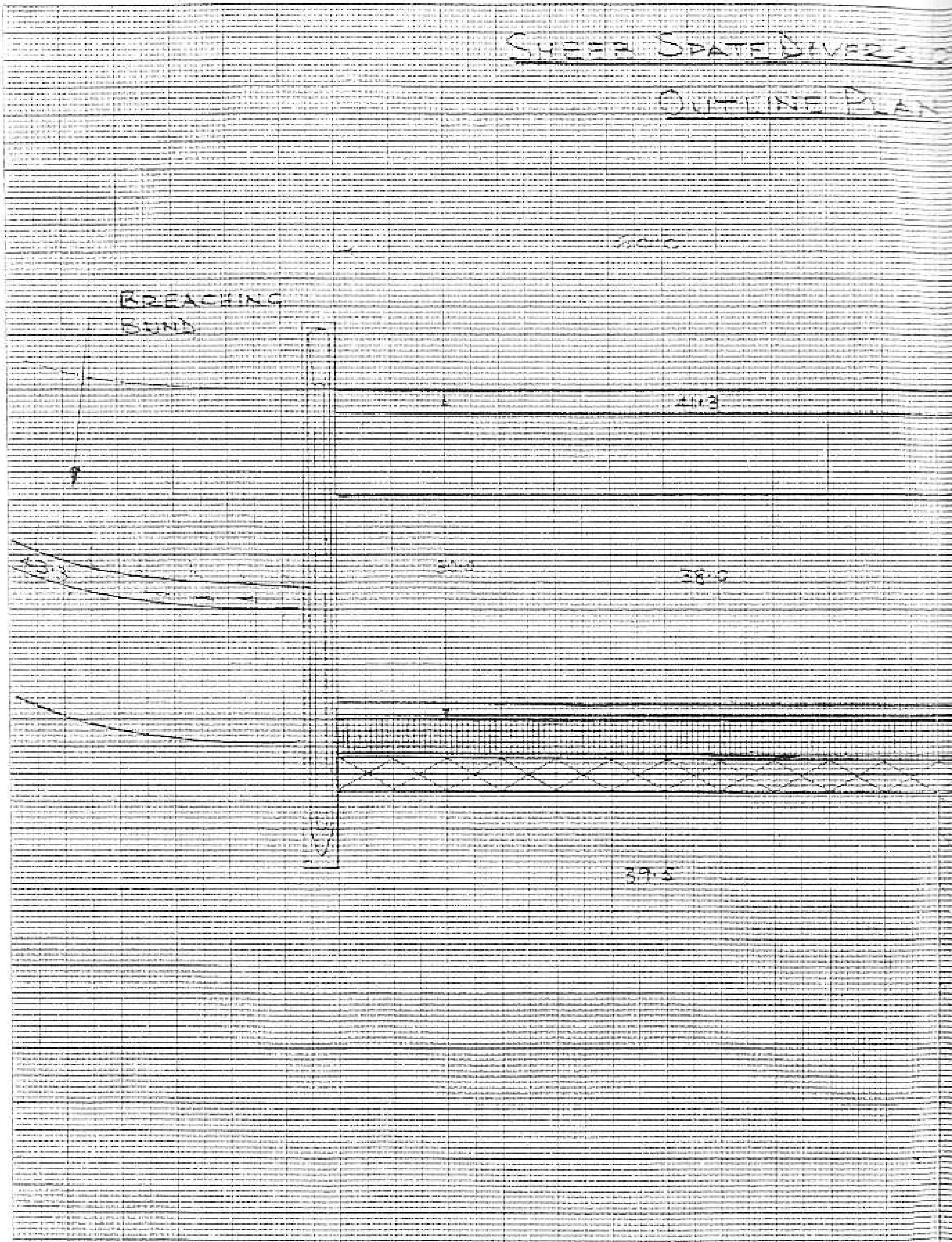

SHEER SPATE DIVERS
OUTLINE PLA
BREACHING BUND
SCALE 1:500

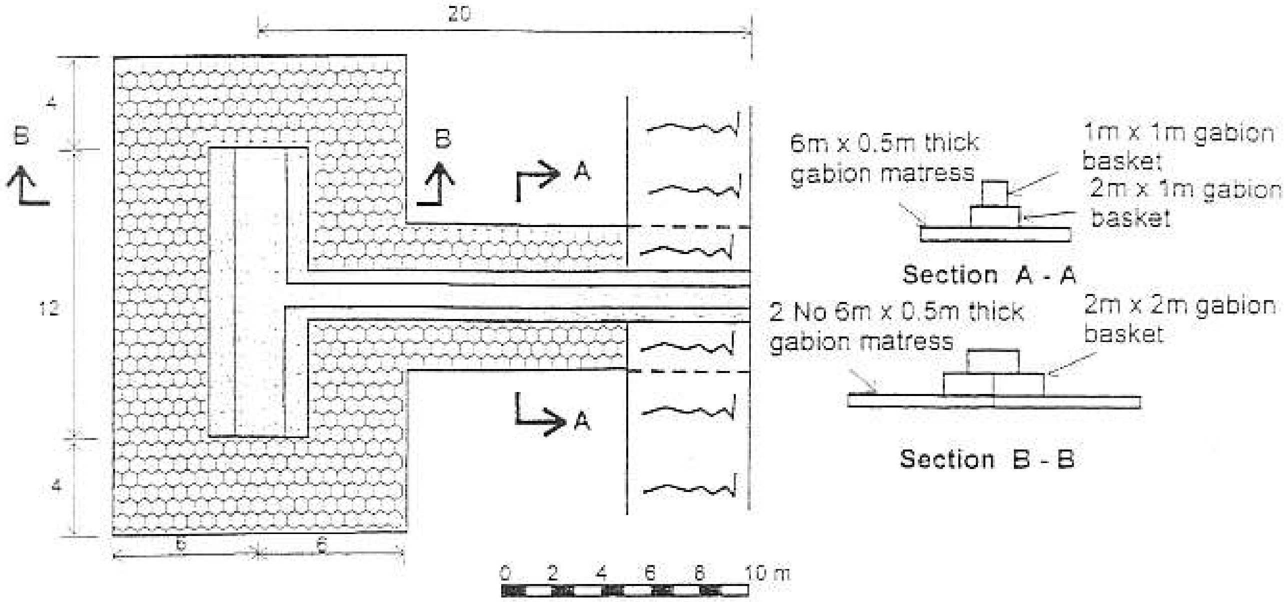
FIG. 6 Typical Layout for Spate Diversion Headworks

Dam building, Western Aden Protectorate

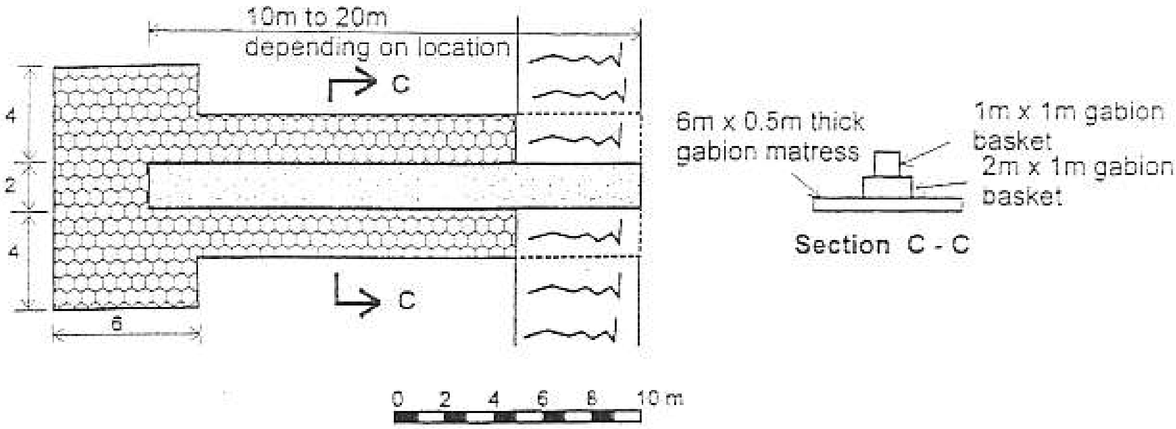
Dam building, Western Aden Protectorate

FIG.7 Diversion Bund Rebuilding in Yemen in 1950's

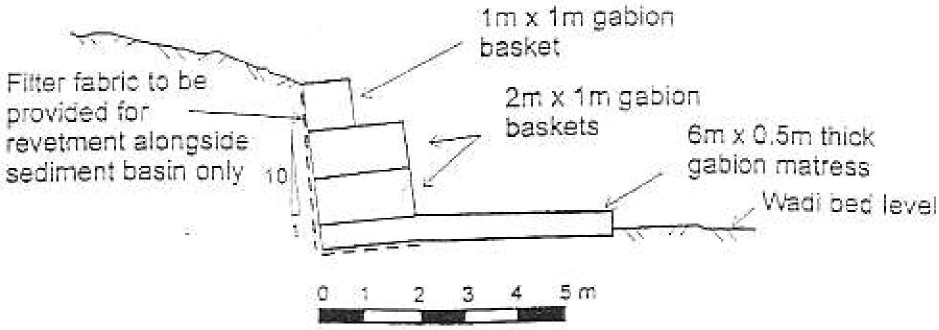
Typical Erosion Control Works



Hammerhead Groyne



Standard Groyne



Standard Wadi Bank Revetment

FIG. 8 Typical Erosion Control Works Using Gabions

TYPICAL IMPROVEMENTS PROPOSED FOR TRADITIONAL SPATE CANAL INTAKE

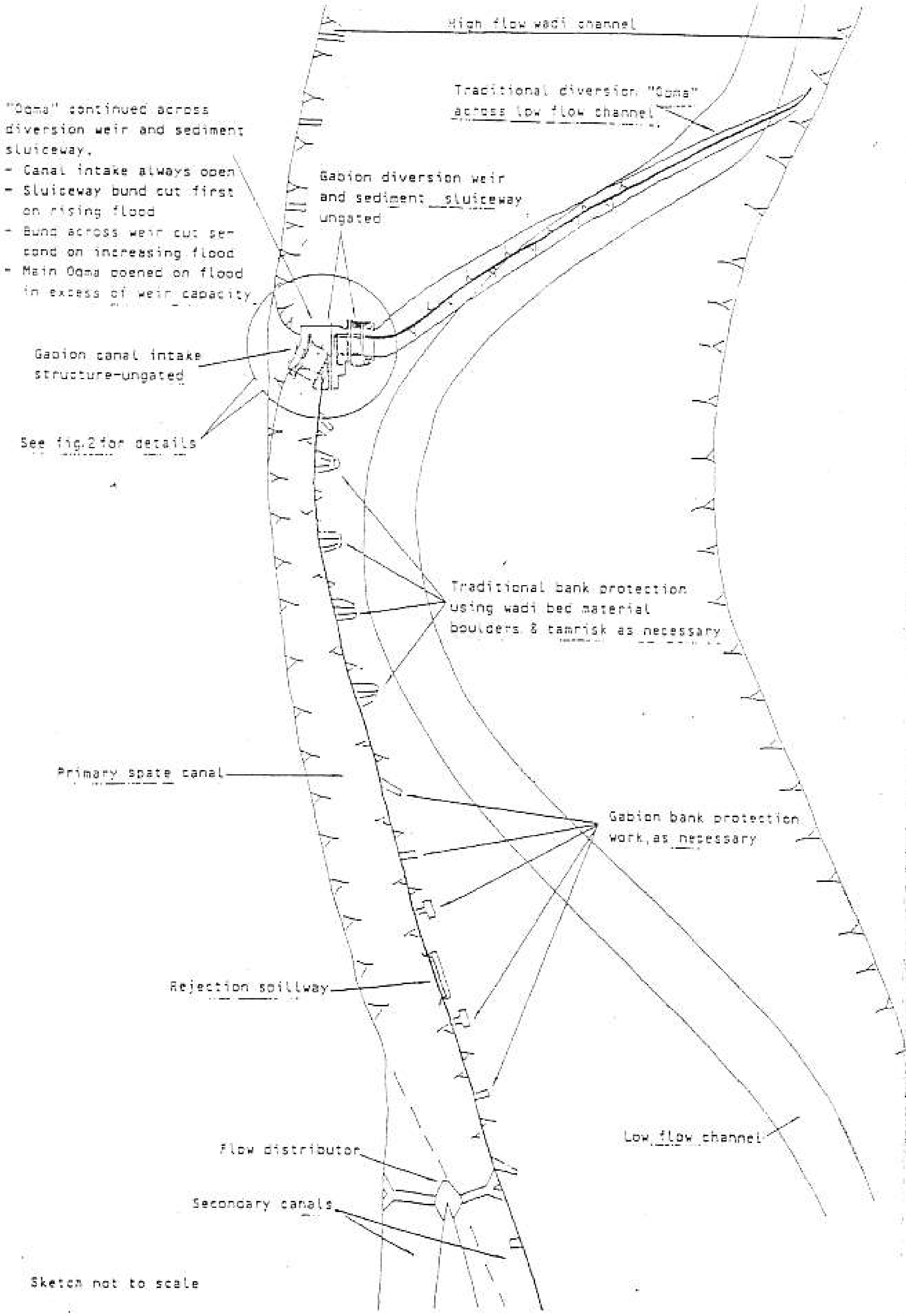


FIG. 9 Typical Improvements Proposed for Traditional Spate Canal Intake

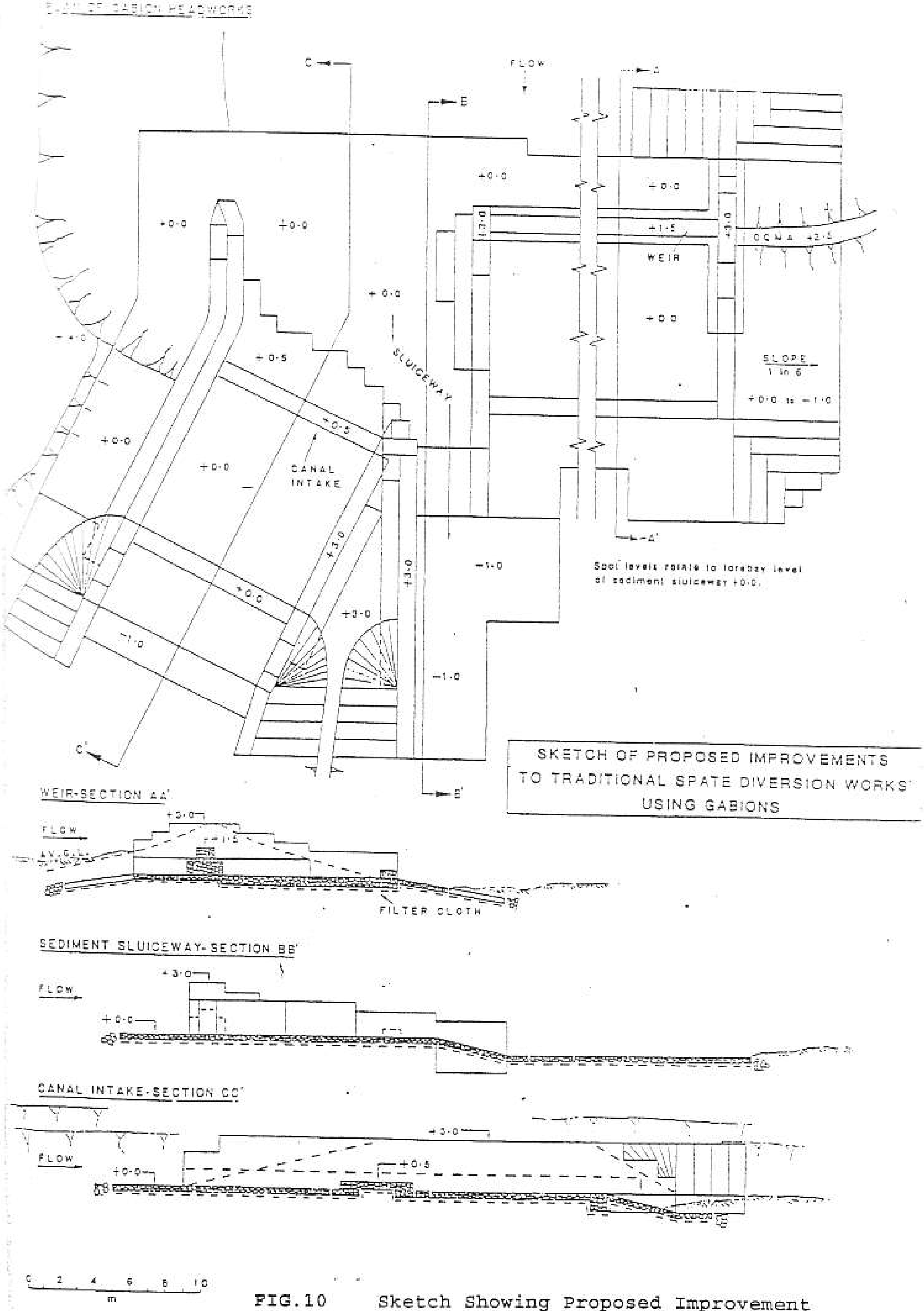
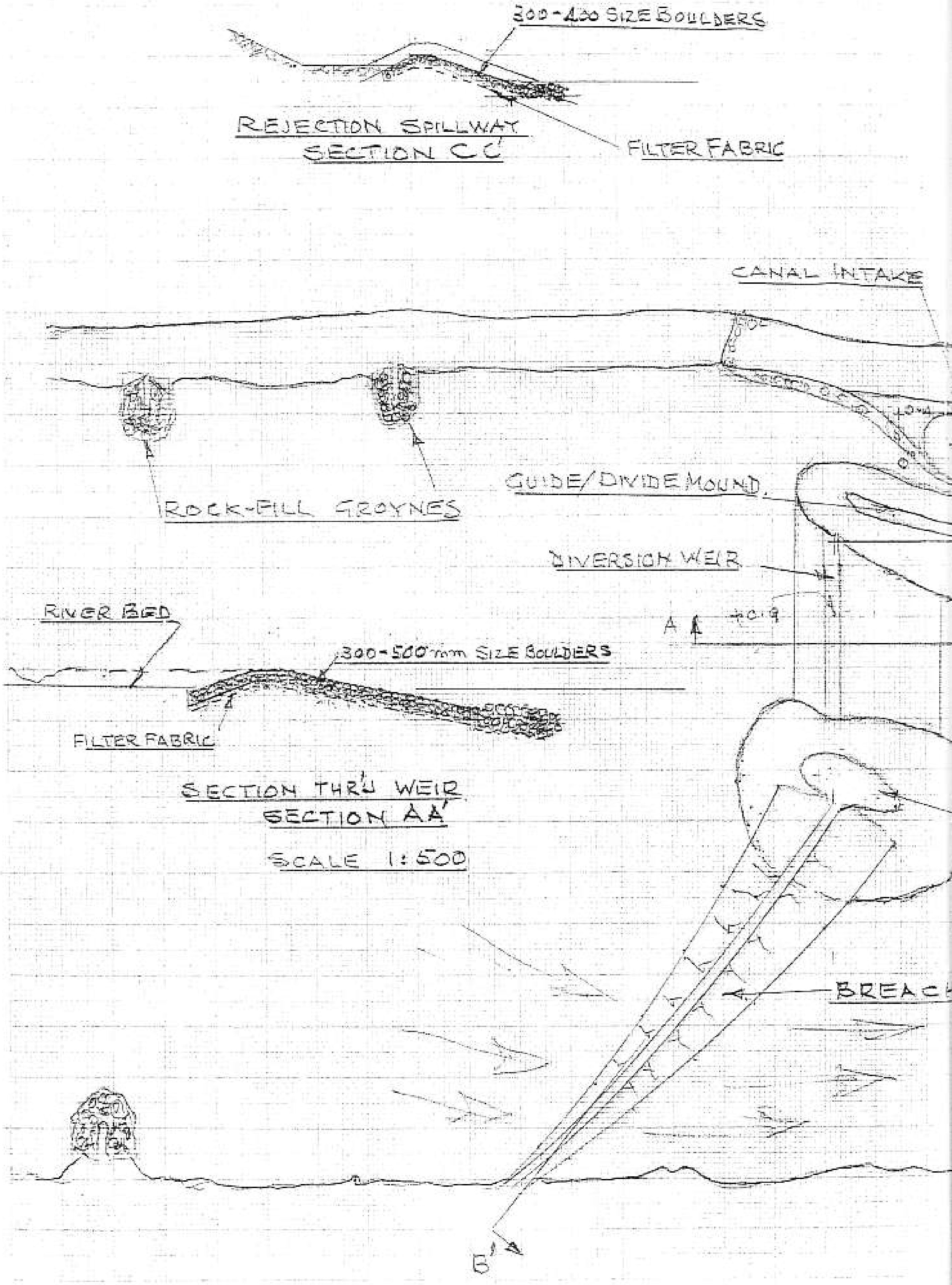


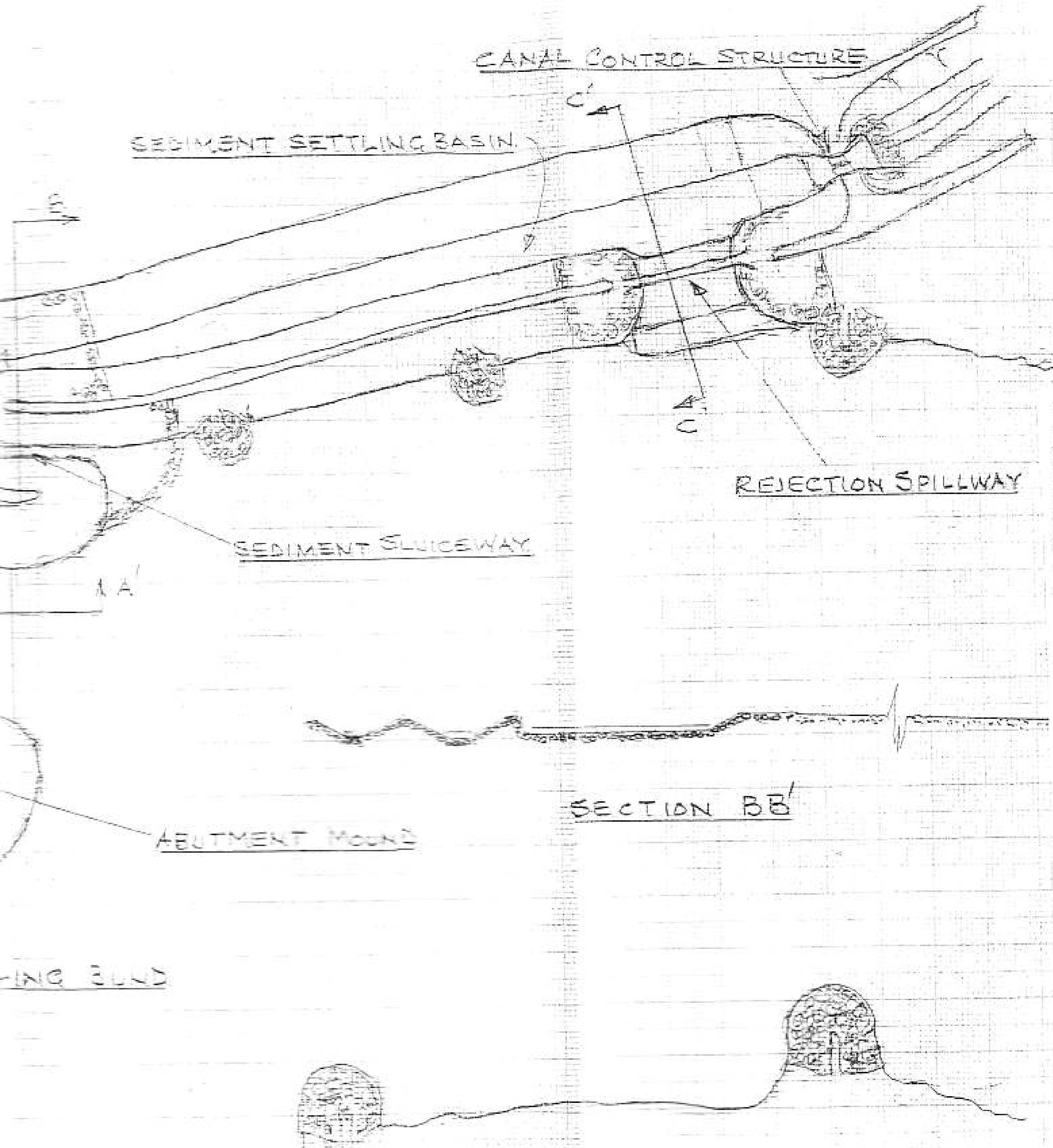
FIG.10 Sketch Showing Proposed Improvement to Traditional Spate Diversion Works



300-400 SIZE BOULDERS
FILTER FABRIC
REJECTION SPILLWAY
SECTION CC'
CANAL INTAKE
GUIDE/DIVIDE MOUND
ROCK-FILL GROYNES
DIVERSION WEIR
RIVER BED
300-500 mm SIZE BOULDERS
FILTER FABRIC
SECTION THR'U WEIR
SECTION AA'
SCALE 1:500
B'

GENERAL ARRANGEMENT FOR SPATE DIVERSION HEADWORKS USING BOULDER RIPRAP AND ROCK-FILL WEIRS.

SCALE 1:1000



SCALE 1:1,000

FIG. 11 General Arrangement for Spate Diversion Headworks Using Boulder Riprap and Rock-fill Weirs

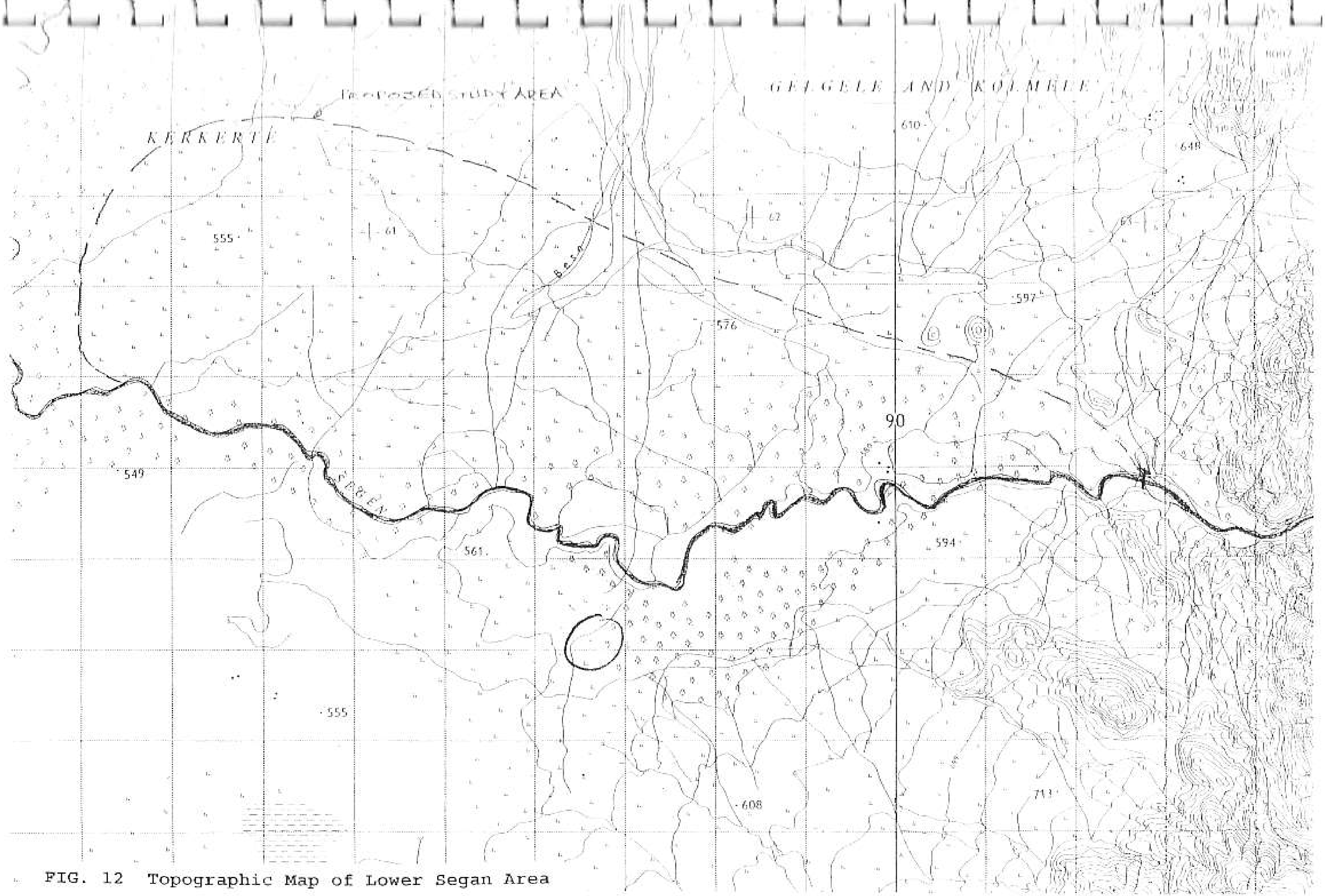


FIG. 12 Topographic Map of Lower Segan Area

APPENDIX A

Diary of Visit of Spate Irrigation Consultant for Review of Methods for Improvement of Spate Irrigation

Wed. 17/5/00 Dep. Heathrow 21.30 accompanied by Vincent Gainey, Programme Officer, Farm-Africa, London.

Thur. 18/5/00 Arr. Addis Ababa 08.45. Briefing meeting at Farm-Africa Office by Amare Mengiste, Project Coordinator for the Konso Capacity Building Project. Dep. Addis 11.45. Arr. Shashamene 17.30 and overnight there.

Fri. 19/5/00 Dep. Shashamene 06.00. Arr. Konso 13.30. Meetings and discussions with Vince Gainey, Michael Assefa, Resources Planner and Acting Project Coordinator and Mengistu Gebre, Water Engineer. Discussed visit programme and spate irrigation problems encountered by the farmers. Later, the group visited Kolme and Busso Mechelo areas to see traditional land use and practices in the higher lands - terraces, water spreading, small storage ponds.

Sat. 20/5/00 Visit programme agreed. Visited Jarso in Yandafero area to see traditional spate irrigation and land use practices in the lowland areas. Also saw very severe deepening of the bed of the river Yanda, about 3-4 metres, which extends almost from its confluence with the Segan for some 6 km up the Yanda. As a result several spate diversions intakes can no longer command their former areas. Discussions with community members and practitioners.

Afternoon, visited Beayde and Fuchucha on the Taho River with Mengistu Gebre to see traditional spate irrigation intakes, which were badly damaged by recent flood flows.

Discussions of outline concept for dealing with the river bed erosion problem on the Yanda.

Sun. 21/5/00 Visited Segan River area around the confluence of the Yando with the Segan.

Concept plan for check dam on the Yando prepared for discussion.

Mon. 22/5/00 Discussions in office, then visited possible sites for a check dam on the Yando for the purpose of arresting bed erosion, which is causing severe damage and loss of command at several traditional intakes.

Afternoon Konso Office. Discussions of field findings and proposals for training and workshop.

Tues. 23/5/00 Morning, training to professionals - discussions with Mengistu Gebre on several aspects of spate irrigation. Afternoon, Workshop session with discussion group of 24, comprising DA's, Foremen, EGS Supervisors & Farmers - discussion of problems, possible solutions and potential development sites.

Wed. 24/5/00 Vince Gainey departs Konso. Visit to upper Taho areas with Mengistu Gebre and two farmers to see two intake sites. First, at Hagayawa left bank intake site, serving the Merssa area of about 1,200 ha and farmed by about 600 families. Second, an upper intake site serving the right bank Kantada area of some 800 ha farmed by about 400 families. Both intakes completely destroyed by large floods and require reconstruction.

Afternoon, very useful further discussions with Mengistu Gebre on some technical aspects of spate irrigation engineering.

Thur. 25/5/00 Visit potential sites at Segan Gete and Burgitcha in the Tebela and Kuchale Kebele on the lower Segan River about 6 km from its confluence with the Weito River. A few thousand hectares of land which can be commanded from the upper site at Segan Gete. An intake and canal is at present being constructed at this site by the Sustainable Development Department, Awassa. Lower down the river at Burgitcha, farmers have built a traditional diversion and canal and are irrigating a small area for growing maize and sorghum. The remainder of the area is generally used for livestock and cattle rearing. Six hour round trip.

Fri. 26/5/00 Visit to Middle Segan area on road to Yabelo had to be cancelled as road impassable at river crossings. Visited proposed dam sites on the Yanda. The preferred site is between Gandayla on the right bank and Komboroti on the left bank. The other two sites were about 1 km and 1.6 km upstream of the preferred site. Large reservoir storage, with side spillway on a saddle.

Afternoon, final discussions with Mengistu Gebre on further technical aspects of spate irrigation engineering. Discussion of findings with Michael Assefa and Mingistu Gebre

Sat. 27/5/00 Winding up meeting. Depart Konso - Overnight Shashamene.

Sun. 28/5/00 Depart Shashamene. Return to Addis.

Mon. 29/5/00 Debriefing with John Fox and Amare Mengiste in FARM Office. Report preparation.

Tues. 30/5/00	Report writing.
Wed. 31/5/00	Report writing.
Thur. 1/6/00	Report writing. Presentation of draft report.
Fri. 2/6/00	Dep. Addis Ababa 12.15. Arr. Heathrow 19.55

Traditional spate irrigation and wadi development schemes

R. F. Camacho
Water Control Engineering Consultant to FAO
Former Associate Sir William Halcrow and Partners, UK

1. Introduction

Spate irrigation is important in arid regions, such as the South-West part of the Arabian peninsular, where catchments are rocky and steep and the annual rainfall—which may vary from 50 to 400 mm in the mountainous areas—is seasonal. This seasonal rainfall produces spate flows of short duration and these are the main source of irrigation water, since there is very little direct rainfall on the larger cultivable areas located in the deltas of the wadis, where layers of alluvial silt have been deposited.

Spate irrigation greatly affects the livelihood of the inhabitants of these regions as they are largely dependent on agriculture and animal husbandry. When the floods arrive, farmers organize immediate water distribution with occasional disputes over water rights; but in general there is a commonly accepted system of water rights in each wadi.

In these regions there is a need for better control of spate flows and improved irrigation facilities to stabilize, as much as possible, the agricultural area under cultivation in any one year, thus allowing food crop cultivation, mainly sorghum, and perhaps a cash crop (cotton, melons etc.) by a single watering.

2. Traditional irrigation methods

Traditional irrigation methods tend to be elementary but in overall terms are effective to a limited extent only. Local farmers traditionally build an earthen bank or "Ogma" of wadi bed material across the low flow channel of the wadi, with the object of diverting the entire low flow channel of the wadi, with the object of diverting the entire low stage of the spate flow to their fields. During a large spate, as there is no provision for a spillway, the "ogma" is either breached deliberately or it is over-topped and breaches as the flood rises. On such occasions a major section of the "Ogma" is washed away, often before its total command area has been watered, and water cannot therefore be diverted again to the fields until the "Ogma" has been rebuilt. How soon this can be effectively achieved will depend on subsequent wadi flows, availability of machines or animal power and occasionally the "Ogma" cannot be rebuilt before the ensuing spate.

Another local system of diversion is by short lengths of small earth banks projecting into the wadis in the form of spurs, which deflect a portion of the spate flow over the adjacent fields.

The traditional systems are relatively cheap to build, but usually very expensive to maintain. The initial capital cost and the annually recurrent maintenance costs are often the same. If small to medium size spates arrive, the "Ogma" can be effective, but medium to large spates can result in the expenditure of much effort and money with very little benefit, as all the "Ogmas" and spurs are swept away to the sea or into the desert. However, the basic principle is sound enough—that of diverting water at low stage and of allowing large spates to pass unchecked.

The primary canals taking off from the wadis have a large capacity (duty) in relation to the area irrigated because of the short duration of the spate flows. The primary canal sub-divides to smaller canals as it reaches the irrigable area, and farmers exercise traditional forms of control so that the higher lands receive water first. Lower-lying areas are then irrigated after the upper lands have received a sufficient supply, or when flood levels are too low.

There are virtually no permanent structures for the control and distribution of spate flows in the canal system. Diversions are made by blocking the canals and distributaries with temporary earth bunds and cutting them when demands are met. Irrigation supplies within the command area are distributed on a field-to-field basis, the basin bunds (about 1.0 m high) being breached once an adequate depth of water (about 0.5 m net) has been applied to the field.

The construction and replacement of the "Ogmas" and distribution of flows through the canals are communal activities organized by the farmers. Costs are shared on the basis of benefits received, which depends on the area of land, its evaluation, and its proximity to the irrigation supply.

3. Characteristics of spate hydrology and establishment of flood warning systems

Spate hydrology is characterized by a great variation in the size and frequency of floods which directly influence the availability of water for agriculture in any one season. Cropped areas and crop production vary considerably over the years because of the large variation in wadi run-off from year to year, season to season and day to day. In addition, the wadis are subject to devastating floods, which damage or destroy irrigation structures and agricultural lands.

Flood hydrographs characteristically indicate a sharp rise to the peak discharge but of short duration. A flow is reported to have increased one hundred times in 15 minutes on a Tihama Wadi in YAR. On Wadi Tuban, PDRY, the flow on 8 September 1959 increased from 14 m^3/s to 2114 m^3/s in one hour, an increase of 151 times. On Wadi Hajr, PDRY, on 4 April 1964, the flow rose from 114 m^3/s to 3 400 m^3/s in 2 hours, an increase of 30 times. The peak flow reduces by a factor of 20 to 50.

The slopes of the wadis are very steep—up to 1 percent—and this produces high velocities. The sediment load is generally very high, from 3 to 7 percent by weight with particle size varying from silt to the very large boulders. Floating debris and trash is often a very serious problem. Trees and branches create blockages in the canal head regulators and sediment excluders, and are difficult to clear before the water recedes; thus the flow is lost. Existing headworks are generally inaccessible to machine for clearance. At the new diversion weir on Wadi Rima, 600 m of trash built up at the canal head regulator in one night. Thus project design must take particular account of critical factors such as:

- ☐ flow frequency and sediment analysis;
- ☐ the exceptionally high sediment loads and the effects of the coarse fractions (gravel and boulders) carried by wadi flows; and
- ☐ the quantity and size of the floating debris and trash.

Although data have become available from the different wadi studies in YAR, PDRY and Saudi Arabia, little continuity and coordination in hydrometric data collection and processing is evident.

A better insight into spate hydrology and the possible introduction of a flood warning system may significantly improve spate water management and distribution, but this also requires a very good system of communication. In the case of excessive floods and warning system may reduce the loss of human life, livestock and settlements.

4. Assessment of water resources, sediment transport and optimal water balance

The assessment of surface and ground water resources is one of the most crucial aspects for the development of spate irrigation and, frequently when feasibility studies are commissioned, there is little or no rainfall or flow data on which to base an assessment. A preliminary assessment often has to be made while data are being collected, and this has to be carried out by correlation with other wadis which have records of flows and groundwater availability. It is important to decide whether the data collected in the first year or two fall into the category of a dry, wet or average year by correlation with data from other similar areas. In the same way, sediment transport must be measured and compared with data from other wadis as it might not always be possible to measure during a wide range of flows. Sediment transport capacity of these wadis is generally under-estimated.

An overall water balance needs to be prepared when alternative development concepts are being studied. A very approximate water balance for the Abyan Delta in PDRY is shown in Table 1. It indicates that a more efficient irrigation scheme, which reduces losses and thus groundwater recharge, will result in a smaller groundwater development. Benefits on a delta-wide basis must be thoroughly assessed for each option.

Table 1 Overall water balance in Abyan Delta (Mm^3)

	Surface Water	Groundwater recharge	discharge
Inflow (mean annual)			
Wadi Bana	+162		
Wadi Hassan	+32		
Wadi Maharia	+2		
Total	+196		
Surface water use/losses			
Net consumption (500 mm on 13 200 ha)	-66	-	
Wadi Seepage Losses (30% of inflow)	-60	+55	
Canal and Field Application Losses (25% of inflow)	-50	+40	
Escape to Sea, D/S of Ogma Sada (10% of inflow)	-20	+10	
Total	-196		
Groundwater abstraction/losses			
From Wells for Irrigation (Net of Return Flows)			-60
From Wells for Domestic Use			-5
Evaporation and Evapotranspiration			-30
Losses from Shallow Water Table			
Subsurface Outflow to Sea and Adjacent Areas			-10
Totals		+105	-105

The rapid expansion of deep tubewells has considerably lowered groundwater levels in several wadis and is a very serious threat to areas of permanent agriculture. Older and shallower wells have dried up as water levels have dropped below present pump intakes. Deterioration of water quality for irrigation has also taken place in several areas adversely affecting agricultural production. Groundwater extraction therefore requires close monitoring and continuous data collection.

The critical points to keep in mind when deciding which overall concept to adopt for planning purposes are firstly, that a great deal of flexibility should be built into the overall scheme design because of the general paucity of data and secondly, that the planned development must be based on a realistic water balance which will maximize overall wadi production per unit of water.

5. Soils and land capability

The most important factor in soil classification for spate irrigation is the water retention capacity. The classification for spate schemes is based on this and a modified USBR standard.

The soil classification in wadi developments usually conforms with the order of water rights as silt-laden flood water seems to improve soil structure. The soils with the best water retention capacity are usually to be found in the upper part of the wadi alluvial fan. Some of the best soils in these wadis can only hold 35 cm of available water and thus it may be inefficient and wasteful to provide a net field application depth of more than 40 cm on average for the whole spate scheme area until improvements to the distribution system are carried out.

Traditional spate irrigation is based on a field-to-field system of distribution and though a field may be filled to a depth of 80 cm, the water is not allowed to stand long enough to soak-in before the field bund is cut and the water drawn down and transferred to fields downstream. Thus it is doubtful whether a net field requirement of more than 40 to 45 cm can actually be beneficially utilized, due to the water retention capacity of the soils and with the traditional field-to-field system of spate irrigation.

6. Land tenure and water rights

These are absolutely essential aspects of any feasibility study for improvement of spate irrigation. A cadastral survey to define land ownership and a study of water rights are required, not only for proper planning and design of the distribution system and operating procedures, but also for the recovery of O and M and development costs by assessment, levying and collection of irrigation charges.

7. Alternative cropping patterns for wadi development

Alternative cropping patterns will be postulated at the feasibility stage and will depend, to a very large extent, on the forecast of water which will become available as a result of the development plan adopted. Such a planned cropping pattern can only be enforced on State farms, but independent farmers will only change their cropping pattern in response to market demand and an improvement in the probability of receiving an irrigation supply. Only when his subsistence is no longer at risk will the farmer change to higher value crops.

The cropping pattern will comprise a variety of crops such as bananas, fruit trees, vegetables etc., that can be grown on the small area (about 10 percent of the total area) in the upper reach of the wadi which can be irrigated almost perennially with the spring flow from the wadi bed. A further area (about 30 percent) can grow two crops on two waterings per crop and the remaining 60 percent will only be able to produce one crop per year on a single irrigation. This area will have a probability of irrigation from 30 percent to 70 percent. The area irrigated from groundwater will have a cropping intensity possibly slightly over 200 percent and will be able to produce a variety of high value crops.

8. Economic development concepts

8.1 Storage reservoirs

The possible construction of a storage reservoir on the wadi would provide a costly, though technically attractive, means of regulating the highly variable spate flows on short duration with very sharp peaks, into an almost perennial irrigation supply. The heavy silt load carried by the floods would, however, cause a rapid loss of live reservoir storage at a rate of the order of 3 percent per annum.

In the wadis of the South-West Arabian peninsular, large storage dams would command relatively small areas and involve high capital investment per hectare, in addition to the cost of the distribution system. Also the operation and maintenance of the perennial water supply system would require a complete change in traditional irrigation and agricultural practices. As the reservoir silted up, large flood flows would begin to overtop the spillway more frequently and the wadi irrigation system would gradually revert to traditional methods of diversion. The construction of a storage dam sometimes deprives farmers in the lower wadi area of their former supply of spate water, however erratic it may have been.

8.2 Spate breakers

A spate breaker is a small dam with a reservoir capacity of say, 8 m to 15 m which would be sufficient to absorb the larger spates with high peak flows that would otherwise wash away almost all of the "Ogmas" of the traditional system. These large spate flows would be immediately routed through the spate breaker and the storage would be drawn down in 2 to 4 days, perferably before the ensuing spate arrived.

Unfortunately spate breakers, besides being very costly, have a very much shorter life than that of a storage reservoir, due to rapid sedimentation. This is caused by the smaller storage capacity, the low ratio of storage volume to average annual flow volume, and high sediment loads. The proposed spate breaker at Khola on Wadi Zabid in YAR was estimated to have an effective life of 5 to 10 years and was therefore not implemented.

8.3 Schemes for spate diversion with wadi training and bank protection works

The water resources of the Yemen Tihama wadis fall into three fairly clearly defined categories:

i. the base flow and minor spates;
ii. spate flows;
iii. groundwater.

The base flow and minor spates are generally sufficient to provide two crops per year on about 30 percent of the traditionally irrigated areas. With improved irrigation this area could be increased to about 40 percent.

Spate flows provide irrigation for just over one crop on about 60 percent of the traditionally irrigated area but with a variable degree of probability.

Groundwater is abstracted both in traditionally irrigated areas and rainfed areas of the Wadi. The cropping intensity is about two crops per year.

In order to optimize the use of surface and groundwater within a situation of water balance, alternative concepts, which seek to maximize the overall value of wadi production per unit of water and in economic terms, need to be thoroughly examined and discussed. The alternative with the highest overall project efficiency or the lowest cost per hectare irrigated will not necessarily be the best scheme to adopt. Other factors, both economic and social, are very important and also need to be carefully considered.

Two different concepts for utilization and diversion of spate flows have been implemented in Wadi Zabid and in Wadi Rima.

In Wadi Zabid the concept of improving spate diversion throughout the length of the wadi was adopted. The farmers in the middle and lower reaches of the wadi who formerly had to cope with the larger unmanageable spates have been provided with a more reliable means of diversion to their fields. These farmers have been given at least equal, if not more, attention than the farmers in the upstream areas who in the past have been able to manage the base flow and small spates without great difficulty, using their traditional methods. The incremental benefits to farmers will therefore be relatively small.

In Wadi Rima the main investment in irrigation works was made in an effort to convert a traditional spate irrigation area to a conventional perennial irrigation scheme. The total diversion capacity is 15 m^3/s from a single weir with one offtake. The commanded area is about 8 000 ha and the resulting water duty is what one would expect for a perennial scheme, but not for spate irrigation. Thus, the scheme largely benefits the farmers in the upper reaches of the wadi who already had a high probability of irrigation and the means of managing and diverting the relatively small flows that they required.

The farmers lower down the wadi have apparently not received as much assistance as they might have expected to get from the formal project works. This is evidenced by the farmers' insistence that their traditional diversion works (free wadi offtakes) and canals must be retained and culverts or aqueducts provided, where the traditional canal crosses the new canal from the headworks. The selection of the scheme seems to have been based on the overall efficiency of the irrigation system.

In Wadi Mawr one weir with an offtake capacity of 40 m^3/s is being provided to command an area of some 18 000 ha. Here again the irrigation duty of about 21 sec/ha is more appropriate for a perennial scheme than for spate irrigation. The investments in irrigation works are for improvements in abstraction of base flow and small spates; to a large extent, significant improvements to traditional works, which attempt to divert larger flows downstream, have been omitted. Further diversion works will eventually be required downstream.

A review of the overall wadi development concepts should therefore be one of the main issues to be decided in Phase 1 of any feasibility study. This is linked with the assessment of water resources and the optimal water balance. Benefits on a delta-wide basis must be thoroughly

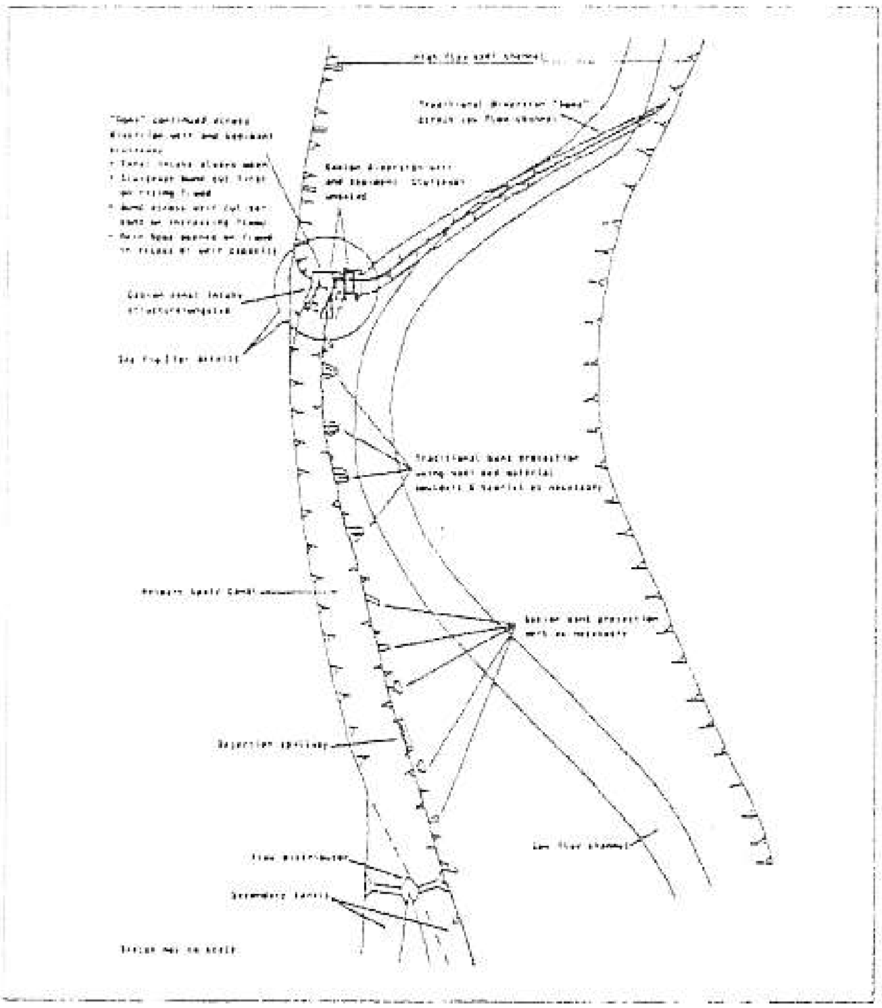


Figure 1 Typical improvements proposed for traditional spate canal intake

assessed for each option to ensure that the total water resource is being used to the best advantage of the farmers within the water right area and within an overall water balance.

It is often difficult to show that spate irrigation schemes are viable in normal economic terms. The proposed alternatives usually all show estimated returns of less than 15 percent, but some form of improvement works must be carried out if only on the basis of social need. In any event the incremental benefits from improvements to spate irrigation schemes are generally limited by the constraints of a single watering, the water retention capacity of the soil and the root zone. In the lower reaches of the wadi the lower probabilities of irrigation reduce further the already limited incremental benefit and call for unconventional solutions if we are to help farmers to improve water control.

In arid zones where water is an absolutely vital commodity, spate spreading schemes have a special significance in stabilizing agricultural production and generally improving the well-being of the people in those areas. The yardstick for the selection of a spate scheme should perhaps be the option which gives the most effective utilization of the total surface water resource yielding the highest value of production per unit of water at the least cost. If this is considered a suitable yardstick then, with the degree of reliability of irrigation of the traditional command areas reducing from the top to the lower areas of the wadi, so also will the scale of investment have to be reduced from a concrete weir and headworks with a degree of security at the head of the wadi to the traditional, simple but large, diversion bund at the lower end of the wadi. A range of diversion structures representing reducing levels of investment and increasing levels of risk needs to be designed to match the lower degree of reliability of irrigation further down the wadi giving lower returns.

The concept of 80 percent probability of irrigation usually adopted for perennial schemes is unacceptable for spate schemes. Scarce water resources are too valuable and every means must be devised to use them effectively. Thus, the third diversion structure might comprise a relatively short gabion weir with a breaching bund capable of withstanding floods with a return period of say five years. The weir would be attached to a canal head control structure and sediment excluder providing irrigation on one side of the wadi only. Any downstream weirs might be designed for floods with even lower return periods.

The traditional diversion bund, "Ogma", has no spill-

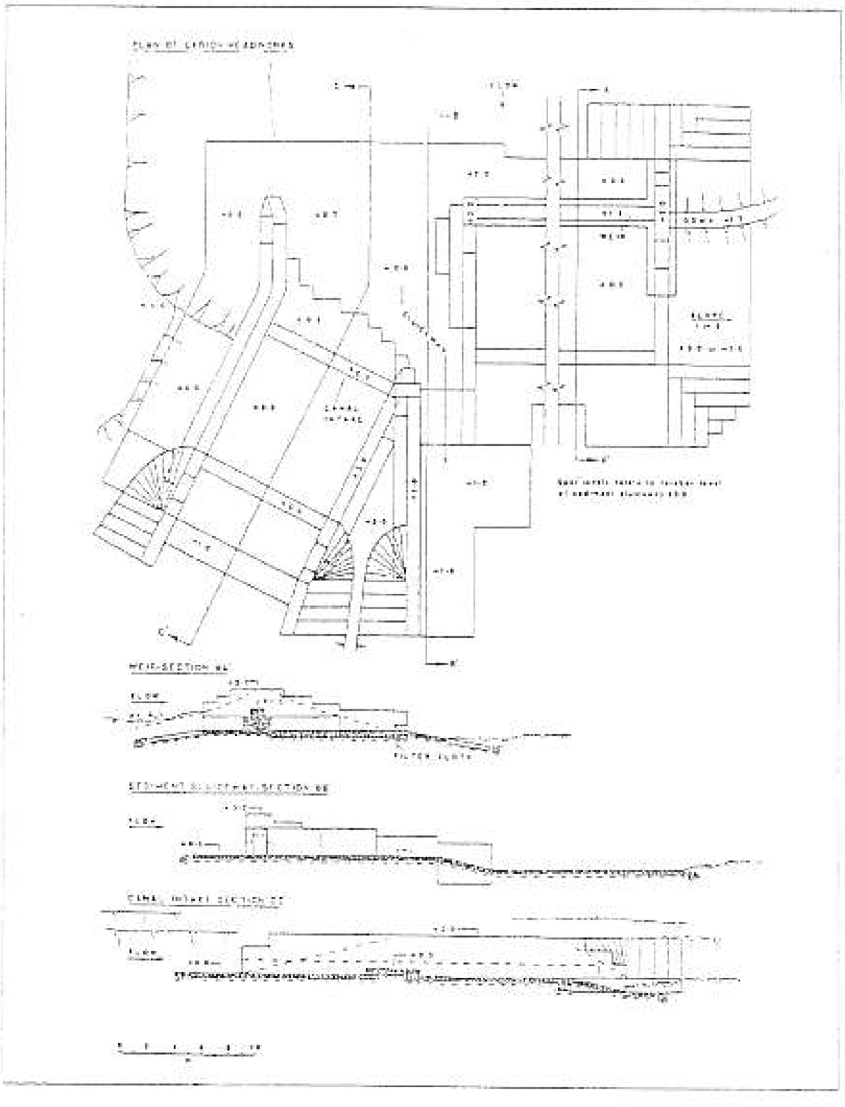


Figure 2 Sketch of proposed improvements to traditional spate diversion works using gabions

way and this is quickly breached and diversion ceases if the flood rises sharply, as it frequently does. The volume of spate flow which can be diverted can be substantially increased by installing some spillway capacity in the traditional "Ogma".

Estimates of water availability and areas irrigated, based on an analysis of simulated flood hydrographs for Wadi [illegible] indicate that volumes diverted and areas irrigated can be doubled or trebled by installing a permanent weir. It should therefore be possible to obtain a modest increase of about 30 percent in the areas irrigated, where some relatively small spillway capacity is installed in "Ogma".

The designs of the low-cost headworks must be capable of construction by groups of farmers with some unskilled labour, and with a minimum of technical supervision.

Typical layouts for such diversion works are shown in figures 1 and 2, which show the kind of improvements proposed for incorporation in a traditional spate diversion "ogma".

A typical gabion headworks (such as shown in the section in figure 2) would be incorporated in the "ogma" at the spate canal intake. It would provide a weir of about 100 m^3/s capacity, an ungated canal intake of 10-20 m^3/s capacity, and a sediment sluiceway of about half the intake capacity. Some wadi training groynes may be required upstream to give some protection to the headworks against outflanking, and groynes may be required downstream to protect the canal bank from scour by wadi flows.

The improved traditional spate diversion works would be operated in the manner to which the local farmers are well accustomed. A bund would be built across the front of the diversion weir to a height just below the lowest level of the "ogma", *i.e.* to about 0.5 m above the crest level of the weir. This bund would be extended across the sluiceway to a height equal to the crest level of the weir. The canal intake would always remain open. Thus, an initial small spate would be diverted to the canal intake and on a rising flood, as the canal reaches its design capacity, the bund across the sluiceway would be cut to divert some of the increasing bed load and control the rising flood level. As the flood level increases, the bund across the weir would be cut, and when the capacity of the canal intake, sluiceway and weir is exceeded the "Ogma" would be over-topped, allowing the larger floods to pass down the wadi. The "Ogma" and bunds would be replaced by bulldozers when the flood recedes.

By ensuring that the "Ogma" is over-topped at a breaching section set a good distance from the gabion headworks, and by providing aprons against scour, it is anticipated that the gabion abutment wall adjoining the "Ogma" would remain intact in all but a major flood.

9. Planning and phasing of development

9.1 Spate management planning

A good spate management plan will make the best use of several components in optimising the value of crop production per unit of water in the overall wadi area. The components include:

- surface and groundwater resources;
- soil and land capacity including a schematic layout showing all cultivable areas;
- water rights and establishment of priorities for water use;
- water retention capacity and root zone parameters of the soils;
- cropping pattern and crop requirements;
- water application depths;
- the irrigation distribution system and the operating code;
- irrigation efficiencies; and
- social implications of different strategies.

If the data on water resources are good enough, spate management planning can now be done with the help of a computer model which would study various options for water management for alternative series of diversion sites and select the best option. The model would:

- optimize and select offtake and canal capacities;
- determine the probability of irrigation of the areas commanded by each diversion work, thus providing the type and cost of improvement works which can be justified and the degree of reliability of the works which has to be accepted if capital and operational costs are to have a reasonably economic relationship to the benefits likely to be derived;
- assist in planning groundwater recharge areas and optimizing operating procedures to minimise losses to the coast or desert.

The programme would simulate the passage of a number of historical or synthesized floods over a typical series of years. For a selection of any number of weirs and any combination of canal sizings, the programme passes each flood in each year down the wadi. At each weir, water is diverted into the supply canal for the duration of each flood. The remaining flood water is passed down the wadi and bed losses and flood attenuation for the wadi reach are deducted. The flood hydrograph is then reformed at the next weir, taking account of the intervening bed losses and flood attenuation, and the process repeated.

In each year, volumes of flood water diverted to each area will be accumulated. When the total water requirement for a seasonal irrigation on an area has been satisfied, no further water is diverted in that season until all of the areas with water rights have been irrigated.

The programme's output will provide the accumulated amount diverted to each area commanded by main canals for each of the seasons considered. From this output, the total area irrigated by each alternative under each combination of canal sizing can be determined. By the area irrigated and their probability of irrigation, the most appropriate scheme for spate diversion and control can be recommended.

9.2 Phasing of development

It is probably better to plan and implement spate improvement works in two separate phases. The first phase would provide diversion and control of spate flows and deliver irrigation supplies to discrete groups of farmers in

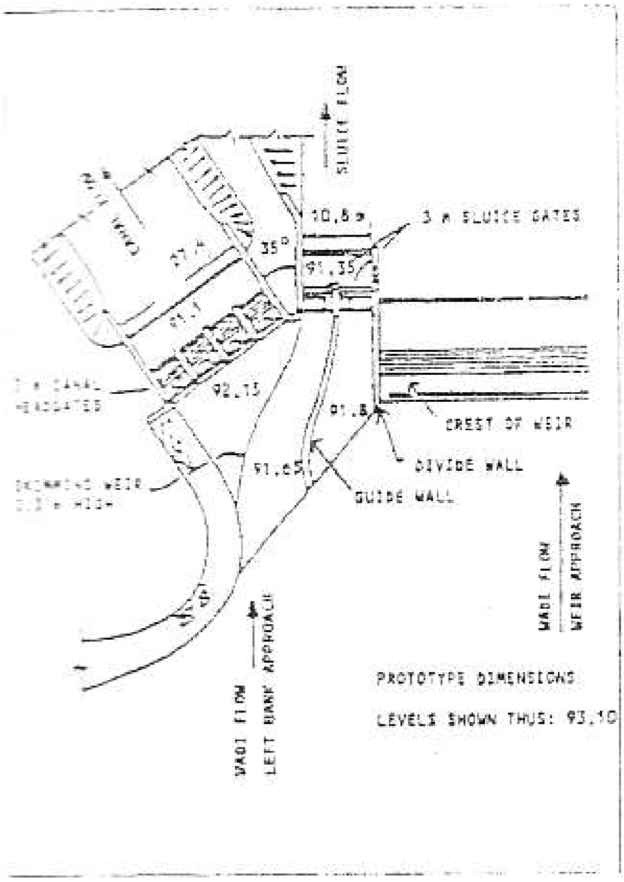
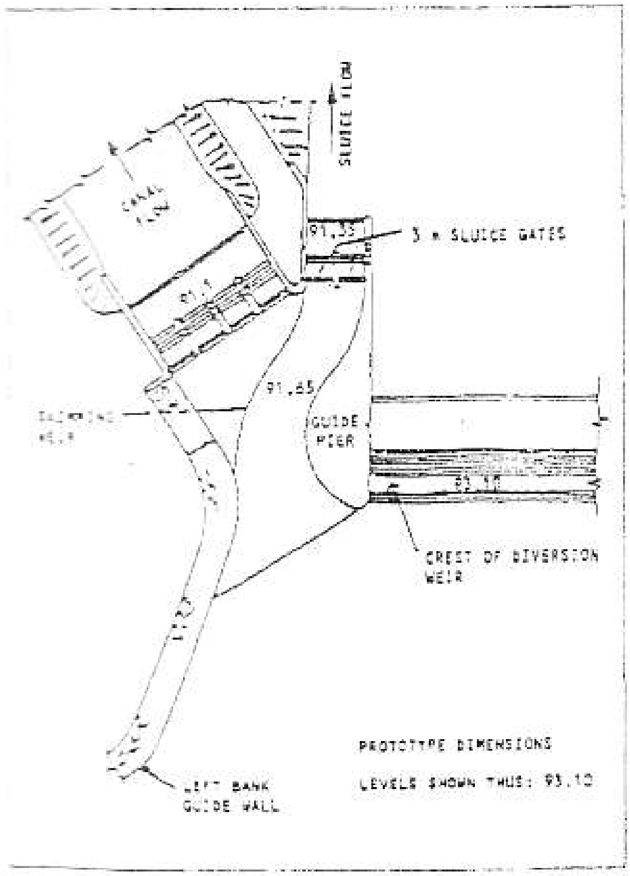


Figure 3 Headworks—preliminary layout

Figure 5 Headworks as modified



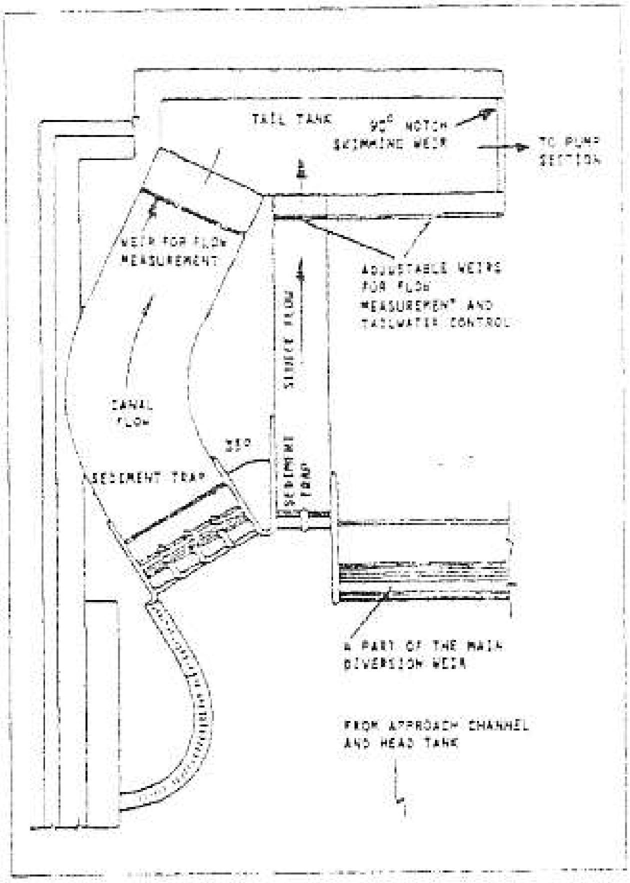
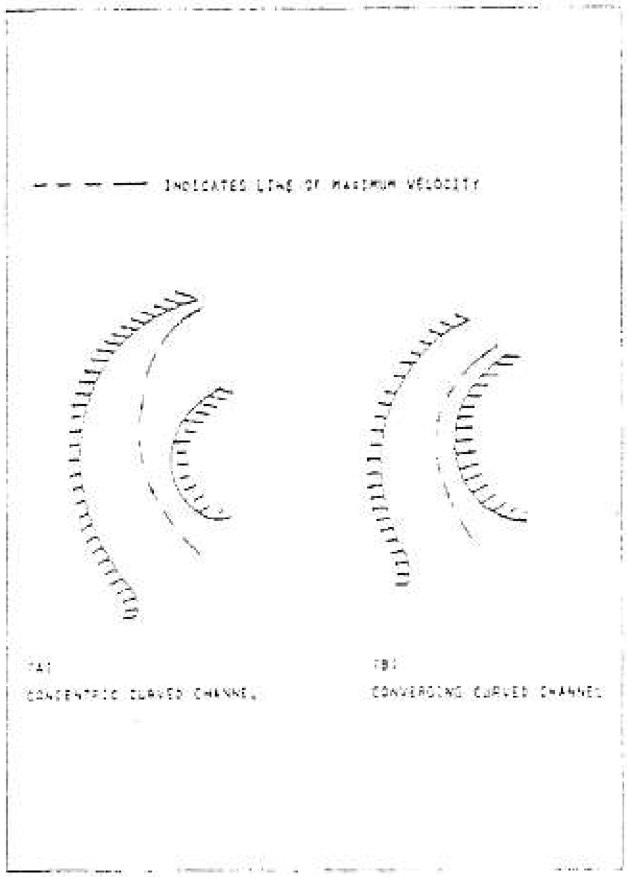


Figure 4 General layout of model area

Figure 6 Lines of maximum velocity in concentric and converging curved channels



quantities that they have been accustomed to diverting. When these farmers have gained complete confidence in the improved diversion system, the next phase should be planned and executed. This second phase would improve distribution and control of the field irrigation system in the areas which are able to grow more than one crop. The field-to-field system of irrigation could be changed to a system of small distributaries delivering supplies to each field, or at least to smaller groupings of fields than at present. After the first phase development has been in operation for a few years, any requirements for minor alterations and improvements will have become evident and these can be incorporated into a second phase for improvement of the secondary irrigation system.

The second phase could be very expensive in both capital and O and M costs if minor distributaries and spate inlet structures are required for each and every field of about 2 to 5 hectares. Land levelling cost would be additional to the distribution cost and unit levelling cost would probably increase with larger field sizes. In order to keep unit costs for secondary works within reasonable limits, an economically acceptable size of irrigation unit has to be decided and this raises the question - what is a manageable quantity of spate water for a small group of farmers to handle, using traditional methods, with some inter-fields irrigation improvements? The largest quantity of spate flow that a small group of farmers could probably manage, with delivery from a single inlet structure, would be about 2 m³/sec and such a group could irrigate about 40 hectares in a day or two, depending on the variation of the flow.

Thus, the command areas controlled by a single distribution structure on the diversion supply canals to be provided in the first phase, could vary from 500 to 1 000 hectares, while in the second phase, the distribution canals and the structures would supply units of about 40 hectares. Groups of farmers would still have a large part to play in the management and distribution of water to their fields in each phase.

10. Design of spate schemes

10.1 Diversion weirs and canal headworks

Diversion weirs are built across the wadis to raise spate flows to a level that will enable the design full supply discharge to be diverted through the irrigation canal head regulator before the flood overtops the crest of the weir.

The weir crest level need only be raised slightly above the average wadi bed level for command purposes, but for efficient sluicing of bed load at low flows, it needs to be at least 1.5 m above average wadi bed level. This height is not ideal for sediment discharges at high wadi flows, but it may not be possible to set it higher if excessive cost of energy dissipation on the diversion is to be avoided.

The weirs are non-regulating, ungated structures designed for permeable foundations. In recent years they have been constructed with crests that slope towards the canal head regulator and sluiceway, to assist in maintaining the approach channel in front of the headworks.

The design of the weirs entails the determination of the stage/discharge relationship, flood discharges for various return periods and width of waterway. The crest level, headwater and tailwater rating curves, intensity of discharge, afflux, high flood level and free-board can then all be calculated. The downstream stilling basin level and the dimension of the energy dissipator and appurtenances, including the depth of sheet piling and length of flexible aprons, will then be determined and designed against uplift pressures and exit gradient.

The canal head regulator is designed to run at full supply discharge, with very little afflux (around 300 mm), just before the flood water overtops the weir crest. The full supply discharge will have been decided at the planning stage and the dimensions of the head reach of the canal determined separately. The width between abutments of the canal head regulator is made approximately equal to the bed width of the canal, to avoid fluming as far as possible, and to provide a skimming weir effect with the maximum intake at low heads. The gate openings and sill level can then be calculated for the full supply discharge and full supply level with minimum loss in head.

The canal head regulator will have to conform to similar design criteria as the weir and the safe downstream floor level should be checked for full supply discharge under the following two situations:

i. with upstream water level at the weir crest level, and
ii. with upstream water level at high flood level.

Condition (ii) will be critical and will determine the safe downstream floor level of the regulator. The velocity at the end sill of the canal head regulator must be checked. The value of the Froude number for this position should not exceed 0.35.

The canal head regulator is set at an angle to the sluiceway for it to function effectively. This angle is dependent on the particular features of the site, but may be varied from almost 0 to 90 degrees and may best be determined by model studies if the magnitude of the work warrants the expenditure.

The control of gravel, shingle and boulders carried into the canals by water diverted from heavy sediment-laden spate flows is a very important feature in the design and operation of wadi irrigation headworks. The problem is one of conducting as much as possible of the stream bed load material through the diversion structure, thereby avoiding its expensive removal from the canal.

The diversion headworks therefore incorporate a scouring sluiceway between the weir and canal head regulator, which is intended to provide good scouring action across the front of the regulator and to intercept as much as possible of the wadi bed load carried by the spate water being diverted for irrigation. The sluiceway also assists the sloping weir in maintaining a channel to the canal head regulator.

The performance of this type of sediment excluder at spate diversion headworks on wadis could be considerably improved by designing the approach to the canal offtake and sediment sluice with a curved converging channel and positioning the weir at the upstream end of the guide pier. In addition to these changes, the bank protection wall should be extended upstream to prevent any pronounced and unfavourable approach from that side. These findings were the result of model studies at the Hydraulics Labora-

Table 2: Mansbury-Rayyan-Bagr Canal, Wadi Zabid, YAR

Range	Discharge (m³/s) Average	% of max	Sediment concentration (ppm)	Probability %	Probability of given discharge being exceeded
0 - 1	0.5	1	1,620	21.5	78.5
1 - 2	1.5	4	2,220	20.3	58.5
2 - 3	2.5	6	2,760	15.3	42.9
3 - 4	3.5	8	3,360	10.1	32.8
4 - 5	4.5	11	3,900	7.5	25.3
5 - 7	5.0	12	4,800	6.4	18.9
7 - 10	8.5	21	6,300	6.0	12.9
10 - 15	12.5	31	8,100	5.7	7.2
15 - 25	20.0	50	11,400	4.8	2.4
40	40	100	18,600	2.3	0.1

ity, University of Southampton in 1978 and Figures 3 to 6 illustrate the basis and outcome of the study.

The most effective method of sediment exclusion is considered to be continuous sluicing adjacent to the canal head regulator in a direction about 60 degrees to the direction of diversion. However, this is not possible in the southwest coastal regions of the Arabian peninsular. Continuous sluicing is only possible when flows in the wadi exceed the full supply discharge of the canal, as wastage of any water which could be diverted would not be tolerated. Intermittent sluicing is sometimes practised when wadi flows are less than the diversion requirement.

The sediment ejector gives no protection against the "wash load" which eventually is deposited in the fields. The coarser fraction—heavy sands and gravel carried in the lower layer of flow—will also enter the headreach of the canal and should be removed by incorporating lined sediment basins and silt ejectors a short distance downstream of the headworks.

For efficient sluicing, the required sluice flow should be one-third to one-half the canal flow, with the upstream level at weir crest level. It may not be possible to set the weir crest at a level which is ideal for sediment exclusion at the higher wadi flows if excessive cost of the diversion works is to be avoided and operational procedures may have to be instituted instead, to reduce the sediment intake during the short peak flows.

In order to preserve the curvature effects of the sluice channel, design velocities should not be too low, hence depths of flow should not be too large. The sill levels of the sluiceways are generally set about 0.6 m to 1.0 m below the sill level of the canal head regulator.

The Tihama Development authority of the YAR has gained valuable experience over the past few years in the operation and maintenance of spate irrigations schemes in Wadi Zabid and Wadi Rima. Due to the sloping weir and the geometry of the headworks at Wadi Rima, the flow intensity towards the head regulator is considerably higher than the average intensity for the wadi. This concentration of flow drives the sediment and trash towards the headworks and the forebay is likely to become blocked with trash while the sediment concentration in the canal will be higher than in the wadi. As the sediment transport capacity in the head reach of the canal will be much less than the wadi because of its flatter slope, heavy deposition of coarse sediment will occur in the head reach and considerable excavation will be frequently required unless corrective measures are taken.

The TDA suggestions for modification of future designs, in order to alleviate some of their O and M problems are:

i. head walls above the scour sluice gates should be omitted to improve the throughput of sediment and trash;
ii. overflow type, falling leaf gates, hinged along the floor, should be installed in the scour sluice, with top level of the gates at weir crest level. This will enable sediment and trash to flow over the gates during higher flows;
iii. trash racks with suitable clearance devices should be installed in front of the main canal head regulator;
iv. a lined settling basin should be provided in the head reach of the main canal;
v. the bed slope in the head reach of the canal should be increased to achieve velocities of about 2.5 m/sec;
vi. a sediment ejector should be provided at the end of the settling basin;
vii. a rejection spillway to deal with excessive discharges into the main canal should be provided;
viii. the scouring sluice and weir should be lined with stone to reduce the abrasion by large cobbles and boulders traveling at high velocities.

These are all worthy of very careful study.

10.2 Canals and distribution structures

The design of spate irrigation canals and distributaries present some unusual problems because of the need to sieze every opportunity of diverting supplies whenever the wadi is in spate. As mentioned earlier there are large fluctuations in the daily wadi flows which necessitate designing canals for a varying discharge rather than the more usual steady flow states. The canal design must provide satisfactory transport of a highly variable sediment load without causing significant scour. The choice of shape and bed slope are the key factors in design.

In the early 1960's, Lacey's equations were used to determine shape, the ratio of surface width to water depth. This ratio was determined by the discharge and the mean particle diameter of the anticipated bed material of the canal which would eventually be that of the dominant sediment load it had to carry. The ratios varied from 10 to 20 for design discharges of about 10 to 40 m /sec. However, much steeper bed slopes than those indicated by Lacey's slope equations were chosen, to provide better sediment transport capacity at the lower discharges. Little or no data were available on sediment concentrations and thus the selection of silt factor and slope was somewhat arbitrarily based on the engineer's judgement.

The equations used for canal design were based on field data mainly from India and Pakistan, where sediment concentrations were thought to be similar. This has proved to be incorrect as sediment concentrations in the wadis of the south-western Arabian peninsular are much larger than those catered for in canal design on the sub-continent at that time.

The TDA now prefers to use the Simons and Albertson method (1960) as their equations are based on a more comprehensive range of field data and cover a wide variation of sediment loads, some of which, in TDA's opinion, are similar to the heavy sediment concentrations carried by canals on their projects.

Flow frequency analysis of existing spate irrigation canals, together with knowledge of their sediment concentrations, bed slopes and information about scour or siltation, will provide a useful insight into future design. Such an analysis has been carried out on the Mansury-Payyan-Bagr canal of the Wadi Zabid Project, which has a design discharge of 40 m /sec, shown in Table 3.

On the basis of this kind of data, the TDA considers that canal bed slopes should be designed for 75 percent of the peak capacity but allowing adequate section for full discharge. They estimate that this will generate velocities of up to 7 percent greater than that by the Simons and Albertson method. Some acceptable scour may occur at peak discharges for a short time, but the canals will have greater sediment-carrying capacity during the predominant low flows. The recommendation to design the bed slope for 75 percent of the peak capacity seems very safe, as on the evidence of Table 3 the 75 percent peak discharge is only likely to be exceeded about 2 percent of the time.

The determination of canal bed slope is probably the most important design factor that influences both the capital cost of the project and its O and M costs. The extent to which canal slopes differ from natural ground slopes will determine the magnitude of the earthworks and the size and number of the water control structures. If steeper canal slopes can be safely adopted, the sediment-carrying capacity will be improved with very significant savings in project cost.

The remodelling of the traditional canals in the middle reach of the Wadi Zabid area provides some useful indicators for future design. The bed slopes of the traditional canals, which must have achieved some sort of regime over the years that accorded with the method of diversion, are shown in Table 3 below. A channel which possesses year-to-year stability is considered to be in regime.

Table 3: Bed Slopes of Old Canals, Wadi Zabid, YAR

Canal	Maximum Capacity (m^3/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 0.0005 to 0.0001. Sluiceways for sediment exclusion are provided at the headworks of these canals but are not very efficient as the sills are not deep enough in relation to the sills of the head regulators and the geometry of the forebay and sluice channel is unsatisfactory. Sediment ejection works were not provided. 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 and this very important issue requires further careful study and consideration.

10.3 Improvements to field systems

It was suggested earlier that distribution canals and control structures to improve the field irrigation system should be carried out initially in areas that are able to grow two crops. In order to keep the cost of these secondary works within reasonable limits, it was also suggested that each inlet structure should supply units of about 40 hectares. The minor distributaries and structures would be about 1 or 2 m /sec capacity so that the farm unit could be irrigated in a day or two.

On some spate schemes the time available for irrigation has been over-estimated and as a result, field inlet structures and minor canals are too small. In such circumstances the farmers make cuts in the canal bank to bypass the inlet or bulldoze a bank across the canal so that the whole flow is diverted until all their needs are met. These are the indications of inadequate design for spate conditions.

Cross control structures in the minor distributaries and field inlet structures should be kept as simple as possible. Small drop structures controlled with stop logs or large gated pipes about 1 m to 1.5 m diameter, could be operated by groups of farmers if their basic method of irrigation had not been varied too much. Farmers will wish to continue to divert the whole flow of the minor distributary in turn and

It is essential to continue to involve them in water distribution and irrigation. Thus, minor canals of the order of 1 to 2 m³/s with checks and field inlets to irrigate about 30 to 50 ha would probably meet their requirements. The field-to-field system could also be improved by additional contour bunding and improvement in the inter-field irrigation by use of filter fabric with boulder protection.

10.4 *Farm access and communications*

Access roads to the irrigation canals and control structures and farm tracks to fields must be planned from inception as an integral part of the irrigation system. These roads are usually on embankments and surfaced with gravel and fine material from the wadi. A reliable communications network is essential for good spate irrigation management because of the much shorter time available for taking decisions on operation.

Telephones and radios are required to connect field personnel with each other and with supervisory staff. The communication network must be dependable and must be maintained in good working condition. The system should include radio communications to all important diversion and distribution sites plus mobile units installed in the vehicles of senior operating and maintenance personnel in touch with a central station at Project Headquarters.

10.5 *Conjunctive use of groundwater and spate irrigation*

The combined use of groundwater and spate irrigation is much discussed but little practised at present. In the Abyan Delta in PDRY, out of a spate area of some 20 000 ha, less than 100 ha uses both sources. In Wadi Tuban the area of conjunctive use is larger but still only represents a very small percentage of the spate area. Does such a system optimise the use of scarce water resources? The answer is not clear. It will certainly increase benefits per unit of land. The use of both water sources must be carefully considered and planned.

The lower wadi areas, where the probability of spate irrigation might be about 20 percent to 30 percent, would be suitable for groundwater development, provided there was a suitable aquifer. In such areas the design of the tubewell system could be built into the existing spate system with distribution mains and hydrants buried in the field bunds. Specific solutions for the design and operation of such systems need to be worked out and the cropping pattern carefully chosen.

10.6 *Deficiencies in planning and design of spate improvements schemes*

The following deficiencies have been encountered:

- ❑ underestimation of maximum flood in the design concept;
- ❑ inadequate provision for sediment exclusion and ejection at canal headworks;
- ❑ insufficient provision of total diversion capacity from the wadi and neglect of investment in downstream areas which have a reduced probability of spate irrigation;
- ❑ underestimation of canal bed slopes and sediment transport capacity;
- ❑ inadequate assessment of the size and capacity of the irrigation inlets to the fields due mainly to a misunderstanding of the probable duration of the spate flows and the reduction in the flood wave in the irrigation channel as the wave passes downstream in the wadi.

11. Operation and maintenance of spate irrigation systems

The main problems encountered in operation and maintenance of spate schemes are concerned with:

- ❑ a clear understanding of local traditions and water rights in relation to the new operating rules;
- ❑ sediment and trash exclusion, ejection and routine clearance;
- ❑ organisation and staff;
- ❑ finance and recovery of water charges.

The avoidance of disputes after implementation, the "misuse" of water by upstream users and, in some cases, by farmers in downstream areas, requires a very clear understanding and appreciation of the traditional water rights and operating arrangements at the planning stage. The recommended scheme should give careful consideration to the social implication of any improvements in the system and traditional concepts should not be discarded without very good justification of all aspects of development. Farmers at the tail end of the traditional schemes should not be deprived of what little rights they had to water without some compensation and without good reason, such as optimisation of benefit per unit of water.

The effects of sediment concentrations in wadi flows on the operation of spate schemes is mentioned above. These problems can be alleviated by improved design but they cannot be eliminated altogether. The cost of maintenance can only be reduced to what may be considered an acceptable level.

The establishment of an O and M organisation with the necessary plant, equipment, workshops and with trained staff to run it effectively, usually takes longer than anticipated. Senior O and M staff should be sent on exchange visits to nearby countries where spate schemes are operated. In many developing countries O and M will need a reliable source of foreign currency to keep the machines in operation. O and M manuals are prepared by the consultants for their project. These should be reviewed annually and amended as the need arises.

The annual O and M cost of spate irrigation schemes should not be entirely dependent on allocations from general revenue. It is usually the first item to be cut at times of financial constraint. A practical means must be devised and instituted for recovering the cost preferably througn well established procedures. Involvement of Farmers' Associations or groups of farmers in O and M should be encouraged.

12. Organization and management

The instrument for development of spate schemes need not be so different in principle from that required by other major projects; but in order to relieve the load on the Chief

Executive and his Deputy of coordinating the efforts of the various departments, it may be worthwhile to consider organising the development agency on a functional basis rather than by discipline, vis: agriculture, engineering, administration, finance and accounts. The basic functional units required would be:

Planning (including monitoring and evaluation);
Design and supervision;
Operations, both agricultural and O and M;
Administration and staff training;
Finance and accounts.

The agency must be given adequate power to perform its duties and responsibilities.

13. Concepts for improvements in spate system design and management

An approach to spate irrigation and wadi development is presented in order to stimulate discussion on economic development concepts.

Present day spate schemes are merely improvements schemes as spate irrigation has been practised for thousands of years on the alluvial coastal plains of the south-west Arabian Peninsular. This fact should not be overlooked. The systems of diversion have been developed by the farmers from experience over the centuries, to utilize and conserve their vital water resource to the maximum.

13.1 Development concepts

One of the most important issues to be decided during phase 1 of the feasibility study is the appropriate development concept to be adopted for improving the traditional spate irrigation system. Should a dam with storage reservoir be constructed or a spate breaker and series of diversion structures or one or more diversion weirs with wadi training and bank protection works?

The social, technical and economic aspects of the alternative concepts should be reviewed during the preliminary appraisal. These should include alternatives for optimal use of surface and groundwater, within a water balance, to maximize overall agricultural production per unit of water and in economic terms for the benefit of all the farmers within the water rights area.

This will entail choices concerning overall project irrigation efficiencies, optimal spate application depth for a single irrigation, and extensive versus intensive use of land. Some questions that arise immediately are:

- which is the target group to benefit from the investments and why was it chosen?
- where should the investment in irrigation facilities be made to obtain the best return per unit of water?
- what is the best means of spreading the benefits to be derived from the total water resource within an overall water balance?
- what methods are to be used for improved conservation of the total water resource?
- will conjunctive use of spate and groundwater optimize the value of the agricultural production per unit of water? and
- is phasing of the improvement works to be advocated? (The first phase for wadi diversion works and the second for improvements in the distribution network.)

13.2 Diversion structures

It is not possible in economic terms to control the probable maximum flood. Thus diversion weirs should have the canal headworks, sediment excluder sluices and sediment control works on one flank and an embankment with a fuse bund on the other flank.

A well-developed traditional spate irrigation system might have, perhaps, 20 or 50 or more individual diversion canals offtaking from the wadi. If this system is to be improved to provide better control of spate flows then it may be necessary to construct some 4 to 6 diversion structures to replace the 20 to 50 traditional offtakes in the wadi.

Each diversion would command an area with a different probability of irrigation. As the probability of irrigation reduces from head to tail of the wadi an analysis is necessary to determine an economic cut-off point for improvements in water control and diversion.

The cut-off point, however or wherever devined, will generally not be accepted bvy the farmers and they usually continue to reconstruct their traditional works in the lower wadi, if no improvements are carried out. This is a clear indication that engineers should always be seeking ways and means of providing better spate control but at a much reduced level of investment in diversion and distribution works in the middle and lower reaches of the wadi to match the lower returns from these areas.

In any event the incremental benefits from improvements to spate irrigation schemes are generally limited by the constraints of a single watering, the water retention capacity of the soil and the root zone. In the lower reaches of the wadi the lower probabilities of irrigation reduce further the already limited incremental benefit and call for unconventional solutions if we are to help farmers to improve water control.

The first diversion work at the head of the wadi could have a weir with a design capacity for the 1 in 20 year flood. The range of diversion structures downstream would have weirs with design capacities of shorter return periods, reflecting acceptance of an increasing degree of risk and reduced levels of investment to match the lower degree of reliability of irrigation further down the wadi giving lower returns. Thus, for example, the last two diversion structures might have a relatively short gabion weir with a breaching bund capable of withstanding a flood in the lower wadi, with a return period of, say, 3 years.

Much more data collection, research and model testing needs to be done on sediment control at spate irrigation headworks.

Diversion structures should be designed for maximum sediment exclusion. The geometry at the canal headworks should provide a converging curved approach channel to the sediment sluiceway, which should have a capacity of about 1/3 to 1/2 that of the full supply discharge of the canal head regulator. The diversion weir should be positioned at the upstream end of the guide pier and the bank protection

... should be extended upstream to prevent any pronounced unfavourable approach flows from that side.

13.3 Some irrigation efficiencies

The overall irrigation efficiency of a spate improvement scheme involving a series of diversion weirs is likely to be of the order of 30 to 40 percent. That is, the net consumptive use of surface water might be only about one-third of the mean annual flow. However, extensive spate spreading is probably the best means of recharging groundwater, much of it through wadi bed seepage and canal losses. Thus the overall water balance would allow somewhat greater groundwater abstraction than for schemes with higher irrigation efficiencies.

13.4 Optimal spate application depth

The optimal spate application depth for a single irrigation should be an average of 400 mm net stored in the soil. This is about as much as the field to field system of irrigation will permit and as much as the average soils in spate areas can retain.

A net water application of 400 mm will allow extensive rather than intensive production and a larger area for spate spreading and groundwater recharge.

13.5 Groundwater recharge

Groundwater recharge schemes, with specially designed recharge basins, do not appear to optimize the value of agricultural production per unit of water. Surface water should first be used for extensive spate irrigation with the wadi bed seepage and canal losses going to groundwater recharge.

13.6 Water use

Conjunctive use of spate water and groundwater on the same plot of land is to be deprecated as it will not optimize production per unit of water. Spate water could be used once in 3 to 4 years on tubewell farms for leaching purposes and to improve soil texture.

13.7 Phasing of development

This development concept lends itself to a phased programme for implementation. The first phase could provide diversion and control of spate flows and deliver irrigation supplies to the existing canal systems in quantities that the groups of farmers have been accustomed to handling.

The operation of the new canal offtakes should be handed over to the groups of farmers responsible for each canal as soon as they have demonstrated that they can handle the system without difficulty, but under general supervision of the authority concerned. When these farmers have gained complete confidence in the improved diversion system the next phase should be planned in close consultation with them.

This second phase would improve distribution and control, where necessary, on the traditional canals. In the areas which are able to grow more than one crop, the field to field system could be changed to a system of small distributaries delivering supplies to each field or at least to smaller groupings of fields than at present.

After the first phase development has been in operation for a few years, there will inevitably be some requirements for minor alterations and improvements; these will have become evident and could be incorporated into a second phase for improvement of the secondary irrigation system.

A phased series of improvement schemes seems much more appropriate and would provide continuity of technical involvement in the area until all systems were operating properly.

Above 60 percent of the total commanded area will receive only one irrigation per year or less and improvements works to deliver water by minor canals to each field will not be justified. In order to keep unit costs for improvements to secondary works within reasonable limits, an economically acceptable size of irrigation unit has to be decided and this raises the question: what is a manageable quantity of spate water for a small group of farmers to handle, using traditional methods, but with some inter-field irrigation improvements? The largest flow would be about 2 m³/s delivered from a single inlet structure. This flow could irrigate about 40 hectares in a day or two, depending on the variation of the flow.

13.8 Water rights and operating procedures

Improvement works as outlined above will be in keeping with traditional water rights and operating procedures. The improvement will be in control of spate flows.

Thus, the command areas controlled by a single distribution structure on the diversion supply canals to be provided in the first phase could vary from 500 to 1 000 hectares, while in the second phase, the distribution canals and the structures would supply units of about 40 hectares. Groups of farmers could then continue to have a large part to play with the management and distribution of water to their fields in both phases.

The farmers' continued involvement in operation and maintenance is absolutely essential.

ERITREA

EASTERN LOWLANDS WADI DEVELOPMENT PROJECT

Working Paper I

Appendix 2

HYDROLOGY

INTRODUCTION

1. There are only three raingauges in or near the catchment of Sheeb and none for Wadi Laba. Unfortunately, daily rainfall data for the Sheeb stations was unobtainable, and the available monthly rainfall data, although covering the previous 25 years, ends in about 1953.

2. At the time of the Formulation Mission, according to the Department of Water Resources (DWR), no flow gauges had ever operated on the wadis contributing flows to the Project areas. Thus, in the Formulation Report mean annual flows were estimated on the basis of the isohyetal map in Working Paper I Figure 2, and assumed run-off coefficients. Estimates of frequency and duration of flow on this basis are given in Working Paper II. However, DWR has in the meantime installed a recording stream gauge on Wadi Laba, just upstream of the proposed diversion structure, and details of the first observations from this are discussed below.

FLOOD ESTIMATION

3. Again, at the time of the Formulation Mission, there had been no previous flood studies in, or relevant to, the Eastern Lowlands, and other than that mentioned below, no records of any historic floods in the Project areas. The only relevant data on floods were of a regional nature, given in various reports on Ethiopian and Southern Arabian rivers, including the Halcrow reports on Wadis Jizan and Dhamad in Saudi Arabia and Wadi Surdud in Yemen, and those by Binnie, also on wadis in Yemen.

4. In the absence of any hydrometeorological data for the catchments of the project areas, the choice of methods for flood estimation was limited. There are a number of standard empirical formulae which have been applied in similar circumstances in such areas, including the Ryves' formula and Creager's formula, which treat maximum floods primarily as a function of the catchment area. The Ryves method was developed in south India for 'areas near hills' and Creager's formula provides an envelope of world floods. The modified B D Richards' deterministic method may also be used in computing the maximum flood, but this requires a subjective assessment of maximum rainfall and other parameters such as the run-off coefficient.

5. However, the magnitude of the maximum flood, whilst of interest, is not critical to the design of spate diversion works, as there is no intention of designing the works to withstand this. The requirement is to know the magnitude of floods of relatively short return period in order to select the design flood.

6. Binnie[1] developed the following regional flood formula for wadis in southern Yemen:

$$\text{Mean Annual Flood } (m^3/s) = 3.27\ CA^{1.163}\ MSL^{-0.935}$$

where CA represents the catchment area in km^2, and MSL represents main stream length in km.

7. Binnie also derived the following growth factors for floods of increasing return period:

Return Period (years)	Growth Factor (>mean annual flood)
5	1.5
10	2.2
20	3.1
50	4.6

8. As the Binnie formula is the most relevant for the project catchments this was used for estimating floods at the proposed diversion sites.

9. The project area at Sheeb receives spate flows from three distinct zones: the main source, Wadi Laba; Mai Ule, a separate wadi further to the north[2]; and through a number of minor wadis to the north of Mai Ule which appear to contribute only a small proportion of the total inflow and drain into a depression immediately to the west of the irrigated area.

10. The project area at Wadi Labka receives most of its inflow from that wadi, although the command area of Ghadim Halib on the left bank of Wadi Labka also receives a contribution from other minor wadis (see Figure 4 in Working Paper I).

11. The catchment characteristics of the project areas, down to the assumed position of the first diversion on each wadi, are given in Table 1 which also shows the estimated floods from Binnie's formula.

[1] Binnie and Partners. Feasibility Study for Long Term Wadi Rehabilitation, Southern Yemen. Final Report 1988.

[2] See Figure 3 in Working Paper I.

Table 1. Catchment Characteristics of the Project Areas and Their Estimated Peak Floods

	Units	Sheeb		Wadi Laba
		Wadi Laba	Mai Ule	
Catchment Characteristics				
Catchment Area	km^2	660	170	1 390
Main stream length	km	50	29	63
Highest point (elevation)	m	2 585	1 960	2 231
Elevation at diversion site	m	230	260	360
Peak Flows				
Mean annual flood	m^3/s	160	55	310
5 year flood	m^3/s	240	85	465
10 year flood	m^3/s	355	120	680
20 year flood	m^3/s	500	170	955
50 year flood	m^3/s	740	255	1 420

HISTORIC FLOODS

12. For comparison with the estimated floods in Table 1, the Formulation Mission investigated an historic flood which was reported to have occurred in Wadi Laba in about 1970. According to members of the Baito this was the largest flood in living memory, and the Mission was shown the high flood level at a point in the wadi just upstream of the site of the proposed diversion works[1].

13. The cross section of the wadi at this point, and at a point some 100 m upstream, was surveyed and the cross sections plotted. The discharge was then computed using a computer programme for solving the Manning equation:

$$Q = 1/n \times AR^{0.67}S^{0.5}$$

where
- Q = discharge in m^3/s
- A = cross sectional area of flow in m^2
- R = cross sectional area of flow divided by the wetted perimeter of flow, in m
- S = slope of channel

and n is a dimensionless coefficient which depends on the roughness of the channel.

14. Ven te Chow gives the value of n for various types of channel, bed material and condition, from which the following range of n can be deduced for the cross section surveyed at Wadi Laba:

---

[1] About halfway between the proposed site of the diversion works and the recently installed stream gauge.

	Range of n
Bed material of boulders, mean depth of flow about 2 m	0.040 - 0.070
Add for bank irregularities, depending on % of bank to wetted perimeter	0.000 - 0.020
Overall	0.040 - 0.090

15. Bank irregularities for the cross section concerned are not significant, because the ratio of bank to wetted perimeter is small, and so the contribution to overall n would also be small, say 0.002. Thus the range of n would be 0.040 to 0.072.

16. From the Manning equation, for a bed width of 68 m, a bed slope of 0.0069 and bank slopes of 4.76, the computed discharge, Q, at the depth at the high flood level indicated on site, was 481 m^3/s for n = 0.040, and 281 m^3/s at n = 0.072.

17. Obviously the selection of a value for n is critical and requires some judgement. From Table 1 above it can be seen that a Q of 481 m^3/s is equivalent to a flood of estimated return period about 15 to 17 years, and a Q of 281 m^3/s is equivalent to a flood of estimated return period about 7 years. Conversely, if the historic flood was assumed to have a return period of 20 years (ie Q = 500 m^3/s), n would have a value of 0.40.

THE NEW GAUGE ON WADI LABA AND 1994 SEASON FLOODS

18. Hydrographs recorded at the new gauge on Wadi Laba during the 1994 flood season from 1 July to 16 August are shown in Figures 1-3. The provisional rating curve in Figure 4 remains to be confirmed, as NRCE[4], who were responsible for the installation of the gauge, are not yet satisfied that the datum is correct.

19. As can be seen, there was a large flood on 22 July which swept the pressure transducer away[5] and as a result interrupted recording for about a week. Unfortunately it is not known whether the record shows the peak stage or whether the transducer was swept away before the peak was reached. However, if for the time being the provisional rating curve is taken as being correct and zero on the hygrograph corresponds to 6.00 on the rating curve, this flood had reached a magnitude of about 450 m^3/s by the time the recording was interrupted. This approaches the upper estimated magnitude (481 m^3/s) of the 1970 flood, and would indicate that the flood estimates above are of the right order of magnitude.

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4 Natural Resources Consulting Engineers Inc.

5 This flood incidentally also caused considerable loss of irrigable land in Sheeb, as it appears to have been diverted over the land by the permanent diversion structure recently built by MoA.

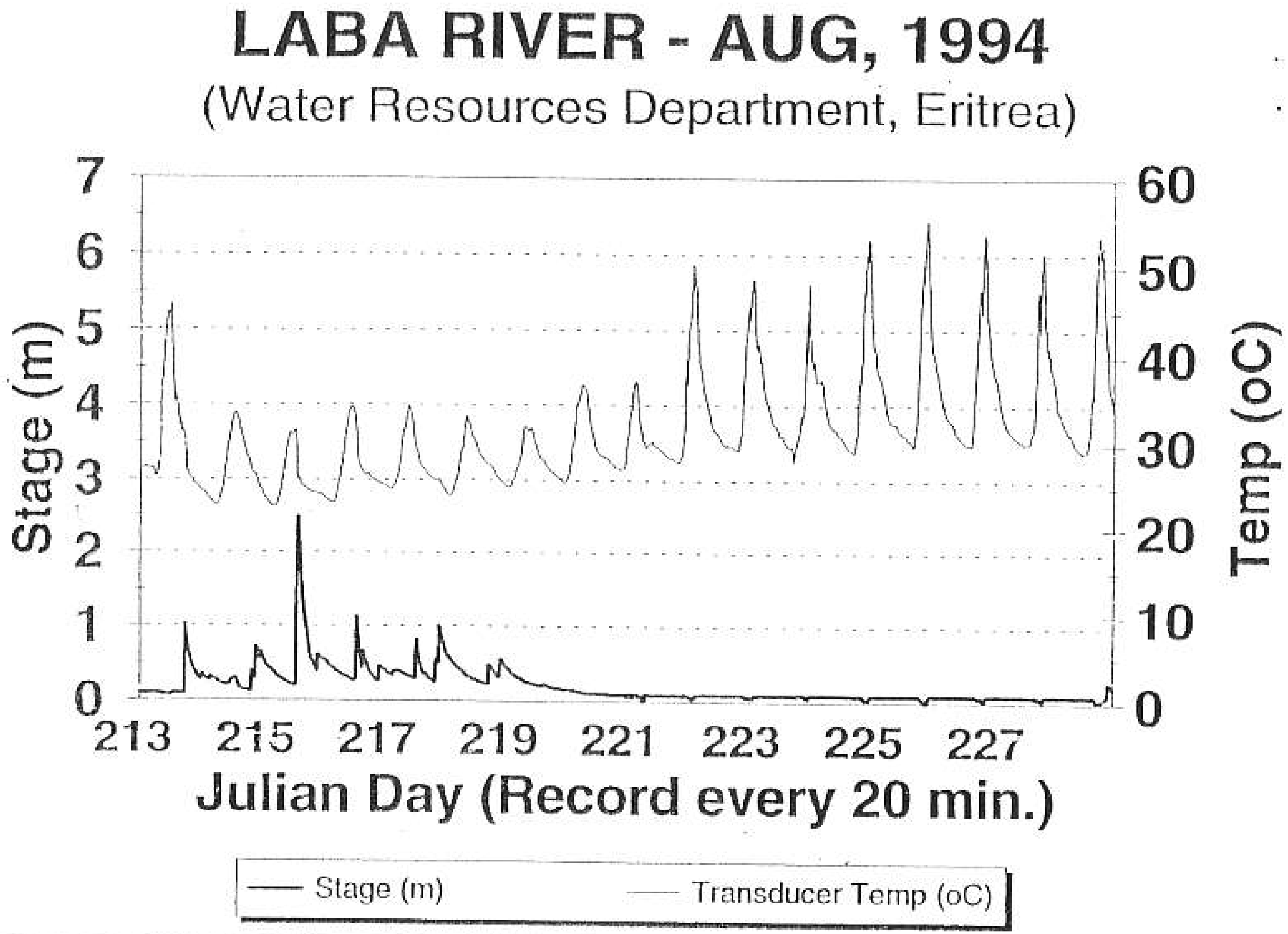
20. On the other hand, according to NRCE their datum could be in error by 2 m, in which case the flood of 22 July would have been considerably more than suggested. This is thought to be unlikely because the Mission was able to inspect the high flood mark and compared it with that measured for the 1970 flood: although the 22 July flood was higher it was not unduly so. It is also interesting to note that there was another large flood on 30 August[6], on the night before the Mission's visit, for which the hydrograph was unfortunately not yet available.

21. Nevertheless, the remaining doubts regarding the historic and recent floods must be resolved, but this will take more than a visit of a day or so to achieve, and for this reason it is essential that flood studies be carried out during the pre-construction phase of the project. The recent work of NRCE must be compared with the information from the 1970 flood, and the cross sections re-surveyed to pick up the new high water marks. In this respect Mr Afewerke Yohannes of MoA knows the exact location of the sections surveyed.

22. Otherwise, the hydrographs for 1994, which was considered to be an average to good year, appear to confirm the assumptions regarding the number and duration of floods, and if anything indicate a stronger "base" flow during the recession between spates, particularly in July.

6 This also caused extensive loss of agricultural land because it was diverted by the MoA structure.

Figure 1. Hydrograph for Wadi Laba - August 1994

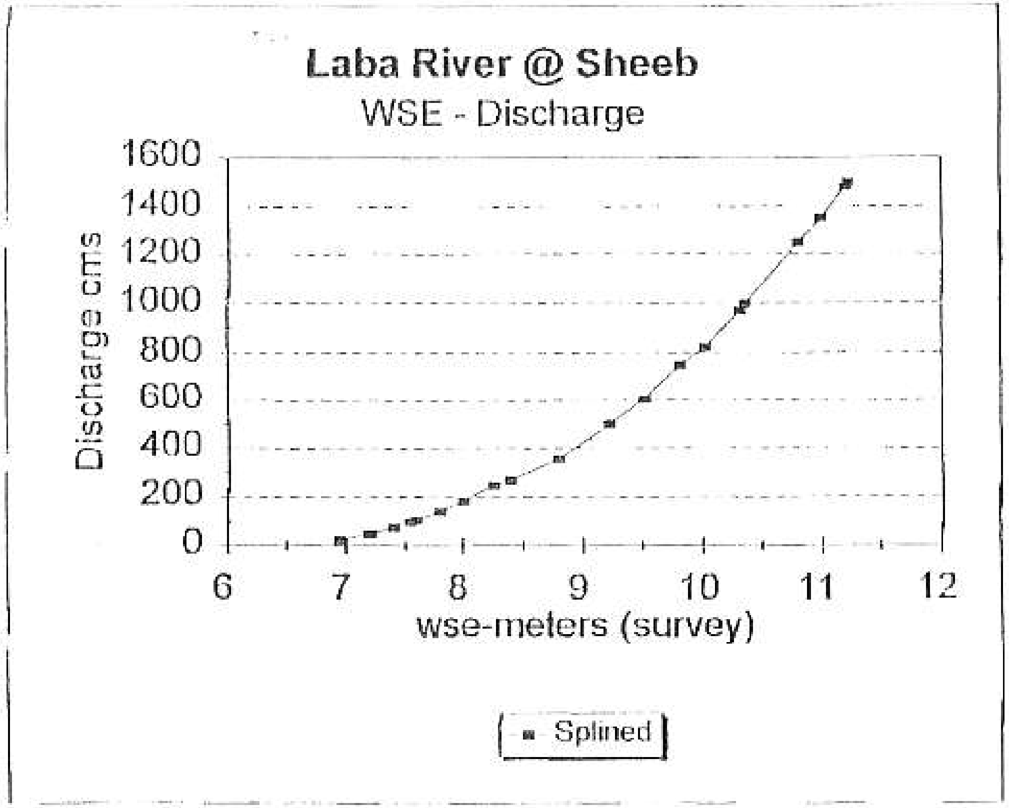


(Source: Natural Resources Consulting Engineers Inc.)

Figure 4. Provisional Rating Curve for Gauge on Wadi Laba

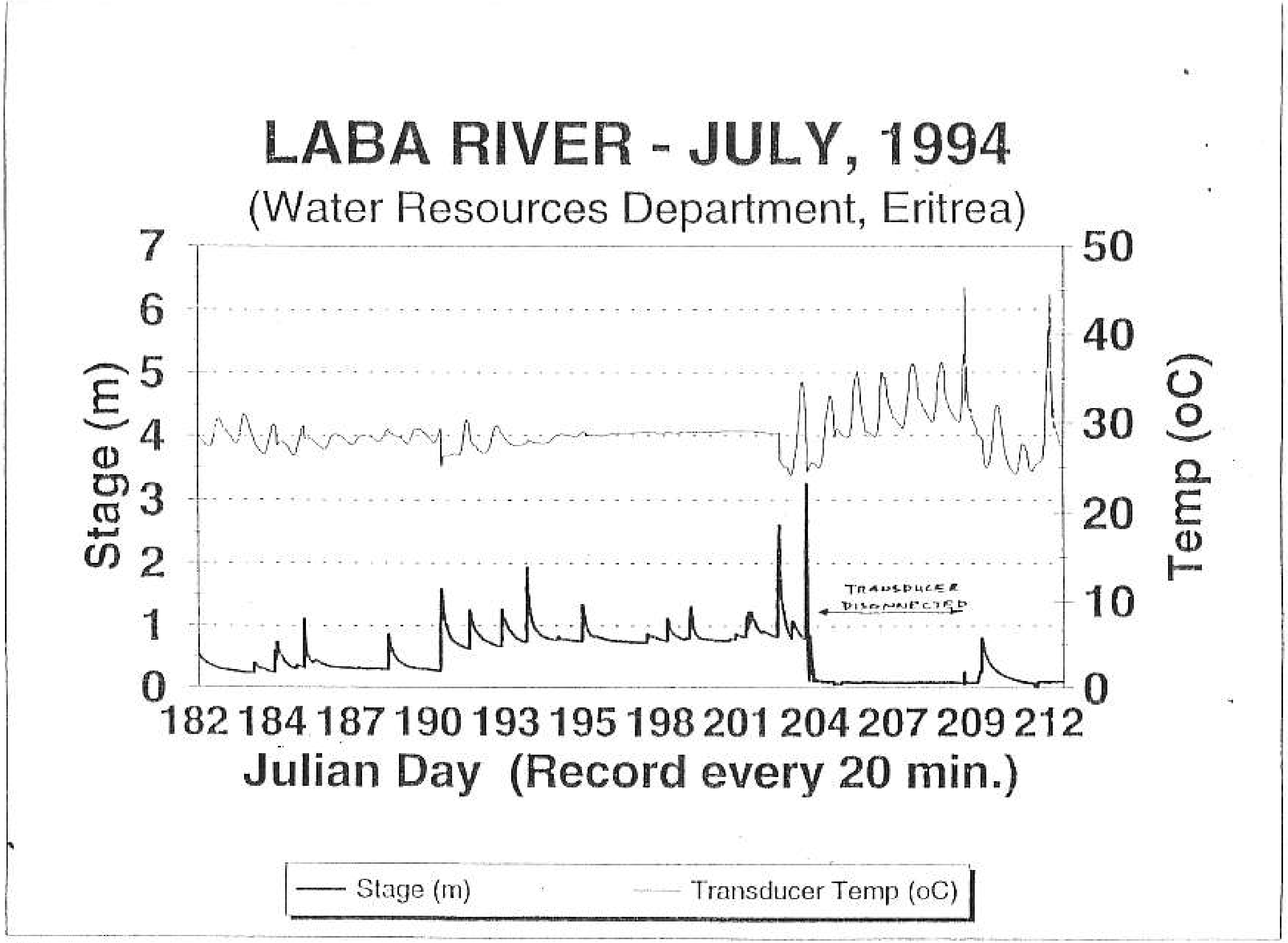
Laba River @ Sheeb
Hec 2 (large bold)and splined

wse	q(cms)
6.945	**25**
6.95	25.5
7.194	**50**
7.2	50.5
7.4	77
7.545	**100**
7.6	108
7.8	142.5
8	184.5
8.256	**250**
8.4	271
8.8	358
9.216	**500**
9.5	604.5
9.8	750
10	824.5
10.3	972.5
10.348	**1000**
10.792	**1250**
10.8	1253.5
11	1356
11.2	1482
11.226	**1500**



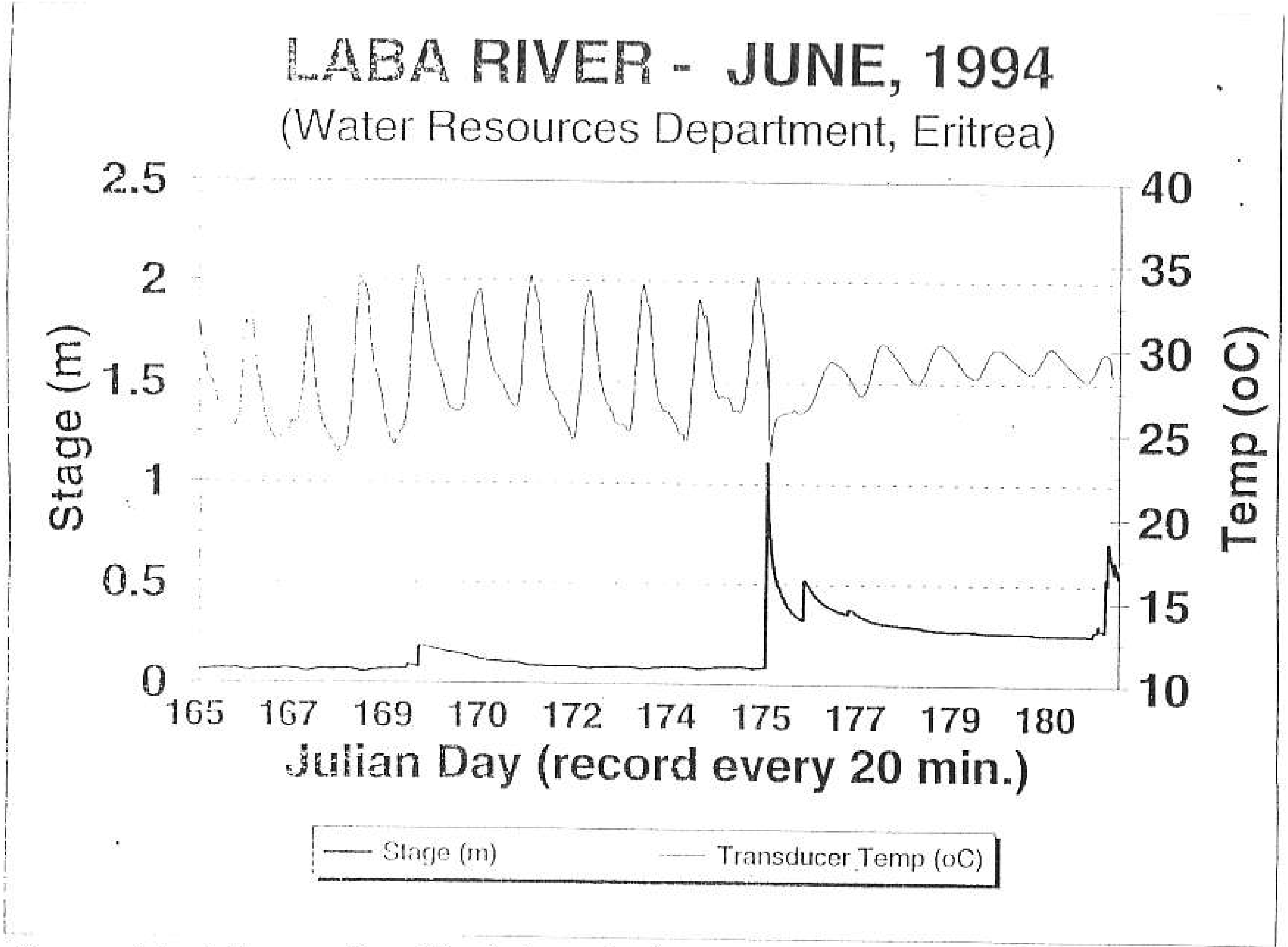
(Source: Natural Resources Consulting Engineers)

Figure 2. Hydrograph for Wadi Laba - July 1994



(Source: Natural Resources Consulting Engineers Inc.)

Figure 1. Hydrograph for Wadi Laba – June 1994



(Source: Natural Resources Consulting Engineers Inc.)

82. It is therefore intended that the strategy for encouraging beneficiary contribution will be to focus on the distribution structures: as there is, at this stage, a limit on the number of distribution structures which can be provided, and some flexibility in their location, there will no doubt be competition among the various farmers groups for these structures. The approach will therefore be to give priority to providing structures on those sites where farmers have demonstrated their commitment by stockpiling, at or near the site, the full requirement of locally available materials, such as stone and sand, and signifying their agreement to providing the agreed subsequent labour quota. Where there is no such support from farmers, no structure should be built.

C. Expected Benefits

83. As mentioned, the objective of improvements to the existing spate irrigation systems is to enhance the annual probability of irrigation over the existing area of command, thus increasing the area irrigated on average each year.

84. Whilst the structures proposed under the Project are semi-permanent only (by virtue of the breaching bund), in years of average floods the structures will act in the same way as a permanent weir, and volumes diverted and areas irrigated will be increased in the same proportions. Thus while present diversion efficiencies are estimated at around 35%, a conservative "with project" increase to 65% can be assumed for Wadi Laba and Mai Ule, and 60% for Wadi Labka where floods are more flashy, and with sharper peaks than Wadi Laba.

85. Table 2 summarizes the hydrological features of the Project areas and gives an estimate of the mean annual runoff volume in each of the areas. Table 3 shows the Mission estimates of the areas presently under command from each of the wadis being considered. Table 6 shows an estimate of the average area irrigated annually without Project, and, as a result of improved efficiencies, with Project.

Table 6. Estimate of Average Area Irrigated Annually With and Without Project

Project Area/Catchment	Sheeb: Wadi Laba		Sheeb: Mai Ule		Wadi Labka		Total	
	Without	With	Without	With	Without	With	Without	With
Mean annual flow volume (Mm³)	52.8		9.35		37.5		99.65	
Diversion Efficiency (%)[a]	35	65	35	65	35	60		
Conveyance Distribution Efficiency (%)	30	40	30	40	25	35		
Field Application Efficiency (%)	50	50	50	50	50	50		
Overall Irrigation Efficiency (%)[b]	15	20	15	20	13	18		
Average Net Irrigation Requirement (m³/ha)[c]	3 550		3 550		3 550		3 550	
Average Gross Irrigation Requirement (m³/ha)	23 667	17 750	23 667	17 750	27 308	19 722		
Command Area (ha)[d]	2 385		640		1 645		4 670	
Average Area Irrigated Annually (ha)	781	1 934	138	342	481	1 141	1 400	3 417
Percentage of Command Area	33	81	22	53	36	69	30	73
Incremental Area Irrigated Annually (ha)	-	1 153	-	204	-	660	-	2 017

a/ The diversion efficiency is the percentage of the annual spate flows that can be diverted for irrigation, the remainder being wadi seepage losses and flows passing through the sediment sluiceway and over the spill weir to downstream users or to the sea.

b/ The overall spate irrigation efficiency is the conveyance/distribution efficiency multiplied by the field application efficiency, averaged for the whole area commanded by the diversion headworks.

c/ The average gross spate irrigation requirement is the average net irrigation requirement divided by the overall spate irrigation efficiency.

d/ All areas may be taken as net, as only irrigated areas were planimetered from the aerial photography.

56. Conveyance and distribution efficiencies are also expected to increase as a result of provision of control structures within the distribution system. However, because structures will not be provided throughout the system, full control will not be possible and so only a modest increase in efficiency has been assumed. Although improved distribution efficiency and better control could also lead ultimately to some improvements in field application efficiency, the field to field system would remain and at this stage it has been assumed that there would be no increase in field application efficiency.

57. Efficiencies at Wadi Labka are taken to be slightly lower than in Sheeb because of the distance between the proposed headworks and the irrigable area, and because of a slightly lower density of proposed cross regulator and offtake structures.

58. Irrigation water requirements were derived assuming the following indicative cropping pattern throughout the Project areas (see Appendix 3):

Crop	Planted Area (ha)
Sorghum - main crop	0.65
Sorghum - ratoon crop	0.40
Maize	0.25
Sesame	0.20
Groundnut	0.15
Total cropped	1.65
Cropping Intensity (%)	165

89. Two levels of effective rainfall were considered in estimating the net irrigation requirement, based on:

- the occurrence in the Project areas of the mean monthly rainfall; and

- the "dependable" rainfall, or rainfall expected to be exceeded in about four years out of five, which is much less than the mean rainfall.

90. Since the net irrigation requirement based on the latter represents the peak requirement this is used in sizing canals and structures. For the purpose of sizing the irrigable area however the net irrigation requirement based on mean effective rainfall is considered. From Appendix 3, this is estimated to be about 3 550 m^3/ha/annum.

91. As can be seen from Table 6, the total increment in irrigable area in an average year "with Project" is estimated to be just over 2 000 ha net; the direct benefit from the proposed infrastructural improvements would therefore be the total crop production on this area.

VI. ARRANGEMENTS FOR IMPLEMENTATION OPERATION AND MAINTENANCE

A. Institutional Responsibility and Professional Staffing

1. General

92. The project will be executed by MoA, which will establish, under the chairmanship of the Head of its Programming and Planning Department, a Project Steering Committee that will comprise the Heads of all departments concerned. The duties of the committee will be programming, budgeting, supervision and monitoring of all Project activities. Implementation of the component for spate irrigation improvement will be under the direct control and supervision of the Head of the Soil and Water Conservation and Irrigation Division of MoA.

93. Strengthening of national capacity to undertake similar future developments is to be one of the major thrusts of the project. However genuine benefit, in terms of increased capacity, will only be gained if Eritrean professionals are involved in all aspects and stages of the developments, and skills are developed within the country through "learning by doing". As a fundamental principle therefore the project will, to the maximum possible extent, be staffed and implemented by Eritreans, rather than

APPENDIX C

STATE OF ERITREA

EASTERN LOWLANDS WADI DEVELOPMENT PROJECT

FORMULATION REPORT

<u>Working Paper II</u>

SPATE IRRIGATION SCHEME:
DESIGN PHILOSOPHY AND CALCULATIONS FOR DIVERSION WORKS

STATE OF ERITREA

EASTERN LOWLANDS WADI DEVELOPMENT PROJECT

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SPATE IRRIGATION SCHEME:
DESIGN PHILOSOPHY AND CALCULATIONS FOR DIVERSION WORKS

I. CONCEPTS FOR IMPROVEMENT OF TRADITIONAL SPATE SYSTEMS

1. Spate systems have been developed by farmers over the centuries, to utilize and conserve their vital water resources to the maximum. One of the most important issues to be decided during the initial phase of the feasibility study is the appropriate development concept to be adopted for improving the traditional spate irrigation system. Should a dam with storage reservoir be constructed or a spate breaker and series of diversion structures or one or more diversion weirs with wadi training and bank protection works?

2. The social, technical and economic aspects of the alternative concepts should be reviewed during the preliminary appraisal. These should include alternatives for optimal use of surface and groundwater, within a realistic and sustainable water balance, to maximize overall agricultural production per unit of water and in economic terms for the benefit of all the farmers within the water rights area.

3. This will entail choices concerning overall project irrigation efficiencies, optimal spate application depth for a single irrigation, and extensive versus intensive use of land. Some questions that arise immediately are:

- which is the target group to benefit from the investments and why was it chosen?
- where should the investment in irrigation facilities be made to obtain the best return per unit of water?
- what is the best means of spreading the benefits to be derived from the total water resource within an overall water balance?
- what methods are to be used for improved conservation of the total water resource?
- will conjunctive use of spate and groundwater optimize the value of the agricultural production per unit of water? and
- is phasing of the improvement works to be advocated? (The first phase for wadi diversion works and the second for improvements in the distribution network.)

A. Storage Dams

4. The possible construction of a storage reservoir on the wadi would provide a costly, though technically attractive, means of regulating the highly variable spate flows of short duration with very sharp peaks, into an almost perenial irrigation supply. The heavy silt load carried by the floods would, however, cause a rapid loss of reservior storage at a rate in the order of 3 percent per annum. Ground water recharge would also be severely reduced.

5. The large storage dams would command relatively small areas and involve high capital investment per hectare, in addition to the cost of the distribution system. Also the operation and maintenance of the perennial water supply system would require a complete change in traditional irrigation and agricultural practices, and could lead to intensive rather than extensive

use of land, when in fact water not land is the limiting resource.

6. As the reservoir silted up, large flood flows would begin to overtop the spillway more frequently and the wadi irrigation system would gradually revert to traditional methods of diversion. The construction of a storage dam is likely to deprive farmers in the downstream areas of the wadi of their former supply of spate water, however eratic it may have been.

B. Spate Breakers

7. A spate breaker is a small dam with a reservoir capacity which would absorb the larger spates with high peak flows that would otherwise wash away all the brushwood and earthern bunds of the traditional systems. These large spate flows would be immediately routed through the spate breaker and the storage would be drawn down in 2 to 4 days, preferably before the ensuing spate arrived.

8. Unfortunately, spate breakers, besides being very costly, have a very much shorter life than a storage reservoir, due to rapid sedimentation. This is caused by the smaller storage capacity, the low ratio of storage volume to average annual flow volume, and the high sediment loads. A proposed spate breaker at Khola on Wadi Zabid in Yemen was estimated in 1971 to have an effective life of 5 to 10 years and was therefore not implemented.

C. Schemes for Spate Diversion with River Training and Bank Protection Works

9. The water resources of the Eritrean wadis fall into three fairly well defined categories:

- i base flow and minor spates;
- ii spate flows;
- iii groundwater.

10. The base flow and minor spates are generally sufficient to provide two crops per year, usually the first ratoon of sorghum or replanting with maize, on about 20 to 30 percent of the traditionally irrigated areas. With improved irrigation methods this area could increase to about 30 to 40 percent.

11. Spate flows provide irrigation for just over one crop on about 65 percent of the traditionally irrigated area, but only with a variable degree of probability.

12. There is virtually no groundwater abstracted for irrigation in the spate irrigated areas of Sheeb and Wadi Labka. In areas where groundwater is developed the cropping intensity is generally about two crops per year.

13. A review of concepts for improving traditional spate schemes must be linked with the assessment of water resources and the optimal water balance. Benefits from the total water resource must be assessed for each option to ensure that the total water resource is being used to the best advantage for farmers within the water rights area of the whole wadi and within a sustainable water balance.

14. In arid zones, where water is an absolutely vital comodity, improvements to spate schemes have a special significance in stabilizing agricultural production and generally improving the well being of the people in those areas. The yardstick for the selection of a spate improvement scheme should perhaps be the option which gives the most effective utilization of the total water resource, yielding the highest value of production per unit of water at the least cost.

15. If this were considered a suitable yardstick then, with the degree of

reliability of irrigation of the traditional command areas reducing from the top to the lower areas of the wadi system, so also will the scale of investment have to be reduced. Thus a concrete weir and headworks with a fair degree of security might be justified at the head of the wadi, while only a traditional large earth bund could be justified at the lower end. A range of diversion structures representing reduced levels of investment and increasing levels of risk need to be designed to match the lower degree of reliability of irrigation further down the wadi, giving lower returns.

16. There must be farmer participation in the work if costs are to be reduced and thus the designs of low-cost diversion works for the lower reaches of the wadi must be capable of construction by groups of farmers with some unskilled labour, and with the minimum of technical supervision.

17. Very preliminary estimates of water availability at diversion sites, based on analysis of typical flood hydrographs indicate that volumes diverted and areas irrigated can be considerably increased by installing some relatively small spillway capacity in the traditional earthen diversion bund or "agim".

18. Figure WP2 - 1 shows an assumed but typical hydrograph of a flood with a return period of 5 to 10 years that could occur in Wadi Laba. If the proposed improved diversion works were provided, comprising a 33 m^3/s canal head regulator, a sediment excluder sluice and a permanent diversion spillweir, the indications are that the flow volume diverted could be doubled or trebled.

19. In order to calculate the improved diversion ratio with a higher degree of accuracy, the following data and site information would be required:

- The headwater and tailwater stage discharge curves for the wadi with the traditional diversion;
- The stage discharge curve at the head of the traditional primary diversion channel and the stage (level) at which it will break back to the wadi or do severe damage to the agricultural lands;
- The headwater and tailwater rating curves with the improved diversion headworks, (see assumed stage/discharge relationship shown at Figure WP2 - 2);
- A series of actual flood hydrographs of various estimated return periods or an agreed typical spate hydrograph for the wadi;

20. However, it can be confidently predicted that an improved headworks, designed in the manner outlined in this Working Paper II, would at least double the flow diversion capability at the site.

II. DESIGN OF SPATE DIVERSION WORKS

A. General Requirements for Spate Diversion Works

21. The requirements for improved spate irrigation diversion works are a weir, a sediment excluder sluiceway and a canal head regulator. The irrigation offtake should be set at an angle to the sluiceway for it to function effectively. The problem is one of diverting the low flows and conducting as much as possible of the wadi's bed load material through the diversion structure, thereby reducing the cost of its removal from the canals.

22. It is not possible in economic terms to provide diversion weirs for spate irrigation schemes which will control the probable maximum flood. Thus

diversion weirs should have the canal head regulator and sediment excluder sluice on one flank and a breachable bund on the other flank.

23. The design of the weir entails the determination of the headwater and tailwater rating curves, intensities of discharge, differential heads, high flood levels, free-board and top levels of the fusible dike.

24. The stilling basin level and dimensions of the energy dissipator and appurtenances, including the depth of cut-offs and length of flexible aprons, will then be determined and designed against uplift pressure and exit gradient. A safe exit gradient calculated according to Khosla should be adopted and protection against scour should be provided according to Lacey.

25. The following calculations are intended as a worked example only and are merely indicative at this stage, i.e. they must be repeated at the detailed design stage after checking all parameters.

B. Sheeb Diversion Weir Design

26. In the case of the proposed Sheeb weir design, a tentative estimate of the flood with a 50 year return period could be of the order of 800 m^3/s. The planned diversion requirement will be about 33 m^3/s to irrigate about 1,700 hectares on the right bank, and 10 m^3/s for 500 hectares on the left bank, for crops that can mature on spate irrigation. Clearly, the diversion weir must be designed with a breachable bund and a design flood discharge of 300 m^3/s, having a return period of about 6 years, is recommended. The weir would be about 60 metres long; if it was made much longer it would encroach to too great an extent on the waterway required, when the bund breaches, to pass a flood of say 550 cumecs, having a return period of about 20 years.

27. The incremental benefit will arise from better control and diversion of small to medium floods and allowing the larger flood flows to pass virtually unchecked, in the same way as traditional schemes do, but without regular damage.

28. Diversion weirs are constructed in wadis to raise the flood flows to a level that will enable the design full supply discharge to be diverted through the canal head regulator before the flood overtops the crest of the weir.

29. The weir crest level needs only be raised slightly above the average wadi bed level for command purposes, but for efficient sluicing of the bed load at low flows it needs to be at least 1.5 m above **average** wadi bed level. This height is not always ideal for sediment discharge at high wadi flows, but it may not be possible to set it very much higher if excessive costs of energy dissipation on the diversion weir are to be avoided.

30. At the proposed Sheeb diversion site, the preliminary design can be based on the following levels taken from the site survey plan:

- Lowest wadi bed level = 39.0 m.
- Average wadi bed level = 39.5 m.

31. The elevation of the invert (floor level) of the sediment excluder sluiceway should be set at the **average** wadi bed level or about 0.30 m above the level of the low flow channel, i.e.about 39.3 m.

32. The sill level of the canal head regulator (CHR) should be set about 0.60 m above the invert of the sediment sluiceway, i.e. at about 40.0 m and the crest level of the weir should be set at about 1.3 m above the sill level of the canal head regulator, i.e. at 41.3 m.

33. The weir will be an ungated structure designed for permeable foundations. The crest of the weir should be designed with a slight slope

along its length with the fall towards the CHR and sluiceway, to assist in maintaining the approach channel in front of the headworks, and in clearing any sediment deposited downstream of the sluice. The weir crest level at the right abutment to the breachable bund should be set at 41.6 m, giving the crest a slope of 1 in 200.

34. Assuming that a design flood discharge of 300 m^3/s with a return period of about 7 years, is adopted for Sheeb diversion works, based on a peak diversion canal flow of 33 cumecs, a peak sediment sluice flow of 20 cumecs and a design weir discharge of 300 cumecs, then the design intensity of flow over the weir should be kept fairly low and restricted to 3 to 5 m^3/s per metre. This will allow for a larger discharge to safely pass over the weir while the breachable bund is eroding. Adopting a design intensity of 5.0 $m^3/s/m$, a length of masonry/concrete weir of 60 m would be required, assuming a coefficient of discharge of 1.7, and a flood rise above the crest of about 2.05 m.

E. Headwater and Tailwater Rating Curves

35. It will be important to establish a good headwater and tailwater rating curve including an assumed retrogressed downstream wadi bed level of about 1 m at low flows.

36. The stage/discharge relationship and the headwater and tail water rating curves can be determined by the following procedures and computations:

(1) The cross sectional areas and the wetted perimeters are measured for several stages of flow.

(2) The hydraulic mean depth R = A/P can then calculated for each stage.

(3) The average flow velocity in the wadi is then calculated for each stage, based on Manning's formula:

$$V = 1/n * R^{0.667} * S^{0.5}$$

where 'n'is the Manning roughness coefficient and 'S' is the bed slope.

Values of Manning's 'n' are based on experience and empirical data. V.T.Chow has proposed a useful method of assessing 'n'. He breaks down 'n' into a number of factors relating to the material involved (n_0), degree of irregularity (n_1), variations of channel cross section (n_2), relative effect of obstructions (n_3), vegetation (n_4) and degree of meandering (m_5). These effects are considered independently, then:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4) m_5$$

Table WP2 - 1 gives suggested values for these factors.

(4) The velocities and the discharges (Q = A*V) are then calculated and a stage/discharge relationship or tailwater rating curve can then be determined.

37. The headwater rating curve can now be determined, using the broad crested weir formula, for a masonry and concrete weir:

$$Q = 1.7 * L * H^{1.5}$$

where:
Q = Discharge in cumecs
L = Length of the weir in metres and
H = Total enegry head above crest in metres

38. However, in order to relate the headwater curve to the tailwater rating curve the crest level of the weir must first be determined, as indicated above, i.e. for Sheeb it will be 41.3 m.

39. An approximation of the tailwater rating curve at the proposed diversion site at Sheeb is indicated in Table WP2 - 2 which also gives the headwater and tailwater levels as well as the canal, sluice and weir flows for wadi discharges up to about 700 cumecs.

40. The preliminary headwater and tailwater rating curves are shown at Figure WP2 - 2 including an assumed retrogressed downstream wadi bed level of about 1.0 m around the exit of the sediment sluice.

2. High Flood Level and Freeboard

41. The abutment walls should have a freeboard of 1.0 m above the 20 year high flood level, and the afflux for the 50 year flood should be checked against the design top level of the abutments. A preliminary estimate of the top level of the abutment walls is about El. 45.0 m, see Figure WP2 - 2. This must be checked for flood flows of 536 and 796 cumecs with the breaching bund completely removed.

3. The Breachable Diversion Bund

42. The breachable bund should slope from the top of the abutment between the weir and the breachable bund (El. 44.3 m) down to the upstream design high flood level (43.3 m). It should be made from the wadi bed material. Finer bed material should be selected for the length of bund rising from the design high flood level and any coarser material available, such as gravel, shingle and boulders should be used in the length adjoining the abutment to the weir. The upstream slope of the bund, generally, should be at 1 in 3 and the downstream slope at 1 in 2 with a top width of about 2 m maximum of uncompacted material.

4. Determination of Depths of Scour and Protection Works

43. The abutment between the weir and the breachable bund must be protected against depths of scour in accordance with Lacey's formula:

$$R = 1.35(q^2/f)^{1/3}$$

where:
R = Depth of scour in metres;
q = Discharge in cubic metres per second per metre run
f = Lacey's silt factor = $1.76(M_r)^{0.5}$
M_r = The average particle size of the boundary material in the wadi bed or channel, in mm.

44. Lacey's silt factor for different bed materials is given in Table WP2 - 3 and the relation between R and q for different values of silt factor f are given in Figure WP2 - 3.

45. The silt factor "f" may be determined by a grading analysis of about three fairly large samples of wadi bed material taken from the diversion site. Large samples will be required because of the boulders and cobbles seen at the diversion sites.

46. According to Lacey, the scour can be categorised in four classes as given below:

Recommended Scour Depths

Class A	Straight reach	1.25R
Class B	Moderate bend	1.50R
Class C	Severe bend	1.75R
Class D	Right angled bend	2.00R

47. The following assumptions and calculations will indicate how the design scour depth and necessary protection, required for the abutment between the weir and the breachable bund, may be determined for a flood with a return period of 20 years.

Assume Q_{20} = 536 m^3/s.
The Lacey width for this flow = 4.75 $Q^{0.5}$;
Thus the flow requires a width of 110 m.

Then q_{20} = 536 / 110 = 4.87 m^3/s per metre.

Assume 'f' = 4.75 for medium gravel (for heavy gravel 'f' = 9.0 and for small boulders 'f' = 12.0); an analysis of a large sample of the bed material at the site is required to check the silt factor 'f'. Then scour depth R = 2.31 m.
The design scour depth should be taken as 2R = 4.62 m below the estimated high flood level of about 43.3 m.
The elevation of the design scour depth = 43.3 - 4.62 = El. 38.68 m.

Assume foundation level of the abutment wall to be set at toe of downstream cutoff.
Elevation of toe of cutoff of the weir = 1.25R below tailwater level for the design flood;
then for q = 5.0 cumecs/m and f = 4.75, R = 2.35 m.
Tailwater level at 328 cumecs (weir + sluice) could be about el. 41.7 m. The tailwater rating curve has not yet been determined. Therefore toe of cutoff should be at:
41.7 - 1.25R = 41.7 - 2.94 = El. 38.76 m.

The toe of cutoff and the foundation level of abutment wall are very close and can both be set at 38.5 m.

48. The cut off depths upstream and downstream of the weir are determined in relation to the calculated depth of scour (R) given by the Lacey equation shown above.

49. For the design of the cut-offs it will be sufficient to take them down to a depth of scour equal to "R" on the upstream side and 1.25R downstream.

50. For the proposed Sheeb diversion weir the required cut off depths are shown below for a critical range of flood flows and assuming that the silt factor "f" = 4.75 at this site.

D/S Flow cumecs 1	Weir Flow cumecs 2	U/S WL El.m. 3	D/S WL El.m. 4	"R" (m.) 5	1.25R (m.) 6	U/S c/o'd' El.m. 7=3-5	D/S c/o'd' El.m. 8=4-6
310	290	43.3	41.6	2.4	3.00	40.9	38.6
550	530	44.0	42.6	3.1	3.9	40.9	38.7

51. For the design of flexible aprons upstream and downstream of the weir, the depth of scour to be guarded against should be 1.5R and 2R respectively to suit local concentrations of flow. These scour depths are measured below the flood levels to which they apply. If the upstream cut off is taken down to 1.5R (El. 39.7 m) the depth of scour will be guarded against and no flexible protection may be necessary upstream. However, this must be checked again for the 1 in 20 year flood of about 550 cumecs. The breaching bund will have been overtopped and the breach will be developing - deepening and widening. The upstream flood level could be about El. 44.0 m. Assuming a total energy head on the spillweir of (44.0 - 41.3) = 2.7 m, the intensity of discharge over the weir will be q_{u} = 7.54 $m^3/s/m$. Based on these assumptions, the weir will be discharging about (7.54 * 60) = 452 cumecs and about 100 cumecs will be passing through the breaching bund. For this case the required cutoff depth would be need to be at El. [44.0 - (1.5 * 3.1)] = El. 39.3 m.

52. Upstream and downstream scour depths at the diversion weir are shown in the Table below, for the design high flood level, assuming the Lacey silt factor "f" = 4.75, at the selected diversion site.

D/S Q cumec 1	Weir Q cumec 2	U/S WL El.m 3	D/S WL El. m 4	"R" m. 5	1.5*R m. 6	2.0*R m. 7	U/S D Scour El. m 8=3-6	D/S D scour El. m 9=4-7
310	290	43.3	41.6	2.4	3.6	4.8	39.7	36.8

53. Downstream protection works are usually provided by a concrete blockwork apron laid upon an inverted filter adjacent to the end sill, and then by a launching apron, see Figure WP2 - 4.

54. Loose launching aprons are provided to prevent the scour hole moving too close to the cut off wall and the downstream floor of the structure and thus causing it to be undermined. It is essential that the stone to be used in the launching apron should be large enough to remain in place during the peak floods. On the basis of experiments, the U.S. Bureau of Reclamation has recommended the following relationship between the velocity of flow and the mean diameter of the stone:

$$Va = 4.915\ (d)^{0.5}$$

where

Va = average velocity of flow in m/sec, and
d = mean diameter of the stone in metres.

55. The above equation holds good for stones with a specific gravity of 2.65, i.e. most stones.

56. Generally it is assumed that the stones launch at a slope of 1V to 2H. The length of the flexible apron should be at least 1.5 times the depth of scour below the downstream floor level. Assuming the downstream floor for Sheeb spillweir is set at about El. 39.0 m, then the length of downstream flexible apron should be 1.5*(39.0 - 36.8) = 3.3 m.

57. The thickness of the apron should be 2 times the mean diameter of the stone. As the maximum average velocity Va downstream of the weir could be about 2.5 m/s, then the mean diameter of the stone d = 0.26 metres. Adopt stone of mean diameter 0.30 metres and a minimum thickness of apron should be 0.60 m.

5. Stilling Basin Design

58. Stilling basin are required below hydraulic structures such as weirs, sediment excluder sluices, canal head regulators and other canal control structures to reduce high velocity flows to a velocity which will not cause

excessive erosion of the natural river bed or downstream channel. Without adequate energy dissipators there is a danger of undermining of structures and damage from waves in the channel downstream.

59. A stilling basin with a horizontal floor of appropiate length for a hydraulic jump is a very efficient means of dissipating energy. An estimate of the depth of flow, the velocity of flow and the *Froude number* is required to design the stilling basin. This is done by carrying out an energy balance between the top of the weir, where flow conditions are governed by the weir equation, and the and the downstream toe of the weir, neglecting the losses due to friction and turbulence between the two sections. The energy balance equation is expressed as follows:

$$D_1 + Z + (V_1/2g)^2 = D_2 + (V_2/2g)^2$$

60. The above dimensions and symbols are shown diagramatically on Figure WP2-5.

61. The left side of the energy balance equation can be determined since all the parameters are known, but the right side of the equation has to be solved iteratively, since D_2 and V_2 are interdependent.

62. Once the depth of flow and the velocity at the bottom of the weir slope have been determined, the the *Froude number* at the downstream toe of the weir can be calculated from the following equation:

$$F_2 = \frac{V_2}{(g \times D_2)^{0.5}}$$

63. The *Froude number* is a dimensionless parameter and is the ratio of inertia forces to gravity forces. When it is unity, the water is flowing at critical depth and thus a hydraulic jump cannot form.

64. The practical design of energy dissipaters is based on hydraulic principles, but because of the use of various baffles and other types of friction blocks to stabilize the hydraulic jump and shorten the length of the apron, most design of stilling basins is done empirically using the results of model studies.

65. The best reference is the Hydraulic Design of Stilling Basins and Energy Dissipaters, published by the USBR in 1983. The recommendations and stilling basins proposed in this reference are summarized below.

Basin Type 1.

This has a plain horizontal basin on which the hydraulic jump can form. There are no chute or baffle blocks, but a small end sill is sometimes provided. The formation of the jump on the stilling basin is governed by the following equation:

$$\frac{D_3}{D_2} = 0.5\{[1 + (8 \times F_2^2]^{0.5} - 1\}$$

The above equation is shown graphically on Figure WP2 - 6, which may be used as an alternative to the equation.

Having already determined D_2 and F_2, the conjugate depth of flow downstream of the jump, D_3, can now be calculated. This tailwater depth to the hydraulic jump will only occur if the water level in the channel downstream of the stilling basin, which is governed by the downstream sections and conditions, is at a higher level than D_3. If this is not the case, then the stilling basin floor level must be lowered and the calculations repeated until satisfactory conditions are achieved.

When a satisfactory jump has been determined, the graphs given on Figures WP2 - 7 can be used to calculate the length of the jump. Then the length of the stilling basin may be taken as the length of the jump as determined above multiplied by 1.2, to allow a factor of safety.

Basin Type 2

This type of basin is used on high spillways, large canal structures, etc. for Froude numbers above 4.5. The basin is equipped with chute blocks and a dentated end sill which allows a reduction in the basin length of about 33% over Basin Type 1.

Basin Type 3

This type is used on small spillways, outlet works and small canal structures where the velocity at the upstream end of the basin does not exceed 15 to 18 metres per second and the Froude number is above 4.5. The jump and stilling basin length is reduced by about 60% over Basin Type 1 with chute blocks, baffle piers and solid end sill.

Basin Type 4

This basin is used to reduce excessive waves created by imperfect jumps and is chosen when the Froude number is in the range 2.5 to 4.5.

66. A modified USBR Basin Type 1 incorporating a solid end sill is recommended for all three wadis, because of the severe abrasion that is likely to occur on the chute blocks and baffle piers, due to gravel, shingle and boulders in the bed load travelling at high velocity.

67. The following calculations are presented as a worked example and might possibly apply to the proposed site at Sheeb diversion weir:

Design max. head on weir = 2.0 m. = H

$q = 1.7 * (2)^{1.5} = 4.81$ cumecs/m

Approx. vel. of approach = V_1 = 4.81/2 = 2.4 m/sec.

Vel. of approach head = $(V_1)^2/2g$ = 0.29 m. = H_a

Discharge downstream of weir at high flood level (HFL) for weir plus sand sluice, (from Table WP2 - 2.1), = 290 + 20 = 310 cumecs.

Tailwater level for Q = 310 cumecs, from Table WP2 - 2.2 is say El. 41.5 m. This is a very rough approximation as a tail water rating curve has not been determined from wadi sections downstream of the proposed weir site as described in section 2.3 above.

Assume regression downstream of weir of only 0.5 m, as the breachable diversion bund feature, will allow movement of sediment past the structure for flood flows in excess of 335 cumecs, which will tend to replace downstream bed scour and stabilize the bed.

Therefore, assuming a design tailwater level = El. 41.0.

Try jump height D_2 = 1.5 * h = 3.0 m.

Thus, try floor level of stilling basin at 41.0 - 3.0 = El.38.0.

Weir crest level = 41.3 m
Stilling basin level = 38.0 m

Z = 3.3 m below weir crest level

Using the energy balance equation given above:

$D_1 + Z + (V_1)^2/2g = D_2 + (V_2)^2/2g$

$2.0 + 3.3 + 0.29 = 5.59 = Y_2 + (V_2)^2/2g$

Try $D_2 = 0.48$ m. Thus $V_2 = 4.81/0.48 = 10.02$ m/sec

$D_2 + (V_2)^2/2g = 0.48 + 5.12 = 5.60$, which is close enough to 5.59.

Adopt $D_2 = 0.48$ m.

Froude No. $F_2 = V_2/(g * D_2)^{0.5} = 4.62$

From Figure WP2 - 2.5, for Froude No. = 4.62:
The jump height = $D_3 = 6.0 * D_2 = 2.88$ m.

TWL = 38.0 + 2.88 = 40.88 < 41.0, the regressed TWL by 0.12 m. and less than the unregressed TWL of 41.5 by 0.62 m.

Adopt stilling basin floor level of El. 38.0.

Refering to Figure WP2 - 2.6, for the Type I basin, for a Froude No. = 4.62; the length of the stilling basin $L = 5.93 * D_3 = 17.1$ m. Allowing for a safety factor, the required length would be 17.1 m * 1.2 = 20.5 m.

Again, from Figure WP2 - 2.6, for a Type II basin, for $F_2 = 4.62$; Length of stilling basin $L = 3.80 * D_3 = 10.94$ m.

For the present, adopt the Type I basin generally. The Type II basin would be subject to severe abrasion that is likely to occur to the chute blocks and dentated end sill due to gravel, shingle and boulders in the bed load, travelling at high velocity. Maintenance of the chute blocks and dentated end sill would be difficult for the farmers.

5. Design of Weir for Sub-surface Flow

(a) Exit Gradient and Floor Length

68. The total length of impervious concrete floor can be determined by using Khosla's theory for safe exit gradient. The critical condition for seepage head usually occurs when water is at crest level of the weir and there is no water on the downstream side, but for the proposed diversion weir at Sheeb the head across the structure may increase at the design high flood level. This must be checked when the tailwater levels are determined, and guarded against for sub-surface flow.

69. A vertical cut off wall must be provided at both the upstream and downstream ends of the impervious floor. The proposed floor length and depth of downstream cut off must be checked for safe exit gradient.

70. At each point under the floor. There is a certain residual uplift pressure and a certain rate at which the head is being lost, indicating a pressure gradient. The pressure gradient at the exit point is called the exit gradient. The purpose of the design length of impervious floor is to restrict the exit gradient below the critical value so that the soil particles are not disturbed, i.e. go into suspension.

71. Khosla and his associates determined that for a standard form consisting of a floor of length b, with a downstream cut off of depth d, the exit gradient Ge, is given by the equation:

$$Ge = H/d * 1/[3.1416 * (F)^{0.5}]$$

where $F = [1 + (1 + (b/d)^2)^{0.5}]/2$

Ge = The exit gradient which should not exceed the figures given in the Table below.

H = The head difference across the structure in metres.

d = The depth of the downstream cutoff in metres.

b = The length of the structure in metres.

72. For a known value of H, the upstream head, the values of H/d and b/d can be determined and the exit gradient can be read off the graph at Figure WP2 - 1.6. It will be obvious after inspection of the above equation that if d = 0 the value of Ge becomes infinity. Thus it is essential that a vertical cut off should be provided at the downstream end of the floor.

73. To safeguard against piping and undermining of the floor, the exit gradient must not be allowed to exceed a certain safe limit for different soils as given below. The objective of the design is to keep the uplift pressures as low as possible, consistent with safety at the exit, in order to keep the floor thickness to a minimum. Thus the downstream cut off depth and an acceptable exit gradient determines the minimum overall length of the impervious floor.

Type of Soil	Safe Exit Gradient (Ge)
Very fine sand or slit	1/7 to 1/8
Medium & fine sand	1/6 to 1/7
Gravel & coarse sand	1/5 to 1/6
Boulders, shingle & gravel	1/4 to 1/5
Very hard clay to soft clay	1/2 to 1/3

74. A safe exit gradient at Sheeb diversion site, which has a wadi bed of boulders, cobbles, shingle, gravel and coarse sand, would be:

$$Ge = 1/5 = 0.20.$$

75. Thus, by trial and error and reference to the chart at Figure WP2 - 8, the floor length and depth of downstream cut off can be determined.

76. When H = 3.3 m and Ge = 0.20; read off value of b/d for given values of H/d and Ge on Figure WP2 - 8.

Cutoff Depth d (m)	H/d	b/d	Floor Length b (m)
1.5	2.20	24.0	36.0
2.0	1.65	13.0	26.0
2.5	1.32	8.0	20.0
3.0	1.10	5.0	15.0
3.5	0.94	3.5	12.3

77. A floor length of 20 m and a downstream cut off of 2.5 m would provide a safe exit gradient, Ge = 1/5 = 0.20. The floor length required for satisfactory energy dissipation and the cut off required to protect against scour depth must now be determined and checked for exit gradient. The upstream cut off provides an additional safety factor.

78. Now check for Lane's weighted creep ratio. Lane analysed a large number of structures on pervious foundations and evolved Lane's Weighted Creep Theory, where the weighted creep length (L_w) is given as

$$L_w = \frac{1}{3}N + V$$

where

N is the sum of all the horizontal contacts and all the sloping contacts less than 45°

V is the sum of all the vertical contacts plus the sloping contacts greater than 45°

79. To ensure safety against piping, (L_w) must not be less than C * H, where H is the seepage pressure head, i.e. the difference between the upstream and downstream water levels, while C is an empirical coefficient depending on the nature of the soil. Values of C - safe weighted creep ratios, for different kinds of soil are given in the table below.

Lane's Creep Coefficient

Material	Safe weighted creep ratio "C"
Very fine sand or silt	8.5
Fine sand	7.0
Coarse sand	5.0
Gravel and sand	3.5 to 3.0
Boulders, gravel and sand	2.5 to 3.0
Clayey soils	3.0 to 1.6

80. Checking the cross-section of the proposed diversion weir shown on Fig. WP2-9.

Sum of the vertical creep lengths V = 2 + 2 + 1.8 + 2 = 7.8 m.

Sum of the horizantal creep lengths N = 30.0 m.

Total creep length L_w = V + N/3 = 7.8 + 30.0/3 = 17.8 m

Differential head across the weir at the design high flood level, just before the breachable bund is cut would be (Hd) about 2.0 m.

Lane's weighted creep coefficient $C = L_w/Hd = 17.8/2.0 = 8.9$

Differential head across the weir with no flow over the weir, but with water at weir crest level and no flow downstream would be 3.3 m.

For this condition $C = 17.8/3.3 = 5.39$

	"C"
Boulders, gravel and sand	2.5 to 3.5

81. Thus, OK for both cases. A check must be carried out after an analysis of the wadi bed material at the proposed diversion site.

E. Uplift Pressures and Floor Thickness

82. The thickness of the floor is calculated on the basis of uplift pressure, which is caused by seepage beneath the hydraulic structure, due to a head of water upstream. The determination of the uplift pressure at the base of the structure is possible by several methods, four of which are:

(i) A graphical method,
(ii) Using Lane's weighted creep theory,
(iii) Distributing the head loss linearly down the upstream cutoff, between the toes of the upstream and downstream cutoffs, and up the downstream cutoff to the downstream bed level, and
(iv) Khosla's method of independent variables.

83. Khosla's method is the best of these, but it is a very complex theory involving lots of calculations. Khosla's curves have reduced this work, but still the theory is difficult to understand. Lane's theory and Method (iii) are very simple and requires very elementary calculations, which can be quickly checked by using a graphical method.

84. According to Lane's theory the uplift pressure at any point on the foundation can be computed from the formula:

$$Px = H\,[1 - L_x/L_w]$$

where Px = Uplift pressure on the foundation at point X, in feet of water.
H = Head of water on the base in feet.
L_w = Lane's weighted creep length, calculated as follows:
$L_w = 1/3\ N + V$
where N is the sum of all the horizontal contacts along the base and all the sloping contacts less than 45° and V is the sum of all the vertical contacts plus sloping contacts greater than 45°.
L_x = Weighted creep length up to point X.

85. Uplift pressures are usually greatest at the upstream end of the stilling basin of a weir, where there is little depth of water upstream of the hydraulic jump. The critical cases occur:

(1) when the upstream water level is at weir crest level, no flow over the weir and the stilling basin is dry; and

(2) at design design maximum flood flows, just before the breachable bund is overtopped.

86. Whilst the most critical section is at the toe of the weir slope, the uplift at several sections along the stilling basin should be checked so that the stilling basin floor thickness can be progressively reduced as the uplift pressure decreases. For rigorous analysis, the shape of the hydraulic jump should be plotted, according to the shape profiles given in USBR EM No 25 -

1983.

87. The following calculations are presented as a worked example for a preliminary design, to obtain approximate quantities and an estimate for the proposed diversion weir at Sheeb.

Check factor of safety (FoS) against uplift pressures, using Method (iii) above.

Consider five points, A to F shown on the cross-section of the weir at Figure WP2 - 9.

Total creep length = 2.3 + 30 + 3 = 35.3 m.

Creep length to A = 2.3 + 4.0 = 6.3 m. gr = 6.3/35.3 = 0.178
Creep length to B = 2.3 + 9.0 = 11.3 m. gr = 11.3/35.3 = 0.320
Creep length to C = 2.3 + 14.0 = 16.3 m. gr = 16.3/35.3 = 0.462
Creep length to D = 2.3 + 18.5 = 20.8 m. gr = 20.8/35.3 = 0.589
Creep length to E = 2.3 + 24.0 = 26.3 m. gr = 26.3/35.3 = 0.745
Creep length to F = 2.3 + 29.2 = 31.5 m. gr = 31.5/35.3 = 0.892

88. When the distribution of uplift pressures along the base of the weir has been calculated the uplift pressures and the resisting weights may be calculated at selected critical points. Using the symbols given in Figure WP2 - 2.8, the factor of safety against uplift may be calculated from the following:

$$\text{FoS} = \frac{\text{Weight of concrete and water on the weir}}{\text{Upward pressure from the seepage water}}$$

$$\text{FoS} = \frac{(d_c * t) + (d_w * H_2)}{[H_1 - \{(H_1 - H_3) * gr\}]d_w}$$

where:

d_c = density of concrete - 2.4 tonne/m^3
d_w = density of water - 1.0 tonne/m$_3$
t = thickness of stilling basin at selected point
gr = gradient of the equipotential lines, i.e. the number of equipotential lines upstream of the selected point divided by the total number of equipotential lines.
H_1, H_2 and H_3 are the heads as shown on Figure WP2 - 10.

89. The thickness of the base slab is then adjusted to give a factor of safety against uplift of between 1.1 and 1.3 depending on the degree of accuracy to which the soils and other parameters are known.

(1) Check FoS against uplift when no flow over the weir, but with the U/S water at weir crest level.

Point A.

$$\text{FoS} = \frac{(2.4 * 1.6) + (1.0 * 0)}{[2.5 - \{(2.5 - 0.9)0.178\}]1.0} = \frac{3.84}{[2.5 - 0.28]1.0} = \frac{3.84}{2.22} = 1.73$$

Therefore OK.

Point B.

$$\text{FoS} = \frac{(2.4 * 2.0) + (1.0 * 0)}{[5.3 - \{(5.3 - 2.0)0.32\}]1.0} = \frac{4.8}{[5.3 - 1.06]1.0} = \frac{4.8}{4.24} = 1.13$$

Therefore OK.

Point C.

$$FoS = \frac{(2.4 \;*\; 2.0) \;+\; (1.0 * 0)}{[5.3 - \{(5.13 - 2.0)0.462\}]1.0} = \frac{4.8}{[5.3 - 1.45]1.0} = \frac{4.8}{3.85} = 1.25$$

Therefore OK.

Point D.

$$FoS = \frac{(2.4 \;*\; 1.6) \;+\; (1.0 * 0)}{[4.9 - \{(4.9 - 1.6)0.589\}]1.0} = \frac{3.84}{[4.9 - 1.94]1.0} = \frac{3.84}{2.96} = 1.30$$

Therefore OK.

Point E.

$$FoS = \frac{(2.4 \;*\; 1.2) \;+\; (1.0 \;*\; 0)}{[4.5 - \{(4.5 - 1.2)0.745\}]1.0} = \frac{2.88}{[4.5 - 2.46]1.0} = \frac{2.88}{2.04} = 1.41$$

Therefore OK.

Point F.

$$FoS = \frac{(2.4 \;*\; 0.8) \;+\; (1.0 \;*\; 0)}{[4.1 - \{(4.1 - 0.8)0.892\}]1.0} = \frac{1.92}{[4.1 - 2.94]1.0} = \frac{1.92}{1.16} = 1.65$$

[illegible]

(2) Now check FoS against uplift when the design flood is passing over the weir, just before the breachable bund is overtopped or cut.

Point A.

$$FoS = \frac{(2.4 \;*\; 1.6) + (1.0 \;*\; 1.0)}{[4.5 - \{(4.5 - 2.6)0.178\}]1.0} = \frac{4.84}{[4.5 - 0.34]1.0} = \frac{4.84}{4.16} = 1.16$$

Therefore OK.

Point B.

$$FoS = \frac{(2.4 \;*\; 2.0) + (1.0 \;*\; 0.48)}{[7.3 - \{(7.3 - 2.48)0.32\}]1.0} = \frac{5.28}{[7.3 - 1.45]1.0} = \frac{5.28}{5.76} = 0.92$$

Therefore unsafe. Increase floor thickness to 3.0 m at point B and check again.

$$FoS = \frac{(2.4 * 3.0) + (1.0 * 0.48)}{[8.3 - \{(8.3 - 3.48)0.32]1.0} = \frac{7.68}{[8.3 - 1.54]1.0} = \frac{7.68}{6.76} = 1.14$$

[illegible]

Point C.

$$FoS = \frac{(2.4 \;*\; 2.0) \;+\; (1.0 * 3.0)}{[7.3 - \{(7.3 - 5.0)0.462\}]1.0} = \frac{7.8}{[7.3 - 1.06]1.0} = \frac{7.8}{6.24} = 1.25$$

Therefore OK.

Point D.

$$FoS = \frac{(2.4 \;*\; 1.2) \;+\; (1.0 * 4.2)}{[6.5 - \{(6.5 - 4.2)0.589\}]1.0} = \frac{7.08}{[6.5 - 1.35]1.0} = \frac{7.08}{5.15} = 1.37$$

[illegible]

Point E.

$$FoS = \frac{(2.4 \;*\; 0.8) \;+ (1.0 * 3.7)}{[6.1 - \{(6.1 - 3.7)0.745\}]1.0} = \frac{5.62}{[6.1 - 1.79]1.0} = \frac{5.62}{4.31} = 1.30$$

[illegible]

Point F.

$$FoS = \frac{(2.4 \;*\; 0.8) \;*\; (1.0 * 3.7)}{[6.1 - \{(6.1 - 3.7)0.892\}]1.0} = \frac{5.62}{[6.1 - 2.14]1.0} = \frac{5.62}{3.96} = 1.42$$

Therefore OK.

C. Canal Head Regulator

90. The canal head regulator (CHR) should be designed to pass the full supply discharge with the upstream level at weir crest level.

91. The order of magnitude of head losses are likely to be as follows:

From weir crest to upstream of the CHR	0.15 m
Total losses through the CHR	0.30 m
Total head loss from weir crest to FSL	0.45 m

92. The sill level of the CHR should be set about 1.3 m below the crest level of the weir. This would provide about 1.15 m depth on the sill which should be adequate to keep the loss in head through the regulator low, at the same time creating a skimming weir effect which would divert the top layer of the flow entering the intake bay when the sediment excluder sluice was in operation.

93. For Sheeb CHR the sill level has been set provisionally at El.40.0 m.

94. A preliminary determination of the full supply discharge for the Sheeb left bank primary canal, based on analyses of scheme water requirements and a seasonal flow duration, is given in Section E3, as 33 m^3/s.

95. Assuming that the canal head regulator full supply discharge of 33 m^3/s is adopted the dimensions of the head reach of the canal has been determined in Section D3, based on Simons and Albertson design table for Type 5 canal, Table 8A5. This indicates that a bed width of about 11.5 m and a water depth of about 1.59 m will be required, for a bed slope of 1 in 1416 and with side slopes of 1:1.3.

96. The width between abutments of the canal head regulator should be made approximately equal to the bed width of the canal, to avoid fluming as far as possible, and to provide the skimming weir effect with low velocities and maximum intake with low head loss, during small wadi flows.

97. For Sheeb CHR, with a design full supply discharge of 33.0 cumecs for a head on the sill of 1.20 m, the width of free opening required will be:

$$L = 33/1.7 * (1.15)^{1.5} = 15.7 \text{ m.}$$

98. Adopt 5 gate openings each 3.0 m wide, with 4 piers each 0.75 m wide; thus the total width between abutments will be 18.0 m.

99. The canal head regulator will have to conform to similar design criteria as the weir and the stilling basin floor level should be determined for full supply discharge under the following conditions:

(i) with the upstream water level at weir crest level; and
(ii) with the upstream water level at high flood level.

Condition (ii) may be critical and may determine the safe downstream floor level of the regulator.

100. For Sheeb CHR - Condition (ii):

$Q = 33$ cumecs; $q = 33/15 = 2.2$ cumecs/m.

Vel. of approach = $V_1 = 2.2/3.3 = 0.67$ m/sec.

Vel. head = $(V_1)^2/2g = 0.02$ m.

Assume maximum canal water level = $2/3 * 1.15$ m = 0.77 m above sill of

CHR at Q = 33 cumecs, i.e. El. 40.77 m.

Canal water depth = 1.70 m.approx. and assume height of end sill is about 0.40 m. Thus, try stilling basin level at 0.77 - 1.7 - 0.4 = -1.33 m. below sill level of CHR, i.e. stilling basin level = 40.0 - 1.33 = El. 38.67 m.

Again using the energy balance equation, which is expressed as follows:

$$D_1 + Z + (V_1/2g)^2 = D_2 + (V_2/2g)^2$$

Then: 3.3 + 1.33+ 0.02 = 4.65 = $D_2 + (V_2)/2g$

Try D_2 = 0.235 m; then V_2 = 2.2/0.235 = 9.36 m/sec.

$D_2 + (V_2)^2/2g = 0.235 + 4.46 = 4.70$

Froude No. F = $9.36/(g * 0.235)^{0.5} = 6.16$

From Figure WP2 - 6, for F = 6.16:
TW Depth = D_3 = 8.1 * D_2 = 1.90 m.

Canal WL = El. 40.77 minus stilling basin level of 38.67= 2.1 m > jump depth of 1.90 m.

Length of stilling basin from Figure WP2 - 7;
for F = 6.16 for Type II basin:

L = 4.2 * D_3 = 7.98 m.

iii. Now check for Sheeb CHR - Condition (i):

Q = 33 cumecs; q = 33/15 = 2.2 cumecs/m.

Vel. of approach = V_1 = 2.2/1.15 = 1.91 m/sec.

Vel. head = $(V_1)^2/2g$ = 0.186 m.

Assume maximum canal water level = 2/3 * 1.15 m = 0.77 m above sill of CHR at Q = 33 cumecs, i.e. at >El. 40.77 m.

Canal water depth = 1.70 m approx. and assume height of end sill is about 0.4 m. Thus, try stilling basin level at 40.77 - 1.77 - 0.4 = 38.67 m.

Again using the energy balance equation.

Then: 1.15 + 1.33 + 0.186 = 2.67 = $D_2 + (V_2)/2g$

Try D_2 = 0.33 m; then V_2 = 2.2/0.33 = 6.67 m/sec.

$D_2 + (V_2)^2/2g = 0.33 + 2.26 = 2.60$. Close enough.

Froude No. F = $6.67/(g * 0.33)^{0.5} = 3.71$

From Figure WP2 - 6, for F = 3.71;
TW Depth = D_3 = 4.8 * D_2 = 1.58 m.

Canal WL = El. 40.77 minus stilling basin level of 38.67 = 2.1 m > jump depth of 1.58 m.

Length of stilling basin from Figure WP2 - 7;
for F = 3.71 for Type I - natural jump:

$L = 5.7 * D_3 = 9.01$ m.

102. Thus, it will be acceptable to adopt a Type II stilling basin with a level at El. 38.6 and having a stilling basin length of 9.0 m.

103. Now refer to Figure WP2 - 11:

Height of dentated sill = $h_2 = 0.2*D_3 = 0.2 * 1.90 = 0.38$ m.

Width of dentation = $W_2 = 0.15 * D_3 = 0.15 * 1.90 = 0.285$ m.

Spacing of dentation = 0.29 m.

Height of chute blocks = 0.3 m.

Width of blocks = 0.3 m.

Spacing of blocks = 0.3 m.

104. The velocity at the end sill of the CHR must be checked and should give a Froude number of less than 0.35.

105. At the end sill of Sheeb CHR the Froude No. at condition (ii) is 0.29.

$$\text{Vel. at end sill} = \frac{33}{1.58 * 18} = 1.16 \text{ m/sec.}$$

$$\text{Froude No. } F = \frac{1.16}{(g * 1.58)^{0.5}} = 0.29$$

Therefore OK.

106. The downstream wing walls, the tops of which should slope down, at about 1V to 2H, to the canal bed should be joined to the canal section by a bowl-shaped quadrant in anticipation of unavoidable bank scour.

107. The canal head regulator should be set at an angle to the sediment excluder sluice for it to function effectively. The angle is partly dependent on the particular features of the site and may vary from about 30 to 60 degrees. This angle and the geometry of the approach channel to the headworks may best be determined by model studies if the magnitude of the works warrants the expenditure. As a guide, 40° should be generally acceptable on many sites.

D. Sediment Control Arrangements

1. Introduction

108. Flood irrigation systems have evolved, in some cases over centuries, to enable water to be diverted from unpredictable flashy flows carrying high sediment concentrations. The features of traditional intakes that enable them carry out this function successfully are :

1) The diversion spurs or bunds are cut or breached when the flows entering offtakes become excessive, and thus the very high sediment concentrations, carried by the larger flood flows, are not diverted to canals.

2) Canals have very steep slopes, and thus a high sediment transporting capacity.

3) There are no permanent structures in canals to pond or retard flows, thus minimising the possibility of sediment settling on the bed.

109. Serious sediment problems have been encountered in some schemes where existing traditional intakes have been replaced with raised weirs and gated canal intakes. Problems include loss of control over low flow channels, resulting from the inevitable rise in bed levels upstream from a permanent weir, and excessive canal sedimentation.

110. Traditional systems do not usually suffer from these problems. The alignment of diversion spurs can be adjusted to follow changes in the location of low flow channel, and bed scour in large floods maintains average bed levels. Canal sedimentation is minimised by the exclusion of high flows and the steep canal slopes.

111. The intake structures described herein are intended to retain these desirable features. However the introduction of a spillway section to the bund will enable farmers to divert water from higher river flows than in the past. As these higher flows carry higher sediment concentrations some form of control over both the size and the concentration of sediments admitted to canals is necessary.

112. The following section describes the sediment control structures that are proposed.

2. Need for sediment control

113. The sediment concentrations that can be transported by a channel reduce as the discharge is reduced, and also reduce as the channel slope becomes flatter. Thus when a proportion of the flow in a wadi is diverted to a canal, where typically both the discharge and the slope are smaller than the wadi, the reduction in sediment transporting capacity results in sediment settling to the canal bed. If sediment loads are high, siltation will rapidly start to reduce the canal capacity. The objective of sediment control is to match the size range and sediment concentration of the sediments that are admitted to canals to that which the canal can transport.

114. Transporting capacity of a channel also depends strongly on sediment size. For very fine sediments, the transporting capacity in traditional canals is very large and silts and clays are usually transported through to the fields. There is no need to control sediments in this size range, which are anyway regarded as a valuable source of fertiliser by farmers.

115. Coarse sediments, which can be transported at medium to large flows in the river, will deposit in the head reachs of canals, and should be excluded at the intake. The size of sediment that has to be excluded depends on the slopes and discharges of the canal and some other factors, but in spate schemes it is usually sufficient to exclude all sediments larger than coarse sand.

3. Sediment control structures

116. The intake proposed for spate schemes includes a curved channel sediment excluder. The purpose of the excluder is to divert the coarsest sediments, which move on or near the wadi bed, away from the canal intake.

117. The performance of the sediment excluder structure depends on both sediment sizes and hydraulic conditions in the wadi. It can be predicted, but the techniques involved are somewhat too complex for inclusion here.

118. The excluder will only be operated during periods when the wadi flow exceeds the canal flow. Thus intakes will often be operated for considerable periods without sediment exclusion. There is also the possibility that farmers will not operate the excluder at high flows, by not opening the sluiceway gate. Therefore some form of secondary sediment control is needed if canal sedimentation is to be minimised.

119. Canal sediment ejectors used in perennial irrigation systems are unacceptable for spate schemes as they require water to be wasted from the canal for sediment flushing, and would have control gates that would need to be adjusted during floods.

120. It is proposed that a simple sediment settling basin is provided at the head of primary spate canals. The purpose of the settling basin is to trap the coarse sediments that pass through the intake, before they settle in the canal system. The settling basin can be constructed by widening and deepening the canal reach at a point a little downstream from the intake, a distance of 10 times the canal width, where the turbulence introduced at the headworks has decayed. It would have to desilted at intervals.

121. The advantage of a basin, over the alternative of allowing sedimentation in the canal, is that some deposition can be accommodated before the flow capacity of the canal is reduced. The same quantity of sediment has to be removed from the canal in either case.

122. It is recommended that settling basins are included at all improved spate diversion works. The leading dimensions of the basin are determined by calculations which are too lenghty for inclusion in this Working Paper.

4. Sediment excluder sluice

123. As indicated above, the control of coarse sand, gravel and shingle which would be carried into the canals by water diverted from the heavy sediment laden flood flows, is a very important feature in the design and operation of any improved spate irrigation headworks. The problem is one of conducting as much as possible of the stream bed load material through the diversion structure, thereby avoiding its expensive removal from the canal.

124. The irrigation headworks should therefore incorporate a scouring sluiceway, between the weir and the canal head regulator, which is designed to provide good scouring action across the front of the regulator and to intercept as much as possible of the wadi bed load and some of the coarser fractions, carried in the lower layer of flood water being diverted for irrigation. The sluiceway will also assist the sloping weir in maintaining an approach channel to the canal head regulator.

125. The performance of this type of sediment excluder at spate diversion headworks on wadis can be considerably improved by designing the approach to the canal offtake and sediment sluice with a curved converging channel and positioning the weir at the upstream end of the guide pier. In addition to these features, any bank protection wall should be extended upstream to prevent any pronounced and unfavourable approach flows from that side. These findings were the result of model studies at the Hydraulics Laboratory, University of Southampton, UK, in 1978 and Figure WP2 - 12 illustrates the basis and outcome of the study.

126. The principle of the curved channel excluder is that the secondary currents develop a helicoidal flow pattern which cause a large proportion of the sediment close to the bed to move towards the convex wall of the channel and away from a relatively long skimming weir on the outer or concave wall of the channel over which the diverted supply must flow to the canal head regulator. The amount of sediment entering the canal is thereby reduced.

127. The most effective method of sediment exclusion is considered to be continuous sluicing adjacent to the canal head regulator in a direction about 30° to 60° to the diverted canal flow. This will not be feasible in the wadis of Eritrea as continuous sluicing is only possible when flows in the wadi exceed the full supply discharge of the canal and this only occurs for a small percentage of the time during the main irrigation season. However,

intermittent sluicing may be possible when wadi flows are less than the diversion requirement.

128. For efficient sluicing, the required sluice flow should be one-third to one-half of the canal flow, with the upstream water level at weir crest level. Sediment exclusion will be less effective at higher wadi flows of short duration. However, operational procedures could be instituted to reduce the intake of sediment, for example, by closing the gates of the canal head regulator for a short while.

129. The gate of the sediment excluder should be designed for a flow of about one half of the CHR discharge with the upstream water level at weir crest level. It is recommended that the minimum width of the gate should be 2.0 metres to reduce the possibility of the opening being blocked with trash. The gate height should be about 2.0 metres high so that all flows less than the CHR full supply discharge can be controlled up to weir crest level.

130. In order to preserve the curvature effects of the sluice channel and induce the secondary helicoidal flow pattern, design velocities should not be too low and thus depths of flow should not be too large. The floor level of the sluiceway should be set about 0.6 to 0.8 metres below the sill level of the canal head regulator and at average wadi bed level or about 0.5 metres above the level of the low flow channel.

131. Ideally, a headwall should ***not*** be constructed above the sediment sluice. Its omission will improve the throughput of sediment and trash. The scour sluice gate should be designed so that the bottom of the gate can be raised above the design high flood level and in the event of mal-function, for it to withstand the forces associated with spilling over the gate. However, gates such as those described above can be quite expensive and in order to economise it is sometimes necessary to provide headwalls and orifice gates.

132. The sediment excluder at the headworks gives no protection against the "wash load" which will eventually be transported to the fields, provided the canal system is designed with sufficient slope. Some of the coarser fraction, heavy sands and gravel, carried in the lower layer of flow, will also enter the head reach of the canal. The objective of the sediment excluder is to minimise this quantity of sediment, which ultimately has to be removed by oxen and scrapers or by mechanical excavation.

133. A similar series of calculations to those required for the design of the weir and the canal head regulator will be necessary for the design of the scouring sluice. In addition a check must be made to ensure that the downstream floor level of the energy dissipator is so designed that no damage will occur if the sediment sluice is left fully open at all stages of flood flows.

134. The following calculations apply to the design of the stilling basin of the proposed sediment excluder sluice at Sheeb:

135. The sluiceway should be about 4.0 m wide and would have 2 gates each 2.0 m wide and 1.5 m high and the sluice opening would have a headwall, to reduce the cost of the gates which would otherwise be required.

136. The formula generally adopted for submerged orifices on medium sized regulators is:

$$Q = C \times A\ (2*g*h)^{0.5}$$

where Q = Discharge in m^3/s;
A = Gate opening in m^2;
h = Differential energy head in metres; and
C = Coefficient of discharge.

137. The approximate values of C for sluice and regulator openings in masonry piers are:

Regulator openings up to 1.8 m wide with recesses in the piers	C = 0.62
Ditto between 1.8 m and 4.0 m wide	C = 0.72
Ditto over 4.0 m wide	C = 0.82

A value of C = 0.72 was adopted for the sluice orifices.

The estimated differential head on sluice gates at the design HFL = 1.55 m = h

138. Design sluice discharge at HFL assuming that the gate is left fully open would be:

$Q = 0.72 * 4.43 * 4.0 * 1.5 * (1.55)^{0.5} = 23.82$ cumecs.
say 24.0 cumecs

139. However, if the orifices are not submerged and the supercritical flow pushes the water downstream, away from the openings, then the head on the gates will (4.0 - 0.75) = 3.25 m and the discharge at HFL assuming the gates are left fully open would be:

$Q = 0.72 * 4.43 * 4 * 1.5 * (3.25)^{0.5} = 34.5$ cumecs.

Provide for higher discharge of Q = 34.5 cumecs.

q = 34.5/4 = 8.62 cumecs/m.

Approx. vel. of approach = V_1 = 8.62/4 = 2.16 m/sec.

Vel. of approach head = $(V_1)^2/2g$ = 0.24 m. = ha

Discharge downstream of weir at high flood level (HFL) for weir plus sand sluice, (from Table WP2 - 2), = 288 + 34.5 = 322.5 cumecs.

Tailwater level for Q = 322 cumecs, from Table WP2 - 2 is El. 41.6 m.

140. Assume regression downstream of weir of only 0.3 m, as breachable diversion bund feature, will allow movement of sediment past the structure for flood flows in excess of 330 cumecs, which will tend to replace downstream bed scour and stabilize the bed.

Therefore, design tailwater level = El. 41.30 m.

Try jump height D_3 = 4.5 m.

Thus, try floor level of stilling basin at 41.3 - 4.5 = El. 36.8 m, say El. 36.5.

Sill level of sluice = 39.3
Stilling basin level = 36.5
Z = 2.8 m below sill level of sluice

Using energy balance equation:

$D_1 + Z + (V_1)^2/2g = D_2 + (V_2)^2/2g$

$4 + 2.8 + 0.11 = 6.91 = D_2 + (V_2)^2/2g$

Try D_2 = 0.79 m. Thus V_2 = 8.62/0.79 = 10.91 m/sec

$D_2 + (V_2)^2/2g = 0.79 + 6.07 = 6.86$, which is close enough to 6.91.

Adopt $D_2 = 0.79$ m.

Froude No. $F_2 = V_2/(g * D_2)^{0.5} = 3.92$

From Figure WP2 - 2.5, for Froude No. = 3.92:
The jump height = $D_3 = 5.0 * D_2 = 3.95$ m.

TWL = 36.50 + 3.95 = 40.45 < 41.3, the regressed TWL by 0.85 m.

Adopt stilling basin floor level at El. 36.5 m.

Refering to Figure WP2 - 7, for the Type I basin, for a Froude No. = 3.92; the length of the hydraulic jump, L = 5.78 * D_3 = 22.83 m. Therefore, adopt a stilling basin floor length of 1.2 * 22.83 m = 27.4 m, to allow a factor of safety.

141. Now check sediment excluder design for normal maximum operational discharge at weir crest level, i.e. 0.6 * 33 = 19.8 cumecs, say 20.0 cumecs.

Design max. head on sill of sluice = 2.0 m. = h

q = 20/4 = 5.0 cumecs/m

Approx. vel. of approach = V_1 = 5.0/2 = 2.5 m/sec.

Vel. of approach head = $(V_1)^2/2g$ = 0.32 m. = ha

Discharge downstream of weir = 20.0 cumecs.

Tailwater level for Q = 20.0 cumecs, from Table WP2 - 2 is El. 39.4 m.

Assume regression downstream of weir of 0.6 m. Therefore, design tailwater level = El. 38.8 m.

Try jump height D_3 = 2.3 m.

Floor level of stilling basin = 38.8 - 2.3 = 36.5 m.
Adopt El. 36.5 m.

Sill level of sluice = 39.3
Stilling basin level = 36.5
Z = 2.8 ft below weir crest level

Using energy balance equation

$D_1 + Z + (V_1)^2/2g = D_2 + (V_2)^2/2g$

$2 + 2.8 + 0.32 = 5.12 = D_2 + (V_2)^2/2g$

Try D_2 = 0.53m. Thus V_2 = 5.0/0.53 = 9.43 m/sec

$D_2 + (V_2)^2/2g = 0.53 + 4.54 = 5.07$, which is close enough to 5.12.

Adopt D_2 = 0.53 m.

Froude No. $F_2 = V_2/(g * D_2)^{0.5} = 4.14$

From Figure WP2 - 2.5, for Froude No. = 4.14:
The jump height = $D_3 = 5.25 * D_2 = 2.78$ m.

TWL = 36.5 + 2.78 = 39.28 > 38.8, the regressed TWL by 0.48 m. and would be less than the unregressed TWL of 39.4 by 0.12 m.

142. The stilling basin floor level still needs further review.

143. Calculations for uplift pressure, exit gradient, scour depths, flexible protection, etc. should carried out on a similar basis to the weir.

144. It may be necessary to line the scouring sluice sill, walls and glacis with stone to reduce abrasion of the concrete by cobbles and gravel travelling at high velocities. At Sheeb and Wadi Labka this kind of damage would be quite severe.

145. An outline plan showing the geometry of a typical spate irrigation diversion works with a curved channel sediment excluder of concrete and masonry construction for Sheeb is shown in Figure WP2 - 13. Typical cross-sections of the proposed flood diversion works for Sheeb are shown at Figures WP2-14 and WP2-15.

5. Sediment sluiceway design details

a) Velocities in the curved channel

146. The leading dimensions of the intake structure will have been set by the procedures described earlier. The width of the sluice channel at the gate section will have been set so that the sluice discharge is between 0.3 and 0.6 times the canal discharge. In this section the procedure for checking the entry and exit widths for the curved channel is described. This is necessary to ensure that:

a) The mean velocity through the curved channel does not reduce towards the gate. (A mild increase is in fact desirable.)

b) The mean velocity in the curved channel is at least equal to the mean velocity in the river upstream.

147. All calculations are carried out for 2 cases, firstly where the water level is at weir crest level, the sluice gate is fully open, and all the river flow is passing through the intake structure, and secondly when the water level is at bund breaching level. This is usually the weir crest elevation plus about 1.5 to 2.0 m.

148. The following procedure is suggested:

a) Select a representative cross section for the river upstream from the intake location.

b) Carry out a slope area calculation to determine the mean velocity at the cross section when the discharge equals the sum of the sluice and canal discharge.

c) Calculate the mean velocity at the sluice gate section of the curved channel. This is calculated from the width of the curved channel, the depth of flow, and the sluice discharge when the gate is fully open.

d) Check that the velocity calculated in (c) is equal to and preferably larger than the velocity calculated in (b).

e) Repeat the procedure set out in (c) for a location halfway along the skimming weir, where the discharge can be assumed to be the sluice discharge plus half the canal discharge, and at the entrance section where the discharge is the sum of the canal and sluice discharges.

The widths of these sections can be adjusted as required to ensure that the velocity in the curved channel does not reduce in the downstream direction.

E. Primary Flood Canal

1. Approach to design

149. The design of spate irrigation canals and distributaries present some unusual problems because of the need to take every opportunity to divert supplies whenever the wadi is in flood. There are large fluctuations in the daily wadi flows which necessitate designing canals for a varying discharge rather than the more usual steady flow state.

150. The canal design must provide satisfactory transport of a highly variable sediment load without causing significant scour. The choice of shape and bed slope are key factors in the design.

151. Whereas, the opening for the canal head regulator should be designed for the full supply discharge, the dimensions of the canal should be designed for about 70% of the full supply discharge, because significant scour should not occur during the relatively short time that the canal flow is in the range of 70% to 100% of the full supply discharge. The sediment deposited during the lower range of flows will tend to heal any tendency to scour.

152. When new designs have to be prepared experience from spate irrigation systems in the Yemen suggests that the slopes should be based on the canals maximum discharge capacity reduced by a factor of around 0.7. This is because flood canals operate for most of the time at discharges that are much smaller than their maximum capacity. The cross section, particularly the provision of free-board, has to be adequate for the maximum discharge, but if the slope is also set by the maximum discharge will too small at the most frequent operating discharge, and sediment deposition will occur. (Larger slopes are required to maintain "regime " in canals as the discharge is reduced.) As very high flows are usually only maintained for short periods, and will be carrying high sediment loads, there is little chance of serious scour problems occurring.

153. It is not anticipated that the canals in existing traditional irrigation systems will be redesigned. However in some schemes canals will have to be extended, and in others the existing canals may not be capable of passing the required flows.

154. Where existing canals are performing satisfactorily the design of new canals should be based on the slopes and cross sections of existing canals, providing they are to carry similar discharges. If the discharge capacity is to be changed then the design tables presented in the next section can be used. However in this case survey data should be used to select the design table that mostly closely predicts exist conditions.

155. A rejection spillway, to deal with excessive discharges which could enter the primary canal, could be installed, if necessary, in the head reach, at an appropriate site downstream of the CER.

156. Finally, if new canal control structures are to be provided they must be designed so that ponding of flows is minimised to so as to reduce local sedimentation. The basic requirements for standard canal control structures are that they should be fairly simple in design, flexible in operation, accurate in measurement of flows and capable of transporting its proportion the sediment load.

2. Design Tables

157. Five canal design tables based on the Simons and Albertson, 1960, regime equations are presented as Tables SA1 to SA5.

158. The Simons and Albertson equations were developed from a much wider data set covering a range of canal types than the more familiar Lacey equations, and have a number of other advantages. The most important of these is that they distinguish between canals carrying moderate and high sediment loads, and successfully predict the slopes of existing stable traditional canals. (The Lacey equations were developed for canals carrying no more than 400 ppm of sediment.)

159. The tables present the following canal parameters as a function of discharge:

b, the mean width, (m)

d, the canal depth, (m)

v, the mean velocity, (m/sec)

S, the canal slope, (m/m)

ss, the side slope, (m/m)

Manning's "n", derived from the predicted velocity, hydraulic radius and slope.

160. Tables for five canal types are listed:

Type 1, Sand bed and banks

Type 2, Sand bed and cohesive banks

Type 3, Cohesive bed and cohesive banks

Type 4, Coarse noncohesive material

Type 5, As type 2 but with heavy sediment load, 2000 to 8000 ppm.

161. The table for type 5 canals is the most generally applicable for conditions on the Eastern Lowlands of Eritrea, where sediment concentrations could possibly exceed 8000 ppm. If canal beds are formed from very coarse sediments, ie gravel cobbles and boulders, then the type 4 tables should be used. The type 3 table may be appropriate for canals running through cohesive sediments and generally are not applicable in the Sheeb and Wadi Labka areas.

162. Guidance as which table is most appropriate is best obtained by comparing predicted slopes and widths with those obtained from surveys of existing canals. (The discharge of existing canals can be estimated from slope area calculations using flow depths for maximum flows estimated by farmers.)

3. Sheeb Primary Canal Design

163. A preliminary estimate of the full supply discharge of Sheeb left bank primary canal is determined by the formula:

$$Q = \frac{2.77 * A * W}{E * N}$$

where Q = design full supply discharge in m^3/s;
A = net commanded area in hectares;

W = average irrigation application depth in metres;
E = overall irrigation efficiency of the whole system;
N = the average number of hours per crop season, that the wadi flows in excess of the design canal discharge.

164. Information obtained by the Mission from farmers in the area and from officials of the MoA concerning the frequency and duration of floods in Wadi Laba is summarised below:

- About 15 floods occur at the proposed diversion site at Wadi Laba in an average year.
- About 5 to 8 floods occur in a dry year at this site.
- The flood flows run for about 8 hours and the recession flows last for about 48 hours.
- In 1977 there were 20 floods in the period July to September in Wadi Laba - 8 big floods and 12 medium size floods.
- In 1984 there were 15 floods in July - September in Wadi Laba - 10 big floods and 5 medium size floods.
- An analysis of the duration of flood flows entering Foro Dam indicates that the median duration (50% probability) is 11 hours.

165. Field inspections of the primary irrigation canal for the Sheeb area, indicated that the capacity was of the order of 40 cumecs.

166. In order to obtain a preliminary estimate of the full supply discharge for Sheeb left bank main canal the following assumptions have been made:

- a net commanded area on the left bank of 1710 hectares, for the Bizurs, Errem, Dogoli and Difa areas.
- a peak water requirement of 0.44 metres applied to this area in two or three applications, during the period July to September.
- an overall irrigation efficiency of 30%,
- and a flow duration of 210 hours (15 floods flowing on average for 14 hours), at or near the design discharge, in the main flood season July to September.

then the canal full supply discharge should be

$$Q = \frac{2.77 * 1710 * 0.44}{0.3 * 210} = 33.1 \text{ cumecs}$$

say 33 cumecs

167. Thus the canal should be designed for 23.1 cumecs (33 * 0.7).

168. The sediment excluder sluice should have a capacity of about 20 cumecs (33 * 0.6) when the upstream water level is at weir crest level and the minimum width of the gate should be 2 metres to reduce the possibility of the intake bay being blocked with trash.

169. The dimensions of the canal, immediately downstream of the canal intake, adopting a Simons and Albertson Type 5 canal, are given below.

Full Supply Discharge (FSD)	= 33 cumecs
Canal Design Discharge	= 23.1 cumecs
Bed Width (b)	= 11.5 m

Canal Slope for 23.1 cumecs = 0.000706 (1 in 1,416)

Actual Slope in Head Reach = 0.010 (1 in 100)

Side slope = 1 in 1.3

Flow Depth (y) at 23.1 cumecs = 1.40 m

Mean Velocity (v) = 1.29 m/s

Using Manning's equation for the channel dimensions adopted above then:

Flow Depth (y) at 33 cumecs = 1.70 m

Mean Velocity (v) = 1.42 m/s

Manning's "n" = 0.023

Bed Level (40.0 + (1.15 * 0.667) - 1.7) = El. 39.07

170. The primary canal will supply both the Bizurs/Errem and the Dogoli/Difa command areas. A flow division structure will be required a short distance downstream of the settling basin to provide about 20.0 cumecs to Bizurs/Errem and about 15.0 cumecs to Dogoli/Difa areas.

171. The flow distributor could take the form of two meter flumes, with the crest level of the Bizurs/Errem outlet approximately 0.10 m lower than that of Dogoli/Difa. Assuming a 1.3 m depth of flow on the crest of the Bizurs outlet and 1.2 m on Dogoli, then four gates each about 2 m wide and 1.3 m high may be required for Bizurs and three gates each 2 m wide and 1.2 m high could be required for Dogoli/Difa. The differences in heights and widths will have to be adjusted when the irrigable command areas and flow durations have been finally determined.

4. Wadi Labka Primary Canal Design

172. Information obtained by the Mission from farmers in the Wadi Labka area and from officials of the MoA concerning the frequency and duration of floods in Wadi Labka is summarised below:

- Water flows in the wadi for about 45 days;
- Big floods run for about 5 hours and the flow takes about 7 hours to recede. The flood rises to its peak in about 15 to 30 minutes;
- On average 8 to 10 big floods occur each year and the diversion works break and have to be repaired about 7 times a year.

173. In order to obtain a preliminary estimate of the full supply discharge for Wadi Labka main canal the following assumptions have been made:

- a net commanded area on the right bank of 1090 hectares, for the Gamahir, Abarara and Atombasa areas.
- a peak water requirement of 0.44 metres applied to this area in two or three applications, during the period July to September.
- an overall irrigation efficiency of 25%,
- and a flow duration of 150 hours, assuming and average of 9 big floods and 4 medium size floods flowing on average for about 12 hours, at or near the design discharge, in the main flood season July to September.

Then the canal full supply discharge should be

$$Q = \frac{2.77 * 1090 * 0.44}{0.25 * 150} = 35.4 \text{ cumecs}$$

say 35 cumecs

174. Thus the canal should be designed for 24.5 cumecs (35 * 0.7).

175. The sediment excluder sluice should have a capacity of about 21 cumecs (35 * 0.6) when the upstream water level is at weir crest level and the minimum width of the gate should be 2 metres to reduce the possibility of the intake bay being blocked with trash.

176. The dimensions of the Wadi Labka canal, immediately downstream of the canal intake, adopting a Simons and Albertson Type 5 canal, are given below.

Full Supply Discharge (FSD)	= 35 cumecs
Canal Design Discharge	= 24.5 cumecs
Bed Width (b)	= 11.9 m
Canal Slope for 24.5 cumecs	= 0.000696 (1 in 1,437)
Actual Slope in Head Reach	= 0.010 (1 in 100)
Side slope	= 1 in 1.3
Flow Depth (y) at 23.1 cumecs	= 1.43 m
Mean Velocity (v)	= 1.30 m/s

Using Manning's equation for the channel dimensions adopted above then:

Flow Depth (y) at 33 cumecs	= 1.73 m
Mean Velocity (v)	= 1.43 m/s
Manning's "n"	= 0.023

Bed Level (46.6 + (1.2 * 0.667) - 1.73) = El. 45.67

177. The primary canal will supply both the Gamahir, Abarara and the Atombasa command areas. A flow division structure will be required some 200 metres downstream of the canal head regulator to provide about 25.0 cumecs to the Abarara/Atombasa areas and about 15.0 cumecs to Gamahir area.

5. Mai Ule Primary Canal Design

178. Virtually no data was obtained conserning flood flows at Mai Ule and therefore in order to obtain a preliminary estimate of the full supply discharge for Mai Ule right bank main canal the following assumptions have been made:

- a net commanded area on the right bank of 720 hectares, for the Khorfotat and Tiluk areas.
- a peak water requirement of 0.44 metres applied to this area in two or three applications, during the period July to September.
- an overall irrigation efficiency of 30%,
- and a flow duration of 130 hours, assuming 11 floods flowing on average for about 12 hours, at or near the design discharge, in the main flood season July to September.

Then the canal full supply discharge should be

$$Q = \frac{2.77 * 720 * 0.44}{0.3 * 130} = 22.5 \text{ cumecs}$$

179. Thus the canal should be designed for 15.75 cumecs (33 * 0.7).

180. The sediment excluder sluice should have a capacity of about 13.5 cumecs (22.5 * 0.6) when the upstream water level is at weir crest level and the minimum width of the gate should be 2 metres to reduce the possibility of the intake bay being blocked with trash.

181. The dimensions of the canal, immediately downstream of the canal intake, adopting a Simons and Albertson Type 5 canal, are given below.

Full Supply Discharge (FSD)	= 22.5 cumecs
Canal Design Discharge	= 15.75 cumecs
Bed Width (b)	= 9.38 m
Canal Slope for 15.75 cumecs	= 0.000773 (1 in 1,294)
Actual Slope in Head Reach	= not known
Side slope	= 1 in 1.33
Flow Depth (y) at 15.75 cumecs	= 1.22 m
Mean Velocity (v)	= 1.22 m/s

Using Manning's equation for the channel dimensions adopted above then:

Flow Depth (y) at 22.5 cumecs	= 1.46 m
Mean Velocity (v)	= 1.36 m/s
Manning's "n"	= 0.023

182. The primary canal will supply both the Khorfotat, Tiluk and the Tiluk Talai command areas. Two flow division structure will be required a short distance downstream of the canal head regulator to provide about 17.5 cumecs to Tiluk and about 5.0 cumecs to Khorfotat areas.

III. OPERATION OF THE DIVERSION HEADWORKS

183. A headworks attendant, chosen by the farmers committee, should be appointed for the maintenance and operation of the irrigation headworks. He should reside at or near the headworks in a house provided for him, and built as an essential part of the project, at the same time as the diversion weir is built. He should be provided with a smallholding adjacent to the house.

184. The headworks attendant should be present whenever there are flood flows in the wadi. He should receive proper training in operation procedures from MoA, as he will be a very important person for achieving an effective flood spreading scheme and minimising the sediment intake into the system.

185. Much of the success of the irrigation scheme will depend on the operation of the diversion works which can best be described for the following conditions of flow.

186. *Condition 1:* When flow in the Wadi is less than the design full supply

discharge (FSD), the gates of the canal head regulator (CHR) will each be raised by an equal height to obtain the maximum discharge with the least loss of head through the structure. All the gates should be raised clear of the water surface; this will reduce velocities and head losses for this condition. The design FSD will occur when the upstream (u/s) level is at weir crest level and all gates are open sufficiently to raise the water level in the canal to the mark on the gauge post downstream (d/s) indicating the full supply level (FSL).

187. *Condition 2:* When the wadi flow increases to a discharge which is equal to or less than the design FSD of the CHR and the sediment sluice, then the sediment excluder sluice should be opened to provide continuous sluicing across the front of the CHR without reduction in the u/s water level, which should be kept at weir crest level.

188. *Condition 3:* When the wadi flow exceeds the design FSD of the CHR and the sediment sluiceway together, the weir will be overtopped and the gates of the CHR should both be lowered to adjust the openings to provide the required discharge or the FSD. The gate of the sluiceway should be kept fully open.

189. *Condition 4:* When the wadi flow greatly exceeds the design FSD of the CHR and sediment sluiceway, the CHR should be closed for a few hours until the peak flow passes. The sediment load will be very high and much trash may be carried by the flood, therefore the sluice gate should be fully opened and lifted clear of the water surface and the CHR should be closed while the peak flow passes.

190. Hydraulic model tests, besides optimising the approach channel geometry,would also assist greatly in determining the best operational procedures for restricting to a minimum the entry of sediment into the canal system.

TABLE WP2 - 1

ROUGHNESS COEFFICIENTS FOR MANNING'S EQUATION

(Ref. V T Chow)

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5$$

Factors	Channel Conditions		Values
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010-0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very high		0.050-0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

SHEEB SPATE IRRIGATION SCHEME
INTAKE FRONTWATER CALCULATION SPREADSHEET

DATA METRIC UNITS

INVERT OF SAND SLUICE	39.30	(M)
INVERT OF CANAL INTAKE	40.00	(M)
WEIR CREST LEVEL	41.30	(M)
WIDTH OF SAND SLUICE	4.00	(M)
WIDTH OF CANAL INTAKE	15.00	(M)
WEIR CREST LENGTH	60.00	(M)
Cd CANAL INTAKE	1.70	
Cd SLUICE	1.70	
Cd WEIR	1.70	
CANAL CUT OFF LEVEL	41.20	(M)
SLUICE GATE OPENED LEVEL	41.30	(M)
SLUICE GATE ABOVE WS	40.80	(M)
CANAL RISE ABOVE CUT OFF	4.00	(M3/S PER METRE)

STAGE	HEAD CANAL	HEAD SLUICE	HEAD WEIR	Q CANAL	Q SLUICE	Q WEIR	QTOT	TW LEVEL		Q (W +S)	Dif.Head Sluice
39.30	-0.70	0.00	-2.00	0.00	0.00	0.00	0.00	38.80	0.00	0.00	0.50
39.80	-0.20	0.50	-1.50	0.00	0.00	0.00	0.00	38.80	0.00	0.00	1.00
40.30	0.30	1.00	-1.00	4.19	0.00	0.00	4.19	38.80	0.00	0.00	1.50
40.80	0.80	1.50	-0.50	18.25	0.00	0.00	18.25	38.80	0.00	0.00	2.00
41.30	1.30	2.00	0.00	33.92	19.23	0.00	53.15	39.40	33.92	19.23	1.90
41.80	1.80	2.50	0.50	35.92	26.88	36.06	98.86	40.00	35.92	62.94	1.80
42.30	2.30	3.00	1.00	37.92	35.33	102.00	175.25	40.60	37.92	137.33	1.70
42.80	2.80	3.50	1.50	39.92	44.53	187.39	271.83	41.20	39.92	231.91	1.60
43.30	3.30	4.00	2.00	41.92	54.40	288.50	384.82	41.75	41.92	342.90	1.55
43.80	3.80	4.50	2.50	43.92	64.91	403.19	512.02	42.30	43.92	468.10	1.50
44.30	4.30	5.00	3.00	41.80	76.03	530.01	647.83	42.85	41.80	606.03	1.45
44.80	4.80	5.50	3.50	47.92	87.71	667.89	803.52	43.40	47.92	755.60	1.40
45.30	5.30	6.00	4.00	49.92	99.94	816.00	965.86	43.90	49.92	915.94	1.40

Table WP2 - 2

Table WP2 - 3

Lacey's Silt Factor for Different Bed Materials

Lacey's silt factor for different materials

Type of material	Size of grain in mm	Silt factor "f"
Silt :		
Very fine	0.052	0.40
Fine	0.081	0.50
Fine	0.120	0.60
Medium	0.158	0.70
Standard	0.323	1.00
Sand :		
Medium	0.505	1.25
Coarse	0.725	1.50
Bajri & Sand :		
Fine	0.888	1.75
Medium	1.290	2.00
Coarse	2.422	2.75
Gravel :		
Medium	7.280	4.75
Heavy	26.100	9.00
Boulders :		
Small	50.100	12.00
Medium	72.500	15.00
Large	183.800	24.00

WADI LEBKHA SPATE IRRIGATION SCHEME
INTAKE FRONTWATER CALCULATION SPREADSHEET

DATA METRIC UNITS

INVERT OF SAND SLUICE	46.00	(M)
INVERT OF CANAL INTAKE	46.60	(M)
WEIR CREST LEVEL	48.00	(M)
WIDTH OF SAND SLUICE	4.00	(M)
WIDTH OF CANAL INTAKE	15.00	(M)
WEIR CREST LENGTH	125.00	(M)
Cd CANAL INTAKE	1.70	
Cd SLUICE	1.70	
Cd WEIR	1.70	
CANAL CUT OFF LEVEL	47.85	(M)
SLUICE GATE OPENED LEVEL	48.00	(M)
SLUICE GATE ABOVE WS	47.50	(M)
CANAL RISE ABOVE CUT OFF	4.00	(M3/S PER METRE)

STAGE	HEAD CANAL	HEAD SLUICE	HEAD WEIR	Q CANAL	Q SLUICE	Q WEIR	QTOT	TW LEVEL		Q (W + S)	Dif.Hea Sluice
46.00	-0.60	0.00	-2.00	0.00	0.00	0.00	0.00	45.50	0.00	0.00	0.5
46.50	-0.10	0.50	-1.50	0.00	0.00	0.00	0.00	45.50	0.00	0.00	1.0
47.00	0.40	1.00	-1.00	6.45	0.00	0.00	6.45	45.50	0.00	0.00	1.5
47.50	0.90	1.50	-0.50	21.77	0.00	0.00	21.77	45.50	0.00	0.00	2.0
48.00	1.40	2.00	0.00	36.24	19.23	0.00	55.47	45.80	36.24	19.23	2.2
48.50	1.90	2.50	0.50	38.24	26.88	75.13	140.25	46.30	38.24	102.01	2.2
49.00	2.40	3.00	1.00	40.24	35.33	212.50	288.07	46.80	40.24	247.83	2.2
49.50	2.90	3.50	1.50	42.24	44.53	390.39	477.15	47.30	42.24	434.91	2.2
50.00	3.40	4.00	2.00	44.24	54.40	601.04	699.68	47.90	44.24	655.44	2.1
50.50	3.90	4.50	2.50	46.24	64.91	839.98	951.13	48.50	46.24	904.89	2.0
51.00	4.40	5.00	3.00	41.80	76.03	1104.18	1222.01	49.10	41.80	1180.21	1.9
51.50	4.90	5.50	3.50	50.24	87.71	1391.43	1529.38	49.70	50.24	1479.14	1.8

Table WP2 - 4

TABLE SA1

SIMONS AND ALBERTSON CANALS

Design Table for Type 1 Canal
(Sand Bed and Banks)

Discharge Q (m^3/s)	Bed Width b (m)	Flow Depth d (m)	Av. Velocity v (m/s)	Canal Slope S (m/m)	Side Slope ss (m/m)	Manning Coef. 'n'
0.25	1.942	0.383	0.238	0.000228	2.375	0.027
0.50	3.021	0.492	0.263	0.000184	2.059	0.027
0.75	3.849	0.569	0.276	0.000163	1.918	0.027
1.0	4.547	0.632	0.289	0.000149	1.835	0.027
1.5	5.718	0.731	0.306	0.000131	1.738	0.026
2.0	6.705	0.811	0.319	0.000120	1.683	0.026
2.5	7.572	0.878	0.329	0.000112	1.647	0.026
5.0	10.987	1.127	0.362	0.000090	1.572	0.025
7.5	13.605	1.305	0.384	0.000080	1.549	0.025
10.0	15.812	1.447	0.399	0.000073	1.542	0.025
12.5	17.757	1.568	0.412	0.000068	1.541	0.024
15.0	19.515	1.674	0.423	0.000064	1.543	0.024
17.5	21.132	1.770	0.432	0.000061	1.547	0.024
20.0	22.636	1.857	0.440	0.000059	1.552	0.024
25.0	25.387	2.012	0.454	0.000055	1.562	0.024
30.0	27.873	2.149	0.466	0.000052	1.573	0.023
35.0	30.159	2.272	0.476	0.000049	1.584	0.023
40.0	32.287	2.383	0.485	0.000047	1.595	0.023
50.0	36.176	2.595	0.499	0.000044	1.608	0.023
60.0	39.693	2.729	0.516	0.000043	1.651	0.023

TABLE SA2

SIMONS AND ALBERTSON CANALS

Design Table for Type 2 Canal
(Sand Bed and Cohesive Banks)

Discharge Q (m^3/s)	Bed Width b (m)	Flow Depth d (m)	Av. Velocity v (m/s)	Canal Slope S (m/m)	Side Slope ss (m/m)	Manning Coef. 'n'
0.25	1.273	0.356	0.346	0.000384	2.362	0.022
0.50	2.074	0.456	0.381	0.000310	2.032	0.022
0.75	2.689	0.528	0.403	0.000274	1.859	0.022
1.0	3.208	0.586	0.420	0.000251	1.761	0.022
1.5	4.078	0.678	0.445	0.000222	1.644	0.022
2.0	4.811	0.752	0.463	0.000203	1.575	0.022
2.5	5.457	0.814	0.477	0.000189	1.529	0.022
5.0	7.992	1.045	0.526	0.000153	1.422	0.021
7.5	9.973	1.201	0.557	0.000135	1.382	0.021
10.0	11.577	1.342	0.580	0.000124	1.363	0.021
12.5	13.021	1.454	0.598	0.000116	1.352	0.020
15.0	14.327	1.552	0.614	0.000109	1.346	0.020
17.5	15.528	1.641	0.627	0.000104	1.343	0.020
20.0	16.646	1.722	0.639	0.000100	1.342	0.020
25.0	18.689	1.866	0.659	0.000093	1.343	0.020
30.0	20.536	1.992	0.676	0.000088	1.346	0.020
35.0	22.235	2.106	0.691	0.000084	1.350	0.020
40.0	23.816	2.210	0.704	0.000081	1.355	0.020
50.0	26.705	2.395	0.726	0.000076	1.365	0.019
60.0	29.317	2.557	0.746	0.000071	1.370	0.019

TABLE SA3

SIMONS AND ALBERTSON CANALS

Design Table for Type 3 Canal
(Cohesive Bed and Cohesive Banks)

Discharge Q (m^3/s)	Bed Width b (m)	Flow Depth d (m)	Av. Velocity v (m/s)	Canal Slope S (m/m)	Side Slope ss (m/m)	Manning Coef. 'n'
0.25	0.973	0.299	0.487	0.000567	2.737	0.017
0.50	1.650	0.384	0.536	0.000455	2.301	0.017
0.75	2.170	0.444	0.568	0.000400	2.100	0.017
1.0	2.608	0.492	0.591	0.000366	1.978	0.017
1.5	3.343	0.570	0.626	0.000321	1.832	0.017
2.0	3.963	0.632	0.651	0.000293	1.745	0.016
2.5	4.509	0.685	0.672	0.000273	1.686	0.016
5.0	6.650	0.879	0.740	0.000220	1.546	0.016
7.5	8.294	1.017	0.784	0.000193	1.490	0.016
10.0	9.680	1.128	0.816	0.000176	1.460	0.016
12.5	10.900	1.223	0.842	0.000164	1.443	0.015
15.0	12.004	1.305	0.864	0.000155	1.432	0.015
17.5	13.019	1.380	0.882	0.000148	1.424	0.015
20.0	13.964	1.448	0.899	0.000141	1.420	0.015
25.0	15.690	1.569	0.928	0.000132	1.415	0.015
30.0	17.251	1.675	0.952	0.000124	1.414	0.015
35.0	18.686	1.771	0.972	0.000119	1.415	0.015
40.0	20.022	1.858	0.991	0.000114	1.417	0.015
50.0	22.463	2.014	1.022	0.000106	1.423	0.015
60.0	24.671	2.150	1.048	0.000100	1.430	0.014

TABLE SA4

SIMONS AND ALBERTSON CANALS

Design Table for Type 4 Canal
(Coarse noncohesive material)

Discharge Q (m^3/s)	Bed Width b (m)	Flow Depth d (m)	Av. Velocity v (m/s)	Canal Slope S (m/m)	Side Slope ss (m/m)	Manning Coef. 'n'
0.25	0.640	0.186	0.983	0.010687	4.233	0.025
0.50	1.179	0.239	1.083	0.009067	3.513	0.026
0.75	1.593	0.276	1.147	0.008235	3.179	0.027
1.0	1.942	0.306	1.194	0.007692	2.975	0.027
1.5	2.528	0.354	1.263	0.006987	2.728	0.028
2.0	3.021	0.393	1.315	0.006526	2.579	0.028
2.5	3.456	0.426	1.357	0.006189	2.478	0.028
5.0	5.162	0.546	1.495	0.005251	2.228	0.028
7.5	6.471	0.632	1.583	0.004769	2.127	0.028
10.0	7.575	0.701	1.646	0.004455	2.063	0.028
12.5	8.547	0.760	1.700	0.004225	2.026	0.028
15.0	9.426	0.812	1.744	0.004046	2.001	0.028
17.5	10.235	0.858	1.782	0.003901	1.982	0.028
20.0	10.987	0.900	1.816	0.003779	1.969	0.028
25.0	12.362	0.975	1.873	0.003584	1.951	0.028
30.0	13.605	1.042	1.922	0.003433	1.941	0.028
35.0	14.748	1.101	1.964	0.003309	1.935	0.028
40.0	15.812	1.155	2.001	0.003206	1.932	0.028
50.0	17.757	1.252	2.064	0.003041	1.931	0.028
60.0	19.515	1.337	2.118	0.002912	1.933	0.028

TABLE SA5

SIMONS AND ALBERTSON CANALS

Design Table for Type 5 Canal
(Type 2 with heavy sediment load, 2000 to 8000 ppm)

Discharge Q (m^3/s)	Bed Width b (m)	Flow Depth d (m)	Av. Velocity v (m/s)	Canal Slope S (m/m)	Side Slope ss (m/m)	Manning Coef. 'n'
1.25	0.603	0.275	0.685	0.002067	2.850	0.020
1.50	1.127	0.353	0.754	0.001753	2.363	0.021
1.75	1.529	0.408	0.798	0.001592	2.136	0.022
2.0	1.868	0.453	0.831	0.001487	1.997	0.022
2.5	2.437	0.524	0.880	0.001351	1.829	0.022
3.0	2.917	0.581	0.916	0.001262	1.726	0.022
3.5	3.339	0.629	0.945	0.001197	1.658	0.022
5.0	4.997	0.808	1.041	0.001015	1.488	0.023
7.5	6.269	0.935	1.102	0.000922	1.415	0.023
10.0	7.341	1.037	1.147	0.000861	1.374	0.023
12.5	8.286	1.123	1.184	0.000817	1.348	0.023
15.0	9.140	1.200	1.214	0.000782	1.331	0.023
17.5	9.925	1.268	1.241	0.000754	1.318	0.023
20.0	10.656	1.331	1.264	0.000731	1.308	0.023
25.0	11.992	1.442	1.304	0.000693	1.295	0.023
30.0	13.200	1.540	1.338	0.000664	1.286	0.023
35.0	14.311	1.628	1.367	0.000640	1.283	0.023
40.0	15.345	1.708	1.393	0.000620	1.279	0.023
50.0	17.234	1.850	1.437	0.000588	1.279	0.023
60.0	18.942	1.976	1.474	0.000563	1.280	0.023

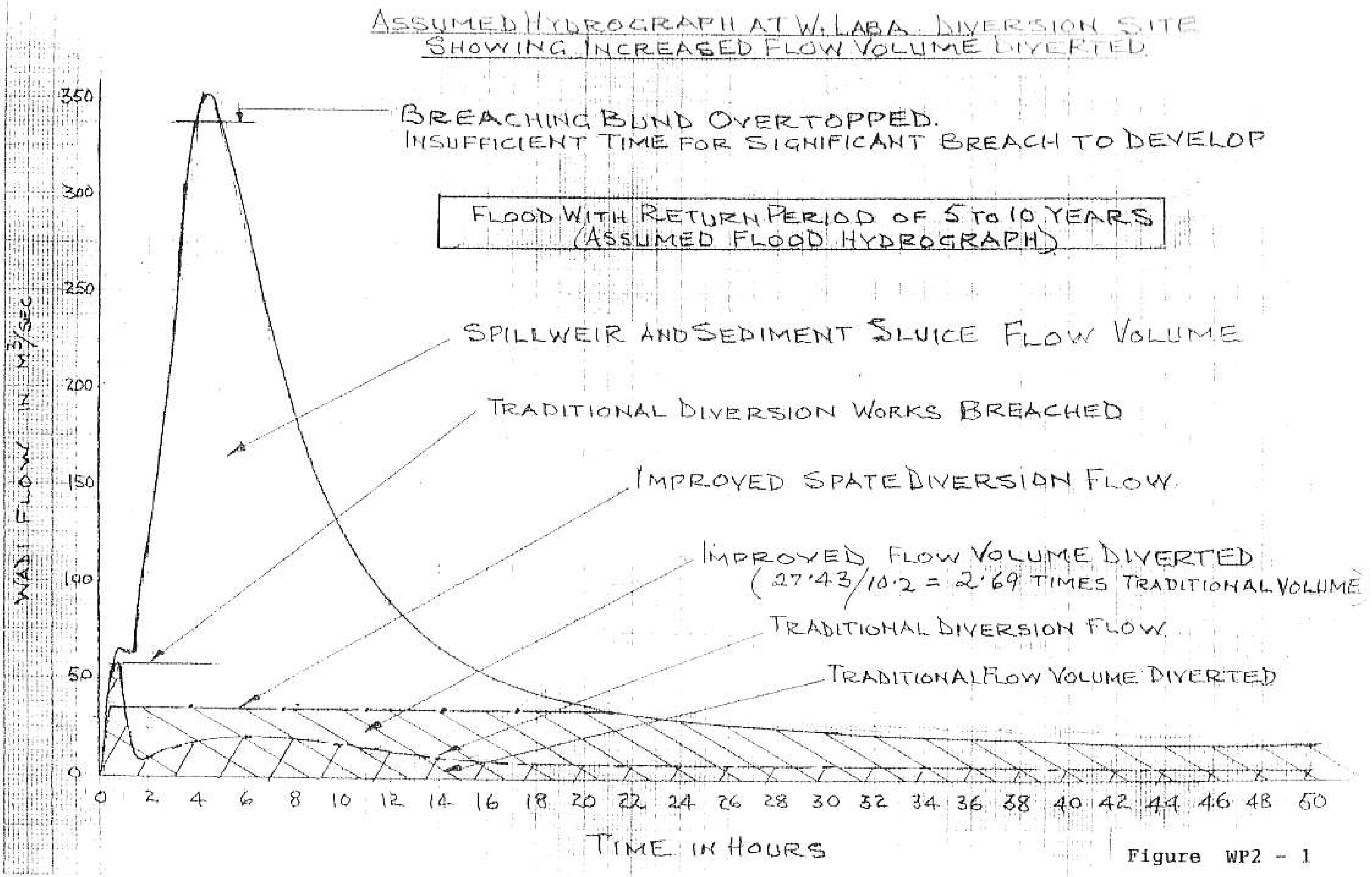

ASSUMED HYDROGRAPH AT W. LABA. DIVERSION SITE
SHOWING INCREASED FLOW VOLUME DIVERTED
BREACHING BUND OVERTOPPED.
INSUFFICIENT TIME FOR SIGNIFICANT BREACH TO DEVELOP
FLOOD WITH RETURN PERIOD OF 5 TO 10 YEARS
(ASSUMED FLOOD HYDROGRAPH)
SPILLWEIR AND SEDIMENT SLUICE FLOW VOLUME
TRADITIONAL DIVERSION WORKS BREACHED
IMPROVED SPATE DIVERSION FLOW.
IMPROVED FLOW VOLUME DIVERTED
(27'43/10'2 = 2'69 TIMES TRADITIONAL VOLUME
TRADITIONAL DIVERSION FLOW.
TRADITIONAL FLOW VOLUME DIVERTED
WADI FLOW IN M3/SEC
350
300
250
200
150
100
50
0
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50
TIME IN HOURS

Figure WP2 - 1

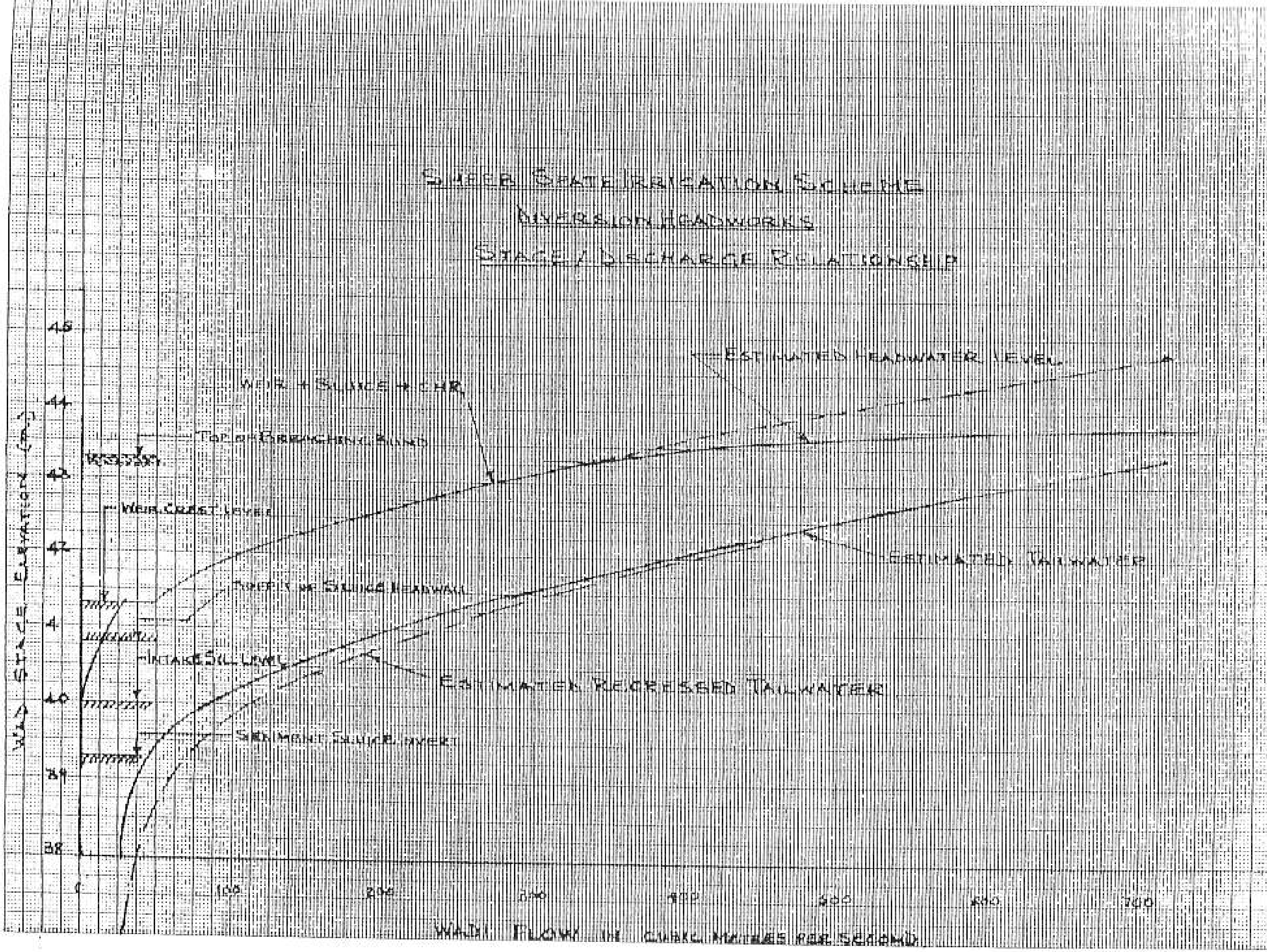

SHEEB STATE IRRIGATION SCHEME
DIVERSION HEADWORKS
STAGE / DISCHARGE RELATIONSHIP
ESTIMATED HEADWATER LEVEL
WEIR + SLUICE + CHR
TOP OF BREACHING BUND
WEIR CREST LEVEL
TOP OF SLUICE HEADWALL
INTAKE SILL LEVEL
ESTIMATED TAILWATER
ESTIMATED REGRESSED TAILWATER
SEDIMENT SLUICE INVERT
WADI STAGE ELEVATION (m)
38
40
42
44
0
100
200
300
400
500
600
700
800
WADI FLOW IN CUBIC METRES PER SECOND

Figure WP2 - 2

Relation between R and q for different values of f

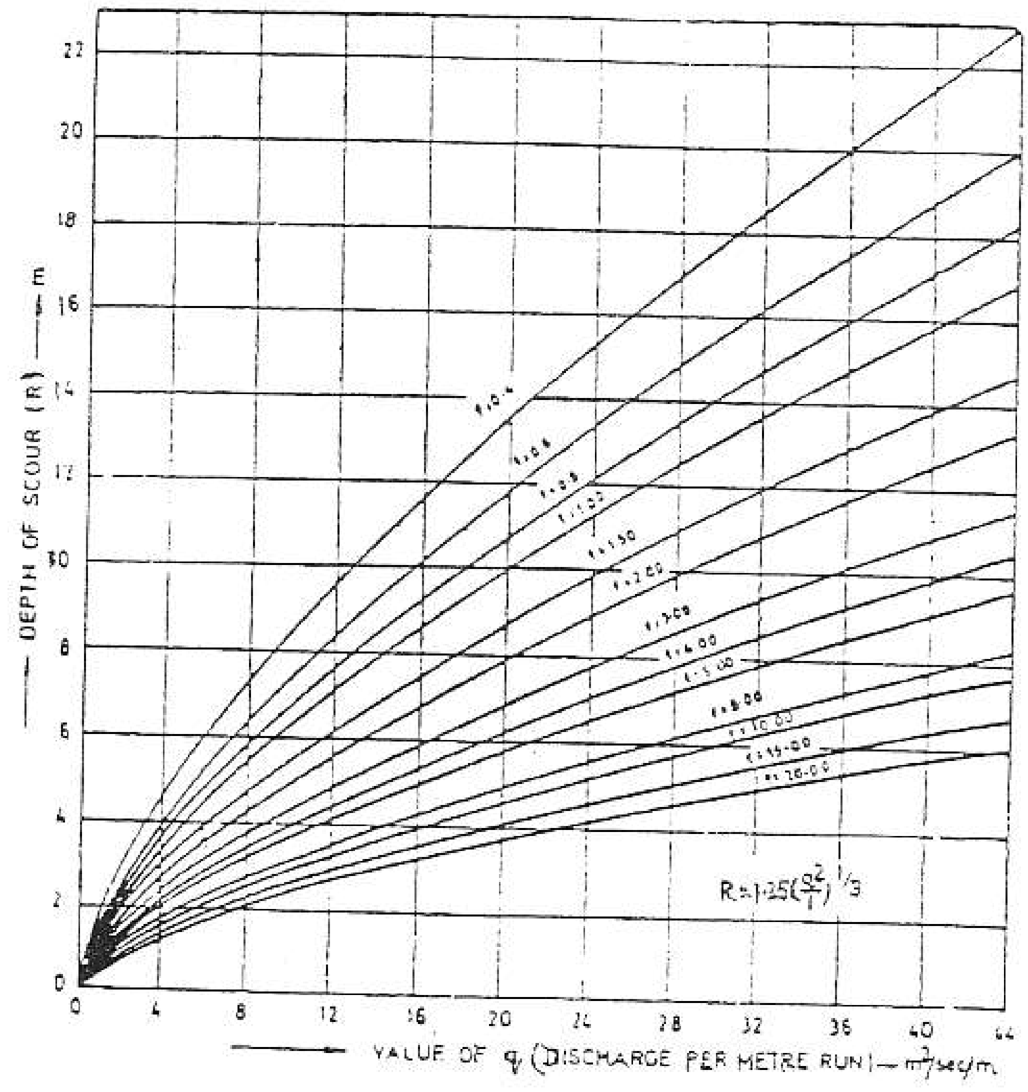
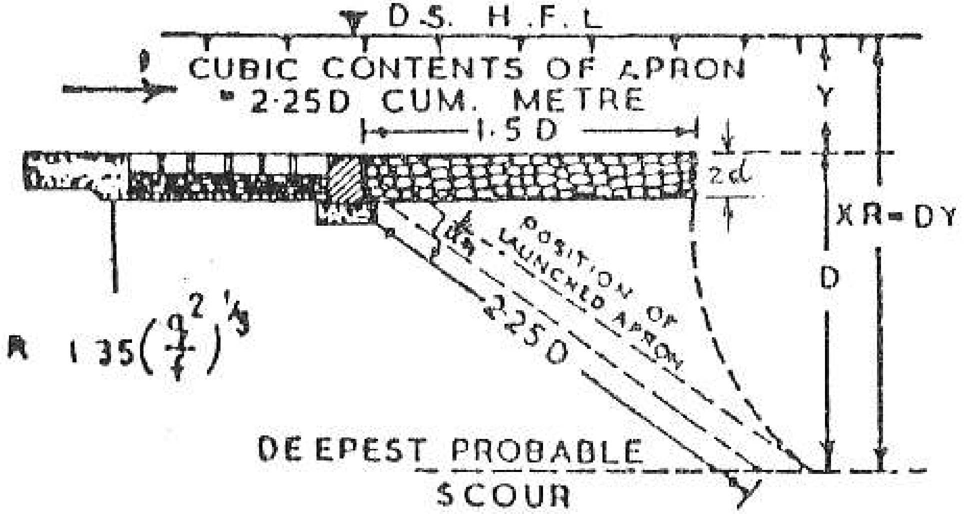


Figure WP2 - 3



Downstream Protection and Launching Apron

Figure WP2 - 4

FLOW OVER WEIR

DIMENSIONS AND SYMBOLS USED IN ENERGY BALANCE EQUATION

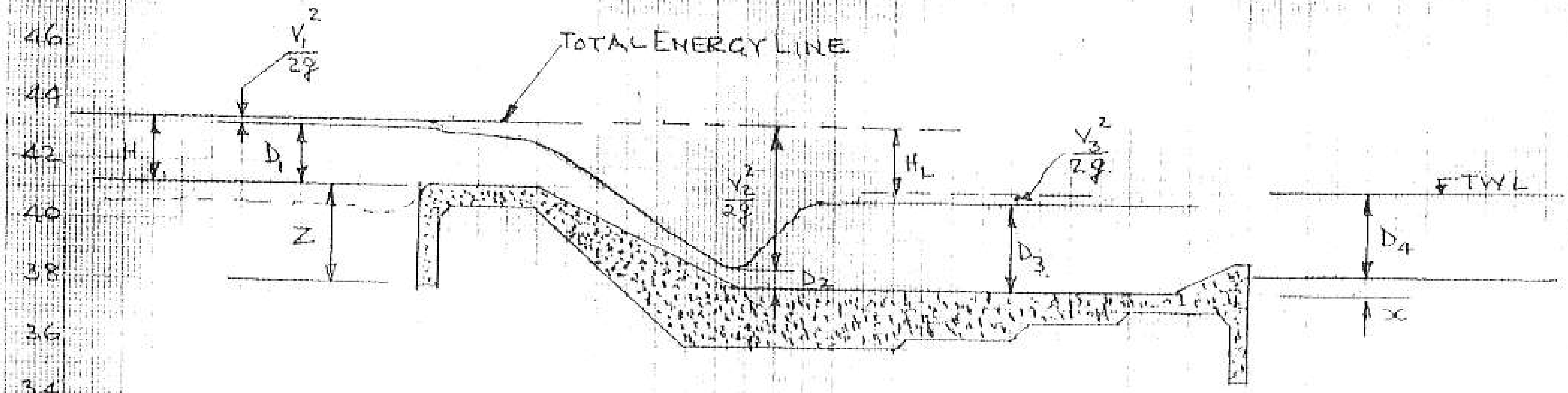
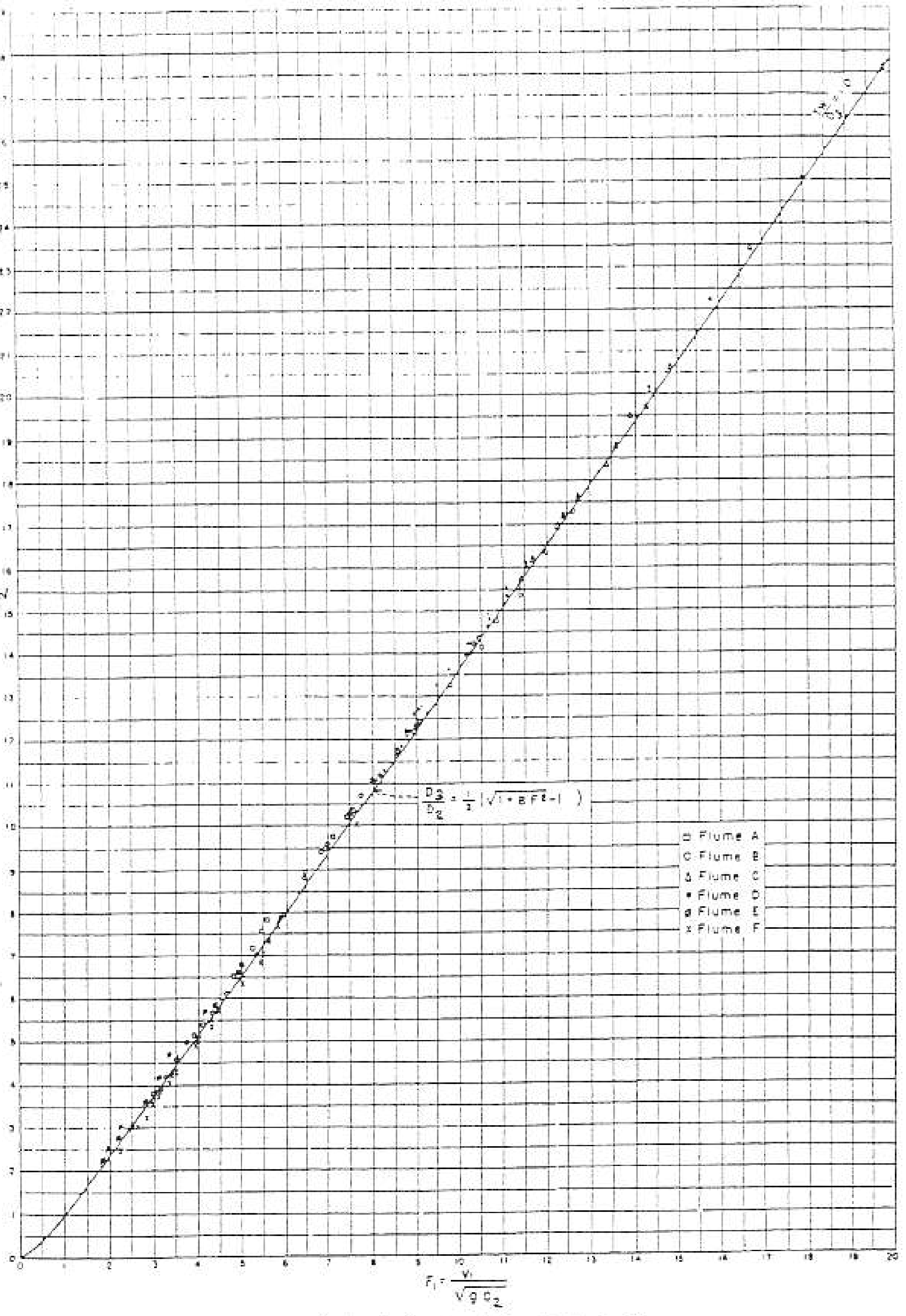
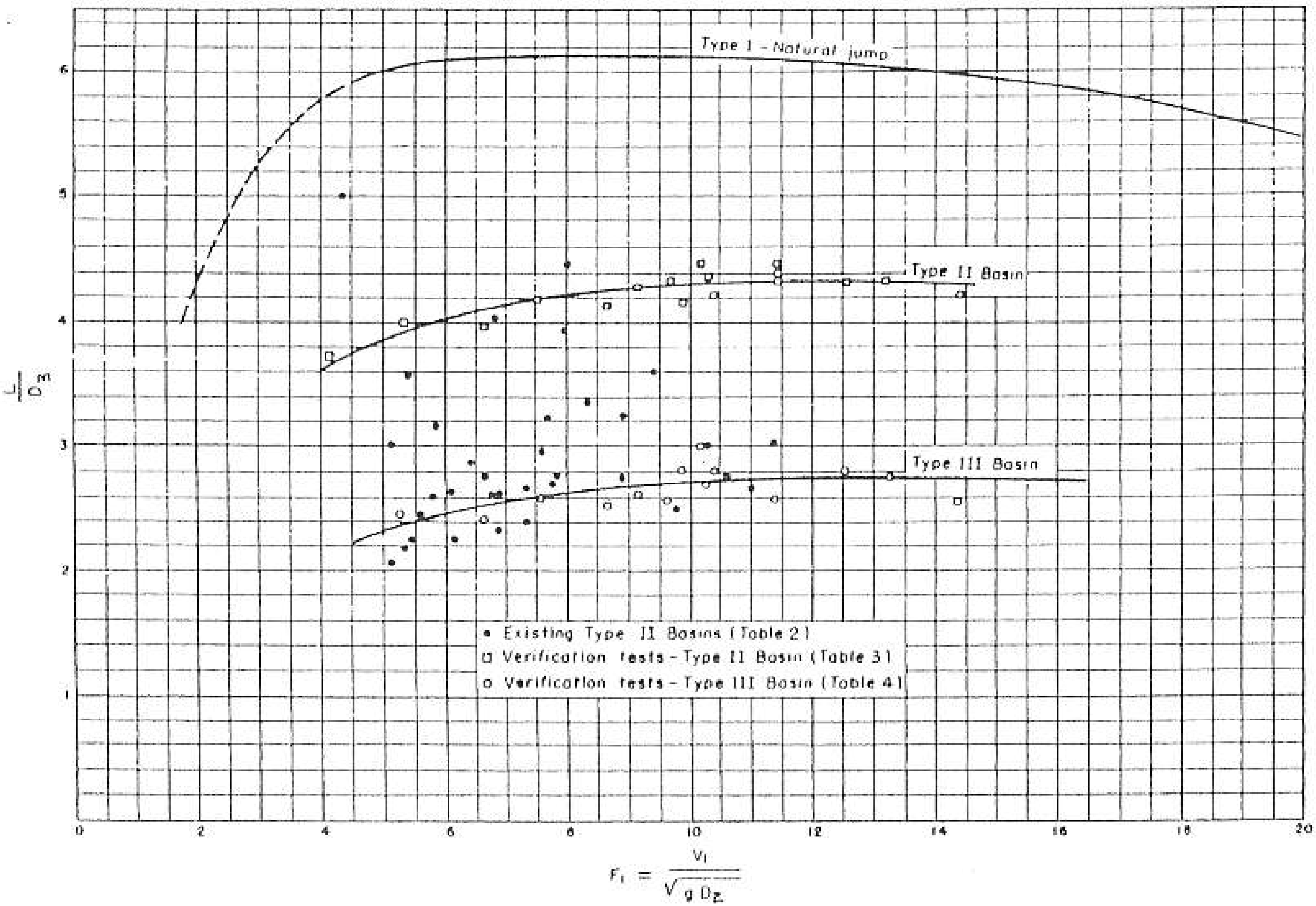


FIGURE WP2-5



Ratio of tail water depth to D_2 (Basin I).

Figure WP2 - 6



Length of jump on horizontal floor (Basins I, II, and III).

Figure WP2 - 7

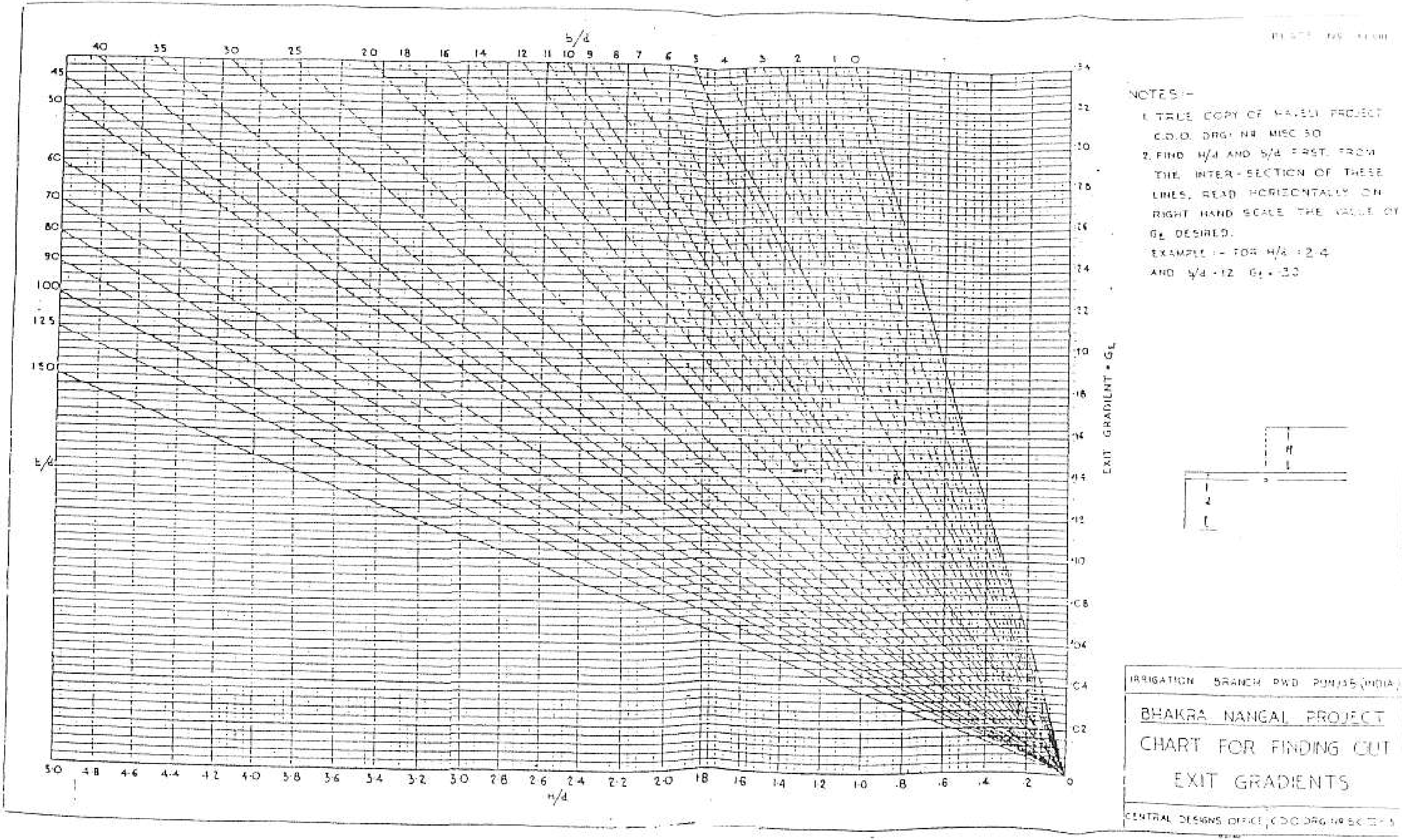
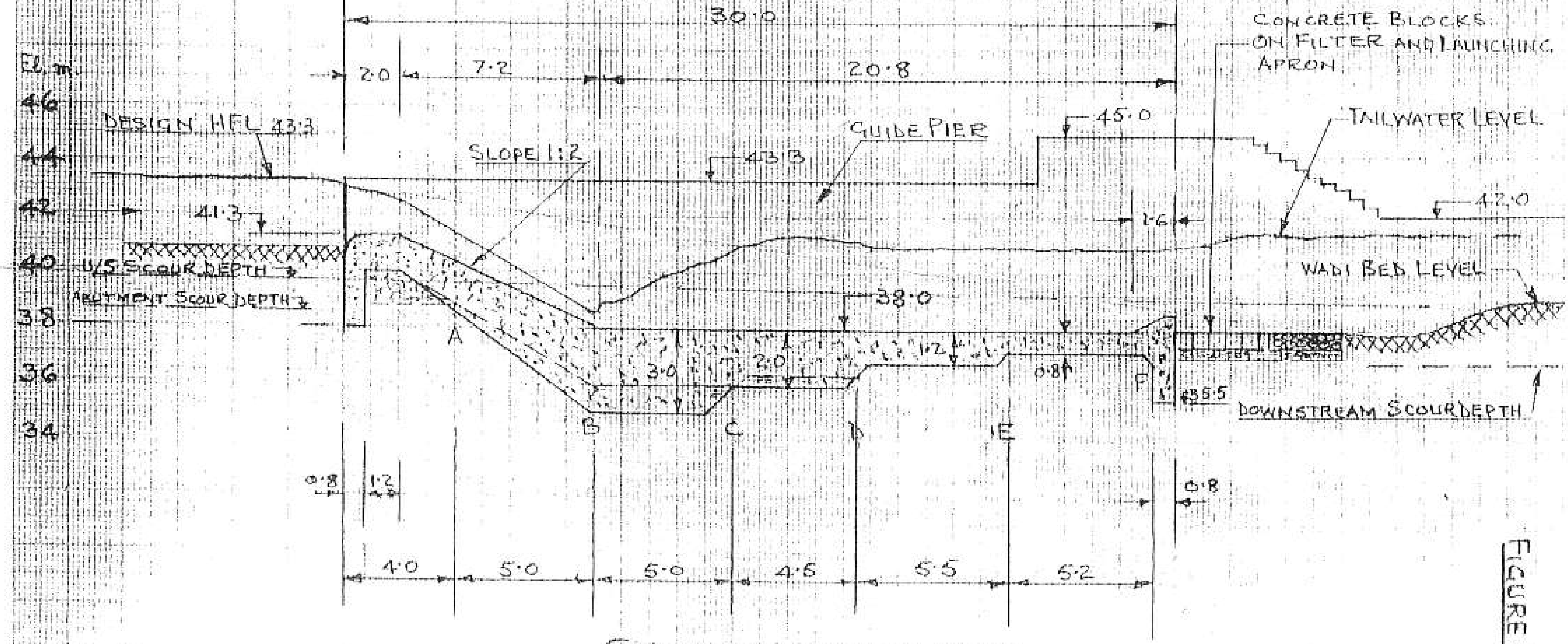

b/d
H/d
EXIT GRADIENT = G_E
NOTES :-
1. TRUE COPY OF ... PROJECT C.D.O. DRG. NO. MISC 30
2. FIND H/d AND b/d FIRST. FROM THE INTER-SECTION OF THESE LINES, READ HORIZONTALLY ON RIGHT HAND SCALE THE VALUE OF G_E DESIRED.
EXAMPLE :- FOR H/d = 2.4 AND b/d = 12 G_E = .30
IRRIGATION BRANCH PWD PUNJAB (INDIA)
BHAKRA NANGAL PROJECT
CHART FOR FINDING OUT EXIT GRADIENTS
CENTRAL DESIGNS OFFICE

Figure WP2 - 8

PROPOSED DIVERSION WEIR FOR SHEEB SPATE IRRIGATION SCHEME

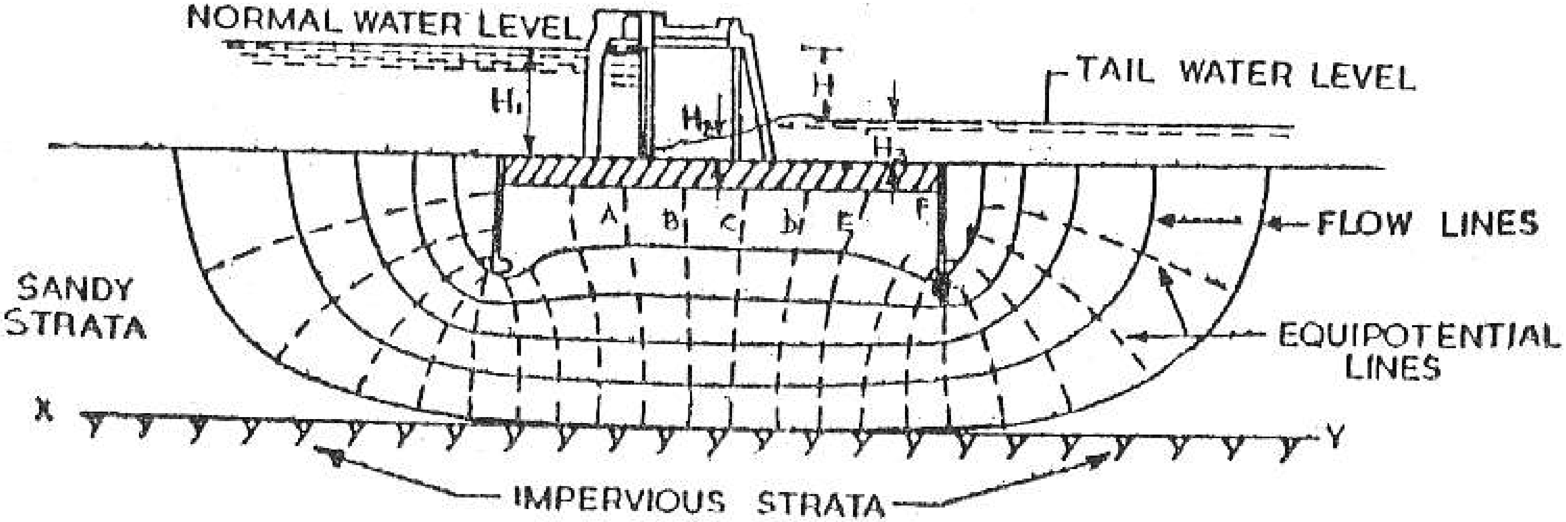
PRELIMINARY CROSS-SECTION.



SECTION THROUGH WEIR.

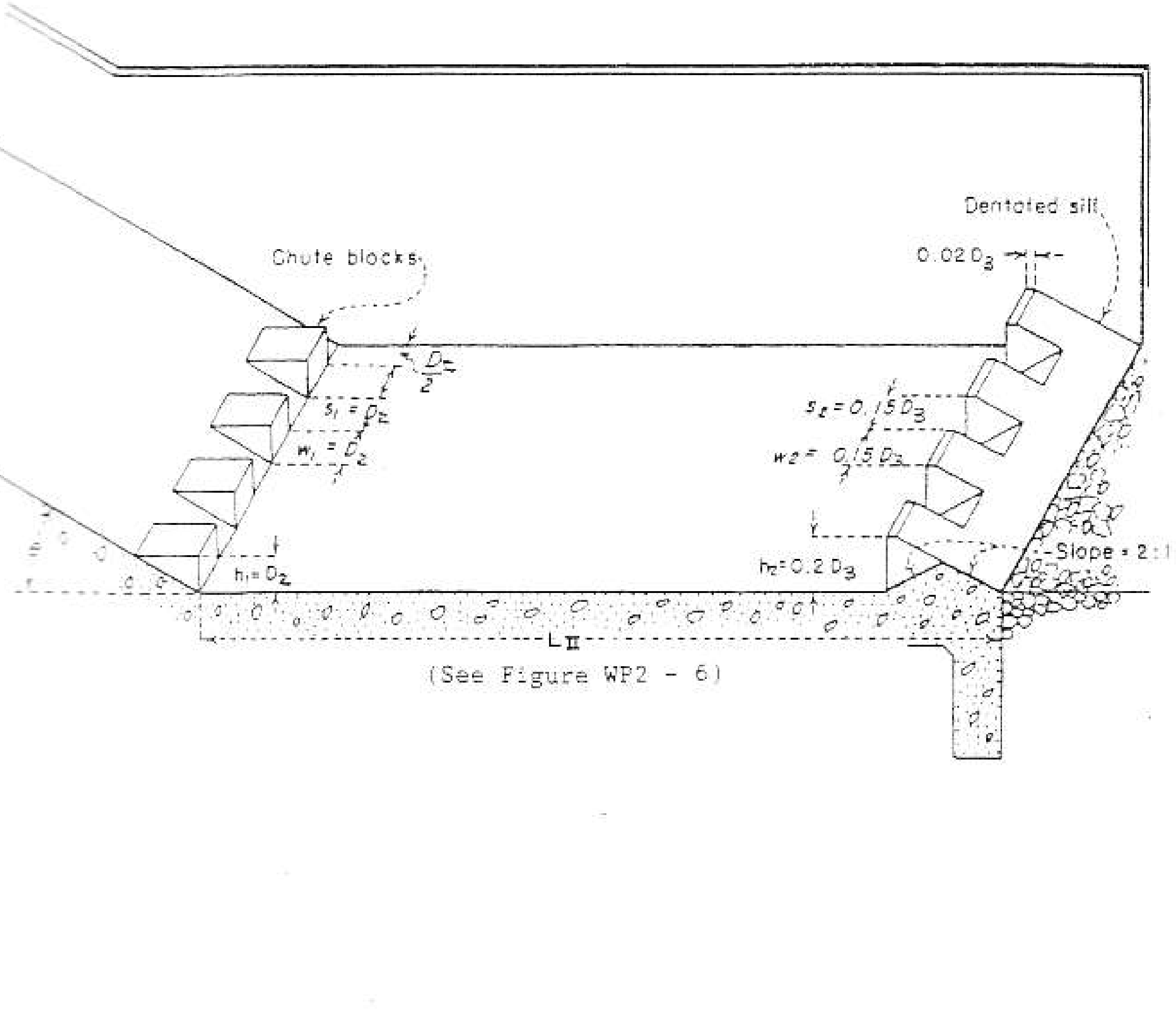
SCALE 1:200

FIGURE WP2-9



Flow Net Under a Weir
Showing Distribution of Uplift Pressure

Figure WP2 - 10



Recommended Proportions for USBR Type II Basin

Figure WP2 - 11

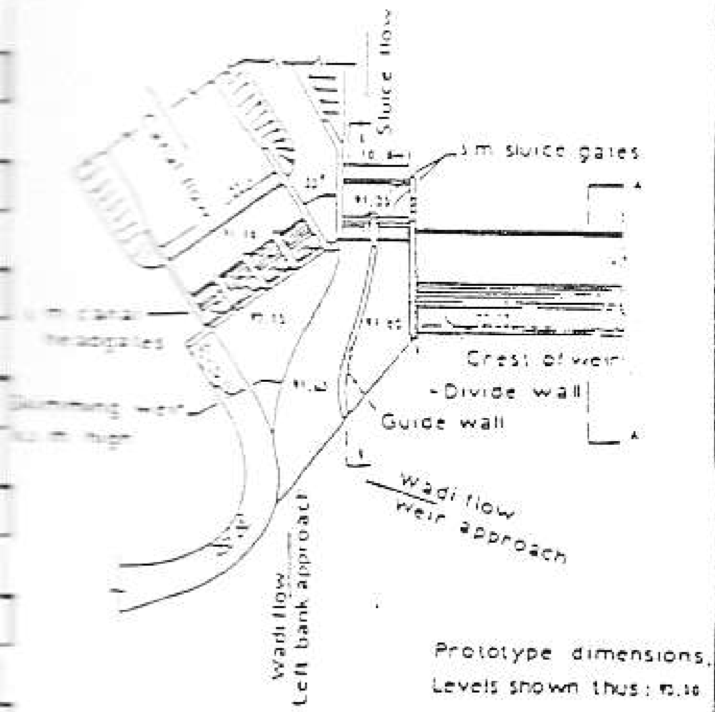


Fig 1 [illegible] m³/s headworks: preliminary layout.

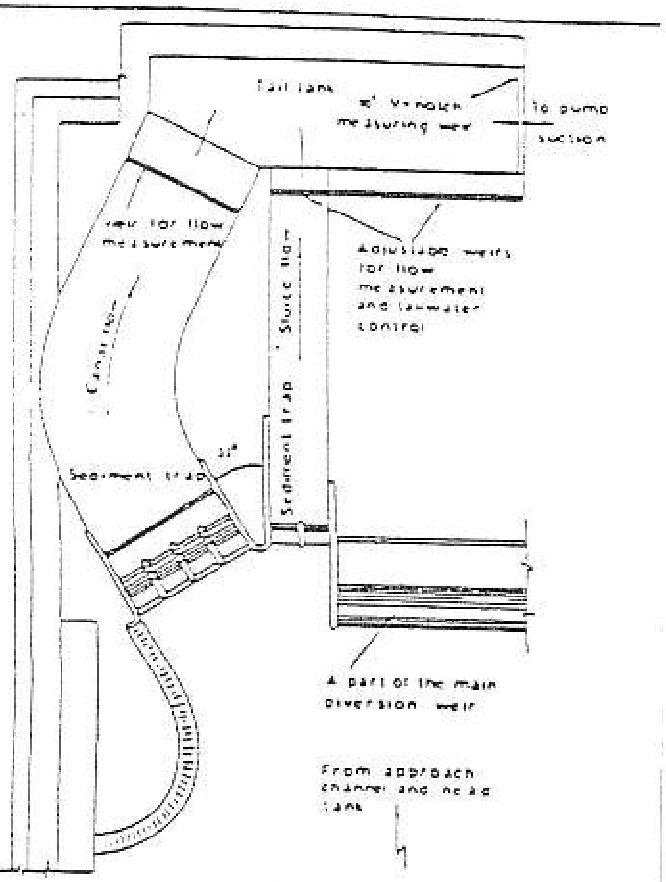


Fig 2 General layout of model area

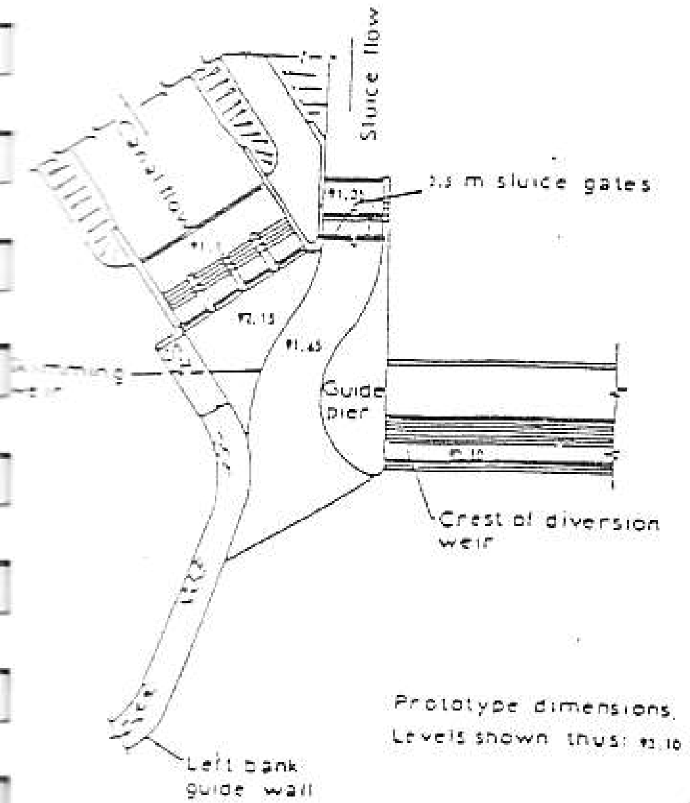


Fig 3 [illegible] m³/s headworks as modified.

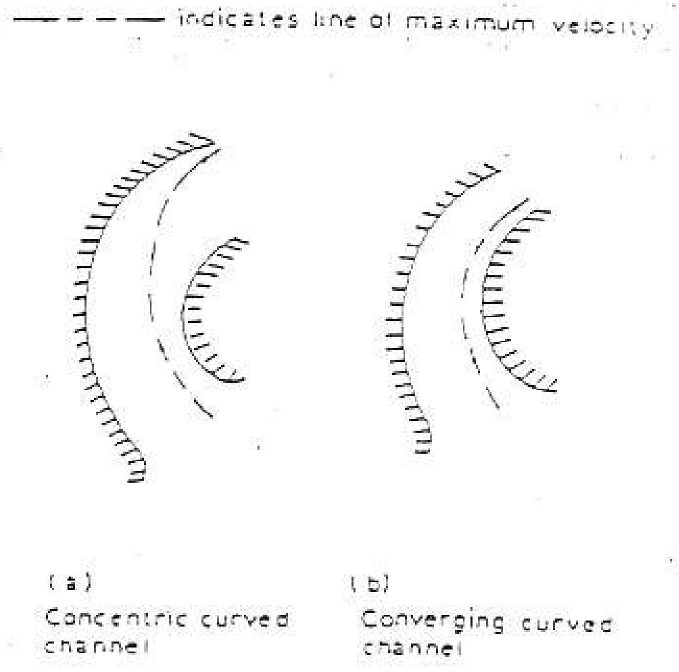


Fig 4 Lines of maximum velocity in concentric and converging curved channels

SHEEB STATE DIVERSION HEADWORKS

OUTLINE PLAN

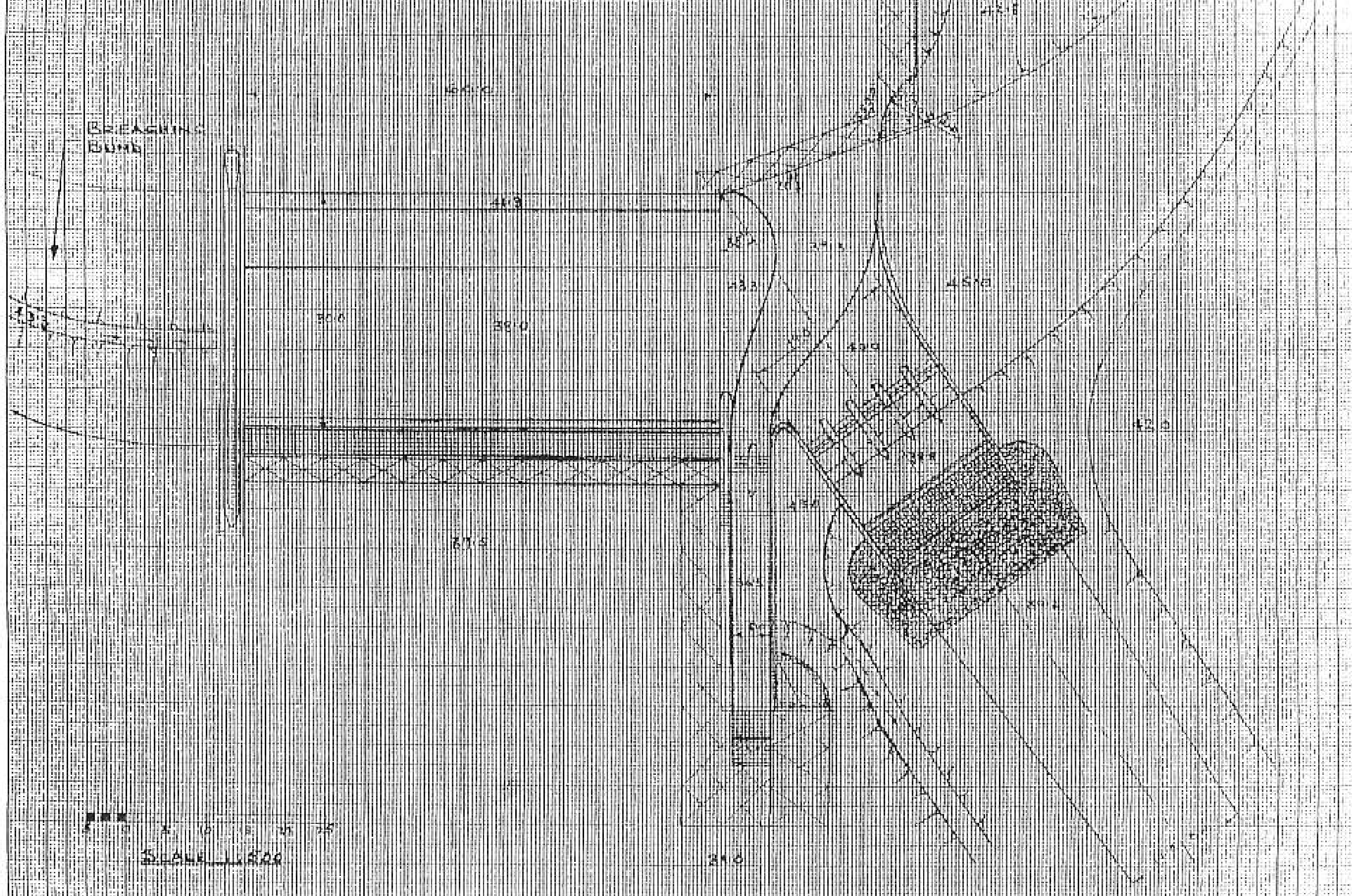
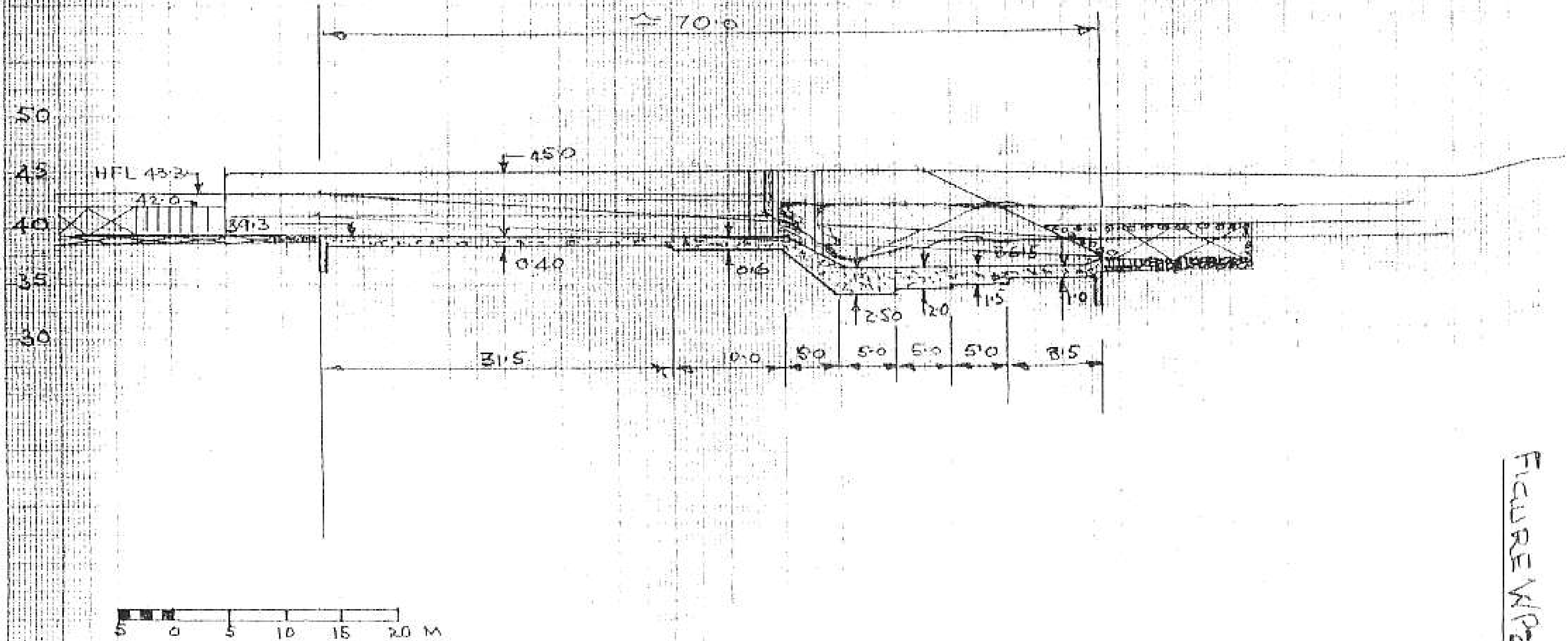


FIGURE WP2-13

SHEB SPATE DIVERSION HEADWORKS

SEDIMENT EXCLUDER SLUICE

LONGITUDINAL SECTION



SCALE 1:500

FIGURE WP2-14

FIGURE WP2-14

SHEEB SPATE DIVERSION HEADWORKS

CANAL HEAD REGULATOR

LONGITUDINAL SECTION

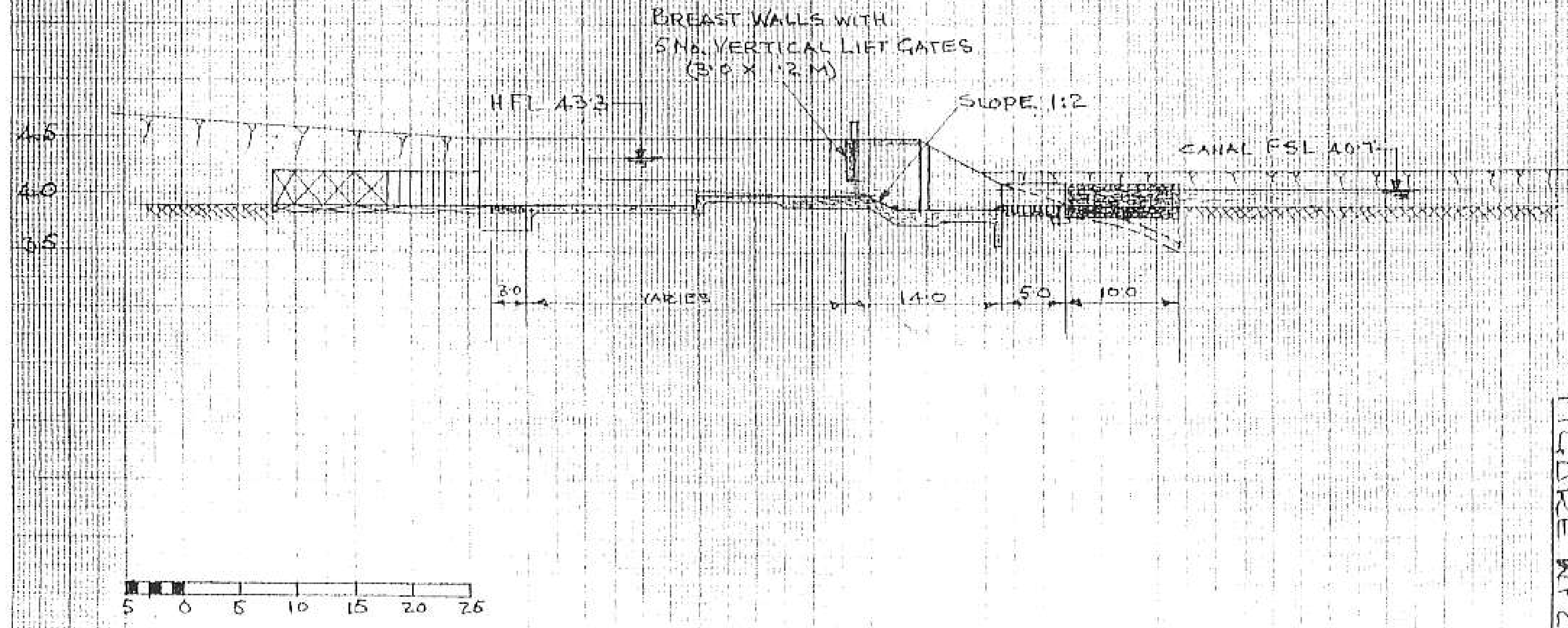


FIGURE WP2 - 15

FIGURE WP2 - 15

APPENDIX D

ETHIOPIA

KONSO SPATE IRRIGATION PROJECTS

TAMARIX FOR WADI BANK PROTECTION

Salah Rouchiche, FAO Forestry Consultant

(Extract from FAO Preparation Mission Report on Second Land and Water Conservation Project - Yemen)

Flood Protection Works

120. **Project Description.** The natural regime of the low flow channel within a wadi is sinuous. Because of the instability of wadi beds, high speed of flow and heavy sediment transport during spate events, wadi boundaries are difficult to control especially in the meandering lower reaches. The flow pattern causes scouring and undercutting of high and vertical wadi banks at the outer curves and sedimentation within the inner curves, thus developing the meander further.

121. It is along the meanders that flood protection works are most needed. They are subdivided in two categories according the type of protection they offer; these are:

- **Bank-protection.** It consists in armoring the outer curves of wadi banks to avoid erosion phenomena leading to bank collapse.

- **Wadi-training** aims at forcing wadi flow to follow a designated course usually within the inner wadi-bed, thus avoiding contact with banks and reducing meandering. The structures used consist of spurs, groynes, deflectors .

122. Unless expensive structures are built, it is practically impossible to provide effective protection against damage from most of the flood events. Usually, bank protection and wadi training structures are designed to withstand floods with a return period of 10 years but the very high investments required for conventional protection

works make it impossible for a country with low resources to ensure a permanent and complete protection.

123. To decrease the costs of protection and ensure the perennity of the structures, the project proposes to experiment a different approach using vegetation in a protective way. The use of vegetation is justified because it not only withstands ordinary floods, but when damage occurs during exceptional floods, regeneration is always possible by sprouting. Moreover, observation in the field show that vegetation reduces speed of flow, allowing for sediment deposit in front, over and behind the vegetative barrier. Sedimentation of coarse material during high and medium flow and of silt mixed with vegetative debris at low flow eventually builds up to form a solid natural protective structure. However, the distribution of natural vegetation in wadis is limited to sites of low speed flow where seeds are deposited and covered with enough sediment to obtain germination. In sites characterized by swift currents, vegetation establishment can only be obtained by planting cuttings deep and offering them protection against scouring and sucking preventing them from being washed away.

124. In order to get rapid vegetation growth, the experimentation would be implemented at the upper and middle reaches of wadis, where spate is available at least once a year. Planting would be carried out after the last spate, when soil moisture content is sufficient for rooting and growth of cuttings.

125. **Basic Design Principles**. The different structures proposed are meant to offer maximum protection from spate to the newly planted cuttings during their establishment phase. The best way to prevent sucking, scouring or undercutting within the planted ditch is to superpose materials closely graded and ranging from large boulders at the top to silt at the bottom. The structures are designed to be flexible and low to reduce resistance to flow and minimize scouring. They would eventually have their emerging part washed away, but they should resist long enough for rooting to take place.

126. Vegetation would be installed in patterns similar to those used in conventional flood protection structures as illustrated in Fig. 6.

- Bank protection would be achieved by armoring the most exposed parts of the outer curves with dense vegetative cover grown under the protection of a provisional retaining wall (Figure 7).

- Wadi training would be achieved by establishing dense vegetative layers grown in spur patterns. Depending upon their position within a wadi section, the vegetative "deflectors" or "groynes" would be protected by the following structures:

  - Provisional spur n. 1 (Figure 8) where flow speed is minimum.

- Provisional spur n. 2 (Figure 9) where flow is very swift.
- Provisional spur n. 3 (Figure 10) where flow is intermediate.

- Foot-Bank Biological Plantation. To reinforce the banks of the wadi, plantations of a mixture of **Tamarix aphylla** cutting and Saccharum aegyptiaca rhizomes will be gradually done on both sides of the wadi, as sedimentation occurs untill all foot-banks are vegetated.

127. Bearing in mind that wadis are important for groundwater recharge, a minimum of vegetative cover should be used to ensure protection.

128. The flood protection structures network represented in the wadi section of Figure 6 is given for the sole purpose of cost estimations. The dimensions and orientations of structures as well as the distances between successive ones are determined by the specificities of the site.

129. The bank retaining walls and provisional spurs may be subject to scouring. It is important to be able to make all necessary observations so as to improve the designs in the future.

130. Implementation and management will be carried out by the local forestry and irrigation units. Observations and technical analysis of the results would be done by the implementation teams with their respective research stations. 10 km in total would be implemented in the following wadis: Hassan, Bana (Abyan), Kabir, Saghir (Tuban), Beihan (Beihan).

REPUBLIC OF YEMEN: Second Land and Water Conservation Project

FLOOD PROTECTION WORKS

WADI-BANK PROTECTION

Illustration of the protection of a 500m wadi-section using provisional defectors (3 types), retaining walls and vegetation

provisional retaining wall

provisional defector type n° 1

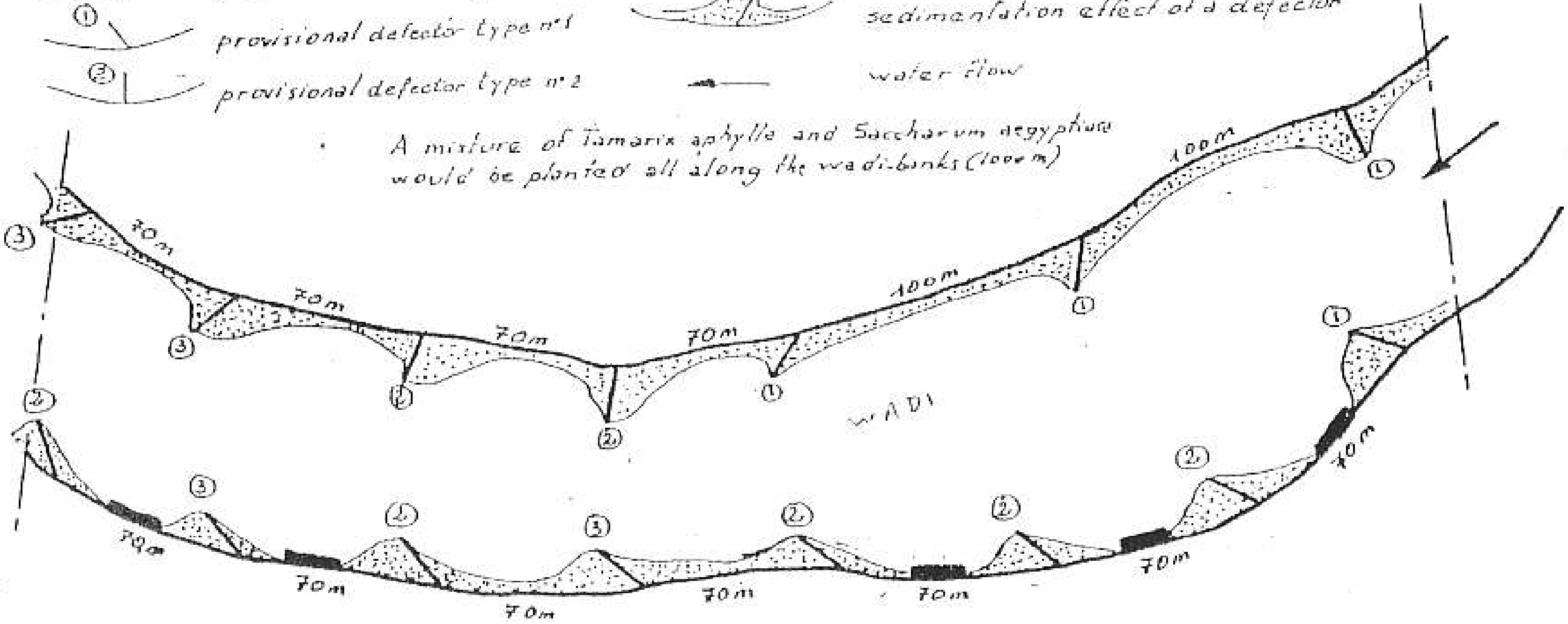
provisional defector type n° 2

provisional defector type n° 3

sedimentation effect of a defector

water flow

A mixture of Tamarix aphylla and Saccharum aegyptium would be planted all along the wadi-banks (1000 m)

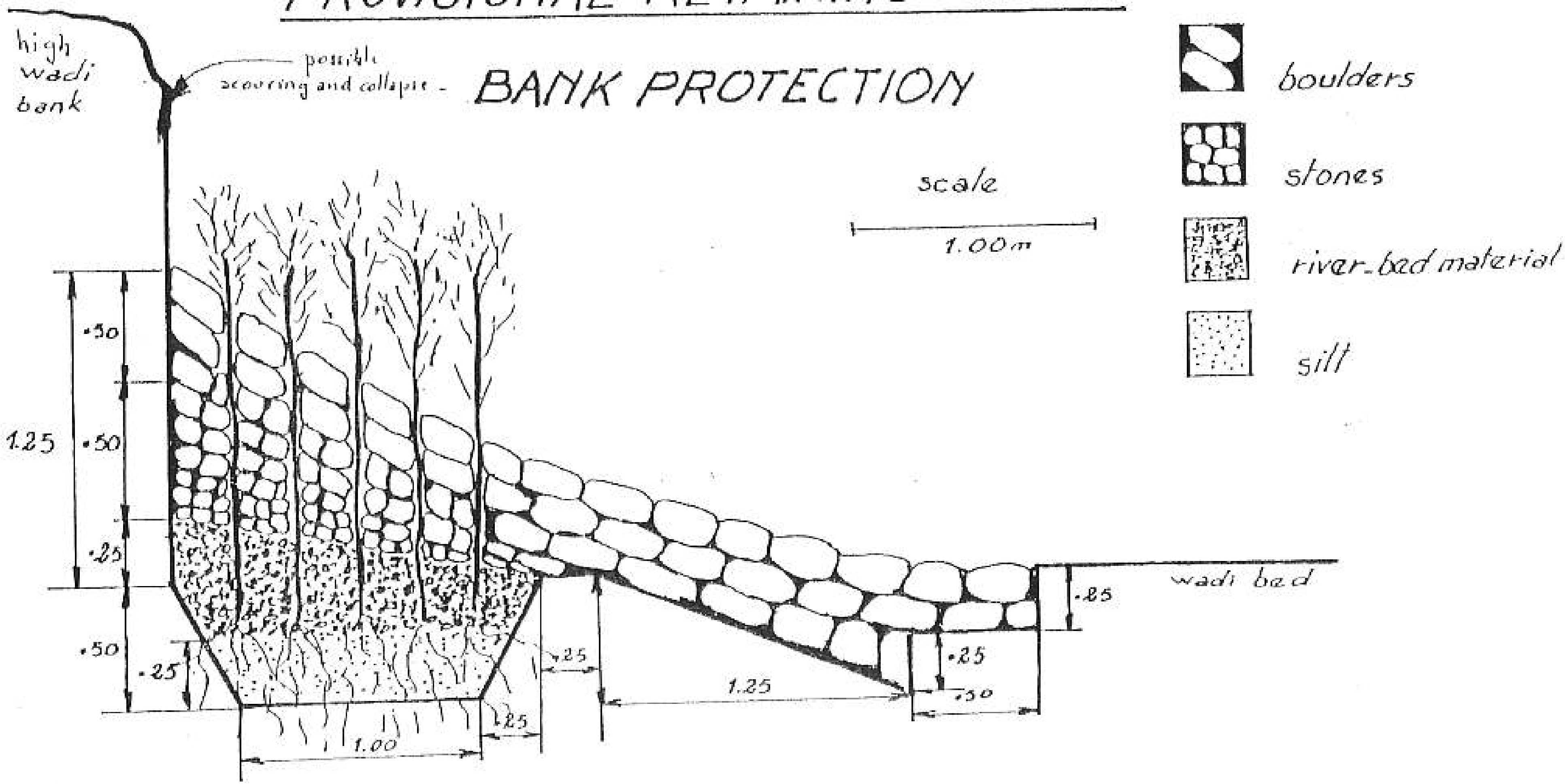


Annex 5
Fig. 6

FLOOD PROTECTION WORKS

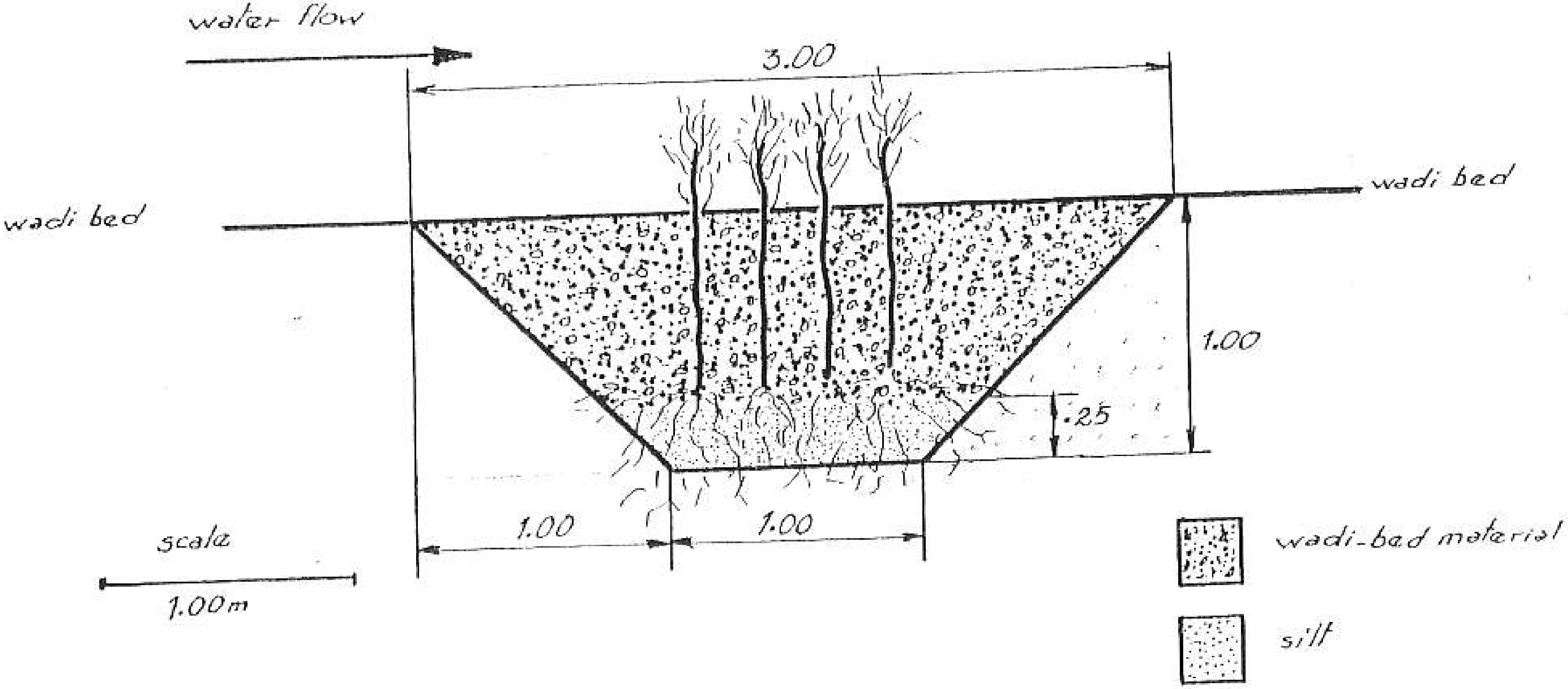
PROVISIONAL RETAINING WALL

BANK PROTECTION



REPUBLIC OF YEMEN: Second Land and Water Conservation Project

FLOOD PROTECTION WORKS

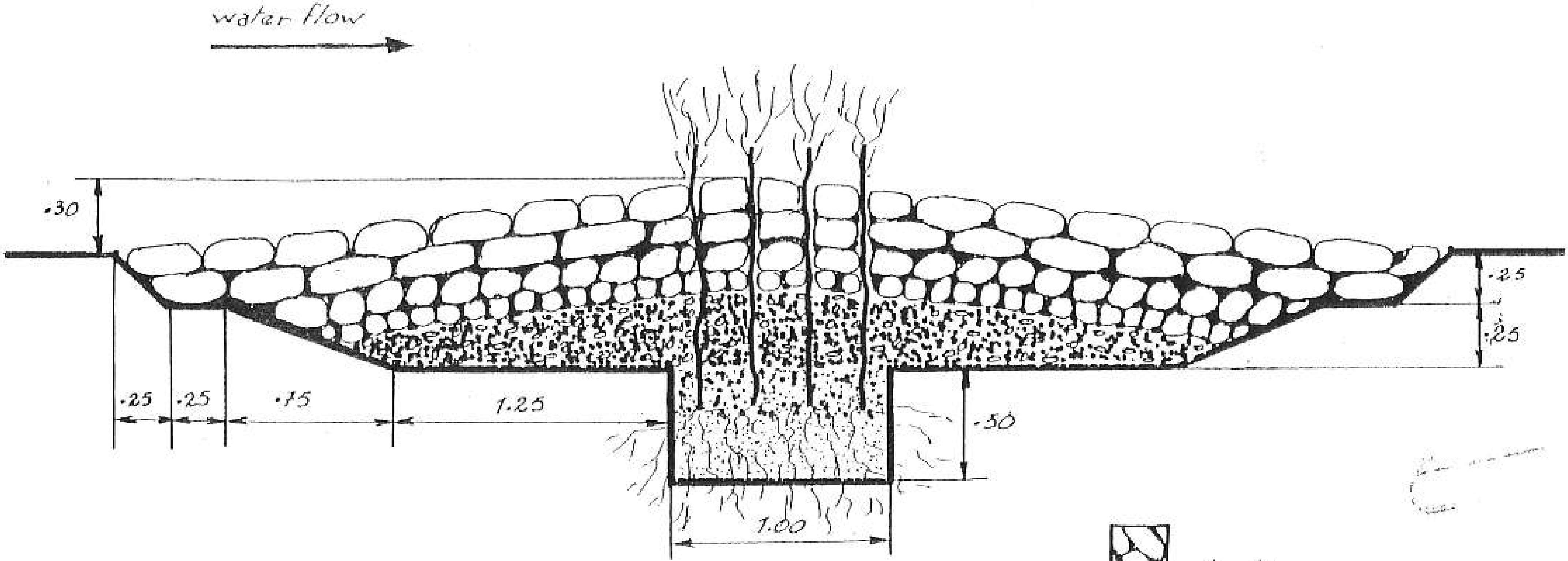


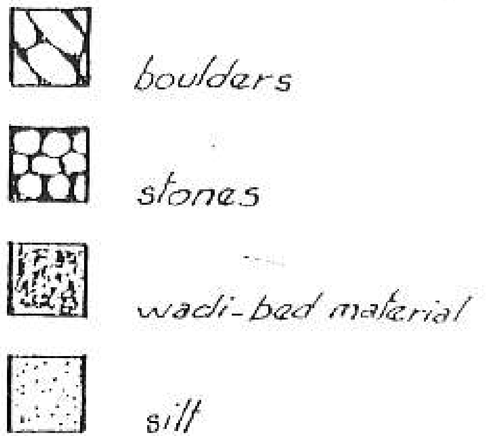
PROVISIONAL SPUR n°1

cross-section

(WADI - TRAINING)

FLOOD PROTECTION WORKS





scala

1.00m

PROVISIONAL SPUR n° 2

cross-section

(WADI TRAINING)

REPUBLIC OF YEMEN: Second Land and Water Conservation Project

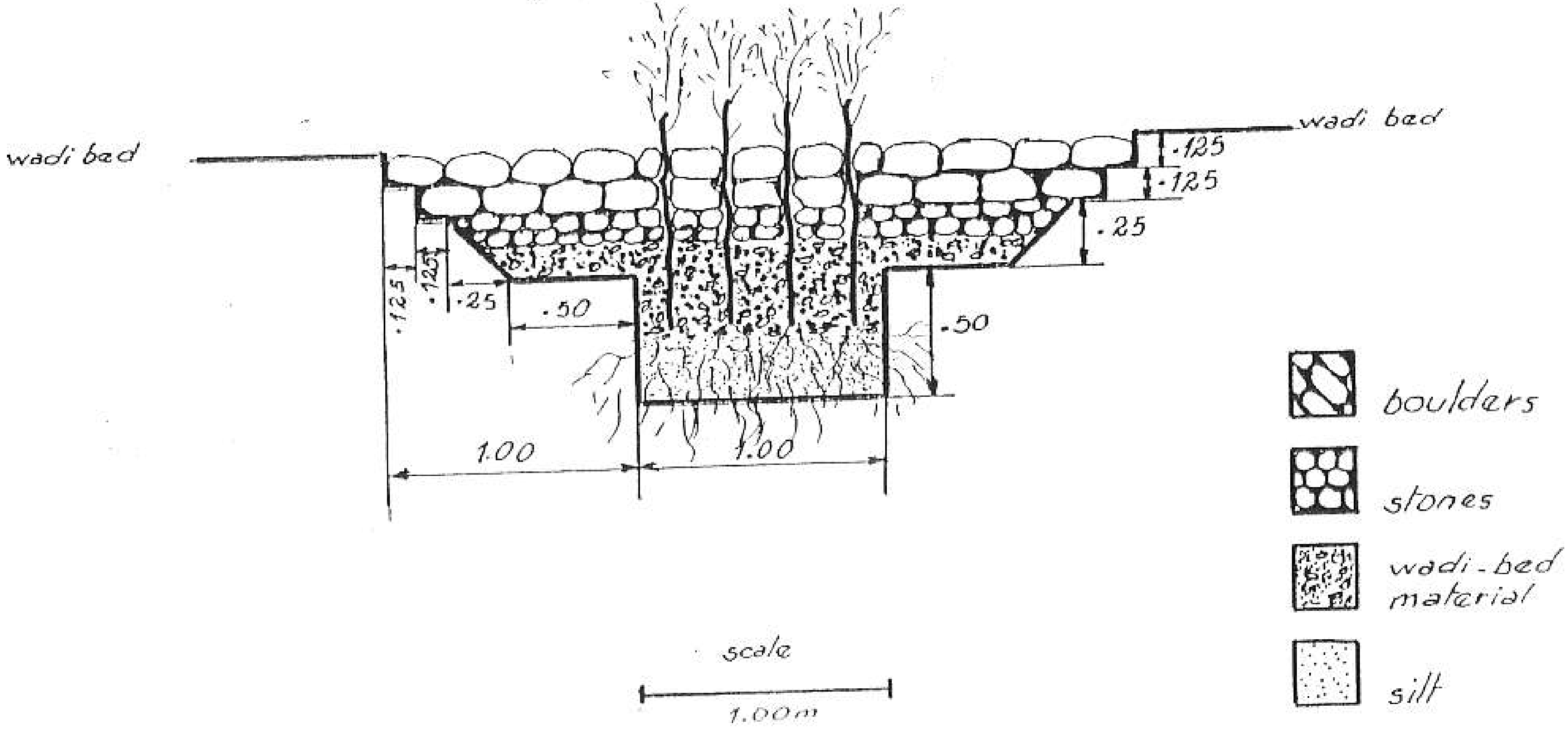
FLOOD PROTECTION WORKS

PROVISIONAL SPUR n°3

cross-section

(WADI TRAINING)

water flow



Multi-purpose Plantation of Water Distribution Canals

14. Technical Charateristics:

- 10 kms of canals distributed along 5 wadis,
- Lay-out: on each side of the canal are 40 m long planted strips separated by openings 10 m wide, i.e. 20 strips on each side of the canal,
- uprooting, total volume manipulated on both sides = 2,000 m x 3 m x 0.25 m = 1,500 m3
- bank reprofiling by a front loader
- planting on 40 strips 40 m wide/km
- each strip has 2 rows of trees planted at 4 m distance within the row (20 trees per strip)
- density of trees/km = 800

15. Norms:

- initial survey of canal section strongly affected by erosion and vegetation = 0.5 md/km
- lay-out of strips = 1 md/km
- uprooting. A front end loader with a 2 m^3 bucket can move a total volume of 50 m^3/hr. For 1,500 m3/km the norm would be 30 hrs/km
- cleaning up vegetation = 50 m/md. The uprooted vegetal material has to be removed from the site to allow for reprofiling and planting.
- bank reprofiling. A front loader can reprofile a strip 3 m wide over a distance of 150 m/hour.
- lay-out of pits (60 x 40 x 40 m) = 400/md
- pit digging = 80/md
- seedlings distribution = 200/md
- planting = 80/md
- protection using vegetal material removed from uprooting operation = 100/md
- finish-up (filling up planting terrace and compacting soil to make the slope uniform) = 400 trees/md

Flood Protection Works

Technical Characteristics And Cost Calculations

Provisional Spur N.1 Fig. 8

16. Characteristic. It is a structure which aims at helping vegetation establishment where sedimentation usually occurs at the inner curve of the wadi. The speed of flow is generally low. The spur is built from the bank, across the wadi-bed over a distance of 20 m (average for conventional works). Its function is to exercise wadi-training by pushing the flow towards the middle of the wadi-bed:

- total length = 20 m
- width = 3 m
- height above ground level = 0 m
- volumes of materials involved: (i) excavated material (digging) = 40 m^3, (cross-section = 2 m^2, length = 20 m); river-bed material = 33.75 m^3, (cross-section = 1.69 m^2, length = 20 m); silt = 6.25 m^3, (cross-section = 0.31 m^2, length = 20 m); planting of tamarix cuttings,total area = 20 m^2, density = 16 cuttings/m^2, total number of cuttings = 320

17. Norms

- digging = 2 m^3/md
- silt collecting and deposit = 2 m^3/md
- wadi-bed material deposit = 4 m^3/md
- wadi-bed material compacting = 40 m^2/md
- tamarix collecting, preparing and planting = 300 cuttings/md

18. Costs

- digging = 20 md = 60 YD
- silt deposit = 3.1 md = 9.3 YD
- wadi-bed material deposit - 8.5 md = 25.5 YD
- Wadi-bed material compacting - 1.5 md = 4.5 YD
- Tamarix planting - 1 md = 3.0 YD

Total cost of provisional spur n.1 construction = 102.3 YD

Provisional Spur N.2 Fig. 9

19. Characteristics. It is a structure which aims at helping vegetation establishment where scouring and under-cutting are stronger. Materials composing the structures are carefully graded so as to avoid any sucking of silt. The structure is kept low in order to allow for minimum scouring from very high speed flows.

- Total length - 20 m,
- total width = 6 m,
- height above ground level = 0.30 m
- volumes of materials involved:

- excavated material (digging) = 60 m^3, (cross-section = 3 m^2, length = 20 m)
- boulders = 29.4 m^3, (cross-section = 1.47 m^2, length = 20 m)
- stones = 18.2 m^3, (cross-section = 0.91 m^2, length = 20 m)
- wadi-bed material = 24.4 m^3, (cross-section = 1.22 m^2, length = 20 m)
- silt = 5 m^3, (cross-section = 0.25 m^2, length = 20 m)
- planting of tamarix cuttings
  total area = 20 m^2, density = 16 cuttings/m^2, total number of cuttings = 320

20. Norms

- digging = 2 m^3/md
- silt deposit = 2 m^3/md
- wadi-bed material deposit - 4 m^3/md
- stones deposit = 1 m^3/md
- boulders deposit = 1 m^3/md
- tamarix collecting, preparing and planting = 300 cuttings/md

21. Costs (labour = 3 YD/day)

- digging - 30 md = 90 YD
- silt deposit - 2.5 md = 7.5 YD
- wadi-bed material deposit - 6.1 md = 18.3 YD
- stones deposit - 18.2 md = 54.6 YD
- boulders deposit - 29.4 md = 88.2 YD
- tamarix planting 1 md = 3.0 YD

Total cost of **provisional spur n.2 construction = 261.6 YD**

Provisional Spur N.3 fig. 10

22. Characteristics. This type of structure aims at helping vegetation establishment where wadi-flow is of medium speed. Some scouring and under-cutting may still occur, and silt must be protected from sucking effect.

- total length = 20 m
- total width = 3 m
- height above ground level = 0 m

- volumes of materials involved in spur construction:

- excavated material (digging) = 35.6 m^3, (cross-section = 1.78 m^2, length = 20 m)
- boulders = 14.4 m^3, (cross-section = 0.72 m^2, length = 20 m)
- stones = 6 m^3, (cross-section = 0.3 m^2, length = 20 m)
- wadi-bed material = 10.4 m^3, (cross-section = 0.52 m^2, length = 20 m)
- silt = 5 m^3, (cross-section = 0.25 m^2, length - 20 m)
- planting of tamarix cuttings
  total area = 20 m^2, density = 16 cuttings/m^2, total number of cuttings = 320

23. Norms

- digging = 2 m^3/md
- boulders collecting and deposit = 1 m^3/md
- stones collecting and deposit = 1 m^3/md
- wadi-bed material deposit = 4 m^3/md
- silt collecting and deposit = 2 m^3/md
- tamarix cuttings (collecting, preparing and planting) = 300/md

24. Costs (Labour costs = 3 YD/day)

- digging 18 md = 54.0 YD
- silt deposit - 2.5 md = 7.5 YD
- wadi-bed material deposit - 2.6 md = 7.8 YD
- stones deposit = 6 md = 18.0 YD
- boulders deposit = 14.4 md = 43.2 YD
- tamarix planting = 1 md = 3.0 YD

Total cost of provisional spur n.3 = 133.5 YD

Provisional Retaining Wall Fig. 7

25. Characteristics. It is a structure which aims at helping vegetation establishment where bank under-cutting and collapse usually occur at the outer curve of the wadi. The technical characteristics of the provisional retaining wall are as follows:

- total length = 20 m (average length for conventional works, which could be modified according to sites and total width of wadi)
- width = 3.50 m
- height above ground level = 1.25 m
- volumes of materials involved:

- excavated material (digging) = 22 m^3, (cross-section = 1.1 m^2, length: 20 m)
- silt = 5.6 m^3, (cross-section = 0.28 m^2, length = 20 m)
- river-bed material = 9.4 m^3, (cross-section = 0.47 m^2, length = 20 m)
- stones = 6.8 m^3, (cross-section = 0.34 m^2, length = 20 m)
- boulders = 26.8 m^3, (cross-section = 1.34 m^2, length = 20 m)
- planting of tamarix cuttings
  total area = 20 m^2, density = 16 cuttings/m^2, total number of cuttings = 320)

26. The upper part of the bank may collapse over the spur. This would provide greater protection to the spur and the cuttings. The material (silt) which has collapsed will be slowly washed away by water flow until the cuttings emerge again.

27. Norms

- digging = 2 m^3/md
- straightening bank wall up to 1.30 m above wadi-bed level = 10 m/md
- silt deposit = 4 m^3/md
- wadi-bed material deposit = 4 m^3/md
- stones deposit = 1 m^3/md
- boulders deposit = 1 m^3/md
- tamarix collecting, preparing and planting = 300 cuttings/md

28. Costs

- digging = 11 md = 33.0 YD
- straightening bank wall = 2 md = 6.0 YD
- silt deposit = 1.4 md = 4.2 YD
- wadi-bed material deposit = 2.4 md = 7.2 YD
- stones deposit - 6.8 md = 20.4 YD
- boulders deposit - 26.8 md = 80.4 YD
- planting tamarix - 1 md = 3.0 YD

Total cost of provisional retaining wall construction = 154.2 YD

Foot-Bank Biological Protection: Plantation along The Banks of Species Such As **Tamarix Aphylla** and **Saccharum Aegyptiaca.**

29. Characteristics. The calculations are made for a wadi section 500 metres long, i.e. for 1,000 metres of bank-sides:

- total length 1,000 m (500 m tamarix, 500 m saccharum)
- number of rows 6 (total width 1.5 m)
- distance within rows - 50 cm
- total number of tamarix cuttings: 500 x 2 x 6 = 6,000 cuttings
- total number of saccharum rhizomes: 500 x 2 x 6 = 6,000 rhizomes
- planting would be done with tamarix and saccharum alterning every 10 metres. A ditch 1.5 wide and 1 m deep would be dug mechanically along the banks. Total volume of ditch = 1,500 m^3

30. Norms

- ditch opening - capacity 20 m^3/hour (Poclain type machine)
- ditch refilling, before planting - Refilling would be done with bulldozer type D6 with a capacity of 40 m^3/hour
- tamarix planting. The cuttings which are 1 m long would be planted 80 cm deep. One man could collect, prepare and plant 300 cuttings/day
- saccharum planting. Collecting, preparing and planting of saccharum rhizomes at 20 cm depth could be done at a rate of 400/md.

31. Costs

- ditch opening = 75 hours at 10 YD/hour = 750.0 YD
- ditch refilling before planting = 37.5 hours at 12 YD/hour = 450.0 YD
- collecting, preparing and planting tamarix cuttings. 20 md = 60.0 YD
- collecting, preparing and planting saccharum rhizomes = 15 md = 45.0 YD

Total cost of 1,000 m bank biological protection = 1,305 YD

Total Costs of Flood Protection over a 500 m Long Wadi Section

Table 9 shows, according to position in wadi, the types and numbers of protection works that would be necessary for a 500 m long wadi section. They are detailed as follows:

	cost
- N.1 provisional spurs - 4 units at 102.3 YD/unit =	409.2 YD
- N.2 provisional spurs - 7 units at 261.6 YD/unit =	1,831.2 YD
- N.3 provisional spurs - 4 units at 133.5 YD/unit =	534 YD
- provisional retaining walls - 5 units at 133.5 YD/unit =	771 YD
- 1,000 m of foot-bank biological protection (plantation) =	1,305 YD
- Total	= 4,850 YD

Cost of protection a 500 m long wadi section = 4,850 YD