Considerations for the Design of River Abstraction Works in South Africa

Report to the
Water Research Commission

by

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Executive Summary

The South African climate oscillates between drought and flood. This leads to extremes in river flow and sediment transport. While storms last for minutes to days, the hydrological critical low flows can last for years during droughts. The strong variation in river flow associated with even higher variability in sediment loads make the design of river pumpstations and diversions highly complex in South Africa, especially if the water supply should have a low risk of failure.

Diversion structures are generally used to divert water from an existing natural watercourse into a water supply conveyance system. For large diversions, such as the headworks for an irrigation main canal system that normally require a headpond, the diversion structure can include a weir, sluiceway, intake and fishway.

This study focused on the following aspects of sediment control at abstraction works:

a) Review of international state of the art technologies to control the sediment.
b) Investigation of optimum abstraction location on a river bend
c) Review of typical South African abstraction case studies.
d) Assessment of flushing channels in abstraction works with field testing.
e) Development of guidelines for the planning and design of river abstraction works in South Africa.

This report focused on (e) above and should be read in conjunction with the Volume 1 report of this study: Sediment Control at River Abstraction Works in South Africa by Brink et al., (2005).

The intake should be located on a stable reach to ensure that the intake is directed to the main current and that the flow path does not wander. The optimal location of an intake is usually just below the vertex of a concave bank (Tan, 1996). Historical aerial photography and satellite images are very important to evaluate the stability of a particular river reach.

The relationship between the central angle of a bend and the optimal location based on experimental data (SC and Ches, 1992) is given in Table 2-2.

Table 2-1 Relationship between central angle of a bend and optimal location of intake (SC and Ches, 1992)

Central angle of bend (˚)	<45	60	90	120	150	180
Optimal location of intake (˚)	0 (end)	45	60	80	95	110

The following are proposed designs for South African conditions, in order of priority:

a) Pumpstation/diversion without a weir, located on the outside of a stable river bend
b) Low weir with low level flushing gate(s) in a gravel trap, and pump canals/sand trap that can be flushed, combined with a bend in the river
   - Design with submerged gravel trap wall
   - Design to control high bedload
   - Design with high gravel trap wall
c) Barrage on river with large gates across the river, combined with a stable bend in the river
d) Weir, flushing canals, deep sand trap (pit) and jet pumps to clean the pit
e) Sand pump system with infiltration gallery

The benefits and disadvantages of these systems are given in Table 1.

Details are provided in this report on pumpsump design and pump selection. Diversion weir hydraulic aspects and energy dissipation are discussed. Environmental flow releases, fishway design and canoe chute design aspects are also addressed.

A review of sand trap types and design principles is given, with recommendations on sediment flushing in terms of duration and minimization of the downstream environmental impact.

Table 1 Priority List of Abstraction Works Types for Sediment Control in South Africa

Description	Benefits	Disadvantages
1. Pumpstation/Diversion without a weir, located on the outside of a stable river bend (Figure 3-1)	• No/small impact on river sediment balance • Cheap and ideal for small, relatively high risk of supply failure designs, such as irrigation • Full use of secondary currents to limit coarse sediment diversion • Diversion structure protruding into flow help to create deep pool at intake	• Low flow in sand bed river could meander away from the bank if not well positioned • Flushing potential of abstraction works limited if river is not steep locally • Pump intake head limited during low flow conditions
2. Low weir with low level flushing gate(s) in a gravel trap and pump canals/sand trap that can be flushed, combined with a stable bend in the river	• Flushing locally at diversion possible with additional head created by weir • Low maintenance on weir versus gated structure (barrage)	• Sediment deposition in the river upstream of the weir with raised flood levels upstream • Sedimentation also reduces the balancing storage at the weir and most diversion weirs are filled up with sediment in a short period of time
2a) Submerged gravel trap wall (Figure 3-10)	• Flushing of gravel trap effective along length of the intake.	
2b) Design to control high bedload: Boulder trap and Gravel trap (Figures 3-14 and 3-15)	• Use of curved channel flow to divert bedload away from intakes • Effective flushing of both traps • When gates at taps are opened during floods, flood levels are reduced	
2c) Design with high gravel trap wall (Figure 3-2)	• Trees cannot enter the scour chamber (F)	• Its not possible to flush the upstream parat of the scour chamber (F) • Secondary current development to create local scour against the structure will be limited due to the presence of the weir and its low notch weir is often at a similar level as the invert of the open intake (C), which makes it susceptible to sediment deposition • During flushing the gravel trap (F) acts like a side channel spillway with the risk that water could jump up on the opposite side through the trash rack

Table 1 Priority List of Abstraction Works Types for Sediment Control in South Africa (continued)

Description	Benefits	Disadvantages
3. Barrage on river with large gates across the river, combined with bend	• Passing sediment through during floods (sluicing) with limited damming and sediment deposition, thereby also limiting flood levels upstream. • Probably the only feasible design on large rivers with high sediment yields such as the Limpopo and Olifants (Limpopo Province), where balancing storage is required on the river. • Balancing storage is created upstream of the barrage	• Expensive design with high maintenance and operational cost • Judicious operation required during floods for safety and to limit sediment deposition • Possible tree blockage if gates are too small • Anaerobic sediment flushing could lead to fish kills if the flushing duration is long during relatively small floods
4. Weir, flushing canals, deep sand trap (pit) and jet pump technology (Figure 3-17)		• Ineffective (short length and rapid transitions) and expensive sandtrap when about 70 % of the sediment transported during floods is silt and clay. • Heavy reliance on power supply for the jetpump, especially during floods • Jet pump technology which is not well proven • When fine sediment (silt and clay) deposits in the pit during periods when the main pumps are not working, it consolidates which could make it difficult for the jetpump to remove since its more suitable to re-entrain sand.
5. Sand pump systems with infiltration gallery (Figure 3-23)	• The pumps can be placed relatively far away from the river on the river bank, well protected from floods, with limited risk of blockage of the intake pipes • No weir is required or hydraulic structure in the river, and it is therefore a relatively cheap design	Damage to the suction pipe system is a high risk during a flood event, and therefore its recommended that sandpump systems are only considered for irrigation supply or standby/emergency supply for potable water use.

It is recommended that the design of a river abstraction works is based on the design guidelines reviewed and developed in this study. The following are some of the key aspects to consider:

- Assess river stability from historical aerial photos and satellite images.
- Consider low flow conditions and flood flows and the variability in sediment loads. The environmental flow requirement must be released downstream during low flow periods and the diversion must operate during floods.
- Locate the diversion on the outside curve of a river bend to limit coarse sediment diversion and to scour a deep pool at the intake during floods, which should still be present during low flow periods.
- Use a mathematical model and/or physical hydraulic model to simulate the sediment dynamics to select the best position and orientation of the diversion at important abstraction works.
- Fine sediment will enter the diversion, therefore allow for flushing under gravity back to the river. Even the pump canals can be flushed.
- A gravel trap should be provided upstream of the pump/diversion canals.
- Robust pumps, preferably submersible, should be selected to handle the coarse sediments.

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Mr R Dube	SA Water Research Commission (Chairman, WRC)
Prof N Armitage	University of Cape Town
Mr NJ van Deventer	Dept. of Water Affairs and Forestry
Mr JK Hauman	PD Naidoo and Associates
Prof CS James	University of Witwatersrand

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1 INTRODUCTION

The South African climate oscillates between drought and flood. This leads to extremes in river flow and sediment transport. While storms last for minutes to days, the hydrological critical low flows can last for years during droughts. The strong variation in river flow associated with even higher variability in sediment loads make the design of river pumpstations and diversions highly complex in South Africa, especially if the water supply should have a low risk of failure.

Diversion structures are generally used to divert water from an existing natural watercourse into a water supply conveyance system. For large diversions, such as the headworks for an irrigation main canal system that normally require a headpond, the diversion structure can include a weir, sluiceway, intake, and fishway. Provisions for allowing the river to return to near natural levels in the dry season (i.e. in general, water is not diverted into the conveyance system during the dry period), permitting boat/canoe access or for fish migration may also be required.

Generally, the diversion structure is located within a stable channel. An actively progressing meaning channel should be avoided since floods may cause the channel to erode and bypass the structure. The foundation conditions at the proposed site for the structure should preferably consist of competent soils or rock with adequate bearing capacity and relatively low permeability.

The intake structure is normally located on the outside bend of the channel to minimize the intrusion of sediments (sand, gravel and boulders) into the conveyance system. Locating a gated sluiceway immediately adjacent to the intake will also help to minimize the sediment load and debris that may enter the conveyance system.

In general, the crest of the sluiceway is normally in-line with the crest of the weir (since both discharge into the downstream channel), while the adjacent intake is set at an angle (preferably 45 º) in order to minimize vortex zones, head losses, and the tendency to trap floating debris.

In cases where provisions are needed to facilitate the movement of fish past the diversion structure, a fishway is provided.

River abstraction structures serve to divert water from river streams as well as to limit the sediment load that enters the diversion system. One of the key features of a diversion structure is the location. By ensuring that the structure is properly located i.e. on a stable bank in a stable river reach, the reliability of the delivered water can be enhanced. The effect of the diversion structure on the morphology of the river can also be limited by ensuring that sediment transport is maintained through the structure. This can be achieved by limiting the sediment that enters the diversion structure and by removing the coarse sediments from the diverted water and returning them to the river.

River bends prove to be ideal for abstraction works and a diversion structure should be on the outside of the bend to take advantage of secondary (spiral or curvilinear) flow which creates a deep pool on the outside, which is very important when abstracting water during droughts. Secondary (spiral) flow has the tendency to direct the heavy sediment laden bottom layers away from the diversion structure and to allow the top layers, with lower sediment concentration, to be directed towards the diversion structure. If the diversion structure can take advantage of the spiral flow, less sediment will be diverted. This is important in minimising sedimentation in the diversion structure. Hydraulic theory can be used to establish the optimum location of abstraction works at a river bend.

One of the first descriptions of curvilinear flow was provided in the 19th century by Thompson (1876). Thompson stated that the bend flow phenomenon will only occur if there is a horizontal pressure greater on the outside of a curved path than on the inside. The result is that the water surface is super-elevated at the outer (concave) bank. Along any vertical section the pressure gradient acting towards the centre of the curvature has to be the same, since the cross slope of the water surface at the top determines it. Thus, the centrifugal acceleration has to be the same down any vertical section. This implies that the velocity is smaller near the bottom and the bottom filaments have to move in curves of smaller radii than the top ones thus giving rise to secondary (spiral) flow. The secondary (spiral) flow is directed towards the centre of curvature of the channel and will tend to move the bed sediment away from the outer (concave) bank towards the centre. For continuity there must exist an opposite cross flow at the surface that tend to push the filaments at the top to the outer (concave) bank. Figure 1-1 shows the developed secondary (spiral) flow. The remains of diversion schemes which use the spiral flow effect date back to ancient Mediterranean civilizations and in China to 2000 BC of which some are still operational.

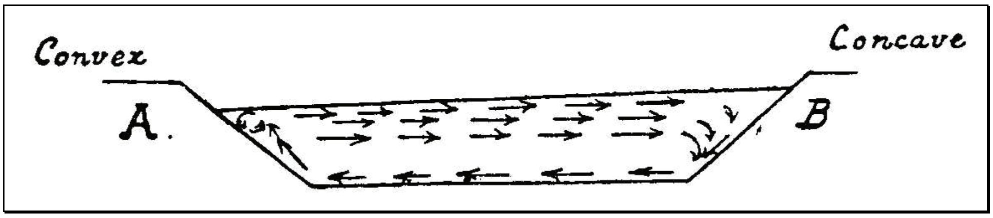


Figure 1-1 Secondary (Spiral) flow (Thompson, 1876)

Various classifications of intakes are found in the literature. Intakes are generally classified according to hydraulic or sediment control principles. Scheuerlein (1984) classified intake types according to their hydraulic and sediment control principles. According to hydraulic principles the intakes were classified as lateral intakes, frontal intakes, bottom intakes and suction intakes. For sediment control a different classification seemed appropriate and the classification with respect to the mechanism of sediment transport is the control of bed load sediment rejection sediment extraction sediment ejection and the control of suspended load.

Raudkivi (1993) classified the different types of intakes according to their hydraulic and sediment control aspects as follows: intakes on river bends, intakes with dividing walls, intakes with under sluices, intakes with excluder tunnels, intakes with baffles, guide vanes and deflectors. According to the above classification there is no clear subdivision apart from the hydraulic and sediment aspects. A clearer subdivision should probably distinguish between intakes with dams, weirs or barrages, and intakes without a weir.

Vanoni (1977) stated that water should be diverted according to the following three principles: direct only water into the diversion structure and return the sediment to the river, or design the canal system hydraulically so that the water with its sediment will be transported out onto the land with a minimum of sediment deposited in the diversion structure, or design the diversion structure to direct as little sediment as practically possible into the diversion channel and remove the deposited sediment by the most inexpensive available method. Of the above-mentioned diversion principles, the third principle is recommended.

Scheuerlein (1984) recommended that the principle of sediment rejection be applied to diversion structures. The principle of sediment rejection is based on allowing the upper, clearer layers of the flow to enter the intake while the lower sediment laden layers are prevented from entering the intake. Advantage can be taken of the river bend phenomenon where the developed secondary current provides favourable flow patterns at the intake. Thus intakes should be located on the outer (concave) bank of a bend to take advantage of this phenomenon. When the intake does not operate in combination with a diversion dam or weir the sediment rejection technique can be applied to divert up to 50% of the total river flow without experiencing bed load problems (Scheuerlein, 1984).

Several handbooks have been written on the topic of control of sediment extraction at river diversions, such as by Raudkivi (1993) and Bouvard (1992) and the question could rightfully be asked why is yet another study needed? The fact is that even though international guidelines are available in the literature, many of the South African abstraction works and pumpstations are experiencing serious problems with sedimentation control. The typical sediment related problem experienced at South African abstraction works are:

a) Changes in the river plan form and geomorphology (which could be natural), with a shallow shifting low flow channel during droughts.
b) Sediment deposition at pump intakes where low velocities are part of the pump sump design. This could lead to damage at start-up of the pump and cause abrasion of the pipeline.
c) Build-up of cohesive sediment in the intake which could be difficult to flush out.
d) High sediment load diversion which is mainly fine and difficult to settle out.
e) Sediment deposition in pools due to a dam upstream causing flood peak attenuation and narrowing of the river.
f) Increased sediment yields due to land degradation.
g) Sediment build-up caused by a downstream dam, which could be higher than the full supply level of the dam.
h) Wrongly positioned abstraction works on the inside of the bend.
i) Flushing facilities are in most cases not provided
j) Incorrect pump selection which cannot deal with the coarse sediment.
k) Underestimation of the operational and maintenance costs that will be incurred with a relatively cheap initial design which does not cater for sediment control.

l) Weirs have to be constructed to dam the water to provide positive suction heads at pumps during droughts, but this leads to slower flow velocities and a rapid rate of sediment deposition upstream of the weir. In Europe barrages are constructed across the river with large gates that allow floods to pass freely with little damming and therefore sedimentation is limited. These structures are expensive, requires high maintenance and are not always practical considering the large variation in flow depths from droughts to major floods in South Africa which could be from 0 m to 15 m depth.

Most of international literature especially from Europe are based on:

- Large rivers with high base flows due to snow melt
- Relatively coarse sediment
- Relatively steep rivers
- Relatively small sediment concentrations
- Expensive hydraulic structures and controls

This research project commenced in 2002 and had a duration of 2 years. The project was carried out by Ninham Shand (Pty) Ltd who has had extensive design experience of abstraction Works in Southern Africa, in association with the Department of Civil Engineering, University of Stellenbosch.

This study focused on the following aspects of sediment control at abstraction works:

a) Review of international state of the art technologies to control the sediment.
b) Investigation of optimum abstraction location on a river bend
c) Review of typical South African abstraction case studies.
d) Assessment of flushing channels in abstraction works with field testing.
e) Development of guidelines for the planning and design of river abstraction works in South Africa.

This document contains the guidelines mentioned in (e) above, while (a) to (d) are included in a separate report (Brink et al., 2005).

2 ABSTRACTION LOCATION AND SECONDARY FLOW

The principle of sediment rejection where as little sediment as possible is abstracted from the main channel is recommended. This can be achieved with the aid of the secondary flow that develops in bends that creates a spiral motion. The spiral motion moves the sediment laden bottom flow towards the inside of the bend, while the upper flow with less suspended sediment move towards the outside of the bend where the diversion is located (Figure 2-1).

In South African rivers 60 to 80% of the transported sediment is not sand (bedload) but silt and clay. These fine fractions (often called washload) has a near uniform vertical and lateral distribution and therefore it is difficult to apply the sediment rejection principle when considering the total load, using secondary currents at a bend or by elevated intakes. Diverted fine sediment could lead to sedimentation in the diversion structure, but is often not harmful to pumps and pipelines. Pumps and pipelines are however generally very sensitive to sand transport and bedload sediment rejection is an important consideration in most South African river diversion designs.

The local geology of the proposed diversion structure is an important parameter to take into account. Factors such as the stability of the riverbanks and additional stabilisation measures to stabilise the intake should all be taken into account when selecting the diversion location. Special attention should be given to bends of meandering rivers since they generally erode rapidly, and cut-offs could occur that may lead to the river bypassing the intake altogether. Braided gravel-rivers can create essentially the same problems as meandering rivers. The individual stream channels change their locations and therefore the channel pattern could be different from that before. Therefore, favourite locations for diversion structures include stable bends, cliff faces and gorges (Raudkivi, 1993).

Bouvard (1992) and Avery (1989) found that the diversion location should always be located on the concave bank. Secondary current phenomena in bends will concentrate bed load material on the inside (convex) of the bend. Off-takes should almost never be placed on the convex side of the bend.

The intake should be located on a stable reach to ensure that the intake is directed to the main current and that the flow path does not wander. The optimal location of an intake is usually just below the vertex of a concave bank (Tan, 1996). Historical aerial photography and satellite images are very important to evaluate the stability of a particular river reach.

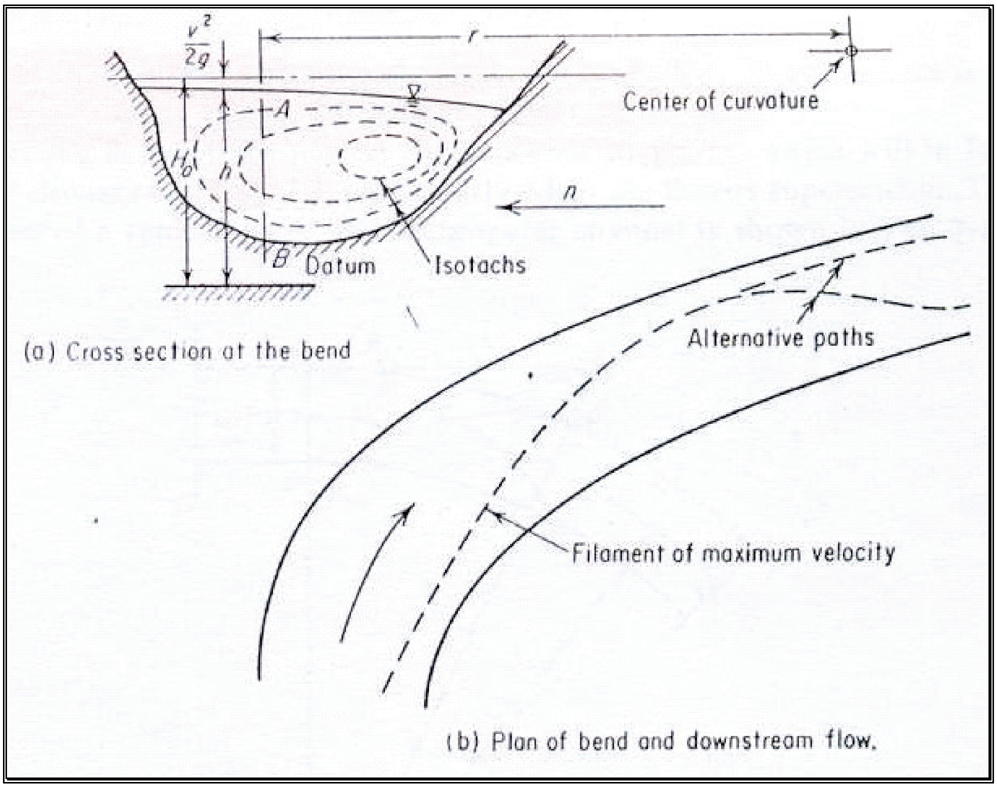


Figure 2-1 Flow behaviour at a channel bend (Henderson, 1967)

Several empirical rules were developed in the past to determine the best location of the intakes on the outside of a bend in the river. The predicted positions of the various scour holes on the many different channels call for a look at the accuracy of certain models claiming to predict the scour position.

The relationship between the central angle of a bend and the optimal location based on experimental data (SC and Ches, 1992) is given in Table 2-2.

Table 2-2 Relationship between central angle of a bend and optimal location of intake (SC and Ches, 1992)

Central angle of bend (˚)	<45	60	90	120	150	180
Optimal location of intake (˚)	0 (end)	45	60	80	95	110

SC and Ches (1992) found that the optimal location of an intake can also be related to the width and radius of curvature of a bend by the following empirical equation.

$$L = \xi\, B \sqrt{\frac{4R_c}{B} + 1} \qquad \text{(2-1)}$$

where L = distance

ξ = 0.8 (coefficient)

R_c = average radius of curvature

B = river width

The method of Raudkivi states that the deepest scour hole will form two breadths away from the upstream river axis intercept point with the bank. It was found as part of this reseach project that in a 0.3m curved channel sediment related tests corresponded very well with this method. A 0.6m laboratory channel, however, did not conform to this method. The scour hole formed further downstream from where it was predicted (Brink et al., 2005).

A similar verdict exists for sinusoidal channels which were analysed by mathematical model. The 70m wide channels formed their deepest scour holes where the Raudkivi predicted, but 20m wide channels did not concur. Their scour holes were to be found further downstream. Certain trends can however, be identified. By inspecting the figures of all the sediment related tests that were conducted it can be seen that the wider a channel is, the further downstream from the curve apex the deepest scour will be observed (Brink et al., 2005).

Two dimensional (quasi-3D) mathematical modelling or physical hydraulic modelling of the sediment dynamics around a river bend should be carried out at important pumpstations or diversions. Figure 2-1 shows a mathematical model simulation of two river bends and the location of the deep scour holes that forms during a flood that was carried out during this study (Brink et al., 2005). A 70m wide channel with a sinuosity of 1.24 and a sediment size of 0.5mm were used. A discharge of 300m^3/s through the channel resulted in a water depth of 2.04m. This discharge relates to a 1:10 year recurrence interval flood for the size of the river based on regime theory. After a simulation time of 11 hours the bed level changes shown in Figure 2-1 occurred.

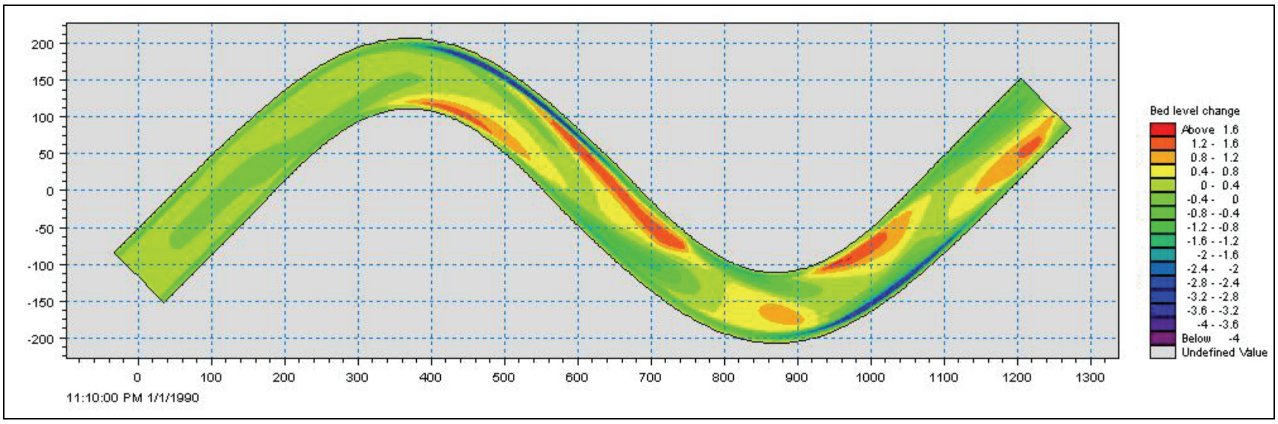


Figure 2-2 Simulated scour and deposition in a 70 m wide river channel (Sin 1.24 Q=300m^3/s, d=0.5mm)

The deepest scour hole present is 4.31m deep. It occurred downstream of the vertex of the curve roughly two breadths after the upstream intersection point. Many sand banks also formed due to the large volume of sediment that was scoured out at the bends. The sandbanks also indicate that the first bend does not influence the scour observed in the second bend. This fact can be more readily seen in Figure 2-2. It shows the helical flow intensity in the channel and it is clear that almost no secondary flows exist in the middle part between the two curves.

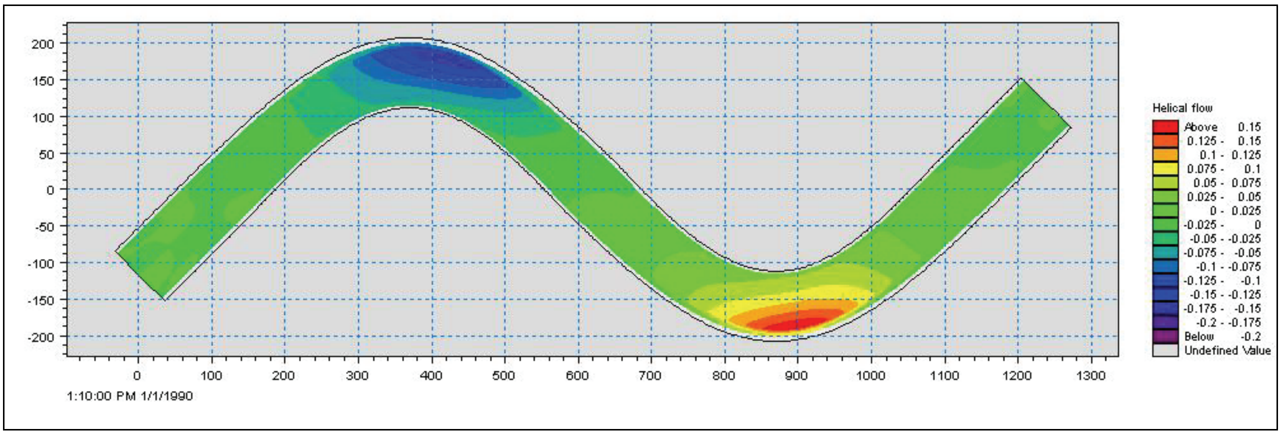


Figure 2-3: Simulated helical flow intensity for a sin 1.24 channel

In general Table 2-1 is recommended for use to predict the location of deepest scour around a river bend for planning purposes. Mathematical modelling (2D with spiral flow module) is however recommended for detailed design. More details on calibration and simulations with sediment transport at a river bend are provided in Brink et al. (2005).

3 ABSTRACTION TYPES

3.1 GENERAL

Abstraction work intakes can be grouped into the following categories:

a) Intakes without a weir or barrage

- artificial bend intakes
- bank intakes
- bottom intakes
- submerged intakes

b) Intakes with a weir or barrage

- bend intakes
- bank intakes
- frontal intakes
- tiered intakes
- bottom grate-type intakes

c) Sand abstraction systems

It is possible to improve the efficiency (to divert less sediment) of intakes (categories (a) and (b)) by using groynes, guide banks and walls, guide vanes, dividing walls, sand guiding sills and sediment intercepting galleries.

Design details, benefits and disadvantages, and some case studies of the above intake types which are not all suitable for South African conditions, are discussed in detail in Volume 1 of this study (Brink et al., 2005).

When selecting a diversion type the most important consideration is to maintain the natural sediment balance in the river as far as possible. If this is not done the long-term implications could be a non-sustainable design. Furthermore the diversion must be able to cope with extreme conditions in the climate such as several years of drought and periods of large floods with high sediment loads.

The following are proposed designs for South African conditions, in order of priority:

f) Pumpstation/diversion without a weir, located on the outside of a stable river bend
g) Low weir with low level flushing gate(s) in a gravel trap, and pump canals/sand trap that can be flushed, combined with a bend in the river
h) Barrage on river with large gates across the river, combined with a stable bend in the river
i) Weir, flushing canals, deep sand trap (pit) and jet pumps to clean the pit
j) Sand pump system with infiltration gallery

3.2 Pump station/diversion without a weir, located on the outside of a stable bend

3.2.1 General

A typical layout of such a pumpstation is shown in Figure 3-1. The abstraction works location should be determined as described in Section 2.

3.2.2 Benefits

- No/small impact on river sediment balance
- Cheap and ideal for small, relatively high risk of supply failure designs, such as irrigation
- Full use of secondary currents to limit sediment diversion
- Diversion structure protruding into flow help to create deep pool at intake

3.2.3 Disadvantages

- Low flow in sand bed river could meander away from the bank if not well positioned
- Flushing potential of abstraction works limited if river is not steep locally
- Pump intake head limited during low flow conditions

Figure 3-1 Lower Mfolozi River pumpstation

3.3 Low weir with low level flushing gate(s) in a gravel trap and pump canals/sand trap that can be flushed, combined with a stable bend in the river

3.3.1 General

These designs usually use the head created by the weir to flush sediment locally out of the abstraction works. Designs could have a high wall at the sluiceway (gravel trap), but usually the gravel trap wall is submerged. In conditions of high bedload transport (cobbles and boulders), a boulder trap could be designed in addition to the general trap, and is often curved to limit sediment diversion.

3.3.2 Benefits and disadvantages

- Limited flushing locally at diversion possible with additional head created by weir

- Low maintenance on weir versus gated structure (barrage)

Disadvantages:

- Sediment deposition in the river upstream of the weir with raised flood levels upstream
- Sedimentation also reduces the balancing storage at the weir and most diversion weirs are filled up with sediment in a short period of time

3.3.3 DESIGN WITH HIGH GRAVEL TRAP WALL

It is difficult to recommend one specific abstraction works design due to site specific conditions. Rooseboom (2002) however proposed a general design with the components shown in Figure 3-2.

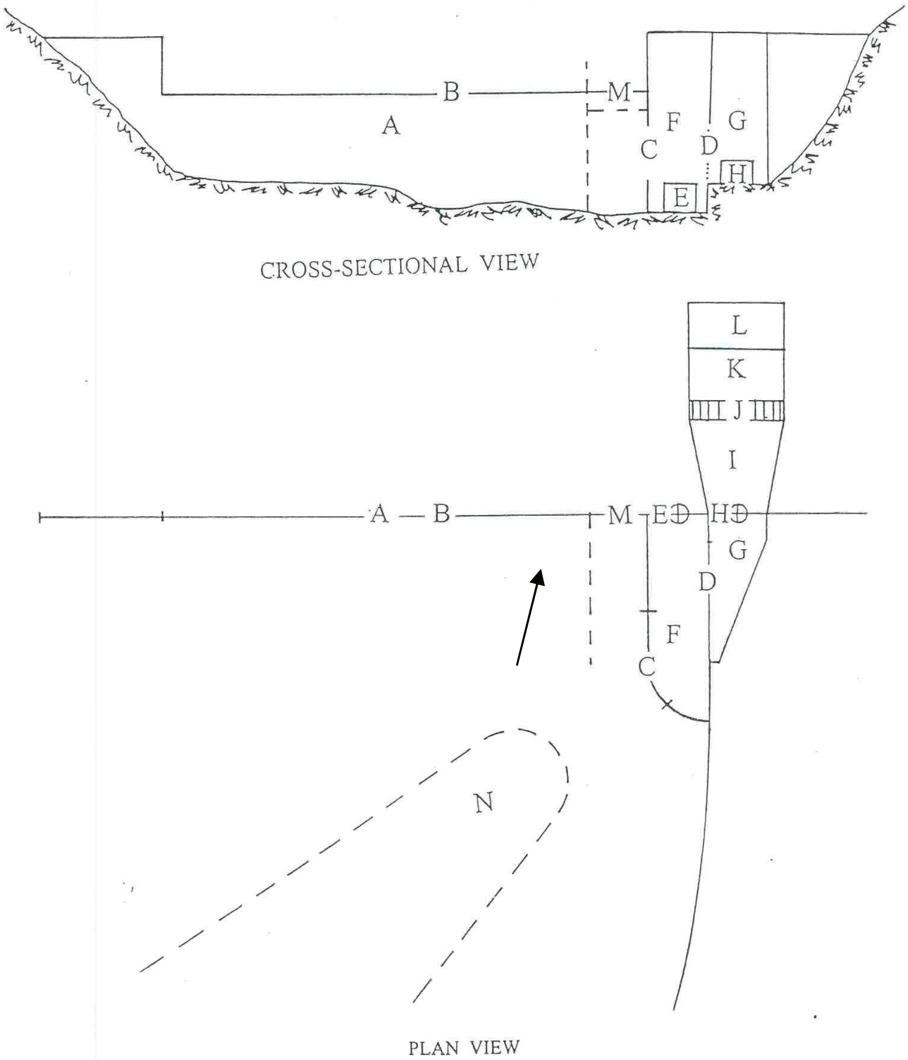


Figure 3-2 Proposed typical diversion layout for South Africa (Rooseboom, 2002)

Figure 3-2 Legend:

Weir	A	Control gate(s)	H
Spillway	B	Transition channel(s)	I
Open intake through non-submerged wall	C	Vortex suppressor	J
Screen intake (trashrack)	D	Settling basin	K
Scour gates	E	Pumps	L
Scour chamber	F	Low notch weir	M
Collection channel	G	Groyne	N

The open intake (C) should keep floating debris out by placing the soffit of the intake below the water level which is normally the weir crest level at low flows. Flow velocities through the opening must also be low enough to prevent objects from being sucked through. The bottom of the opening must be high enough to create sufficient gradient to flush out sediments from the scour basin and to allow for sediment deposition between flushings.

The screen (D) stops suspended debris. The screen openings are determined by the sediment diameter the pumps can deal with. The upper edge of the screen should be below the water surface to limit the entanglement of floating debris.

The scour gate (E) must be low enough to keep sediment levels down and must discharge freely.

The scour chamber (F) traps sediment, but is also shaped to induce scour along its outside perimeter, similar to a bridge pier, to limit sediment build-up around the intake during floods. The outer wall of the scour chamber should be streamlined and its downstream section should run parallel with the flow direction (in plan) to pass floating debris over the spillway.

In the collection channel (G) the velocities should be relatively high and constant to limit sediment deposition. The channel floor is therefore raised and it widens downstream.

Control gates (H) should be kept as small as possible due to their high cost, but this leads to high downstream velocities which should be dissipated to have smooth uniform flow conditions at the pumps.

A settling basin (K) can be used to settle out sand.

The pump layout (L) can be either a wet well or drywell installation.

The low notch weir (M) serves two purposes in that it maintains the low flow channel near the intake and it passes floating debris. A guide wall upstream of the low notch will further help to pass the floating debris.

Groynes (N) can be used to concentrate the flow at the intake and to increase the curvature of the flow lines to create an outside of bend pattern at the intake.

Comments on this layout:

- Its not possible to flush the upstream part of the scour chamber (F), but this should not affect the operation

- The pump canals (K) and (L) cannot be flushed and sediment has to be removed mechanically
- Trees cannot enter the scour chamber (F)
- Secondary current development to create local scour against the structure will be limited due to the presence of the weir and its low notch weir is often at a similar level as the invert of the open intake (C), which makes it susceptible to sediment deposition
- During flushing the gravel trap (F) acts like a side channel spillway with the risk that water could jump up on the opposite side through the trash rack

3.3.4 OTHER TYPICAL DESIGNS WITH SUBMERGED GRAVEL TRAP WALL

Following a comprehensive literature survey of international and South African diversion works, the general layouts with similarities as shown in Figures 3-3 to 3-11 were found.

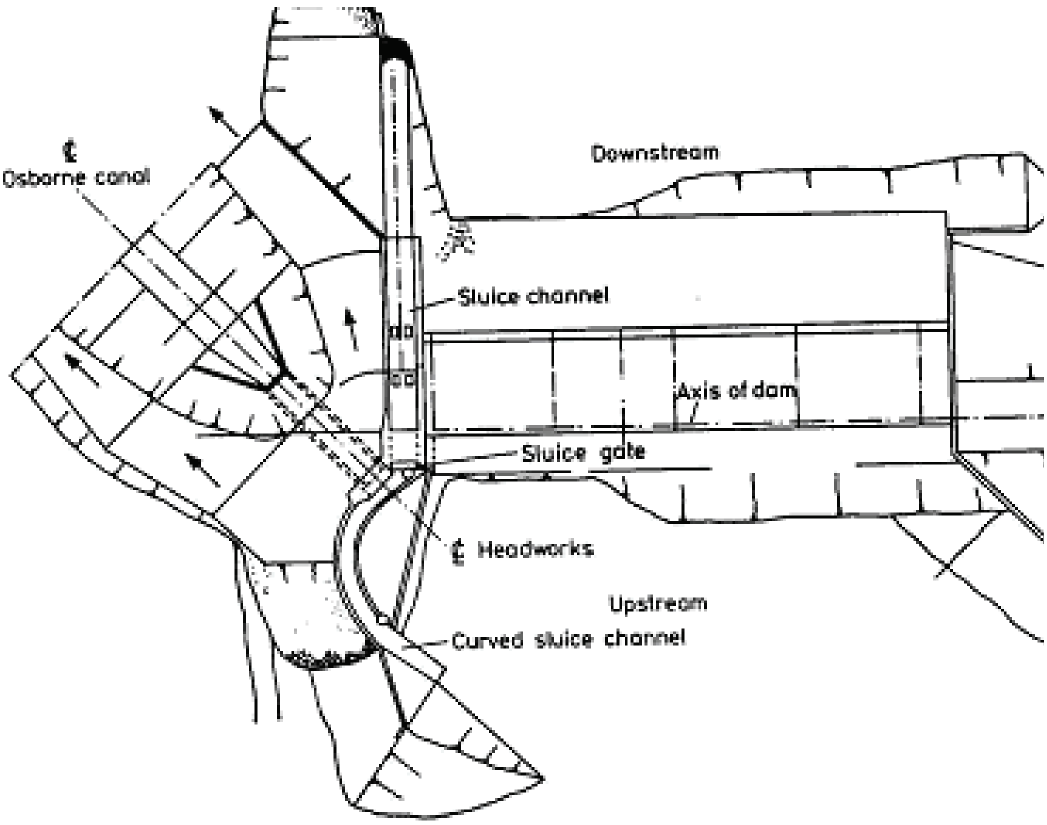


Figure 3-3 Separate curved sluice channel for sediment exclusion (Avery, 1989)

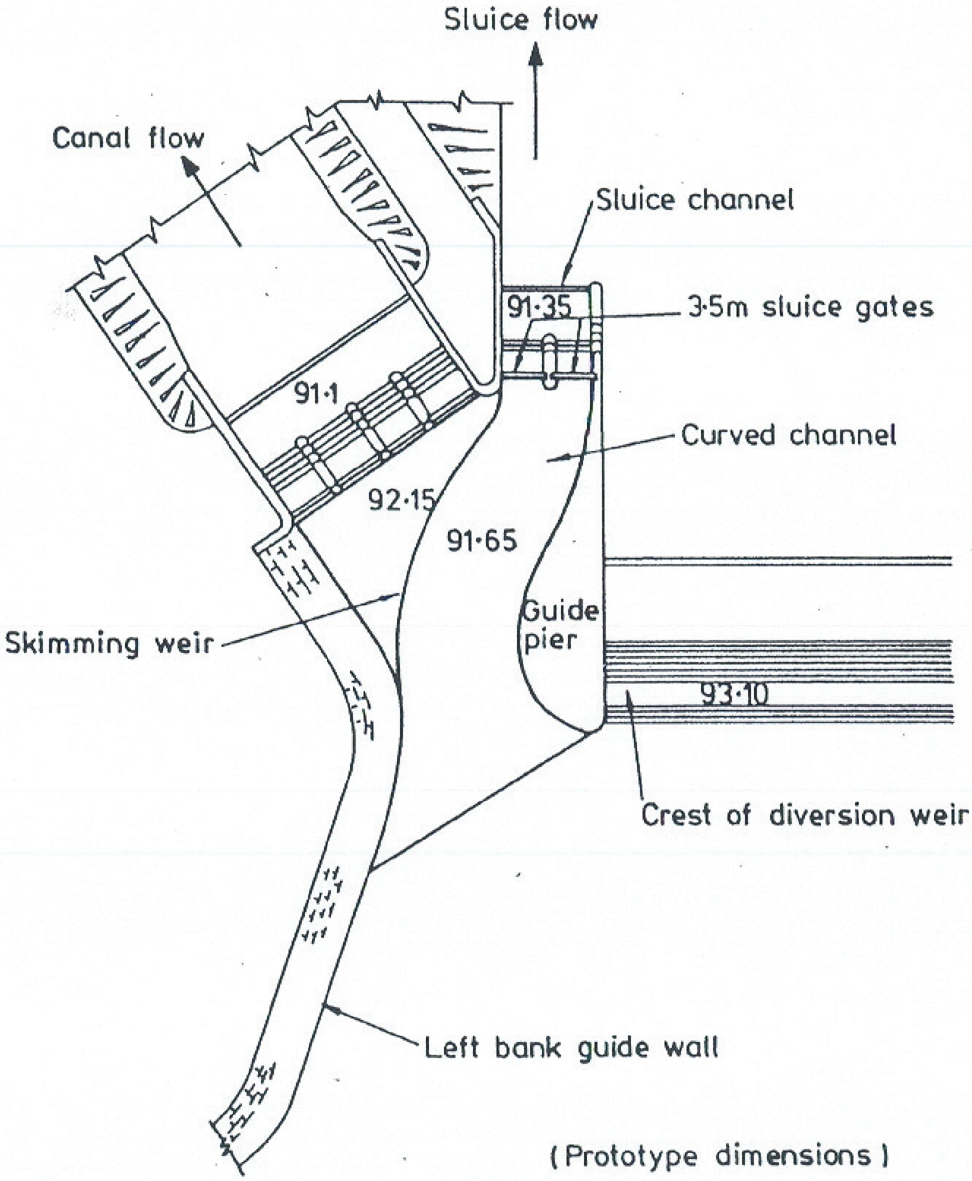


Figure 3-4 Separate curved sluice channel for sediment exclusion with a skimming weir (Avery, 1989)

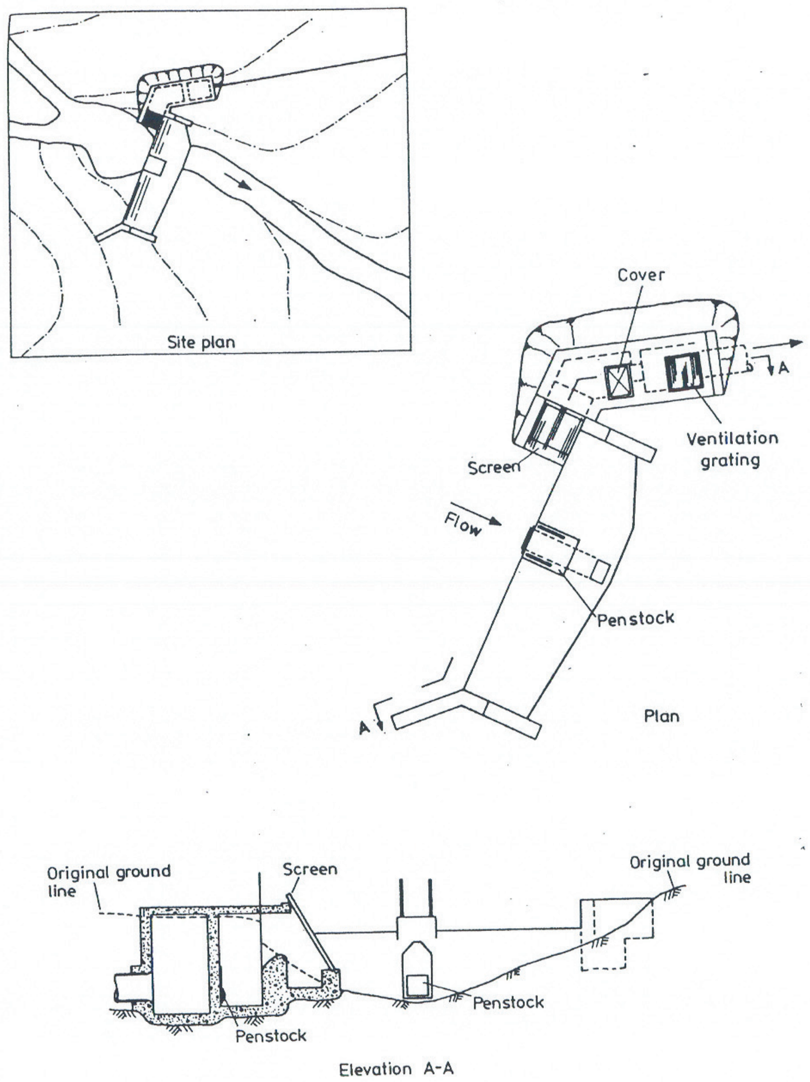


Figure 3-5 Side intake with a cross weir (Avery, 1989)

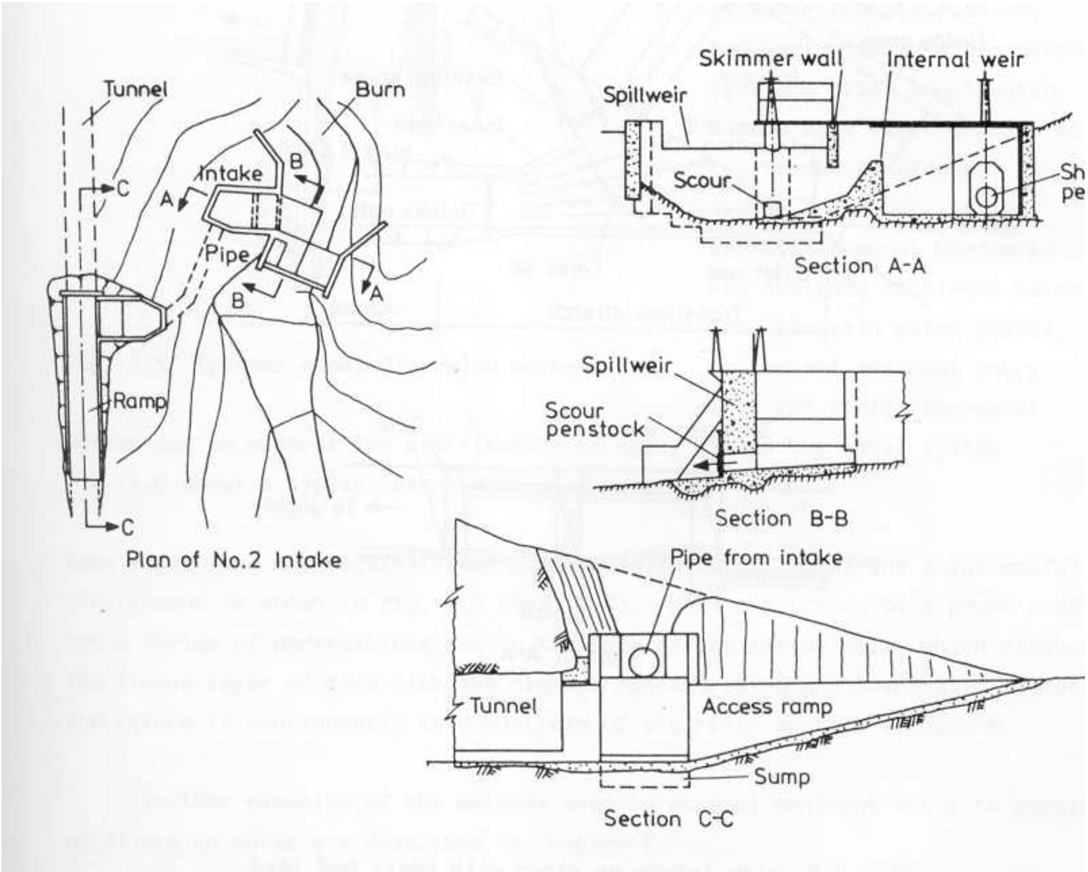


Figure 3-6 Screenless side intake with a cross weir (Avery, 1989)

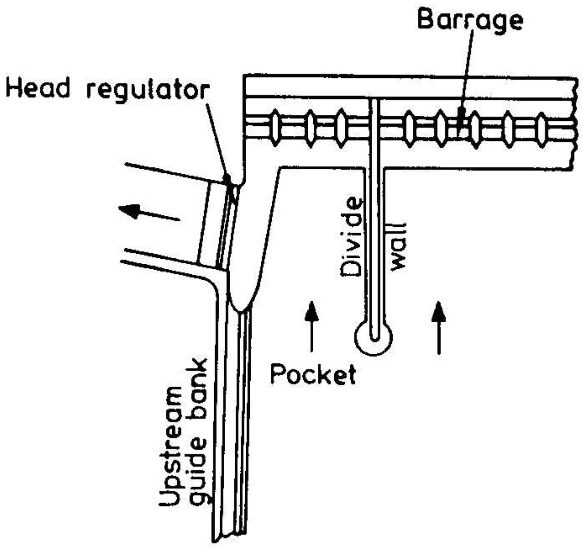


Figure 3-7 Low head river or canal diversion works (Avery, 1989)

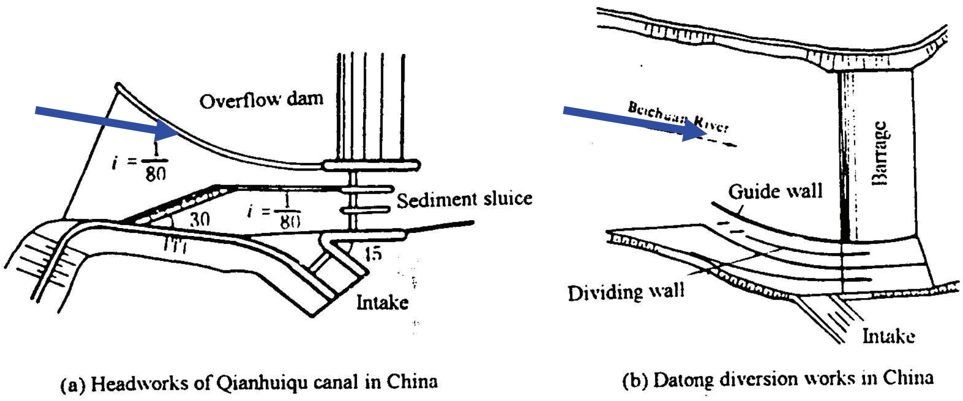


Figure 3-8 Curved sluicing flumes at (a) Headworks of Qianhuiqu canal, China and (b) Datong diverson works, China (Tan, 1996)

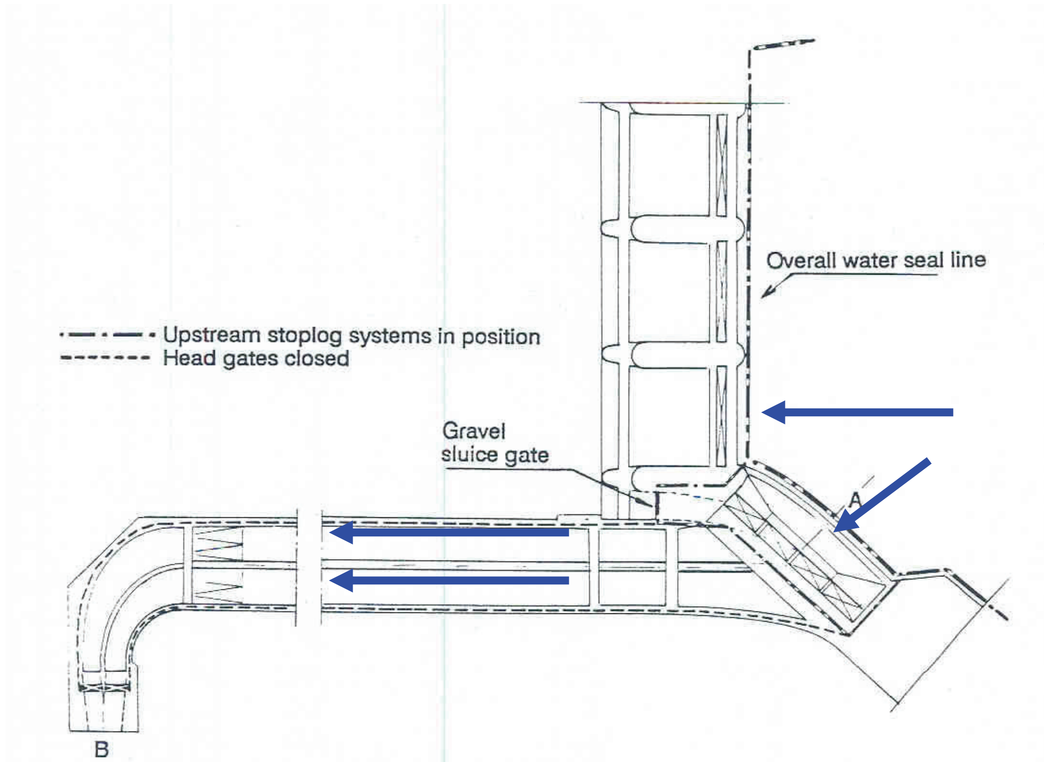


Figure 3-9 Pressy Water Intake (Bouvard, 1992)

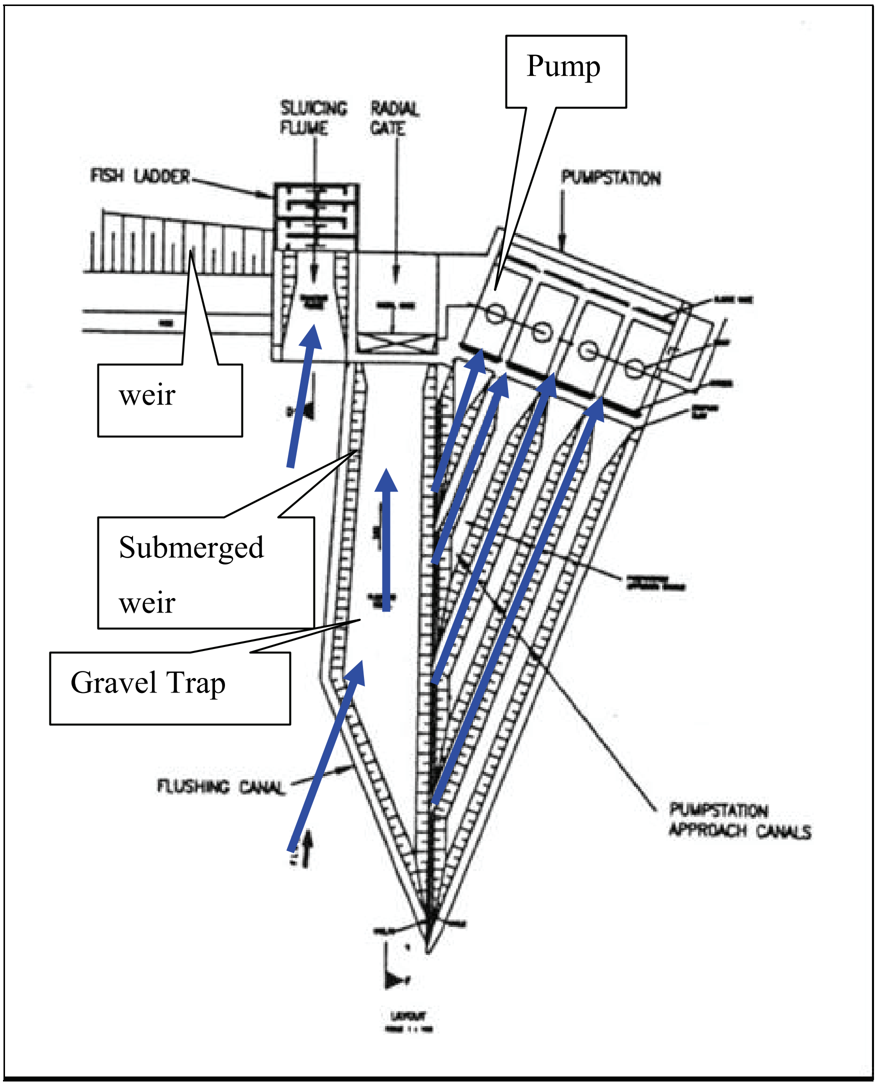


Figure 3-10 Lebalelo pumpstation layout, Olifants River

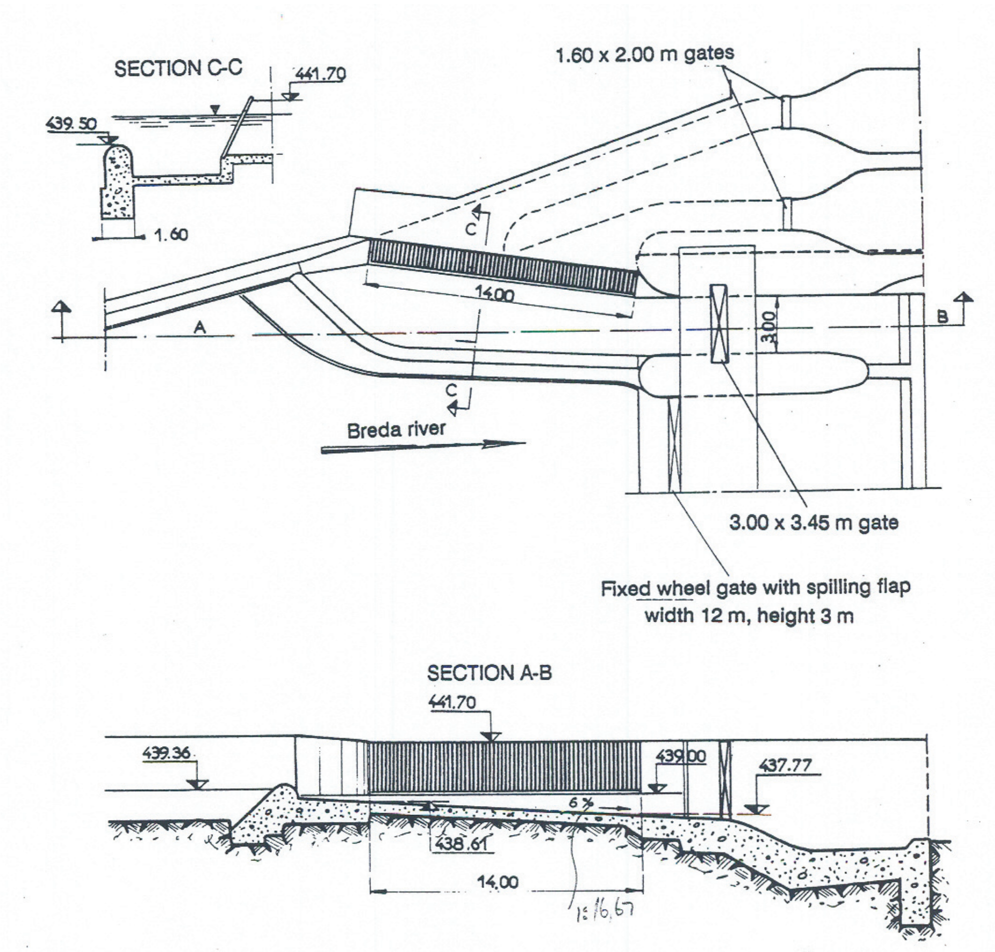


Figure 3-11 Overpour-channel gravel sluice at the water intake on the Breda (Bouvard, 1992)

The above designs have a have number of aspects in common:

- River curvature is used to locate the diversion, and the designs try to exclude any bedload by using layouts as shown in Figure 3-12 and with the inclusion of a gravel trap to flush coarse sediment downstream.

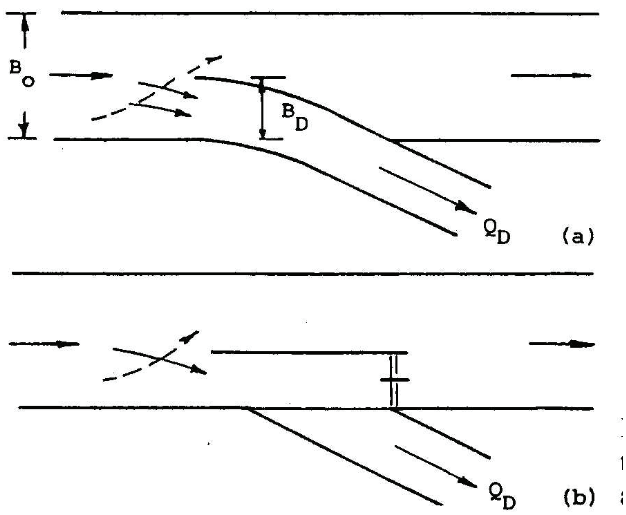


Figure 3-12 Creation of flow curvature with the aid of dividing walls and/or sluices (Raudkivi, 1993)

- The sluiceway structure is ordinarily designed to prevent larger sediments from entering or being deposited in front of the adjacent intake structure. This may be accomplished by providing a radial (undershot) gate on the sluiceway that would be operated as needed to draw or flush the sediments away from the intake.

  The potential for the sluiceway to also pass floating debris may be considered, depending on the specifics of the layout and expected operating conditions.
- Upstream of the gravel trap a submerged weir is found that acts as control during flushing of the gravel trap. The crest level of this weir must not be too low below the operating water level, so that the gravel trap flow does not submerge the control during flushing. The submerged weir should have a negative slope on its crest. The bed slope of the gravel trap should be at least 1:50 with scour velocities between 2 to 4 m/s. Critical conditions for re-entrainment of the coarse sediment has to be considered, and in the rough turbulent flow zone this equites to d = 11Ds, where d = sediment diameter that could be re-entrained, D = flow depth in the gravel trap during flushing, and s is the energy slope, approximately equal to the bed slope.
- The intake structure is ordinarily located immediately adjacent to the sluiceway. For flow control, the intake structure may be equipped with either slide gates or radial gates depending on the required diversion capacity. The inverts of the intake gates are

typically set above that of the sluiceway gate as an added measure to keep sediments out of the conveyance system.

- A trashrack may be required to keep floating debris or fish out of the conveyance system. A trashrack is placed upstream of the intakes to the sand trap or pump canals.
- The benefit of the Lebalelo design is that the pump canals are covered and do not form any obstruction to the flow or debris during floods.
- With a submerged graveltrap wall the sediment deposited upstream along the length of the weir can be flushed out to maintain the low flow channel, while with the Rooseboom (2002) layout only local scour will occur at the side wall intake and more reliance will be placed on floods to scour sediment along the length of the wall (Figure 3-13).
- It is also important that the diversion structure is not located against the river bank, but pushed into the main channel to ensure the low flow channel remains against the structure.

Figure 3-13 Gravel trap flushing, Lebalelo

In bedload dominated rivers such as in the Western Cape, it is recommended that a boulder trap is added to the gravel trap in the design, as was used in the final design of the Berg River Supplement Scheme.

3.3.5 DESIGN TO CONTROL HIGH BEDLOAD

The design of the Berg River Supplement Scheme abstraction works, located about 13 km downstream of the Berg River Dam on the Berg River, is a design that incorporates most of the above design principles used in the Lebalelo design, but also includes a fishway, canoe chute, curved concrete approach channel with boulder and gravel traps that can be flushed, a submerged boulder deflection wall, sand traps that can be flushed, and water level control gates. This design is the most suitable when the river transports a high bedload of cobbles and boulders. A photograph of the hydraulic model of the Berg River abstraction works is shown in Figures 3-14 and 3-15.

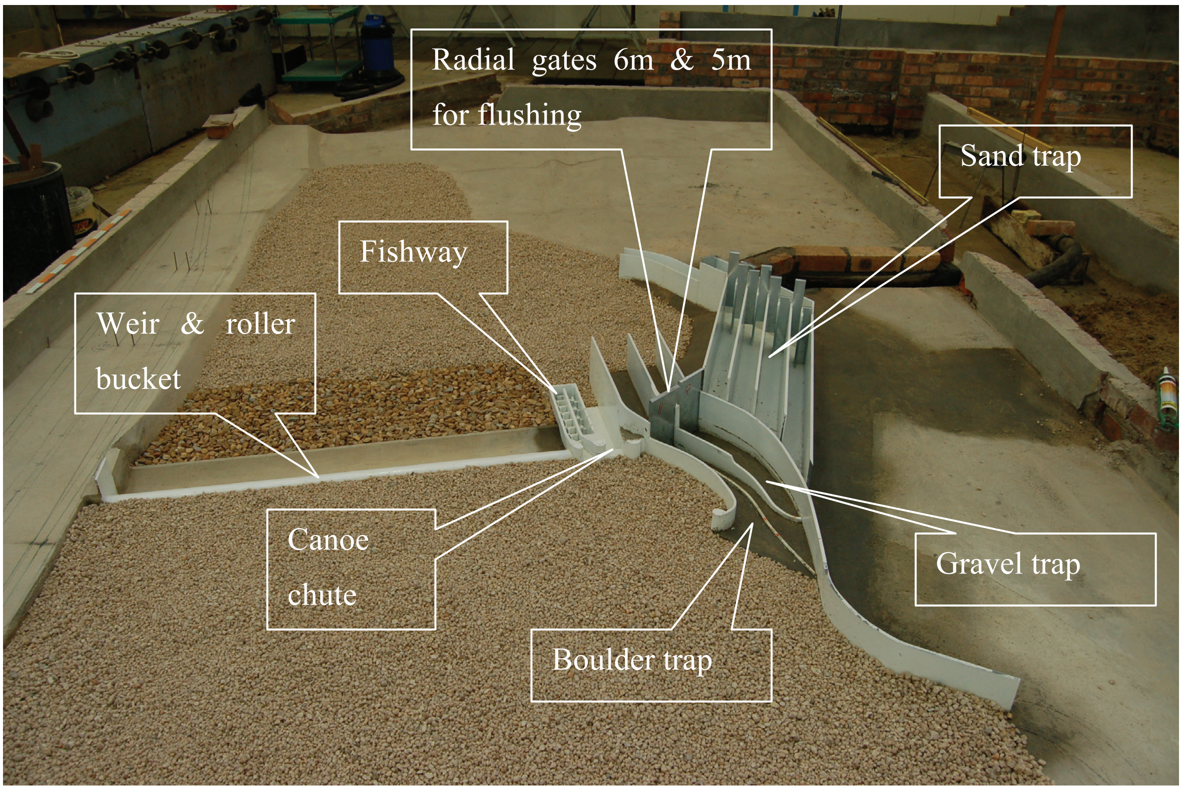


Figure 3-14 Berg River abstraction works layout

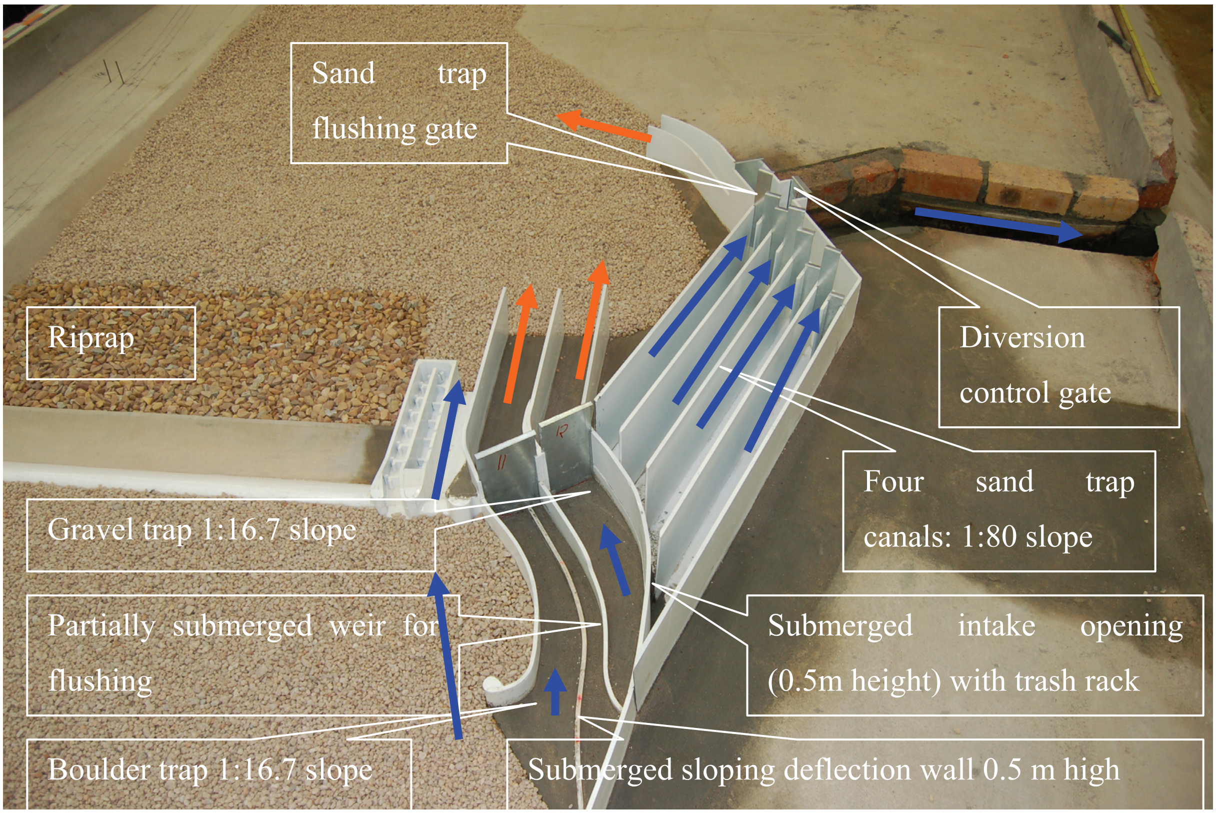


Figure 3-15 Boulder, gravel and sand traps at Berg River abstraction works

a) Design components:

- Boulder trap with 6 m wide flushing sluice gate. The boulder trap will allow flushing of boulders from time to time, which will keep the upstream end of the sand trap intake openings clear of boulders, which can only be removed mechanically in a design without a boulder trap. The flow through the boulder and gravel trap is curved so that cobbles and boulders transported at the bed are diverted to the inside of the bend during a flood, away from the intakes
- Gravel trap with 5 m wide flushing sluice gate
- Sand trap (4 canals)
- Canoe chute that's uncontrolled and can be used all the time, designed to pass 1.32 m^3/s at full supply level (weir crest level) (FSL) as part of the (instream flow requirement) IFR
- Fishway designed to pass 0.18 m^3/s at FSL as part of the IFR of 1.5m^3/s (minimum) to 2.9 m^3/s (maximum)
- Additional IFR flow above 1.5 m^3/s is obtained by opening the boulder trap radial gate

In this design the gates at the end of the sand traps are open during diversion and the water level and diversion discharge control is through the automatic radial gate located on the canal to the balancing dam. This means that up to 6m^3/s diversion the water level stays at FSL of 138.36 m. With this design the diversion efficiency will be high for river inflows in the range above the [IFR] to [6 m^3/s plus the IFR], since the diversion efficiency will become 100 % in this range.

b) Proposed operation:

i) Diversion/Deposition mode

- At inflows of < 1.5 m^3/s, the uncontrolled canoe chute and fishway pass all flow downstream
- If the IFR requirement is say 3 m^3/s, the additional flow is obtained by adjusting the boulder trap gate
- Diversion starts when the IFR is exceeded, by automatic gate downstream of the sand trap on the canal to the balancing dam
- The automatic gate downstream of the sand trap also limits the diversion to 6 m^3/s or stops diversion when the balancing dam is full, or closes when the water level at the weir drops below FSL
- If the river inflow is above 6 m^3/s + IFR, the boulder trap gate opens further to maintain the FSL of 138.36 m as long as possible, which will limit sediment deposition upstream during floods

ii) Flushing mode

- Sand traps close their end gates except one, automatic diversion gate closed, flushing radial gate opens. Flushing one canal at time
- Boulder and gravel trap flushing from time to time during floods

c) Conclusions on abstraction works designed with sluiceways

Basically the design shown in Figure 3-10 (with sand trap or with pump canal settlers) is recommended for use in South Africa for sand bed rivers, and the design such is in Figure 3-14 for high bedload rivers. The design by Rooseboom (2002) (Figure 3-2) will also work in sand bed rivers, but the following has to be considered:

- the spiral flow scour created upstream of the structure could only have a limited effect due to the effect of the weir and the typically long length and low invert level of the intakes that have to be free of sediment
- The gravel trap cannot be flushed effectively, especially at its upstream end

- The pump canal cannot be flushed

3.4 Barrage on river with large gates across the river, combined with bend

Gates could be radial gates such as at Phalaborwa Barrage, Olifants River, or hinged at the floor with automatic hydraulic water level control to fold flat on the bed during a flood such as at Maccaretane Barrage in Mozambique on the Limpopo River. These structures should be considered when the diversion discharge is high, when some balancing storage is required in the river and especially when the impacts of the structure on the river (flood levels and sedimentation) should be minimized. With regular flushing during floods at least 40 % of the original storage capacity can be maintained, but this could be as high as 80 %. To limit the environmental impact of flushing of sediment through the barrage, its however important that flushing is carried out during all floods and on a regular basis, with water level drawdown.

a) Benefits:

- Passing sediment through during floods (sluicing) with limited damming and sediment deposition, thereby also limiting flood levels upstream.
- Probably the only feasible design on large rivers with high sediment yields such as the Limpopo and Olifants (Limpopo Province), where balancing storage is required on the river.
- Balancing storage is created upstream of the barrage

b) Disadvantages:

- Expensive design with high maintenance and operational cost
- Judicious operation required during floods for safety and to limit sediment deposition
- Possible tree blockage if gates are too small
- Anaerobic sediment flushing could lead to fish kills if the flushing duration is long during relatively small floods

3.5 WEIR, FLUSHING CANALS, DEEP SAND TRAP (PIT) AND JET PUMP TECHNOLOGY

Recently, the Department of Water Affairs and Forestry (DWAF) favours as design which uses a jet pump to clean the sand trap which forms an integral part of the abstraction works. In February 2003 the Hoxani pumpstation was commissioned on the Sabie River. Figure 3-16 shows the weir across the river with the abstraction works on the left bank. There is also another pumpstation on the right bank, not shown, which is actually on the outside of the bend in the river. Figure 3-17 shows the plan layout of the abstraction works. Water flows into the first canal which now also has a Crump weir at its upstream end, and is diverted to a second canal over a side weir. Both canals have radial gates that are currently operated in closed position, but can be opened to flush out deposited sediment. The canals are about 23 m in length. From the second canal water is diverted into the sediment pit through a grid in the top of the canal and opening in the side wall. This grid which is submerged acts as trash rack since the specified jetpump can only pump sediment particles smaller than 40 mm. The jet pump uses water from the main pumps to operate.

The sediment pit (Figure 3-18) has a concrete roof slab to prevent sediment entering during floods. The pit has steep side slopes, is excavated in rock and is 7 m deep. The plan dimensions of the pit at the surface is about 9 m x 6 m (width x flow length). With such a short length only coarse sand would settle out. The full width of the pit is also not effective as has been observed in the field. Also typically due to high turbulence the entrance zone is also less effective in depositing fine sand. The effective flow depth at the pumps would be about 3 m, with the result that fine sand (0.03 mm) can reach the pumps when the approaching flow velocity through the pit is about 0.15 m/s.

The pump location perched at the end of the pit is not an ideal layout since the approaching flow pattern is not uniform, but the flow velocities are low.

The weir is about 2 to 2.5 m high above the bed level.

The key concerns with this layout are:

- Ineffective (short length and rapid transitions) and expensive sandtrap when about 70 % of the sediment transported during floods is silt and clay.
- Heavy reliance on power supply for the jetpump, especially during floods
- Jet pump technology which is not well proven

The last two bullets above is illustrated best when fine sediment (silt and clay) deposits in the pit during periods when the main pumps are not working and consolidates which could make it difficult for the jetpump to remove since its more suitable to re-entrain sand.

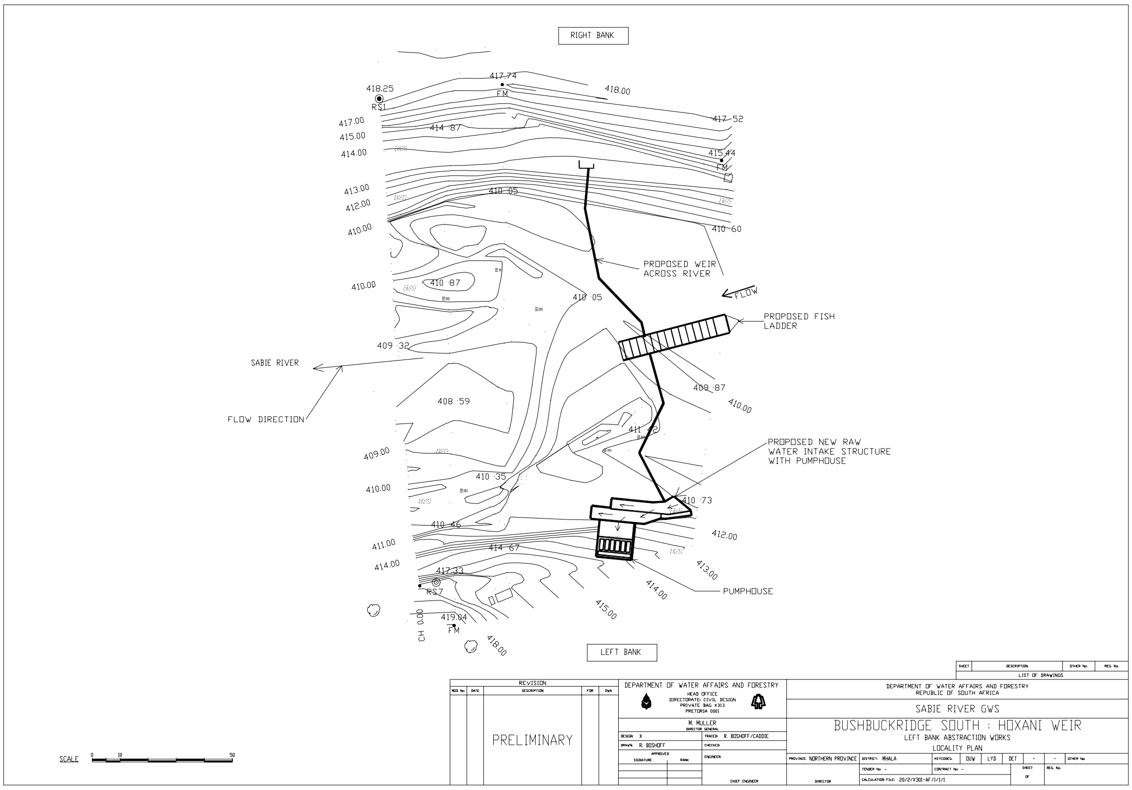


Figure 3-16 Hoxane abstraction works on the Sabie River (DWAF, 2003)

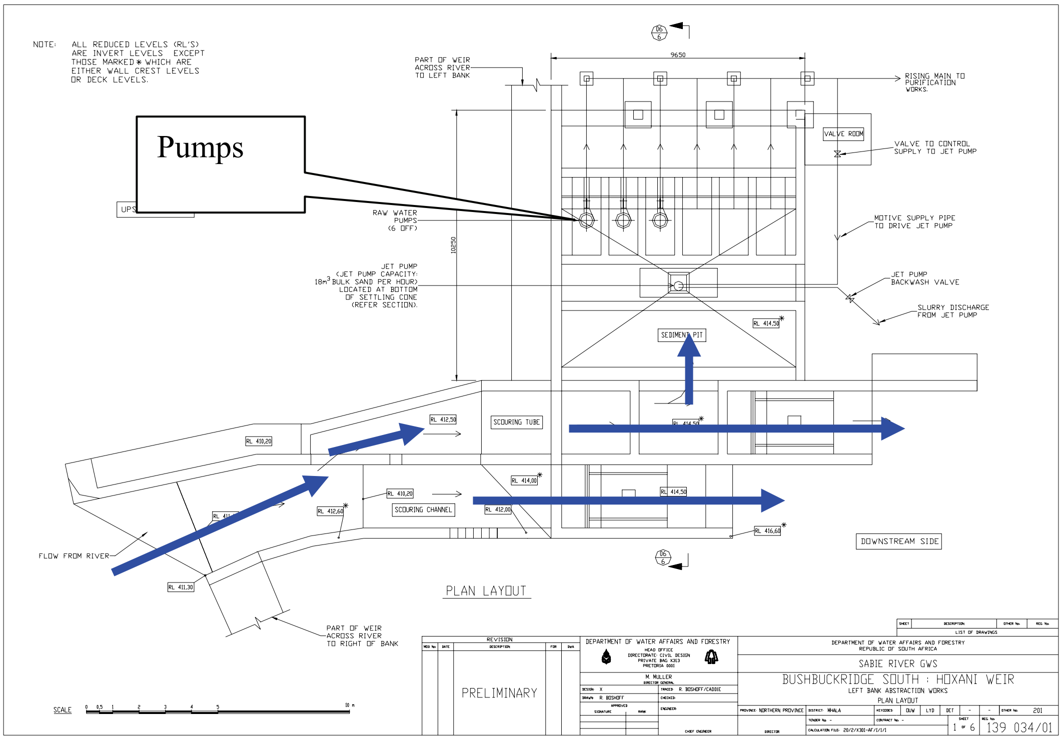


Figure 3-17 Hoxane abstraction works layout (DWAF, 2003)

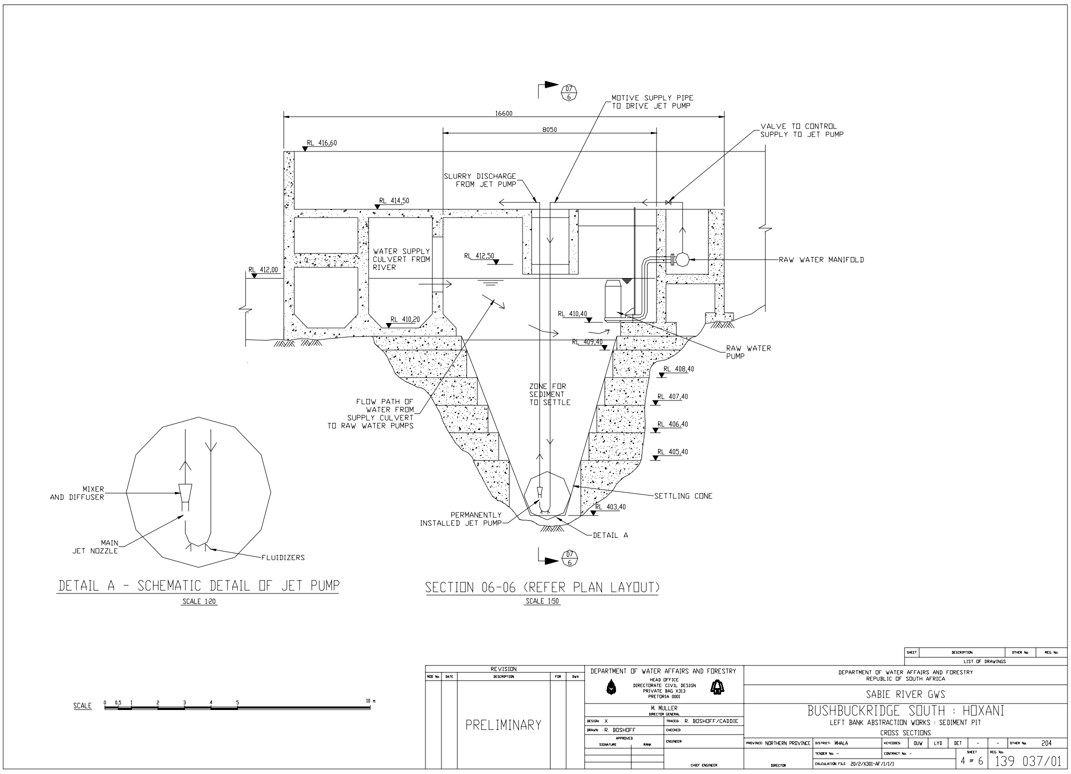


Figure 3-18 Hoxane Sand trap (pit) elevation with jet pump (DWAF, 2003)

3.6 SAND PUMP SYSTEM WITH INFILTRATION GALLERY

3.6.1 TYPES OF SYSTEMS

The sand abstraction systems that one encounters in the field are as varied as the people who design them. Systems can, however be described under various general types of abstraction systems, which are discussed below.

3.6.2 CAISSON TYPE SYSTEMS

These systems generally incorporate a large diameter vertical caisson installed in the sand of the riverbed or the riverbank. The level of sophistication of caisson type systems varies from simple systems constructed using precast hollow blocks or even large perforated steel drums to structures constructed using precast manhole rings, reinforced concrete or no-fines concrete.

Provision must be made for infiltration of water into the caisson. This can be done in various ways, including:

- The use of no-fines concrete for the construction of the caisson. No-fines concrete is permeable, thus water can infiltrate into the system through the entire caisson surface.
- The inclusion of slots or openings in the caisson walls. Slots should be covered with a screen or mesh to prevent the ingress of sand into the caisson. A typical caisson with slot openings is shown in Figure 3-19.
- Use of selected aggregate or stone to construct the floor of the caisson. This will allow for infiltration of water through the floor of the caisson. Again, it is recommended that the surface of an aggregate floor be covered with screen mesh to prevent the ingress of fines into the system and retain the aggregate in place. Where caissons have both slot openings in the wall and an aggregate floor, it has been found that the majority of water infiltrates through the floor of the caisson. It is therefore recommended that caissons be constructed with a permeable floor, either packed aggregate or no-fines concrete.
- Horizontal well screens connected to the caisson can be installed in the sand bed surrounding the caisson. Water infiltrating the well screens will flow into the caisson. If multiple screens are installed these will be arranged radially from the caisson. A typical system of this type is shown in Figure 3-20.

Caissons lend themselves to the installation of submersible pumps. Water can then be pumped directly from the caisson to storage or treatment facilities. Alternately, an outlet pipe can be installed in the bottom of the caisson, connecting the caisson to the wet well of a pump station, or being connected to the suction of a centrifugal or mono-type pump. Where more than one caisson is installed at a site, a common collector pipe can be used to deliver water from all caissons to the pump station.

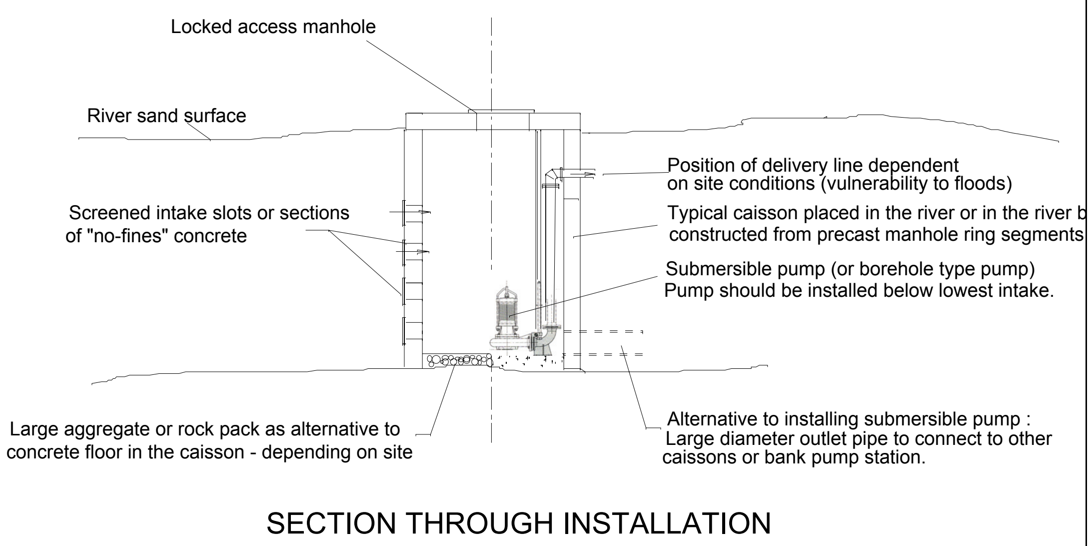


Figure 3-19 Caisson with Intake Slots or Sections

It is recommended that where possible caissons are founded on bed rock, being anchored to the rock by means of rock dowels or similar.

Although numerous successful applications of caissons exist, it has been found that caissons with slot openings are not ideally suited to conditions where there are a high percentage of fines in the river sand. Fines tend to enter the caisson and there is a build up of sludge in the bottom of the caisson under these conditions. Caissons with horizontal well screens with appropriate slot sizes would be more suited to these conditions.

Caissons are also not ideally suited to installation in rivers where extremely high flood flows occur, with associated bed fluidisation of the sand at depth. Caissons can then begin to "float" within the sand and can be overturned. Caissons installed in the riverbank are less susceptible to flood damage. The parameters of the alluvium in the riverbank must, however, be carefully investigated to ensure that sufficient infiltration can be achieved if the caisson is to be installed in the riverbank.

Caisson type systems are most suited to conditions where:

- The depth of the sand bed varies between 3 and 5 m. This gives sufficient depth for infiltration into the caisson, and allows for founding the caisson on the bedrock.
- The fines content of the river sand is not high, unless horizontal well screens are to be used.

A disadvantage of caisson type systems is that they do not lend themselves to backwashing or development of the sand around the caisson.

3.6.3 INFILTRATION GALLERIES WITH HORIZONTAL WELL-SCREENS

Infiltration galleries with horizontal well screens incorporate a horizontal gallery installed in the riverbed or riverbank. Horizontal well screens are connected to the gallery, near the invert of the gallery. These well screens project into the riverbed. The well screens are normally parallel to each other, although screens can be installed in the ends of the gallery, projecting perpendicularly to the screens installed in the sides of the gallery. A typical infiltration gallery with horizontal well screens is shown in Figure 3-20.

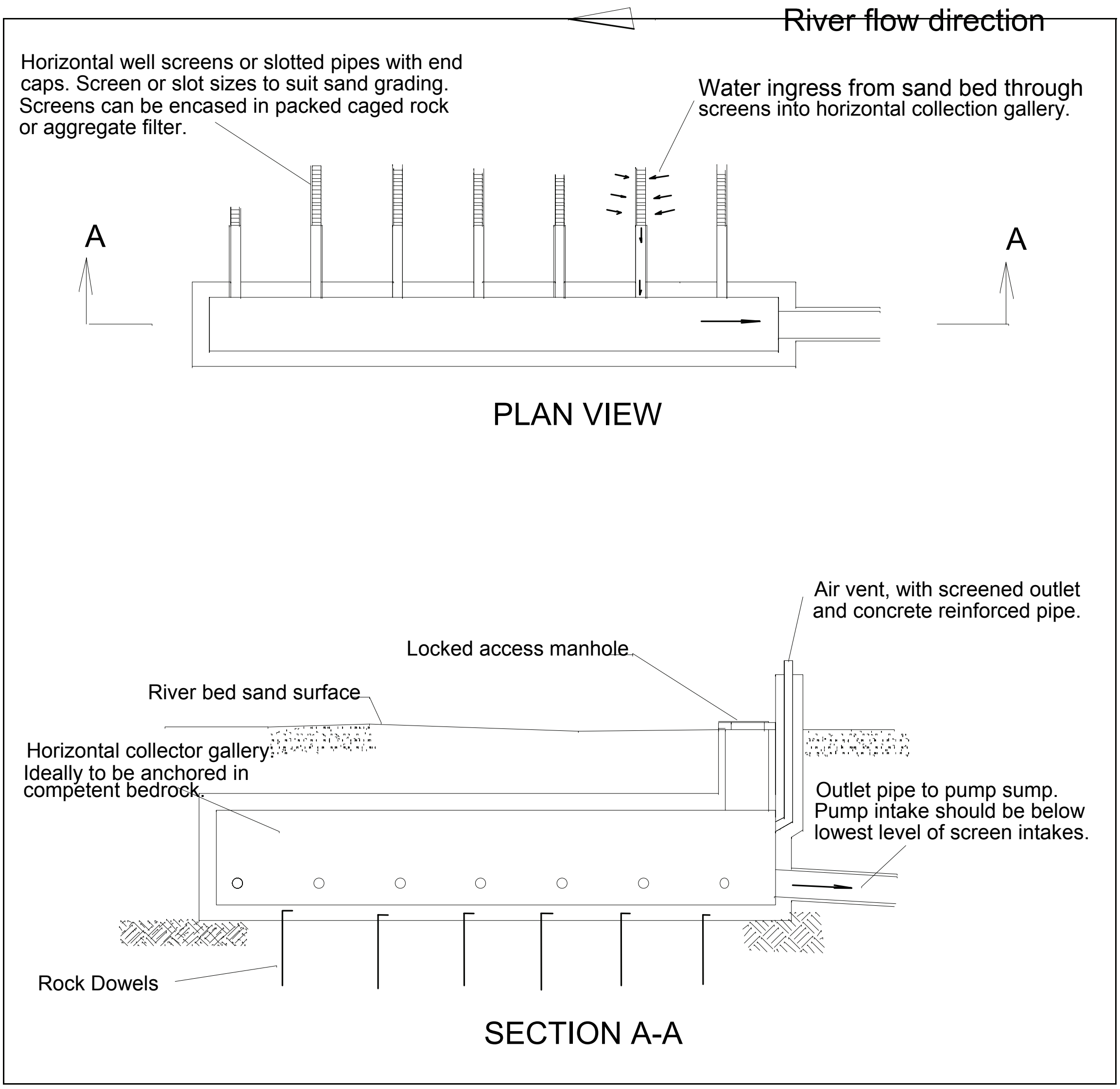


Figure 3-20 Collector Gallery with Horizontal Well Screens

The length, diameter and slot size of the well screens will be determined by the parameters of the sand, and the required yield of the system. The screen diameter is also governed by the need to minimise head loss for flow through the screen pipe.

The collector gallery would normally be constructed from reinforced concrete, but other construction materials such as blockwork could also be used. It is recommended that the gallery be founded on bedrock, being anchored to the rock by means of rock dowels or similar.

Construction of these types of systems can be lengthy, and, when compared to other systems, costly. Systems with a collector gallery and horizontal well screens are therefore more suited to applications where demand is relatively high, and higher capital costs can be justified. For systems where demands are lower, other types of systems would be more suitable.

These systems are less susceptible to flood damage than are banks of vertical or horizontal well screens. This is generally because the well screens can be installed at greater depth than systems where the wells screens are connected to a manifold. The collector gallery also provides some anchorage under flood flow conditions.

Similar to caissons, these systems lend themselves to the installation of submersible pumps. Water can then be pumped directly from the collector gallery to storage or treatment facilities. Alternately, an outlet pipe can be installed in the bottom of the gallery, connecting the gallery to the wet well of a pump station, or being connected to the suction of a centrifugal or mono-type pump.

A disadvantage of infiltration galleries with horizontal well screens is that they do not lend themselves to the incorporation of the facility for backwashing the screens, or to the development of the sand around the screens at the time of construction. The sand can be developed by isolating each screen independently, but it is far easier to develop the sand when installing banks of vertical or horizontal well screens connected to a manifold.

Design criteria applying specifically to bed-mounted galleries include the following:

a) The screen burial depth should be 0.9 to 1.5 m below the stream bed. There should be 0.3 m of filter pack beneath the screen.
b) To minimize excessive sedimentation on the gallery surface, the stream selected should have a velocity of at least 0.3 m/s.
c) Space the screens approximately 3 m apart.

d) If the stream has a large bed load transport, a single screen should be oriented parallel to the bank, but not in the main channel if possible.
e) Screens should always be placed in the straight reaches of the river or stream, not near the meander bends to limit scour

Field experience indicates that actual infiltration rates from streams and lakes range from 0.5 to 3.1 m^3/day per m^2 per m of head loss. In general, the infiltration rate will be high when the stream gradient is steep and the bed load is coarse. Infiltration rates from lake beds will ordinarily decrease more with time when compared with streams, unless wave activity is particularly vigorous and the bottom is continually disturbed so that fine sediment cannot settle. Wave energy can be transmitted to the bottom if the water depth over the gallery is less than one-half the typical wave length (distance from wave crest to wave crest).

The screens and filter pack material used for infiltration galleries may become partially plugged with sediment over time. Thus, it is good engineering practice to estimate the plugging potential and allow for excess entrance area to maintain the required flow. To maintain yield over time, the actual open area of the screens should be twice the required open area, that is the screen length should be doubled. Backwashing capabilities may be specified for some infiltration galleries. The flushing rate is usually twice the pumping rate for the screen configuration. For example, if a series of three infiltration gallery screens were producing 16 400 m^3/day each screen should be backwashed at a rate of 10 900 m^3/day. Backwashing techniques include (1) gravity backwashing, (2) piping and valve systems to pump from several screens while backwashing others, and (3) air backflushing.

3.6.4 HORIZONTAL WELL SCREENS CONNECTED TO A MANIFOLD

Banks of horizontal well screens can be installed, connecting all the screens to a common manifold. The manifold can then be connected either directly to the intake of either centrifugal or mono-pumps, or water can flow under gravity to the wet well of a pumpstation. A typical horizontal well screen type system is shown in Figure 3-21.

The banks of screens can be installed in either the riverbed or riverbank. In general, although more susceptible to flood damage when installed in the riverbed, yields will be better than if the screens are installed in the riverbank.

When installing systems of this type, it is recommended that the manifold be installed on the riverbank. It is then possible to incorporate isolating valves at the head of each well screen, so that each screen can be isolated from the rest of the system. This is, however, not always possible, particularly when the riverbed is extremely wide and the main flow of the river (surface or subsurface) is not close to the riverbank.

The length, diameter and slot size of the well screens will be determined by the parameters of the sand, and the required yield of the system. The screen diameter is also governed by the need to minimise head loss for flow through the screen pipe.

It is recommended that the well screens be connected to the manifold by a length of flexible pipe (helical). This will allow for some movement of the well screens within the sand bed, without shearing the screens at the manifold.

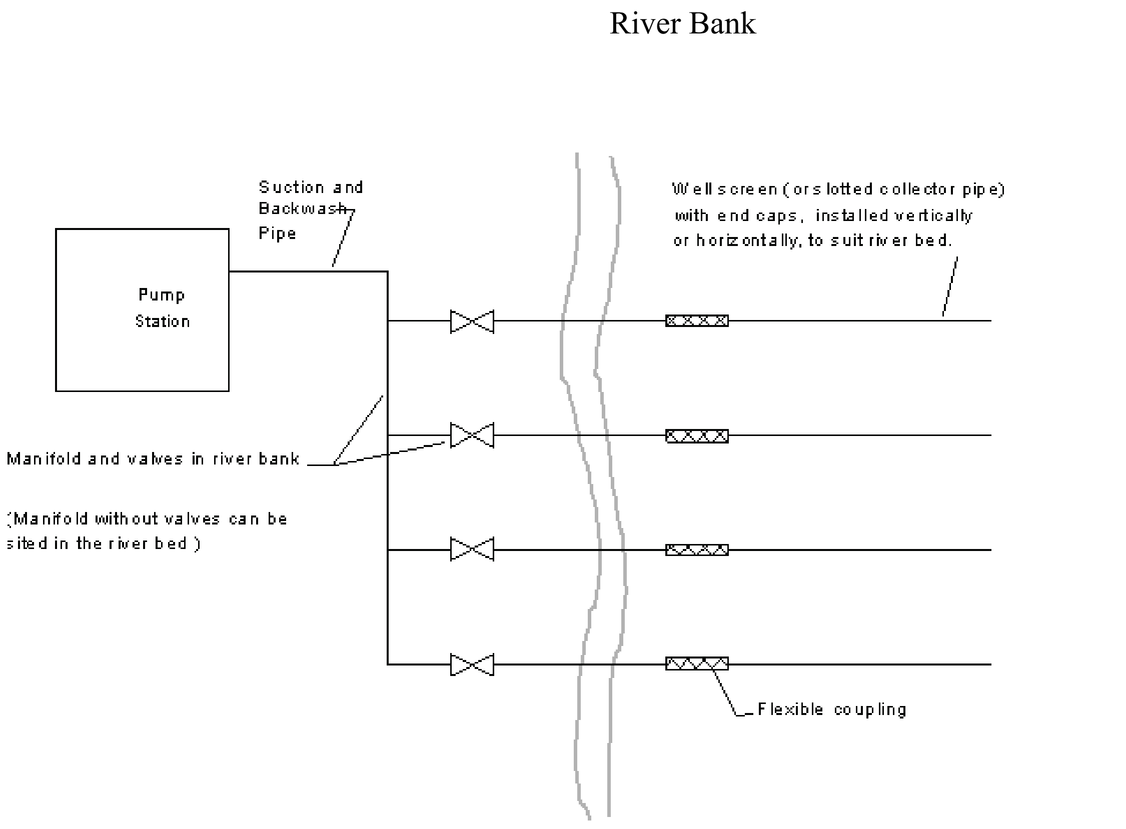


Figure 3-21 Horizontal / Vertical Well Screen System connected to a Manifold

It has generally been found that the performance of systems that are backwashed regularly as part of normal operation procedures is better than that of systems that are not backwashed. Backwashing of the screens will remove any fines that have accumulated in the screens and will assist in breaking down and controlling the development and build up of scale and / or biofilm in the screens.

It is therefore recommended that where possible the system be designed such that it can be backwashed. This can either be done by incorporating a return flow from the storage tanks, with a bypass around the pumps, or by incorporating a circular return system, whereby water abstracted from one screen is pumped back into another screen to backwash it. The former system is however, preferred. In the case of the latter system, problems can arise if fines and biofilm etc. being washed out of the screen being backwashed is sucked into the screen that is being used to abstract the water for backwashing. This could lead to clogging of some of the screens.

It is also preferable that one screen is backwashed at a time. If the entire bank is backwashed simultaneously, the backwash water will follow the path of least resistance, flowing out of the cleanest screens, and not effectively backwashing those screens that most require backwashing.

Well screens can be susceptible to flood damage, particularly when the screens are installed at relatively shallow depths in the sand. Measures should therefore be taken to protect the screens against flood damage. Alternatively, one can simply accept that the screens will occasionally be damaged or lost, replacing them if required. The latter approach has often been found to be more economical, particularly where the operating body has the skill to rapidly and easily replace the screens. This approach would not, however, be suited to schemes where the operating body do not have the skills to replace the screens, or are not likely to have the capital resources to pay for replacing lost screens e.g. large community water supply schemes. When a well screen is damaged by a flood it is not always possible to replace the screen due to the long duration of high flow conditions. Sand pump systems are therefore more suitable for irrigation or backup domestic water supply.

Methods and measures that have been implemented to protect horizontal well screens include:

- Installing the screens at the greatest possible depth, just above bedrock level.
- Tying the screens to anchor blocks buried in the sand.
- Encasing the screens in gabions or packed rock.
- Laying the screens on a mass concrete foundation and encasing them in aggregate enclosed in a mesh cage, which is bolted to the concrete foundation. The cage is then covered with rock. (designed by Bradford, Conning and Partners, Newcastle) – see Figure 3-22. The system is relatively expensive to construct when compared to other horizontal screen systems, but it is believed that this system will remain successfully in operation for many years. The design has the added advantage of incorporating a graded filter around the screens. Problems of low yield are also therefore less likely to occur at this system.
- Attempts have also been made to anchor the well screens to the bank by means of a chain connected to rings on the end of the well points. This has, however, not been very successful.

1. SUBSOIL DRAINS UNDER CONSTRUCTION - MASS CONCRETE BASE WITH INVERTED STAINLESS STEEL U-BRACKETS BOLTED TO CONCRETE. 6 mm STONE CONTAINED BY 4,75 X 1,6 mm GALVANISED MESH. 3 X 200ϕ SLOTTED uPVC PIPES CONVEYS WATER TO MAIN PIPE AT BOTTOM OF WEIR (MAIN PIPE NOT SHOWN ON PHOTO).

2. 3 x SUBSOIL DRAINS BEING COVERED WITH BOULDERS FOR FURTHER PROTECTION AND IMPROVED FILTRATION. PRIMARY COLLECTION CHAMBER UNDER CONSTRUCTION.

Figure 3-22 Installation of sub-soil screens enclosed in graded filters and cages. Photographs and text courtesy of Bradford Conning and Partners, Newcastle

3.6.5 IMPORTANT DESIGN CRITERIA FOR INFILTRATION GALLERIES

A major design principle for infiltration galleries involves the orientation of the screen relative to the surface water or groundwater flow directions. For bed-mounted galleries, the screen is oriented perpendicular to the stream flow. For bank-mounted galleries, the screen is placed perpendicular to the groundwater flow to minimize the head loss; that is, the screen is placed parallel to the stream or river.

Important design criteria of infiltration galleries include:

a) Entrance velocity through the screen slot openings should be 0.03 m/s or less.

b) Axial velocity inside the screen should be 0.9 m/s or less, so that the head loss, h, will be 0.3 m or less. The following equation is used to determine flow velocity:

$$V = \frac{1.16x10^{-3}Q}{\pi r^2} \qquad (3\text{-}1)$$

Where

V = velocity, in m/s

Q = yield, in m^3/day

r = radius, in m

c) Screen slot size is based on the grain-size distribution of the filter pack; always retain 100 percent of the filter pack.

d) Use 304 stainless steel for fresh water, and 316 stainless steel or monel for salt water. Do not use monel if the Ryznar stability index is 9.5 or greater.

e) Filter pack recommendations:

- The surface area of the filter pack material is determined on the basis of water entering the pack at a rate of 117 to 293 m^3/day per m^2 of surface area. The actual hydraulic conductivity of the pack is usually much higher.
- Filter pack design is similar to that for a vertical well, but with a slightly more liberal multiplier of 6 to 7 times the 70-percent-retained size.
- Filter pack material should be clean, siliceous, rounded and uniform.

A typical large sand pump system at Magudu on the Komati River best illustrates the design and operational aspects of such a system.

3.6.6 Magudu sand pump system with infiltration gallery on the Komati River

This irrigation pumpstation constructed recently on the Komati River in Mpumalanga, supplies 373 ha irrigation to the Mawewe tribal authority. It consists of 2 parallel pumps on the river bank, with each having a set of 4 suction pipes located in the river bed (Figures 3-23 and 3-24). Two KSB Omega 250 – 600 A's were installed with a duty point of 777m³/h @ 115m each. The total pump capacity is 432 *l*/s (Burger du Plessis).

The 300 mm diameter suction pipes located in the alluvial river bed are each 42 m long and about 3 m apart.

The benefits of such a design are:

- the pumps can be placed relatively far away from the river on the river bank, well protected from floods, with limited risk of blockage of the intake pipes
- no weir is required or hydraulic structure in the river, and it is therefore a relatively cheap design

Concerns are related to floods:
No large floods have been experienced since operation of the pumpstation started, and this is probably the biggest concern with the design. On the Komati River in 2000 the water depth was recorded as 15 m deep by DWAF during a major flood. The maximum dune height that formed in the river would have been in the order of 15/6 = 2.5 m which is deeper than the 1.7 m depth to the pipes. Also general scour of the river would occur during a flood. Damage to the suction pipe system is therefore quite possible during a future flood event.

Figure 3-23 Magudu suction pipe system under construction (Burger du Plessis Consulting Engineers)

In the river bed, the river sand is replaced to a depth of 1.7 m by a filter (Figure 3-24) layer consisting of:

- 0.9 m layer of coarse 6.7 mm gravel around the suction pipes
- 0.7 m layer of 2.4 to 4.8 mm sand filter on top of gravel layer
- 0.1 m layer of in situ river sand

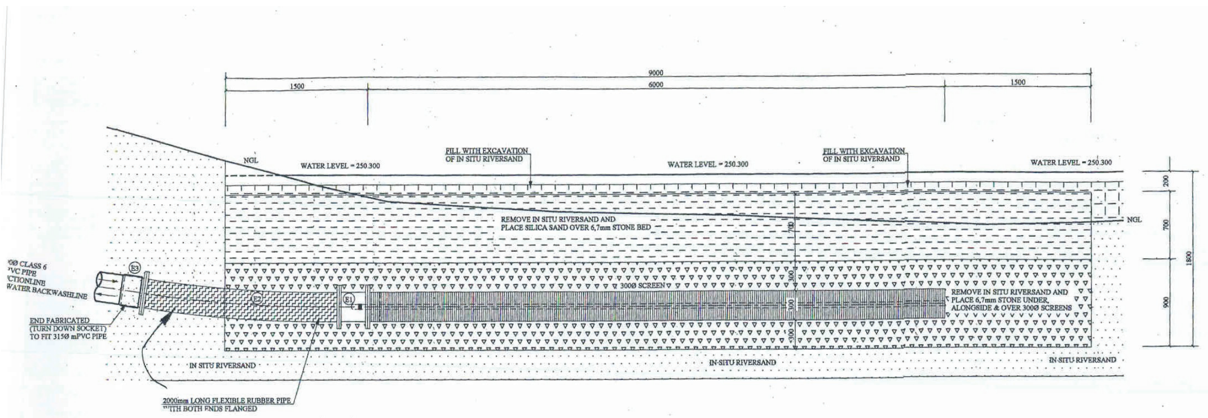


Figure 3-24 Magudu infiltration gallery design (Burger du Plessis Consulting Engineers)

Fine silt and clay have deposited through the filter layer and blocked the openings of the suction pipes. But this has been catered for in a backwash system. The system is backwashed every day for one hour with a 50% water and 50% air mixture at a rate double the pump delivery. The mixture is pumped through a 50 mm diameter pipe which is located centrally in each of the suction pipes, to further clean the graded filter around the pipe. A similar design was constructed and tested at the Hydraulic Laboratory of the University of Stellenbosch. Figures 3-25 and 3-26 show the slotted screen in the laboratory. Tests were carried out with a 3 m screen and a gravel pack at 1:1 scale, using clay to partially block the flow, and then followed by backwashing with air or water, or a combination of air and water.

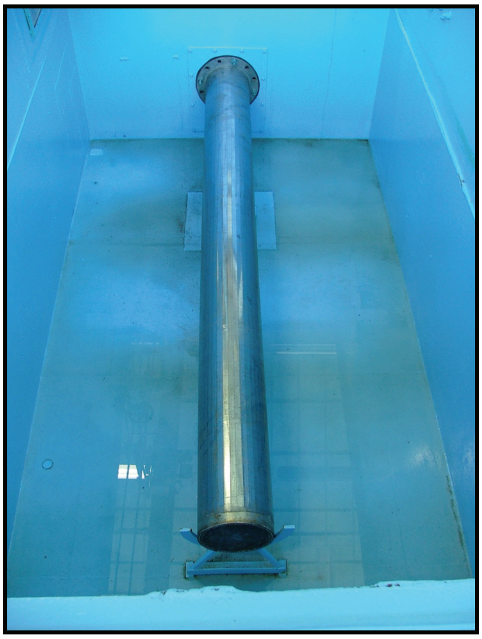


Figure 3-25 3m Stainless steel, Johnson continuous slot well screen with 0.8mm slot opening.

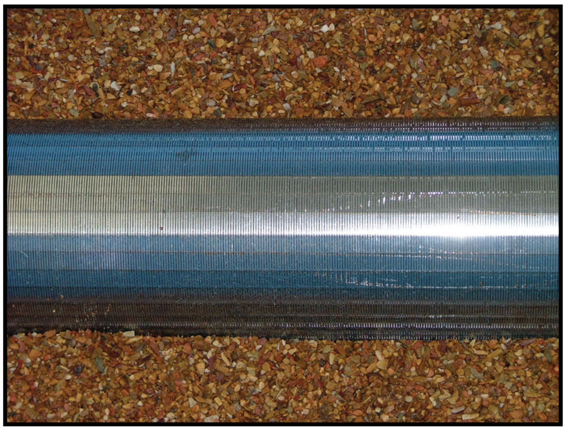


Figure 3-26 Stainless steel, Johnson continuous slot well screen embedded in a gravel pack.

Figures 3-27 and 3-28 show the 50mm PVC slotted backwash pipe in its rod-constructed frame placed inside the Johnson slotted screen.

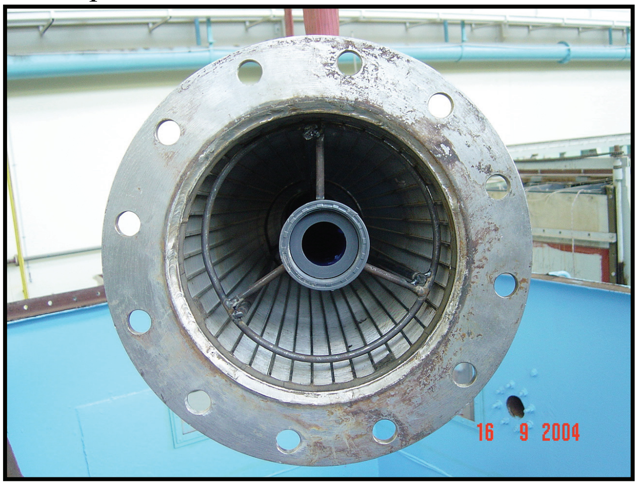


Figure 3-27 Backwash pipe inside slotted screen

Figure 3-28 Backwash pipe (50 mm diameter) and its supporting frame

Figures 3-29 to 3-32 show the filled container and air backwash procedure functioning accordingly at a pressure of 1 bar and the view of the air strip (air bubbles) on the surface of the water:

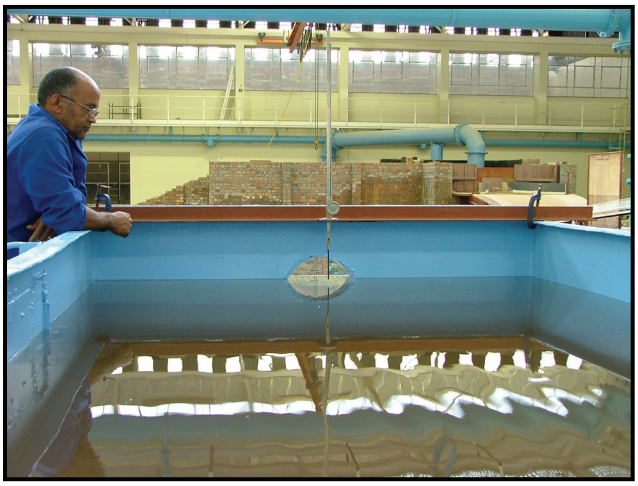


Figure 3-29 Container filled with clean (sediment free) water to a depth of 1.8m.

Figure 3-30 During air backwash procedure, sufficient air pressure reached the end of the screen.

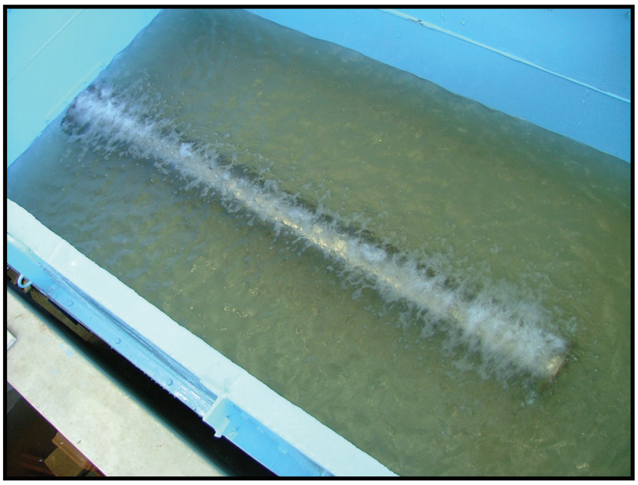


Figure 3-31 Distribution of airflow over the length of the well screen seems to appear evenly.

Figure 3-32 Air backwash procedure with water depth at 1.8m. A 450mm air strip appears on the surface of the water.

Figure 3-33 shows the air backwash procedure functioning with the filter bed in place.

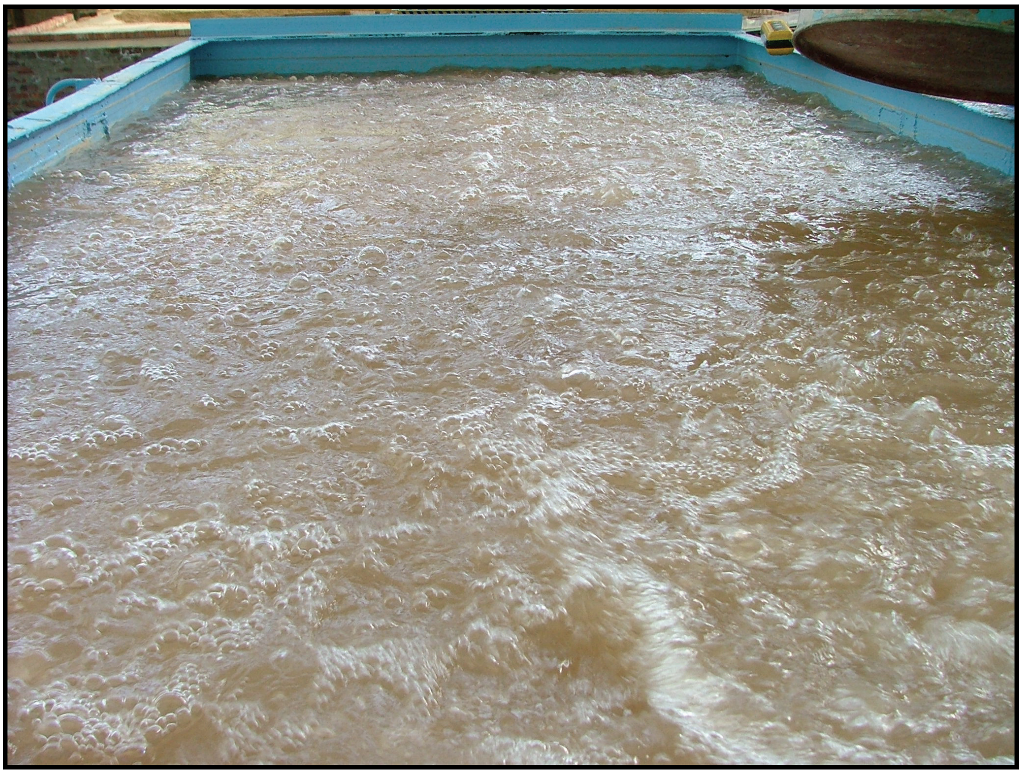


Figure 3-33 Even distribution of air over the test area with filter bed in place

Figure 3-34 shows the backwash during the overlap period between air and water backwash simultaneously.

Figure 3-34 Top view of bubble-strip width during overlap between air and water backwash.

Based on the laboratory tests, the following recommendations are made:

- Assuming that the flow pattern of air and water through the gravel layer is approximately parabolic with air having a wider surface width to that of the width of catchment area of the buried well screen and the combined force of air and water causes this surface width to decrease, cleaning less of a catchment area above the well screen. It is therefore recommended to backwash air and water separately instead of combining air and water backwash.

- From the tests done, a combination of 4 minutes of air supply at a pressure of 1 bar and thereafter a 4 minute duration of water backwash at a rate of 30 *ℓ/s,* which is twice that of the desired abstraction, was found to be the most efficient backwash procedure and may be implemented in practice as the most efficient backwash procedure for a 3m length well screen.

- A system may for instance have a pumping cycle of 24 hours and then be backwashed after each cycle if necessary, but continuous monitoring is required in cases where the water is highly contaminated with sediment. Backwash then needs to be done as soon as the abstraction capacity decreases with 10% or when the negative pressure of the suction head of the hydraulic pump drops below a specified value, in order to protect the pump.

- Knowing that abstraction of water is expensive, the 4 minute duration of water backwash at a rate of 30 ℓ/s may be kept constant, increasing only the duration of air, loosening as high a percentage of the clogged sediment as possible.

- For a well screen longer or shorter than 3m, only the duration of this backwash procedure and the pressure of the air backwash must be altered, not the capacity of water backwash. Water backwash should be equal to twice the abstraction rate.

4. INTAKES AND PUMP SUMPS

Most South African abstraction works are relatively small (< 4 m^3/s) and pumps are often used to abstract water from the river. It is therefore important to understand the hydraulic requirements of a pump and pumpstation.

4.1 GENERAL

The intake chamber of a vertically installed pump should be designed so that undisturbed flow to the pump is ensured for each operating condition and for all inlet water levels. This is particularly important for pumps of high specific speed (mixed flow and axial flow pumps), as they are more sensitive to inlet flow conditions than centrifugal pumps.

Operation of the pump will be trouble-free if the flow to the pump impeller inlet is swirl-free and if there is an uniform velocity profile across the entire impeller entry area. Additionally, the formation of air entraining vortices in the intake chamber must be prevented, when operating at minimum liquid levels. If these conditions are not met, the performance of the pump, in terms of flowrate and efficiency, will be impaired. In the worst cases damage due to vibrations and cavitation could occur.

4.2 DESIRABLE FLOW CONDITIONS AT INTAKE

The ideal flow conditions at a pump intake section are:

a) **single phase**: the water should contain no entrained air. For a centrifugal pump 3 % of free air has been found to decrease efficiency by 15 % and axial flow pumps are even more sensitive. There are several ways related to intake or sump geometry, in which air can be entrained. If the water level in the sump is very close to the top of the intake air may be drawn through the intake continuously or by "gulping". This tendency will be increased by high intake velocities, particularly where approach velocities are relatively low and the draw-down of the water surface near the intake is accentuated. A minimum submergence of the intake is therefore necessary.

   Air can also be entrained by a falling jet which is common where water enters the sump over a weir. A minimum sump length could be calculated from the approach velocity and water depth in the sump and the rising velocity of air bubbles taken as about 0.2 m/s for 2 to 5 mm diameter bubbles. Intense vortex action can also lead to air entrainment.

b) **uniform flow**: the flow velocity should be constant in magnitude and direction across the flow section. Intense swirling can cause rapid changes in local pressure on the pump impeller which can lead to cavitation and severe vibration. Axial type pumps are generally most susceptible to this type of damage. Intense swirling is associated with vortex action. In addition to surface vortices there may be submerged vortices which originate at the floor or walls of the sump rather than the water surface. The large scale vortices are generally caused by general circulation in the sump which is amplified as the flow converges towards the intake. The general circulation may be initiated by the distorted velocity profile of the approach flow and the sump geometry.

c) **steady flow**: the flow velocity should be constant in magnitude and direction with respect of time. Unsteady flow can cause vibrations with time and cause bearing wear. In addition to the small scale turbulence which can normally be expected, large scale turbulence where the eddy size is of the same order of magnitude as the intake cross-section sometimes occurs. The major causes of this are unsteady flow patterns arising from obstacles in the sump or poor inlet conditions and vortex shedding from pillars or other pumps. Another common cause of unsteadiness is the presence of stagnant regions of water above or behind the intake.

4.3 INTAKE AND SUMP DESIGN

Four functional zones can be identified in a typical sump-intake arrangement:

a) Inlet from the supply source
b) Approach section which may contain screens for removing solid matter and gates or dividing walls for directing the flow to the appropriate sump.
c) The sump which is generally rectangular in plan with a flat floor. The purpose of the sump is to damp out distorted flow patterns. The pump may be installed in the sump (wet well arrangement) or in a dry well at the end of the sump.
d) The zone in the pipe between the pump intake and the pump.

A typical sump arrangement is shown in Figure 4-1, while wet well and dry well sumps are shown in Figure 4-2.

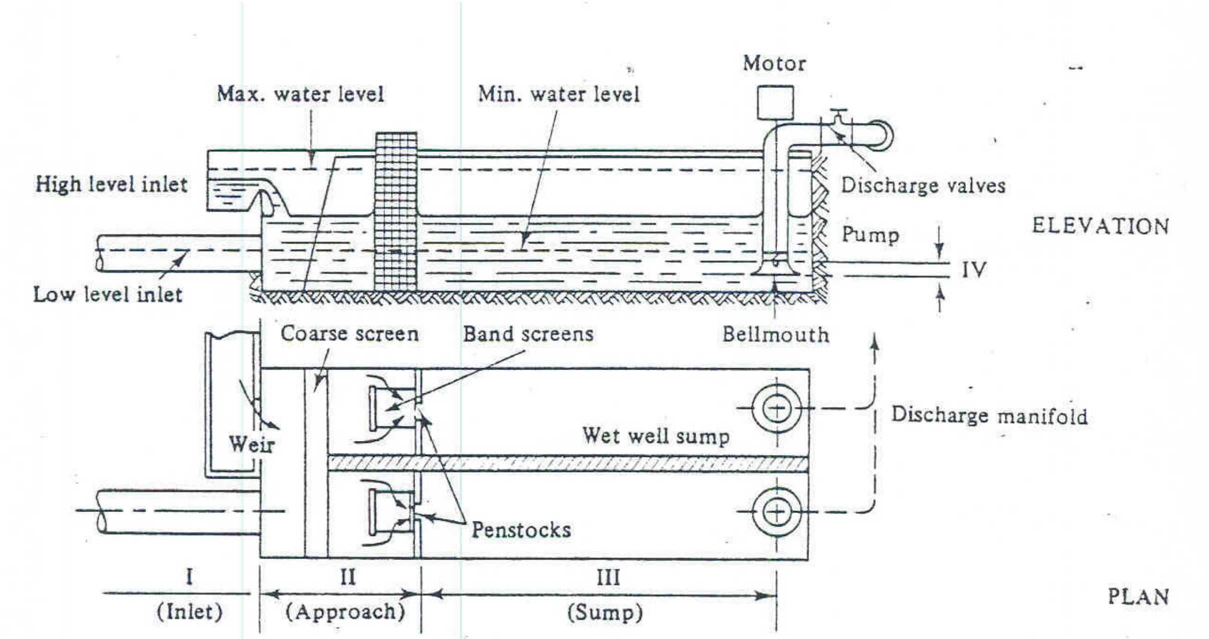


Figure 4-1 A typical sump arrangement

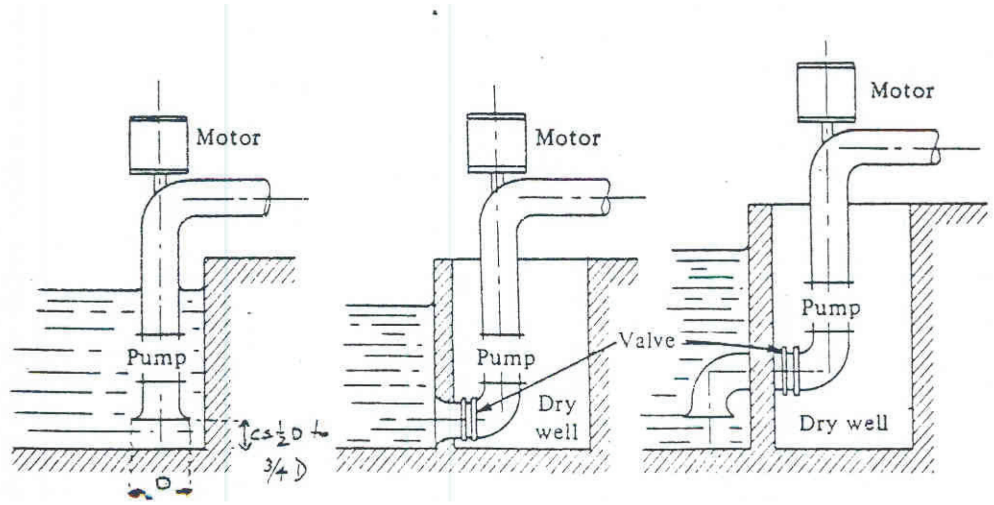


Figure 4-2 Wet well and dry well arrangements

A wet well with a submersible pump has the advantage of simplicity of design and hence relatively low cost. Maintenance requires removal of the pump, but this does not require drainage since the pump can be raised vertically. In river conditions with the correct selection of pumps that can pump coarse sediment, this is a very reliable solution.

A dry well has the benefit that the pumps are accessible for maintenance at all times. The intake for a dry well may be horizontal or a turned down bellmouth in the sump, depending on minimum water levels required in the sump. Prosser (1977) recommends a minimum submergence of 1.5 times the bell mouth diameter for turned down intakes and 1.0 times the diameter for horizontal intakes, but this is only a general recommendation.

General guidelines for sump design are:

a) Flow towards the intake should be uniform across the width of the channel
b) Excess energy associated with changes in level down slopes or over a weir should be dissipated well away from the intake
c) Any obstructions should be streamlined to avoid flow separation near the intake
d) Potentially stagnant regions should be filled in
e) Average velocities should be kept low: about 0.6 m/s for flow into the pumping station and 0.3 m/s approaching the bellmouth.

Pump inlets should never be placed in series in the sump due to the non-uniform and unsteady flow patterns which will be created at the downstream pump inlets. The pumps should rather be separated by walls in the sump in a unitised design which is preferred (Figure 4-3) or an open sump design can also be used as shown in Figure 4-4

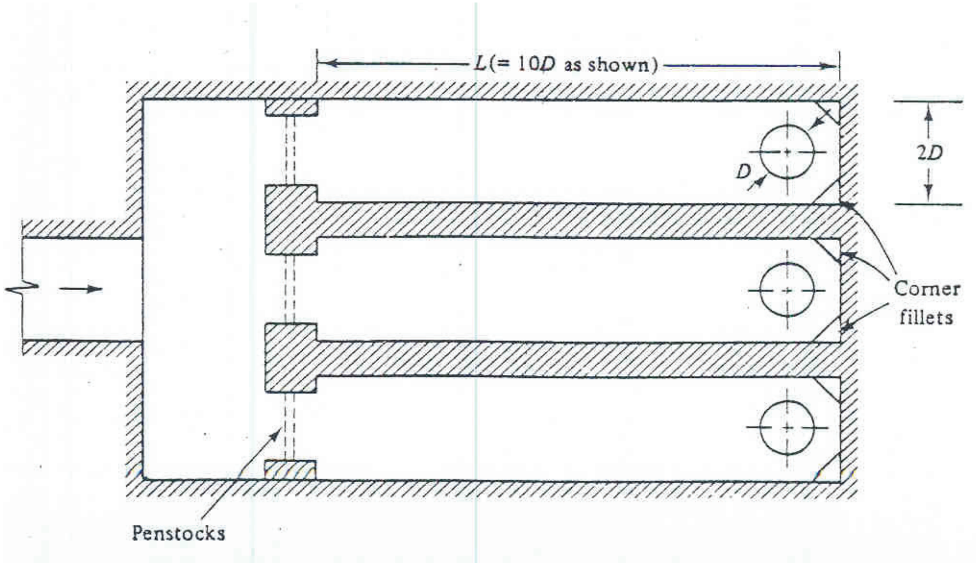


Figure 4-3 Unitised sump design

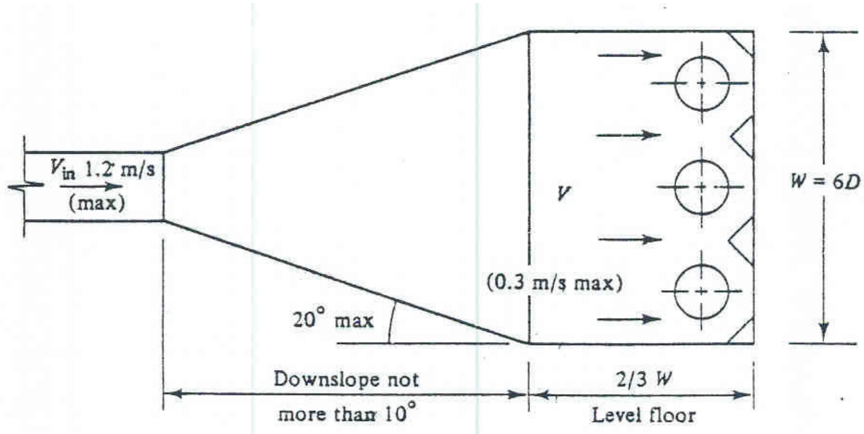


Figure 4-4 Open sump design

4.3.1 OPEN INTAKE CHAMBERS

If a single pump is installed in an intake chamber, recommendations for the principal dimensions may be taken from Figure 4-5. A Uniform channel cross-sectional area over a length of at least 5D upstream of the pump should be provided. The flow velocity in the intake channel should not exceed 0.5 m/s. The reference dimension “D” corresponds to the outside diameter of the suction bellmouth.

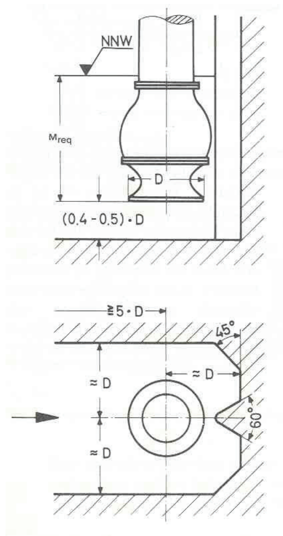


Figure 4-5 Single pump intake chamber

The minimum submergence M_{req} is defined as the distance from the bottom edge of the suction bellmouth to the lowest inlet water level (NNW). For the installation of vertical pumps no general standard values can be given, but must be determined by the pump manufacturer in each individual case. The minimum submergence must ensure that cavitation does not occur at any point inside the pump. If several pumps have to be installed in one intake chamber, separate bays for the individual pumps provide the best solution (Figure 4-6).

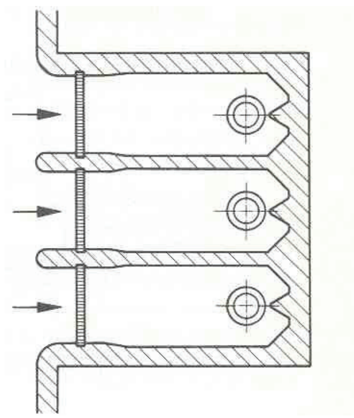
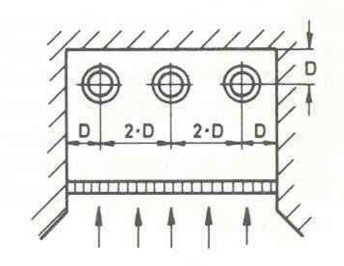


Figure 4-6 Intake chamber with separate bays

If this solution is not possible, an arrangement similar to Figure 4-7 should be used. The distances quoted should be considered as nominal values. In very difficult cases guide walls (baffles) may have to be provided (Figure 4-8), the siting of such walls should be agreed with the pump manufacturer.



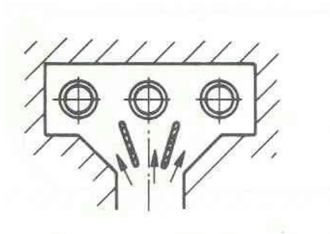
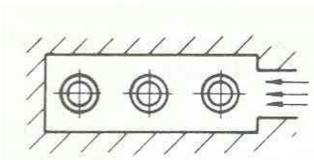


Figure 4-7 Intake chamber without bays **Figure 4-8 Intake chamber with guide walls**

4.3.2 AVOIDABLE MISTAKES IN THE DESIGN OF INTAKE CHAMBERS

- In the arrangements shown in Figures 4-9 and 4-10, the liquid enters at one end of the intake chamber. The flow to the individual pumps is non-uniform and the pumps will affect each other.



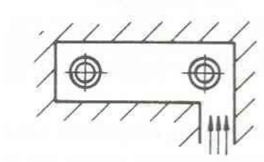


Figure 4-9 Incorrect pumps in series design **Figure 4-10 Wrong intake design**

- Several pumps arranged asymmetrically in one intake chamber.
- Sudden expansion or contraction of the supply channel.
- The length of supply channel with a uniform cross-sectional area is too short.
- Beams, steps or pipes at the bottom of the intake chamber immediately upstream of the pump.
- Suction bellmouth too close to the bottom of the suction chamber.
- The liquid supplied to the intake chamber is discharged above the level of the pumped liquid.

4.3.3 COVERED INTAKE CHAMBERS

If, for any reason, the extended supply channel length ($l \geq 5 \cdot D$) which is required for trouble-free pump operation cannot be provided, an alternative consists in fitting a sloping cover to the intake chamber. These covers are very effective in reducing swirl. Approximate recommendations for the principal dimensions can be taken from Figure 4-11. However, in each case the dimensions should be determined by the pump manufacturer.

If, due to the particular site conditions, widening of the intake chamber (oblique side walls, sloping bottom ending before the suction bellmouth) cannot be avoided, the use of a cover can provide the necessary acceleration of the inlet flow to achieve a more uniform velocity profile.

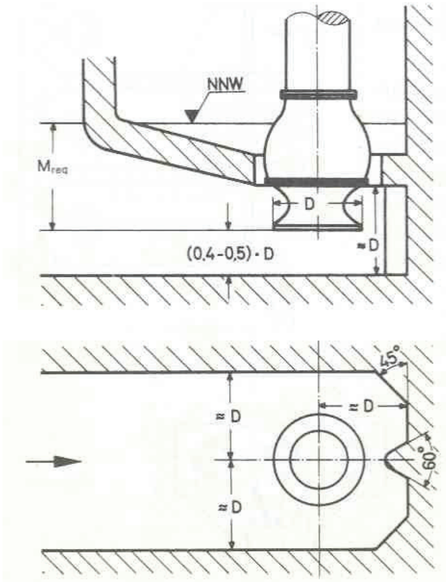


Figure 4-11 Covered intake chamber

4.3.4 INLET ELBOWS

Minimum dimensions are obtained with inlet elbows which – similar to the well-known turbine elbows – are shaped as "accelerating elbows" (Figures 4-12 and 4-13). If the flow velocity is accelerated by a factor of 4 or 5, the length of the elbow = distance between the inlet area and the centre of the pump, $l_{Kr} \approx 4$ x impeller inlet diameter, is sufficient to achieve a uniform velocity distribution at impeller entry.

The inlet cross-sectional area A_1 of the elbow should be large so that the velocity at entry to the elbow is insufficient to allow formation of air entraining vortices or to allow air to be drawn into the pump. In each individual case an economic assessment of the design has to be made to determine whether the higher construction costs of an inlet elbow are acceptable compared to the lower costs of a single intake chamber. The design and construction of an inlet elbow is more complex than that of an intake chamber, and in some cases deeper excavations may be necessary.

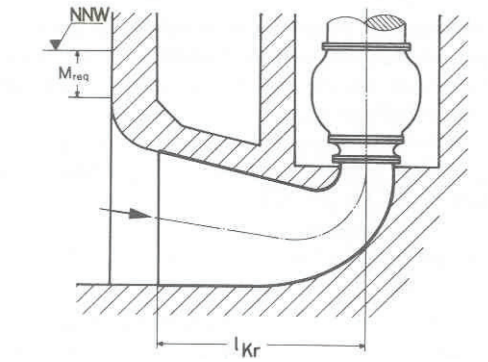


Figure 4-12 Inlet elbow section

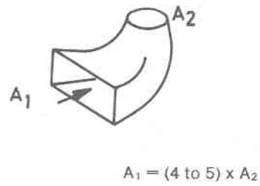


Figure 4-13 Inlet elbow contraction

Dry well pump installation with the aid of inlet elbows was used in a design of the Fairbreeze pumpstation on the Thukela River, which has the additional benefit that the pumps do not have to be raised/or are in the way during flushing of their approach canals (Figure 4-14).

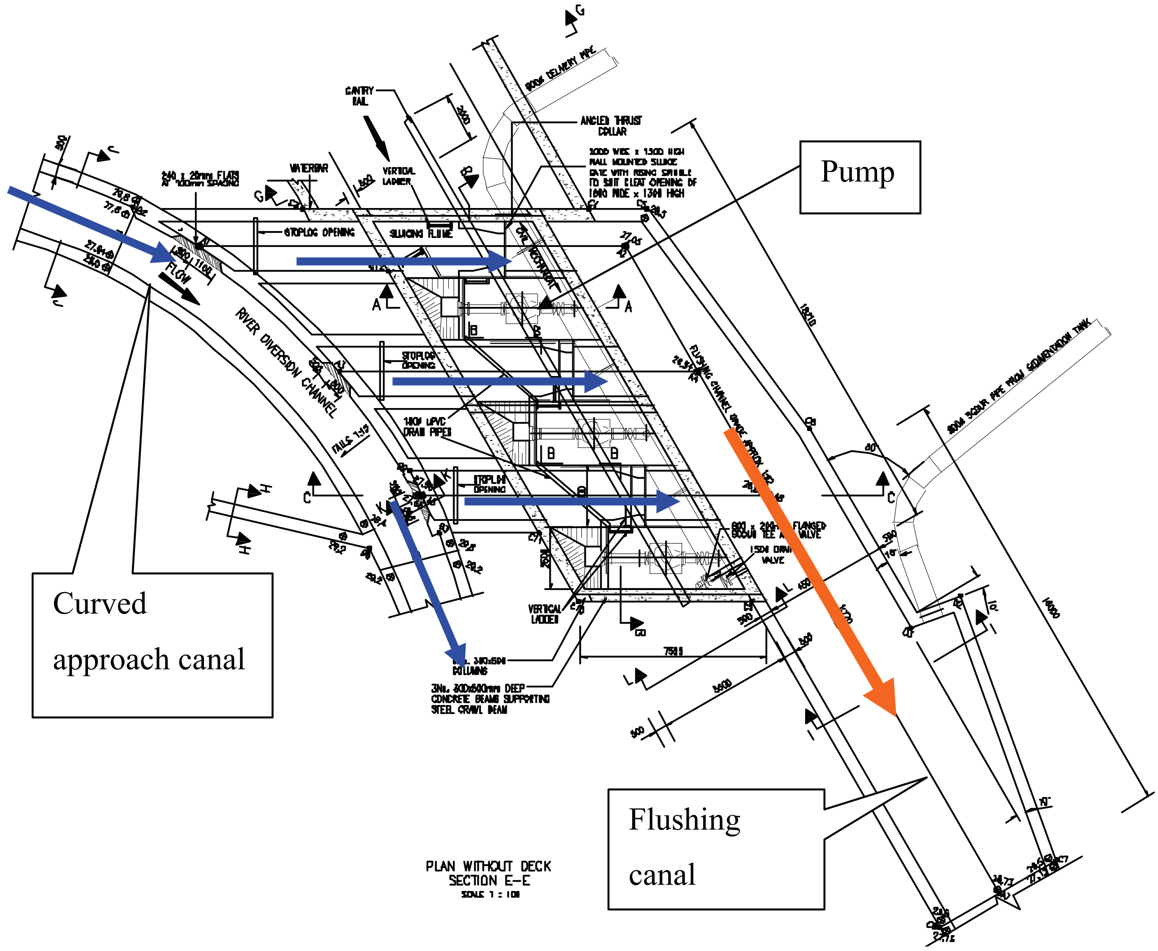


Figure 4-14 Plan layout of pump flushing canals with dry well pump installation (proposed Fairbreeze, Thukela River design)

5 PUMP CANALS WITH FLUSHING

It is possible to design the pump canals so that they can be flushed. With submersible pumps in the canal, it is best to design the flushing with the pump raised prior to flushing due to the hydrodynamic force on the pump. As safety factor the canal should also be wide enough so that a hydraulic jump does not form at the pump if the pump is not raised.

Figures 5-1 and 5-2 show one of the Lebalelo pump canals during flushing (in this case the pump and fine screen were not raised). The hydraulic control is upstream at the trash rack and a steep drop upstream creates supercritical flow right through the canal. In the design the tailwater level should be considered to ensure that a hydraulic jump does not form in the canal during flushing.

Figure 5-1 Pump canal flushing at Lebalelo looking upstream

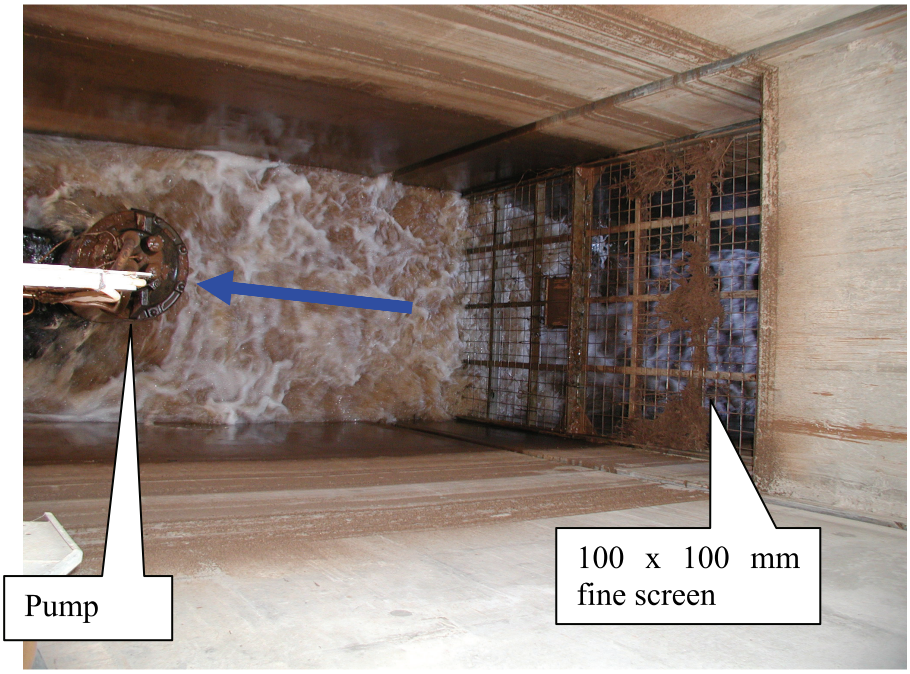


Figure 5-2 Flushing around pump at Lebalelo pump station

6 PUMP SELECTION

For a pumpstation to operate under flood conditions and high sediment loads it is important that suitable pumps are selected.

The following main pump types can be used in abstraction works:

a) The **centrifugal pump**. The capacity of traditional dry well centrifugal pumps for reasonably economic working may vary from a minimum of about 7 ℓ/s up to 700 ℓ/s and more, with heads varying from 3 m to about 45 m. With small to medium capacities the pump should be of the unchokeable type wherein any solid, up to a maximum of about a 100 mm sphere, that may enter the pump suction will be passed through the pump.

The recessed impeller type of certrifugal pump (also called vortex or torque flow pumps), although of lower efficiency, is less likely to be affected by fibrous material and can be easier to open up for maintenance. Centrifugal screw pumps also fall in this category.

Submersible centrifugal pumps are available for a similar range of duties. They are self-priming with both pump and motor totally submersible and are accordingly suited for use in wet wells or in dry wells.

Immersible or submersible pumps that can handle coarse sediments should be considered in river pumpstation designs (Figures 6-1 and 6-2).

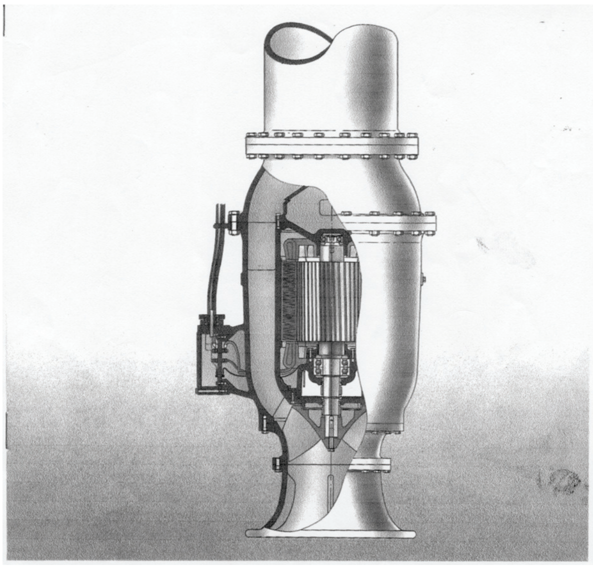


Figure 6-1 Immersible pump

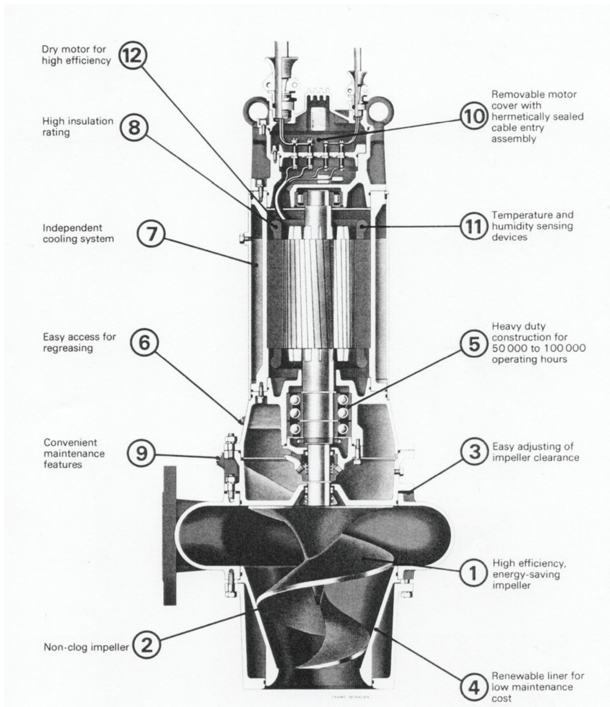


Figure 6-2 Components of a submersible pump that can handle solids without clogging

Cooling is a problem in a dry well and special design precautions may be necessary. The discharge connection of the pump is adaptable for either a flexible hose or static pipework.

For wet well installation submersible centrifugal pump units are available which will slide down guides and seat automatically on the permanent discharge connection. The weight of the pump forces the mating flanges into contact thus providing a seal on the discharge side.

Instead of installing complicated and expensive drive shafts, submersible pumpsets are recommended for this purpose where practicable. This simplifies maintenance. Where a large percentage of rags are present problems have been experienced with pumps installed in wet wells. Rags tend to accumulate around the guide rails. The danger exists that the rags may accumulate on the flange of the duckfoot bend when the pump is raised for inspections. When the pump is lowered again the rags may jam between the pump and bend flanges. Poor sealing results and the pump vibrates, causing the bearing and mechanical seals to fail. Where there is doubt that the pump may become choked by rags, etc. provision should be made at the design stage for installing at least the civil works required to accommodate screens to remove excess fibrous materials.

b) The **mixed flow pump**. The mixed flow pump is more efficient where the volume to be pumped is large and where the head lies in the range of 6 m to 18 m.

c) The **axial flow or propeller-type pump**. The axial flow pump is suitable where large volumes have to be pumped against low heads.

To reduce erosion from grit and cavitation improved metallurgy should be specified.

Potential problems in operating variable-speed pumps at conditions that induce high radial loads should be recognised. To forestall failures and damage such as bearing, wearing ring, and shaft sleeve wear that sustained operation at low speed can cause custom-engineered pumps with heavier shafts, bearings, and bearings frames should be specified where large pumps have to be installed. This is usually not economical where pumps < DN 350 are installed. It is then necessary to obtain detailed drawings from the tenderers to ascertain the shaft and bearing sizes offered. The impeller shaft of all pumps > DN 150 should be tapered (rather than parallel as is the norm on cheaper pumps). Do not necessarily select the cheapest equipment offered. Invariably equipment that is cheaper is insufficiently robust for arduous duty.

The material from which the mechanical seals are made must also be determined. Silicon carbide is considerably more expensive than Tungsten carbide, but is much more satisfactory where grit is present in the pumped liquid. The outer seal should therefore be specified as Silicon Carbides on ceramic, or Silicon Carbide on Silicon Carbide, while carbon on ceramic will suffice for the innerseal for pumps up to DN 300.

Submersible motors which are designed to circulate the pumped liquid to provide cooling should not be used as grit present in the liquid tends to settle in the heat exchanger cooling tubes, causing them to block. Overheating results. The most satisfactory designs consists of circulating the lubricating oil in the mechanical seal oil bath to the motor stator windings. The pump volute then acts as a heat exchanger to keep the oil cool. Some motors are fitted with fans sealed to IP 68, to circulate air which is cooled in the same way. These are also satisfactory.

Motors which require submergence to keep them cool should not be used in either wet or dry wells.

Special care must be taken by the user in the selection of submersible pump units to ensure that entry cables are truly to IP 68 and that the ingress of moisture is properly monitored.

The pumpstation should be designed for a life span of at least 30 years. The pumps installed are frequently replaced after about 15 years. Larger pumps can then be installed if required. The civil works must be designed to accommodate the larger equipment. (Future installation).

Typically 4 pumps are used: (3 + 1 standby). The nominal peak pumping rates for each pump is about one-third of the station's peak capacity.

As inflow increases and exceeds the capacity of the on-line equipment, an additional pump is started. All in-service pumps must operate at the same speed in load-sharing operation, minimising the wear on the equipment caused by operating a pump at very low flows, as happens in staggered operation. Maintenance costs are thus reduced.
It is usually difficult to select a single pump to accommodate the full range of flows. However, as a rule, it is possible to limit the number of pumps to three or four by taking full advantage of the capacity available using variable-speed drives.

Where pipes with flexible couplings are installed it is essential that concrete thrust blocks be installed where necessary.

Where it is anticipated that an additional rising main will be required at some future date, provision (space) for its installations must be provided at stage 1. Provisions must also be made for cross connection of the two mains. All the necessary valves and connections to change over automatically from one to the other main or to operate them in parallel (depending on flow-rate) must be made, as it becomes extremely difficult to install at a later stage.

It must be borne in mind that the system is such that under normal operating conditions the velocity in the pumping main does not drop below 0.6 m/s. At very low flows it is not practicable to maintain this velocity. Provision must therefore be made at the design stage to be able to pump at least 1.2 m/s for short periods at least twice a week to flush the pipe system.

Additional emphasis: The operation and maintenance staff must be motivated to believe that proper attention to the support features of a pumping facility is important. Support systems that are sometimes given little attention but can make a major difference in overall performance and ease of maintenance include dry well sumps and sump pumps, cranes, hoists, lifting eyes, access hatches, alarms, doorways, and landscaping.

7 TRASHRACKS AND SCREENS

Trashracks are designed to filter material floating on or just below the surface. Bar spacing are based on sediment and debris size to exclude, and in some countries also on fish.

The bars are usually placed vertically for easier rack cleaning. Head losses of up to 0.5 m are possible if the rack is dirty and partly clogged, but this could be high for low head diversions. Automatic rack cleaning systems will reduce head losses, but will only be considered at large diversions. Hand rake systems usually have bars to the top at the working platform and the trashrack is tilted 20 ˚ to the vertical for easier cleaning.

Tapered bars (Figure 7-1) increase the rigidity of the bars, make it more difficult for material to jam between them, and facilitate cleaning. Round bars usually lack rigidity.

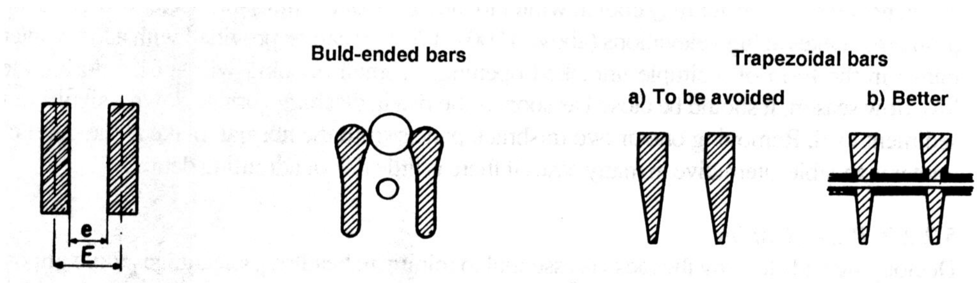


Figure 7-1 Trashrack bar designs (Bouvard, 1992)

To limit blockage of the trashrack the soffit of the trashrack should be below the minimum operating level of the abstraction works.

8 DIVERSION WEIR DESIGN

An uncontrolled weir structure, sometimes in conjunction with an earth embankment, may be provided across the watercourse in order to raise the water level sufficiently to permit water to be diverted through the intake structure.

Typically for the weir structure, an ogee weir configuration with a hydraulic jump stilling basin is employed; however, in some cases a trapezoidal weir may be considered. Water flowing over the weir will cause a submerged roller to form immediately downstream that can be hazardous to canoeists and swimmers. Consequently, when choosing a site for the weir, its proximity to a populated area and safety requirements should be considered.

The concrete weir should be founded on rock whenever possible for stability and reducing cost. The following should be considered when designing the weir:

- Make the weir as low as possible to limit its impact on the river
- Use an ogee crest based on a design flood
- Establish tailwater levels from field observations
- Hydrostatic (water and deposited sediment upstream) and hydrodynamic forces
- Aeration by piers on the crest of the weir, or splitters with pipe/gallery, if an ogee weir is not used

- Multi-notch weir for improved flow measurement and forcing low flow towards the intake
- Depending on the foundation conditions and structure arrangements, seepage control and drainage methods would be required

9 ENERGY DISSIPATION

Suitable energy dissipation downstream of a weir is extremely important when the weir is not founded on rock, such as at the Mhlathuze River weir of Mhlathuze Water. In an alluvial river a standard stilling basin or a roller bucket will be able to dissipate the energy, but the former structure is expensive due to its long length and uplift forces. A solid roller bucket with a 4 to 5 m radius with invert 1.5 to 2 m below the river bed level, ogee crest, and roller bucket ending slightly above the river bed level, provides a compact solution when the required weir height is about 2.5 to 5 m (Figure 9-1). The total length of the bucket is about 8 m, but riprap (dumped rock) is required downstream for at least 20 m with a suitable filter design. The roller bucket requires the same tail water levels for stable energy dissipation as for a stable hydraulic jump on a horizontal bed. A key benefit of a roller bucket, is its capability to roll sediment upstream towards the bucket in the boil. Solid and slotted rolled buckets can be used (Figure 9-2).

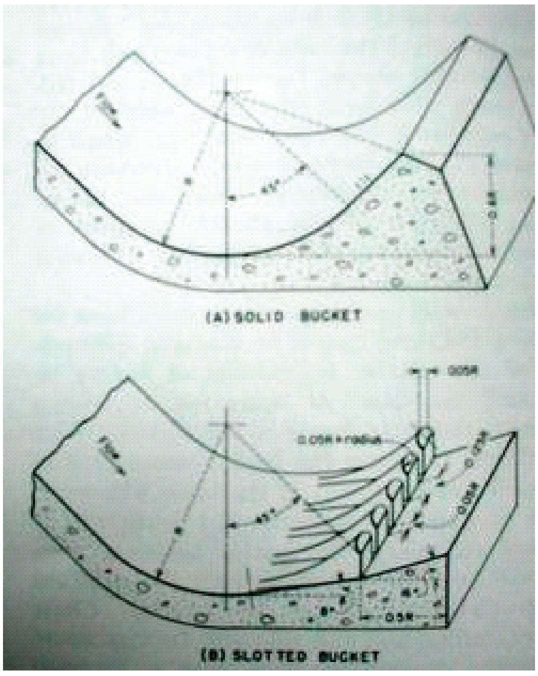


Figure 9-1 Roller buckets

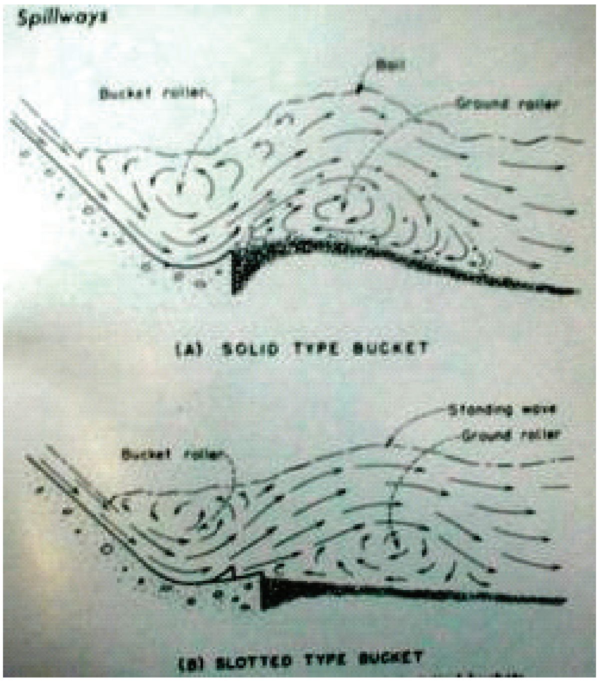


Figure 9-2 Roller bucket flow patterns

When designing a roller bucket a physical model study is recommended (Figure 9-3). The dimensions of the roller bucket can be optimised in the model by placing a movable bed first in the area where the bucket is to be located and testing for a range of flows the scour patterns downstream of the weir to select the most suitable radius and invert level of the roller bucket.

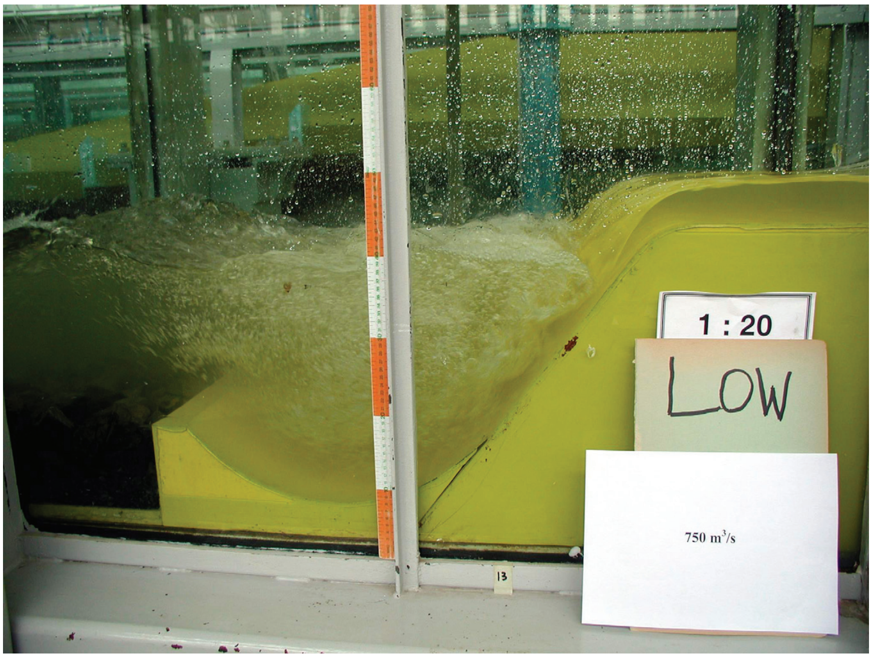


Figure 9-3 Solid roller bucket at 750 m^3/s

A slotted bucket can also be considered, which creates horizontal jets in the slots with energy dissipation as the water leaves the bucket and the bucket can be placed slightly shallower than the solid roller bucket (Figure 9-4).

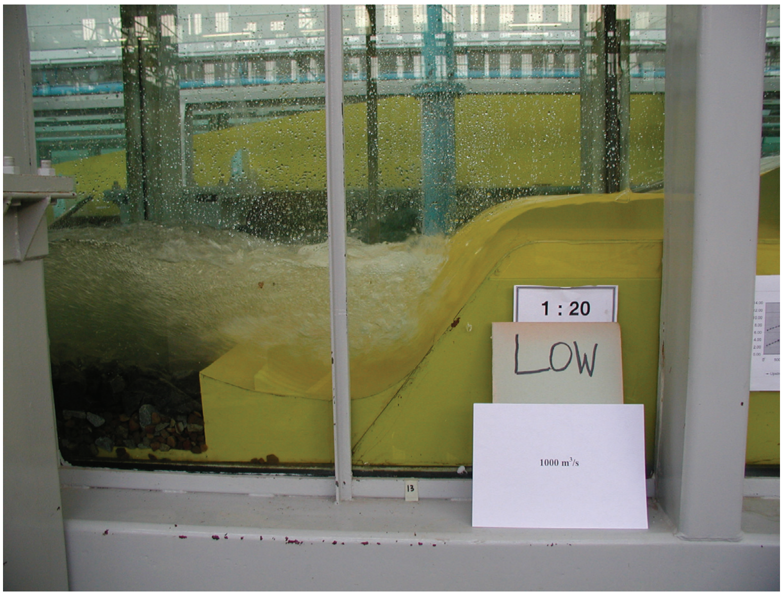


Figure 9-4 Slotted roller bucket at 1000 m^3/s

Two further aspects should be considered when designing the roller bucket:

- possible bed degradation downstream of the structure (general scour and/or local due to turbulence)
- plunging flow when the weir is nearly submerged, with the upstream jet trajectory passing over the roller bucket and hitting the riprap downstream

Figure 9-5 shows a hydraulic model of a roller bucket, while figures 9-6 and 9-7 show flood flow conditions in the field and in the model.

When the weir is constructed on rock, costs can be significantly reduced since a stilling basin may not be required.

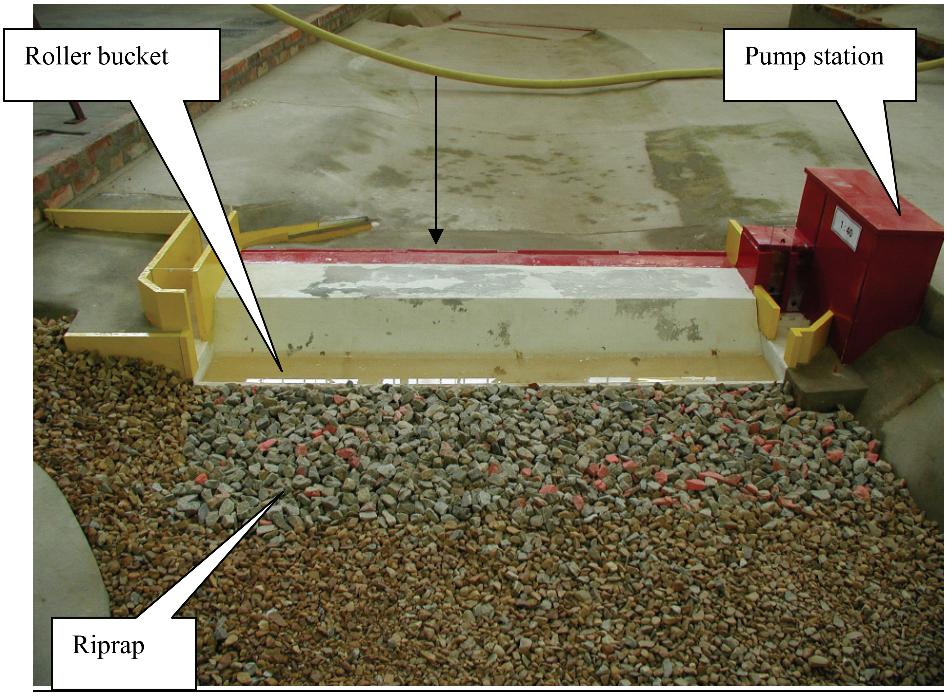


Figure 9-5 Hydraulic model of the Mhlathuze weir

Figure 9-6 Mhlathuze River flood of 2002 at 1500 m^3/s

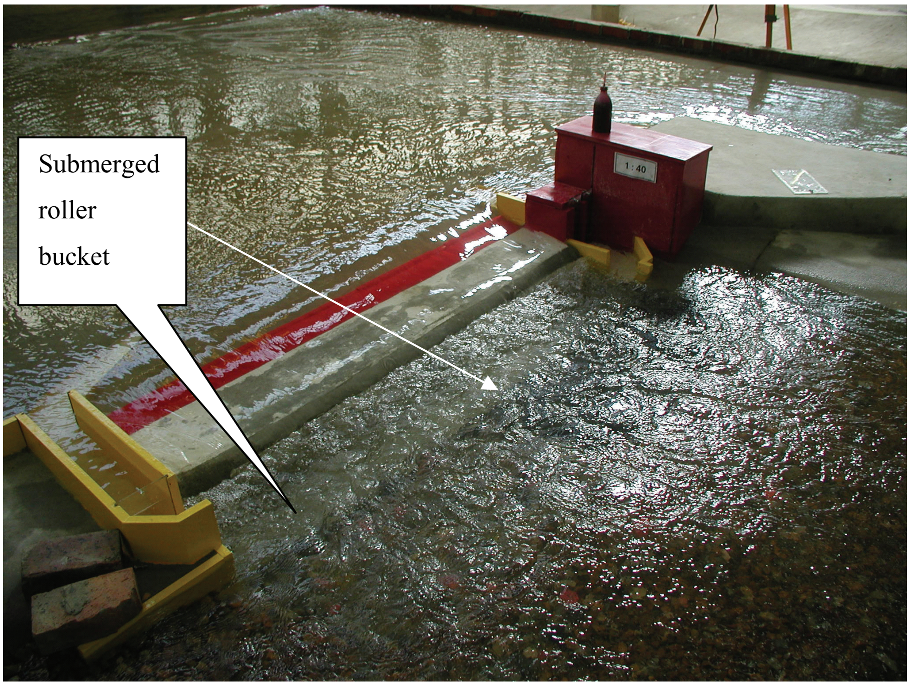


Figure 9-7 Mhlathuze River flood of 1500 m^3/s with 72 m long solid roller bucket

10 ENVIRONMENTAL FLOW REQUIREMENTS

The design must make provision to allow the in stream flow requirement (IFR) to pass without abstracting it. This can be achieved by having a minimum operating level (MOL) with the IFR release at its minimum at this MOL. Possible future changes in the IFR should also be considered. Typically the IFR flow requirement could be adjusted once a month if it is gate controlled.

A low level outlet conduit, normally gated, may be provided to allow riparian flows to be released particularly when the sluiceway gate is closed and low flow conditions exist within the watercourse. Depending on the size and configuration of the structure, it may be appropriate to combine the low level outlet conduit as part of one of the other structure components.

11 SAND TRAP (SETTLER) DESIGN WITH FLUSHING: CONTINUOUS OR INTERMITTENT

11.1 BACKGROUND

Although the design of sand traps do not form part of the scope of this project, their design principles are important for the design of pump canals, and gravel traps and it is important to understand their functioning to incorporate them where necessary as part of the diversion works.

Not many sand traps have been designed and constructed in South Africa, due to a number of reasons:

- Generally the sediment is so fine and with suitable pump selection that can handle solids, no problems are experienced at the river pumpstation, as long as the sand fraction is kept away from high lift pumps
- Reservoir sedimentation of off-channel storage caused by diverted water was not a major problem, but environmental considerations when sediment has to be removed are becoming very important
- Due to the relatively fine sediment (high percentage silt and clay) in many rivers during floods, a sand trap will not be very effective
- The high flood levels compared to drought flow conditions in South Africa make it difficult to construct a sand trap on the river bank that operates under gravity

11.2 SAND TRAP TYPES

Sand traps can be classified in four categories based on their mode of flushing. One of the best descriptions of sand trap design is given in Bouvard (1992).

11.2.1 TRAPS WITH OUTLET SEDIMENT SCOUR

In this design the sediment is removed at the outlet. The flow conditions in the sand trap are such that all the sediment is transported though the sand trap, with the coarse sediment as bed load, and all that is needed is a scour outlet in the floor at the downstream end. The scour gate is permanently open.

11.2.2 Traps with distributed sediment excluder

The Dufour type sand trap has an internal sediment excluder. It has two or more parallel channels with sloping bottoms. Material depositing on the slopes rolls down to a lengthwise gutter along the centreline prior to removal through a scour outlet on the floor. The gutter removes the outflow from the excluder at a velocity of 2 to 2.5 m/s. The exclusion flow usually amounts to 5 to 10 percent of the flow entering the sand trap. The Bieri type sand trap is similar to the Dufour type, but the sediment washout flow is controlled by a hydraulically-actuated steel excluder 15 to 30 m long. The excluder has several sediment excluder orifices on the floor.

Sand traps are typically designed to exclude sediment larger than 0.3 mm in diameter at 100 % efficiency, and are seldom designed to trap sediment smaller than 0.1 mm and then it is not at 100 % efficiency.

Sand trap flow velocities are typically in the range 0.3 m/s to 0.5 m/s.

11.2.3 Traps with intermittent flushing of deposited sediment (settlers)

Settlers are exclusion systems from which the deposited sediment is removed periodically. Settlers are ideally suited for South African systems with their typical throughput of 4 to 6 m^3/s. They operate on quite a different manner from sand traps and consist essentially of a channel with a mean flow velocity of 0.1 to 0.2 m/s. Sediment entering the settler first deposits at the upstream end, and the settler gradually fills up from upstream. When the load deposited in the settler becomes critical, a sluice gate at the downstream end of the settler is opened and the sediment is flushed back to the river. During flushing no water can be diverted. Settlers are thus operated in two distinct modes: deposition and flushing. The inconvenience of interrupting the diversion of water during flushing is however not as serious as it might seem. If the structure is correctly designed washout will be hydraulically efficient and will seldom take more than 15 minutes even for long settlers. Compared with continuous exclusion systems where water is inevitably wasted, settler can actually reduce water wastage by the short duration of flushing required.

Settler volume is based on the sediment transport capacity and sediment load, but as a general rule the volume should not be more than about 200 times the discharge per second into the settler, while in practice the volume is usually closer to 100 m^3 per m^3/s of inflow.

Settlers should be absolutely straight and symmetrical from one end to the other, since flushing flow occurs under supercritical flow conditions where it is difficult to change direction. Trashracks may be required upstream of settlers and the sediment particle size entering the settler determines the slope of the settler. Typical settler bed slopes should be 2 to 3 %, but not steeper than 5 % since the bed expansion will induce extra turbulence.

During flushing sediment scour takes place retrogressively, starting at the downstream end and cutting back upstream. The effective energy slope created at the steep front of the retrogressive erosion is very effective to remove even cohesive, consolidated sediment. The critical bed slope (So) for flushing of non-cohesive sediments required for a settler can be determined by using the Shields or Liu diagrams, or Bouvard (1992) proposed:

$$So = 0.44\ d^{(9/7)}/(q^{(6/7)})$$

where
q = unit discharge (m^3/s)
d = sediment particle diameter (m)
A safety factor of 1.5 is usually applied to this slope in practice to make it slightly steeper and more effective for flushing.

Settlers typically cannot exclude sediment particles smaller than 0.1 mm, but can be designed to trap some of the silt load such as at the Lebalelo and Vaal Dam (VRESAP) schemes.

Figure 11-1 shows essential design features of a settler.

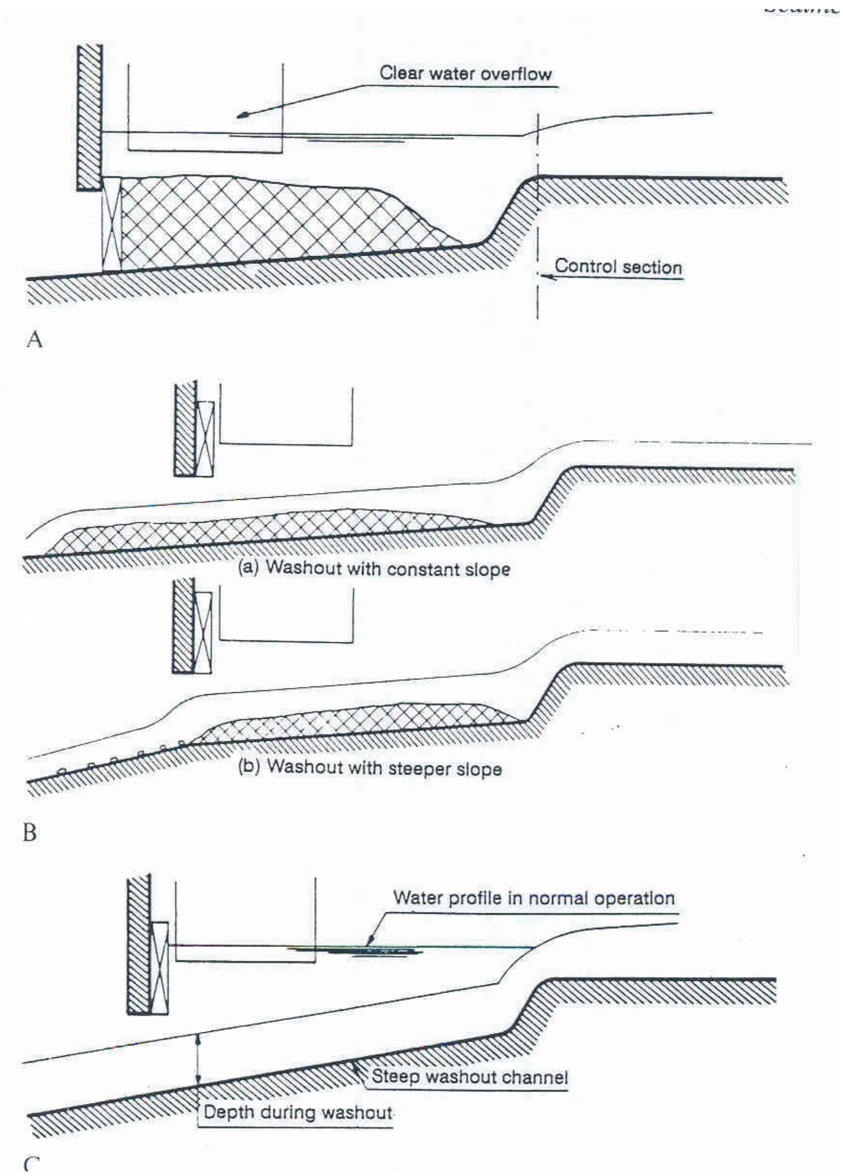


Figure 11-1 Essential design features of a settler (Bouvard, 1992).

During floods efficient flushing requires that the scour flow be limited from upstream, rather than by a gate at the downstream end. An upstream end gate also makes it possible work inside a settler if maintenance work is required, while other settlers are operational. The downstream gate should be at the bed level, should preferably be as wide as the canal and should allow free outflow conditions during flushing.

The settler type of design has very few mechanical components, is an open system which can be operated without risk of clogging of pipes or openings, and is effective with limited water wastage during flushing.

At Lebalelo on the Olifants River, the low head pumpstation at the river pumps water to a settler consisting of 4 parallel canals, each 2.5 m wide and 80 m long. In deposition mode the river water is discharged at the deep end of the settler where it is 6 m deep, and clear water is extracted from the surface at the downstream end where the depth is only 3 m. The settler was designed to trap some of the silt, not only sand, by using a one dimensional model with an advection-dispersion module to describe fine sediment transport and deposition in the settler. The design efficiency is 70 % trapping. The first part of the settler where the pipes enter is inefficient due to the turbulence generated.

What makes the Lebalelo design unique is that the flow direction is reversed during flushing by valves, and the steep bed is used to scour out sediment within a short period of time. One canal is then flushed at a time, by using pumped water from the river and under free outflow conditions in the settler. Short duration flushing during floods are preferred to continuous flushing throughout the year including low flow periods when considering the environmental impact.

Figures 11-2 to 11-6 show details of the Lebalelo settler in operation.

Figure 11-2 Depositional mode

Figure 11-3 Settler flushing during partial water level drawdown, Lebalelo

Figure 11-4 Settler outlet gate during flushing

Figure 11-5 Settler flushing completed at Lebalelo with free flow conditions, Olifants River

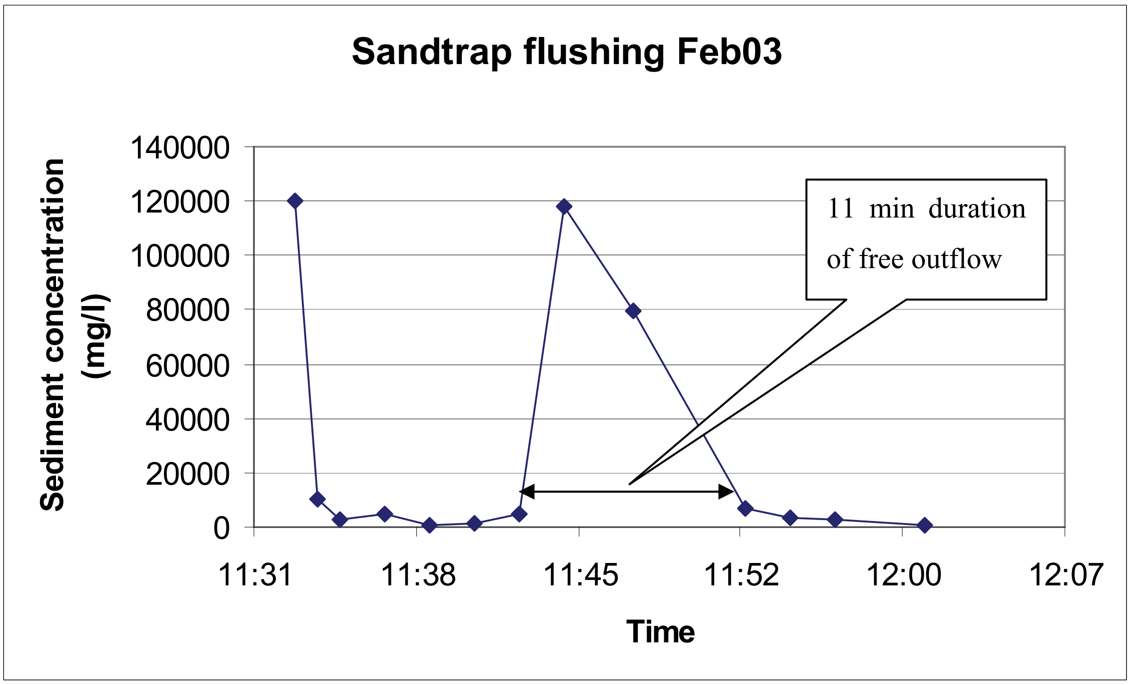


Figure 11-6 Observed outflow sediment concentrations at settler flushing at Lebalelo

11.2.4 TRAPS WITH CONES, CONTINUOUS FLUSHING AND MECHANICAL/CHEMICAL SEDIMENT EXCLUSION

Other types of sediment excluders have been constructed in South Africa based on technology used in water purification technology. Such systems have been constructed at the Phalaborwa Barrage, Upington and Craighead abstraction works (Figure 11-7). The design uses a conical shaped reservoir with the turbid water pumped in from the bottom in the centre. Deposited sediment is flushed out at the bottom from time to time, returning to the river, or a continuous flushing system is installed. Clear water is extracted from the surface. Sometimes chemicals are added to induce flocculation and sediment deposition, but this is expensive. Moving mechanical scrapers are also added in some designs to facilitate removal of deposited sediment.

The retention time in the trap should be long enough to allow the design sediment particles to settle out. A particle diameter of 0.1 mm for example has a settling velocity of 0.007 m/s and will therefore take 2 min 23 s to settle through 1 m of water depth. If the design sediment diameter to deposit is however fine sand at 0.03 mm, it will take 30 minutes to settle through 1 m water depth. The volume of the trap and flow area should therefore be quite large.

Another risk with this type of design with a conical bottom shape is that cohesive sediment does not slide down the bed slope. When water jets are incorporated, slumping of the fine sediment could cover and clog the flushing bottom outlet and pipe system completely.

These traps are generally very expensive to build and to operate.

Figure 11-7 Craighead sand trap, Amatola Water

12 Flow measurement

The diversion weir can often be converted to measure flow. For reliable discharge measurement the requirement is that the weir should consist of several notches with steps of about 300 mm, and a Crump weir crest is preferred. This layout is however not suitable for effective energy dissipation downstream when the weir is not constructed on rock. Flow gauging at a river diversion should therefore not determine the hydraulic layout and is of secondary importance to the diversion hydraulics and sediment control functioning.

13 Fishway types and layout considerations

13.1 Background

Typically, a fishway is required to allow fish to travel past the diversion structure.

The type and design requirements of the fishway is dependent on a number of variables including: the species, size and numbers of fish expected to use the fishway; the range of discharge water levels, depths, and flow velocities within the watercourse during the migratory period; the difference in water level across the structure; attraction and guidance flow requirements at the fishway entrance; and lighting conditions.

Special care is required to properly locate the fishway relative to the other structure components such that fish are attracted towards and can readily find the fishway, as well as in choosing the correct design so that fish can traverse the fishway. Particular care is also required in determining the attraction flow characteristics that will be needed to help fish find the fishway entrance, and similarly, the guidance flow characteristics needed for fish to make their way through the fishway. As a result, input form a fish biologist respecting the location, orientation, type, and design requirements for the fishway is normally required.

Fishways have typically been comprised of hard-engineered structures constructed using concrete, metal or wood; however more recently, consideration is being given to the use of naturalized fishways constructed using materials such as large boulders.

Many diversion weirs are low enough to be submerged during floods and allow migration of fish, but during low flow periods fish cannot cross the weir. Fishways are therefore needed at weirs. The hydraulic design of fishways has been well researched internationally and a very useful reference is by Vigneux and Larinier, (2002). Currently in South Africa at least two WRC studies address the hydraulic design aspects of fishways. The selection of a specific design depends on the characteristics of the fish.

13.2 FISHWAY POSITIONING

A fishway should be positioned near one of the banks, at the most upstream extent of the weir if possible, but it is also important that low flow must be able to reach the fishway under all conditions. Fish will tend to swim upstream during migration along the banks and when they reach the obstruction (weir), they could traverse the river probably to the most upstream flow turbulence near the weir. Therefore at its downstream end the fishway must have sufficient turbulence created by its flow to attract fish.

A fishway located across the weir spillway face of existing structures has often been proposed in South Africa to reduce the gradient and cost. This layout will however affect the energy dissipation in the stilling basin during a flood and the hydrodynamic forces are extremely high.

Figures 13-1 and 13-2 provide good and bad layouts of the fishway and weir.

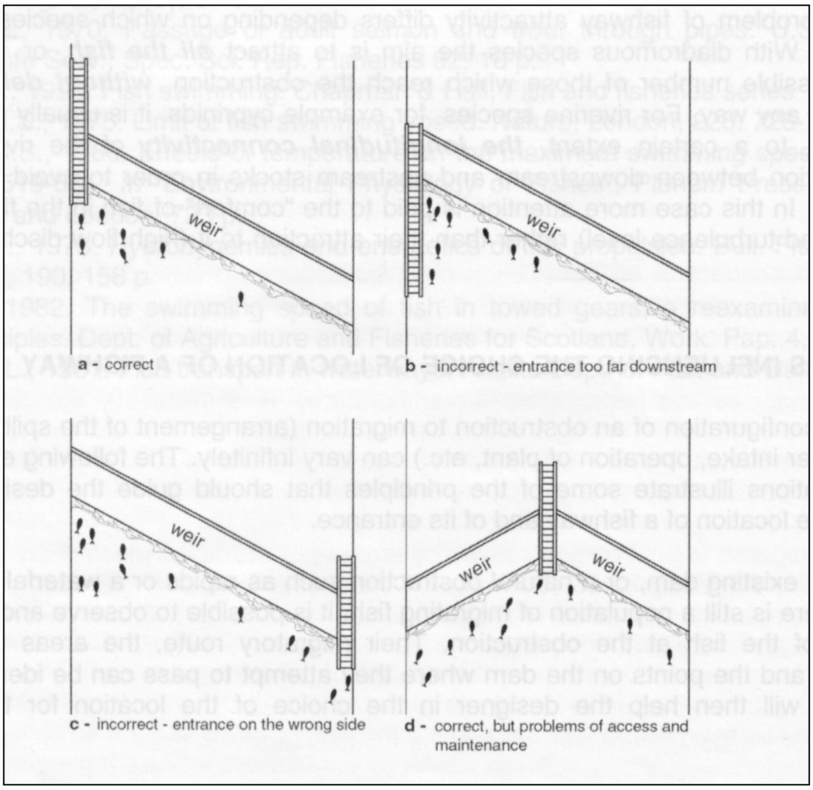


Figure 13-1 Schematic plans illustrating the installation on an oblique weir (Vigneux and Lariner, 2002)

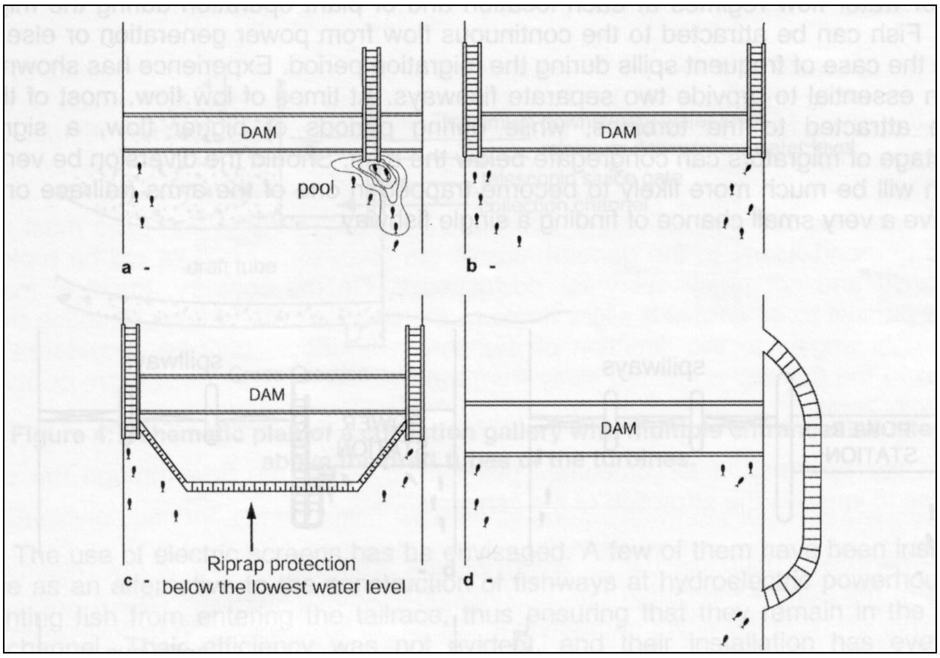


Figure 13-2 Installation of a fishway on a weir that is at right angles to the flow direction (Vigneux and Lariner, 2002)

13.3 FISHWAY TYPES

The type of fishway which is most frequently used is the pool fishway. This consists of a series of pools in steps leading from the river at the foot of the obstruction to the river above. The pools have a double function: they ensure a proper dissipation of energy and provide resting areas for the fish. The slope is typically 10% to 15%.

Baffle type fishways have baffles on the bottom and or sides of a flume. The flow is more intensively aerated and turbulent than in conventional pool fishways (Figure 13-3).

Pool and orifice type. This configuration consists of a series o stepped pools that are separated by weirs or cross walls. Water flows from pool to pool through a submerged orifice.

Denil type. This configuration consists of a steep flume or trough which has vanes installed on the sides and bottom.

Pool and jet type. This configuration is similar to the pool and orifice type except the orifice consists of a vertical slot that extends for the full height of the weirs or cross walls.

Partial blockage of the flow passages, particularly for engineered fishways, could result in unfavourable conditions such as reduced opening sizes and higher velocities. Consequently, consideration should be given to incorporating a trash rack at the upstream end of the fishway.

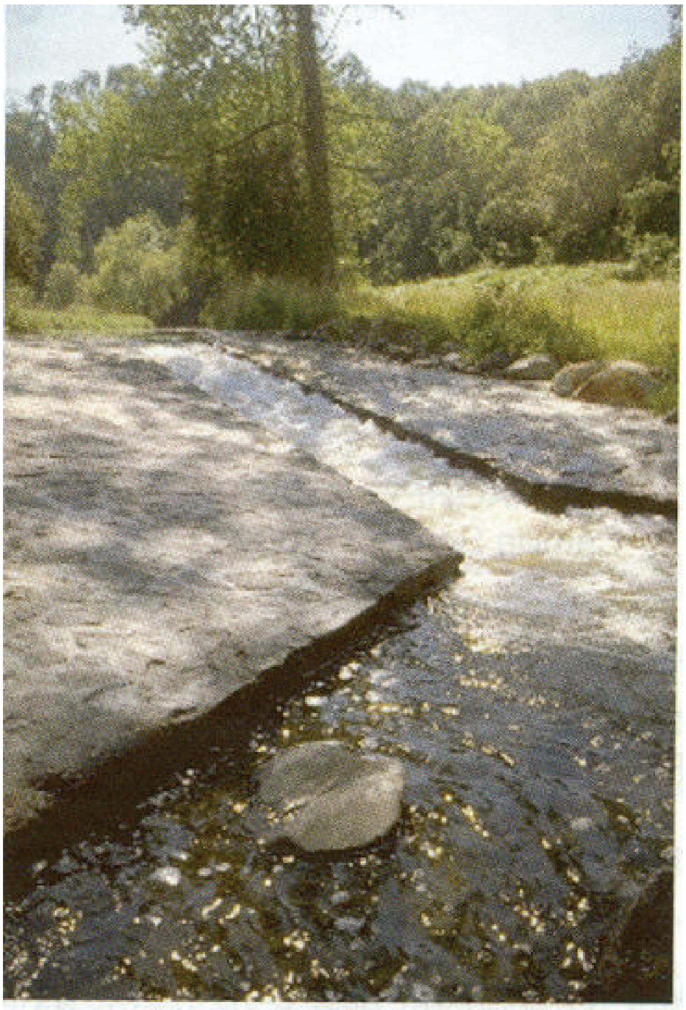


Figure 13-3 Baffle type fishway in operation (Vigneux and Lariner, 2002)

Naturalized fishways are a relatively recent development. They are gaining attention and becoming more acceptable to the public and regulatory agencies primarily because they are designed to simulate the natural stream and are constructed using natural materials.

Pool and riffle type. This configuration consists of a stair-step arrangement comprised of a short steep channel section followed by a pool section.

Rocky ramp type. This configuration consists of a long sloping channel impregnated with large boulders to produce a “boulder garden” effect.

The natural bypass channel, or division channel fishway consists of a channel excavated from upstream to downstream in one of the banks of the river. The channel is roughened with artificial or natural obstacles. The slope of such a facility may not be more than a few percent, and because of its length the use of this type of installation is therefore often constrained by the limited space available. On the other hand it usually blends in very well with the landscape.

The use of rocks to create an artificial canal on one bank is the most natural and should be considered wherever possible, but constraints are a long reach due to slope requirements and relatively large discharge to create sufficient depth. The fishway should preferably not be steeper than natural gradients on that specific river, where one would perhaps find a maximum natural gradient of say 1:100 which will result in an extremely long fishway. Such a design has been used successfully on the Sabie River in the Kruger National Park as shown in Figure 13-4.

Figure 13-4 Sabie River “natural” fishway created by large rocks

13.4 CRITERIA FOR THE CHOICE OF A FISHWAY

When several migration species are involved the pool fish pass or the natural bypass canal are the best solutions, as they are much less selective than baffle fishways.

Pool fish passes with notches, specifically vertical slot fish passes are particularly well suited to sites where upstream and/or downstream water level fluctuations are significant.

Baffle fishways are suitable for species with sufficient swimming capacity in terms of both swimming speed and endurance. They are generally not suitable for small fish measuring less than 0.3 m in length.

Baffle and deep pool fishways are not installed in rivers with coarse bed material, such as gravel and cobbles which are likely to be deposited between the baffles.

13.5 DESIGN OF A VERTICAL SLOT FISHWAY

The vertical slot fishway is probably one of the best designs for South African conditions. The vertical baffles create resting areas, while the slots control the discharge and create the required flow depth and hydraulic conditions. The slots are placed in line near one side of the fishway and the baffles should be far enough apart to limit vertical flow patterns. The dimensions of the slotted vertical fishway is based on unit input stream power.

These fishways have been well tested in scale models and are widely applied. Figure 13-5 shows a layout of such a fishway. Generally a step of at least 200 mm is installed at the base of the slot.

Figure 13-5 Vertical slot fishway

The difficulty of passage for migrators increases with the turbulence and aeration in the pools. The required minimum size of the pools between the slots is given by the following stream power (P) equation:

$P = \rho g Q dH/_{Vol}$

Where P= volumetric dissipated power (watts/m^3)

ρ = density of water

g = gravitational acceleration

Q = discharge in fishway

dH = head difference between pools

Vol = Volume of water in the pool

200 Watts/m^3 is generally taken as the upper limit for adult salmon and sea trout fish passes and lower levels are advisable < 150 Watts/m^3 for small fishways.

To design a fishway in a specific river it is probably best to consider the local hydraulic conditions in the river reach. This was done for the upper Berg River where a fishway had to be designed at the diversion structure. By surveying a representative reach (100 m length) and measuring water levels and flow velocities during low flow periods in summer and winter, it was possible to determine the stream power distribution in the river reach (See Figures 13-6 to 13-8). The unit input stream power (P) is calculated from the survey data with a grid spacing of 2 m (across river) x 5 m used at this site, by using the following equation:

$$P = \rho g V S_f$$

Where ρ = water density, g = gravitational acceleration, V = measured average flow velocity in a grid point, S_f = Local energy slope at grid point which can be calculated from the water level survey. Figure 13-9 shows the plan plot of stream power and it is clear that a fishway design of say $\leq$ 150 W/m^3 would be suitable on this river for low flow conditions.

Figure 13-6 Berg River 100 m reach viewed from downstream (site 1)

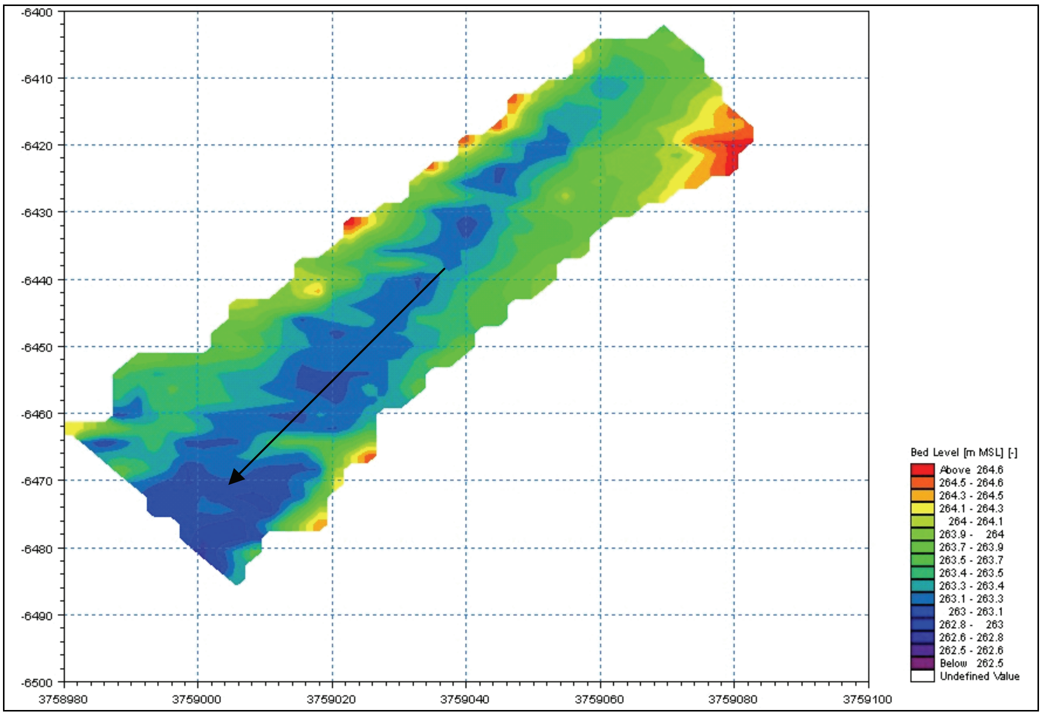


Figure 13-7 Summer 2003 bed level survey at Site 1

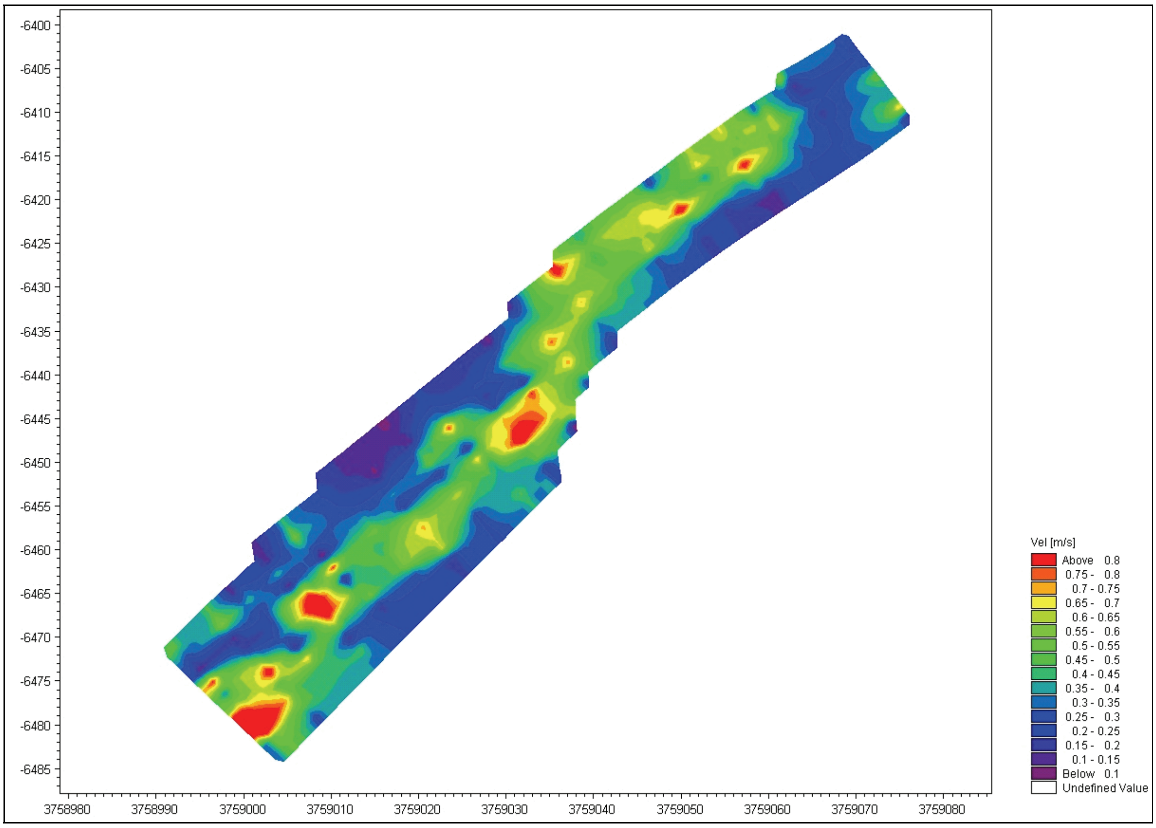


Figure 13-8 Winter 2003 measured velocity distribution at Site 1

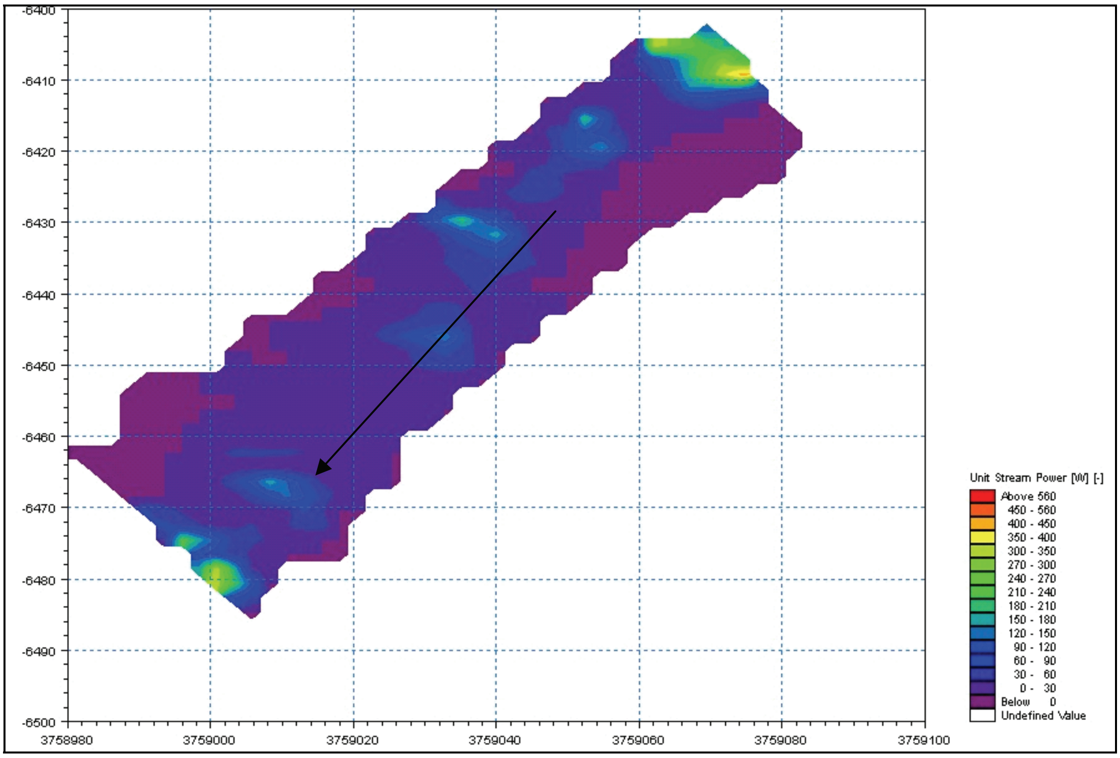


Figure 13-9 Plan plot of observed unit input stream power at site 1

14 CANOE CHUTE REQUIREMENTS

Canoe races are held annually on many South African rivers and when a low weir is constructed, where possible a safe canoe chute should be incorporated in the design. Canoeists can safely negotiate a weir of 2 to 3 m high, usually by crossing at the low flow notch straight in line (Figure 14-1) with the flow when the downstream depth is deep enough to prevent the nose from hitting the bed, or by approaching the weir slowly nearly parallel to the weir and to slide down the weir face usually with some paddle support against the weir (Figure 14-2). In both cases the hydraulic jump downstream, especially if it is a stable or submerged jump, could trap the canoeist which could be highly dangerous, especially if the canoe capsizes.

Figure 14-1 Straight weir descent on the Great Fish River

Figure 14-2 Near parallel descent with the weir on the Great Fish River

On the Msinduzi River at Pietermaritzburg a relatively high stepped concrete chute was provided to dissipate the energy on the steps and it works fairly well but is difficult to negotiate for novices, especially when re-entering the river and to keep straight on the chute. On the Berg River at Paarl a weir and canoe chute was constructed during the 1980s. The chute is however very steep and the river downstream shallow with a hydraulic jump. This chute is dangerous under all flow conditions.

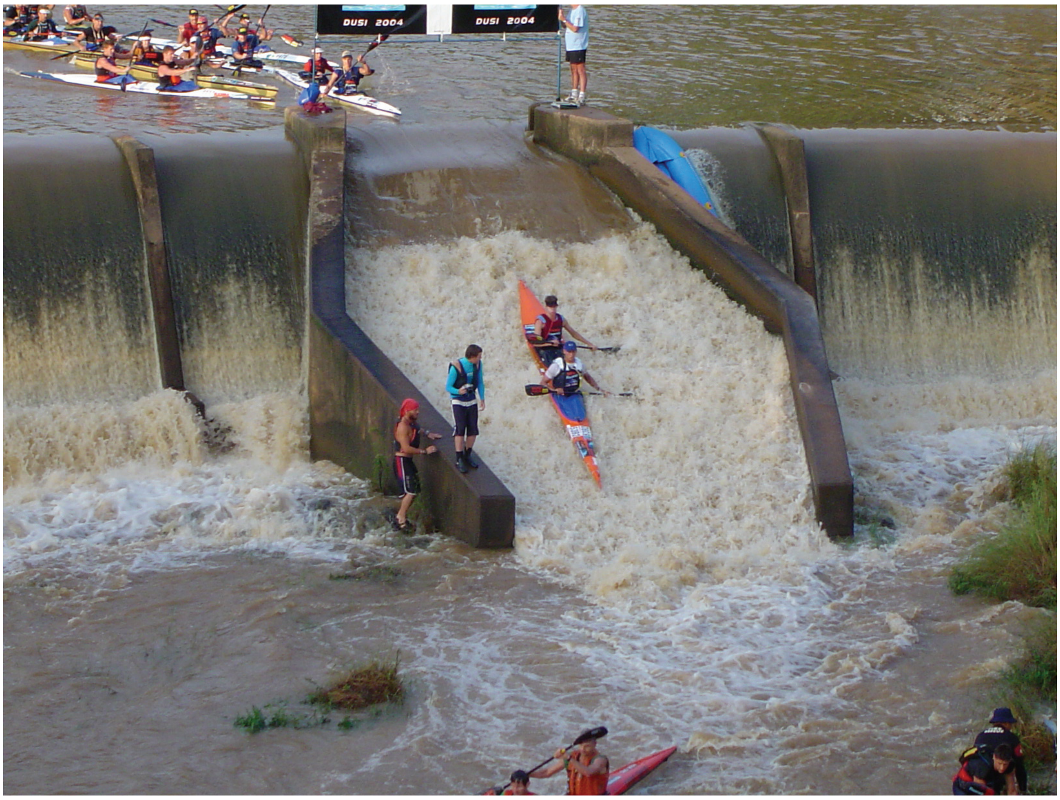


Figure 14-3 Mzinduze canoe chute, Pietermaritzburg

The problem with a canoe chute design is that a stable hydraulic jump is required for the design of the weir, especially if the weir is not constructed on rock. The stable jump creates the so called stopper which can be dangerous. Figure 14-4 shows a stable hydraulic jump downstream of the weir. Therefore the chute should have a small gradient, with bed roughness (steps) to improve energy dissipation, and at the toe of the canoe chute the water jet should be as horizontal as possible to wash out the hydraulic jump. There is however an alternative design that could be considered such as a step-pool design with horizontal contractions, with each pool longer than the canoe length. This design requires a long chute and a high base flow in the river.

Figure 14-4 Stable hydraulic jump at bottom of weir stopping the canoeist

It is recommended that a physical model study is carried out of a canoe chute design, because an incorrect design could result in loss of life and even possible failure of the weir.

15 WEIR SEDIMENTATION

A weir creates a reservoir upstream which the river can fill quickly with sediment, in fact at most weirs they will silt up within the first year to an equilibrium level which is close to the crest of the weir near the weir and extending above full supply level further upstream. The consequences are twofold:

- the intakes could be covered with silt and the sediment diversion ratio could increase drastically

- Flood levels upstream will rise which could impact on expropriation of land and the cost of the abstraction works.

Even if the sedimentation rate is slower, the long-term equilibrium condition must be used to design a sustainable solution. A hydrodynamic mathematical model linked to a sediment transport module should be used to determine the equilibrium sedimentation profile based on field data calibration wherever possible, since empirical methods cannot predict the sediment trap efficiency of a weir accurately. It is also important to consider cohesive (fine) sediment deposition and erosion.

16 SIZING OF DIVERSION CAPACITY AND EFFICIENCY

Most river abstraction works in South Africa divert water to a balancing dam. Both the diversion discharge capacity and the balancing dam size have to be optimised together. Recorded river flow data should be selected and routed to the diversion site. This flow record should be tested for representivity by plotting cumulative flow against time.

Hourly time steps over say a 20 year period should be used to determine the water balance in the diversion and the required dimensions and risk of failure. An economic analysis should be carried out to determine the best design.

17 SEDIMENT DIVERSION EFFICIENCY

A time series of sediment loads in the river as well as the sediment characteristics of transported sediment are required. This data could be obtained from suspended sediment sampling in the river at the diversion site including as much flood data as possible. The sediment load-discharge rating thus obtained can be integrated with the flow record to establish the time series of sediment loads and a sediment yield, which should be compared with the regional sediment yield of the river. By using say one hour time steps it is possible to calculate the diverted sediment load.

The next step is to calculate sediment transport and deposition through the gravel trap and sand trap/pump canals. A one-dimensional mathematical model can be used to determine the trap efficiency of the sandtrap/pump canals and will also give the sediment fractions reaching the pumps. A multifraction sediment transport model with clay and silt fractions should be used.

18 MINIMIZATION OF THE IMPACT OF FLUSHING ON THE RIVER

Concerns in terms of the downstream river geomorphology are:

- the possible deposition and build up of flushed sediment locally
- high sediment concentrations during flushing, especially during low flow periods when only the instream flow requirement is released.
- Long duration flushing

If intermittent flushing is carried out of the gravel trap, pump canals and sand traps, the impact on the downstream river should be limited, especially if the duration of flushing is short. Short flushing duration also means less water wastage and can be obtained by designing hydraulically steep canals of 2 to 5 %. All the canals also do not have to be flushed at the same time.

The impact of a specific flushing operation is site specific and local suspended sediment concentration data should be obtained to establish the design and operating rules. Figure 18-1 shows observed suspended sediment loads in the Thukela River at Mandini obtained over a 10 year period where the sediment yield is 400 ton/km^2.year with a total mean load of close to 10 million tons per year. The bed load sediment transport should be added to this data, and if not measured is often taken as 25 % which also compensates for non-uniformity across the river. Flushing should not be carried out when the river flow is at the IFR, but rather during freshets or floods. The sediment loads in the river downstream (not at the flushing gate but say 100 m downstream in the river) during flushing should not exceed the maximum range (or say 95 percentile value) of observed sediment loads for a specific river flow (Figure 18-1).

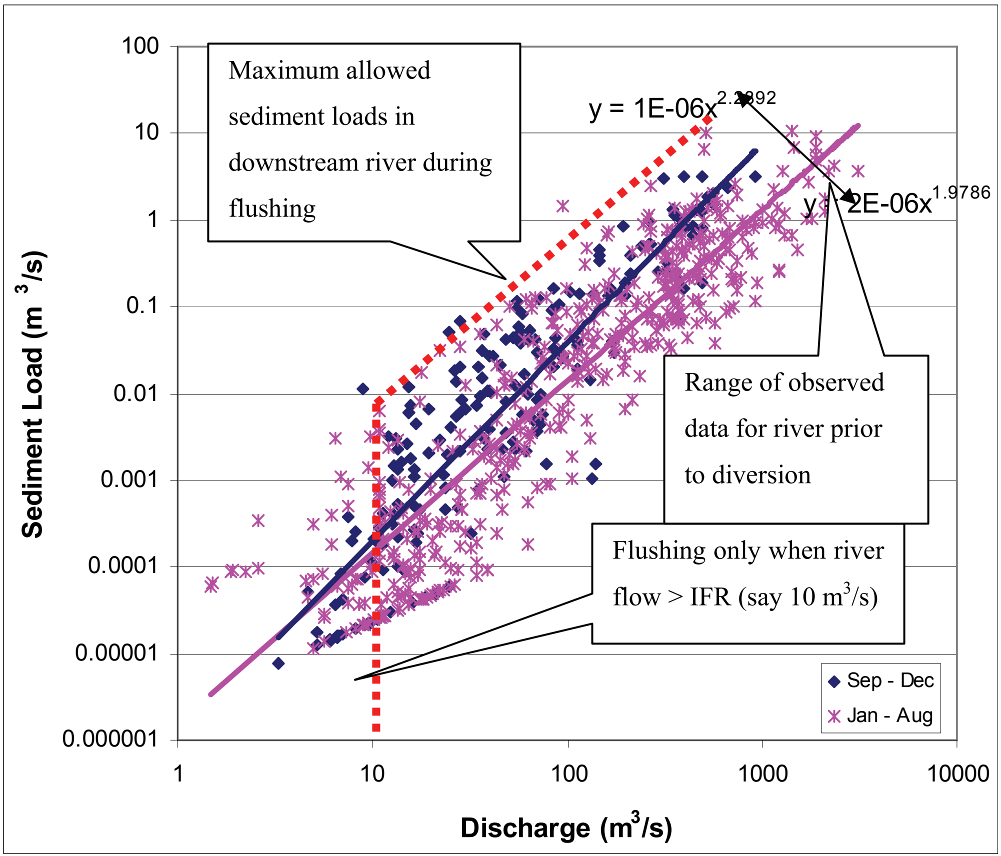


Figure 18-1 Observed sediment load-discharge relationship for the Thukela River, Mandini, and possible flushing design

The required sediment loads in the river downstream can be controlled by monitoring and adjustment of the flushing gates to limit sediment transport. Release of clear water at the end of the flushing period will also help to minimize the impacts.

19 Operation and Maintenance

It is important that an operational and maintenance manual is written and implemented. Where flushing facilities have been implemented it is important that judicious operation is carried out to control sedimentation.

At existing pumpstations, emergency sediment removal by jet pumps is possible in sandbed rivers without a weir. In a WRC study by Bosman et al. (2003), the use of jetpumps to dredge sand at conventional pump intakes was investigated.

19.1 JET PUMP TECHNOLOGY FOR SAND DREDGING AT PUMP INTAKES AS EMERGENCY SOLUTION

Jet pumps have been used for maintenance dredging at ports in South Africa over many years. The benefit of these pumps are that no moving parts come in contact with the sediment. Clear water is pumped through a nozzle into a mixing chamber where water and sediment mixes for transport through a pipeline (Figure 18-1). Water jets can also be used to loosen sand. A jet pump is generally not as efficient as a centrifugal pump.

In a recent research project sponsored by the WRC, jet pump technology for the removal of sand at DWAF pumpstations in Mpumalanga was investigated (Bosman et al, 2003).

Where pumpstations are not located and designed properly, the pool where water is abstracted from could fill with sand. Jet pump technology could help to remove the sediment, as shown in Figures 18-2 to 18-4, with fixed or movable installations. The jet pumps are driven by water from the main pumps in the pumpstation.

The field tests by DWAF showed that dredging with a jet pump is expensive and should be seen as a last resort. Added to this is also the problem of disposal of dredged material (Figure 18-5).

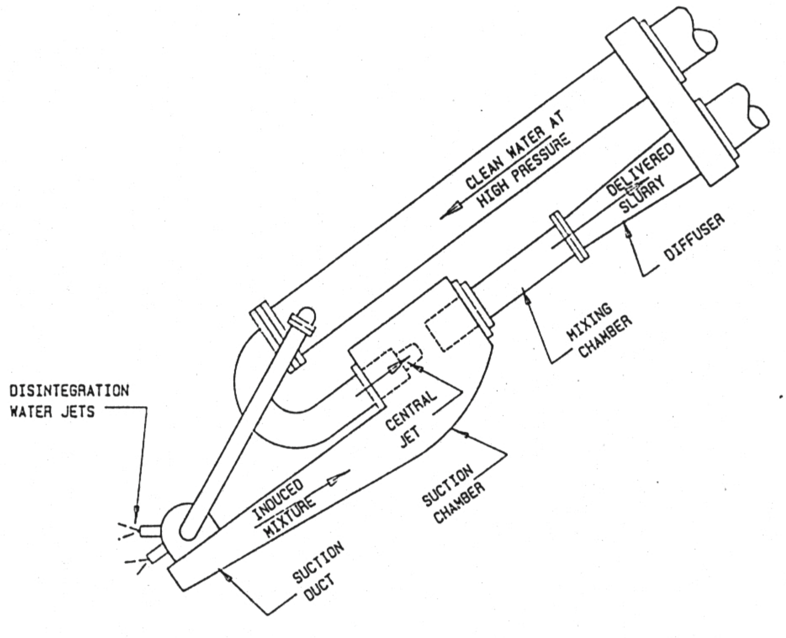
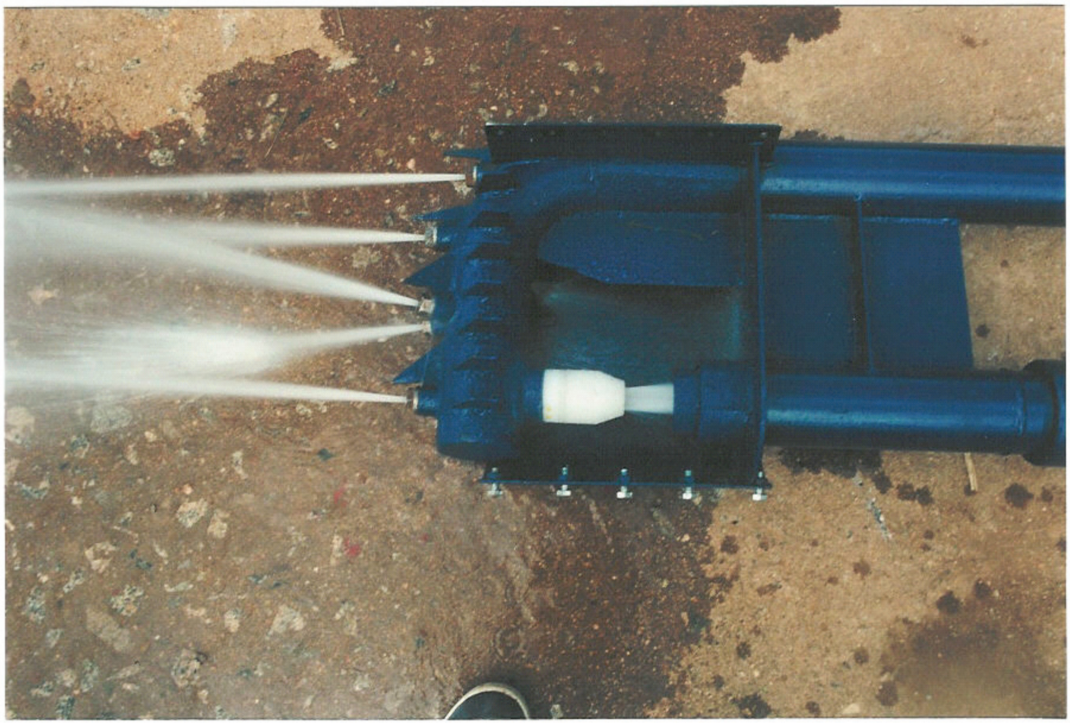


Figure 18-1 Jet pump technology (Bosman et al. 2003)

(a) Jet pump motive supply tap off from raw water discharge pipe in foreground and installed fixed jetpump in background.



(b) Close up of activated GENFLO SANDBUG® activated showing main and fluidizer/disintegrater nozzles. (half of cowl removed).

Figure 18-2 Movable jet pump operating to create pool (top) and jet pump with nozzles (bottom) (Bosman et al, 2003)

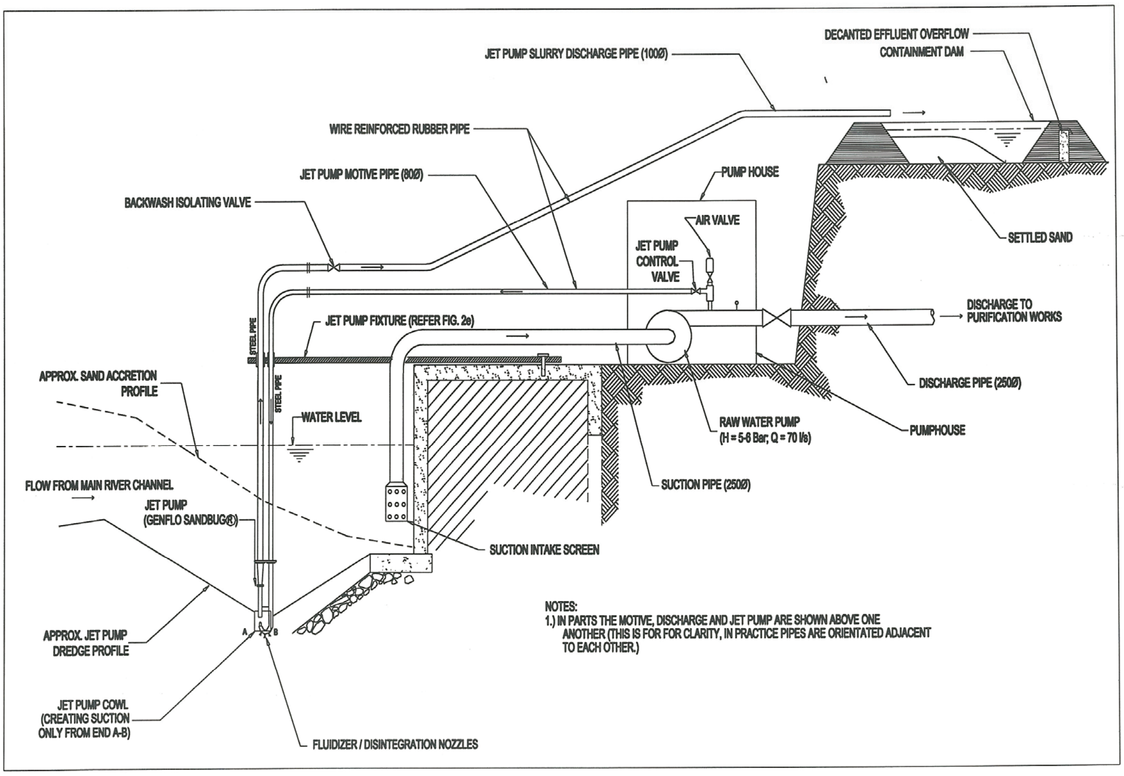


Figure 18-3 Typical design of a fixed jet pump test system at a river pump station (Bosman et al., 2003)

Figure 18-4 Mobile jet pump system mounted on trailer in operation (Bosman et al, 2003)

Figure 18-5 Dredged material disposal creates ecological problems and is expensive.

20 Conclusions and Recommendations

This document should be read in conjunction with the Volume 1 document of this series: "Sediment Control at River Abstraction Works in South Africa", by CJ Brink, GR Basson and F Denys (2005), which gives more in depth descriptions of diversion/abstraction works types as well as covering theoretical aspects.

The following are proposed designs for South African conditions, in order of priority:

a) Pumpstation/diversion without a weir, located on the outside of a stable river bend
b) Low weir with low level flushing gate(s) in a gravel trap, and pump canals/sand trap that can be flushed, combined with a bend in the river. The recommended layout should include a submerged gravel trap such as in Figure 20-1, or a boulder and gravel trap as shown in Figure 20-2
c) Barrage on river with large gates across the river, combined with a stable bend in the river
d) Weir, flushing canals, deep sand trap (pit) and jet pumps to clean the pit
e) Sand pump system with infiltration gallery

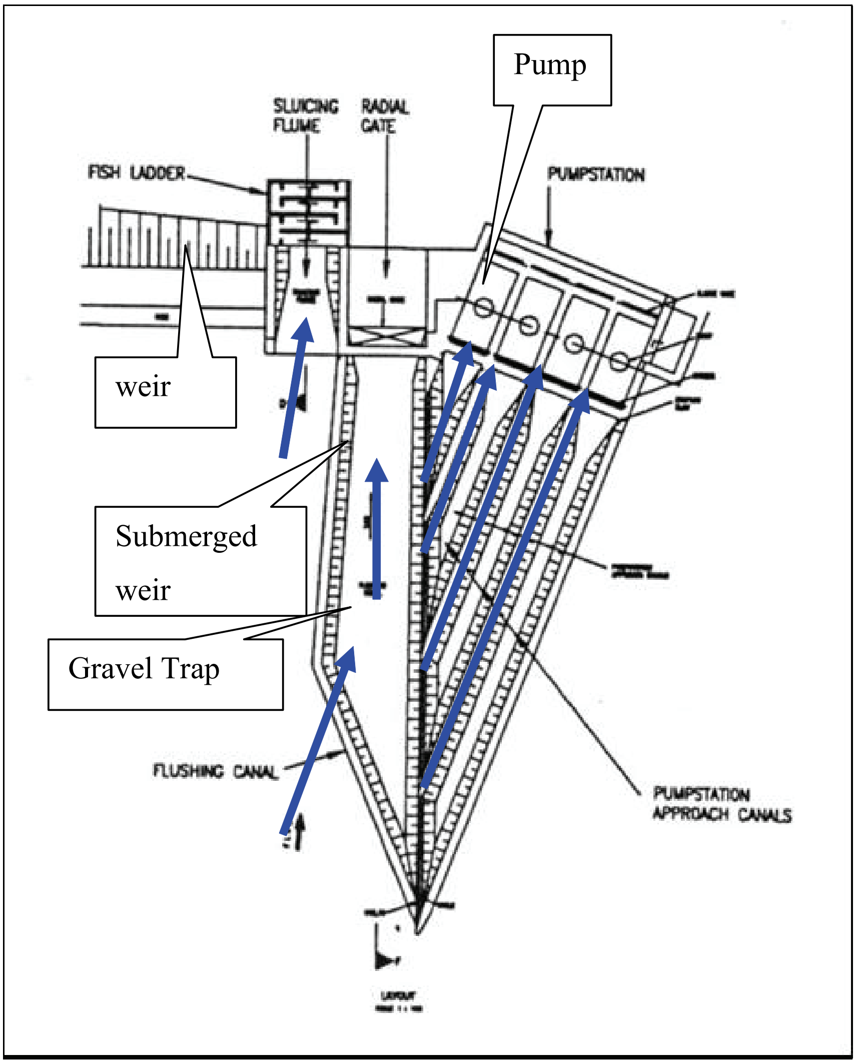


Figure 20-1 Lebalelo pumpstation layout, Olifants River (gravel trap)

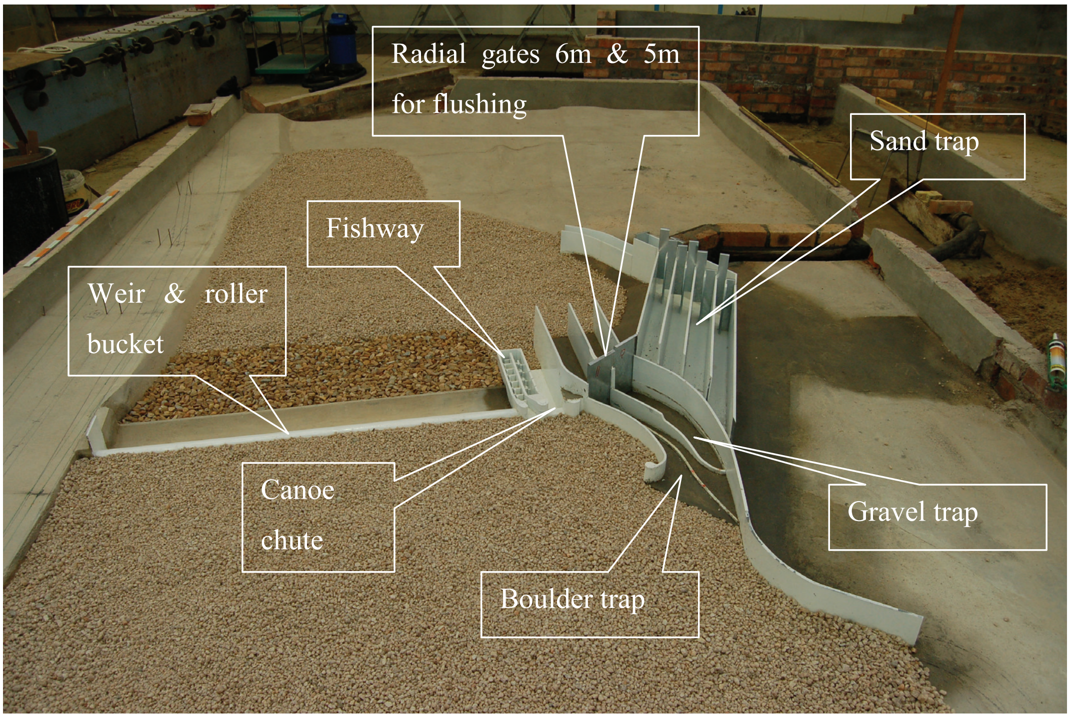


Figure 20-2 Berg River abstraction works layout (gravel and boulder traps)

It is recommended that the design of a river abstraction works is based on the design guidelines reviewed and developed in this study. The following are some of the key aspects to consider:

- Assess river stability from historical aerial photos and satellite images.
- Consider low flow conditions and flood flows and the variability in sediment loads. The environmental flow requirement must be released downstream during low flow periods and the diversion must operate during floods.
- Locate the diversion on the outside curve of a river bend to limit coarse sediment diversion and to scour a deep pool at the intake during floods, which should still be present during low flow periods.
- Use a mathematical model and/or physical hydraulic model to simulate the sediment dynamics to select the best position and orientation of the diversion at important abstraction works.
- Fine sediment will enter the diversion, therefore allow for flushing under gravity back to the river. Even the pump canals can be flushed.
- A gravel trap should be provided upstream of the pump/diversion canals.
- Robust pumps, preferably submersible, should be selected to handle the coarse sediments.

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