Report # 1

Report on Spurwing Dam,
Letata, Dullstroom

for

Cutwater Farms (Pty) Ltd

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Summary

An inspection of the Spurwing dam was made by Dams for Africa in March 2004. The state of the dam is discussed in this report. Reference is made to dam safety legislation in South Africa. The dam's functionality is considered in terms of the stability of the embankment, its impermeability, and the adequacy of the two spillways in dealing with various types of floods. The principles and requirements of a spillway are set out, and the deficiencies in the two spillways are considered against this backdrop. It is recommended that the right flank spillway be re-engineered. Three options are discussed and one is recommended. A preliminary/generic design is outlined, and a course for future action proposed.

Report on Spurwing Dam, Letata, Dullstroom

1.0 Background and Brief

On 19-03-04 Dams for Africa were appointed by Mr Guy Fletcher representing Cutwater Farms (Pty) Ltd. to investigate the Spurwing Dam on Letata Farm near Dullstroom.

In the brief (see appendix) it states that the dam was built about 12 years ago, with the spillway originally on the left flank of the dam. (Note that in the design of dams, convention dictates that you face the dam as if you were the river flowing into the dam when referring to its left or right flank).

In 2002 and 2003 the runoffs into the spillway resulted in creeping erosion which was apparently serious enough for the decision to be made to cut another spillway on the right flank. This spillway is referred to hereafter as the *right flank spillway* and has been in operation for about a year.

In the mean time the eroded areas of the original *left flank spillway* have been filled in and grass has been planted. The mouth of this spillway was closed off by constructing a berm about a meter in height and four meters wide across the full width of the mouth of the spillway. The height of this berm is about 1,00m above the right flank spillway's overflow level, referred as the *full supply level (FSL),* and about 0,45m below the crest of the embankment (see figure 1), so that it will allow the old spillway to act as an auxiliary spillway in times of exceptional rainfall to supplement the capacity of the right flank spillway.

In recent months heavy rainfalls have resulted in significant flows through the right flank spillway, resulting in unacceptable levels of erosion in the spillway and adjoining embankment. This prompted the question: was the spillway correctly designed and constructed? After some discussions and consultation Dams for Africa were appointed to inspect the dam and highlight areas requiring maintenance. In particular the adequacy and functionality of the right flank spillway was to receive special attention. A copy of the client's brief is attached in the appendix.

While this report covers in considerable detail the various points raised in the brief, it also makes recommendations for corrective action and includes three options of a conceptual/preliminary design of a proposed '*weir-spillway*' on the right flank.

2.0 Site Visit and Inspection

On 22-03-04 the writer was accompanied by two final year civil engineering students from Wits University (Elvis and Franclyn) acting as assistants. They were met by Mr Ian Cochran and his wife Sue, who were most helpful and hospitable, offering us tea and later lunch. They also took us on an orientation drive showing us all the various dams on the farm and their associated catchments, ending with an overview of the Spurwing dam. Useful background knowledge was gained from this tour.

After lunch at the lodge a survey was made of the crest of the dam beginning on the left side of the auxiliary left flank spillway and ending on the right side of the main right flank spillway. From this information figure 1 has been constructed showing the profile of the dam, with all heights being referenced to the invert level of the 600mm outlet valve.

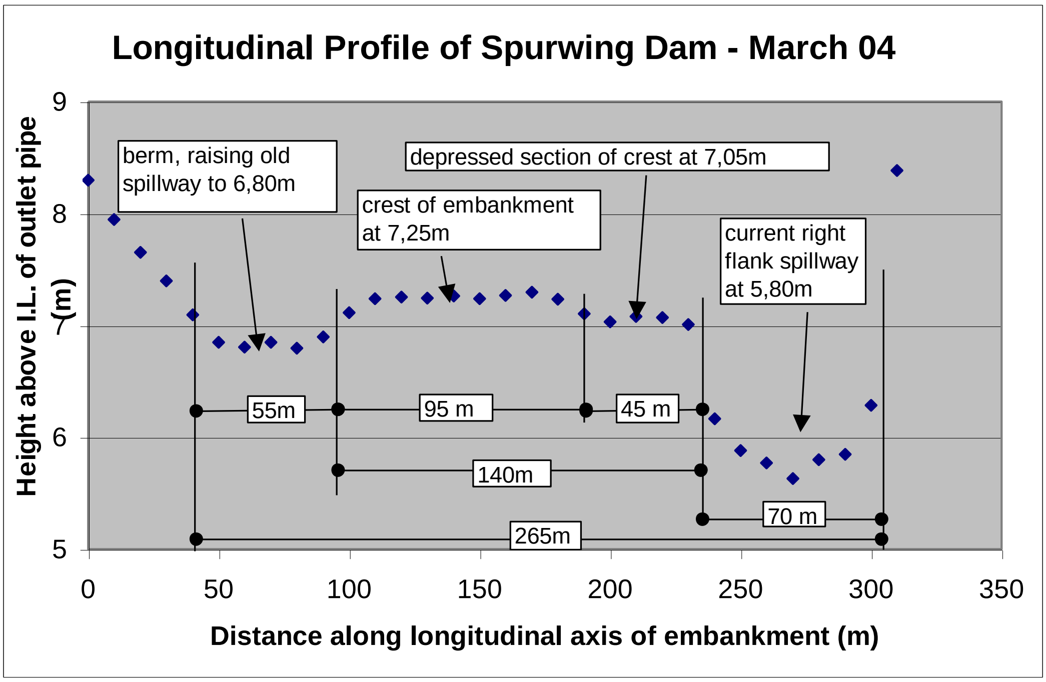

Figure 1 – Longitudinal Profile of Spurwing Dam. All levels are referenced to the invert level of the outlet valve, and are spaced 10m apart. The current right flank spillway has on average an elevation of 5,8m, while the left flank spillway begins to function when the water rises to a height of 6,8m, as this is the elevation of the crest of the road/berm that now traverses it. The crest of the embankment between the two spillways is at an elevation of approximately 7,25m, except for a 45m stretch on the right which is about 0,2 m lower.

An inspection was also made of the downstream embankment, particularly its lower third and toe, to look for evidence of seepage.

Finally a careful inspection of the right flank spillway was made. Levels and measurements were taken, including some measurements of the embankment in the vicinity of the spillway.

No drawings were available of the construction of the embankment or spillways. Apparently there are photographs of the erosion of the old left flank spillway, but these could not be located at the time of the inspection.

3.0 Dam Safety Legislation in South Africa

Since 1986 legislation has been in place to ensure that dams with a safety risk are safely engineered, constructed and /or altered in South Africa. A dam with a *safety risk* is defined as an impoundment with a height of a least 5m and a storage of at least 50000m^3. It is further necessary to obtain a permit to construct, alter, or enlarge any dam with a safety risk from the Dam Safety Office of the Department of Water Affairs and Forestry (DWAF) and such a permit will only be issued on their approval of the design of the measures proposed for the works. Depending on its size and hazard potential, a dam with a safety risk is classified as Category I to III and appropriate levels of risk are assigned for design, safety, and evaluation of flood conditions. Even a category I dam must be designed by a professionally qualified engineer.

On the other hand if a dam is less than 5m in height, or has less than 50,000m^3 storage capacity, no specific design criteria are prescribed in terms of the dam safety legislation and no approvals for construction, alteration, or enlargement are necessary.

In the following paragraph evidence is presented to show that the Spurwing dam is on the borderline of category I, possibly just below.

- Depth of Spurwing dam - Figure 1 shows that the right flank spillway is on average 5,8m above the outlet valve's invert level, which in turn is estimated at 0,6m above the riverbed. Thus the elevation of the water at FSL (the level at which water starts running down the spillway) relative to the riverbed is 5.8 + 0,6 = 6,4m.

- Impoundment volume of Spurwing dam – Owing to the onset of darkness and a storm, there was insufficient time to accurately assess the capacity of the reservoir. However based on the many photographs that were taken earlier, the dimensions given in figure 1, and visual estimates of distances, surface area and topography, it appears that the capacity of the reservoir at FSL may be slightly less than 50000m^3.

In this case the Spurwing dam is not a category 1 dam, and is therefore not considered to have a safety risk. Although the FSL of 6,4m (overflow level above river bed) exceeds the 5m criterion, its capacity is less than 50000m^3. Clearly the dam's impoundment capacity should be confirmed as this has legal and other implications, and this is included as a recommendation (see section 11).

Given this uncertainty, and considering that the dam is close to the category I borderline if it does not actually cross it, it seems prudent to analyse the dam assuming that it is a category I dam, and the design of betterments will accordingly follow the guidelines of the South African National Committee on Large Dams (SANCOLD) for a category I dam, but without seeking the formal approvals from DWAF until the categorization of the dam is known with more certainty.

The chief implication of treating the Spurwing dam as a category I dam in terms of the South African safety legislation is that the *Recommended Design Flood (RDF)* will equate to an event with a 20 year *recurrence interval (RI)* and the *Safety Evaluation Flood (SEF)* will equate to a flood equivalent to *RMF*$_{-\Delta}$, which is approximately equivalent to the 200 year RI flood. These terms and their implications are discussed further on.

4.0 Dam Functionality

A storage dam should satisfy three important criteria. Firstly its embankment should be structurally stable, secondly it should not leak, and thirdly the spillway design should have sufficient capacity to allow flood waters to pass through without causing damage to the spillway or embankment and without the level of the water rising too high.

These three aspects will now be considered in more detail and comments on the adequacy of the existing structure will be made where appropriate.

4.1 Embankment Stability

The embankment should be structurally stable in the sense that it should retain the water without being pushed downstream, without overturning, and without bursting. In an earth embankment dam, this is achieved by ensuring that the downstream and upstream faces of the embankment are not too steep for the type of material used to construct the dam and with due consideration to the material on which the embankment is founded. Furthermore drainage for the downstream shoulder of the embankment should ideally be provided, especially in a dam with a clay core, to increase its stability.

In the absence of drawings it is not known what provision was made for the drainage of the embankment’s downstream shoulder, but the virtual absence of surface seepage on the embankment’s downstream slope and toe indicates that special drainage measures were not necessary.

The inclination of the downstream and upstream slopes of the embankment appear to lie in the zone of 1 in 1,5 through 1 in 2. This is relatively steep, but is acceptable where the material used to construct the dam has good shear strength. This appears to be the case, given the absence of slides and other indications of inadequate shear strength. The only evidence of distress is at a zone affected by flows over the current right flank spillway, where the toe and lower regions have been washed away by discharges over the spillway (see figure 2). This should not be seen a weakness in the design/construction of the embankment, but rather poor spillway design (discussed later). The eroded areas in the embankment should be repaired at the same time as the spillway is re-engineered.

It is evident from figure 1 that the crest of the embankment is virtually horizontal. This is generally an indication of an embankment that was well compacted during the construction stage. All embankment dams consolidate (compact under their own weight with time) to a greater or lesser extent, and clearly consolidation will be greater where the embankment is highest. So for an embankment to still be virtually horizontal and not V shaped after 12 years implies that it was very well compacted at the construction stage. A well compacted embankment generally has good shear strength and hence the various forms of shear failure that are possible are most unlikely even for the increased forces associated with a possible overtopping event (to be discussed later). It also means that it will not erode as easily as would a poorly compacted embankment in the event of water flowing over the crest and down the embankment during overtopping.

However, there are some unknowns that do not allow a greater degree of assurance. These chiefly centre around a lack of knowledge of the design and the materials that were used. (For example, was the dam analysed for stability assuming the level of water would rise the full extent of the freeboard or even higher? Does the embankment have a central clay core to minimise seepage, or is it a homogenous embankment constructed from a single uniform material? How deep is the embankment founded into the residual ground? What is the shear strength and bearing strength of this residual ground?).

To end off this section on a positive note it may be said that small dams that fail often do so within the first year or two after construction because of inadequate compaction, and/or the use of unsuitable materials. Thereafter the process of consolidation begins to take effect, increasing the shear strength of the structure and decreasing the likelihood of failure. The virtual absence of settlement in the case of the Spurwing dam, particularly at the deepest section of the embankment, is evidence that the dam was well compacted at the construction stage, and now with 12 years of ongoing consolidation, the likelihood of failure is extremely slight.

4.2 Embankment Impermeability

No earth embankment constructed from a homogenous material is ever impermeable. A simple seepage analysis of a homogenous embankment will reveal that water will eventually emerge from the downstream face to approximately 1/3height, no matter how impermeable the constituent material. When the material is particularly impermeable, the emergence of seepage might only be seen after several 10s of years. Moreover the rate of evaporation and transpiration where grass is present might exceed the rate of seepage, giving the impression that there is no seepage.

That some water is seeping through the Spurwing dam is indicated by the observation that vegetation at the toe of the dam is more lush than higher up the embankment. Further evidence is that the surface of the soil has a moist sheen in this zone.

There are however no trickle flows coming out of the embankment and no standing water in front of the toe of the embankment (other than where water is let through the valve). It may therefore be said that the measures taken to ensure that the embankment does not leak are effective. It may be that no special measures were taken other than using ground that has a sufficient clay/silt content to be relatively impervious when well compacted. This water may even be migrating directly through the foundation layers. However it is so slight that the structure may be regarded as ‘not leaking’.

The presence of the moisture is of more concern in the effect it has on reducing the shear strength of the embankment and founding layers, although the apparent absence of settlement of the crest of the dam (discussed in 4.1), or of any form of shear failure, suggests that this is not a matter for concern.

There is no evidence of anthills, or rodent tunnels, or trees, which can all be detrimental to the impermeability of the embankment. A few shrubs were in evidence, which should be removed, unless it known that their root systems are not deep.

4.3 Spillway Design

In this section some of the principles and requirements of spillway design are reviewed, in order to lay a foundation for the evaluation of the existing spillway in the next section.

The design of a spillway should satisfy the following requirements:

4.3.1 Ideally the spillway should have *sufficient capacity* to allow flood waters to pass through without causing the level to rise to the point where water goes over the crest of the embankment resulting in erosion, which may result in a breach and consequent emptying of the impounded water.

A spillway may discharge the incoming water either down a relatively long channel at a particular slope, or discharge the flow at one defined point, as in a weir. A combination of these two options is also possible. Expressions (1) and (2) below may be used to determine the flow down a channel, while expression (3) should be used to determine the flow through a weir.

The average velocity of the water flowing in a channel with a constant gradient (i), width (b) and roughness (n) may be determined using Manning's formula which may be stated as:

Velocity, V $= 1{,}5 / n \times (\text{cross sectional area/wetted perimeter})^{0{,}666} \times (\text{gradient})^{0{,}5}$

$$= 1{,}5 / n \,.\, [(b \times d) / (d + b + d)]^{0{,}666} \,.\, i^{\,0{,}5} \quad \ldots\ldots \quad (1)$$

and the corresponding flow by

Q $= \text{velocity} \times \text{cross sectional area}$

$$= V \,.\, d \,.\, b \ldots\ldots \quad (2)$$

From these expressions it is evident that for a given width (b) and roughness (n), the velocity (and hence the flow) will increase as the gradient (i) increases. However, by increasing the velocity the potential for erosion also increases.

It may also be shown that for any given velocity, the greater the gradient, the less will be the depth of flow (d), allowing a shallower channel (which is cheaper as less excavation is required).

In the case of a dam that has an earth embankment it is especially important that the spillway be correctly designed and sized (b, d) so that water ideally does not go over the top of the embankment, especially in dams that impound large volumes or have significant and lasting inflows.

4.3.2 A spillway should be *cost effective.* Where conditions are ideal it is possible to construct a cost effective spillway that can fully contain even the worst predicted flows. On the other hand, in unfavourable topography, the cost of a building a spillway that can accommodate serious flood events could exceed the cost of the whole embankment, especially for relatively small dams. In such situations it may be feasible to make use of an auxiliary spillway on the opposite side to contain some of the peak flow, and even allow some of the flow over the embankment for a limited period, providing the embankment is able to withstand this. A well consolidated and grassed embankment should be able to withstand overtopping of 1m for a period of one hour without failing, and generally this is long enough to cater for surges from burst upstream dams or localised cloudbursts. Some repairs and maintenance may be required after the event, but the cost of such remedial work may be insignificant relative to the cost of a conservative spillway.

One prerequisite if this option is adopted is that the crest of the embankment should be at a constant elevation all along its length to ensure that no concentrated flows of longer duration occur at specific points, which may lead to excessive erosion or even breaching.

4.3.3 The spillway should have *sufficient length* to discharge the water well beyond the toe of the downstream embankment to prevent erosion of the latter.

4.3.4 The spillway should have a relatively *gentle gradient* to prevent water from flowing too fast in its channel, as the erosion potential of water increases exponentially with velocity. Typically channel gradients may vary between 0,1% through 1,5%. Well grassed spillways can withstand velocities of 6m/s for the limited periods of peak flows, which is what may be expected for a 1,7% gradient. On the other hand where the floor of the spillway is rock, it can have virtually any slope.

4.3.5 Where the topography does not allow a spillway with a gentle gradient, it is necessary to create a series of steps at controlled points, or waterfalls, that allow finite reductions in elevation that are able to *dissipate the energy* of the water without being eroded. Often gabion weirs are used for this purpose.

The flow through a weir may be determined by the expression:

$$Q = C.\ (2gd)^{0,5}.bd \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

Where:

C is the discharge coefficient through a weir (generally varies between 0,35 and 0,5), g is the gravitational constant, 9,81 m/s^2, d (in m) is the vertical distance between the crest of the weir and the surface of the water before its velocity accelerates at the crest.

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Where a series of weirs is used in a channel (see figure 4 (a) and (b)) the gradient *between* weirs should be gentle, particularly if it is unlined or grassed, to prevent erosion of the channel bed.

Thus in a spillway where a series of weirs are positioned, expressions (1) and (2) relating to the flow in the channel should be considered together with expression (3) which governs the flow through a weir. Expressions (1) and (2) are used to determine the required depth of the channel connecting the weirs.

4.3.6 A spillway should ideally have a *constant gradient and width.* Any reduction in gradient will result in the depth of flow increasing (see expresion (1)), necessitating an increase in the depth of the spillway. Likewise the width should be either constant or increasing. Depending of the velocity of the flow, a reduction in the width may result in increased velocity (venturi effect, resulting in greater erosion) or an increase in the depth (requiring a deeper spillway).

4.3.7 A spillway's channel should *prevent cross flows.* To achieve this the floor of the spillway should be horizontal in the direction perpendicular to the intended flow.

5.0 Evaluation of Right Flank Spillway

The right flank spillway will now be critically examined in the light of the requirements discussed in 4.3.1 through 4.3.7:

5.1 In the first instance the spillway should have *sufficient capacity* (see requirement 4.3.1).

Clearly the greater the inflow into the dam the greater must be the cross sectional area (b x d) of the spillway. In determining this flow the main considerations are the size of the catchment expressed in km^2 and how much rain the area receives. Rainfall must be considered both in terms of annual rainfall, and in terms of intensity, and to do this Southern Africa has been divided into zones that have various *'K' values* as given in the document *TR137 'Regional Flood Peaks in Southern Africa'* published by the Department of Water Affairs and Forestry.

TR137 also describes a method for determining the inflow into a dam, where use has been made of the concept of the *regional maximum flood (RMF)*, which is an empirically established upper limit of flood peaks that can be reasonably expected at a given site. The method is based on maximum flood peaks recorded since 1856 at more than 500 sites in Southern Africa. Having established the RMF, the given tables/formulas can be applied to obtain the floods corresponding to recurrence intervals of 50, 100 and 200 years. These values may be extrapolated to obtain a flood with a 20 year RI.

The term *$RMF_{-\Delta}$* is related to RMF but applies to a region one further down on the K scale (the next driest region). It is significant in that the dam safety legislation uses this flow to establish the *Safety Evaluation Flood (SEF).* The SEF in South Africa is defined as the maximum flood during which <u>catastrophic failure of the dam must not be experienced, although significant damage can be tolerated</u>. Accordingly it would be quite permissible to design the spillway capacity of the small Spurwing dam to allow some overtopping over the crest of the embankment during the SEF flood event.

The RMF and $RMF_{-\Delta}$ can be instantly calculated from formulas if the geographic position of the site and its effective catchment area are known, as is the case in question. It is evident from the 1:50000 topographical map '2530AC Dullstroom' that the catchment area is relatively small, 28 km^2. Applying the methods described in TR137 the following flood values have been determined:

RMF = $355m^3/s$
$RMF_{-\Delta}$ = $218m^3/s$: choose as the SEF for a category I dam
200 year flood = $214m^3/s$
100 year flood = $176m^3/s$
50 year flood = $137m^3/s$
20 year flood = $86m^3/s$: choose for the RDF for a category I dam

In terms of the dam safety legislation for a category I dam, the two most significant floods in this list are the 20 year RI flood, and the $RMF_{-\Delta}$ .The former is chosen as the Recommended Design Flood, and the latter as the Safety Evaluation Flood.

One way of applying these values to the Spurwing dam is to design the right flank spillway to cope with the RDF by itself, with no water having to pass through the auxiliary spillway on the left flank. This implies that the maximum elevation of the water should not exceed 6,80m (see figure 1).

Using expression (3), and assuming a discharge coefficient, C = 0,385, Q = 86m3/s, and d = 1,0m (determined as the difference in height between the crest of the berm in front of the left flank spillway and the sill of the right flank's spillway, i.e. 6,80m – 5,80m), the value for b, the width of the proposed new 'weir-spillway' on the right flank, may be resolved as 51m.

It remains to check what happens in the event of a SEF. In this event water will flow into the auxiliary spillway, and even a limited flow over the crest of the embankment is permissible. Again, using expression (3), it may be calculated that when the water reaches an elevation of 7,366m, the SEF flow of 218m3/s is achievable. In this case 169m^3/s travels through the proposed 51m wide spillway on the right flank, 40m^3/s travels through the old left flank spillway, and 9m^3/s goes over the crest of the embankment. As this latter flow is a relatively small flow and as it is spread over a distance of 140m (see figure 1) it goes over as a relatively thin sheet of 116mm in height, which is not considered destructive, especially for a well grassed well consolidated embankment. (As a rule of thumb a well consolidated spillway can usually resist overtopping by a depth of approximately 1m for a duration of at least 1 hour before failure occurs). It should also be remembered that significant damage can be tolerated in the SEF so long as it does not lead to catastrophic failure (such as bursting or overturning).

Cresting of the embankment is most likely to occur as a result of upstream dams bursting during exceptional storms. This however will only be problematic where relatively large storage dams fail, which is unlikely as such dams must be designed by professional engineers. Moreover, there do not appear to be any such dams upstream of Spurwing, (refer to the topographical map). Only at E30°02'30", S25°23'05" is there a dam which seems to correspond to Spurwing in surface area, but this does not represent a major threat since it may be calculated that even if this dam has a similar capacity to Spurwing, of approximately 50000m^3, its entire volume will take less than four minutes to pass if flowing at the rate of a SEF (218m^3/s).

Downstream of the Spurwing dam, the gradient of the river is relatively flat (based on the contours on the topographical map); certainly it is much flatter downstream than upstream. Furthermore, the river appears to run in a relatively narrow valley in this region. These two factors combined will result in a choking effect, causing the water to backup upstream. In extreme flows where water goes over the embankment, the toe of the Spurwing dam will likely be inundated by this tailwater, and this will protect it from water that may crest the embankment, so that only the higher parts of the embankment will be subjected to the erosive forces from the sheet of water flowing down it.

All the above arguments show that the 51m wide right flank spillway supplemented as it is by the auxiliary spillway and some overtopping over the embankment's crest has sufficient capacity for the SEF. Therefore the current width of 70m (see figure 1), is more than sufficient. Such a wide spillway would be comforting if it was correctly designed and constructed, which unfortunately is not the case. This becomes evident as the other features of this spillway are examined with reference to the other requirements for a spillway, as will become apparent in the next few paragraphs.

5.2 The requirement of a *gentle gradient* (see requirement 4.3.4) is completely ignored, resulting in high velocities that have led to alarming levels of erosion (notice the uneven bed and small dongas in figure 2), especially considering that it has only been functioning for one year. The gradient (horizontal distance / vertical distance x 100) approaches 75% in places leading to super critical flows, hydraulic jumps and turbulence – all very erosive. The channel of a spillway should ideally have a gradient of 1% or less to restrict the velocity of the water to non erosive levels, unless it is founded on rock or is lined with concrete. Where topography does not allow a gentle gradient, finite reductions in elevation through energy dissipating weirs are required, so that the channel between the weirs *can* have a gentle gradient.

5.3 The requirement that a spillway should have *sufficient length* (see 4.3.3) to discharge the water well downstream of the embankment is overlooked. In fact the water flowing on the left side of this spillway flows directly down a portion of the embankment which happens to be in its path! (see figure 2). This has resulted in significant erosion to this zone of the embankment. It appears that, more recently, an attempt has been made to create a rock-drain in front of the embankment to divert the water to the right and so prevent the flow from going over the downstream face of the embankment. However, this measure will only be effective for small flows. Large flows will soon completely fill the drain and water will go over the embankment again, and it is precisely these large flows that are the most erosive. Furthermore, this water, small of large, can be seen to have flowed along the toe of the embankment, eroding it for a considerable distance.

5.4 A spillway should *prevent crossflows* (see requirement 4.3.7). In the current right flank spillway there is a very pronounced cross flow from right to left that directs the water against the toe of the embankment for a considerable distance (see figure 2), and this compounds the erosion already occurring as a result of the action described in 5.3.

5.5 In the foreground of figure 2 a small gabion structure may be seen, and there are a number of these structures immediately upstream of the one in view.

Presumably they were intended to keep the residual flow of the river away from the toe of the embankment. However, they are too insignificant to cope with flows generated by heavy rainfall in the catchment, let alone flood events. Therefore the proposed design calls for a series of weirs of a much sturdier configuration that span the full width of the 51m channel, creating steps that lower the water up to 2m at a time (see figure 4). The gabion cages and rocks that have been used in these structures can be reused to construct the new structures.

Figure 2 – This picture illustrates how several of the requirements for a spillway have been overlooked. The spillway is far too steep (see requirement 4.3.4) and consequently significant erosion is evident. It is also evident that the water that flows over the left side of the spillway discharges its water directly down the embankment (see requirement 4.3.3). The crossflow in the spillway channel (see requirement 4.3.7) results in water flowing against the toe of the embankment, and this has resulted in erosion for a considerable distance.

5.6 The only thing that might be said in favour of this spillway is that it is so wide that water will never go over the crest of the embankment in a serious flood event. But this width creates other problems that have already been discussed, such as discharging water over a portion of the downstream embankment, leading to erosion of the latter. However even a 51m wide spillway, as proposed in this option, would encroach on the downstream embankment.

6.0 Old left flank Spillway

Thus far only the right flank spillway, with its many shortcomings has been evaluated. Before recommending a new design for the right flank spillway, consideration should be given to once again making the old left flank spillway the main spillway.

There are advantages and disadvantages:

6.1 The strongest point in favour of again making the left flank spillway the main spillway is that there is a small hill positioned immediately downstream of the point where the mouth of the spillway meets the beginning of the embankment. This means that water flowing down the spillway must go away from the embankment for some distance before it can turn towards the river, and thus the spillway's discharge is kept away from the embankment, precluding its erosion. In effect this serves the same purpose of a long spillway (see requirement 4.3.3). The bed of the spillway also slopes gently upwards from either side of its centreline, and these slopes are reasonably symmetrical about this line. These attributes will limit crossflows (see requirement 4.3.7).

6.2 However, there are also some disadvantages to re-positioning the main spillway to the left flank of the dam. There is a significant drop, of about 2,5m, before the spillway reaches the plain of the river, and it will therefore be necessary to dissipate the energy (see requirement 4.3.5) by introducing a substantial weir structure with a non erodeable bed to cater for the concentrated flows at this point. Added to this its width is significantly reduced in this region, resulting in a channelling effect that increases the unit energy of the water. Even above this point the spillway does not have a constant gradient and width (see requirement 4.3.6), neither can the gradient in this zone be classified as 'gentle' (see requirement 4.3.4), so that several gabion barriers will in addition be required along the length of the channel at strategic positions.

A further complication is that the boundary fence is very close to this point (if my memory serves me correctly), and it is conceivable that in order to achieve sufficient width the eventual weir structure will encroach over the boundary. This introduces other extraneous factors such as obtaining the permission and cooperation of the neighbours if this option is to be pursued.

It is therefore evident that relocating back to the left flank may be as costly and problematic as correcting the right flank. But this cannot be said with certainty. There was insufficient time to make a more detailed evaluation of the left flank, which should be done before a final decision is made to re-engineer the right flank as this may be the most cost effective.

Notwithstanding for the time being it will be assumed that the option of re-engineering the right flank is the more economical option, and this solution is pursued in 7.1. as the major component of the preliminary design.

7.0 Preliminary Design

Six areas requiring attention have been addressed in the preliminary design. These may be located in figure 3 and are fully described in 7.1 through 7.6. The preliminary design is further supported by figures 4 through 7.

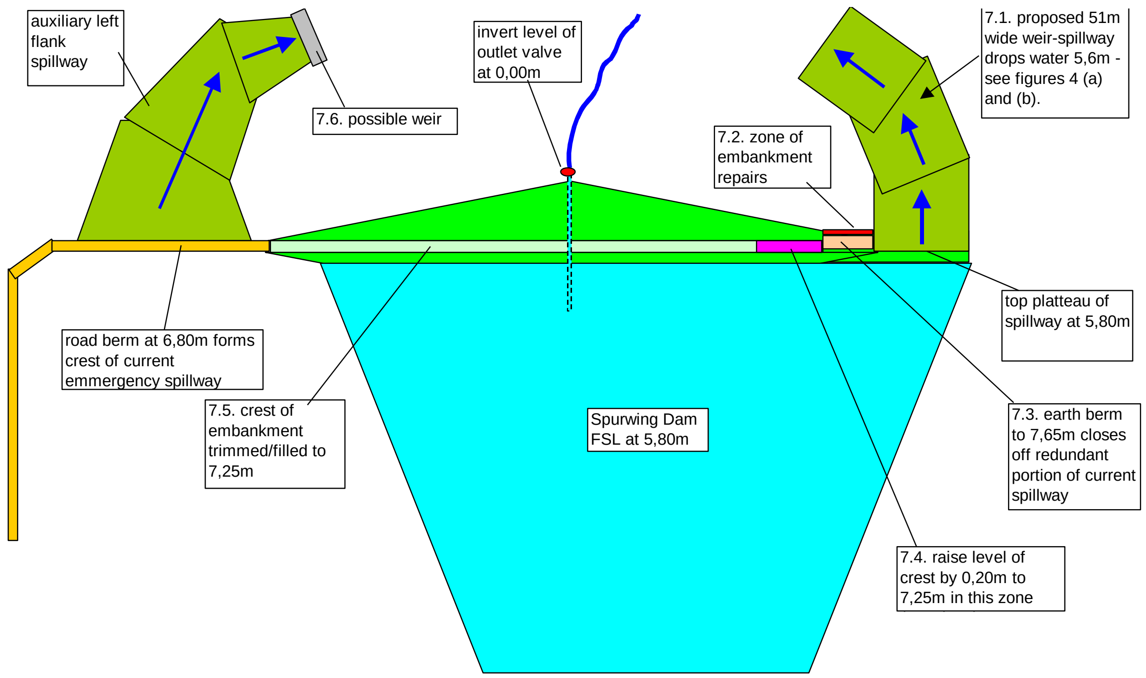

Figure 3 – Schematic Plan of Spurwing Dam indicating proposed scheme of work (7.1 to 7.6).

7.1 Weir-spillway

7.1.1 Preliminary design weir-spillway

In section 5 a case for a 51m wide weir-spillway was made to serve as a means of conveying storm water around the dam. In this process the energy of the water is dissipated by lowering it 5,6m from the elevation of 5,80m (FSL) to a safe 0,20m via four weirs (see figure 4(a)) strategically positioned in an excavated/profiled channel (see figure 4(b)). All levels are referenced to the invert level of the outlet valve at 0,00m. Between the weirs the excavated channel will vary in depth from approximately 1,5m to 2,5m, and have a gradient not exceeding 1%. The total length of the spillway is expected to be of the order of 60m, and the weirs are spaced approximate 20m apart.

Note that these dimensions and details pertain to the preliminary design, and some changes are likely in the final design.

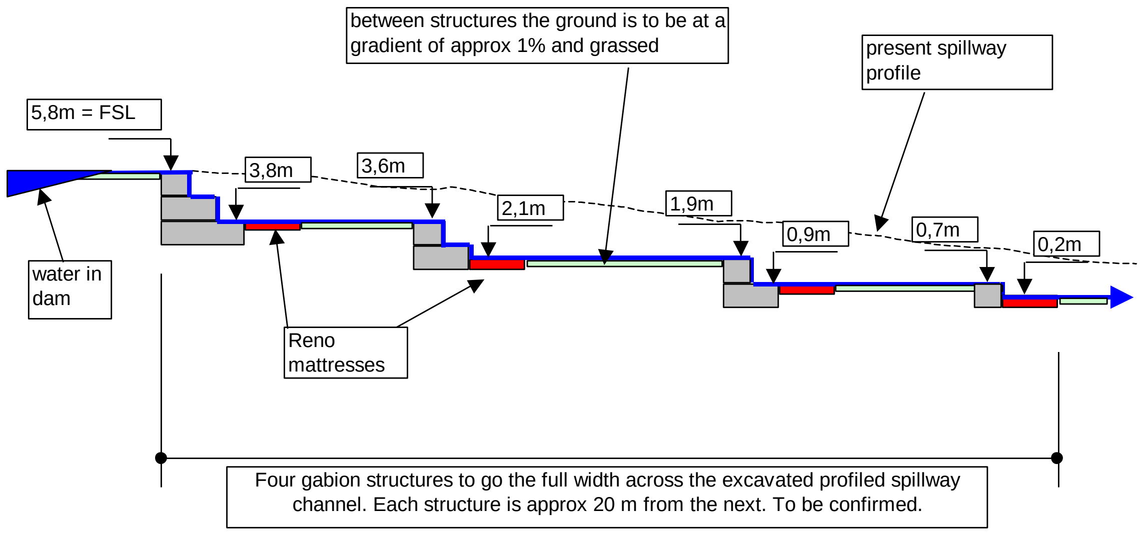

Figure 4 (a) - Section through longitudinal profile of weir-spillway showing where the various changes in elevation are likely to occur. Background structures are not shown.

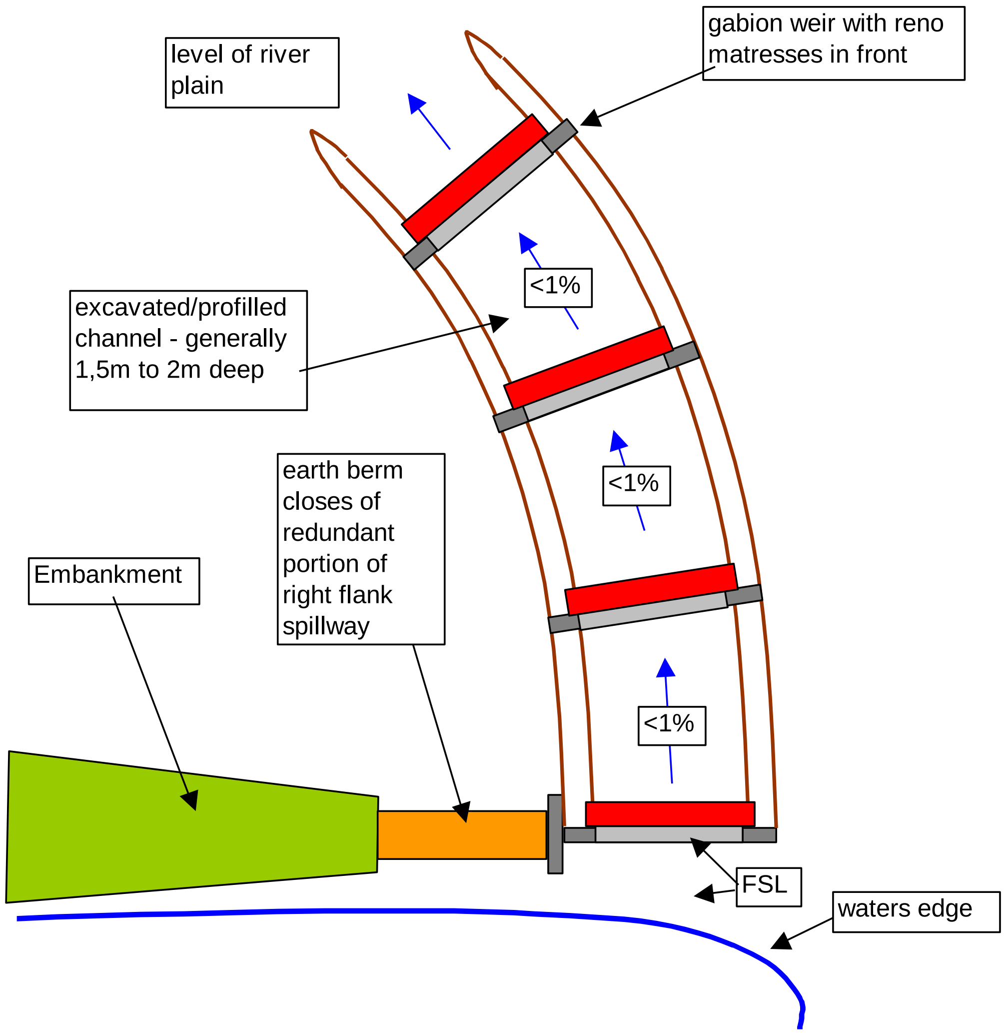

Figure 4 (b) – Plan view of spillway, showing the four gabion structures constructed across the excavated/profiled channel at strategic positions

7.2.2 Spillway capacities and floods

When heavy rain falls on the 28km^2 catchment, the water in the dam will steadily rise from its FSL (5,80m) until it reaches the level of the auxiliary spillway (6,80m). Now 86m^3/s will pass through the proposed weir-spillway and this will be adequate for most storm events, up to a flood with a 1 in 20 year RI. At this point the level of the water is 450mm below the crest of the embankment. In the event of a flood with a 1 in 50 year RI, the water will rise to within 175mm of the embankment's crest, and now 124m^3/s will pass through the proposed right flank weir-spillway, while 14m^3/s will go through the left flank spillway. For a flood with a 1 in 100 year RI, the water will rise to within 10mm of the embankment's crest, and 149m^3/s will pass through the proposed right flank weir-spillway, and 27m^3/s will go through the left flank spillway. These values are tabulated in table 1 below. It may be seen from this table that for the 1 in 200 year RI flood, the $RMF_{-\Delta}$ and the RMF the level of the water has risen to the extent that it goes over the top of the embankment, but even the 409mm overspill of the RMF is permissible for short durations in terms of the SEF criterion for category I dams of tolerating 'significant damage' providing it does not result in 'catastrophic failure'.

Table 1 : Spillway Capacities

	elevation of water in relation to crest of embankment	width of spillway	flow through dam
	mm	m	m^3/s
20 year flood (Q=86m3/s)			
A. proposed right bank weir-spillway	-450	51	86
B. old left bank spillway	-450	55	0
C. over embankment	-450	140	0
Total throughput			**86**
50 year flood (Q=137m3/s)			
A. proposed right bank weir-spillway	-175	51	124
B. old left bank spillway	-175	55	14
C. over embankment	-175	140	0
Total throughput			**138**
100 year flood (Q =176m3/s)			
A. proposed right bank weir-spillway	-10	51	149
B. old left bank spillway	-10	55	27
C. over embankment	-10	140	0
Total throughput			**176**
200 year flood (Q = 214m3/s)			
A. proposed right bank weir-spillway	106	51	167
B. old left bank spillway	106	55	39
C. over embankment	106	140	8
Total throughput			**214**
SEF flood (Q=218m3/s)			
A. proposed right bank weir-spillway	116	51	169
B. old left bank spillway	116	55	40
C. over embankment	116	140	9
Total throughput			**218**
RMF flood (Q=355m3/s)			
A. proposed right bank weir-spillway	409	51	218
B. old left bank spillway	409	55	75
C. over embankment	409	140	62
Total throughput			**355**

At the mouth of the spillway, the top of the spillway sidewalls should be 0,4m higher than the embankment’s crest. This will ensure that flood waters are confined to the spillway, even for the RMF.

It bears repeating that reducing the width of the spillway from 70m to 51m not only has cost advantages, but is a practical necessity in terms of limiting the degree of encroachment on the embankment.

Figure 5 – Example of a weir-spillway in Italy, constructed of gabions and reno-mattresses. The large drops in elevation take place at the gabions, allowing much reduced velocities in the channel between the gabions.

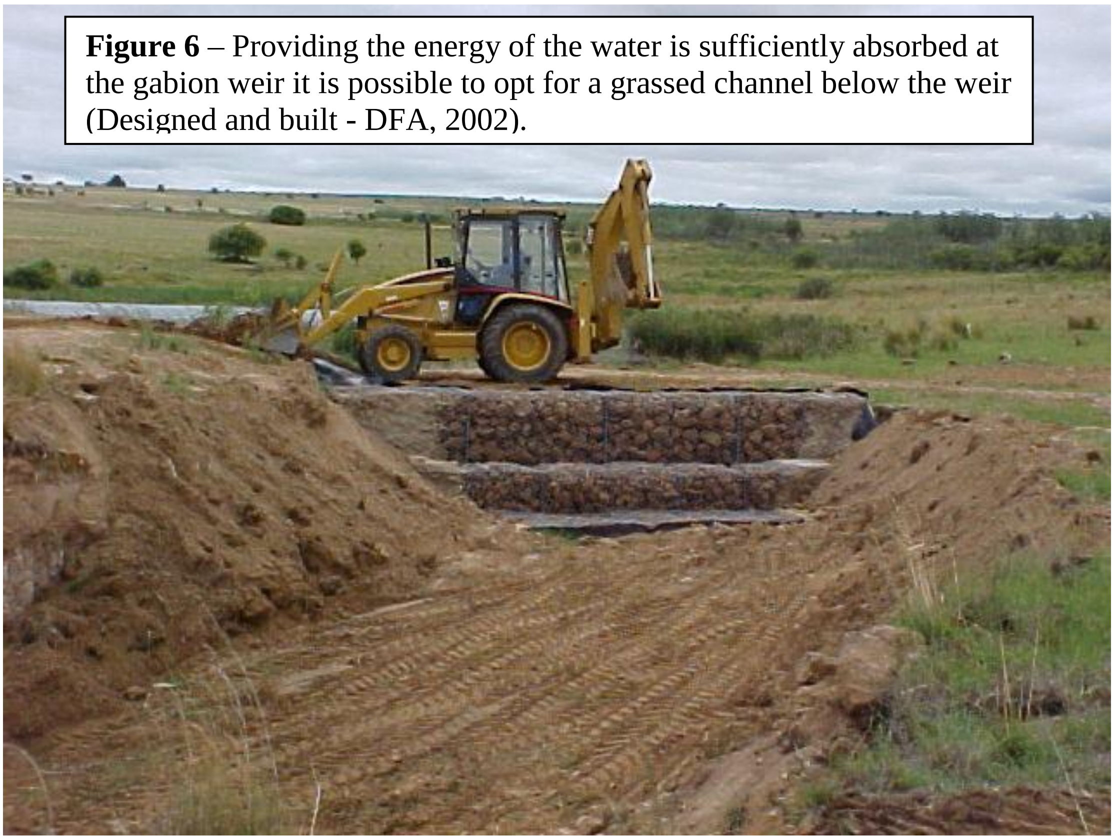

Figure 6 – Providing the energy of the water is sufficiently absorbed at the gabion weir it is possible to opt for a grassed channel below the weir (Designed and built - DFA, 2002).

Figure 7 – Where ground is well consolidated it may be cost effective to construct the weir from a more rigid system such as random-rock-masonry as shown here. Much of the ground from which the re-engineered spillway is to be constructed is either undisturbed or well compacted, making this type of construction an option in the final design, if better suited than gabions. This diversion weir is situated in a gorge that has a 130km^2 catchment. (Designed and built – Dams for Africa 2003/4).

7.2 Repairs to embankment

As explained in 5.3 the current right flank spillway discharges part of its water directly over the embankment. This has resulted in the toe of the embankment being washed away in this area and a degree of shear failure has occurred. The embankment is also very steep in this zone approaching an angle of 1 in 1,3, and this should be brought in line with the rest of the embankment.

As indicated in figure 2 and 3 the effected areas are not large, and the repairs can thus be effected relatively easily with simple compaction equipment.

7.3 Closure of redundant portions of existing spillway

Once the repairs in 7.2 have been completed, an earth berm can be constructed on top of the existing spillway, starting at the point where it currently begins to descend, through to the gabion structure shown in figure 3. The berm will therefore be about 19m long, 3m wide at the top, and 1,85m high. This will not require vast quantities of ground, which can be sourced from the excavated channel of the re-engineered right hand spillway. Again simple compaction equipment will suffice. A height tolerance of plus/minus 25mm is acceptable. Note that the elevation of this portion is somewhat higher that the rest of the embankment’s crest, to protect the toe of the embankment in this region from any further erosion in the event that overtopping occurs over the rest of the embankment (as may be expected for a 1 in 200 RI flood and upwards).

7.4 Raise level of crest

As indicated in figures 1 and 3, there is a portion of the crest, approximately 45m in length, that requires additional material to raise it by approximately 0,2m, so that it is at the same elevation as the rest of the crest. This will enable the crest to act as an emergency spillway, for floods exceeding 1 in 100 years. A tolerance of plus/minus 25mm is acceptable for the final elevation of the crest. Once again this is a simple process of clearing existing vegetation, bringing in the correct ground, spreading over the existing crest, and compacting with relatively small roller vibrators.

7.5 Trimming and filling of crest

The crest of the embankment has a few high and low spots (over and above what is mentioned in 7.4), as may be discerned from figure 1. This will require some trimming and filling in a few places. A tolerance of plus/minus 25mm is acceptable.

7.6 Weir in Auxiliary left flank Spillway

Consideration should also be given to constructing a weir in the auxiliary spillway as indicated in figure 3, even if it is to remain as the auxiliary spillway. At the point indicated the spillway descends steeply, and a weir (either gabion, or random-rock-masonry) that is designed to dissipate the energy would virtually eliminate maintenance. Note that this spillway will only start to function the moment the flow exceeds $86m^3/s$, i.e. above the 1 in 20 year RI flood. However, it may be decided that the cheaper option is to simply do periodic repairs, since generally the proposed weir-spillway on the right flank will cater for most events.

There is some evidence of maintenance work having been done here – in the form of filling with loose rocks. While this form of repair is good for stabilizing saturated unstable slopes, giving support and facilitating drainage at the same time, it is not ideal for preventing dongas in spillways from creeping upslope, as the water simply gets in from the top and continues to erode behind the rocks, flushing out the soft ground through the rocks, albeit at a slower rate. (There is also evidence of this type of repair work on the right flank, see figure 2, which as explained only slows down the rate of erosion. *Warning:* This flushing out process can also happen to gabion structures that are not correctly conceived and constructed).

8.0 Other Options

The cost of a 51m wide spillway that effectively has four step-down weirs (see figure 4 (a) and 4 (b)) and has an associated excavated channel that is approximately 60m long, may prove to be too costly.

The shareholders may therefore wish to reduce the width of the channel/weirs of the right flank spillway by bringing the auxiliary spillway into use sooner, to assist with the 1:20 year flood, the RDF in terms of a category I dam. (It is 86m3/s for this catchment).

8.1 20m Wide Spillway

Applying this principle to the fullest extent, the level of the water may be designed to reach (but not overtop) the crest of the embankment in a 1 in 20 year RI flood. With a 20m wide right flank spillway which starts overflowing at 5,80m and a 55m wide left flank spillway that starts overflowing at an elevation of 6,8m, it is possible to accommodate the 1 in 20 year RDF of 86m3/s, with 58m^3/s going down the right flank spillway, and 28m^3/s going down the auxiliary.

A flow of 218m^3/s must still be accommodated for the SEF, and for this to happen the level of the water must overtop the crest of the embankment by 400mm, allowing 84m^3/s to pass through the right flank, 74m^3/s through the left flank, and 60 m^3/s over the top.

Note that the above approach only just satisfies the requirements of the dam safety legislation for a category I dam. Furthermore, the ability of the embankment to withstand the erosive effects of overtopping becomes a much greater factor, since every flood event over the 1 in 20 event will cause overtopping.
With relatively large volumes of water going through the auxiliary spillway quite often it may become necessary to upgrade it sooner rather than later according to the preliminary design described in 7.6.

8.2 26m Wide Spillway

In this option the width of the right flank spillway has been designed so that overtopping of the embankment only occurs at or above the 1 in 50 year RI. Furthermore, in order to limit the width of the right flank spillway to limit encroachment towards the embankment as far as possible, the height of the berm on the left flank spillway has been reduced from 1m to 0,7m, thus effectively allowing more water down the left flank spillway. In this case, for the RDF, the water is 280mm *below* the embankment’s crest, with 56m^3/s passing through the right flank spillway and 30m^3/s passing through the left flank spillway. For the SEF it overtops by only 240mm, with 97m^3/s passing through the right hand spillway, 92m^3/s passing through the left hand spillway, and 28m^3/s over the embankment.

In the writer’s opinion this option is a good balance between cost and comfort, and is recommended for the Spurwing dam. Once again any overtopping events that occur (which should occur statistically every 50 years) should be followed by a careful inspection of the embankment to see how it performed.

9 Erosion protection of Embankment on upstream side

There does not appear to be any significant erosion from wave action on the upstream side of the embankment, indicating that the rocks (rif-raf) that have been positioned there to protect the embankment are working effectively.

10 State of Other Dams on Letata

In 2.0 mention was made of a brief visit to the other dams on Letata, including Oribi, Wickham's fancy, Eagle Owl, Dragon Fly, Zulu, Dog Nobler, Coachman, and Murray's Island. The following observations can be made:

These dams are all very small with embankments well below 5m and impoundment capacities well below 50000m3, and as such do not constitute a safety risk. With the exception of Murray's Island (and thus Spurwing) the other dams have much smaller catchments, and their spillways appear to be minimally eroded. Several of the spillways appear to be situated on bedrock, which is ideal. Various degrees of seepage were evident on the downstream slopes of some of the embankments, but only that observed in the Oribi dam is serious enough to merit further investigation. No anthills or rodent tunnels were noticed on the embankments, but on some of them a number of larger bushes had established themselves. In the interests of preserving impermeability as well as shear strength and hence stability, it is recommended that these be removed, unless they can be identified by a botanist as having shallow root systems, certainly less than 1m at full maturity.

A leak in Murray's dam was detected, the water flowing beneath the road and on towards Spurwing dam. The water is presumed to be flowing through a 'pipe' consisting of rocks under the road that have voids between them. This should be investigated. It should be relatively simple to plug this 'pipe' by applying and compacting a few wheelbarrows of clayey fill at its mouth on the upstream side.

11 Conclusion and Recommendation

The general impression gained is that the embankment is soundly engineered and constructed; it is both stable and virtually impermeable. A small amount of work on the crest will ensure a constant elevation throughout its length, which will be beneficial in the event that the embankment is overtopped in exceptional flood events.

However the design and construction of the right flank spillway is not in accordance with sound practice, as evidenced by the significant levels of erosion that have occurred in the space of one year.

The following is recommended:

The proposed remedial steps outlined in 7.1 through 7.6 should be taken to rectify the various deficiencies. This should be preceded by finalizing the design, including working drawings, based on the requirements and principles set out in sections 4 and 5, and the preliminary designs discussed in sections 7 and 8. Thereafter an estimate of quantities and costs should be made. Finally a suitably qualified and experienced contractor with a good track record should be appointed to do the work.

It is recommended, at the least, that another brief inspection be made of the left flank spillway, before a final decision is made to re-engineer the right flank spillway. Assuming that the right flank spillway is retained, (which is likely) it is proposed that a 26m wide channel about 2m deep and 60m long be excavated/profiled according to carefully established levels, and four weirs built as energy dissipaters at strategic points along the channel. Figure 6 gives an indication of how the channel would be profiled between the weirs, except that the Spurwing weir would be significantly wider.

The storage capacity of the dam should be more accurately determined to establish if it exceeds 50000m3. This exercise should ideally be done with the dam as empty as possible so that accurate levels of the dam’s basin can be established.

The solutions proposed are considered appropriate in the circumstances of the project and can be implemented relatively quickly.

Finally, Dams for Africa wish to express their appreciation to the shareholders of Cutwater Farms (Pty) Ltd for entrusting this investigation to them.

Yours faithfully

Nicholas Papenfus
Director - Dams for Africa

Appendix

Brief for Nicholas Papenfus Dams for Africa from Cutwater Farms (Pty) Ltd

Spurwing dam

1: Background

The dam was built around twelve years ago. Ivan Schwarz who was deputy Professor of the Hydrology Department at Wits designed the wall and calculated the length of spillway needed using a map of the catchment area and 100 year rainfall records for the area.

During 2002 and 2003 the level of the dam rose to the spillway level on a number of occasions. These runoffs started eroding the downstream section of the spillway and started eating up towards the dam wall.

A contractor was called in to repair the eroded piece of spillway. This contractor recommended the moving of the spillway to the other side of the dam wall. This has been done and the construction of the new spillway, with a gabioned runoff area has largely been done.

The erosion of the spillway resulted in complaints to the department of water affairs by a downstream syndicate.

2. Brief

2.1 Could you please confirm that the original calculation of the spillway size of 60 meters is correct.

2.2 Could you please inspect the dam as it stands now and confirm that

- The spillway size is sufficient for the anticipated runoff from your new calculations
- That the design of the new spillway is optimal for the dam in terms of ease of maintenance.
- Raise for our attention any other areas of the dam wall which may require maintenance.

3. Invoices

Would you please invoice Cutwater Farms (Pty) Ltd and post it to Guy Fletcher at PO Box 2079 Houghton 2041

About Dams for Africa

Dams for Africa (Pty) Ltd design/construct/rehabilitate water related infrastructure to **empower communities** in remote rural areas. Typical projects include dam rehabilitation, canal, weir and reservoir construction, installation of pipelines, boreholes and irrigation systems.

DFA recognises the need to be **flexible** and will tailor its involvement according to each need, from minor consultations to relatively large turnkey projects.

DFA's contribution to a **typical project** may take the form of an initial feasibility study, followed by design and/or construction.

Whenever practical **labour intensive** methods will be used in the construction process, sourced from local community.

DFA is also in a position to provide the necessary hydrological, topographical, geological, ecological and social impact **studies**, and attend to the technicalities and legalities associated with water related infrastructure.

Dams for Africa fully appreciates the need to **network** and co-operate with partners such as:

1. *Community based organizations* that are in touch with the needs of the resident population.

DFA is aware of the importance of *community involvement* and is, if required, prepared to participate in all stages of this process. This would include a response-to-need request as the first step, assistance with visualization, participation in negotiations, recruitment and training of local residents for the construction stage, facilitation of training in subsequent agriculture and irrigation, and ongoing mentoring as may be required.

2. *Donors/funders* including government and financial institutions.

DFA is prepared to participate in *fundraising* for worthwhile projects, and in the production of 'bankable' documentation.

3. *Training organizations* who teach on farming methods, produce marketing, and who know the value of ongoing mentoring.

DFA would like to know that its engineering contribution is placed in the hands of a motivated community that has been *equipped* with the necessary skills to put the water infrastructure to good use for many years to come.

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